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HAYWARD BAKER
Geotechnical Construction
KELLER

Vancouver Energy Terminal
Ground Improvement Design- Areas 300 & 400

Submitted to:

R&M Engineering Consultants
5280 South Commerce Drive, Suite E-130
Murray, Utah

Submitted by:
Hayward Baker Inc.

April 15, 2015

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HBI Project No.: 860058

April 15, 2015

R&M Engineering Consultants
5280 South Commerce Drive, Suite E-130
Murray, Utah

Attention: Mr. Norman Bennion, PE, SE

Subject: Tesoro Savage Petroleum Terminal LLC
Ground Improvement Design- Areas 300 & 400

Hayward Baker Inc. (HBI) is pleased to present the design for ground improvement for the structures and major transfer pipelines to be constructed in Area 300 and Area 400 of Tesoro Savage Petroleum Terminal LLC's planned facility located at the Port of Vancouver, WA. The purpose of the ground improvement program is to enhance the safety, stability and serviceability of the proposed improvements. This is accomplished by increasing the strength of the ground to the point where the ground can safely support the anticipated structures under static loads as well as during and after the design level earthquake. Additional information is provided in the attached report.

The design provided herein has been prepared for the exclusive use of HBI and our client under the following strict limitations:

1. Only HBI may construct the work described by the design, and
2. The design may not be used by others for any purpose.

HBI appreciates the opportunity to be of service. Please feel free to contact me at (206) 223-1732 with any questions, comments or concerns.

DRAFT

Mark Rohrbach, PE, GE, P.Eng.
Sr. Engineer
Hayward Baker Inc.

Mike Blanding
Branch Manager
Hayward Baker Inc.

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1.0 EXECUTIVE SUMMARY

The proposed project consists of the construction of a facility that will provide transloading services for pipeline quality crude oil from railcars to marine vessels. The proposed development will occupy portions of Terminals 4 and 5 and Parcel 1A at the Port of Vancouver, WA. The project includes specific areas that have been designated Areas 200 through 600 and covers an area of approximately 47 acres. This report addresses ground improvement for Area 300 and Area 400. Area 300, storage, includes six oil storage tanks and an associated transfer pipeline. Area 400, marine terminal, includes the portion of the transfer pipeline system located along the Columbia River shoreline, the dock abutment and several ancillary support structures.

According to the project geotechnical reports (GRI, December 20, 2013 & September 5, 2014), the subsurface soils at this site and in their present condition are subject to significant static settlement due to compression of near-surface soft and loose soils. In addition, the sands and silty sands have a potential to liquefy under the design earthquake. The geotechnical report states that ground improvement will likely be required to limit static and liquefaction-induced soil movement beneath the tanks, transfer pipeline and near the dock abutment. The ground improvement program described herein mitigates the potential static and seismic ground movement risks in accordance with project specific soil movement tolerances and applicable building codes (e.g. ASCE 7-10, IBC 2012, API 650, API 653, ASCE 61-14). For this project HBI determined three ground improvement techniques are required. These techniques include:

- **Vibro-Stone Columns (Stone Columns):** This ground improvement technique uses specialty purpose-built vibrating probes to densify and reinforce the soils while constructing a stone column of approximately 0.9m (3 foot) diameter. Vibro-Stone Columns are commonly called stone columns.

- **Jet Grout (JG):** This ground improvement technique involves drilling a relatively small hole to a target depth then using high pressures to create high velocity grout that erodes away the existing weak soil and replaces it with a high strength mixture of soil and grout to form a nearly homogeneous low strength concrete commonly referred to as soilcrete. Depending on project requirements jet grouting can be used to improve ~25% to ~100% of the soil in a given area.

- **Deep Soil Mixing (DSM):** This ground improvement technique involves using high torque equipment to mechanically mix grout with native soils to create a nearly homogeneous mixture of weak concrete called soilcrete. DSM is a top down construction technique. Often predrilling can be used to simplify disposal of construction spoils and waste soil. Depending on project requirements DSM can be used to improve ~10% to ~90% of the soil in a given area.

In Area 300, the tank area, HBI's analysis has shown that a ground improvement program below the tanks is necessary. In general the ground improvement below the tanks consists of stone columns three feet in diameter and spaced at 8.2 ft. on centers (square grid spacing) and a systematic first filling of each tank with water. The length and tip elevation for the stone columns is different for each tank. The tip elevations range from about Elevation -14 ft. to about Elevation -18.5 ft. Additional information is provided in the narrative of this report and on the Design Concept Drawings. At Tank 1, the base design is supplemented with two additional rings of stone columns to address potential differential settlement concerns in unusually poor soil conditions.

Also in Area 300, HBI's analysis indicates that ground improvement below the proposed transfer pipeline is also necessary. In general, ground improvement below the transfer pipeline consisting of stone columns three feet in diameter and spaced at about 7.7 ft. on centers will achieve the required static and seismic

ground behavior. The length and tip elevations for the stone columns supporting the proposed transfer pipeline varies along the alignment of the transfer pipeline. Stone Column tip elevations range from about Elevation 5 ft. to about Elevation -16 ft. Additional information is provided in the narrative of this report and on the Design Concept Drawings.

In Area 400, the marine terminal, the transfer pipeline alignment is generally parallel with the river and about 94 ft. northeast of the ordinary high water line (OHWL). HBI's analysis has shown that ground improvement below the proposed transfer pipeline is necessary. In the transfer pipeline portion of Area 400 stone columns alone are not adequate to provide the required stability; therefore, the design includes a series of jet grout columns, DSM panels and stone columns. The DSM panels are used to limit the potential liquefaction below the pipe alignment. The jet grout column is used to provide the vertical support of the pipe-rack foundation. The stone columns form a nonliquifiable buttress that stabilizes the shoreline area. The DSM panels are spaced approximately 35 ft. apart, 55 ft. long, ±6 ft. wide and extend to Elevation -18 ft. The jet grout column below the DSM panel, is 8 ft. in diameter and extends to Elevation -50 ft. The stone columns are located between the DSM panels and the top of the bank are 3 ft. in diameter and spaced at 8 ft. on centers. The stone columns extend to the nonliquefiable soils at approximately Elevation -50 ft. Additional information is provided in the narrative of this report and on the Design Concept Drawings.

Also in Area 400, HBI's analysis indicates that ground improvement in the vicinity of the proposed dock abutment is necessary. In this area, the transfer pipeline support transitions from foundations bearing on improved soils to pipe-racks supported by the proposed dock structure. This transition occurs at the dock abutment. At the dock abutment a block of ground improvement approximately ±160 ft. long (parallel to the river) and ±72 ft. long (perpendicular to the river) is necessary. This ground improvement block consists of three zones. The southern zone begins at the OHWM and extends ±40 ft. landward (northeast). The soil in this area is not adequate to support the abutment piles so jet grouting methods will be used to achieve full replacement (Ar = 100%) The central zone contains jet grout columns 6 ft. in diameter and spaced to achieve a 40% replacement ratio (Ar = 40%). The third (northern) zone contains the DSM/Jet Grout section described above for support of the pipe-rack foundations. Additional information is provided in the narrative of this report and on the Design Concept Drawings.

2.0 INTRODUCTION AND NOTICES

For this project HBI is the ground improvement design-build contractor and the ground improvement design engineer. In this capacity, HBI performed this evaluation in a manner consistent with the standard of care ordinarily exercised by members of the geotechnical engineering community currently practicing in the area. Tesoro Savage Petroleum Terminal LLC is the Owner. HBI is working for Tesoro Savage Petroleum Terminal LLC, through R&M Engineering Consultants. Although there are expected to be other contractors working on this project, none have authority over HBI.

Any designs prepared and furnished by HBI, including any designs, means, and/or methods (collectively, "Designs"), have been prepared based upon, and in anticipation of, HBI performing the work called for in such Designs. As such, the Designs are specifically tailored for HBI's purposes and have been prepared for exclusive use by HBI. Owner and/or Contractor shall not use or control the Designs without the prior written consent of an authorized representative of HBI, and then only as absolutely necessary to perform such permitted use. The Designs are subject to protection under the Copyright Act of 1976 and Architectural Works Copyright Protection Act of 1990. Owner and/or Contractor shall not copy, reproduce, emulate or model the Designs or disclose, publish, or disseminate the Designs to any third parties without the prior written approval of an authorized representative of HBI. HBI is and shall continue to be the sole owner of the Designs.

Owner and/or Contractor shall not permit other contractors to use the Designs on this Project or use the

Designs on other projects without the prior written approval of an authorized representative of HBI. HBI makes no warranties or guarantees as to the suitability of the Designs for use by others or for other applications or projects. Any such use shall be at Owner and/or Contractor's sole risk, and the Owner and/or Contractor shall indemnify, defend, insure, and hold harmless HBI, its agents, consultants, officers, directors and employees from and against any and all claims, damages, losses, costs and expenses, including but not limited to attorney's fees, costs and expenses, arising out of or resulting from use of the Designs by any party other than HBI.

In addition to this introduction this report is divided into the following sections:

- Section 3.0 summarizes the basis of design information upon which HBI'S design is based.
- Section 4.0 summarizes the analytical methods used during the design process.
- Section 5.0 summarizes the results of our analysis and demonstrates compliance with all required performance requirements.
- Section 6.0 describes the test sections that will be constructed to verify the adequacy of the design.
- Section 7.0 describes the verification and testing that will be performed to evaluate the test sections and the subsequent production work.
- Section 8.0 summarizes HBI's scope of work.
- Section 9.0 lists technical references reviewed or cited as part of the design process. Project specific references are included in the text of Section 2.0.

The soil conditions in Area 300 and the performance requirements are notably different than those of Area 400. To simplify presentation of the design each section of this report provides information common to both areas, then Area 300 followed by Area 400.

3.0 BASIS OF DESIGN

This design requires the tanks to be supported by a load transfer platform (LTP) a minimum of 4 ft. thick. The LTP will consist of compacted structural fill with an in-place density of about 130 pcf and an angle of internal friction of at least 36 degrees. The bottoms of the tanks are at Elevation 31.75'. This design is based on HBI's understanding of the project. Our understanding of the project is based on our review of the following documents and performance requirements articulated in our contract.

3.1 Reference Documents

Although many documents were reviewed, only those which provided information that directly affects our design are listed below.

- "Calculations for Flat Bottom Tank Ring Foundation", not dated, by R&M Engineering Consultants. This document appears to be a MathCAD file titled "13014 Savage POV, Typ. 350,000 bbl Crude Oil Tank Foundation";
- Design Concept Drawings "Area 300 Storage Tank Foundation, 0300-FD-004 REV HH", Dated November 14, 2014, by R&M Engineering Consultants;
- Provided TSPT Site Plan CAD drawing by R&M dated November 12, 2104, by R&M Engineering;
- Design Concept Drawings "Footing & Foundation Plot Plan Area 300, 0300-FD-001 REV HH", Dated November 14, 2014, by R&M Engineering Consultants;
- "Ground Improvement Port of Vancouver", dated October 17, 2013 and prepared by R&M Engineering Consultants;
- "Tesoro Savage Vancouver Energy Distribution Terminal – Dock Facility, Port of Vancouver, USA", dated September 5, 2014 and prepared by GRI; and

- "Tesoro Savage Vancouver Energy Distribution Terminal – Upland Facility, Port of Vancouver, USA", dated December 20, 2013 and prepared by GRI (see below).

If any of these documents are changed or altered in any, way HBI may need to modify our design. HBI understands GRI is currently preparing a revision to their December 20, 2013 which will include a clear recommendation for the regarding the design earthquake(s) corresponding to IBC 2012 Site Class D/E. We further understand these recommendations will be consistent with conversations which occurred during our April 10, 2015 meeting and that the revised report will be published in April 2015.

3.2 Performance Requirements

In addition to the documents listed above our design also considered the design and performance requirements articulated to us by Tesoro Savage Petroleum Terminal LLC and its consulting team. These requirements which are different for each area and structure and are summarized below. HBI was not a party to the selection of the design and performance criteria. As part of our professional due diligence we have confirmed that these values are reasonable and slightly more stringent than what we would normally expect for a project of this type in this location.

3.2.1 Performance Requirements for Area 300

The Performance Criteria for Area 300 are:

- Static settlement at the perimeter of the tanks:
  2" total settlement (post hydro-test), and
  1" differential settlement (post hydro-test).

- Seismic (Liquefaction) settlement at the perimeter of the tanks:
  2" total settlement, and
  1" differential settlement.

- Combined Seismic/Static Settlement at the center of the tanks:
  8 inches (post hydro-test).

- Design groundwater elevation:
  Elevation +12 Ft.

- Design earthquake corresponding to IBC 2012 Site Class D/E:
  PGA=0.37g
  Mw=9.0

For your convenience we have summarized the most important design parameters in Table 1 (below)

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Table 1. Summary of Ground Improvement Design Parameters for Area 300

Parameter	Value	Basis
Earthquake Magnitude	9	GRI
PGA	0.37	GRI
Groundwater Elevation in Area 300 (ft.)	12	GRI
Elevation of top of soil profile (ft.)	Varies (±27.5)	Based on survey data
Elevation of bottom of soil profile	Varies	Varies with top and tip of CPT*
Elevation of Installation	Ex. Grade -4ft.	HBI Proposal Dated 11/26/13
Bottom of Tank Elevation (ft.)	31.75	Drawing 0300-FD-004
Thickness of soil profile to analyze	Length of CPT	Varies per CPT*
Critical Facility	NO	IBC 2012
Bearing Pressure (In Service)	2500 psf	GRI / R&M Calculations
Bearing Pressure (Hydro-test)	2882 psf	R&M Calculations
Tank Diameter	240 ft.	Drawing 0300-FD-004
Tank Shell Capacity	400,000 bbl	BergerABAM
Tank Maximum Fill Volume	360,000 bbl	BergerABAM
Tank Working Volume	340,000 bbl	BergerABAM
*All CPTs were advanced to practical refusal which occurred atop the gravel at about Elevation -24 ft. to -30 ft. The gravel is reported to have a low potential for static or seismic settlement.		

3.2.2 Performance Requirements for Area 400

- The transfer pipeline is located about 80 ft. from top of bank

- Static displacement of pipe-rack foundations (both vertical and horizontal)
  2" total settlement (within 30 ft. pipe span)
  1" differential settlement (within 30 ft. pipe span)

- Seismic displacement of pipe-rack foundations (both vertical and horizontal)
  2" total settlement (within 30 ft. pipe span)
  1" differential settlement (within 30 ft. pipe span)

- Lateral Spreading (both vertical and horizontal)
  2" total

- Static settlements of structures in improved areas:
  1" total settlement
  0.5" differential settlement

- Seismic settlements of structures in improved areas:
  2" total settlement
  1" differential settlement

- Design groundwater elevation along the shoreline:
  Elevation +7 ft.

- Design earthquake corresponding to IBC 2012 Site Class E:
  PGA=0.37g
  Mw=9.0

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For your convenience we have summarized the most important design parameters in Table 2 (below)

Table 2. Summary of Ground Improvement Design Parameters for Area 400

Parameter	Value	Basis
Earthquake Magnitude, Mw	9	GRI
PGA	0.37g	GRI
Groundwater Elevation Area 400 (ft.)[1]	+ 7.5 ft.	GRI
Elevation of top of soil profile	Varies (±27.0 ft)	Based on survey data
Elevation of bottom of soil profile	Varies (-56 to -66 ft)	Varies with top and tip of CPT[2]
Thickness of Soil Profile to analyze	Length of CPT	Varies per CPT[2]
Critical Facility	NO	IBC 2012
Cross-section A-A' & A1-A1'	NA	GRI and project drawings
1. This is the groundwater elevation used for analysis and assumed to exist at the time of field explorations (SPT and S/CPT testing). 2. All CPTs were advanced to practical refusal which occurred atop the gravel at about Elevation -53ft.		
The gravel is reported to have low potential for static or seismic settlement		

3.3 Site Soils

The GRI report included seven drilled geotechnical borings and four CPT soundings in Area 300. HBI supplemented this exploration program by advancing an additional 43 Seismic Cone Penetration Tests and traditional (not seismic) Cone Penetration Tests (together S/CPT) soundings and an additional eight drilled geotechnical borings in Area 300.

The GRI report included seven drilled geotechnical borings and one CPT soundings in Area 400. HBI supplemented this exploration program by advancing an additional six S/CPT soundings and an additional two drilled geotechnical borings in Area 400.

The supplemental field explorations were completed between November 10 and 26, 2014. The eight drilled geotechnical borings were advanced for the purpose of obtaining laboratory samples and confirming the soil types and validating HBI's estimates of CPT based soils fines content.

S/CPT soundings were completed by In Situ Engineering of Snohomish, WA using both truck and track mounted rigs. The CPT rigs were calibrated for accuracy in May of 2014 (see Appendix A). Gregory Drilling of Redmond, WA completed the drilled geotechnical borings using mud rotary drilling methods and a truck mounted CME-75 drill rig (Rig No. 304). Drilling of the geotechnical borings included obtaining Standard Penetration Test (SPT) and thin wall Shelby Tube samples. The SPT samples were obtained in general accordance with ASTM D1586. The SPT sampler was driven with an automatic hammer operating with an efficiency of 80% (see Appendix A). The logs of HBI's S/CPT soundings are provided in Appendix B. The logs of HBI's geotechnical drilled borings are provided in Appendix C.

All HBI exploration locations were surveyed by Mackay Sposito of Vancouver, WA. The S/CPT and SPT locations are provided on the exploration location maps in Appendices B and C. The survey data sheets are also provided in Appendices B and C. The survey data sheets include the surface elevation of each exploration location.

Geotechnical laboratory testing of soil samples collected during our field investigation was performed by E3RA of Tacoma, WA and HartCrowser of Seattle, WA. Geotechnical laboratory testing included

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Atterberg Limit testing (ASTM D4318), Percent Passing #200 Sieve including natural moisture content (ASTM D1140), Determination of Unit Weight (ASTM D2937 or ASTM C29), and 1-D Consolidation of Saturated Cohesive Soils using Controlled-Strain Loading (ASTM D4186). E3RA performed the basic index testing and HartCrowser performed the highly specialized testing (as well as some index tests for correlation with E3RA's results). The results of the supplementary laboratory program are included on the boring logs and in Appendix D.

3.3.1 Area 300 Soil Conditions

In general the soil profile in Area 300 consists of three to ten ft. of loose to very dense granular soils overlying 5 to 20 ft. of very soft to stiff fine grained soils with interbedded layers of loose sand. This layer overlies 5 to 20 ft. of clean to silty sand (primarily SP-SM) which in turn overlies the very dense gravel described in the GRI report. HBI did not develop a generalized design soil profile for this area; instead HBI evaluated the entirety of each CPT sounding and developed a ground improvement design appropriate for each soil profile encountered below each tank.

HBI's exploration and testing program specifically targeted the very soft to stiff fine grained soils below the surficial loose to very dense granular soils. This was done because this layer was considered critical to tank static settlement and tank edge stability calculations. In general our targeted exploration program concluded that:

- The soils described as "Soil Zone 3 - Clay" on the CPT logs are quite plastic and compressible.
- In the field and during construction, it will be very difficult to distinguish between soils with high plasticity and soils with low to moderate plasticity.
- The "Soil Zone 3 - Clay" soils are sensitive and widespread.

3.3.2 Area 400 Soil Conditions

In area 400, within approximately 100 ft. of the shoreline and below the riprap and asphalt pavement, the subsurface soils consist of predominately loose sandy soils, with thin layers of medium stiff/medium dense sandy silt/clayey and reverted back to silty sand/clean sand extending down to 65 ft. to 70 ft. below ground surface (approximately Elev -40 ft. to Elev. -45 ft.). Below Elev. -40 ft. to Elev. -45 ft., the sand becomes denser and eventually transitions into dense gravel.

In area 400, where lateral spreading analysis is necessary, a design soil profile was developed for the purposes of lateral spreading analysis. The design soil profile includes three soil units which are based on the physical characteristics and engineering properties of the soils encountered by the explorations. These three soil units are:

1. Fill/Silty Sand, (SP, SP-SM, SM),
2. Silt (ML, SC.), and
3. Gravel

4.0 ANALYTICAL APPROACH

The structures to be constructed in Area 300 are different than the structures proposed for Area 400. In addition the soil conditions at Area 300 are different than the soil conditions at Area 400. In recognition of these differences HBI's design uses different methods and different design soil profiles to evaluate the existing soil conditions and the anticipated post improvement conditions and performance.

4.1 Derivation of Soil Properties

4.1.1 Area 300 – Soil Properties

As discussed above, in Area 300 the soil profile consists of three to ten feet of loose to very dense granular

soils overlying 5 to 20 feet of very soft to stiff fine grained soils with interbedded layers of loose sand. This layer overlies 5 to 20 feet of clean to silty sand (primarily SP-SM) which in turn overlies the very dense gravel described in the GRI report. HBI did not develop a generalized design soil profile for this project; instead HBI evaluated the entirety of each CPT sounding and developed a ground improvement design appropriate for each soil profile encountered below each tank. The basis for the material properties used for design is summarized below.

- Soil Density (γ): Based on relative density as determined using correlations with SPT and CPT testing and index values published by Bureau of Reclamation (1987).
- Angle of Internal Friction (ϕ);
  - For sandy soils, the friction angle is calculated as ϕ=SQRT(20*$(N_1)_{60}$)+20; after Hatanaka and Uchida (1996)
- Es=3.0·q_c.
- Soil Cohesive Strength, (c): Based on shear vane and pocket penetration tests performed by HBI personnel and by our specialty geotechnical testing laboratory, HartCrowser. These test results were then compared to published correlations with SPT and CPT for consistency.
- Initial Void Ratio (e): Based on laboratory testing performed by HartCrowser.
- Coefficient of Compressibility (C_C), Based on laboratory testing performed by HartCrowser.
- Coefficient of Recompression (C_r), Based on laboratory testing performed by HartCrowser.

4.1.2 Area 400 – Soil Properties

As noted above, in Area 400, where lateral spreading analysis is necessary, a design soil profile was developed for the purposes of lateral spreading analysis. The design soil profile includes three soil units which are based on the physical characteristics and engineering properties of the soils encountered by the explorations. These three soil units are:

1. Fill/Silty Sand, (SP, SP-SM, SM),
2. Silt (ML, SC), and
3. Gravel

The drained soil strength properties used for these soil groups are provided in Table 3 (below).

Table 3. Soil Properties Area 400

	Fill/Silty Sand	Silt	Gravel	Basis
Unit Weight (γ, psf)	120	115	135	Typical values by designer
Friction Angle (ϕ, degrees)*	33	25	50	ϕ=MIN(17.6+11log(q_{t1}), 2)
Cohesive Strength (c, psf)	200	300	0	Typical values by designer
* after Kulhawy & Mayne, et al. 1990				

4.2 Evaluation of Existing Soil Conditions

The first step in the analysis is to determine the total and effective stress as a function of depth along the entire soil profile evaluated. The CPT data are used along with various published correlations and a site-specific correlation relating Ic to fines content to evaluate the soil type, fines content, and in situ stress for the entire soil profile at each CPT test location.

The liquefaction potential of the unimproved soils is calculated using the CPT data and the method of analysis proposed by Boulanger and Idriss (2014). The factor of safety against liquefaction for the initial

conditions (no ground improvement) is plotted in blue on the calculation summary sheets included in Appendix E.

The anticipated pre-improvement settlement is calculated using the method proposed by Tokimatsu and Seed (1987), and the predicted settlements are provided on the lower left portion of the calculation summary sheets in Appendix E.

4.3 Evaluation of Ground Improvement

Two methods are used to evaluate the expected degree of densification resulting from the proposed ground improvement program. They are:

1. The Priebe method (Priebe, 1995), and
2. The Baez method (Baez, 1995).

HBI's method to predict the degree of ground improvement related to the construction of stone columns considers both the theoretical basis and the limitations of both of the above methods. For example the Priebe method has been used to evaluate area replacement ratio (Ar) values as high as 40% and relies primarily on the strength of the constructed stone column but does not consider densification. The Baez method includes site research for Ar values up to 12%, endeavors to predict densification and does not rely on the strength of the stone column. Experience has shown the approach used for this project predicts actual densification resulting from the ground improvement program with reasonable accuracy. On this project, the accuracy of the model used will be verified by post construction verification testing of a test section. Based on the results of the test section, the stone column layout may be modified.

These calculations also rely on the following published correlations which are used when determining pre-improvement soil properties. For completeness these relationships are provided (again) below:

- For sandy soils, the friction angle is calculated as ϕ=SQRT(20*$(N_1)_{60}$)+20; after Hatanaka and Uchida (1996).
- The soil modulus (Es) is calculated using correlations from Duncan and Bursey (2007) and originally from Tan, et al. (1991).

After these calculations are complete, the in-situ stresses are recalculated using the revised soil unit weights based on the previously calculated soil type and a factor related to the Ar. The soil type and percent fines values are not altered but a new soil density profile is created.

4.4 Evaluation of Improved Soil Conditions

Using the predicted properties of the improved soil profile, the factor of safety against liquefaction for the improved soil is calculated using the method proposed by R.W. Boulanger and I.M. Idriss (April 2014).

The improved factor of safety against liquefaction for the improved case, but without consideration of soil reinforcement effects, is plotted in purple on the calculation summary sheets. The improved factor of safety against liquefaction with consideration of soil reinforcement effects based on the procedure suggested by Rayamajhi (2014) is plotted in red on the calculation summary sheets.

As above, the predicted post-improvement liquefaction induced settlement is calculated using the method proposed by Tokimatsu and Seed (1987), and provided in the lower left portion of the calculation summary sheets included in Appendix E.

With regard to static settlement, long term consolidation settlement is closely related to the drainage path

for water trapped within fine grained soils. For purposes of calculation of consolidation settlement the stone columns are considered to be highly permeable. Thus, within the ground improvement zone the drainage distance is related to the soil profile and the stone column spacing and below the ground improvement zone the drainage distance is related only to the soil profile. Therefore, traditional consolidation settlement is expected to occur more quickly within the treatment zone than below the treatment zone.

During the service life of each structure, it will be impossible to distinguish between elastic (a.k.a. immediate) settlement and rapidly occurring consolidation settlement that occurs within the treatment zone. Although interesting from an academic perspective, the distinction is not significant from a practical perspective. From a practical perspective what matters is how much will the structures settle during the hydro-test and how much the structures will settle after the final piping is attached. *To simplify presentation of calculation results all settlement that is expected to occur during the first filling/loading (i.e during hydro-testing) is referred to as "rapid" settlement and all settlement that is expected to occur following first filling is considered "long-term".* In an effort to prevent confusion, clarifying notes have been added throughout this report.

4.4.1 Static Elastic Settlement of Tanks (Area 300)

Pre-improvement and post-improvement elastic settlement is calculated using the analysis method proposed by Schmertmann & Hartman (1978).

These calculations also rely on the following simplifications and published correlations:

- $Es=3.0 \cdot q_c$
- Use of a square footing of equivalent area (for Ø=240 ft; L=212' and B=212')
- Use no reduction of settlement magnitude for flexible footings.
- Depth of footing is 2.0 ft. (this is to model the tank not the ring beam)
- The upper 1.5 ft. of soil directly below the bottom of the footing will result in negligible settlement because it is either structural fill or has been proof rolled.
- Silty and clayey soil within the ground improvement zone will experience some elastic settlement and some time-dependent settlement.
- When calculating settlement for the improved soil profile, the Priebe 'n' value is applied as recommended by Priebe (1995).

Please note that the Schmertmann & Hartman method specifically includes a term for long term creep of granular soils. Using the Schmertmann & Hartman method elastic creep is considered using the relationship $C2=1+0.2 \cdot \log(t/0.1)$, where 't' is the time factor measured in years. For this application of this project ' t ' has been set to 50 years.

Settlement limitations for the perimeter of the tanks are more restrictive than for the center of the tanks. For each CPT, the settlement beneath the center of the tank is calculated using the above procedure and a soil profile indicated by the CPT data. Settlement beneath the edge of the tank is calculated based on the settlement at the center of the tank and applying a method of stress distribution derived from the Boussinesq stress distribution solution. This results in a reduction in soil settlement of about 50%. In other words, the settlement at the edge of the tank will be about one-half of the settlement at the center of the tank.

4.4.2 Static Consolidation Settlement of Tanks (Area 300)

Pre-improvement and post-improvement consolidation settlement is calculated using traditional 1-dimensional consolidation theory where the stress increase is calculated by integrating the Boussinesq point load solution over a loaded area (e.g. Perloff and Baron, 1976). Doing this for a uniformly loaded area produces the following equation for the increase in stress beneath the center of the area.

$\Delta\sigma = [1 - (1/(1 + (a/z)^2)^{1.5})]\ \sigma$, where z is the depth at which the change in stress is being calculated, 'a' is the radius of the circle, and σ is the applied uniform surface loading

These calculations also rely on the following published correlations/simplifications:

- Depth of footing is 2 ft. (this is to model the tank not the ring beam)
- The upper 1.5 ft. of soil directly below the bottom of the footing will result in negligible settlement because it is either structural fill or has been proof rolled.
- Silty and clayey soil within the ground improvement zone will experience some elastic settlement and some time-dependent settlement.
- When calculating settlement for the improved soil profile, the Priebe 'n' value is applied as recommended by Priebe (1995).

The settlement beneath the edge of each tank is calculated based on the expected settlement beneath the center of the tank using the procedure referenced above for elastic settlements.

Please note that evaluation of long term secondary compression/consolidation of fine grained soils considers the hydro-testing described in Section 7.2 and Table 18. Following the Hydro-test soils will have an overconsolidation ratio (OCR) of ±1.15 suggesting that secondary compression/consolidation settlement will be small and less than 0.25 inches.

4.5 Edge and Base Shear Stability of Tanks (Area 300)

Edge and base shear stability checks were performed for both the full service load condition and for the full hydro-test load condition. HBI used the procedures recommended by Duncan and D'Orazio (1984). These checks were performed assuming no strength gain of the fine grained soils occurs during incremental loading of the tanks and are provided in Appendix E.

4.6 Pipe-rack Foundation Support

4.6.1 Pipe-rack Liquefaction Settlement

The potential for settlement of the pipe-rack foundations was evaluated using the analysis methods described above for Area 300 (Boulanger and Idriss (2014), Tokimatsu and Seed (1987), etc.). Various initial evaluation results such as factor of safety, liquefaction-induced settlement, and estimated in-situ effective friction angles for the pre-improvement are plotted in red on the calculation summary sheets provided in Appendix F. These results indicate that the unimproved soil settlement is not acceptable and ground improvement is necessary.

In Area 400 where the soils are considered more easily densified, HBI's design relies more heavily on the Baez method than it did to predict the degree of ground improvement in Area 300. Using the predicted properties of the improved soil profile, the factor of safety against liquefaction for the improved soil is calculated using the method proposed by Boulanger and Idriss (2014). The improved factor of safety against liquefaction, expected improved liquefaction-induced settlement, and expected improved effective friction angle for the improved case are plotted in blue on the calculation summary sheets.

On this project the degree of ground improvement required to satisfy the seismic settlement criteria of the pipe-rack foundations is less than the amount of ground improvement required to adequately address the lateral spreading hazard. Additional information is provided in Sections 4.6.2 and 4.6.3 (below).

4.6.2 Pipe-rack Lateral Spreading Analysis

Lateral spread has generally been observed to take place in the direction of a free-face. Displacement magnitudes within lateral spreads exhibit strong spatial variations, both with distance from a free-face and

with position parallel to a free-face. Other factors such as earthquake magnitude, distance from the earthquake epicenter, thickness of the liquefiable layers, and the fines content and particle sizes of the liquefiable layers also affect the amount of lateral ground displacement. Because the dock terminal area includes a free-face slope along the Columbia River, the subsurface soils have very high lateral spreading potential.

According to FHWA-NHI-11-032, assessment of lateral spread hazard must consider dynamic loading as a "driving force", in addition to gravitational forces. HBI selected the widely used Newmark sliding block analysis technique to estimate lateral displacement. The Newmark sliding block method is a two-step method that considers loading at the site:

> **Step 1:** A pseudo-static slope stability analysis is performed to evaluate the horizontal acceleration required to yield the slope (i.e. reach FS = 1.0). The horizontal acceleration that causes yielding of the slope or free face is referred to as the yield acceleration (k_y).
>
> **Step 2:** After the yield acceleration is found, the Newmark displacements are determined by integrating incremental seismic displacements that occur when dynamic/seismic movement momentarily exceeds the yield acceleration of the slope. The total anticipated Newmark lateral displacement was then evaluated using the Bray and Rathjie (1998) simplified method for the post-improvement slope condition.

For both steps the analysis is performed using the soil strength properties that are likely to exist at the time lateral spreading is expected to occur. On this project this occurs during and after the design seismic event. During this time the not-improved soils are assumed to have liquefied. HBI evaluated the residual strength for liquefied soil using the method proposed by Idriss & Boulanger in 2008. This method uses empirical data to estimate residual shear strength of liquefied soil and is well suited for this application (because the liquefiable layer is quite well-defined and continuous along the shoreline, thus it is concluded that void redistribution effects are expected to be negligible). HBI also performed a limited parametric analysis and concluded that based on all of the information available a residual friction angle of $\phi'=10.5^{o}$ is appropriate for use in the lateral spreading analysis.

HBI evaluated the potential for lateral spreading at the site using the method proposed by Kitazume & Terashi (2013) against soil extrusion for soil between adjacent Deep Soil Mixed panels. Because the Kitazume & Terashi based factor of safety against soil extrusion is greater than 1.0, simplified cross-sections were analyzed. These simplified cross sections were evaluated using the computer program Slope/W 2012 by Geostudio. These calculations are summarized in Appendix F.

4.6.3 Geotechnical Jet Grout Column Capacity and Settlement

As stated above, HBI plans to use a single 8 ft. diameter JG column to provide the load transfer of DSM columns directly underneath each pipe-rack footing to the underlying non-liquefiable soils. This hybrid DSM-JG column will penetrate into dense gravel layers at approximately Elev. -50ft and will function as an end-bearing column. To estimate the vertical capacity and anticipated vertical settlement during the design earthquake event, this hybrid DSM-JG column is modeled as an equivalent 6 foot diameter continuous soilcrete column surrounded by roughly 75 ft. of downdrag soils. Allowable design geotechnical DSM-JG column loads and anticipated settlement were calculated using AllPile 7.4 software by CivilTech using the soil properties provided in Table 4 (below).

The simplified soil profile used for design is as summarized in Table 4:

Table 4. Soil Properties Used for AllPile Analysis

Depth	Soil Type	Friction Angle (°)	Cohesion (psf)	Comment
0 - 45 ft.	DSM-JG column	0	2250	Hybrid DSM-JG column flanked by 6 foot diameter DSM columns.
45 - 75 ft.	Liquefied Sand	10.5	0	

HBI has designed the soilcrete to have an average target 28-days unconfined compressive strength (UCS) of 200 psi. Detailed Calculations are provided in Appendix F.

The allowable geotechnical compression load and expected settlement for DSM columns have been calculated considering potential downdrag of the soil layers extending from existing grade to approximately 75 ft. deep. A conservative negative friction equal to 100 percent of the static positive friction value was applied for soils.

4.6.4 Pipe-rack Static Settlements

Based on the AllPile analysis, it is anticipated that static settlement of the pipe-rack foundation supported by the improved ground is less than one half of an inch.

4.7 Dock Abutment Support

4.7.1 Dock Abutment Liquefaction Settlement

The liquefaction potential of the soils around the abutment was evaluated using the methods described in Section 4.6.1 (above).

4.7.2 Dock Abutment Lateral Spreading Analysis

The potential for and magnitude of lateral spreading in the area of the dock abutment was evaluated using the methods described in section 4.6.2.

4.7.3 Dock Abutment Static Settlements

Based on the AllPile analysis, it is anticipated that static settlement of the pipe-rack foundations supported by the improved ground is less than one half of an inch.

5.0 SUMMARY OF CALCULATIONS

5.1 Area 300

HBI's analysis indicates that the combination of stone column length and replacement ratio listed in Table 5 is adequate to achieve the required static total and differential as well as seismic total and differential performance objectives. This conclusion is based on the results of our calculations as summarized in Tables 6 through 12 (below). Actual tank perimeter settlements are expected to be an average of the calculated perimeter settlement with differential settlements being approximately half of the average total settlement.

For the purposes of this project rapid settlement is defined as settlement expected to occur during construction and hydro-testing and long term settlement is defined as settlement expected to occur after the tank is placed into operation. For more information please review the last paragraph of Section 4.4 (above).

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Table 5. Summary of Ground Improvement Design

Tank	Working Elevation	Tip Elevation	SC Length	Ar (sand)	Ar (silt)
1	24.9 ft.	-15.1 ft.	40 ft.	10.5%	22%
2	24.9 ft.	-18.1 ft.	43 ft.	10.5%	11%
3	24.6 ft.	-11.4 ft.	36 ft.	10.5%	11%
4	23.6 ft.	-11.4 ft.	35 ft.	10.5%	11%
5	22.5 ft.	-13.5 ft.	36 ft.	10.5%	11%
6	24.1 ft.	-13.9 ft.	38 ft.	10.5%	11%
Transfer pipeline	Varies	Varies	Varies	10.5%	12%

Table 6. Summary of Analysis for Tank 1

Length = 40 ft.; Ar=10.5%				Improved		Original	
Soil Profile	Exploration Depth	Orig. Liq Sett.	Imp. Liq Sett.	Rapid* Sett.	Long Term Sett.	Rapid* Sett.	Long Term Sett.
HB-CPT-111	56.8-ft	2.87-in.	1.14-in.	0.98-in.	< 0.5-in.	0.44-in.	1.96-in.
HB-CPT-112	56.8-ft	1.96-in.	0.54-in.	0.98-in.	< 0.5-in.	0.42-in.	1.95-in.
HB-CPT-113	59.2-ft	2.65-in.	1.42-in.	1.5-in.	< 0.5-in.	0.57-in.	3.0-in.
HB-CPT-115	55.0-ft	3.79-in.	1.92-in.	0.74-in.	< 0.501-in.	0.72-in.	0.01-in.
X-CPT-3	54.8-ft	2.99-in.	1.51-in.	0.97-in.	0.04-in.	0.82-in.	1.96-in.
Average*:		**2.82-in.**	**1.26-in.**	**1.05-in.**	**0.02-in.**	**0.54-in.**	**1.73-in.**
HB-CPT-114(s)	55.3-ft	2.66-in.	1.49-in.	2.26-in.	0.05-in.	1.34-in.	4.49-in.

*Because of the project acceptance criteria the settlement at the center of the tank (HB-CPT-114s) is not included in the calculation for average settlement.

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Table 7. Summary of Analysis for Tank 2

Length = 43 ft.; Ar=10.5%				Improved		Original	
Soil Profile	Exploration Depth	Orig. Liq Sett.	Imp. Liq Sett.	Rapid Sett.	Long Term Sett.	Rapid Sett.	Long Term Sett.
HB-CPT-121	53.8-ft	5.01-in.	1.26-in.	1.1-in.	0.08-in.	0.57-in.	1.46-in.
HB-CPT-124	34.5-ft	2.66-in.	.5-in.	0.52-in.	0.06-in.	0.29-in.	0.65-in.
HB-CPT-126	57.1-ft	3.8-in.	1.92-in.	0.58-in.	0.1-in.	0.71-in.	0.84-in.
HB-CPT-125	57.4-ft	6.22-in.	2.34-in.	0.78-in.	0.11-in.	0.6-in.	1.-in.
HB-CPT-122	57.1-ft	4.26-in.	2.17-in.	1.5-in.	0.11-in.	0.8-in.	2.-in.
Average*:		**4.39-in.**	**1.64-in.****	**0.9-in.**	**0.09-in.**	**0.59-in.**	**1.19-in.**
HB-CPT-123(s)	56.8-ft	4.26-in.	2.17-in.	0.68-in.	0.2-in.	1.03-in.	0.88-in.

*Because of the project acceptance criteria the settlement at the center of the tank (HB-CPT-123s) is not included in the calculation for average settlement.

Table 8. Summary of Analysis for Tank 3

Length = 36 ft.; Ar=10.5%				Improved		Original	
Soil Profile	Exploration Depth	Orig. Liq Sett.	Imp. Liq Sett.	Rapid Sett.	Long Term Sett.	Rapid Sett.	Long Term Sett.
HB-CPT-131	53.6-ft	3.76-in.	1.77-in.	1.09-in.	0.08-in.	0.42-in.	1.37-in.
HB-CPT-135	53.0-ft	3.75-in.	2.16-in.	0.99-in.	0.11-in.	.54-in.	1.25-in.
X-CPT-1	54.0-ft	4.18-in.	1.42-in.	0.82-in.	0.1-in.	0.52-in.	1.05-in.
HB-CPT-133	53.0-ft	3.41-in.	1.51-in.	1.24-in.	0.04-in.	0.35-in.	1.61-in.
HB-CPT-132	53.8-ft	4.01-in.	2.19-in.	2.94-in.	0.1-in.	0.83-in.	3.86-in.
Average*:		**3.82-in.**	**1.81-in.**	**1.42-in.**	**0.09-in.**	**0.53-in.**	**1.83-in.**
HB-CPT-134(s)	53.5-ft	2.6-in.	1.07-in.	1.14-in.	0.02-in.	0.84-in.	1.39-in.

*Because of the project acceptance criteria the settlement at the center of the tank (HB-CPT-134s) is not included in the calculation for average settlement.

Table 9. Summary of Analysis for Tank 4

Length = 35 ft.; Ar=10.5%				Improved		Original	
Soil Profile	Exploration Depth	Orig. Liq Sett.	Imp. Liq Sett.	Rapid Sett.	Long Term Sett.	Rapid Sett.	Long Term Sett.
HB-CPT-141	57.3-ft	2.7-in.	1.84-in.	1.82-in.	0.09-in.	0.68-in.	2.4-in.
HB-CPT-145	56.8-ft	3.07-in.	2.26-in.	1.76-in.	0.1-in.	0.76-in.	2.33-in.
HB-CPT-144	55.8-ft	2.89-in.	1.74-in.	1.9-in.	0.08-in.	0.93-in.	2.63-in.
X-CPT-4	54.3-ft	1.44-in.	.5-in.	2.41-in.	0.07-in.	0.77-in.	3.23-in.
HB-CPT-142	54.3-ft	2.22-in.	1.89-in.	3.66-in.	0.15-in.	1.6-in.	4.98-in.
Average*:		**2.46-in.**	**1.65-in.**	**2.31-in.**	**0.1-in.**	**0.95-in.**	**3.11-in.**
HB-CPT-143(s)	57.6-ft	3.38-in.	1.88-in.	3.46-in.	0.21-in.	1.7-in.	4.65-in.

*Because of the project acceptance criteria the settlement at the center of the tank (HB-CPT-143s) is not included in the calculation for average settlement.

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Table 10. Summary of Analysis for Tank 5

Length = 36 ft.; Ar=10.5%				Improved		Original	
Soil Profile	Exploration Depth	Orig. Liq Sett.	Imp. Liq Sett.	Rapid Sett.	Long Term Sett.	Rapid Sett.	Long Term Sett.
HB-CPT-151	55.5-ft	4.03-in.	2.77-in.	1.39-in.	0.08-in.	0.8-in.	1.94-in.
HB-CPT-154	52.4-ft	3.05-in.	1.38-in.	.67-in.	0.07-in.	0.39-in.	.85-in.
HB-CPT-156	47.2-ft	1.9-in.	0.5-in.	1.43-in.	0.05-in.	0.49-in.	1.9-in.
HB-CPT-155	56.1-ft	2.2-in.	1.05-in.	1.75-in.	0.02-in.	0.4-in.	2.33-in.
HB-CPT-152	55.9-ft	2.78-in.	0.99-in.	2.47-in.	0.05-in.	0.85-in.	3.36-in.
Average:		**2.79-in.**	**1.34-in.**	**1.54-in.**	**.05-in.**	**.59-in.**	**2.08-in.**
HB-CPT-153(s)	12.8-ft	--	--	--	--	--	--

Table 11. Summary of Analysis for Tank 6

Length = 38 ft. Ar-10.5%				Improved		Original	
Soil Profile	Exploration Depth	Orig. Liq Sett.	Imp. Liq Sett.	Rapid Sett.	Long Term Sett.	Rapid Sett.	Long Term Sett.
HB-CPT-161	54.3-ft	5.14-in.	2.1-in.	1.43-in.	0.07-in.	1.07-in.	2.13-in.
HB-CPT-165	54.8-ft	4.52-in.	2.43-in.	1.26-in.	0.08-in.	0.85-in.	1.79-in.
HB-CPT-164	53.5-ft	4.55-in.	2.33-in.	.77-in.	0.08-in.	0.77-in.	1.14-in.
HB-CPT-166	55.5-ft	2.58-in.	0.76-in.	1.98-in.	0.02-in.	0.45-in.	2.65-in.
HB-CPT-162	54.6-ft	4.09-in.	1.94-in.	1.59-in.	0.03-in.	0.39-in.	2.11-in.
Average*:		**4.1-in.**	**1.92-in.**	**1.41-in.**	**.05-in.**	**.71-in.**	**1.97-in.**
HB-CPT-163(s)	53.8-ft	3.75-in.	1.95-in.	0.65-in.	0.09-in.	0.39-in.	.8-in.

*Because of the project acceptance criteria the settlement at the center of the tank (HB-CPT-163s) is not included in the calculation for average settlement.

Table 12. Summary of Analysis of Area 300 pipeline Pipeline

Length Varies; Ar=12%					Improved		Original	
Soil Profile	Treatment Length	Depth	Orig Liq Sett.	Imp. Liq Sett.	Rapid Sett.	Long Term Sett.	Rapid Sett.	Long Term Sett.
HB-CPT-171	25	54.3-ft	1.23-in.	1.34-in.	1.39-in.	0.24-in.	1.94-in.	0.94-in.
HB-CPT-172	47	58.9-ft	3.91-in.	2.13-in.*	0.24-in.	0.06-in.	0.69-in.	0.47-in.
HB-CPT-173	45	57.6-ft	3.63-in.	1.89-in.	1.13-in.	0.08-in.	1.84-in.	0.92-in.
HB-CPT-174	34	56.4-ft	2.23-in.	1.98-in.	0.85-in.	0.09-in.	1.14-in.	0.4-in.
HB-CPT-175	46	55.1-ft	3.63-in.	1.9-in.	0.91-in.	0.1-in.	1.1-in.	0.74-in.
HB-CPT-176	37	55.0-ft	3.17-in.	1.78-in.	1.1-in.	0.05-in.	0.68-in.	1.45-in.
HB-CPT-177	44	54.3-ft	4.45-in.	1.93-in.	0.23-in.	0.02-in.	0.42-in.	0.41-in.

*Actual settlement is expected to be less due to support and reinforcement offered by surrounding stronger soils.

5.2 Area 400

HBI's analysis indicates that the combination of stone column, jet grout, and deep soil mixing is adequate to achieve the required static total and differential as well as seismic total and differential performance objectives. This conclusion is based on the results of our calculations as summarized in Tables 13 through 16 (below).

Table 13. Pre-Improvement Liquefaction-Induced Settlement (LQ-Induced Settlement)

Soil Profile	Top of LQ Layer [ft]	Bot of LQ Layer [ft]	LQ-Layer Thickness [ft]	LQ-Induced Settlement [inches]
HB-CPT-191	19	80	61	>12
HB-CPT-192	22	80	58	>12
HB-CPT-193	20	80	60	>11
HB-CPT-194	21	80	59	>12
HB-CPT-195	20	80	60	>12
HB-CPT-196	23	80	57	>11

Table 14: Post-Improvement Liquefaction-Induced Settlement (LQ-Induced Settlement)

CPT [-]	Top of LQ Layer [ft]	Bot of LQ Layer [ft]	LQ-Layer Thickness [ft]	LQ-Induced Settlement [inches]
HB-CPT-191	20	25	5	< 1.0
HB-CPT-192	NA	NA	NA	< 1.0
HB-CPT-193	62	65	3	< 1.0
HB-CPT-194	NA	NA	NA	< 1.0
HB-CPT-195	61	62	1	< 1.0
HB-CPT-196	34	36	2	< 1.0

Table 15: Summary of Lateral Spreading With and Without Ground Improvement

Section	Parameter	Condition	Unimproved	With Ground Improvement
Pipe-Rack Foundations	ky	Static	0.00	NA
		Pseudo-Static	< 0.05	0.140
	Disp (in)	Static	0.00	< 1.0
		Pseudo-Static	Not Acceptable	1.924
Dock Abutment	Ky	Static	0.00	NA
		Pseudo-Static	< 0.05	0.185
	Disp (in)	Static	0.00	< 1.0
		Pseudo-Static	Not Acceptable	1.016

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Table 16: Geotechnical DSM-JG Column Capacity and Settlement

Dia [ft]	Column Length [ft]	Loading Condition [-]	Factor of Safety [-]	$Q_{a\_compression}$ [kip]	Anticipated Settlement [in]
6	75	Static	3	391	<1
		Seismic+Static	1.5	103	<2

6.0 DEMONSTRATION TEST SECTIONS

6.1 Area 300

A sufficient number of demonstration test sections shall be constructed at the project site to confirm the proposed design is sufficient to satisfy the project acceptance criteria. Each test section shall include no less than 9 stone columns. The test section shall have the same stone column dimensions (diameter and spacing) as is anticipated for the production work and be constructed with a stone of the gradation to be used for production work. A typical stone gradation is provided in Table 14 (below).

At least 7 days following the stone column installation, two (2) post-improvement CPTs shall be conducted. One of the post-improvement CPTs (CPT-A) shall be located equidistant from the four closest columns and one CPT (CPT-B) shall be half way between CPT-A and a stone column within the test section.

Both post-improvement CPT logs will be reviewed and compared with the closest pre-treatment CPT. Based on HBI's evaluation of the completed demonstration test(s), HBI may alter the proposed ground improvement program.

A typical gradation for stone used for construction of stone columns is provided in Table 17 (below).

Table 17. Typical stone gradation used for Stone Columns

Sieve/Diameter	Percent Passing
3"	100
2.5"	100
2.0"	80 – 100
0.75	30 – 60
0.5	5 – 15
No. 4	0 – 3
No. 200	0 - 1

6.2 Area 400

6.2.1 Jet Grout Columns

Within Area 400, at minimum one test section will be conducted to verify that the proposed equipment, procedures, and construction methodology can achieve the design geometry and soilcrete strength. The test section will include at least three Jet Grout columns. At least four wet samples will be collected. Each of these samples will be collected at a different depth. These samples will be placed (cast) in cylinders for

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testing. At minimum one set of three dry samples taken through the process of core drilling will be obtained and tested. These dry and wet samples will be tested in unconfined compression to see if they meet the target design strength (an average 200 psi with no more than 10% less than 75 psi). In addition the methods used to check jet grout column diameter will consist of some/all of the following:

- Excavation of portions of the completed columns,
- Exploratory core drilling through the extents of the column,
- Or other methods approved by the ground improvement engineer.

6.2.2 Deep Soil Mixed Column

At minimum, one test section shall be conducted to verify that the proposed equipment, procedures, and mix design can uniformly mix the onsite soils and achieve the design strength. The test section shall include at least 3 fully mixed 6 foot diameter columns to the design depth. Laboratory testing will be used to identify initial mix designs used in the field program. Bulk soil samples from the site will be obtained by HBI. A suite of three mix designs is required. At minimum one full-depth core sample from the test elements shall be performed. At minimum three sets of 4 wet samples each collected at random depths. These samples shall be placed (cast) in cylinders for testing. At minimum one set of three dry samples taken from the core will be tested. These dry and wet samples will be tested in unconfined compression to see if they meet the target design strength (an average 200 psi with no less than 10% of breaks below 75 psi)

7.0 VERIFICATION AND TESTING

7.1 Stone Column Verification Tests

Within Area 300 and Area 400, following installation of the stone columns, verification testing will be performed using a series of CPT soundings. If necessary, the CPT testing may be complemented by geotechnical drilled borings and SPT testing. Production work within the proposed tank areas shall be evaluated by CPT methods at a frequency of four CPTs per tank. Production work within pipeline areas shall be evaluated using one CPT per 15,000 square feet of ground improvement area. The CPT soundings shall extend at least three feet below the bottom of the immediately adjacent stone columns. CPT testing should be performed at least 7 days after stone column installation.

The CPT data will be analyzed by the ground improvement design engineer to evaluate achievement of HBI's settlement design criteria (see Section 2 of this report). HBI's Engineer shall use analysis methods in general agreement with those described in Section 3 of this report. In the event that the required settlement criteria is not met, HBI shall perform additional exploration, laboratory testing and analysis, as necessary, to determine the extent of the area of non-compliance and prepare a supplemental ground improvement program to achieve the required settlement criteria. If such supplemental ground improvement is necessary, the affected area shall again be verification tested using CPT soundings as described above except that at least 2 CPT soundings will be placed within the affected zone.

7.2 Tank First Filling and Hydro-Testing

Due to the presence of sensitive fine grained soils, HBI understands that Tesoro Savage Petroleum Terminal LLC plans to hydro-test each tank by slowly filling each tank with water over the period of one month. The preliminary plan is to fill each tank to the 25% level and maintain that loading for 1 week, followed by filling another 25% and maintaining the load for another week. This process will continue until the tanks are completely full and the full load has been maintained for 1 week. The total time required for the hydro test is expected to be approximately one month.

HBI anticipates that the ground improvement system will accelerate drainage of the fine grained soils. This

accelerated rate of consolidation could be used to increase the effectiveness of the currently planned hydro-test. If temporary piezometers are installed below the tanks, a minor modification to the currently planned first filling schedule is possible. The loading schedule provided in Table 18 uses the same test duration, but provides for longer hold times at the higher loads. This will result in better long term performance of the tanks.

Table 18. Possible First Filling Schedule

% of Full	Anticipated Hold	Settlement monitoring frequency
25	2 days	Every 4 hours
50	4 days	Every 12 hours
75	6 days	Daily
100	18 days	Days 1-6 Daily Days 7-12 every 2 days
Note: VWP readings shall be continuously available to Hayward Baker during first filling and hydro-testing.		

HBI recommends that settlement monitoring points shall be established and the tanks be filled as recommended by API 650 and API 653.

One of the benefits of hydro-testing each tank with water and holding the largest load for more than seven days is the over-consolidation of the fine grained soils and reduction of much of the anticipated secondary compression.

8.0 SCOPE OF WORK

The Design Concept Drawings in Appendix F show the layout for the stone column ground improvement plan. As described herein, the ground improvement program will mitigate both static and liquefaction induced settlement concerns. The stone columns will be installed from a temporary working grade which varies from tank to tank and ranges from Elevation 22.6 ft. to Elevation 24.9 ft. The working grade is about 4 ft. below the grade that existed at the time of our exploratory program.

Confirmation of the densification achieved by the stone column program will be evaluated by performing a series of CPTs after the stone column construction.

Following stone column installation the top 2 ft of the working surface should be replaced with granular fill and compacted by others to at least 95% relative compaction according to ASTM D1557. Then the site will be filled to the appropriate elevation and the tank load transfer platform will be constructed by others.

A brief description of Stone Column installation techniques are as follows:

> **Vibro-Stone Columns:** This ground improvement technique uses specialty purpose-built vibrating probes to densify and reinforce the soils while constructing a stone column of approximately 0.9m (3 ft.) diameter. Please refer to the drawings in Appendix G for layout and spacing.
>
> The installation process imparts energy by means of vibrations produced by a rotating eccentric weight mounted on a shaft near the tip of the probe. The vibrator is suspended from a crane and lowered into the ground under its own weight, vibrations, and air jetting. Upon reaching the design depth, the vibrator is lifted in stages as the stone is fed through a side pipe and expelled at the tip of the vibrator. Cohesionless soils are densified while cohesive soils are reinforced by the

installation of the stone column. For this project, pre-auguring through the fine grained soils is necessary to ensure the installation process does not damage sensitive fine grained soils.

Installation of stone columns by the dry bottom feed method displaces the ground that was not removed by pre-auguring. Verification of improvement is undertaken by HBI using cone penetration testing after stone column installation. Acceptance of the work will be in accordance with the acceptance criteria outlined in this document.

DSM Columns

Deep Soil Mixing is a ground improvement technique that improves the characteristics of weak soils by mechanically mixing them with cementitious binder slurry. To construct columns, a powerful drill advances drill steel with radial mixing paddles located near the bottom of the drill string. The binder slurry is pumped through the drill steel to the tool as it advances and additional soil mixing is achieved as the tool is withdrawn. The process constructs individual soilcrete columns, rows of overlapping columns or 100% mass stabilization, all with a designed strength and stiffness. The technique has been used to increase bearing capacity, decrease settlement, increase global stability, and mitigate liquefaction potential for planned structures, tanks, embankments and levees.

Wet soil mixing has also been used to construct in situ gravity retaining structures, and to facilitate tunnel construction or remediate the impact tunneling may have on nearby structures. Soil stabilization by wet soil mixing can provide structural support and/or it can greatly reduce lateral loads on bulkhead walls. Mixing shaft speed, penetration rate, batching, and pumping operations are typically adjusted after constructing one or more test columns in a convenient area on site. Pre-production laboratory testing is used to prescribe mix methodology, energy, and the grout slurry system.

HBI has developed proprietary special equipment and software for the real-time monitoring of all mixing parameters during the wet soil mixing process (DAQ). Verification of improvement is undertaken by HBI using DAQ, wet-sampling, and coring.

Jet Grout:

Jet grouting is a grouting technique that creates in situ geometries of soilcrete (grouted soil), using a grouting monitor attached to the end of a drill stem. The jet grout monitor is advanced to the maximum treatment depth, at which time high velocity grout jets (and sometimes water and air) are initiated from ports in the side of the monitor. The jets erode and mix the in situ soil as the drill stem and jet grout monitor are rotated and raised. The jet grouting process constructs soilcrete full columns designed strength and geometry. Since the geometry and physical properties of the soilcrete are engineered, the properties of the soilcrete are readily and accurately predictable.

Jet grouting's ability to construct soilcrete in confined spaces and around subsurface obstructions such as utilities, provides a unique degree of design flexibility. Verification of improvement is undertaken by HBI using DAQ, wet-sampling, and/or coring.

Jet grouting produces spoils which require proper field management/control and proper disposal. Field management and control of jet grout spoils to typically the responsibility of the project general contractor and is usually accomplished using a combination of ditches, collection/settling/curing ponds. In the state of Washington off-site disposal of jet grout spoils is typically necessary.

DRAFT

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NOTES REGARDING THE APPENDICES:

1. *The appendices are organized by the proposed structure (i.e. Tanks 1 through 6 and the pipe line).*
2. *To facilitate ease of use each appendix has a blue colored cover sheet and the sections within the appendix are separated by yellow colored sheets.*
3. *In general, the colored sheets serve the purpose of tabs in a three ring binder.*

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Appendix A

CPT and SPT Calibration Information

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Appendix A
SPT and CPT calibration information

SPT calibration information

Robert Miner Dynamic Testing, Inc.

Dynamic Measurements and Analyses for Deep Foundations

December 23, 2014

Mr. Larry Gregory
Gregory Drilling
17609 NE 70th St
Redmond, WA 98052

Re: Penetration Test Energy Measurements
Bore Hole: E011, December 22, 2014
Truck Mounted CME Rig No. 304, 140lb ram, NW-J Rod
University of Washington East Campus, Seattle, Washington
RMDT Job No. 14F51

Dear Mr. Gregory,

This letter presents energy transfer measurements made during Standard Penetration Tests for the drill hole and drill rig referenced above. Robert Miner Dynamic Testing, Inc. (RMDT) made dynamic measurements with a Pile Driving Analyzer® as a hammer advanced the NW rod during sampling with a split spoon sampler.

The purpose of RMDT's testing was the measurement of energy transferred to the drill rods. Measurements were made on a section of NW gauge rod at the top of the drill rod. Strain gages and accelerometers on the rod were connected to a Pile Driving Analyzer® (PDA) which generally processed acceleration and strain measurements from each hammer blow and stored both the measurements and computed results. Measurements and data processing generally followed the ASTM D 4633-10 standard. Energy transfer past the gage location, EFV, was computed by the PDA using force and velocity records as follows:

$$EFV = \int_a^b F(t)\ v(t)\ dt$$

The value "a" corresponds to the start of the record which is when the energy transfer begins and *"b"* is the time at which energy transferred to the rod reaches a maximum value. Appendix A contains more information on our measurement equipment and methods of analysis. The EFV energy calculation is identical to the EMX energy result discussed in Appendix A. The EFV and EMX values apply to the sensor location near the top of the rod.

Mailing Address: P.O. Box 340, Manchester, WA, 98353, USA **Phone:** 360-871-5480
Location: 2288 Colchester Dr. E., Ste A, Manchester, WA, 98353 **Fax:** 360-871-5483

TEST DETAILS

Testing occurred on December 22, 2014. Boring E011 was advanced in between Loew Hall and The Mechanical Engineering Building off East Stevens Way NE on the University of Washington Campus in Seattle, Washington. NW size rod was used to advance a standard split spoon sampler. The automatic hammer in use during our testing was manufactured by Central Mine Equipment (CME) and was reported to use a 140 lb ram. The drill rig was a truck-mounted CME-75 referred to as Rig No. 304 (LN:B01136D) by the operator.

RESULTS

A summary of testing and monitoring results is given in Table 1. The tabulated results include the starting sample depth, the penetration resistance, the number of hammers blows in our data set, measured energy transfer, EFV, the computed transfer efficiency, ETR, and the hammer blow rate, BPM. Appendix B contains detailed numeric results for each individual test.

Energy measurements must be divided by the theoretical free fall energy of the hammer to obtain an efficiency. A 140 lb ram raised 30 inches above an impact surface has 350 lb-ft of potential energy. Thus, the transfer energy results for sampling with the 140 lb ram may be divided by 350 lb-ft to yield the ratio of the delivered energy to the nominal potential energy. This efficiency ratio, ETR, is given for each sample interval as a percent efficiency.

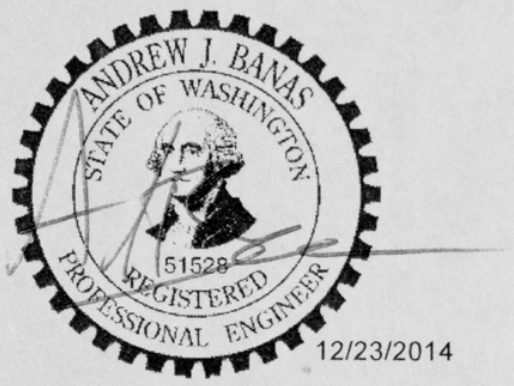
Table 1. Summary of Test Details and Results for 140 LB Ram (Split Spoon)

Sample Name	Penetration Resistance (Blow/Set)	Number of Blows in Data Set	Average Transfer Energy EFV (lb-ft)	Average Transfer Efficiency ETR (percent)	Average Hammer Blow Rate BPM (blow/min)
25 ft Sample	50/1 in	50	282	81	56
35 ft Sample	50/4 in	50	281	80	55
50 ft Sample	50/5 in	50	270	77	54
65 ft Sample	50/6 in	50	287	82	53
70 ft Sample	50/5 in	50	275	79	55
Average for Split Spoon Samples:			**279**	**80**	**55**

Five sample returns were monitored while the 140 lb ram and standard split spoon sampler were in use. The overall average ETR and hammer blow rate was 80 percent and 55 blows per minute, respectively.

It was a pleasure to assist you and to participate on this project with the staff of Gregory Drilling. Please do not hesitate to contact us if you or your client have any questions about this report.

Sincerely,



Andrew Banas, P.E.
Staff Engineer
Robert Miner Dynamic Testing, Inc.

APPENDIX A
AN INTRODUCTION INTO DYNAMIC PILE TESTING METHODS

The following has been written by Goble Rausche L kins and Associates, Inc. and may only be copied with its written permission.

BACKGROUND

Modern procedures of design and construction control require verification of bearing capacity and integrity of deep foundations during preconstruction test programs and also production installation. Dynamic pile testing methods meet this need economically and reliably, and therefore, form an important part of a quality assurance program when deep foundations are executed. Several dynamic pile testing methods exist; they have different benefits and limitations and different requirements for proper execution.

The Case Method of dynamic pile testing, named after the Case Institute of Technology where it was developed between 1964 and 1975, requires that a substantial ram mass (such as that of a pile driving hammer) impacts the pile top such that the pile undergoes at least a small permanent set. The method is therefore also referred to as a "High Strain Method". The Case Method requires dynamic measurements on the pile or shaft under the ram impact and then an evaluation of various quantities based on closed form solutions of the wave equation, a partial differential equation describing the motion of a rod under the effect of an impact. Conveniently, measurements and analyses are done by a single piece of equipment: the Pile Driving Analyzer® (PDA). However, for bearing capacity evaluations an important additional method is CAPWAP® which performs a much more rigorous analysis of the dynamic records than the simpler Case Method.

A related analysis method is the "Wave Equation Analysis" which calculates a relationship between bearing capacity and pile stress and field blow count. The GRLWEAP™ program performs this analysis and provides a complete set of helpful information and input data.

The following description deals primarily with the Case Method or "High Strain Test" Method of pile testing, however, for the sake of completeness, the "Low Strain Test" performed with the Pile Integrity Test™ (PIT), mainly for pile integrity evaluation, will also be described.

RESULTS FROM DYNAMIC TESTING

There are two main objectives of high strain dynamic pile testing:

- *Dynamic Pile Monitoring* and
- *Dynamic Load Testing*.

Dynamic pile monitoring is conducted during the installation of impact driven piles to achieve a safe and economical pile installation. Dynamic load testing, on the other hand, has as its primary goal the assessment of pile bearing capacity. It is applicable to both cast *insitu* piles or drilled shafts and impact driven piles during restrike.

Dynamic Pile Monitoring

During pile installation, the sensors attached to the pile measure pile top force and velocity. A PDA conditions and processes these signals and calculates or evaluates:

- Bearing capacity at the time of testing, including an assessment of shaft resistance development and driving resistance. This information supports formulation of a driving criterion.

- Dynamic pile stresses, axial and averaged over the pile cross section, both tensile and compressive, during pile driving to limit the potential of damage either near the pile top or along its length. Bending stresses can be evaluated at the point of sensor attachment.

- Pile integrity assessment by the PDA is based on the recognition of certain wave reflections from along the pile. If detected early enough, a pile may be saved from complete destruction. On the other hand, once damage is recognized measures can be taken to prevent reoccurrence.

- Hammer performance parameters including the energy transferred to the pile, the hammer speed in blows per minute and the stroke of open ended diesel hammers.

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Dynamic Pile Load Testing

Bearing capacity testing of either driven piles or drilled shafts applies the same basic measurement approach of dynamic pile monitoring. However, the test is done independent of the pile installation process and therefore a pile driving hammer or other dynamic loading device may not be available. If a special ram has to be mobilized then its weight should be between 0.8 and 2% of the test load (e.g. between 4 and 10 tons for a 500 ton test load) to assure sufficient soil resistance activation.

For a successful test, it most important that the test is conducted after a sufficient waiting time following pile installation for soil properties approaching their long term condition or concrete to properly set. During testing, PDA results of pile/shaft stresses and transferred energy are used to maintain safe stresses and assure sufficient resistance activation. For safe and sufficient testing of drilled shafts, ram energies are often increased from blow to blow until the test capacity has been activated. On the other hand, restrike tests on driven piles may require a warm hammer so that the very first blow produces a complete resistance activation. Data must be evaluated by CAPWAP for bearing capacity.

After the dynamic load test has been conducted with sufficient energy and safe stresses, the CAPWAP analysis provides the following results:

- Bearing capacity i.e. the mobilized capacity present at the time of testing
- Resistance distribution including shaft resistance and end bearing components
- Stresses in pile or shaft calculated for both the static load application and the dynamic test. These stresses are averages over the cross section and do not include bending effects or nonuniform contact stresses, e.g. when the pile toe is on uneven rock.
- Shaft impedance vs depth; this is an estimate of the shaft shape if it differs substantially from the planned profile
- Dynamic soil parameters for shaft and toe, i.e. damping factors and quakes (related to the dynamic stiffness of the resistance at the pile/soil interface.)

MEASUREMENTS

PDA

The basis for the results calculated by the PDA are pile top strain and acceleration measurements which are converted to force and velocity records, respectively. The PDA conditions, calibrates and displays these signals and immediately computes average pile force and velocity thereby eliminating bending effects. Using closed form Case Method solutions, based on the one-dimensional linear wave equation, the PDA calculates the results described in the analytical solutions section below.

HPA

The ram velocity may be directly obtained using radar technology in the Hammer Performance Analyzer™. For this unit to be applicable, the ram must be visible. The impact velocity results can be automatically processed with a PC or recorded on a strip chart.

Saximeter™

For open end diesel hammers, the time between two impacts indicates the magnitude of the ram fall height or stroke. This information is not only measured and calculated by the PDA but also by the convenient, hand-held Saximeter.

PIT

The Pile Integrity Tester™ (PIT) can be used to evaluate defects in concrete piles or shafts which may have occurred during driving or casting. Also timber piles of limited length can be tested in that manner. This so-called "Low Strain Method" or “Pulse-Echo Method” of integrity testing requires only the measurement of acceleration at the pile top. The stress wave producing impact is then generated by a small hand-held hammer and the records interpreted in the time domain. PIT also supports the so-called “Transient Response Method” which requires the additional measurement of the hammer force and an analysis in the frequency domain. This method may also be used to evaluate the unknown length of deep foundations under existing structures.

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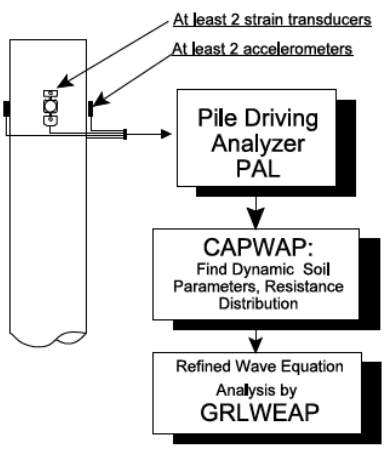
ANALYTICAL SOLUTIONS
BEARING CAPACITY

Wave Equation

GRL has written the GRLWEAP™ program which calculates a relationship between bearing capacity, pile stress and blow count. This relationship is often called the "bearing graph." Once the blow count is known from pile installation logs, the bearing graph yields the bearing capacity. This approach requires no measurements and therefore can be performed during the design stage of a project, for example for the selection of hammer, cushion and pile size.

After dynamic pile monitoring and/or dynamic load testing has been performed, the "Refined Wave Equation Analysis" or RWEA (see schematic below) is often performed by inputting the PDA and CAPWAP calculated parameters. Then the bearing graph from the RWEA is the basis for a safe and sufficient driving criteria.



Case Method

The Case Method is a closed form solution based on a few simplifying assumptions such as ideal plastic soil behavior and an ideally elastic and uniform pile. Given the measured pile top force F(t) and pile top velocity v(t), the total soil resistance is

$$R(t) = \tfrac{1}{2}\{[F(t) + F(t_2)] + Z[v(t) - v(t_2)]\} \qquad (1)$$

where

t = a point in time after impact
t_2 = time $t + 2L/c$
L = pile length below gages
c = $(E/\rho)^{1/2}$ is the speed of the stress wave
ρ = pile mass density
Z = EA/c is the pile impedance
E = elastic modulus of the pile ($\rho\, c^2$)
A = pile cross sectional area

The total soil resistance consists of a dynamic (R_d) and a static (R_s) component. The static component is therefore

$$R_s(t) = R(t) - R_d(t) \qquad (2)$$

The dynamic component may be computed from a soil damping factor, J, and a pile toe velocity, $v_t(t)$ which is conveniently calculated for the pile toe. Using wave considerations, this approach leads immediately to the dynamic resistance

$$R_d(t) = J[F(t) + Zv(t) - R(t)] \qquad (3)$$

and finally to the static resistance by means of Equation 2.

There are a number of ways in which Eq. 1 through 3 can be evaluated. Most commonly, t_2 is set to that time at which the static resistance becomes maximum. The result is the so-called **RMX** capacity. Damping factors for RMX typically range between 0.5 for coarse grained materials to 1.0 for clays. The **RSP** capacity (this method is most commonly referred to in the literature, yet it is not very frequently used) requires damping factors between 0.1 for sand and 1.0 for clay. Another capacity, **RA2**, determines the capacity at a time when the pile is essentially at rest and thus damping is small; RA2

therefore requires no damping parameter. In any event, the proper Case Method and its associated damping parameter is most conveniently found after a CAPWAP analysis has been performed.

The static resistance calculated by Case Method or CAPWAP is the mobilized resistance at the time of testing. Consideration therefore has to be given to soil setup or relaxation effects and whether or not a sufficient set has been achieved under the test loading that would correspond to a full activation of the ultimate soil resistance.

The PDA also calculates an estimate of shaft resistance as the difference between force and velocity times impedance at the time immediately prior to the return of the stress wave from the pile toe. This shaft resistance is not reduced by damping effects and is therefore called the total shaft resistance **SFT**. A correction for damping effects produces the static shaft resistance estimate, **SFR**.

The Case Method solution is simple enough to be evaluated "in real time," i.e. between hammer blows, using the PDA. It is therefore possible to calculate all relevant results for all hammer blows and plot these results as a function of depth or blow number. This is done in the PDAPLOT program.

CAPWAP

The CAse Pile Wave Analysis Program combines the wave equation pile and soil model with the Case Method measurements. Thus, the solution includes not only the total and static bearing capacity values but also the shaft resistance, end bearing, damping factors and soil stiffnesses. The method iteratively calculates a number of unknowns by signal matching. While it is necessary to make hammer performance assumptions for a GRLWEAP analysis, the CAPWAP program works with the pile top measurements. Furthermore, while GRLWEAP and Case Method require certain assumptions regarding the soil behavior, CAPWAP calculates these soil parameters.

STRESSES

During pile monitoring, it is important that compressive stress maxima at pile top and toe and tensile stress maxima somewhere along the pile be calculated for each hammer blow.

At the pile top (location of sensors) both the maximum compression stress, **CSX**, and the maximum stress from individual strain transducers, **CSI**, are directly obtained from the measurements. Note that CSI is greater than or equal to CSX and the difference between CSI and CSX is a measure of bending in the plane of the strain transducers. Note also that all stresses calculated for locations below the sensors are averaged over the pile cross section and therefore do not include components from either bending or eccentric soil resistance effects.

The PDA calculates the compressive stress at the pile bottom, **CSB**, assuming (a) a uniform pile and (b) that the pile toe force is the maximum value of the total resistance R(t) minus the total shaft resistance, SFT. Again, for this stress estimation uniform resistance force are assumed (e.g. not a sloping rock.)

For concrete piles, the maximum tension stress, **TSX**, is also of great importance. It occurs at some point below the pile top. The maximum tension stress can be computed from the pile top measurements by finding the maximum tension wave (either traveling upward, W_U, or downward, W_d) and reducing it by the minimum compressive wave traveling in opposite direction.

$$W_u = \tfrac{1}{2}[F(t) - Zv(t)] \quad (4)$$

$$W_d = \tfrac{1}{2}[F(t) + Zv(t)] \quad (5)$$

CAPWAP also calculates tensile and compressive stresses along the pile and, in general, more accurately than the PDA. In fact, for non-uniform piles or piles with joints, cracks or other discontinuities, the closed form solutions from the PDA may be in error.

PILE INTEGRITY

High Strain Tests (PDA)

Stress waves in a pile are reflected wherever the pile impedance, $Z = EA/c = \rho cA = A\sqrt{(E\rho)}$, changes. Therefore, the pile impedance is a measure of the quality of the pile material (E, ρ, c) and the size of its cross section (A). The reflected waves arrive at the pile top at a time which is greater the farther away from the pile top the reflection occurs. The

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magnitude of the change of the upward traveling wave (calculated from the measured force and velocity, Eq. 4) indicates the extent of the cross sectional change. Thus, with β_i (**BTA**) being a relative integrity factor which is unity for no impedance change and zero for the pile end, the following is calculated by the PDA.

$$\beta_i = (1 - \alpha_i)/(1 + \alpha_i) \quad (6)$$

with

$$\alpha_i = ½(W_{UR} - W_{UD})/(W_{Di} - W_{UR}) \quad (7)$$

where

W_{UR} is the upward traveling wave at the onset of the reflected wave. It is caused by resistance.

W_{UD} is the upwards traveling wave due to the damage reflection.

W_{Di} is the maximum downward traveling wave due to impact.

It can be shown that this formulation is quite accurate as long as individual reflections from different pile impedance changes have no overlapping effects on the stress wave reflections.

Without rigorous derivation, it has been proposed to consider as slight damage when β is above 0.8 and a serious damage when β is less than 0.6.

Low Strain Tests (PIT)

The pile top is struck with a held hand hammer and the resulting pile top velocity is measured, displayed and interpreted for signs of wave reflections. In general, a comparison of the reflected acceleration leads to a relative measure of extent of damage, again the location of the problem is indicated by the arrival time of the reflection. PIT records can also be interpreted by the β-Method. However, low strain tests do not activate much resistance which simplifies Eq. 7 since W_{UR} is then equal to zero.

For drilled shafts and PIT records that clearly show a toe reflection, an approximate shaft profile can be calculated from low strain records using the PITSTOP program's PROFILE routine.

HAMMER PERFORMANCE

The PDA calculates the energy transferred to the pile top from:

$$E(t) = \int_0^t F(t)v(t)\,dt \quad (8a)$$

The maximum of the E(t) curve is the most important information for an overall evaluation of the performance of a hammer and driving system. This **EMX** value allows for a classification of the hammer's performance when presented as the rated transfer efficiency, also called energy transfer ratio (**ETR**) or global efficiency

$$e_T = EMX/E_R \quad (8b)$$

where

E_R is the manufacturer's rated energy value.

Both Saximeter and PDA calculate the stroke (**STK**) of an open end diesel hammer using

$$STK = (g/8)\,T_B^2 - h_L \quad (9)$$

where

g is the earth's gravitational acceleration,
T_B is the time between two hammer blows,
h_L is a stroke loss value due to gas compression and time losses during impact (usually 0.3 ft or 0.1 m).

DETERMINATION OF WAVE SPEED

An important facet of dynamic pile testing is an assessment of pile material properties. Since in general force is determined from strain by multiplication with elastic modulus, E, and cross sectional area, A, the dynamic elastic modulus has to be determined for pile materials other than steel. In general, the records measured by the PDA clearly indicate a pile toe reflection as long as pile penetration per blow is greater than 1 mm or .04 inches. The time between the onset of the force and velocity records at impact and the onset of the reflection from the toe (usually apparent by a local maximum of the wave up curve) is the so-called wave travel time, T. Dividing 2L (L is here the length of the pile below sensors) by T leads to the stress wave speed in the pile:

$$c = 2L/T \quad (10)$$

The elastic modulus of the pile material is related to the wave speed according to the linear elastic wave equation theory by

$$E = c^2\rho \tag{11}$$

Since the mass density of the pile material, ρ, is usually well known (an exception is timber for which samples should be weighed), the elastic modulus is easily found from the wave speed. Note, however, that this is a dynamic modulus which is generally higher than the static one and that the wave speed depends to some degree on the strain level of the stress wave. For example, experience shows that the wave speed from PIT is roughly 5% higher than the wave speed observed during a high strain test.

Other Notes:

- If the pile material is nonuniform then the wave speed c, according to Eq. 10, is an average wave speed and does not necessarily reflect the pile material properties of the location where the strain sensors are attached to the pile top. For example, pile driving often causes fine tension cracks some distance below the top of concrete piles. Then the average c is slower than that at the pile top. It is therefore recommended to determine E in the beginning of pile driving and not adjust it when the average c changes.

- If the pile has such a high resistance that there is no clear indication of a toe reflection then the wave speed of the pile material must be determined either by assumption or by taking a sample of the concrete and measuring its wave speed in a simple free column test. Another possibility is to use the proportionality relationship, discussed under "DATA QUALITY CHECKS" to find c as the ratio between the measured velocity and measured strain.

DATA QUALITY CHECKS

Quality data is the first and foremost requirement for accurate dynamic testing results. It is therefore important that the measurement engineer performing PDA or PIT tests has the experience necessary to recognize measurement problems and take appropriate corrective action should problems develop. Fortunately, dynamic pile testing allows for certain data quality checks because two independent measurements are taken that have to conform to certain relationships.

Proportionality

As long as there is only a wave traveling in one direction, as is the case during impact when only a downward traveling wave exists in the pile, force and velocity measured at the pile top are proportional

$$F = v\,Z = v\,(EA/c) \tag{12a}$$

This relationship can also be expressed in terms of stress

$$\sigma = v\,(E/c) \tag{12b}$$

or strain

$$\epsilon = v / c \tag{12c}$$

This means that the early portion of strain times wave speed must be equal to the velocity unless the proportionality is affected by high friction near the pile top or by a pile cross sectional change not far below the sensors. Checking the proportionality is an excellent means of assuring meaningful measurements.

Measurements are always taken at opposite sides of the pile as a means of calculating the average force and velocity in the pile. The velocity on the two sides of the pile is very similar even when high bending exists. Thus, an independent check of the velocity measurements is easy and simple.

Strain measurements may differ greatly between the two sides of the pile when bending exists. It is even possible that tension is measured on one side while very high compression exists on the other side of the pile. In extreme cases, bending might be so high that it leads to a nonlinear stress distribution. The averaging of the two strain signals does then not lead to the average pile force and proportionality will not be achieved.

When testing drilled shafts, measurements of strain may also be affected by local concrete quality variations. It is then often necessary to use four strain transducers spaced at 90 degrees around the pile for an improved strain data quality. The use of four transducers is also recommended for large pile

diameters, particularly when it is difficult to mount the sensors at least two pile widths or diameters below the pile top.

LIMITATIONS, ADDITIONAL CONSIDERATIONS

Mobilization of capacity

Estimates of pile capacity from dynamic testing indicate the **mobilized pile capacity at the time of testing**. At very high blow counts (low set per blow), dynamic test methods tend to produce lower bound capacity estimates as not all resistance (particularly at and near the toe) is fully activated.

Time dependent soil resistance effects

Static pile capacity from dynamic method calculations provide an estimate of the axial pile capacity. Increases and decreases in the pile capacity with time typically occur (soil setup/relaxation). Therefore, **restrike testing usually yields a better indication of long term pile capacity than a test at the end of pile driving**. Often a wait period of one or two days between end of driving and restrike is satisfactory for a realistic prediction of pile capacity but this waiting time depends, among other factors, on the permeability of the soil.

(A) Soil setup

Because excess positive pore pressures often develop during pile driving in fine grained soil (clays, silts or even fine sands), the capacity of a pile at the time of driving may often be less than the long term pile capacity. These pore pressures reduce the effective stress acting on the pile thereby reducing the soil resistance to pile penetration, and thus the pile capacity at the time of driving. As these pore pressures dissipate, the soil resistance acting on the pile increases as does the axial pile capacity. This phenomena is routinely called soil setup or soil freeze.

(*B) Relaxation*

Relaxation (capacity reduction with time) has been observed for piles driven into weathered shale, and may take several days to fully develop. Pile capacity estimates based upon initial driving or short term restrike tests can significantly overpredict long term pile capacity. Therefore, piles driven into shale should be tested after a minimum one week wait either statically or dynamically (with particular emphasis than on the first few blows). Relaxation has also been observed for displacement piles driven into dense saturated silts or fine sands due to a negative pore pressure effect at the pile toe. Again, restrike tests should be used, with great emphasis on early blows.

Capacity results for open pile profiles

Larger diameter open ended pipe piles (or H-piles which do not bear on rock) may behave differently under dynamic and static loading conditions. Under dynamic loads the soil inside the pile or between its flanges may slip and produce internal friction while under static loads the plug may move with the pile, thereby creating end bearing over the full pile cross section. As a result both friction and end bearing components may be different under static and dynamic conditions.

CAPWAP Analysis Results

A portion of the soil resistance calculated on an individual soil segment in a CAPWAP analysis can usually be shifted up or down the shaft one soil segment without significantly altering the match quality. Therefore, use of the CAPWAP resistance distribution for uplift, downdrag, scour, or other geotechnical considerations should be made with an understanding of these analysis limitations.

Stresses

PDA and CAPWAP calculated stresses are average values over the cross section. Additional allowance has to be made for bending or non-uniform contact stresses. To prevent damage it is therefore important to maintain good hammer-pile alignment and to protect the pile toes using appropriate devices or an increased cross sectional area.

In the United States is has become generally acceptable to limit the dynamic installation stresses of driven piles to the following levels:

90% of yield strength for steel piles

85% of the concrete compressive strength - after subtraction of the effective prestress - for concrete piles in compression

100% of effective prestress plus ½ of the concrete's tension strength for prestressed piles in tension

70% of the reinforcement strength for regularly reinforced concrete piles in tension

300% of the static design allowable stress for timber

Note that the dynamic stresses may either be directly measured at the pile top by the PDA or calculated by the PDA for other locations along the pile based on the pile top measurements.

Additional design considerations

Numerous factors have to be considered in pile foundation design. Some of these considerations include

- additional pile loading from downdrag or negative skin friction,
- lateral and uplift loading requirements
- effective stress changes (due to changes in water table, excavations, fills or other changes in overburden),
- long term settlements in general and settlement from underlying weaker layers and/or pile group effects,

These factors have not been evaluated by GRL and have not been considered in the interpretation of the dynamic testing results. The foundation designer should determine if these or any other considerations are applicable to this project and the foundation design.

Wave equation analysis results

The results calculated by the wave equation analysis program depend on a variety of hammer, pile and soil input parameters. Although attempts have been made to base the analysis on the best available information, actual field conditions may vary and therefore stresses and blow counts may differ from the predictions reported. Capacity predictions derived from wave equation analyses should use restrike information. However, because of the uncertainties associated with restrike blow counts and restrike hammer energies, correlations of such results with static test capacities with have often displayed considerable scatter.

As for PDA and CAPWAP, the theory on which GRLWEAP is based is the one-dimensional wave equation. For that reason, stress predictions by the wave equation analysis can only be averages over the pile cross section. Thus, bending stresses or stress concentrations due to non-uniform impact or uneven soil or rock resistance are not considered in these results. Stress maxima calculated by the wave equation are usually subjected to the same limits as those measured directly or calculated from measurements by the PDA.

Appendix B

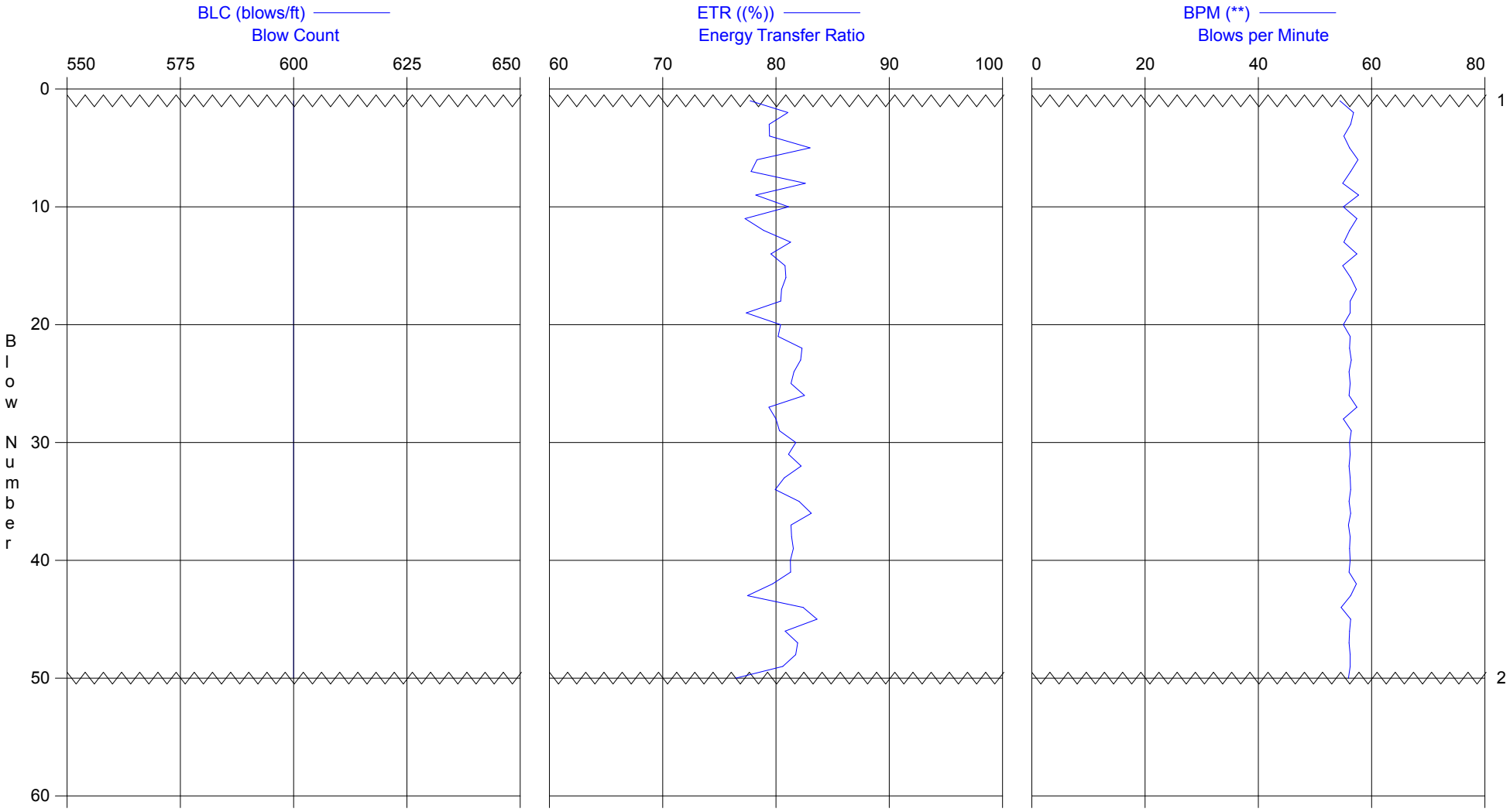
Summary of Case Method Results

Robert Miner Dynamic Testing, Inc. - Case Method & iCAP® Results

PDIPLOT Ver. 2014.1 - Printed: 23-Dec-2014

Test date: 22-Dec-2014

GREGORY DRILLNG, UW CAMPUS - 25FT SAMPLE



1 - Start of test on 12/22/2014 at 1:16:26 PM

2 - End of test on 12/22/2014 at 1:17:18 PM

EX-0001-006741-PCE

Robert Miner Dynamic Testing, Inc.
Case Method & iCAP® Results

PDIPLOT Ver. 2014.1 - Printed: 23-Dec-2014

GREGORY DRILLNG, UW CAMPUS - 25FT SAMPLE
OP: RMDT

BH E011, CME 75 RIG 304
Test date: 22-Dec-2014

AR: 1.43 in^2
LE: 29.00 ft
WS: 16,807.9 f/s

SP: 0.492 k/ft3
EM: 30,000 ksi
JC: 0.35

CSX: Max Measured Compr. Stress
BPM: Blows per Minute
EFV: Energy of FV
ETR: Energy Transfer Ratio
FMX: Maximum Force

VMX: Maximum Velocity
RAT: SPT Length Ratio
EF2: Energy of F^2
RX9: Max Case Method Capacity (JC=0.9)

BL#	depth	BLC	CSX	BPM	EFV	ETR	FMX	VMX	RAT	EF2	RX9
	ft	bl/ft	ksi	**	k-ft	(%)	kips	f/s	[]	k-ft	kips
1	25.00	600	26.9	54.4	0.272	77.7	38	13.3	1.0	0.4	14
2	25.00	600	27.6	56.8	0.284	81.1	39	13.7	1.0	0.4	15
3	25.01	600	27.5	56.3	0.278	79.4	39	13.8	1.0	0.4	16
4	25.01	600	26.9	55.1	0.278	79.4	39	14.0	1.0	0.4	18
5	25.01	600	27.9	56.1	0.291	83.0	40	14.2	1.0	0.4	16
6	25.01	600	27.0	57.6	0.274	78.3	39	14.1	1.0	0.4	16
7	25.01	600	26.8	56.3	0.272	77.8	38	13.4	1.0	0.4	17
8	25.01	600	27.8	54.9	0.289	82.6	40	13.8	1.0	0.4	20
9	25.02	600	26.6	57.7	0.274	78.2	38	13.3	1.0	0.4	20
10	25.02	600	27.6	55.0	0.284	81.1	39	13.5	1.0	0.4	19
11	25.02	600	27.1	57.4	0.270	77.3	39	13.2	1.0	0.4	20
12	25.02	600	27.2	56.1	0.276	78.9	39	13.2	1.0	0.4	22
13	25.02	600	27.8	55.1	0.285	81.3	40	13.5	1.0	0.4	22
14	25.02	600	27.3	57.4	0.278	79.5	39	13.4	1.0	0.4	22
15	25.03	600	28.0	54.9	0.283	80.8	40	13.5	1.0	0.4	22
16	25.03	600	27.9	56.3	0.283	80.9	40	13.6	1.0	0.4	23
17	25.03	600	27.5	57.3	0.282	80.5	39	13.7	1.0	0.4	21
18	25.03	600	27.7	56.2	0.281	80.4	40	13.6	1.0	0.4	23
19	25.03	600	27.3	56.2	0.271	77.4	39	13.2	1.0	0.4	22
20	25.03	600	27.5	55.0	0.281	80.4	39	13.6	1.0	0.4	24
21	25.04	600	27.9	56.2	0.281	80.2	40	13.5	1.0	0.4	25
22	25.04	600	28.3	56.1	0.288	82.3	40	13.8	1.0	0.4	23
23	25.04	600	28.0	56.4	0.288	82.2	40	13.9	1.0	0.4	25
24	25.04	600	27.9	56.0	0.286	81.6	40	13.9	1.0	0.4	26
25	25.04	600	27.7	56.2	0.285	81.3	40	13.9	1.0	0.4	25
26	25.04	600	27.7	56.0	0.289	82.5	40	14.0	1.0	0.4	26
27	25.05	600	27.2	57.4	0.278	79.4	39	13.4	1.0	0.4	26
28	25.05	600	27.4	55.0	0.280	80.0	39	13.6	1.0	0.4	26
29	25.05	600	27.3	56.4	0.281	80.3	39	13.8	1.0	0.4	26
30	25.05	600	27.4	56.1	0.286	81.8	39	14.0	1.0	0.4	27
31	25.05	600	27.7	56.2	0.284	81.1	40	14.1	1.0	0.4	25
32	25.05	600	27.7	56.0	0.288	82.2	40	14.2	1.0	0.4	27
33	25.06	600	27.6	56.2	0.283	80.7	39	14.0	1.0	0.4	28
34	25.06	600	27.5	56.3	0.280	79.9	39	13.9	1.0	0.4	27
35	25.06	600	28.0	56.0	0.287	82.0	40	14.0	1.0	0.4	27
36	25.06	600	27.9	56.3	0.291	83.1	40	14.1	1.0	0.4	26
37	25.06	600	27.6	55.9	0.285	81.3	39	13.8	1.0	0.4	26
38	25.06	600	27.5	56.2	0.285	81.4	39	14.0	1.0	0.4	26
39	25.07	600	27.5	56.1	0.285	81.5	39	14.1	1.0	0.4	27
40	25.07	600	27.7	56.2	0.284	81.3	40	14.1	1.0	0.4	27
41	25.07	600	27.4	56.0	0.285	81.3	39	14.0	1.0	0.4	27
42	25.07	600	27.0	57.3	0.279	79.7	39	13.7	1.0	0.4	26
43	25.07	600	26.8	56.3	0.271	77.5	38	13.4	1.0	0.4	25
44	25.07	600	27.3	54.6	0.288	82.4	39	14.1	1.0	0.4	27
45	25.08	600	27.9	56.3	0.293	83.6	40	14.4	1.0	0.4	27
46	25.08	600	27.8	56.1	0.283	80.8	40	14.0	1.0	0.4	28
47	25.08	600	27.5	56.0	0.287	81.9	39	14.1	1.0	0.4	26
48	25.08	600	27.6	56.2	0.286	81.7	39	14.0	1.0	0.4	27
49	25.08	600	27.4	56.2	0.282	80.6	39	13.9	1.0	0.4	27
50	25.08	600	26.8	55.9	0.268	76.4	38	13.5	1.0	0.4	25
		Average	27.5	56.1	0.282	80.6	39	13.8	1.0	0.4	24
		Std. Dev.	0.4	0.7	0.006	1.7	1	0.3	0.0	0.0	4
		Maximum	28.3	57.7	0.293	83.6	40	14.4	1.0	0.4	28

Total number of blows analyzed: 50

BL#	depth (ft)	Comments
1	25.00	Start of test on 12/22/2014 at 1:16:26 PM
50	25.08	End of test on 12/22/2014 at 1:17:18 PM

Time Summary

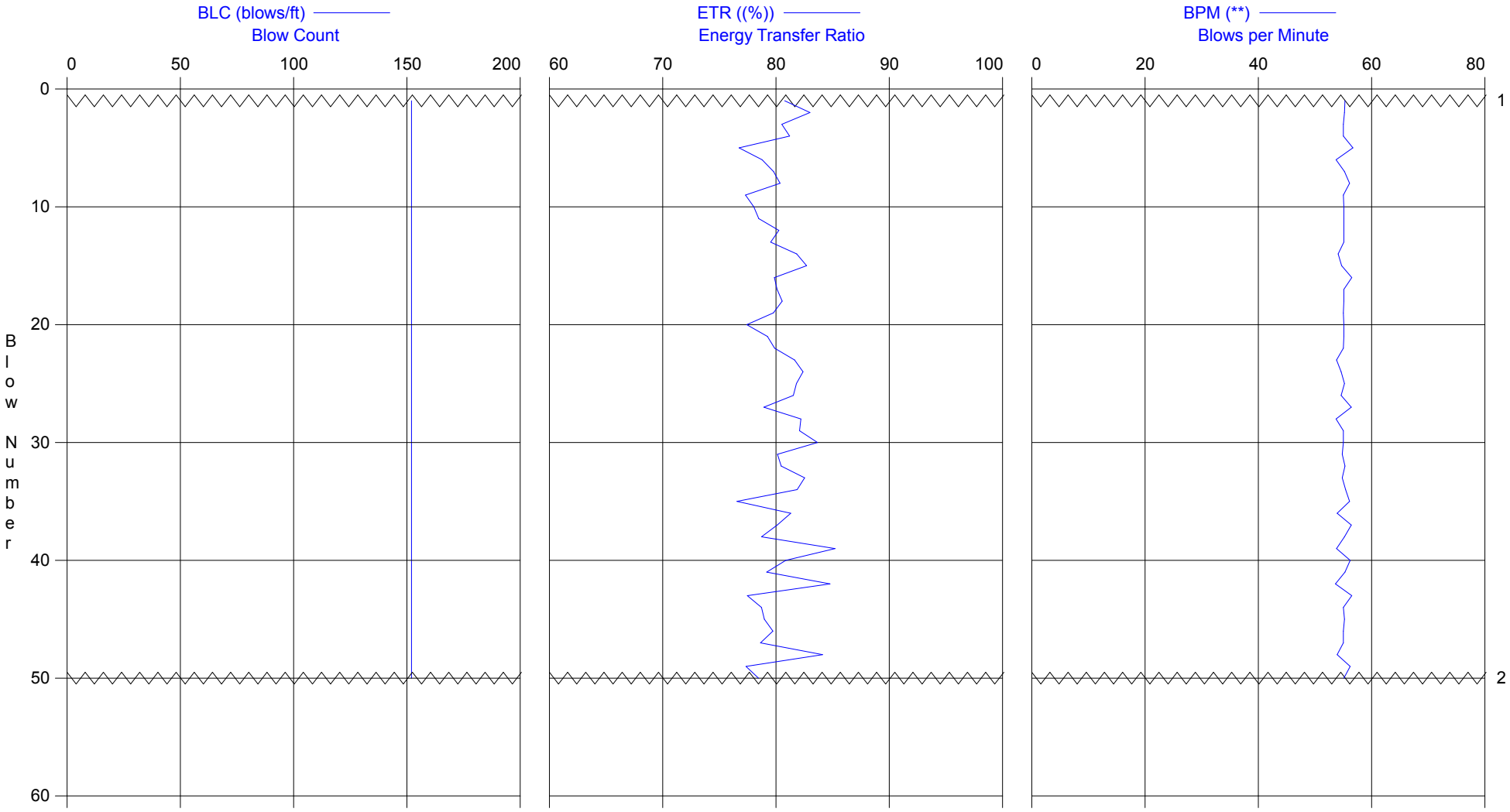
Drive	52 seconds	1:16:26 PM - 1:17:18 PM (12/22/2014) BN 1 - 50

Robert Miner Dynamic Testing, Inc. - Case Method & iCAP® Results

PDIPLOT Ver. 2014.1 - Printed: 23-Dec-2014

Test date: 22-Dec-2014

GREGORY DRILLNG, UW CAMPUS - 35FT SAMPLE



1 - Start of test on 12/22/2014 at 1:38:27 PM

2 - End of test on 12/22/2014 at 1:39:20 PM

EX-0001-006743-PCE

Robert Miner Dynamic Testing, Inc.
Case Method & iCAP® Results

PDIPLOT Ver. 2014.1 - Printed: 23-Dec-2014

GREGORY DRILLNG, UW CAMPUS - 35FT SAMPLE
OP: RMDT

BH E011, CME 75 RIG 304
Test date: 22-Dec-2014

AR: 1.43 in^2
LE: 39.00 ft
WS: 16,807.9 f/s

SP: 0.492 k/ft3
EM: 30,000 ksi
JC: 0.35

CSX: Max Measured Compr. Stress
BPM: Blows per Minute
EFV: Energy of FV
ETR: Energy Transfer Ratio
FMX: Maximum Force

VMX: Maximum Velocity
RAT: SPT Length Ratio
EF2: Energy of F^2
RX9: Max Case Method Capacity (JC=0.9)

BL#	depth	BLC	CSX	BPM	EFV	ETR	FMX	VMX	RAT	EF2	RX9
	ft	bl/ft	ksi	**	k-ft	(%)	kips	f/s	[]	k-ft	kips
1	39.01	152	26.7	55.3	0.283	80.8	38	15.1	1.0	0.4	12
2	39.01	152	26.7	55.2	0.290	83.0	38	15.4	1.0	0.4	12
3	39.02	152	26.6	55.0	0.282	80.5	38	14.8	1.0	0.4	13
4	39.03	152	26.4	55.0	0.284	81.2	38	15.2	1.0	0.4	12
5	39.03	152	26.4	56.7	0.269	76.7	38	14.6	1.0	0.3	13
6	39.04	152	26.5	53.7	0.276	78.8	38	14.7	1.0	0.4	14
7	39.05	152	27.0	55.2	0.279	79.8	39	15.3	1.0	0.4	14
8	39.05	152	26.4	56.1	0.281	80.4	38	15.1	1.0	0.4	12
9	39.06	152	26.0	55.0	0.271	77.3	37	14.7	1.0	0.3	15
10	39.07	152	26.3	55.1	0.273	78.0	38	14.9	1.0	0.4	15
11	39.07	152	25.6	55.1	0.275	78.5	37	15.0	1.0	0.4	14
12	39.08	152	26.4	55.1	0.281	80.3	38	15.0	1.0	0.4	14
13	39.09	152	26.3	55.1	0.278	79.5	38	14.9	1.0	0.4	14
14	39.09	152	26.9	54.1	0.286	81.8	38	15.2	1.0	0.4	14
15	39.10	152	27.1	54.7	0.289	82.7	39	15.3	1.0	0.4	16
16	39.11	152	26.5	56.5	0.280	79.9	38	15.1	1.0	0.4	16
17	39.11	152	26.9	55.1	0.280	80.1	38	15.2	1.0	0.4	17
18	39.12	152	26.8	55.1	0.282	80.6	38	15.2	1.0	0.4	18
19	39.13	152	27.0	55.0	0.279	79.8	39	14.9	1.0	0.4	19
20	39.13	152	26.3	55.1	0.271	77.4	38	14.7	1.0	0.4	21
21	39.14	152	26.9	55.1	0.277	79.2	39	15.0	1.0	0.4	20
22	39.15	152	26.8	55.0	0.280	79.9	38	14.9	1.0	0.4	18
23	39.15	152	27.4	53.8	0.286	81.6	39	15.3	1.0	0.4	19
24	39.16	152	27.3	54.6	0.288	82.4	39	15.2	1.0	0.4	19
25	39.17	152	27.5	55.2	0.286	81.8	39	15.2	1.0	0.4	20
26	39.17	152	26.6	54.6	0.285	81.5	38	14.9	1.0	0.4	18
27	39.18	152	25.7	56.4	0.276	78.9	37	14.4	1.0	0.3	17
28	39.18	152	27.8	53.7	0.288	82.2	40	15.4	1.0	0.4	19
29	39.19	152	27.1	55.0	0.287	82.1	39	15.2	1.0	0.4	18
30	39.20	152	27.1	55.0	0.293	83.7	39	15.2	1.0	0.4	17
31	39.20	152	26.4	54.8	0.280	80.1	38	14.8	1.0	0.4	19
32	39.21	152	26.5	55.3	0.282	80.5	38	14.7	1.0	0.4	19
33	39.22	152	26.8	54.8	0.289	82.5	38	15.0	1.0	0.4	19
34	39.22	152	26.4	55.4	0.287	81.9	38	14.8	1.0	0.4	20
35	39.23	152	25.3	56.1	0.268	76.6	36	14.6	1.0	0.3	20
36	39.24	152	26.7	53.9	0.285	81.3	38	14.9	1.0	0.4	20
37	39.24	152	26.3	56.4	0.280	80.1	38	15.2	1.0	0.4	20
38	39.25	152	26.2	55.2	0.276	78.7	37	14.9	1.0	0.4	19
39	39.26	152	27.7	53.8	0.298	85.2	40	15.6	1.0	0.4	19
40	39.26	152	26.4	56.2	0.283	80.9	38	15.4	1.0	0.4	21
41	39.27	152	26.5	55.3	0.277	79.2	38	15.0	1.0	0.4	20
42	39.28	152	27.2	53.6	0.297	84.8	39	15.9	1.0	0.4	20
43	39.28	152	26.2	56.5	0.271	77.5	37	14.8	1.0	0.3	19
44	39.29	152	26.3	55.0	0.276	78.7	38	15.0	1.0	0.4	19
45	39.30	152	26.0	55.2	0.276	79.0	37	15.1	1.0	0.3	18
46	39.30	152	26.5	55.0	0.279	79.7	38	15.1	1.0	0.4	20
47	39.31	152	26.1	55.0	0.275	78.6	37	15.1	1.0	0.3	21
48	39.32	152	27.3	53.9	0.294	84.1	39	15.7	1.0	0.4	20
49	39.32	152	26.8	56.2	0.271	77.4	38	14.8	1.0	0.3	21
50	39.33	152	26.7	55.1	0.275	78.5	38	15.1	1.0	0.4	20
		Average	26.6	55.1	0.281	80.3	38	15.0	1.0	0.4	17
		Std. Dev.	0.5	0.8	0.007	2.0	1	0.3	0.0	0.0	3
		Maximum	27.8	56.7	0.298	85.2	40	15.9	1.0	0.4	21

Total number of blows analyzed: 50

BL#	depth (ft)	Comments
1	39.01	Start of test on 12/22/2014 at 1:38:27 PM
50	39.33	End of test on 12/22/2014 at 1:39:20 PM

Time Summary

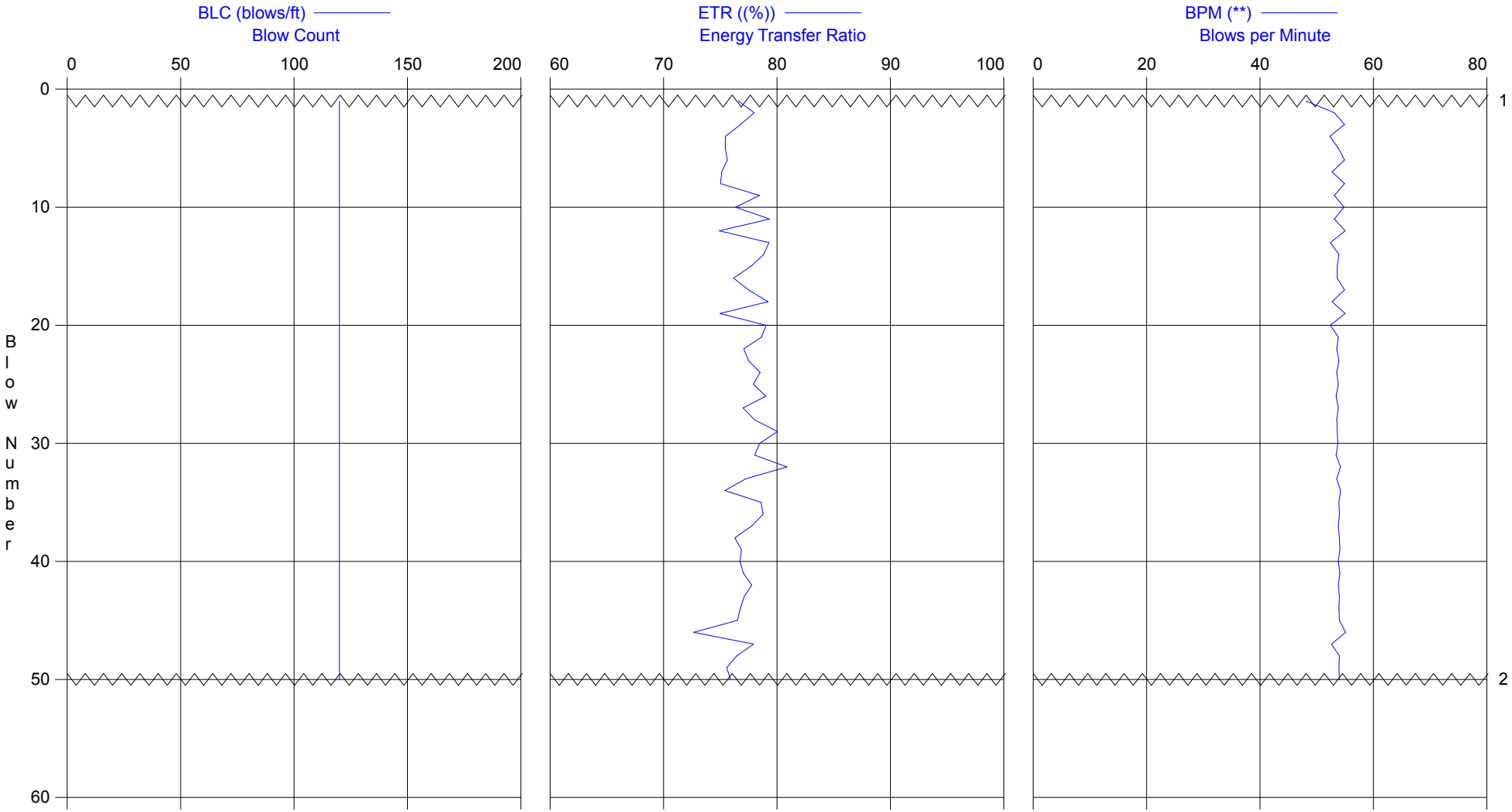
Drive	53 seconds	1:38:27 PM - 1:39:20 PM (12/22/2014) BN 1 - 50

Robert Miner Dynamic Testing, Inc. - Case Method & iCAP® Results

PDIPLOT Ver. 2014.1 - Printed: 23-Dec-2014

Test date: 22-Dec-2014

GREGORY DRILLNG, UW CAMPUS - 50FT SAMPLE



1 - Start of test on 12/22/2014 at 2:18:52 PM

2 - End of test on 12/22/2014 at 2:19:47 PM

EX-0001-006745-PCE

Robert Miner Dynamic Testing, Inc.
Case Method & iCAP® Results

PDIPLOT Ver. 2014.1 - Printed: 23-Dec-2014

GREGORY DRILLNG, UW CAMPUS - 50FT SAMPLE
OP: RMDT

BH E011, CME 75 RIG 304
Test date: 22-Dec-2014

AR: 1.43 in^2
LE: 54.00 ft
WS: 16,807.9 f/s

SP: 0.492 k/ft3
EM: 30,000 ksi
JC: 0.35

CSX: Max Measured Compr. Stress
BPM: Blows per Minute
EFV: Energy of FV
ETR: Energy Transfer Ratio
FMX: Maximum Force

VMX: Maximum Velocity
RAT: SPT Length Ratio
EF2: Energy of F^2
RX9: Max Case Method Capacity (JC=0.9)

BL#	depth	BLC	CSX	BPM	EFV	ETR	FMX	VMX	RAT	EF2	RX9
	ft	bl/ft	ksi	**	k-ft	(%)	kips	f/s	[]	k-ft	kips
1	50.01	120	26.6	48.1	0.268	76.6	38	13.4	0.8	0.4	9
2	50.02	120	26.8	53.1	0.273	78.0	38	13.6	0.7	0.4	9
3	50.03	120	26.7	54.9	0.269	76.8	38	13.7	0.7	0.4	10
4	50.03	120	26.7	52.3	0.264	75.4	38	13.6	0.7	0.3	9
5	50.04	120	26.8	53.8	0.264	75.4	38	13.3	0.7	0.3	10
6	50.05	120	26.6	54.9	0.265	75.6	38	13.2	0.7	0.3	10
7	50.06	120	26.6	52.7	0.263	75.1	38	13.1	0.7	0.3	12
8	50.07	120	26.4	54.9	0.263	75.0	38	13.0	0.7	0.3	12
9	50.08	120	26.8	53.1	0.275	78.4	38	13.3	0.7	0.4	12
10	50.08	120	26.6	54.8	0.267	76.4	38	13.1	0.7	0.4	12
11	50.09	120	26.9	53.1	0.278	79.3	38	13.6	0.7	0.4	12
12	50.10	120	26.1	55.0	0.262	74.9	37	13.2	0.7	0.3	12
13	50.11	120	26.8	52.4	0.278	79.3	38	13.6	0.7	0.4	11
14	50.12	120	26.8	53.9	0.276	78.8	38	13.5	0.7	0.4	12
15	50.13	120	27.0	53.6	0.272	77.7	39	13.5	0.7	0.4	12
16	50.13	120	26.6	53.6	0.267	76.2	38	13.6	0.7	0.4	11
17	50.14	120	26.6	54.9	0.271	77.5	38	13.2	0.7	0.4	12
18	50.15	120	26.9	52.7	0.277	79.2	38	13.6	0.7	0.4	11
19	50.16	120	26.1	55.0	0.262	75.0	37	13.3	0.7	0.3	11
20	50.17	120	26.7	52.4	0.277	79.0	38	13.6	0.7	0.4	12
21	50.18	120	26.6	53.8	0.275	78.6	38	13.6	0.7	0.4	12
22	50.18	120	26.6	53.5	0.270	77.1	38	13.5	0.7	0.4	11
23	50.19	120	26.4	53.9	0.271	77.5	38	13.7	0.7	0.4	12
24	50.20	120	26.7	53.5	0.275	78.5	38	13.7	0.7	0.4	12
25	50.21	120	26.3	53.8	0.273	77.9	38	13.8	0.7	0.4	12
26	50.22	120	26.0	53.4	0.277	79.0	37	13.9	0.7	0.4	13
27	50.23	120	26.2	53.8	0.270	77.0	37	13.7	0.7	0.4	13
28	50.23	120	26.6	53.5	0.273	78.0	38	13.8	0.7	0.4	13
29	50.24	120	26.0	53.6	0.280	80.0	37	14.2	0.7	0.4	13
30	50.25	120	25.9	53.7	0.275	78.5	37	14.0	0.7	0.4	13
31	50.26	120	26.0	53.4	0.273	78.0	37	14.0	0.8	0.3	13
32	50.27	120	26.0	54.2	0.283	80.9	37	14.3	0.7	0.4	13
33	50.28	120	25.6	53.5	0.270	77.2	37	14.1	0.8	0.3	13
34	50.28	120	25.6	54.2	0.264	75.4	37	13.8	0.7	0.3	13
35	50.29	120	25.8	53.9	0.275	78.6	37	14.2	0.7	0.4	13
36	50.30	120	25.6	54.0	0.276	78.8	37	14.2	0.8	0.4	13
37	50.31	120	25.5	53.8	0.272	77.8	36	14.3	0.7	0.3	13
38	50.32	120	25.5	54.0	0.267	76.3	36	14.1	0.8	0.3	13
39	50.33	120	25.6	54.1	0.269	76.9	37	14.0	0.7	0.3	13
40	50.33	120	25.5	53.8	0.269	76.7	36	14.2	0.7	0.3	13
41	50.34	120	25.3	54.1	0.270	77.0	36	14.3	0.7	0.3	13
42	50.35	120	25.6	53.8	0.272	77.8	37	14.3	0.7	0.4	13
43	50.36	120	25.4	54.0	0.270	77.1	36	14.2	0.7	0.3	13
44	50.37	120	25.2	53.9	0.269	76.8	36	14.1	0.7	0.3	13
45	50.38	120	25.5	54.0	0.268	76.5	36	14.3	0.7	0.4	13
46	50.38	120	25.0	55.1	0.254	72.6	36	13.7	0.7	0.3	13
47	50.39	120	25.3	52.6	0.273	77.9	36	14.2	0.7	0.3	13
48	50.40	120	25.3	54.0	0.268	76.4	36	14.0	0.7	0.3	13
49	50.41	120	25.5	53.9	0.265	75.6	37	13.9	0.7	0.3	13
50	50.42	120	25.3	54.0	0.266	75.9	36	14.1	0.7	0.3	14
		Average	26.1	53.7	0.270	77.2	37	13.8	0.7	0.4	12
		Std. Dev.	0.6	1.1	0.005	1.6	1	0.4	0.0	0.0	1
		Maximum	27.0	55.1	0.283	80.9	39	14.3	0.8	0.4	14

Total number of blows analyzed: 50

BL#	depth (ft)	Comments
1	50.01	Start of test on 12/22/2014 at 2:18:52 PM
50	50.42	End of test on 12/22/2014 at 2:19:47 PM

Time Summary

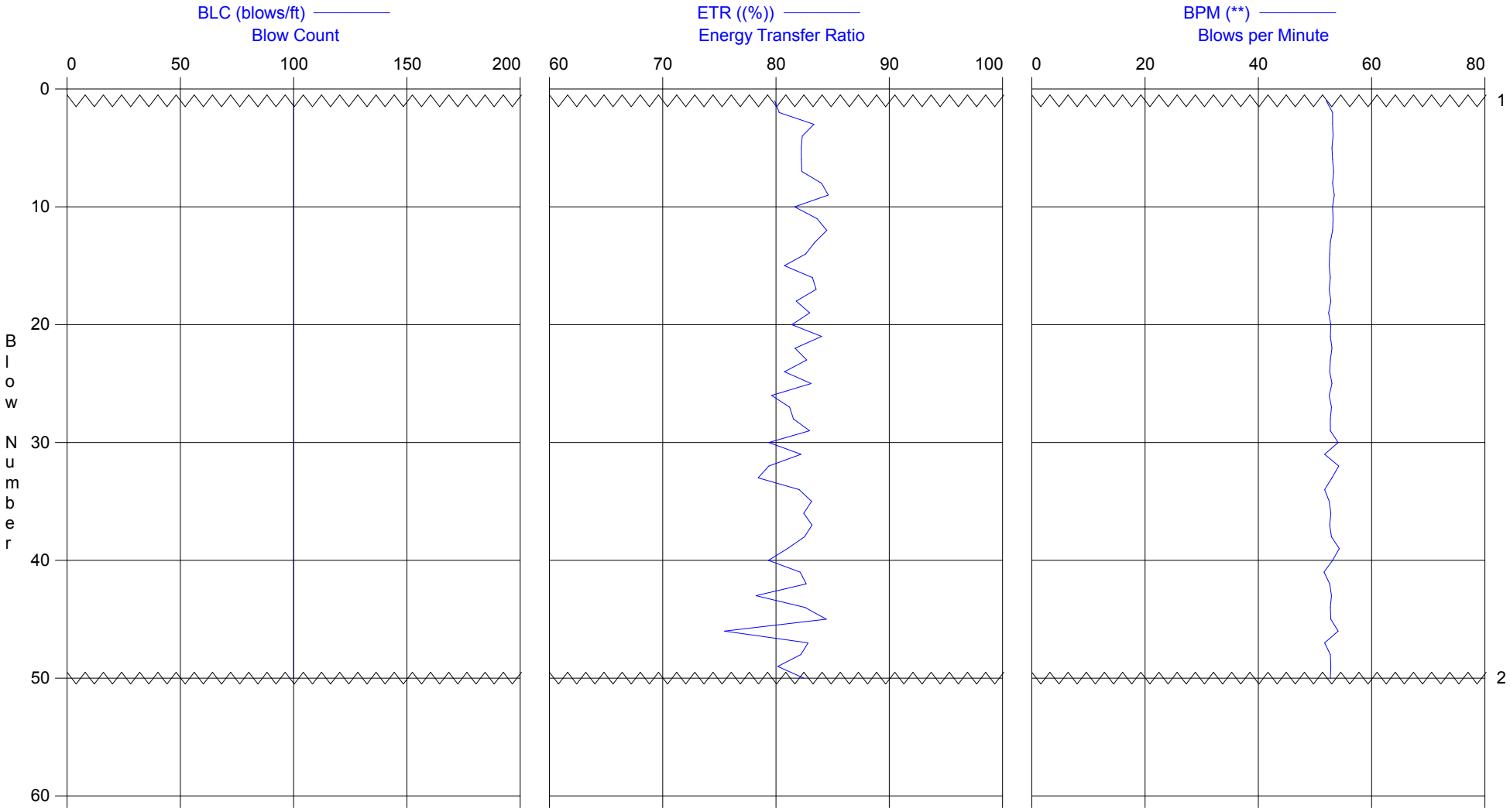
Drive	55 seconds	2:18:52 PM - 2:19:47 PM (12/22/2014) BN 1 - 50

Robert Miner Dynamic Testing, Inc. - Case Method & iCAP® Results

PDIPLOT Ver. 2014.1 - Printed: 23-Dec-2014

Test date: 22-Dec-2014

GREGORY DRILLNG, UW CAMPUS - 65FT SAMPLE



1 - Start of test on 12/22/2014 at 3:09:27 PM

2 - End of test on 12/22/2014 at 3:10:22 PM

EX-0001-006747-PCE

Robert Miner Dynamic Testing, Inc.
Case Method & iCAP® Results

PDIPLOT Ver. 2014.1 - Printed: 23-Dec-2014

GREGORY DRILLNG, UW CAMPUS - 65FT SAMPLE
OP: RMDT

BH E011, CME 75 RIG 304
Test date: 22-Dec-2014

AR: 1.43 in^2
LE: 69.00 ft
WS: 16,807.9 f/s

SP: 0.492 k/ft3
EM: 30,000 ksi
JC: 0.35

CSX: Max Measured Compr. Stress
BPM: Blows per Minute
EFV: Energy of FV
ETR: Energy Transfer Ratio
FMX: Maximum Force

VMX: Maximum Velocity
RAT: SPT Length Ratio
EF2: Energy of F^2
RX9: Max Case Method Capacity (JC=0.9)

BL#	depth	BLC	CSX	BPM	EFV	ETR	FMX	VMX	RAT	EF2	RX9
	ft	bl/ft	ksi	**	k-ft	(%)	kips	f/s	[]	k-ft	kips
1	65.01	100	27.8	52.0	0.280	79.9	40	13.1	0.7	0.4	5
2	65.02	100	28.3	53.1	0.281	80.3	41	13.8	0.7	0.4	6
3	65.03	100	29.3	53.1	0.292	83.4	42	14.1	0.6	0.4	6
4	65.04	100	29.6	53.2	0.288	82.3	42	13.8	0.7	0.4	6
5	65.05	100	29.3	53.0	0.288	82.2	42	14.0	0.7	0.4	7
6	65.06	100	29.0	53.1	0.288	82.2	41	13.2	0.7	0.4	6
7	65.07	100	28.7	53.3	0.288	82.3	41	13.2	0.7	0.4	5
8	65.08	100	29.4	53.1	0.294	84.0	42	14.2	0.7	0.4	7
9	65.09	100	29.6	53.4	0.296	84.6	42	14.1	0.7	0.4	6
10	65.10	100	28.9	53.1	0.286	81.6	41	13.6	0.7	0.4	7
11	65.11	100	29.7	53.2	0.293	83.6	42	14.2	0.7	0.4	9
12	65.12	100	29.8	53.1	0.296	84.5	43	14.1	0.7	0.4	9
13	65.13	100	29.8	52.7	0.292	83.4	43	14.0	0.7	0.4	10
14	65.14	100	30.0	52.6	0.289	82.6	43	14.0	0.5	0.4	9
15	65.15	100	29.0	52.5	0.283	80.7	41	12.9	0.6	0.4	7
16	65.16	100	29.8	52.7	0.291	83.2	43	14.0	0.7	0.4	9
17	65.17	100	29.8	52.5	0.292	83.5	43	13.9	0.7	0.4	9
18	65.18	100	29.1	52.8	0.286	81.8	42	13.4	0.8	0.4	10
19	65.19	100	30.3	52.4	0.290	83.0	43	14.1	0.6	0.4	10
20	65.20	100	28.9	52.8	0.285	81.4	41	13.4	0.7	0.4	9
21	65.21	100	30.0	52.7	0.294	84.0	43	14.1	0.6	0.4	9
22	65.22	100	29.0	53.0	0.286	81.7	41	13.5	0.7	0.4	9
23	65.23	100	29.9	52.7	0.289	82.7	43	14.1	0.6	0.4	10
24	65.24	100	28.8	52.6	0.283	80.7	41	13.4	0.7	0.4	10
25	65.25	100	29.8	53.0	0.291	83.1	43	14.0	0.6	0.4	10
26	65.26	100	28.6	52.5	0.279	79.6	41	13.2	0.7	0.4	10
27	65.27	100	29.1	52.9	0.284	81.2	42	13.5	0.7	0.4	11
28	65.28	100	29.2	52.7	0.285	81.5	42	13.5	0.7	0.4	12
29	65.29	100	29.6	52.7	0.290	83.0	42	13.9	0.7	0.4	11
30	65.30	100	28.4	54.1	0.278	79.4	41	12.7	0.7	0.4	12
31	65.31	100	29.5	51.7	0.288	82.2	42	13.6	0.7	0.4	12
32	65.32	100	28.7	54.2	0.278	79.4	41	13.2	0.7	0.4	12
33	65.33	100	28.2	53.0	0.274	78.4	40	12.6	0.7	0.4	12
34	65.34	100	29.0	51.7	0.287	82.0	41	13.3	0.7	0.4	13
35	65.35	100	29.2	52.5	0.291	83.1	42	13.0	0.7	0.4	12
36	65.36	100	29.5	52.8	0.289	82.4	42	13.8	0.7	0.4	13
37	65.37	100	29.2	52.6	0.291	83.2	42	13.5	0.7	0.4	13
38	65.38	100	29.6	52.9	0.289	82.5	42	14.0	0.7	0.4	13
39	65.39	100	29.1	54.3	0.284	81.0	42	13.8	0.7	0.4	13
40	65.40	100	28.8	53.1	0.278	79.3	41	13.0	0.7	0.4	13
41	65.41	100	29.1	51.6	0.287	82.1	42	13.4	0.8	0.4	13
42	65.42	100	29.2	52.6	0.289	82.7	42	13.4	0.7	0.4	13
43	65.43	100	28.6	52.9	0.274	78.2	41	12.9	0.7	0.4	13
44	65.44	100	29.2	52.7	0.289	82.6	42	13.9	0.7	0.4	13
45	65.45	100	29.6	52.8	0.296	84.4	42	14.1	0.7	0.4	14
46	65.46	100	27.7	54.1	0.264	75.5	40	12.6	0.6	0.3	13
47	65.47	100	29.0	51.7	0.290	82.8	41	13.7	0.7	0.4	14
48	65.48	100	29.2	52.7	0.288	82.2	42	13.2	0.7	0.4	13
49	65.49	100	28.7	52.8	0.281	80.2	41	13.1	0.7	0.4	13
50	65.50	100	29.2	52.7	0.289	82.5	42	13.7	0.7	0.4	14
		Average	29.2	52.8	0.287	81.9	42	13.6	0.7	0.4	10
		Std. Dev.	0.5	0.6	0.006	1.8	1	0.4	0.1	0.0	3
		Maximum	30.3	54.3	0.296	84.6	43	14.2	0.8	0.4	14

Total number of blows analyzed: 50

BL#	depth (ft)	Comments
1	65.01	Start of test on 12/22/2014 at 3:09:27 PM
50	65.50	End of test on 12/22/2014 at 3:10:22 PM

Time Summary

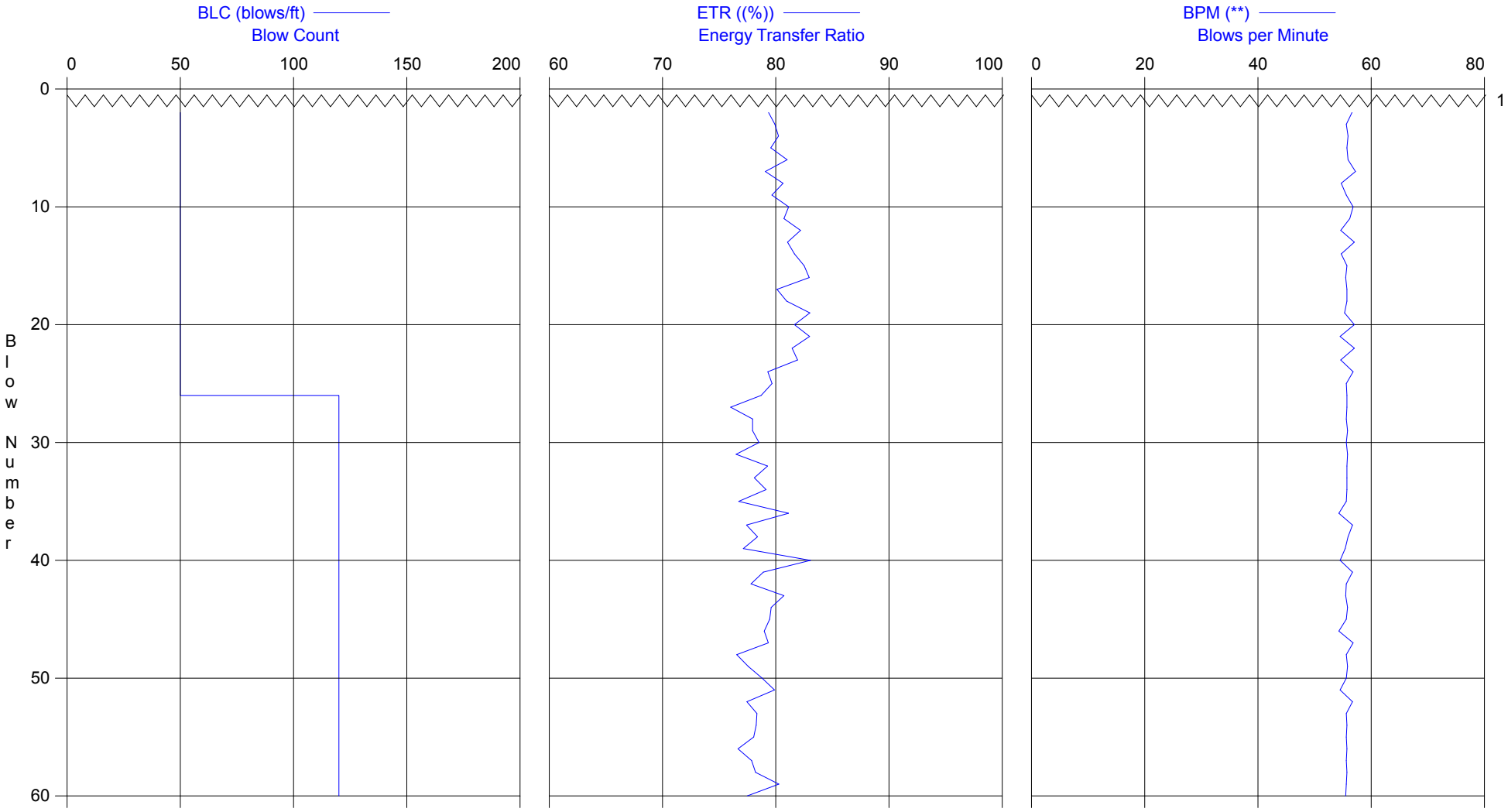
Drive	55 seconds	3:09:27 PM - 3:10:22 PM (12/22/2014) BN 1 - 50

Robert Miner Dynamic Testing, Inc. - Case Method & iCAP® Results

PDIPLOT Ver. 2014.1 - Printed: 23-Dec-2014

Test date: 22-Dec-2014

GREGORY DRILLNG, UW CAMPUS - 70FT SAMPLE



1 - Start of test on 12/22/2014 at 3:30:18 PM

EX-0001-006749-PCE

Robert Miner Dynamic Testing, Inc.
Case Method & iCAP® Results

PDIPLOT Ver. 2014.1 - Printed: 23-Dec-2014

GREGORY DRILLNG, UW CAMPUS - 70FT SAMPLE
OP: RMDT

BH E011, CME 75 RIG 304
Test date: 22-Dec-2014

AR: 1.43 in^2
LE: 74.00 ft
WS: 16,807.9 f/s

SP: 0.492 k/ft3
EM: 30,000 ksi
JC: 0.35

CSX: Max Measured Compr. Stress
BPM: Blows per Minute
EFV: Energy of FV
ETR: Energy Transfer Ratio
FMX: Maximum Force

VMX: Maximum Velocity
RAT: SPT Length Ratio
EF2: Energy of F^2
RX9: Max Case Method Capacity (JC=0.9)

BL#	depth	BLC	CSX	BPM	EFV	ETR	FMX	VMX	RAT	EF2	RX9
	ft	bl/ft	ksi	**	k-ft	(%)	kips	f/s	[]	k-ft	kips
26	74.51	120	28.3	55.7	0.275	78.7	40	14.0	0.5	0.4	7
27	74.52	120	27.8	55.7	0.266	76.0	40	13.4	0.7	0.4	7
28	74.53	120	27.9	55.6	0.273	78.0	40	13.6	0.6	0.4	8
29	74.53	120	28.3	55.8	0.273	78.0	41	13.7	0.5	0.4	8
30	74.54	120	28.2	55.6	0.275	78.5	40	13.7	0.5	0.4	7
31	74.55	120	27.8	55.8	0.268	76.5	40	13.7	0.5	0.4	7
32	74.56	120	28.3	55.7	0.277	79.3	41	14.0	0.5	0.4	7
33	74.57	120	28.5	55.7	0.273	78.1	41	14.1	0.5	0.4	7
34	74.58	120	28.5	55.7	0.277	79.2	41	14.2	0.5	0.4	7
35	74.58	120	27.7	55.6	0.269	76.7	40	13.9	0.7	0.4	7
36	74.59	120	27.8	54.3	0.284	81.1	40	14.4	0.7	0.4	8
37	74.60	120	27.7	56.7	0.271	77.4	40	13.5	0.6	0.4	7
38	74.61	120	28.4	55.9	0.274	78.4	41	13.8	0.5	0.4	8
39	74.62	120	28.3	55.4	0.270	77.1	40	13.7	0.6	0.4	7
40	74.63	120	28.7	54.5	0.291	83.1	41	14.7	0.7	0.4	8
41	74.63	120	28.2	56.7	0.276	78.9	40	13.8	0.5	0.4	8
42	74.64	120	28.3	55.6	0.272	77.8	41	13.6	0.5	0.4	8
43	74.65	120	29.1	55.5	0.283	80.7	42	14.3	0.4	0.4	8
44	74.66	120	28.6	55.8	0.279	79.6	41	14.2	0.5	0.4	8
45	74.67	120	28.7	55.6	0.278	79.5	41	14.1	0.5	0.4	9
46	74.68	120	28.2	54.3	0.276	79.0	40	13.9	0.7	0.4	8
47	74.68	120	28.4	56.8	0.278	79.3	41	13.9	0.5	0.4	8
48	74.69	120	28.2	55.6	0.268	76.6	40	13.1	0.5	0.4	8
49	74.70	120	28.5	55.8	0.271	77.6	41	13.1	0.5	0.4	8
50	74.71	120	28.9	55.6	0.276	78.8	41	13.4	0.7	0.4	9
51	74.72	120	28.8	54.5	0.280	79.9	41	13.4	0.6	0.4	8
52	74.73	120	28.1	56.7	0.271	77.5	40	13.5	0.6	0.4	8
53	74.73	120	28.2	55.6	0.274	78.3	40	13.6	0.5	0.4	8
54	74.74	120	28.4	55.7	0.274	78.3	41	13.4	0.7	0.4	8
55	74.75	120	28.2	55.6	0.273	78.1	40	13.6	0.5	0.4	8
56	74.76	120	28.1	55.7	0.268	76.7	40	13.1	0.6	0.4	8
57	74.77	120	28.3	55.6	0.273	77.9	41	13.2	0.5	0.4	8
58	74.78	120	28.2	55.7	0.274	78.2	40	13.4	0.7	0.4	8
59	74.78	120	28.7	55.6	0.281	80.3	41	13.8	0.5	0.4	8
60	74.79	120	28.2	55.5	0.271	77.5	40	13.3	0.7	0.4	8
61	74.80	120	28.5	55.8	0.275	78.6	41	13.5	0.7	0.4	7
62	74.81	120	27.8	55.5	0.270	77.3	40	13.1	0.7	0.4	7
63	74.82	120	28.4	55.7	0.276	78.8	41	13.5	0.5	0.4	7
64	74.83	120	28.6	55.5	0.278	79.4	41	13.5	0.7	0.4	7
65	74.83	120	28.3	55.8	0.272	77.6	40	13.3	0.7	0.4	7
66	74.84	120	28.4	55.4	0.272	77.7	41	13.2	0.6	0.4	6
67	74.85	120	28.6	55.6	0.272	77.6	41	13.3	0.6	0.4	6
68	74.86	120	28.3	55.6	0.270	77.1	40	13.1	0.6	0.4	6
69	74.87	120	29.0	54.2	0.281	80.3	41	13.5	0.6	0.4	6
70	74.88	120	29.0	55.1	0.282	80.5	41	13.3	0.6	0.4	5
71	74.88	120	28.3	56.6	0.269	76.9	40	13.0	0.7	0.4	5
72	74.89	120	28.8	54.4	0.285	81.3	41	13.9	0.5	0.4	6
73	74.90	120	28.0	56.4	0.272	77.7	40	13.2	0.7	0.4	5
74	74.91	120	28.1	56.0	0.277	79.1	40	13.3	0.7	0.4	6
75	74.92	120	29.3	23.7	0.296	84.5	42	14.0	0.5	0.4	5
		Average	28.4	55.0	0.275	78.6	41	13.6	0.6	0.4	7
		Std. Dev.	0.4	4.5	0.006	1.6	1	0.4	0.1	0.0	1
		Maximum	29.3	56.8	0.296	84.5	42	14.7	0.7	0.4	9

Total number of blows analyzed: 50

BL#	depth (ft)	Comments
1	74.02	Start of test on 12/22/2014 at 3:30:18 PM
75	74.92	End of test on 12/22/2014 at 3:31:39 PM

Time Summary

Drive 1 minute 21 seconds 3:30:18 PM - 3:31:39 PM (12/22/2014) BN 1 - 75

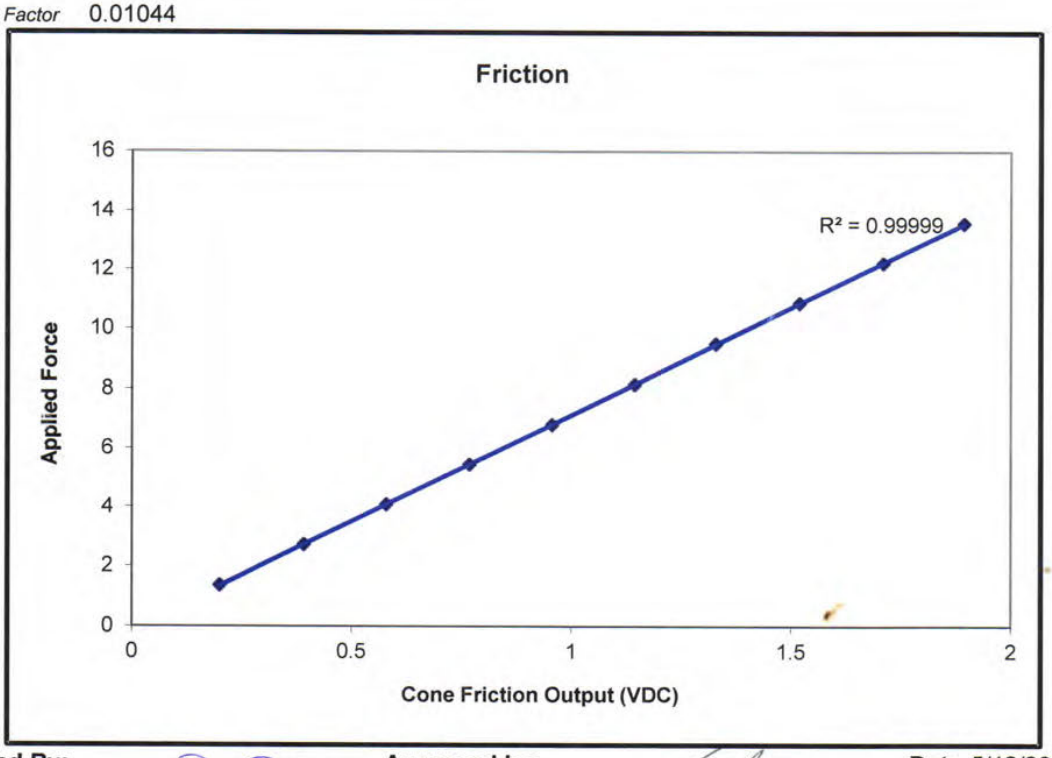
CPT calibration information

250 Beanville Road
Randolph, Vt. 05060
802-728-4588
800-639-6315

Digital Cone Penetrometer Calibration

Cone Serial Number	DDG1254	
Customer	NORTHWEST	0
Reference Load Cell	INTERFACE 1221ANE-50K-B	SN 322089A
Reference Press. Gauge	DRESSER D XD 1000PSIG S/N	3432

	FRICTION
Full range V	1.8804
Baseline V	0.01474
Full Scale Force	1300 KPA
Output Units	TSF
Unit Conv. Factor	0.01044



Calibrated By: L Smith Signed: _____

Approved by: T van der Stok Signed: _____

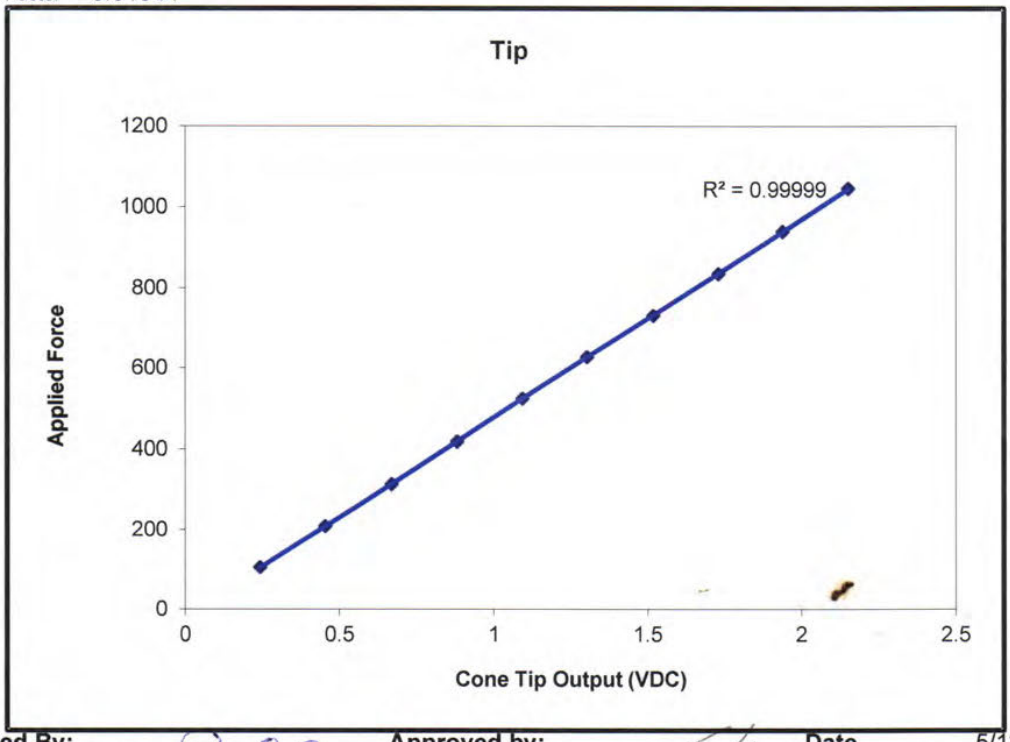
Date 5/12/2014

250 Beanville Road
Randolph, Vt. 05060
802-728-4588
800-639-6315

Digital Cone Penetrometer Calibration

Cone Serial Number	DDG1254	
Customer	NORTHWEST	0
Reference Load Cell	INTERFACE 1221ANE-50K-B	SN 322089A
Reference Press. Gauge	DRESSER D XD 1000PSIG S/N	3432

	TIP
Full range V	2.1169
Baseline V	0.03642
Full Scale Force	100000 KPA
Output Units	TSF
Unit Conv. Factor	0.01044



Calibrated By: L Smith Signed:

Approved by: T van der Stok Signed:

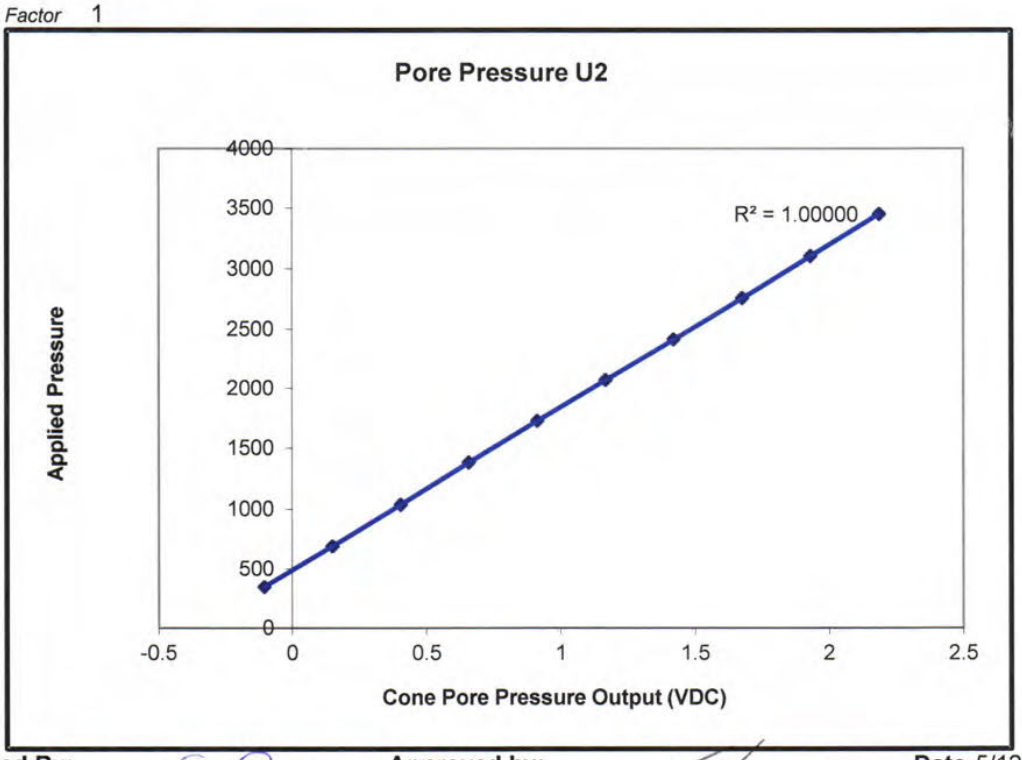
Date 5/12/2014

250 Beanville Road
Randolph, Vt. 05060
802-728-4588
800-639-6315

Digital Cone Penetrometer Calibration

Cone Serial Number	DDG1254	
Customer	NORTHWEST	0
Reference Load Cell	INTERFACE 1221ANE-50K-B	SN 322089A
Reference Press. Gauge	DRESSER D XD 1000PSIG S/N	3432

	PRESSURE U2
Full range V	2.5534
Baseline V	-0.35699
Full Scale Force	3447.5 KPA
Output Units	PSI
Unit Conv. Factor	1



Calibrated By: L Smith Signed:

Approved by: T van der Stok Signed:

Date 5/12/2014

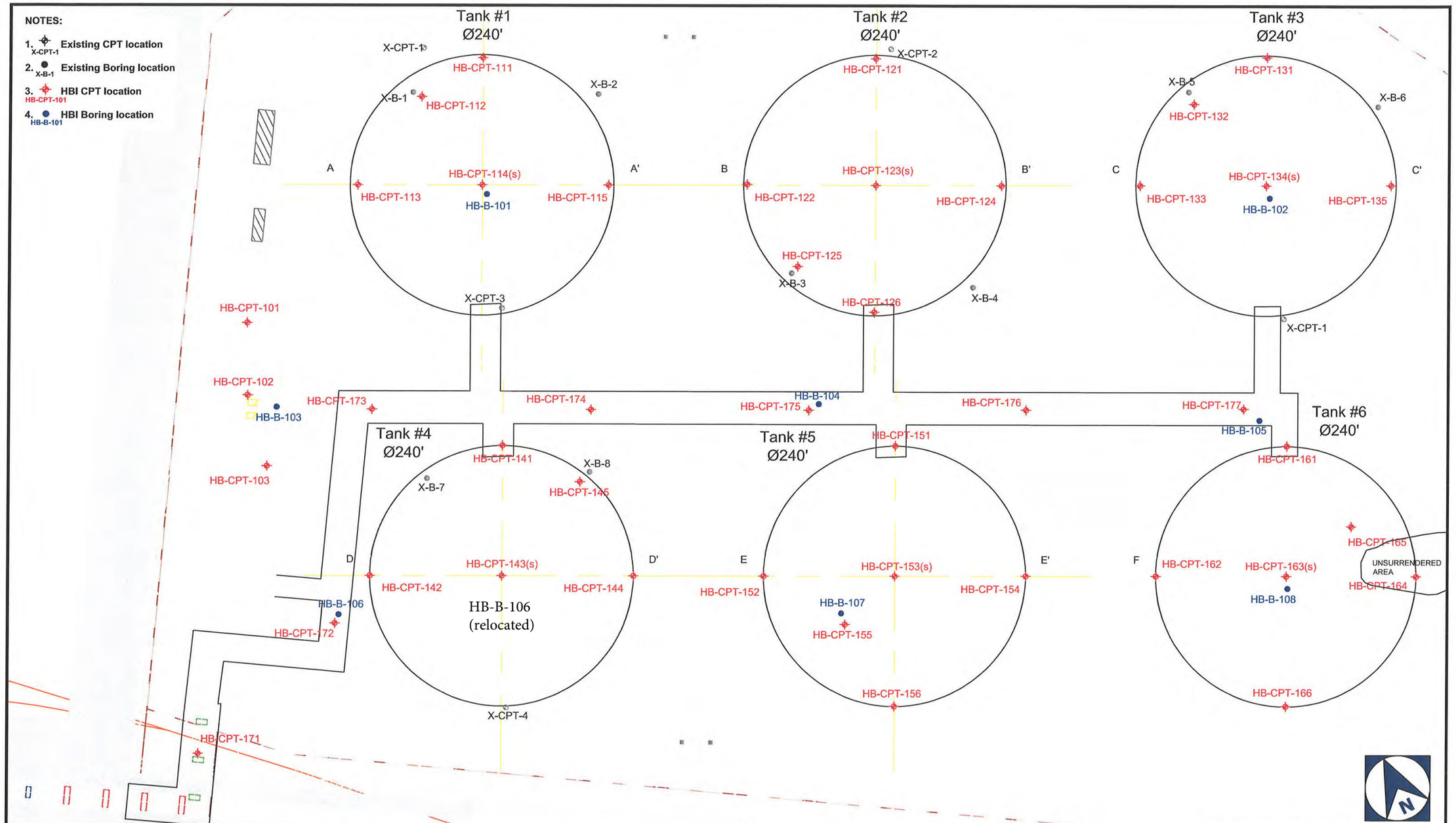
Appendix B

CPT Logs

DRAFT

Appendix B
CPT logs

Exploration Location Map



Notes:
1. Work will conducted in two phases. No work will be conducted in areas marked as Phase 2 until necessary clean-up has been completed and HBI is notified to proceed in those areas.
2. Spoils will be disposed of on site in a location agreed upon with the Owner.
3. No subsurface contamination is anticipated during this work. Contaminated spoils will be handled as per the work plan.
4. HBI will coordinate activities with the Port's project managers daily with regards to the use of Parcel 1A by other contractors.

Scale: 1":80'

0' 40' 80' 160'

PROJECT : Tesoro Distribution Terminal

Vancouver, Washington

SHEET TITLE : Geotechnical Soil Investigation

AREA 300 - Tank Area

DRWG. NO. 860058-001

RELATED DRWG:

DRWG. BY a m m

DATE 19 May 2014

APRVD. BY M Rohrbach

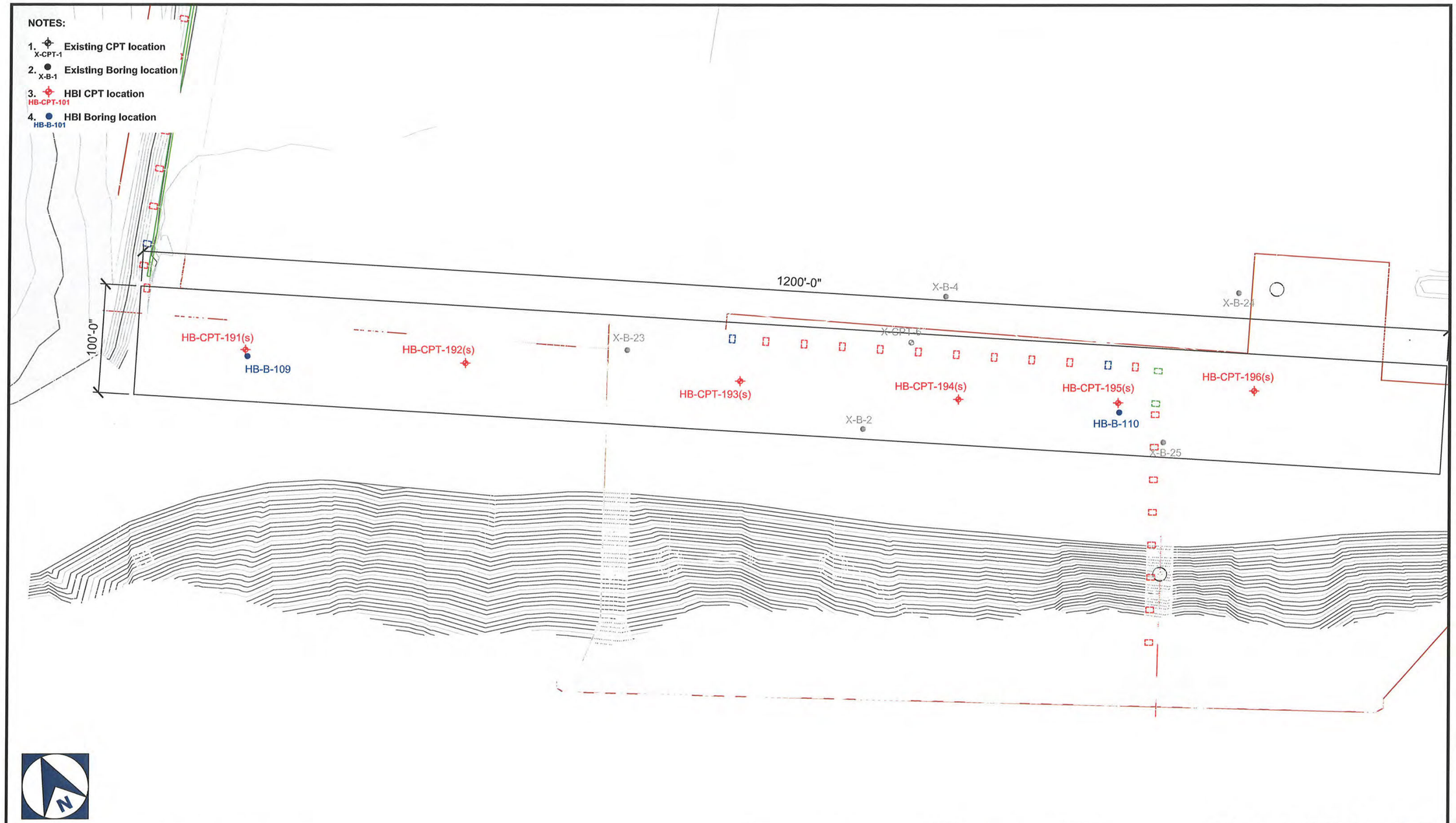
REVISIONS			
NO.	DRAWN BY	DATE	
0	s p b	11 Feb 2014	
1	a m m	24 Apr 2014	
1	d c f	3 May 2014	

HAYWARD BAKER
A MULLER COMPANY
Seattle Branch
3600 S Norfolk Way South
Seattle, WA 98118
Phone 206-723-4080 Fax 206-723-4090

PROJECT NUMBER: 860058

1 Sheets 1 of 2

EX-0001-006759-PCE



Notes:
1. Spoils will be disposed of on site in a location agreed upon with the Owner.
3. No subsurface contamination is anticipated during this work. Contaminated spoils will be handled as per the work plan.
4. HBI will coordinate activities with the Port's project managers daily with regards to the use site by other contractors.

Scale: 1":80'

0' 40' 80' 160'

PROJECT : Tesoro Distribution Terminal

Vancouver, Washington

SHEET TITLE : Geotechnical Soil Investigation

AREA 400 - Dock & Pipeline

DRWG. NO. 860058-002

RELATED DRWG:

DRWG. BY a m m

DATE 19 May 2014

APRVD. BY M Rohrbach

REVISIONS			
NO.	DRAWN BY		DATE
0	s p b	11	Feb 2014
1	a m m	24	Apr 2014

LAYWARD BAKER

A KELLER COMPANY

PROJECT NUMBER: 860058

1 Sheets 2 of 2

EX-0001-006761-PCE

CPT Location Coordinates

Project Name HAYWARD BAKER BORING-CPT (HAYWARD BAKER BORING CPT.DWG)
Project # 15622 (HBI CPT-BORING.csv)
Date 11/6/14
Crew F. COSTANZA, A. DILL

MacKay + Sposito

STA@ 1010	BS@ 205	6400	STA@	BS@	
HT= 6 38 CB	HR= 6 59	6458	HT=	HR=	
STA@	BS@		STA@	BS@	
HT=	HR=		HT=	HR=	
STA@	BS@		STA@	BS@	
HT=	HR=		HT=	HR=	

Point	Northing	Easting	Elev.	Description	Hub Elev.	Cut / Fill	Store No.
6300	120964.5607	1072349.682	0	HB-CPT-196(S)	27.94	~	~~6400~~ 6458
6301	121009.3994	1072232.252	0	HB-CPT-195(S)	27.25	~	6456
6302	121076.6474	1072101.527	0	HB-CPT-194(S)	27.09	~	6455
6303	121179.0322	1071929.39	0	HB-CPT-193(S)	26.88	~	6454
6304	121304.071	1071709.597	0	SET 20' (E) OF PNT HB-CPT-192(S)	27.20 FALLS IN ROCK PILE		6453
6305	121402.5404	1071534.943	0	HB-CPT-191(S)	27.42	~	6452
6306	121396.3319	1071534.061	0	HB-B-109	27.54	~	6451
6307	121001.1169	1072229.35	0	HB-B-110	27.29	~	6457
6308	122119.1913	1073789.129	0	HB-CPT-171	26.34	~	6401
6309	122173.2003	1073953.276	0	HB-CPT-172	27.49	~	6402
6310	122198.8178	1074001.039	0	HB-CPT-142	27.55	~	6403
6311	122146.3409	1074108.956	0	HB-CPT-143(S)	27.34	~	6404
6312	122093.8639	1074216.874	0	HB-CPT-144	27.40	~	6405
6313	122193.2851	1074210.453	0	HB-CPT-145	27.86	~	6406
6314	122254.2583	1074161.434	0	HB-CPT-141	27.93	~	6407
6315	122330.9513	1073960.019	0	HB-CPT-103	27.85	~	6408
6316	122397.3714	1073972.572	0	HB-CPT-102	28.14	~	6409

Date Staked: _____

Checked By: _____

Project Name HAYWARD BAKER BORING-CPT (HAYWARD BAKER BORING CPT.DWG)
Project # 15622 (HBI CPT-BORING.csv)
Date
Crew

MacKay + Sposito

STA@	BS@		STA@	BS@	
HT=	HR=		HT=	HR=	
STA@	BS@		STA@	BS@	
HT=	HR=		HT=	HR=	
STA@	BS@		STA@	BS@	
HT=	HR=		HT=	HR=	

Point	Northing	Easting	Elev.	Description	Hub Elev.	Cut / Fill	Store No.
6317	122336.2939	1074068.906	0	HB-CPT-173	27.93	~	6411
6318	122248.8348	1074248.259	0	HB-CPT-174	28.27	~	6412
6319	122161.3758	1074427.612	0	HB-CPT-175	28.22	~	6413
6320	122162.4349	1074438.423	0	HB-B-104	28.04	~	6414
6321	122376.1051	1073991.536	0	HB-B-103	28.18	~	6410
6322	122178.878	1073959.936	0	HB-B-106	27.59	~	6418
6323	122457.7691	1074001.038	0	HB-CPT-101	28.81	~	6419
6324	122528.2569	1074146.642	0	HB-CPT-113	28.93	~	6420
6325	122478.7869	1074248.449	0	HB-CPT-114(S)	29.26	~	6423
6326	122469.223	1074248.486	0	HB-B-101	29.20	~	6424
6327	122428.4275	1074351.888	0	HB-CPT-115	29.17	~	6425
6328	122576.1083	1074233.972	0	HB-CPT-112	29.39	~	6421
6329	122584.1463	1074299.555	0	HB-CPT-111	29.50	~	6422
* 6330	122373.2375	1074467.403	0	HB-CPT-122	29.33	~	6426
6331	122285.2002	1074475.767	0	HB-CPT-125	28.58	~	6427
6332	122216.4809	1074520.671	0	HB-CPT-126	28.38	~	6428
6333	122270.5412	1074676.133	0	HB-CPT-124	28.57	~	6430

Date Staked: _____

Checked By: _____

Project Name HAYWARD BAKER BORING-CPT (HAYWARD BAKER BORING CPT.DWG)

Project # 15622 (HBI CPT-BORING.csv)

Date

Crew

MacKay Sposito

STA@	BS@		STA@	BS@	
HT=	HR=		HT=	HR=	
STA@	BS@		STA@	BS@	
HT=	HR=		HT=	HR=	
STA@	BS@		STA@	BS@	
HT=	HR=		HT=	HR=	

Point	Northing	Easting	Elev.	Description	Hub Elev.	Cut / Fill	Store No.
6334	122321.2339	1074572.546	0	HB-CPT-123(S)	28.70	~	6429
6335	122426.5527	1074622.973	0	HB-CPT-121	29.73	~	6450
6336	122074.3521	1074607.176	0	HB-CPT-176	28.59	~	6446
6337	121987.5851	1074787.742	0	HB-CPT-177	28.66	~	6437
6338	121971.6681	1074796.496	0	HB-B-105	28.57	~	6438
6339	122214.7396	1074791.464	0	HB-CPT-133	28.64	~	6431
6340	122163.9779	1074895.906	0	HB-CPT-134(S)	28.53	~	6434
6341	122113.8709	1074999.607	0	HB-CPT-135	28.58	~	6436
6342	122152.1173	1074893.749	0	HB-B-102	28.73	~	6435
6343	122261.8277	1074868.885	0	HB-CPT-132	28.65	~	6432
6344	122270.1999	1074947.753	0	HB-CPT-131	28.69	~	6433
6360	122041.387	1074324.792	0	PH2 HB-CPT-152	27.23	~	6447
6361	121968.8459	1074372.004	0	PH2 HB-CPT-155	26.90	~	6448
6362	121988.9102	1074432.709	0	PH2 HB-CPT-153(S)	26.03	~	6416
6363	121880.9926	1074380.233	0	PH2 HB-CPT-156	25.01	~	6449
6364	121936.4332	1074540.627	0	PH2 HB-CPT-154	25.39	~	6445
6365	122096.8275	1074485.186	0	PH2 HB-CPT-151	28.42	~	6415

Date Staked: _____

Checked By: _____

Project Name HAYWARD BAKER BORING-CPT (HAYWARD BAKER BORING CPT.DWG)

Project # 15622 (HBI CPT-BORING.csv)

Date

Crew

MacKay Sposito

STA@	BS@		STA@	BS@	
HT=	HR=		HT=	HR=	
STA@	BS@		STA@	BS@	
HT=	HR=		HT=	HR=	
STA@	BS@		STA@	BS@	
HT=	HR=		HT=	HR=	

Point	Northing	Easting	Elev.	Description	Hub Elev.	Cut / Fill	Store No.
6366	121883.9559	1074648.544	0	PH2 HB-CPT-162	27.91	~	6444
6367	121723.5614	1074703.985	0	PH2 HB-CPT-166	27.44	~	6443
6368	121779.002	1074864.379	0	PH2 HB-CPT-164	FALLS	ON	PIPE
6369	121846.7258	1074830.095	0	PH2 HB-CPT-165	28.71	~	6440
6370	121939.3964	1074808.938	0	PH2 HB-CPT-161	28.65	~	6439
6371	121831.479	1074756.461	0	PH2 HB-CPT-163(S)	27.81	~	6441
6372	121820.7363	1074752.679	0	PH2 HB-B-108	27.73	~	6442
6373	121979.5838	1074373.519	0	PH2 HB-B-107	27.01	~	6417

Date Staked: _____

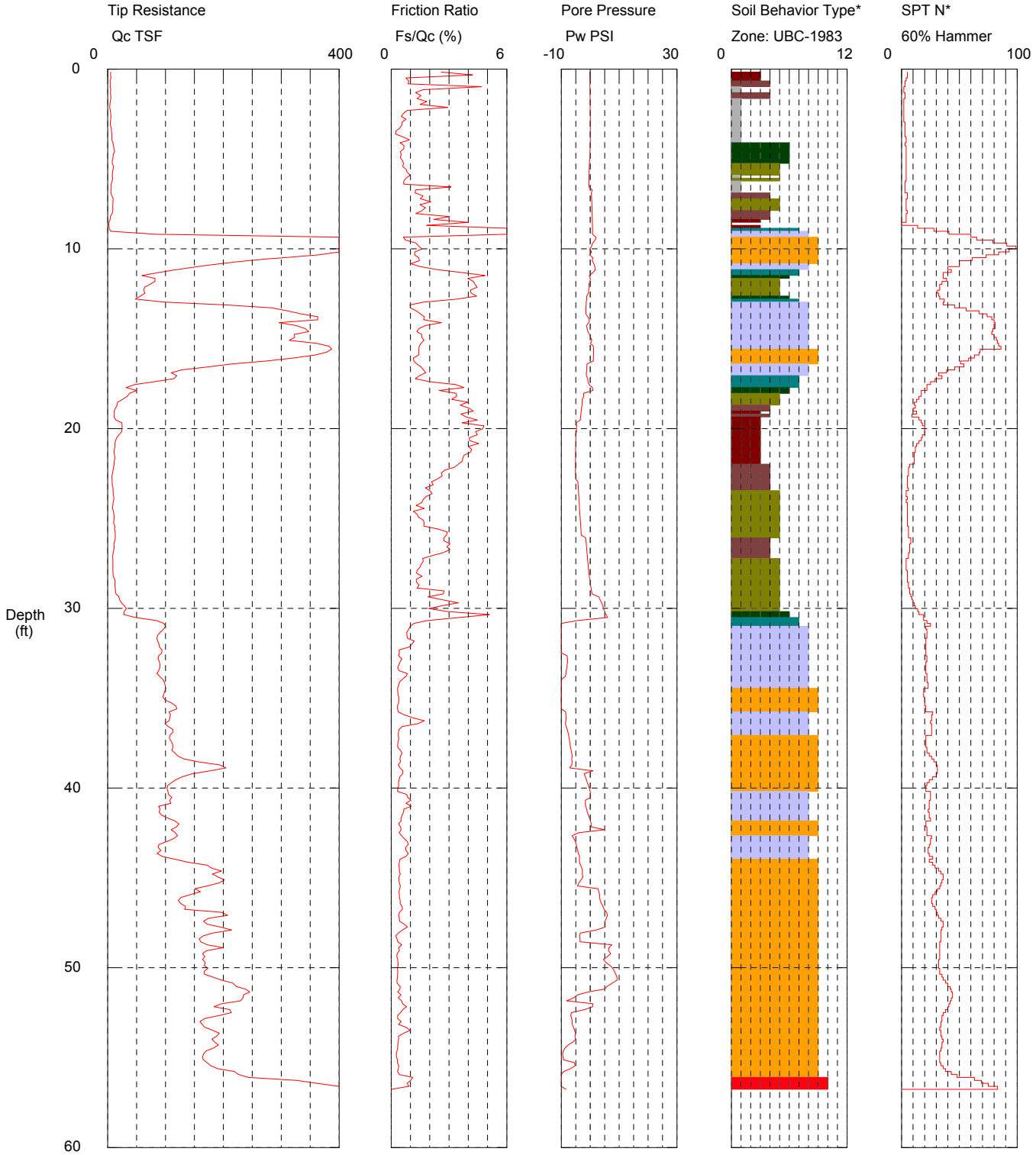
Checked By: _____

Tank 1

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-111
Cone Used: DDG1254

CPT Date/Time: 11/15/2014 10:49:21 AM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 56.76 feet

Depth Increment = 0.164 feet

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

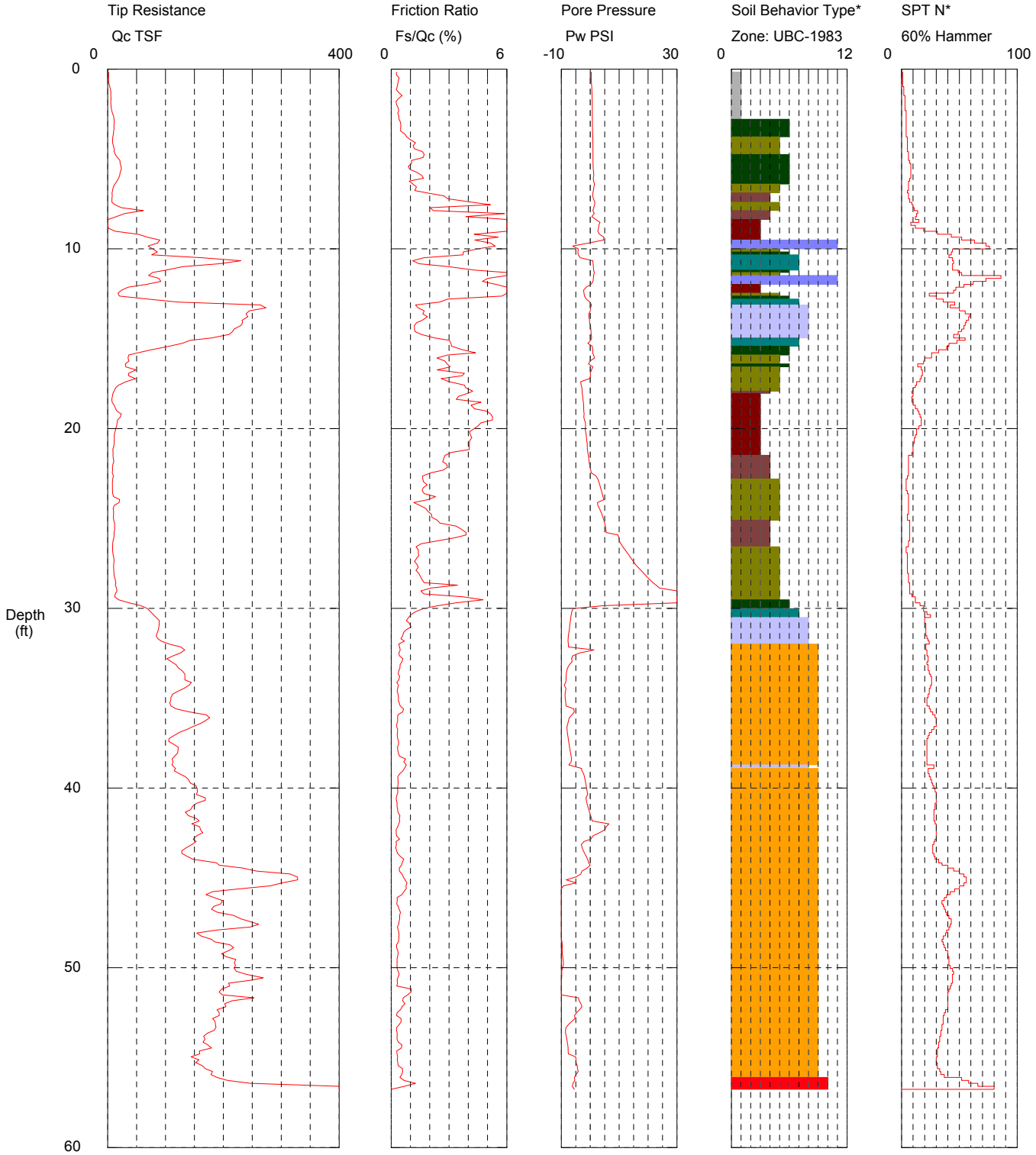
InSitu Engineering Predrilled top 9 feet

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

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-112
Cone Used: DDG1254

CPT Date/Time: 11/14/2014 4:23:51 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 56.76 feet

Depth Increment = 0.164 feet

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

InSitu Engineering

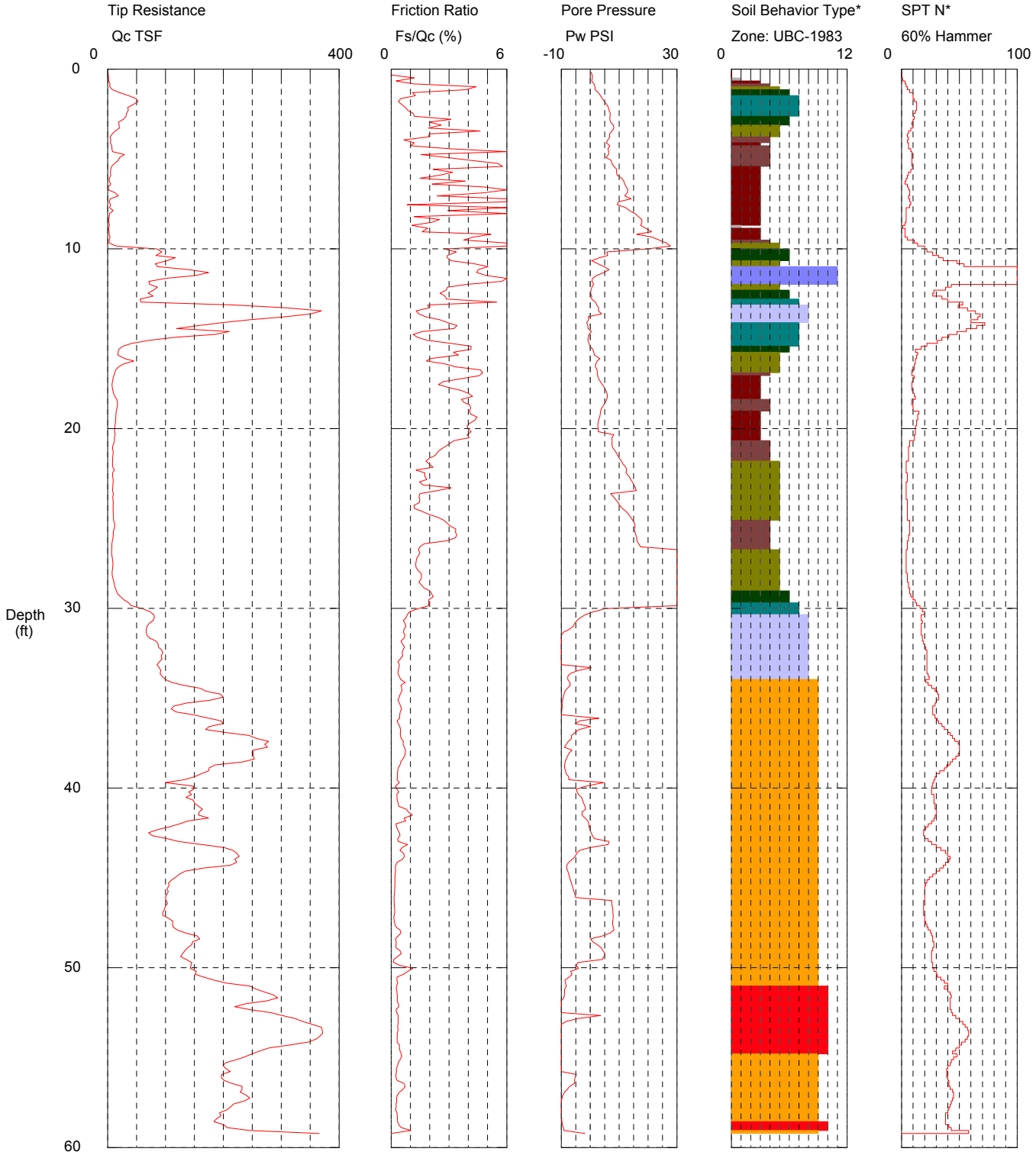
*Soil behavior type and SPT based on data from UBC-1983

EX-0001-006771-PCE

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-113
Cone Used: DDG1254

CPT Date/Time: 11/14/2014 2:21:15 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 59.22 feet

Depth Increment = 0.164 feet

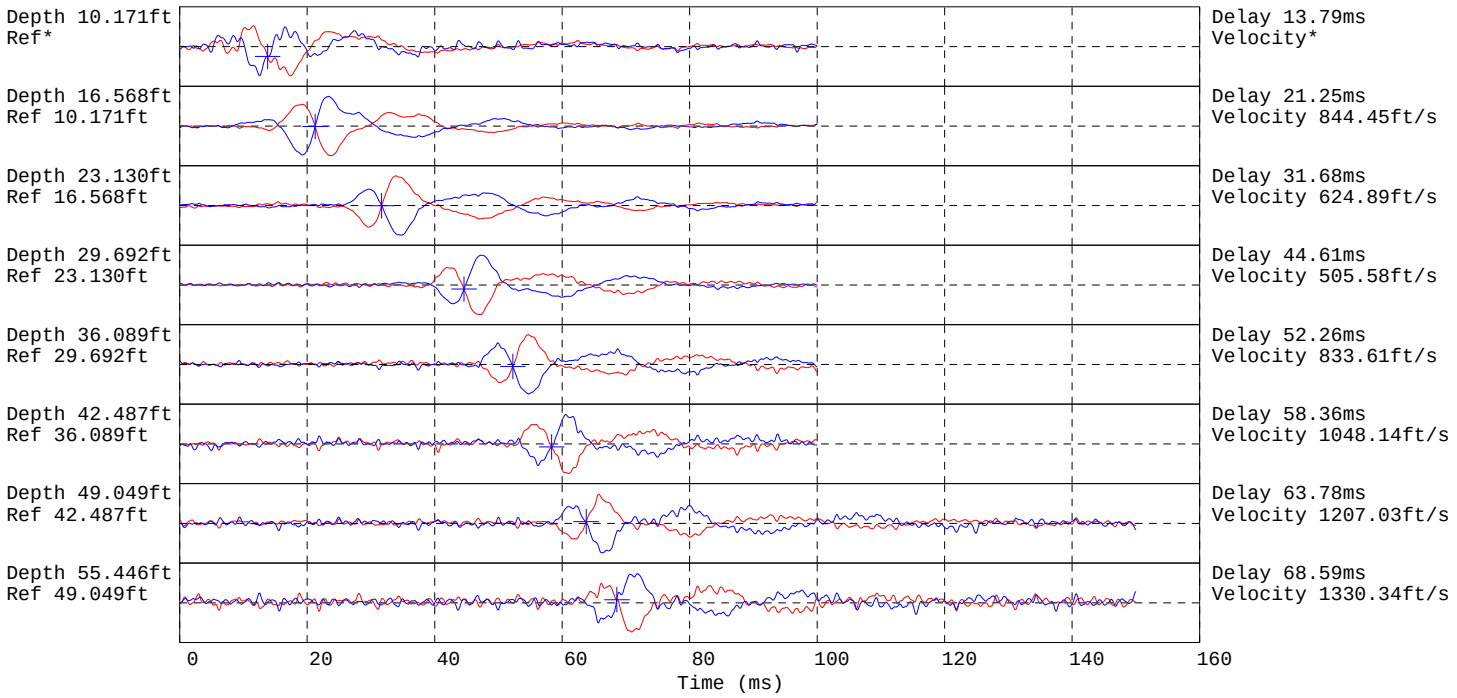
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

InSitu Engineering

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

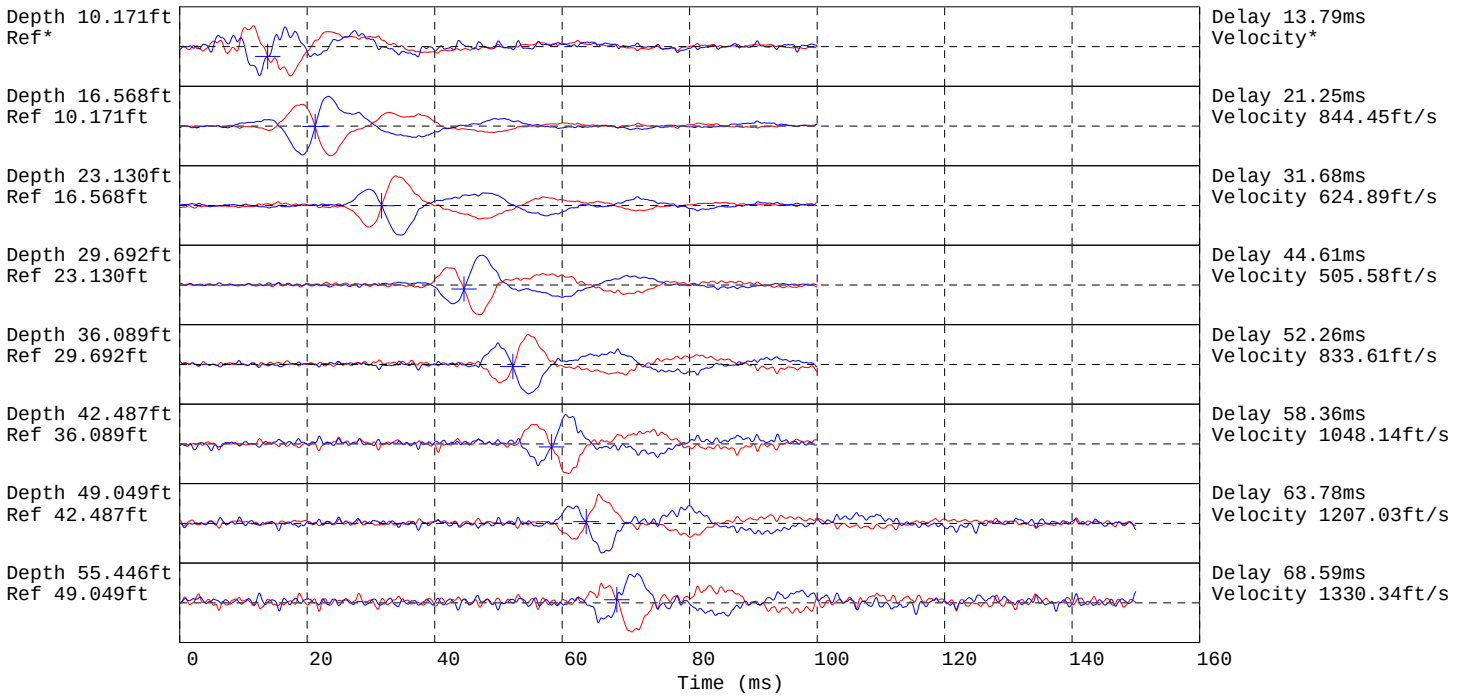
EX-0001-006772-PCE

Tesoro Distribution Terminal
HB-CPT-114(s) Shear Wave Analysis



Hammer to Rod String Distance 0.7 (m)
* = Not Determined

Tesoro Distribution Terminal
HB-CPT-114(s) Shear Wave Analysis

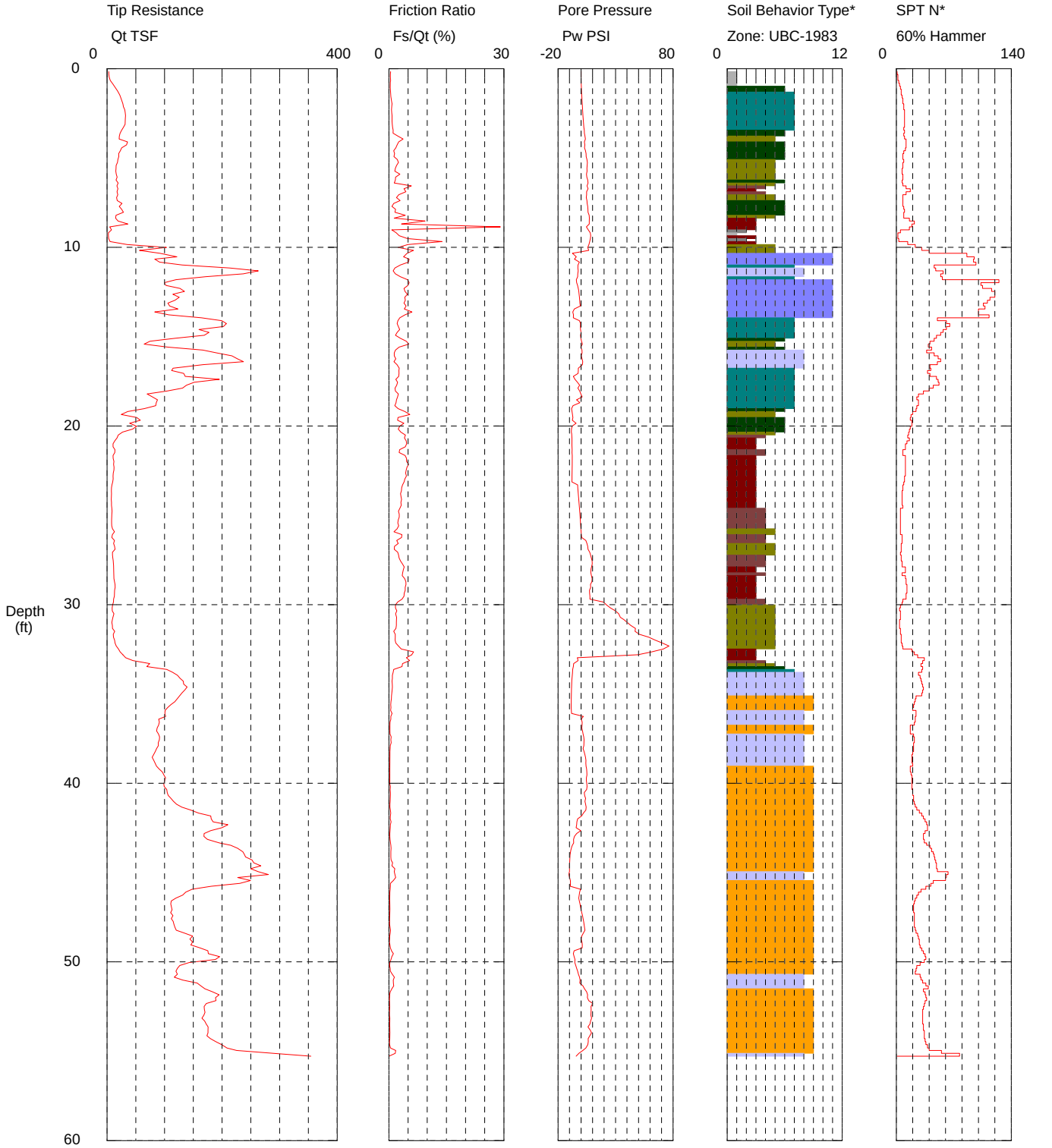


Hammer to Rod String Distance 0.7 (m)
* = Not Determined

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-114(s)B
Cone Used: DDG1254

CPT Date/Time: 11/13/2014 4:27:24 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 55.28 feet

Depth Increment = 0.164 feet

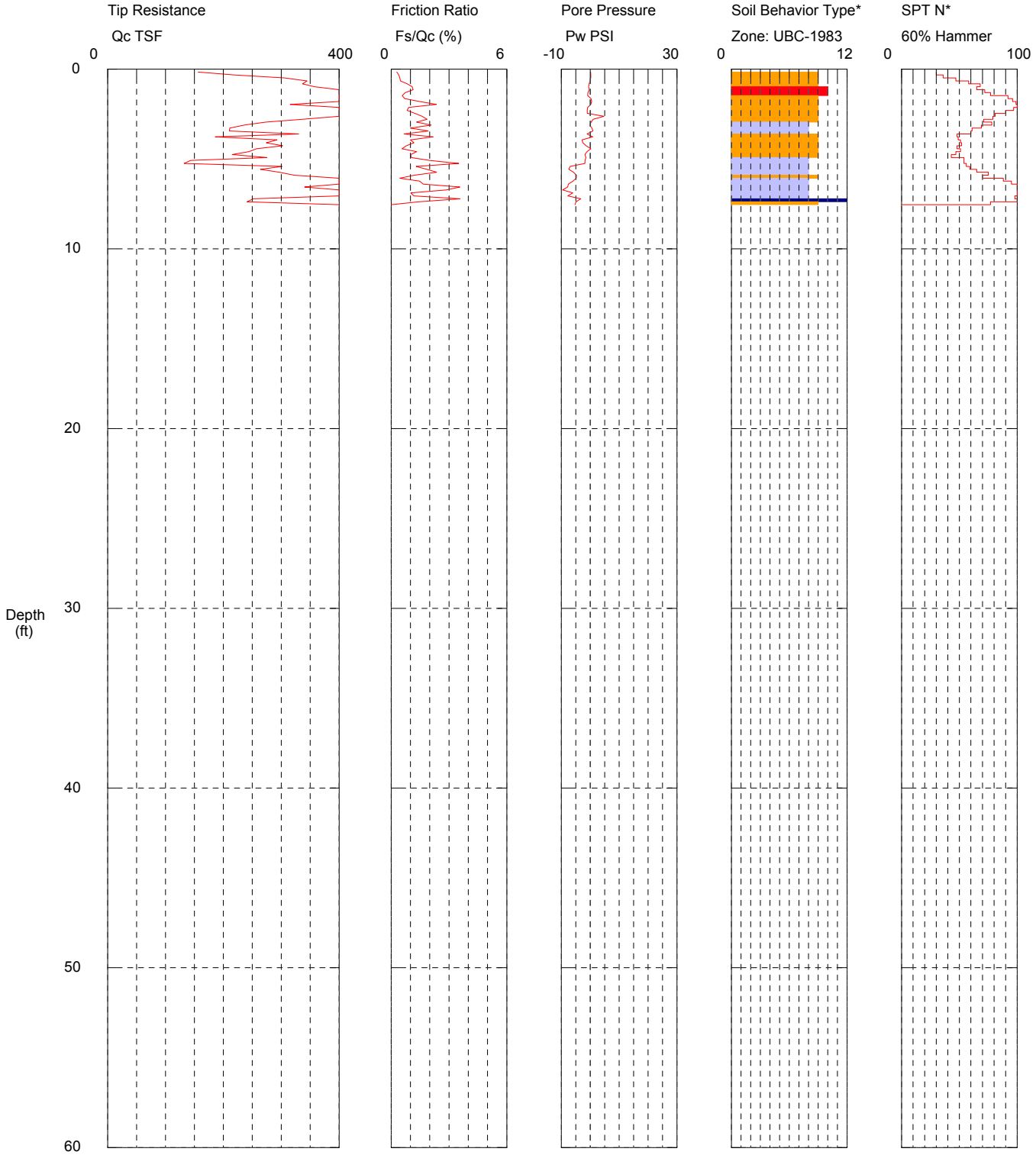
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

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

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-114(s)A
Cone Used: DDG1238

CPT Date/Time: 11/11/2014 3:23:46 PM
Location: Port of Vancouver -Tesoro Distribution Center
Job Number:



Maximum Depth = 7.55 feet

Depth Increment = 0.164 feet

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

InSitu Engineering

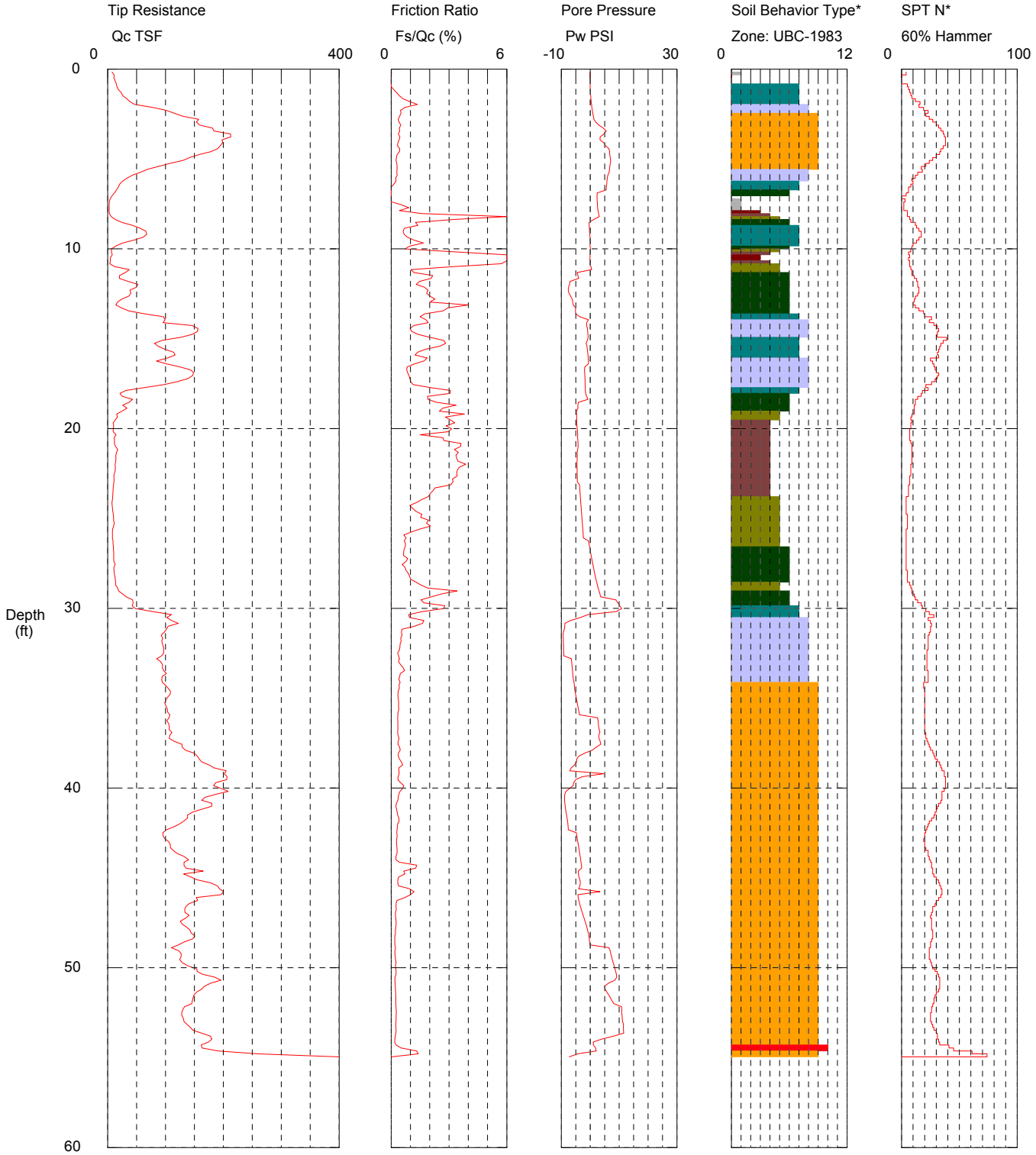
*Soil behavior type and SPT based on data from UBC-1983

EX-0001-006776-PCE

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-115
Cone Used: DDG1254

CPT Date/Time: 11/15/2014 12:37:58 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 54.95 feet

Depth Increment = 0.164 feet

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

InSitu Engineering Predrilled top 13 feet

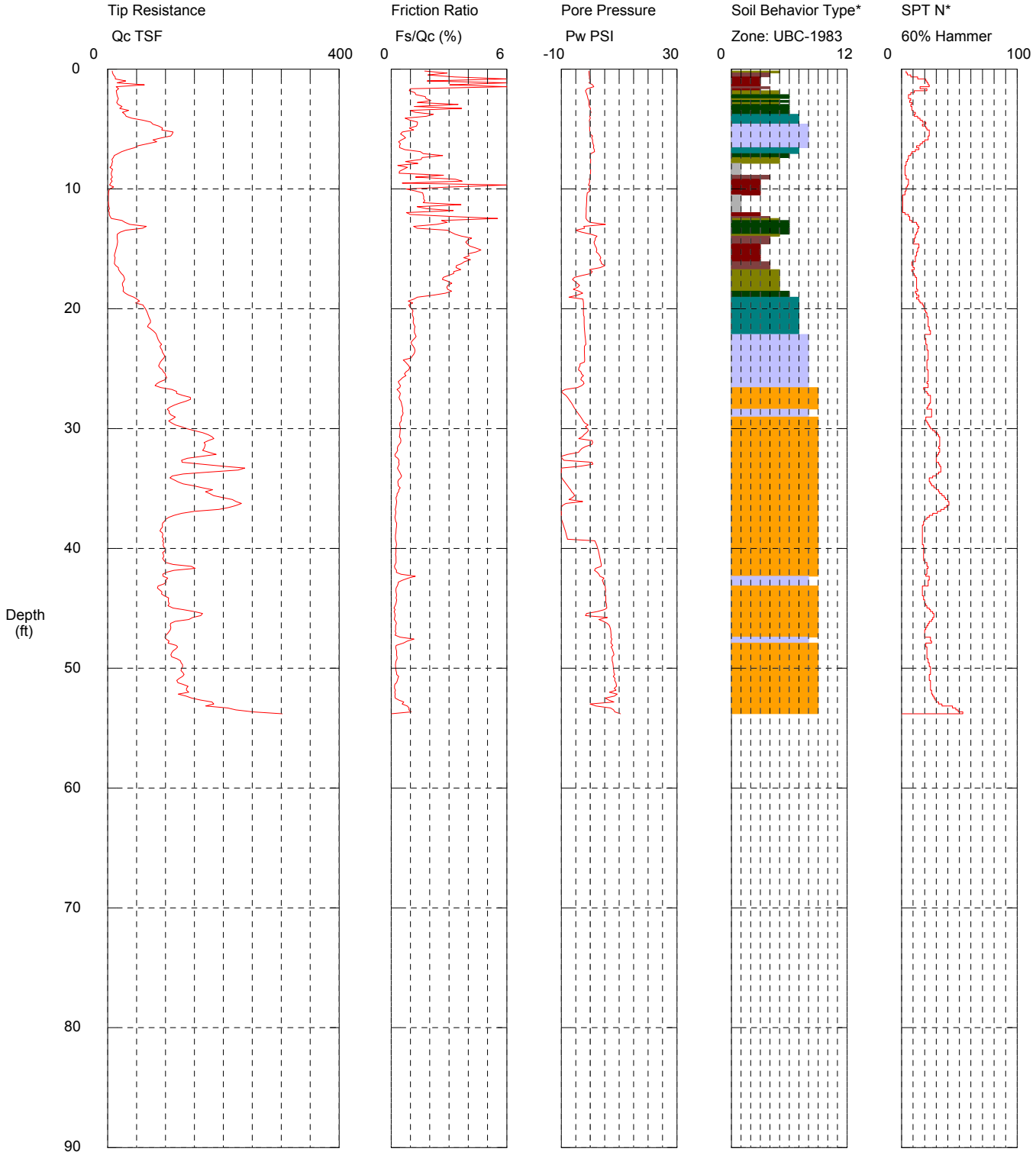
*Soil behavior type and SPT based on data from UBC-1983

Tank 2

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-121
Cone Used: DDG1254

CPT Date/Time: 11/16/2014 11:14:47 AM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 53.81 feet

Depth Increment = 0.164 feet

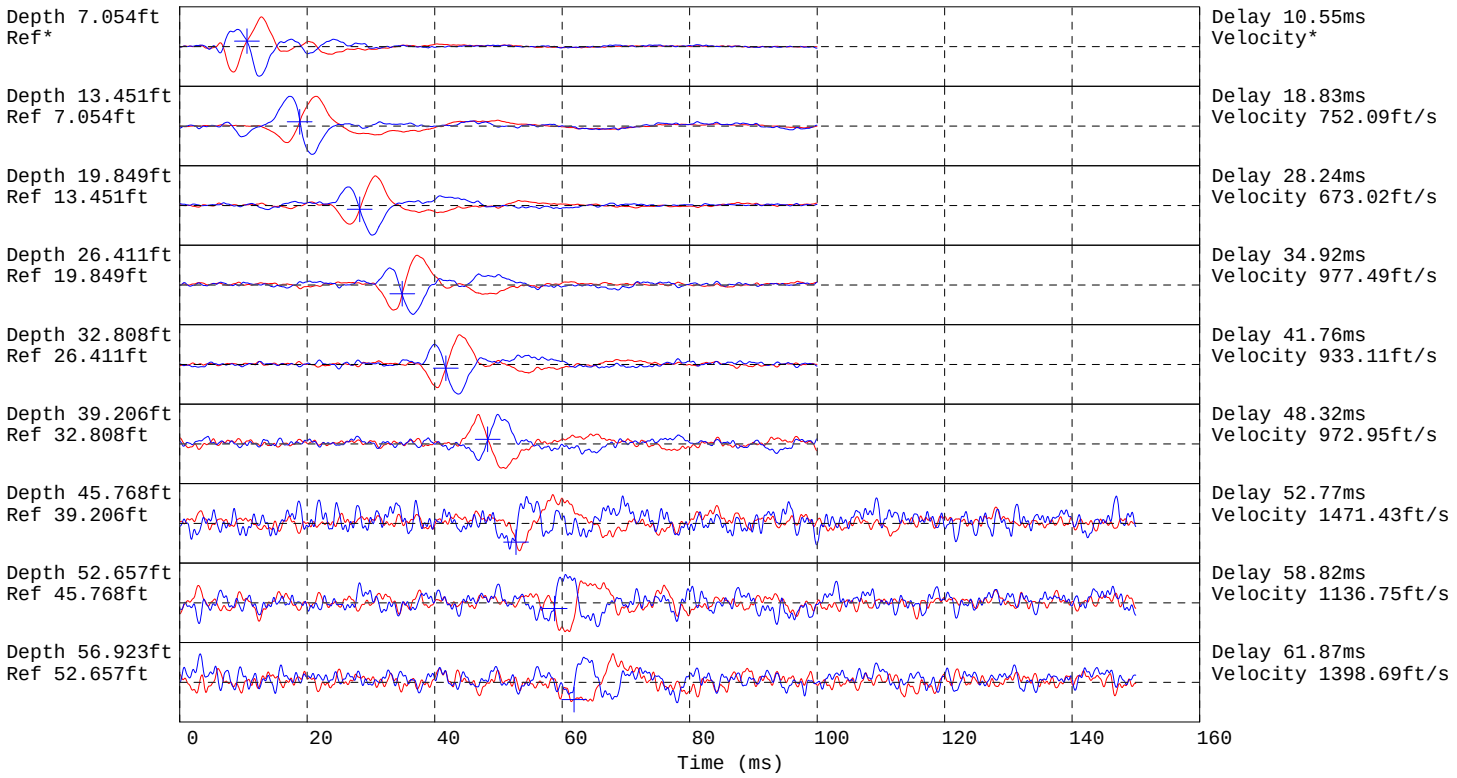
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

InSitu Engineering

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

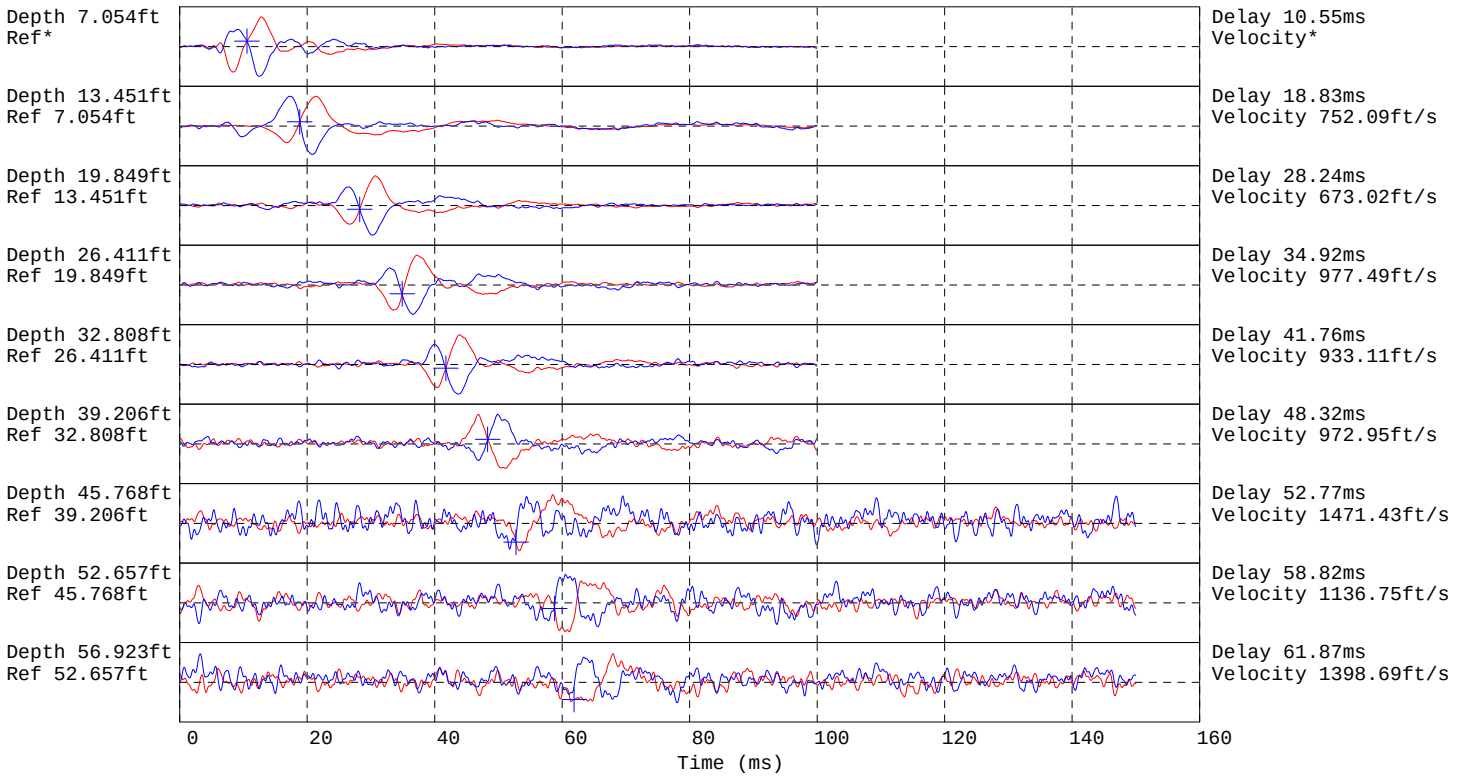
EX-0001-006780-PCE

Tesoro Distribution Terminal
HB-CPT-123(s) Shear Wave Analysis



Hammer to Rod String Distance 0.7 (m)
* = Not Determined

Tesoro Distribution Terminal
HB-CPT-123(s) Shear Wave Analysis

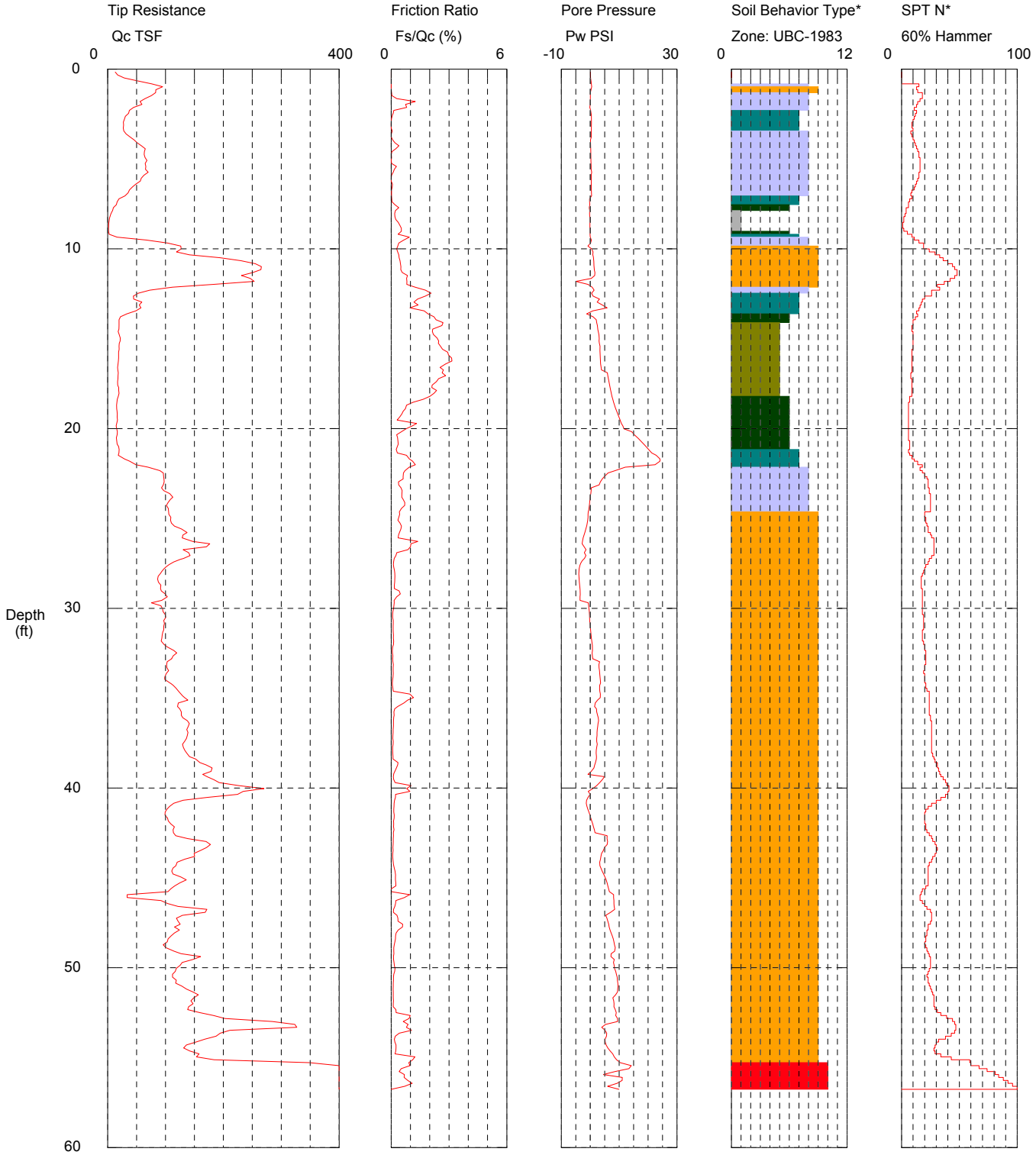


Hammer to Rod String Distance 0.7 (m)
* = Not Determined

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-123(S)
Cone Used: DDG1254

CPT Date/Time: 11/21/2014 2:03:10 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 56.76 feet

Depth Increment = 0.164 feet

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

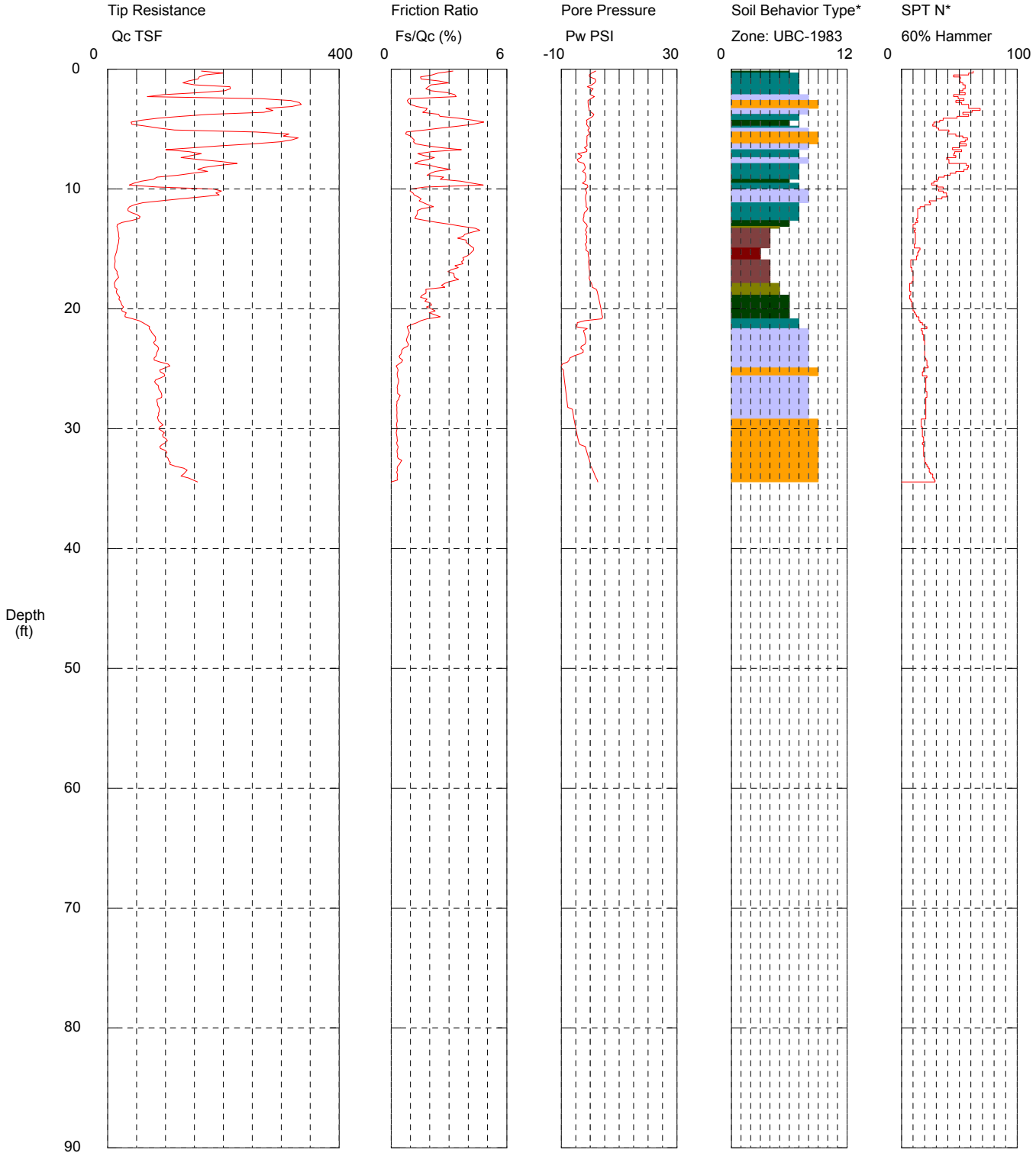
InSitu Engineering Predrilled top 10 feet

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

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-124
Cone Used: DDG1254

CPT Date/Time: 11/16/2014 3:40:09 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 34.45 feet

Depth Increment = 0.164 feet

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

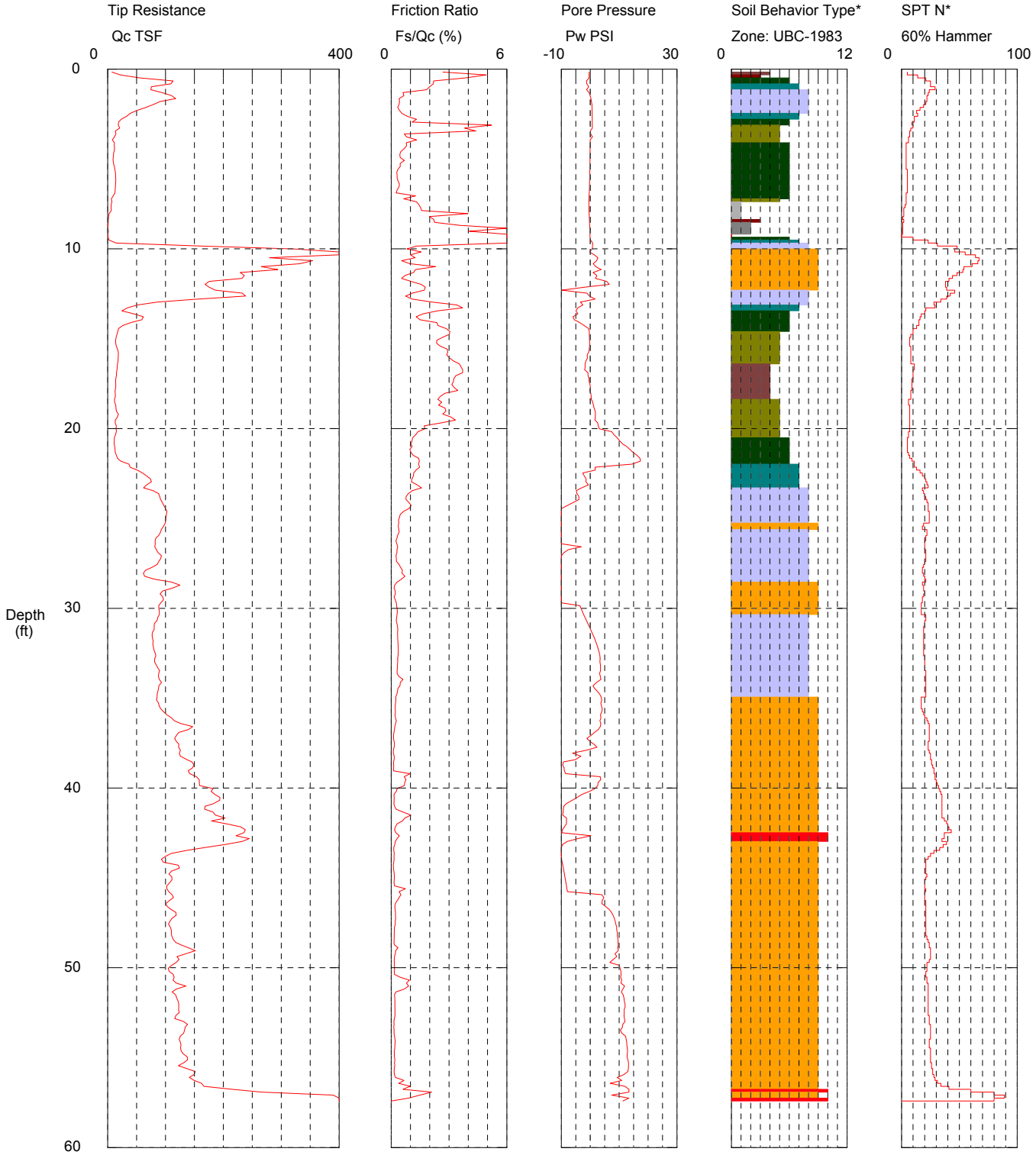
InSitu Engineering Bent rod during push. Terminated early due to bent rod.

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

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-125A
Cone Used: DDG1254

CPT Date/Time: 11/21/2014 9:19:14 AM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 57.41 feet

Depth Increment = 0.164 feet

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

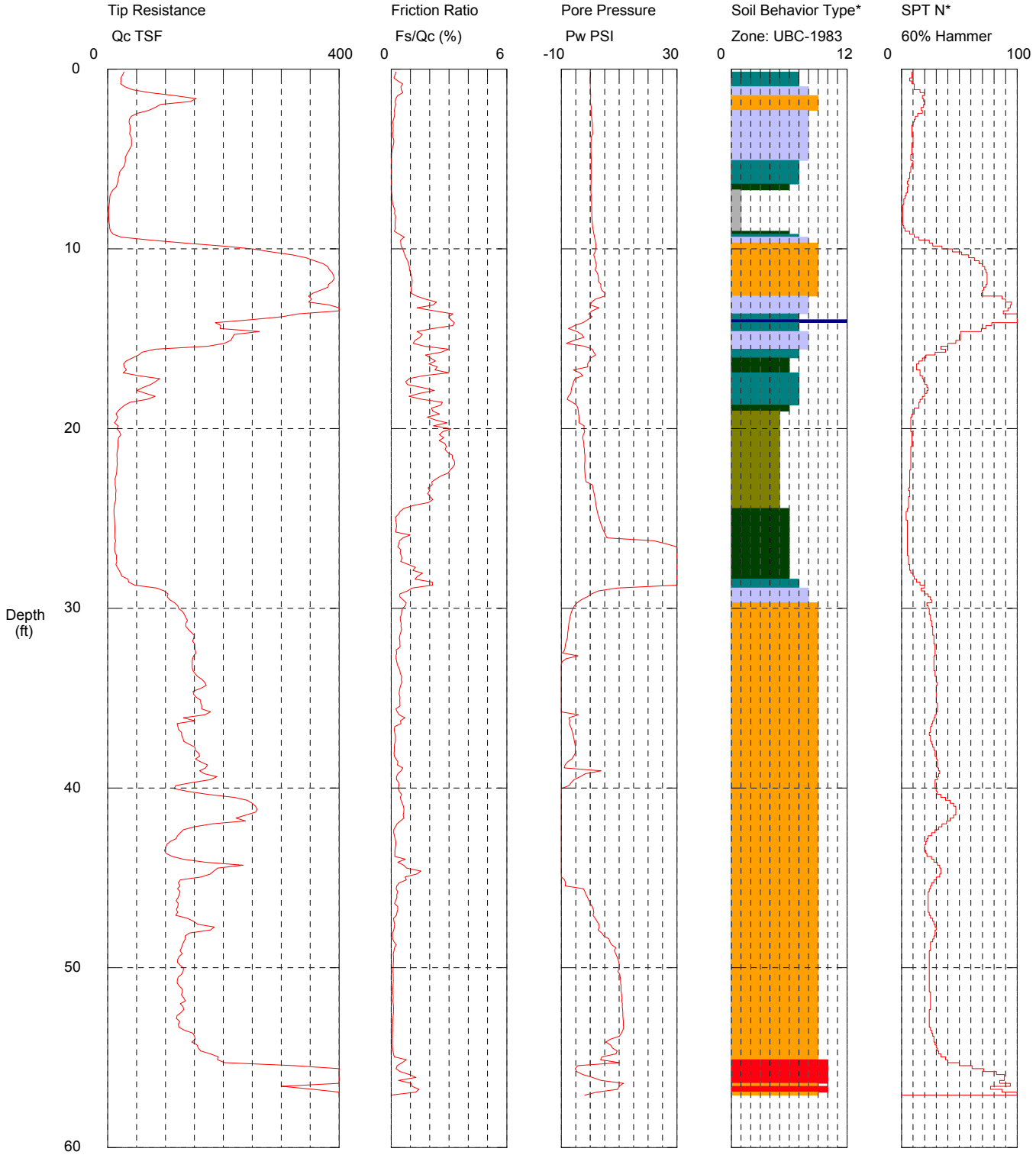
InSitu Engineering Predrilled top 10 feet

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

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-126
Cone Used: DDG1254

CPT Date/Time: 11/21/2014 10:52:24 AM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 57.09 feet

Depth Increment = 0.164 feet

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

InSitu Engineering Predrilled top 10 feet

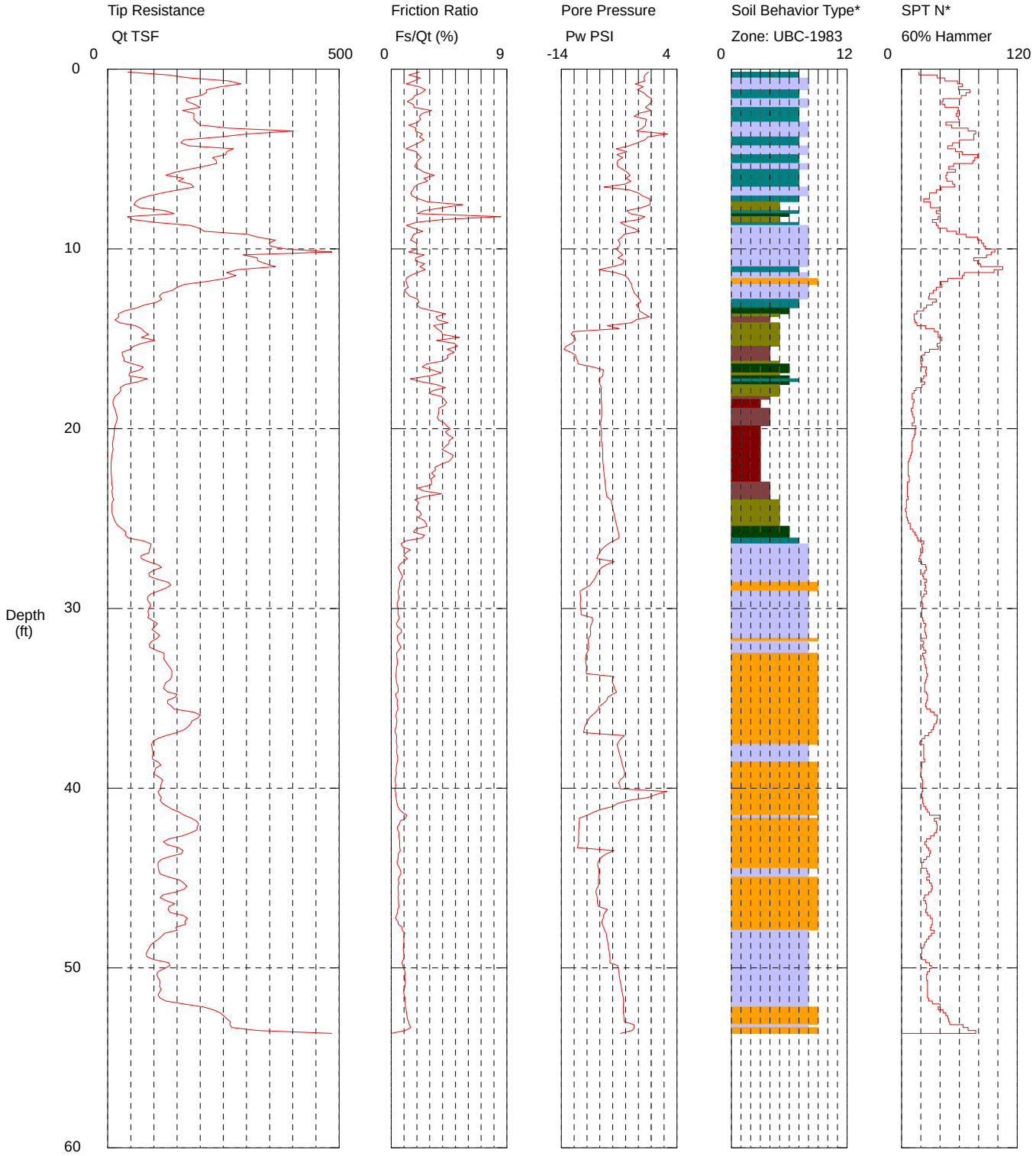
*Soil behavior type and SPT based on data from UBC-1983

Tank 3

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-131
Cone Used: DDG1263

CPT Date/Time: 11/12/2014 2:42:41 PM
Location: Port of Vancouver
Job Number: 860058



Maximum Depth = 53.64 feet

Depth Increment = 0.164 feet

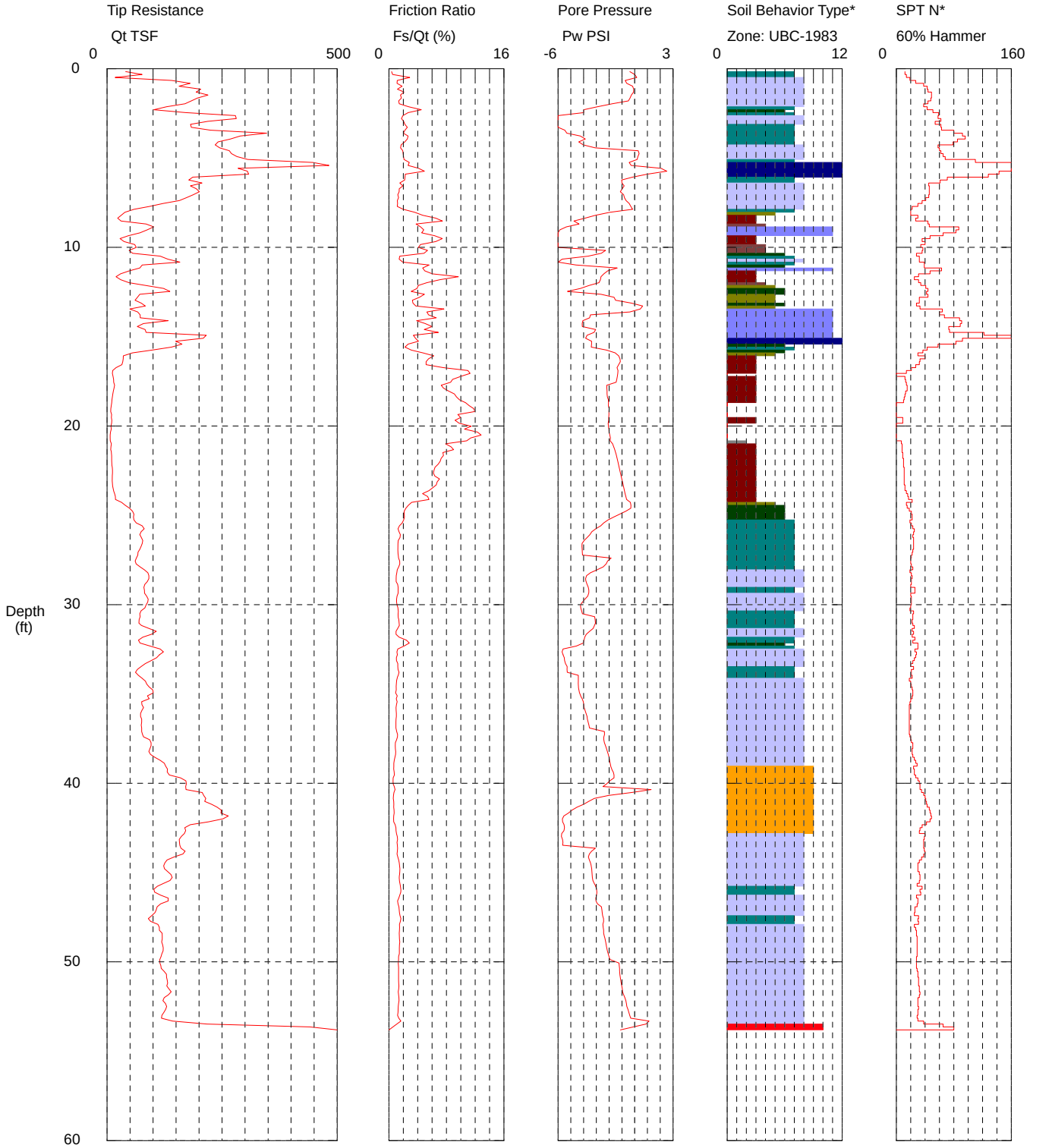
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

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

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-132
Cone Used: DDG1263

CPT Date/Time: 11/12/2014 3:27:24 PM
Location: Port of Vancouver
Job Number: 860058



Maximum Depth = 53.81 feet

Depth Increment = 0.164 feet

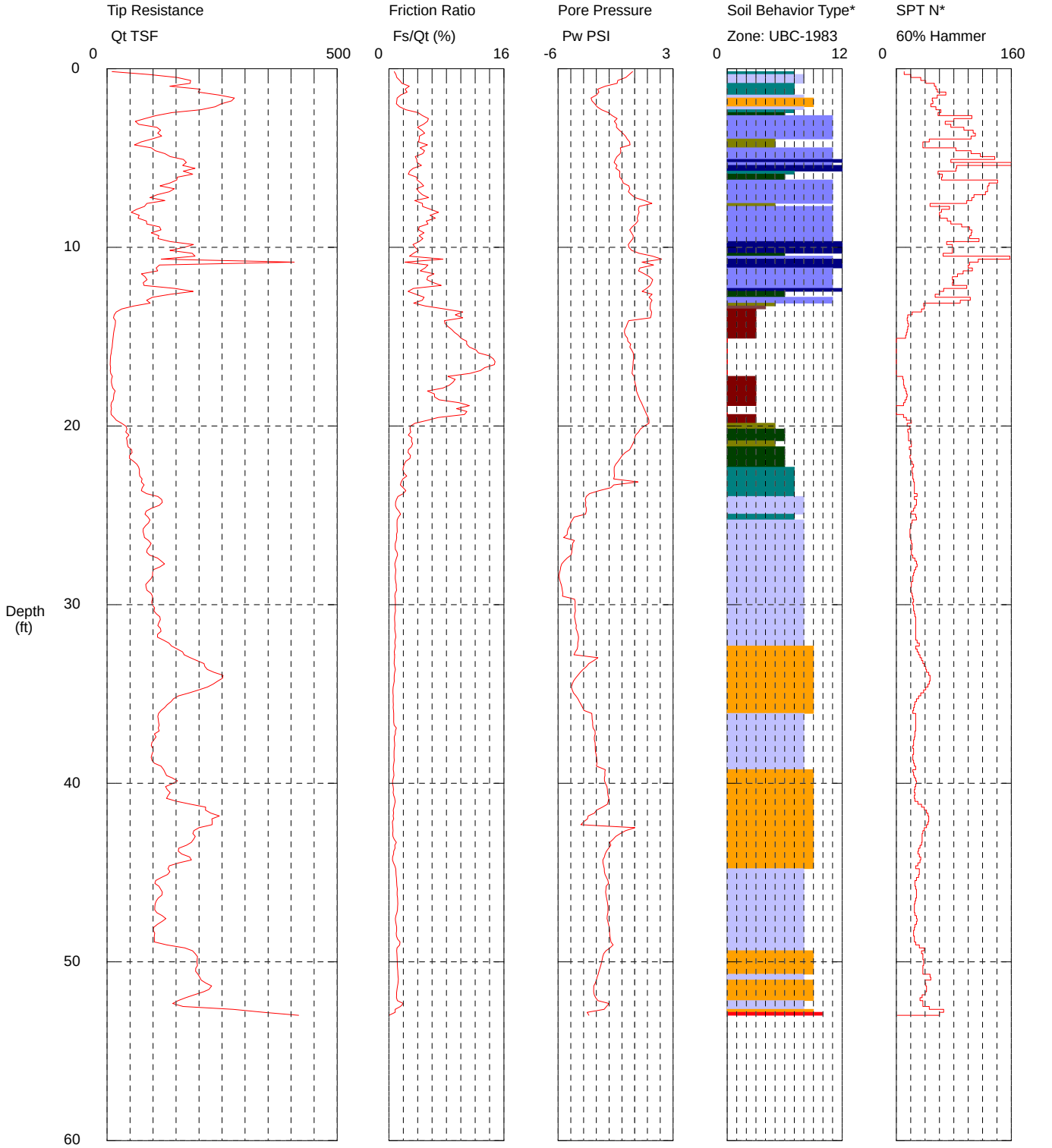
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

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

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-133
Cone Used: DDG1263

CPT Date/Time: 11/12/2014 4:12:29 PM
Location: Port of Vancouver
Job Number: 860058



Maximum Depth = 52.99 feet

Depth Increment = 0.164 feet

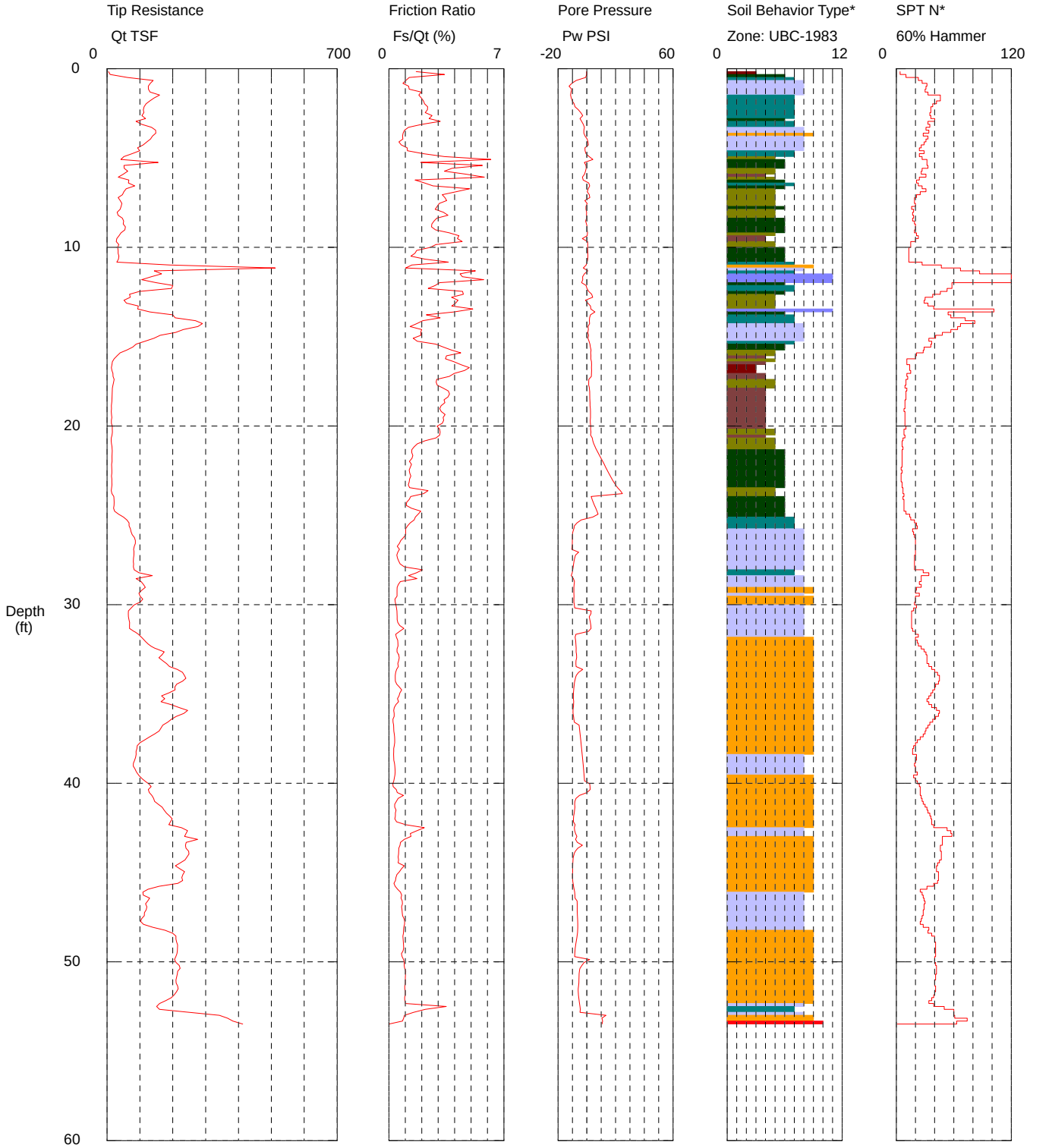
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

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

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-134(s)
Cone Used: DDG1263

CPT Date/Time: 11/11/2014 7:48:33 AM
Location: Port of Vancouver
Job Number: 860058



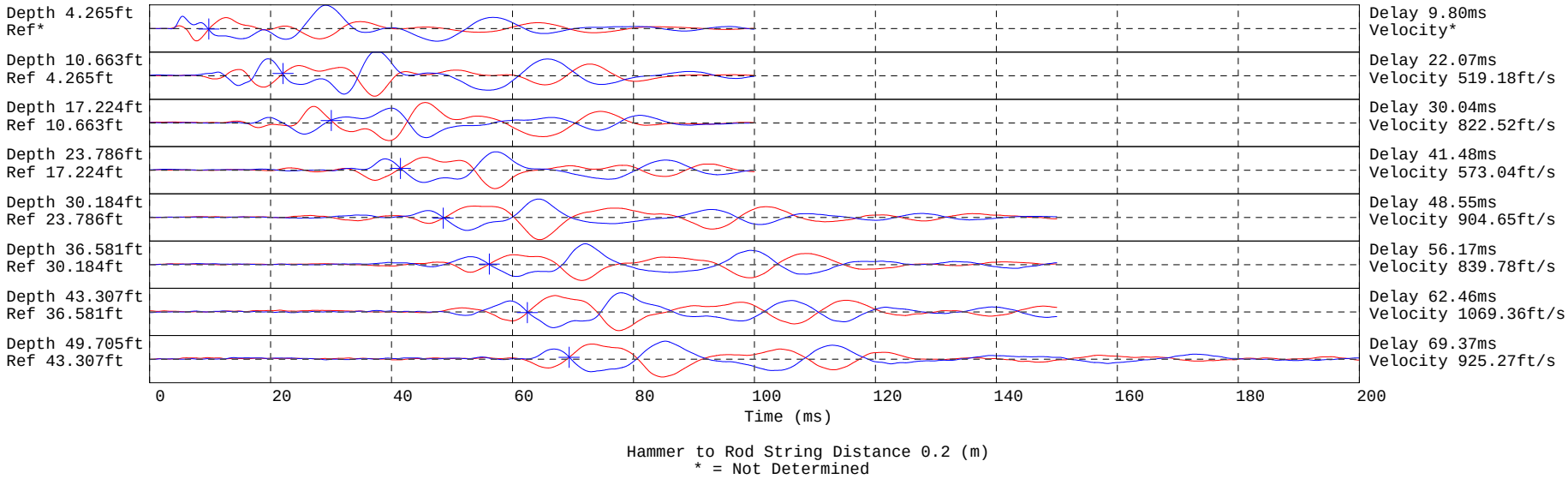
Maximum Depth = 53.48 feet

Depth Increment = 0.164 feet

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

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

Tesoro Distribution Terminal

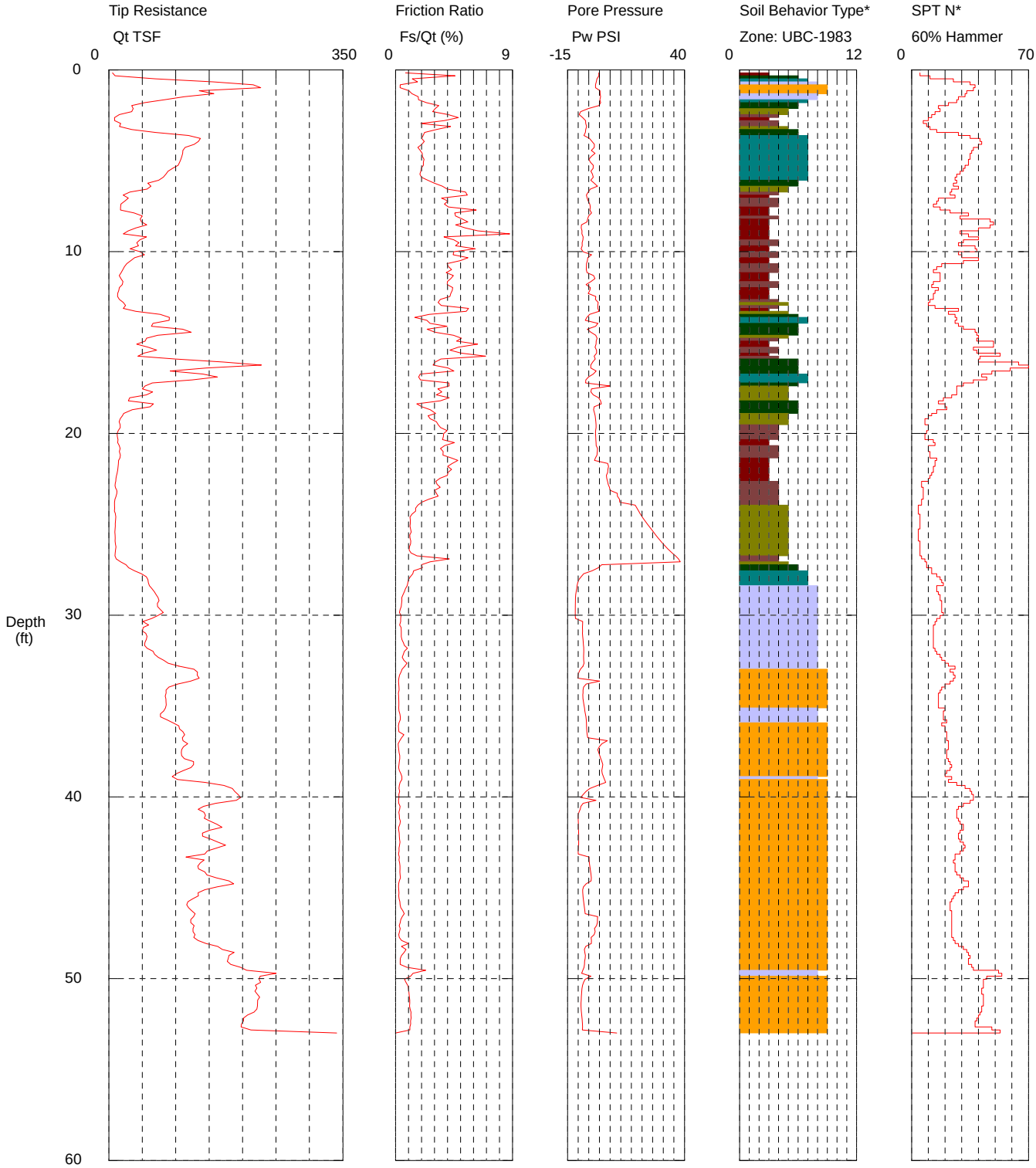


Hammer to Rod String Distance 0.2 (m)
* = Not Determined

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-135
Cone Used: DDG1263

CPT Date/Time: 11/12/2014 1:57:34 PM
Location: Port of Vancouver
Job Number: 860058



Maximum Depth = 52.99 feet

Depth Increment = 0.164 feet

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

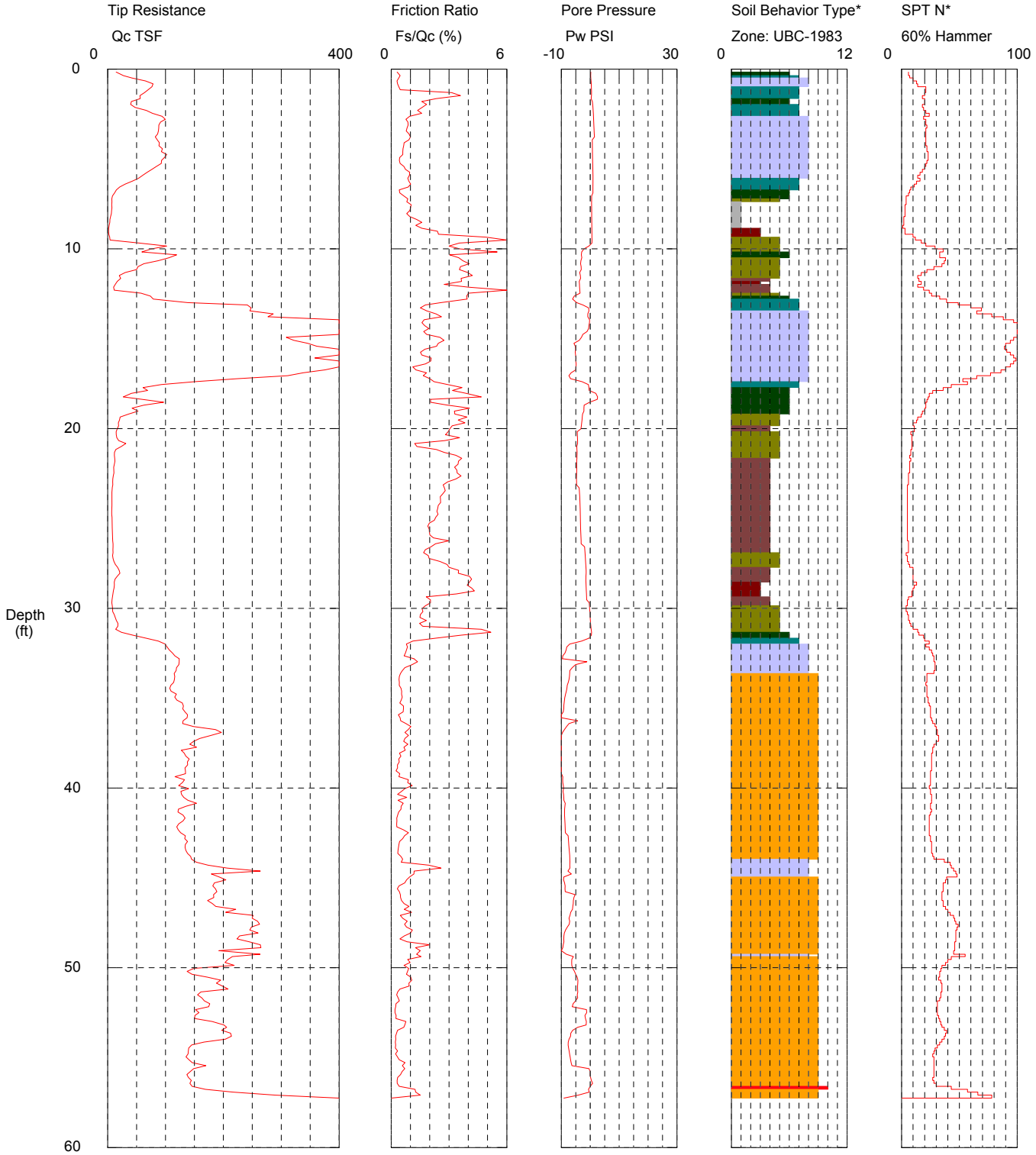
*Soil behavior type and SPT based on data from UBC-1983

Tank 4

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-141
Cone Used: DDG1254

CPT Date/Time: 11/20/2014 1:01:57 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 57.25 feet

Depth Increment = 0.197 feet

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

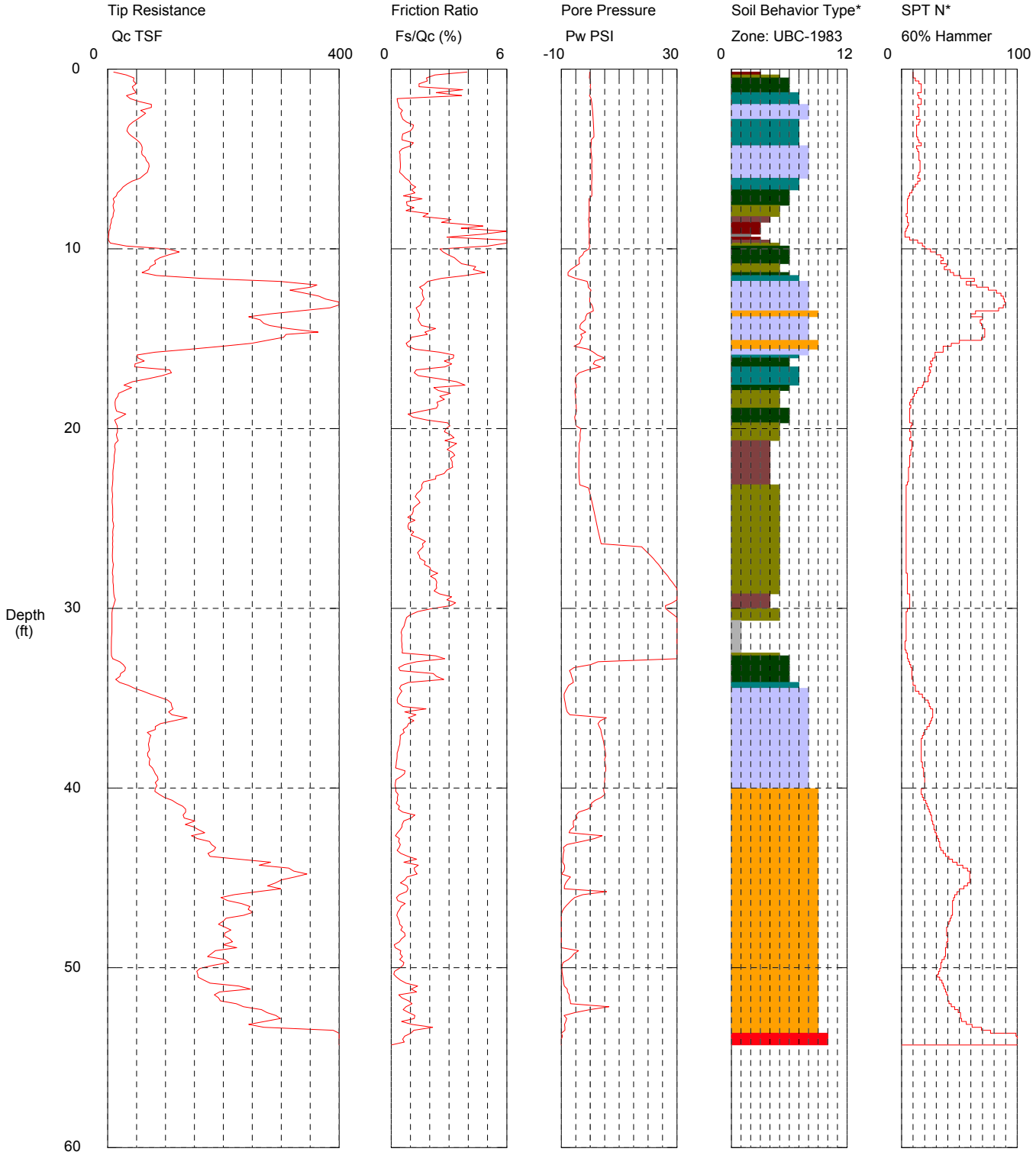
InSitu Engineering Predrilled top 10 feet

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

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-142
Cone Used: DDG1254

CPT Date/Time: 11/20/2014 8:31:51 AM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 54.30 feet

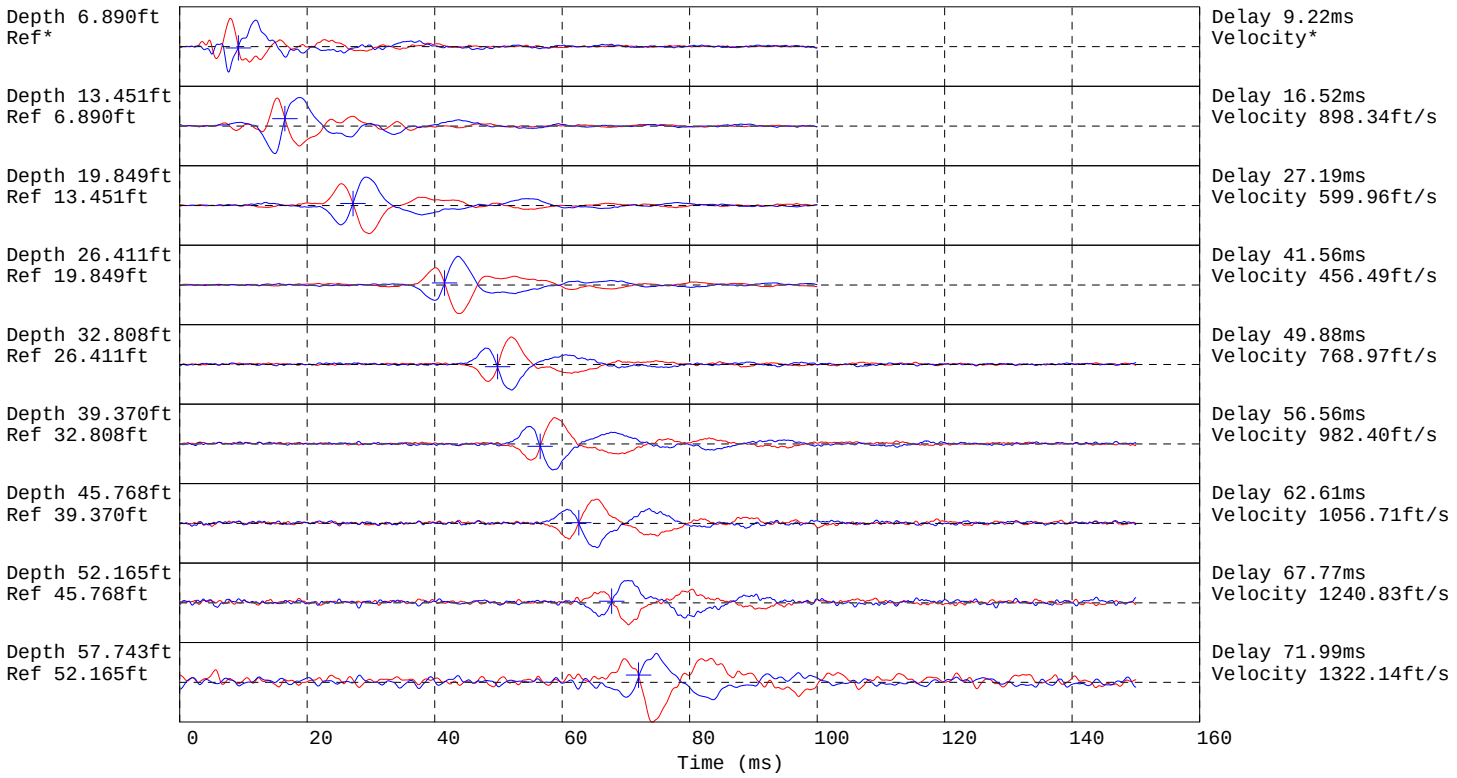
Depth Increment = 0.164 feet

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

InSitu Engineering Predrilled top 10 feet

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

Tesoro Distribution Terminal
HB-CPT-143(s) Shear Wave Analysis

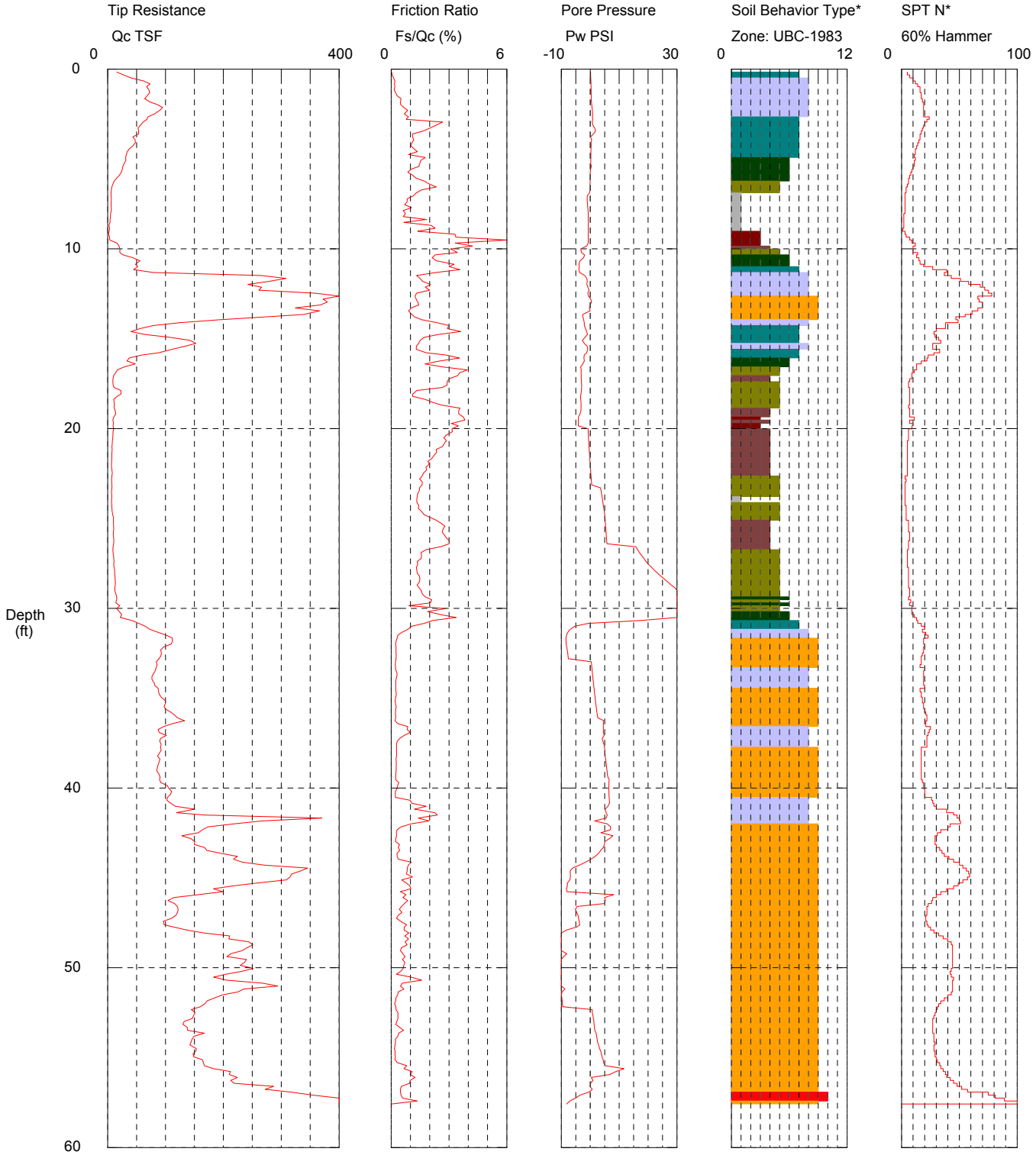


Hammer to Rod String Distance 0 (m)
* = Not Determined

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-143(S)
Cone Used: DDG1254

CPT Date/Time: 11/20/2014 10:53:36 AM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 57.58 feet

Depth Increment = 0.164 feet

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

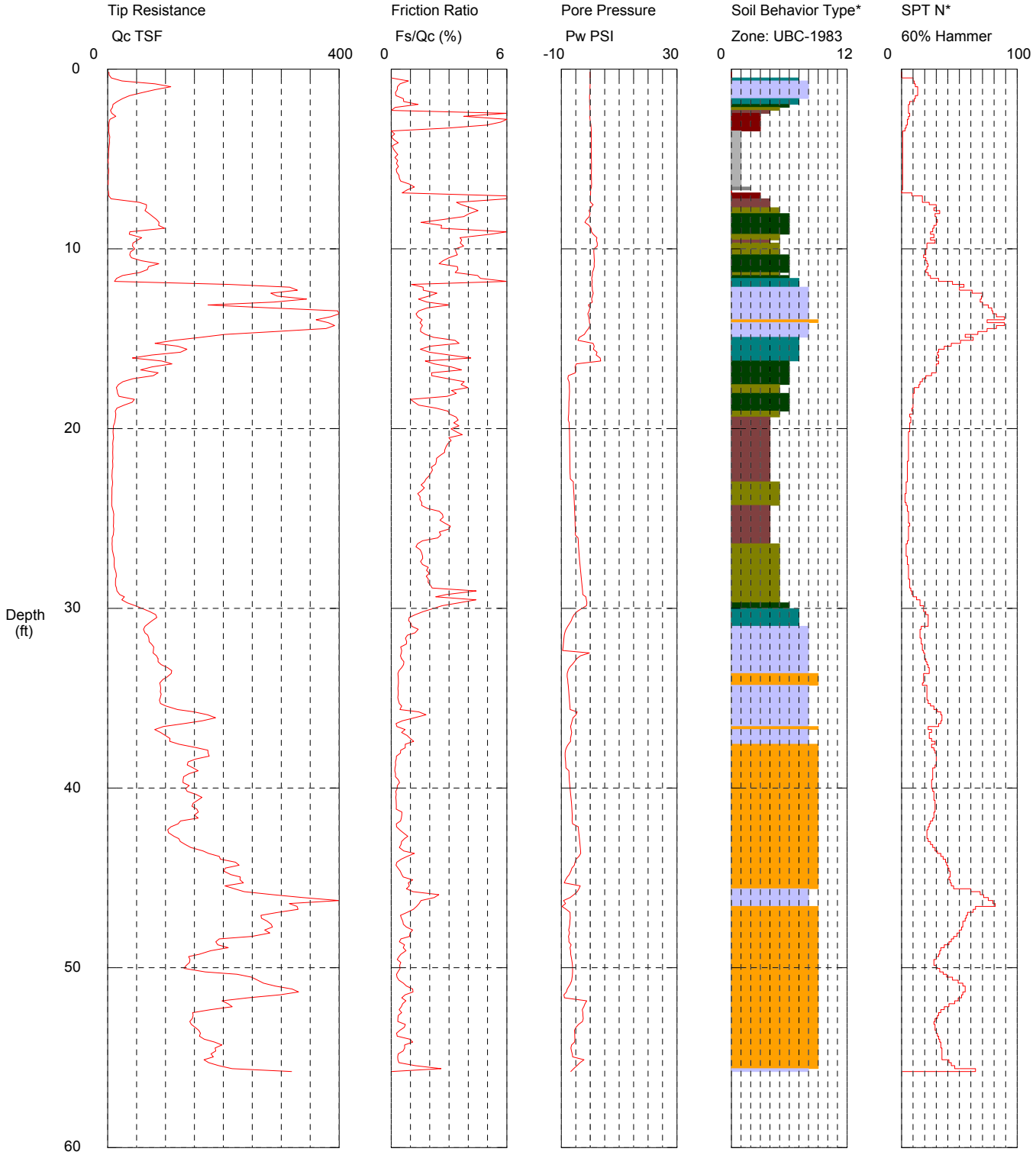
InSitu Engineering Predrilled top 10 feet

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

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-144
Cone Used: DDG1254

CPT Date/Time: 11/20/2014 3:48:24 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 55.77 feet

Depth Increment = 0.164 feet

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

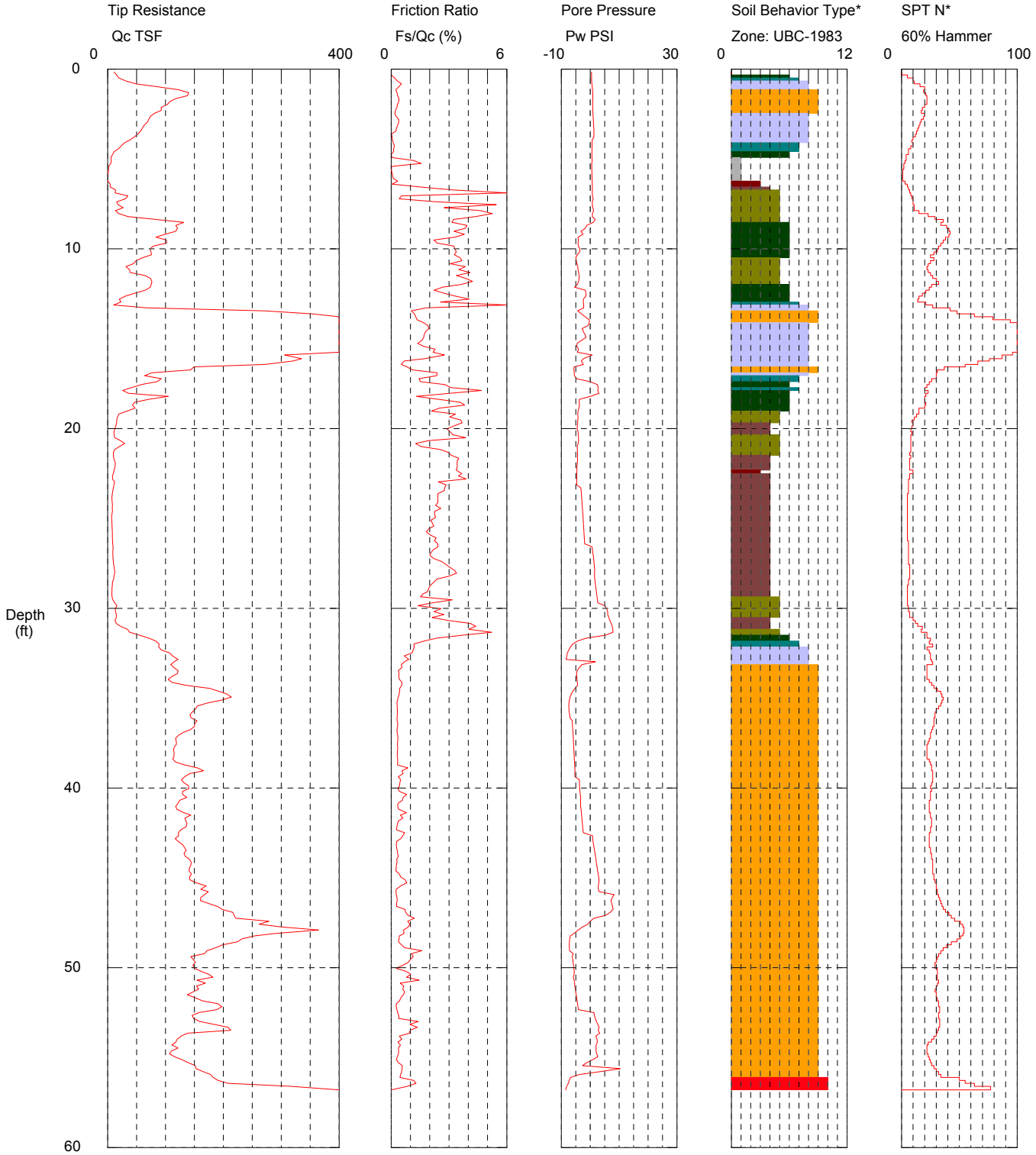
InSitu Engineering Predrilled top 7 feet

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

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-145
Cone Used: DDG1254

CPT Date/Time: 11/20/2014 2:26:23 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 56.79 feet

Depth Increment = 0.164 feet

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

InSitu Engineering Predrilled top 9 feet

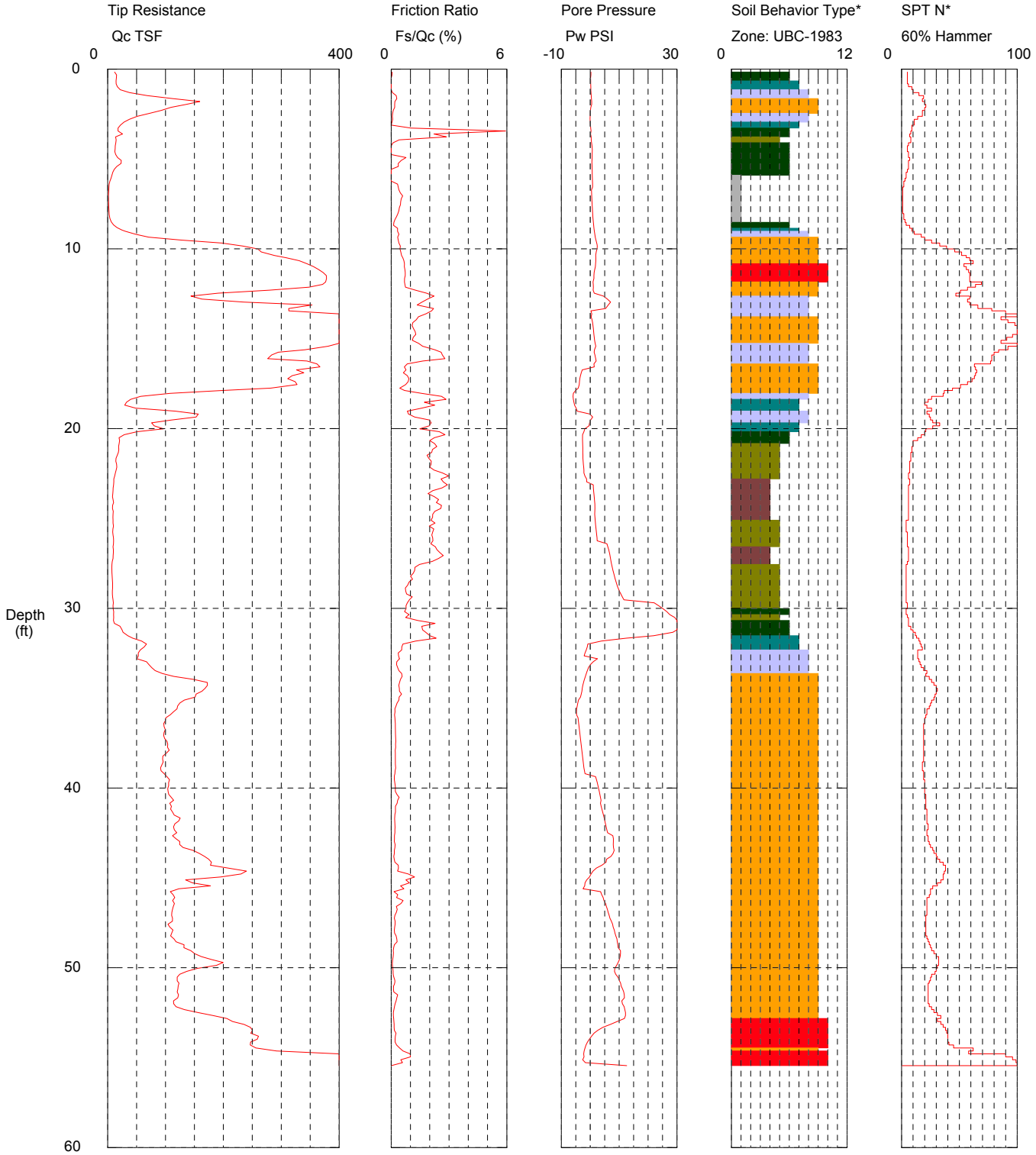
*Soil behavior type and SPT based on data from UBC-1983

Tank 5

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-151
Cone Used: DDG1254

CPT Date/Time: 11/21/2014 4:04:51 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 55.45 feet

Depth Increment = 0.164 feet

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

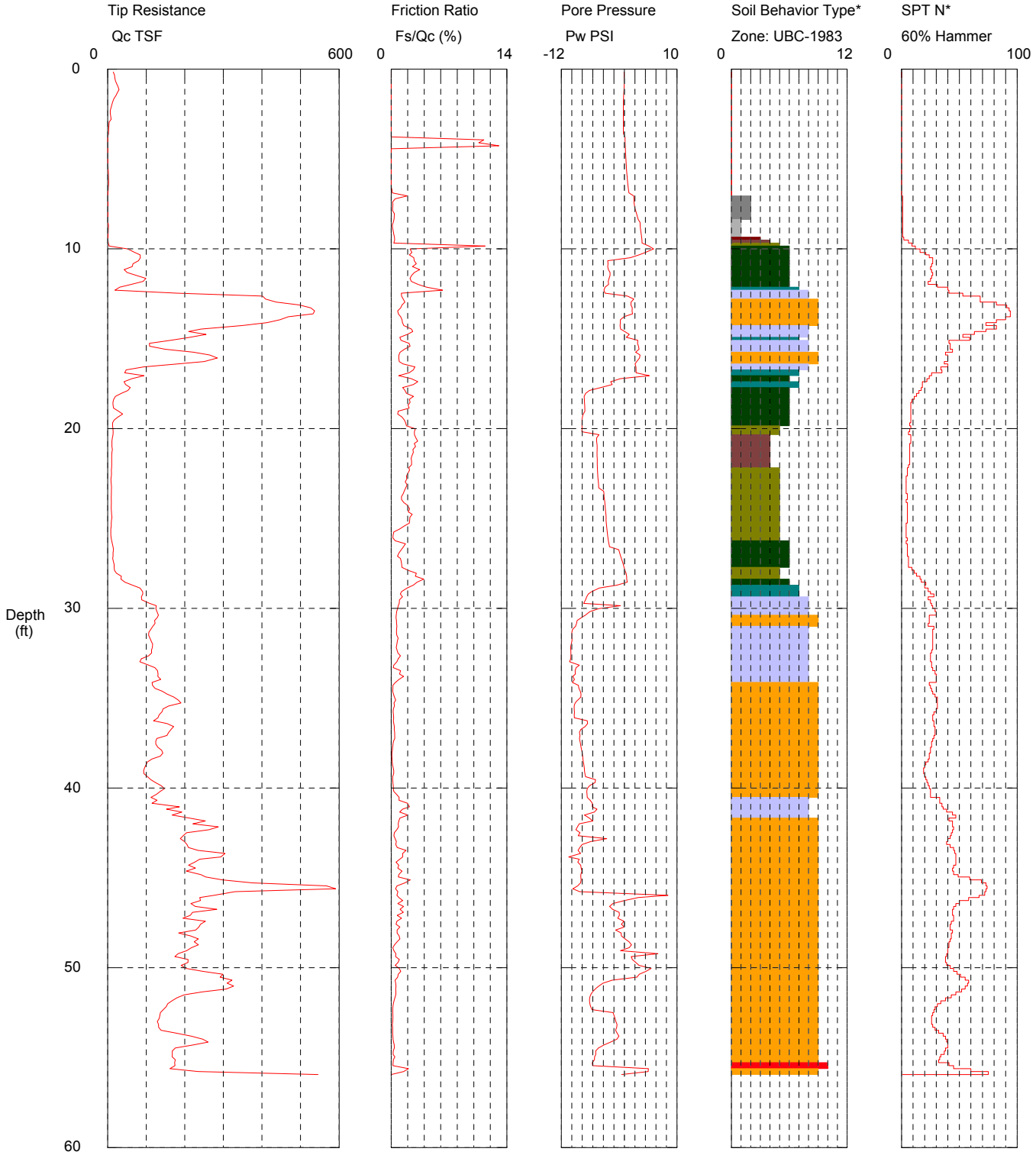
InSitu Engineering Predrilled top 9 feet

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

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-152A
Cone Used: DDG1254

CPT Date/Time: 11/22/2014 2:10:05 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 55.94 feet

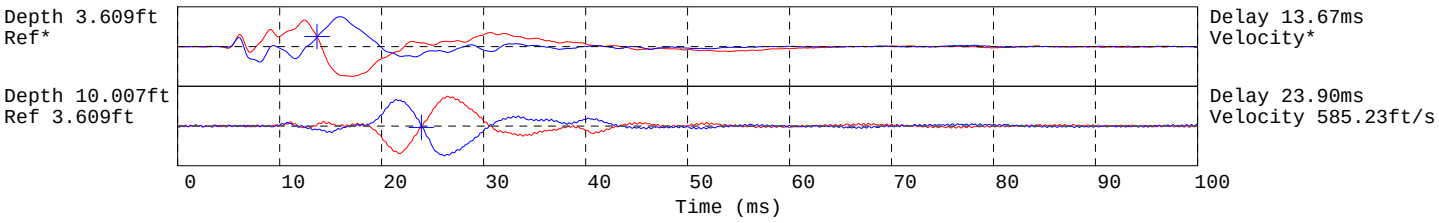
Depth Increment = 0.164 feet

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

InSitu Engineering Predrilled top 13 feet

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

Tesoro Distribution Terminal
HB-CPT-153(s)

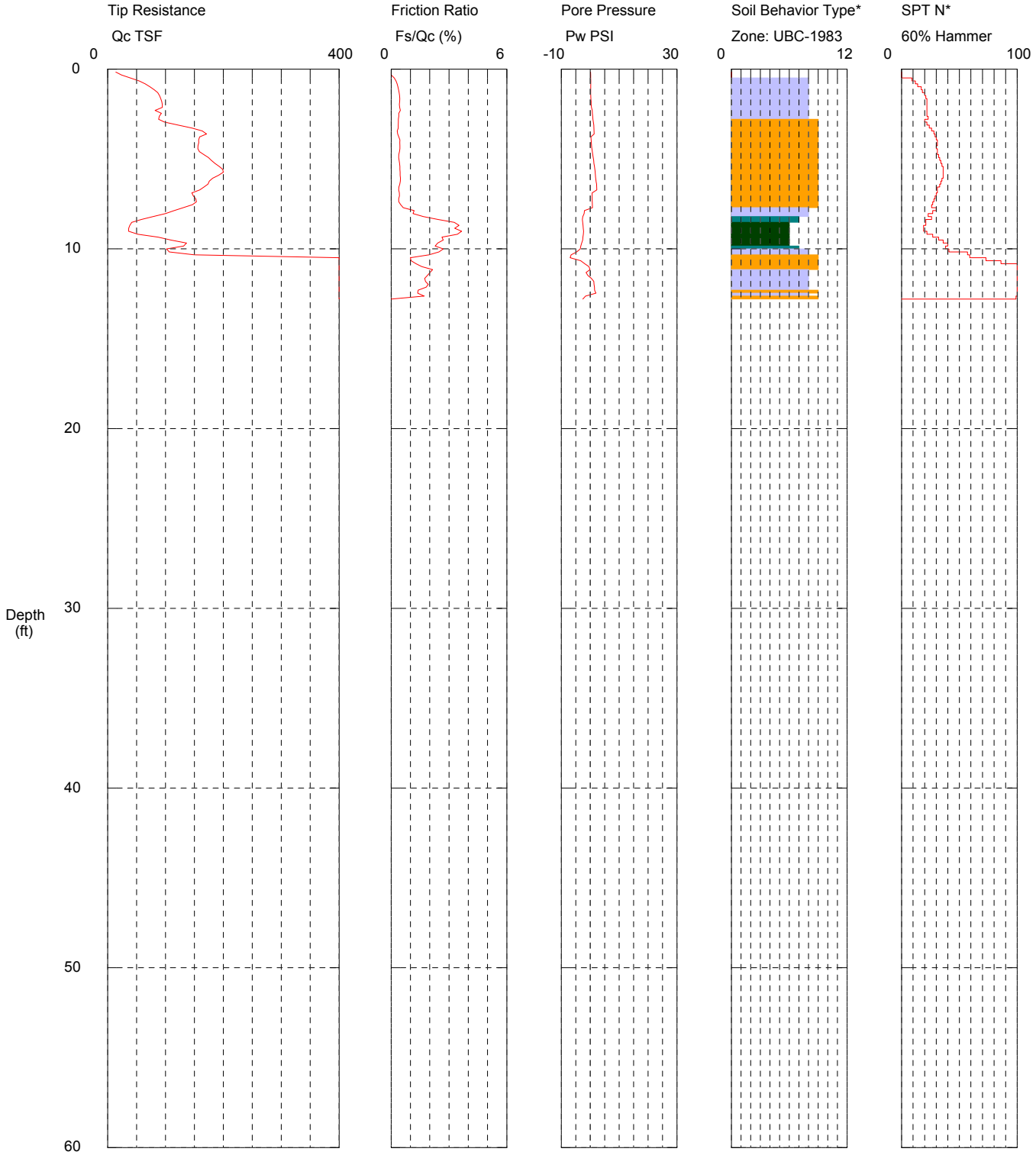


Hammer to Rod String Distance 0.7 (m)
* = Not Determined

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-153(s)
Cone Used: DDG1254

CPT Date/Time: 11/12/2014 12:16:48 PM
Location: Port of Vancouver -Tesoro Distribution Center
Job Number:



Maximum Depth = 12.80 feet

Depth Increment = 0.164 feet

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

InSitu Engineering

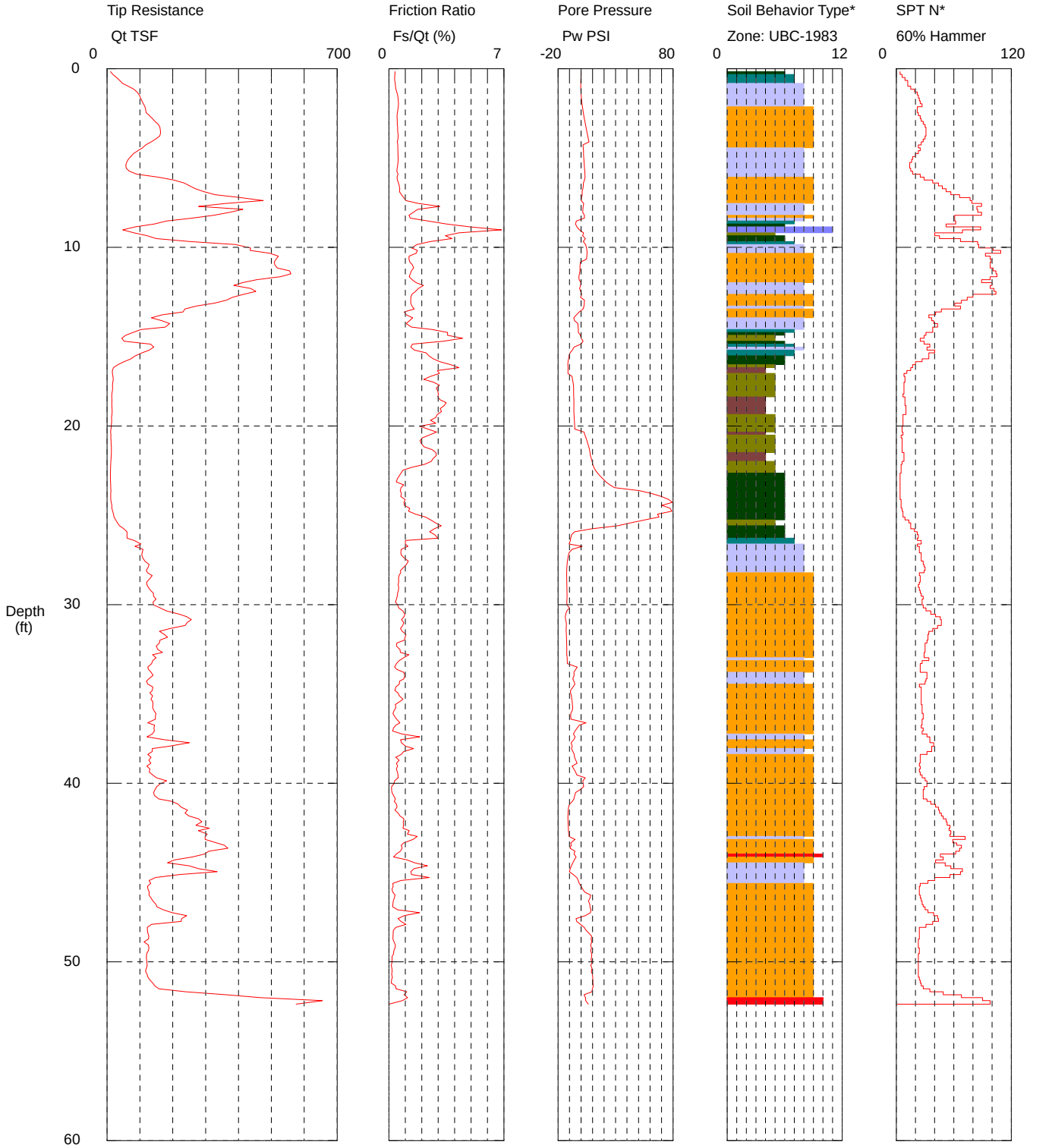
*Soil behavior type and SPT based on data from UBC-1983

EX-0001-006807-PCE

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-154
Cone Used: DDG1254

CPT Date/Time: 11/12/2014 8:31:19 AM
Location: Port of Vancouver
Job Number: 860058



Maximum Depth = 52.36 feet

Depth Increment = 0.164 feet

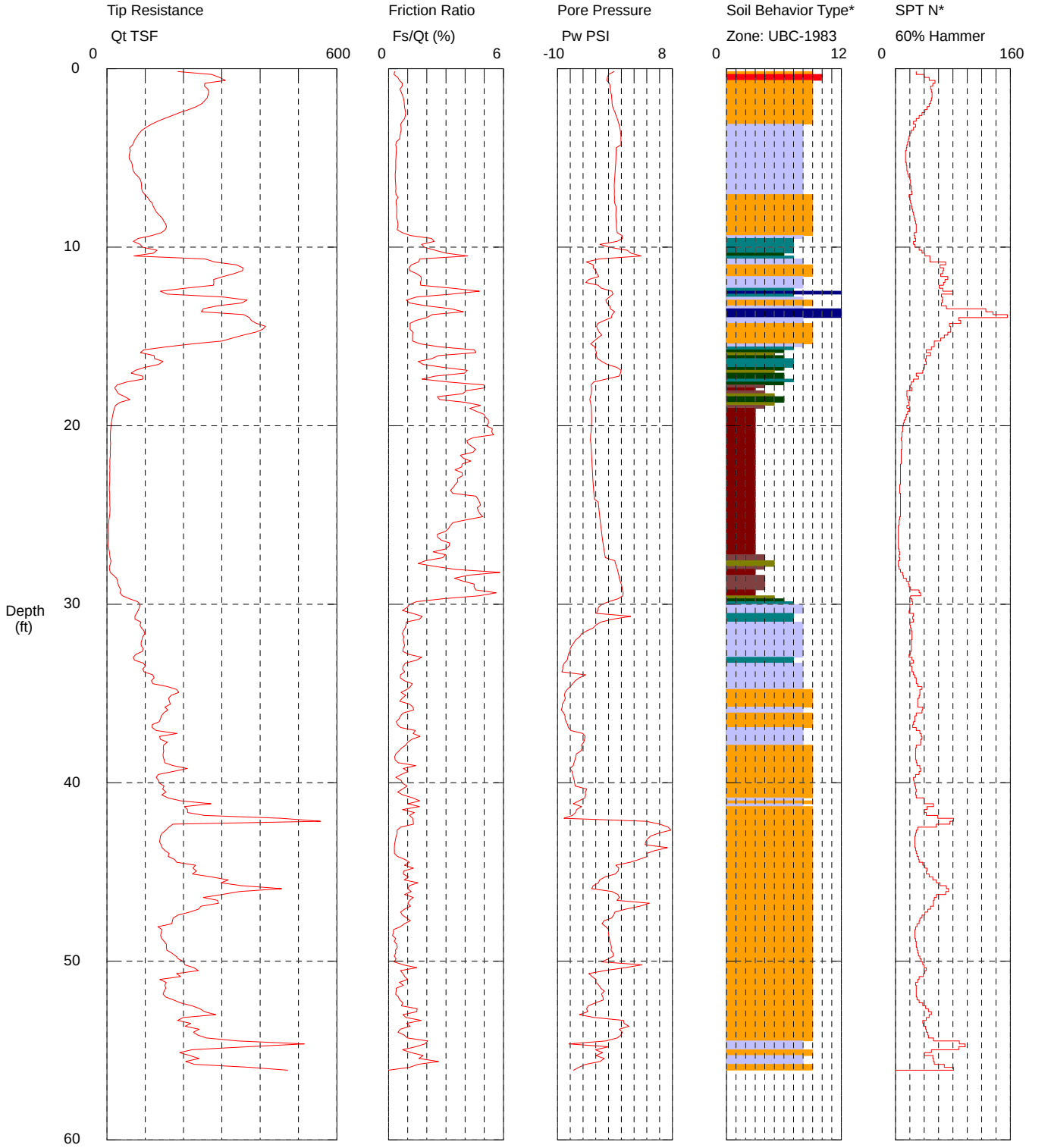
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

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

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-155
Cone Used: DDG1263

CPT Date/Time: 11/10/2014 2:57:41 PM
Location: Port of Vancouver
Job Number: 860058



Maximum Depth = 56.10 feet

Depth Increment = 0.164 feet

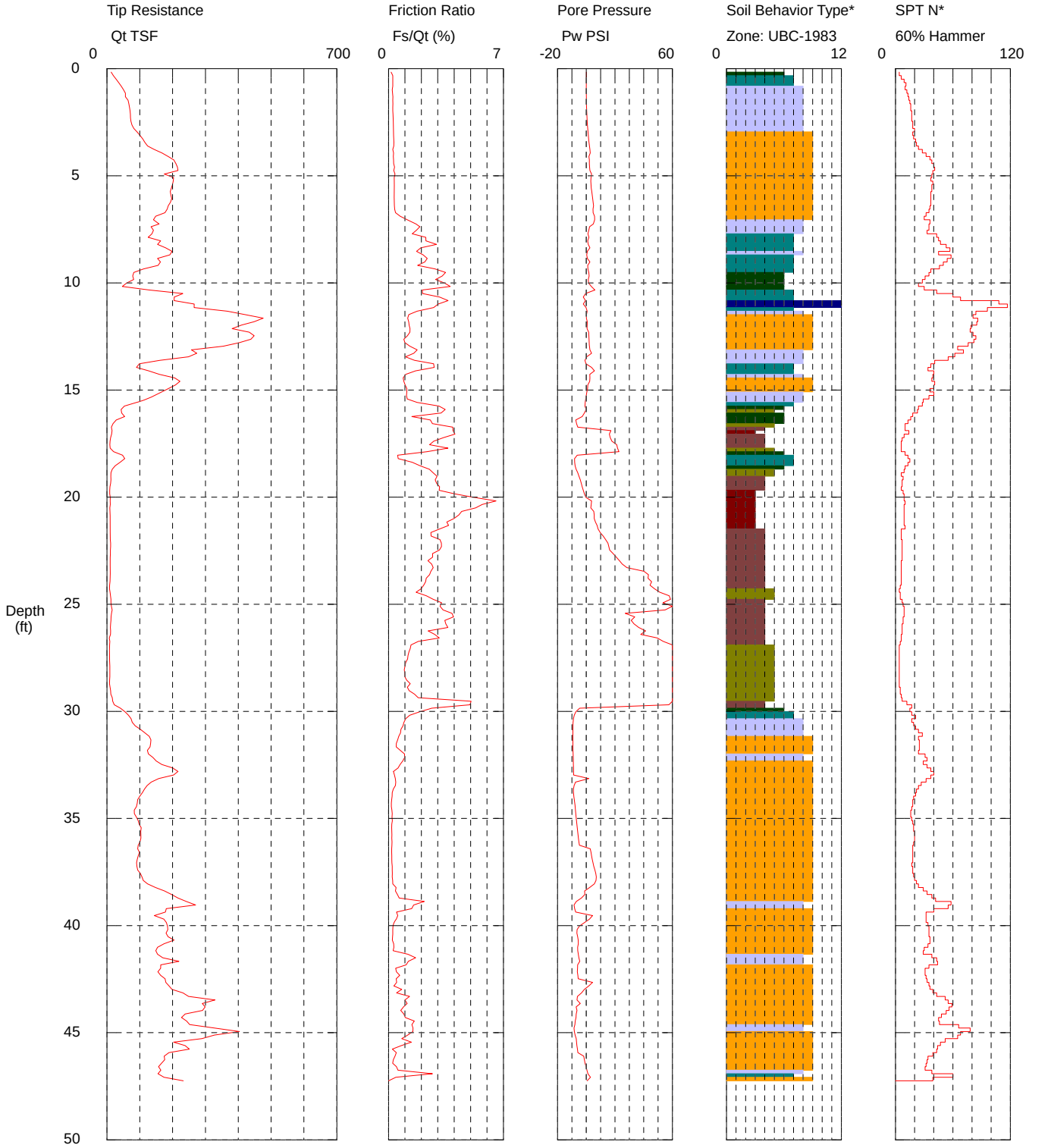
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

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

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-156
Cone Used: DDG1254

CPT Date/Time: 11/11/2014 5:27:30 PM
Location: Port of Vancouver
Job Number: 860058



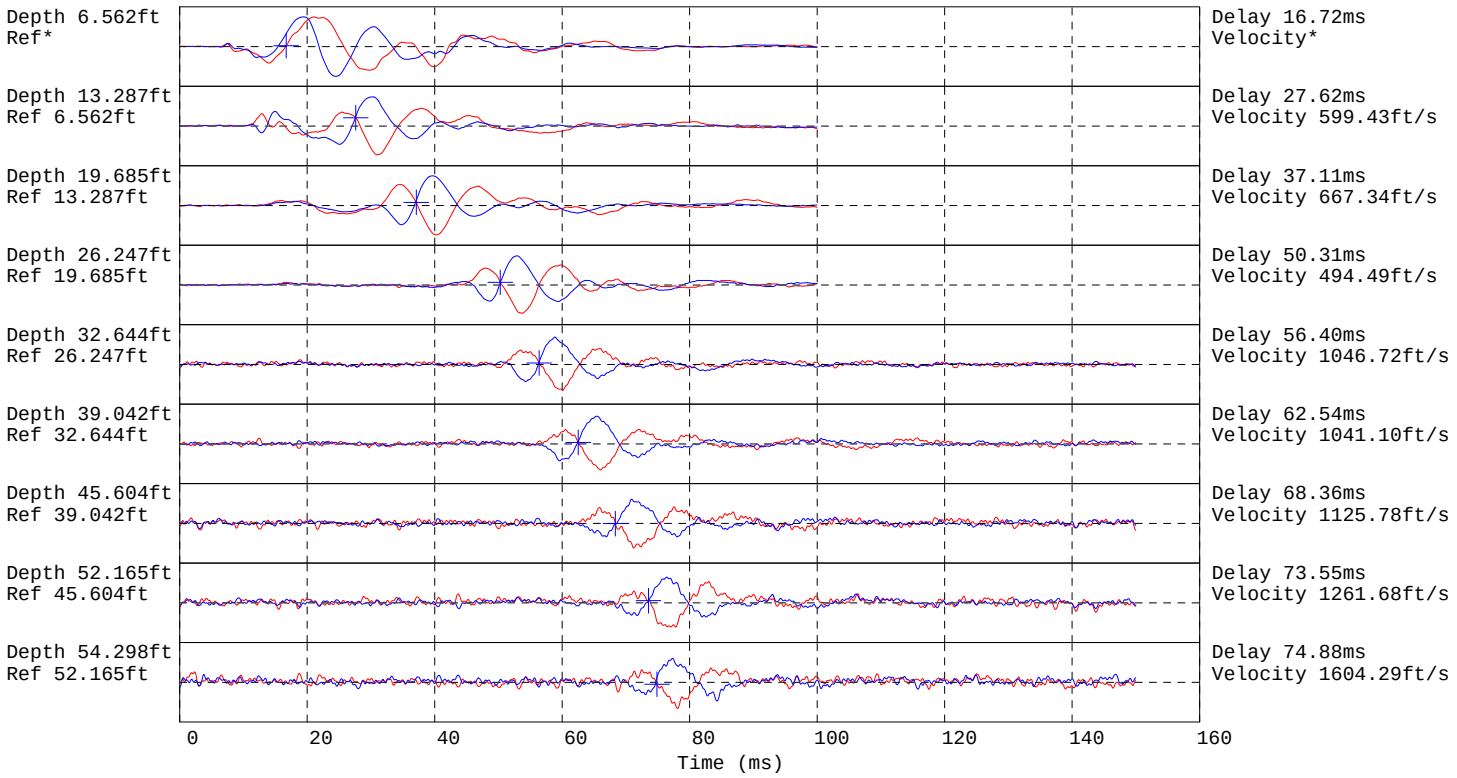
Maximum Depth = 47.24 feet

Depth Increment = 0.164 feet

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

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

Tesoro Distribution Terminal
HB-CPT-153(s)A Shear Wave Analysis



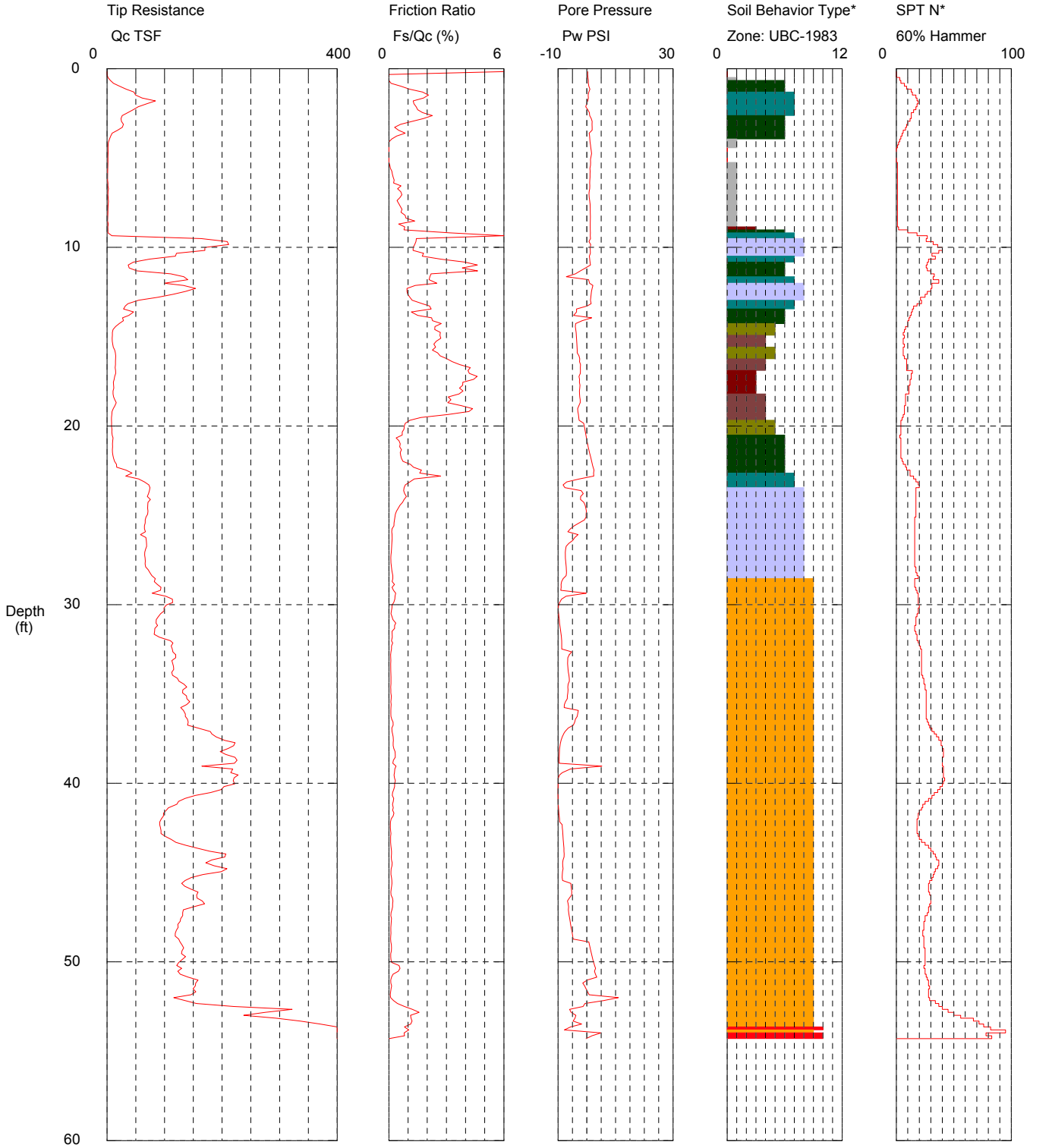
Hammer to Rod String Distance 0.7 (m)
* = Not Determined

Tank 6

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-161
Cone Used: DDG1254

CPT Date/Time: 11/22/2014 10:28:12 AM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 54.30 feet

Depth Increment = 0.164 feet

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

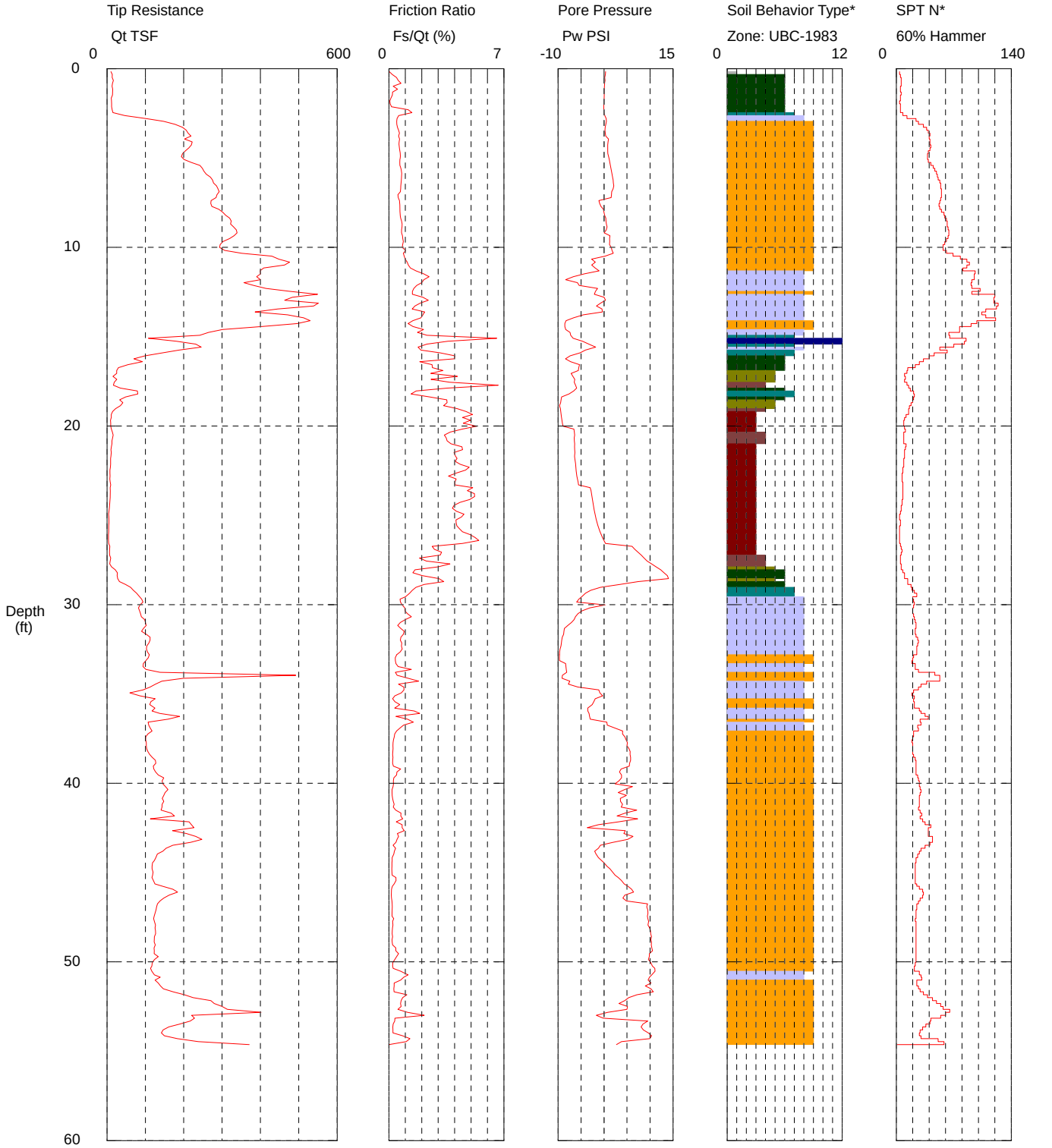
InSitu Engineering Predrilled top 10 feet

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

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-162
Cone Used: DDG1254

CPT Date/Time: 11/12/2014 9:20:32 AM
Location: Port of Vancouver
Job Number: 860058



Maximum Depth = 54.63 feet

Depth Increment = 0.164 feet

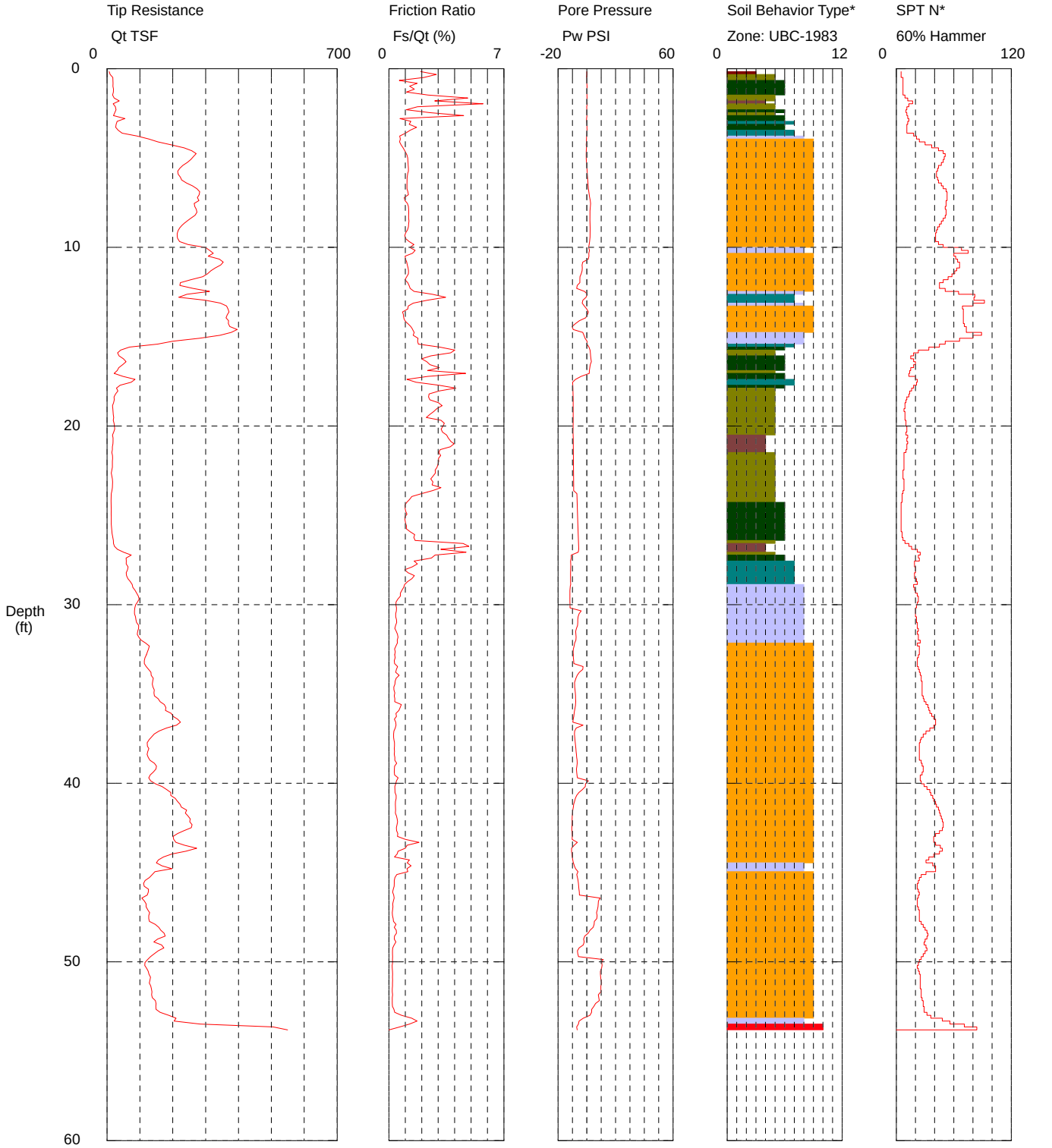
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

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

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-163(s)
Cone Used: DDG1263

CPT Date/Time: 11/11/2014 12:48:55 PM
Location: Port of Vancouver
Job Number: 860058



Maximum Depth = 53.81 feet

Depth Increment = 0.164 feet

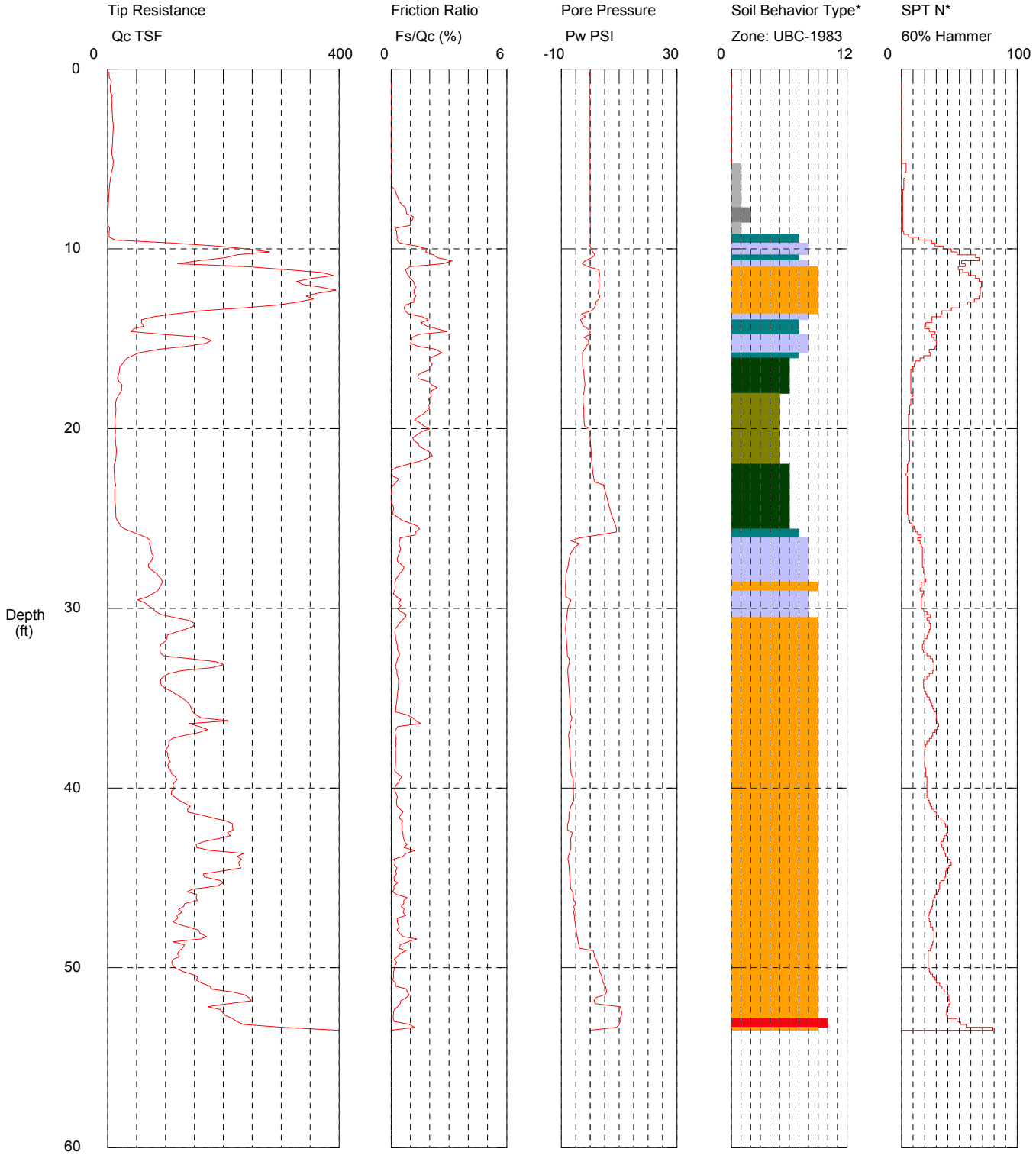
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

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

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-164B
Cone Used: DDG1254

CPT Date/Time: 11/22/2014 12:53:45 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 53.48 feet

Depth Increment = 0.164 feet

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

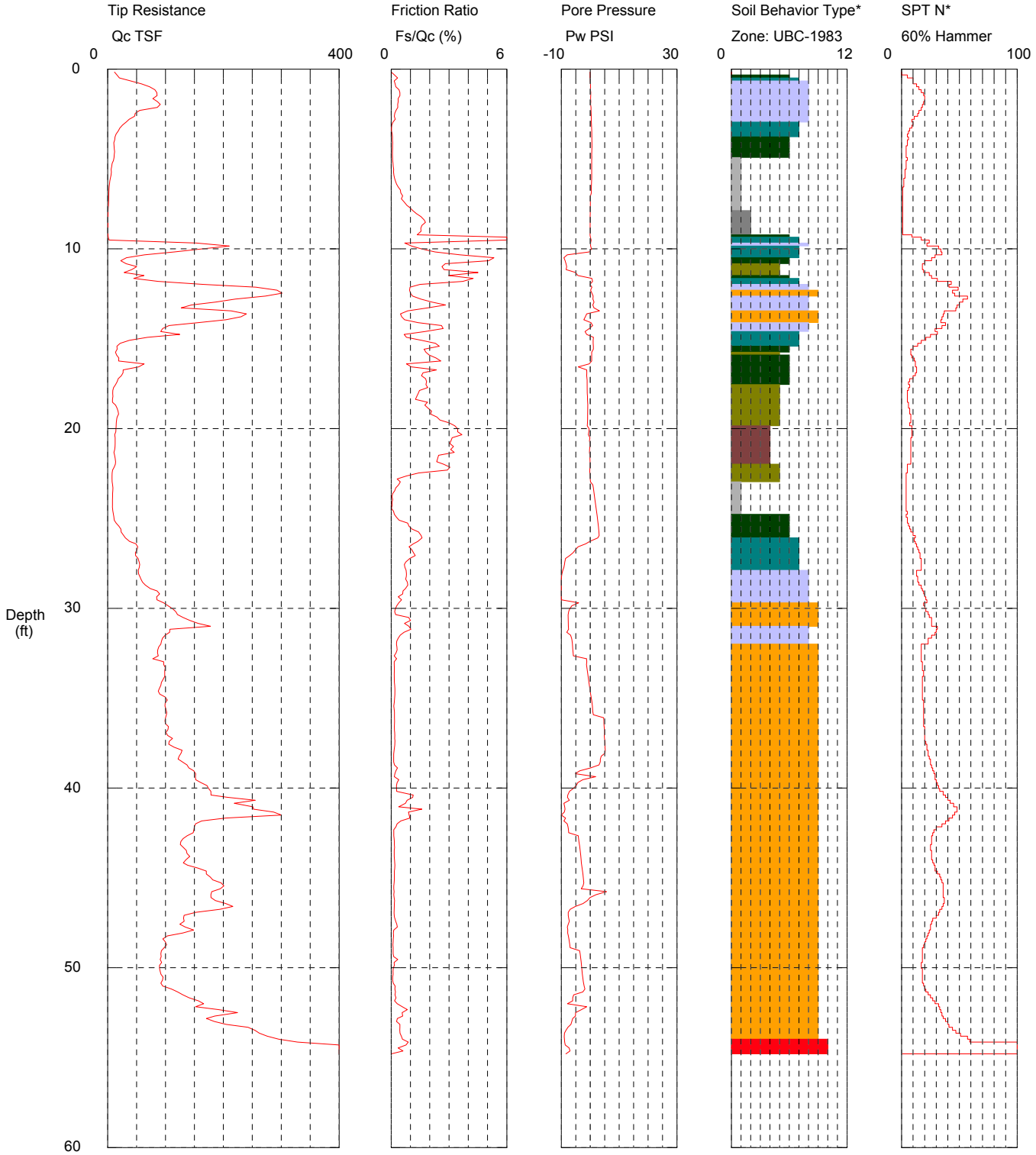
InSitu Engineering Pre-drilled top 10 feet.

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

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-165A
Cone Used: DDG1254

CPT Date/Time: 11/22/2014 11:38:50 AM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 54.79 feet

Depth Increment = 0.164 feet

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

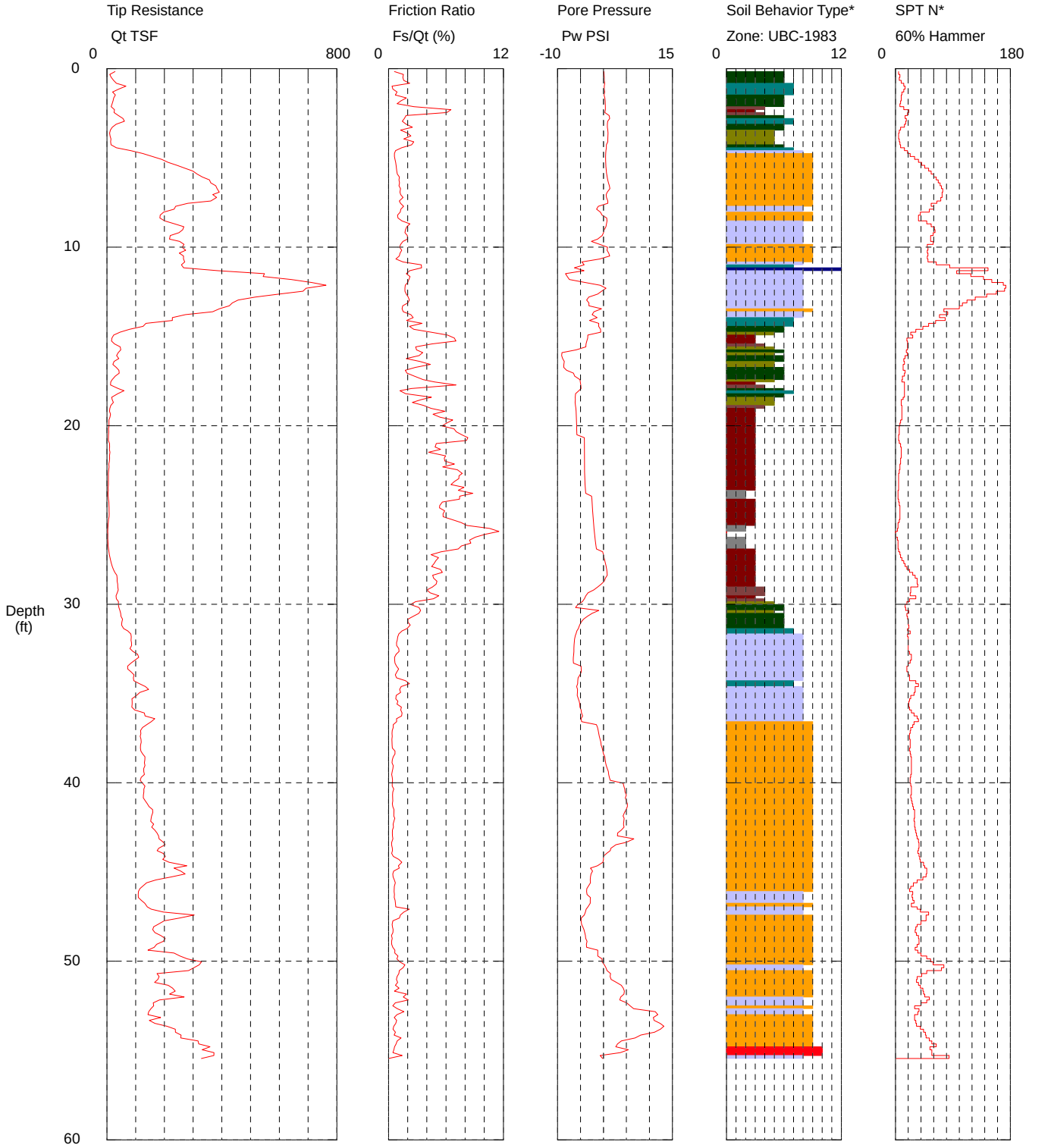
InSitu Engineering Pre-drilled top 10 feet.

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

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-166A
Cone Used: DDG1254

CPT Date/Time: 11/12/2014 10:36:17 AM
Location: Port of Vancouver
Job Number: 860058



Maximum Depth = 55.45 feet

Depth Increment = 0.164 feet

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

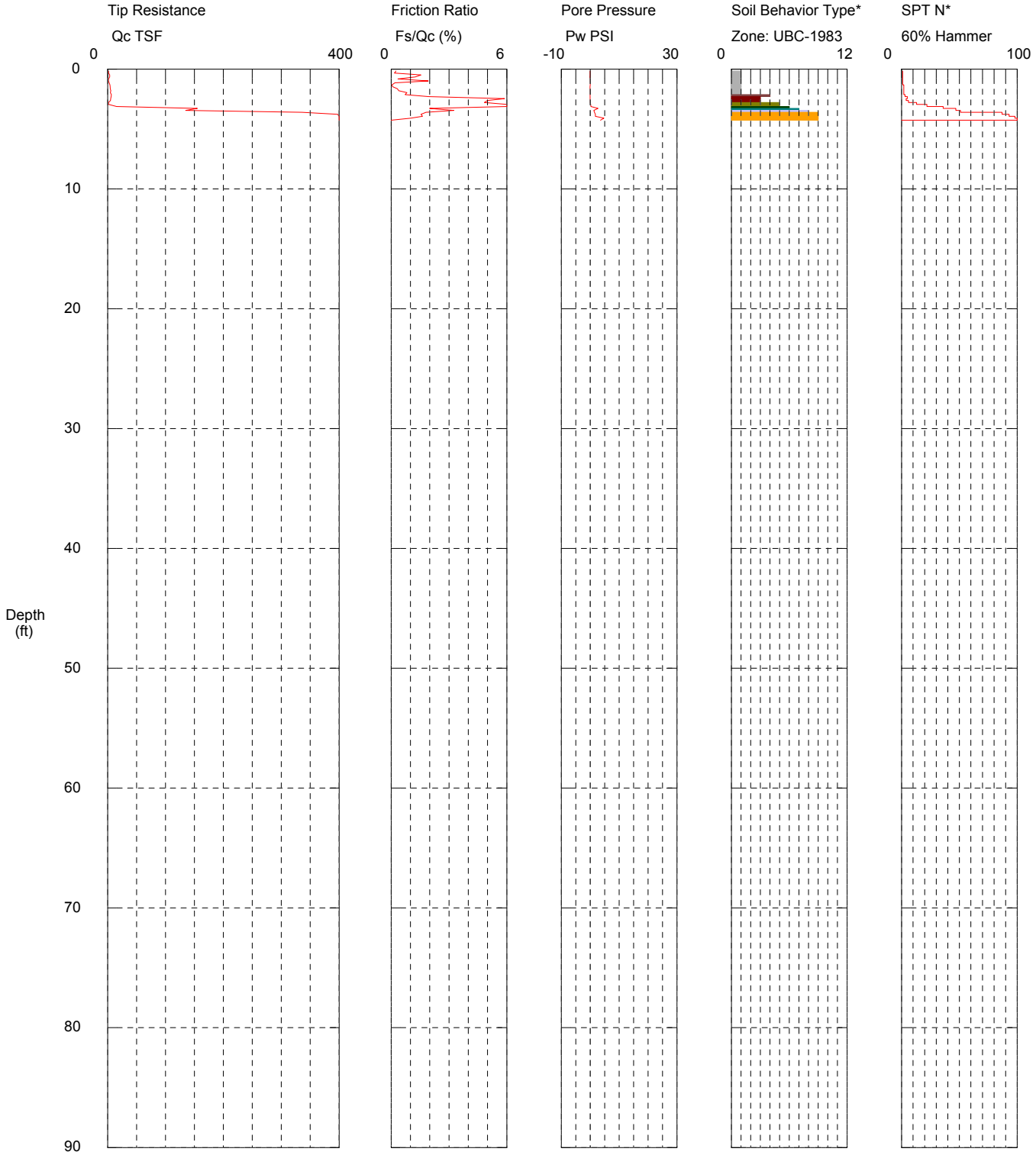
*Soil behavior type and SPT based on data from UBC-1983

Pipeline Area

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-171
Cone Used: DDG1254

CPT Date/Time: 11/16/2014 12:53:57 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 4.27 feet

Depth Increment = 0.164 feet

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

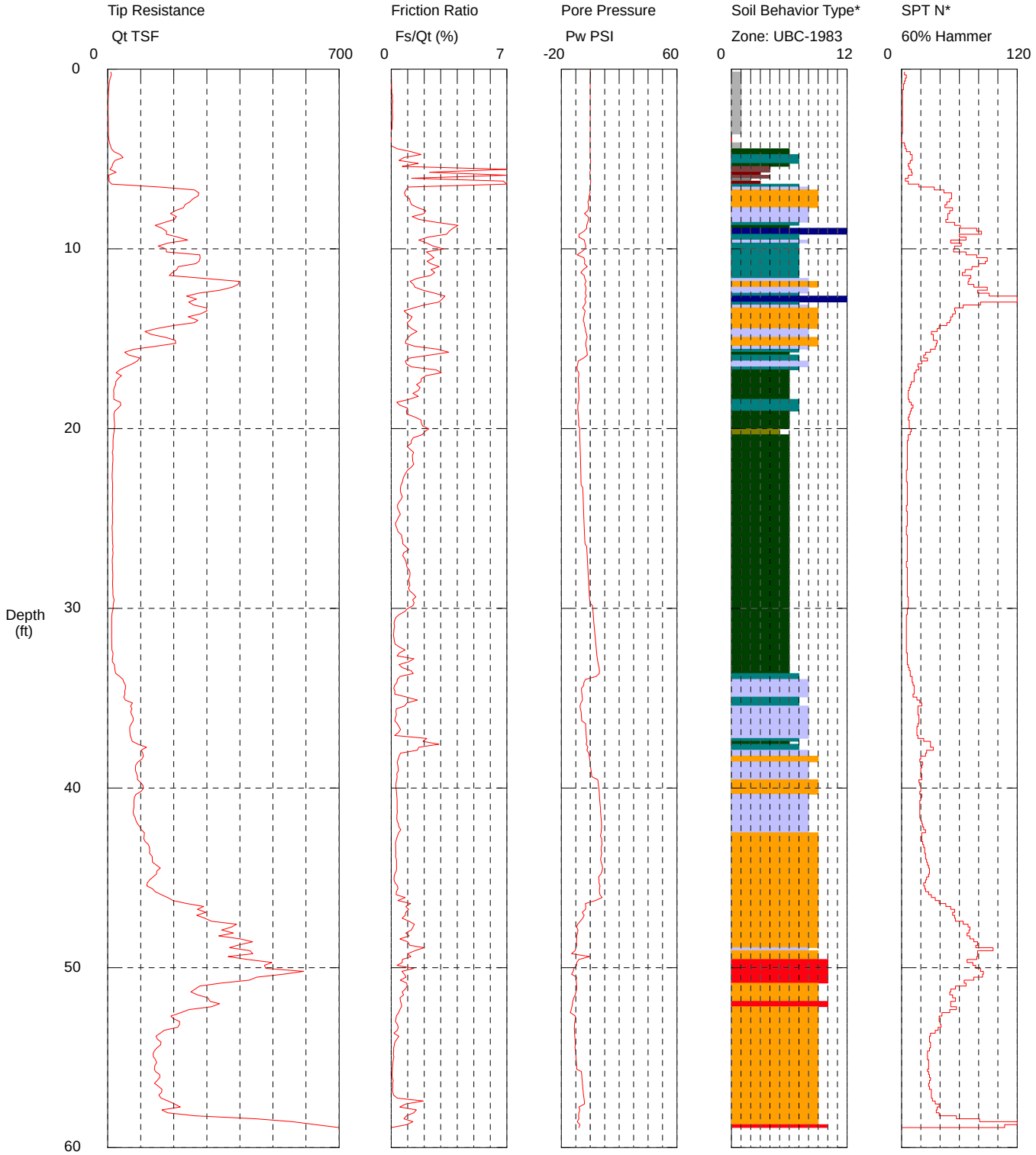
InSitu Engineering Predrilled top 2 feet

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

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-172D
Cone Used: DDG1238

CPT Date/Time: 11/11/2014 10:40:46 AM
Location: Port of Vancouver -Tesoro Distr bution Center
Job Number:



Maximum Depth = 58.89 feet

Depth Increment = 0.164 feet

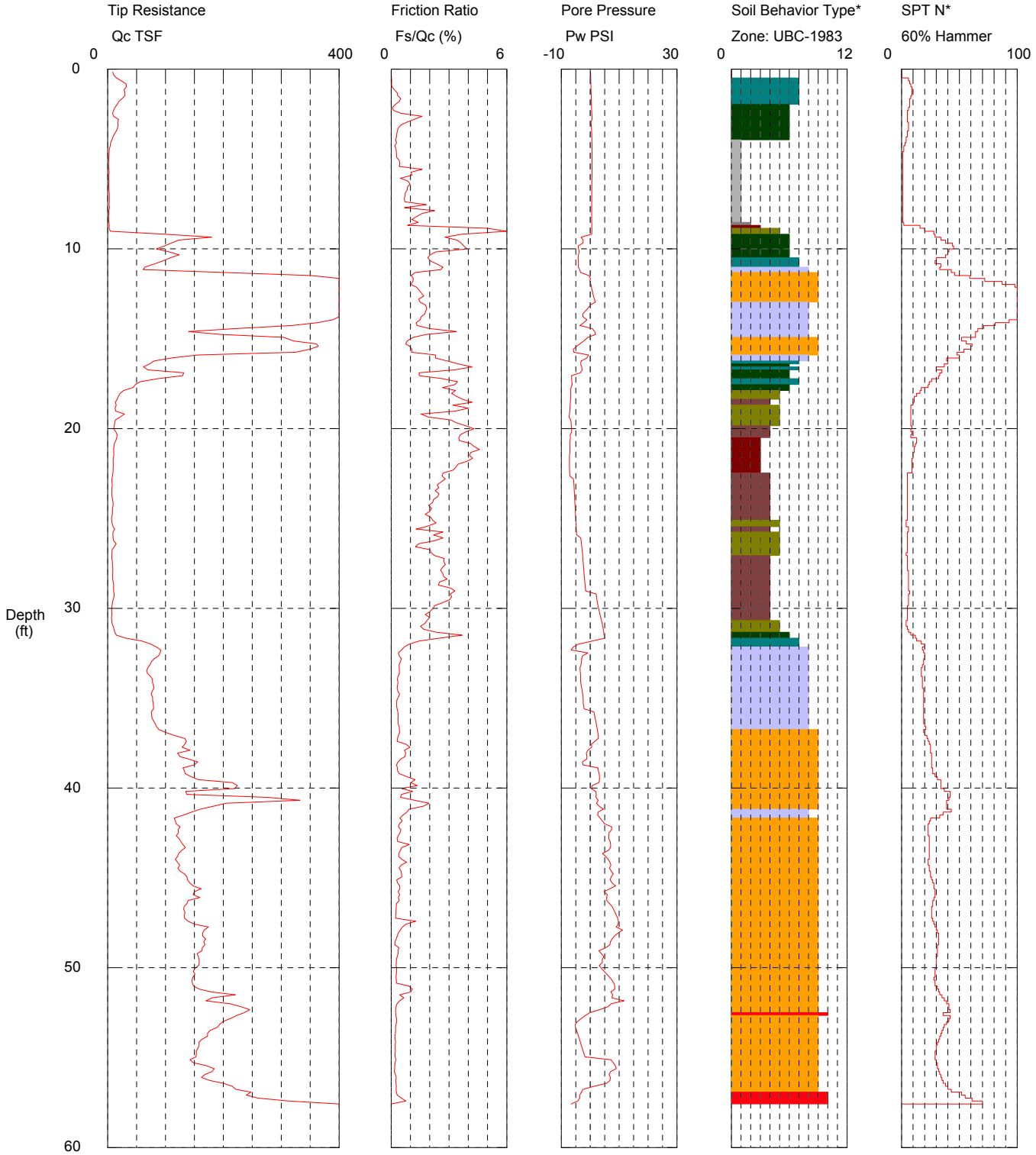
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

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

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-173
Cone Used: DDG1254

CPT Date/Time: 11/19/2014 3:11:06 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 57.58 feet

Depth Increment = 0.164 feet

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

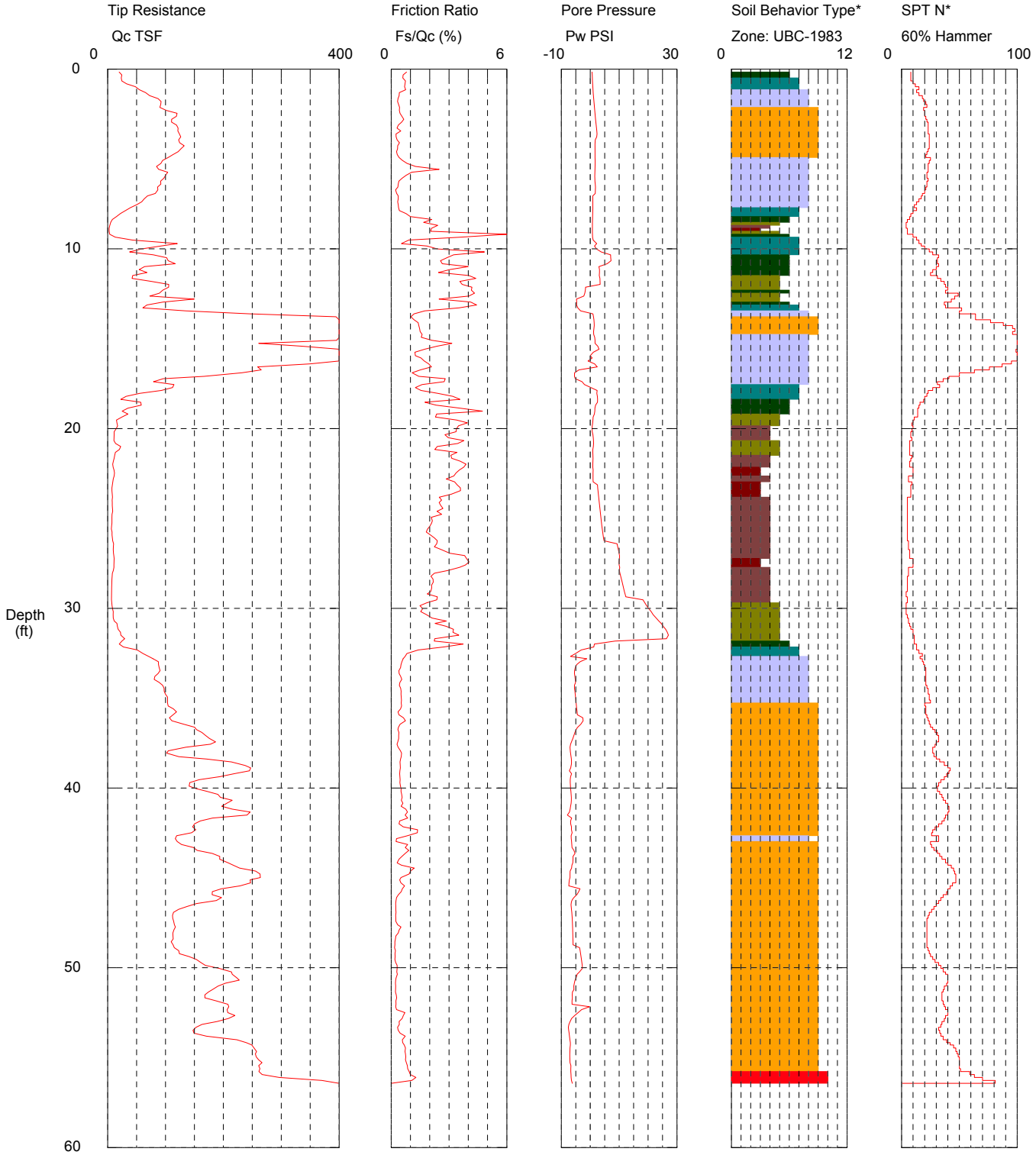
InSitu Engineering Predrilled top 11 feet

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

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-174
Cone Used: DDG1254

CPT Date/Time: 11/19/2014 4:56:50 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 56.43 feet

Depth Increment = 0.164 feet

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

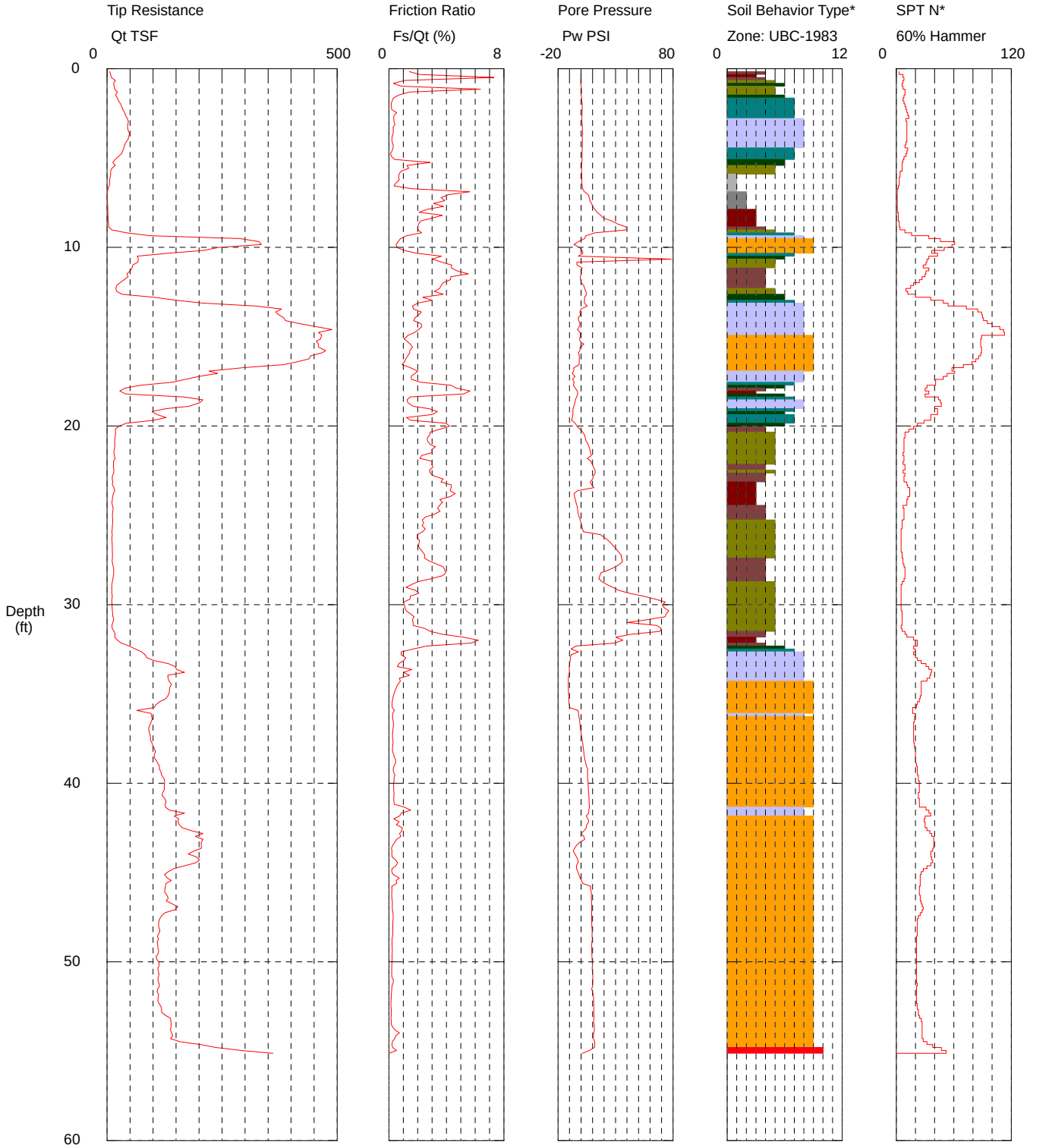
InSitu Engineering Predrilled top 10 feet

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

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-175E
Cone Used: DDG1254

CPT Date/Time: 11/13/2014 2:48:10 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 55.12 feet

Depth Increment = 0.164 feet

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

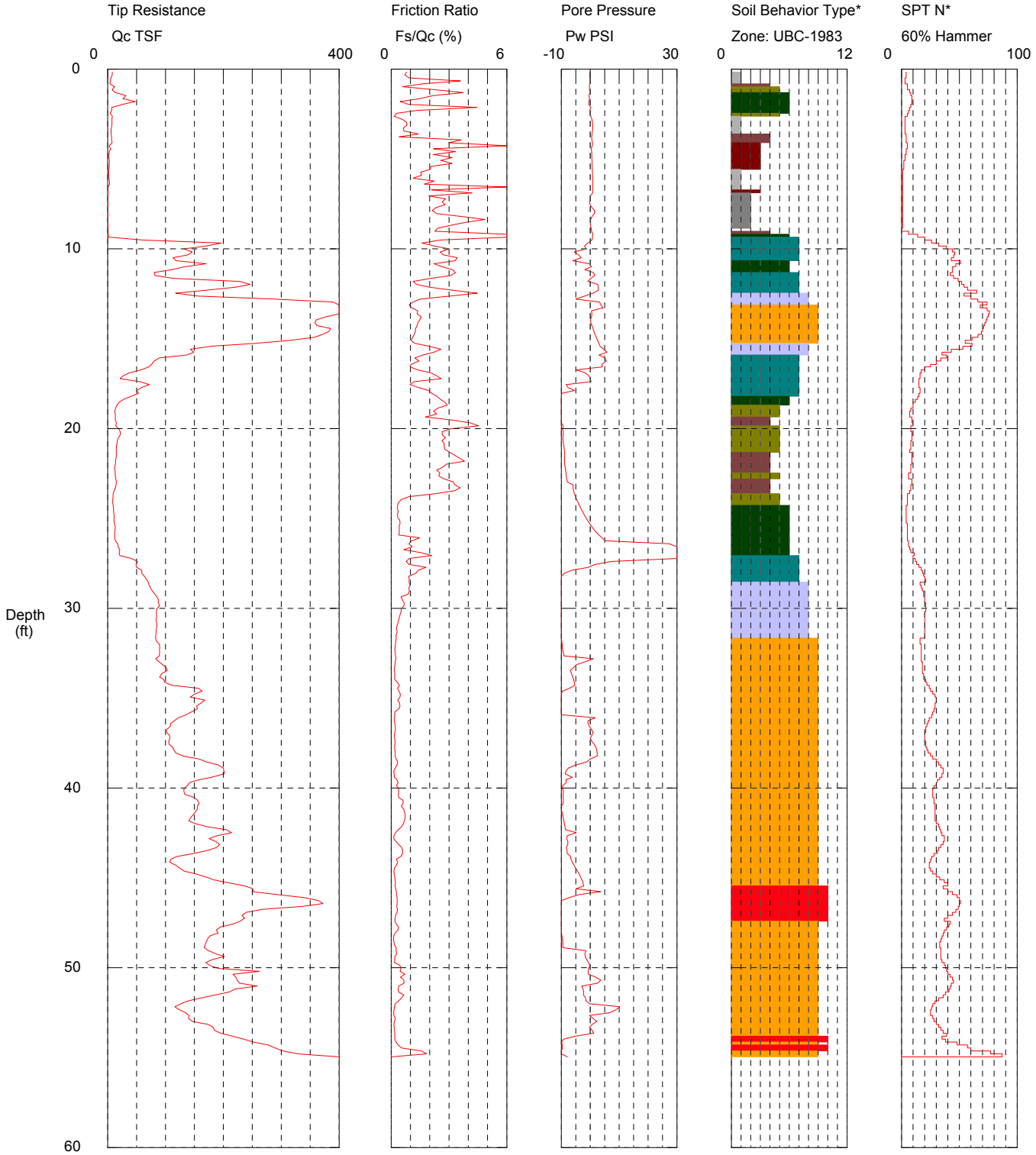
Pre-drilled 9 ft.

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

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-176
Cone Used: DDG1254

CPT Date/Time: 11/22/2014 9:09:38 AM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 54.95 feet

Depth Increment = 0.164 feet

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

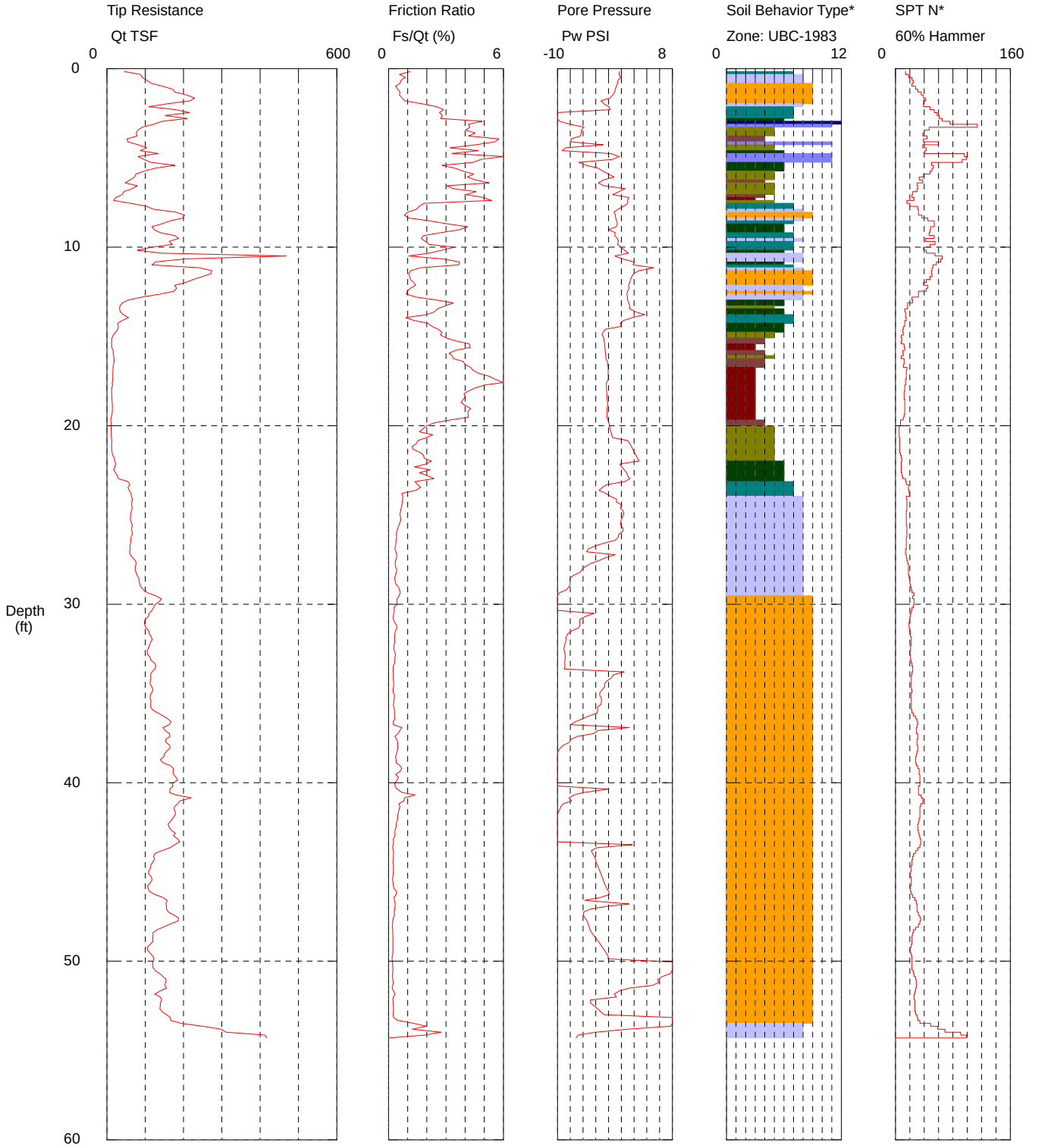
InSitu Engineering Predrilled top 10 feet

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

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-177
Cone Used: DDG1263

CPT Date/Time: 11/10/2014 4:49:19 PM
Location: Port of Vancouver
Job Number: 860058



Maximum Depth = 54.30 feet

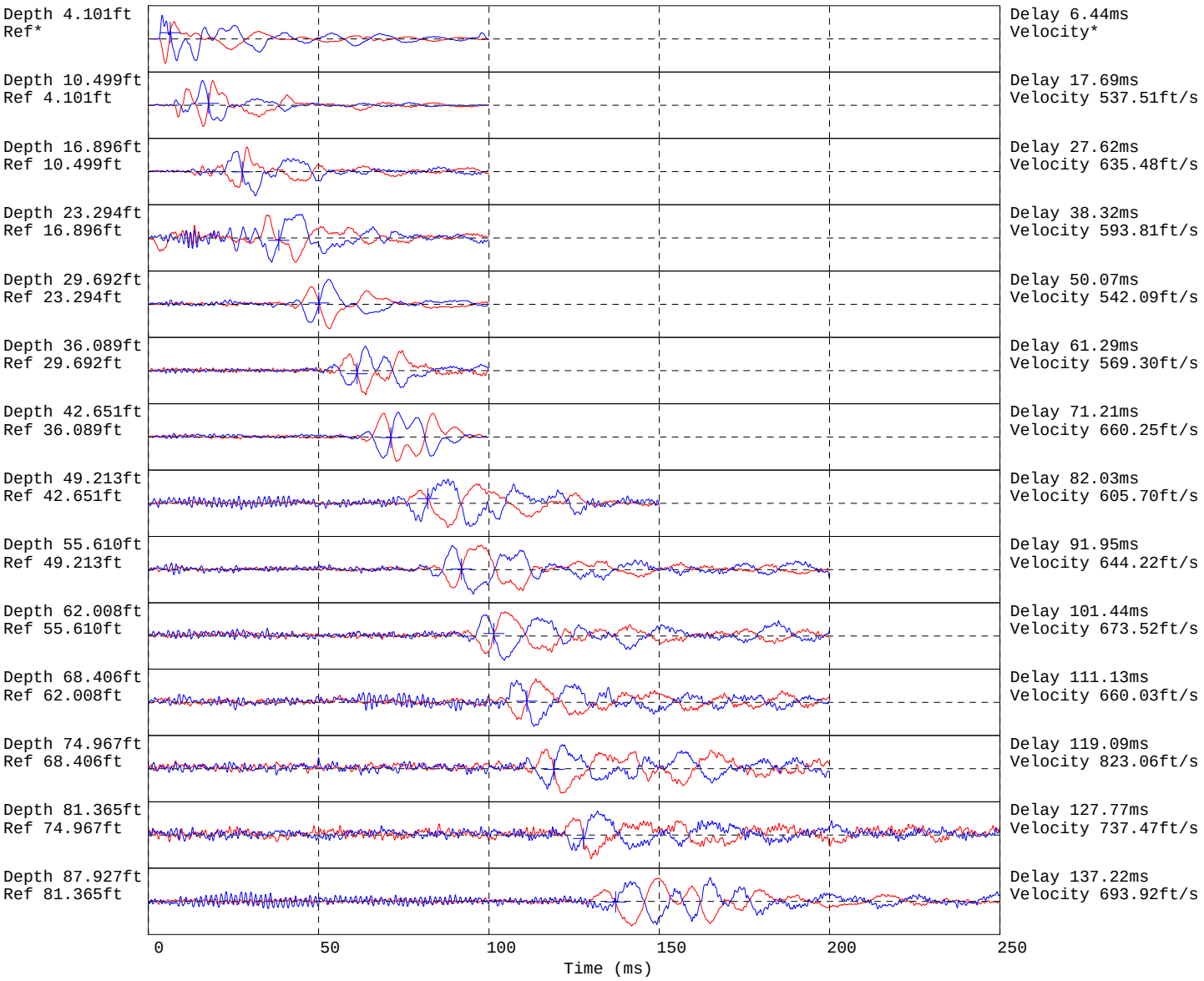
Depth Increment = 0.164 feet

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

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

Dock Area

Tesoro Distribution Terminal
HB-CPT-191(s) Shear Wave Analysis

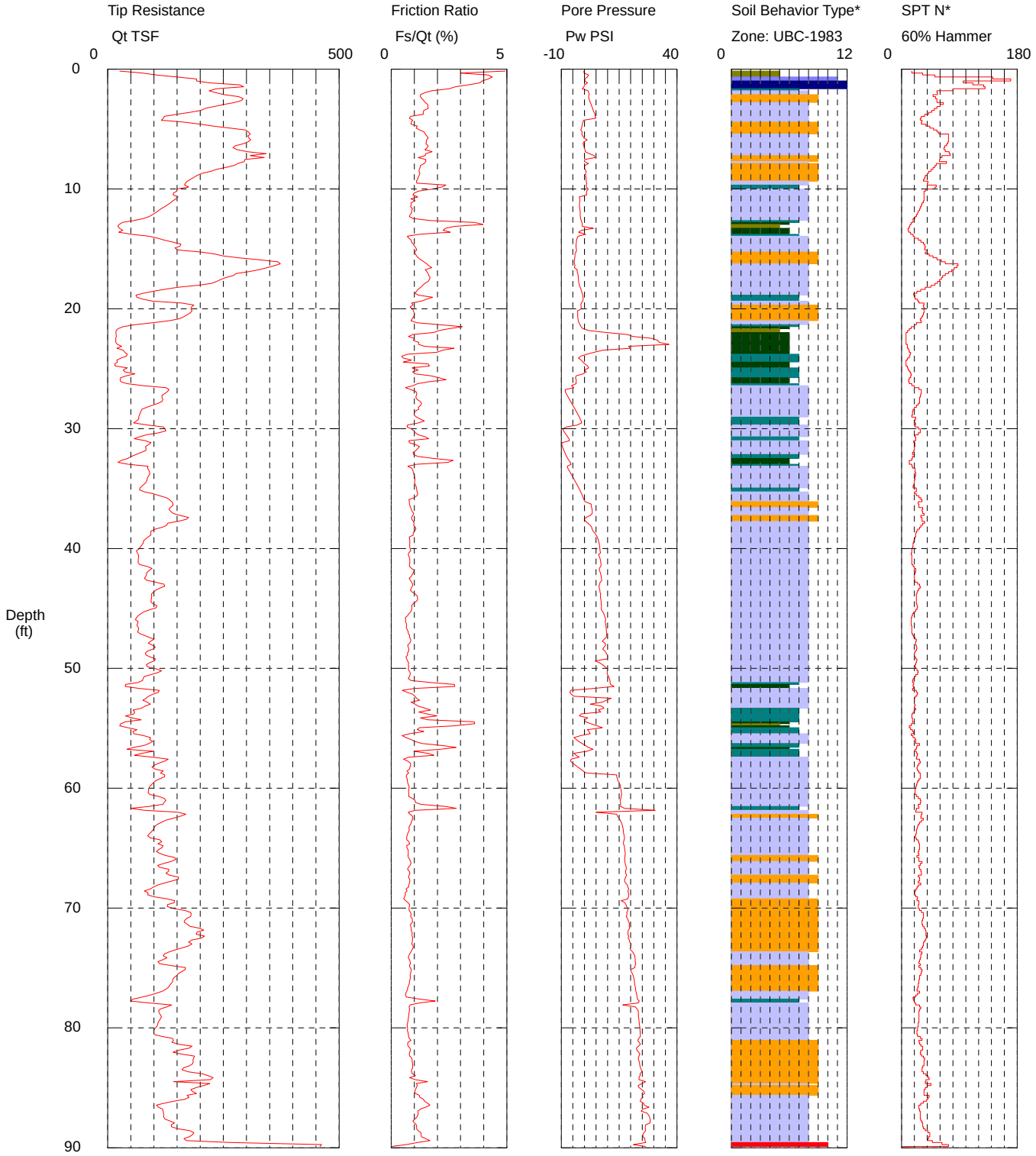


Hammer to Rod String Distance 0.7 (m)
* = Not Determined

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-191(s)
Cone Used: DDG1263

CPT Date/Time: 11/13/2014 9:24:17 AM
Location: Port of Vancouver
Job Number: 860058



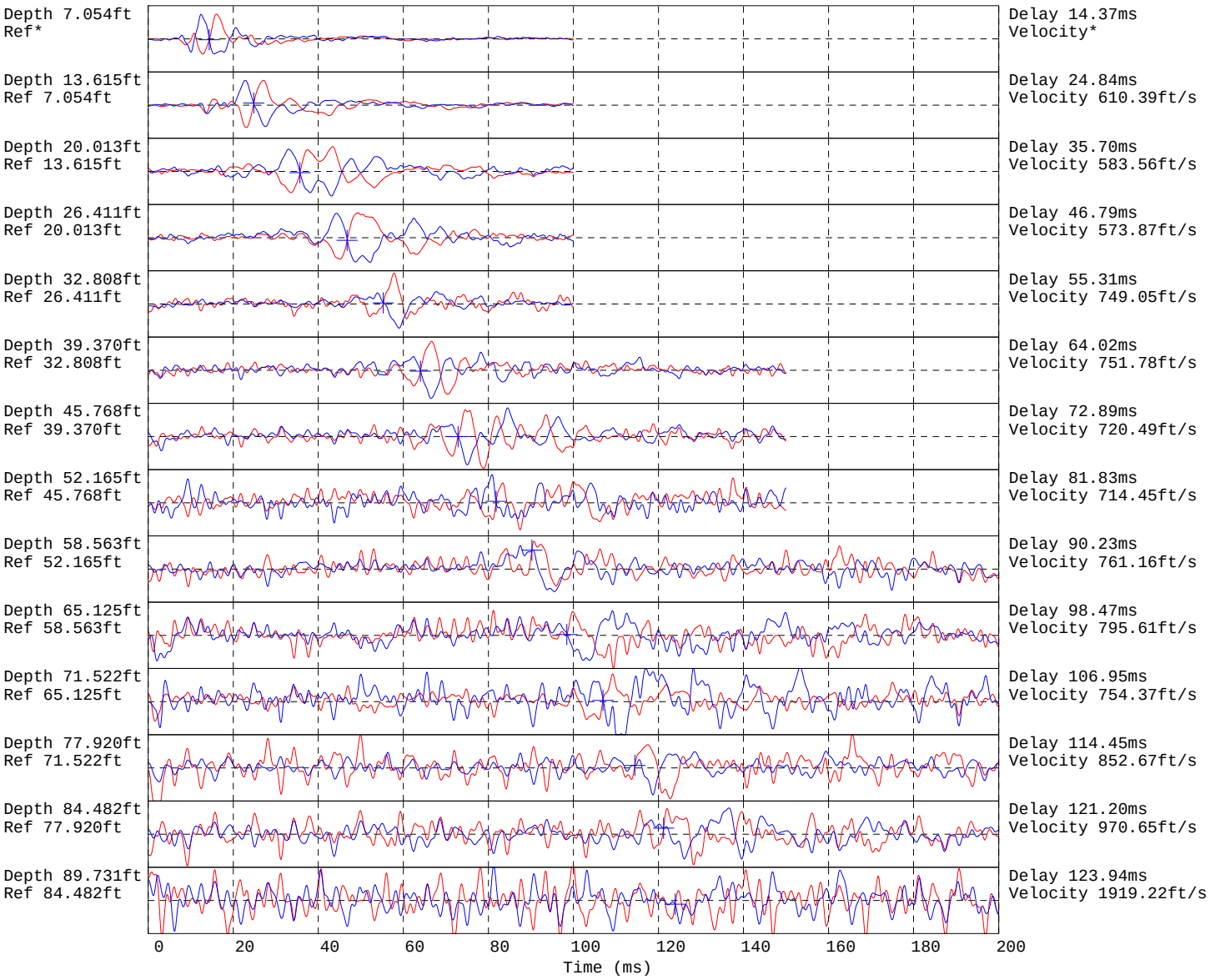
Maximum Depth = 89.90 feet

Depth Increment = 0.164 feet

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

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

Tesoro Distribution Terminal
HB-CPT-192(s) Shear Wave Analysis

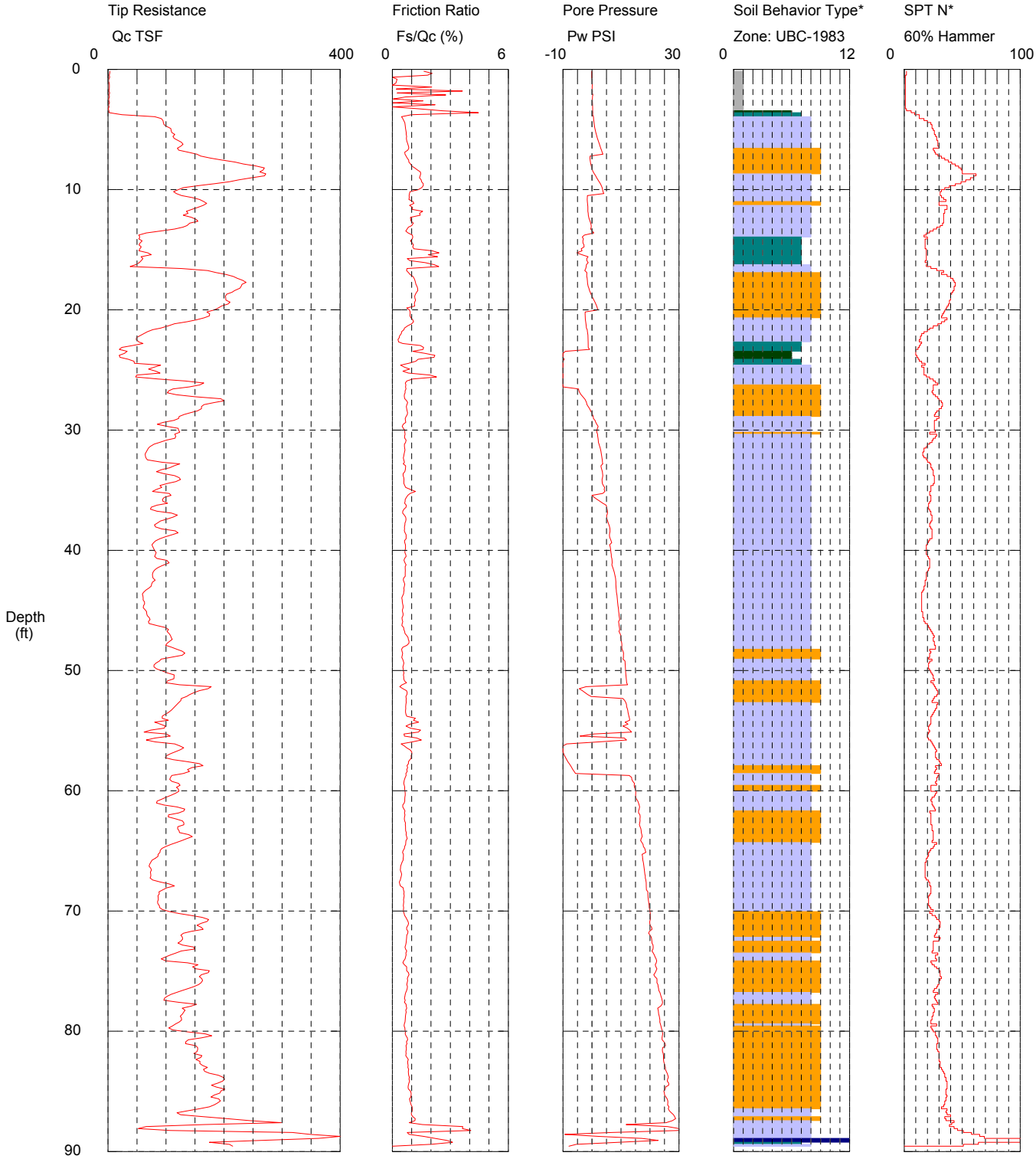


Hammer to Rod String Distance 0.7 (m)
* = Not Determined

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-192(S)
Cone Used: DDG1254

CPT Date/Time: 11/19/2014 8:37:46 AM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 89.57 feet

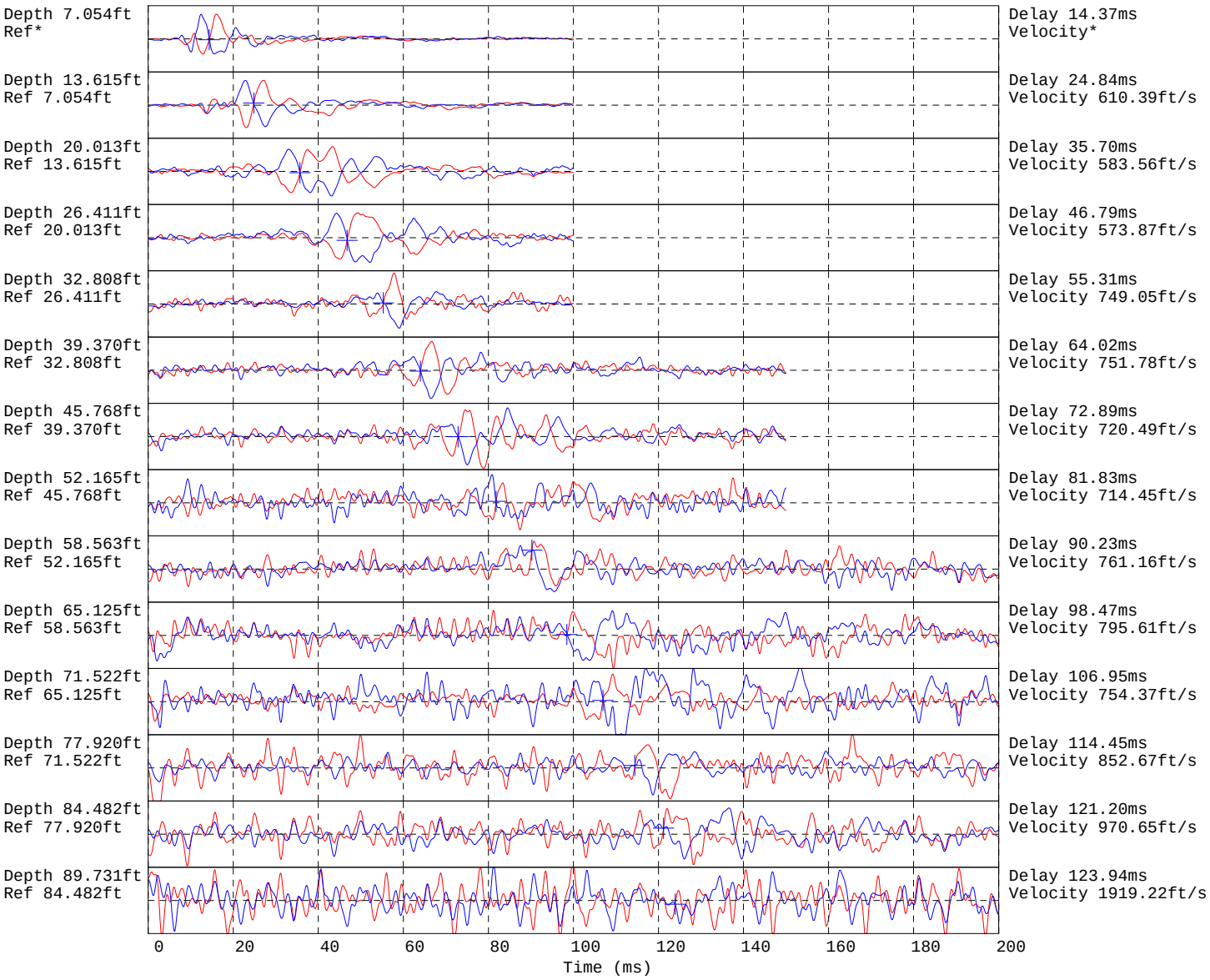
Depth Increment = 0.164 feet

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

InSitu Engineering Predrilled top 3 feet

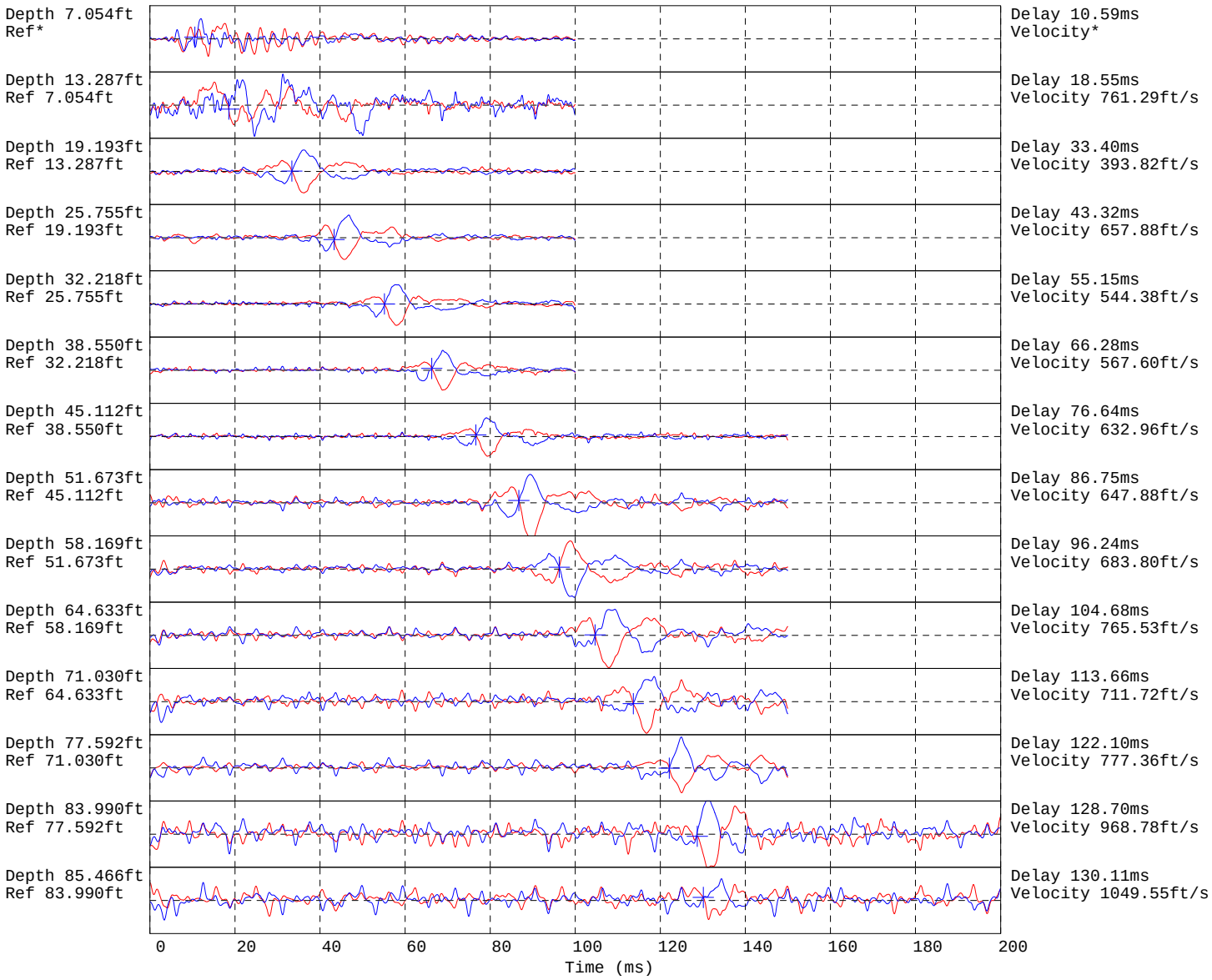
*Soil behavior type and SPT based on data from UBC-1983

Tesoro Distribution Terminal
HB-CPT-192(s) Shear Wave Analysis



Hammer to Rod String Distance 0.7 (m)
* = Not Determined

Tesoro Distribution Terminal
HB-CPT-193(s) Shear Wave Analysis

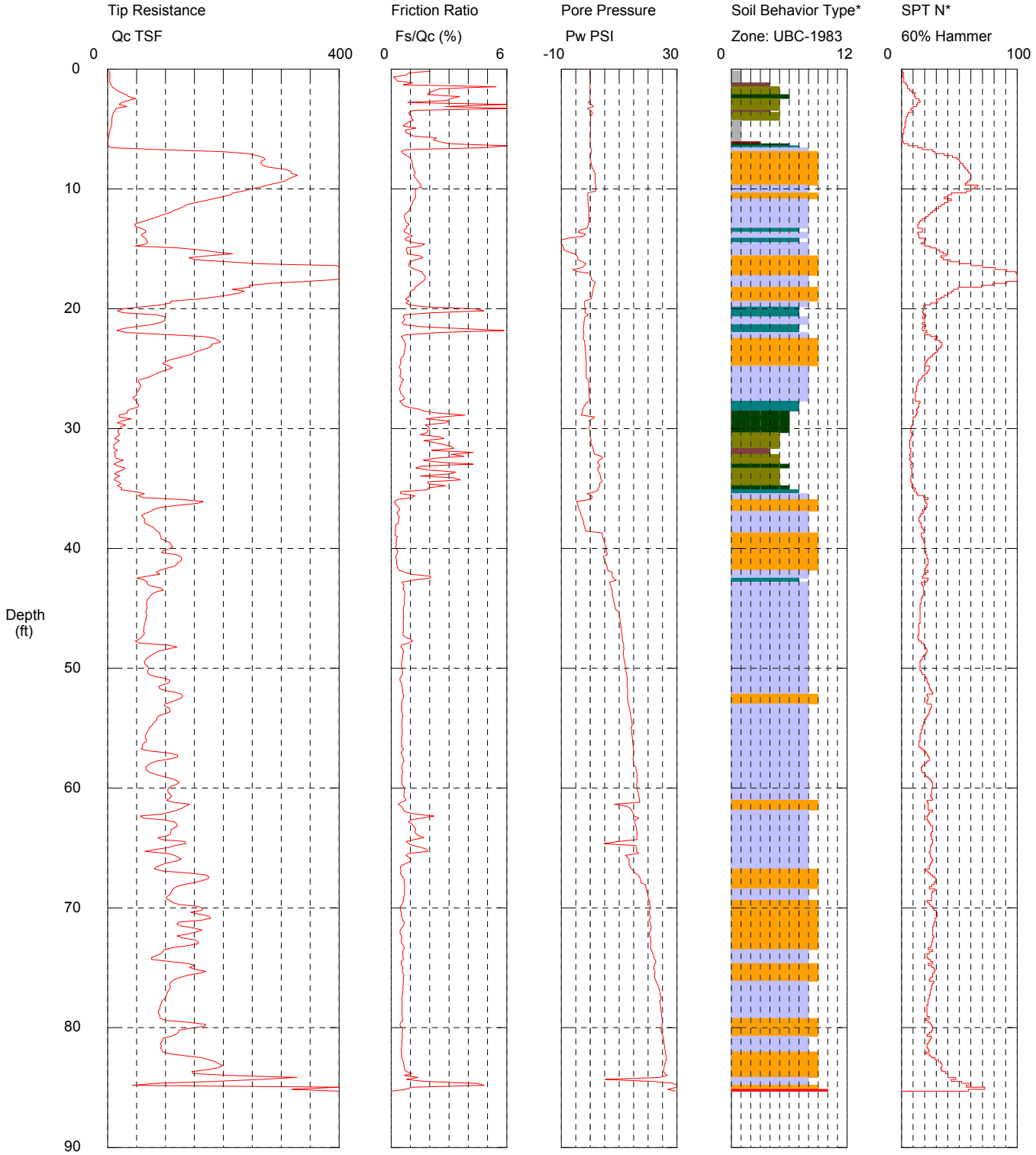


Hammer to Rod String Distance 0.7 (m)
* = Not Determined

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-193(S)
Cone Used: DDG1254

CPT Date/Time: 11/18/2014 2:33:50 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 85.30 feet

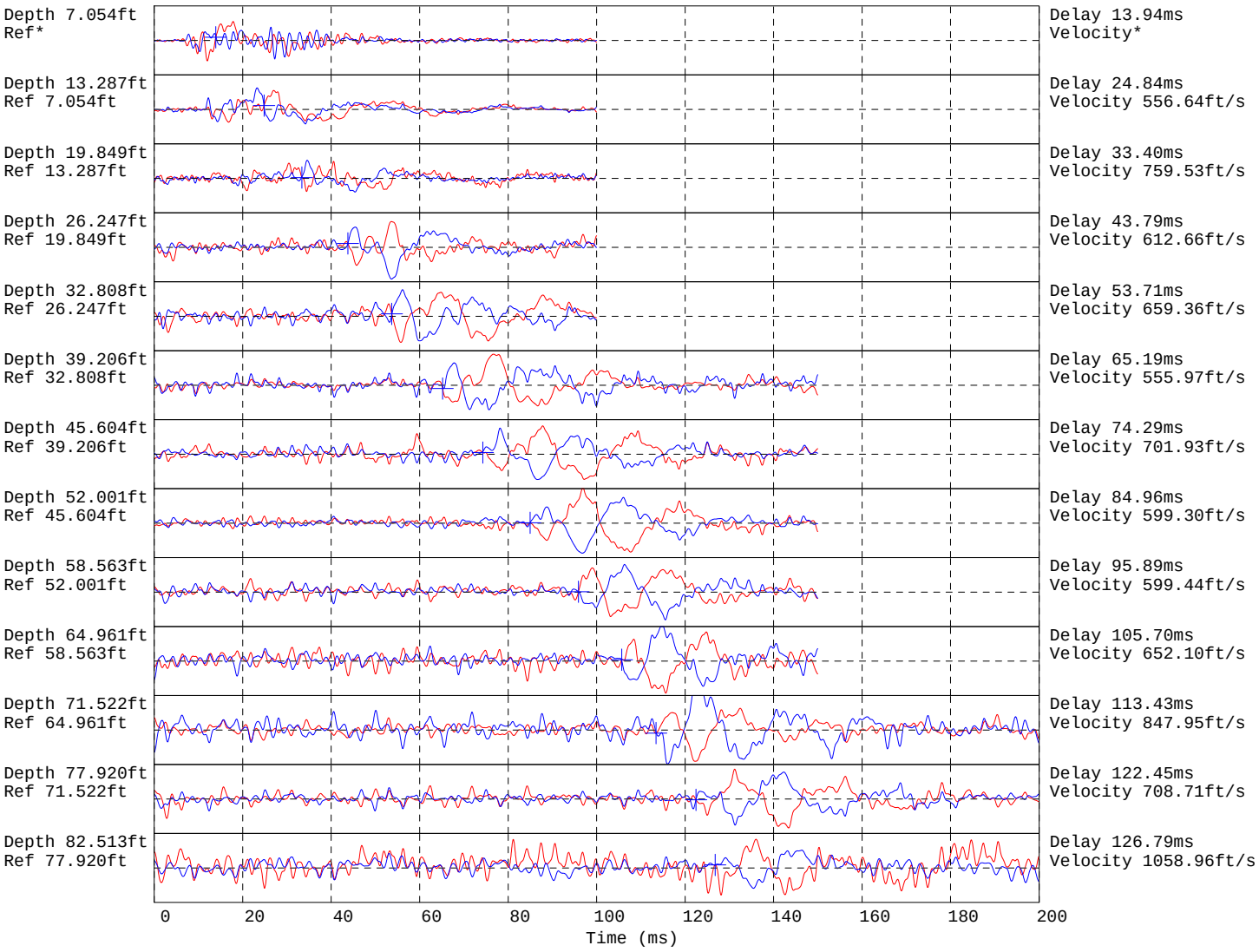
Depth Increment = 0.164 feet

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

InSitu Engineering Predrilled top 6 feet

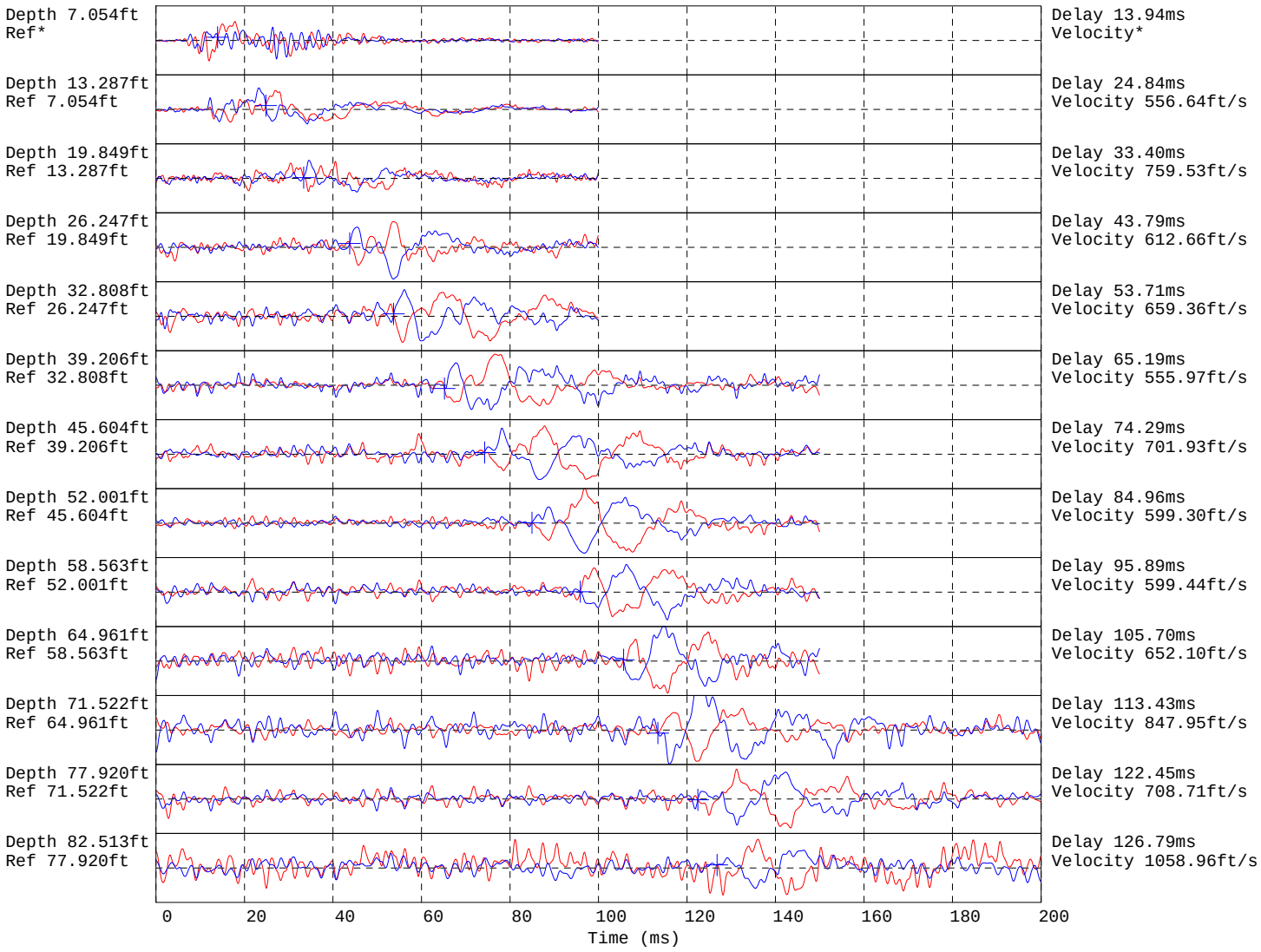
*Soil behavior type and SPT based on data from UBC-1983

Tesoro Distribution Terminal
HB-CPT-194(s) Shear Wave Analysis



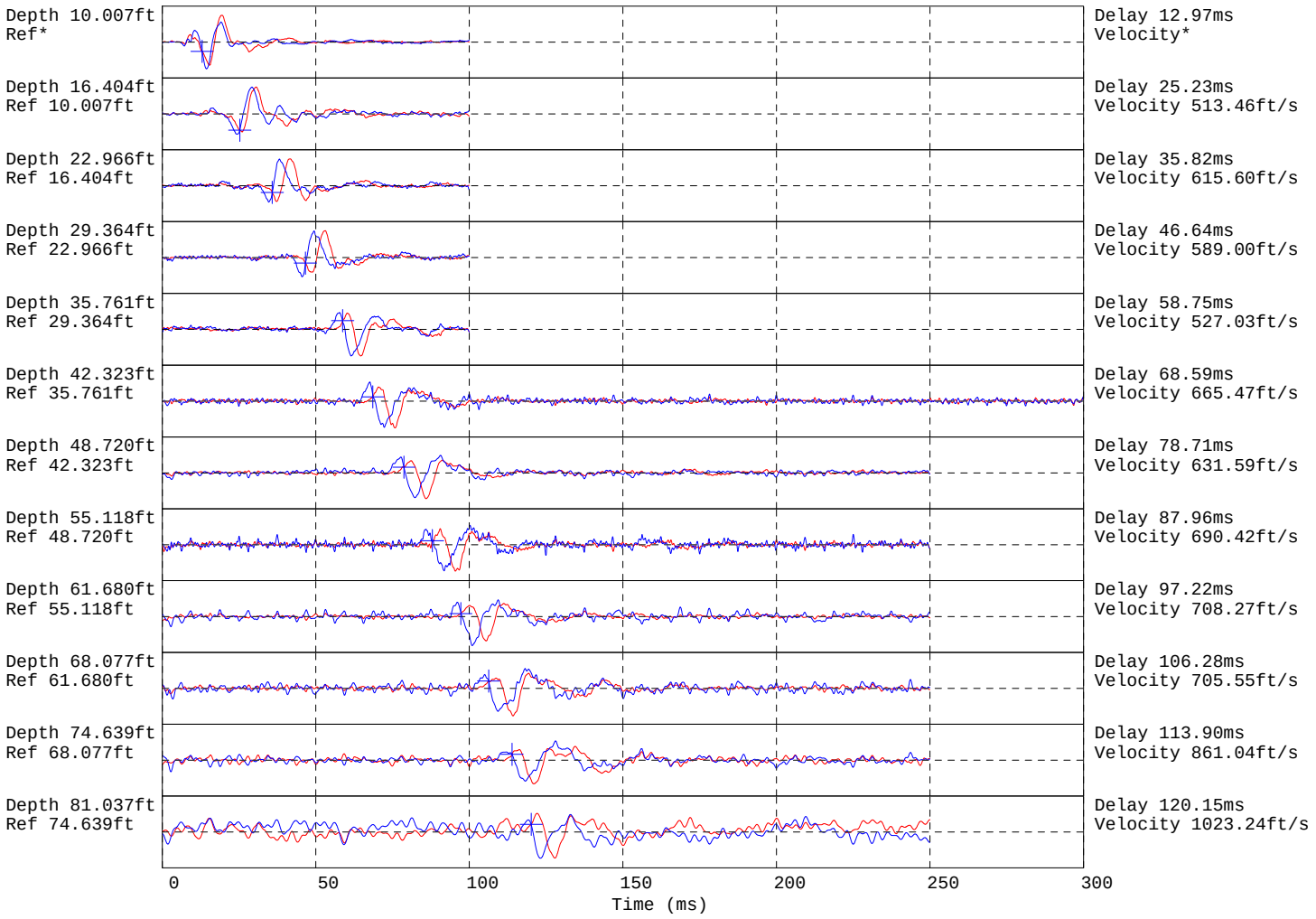
Hammer to Rod String Distance 0.7 (m)
* = Not Determined

Tesoro Distribution Terminal
HB-CPT-194(s) Shear Wave Analysis



Hammer to Rod String Distance 0.7 (m)
* = Not Determined

Tesoro Distribution Terminal
HB-CPT-195(s)C Shear Wave Analysis

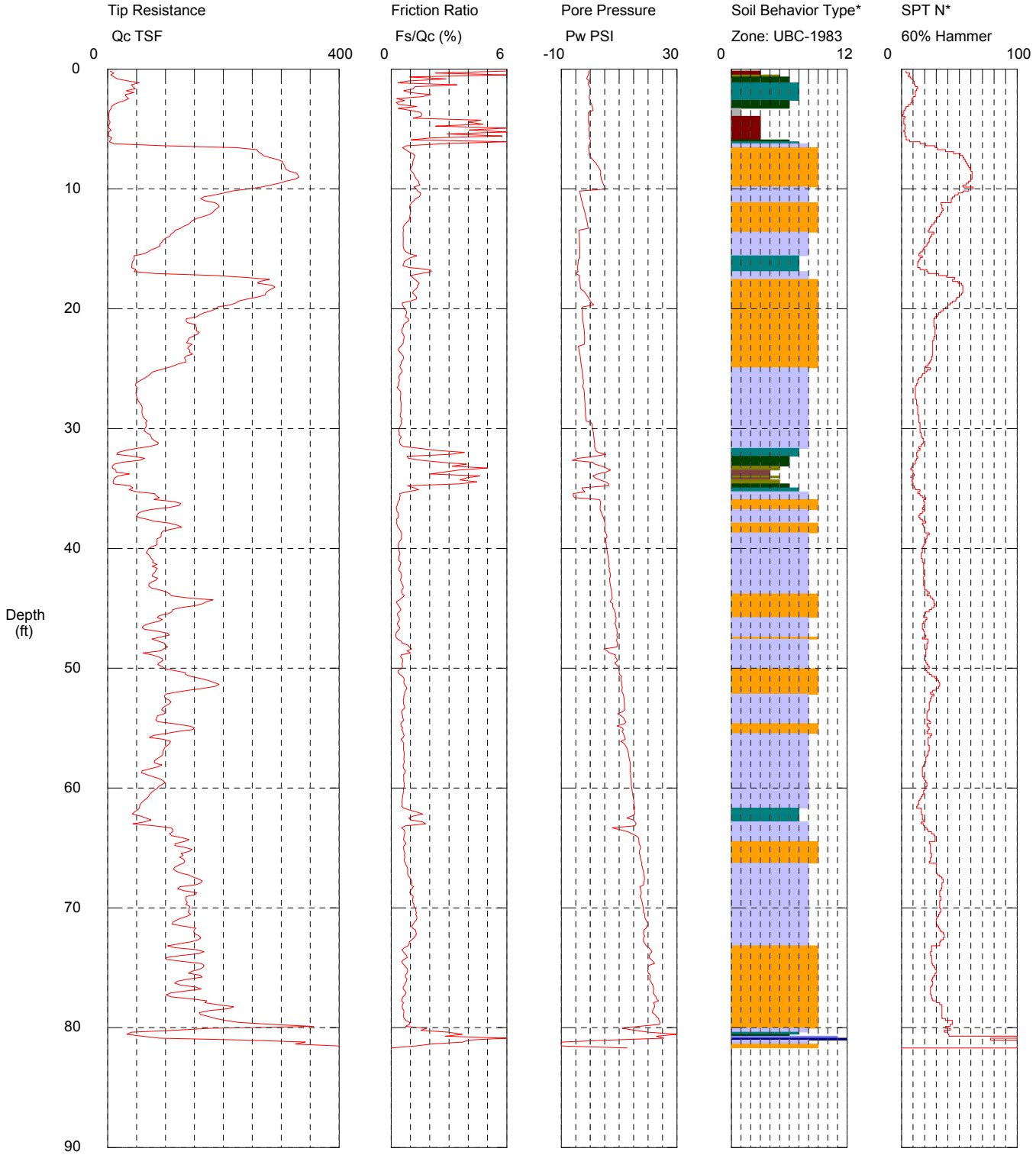


Hammer to Rod String Distance 0.7 (m)
* = Not Determined

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-195(S)C
Cone Used: DDG1254

CPT Date/Time: 11/18/2014 9:09:45 AM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 81.69 feet

Depth Increment = 0.164 feet

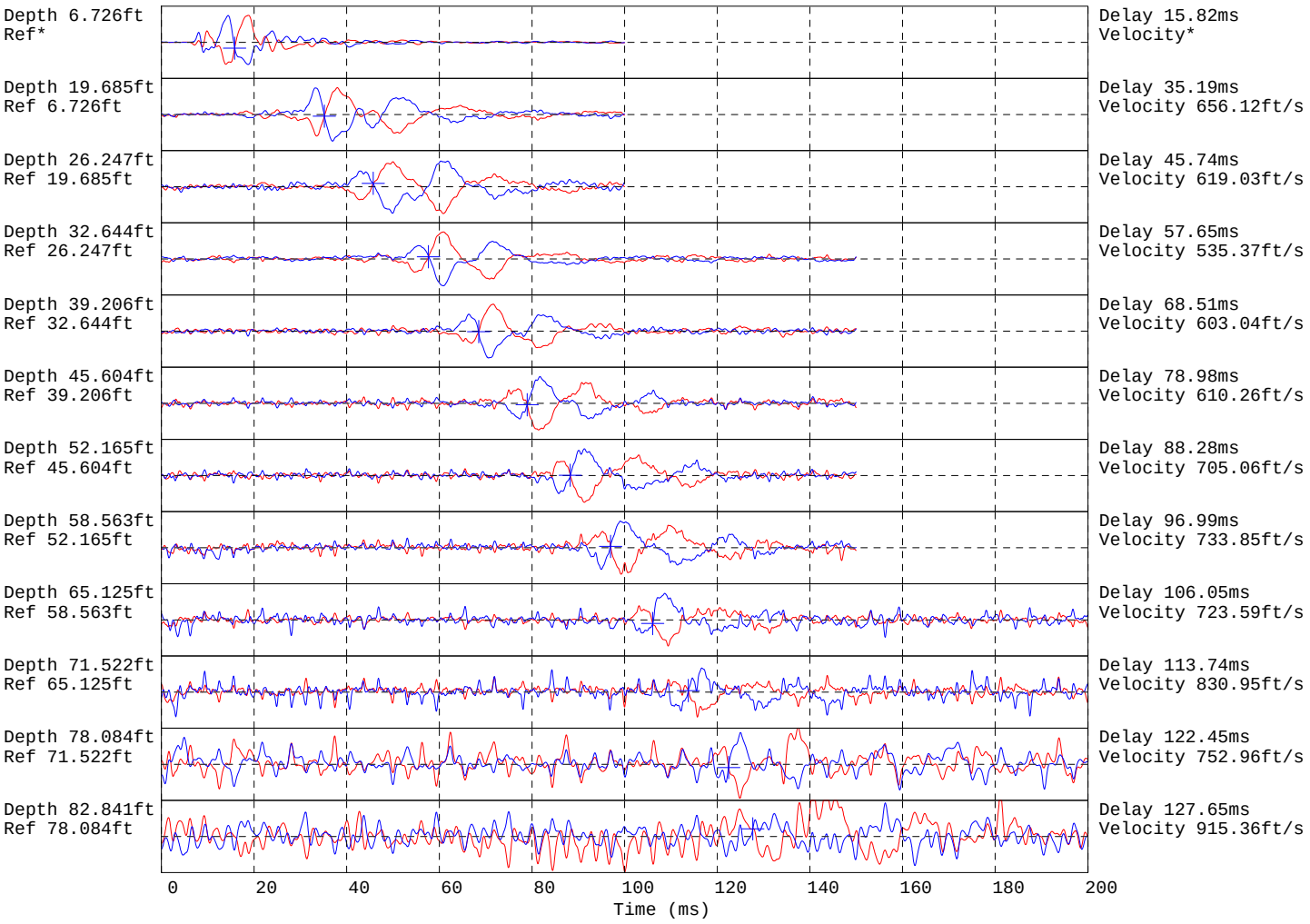
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

InSitu Engineering Predrilled top 6 feet

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

EX-0001-006840-PCE

Tesoro Distribution Terminal
HB-CPT-196(s) Shear Wave Analysis

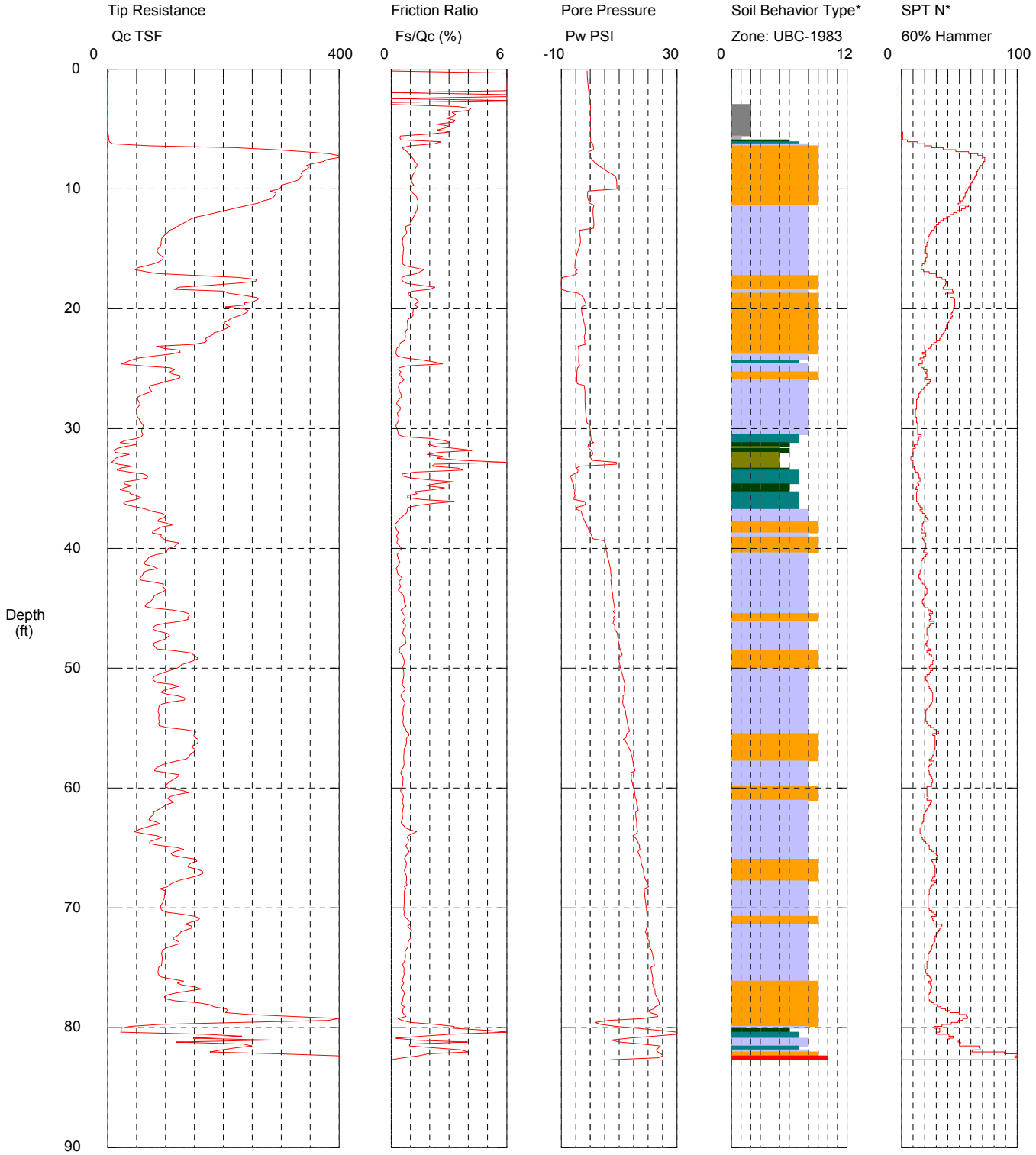


Hammer to Rod String Distance 0.7 (m)
* = Not Determined

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-196(S)
Cone Used: DDG1254

CPT Date/Time: 11/18/2014 11:14:59 AM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 82.68 feet

Depth Increment = 0.164 feet

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

InSitu Engineering Predrilled top 6 feet

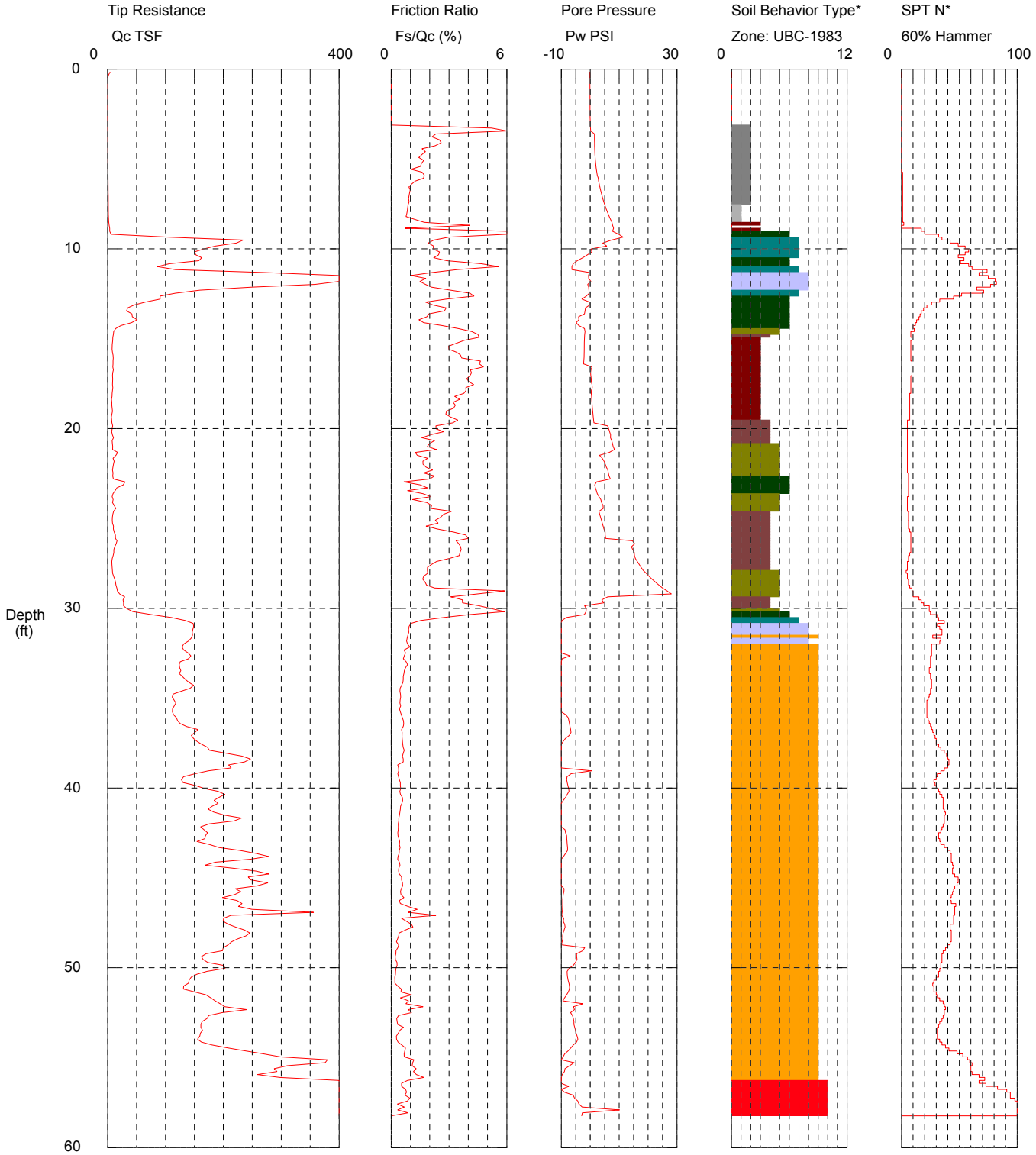
*Soil behavior type and SPT based on data from UBC-1983

Additional CPT Plots

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-101
Cone Used: DDG1254

CPT Date/Time: 11/19/2014 1:20:42 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 58.23 feet

Depth Increment = 0.164 feet

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

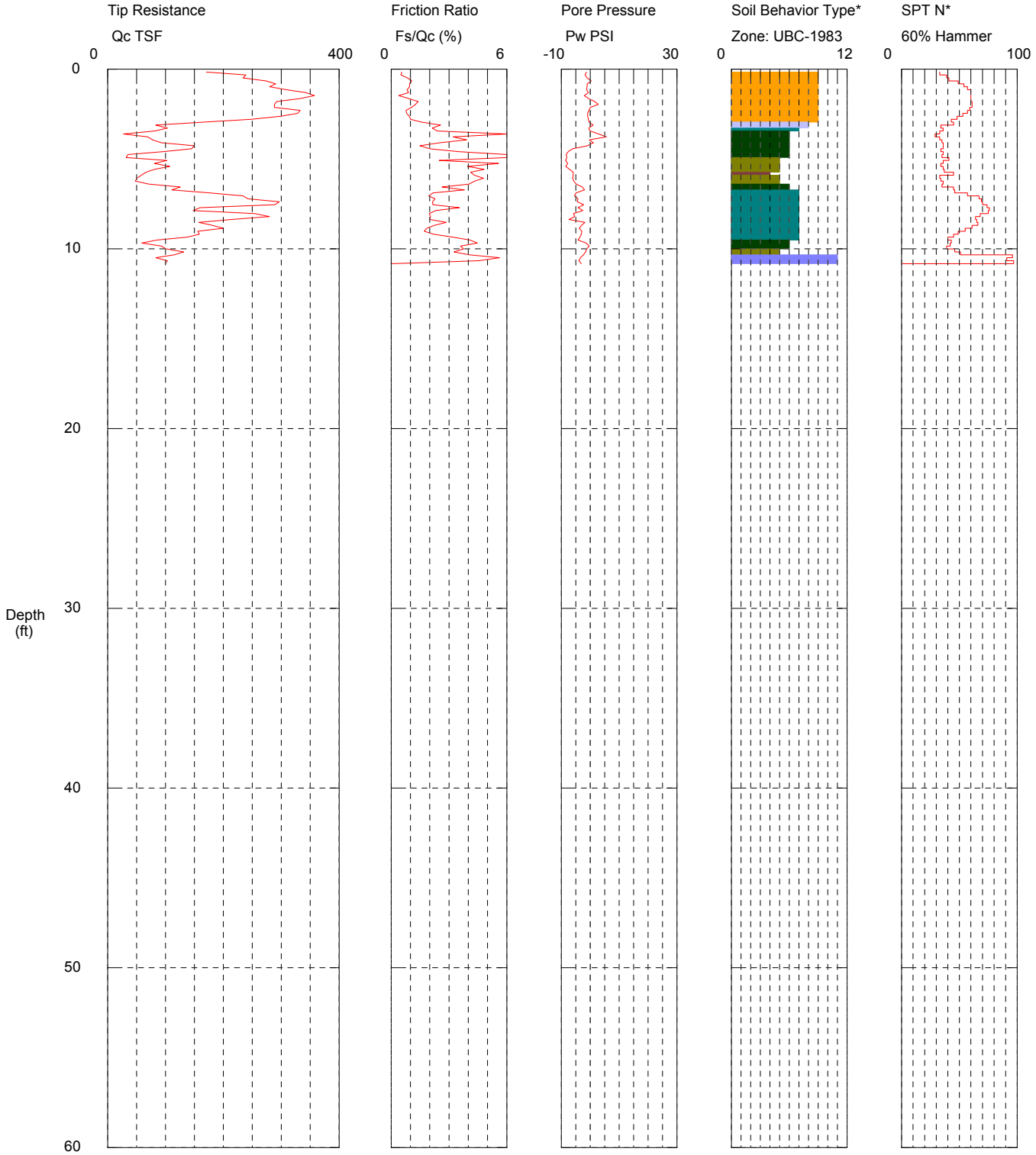
InSitu Engineering Predrilled top 10 feet

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

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-102
Cone Used: DDG1263

CPT Date/Time: 11/10/2014 12:00:49 PM
Location: Port of Vancouver
Job Number: 860058



Maximum Depth = 10.83 feet

Depth Increment = 0.164 feet

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

InSitu Engineering

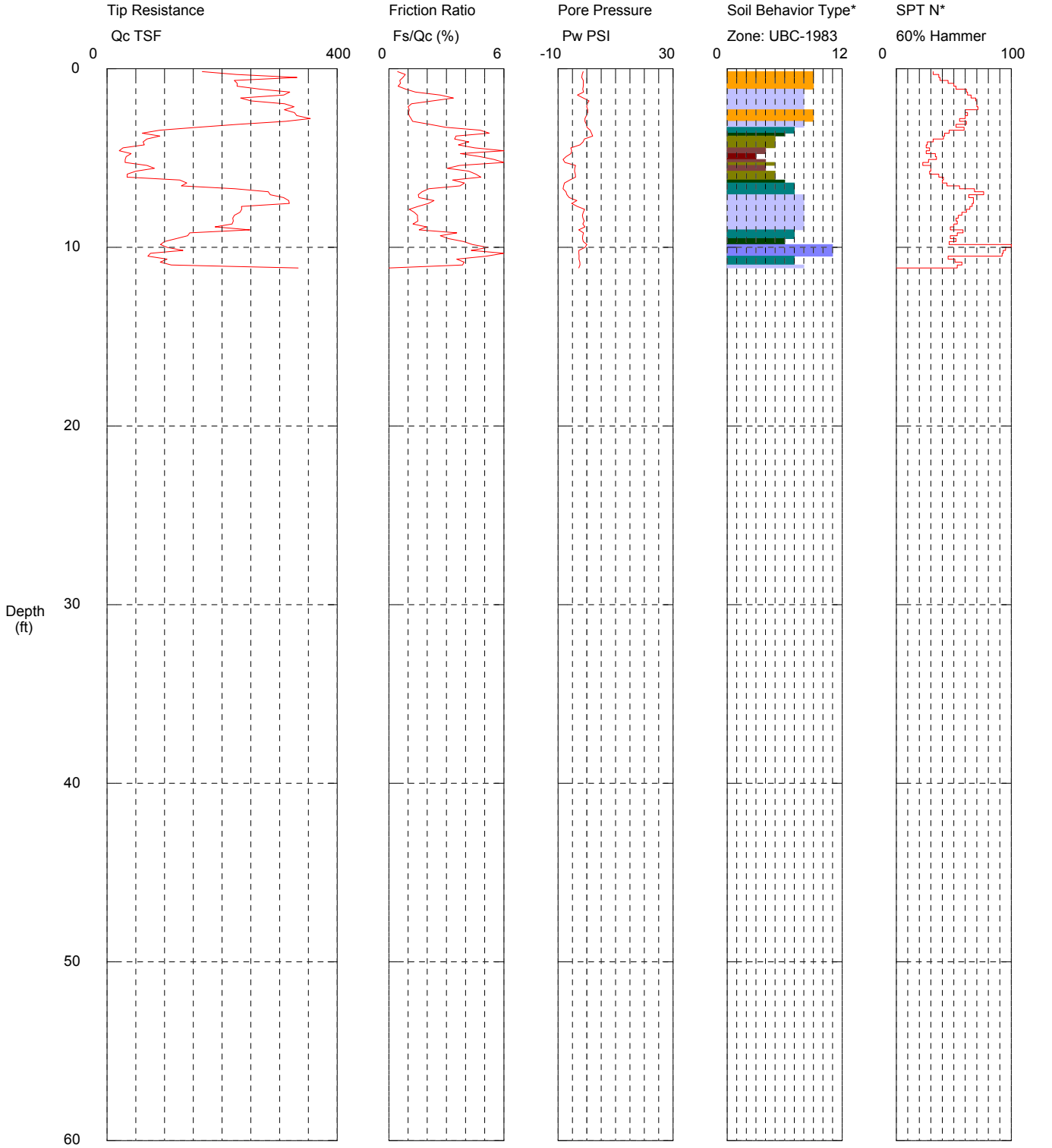
*Soil behavior type and SPT based on data from UBC-1983

EX-0001-006845-PCE

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-102C
Cone Used: DDG1263

CPT Date/Time: 11/10/2014 12:44:33 PM
Location: Port of Vancouver
Job Number: 860058



Maximum Depth = 11.15 feet

Depth Increment = 0.164 feet

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

InSitu Engineering

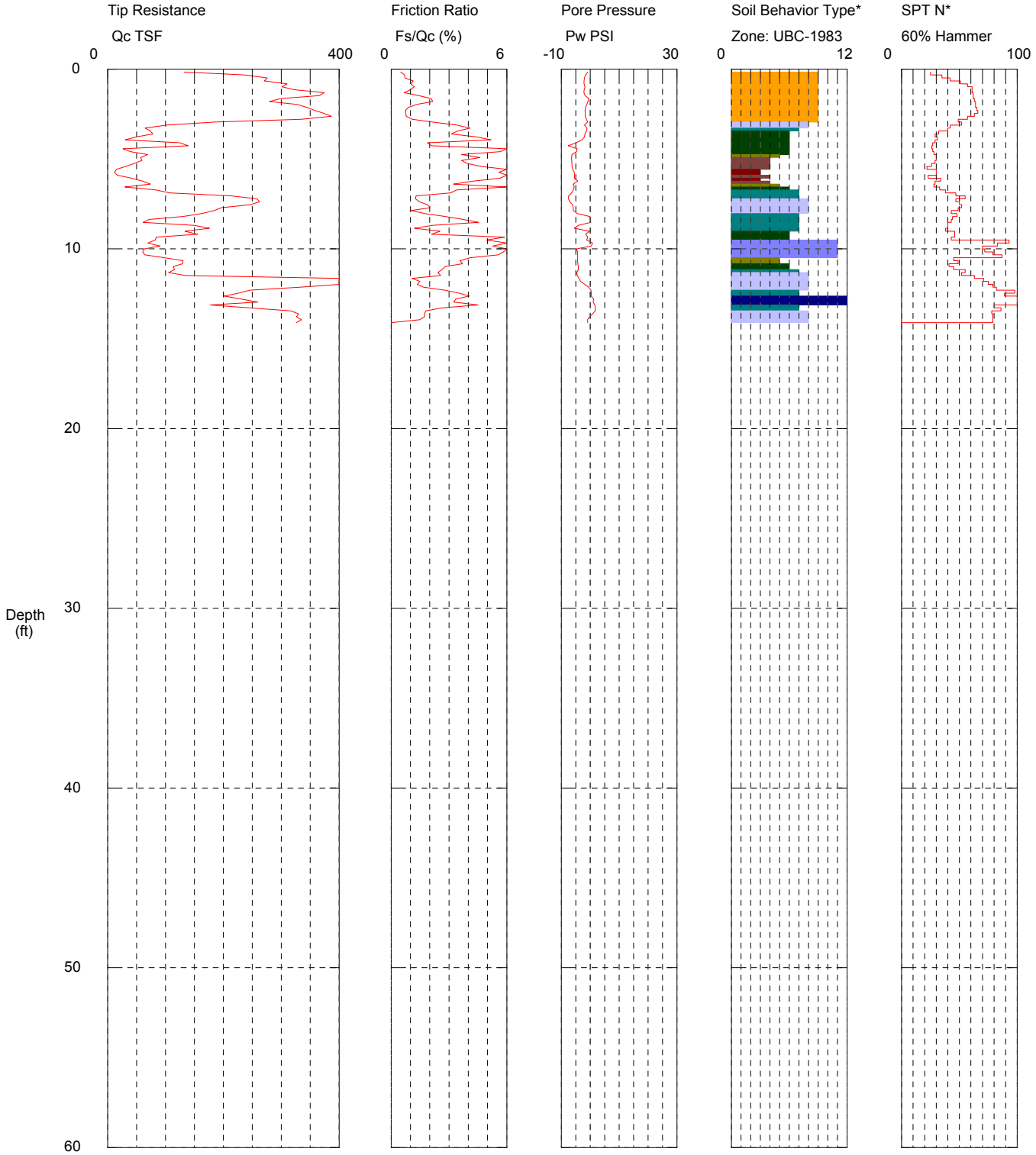
*Soil behavior type and SPT based on data from UBC-1983

EX-0001-006846-PCE

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-102D
Cone Used: DDG1263

CPT Date/Time: 11/10/2014 1:06:10 PM
Location: Port of Vancouver
Job Number: 860058



Maximum Depth = 14.11 feet

Depth Increment = 0.164 feet

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

InSitu Engineering

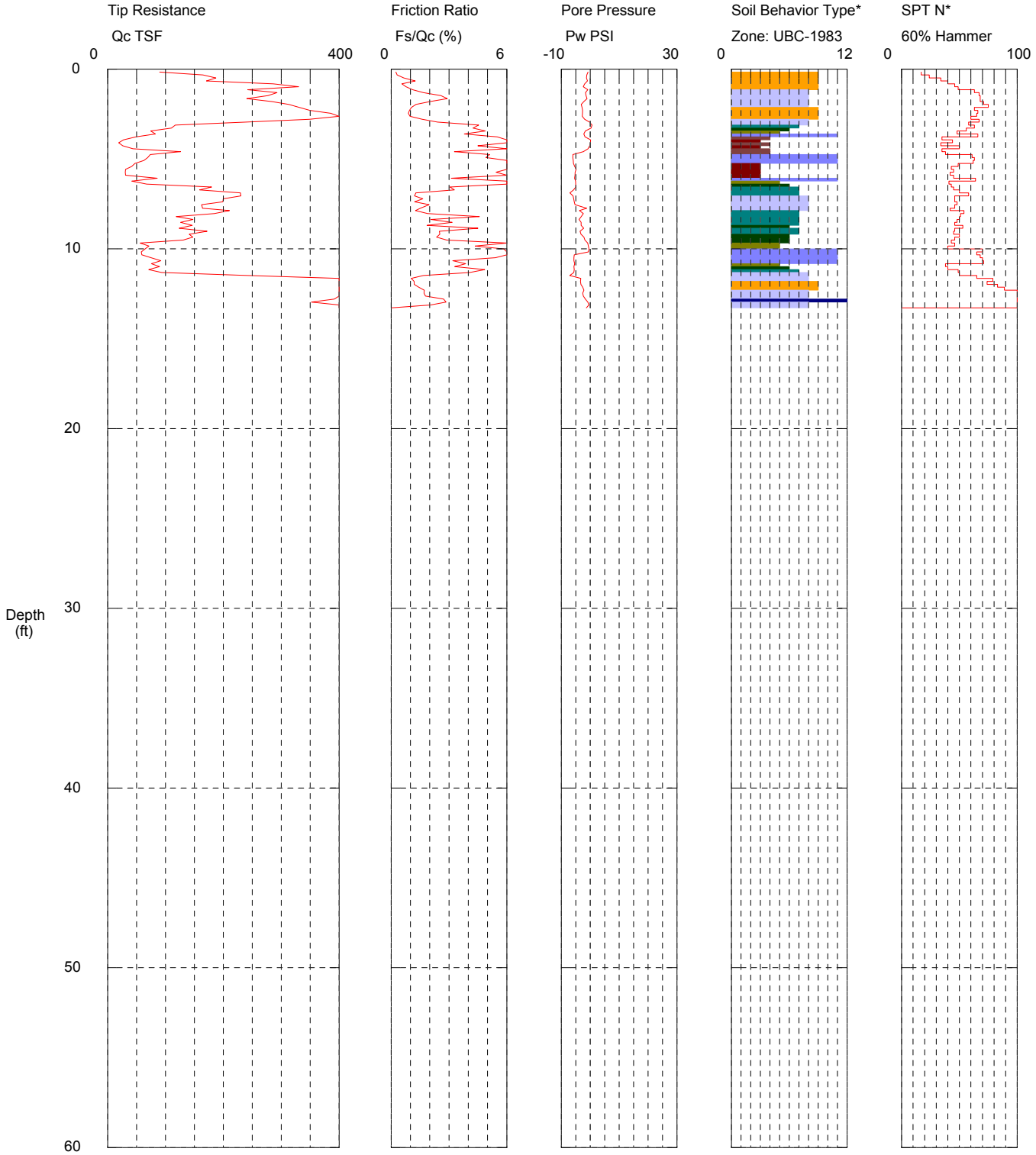
*Soil behavior type and SPT based on data from UBC-1983

EX-0001-006847-PCE

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-102E
Cone Used: DDG1263

CPT Date/Time: 11/10/2014 1:50:07 PM
Location: Port of Vancouver
Job Number: 860058



Maximum Depth = 13.29 feet

Depth Increment = 0.164 feet

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

InSitu Engineering

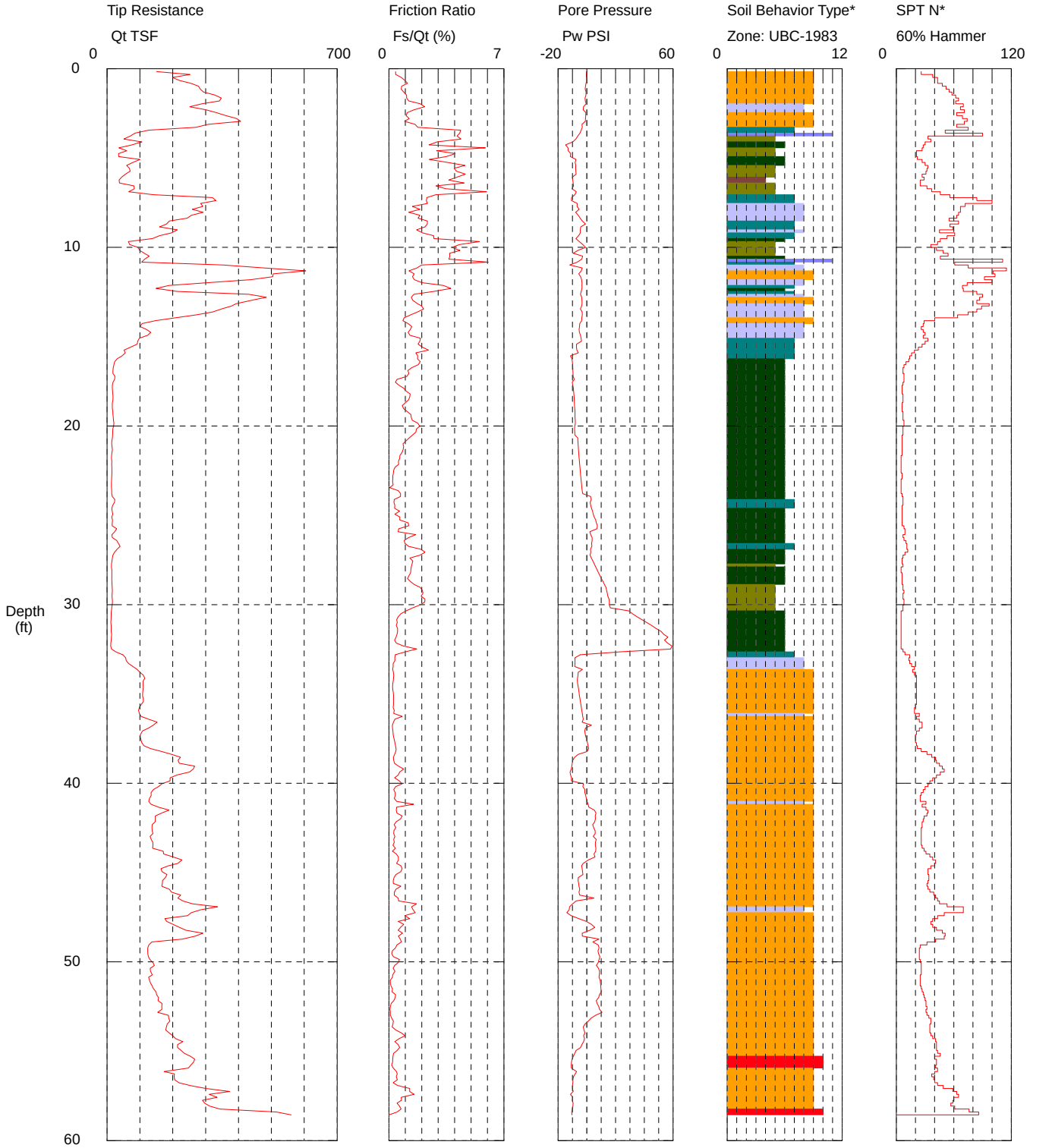
*Soil behavior type and SPT based on data from UBC-1983

EX-0001-006848-PCE

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-102
Cone Used: DDG1238

CPT Date/Time: 11/11/2014 9:04:10 AM
Location: Port of Vancouver -Tesoro Distr bution Center
Job Number:



Maximum Depth = 58.56 feet

Depth Increment = 0.164 feet

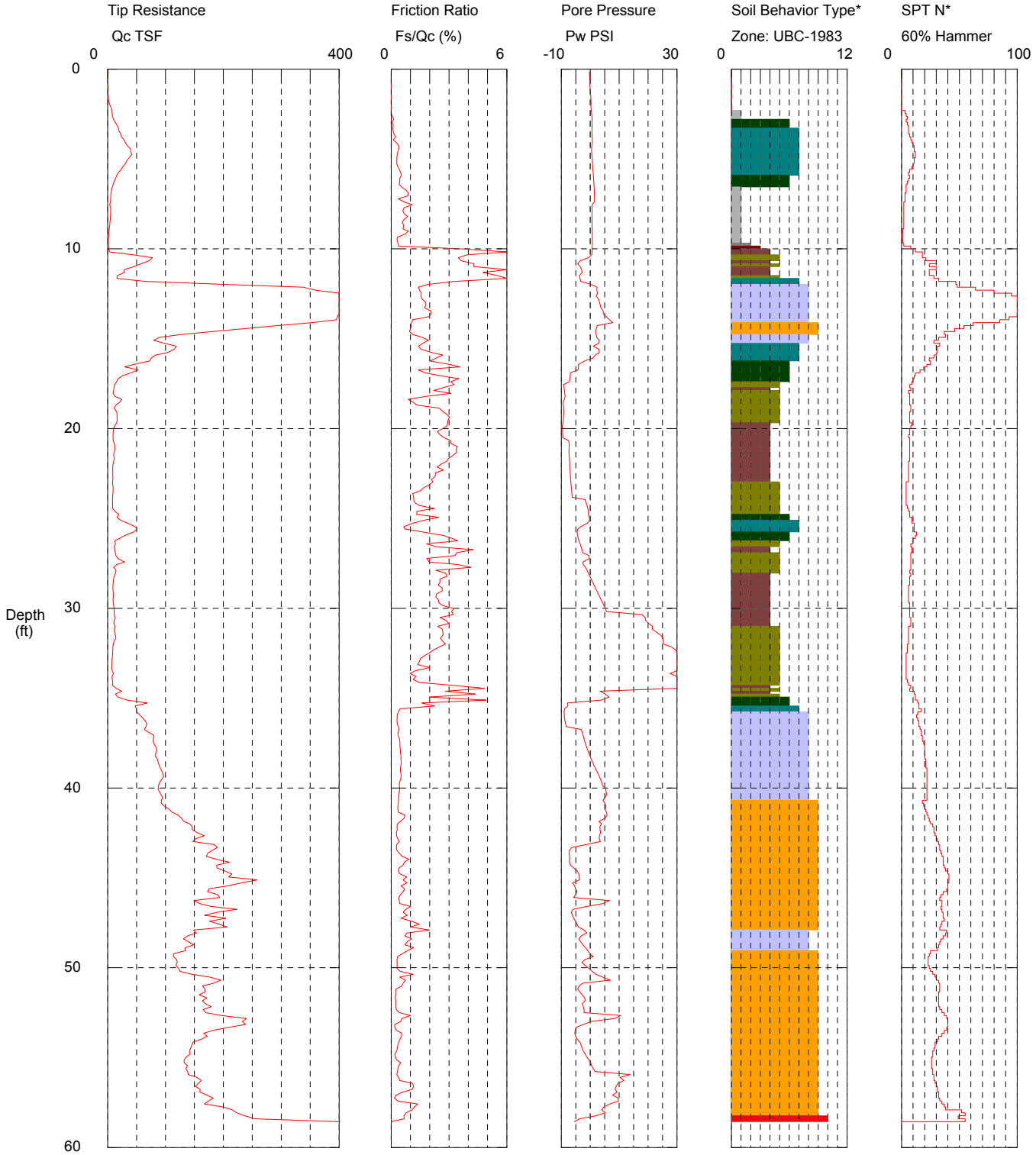
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

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

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-103
Cone Used: DDG1254

CPT Date/Time: 11/19/2014 11:43:07 AM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 58.56 feet

Depth Increment = 0.164 feet

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

InSitu Engineering Predrilled top 10 feet

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

EX-0001-006850-PCE

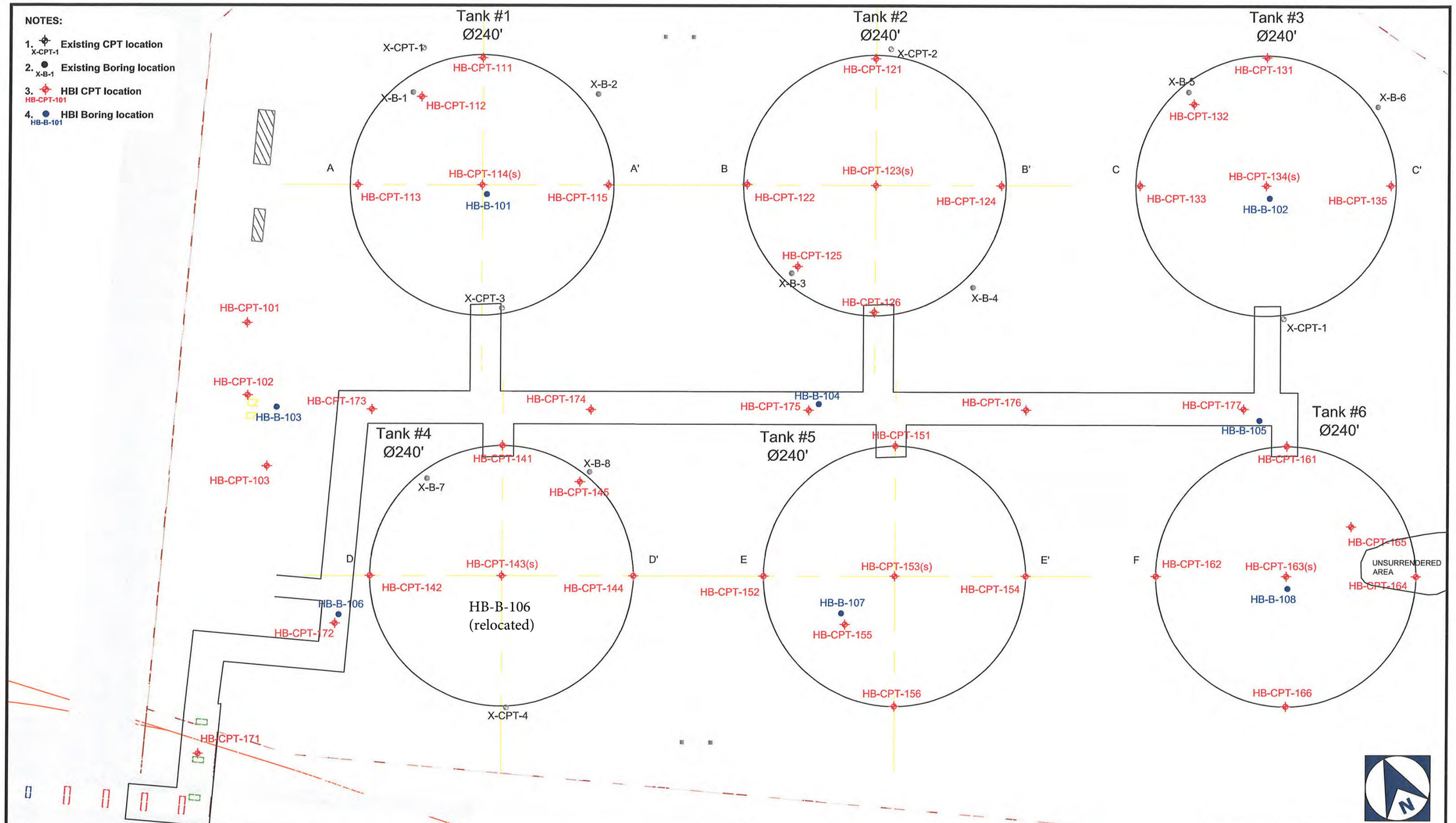
Appendix C

Geotechnical Boring (SPT) Logs

DRAFT

Appendix C
Boring Logs

Exploration Location Map



Notes:
1. Work will conducted in two phases. No work will be conducted in areas marked as Phase 2 until necessary clean-up has been completed and HBI is notified to proceed in those areas.
2. Spoils will be disposed of on site in a location agreed upon with the Owner.
3. No subsurface contamination is anticipated during this work. Contaminated spoils will be handled as per the work plan.
4. HBI will coordinate activities with the Port's project managers daily with regards to the use of Parcel 1A by other contractors.

Scale: 1":80'

0' 40' 80' 160'

PROJECT : Tesoro Distribution Terminal

Vancouver, Washington

SHEET TITLE : Geotechnical Soil Investigation

AREA 300 - Tank Area

DRWG. NO. 860058-001

RELATED DRWG:

DRWG. BY a m m

DATE 19 May 2014

APRVD. BY M Rohrbach

REVISIONS			
NO.	DRAWN BY		DATE
0	s p b	11	Feb 2014
1	a m m	24	Apr 2014
1	d c f	3	May 2014

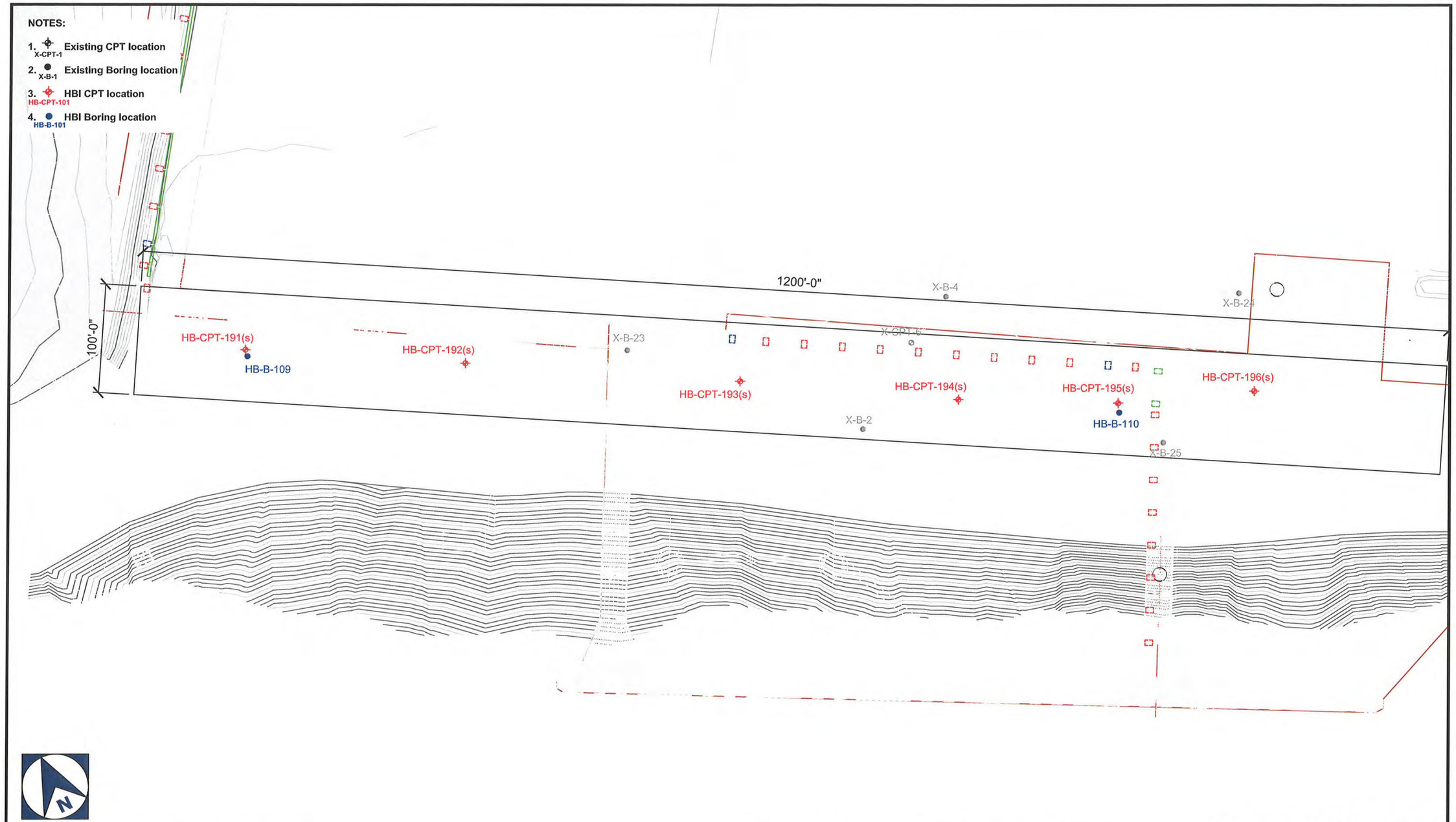
HAYWARD BAKER

A MUELLER COMPANY

PROJECT NUMBER: 860058

1 Sheets 1 of 2

EX-0001-006855-PCE



Notes:
1. Spoils will be disposed of on site in a location agreed upon with the Owner.
3. No subsurface contamination is anticipated during this work. Contaminated spoils will be handled as per the work plan.
4. HBI will coordinate activities with the Port's project managers daily with regards to the use site by other contractors.

Scale: 1":80'

0' 40' 80' 160'

PROJECT : Tesoro Distribution Terminal

Vancouver, Washington

SHEET TITLE : Geotechnical Soil Investigation

AREA 400 - Dock & Pipeline

DRWG. NO.	860058-002
RELATED DRWG:	
DRWG. BY	a m m
DATE	19 May 2014
APRVD. BY	M Rohrbach

REVISIONS

NO.	DRAWN BY	DATE
0	s p b	11 Feb 2014
1	a m m	24 Apr 2014

HAYWARD BAKER

A KELLER COMPANY

PROJECT NUMBER: 860058

1 Sheets 2 of 2

EX-0001-006857-PCE

Boring Location Coordinates

Project Name HAYWARD BAKER BORING-CPT (HAYWARD BAKER BORING CPT.DWG)
Project # 15622 (HBI CPT-BORING.csv)
Date 11/6/14
Crew F. COSTANZA, A. DILL

MacKay + Sposito

STA@ 1060	BS@ 205	6400	STA@	BS@	
HT= 6 38 CB	HR= 6 59	6458	HT=	HR=	
STA@	BS@		STA@	BS@	
HT=	HR=		HT=	HR=	
STA@	BS@		STA@	BS@	
HT=	HR=		HT=	HR=	

Point	Northing	Easting	Elev.	Description	Hub Elev.	Cut / Fill	Store No.
6300	120964.5607	1072349.682	0	HB-CPT-196(S)	27.94	~	~~6400~~ 6458
6301	121009.3994	1072232.252	0	HB-CPT-195(S)	27.25	~	6456
6302	121076.6474	1072101.527	0	HB-CPT-194(S)	27.09	~	6455
6303	121179.0322	1071929.39	0	HB-CPT-193(S)	26.88	~	6454
6304	121304.071	1071709.597	0	SET 20' (E) OF PNT HB-CPT-192(S)	27.20 FALLS IN ROCK PILE		6453
6305	121402.5404	1071534.943	0	HB-CPT-191(S)	27.42	~	6452
6306	121396.3319	1071534.061	0	HB-B-109	27.54	~	6451
6307	121001.1169	1072229.35	0	HB-B-110	27.29	~	6457
6308	122119.1913	1073789.129	0	HB-CPT-171	26.34	~	6401
6309	122173.2003	1073953.276	0	HB-CPT-172	27.49	~	6402
6310	122198.8178	1074001.039	0	HB-CPT-142	27.55	~	6403
6311	122146.3409	1074108.956	0	HB-CPT-143(S)	27.34	~	6404
6312	122093.8639	1074216.874	0	HB-CPT-144	27.40	~	6405
6313	122193.2851	1074210.453	0	HB-CPT-145	27.86	~	6406
6314	122254.2583	1074161.434	0	HB-CPT-141	27.93	~	6407
6315	122330.9513	1073960.019	0	HB-CPT-103	27.85	~	6408
6316	122397.3714	1073972.572	0	HB-CPT-102	28.14	~	6409

Date Staked: _____

Checked By: _____

Project Name HAYWARD BAKER BORING-CPT (HAYWARD BAKER BORING CPT.DWG)

Project # 15622 (HBI CPT-BORING.csv)

Date

Crew

MacKay + Sposito

STA@	BS@		STA@	BS@	
HT=	HR=		HT=	HR=	
STA@	BS@		STA@	BS@	
HT=	HR=		HT=	HR=	
STA@	BS@		STA@	BS@	
HT=	HR=		HT=	HR=	

Point	Northing	Easting	Elev.	Description	Hub Elev.	Cut / Fill	Store No.
6317	122336.2939	1074068.906	0	HB-CPT-173	27.93	~	6411
6318	122248.8348	1074248.259	0	HB-CPT-174	28.27	~	6412
6319	122161.3758	1074427.612	0	HB-CPT-175	28.22	~	6413
6320	122162.4349	1074438.423	0	HB-B-104	28.04	~	6414
6321	122376.1051	1073991.536	0	HB-B-103	28.18	~	6410
6322	122178.878	1073959.936	0	HB-B-106	27.59	~	6418
6323	122457.7691	1074001.038	0	HB-CPT-101	28.81	~	6419
6324	122528.2569	1074146.642	0	HB-CPT-113	28.93	~	6420
6325	122478.7869	1074248.449	0	HB-CPT-114(S)	29.26	~	6423
6326	122469.223	1074248.486	0	HB-B-101	29.20	~	6424
6327	122428.4275	1074351.888	0	HB-CPT-115	29.17	~	6425
6328	122576.1083	1074233.972	0	HB-CPT-112	29.39	~	6421
6329	122584.1463	1074299.555	0	HB-CPT-111	29.50	~	6422
* 6330	122373.2375	1074467.403	0	HB-CPT-122	29.33	~	6426
6331	122285.2002	1074475.767	0	HB-CPT-125	28.58	~	6427
6332	122216.4809	1074520.671	0	HB-CPT-126	28.38	~	6428
6333	122270.5412	1074676.133	0	HB-CPT-124	28.57	~	6430

Date Staked: _____

Checked By: _____

Project Name HAYWARD BAKER BORING-CPT (HAYWARD BAKER BORING CPT.DWG)

Project # 15622 (HBI CPT-BORING.csv)

Date

Crew

MacKay Sposito

STA@	BS@		STA@	BS@	
HT=	HR=		HT=	HR=	
STA@	BS@		STA@	BS@	
HT=	HR=		HT=	HR=	
STA@	BS@		STA@	BS@	
HT=	HR=		HT=	HR=	

Point	Northing	Easting	Elev.	Description	Hub Elev.	Cut / Fill	Store No.
6334	122321.2339	1074572.546	0	HB-CPT-123(S)	28.70	~	6429
6335	122426.5527	1074622.973	0	HB-CPT-121	29.73	~	6450
6336	122074.3521	1074607.176	0	HB-CPT-176	28.59	~	6446
6337	121987.5851	1074787.742	0	HB-CPT-177	28.66	~	6437
6338	121971.6681	1074796.496	0	HB-B-105	28.57	~	6438
6339	122214.7396	1074791.464	0	HB-CPT-133	28.64	~	6431
6340	122163.9779	1074895.906	0	HB-CPT-134(S)	28.53	~	6434
6341	122113.8709	1074999.607	0	HB-CPT-135	28.58	~	6436
6342	122152.1173	1074893.749	0	HB-B-102	28.73	~	6435
6343	122261.8277	1074868.885	0	HB-CPT-132	28.65	~	6432
6344	122270.1999	1074947.753	0	HB-CPT-131	28.69	~	6433
6360	122041.387	1074324.792	0	PH2 HB-CPT-152	27.23	~	6447
6361	121968.8459	1074372.004	0	PH2 HB-CPT-155	26.90	~	6448
6362	121988.9102	1074432.709	0	PH2 HB-CPT-153(S)	26.03	~	6416
6363	121880.9926	1074380.233	0	PH2 HB-CPT-156	25.01	~	6449
6364	121936.4332	1074540.627	0	PH2 HB-CPT-154	25.39	~	6445
6365	122096.8275	1074485.186	0	PH2 HB-CPT-151	28.42	~	6415

Date Staked: _____

Checked By: _____

Project Name HAYWARD BAKER BORING-CPT (HAYWARD BAKER BORING CPT.DWG)
Project # 15622 (HBI CPT-BORING.csv)
Date
Crew

MacKay Sposito

STA@	BS@		STA@	BS@	
HT=	HR=		HT=	HR=	
STA@	BS@		STA@	BS@	
HT=	HR=		HT=	HR=	
STA@	BS@		STA@	BS@	
HT=	HR=		HT=	HR=	

Point	Northing	Easting	Elev.	Description	Hub Elev.	Cut / Fill	Store No.
6366	121883.9559	1074648.544	0	PH2 HB-CPT-162	27.91	~	6444
6367	121723.5614	1074703.985	0	PH2 HB-CPT-166	27.44	~	6443
6368	121779.002	1074864.379	0	PH2 HB-CPT-164	FALLS	ON	PIPE
6369	121846.7258	1074830.095	0	PH2 HB-CPT-165	28.71	~	6440
6370	121939.3964	1074808.938	0	PH2 HB-CPT-161	28.65	~	6439
6371	121831.479	1074756.461	0	PH2 HB-CPT-163(S)	27.81	~	6441
6372	121820.7363	1074752.679	0	PH2 HB-B-108	27.73	~	6442
6373	121979.5838	1074373.519	0	PH2 HB-B-107	27.01	~	6417

Date Staked: _____

Checked By: _____

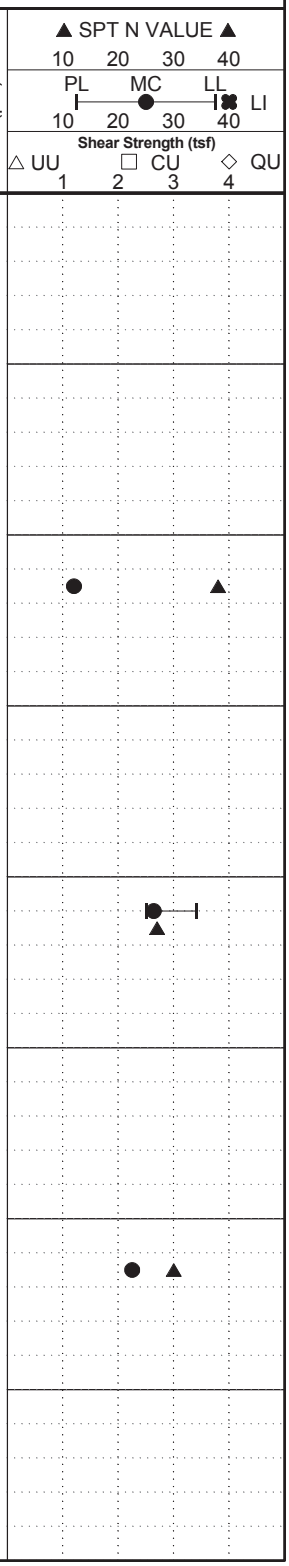
Final Boring Logs

Hayward Baker

BORING NUMBER HB-B-101

PROJECT NAME Tesoro Savage Petro eum Term na
PROJECT LOCATION Port of Vancouver, WA
DATE STARTED 11/20/14 **COMPLETED** 11/20/14
NORTH: 122469.223 **EAST:** 1074248.486
METHOD Mud Rotary, CME 75, Gregory Dr ng
GROUND ELEVATION 29.2 ft
NOTES Automat c hammer eff c ency: 80%
AT TIME OF DRILLING N/A
AT END OF DRILLING N/A

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)
0.0		SM					
2.5							
5.0		24 2					
		SP SM	S 1	33	12 16 22 (38)	8.9	
7.5							
10.0		19 2					
		SP SM	S 2	78	16 14 13 (27)	9.6	
12.5							
15.0		Bor ng s uffed redr to 15' 14 2					
		SM, ~2" ense f ne SP	S 3	89	9 14 16 (30)	32.6	
17.5							
20.0		9 2					



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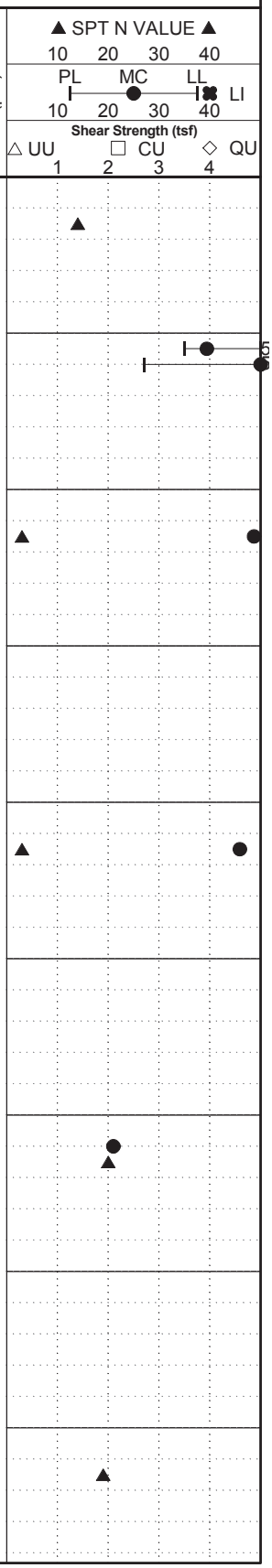
HAYWARD BAKER
Geotechnical Construction
KELLER

Hayward Baker

BORING NUMBER HB-B-101

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)
20.0		CL or CH	S 4	33	2 6 8 (14)		
22.5			TW 1				
25.0		4 2					
		SM or SC	S 5	100	0 2 1 (3)	49.3	
27.5							
		0 2					
		SM					
30.0		(poss b e heave)	S 6	133	0 1 2 (3)	19.2	
32.5		-3 8					
		SP SM					
		Dr er notes, harder dr ng at 33'					
35.0			S 7	67	9 10 10 (20)	7.4	
37.5							
40.0		-10 8					
		SP SM	S 8	22	7 8 11 (19)		



HAYWARD BAKER - LOG POR OF VANCOUVER ESORO PE ROLEUM GPJ A C GD 1/13/15

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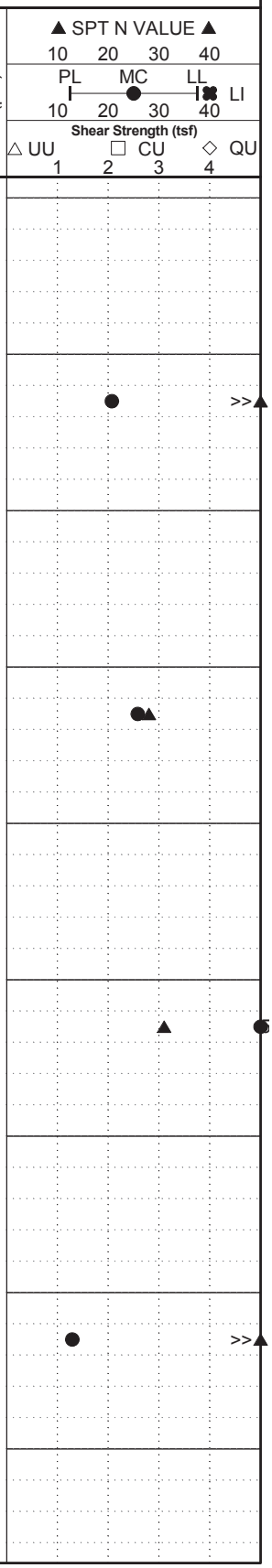
BORING NUMBER HB-B-101

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)
42.5		SP SM *(continued)*					
45.0			S 9	50	20 30 23 (53)	11.9	
47.5							
50.0			S 10	67	10 15 13 (28)	7.8	
52.5							
55.0		-25 8					
		SM MC = 55.9%	S 11	67	14 14 17 (31)	16.8	
57.5							
60.0		-30 8					
		SP SM	S 12	33	26 50 50 (100)	6.1	
62.5							



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BORING NUMBER HB-B-101

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ 10 20 30 40 / PL MC LL LI 10 20 30 40 / Shear Strength (tsf) △ UU 1 □ CU 2 3 ◇ QU 4
65.0		SP SM *(continued)* -35 8						
		SP, w th f ne grave -37 3	S 13	33	14 26 20 (46)	3		● ▲

Hayward Baker

BORING NUMBER HB-B-102

PROJECT NAME Tesoro Savage Petro eum Term na
PROJECT LOCATION Port of Vancouver, WA
DATE STARTED 11/20/14 **COMPLETED** 11/21/14
NORTH: 122152.1173 **EAST:** 1074893.749
METHOD Mud Rotary, CME 75, Gregory Dr ng
GROUND ELEVATION 28.73 ft
NOTES Automat c hammer eff c ency: 80%
AT TIME OF DRILLING N/A
AT END OF DRILLING N/A

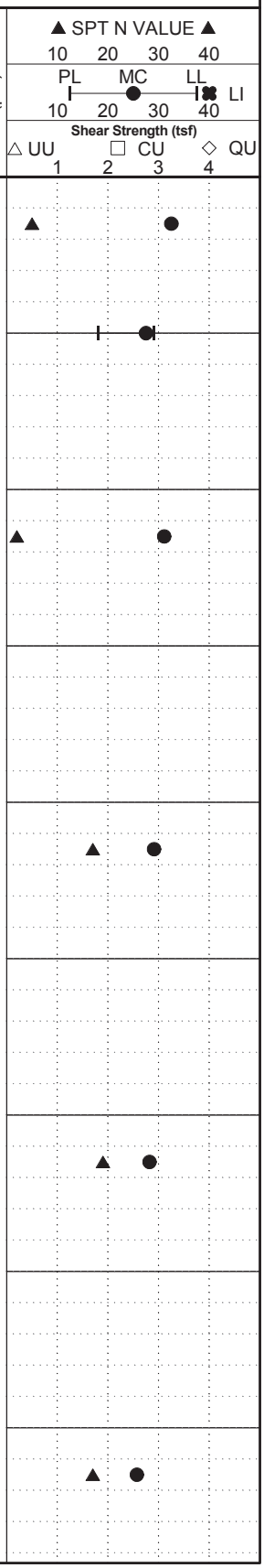
DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ 10 20 30 40 / PL MC LL LI 10 20 30 40 / Shear Strength (tsf) △ UU 1 □ CU 2 ◇ QU 3 4
0.0		SM						
2.5								
5.0			S 1	100	3 4 6 (10)	40.5		▲ ●
7.5								
10.0			S 2	78	1 3 4 (7)	43.8		▲ ●
12.5								
15.0		13 7						
		SM, w th f ne to med um grave s	S 3	67	13 21 15 (36)	19.9		● ▲
17.5								
20.0		8 7						

(Continued Next Page)

BORING NUMBER HB-B-102

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)
20.0		CL	S 4	100	1 2 3 (5)	79.9	
22.5			TW 1				
25.0		3 7					
		SC (poss b e heave)	S 5	133	1 1 1 (2)	25	
27.5							
30.0		-1 3					
		SM	S 6	67	5 8 9 (17)	9.6	
32.5							
35.0			S 7	56	10 11 8 (19)	8.8	
37.5							
40.0			S 8	39	3 8 9 (17)	6	



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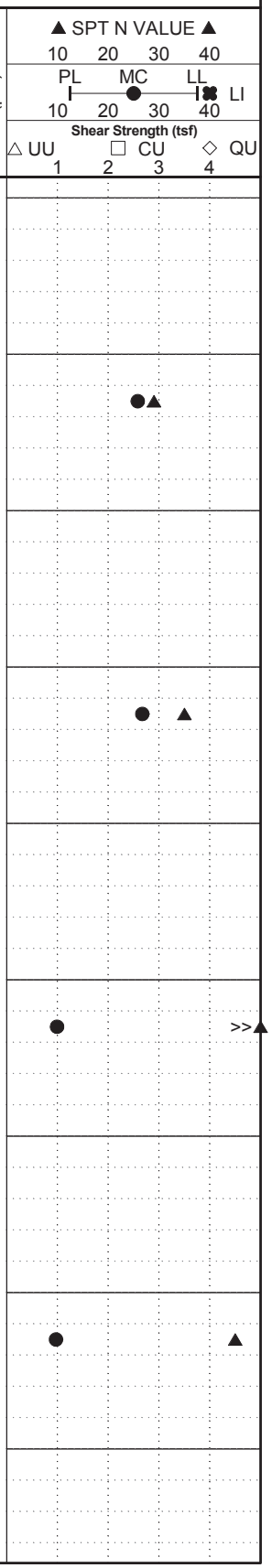
PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)
42.5		SM *(continued)*					
45.0		-16 3					
		SM	S 9	72	1 13 16 (29)	7.9	
47.5							
50.0		-21 3					
		SP SM	S 10	61	11 16 19 (35)	11.7	
52.5							
55.0		-26 3					
		SP or SW, 2 nch ayer f ne angu ar grave s, sand had f ne to med um grave s	S 11		24 28 34 (62)	3.4	
57.5							
60.0		-31 3					
		GP, w th f ne grave (some fractured)	S 12	39	25 23 22 (45)	1.5	
62.5		Dr chatter to 65'					



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BORING NUMBER HB-B-102

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ 10 20 30 40 / PL MC LL LI 10 20 30 40 / Shear Strength (tsf) △ UU 1 □ CU 2 3 ◇ QU 4
65.0		GP, w th f ne grave (some fractured) *(continued)*						
		Dr ho e co apsed to 60' Redr and samp e after 8 12 hours -37 8	S 13	67	15 50 50 (100)	3.7		>>▲

HAYWARD BAKER
Geotechnical Construction
KELLER

Hayward Baker

BORING NUMBER HB-B-103

PROJECT NAME Tesoro Savage Petro eum Term na
PROJECT LOCATION Port of Vancouver, WA
DATE STARTED 11/19/14 **COMPLETED** 11/19/14
NORTH: 122376.1051 **EAST:** 1073991.536
METHOD Mud Rotary, CME 75, Gregory Dr ng
GROUND ELEVATION 28.18 ft
NOTES Automat c hammer eff c ency: 80%
AT TIME OF DRILLING N/A
AT END OF DRILLING N/A

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ 10 20 30 40 / PL MC LL LI 10 20 30 40 / Shear Strength (tsf) △ UU 1 □ CU 2 3 ◇ QU 4
0.0		SM						
2.5								
5.0			S 1	33	7 7 7 (14)			
7.5								
10.0			S 2	39	6 13 13 (26)			
12.5								
15.0			S 3	50	10 13 7 (20)	19		
17.5		Organ cs at 16.5'. 10 7						
20.0		CL 18' 19.5' No Recovery 18.5' 20.5' No Recovery	TW 0	0				

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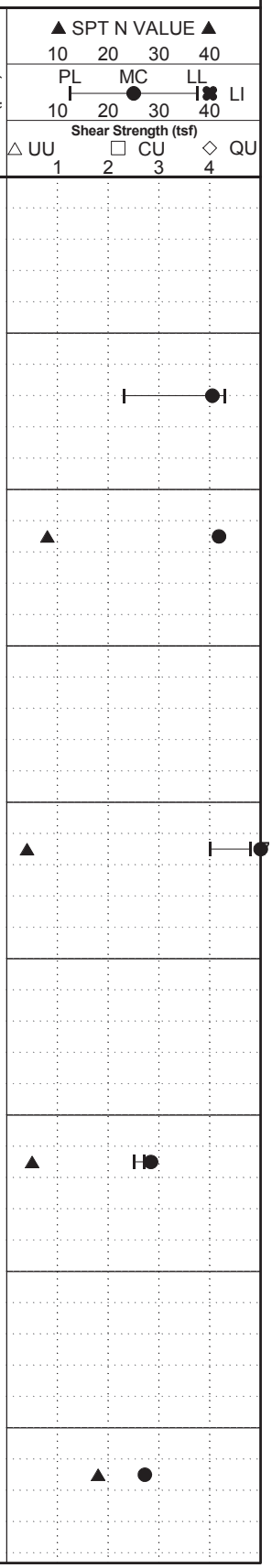
PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)
20.0		CL *(continued)*					
22.5			TW 1	89			
25.0			S 4	89	1 4 4 (8)	44.1	
27.5							
30.0		-1 8					
		ML MC = 72.2%	S 5	100	0 2 2 (4)	93.1	
32.5							
35.0		-6 8					
		SM Dr er reports harder dr ng	S 6	100	1 2 3 (5)	27.1	
37.5							
40.0		-11 8					
		SP SM	S 7	61	8 9 9 (18)	10.2	



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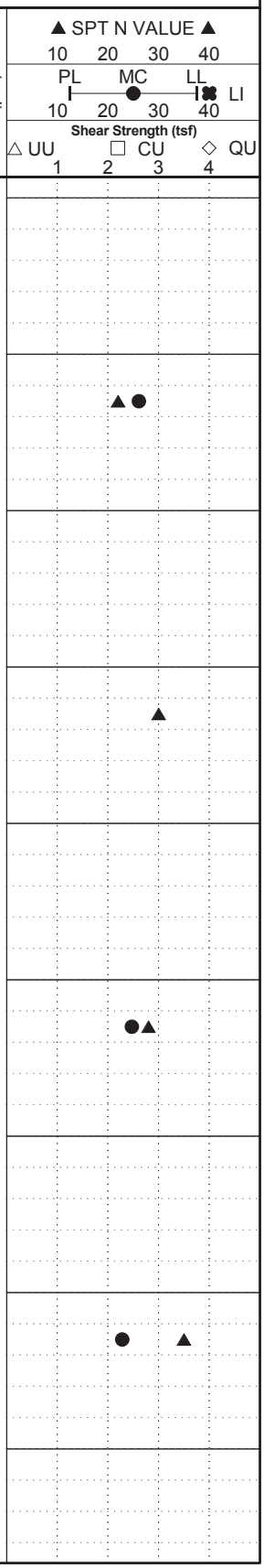
PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)
42.5		SP SM *(continued)* Dr er notes grave at 42.5 feet					
45.0		-16 8 SP SM	S 8	50	6 9 13 (22)	8.1	
47.5							
50.0		Poor recovery, grave n shoe	S 9	11	12 16 14 (30)		
52.5							
55.0		-26 8 SP SM	S 10	39	10 14 14 (28)	7.4	
57.5							
60.0		-31 8 SP SM	S 11	33	16 21 14 (35)	6.3	
62.5							



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PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ 10 20 30 40 / PL MC LL LI 10 20 30 40 / Shear Strength (tsf) △ UU 1 □ CU 2 3 ◇ QU 4
65.0		SP SM *(continued)* -36 8						
		SP or GP, grave stuck n shoe -38 3	S 12	39	11 27 22 (49)	2.9		

HAYWARD BAKER Geotechnical Construction
KELLER

Hayward Baker

BORING NUMBER HB-B-104

PROJECT NAME Tesoro Savage Petro eum Term na
PROJECT LOCATION Port of Vancouver, WA
DATE STARTED 11/21/14 **COMPLETED** 11/22/14
NORTH: 122162.4349 **EAST:** 1074438.423
METHOD Mud Rotary, CME 75, Gregory Dr ng
GROUND ELEVATION 28.04 ft
NOTES Automat c hammer eff c ency: 80%
AT TIME OF DRILLING N/A
AT END OF DRILLING N/A

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ 10 20 30 40 / PL MC LL LI 10 20 30 40 / Shear Strength (tsf) △ UU 1 □ CU 2 3 ◇ QU 4
0.0		SM, w th br ck and grave						
2.5								
5.0			S 1	78	12 34 30 (64)	9.7		>>▲
7.5								
10.0		18 0						
		SP SM, w th f ne grave s	S 2	67	15 16 11 (27)	10.4		▲
12.5								
15.0			S 3	56	5 7 7 (14)			▲
17.5		10 5						
		ML						
20.0								

(Continued Next Page)

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ 10 20 30 40 / PL MC LL LI 10 20 30 40 / Shear Strength (tsf) △ UU 1 □ CU 2 3 ◇ QU 4
20.0		ML *(continued)*	TW 1					
22.5								
25.0								
		3 0						
		CL or SC or ML	S 4	100	2 2 2 (4)			
27.5								
30.0								
		-2 0						
		SC, c ayey sand	S 5	78	0 0 2 (2)			
32.5								
35.0								
		-7 0						
		SM	S 6	78	6 7 7 (14)	16.4		
37.5								
		-10 5						
		SP SM						
40.0			S 7	56	7 9 10 (19)	6.1		

(Continued Next Page)

BORING NUMBER HB-B-104

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ 10 20 30 40 / PL MC LL LI 10 20 30 40 / Shear Strength (tsf) △ UU 1 □ CU 2 3 ◇ QU 4
42.5		SP SM *(continued)*						
45.0		Coarser sand	S 8	44	9 9 10 (19)	6.7		▲ ●
47.5								
50.0			S 9	94	5 10 10 (20)	5.8		▲ ●
52.5								
55.0			S 10	78	9 11 13 (24)	5.5		▲ ●
57.5								
60.0		-32 0 SP SM or GP, w th f ne to med um grave s	S 11	56	32 32 34 (66)	4.6		● >>▲
62.5								

(Continued Next Page)

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ 10 20 30 40 / PL MC LL LI 10 20 30 40 / Shear Strength (tsf) △ UU 1 □ CU 2 3 ◇ QU 4
65.0		SP SM or GP, w th f ne to med um grave s *(continued)*						
		-38 5	S 12	56		5.6		

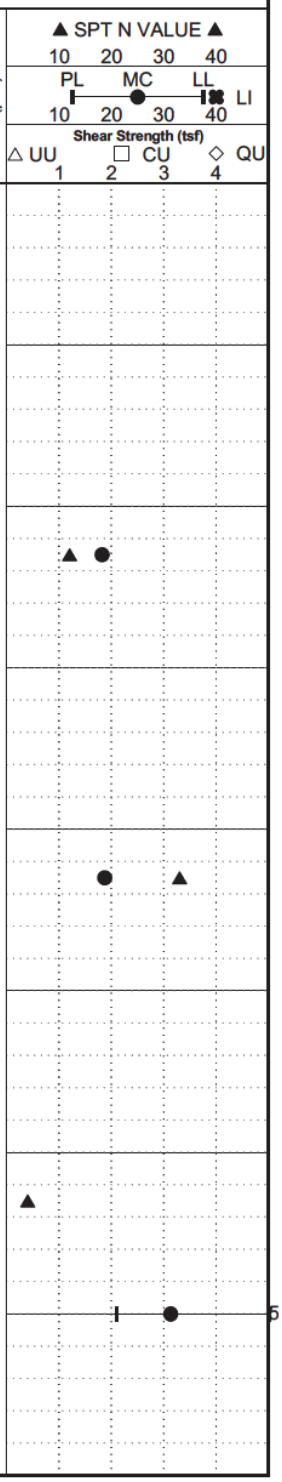
HAYWARD BAKER
Geotechnical Construction
KELLER

Hayward Baker

BORING NUMBER HB-B-105

PROJECT NAME Tesoro Savage Petro eum Term na
PROJECT LOCATION Port of Vancouver, WA
DATE STARTED 11/21/14 **COMPLETED** 11/21/14
NORTH: 121971.6681 **EAST:** 1074796.496
METHOD Mud Rotary, CME 75, Gregory Dr ng
GROUND ELEVATION 28.51 ft
NOTES Automat c hammer eff c ency: 80%
AT TIME OF DRILLING N/A
AT END OF DRILLING N/A

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)
0.0		**SM or SC**					
2.5							
5.0		**1" ense SP (f ne graded sand)**	S 1	89	4 5 7 (12)	28.5	
7.5							
10.0		18 5 **SC, c ayey sand**	S 2	33	5 16 17 (33)	30.7	
12.5							
15.0		13 5 **CH**	S 3		2 1 3 (4)	76.5	
17.5			TW 1				
20.0		8 5					



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HAYWARD BAKER - LOG POR OF VANCOUVER ESORO PE ROLEUM GPJ A C GD 1/13/15

HAYWARD BAKER Geotechnical Construction
KELLER

Hayward Baker

BORING NUMBER HB-B-105

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ 10 20 30 40 / PL MC LL LI 10 20 30 40 / Shear Strength (tsf) △ UU 1 □ CU 2 3 ◇ QU 4
20.0		CL	S 4	100	0 1 1 (2)	57.3		▲ ●
22.5								
25.0		3 5						
		SP SM	S 5	78	4 7 6 (13)	9.5		▲ ●
27.5								
30.0			S 6	78	6 9 10 (19)	6.9		▲ ●
32.5								
35.0			S 7	56	7 8 9 (17)	6.7		▲ ●
37.5								
40.0			S 8	78	13 18 17 (35)	7.3		● ▲

(Continued Next Page)

BORING NUMBER HB-B-105

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)
42.5		SP SM *(continued)*					
45.0			S 9	83	10 10 19 (29)	6.5	
47.5							
50.0			S 10	78	8 11 10 (21)	7.1	
52.5							
55.0		W th f ne to med um grave s near bottom depth n samp e	S 11	78	10 13 34 (47)	8.4	
57.5		Dr ng chatter from 58 feet to 60 feet					
60.0		-31 5 SP or GP, (coarser sand) w th f ne to med um grave s	S 12		42 34 43 (77)	4.9	
62.5							

▲ SPT N VALUE ▲ 10 20 30 40
PL MC LL LI 10 20 30 40
Shear Strength (tsf)
△ UU 1 □ CU 2 3 ◇ QU 4

(Continued Next Page)

Hayward Baker

BORING NUMBER HB-B-105

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ 10 20 30 40 / PL MC LL LI 10 20 30 40 / Shear Strength (tsf) △ UU 1 □ CU 2 3 ◇ QU 4
65.0		-36 5						
		SP SM or GP, w th fractured grave s -38 0	S 13	89	40 50 50 (100)	6.5		>>▲

HAYWARD BAKER - LOG POR OF VANCOUVER ESORO PE ROLEUM GPJ A C GD 1/13/15

HAYWARD BAKER Geotechnical Construction
KELLER

Hayward Baker

BORING NUMBER HB-B-106

PROJECT NAME Tesoro Savage Petro eum Term na
PROJECT LOCATION Port of Vancouver, WA
DATE STARTED 11/24/14 **COMPLETED** 11/24/14
NORTH: Relocated **EAST:** Relocated 10 ft. South of CPT(s)
METHOD Mud Rotary, CME 75, Gregory Dr ng
GROUND ELEVATION 27.59 ft
NOTES Automat c hammer eff c ency: 80%
AT TIME OF DRILLING N/A
AT END OF DRILLING N/A

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ 10 20 30 40 / PL MC LL LI 10 20 30 40 / Shear Strength (tsf) △ UU 1 □ CU 2 3 ◇ QU 4
0.0		SM or SC						
2.5								
5.0		22 6						
		SM, w th f ne to med um grave s	S 1	100	27 20 23 (43)	12.1		● ▲
7.5								
10.0			S 2	6	12 14 11 (25)			▲
12.5		Grave part c es						
15.0			S 3	67	8 5 10 (15)	18.8		▲ ●
17.5								
20.0		7 6						

(Continued Next Page)

BORING NUMBER HB-B-106

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)
20.0		CL	S 4	100	0 1 3 (4)	83.1	
22.5			TW 1	0			
25.0			S 5	17	0 2 2 (4)		
27.5							
30.0		-2 4					
		CL ML, e evated b ow counts (bounc ng on rock)	S 6	11	4 4 10 (14)		
32.5							
35.0		Redr w th 3" sp t spoon recover coarse grave marked as samp e S7A	S 7	0	10 16 17 (33)		
37.5							
40.0		-12 4					
		SP SM	S 8	67	5 8 7 (15)	5.1	

▲ SPT N VALUE ▲ 10 20 30 40
PL MC LL LI 10 20 30 40
Shear Strength (tsf) △ UU 1 □ CU 2 3 ◇ QU 4

(Continued Next Page)

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)
42.5		SP SM *(continued)*					
45.0			S 9	67	10 14 21 (35)	9.1	
47.5							
50.0			S 10	83	10 15 14 (29)	9.3	
52.5							
55.0			S 11	72	10 12 11 (23)	7.4	
57.5							
60.0		-32 4					
		GP, sandy grave	S 12	72	29 37 32 (69)	4.1	
62.5		Dr chatter starts at 61.5 feet					

▲ SPT N VALUE ▲ 10 20 30 40
PL MC LL LI 10 20 30 40
Shear Strength (tsf)
△ UU 1 □ CU 2 3 ◇ QU 4

(Continued Next Page)

BORING NUMBER HB-B-106

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ 10 20 30 40 / PL MC LL LI 10 20 30 40 / Shear Strength (tsf) △ UU 1 □ CU 2 3 ◇ QU 4
65.0		GP, sandy grave *(continued)*						
		-38 9	S 13	78	34 40 47 (87)			>>▲

HAYWARD BAKER
Geotechnical Construction
KELLER

Hayward Baker

BORING NUMBER HB-B-107

PROJECT NAME Tesoro Savage Petro eum Term na
PROJECT LOCATION Port of Vancouver, WA
DATE STARTED 11/22/14 **COMPLETED** 11/22/14
NORTH: 121979.5838 **EAST:** 1074373.519
METHOD Mud Rotary, CME 75, Gregory Dr ng
GROUND ELEVATION 27.01 ft
NOTES Automat c hammer eff c ency: 80%
AT TIME OF DRILLING N/A
AT END OF DRILLING N/A

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ 10 20 30 40 / PL MC LL LI 10 20 30 40 / Shear Strength (tsf) △ UU 1 □ CU 2 3 ◇ QU 4
0.0		SP						
2.5								
5.0								
			S 1	67	5 4 4 (8)	5		▲ ●
7.5								
10.0								
		16 0	S 2	72	6 9 7 (16)	21.4		▲●
		SM						
12.5								
15.0								
			S 3	94	12 24 23 (47)	14.7		● ▲
17.5								
20.0		7 0						

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HAYWARD BAKER - LOG POR OF VANCOUVER ESORO PE ROLEUM GPJ A C GD 1/13/15

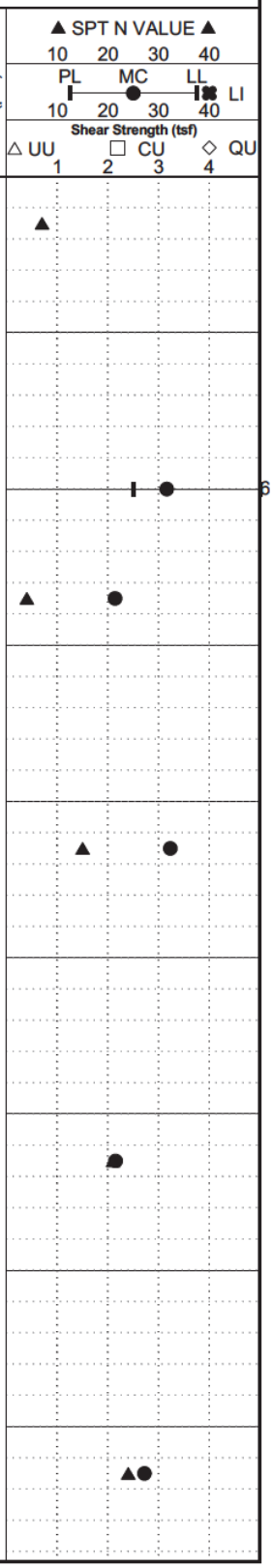
HAYWARD BAKER
Geotechnical Construction
KELLER

Hayward Baker

BORING NUMBER HB-B-107

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)
20.0		CH	S 4	89	1 3 4 (7)		
22.5							
25.0		2 0	TW 1	0			
		SM, c ayey sand					
27.5			S 5	50	1 2 2 (4)	34.9	
30.0		-3 0					
		SP SM	S 6	89	4 7 8 (15)	11.1	
32.5							
35.0		W th f ne grave s Dr chatter from 35.5 feet to 40 feet	S 7	61	8 11 10 (21)	7.9	
37.5							
40.0			S 8	56	9 12 12 (24)	6.3	



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HAYWARD BAKER - LOG POR OF VANCOUVER ESORO PE ROLEUM GPJ A C GD 1/13/15

HAYWARD BAKER
Geotechnical Construction
KELLER

Hayward Baker

BORING NUMBER HB-B-107

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ 10 20 30 40 / PL MC LL LI 10 20 30 40 / Shear Strength (tsf) △ UU 1 □ CU 2 3 ◇ QU 4
42.5		SP SM *(continued)*						
45.0			S 9	67	9 11 14 (25)	6.8		
47.5								
50.0		F ne med um grave s	S 10	67	11 10 12 (22)	6.9		
52.5								
55.0			S 11	67	13 17 17 (34)	6.3		
57.5		Dr chatter from 56.5 feet to 60 feet						
60.0		-33 0 GP, f ne med um fractured grave	S 12	11	50/4 50 50 (100)			>>▲
62.5		Dr ho e co apsed/heaved to 10 feet						

(Continued Next Page)

HAYWARD BAKER - LOG POR OF VANCOUVER ESORO PE ROLEUM GPJ A C GD 1/13/15

BORING NUMBER HB-B-107

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ 10 20 30 40 / PL MC LL LI 10 20 30 40 / Shear Strength (tsf) △ UU 1 □ CU 2 3 ◇ QU 4
65.0		GP, f ne med um fractured grave *(continued)*						
		-39 5	S 13	56	37 50 50 (100)	4.8		>>▲

Hayward Baker

BORING NUMBER HB-B-108

PROJECT NAME Tesoro Savage Petro eum Term na
PROJECT LOCATION Port of Vancouver, WA
DATE STARTED 11/24/14 **COMPLETED** 11/24/14
NORTH: 121820.7363 **EAST:** 1074752.679
METHOD Mud Rotary, CME 75, Gregory Dr ng
GROUND ELEVATION 27.73 ft
NOTES Automat c hammer eff c ency: 80%
AT TIME OF DRILLING N/A
AT END OF DRILLING N/A

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ 10 20 30 40 / PL MC LL LI 10 20 30 40 / Shear Strength (tsf) △ UU 1 □ CU 2 3 ◇ QU 4
0.0		SP SM						
2.5								
5.0								
			S 1	78	5 10 11 (21)	6.6		● ▲
7.5								
10.0								
			S 2	78	10 19 20 (39)	8.6		● ▲
12.5								
15.0		12 7						
		SM	S 3	83	15 23 21 (44)	17.6		● ▲
17.5								
20.0		7 7						

(Continued Next Page)

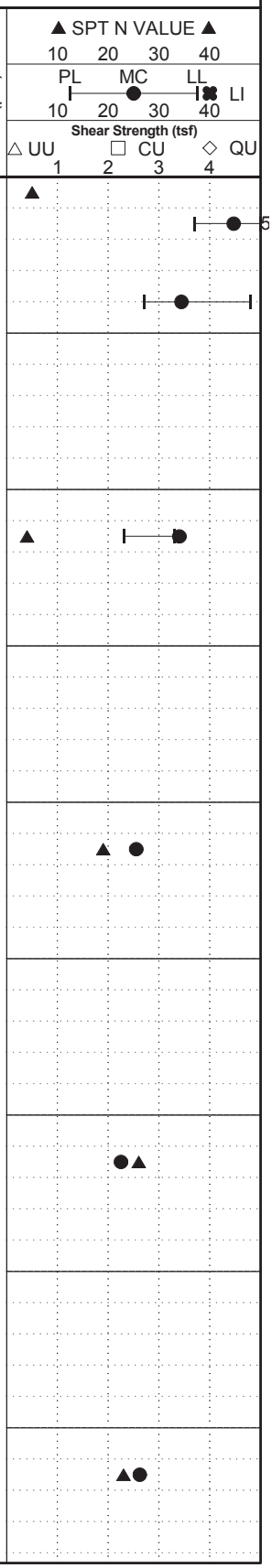
HAYWARD BAKER Geotechnical Construction
KELLER

Hayward Baker

BORING NUMBER HB-B-108

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)
20.0		MH, some organ cs (PP = 1.25 1.5)	S 4	100	2 2 3 (5)	60	
		6 7					
		CL	TW 1	100			
22.5							
25.0							
		2 2					
		ML, (PP = 1.5)	S 5	122	1 2 2 (4)	55.1	
27.5							
30.0		-2 3					
		SM, w th med um grave s	S 6	56	5 8 11 (19)	15.4	
32.5							
		-6 3					
		SP SM					
35.0			S 7	78	9 12 14 (26)	8.1	
37.5							
40.0		-12 3					
		SP, coarse sand	S 8	72	8 10 13 (23)	4.2	



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HAYWARD BAKER Geotechnical Construction
KELLER

Hayward Baker

BORING NUMBER HB-B-108

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ 10 20 30 40 / PL MC LL LI 10 20 30 40 / Shear Strength (tsf) △ UU □ CU ◇ QU 1 2 3 4
42.5		-14 8						
		SP SM						
45.0			S 9	78	11 11 10 (21)	6.1		▲●
47.5								
50.0			S 10	67	9 11 10 (21)	7.4		▲●
52.5								
55.0		W th grave (f ne to med um)	S 11	72	22 32 39 (71)	6.3		● >>▲
		Dr chatter from 56.5' to 60'						
57.5								
60.0		-32 3						
		GP, f ne to med um grave s	S 12	67	29 50 50 (100)	3.6		● >>▲
62.5								

(Continued Next Page)

BORING NUMBER HB-B-108

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ 10 20 30 40 / PL MC LL LI 10 20 30 40 / Shear Strength (tsf) △ UU 1 □ CU 2 3 ◇ QU 4
65.0		GP, f ne to med um grave s *(continued)*						
		-38 8	S 13		45 50 50 (100)	4.5		>>▲

Hayward Baker

BORING NUMBER HB-B-109

PROJECT NAME Tesoro Savage Petro eum Term na
PROJECT LOCATION Port of Vancouver, WA
DATE STARTED 11/24/14 **COMPLETED** 11/24/14
NORTH: 121396.3319 **EAST:** 1071534.061
METHOD Mud Rotary, CME 75, Gregory Dr ng
GROUND ELEVATION 27.54 ft
NOTES Automat c hammer eff c ency: 80%
AT TIME OF DRILLING N/A
AT END OF DRILLING N/A

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ 10 20 30 40 / PL MC LL LI 10 20 30 40 / Shear Strength (tsf) △ UU 1 □ CU 2 3 ◇ QU 4
0.0		SM						
2.5								
5.0								
			S 1	100	5 13 15 (28)	37.5		● ▲
7.5								
		19 0						
		SP SM						
10.0								
			S 2	61	8 11 11 (22)	7.9		● ▲
12.5								
15.0								
			S 3	0	7 15 21 (36)			▲
		Grave stuck n shoe						
17.5								
20.0		7 5						

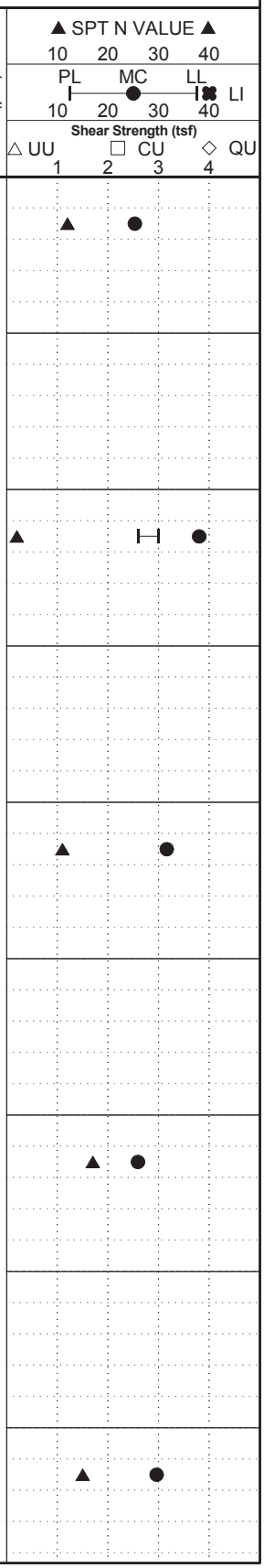
(Continued Next Page)

HAYWARD BAKER - LOG POR OF VANCOUVER ESORO PE ROLEUM GPJ A C GD 1/13/15

BORING NUMBER HB-B-109

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)
20.0		SP, med um to f ne grave s	S 4	56	5 8 4 (12)	4.6	
22.5							
25.0		2 5					
		ML	S 5	100	0 1 1 (2)	63.8	
27.5							
30.0		-2 5					
		SP	S 6	50	6 5 6 (11)	4.9	
32.5							
		-6 5					
		SP SM					
35.0			S 7	78	5 7 10 (17)	6.1	
37.5							
40.0		-12 5					
		SP SM	S 8	67	7 7 8 (15)	7	



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BORING NUMBER HB-B-109

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)
42.5		SP SM *(continued)*					
45.0			S 9	78	5 10 11 (21)	7.3	
47.5							
50.0		-22 5					
		SP	S 10	72	5 7 9 (16)	3.6	
52.5							
55.0		-27 5					
		SM	S 11	78	7 10 10 (20)	13.7	
57.5							
60.0		-32 5					
		SP	S 12	67	7 8 8 (16)	4.5	
62.5							

▲ SPT N VALUE ▲ 10 20 30 40
PL MC LL LI 10 20 30 40
Shear Strength (tsf)
△ UU 1 □ CU 2 3 ◇ QU 4

(Continued Next Page)

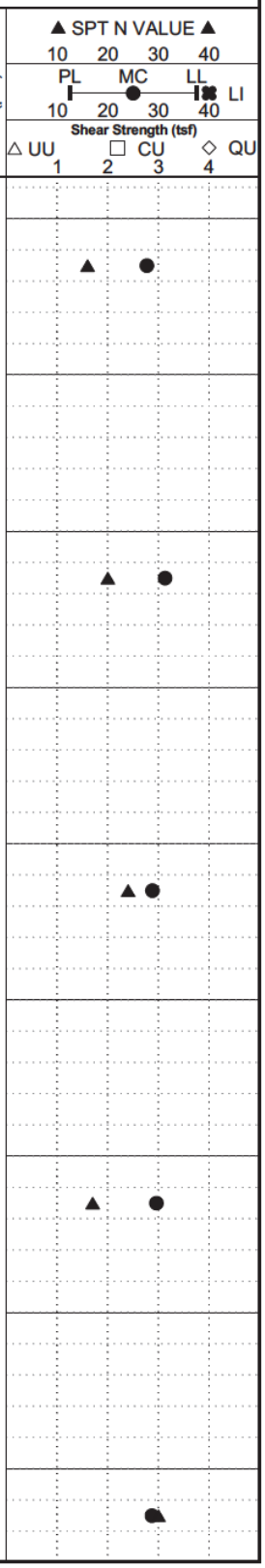
HAYWARD BAKER Geotechnical Construction
KELLER

Hayward Baker

BORING NUMBER HB-B-109

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)
65.0		**SP** *(continued)* -37 5					
		SP SM	S 13	83	7 9 7 (16)	6.6	
67.5							
70.0							
			S 14	67	6 9 11 (20)	5.8	
72.5							
75.0							
			S 15	72	9 10 14 (24)	7.2	
77.5		-51 0					
		SP					
80.0							
			S 16	67	8 8 9 (17)	4.5	
82.5							
85.0		-57 5					
		SP SM	S 17	67	10 15 15 (30)	5.8	



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HAYWARD BAKER - LOG POR OF VANCOUVER ESORO PE ROLEUM GPJ A C GD 1/13/15

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ 10 20 30 40 / PL MC LL LI 10 20 30 40 / Shear Strength (tsf) △ UU 1 □ CU 2 3 ◇ QU 4
		SP SM *(continued)*						
87.5								
90.0								
		-64 0	S 18	72	5 11 13 (24)	5.3		▲ ●

HAYWARD BAKER Geotechnical Construction
KELLER

Hayward Baker

BORING NUMBER HB-B-110

PROJECT NAME Tesoro Savage Petro eum Term na
PROJECT LOCATION Port of Vancouver, WA
DATE STARTED 11/26/14 **COMPLETED** 11/26/14
NORTH: 121001.1169 **EAST:** 1072229.35
METHOD Mud Rotary, CME 75, Gregory Dr ng
GROUND ELEVATION 27.29 ft
NOTES Automat c hammer eff c ency: 80%
AT TIME OF DRILLING N/A
AT END OF DRILLING N/A

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ 10 20 30 40 / PL MC LL LI 10 20 30 40 / Shear Strength (tsf) △ UU 1 □ CU 2 3 ◇ QU 4
0.0		SP SM or GP, 2" SM, 5" GP f ne to med um grave *(33 b ow count samp er ho d at 15" for ~10 counts overstated)						
2.5								
5.0								
			S 1	39	8 14 33 (47)	7.7		● ▲
7.5								
10.0		17 3						
		SP	S 2	100	8 12 14 (26)	3.1		● ▲
12.5								
		13 8						
		SP SM						
15.0								
			S 3	72	5 5 15 (20)	8.6		▲ ●
17.5								
20.0								

(Continued Next Page)

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)
20.0		SP SM *(continued)*	S 4	89	15 16 17 (33)	8.7	
22.5		4 3 SP					
25.0			S 5	72	4 4 4 (8)	4	
27.5		-0 7 SP SM					
30.0			S 6	67	5 5 5 (10)	5.1	
32.5							
35.0		-7 7 SM, a ternat ng bands of SM and SP samp e conta ned wood chunks	S 7	67	2 1 3 (4)	40	
37.5							
40.0		-12 7 SP	S 8	33	4 5 4 (9)	4.8	

▲ SPT N VALUE ▲ 10 20 30 40
PL MC LL LI 10 20 30 40
Shear Strength (tsf) △ UU 1 □ CU 2 3 ◇ QU 4

(Continued Next Page)

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ 10 20 30 40 / PL MC LL LI 10 20 30 40 / Shear Strength (tsf) △ UU □ CU ◇ QU 1 2 3 4
42.5		SP *(continued)*						
45.0			S 9	67	3 6 6 (12)	4.9		▲ ●
47.5								
50.0		-22 7						
		SP SM, w th f ne grave	S 10	67	7 7 9 (16)	5.8		▲ ●
52.5								
55.0			S 11		5 7 9 (16)	7.6		▲ ●
57.5								
60.0		-32 7						
		SP	S 12	67	7 10 10 (20)	4.6		▲ ●
62.5								

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HAYWARD BAKER
Geotechnical Construction
KELLER

Hayward Baker

BORING NUMBER HB-B-110

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ 10 20 30 40 / PL MC LL LI 10 20 30 40 / Shear Strength (tsf) △ UU □ CU ◇ QU 1 2 3 4
65.0		SP *(continued)* -37 7						
		SP SM	S 13	72	10 15 12 (27)	5.5		
67.5								
70.0								
			S 14	83	10 12 12 (24)	5.5		
72.5								
75.0								
			S 15	83	11 10 12 (22)	6		
77.5								
80.0								
			S 16	72	14 17 19 (36)	5.9		
82.5								
85.0		-57 7						
		SM, w th f ne to med um grave s	S 17	67	40 44 50 (94)			>>▲

(Continued Next Page)

HAYWARD BAKER - LOG POR OF VANCOUVER ESORO PE ROLEUM GPJ A C GD 1/13/15

PROJECT NAME Tesoro Savage Petro eum Term na **PROJECT LOCATION** Port of Vancouver, WA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	Percent Fines	DRY UNIT WT. (pcf)	▲ SPT N VALUE ▲ 10 20 30 40 / PL MC LL LI 10 20 30 40 / Shear Strength (tsf) △ UU 1 □ CU 2 3 ◇ QU 4
87.5		SM, w th f ne to med um grave s *(continued)*						
90.0		-62 7						
		SP, w th f ne to med um grave	S 18	67	26 45 50 (95)			>>▲
		-64 2						

Appendix D

Summary of Laboratory Testing

DRAFT

Appendix D
Lab results

Lab Results Summary

PO Box 44840
Tacoma, WA 98448
253-537-9400
253-537-9401 fax

E3RA

January 27, 2015

Hayward Baker, Inc.
11180 E. Marginal Way South
Tukwila, WA 98168

Attention: Mr. Robert Carnevale

Subject: **Lab Testing Results Summary**

Dear Mr. Carnevale:

E3RA, Inc. (E3RA) is pleased to submit the attached summary of laboratory testing results that includes 18 Atterberg Limits, 1 moisture test, and 122 200-wash/moisture tests combined.

Sincerely,

E3RA, Inc.

Boring-Sample #	Sample Depth	Moisture Content (%)	Liquid Limit	Plasticity Index	Soil Description
HB-B-101 S-2	10'-11.5'	18.2%	34	9	ML
HB-B-101 T-1	22.5'	39.4%	54	19	MH
HB-B-101	0	53.6%	51	11	MH
HB-B-102	0	27.1%	28	4	ML
HB-B-103 S-2	10'-11.5'	18.1%	28	4	ML
HB-B-103 S-4	25'-26.5'	41.8%	24		Non Plastic
HB-B-103 S-5	30'-31.5'	72.2%	48	8	ML
HB-B-103 S-6	35'-36.5'	28.4%	27	2	ML
HB-B-103	0	40.7%	39	10	ML
HB-B-104	0	49.3%	52	22	MH
HB-B-105	0	33.5%	49	21	ML
HB-B-106 S-4	0	39.4%	50	19	ML/MH
HB-B-106	0	41.7%	40	13	ML/MH
HB-B-107	0	44.9%	57	26	MH
HB-B-108 S-4	19.5'-21'	45.9%	52	15	MH
HB-B-108 S-5	25'-26.5'	34.1%	33	10	ML
HB-B-108	0	34.1%	40	11	ML/MH
HB-B-109 S-5	25'-26.5'	38.0%	30	4	ML

Boring-Sample #	Sample Depth	Moisture Content (%)	% Fines
HB-B-101 S-1	5'-6.5'	11.7	8.9
HB-B-101 S-2	10'-11.5'	18.2	9.6
HB-B-101 S-3	15'-16.5'	22.8	32.6
HB-B-101 S-5	25'-26.5'	48.7	49.3
HB-B-101 S-6	30'-31.5'	45.9	19.2
HB-B-101 S-7	35'-36.5'	30.4	7.4
HB-B-101 S-9	45'-46.5'	20.7	11.9
HB-B-101 S-10	50'-51.5'	25.8	7.8
HB-B-101 S-11	55'-56.5'	55.9	16.8
HB-B-101 S-12	60'-61.5'	12.9	6.1
HB-B-101 S-13	65'-66.5'	14.8	3
HB-B-102 S-1	5'-6.5'	20.9	40.5
HB-B-102 S-2	10'-11.5'	18.5	43.8
HB-B-102 S-3	15'-16.5'	15.4	19.9
HB-B-102 S-4	20'-21.5'	32.5	74.9
HB-B-102 S-5	25'-26.5'	31.1	25
HB-B-102 S-6	30'-31.5'	29.1	9.6
HB-B-102 S-7	35'-36.5'	28.2	8.8
HB-B-102 S-8	40'-41.5'	25.7	6
HB-B-102 S-9	45'-46.5'	25.8	7.9
HB-B-102 S-10	50'-51.5'	26.7	11.7
HB-B-102 S-11	55'-56.5'	9.9	3.4
HB-B-102 S-12	60'-61.5'	9.7	1.5
HB-B-102 S-13	65'-66.5'	11.2	3.7
HB-B-103 S-2	10'-11.5'	18.1	--
HB-B-103 S-3	15'-16.5'	24.6	19
HB-B-103 S-4	25'-26.5'	41.8	44.1
HB-B-103 S-5	30'-31.5'	72.2	93.1
HB-B-103 S-6	35'-36.5'	28.4	27.1
HB-B-103 S-7	40'-41.5'	27.2	10.2
HB-B-103 S-8	45'-46.5'	26.1	8.1
HB-B-103 S-10	55'-56.5'	24.7	7.4

Boring-Sample #	Sample Depth	Moisture Content (%)	% Fines
HB-B-103 S-11	60'-61.5'	22.8	6.3
HB-B-103 S-12	65'-66.5'	16	2.9
HB-B-104 S-1	5'-6.5'	17.2	9.7
HB-B-104 S-2	10'-11.5'	15.5	10.4
HB-B-104 S-6	35'-36.5'	34.6	16.4
HB-B-104 S-7	40'-41.5'	28.4	6.1
HB-B-104 S-8	45'-46.5'	29.5	6.7
HB-B-104 S-9	50'-51.5'	25	5.8
HB-B-104 S-10	55'-56.5'	28.6	5.5
HB-B-104 S-11	60'-61.5'	11.2	4.6
HB-B-104 S-12	65'-66.5'	12.2	5.6
HB-B-105 S-1	5'-6.5'	18.2	28.5
HB-B-105 S-2	10'-11.5'	18.7	30.7
HB-B-105 S-3	15'-16.5'	37.9	76.5
HB-B-105 S-4	20'-21.5'	38.6	57.3
HB-B-105 S-5	25'-26.5'	34.1	9.5
HB-B-105 S-6	30'-31.5'	29.9	6.9
HB-B-105 S-7	35'-36.5'	29.3	6.7
HB-B-105 S-8	40'-41.5'	27	7.3
HB-B-105 S-9	45'-46.5'	23.9	6.5
HB-B-105 S-10	50'-51.5'	28.6	7.1
HB-B-105 S-11	55'-56.5'	23	8.4
HB-B-105 S-12	60'-61.5'	10.9	4.9
HB-B-105 S-13	65'-66.5'	18.2	6.5
HB-B-106 S-1	5'-6.5'	23.1	12.1
HB-B-106 S-3	15'-16.5'	32.4	18.8
HB-B-106 S-4	20'-21.5'	43.4	83.1
HB-B-106 S-8	40'-41.5'	27.9	5.1
HB-B-106 S-9	45'-46.5'	22.2	9.1
HB-B-106 S-10	50'-51.5'	26.9	9.3
HB-B-106 S-11	55'-56.5'	25.2	7.4
HB-B-106 S-12	60'-61.5'	10.8	4.1
HB-B-107 S-1	5'-6.5'	23.8	5
HB-B-107 S-2	10'-11.5'	17.8	21.4
HB-B-107 S-3	15'-16.5'	18.6	14.7
HB-B-107 S-5	26'-27.5'	21.4	34.9
HB-B-107 S-6	30'-31.5'	32.3	11.1
HB-B-107 S-7	35'-36.5'	21.5	7.9
HB-B-107 S-8	40'-41.5'	27.2	6.3
HB-B-107 S-9	45'-46.5'	26.4	6.8
HB-B-107 S-10	50'-51.5'	22	6.9
HB-B-107 S-11	55'-56.5'	20.7	6.3
HB-B-107 S-13	65'-66.5'	11.3	4.8
HB-B-108 S-1	5'-6.5'	5.7	6.6
HB-B-108 S-2	10'-11.5'	21.2	8.6
HB-B-108 S-3	15'-16.5'	19.9	17.6
HB-B-108 S-4	19.5'-21'	43.5	60
HB-B-108 S-5	25'-26.5'	34	55.1
HB-B-108 S-6	30'-31.5'	25.5	15.4
HB-B-108 S-7	35'-36.5'	22.5	8.1
HB-B-108 S-8	40'-41.5'	26.2	4.2
HB-B-108 S-9	45'-46.5'	25.2	6.1
HB-B-108 S-10	50'-51.5'	24.1	7.4

Boring-Sample #	Sample Depth	Moisture Content (%)	% Fines
HB-B-108 S-11	55'-56.5'	11.3	6.3
HB-B-108 S-12	60'-61.5'	10.2	3.6
HB-B-108 S-13	65'-66.5'	9	4.5
HB-B-109 S-1	5'-6.5'	6.1	37.5
HB-B-109 S-2	10'-11.5'	15.9	7.9
HB-B-109 S-4	20'-21.5'	25.3	4.6
HB-B-109 S-5	25'-26.5'	38	63.8
HB-B-109 S-6	30'-31.5'	31.6	4.9
HB-B-109 S-7	35'-36.5'	25.9	6.1
HB-B-109 S-8	40'-41.5'	29.6	7
HB-B-109 S-9	45'-46.5'	29.5	7.3
HB-B-109 S-10	50'-51.5'	22.3	3.6
HB-B-109 S-11	55'-56.5'	30.5	13.7
HB-B-109 S-12	60'-61.5'	28.5	4.5
HB-B-109 S-13	65'-66.5'	27.7	6.6
HB-B-109 S-14	70'-71.5'	31.3	5.8
HB-B-109 S-15	75'-76.5'	28.8	7.2
HB-B-109 S-16	80'-81.5'	29.6	4.5
HB-B-109 S-17	85'-86.5'	28.7	5.8
HB-B-109 S-18	90'-91.5'	32.3	5.3
HB-B-110 S-1	5'-6.5'	11.5	7.7
HB-B-110 S-2	10'-11.5'	6.1	3.1
HB-B-110 S-3	15'-16.5'	29	8.6
HB-B-110 S-4	20'-21.5'	25.7	8.7
HB-B-110 S-5	25'-26.5'	30.7	4
HB-B-110 S-6	30'-31.5'	32.4	5.1
HB-B-110 S-7	35'-36.5'	42.9	40
HB-B-110 S-8	40'-41.5'	24.4	4.8
HB-B-110 S-9	45'-46.5'	29.2	4.9
HB-B-110 S-10	50'-51.5'	24.9	5.8
HB-B-110 S-11	55'-56.5'	27.2	7.6
HB-B-110 S-12	60'-61.5'	29.2	4.6
HB-B-110 S-13	65'-66.5'	28.9	5.3
HB-B-110 S-14	70'-71.5'	32.7	5.5
HB-B-110 S-15	75'-76.5'	29.2	6
HB-B-110 S-16	80'-81.5'	26.2	5.9
HB-B-110 S-17	85'-86.5'	13.5	8.2
HB-B-110 S-18	90'-91.5'	15.7	3.1

Lab Testing

MEMORANDUM

DATE: January 27, 2015

TO: Mark Rohrbach, PE; Hayward Baker
Jim Brigham, PE; E3RA

FROM: Jeff Wagner, PE; Brice Exley, PE; and Paul Kastens, LG, LEG; Hart Crowser

RE: **Geotechnical Laboratory Test Results**
E3RA and Hayward Baker, Inc.
Report Addendum No. 1
19109-00

This memo is an addendum to our Geotechnical Laboratory Test Results report dated January 20, 2015. In our original report, we described how sample quality was estimated. This memo contains a few additional comments about the calculation of vertical effective stress that should be included in that discussion. The vertical effective stress was estimated, for the purposes of our sample quality calculations, using assumed soil properties. The use of these particular assumptions and estimates is reasonable and adequate for the purposes of assigning a sample quality designation. However, these assumptions and estimates should not be used or relied upon by anyone as a basis for design recommendations.

Closing

This memorandum has been prepared for the exclusive use of E3RA, Hayward Baker, and their design consultants for specific application to the subject project and site. This work has been performed in accordance with generally accepted geotechnical engineering practices in the same or similar localities, related to the nature of the work accomplished at the time the services are performed. No other warranty, express or implied, is made.

We trust that this addendum memo meets your current needs. Please do not hesitate to contact us if you have any questions or comments on the contents of this memo.

\\seattlefs\projects\Notebooks\1910900_Hayward Baker Laboratory Testing\Deliverables\Memos\Final Report Addendum Memo 001.docx

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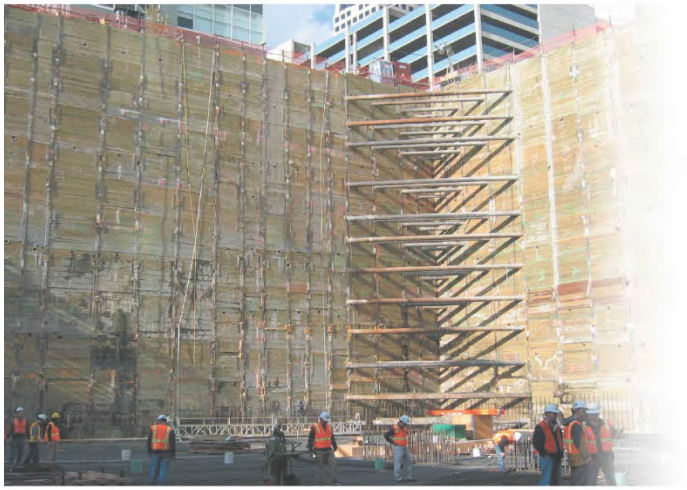
Tel 206.324.9530



Geotechnical Laboratory Test Results

E3RA and Hayward Baker, Inc.

Prepared for

E3RA and
Hayward Baker, Inc.

January 20, 2015
19109-00

HARTCROWSER

EX-0001-006919-PCE

Geotechnical Laboratory Test Results

E3RA and Hayward Baker, Inc.

Prepared for
E3RA and
Hayward Baker, Inc.

January 20, 2015
19109-00

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Geotechnical Laboratory Testing Results

E3RA and Hayward Baker, Inc.

January 2015

This report provides the results of constant rate of strain consolidation (CRS), Atterberg limits, specific gravity, moisture content, torvane, and pocket penetrometer tests that we conducted for E3RA and Hayward Baker, Inc. (HB). The report includes our scope of services, testing methods, and test results.

Our scope of work was to perform approximately eight CRS tests and related geotechnical laboratory tests as directed by HB on a unit cost basis. Our scope included providing a report of the test results to E3RA and HB upon completion of all testing.

TEST SAMPLES

The test samples included eight 3-inch-diameter, labeled Shelby tubes, which are described in Table 1. Six of the Shelby tubes were delivered to our lab in a 5-gallon bucket by Joe Kinerk of E3RA on November 24, 2014. Two more Shelby tubes were delivered on December 1, 2014, by E3RA personnel.

All eight tubes were received capped and taped without wax seals. We make every effort to preserve and maintain sample quality within the lab; however, sample collection, delivery, and preservation methods greatly affect the quality of soil samples which in turn can affect the test results.

Table 1 – Sample Identification

Boring ID	Sample ID	Sample Depth (feet)	Date Received	HC Sample Comments
HB-B-101	T-1	22-24	11/24/14	
HB-B-102	T-1	22-24	11/24/14	
HB-B-103	T-1	22.5-24.5	11/24/14	small diameter, vertical channels observed
HB-B-104	T-1	20-22	11/24/14	low recovery, disturbed tube with voids
HB-B-105	T-1	17-19	11/24/14	
HB-B-106	T-1	21.5-23.5	12/1/14	
HB-B-107	T-1	24-26	11/24/14	
HB-B-108	T-1	21-23	12/1/14	

METHODS AND STANDARDS

We extracted CRS test specimens from their original sample tubes by cutting a segment of the tube, delaminating the soil from the tube with a thin wire and hypodermic tube, extruding the soil sample from the tube shell, and trimming the extruded sample to fit the consolidation ring.

Water Content Determination

Water content was determined in general accordance with ASTM D2216 after cutting the tubes for CRS specimen extraction. Water content was not determined if only very small samples were available or if samples contained large gravel that would result in unrepresentative values. Water content is routinely determined for samples subjected to other testing.

Atterberg Limits

We determined Atterberg limits (AL) for selected fine-grained soil samples. The liquid limit and plastic limit of soil samples were determined in general accordance with ASTM D4318-84. This test relates the plasticity index (liquid limit minus the plastic limit) to the liquid limit. The results of the AL tests and the plasticity characteristics are summarized in the Liquid and Plastic Limits Test Reports on Figures 1 and 2.

Pocket Penetrometer and Torvane

The pocket penetrometer (PP) and torvane (TV) procedures provide the approximate undrained shear strength of a cohesive soil sample.

The PP device consists of a calibrated spring mechanism that measures penetration resistance of a 1/4-inch-diameter steel tip over a given distance. The penetration resistance is correlated to the unconfined compressive strength of the soil, which is typically twice the undrained shear strength of a saturated, cohesive soil.

The TV device consists of a 1-inch-diameter plate with eight equally spaced and radially arranged 1/4-inch vanes. The vanes are pressed into the soil and the device is rotated. The vanes force a shear failure over the area of the plate. The resistance at failure, as measured by a calibrated spring, correlates to the approximate undrained shear strength of the soil sample. Table 5 shows the results of the pocket penetrometer and torvane tests.

Grain Size Analysis

Grain size (mechanical sieve) analysis was performed in general accordance with ASTM D6913-04. The results of the mechanical sieve analysis are shown on a particle size distribution graph on Figure 3. This test was not initially requested for the CRS consolidation test samples, but was performed by request on a sample taken from HB-B-104/T-1.

Constant Rate of Strain Consolidation Test

The one-dimensional constant rate of strain (CRS) consolidation test provides data for estimating settlement. Our CRS testing was done in general accordance with ASTM D4186. After the sample was extracted, a relatively undisturbed, fine-grained sample was carefully trimmed and fit into a rigid steel ring with porous stones placed on the top and bottom of the sample to allow drainage. Vertical strain was then applied continuously to the sample in a way that allowed the sample to partially consolidate

under the given strain rate. Measurements were made of the compression of the sample over time, the total load placed on the sample, and the excess pore pressure at the base of the sample throughout the test. Rebound was measured during the unloading phase. In general, an excess pore pressure ratio of 3 percent is targeted during loading, with an allowance of up to 15 percent without significant concerns about strain rate effects. The test results are plotted in terms of axial strain and coefficient of consolidation versus applied load (stress). The specific gravity of the samples was determined by ASTM D854.

Sample quality was estimated using methods by both Lunne et al. (2006) and Terzaghi et al. (1996) as described in Tables 2 and 3. As needed for sample quality designation (SQD), the preconsolidation stress was estimated using the Casagrande and strain energy methods. *In situ* vertical effective stress was estimated based on the provided soil profile information.

Though we make every effort to refine our estimates, the user of this data should apply their own interpretation and engineering judgment to the consolidation test results. Our interpretations are intended solely for the purpose of SQD. SQD approaches are most applicable to low to medium plasticity clays with overconsolidation ratios (OCR) ranging between about 1 and 4.

Table 2 - Sample Quality by Lunne et al. (2006)

OCR	$\Delta e/e_0$ at σ'_{v0}			
1 to 2	< 0.04	0.04 – 0.07	0.07 – 0.14	> 0.14
2 to 4	< 0.03	0.03 – 0.05	0.05 – 0.10	> 0.10
Quality	1	2	3	4

Notes:

a. 1 = very good to excellent, 2 = fair to good, 3 = poor, 4 = very poor

Table 3 - Sample Quality by Terzaghi et al. (1996)

εv at σ'v0	<1	1 – 2	2 – 4	4 - 8	< 8
Quality	A	B	C	D	E

Notes:

a. A (best) to E (worst)

RESULTS

During the CRS sample extraction procedures, two tube samples had unusual conditions that required consultation with HB before lab testing. HB-B-103 exhibited at least two small voids in the soil and HB-B-104 had significant visible disturbance (Figure 4).

Table 4 identifies the tests that we performed. Table 5 provides a summary of all test results that are not provided in the figures at the end of this report. Figures 5 through 39 provide the results for CRS consolidation tests and any related index testing, along with photographs of selected CRS samples before testing

Table 4 - Summary of Tests Performed

Test	HB-B-101/T-1	HB-B-102/T-1	HB-B-103/T-1	HB-B-104/T-1	HB-B-105/T-1	HB-B-106/T-1	HB-B-107/T-1	HB-B-108/T-1
CRS	1	1	1	N/A	1	1	1	1
AL	1	1	1	1	1	1	1	1
PP	1	1	1	N/A	1	1	1	1
TV	1	1	1	N/A	1	1	1	1
Sieve	N/A	N/A	N/A	1	N/A	N/A	N/A	N/A

Notes:

Because of disturbed sample conditions in tube HB-B-104/T-1, CRS consolidation testing was cancelled on this sample. Atterberg limits and mechanical sieve analysis tests were conducted on the sample at the request of an HB representative.

Table 5 - Summary of Index Tests

Boring/Sample	Depth (feet)	PP (TSF)	TV (TSF)	Specific Gravity
HB-B-101/T-1	22.5	1.3	2.0	2.584
HB-B-102/T-1	22.9	1.8	4.4	2.695
HB-B-103/T-1	23.9	1.0	3.5	2.649
HB-B-104/T-1	22.5	N/A	N/A	N/A
HB-B-105/T-1	18.0	2.0	3.5	2.612
HB-B-106/T-1	22.3	1.0	3.0	2.666
HB-B-107/T-1	25.1	1.3	3.8	2.570
HB-B-108/T-1	21.7	2.0	6.5	2.585

Notes:

Because of disturbed sample conditions in tube HB-B-104/T-1, CRS consolidation testing was cancelled on this sample.

USE OF THIS REPORT

This report has been prepared for the exclusive use of Hayward Baker, Inc., and their subconsultants for specific application to the subject project and site. These tests have been performed in general accordance with the standards identified throughout this report. No other warranty, express or implied, is made.

Notation Index

c_v = vertical coefficient of consolidation (ft^2/day)

e = void ratio (-)

e_0 = initial void ratio (-)

K_c = consolidation stress ratio = σ'_h/σ'_v

LL = liquid limit (%)
LI = liquidity index = (w – PL)/PI
M = constrained (oedometer) modulus = $\Delta\sigma'_v/\Delta\varepsilon_v$ (ksf)
PI = plasticity index (%)
PL = plastic limit (%)
t = time (day, hr, min, s)
w = water content (%)
w_n = natural water content (%)

Δ = change
ΔL = horizontal displacement (in)
ΔH = vertical displacement (change in specimen height) (in)
Δu = equivalent DSS shear induced pore pressure (psf)
ε_v = vertical strain (%)
ε_{vc} = vertical consolidation strain (%)
ε_{vf} = final vertical consolidation strain (%)
ε_{vol} = volumetric strain (%)
γ_t = total unit weight (pcf)
γ_d = dry unit weight (pcf)
% = percentage
σ'_p = preconsolidation stress (psf)
σ'_v = vertical effective stress (psf)

cm = centimeter
ft = feet
in = inches
ksf = kips per square foot
min = minute
mm = millimeter
pcf = pounds per cubic foot
psf = pounds per square foot
s = seconds

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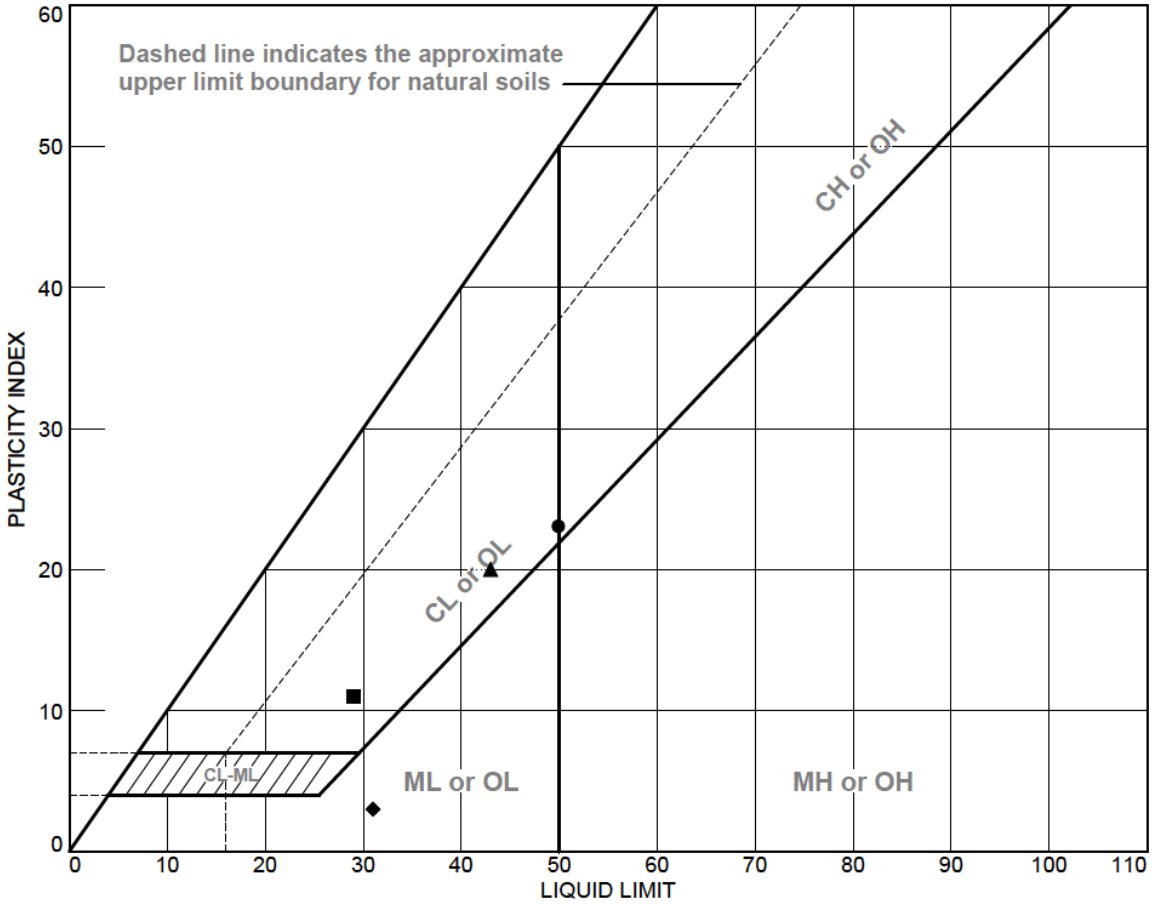
LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA

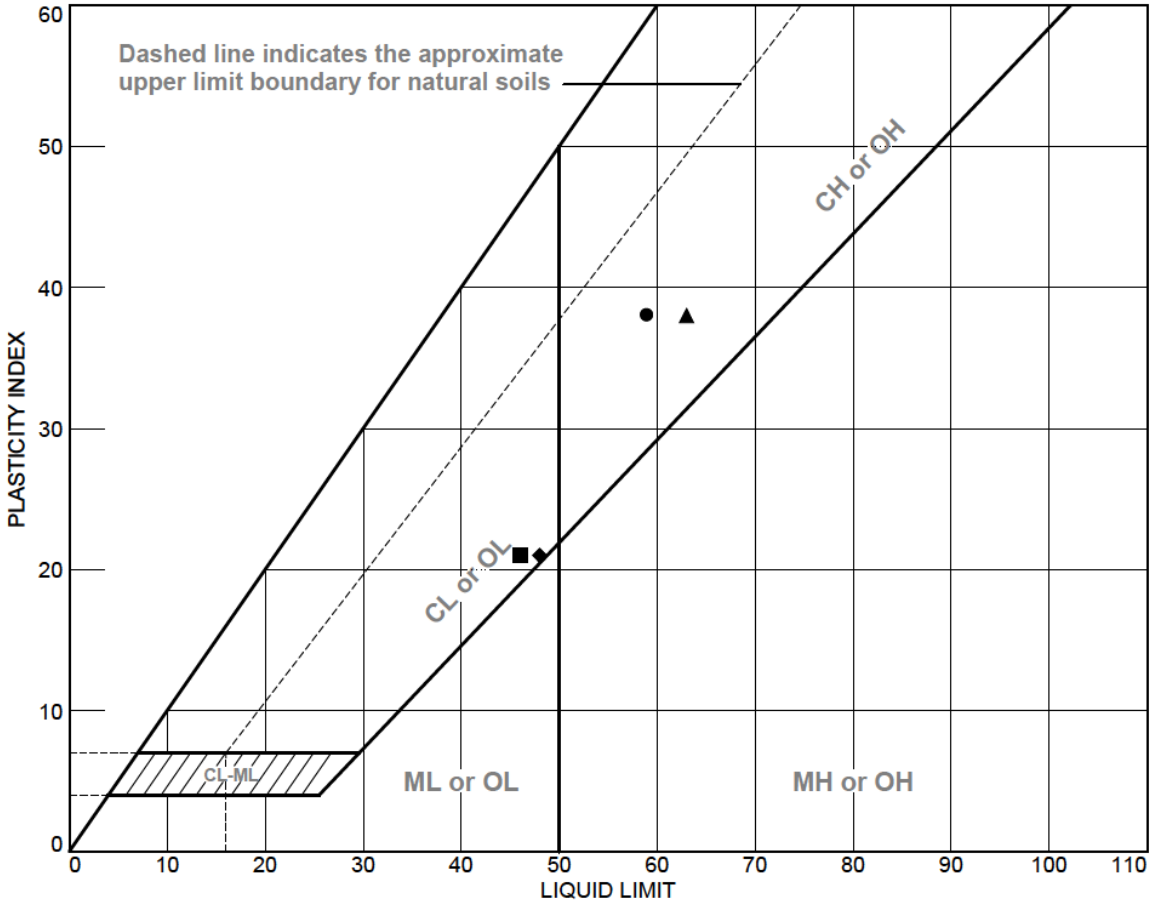
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	HB-B-101	T-1	22.5 ft	52.4	27	50	23	CH
■	HB-B-102	T-1	22.9 ft	27.5	18	29	11	CL
▲	HB-B-103	T-1	23.9 ft	40.5	23	43	20	CL
◆	HB-B-104	T-1	22.5 ft	29.2	28	31	3	ML

HARTCROWSER

Client: Hayward Baker, Inc.
Project: Port of Vancouver

Project No.: 19109-00 **Figure** 1

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA

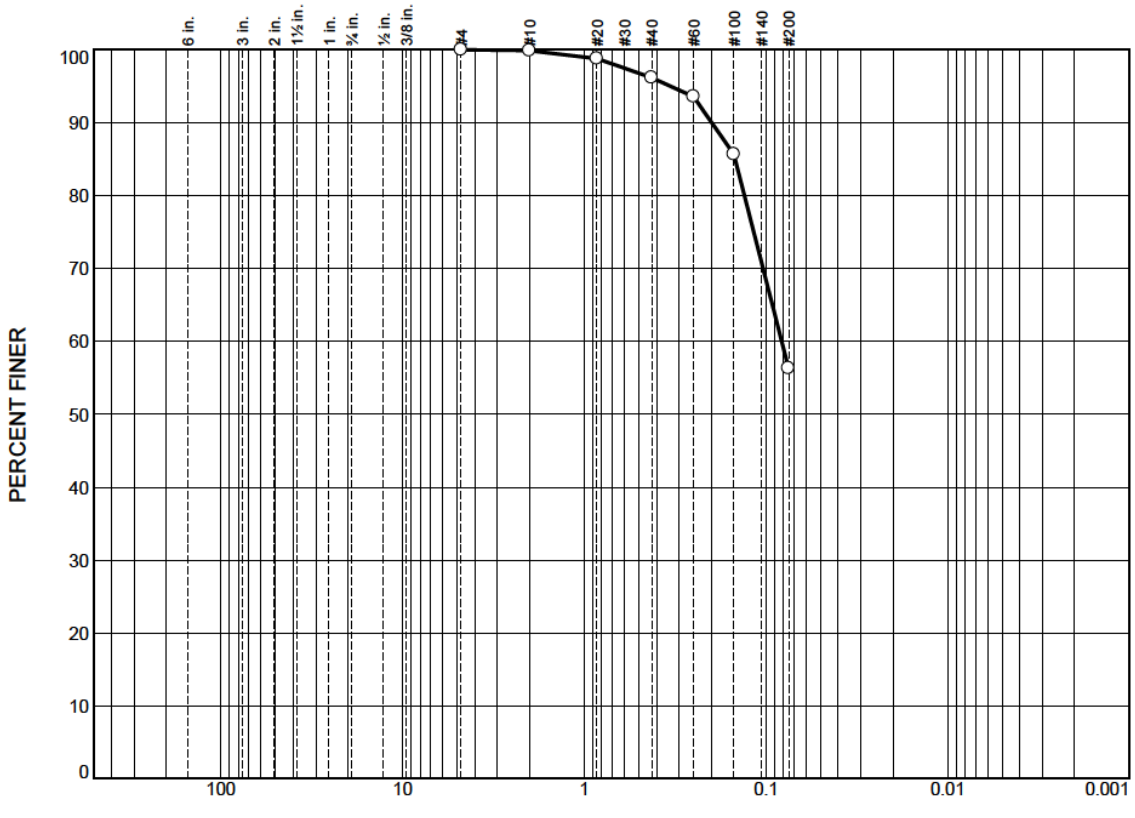
SYMBOL	SOURCE	SAMPLE NO.	DEPTH	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	HB-B-105	T-1	18.0 ft	31.3	21	59	38	CH
■	HB-B-106	T-1	22.3 ft	40.7	25	46	21	CL
▲	HB-B-107	T-1	25.1 ft	31.6	25	63	38	CH
◆	HB-B-108	T-1	21.7 ft	34.4	27	48	21	CL

HARTCROWSER

Client: Hayward Baker, Inc.
Project: Port of Vancouver

Project No.: 19109-00 **Figure** 2

Particle Size Distribution Report



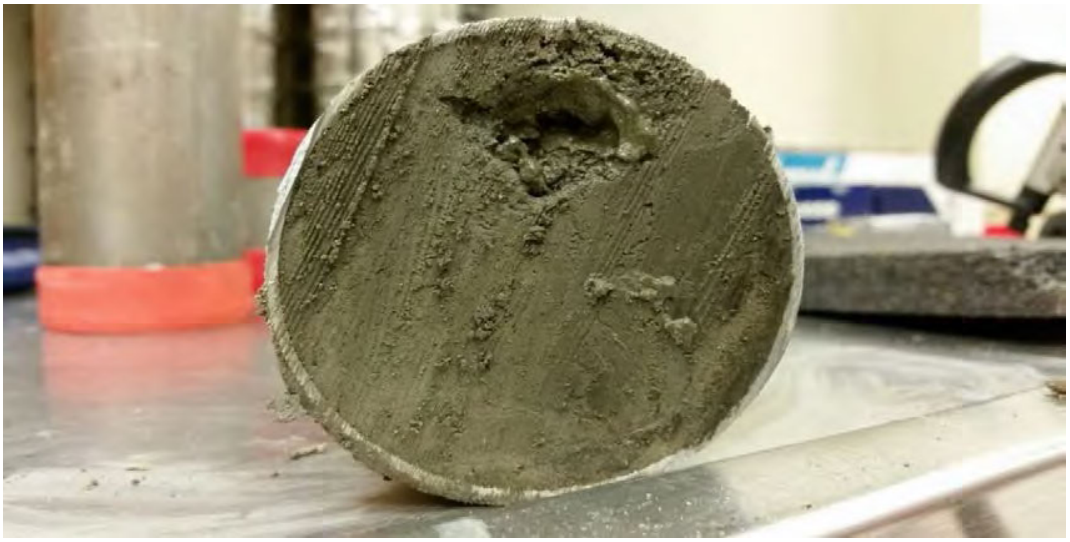
	% +3"	% Gravel	% Sand	% Silt	% Clay
○	0.0	0.0	43.7	56.3	

SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	HB-B-104	T-1	22.5 ft	very sandy SILT	ML

HARTCROWSER

Client: Hayward Baker, Inc.
Project: Port of Vancouver
Project No.: 19109-00 **Figure** 3



Upper tube cross-section photo showing void/disturbed conditions.

Lower tube cross-section photo showing void/disturbed conditions.

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Hayward Baker	
HB-B-104 Tube Photos	
19109-00	12/22/2014
HARTCROWSER	Figure **4**

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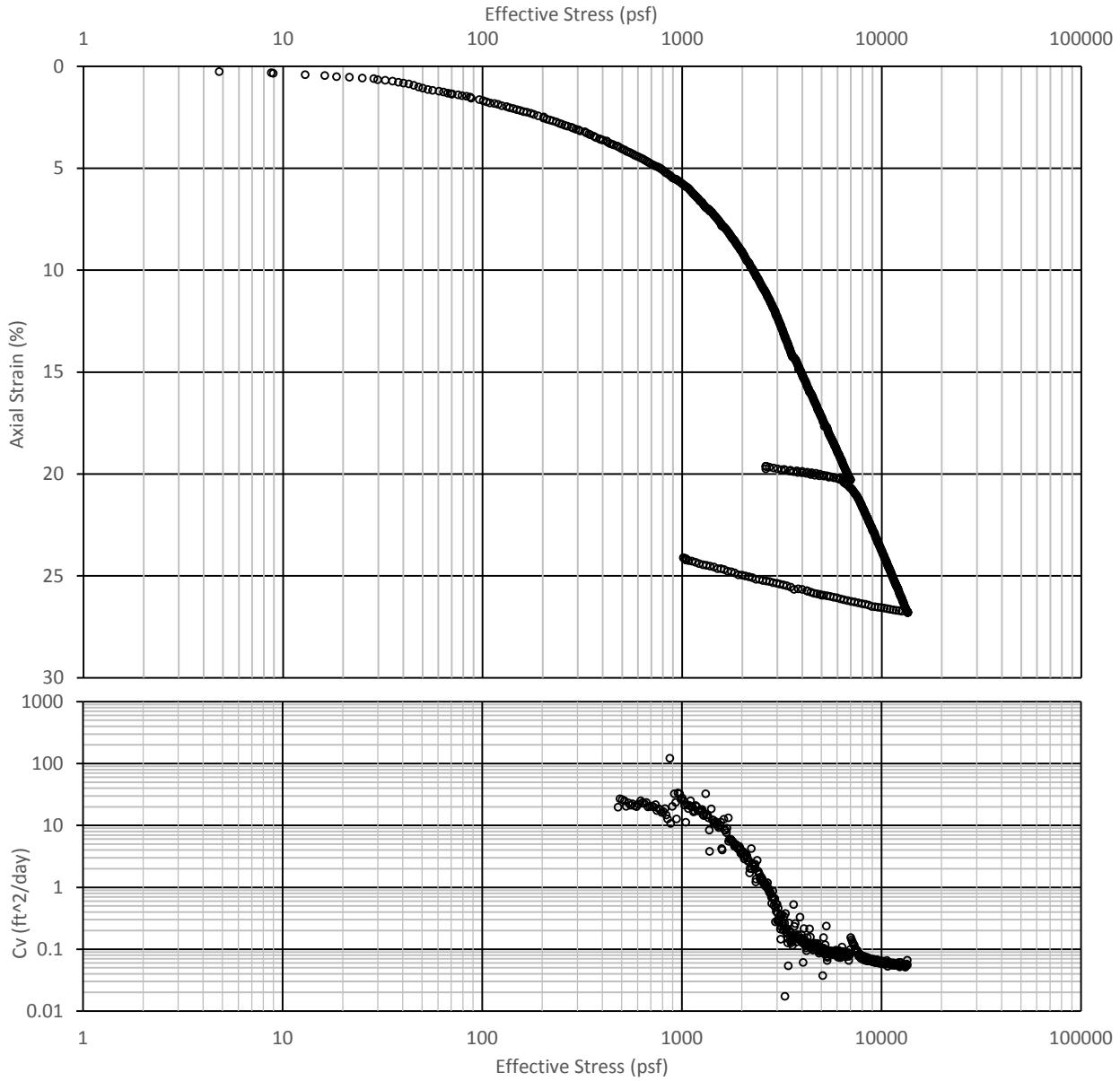

Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
22.5 feet	54	38	50	27	23	CLAY	CH

σ'_{v_o}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1300	1900	2200	1700
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
D		Poor	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	4.65
Total Unit Weight (pcf)	102.35
Degree of Saturation (%)	97.70
Void Ratio (e0)	1.423

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

Hayward Baker

Axial strain and coefficient of consolidation versus logarithm of vertical effective stress for HB-101 T-1 CRS

19109-00 12/1/2014

HARTCROWSER

Figure

5A

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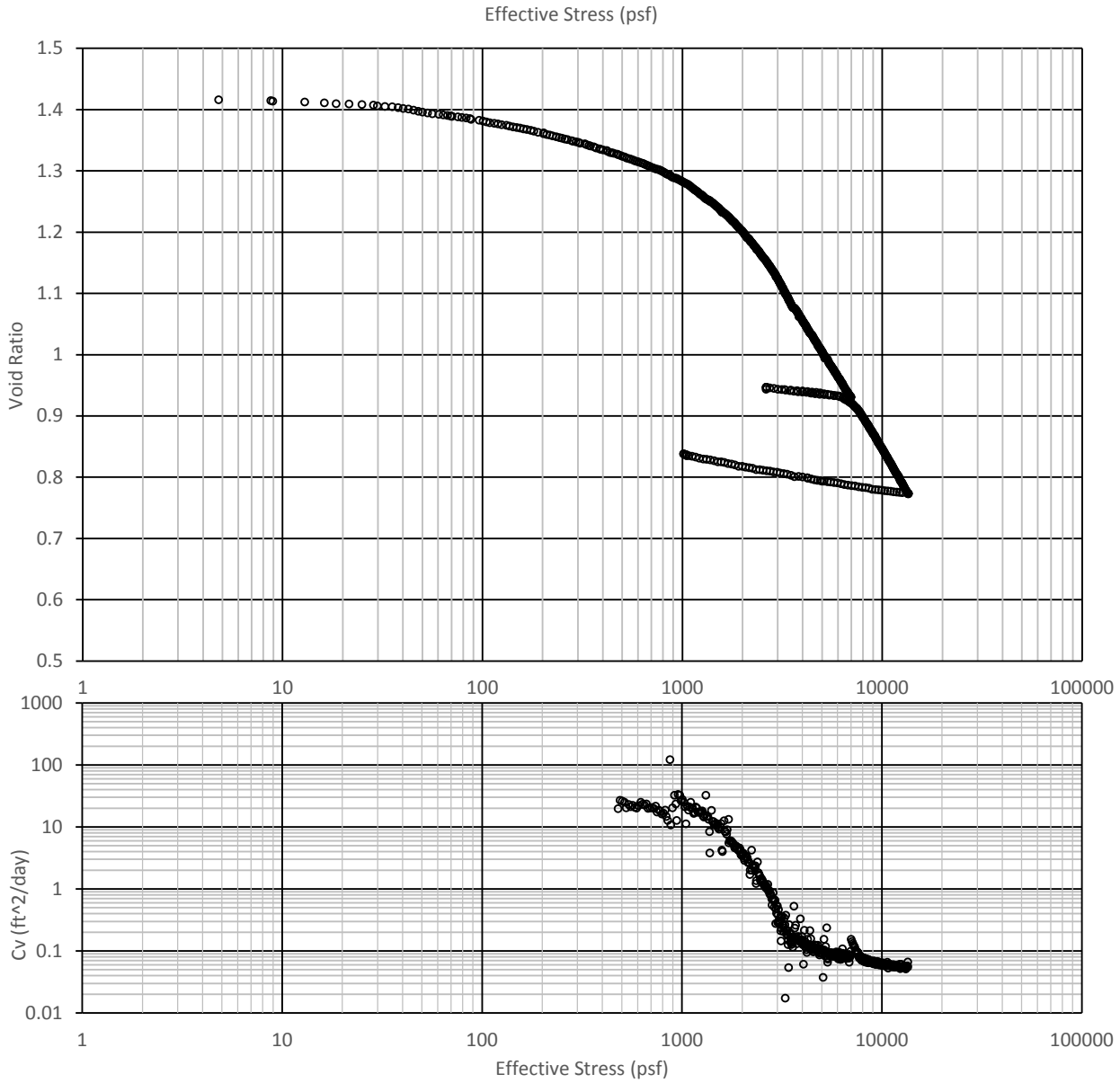

Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
22.5 feet	54	38	50	27	23	CLAY	CH

σ'_{v_o}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1300	1900	2200	1700
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
D		Poor	

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Total Unit Weight (pcf)	102.35
Degree of Saturation (%)	97.70
Void Ratio (e0)	1.423

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

Hayward Baker

Void ratio and coefficient of consolidation versus logarithm of vertical effective stress for HB-101 T-1 CRS

19109-00 12/1/2014

HARTCROWSER

Figure **5B**

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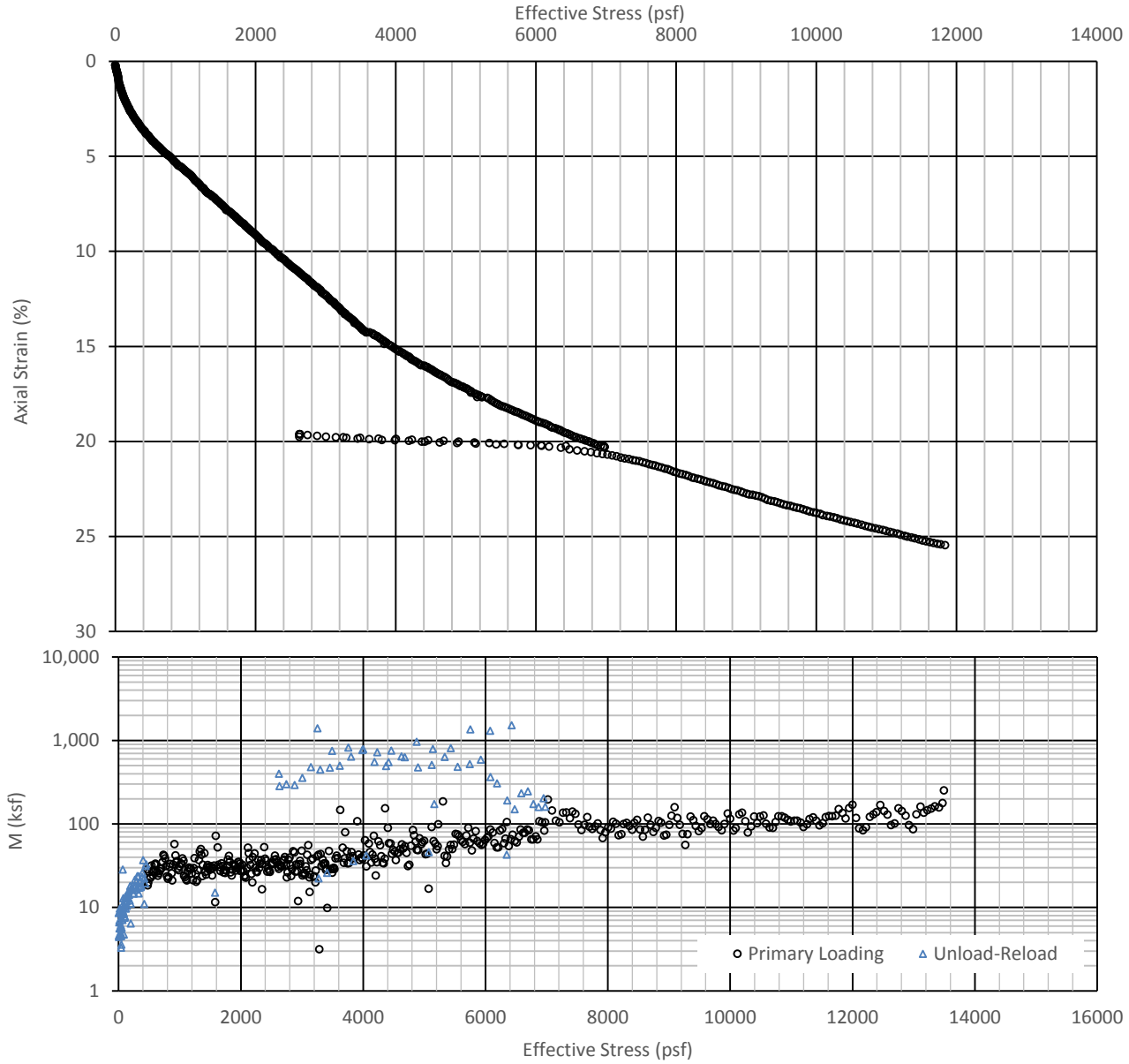

Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
22.5 feet	54	38	50	27	23	CLAY	CH

σ'_{v_o}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1300	1900	2200	1700
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
D		Poor	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	4.65
Total Unit Weight (pcf)	102.35
Degree of Saturation (%)	97.70
Void Ratio (e0)	1.423

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

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Axial strain versus vertical effective stress for HB-101 T-1 CRS

19109-00 12/1/2014

HARTCROWSER

Figure 6

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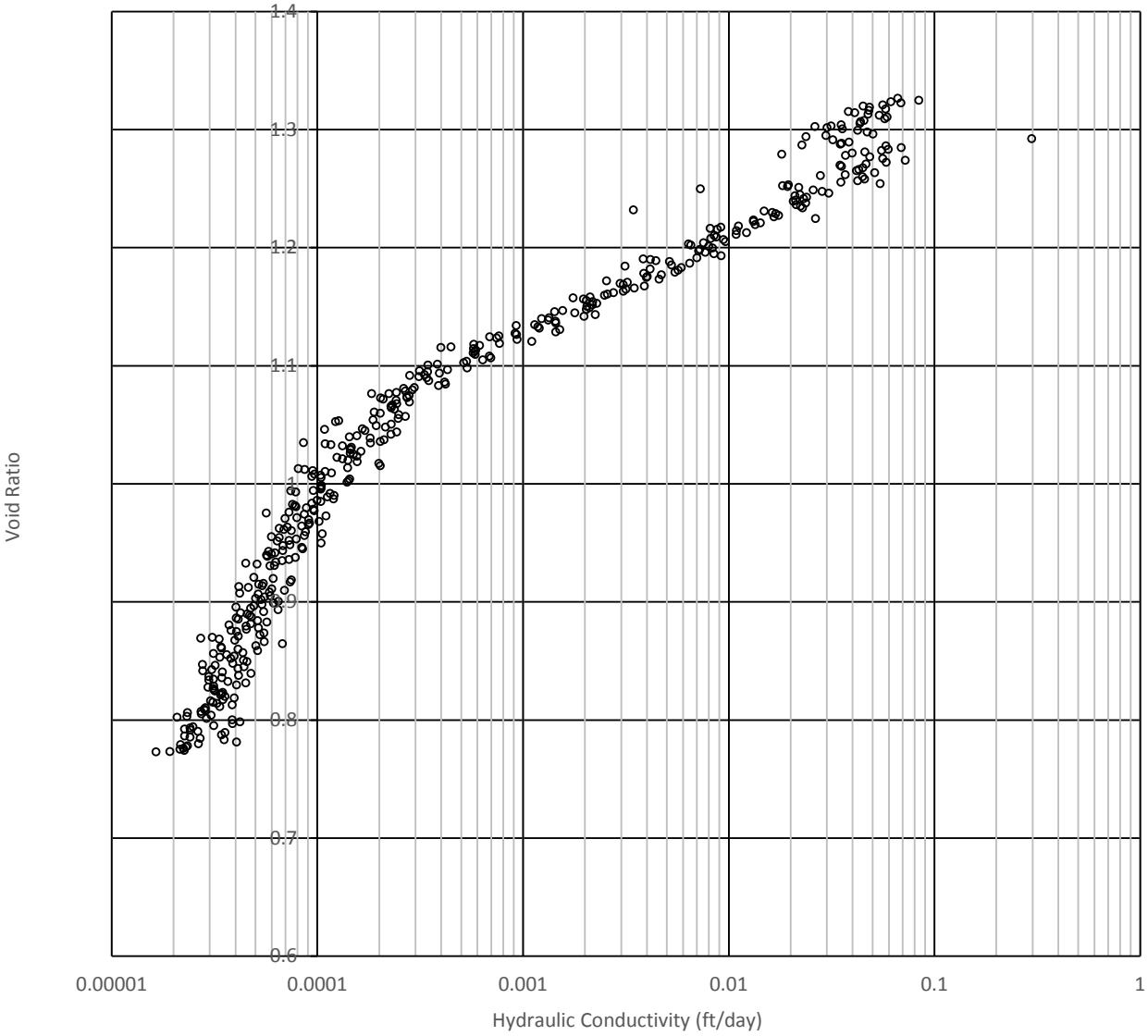

Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
22.5 feet	54	38	50	27	23	CLAY	CH

σ'_{v_o}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1300	1900	2200	1700
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
D		Poor	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	4.65
Total Unit Weight (pcf)	102.35
Degree of Saturation (%)	97.70
Void Ratio (e0)	1.423

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

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Void ratio versus logarithm of hydraulic conductivity
HB-101 T-1 CRS

19109-00 12/1/2014

HARTCROWSER

Figure
7

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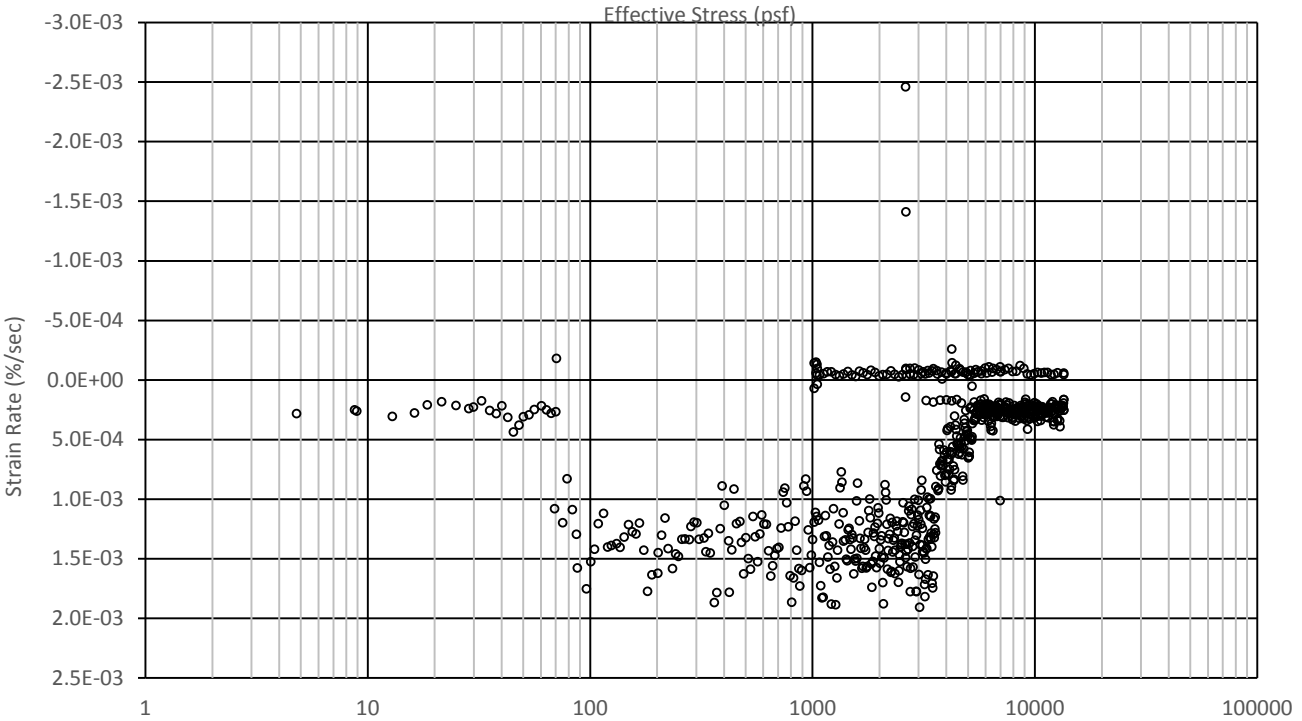

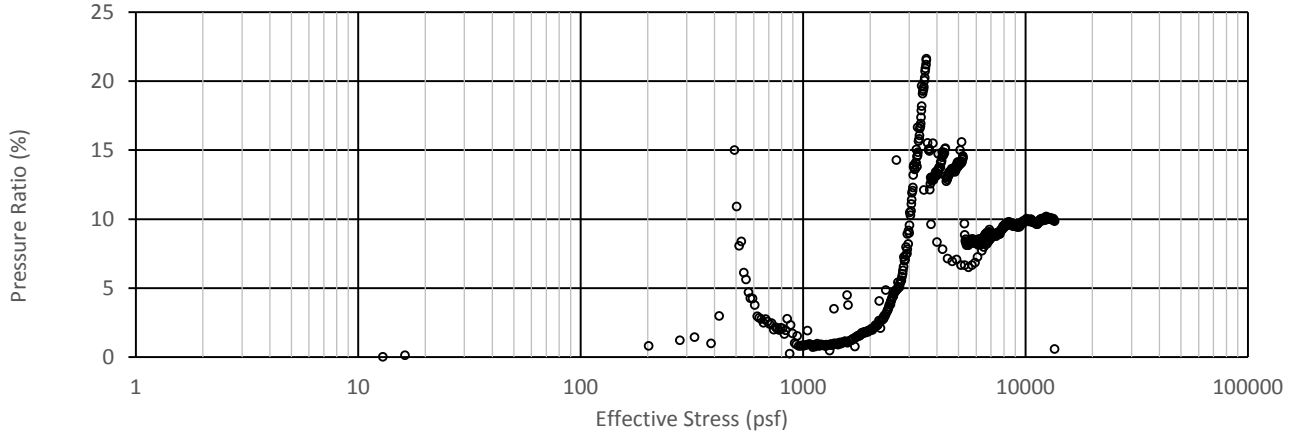

Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
22.5 feet	54	38	50	27	23	CLAY	CH

σ'_{v_o}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1300	1900	2200	1700
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
D		Poor	

Initial Specimen Properties	
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Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

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Axial strain, void ratio, and coefficient of consolidation versus logarithm of vertical effective stress for HB-101 T-1 CRS

19109-00 12/1/2014

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Figure 8

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CRS sample HB-B-101, T-1 (22.5 feet) in CRS ring before testing.

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Hayward Baker	
HB-B-101 CRS Sample Photos	
19109-00	12/1/2014
HARTCROWSER	Figure **9**

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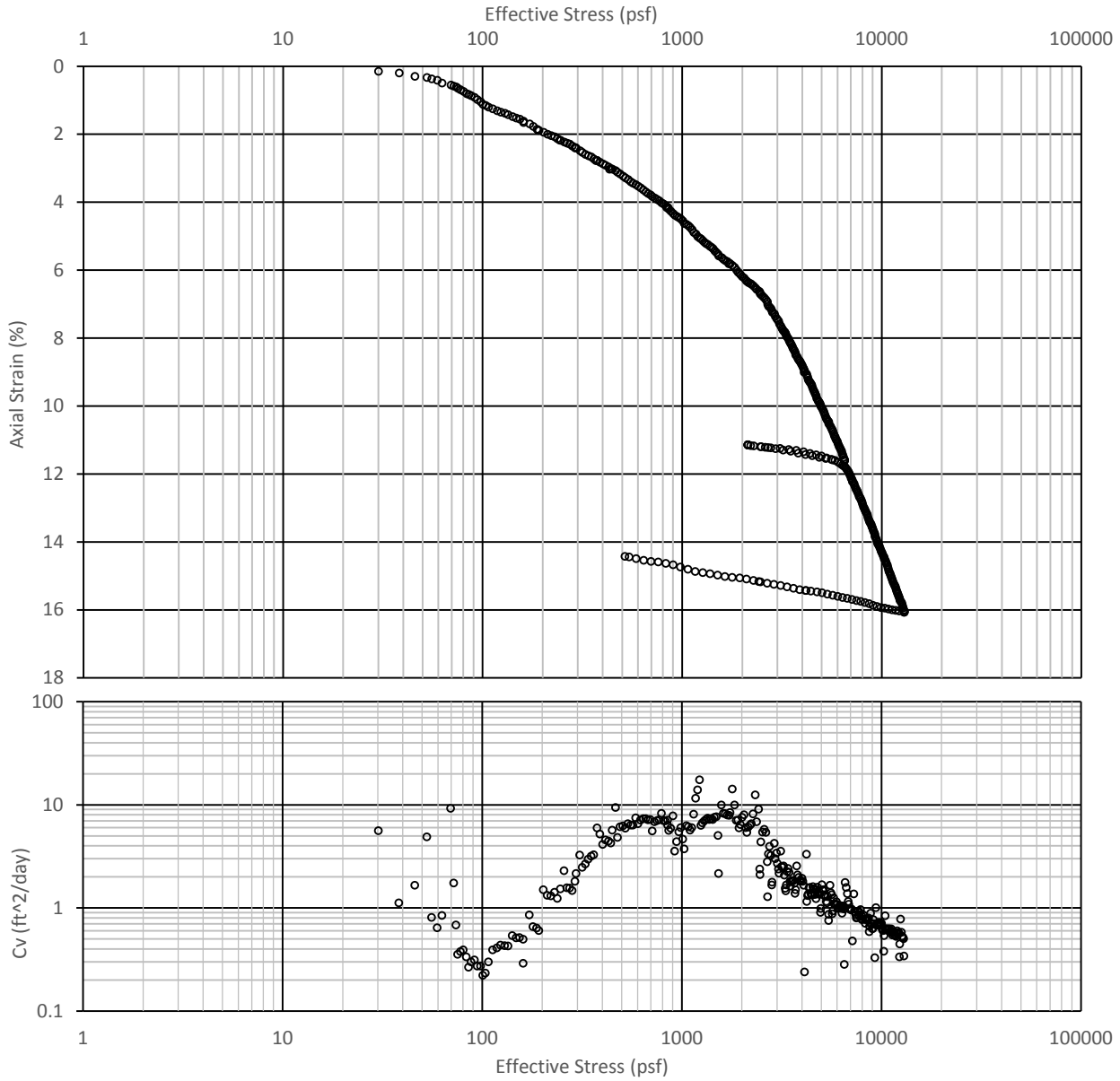

Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
22.9	26	20	29	18	11	CLAY	CL

σ'_{v_o}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1300	2900	3100	2800
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
D		Very poor	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	5.63
Total Unit Weight (pcf)	123.89
Degree of Saturation (%)	98.31
Void Ratio (e0)	0.708

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

Hayward Baker

Axial strain and coefficient of consolidation versus logarithm of vertical effective stress for HB-102 T-1 CRS

19109-00 12/19/2014

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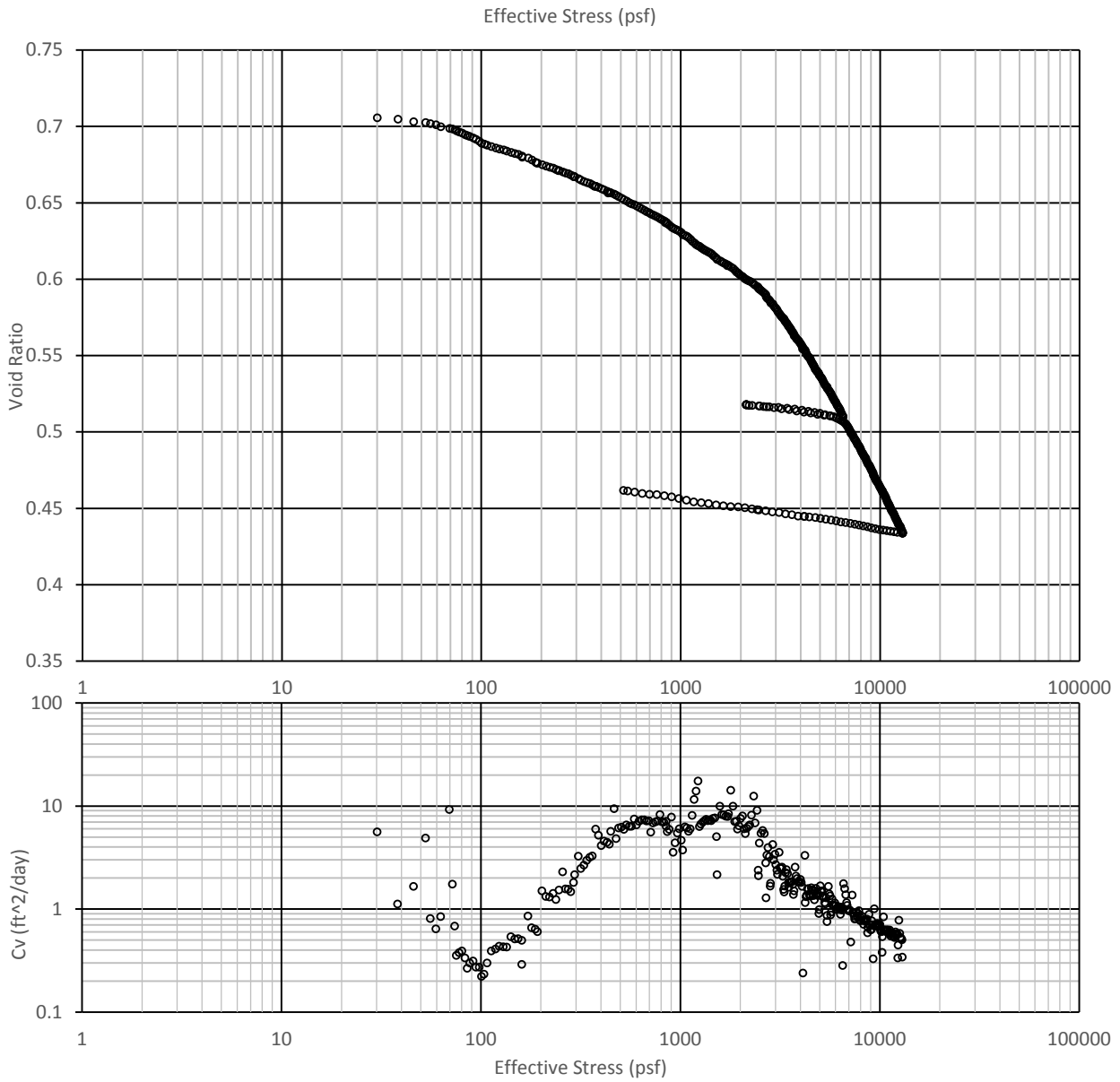
Figure **10A**

initials MM/DD/YY location\filename.xls



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
22.9	26	20	29	18	11	CLAY	CL

$\sigma'v_o$	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1300	2900	3100	2800
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
D		Very poor	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	5.63
Total Unit Weight (pcf)	123.89
Degree of Saturation (%)	98.31
Void Ratio (e0)	0.708

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

Hayward Baker

Void ratio and coefficient of consolidation versus logarithm of vertical effective stress for HB-102 T-1 CRS

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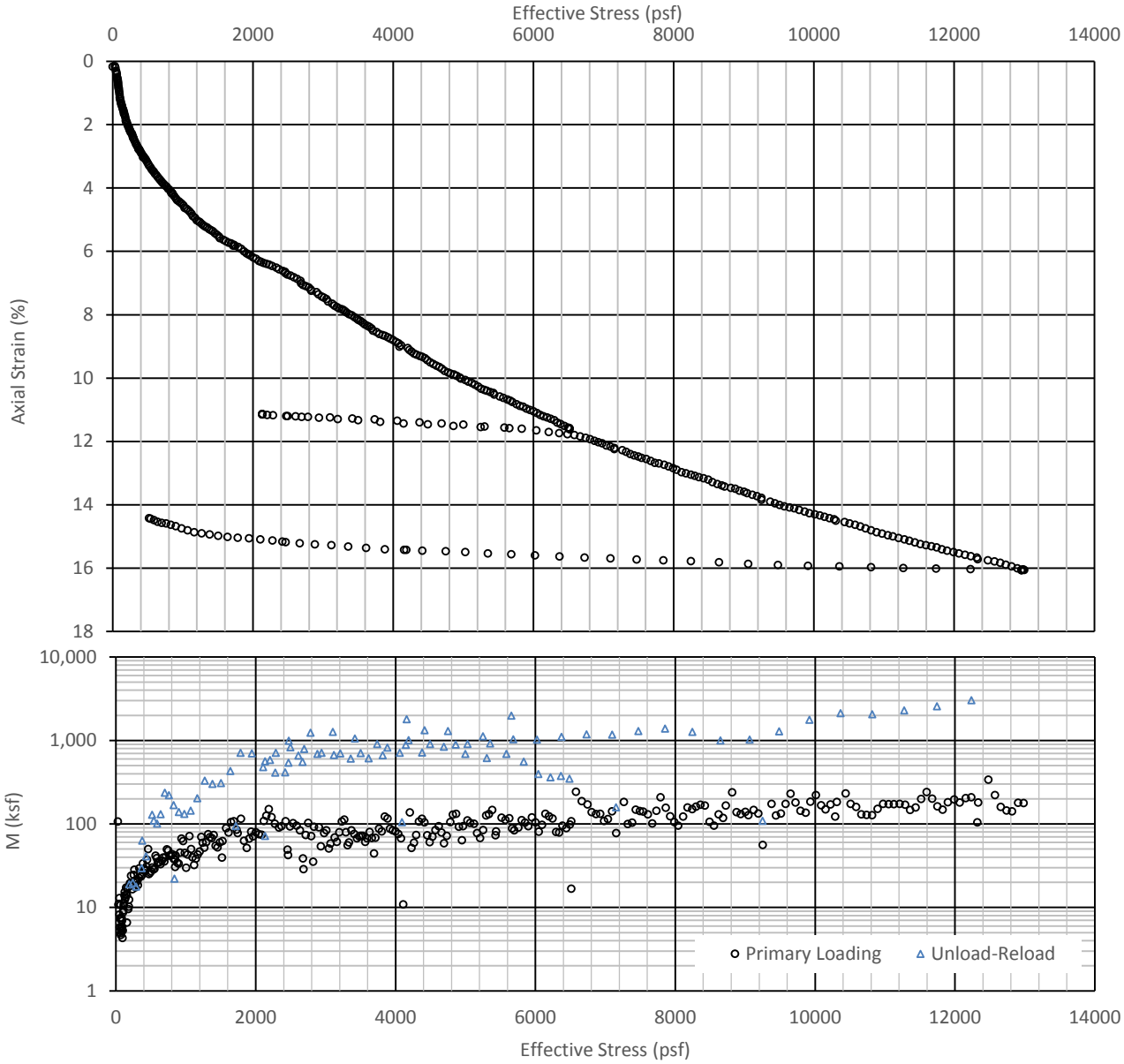
Figure **10B**

initials MM/DD/YY location\filename.xls



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
22.9	26	20	29	18	11	CLAY	CL

σ'_{v_o}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1300	2900	3100	2800
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
D		Very poor	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	5.63
Total Unit Weight (pcf)	123.89
Degree of Saturation (%)	98.31
Void Ratio (e0)	0.708

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

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Axial strain versus vertical effective stress for HB-102 T-1 CRS

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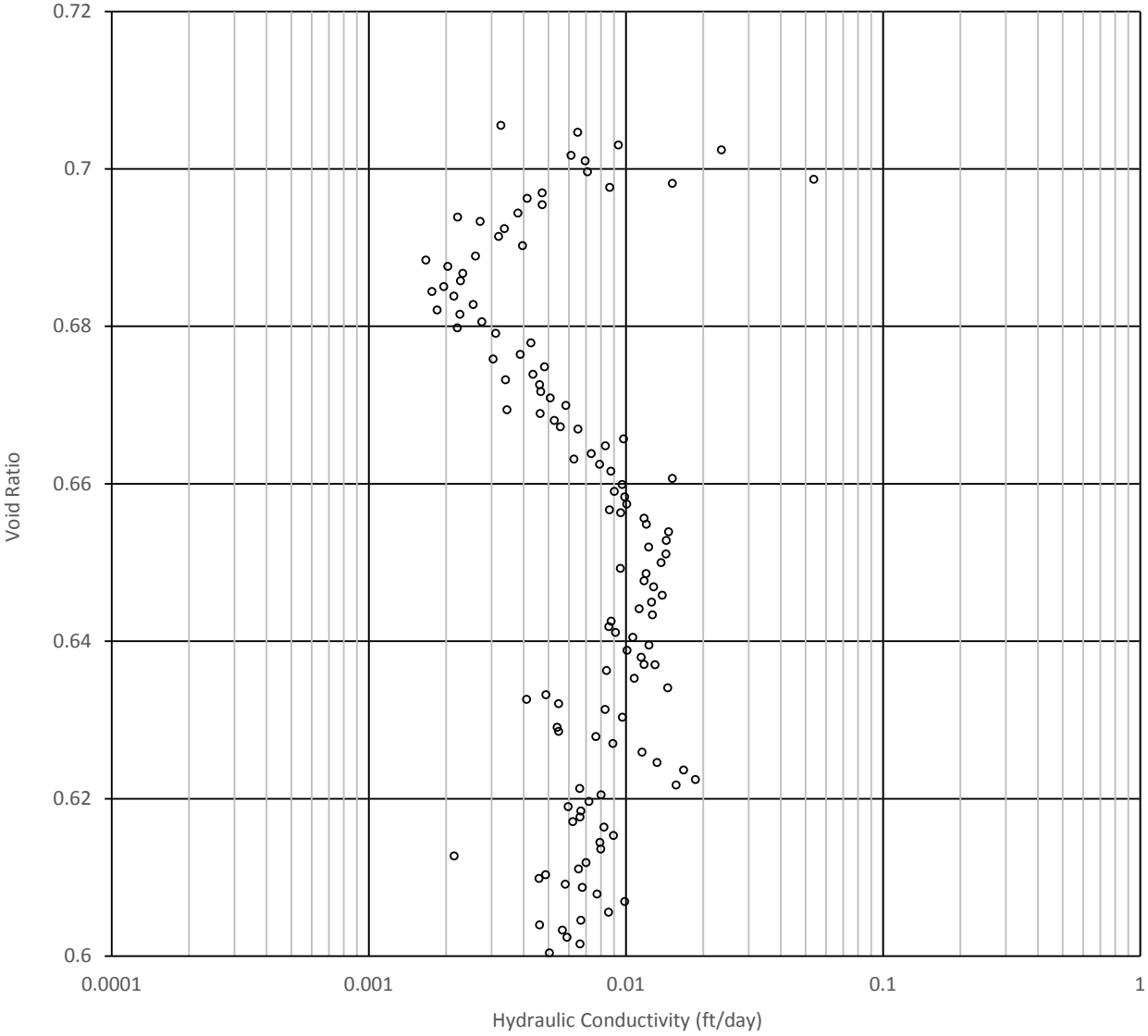
Figure 11

initials MM/DD/YY location\filename.xls



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
22.9	26	20	29	18	11	CLAY	CL

σ'_{v_o}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1300	2900	3100	2800
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
D		Very poor	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	5.63
Total Unit Weight (pcf)	123.89
Degree of Saturation (%)	98.31
Void Ratio (e0)	0.708

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

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Void ratio versus logarithm of hydraulic conductivity
HB-102 T-1 CRS

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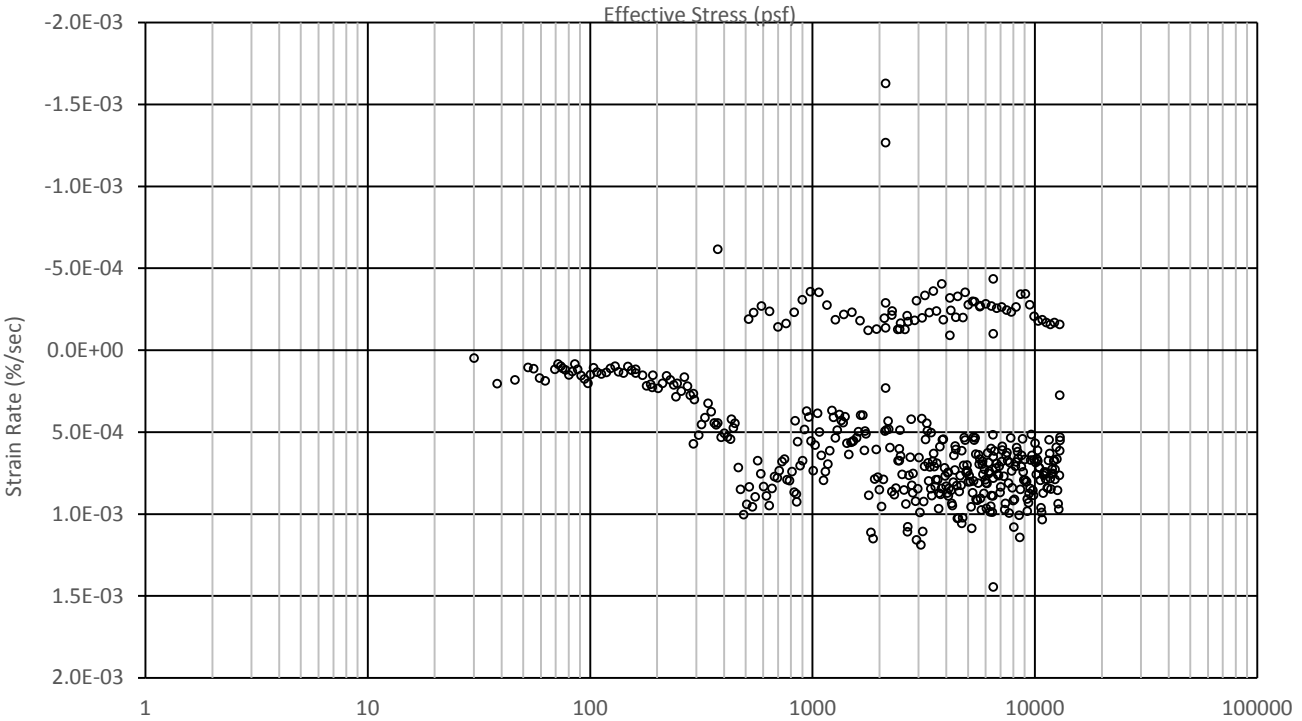
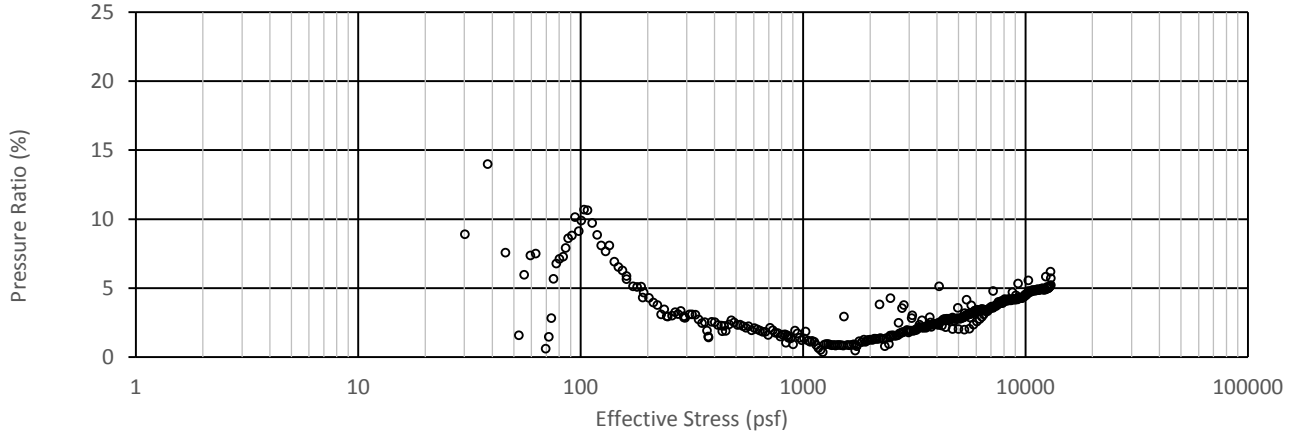
Figure
12

initials MM/DD/YY location\filename.xls





Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
22.9	26	20	29	18	11	CLAY	CL

σ'_{v_o}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1300	2900	3100	2800
Sample Quality Designation			
Terzaghi et al. (1996)	Lunne et al. (1997)		
D	Very poor		

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	5.63
Total Unit Weight (pcf)	123.89
Degree of Saturation (%)	98.31
Void Ratio (e0)	0.708

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

Hayward Baker

Axial strain, void ratio, and coefficient of consolidation versus logarithm of vertical effective stress for HB-102 T-1 CRS

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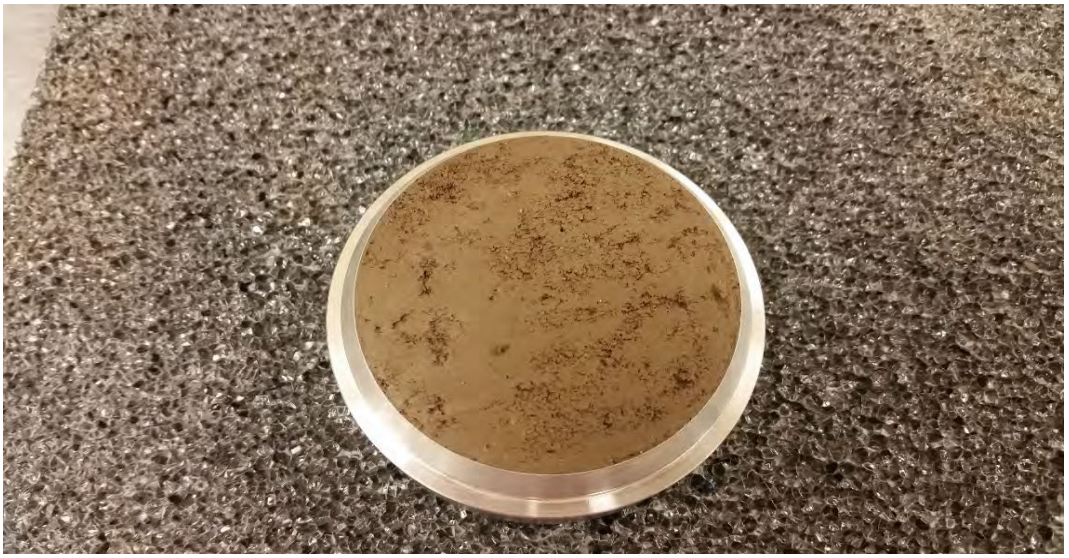
Figure 13

initials MM/DD/YY location\filename.xls



CRS sample HB-B-102, T-1 (22.9 feet) in CRS ring before testing.

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HB-B-102 CRS Sample Photo

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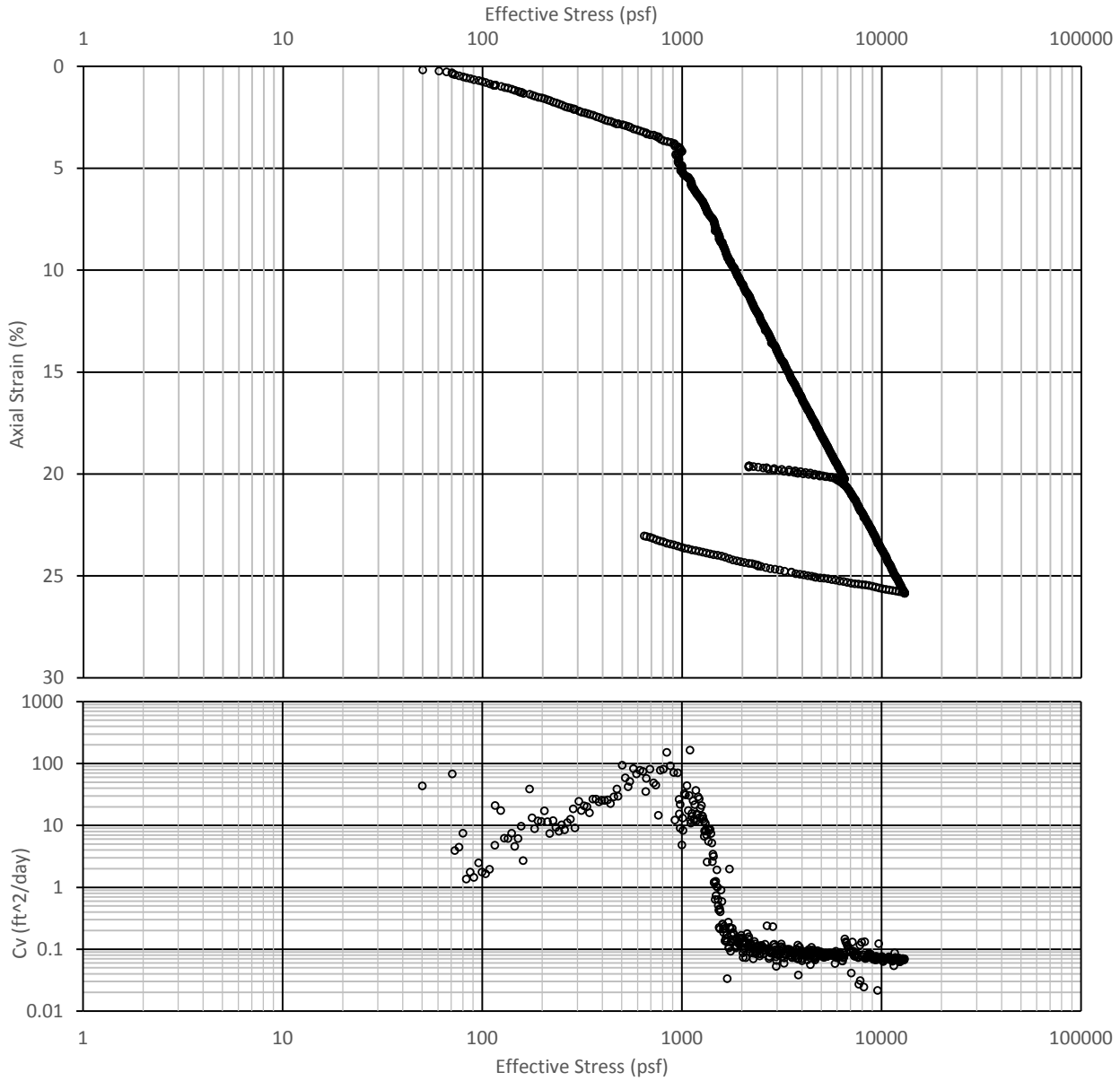
Figure

14

initials MM/DD/YY location\filename.xls

EX-0001-006944-PCE



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
23.9	40	26	43	23	20	CLAY	CL

σ'_{vo}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1000	1000	1250	1000
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
D		Very poor	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	4.93
Total Unit Weight (pcf)	108.53
Degree of Saturation (%)	93.12
Void Ratio (e0)	1.126

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

Hayward Baker

Axial strain and coefficient of consolidation versus logarithm of vertical effective stress for HB-103 T-1 CRS

19109-00 12/8/2014

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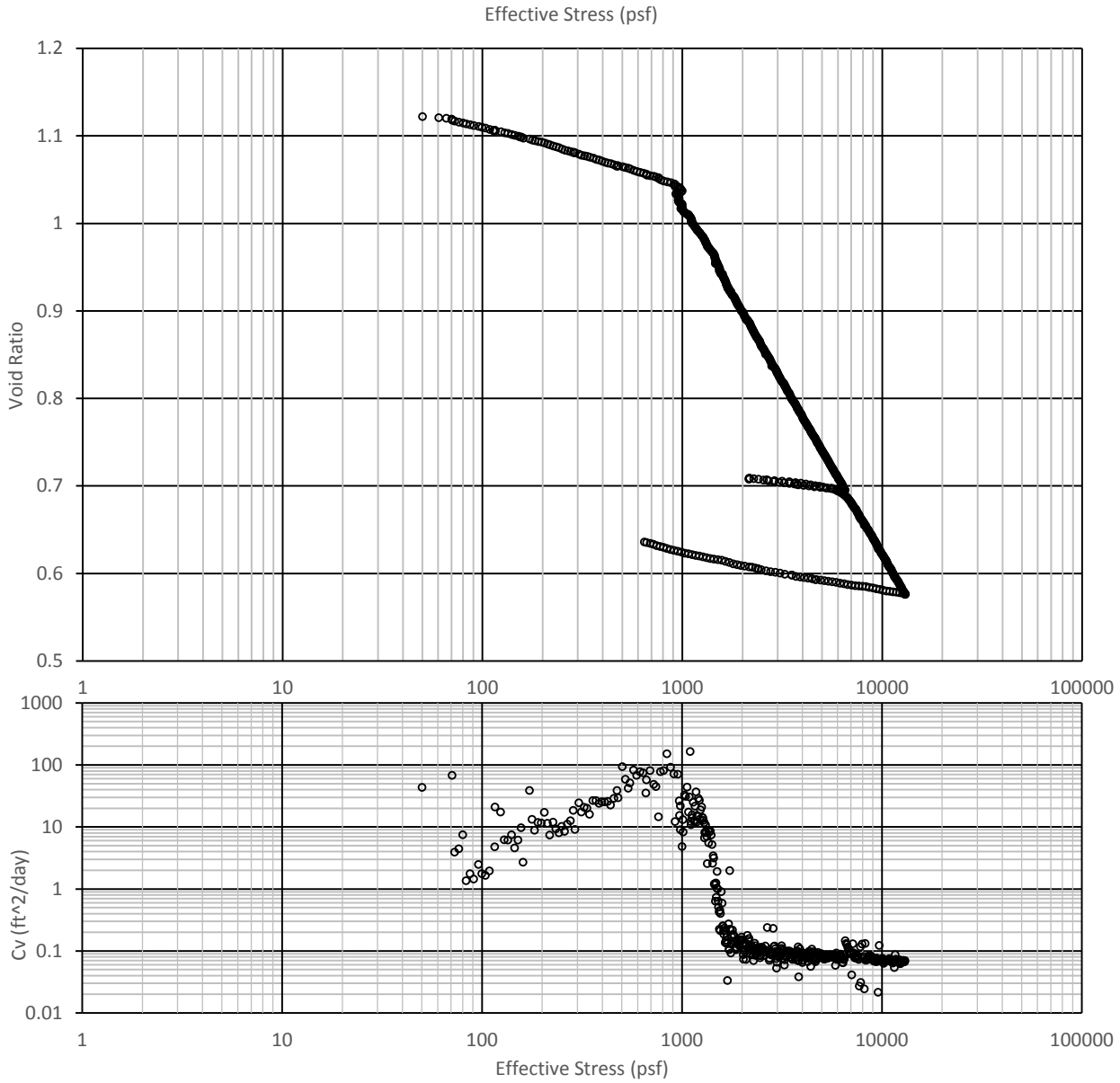
Figure **15A**

initials MM/DD/YY location\filename.xls



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
23.9	40	26	43	23	20	CLAY	CL

σ'_{v_o}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1000	1000	1250	1000
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
D		Very poor	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	4.93
Total Unit Weight (pcf)	108.53
Degree of Saturation (%)	93.12
Void Ratio (e0)	1.126

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

Hayward Baker

Void ratio and coefficient of consolidation versus logarithm of vertical effective stress for HB-103 T-1 CRS

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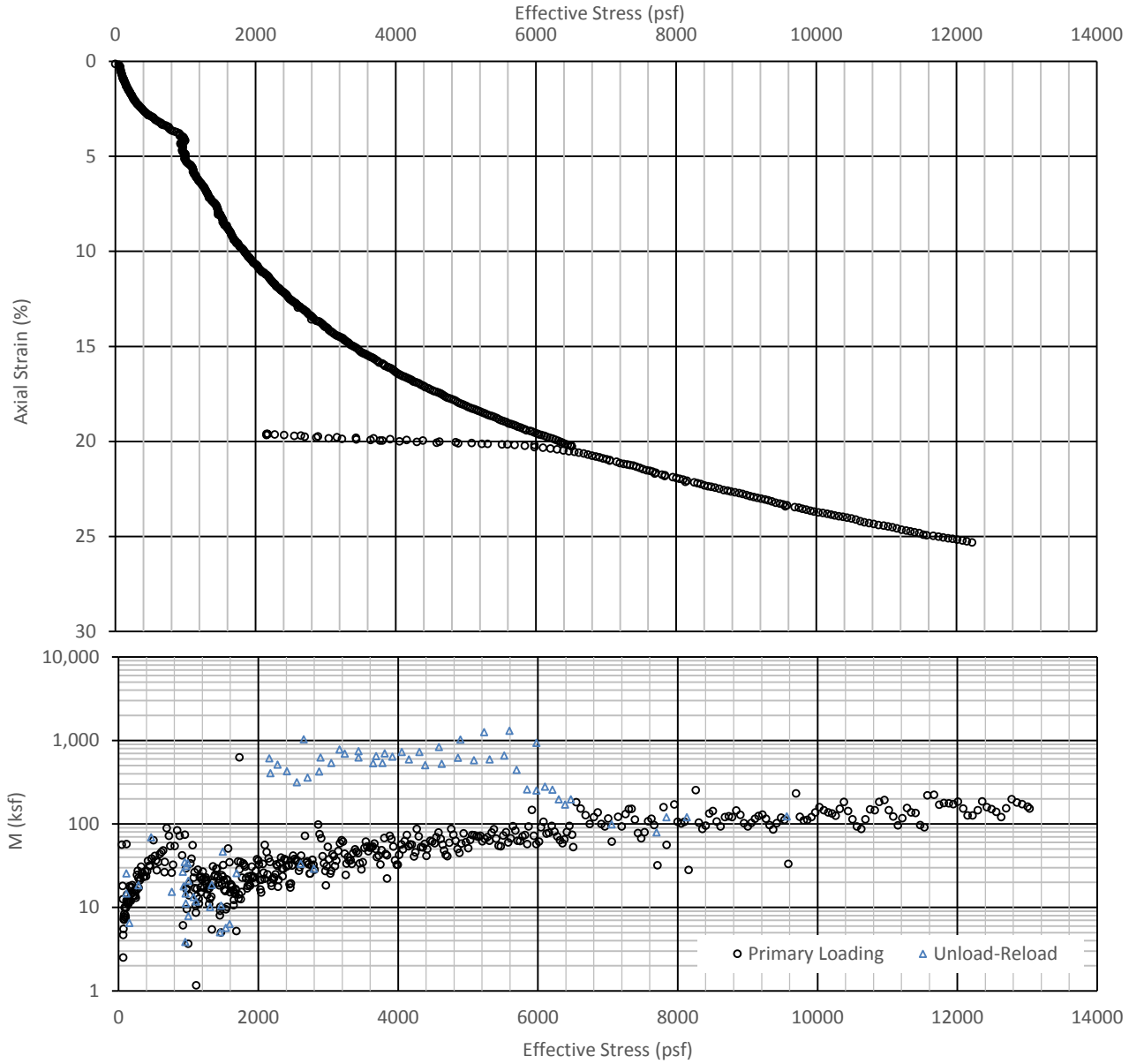
Figure **15B**

initials MM/DD/YY location\filename.xls



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
23.9	40	26	43	23	20	CLAY	CL

σ'_{v_o}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1000	1000	1250	1000
Sample Quality Designation			
Terzaghi et al. (1996)	Lunne et al. (1997)		
D	Very poor		

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	4.93
Total Unit Weight (pcf)	108.53
Degree of Saturation (%)	93.12
Void Ratio (e0)	1.126

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

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Axial strain versus vertical effective stress for HB-103 T-1 CRS

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Figure 16

initials MM/DD/YY location\filename.xls

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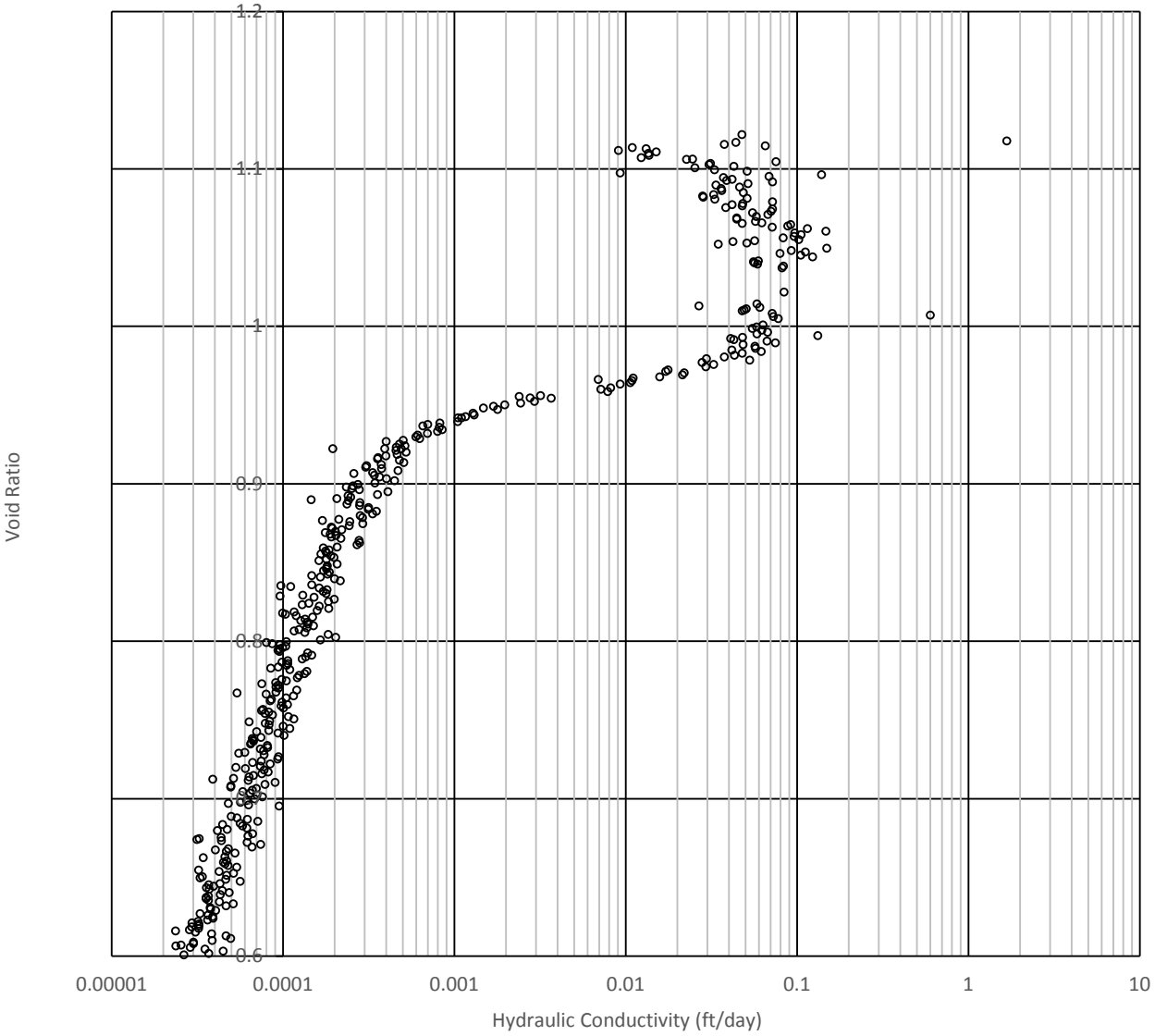

Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
23.9	40	26	43	23	20	CLAY	CL

σ'_{v_o}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1000	1000	1250	1000
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
D		Very poor	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	4.93
Total Unit Weight (pcf)	108.53
Degree of Saturation (%)	93.12
Void Ratio (e0)	1.126

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

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Void ratio versus logarithm of hydraulic conductivity
HB-103 T-1 CRS

19109-00 12/8/2014

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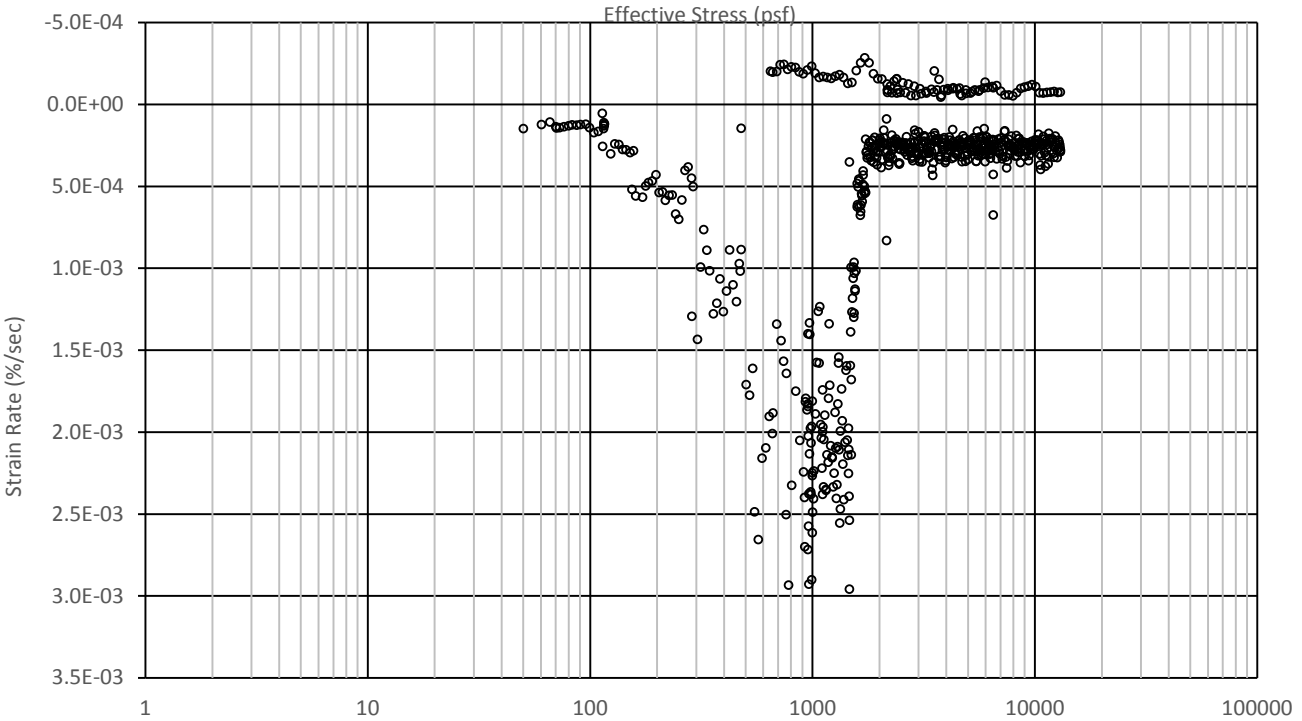
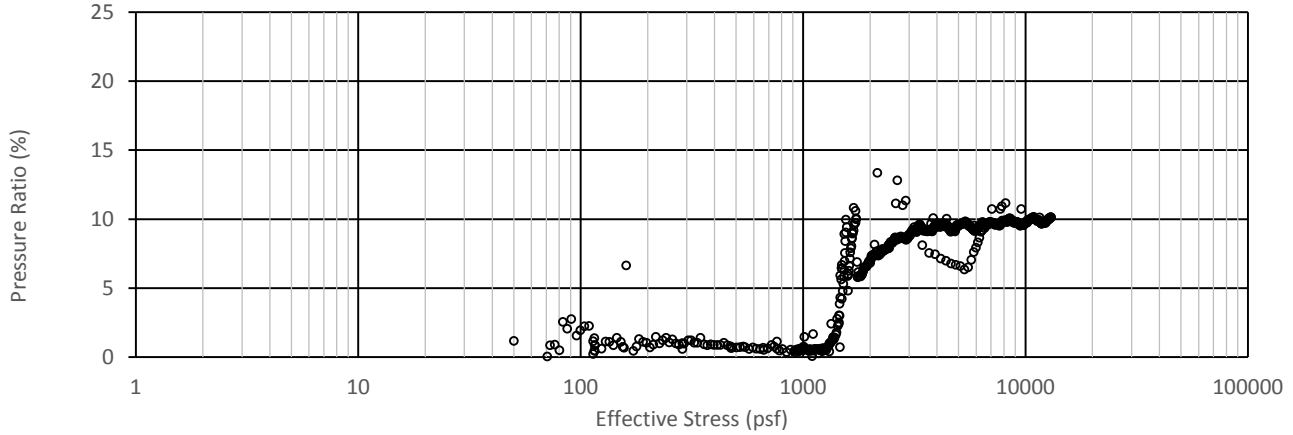
Figure
17

initials MM/DD/YY location\filename.xls





Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
23.9	40	26	43	23	20	CLAY	CL

σ'_{vo}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1000	1000	1250	1000
Sample Quality Designation			
Terzaghi et al. (1996)	Lunne et al. (1997)		
D	Very poor		

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	4.93
Total Unit Weight (pcf)	108.53
Degree of Saturation (%)	93.12
Void Ratio (e0)	1.126

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

Hayward Baker

Axial strain, void ratio, and coefficient of consolidation versus logarithm of vertical effective stress for HB-103 T-1 CRS

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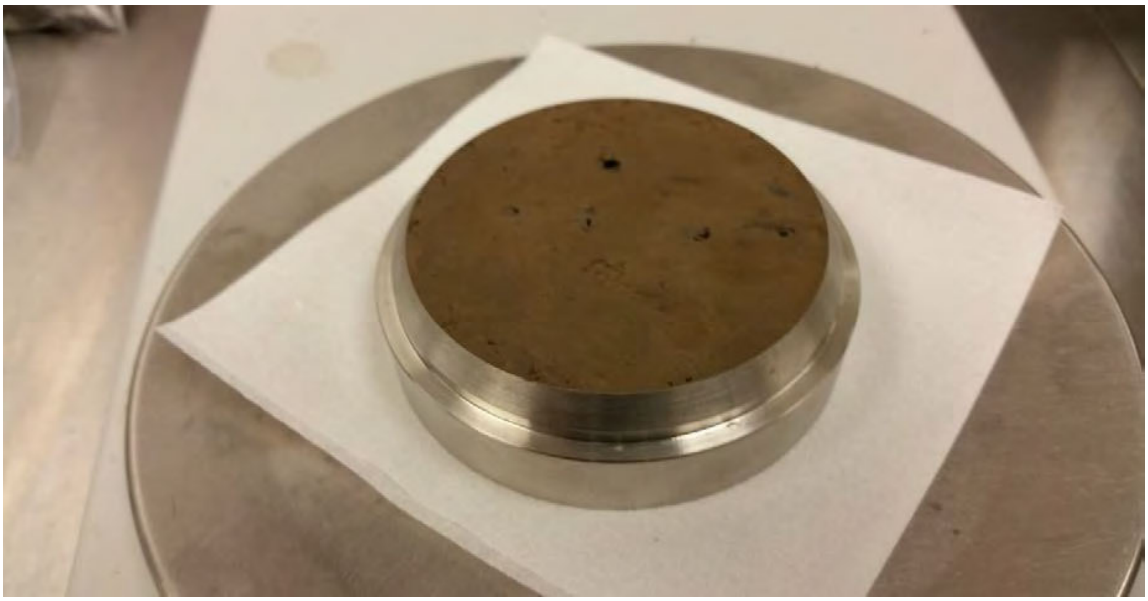
Figure 18

initials MM/DD/YY location\filename.xls

Photo of Tube HB-B-103 cross-section showing vertical channels.

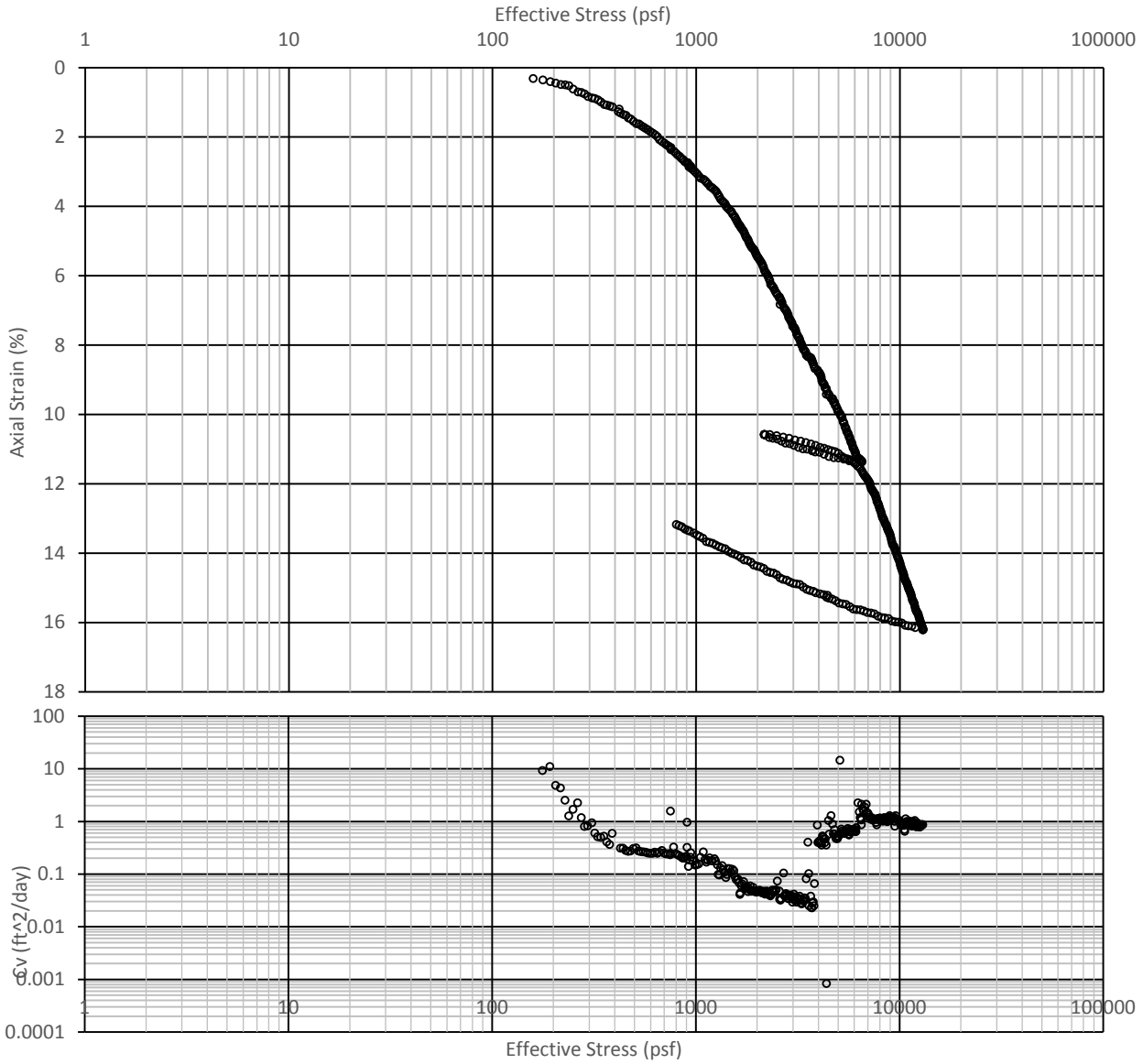


CRS sample HB-B-103, T-1 (23.9 feet) in CRS ring before testing.

initials MM/DD/YY location\filename.xls

Hayward Baker	
HB-B-103 CRS Sample Photos	
19109-00	12/8/2014
HARTCROWSER	Figure **19**

EX-0001-006950-PCE



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
18	31	26	59	21	38	CLAY	CH

σ'_{v_o}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1000	1100	1500	1500
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
C		Good to fair	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	5.41
Total Unit Weight (pcf)	119.11
Degree of Saturation (%)	102.37
Void Ratio (e0)	0.795

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

Hayward Baker

Axial strain and coefficient of consolidation versus logarithm of vertical effective stress for HB-105 T-1 CRS

19109-00 12/4/2014

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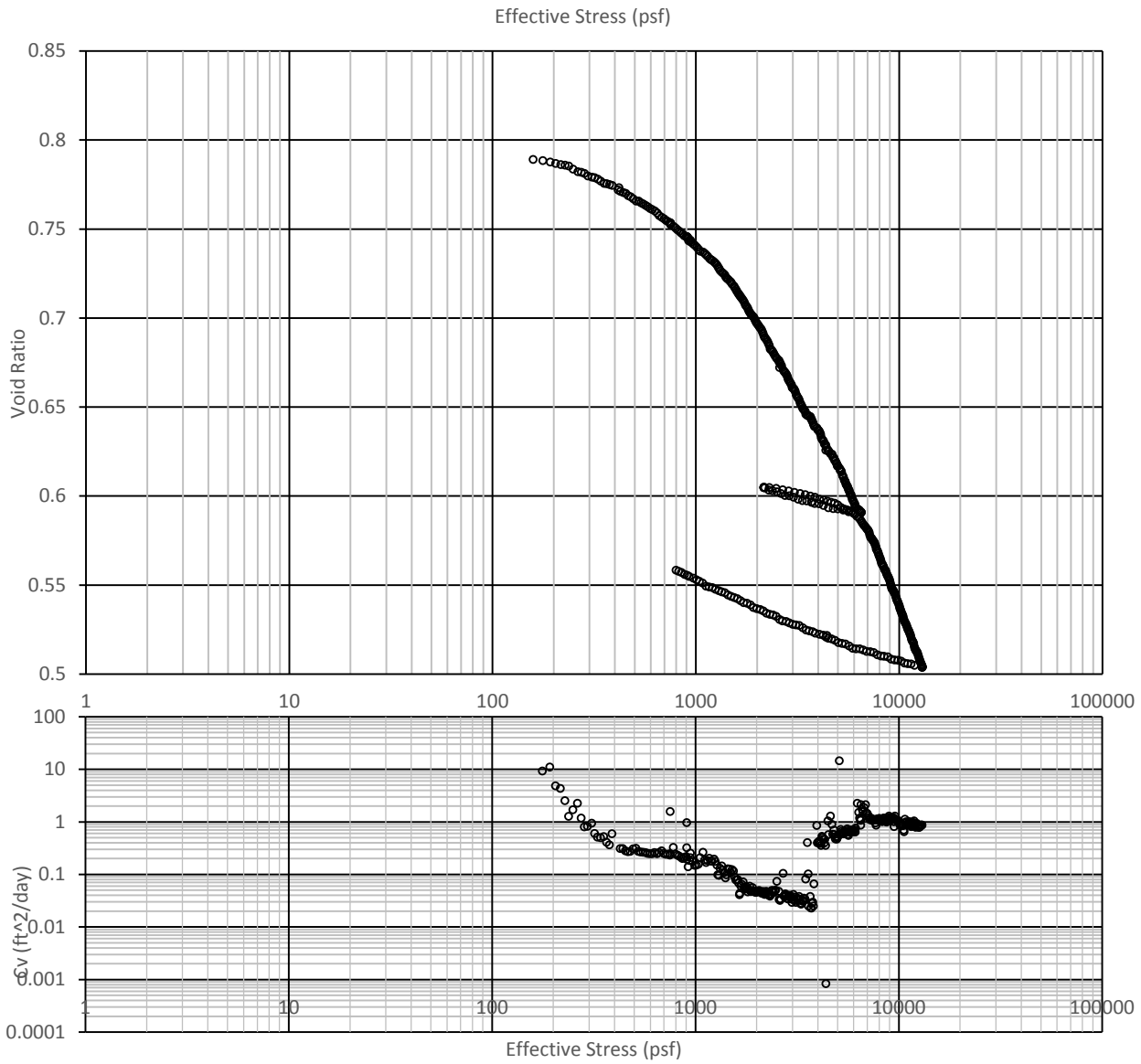
Figure **20A**

initials MM/DD/YY location\filename.xls



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
18	31	26	59	21	38	CLAY	CH

σ'_{vo}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1000	1100	1500	1500
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
C		Good to fair	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	5.41
Total Unit Weight (pcf)	119.11
Degree of Saturation (%)	102.37
Void Ratio (e0)	0.795

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

Hayward Baker

Void ratio and coefficient of consolidation versus logarithm of vertical effective stress for HB-105 T-1 CRS

19109-00 12/4/2014

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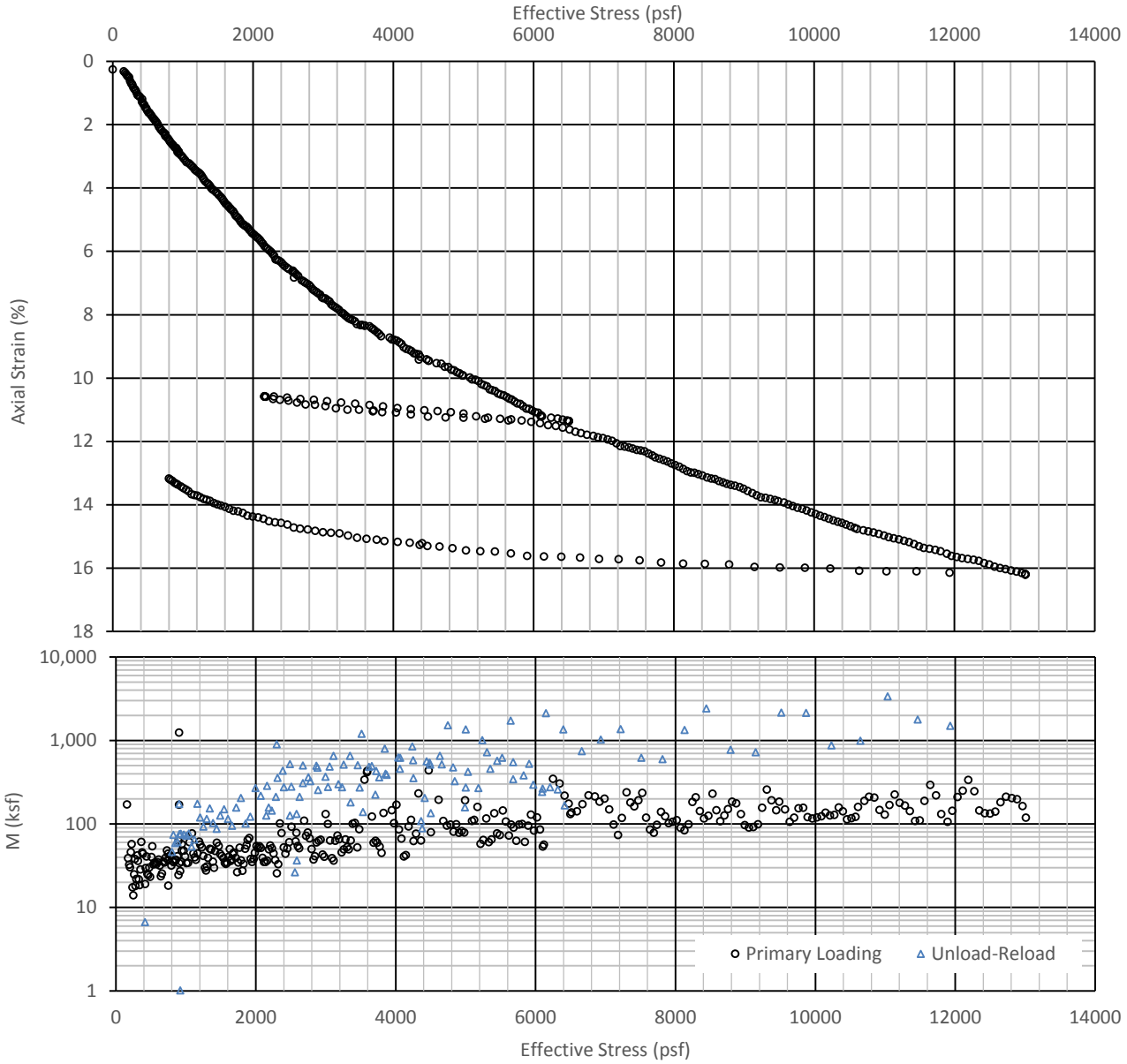
Figure **20B**

initials MM/DD/YY location\filename.xls

EX-0001-006952-PCE



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
18	31	26	59	21	38	CLAY	CH

σ'_{v_o}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1000	1100	1500	1500
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
C		Good to fair	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	5.41
Total Unit Weight (pcf)	119.11
Degree of Saturation (%)	102.37
Void Ratio (e0)	0.795

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

Hayward Baker

Axial strain versus vertical effective stress for HB-105 T-1 CRS

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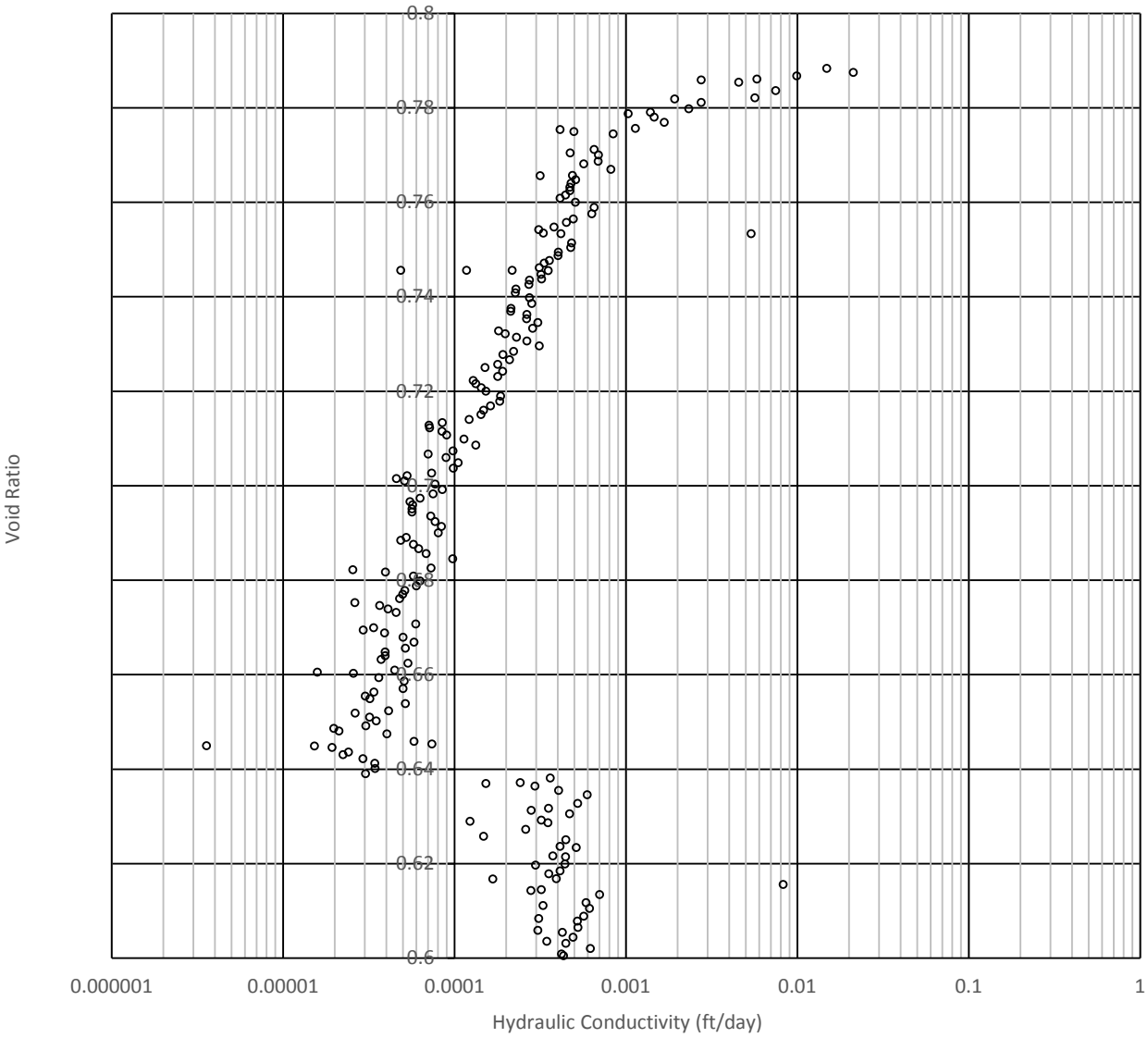
Figure 21

initials MM/DD/YY location\filename.xls



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
18	31	26	59	21	38	CLAY	CH

σ'_{v_o}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1000	1100	1500	1500
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
C		Good to fair	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	5.41
Total Unit Weight (pcf)	119.11
Degree of Saturation (%)	102.37
Void Ratio (e0)	0.795

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

Hayward Baker

Void ratio versus logarithm of hydraulic conductivity
HB-105 T-1 CRS

19109-00 12/4/2014

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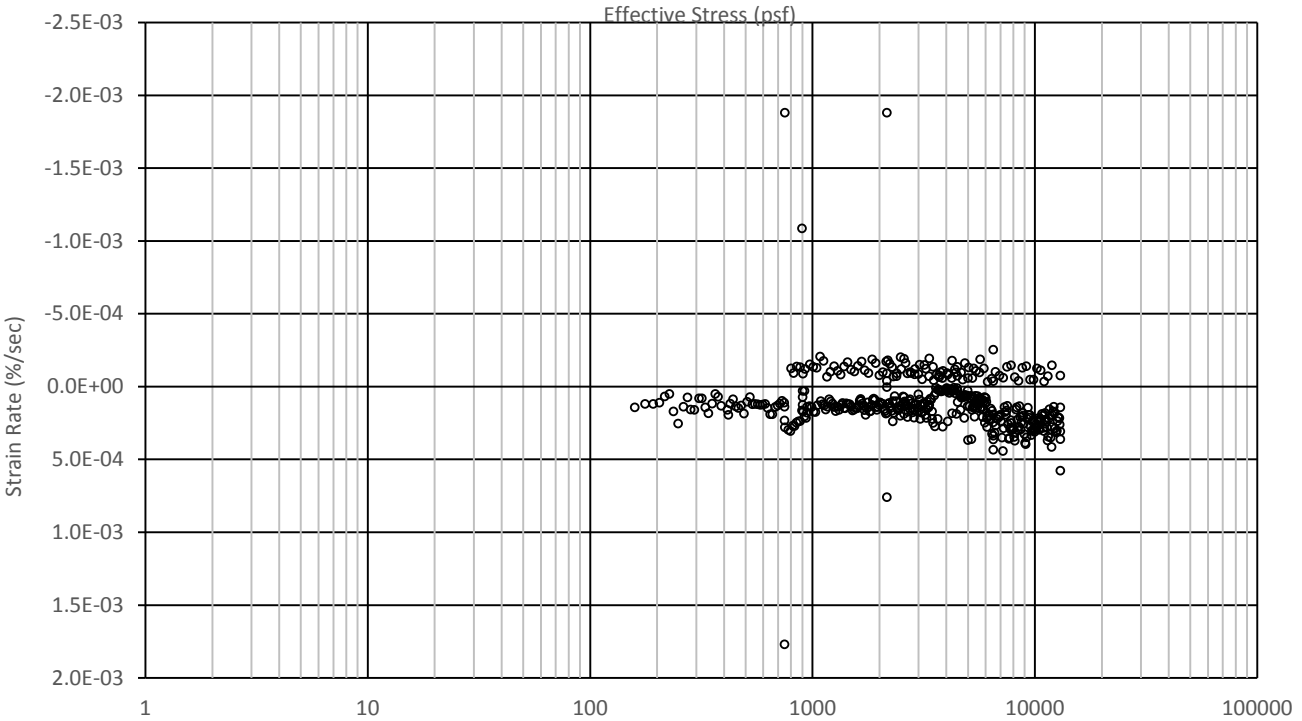
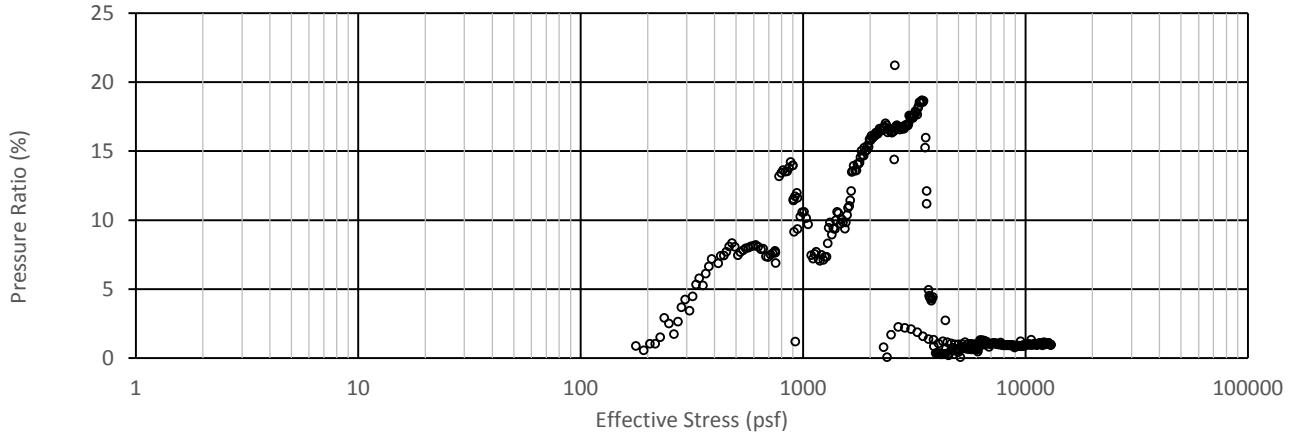
Figure
22

initials MM/DD/YY location\filename.xls

EX-0001-006954-PCE





Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
18	31	26	59	21	38	CLAY	CH

σ'_{v_o}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1000	1100	1500	1500
Sample Quality Designation			
Terzaghi et al. (1996)	Lunne et al. (1997)		
C	Good to fair		

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	5.41
Total Unit Weight (pcf)	119.11
Degree of Saturation (%)	102.37
Void Ratio (e0)	0.795

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

Hayward Baker

Axial strain, void ratio, and coefficient of consolidation versus logarithm of vertical effective stress for HB-105 T-1 CRS

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Figure 23

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CRS sample HB-B-105, T-1 (18.0 feet) in CRS ring before testing.

Hayward Baker

HB-B-105 CRS Sample Photo

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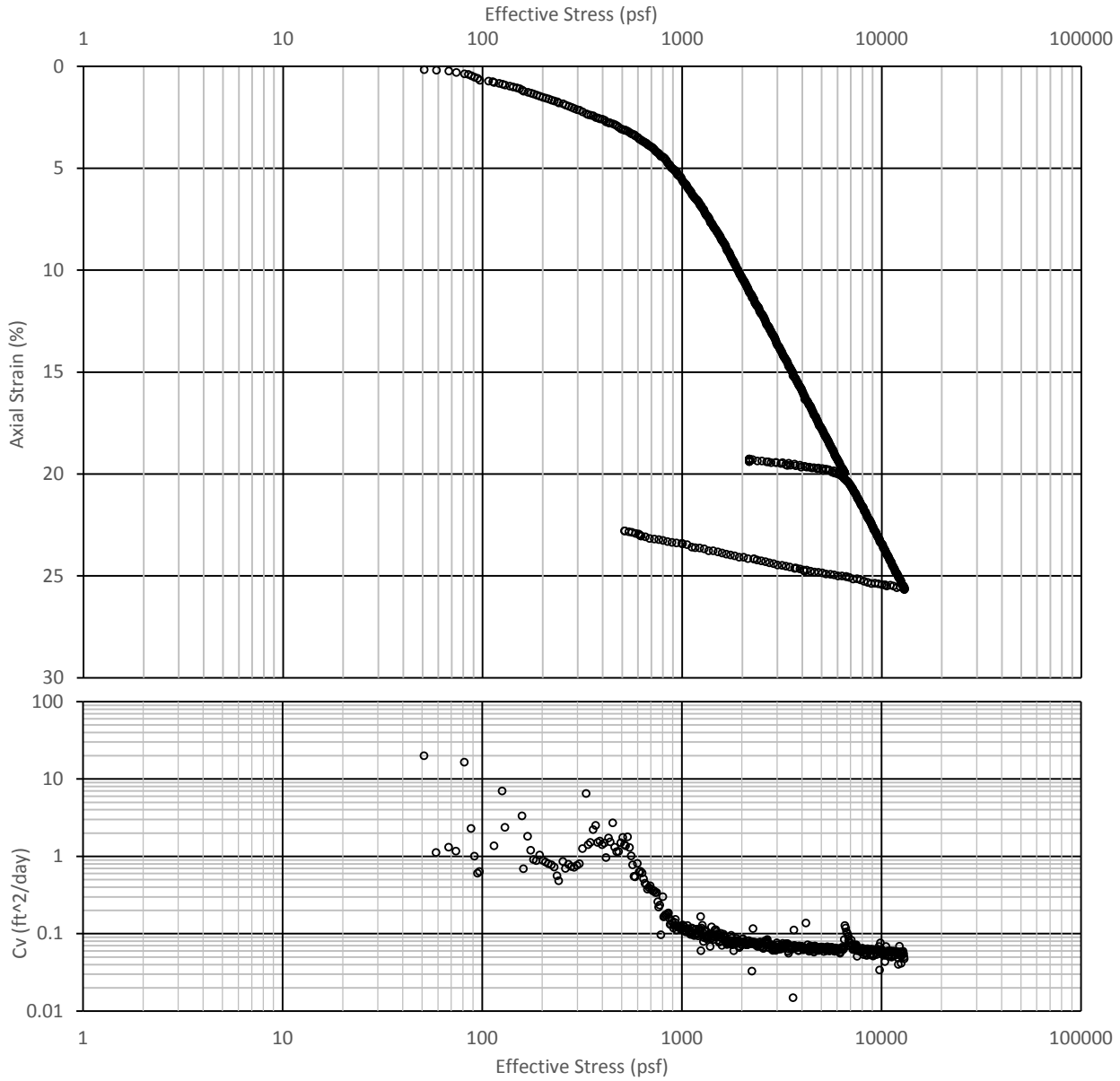
Figure

24

initials MM/DD/YY location\filename.xls

EX-0001-006956-PCE



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
22.3	42	29	46	25	21	CLAY	CL

σ'_{v_o}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1172.98	1000	1100	1050
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
D		Poor	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	5.00
Total Unit Weight (pcf)	110.03
Degree of Saturation (%)	97.28
Void Ratio (e0)	1.142

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

Hayward Baker

Axial strain and coefficient of consolidation versus logarithm of vertical effective stress for HB-106 T-1 CRS

19109-00 12/26/2014

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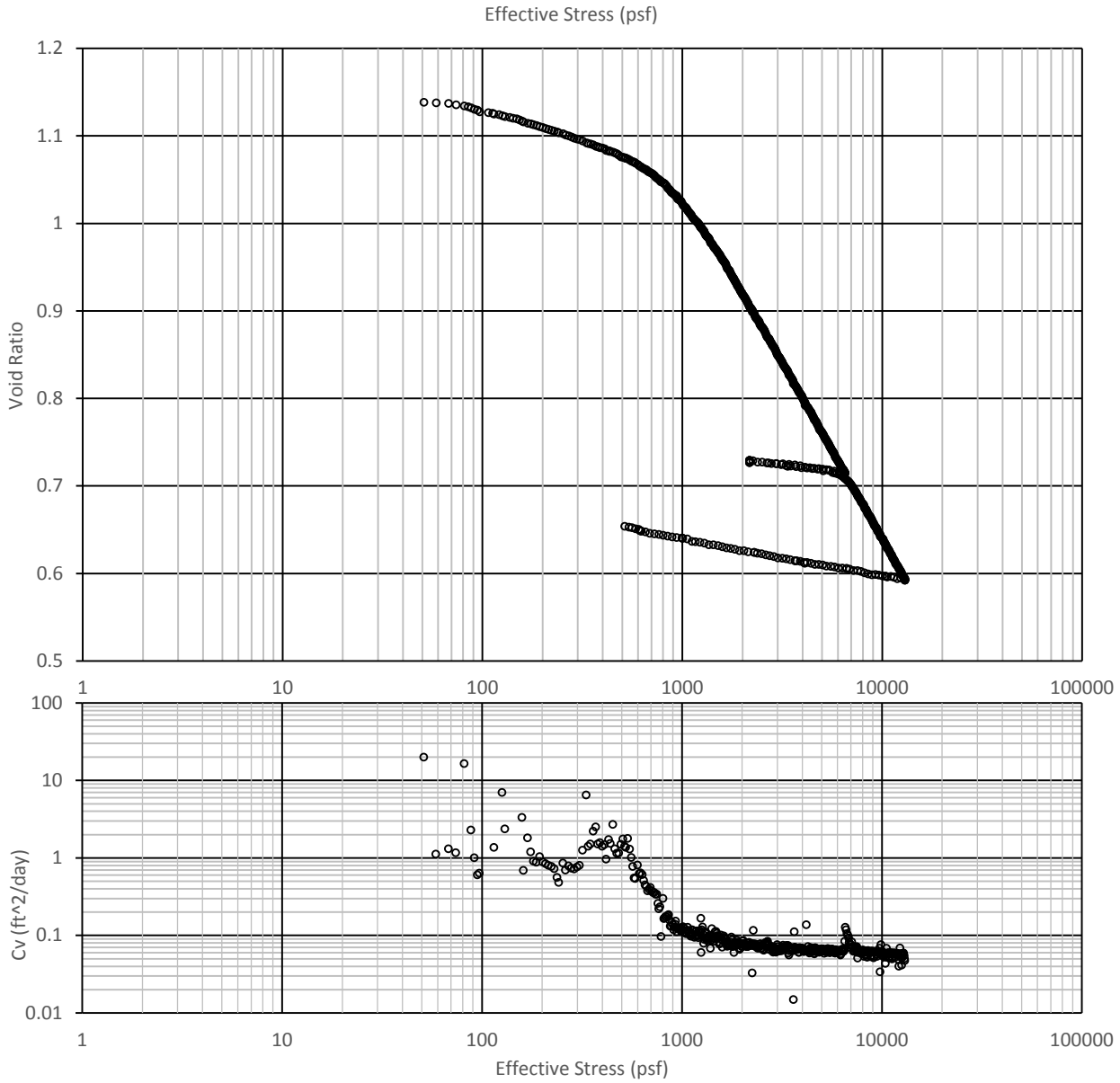
Figure **25A**

initials MM/DD/YY location\filename.xls



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
22.3	42	29	46	25	21	CLAY	CL

σ'_{v_o}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1172.98	1000	1100	1050
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
D		Poor	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	5.00
Total Unit Weight (pcf)	110.03
Degree of Saturation (%)	97.28
Void Ratio (e0)	1.142

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

Hayward Baker

Void ratio and coefficient of consolidation versus logarithm of vertical effective stress for HB-106 T-1 CRS

19109-00 12/26/2014

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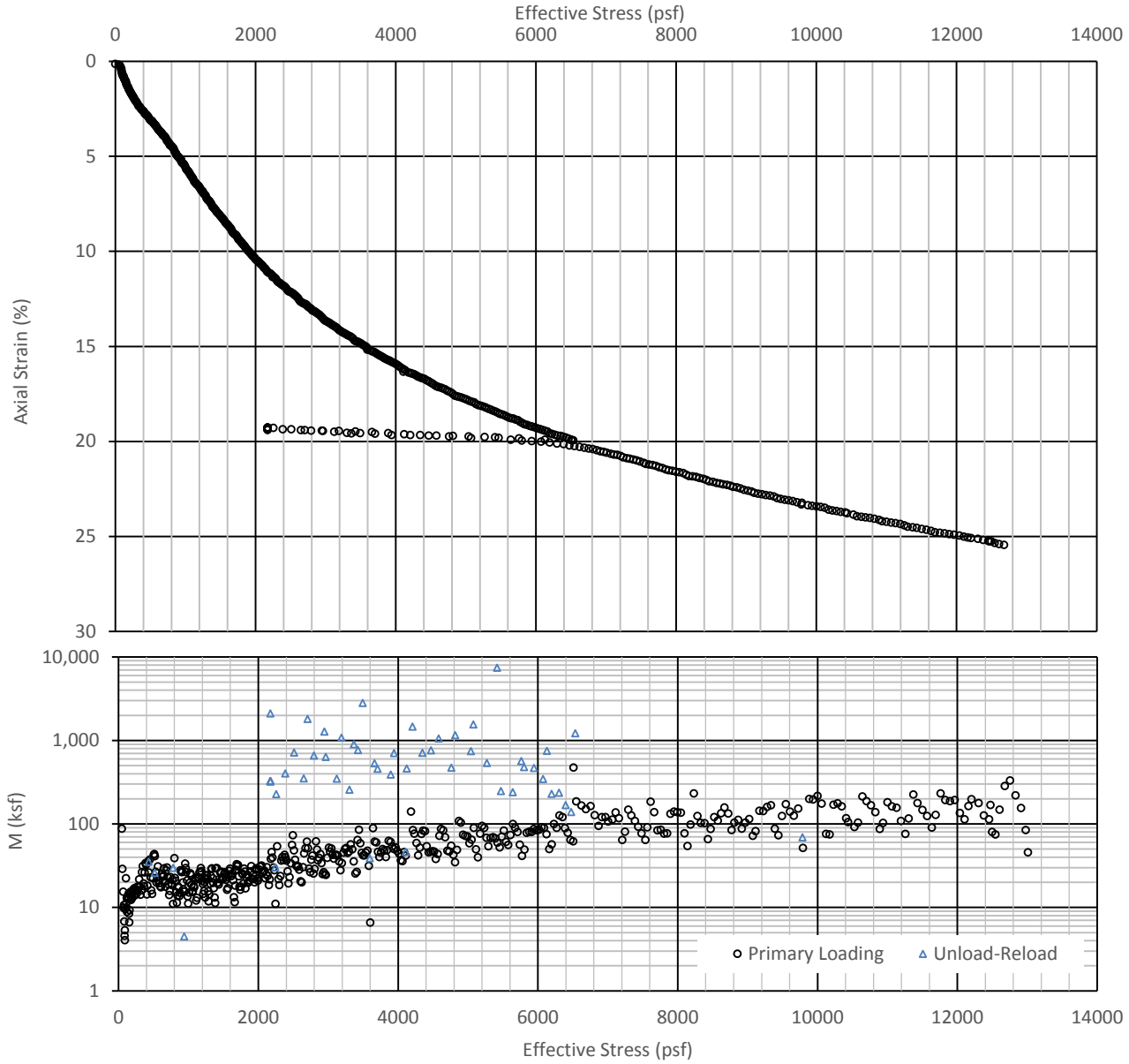
Figure **25B**

initials MM/DD/YY location\filename.xls



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
22.3	42	29	46	25	21	CLAY	CL

σ'_{v_o}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1172.98	1000	1100	1050
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
D		Poor	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	5.00
Total Unit Weight (pcf)	110.03
Degree of Saturation (%)	97.28
Void Ratio (e0)	1.142

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

Hayward Baker

Axial strain versus vertical effective stress for HB-106 T-1 CRS

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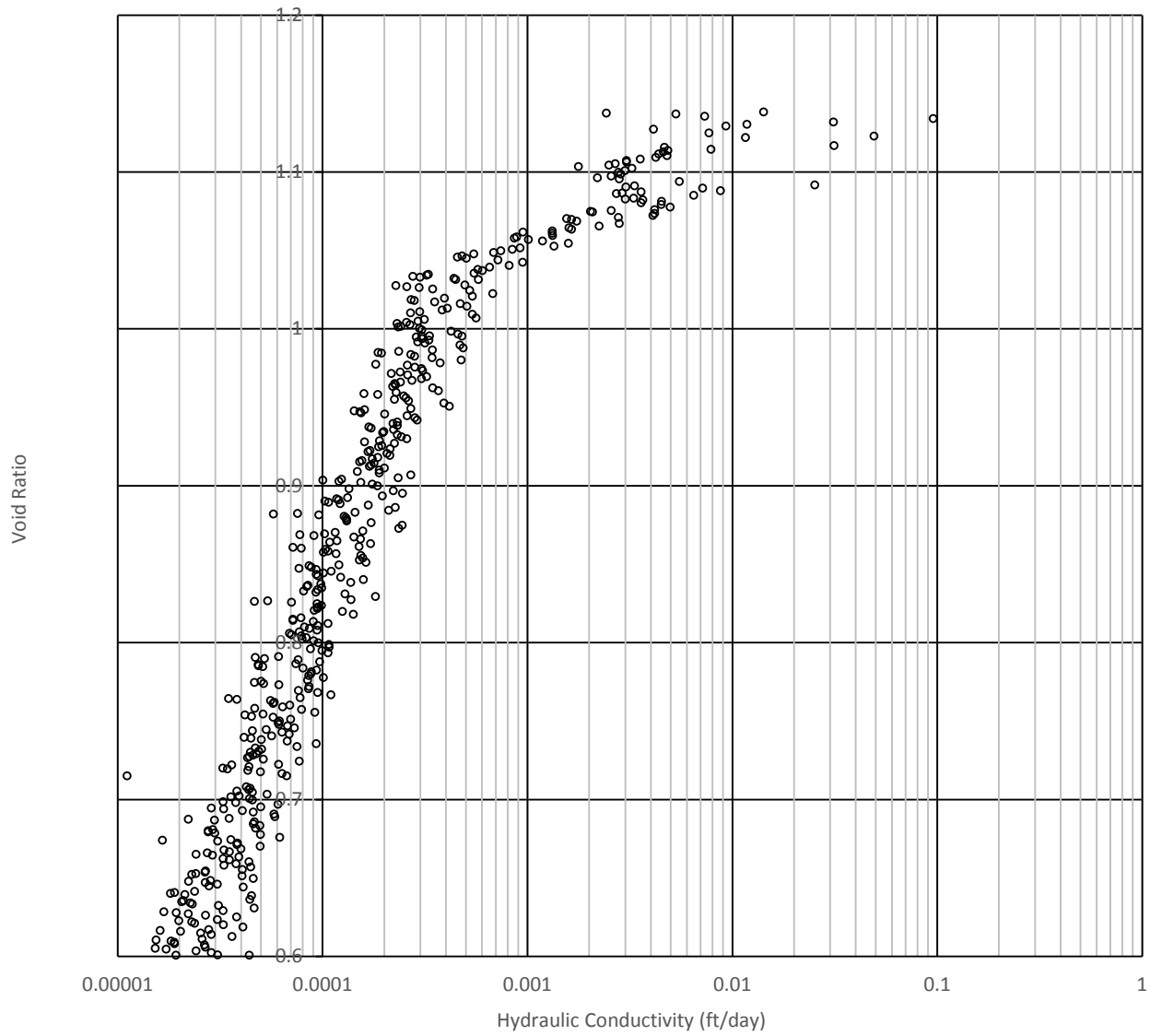
Figure 26

initials MM/DD/YY location\filename.xls



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
22.3	42	29	46	25	21	CLAY	CL

σ'_{vo}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1172.98	1000	1100	1050
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
D		Poor	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	5.00
Total Unit Weight (pcf)	110.03
Degree of Saturation (%)	97.28
Void Ratio (e0)	1.142

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

Hayward Baker

Void ratio versus logarithm of hydraulic conductivity
HB-106 T-1 CRS

19109-00 12/26/2014

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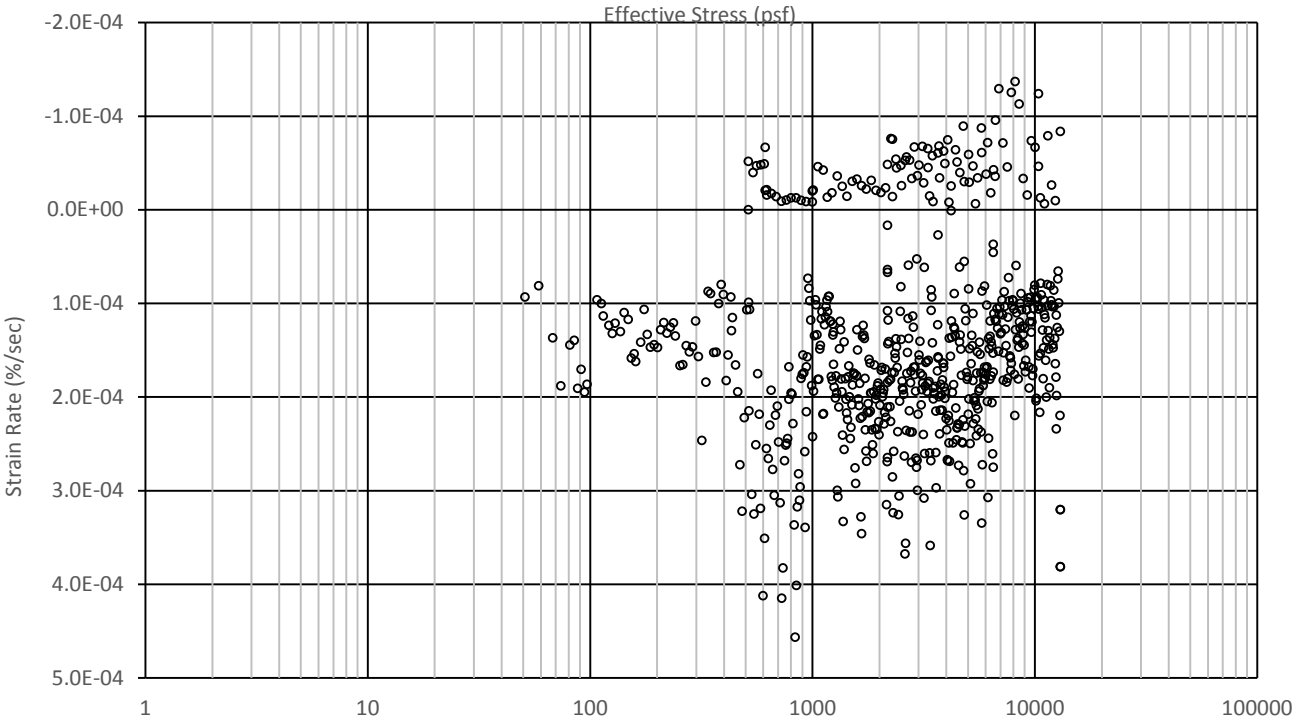
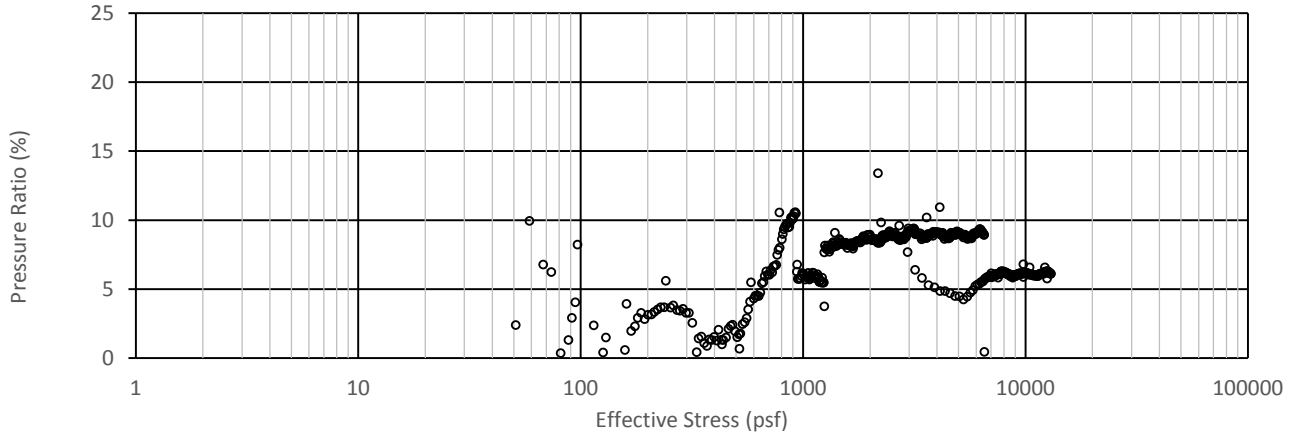
Figure
27

initials MM/DD/YY location\filename.xls





Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
22.3	42	29	46	25	21	CLAY	CL

σ'_{vo}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1172.98	1000	1100	1050
Sample Quality Designation			
Terzaghi et al. (1996)	Lunne et al. (1997)		
D	Poor		

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	5.00
Total Unit Weight (pcf)	110.03
Degree of Saturation (%)	97.28
Void Ratio (e0)	1.142

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

Hayward Baker

Axial strain, void ratio, and coefficient of consolidation versus logarithm of vertical effective stress for HB-106 T-1 CRS

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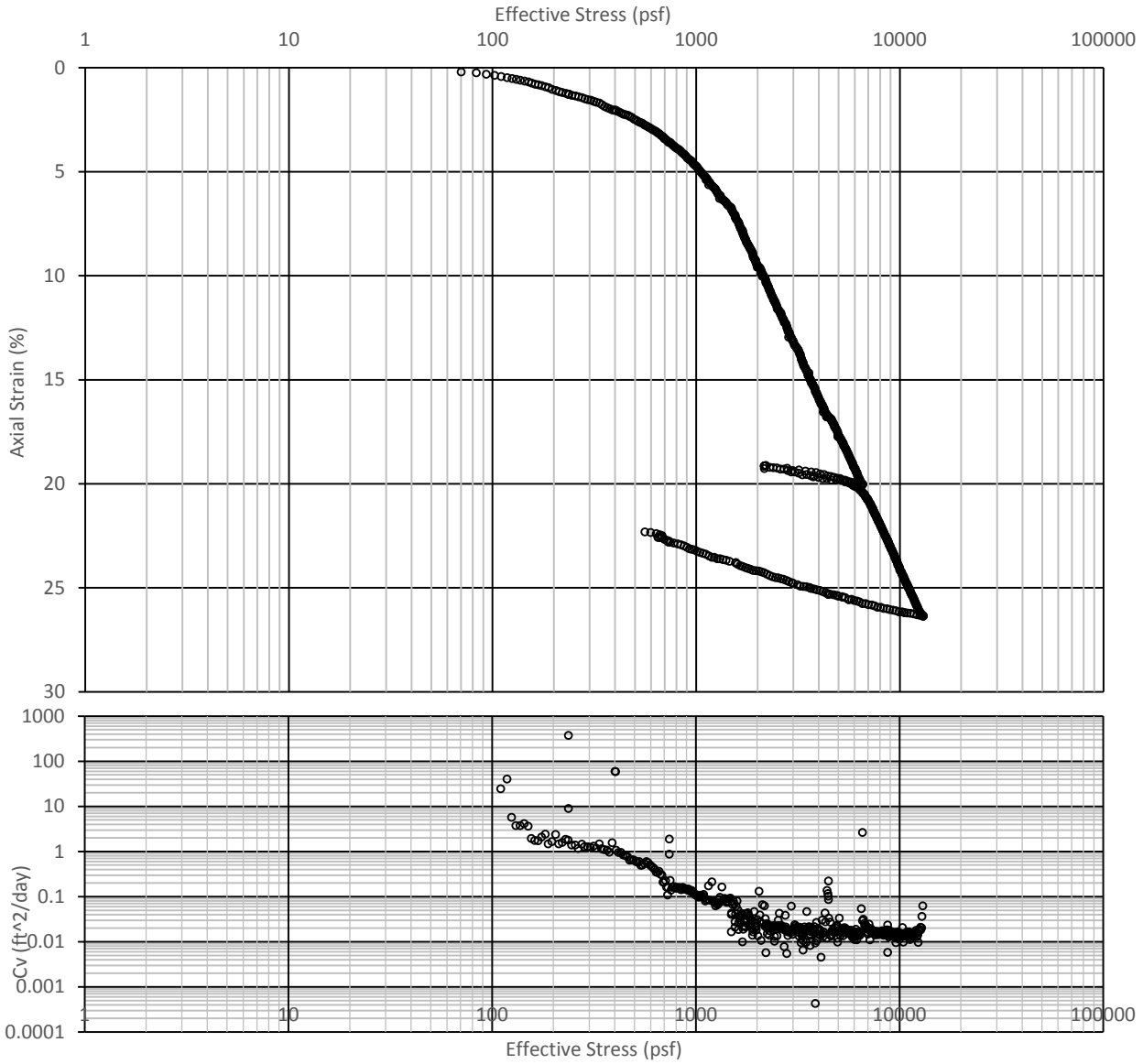
Figure **28**

initials MM/DD/YY location\filename.xls

CRS sample HB-B-106, T-1 (22.3 feet) in CRS ring before testing.

initials MM/DD/YY location\filename.xls

Hayward Baker	
HB-B-106 CRS Sample Photo	
19109-00	12/26/2014
HARTCROWSER	Figure **29**



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
25.1	45	32	63	25	38	CLAY	CH

σ'_{vo}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1100	1100	1300	1150
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
D		Poor	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	4.95
Total Unit Weight (pcf)	108.88
Degree of Saturation (%)	101.93
Void Ratio (e0)	1.136

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

Hayward Baker

Axial strain and coefficient of consolidation versus logarithm of vertical effective stress for HB-107 T-1 CRS

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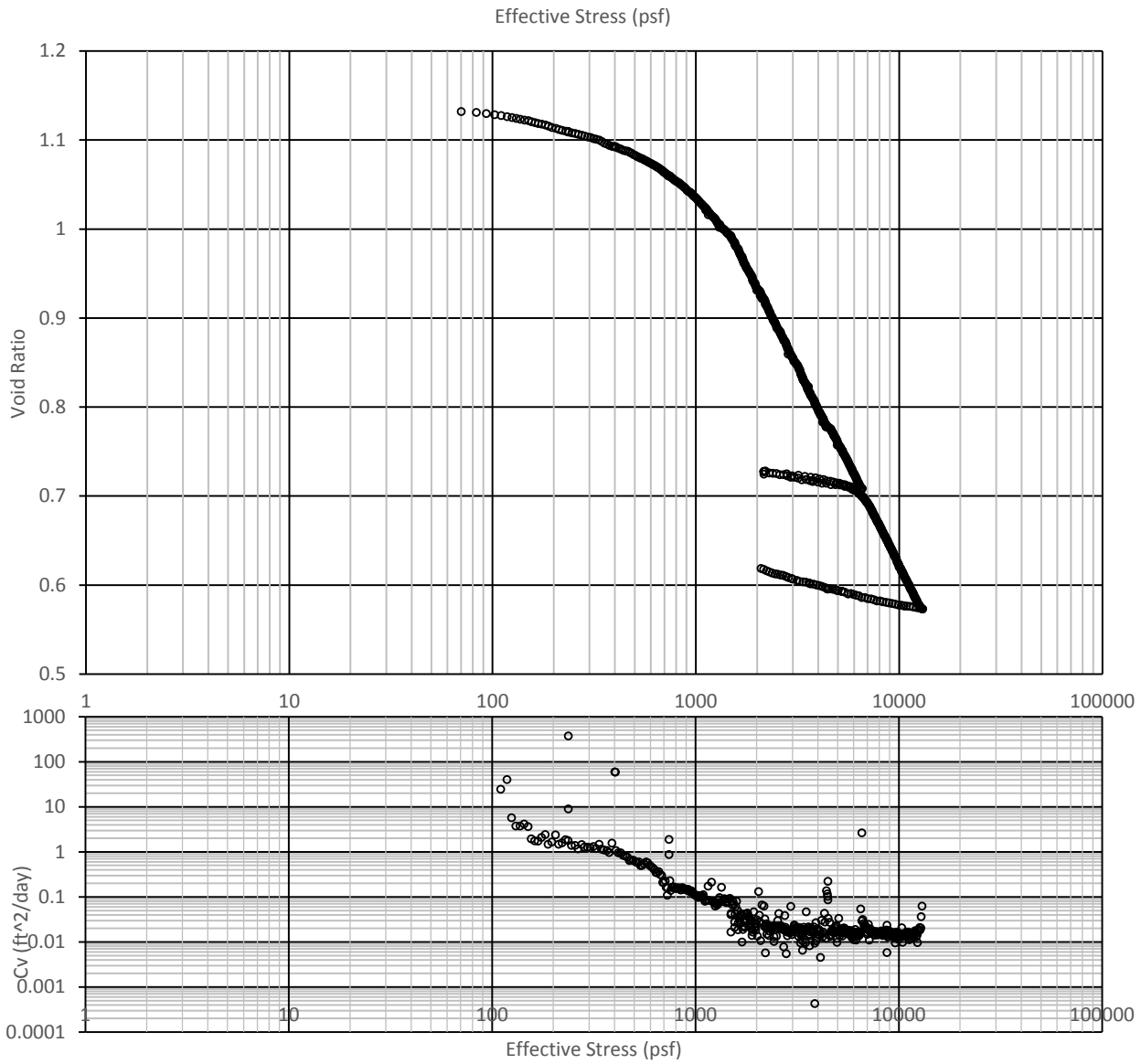
Figure **30A**

initials MM/DD/YY location\filename.xls



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
25.1	45	32	63	25	38	CLAY	CH

σ'_{v_o}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1100	1100	1300	1150
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
D		Poor	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	4.95
Total Unit Weight (pcf)	108.88
Degree of Saturation (%)	101.93
Void Ratio (e0)	1.136

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

Hayward Baker

Void ratio and coefficient of consolidation versus logarithm of vertical effective stress for HB-107 T-1 CRS

19109-00 12/11/2014

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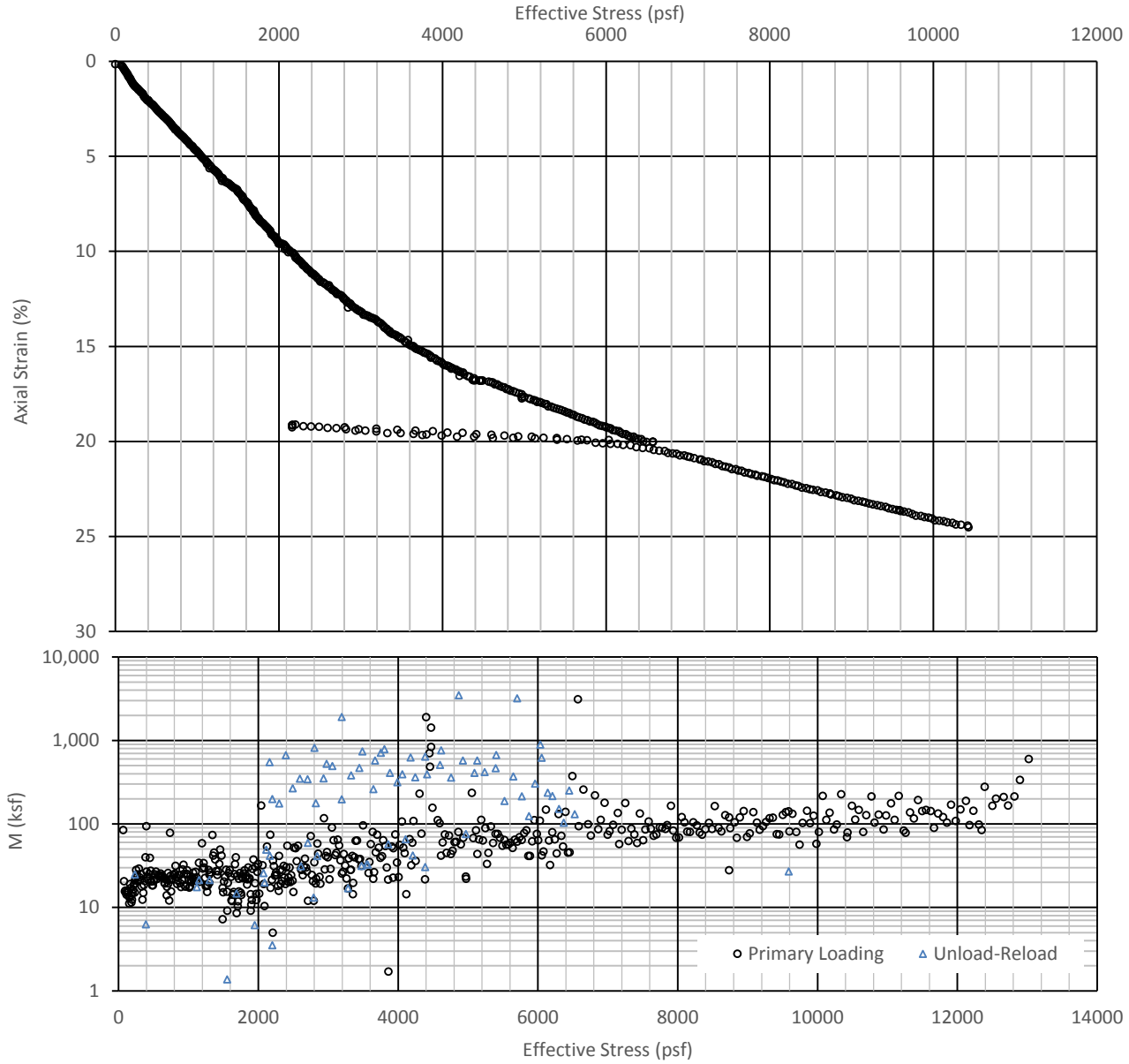
Figure **30B**

initials MM/DD/YY location\filename.xls



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
25.1	45	32	63	25	38	CLAY	CH

σ'_{vo}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1100	1100	1300	1150
Sample Quality Designation			
Terzaghi et al. (1996)	Lunne et al. (1997)		
D	Poor		

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	4.95
Total Unit Weight (pcf)	108.88
Degree of Saturation (%)	101.93
Void Ratio (e0)	1.136

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

Hayward Baker

Axial strain versus vertical effective stress for HB-107 T-1 CRS

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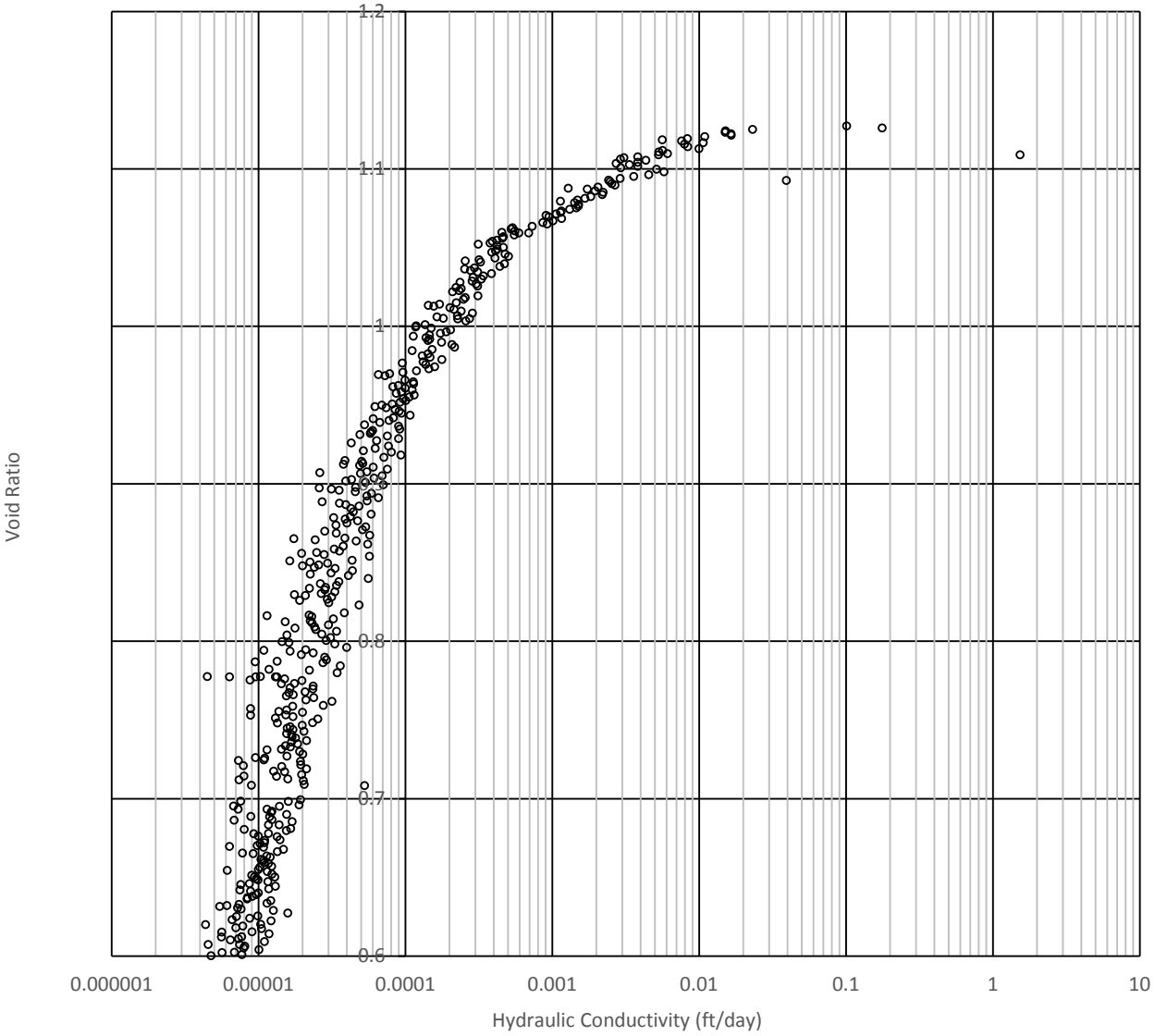
Figure 31

initials MM/DD/YY location\filename.xls



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
25.1	45	32	63	25	38	CLAY	CH

σ'_{vo}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1100	1100	1300	1150
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
D		Poor	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	4.95
Total Unit Weight (pcf)	108.88
Degree of Saturation (%)	101.93
Void Ratio (e0)	1.136

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

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Void ratio versus logarithm of hydraulic conductivity
HB-107 T-1 CRS

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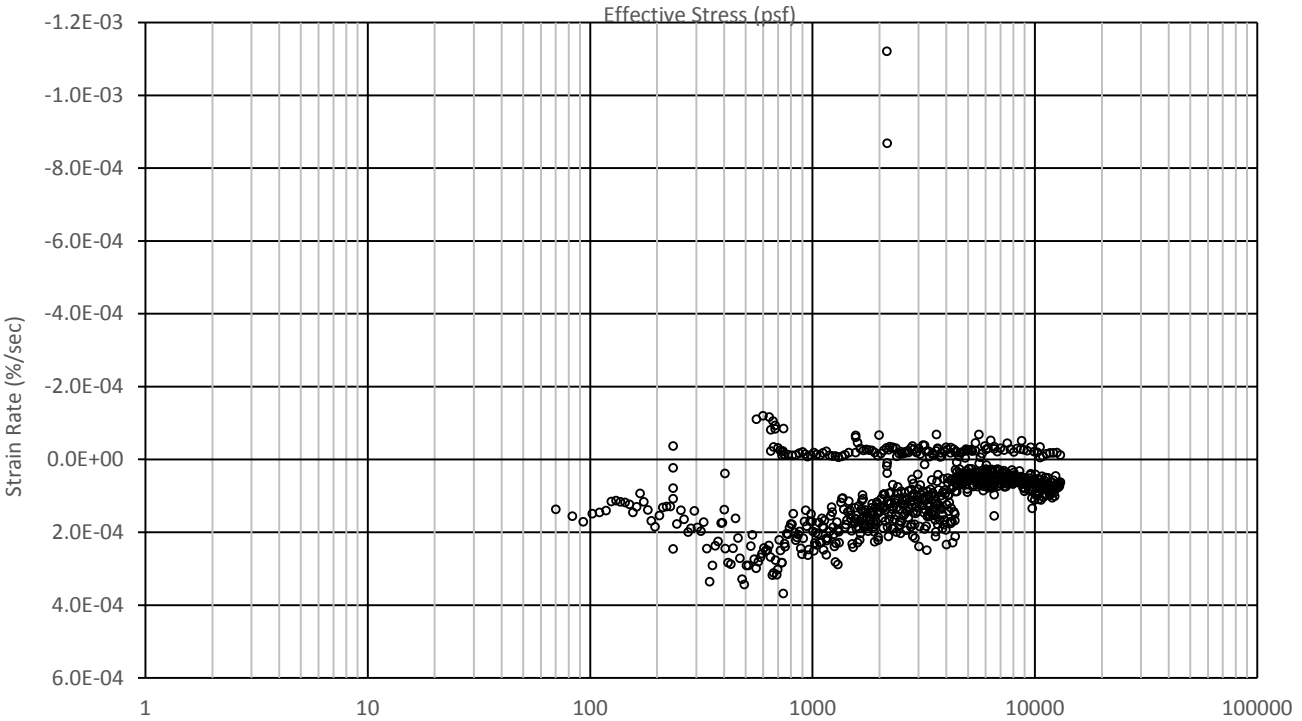
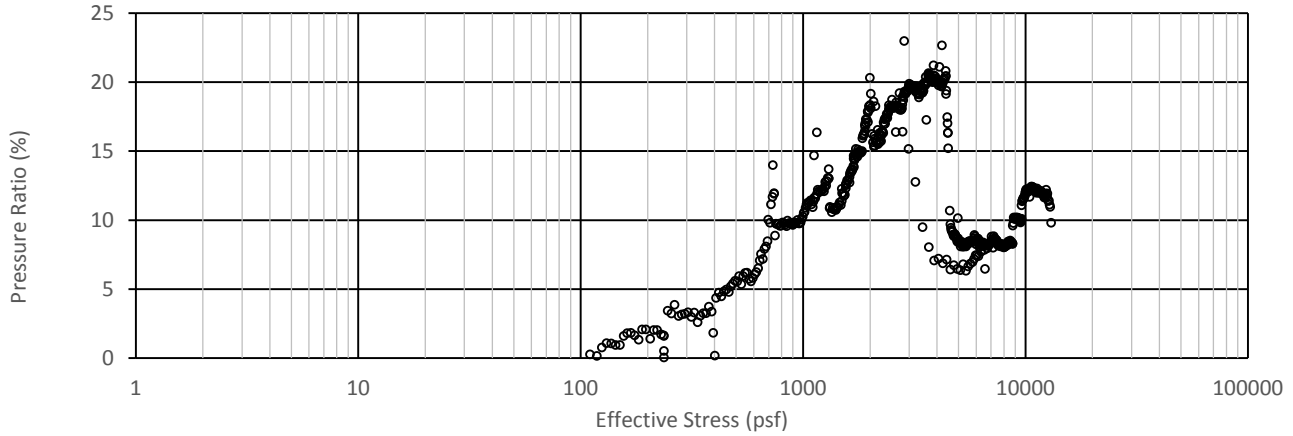
Figure
32

initials MM/DD/YY location\filename.xls





Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
25.1	45	32	63	25	38	CLAY	CH

σ'_{vo}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1100	1100	1300	1150
Sample Quality Designation			
Terzaghi et al. (1996)	Lunne et al. (1997)		
D	Poor		

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	4.95
Total Unit Weight (pcf)	108.88
Degree of Saturation (%)	101.93
Void Ratio (e0)	1.136

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

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Axial strain, void ratio, and coefficient of consolidation versus logarithm of vertical effective stress for HB-107 T-1 CRS

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Figure 33

initials MM/DD/YY location\filename.xls

CRS sample HB-B-107, T-1 (25.1 feet) in CRS ring before testing.

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HB-B-107 CRS Sample Photo

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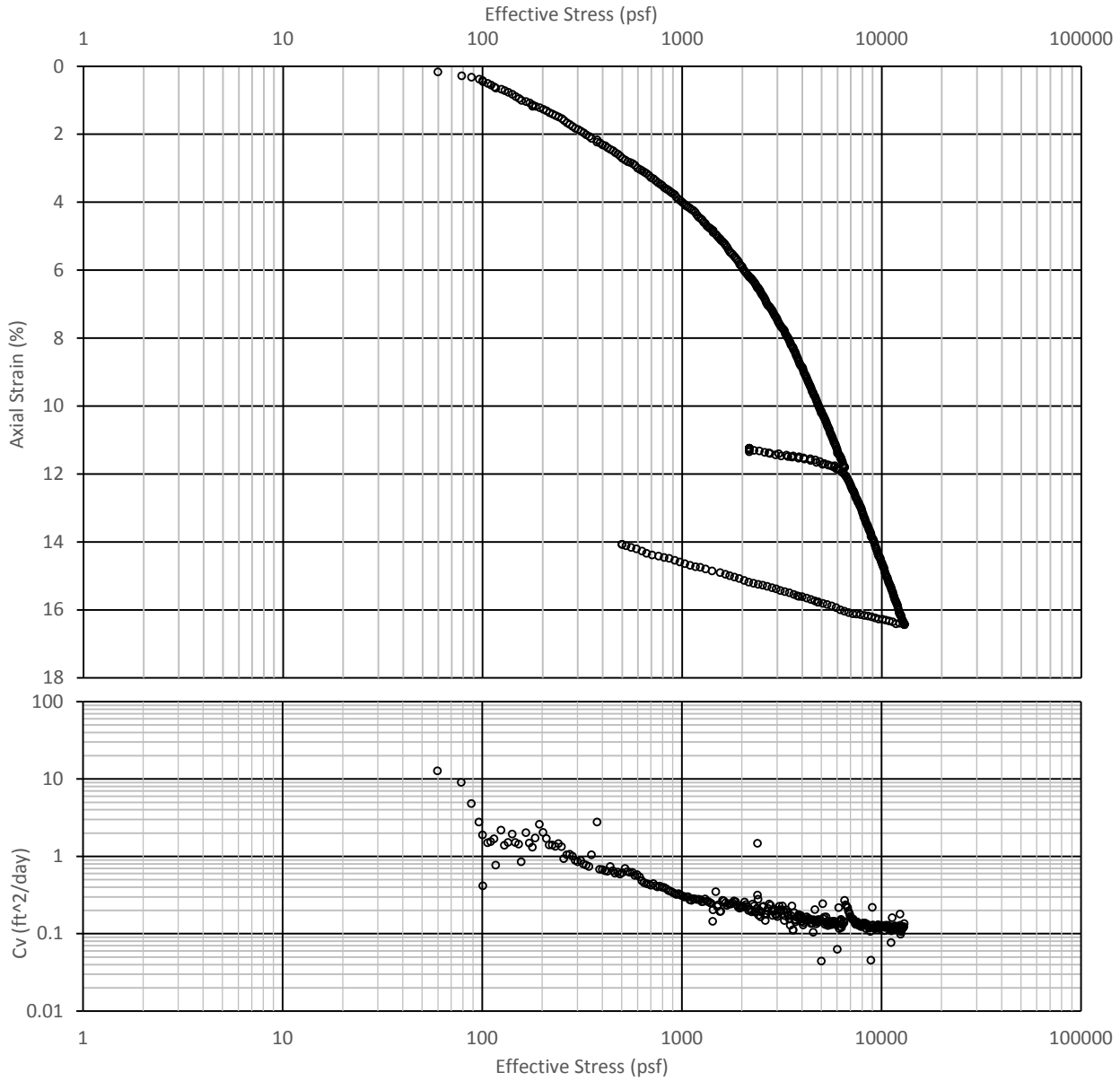
Figure

34

initials MM/DD/YY location\filename.xls

EX-0001-006968-PCE



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
21.7	30	23	48	27	21	CLAY	CL

σ'_{vo}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1150	2100	2550	2100
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
D		Poor	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	5.36
Total Unit Weight (pcf)	117.86
Degree of Saturation (%)	99.46
Void Ratio (e0)	0.778

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

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Axial strain and coefficient of consolidation versus logarithm of vertical effective stress for HB-108 T-1 CRS

19109-00 12/21/2014

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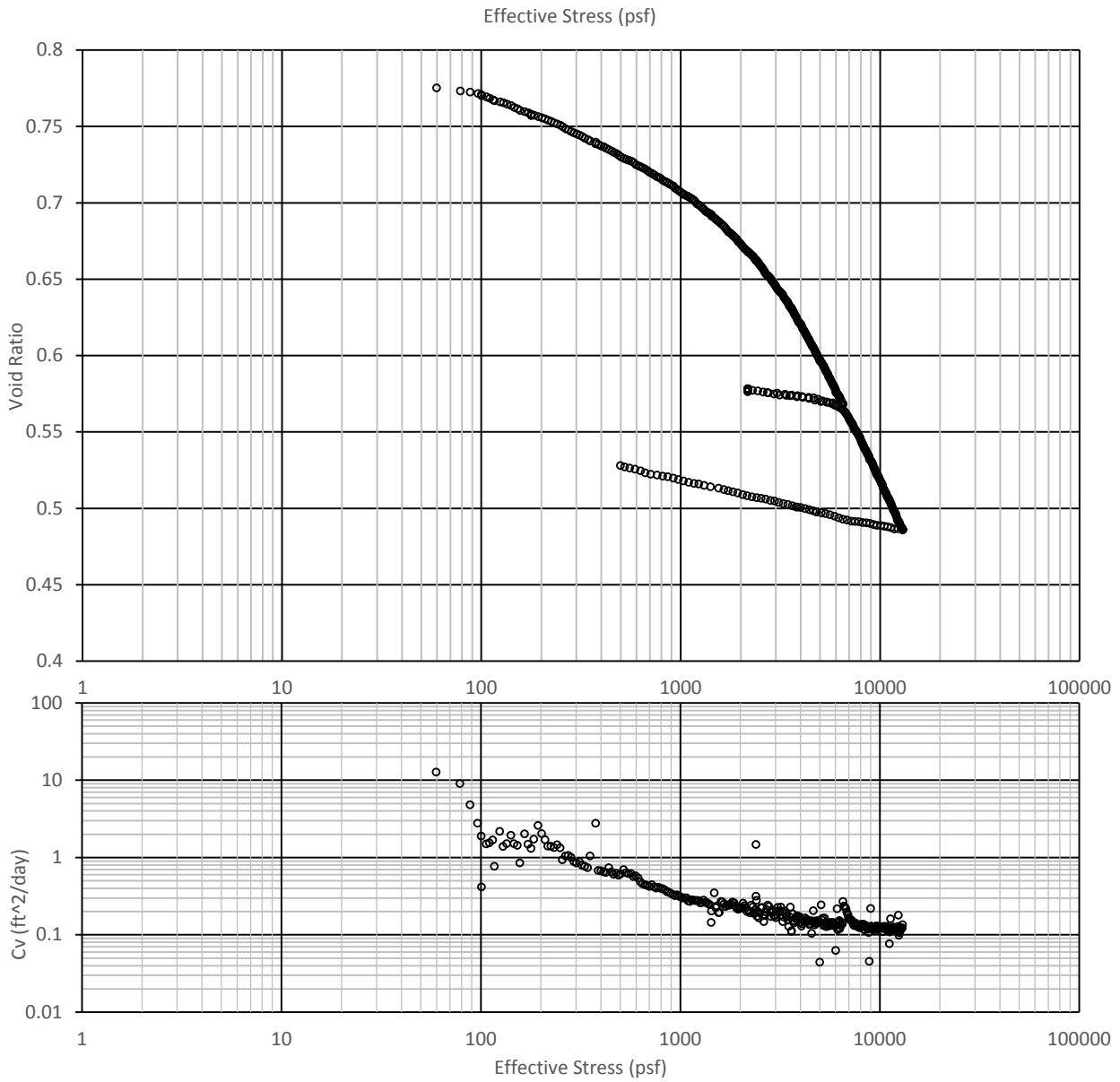
Figure **35A**

initials MM/DD/YY location\filename.xls



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
21.7	30	23	48	27	21	CLAY	CL

$\sigma'v_o$	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1150	2100	2550	2100
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
D		Poor	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	5.36
Total Unit Weight (pcf)	117.86
Degree of Saturation (%)	99.46
Void Ratio (e0)	0.778

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

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Void ratio and coefficient of consolidation versus logarithm of vertical effective stress for HB-108 T-1 CRS

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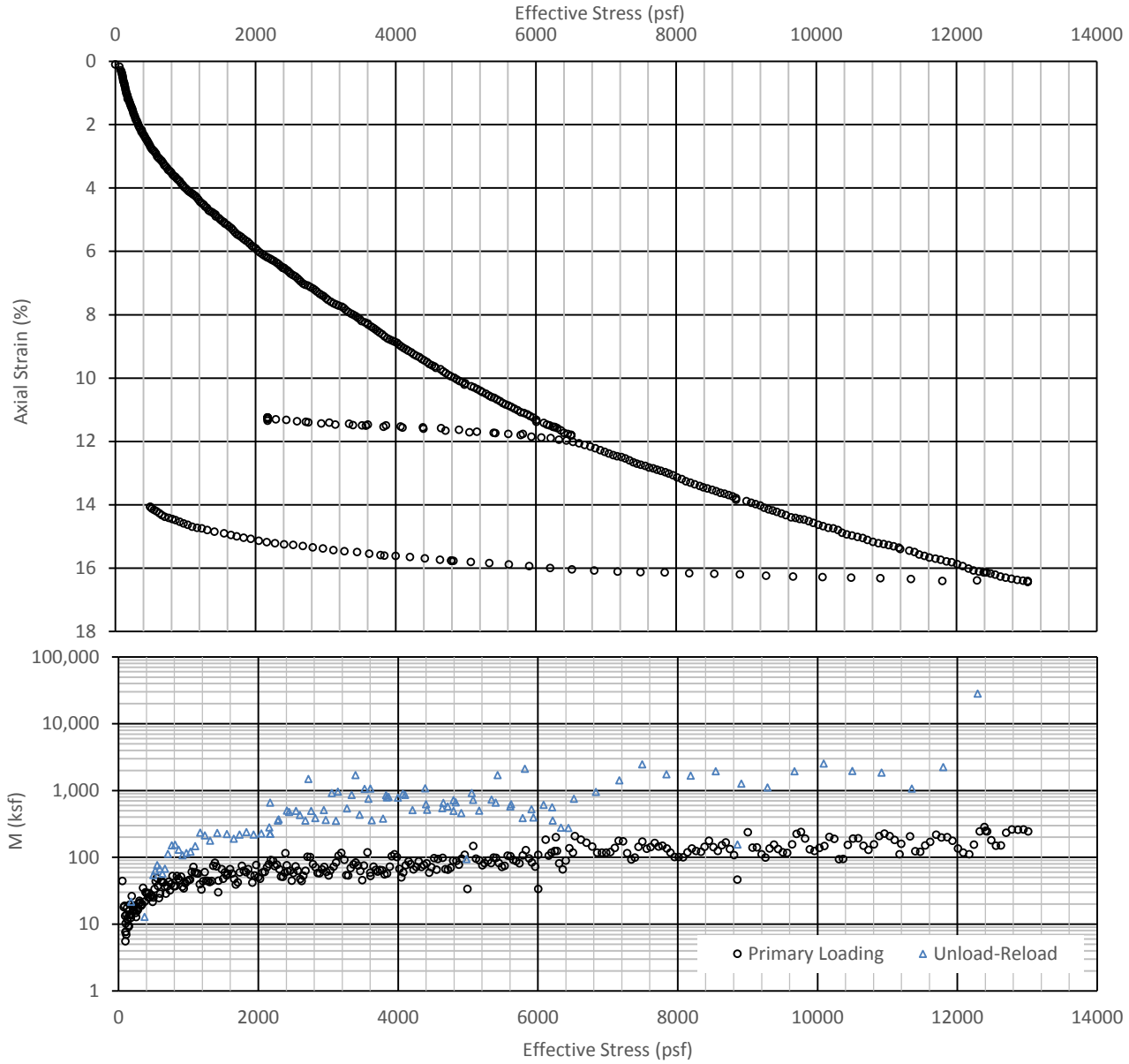
Figure **35B**

initials MM/DD/YY location\filename.xls

EX-0001-006970-PCE



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
21.7	30	23	48	27	21	CLAY	CL

σ'_{vo}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1150	2100	2550	2100
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
D		Poor	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	5.36
Total Unit Weight (pcf)	117.86
Degree of Saturation (%)	99.46
Void Ratio (e0)	0.778

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

Hayward Baker

Axial strain versus vertical effective stress for HB-108 T-1 CRS

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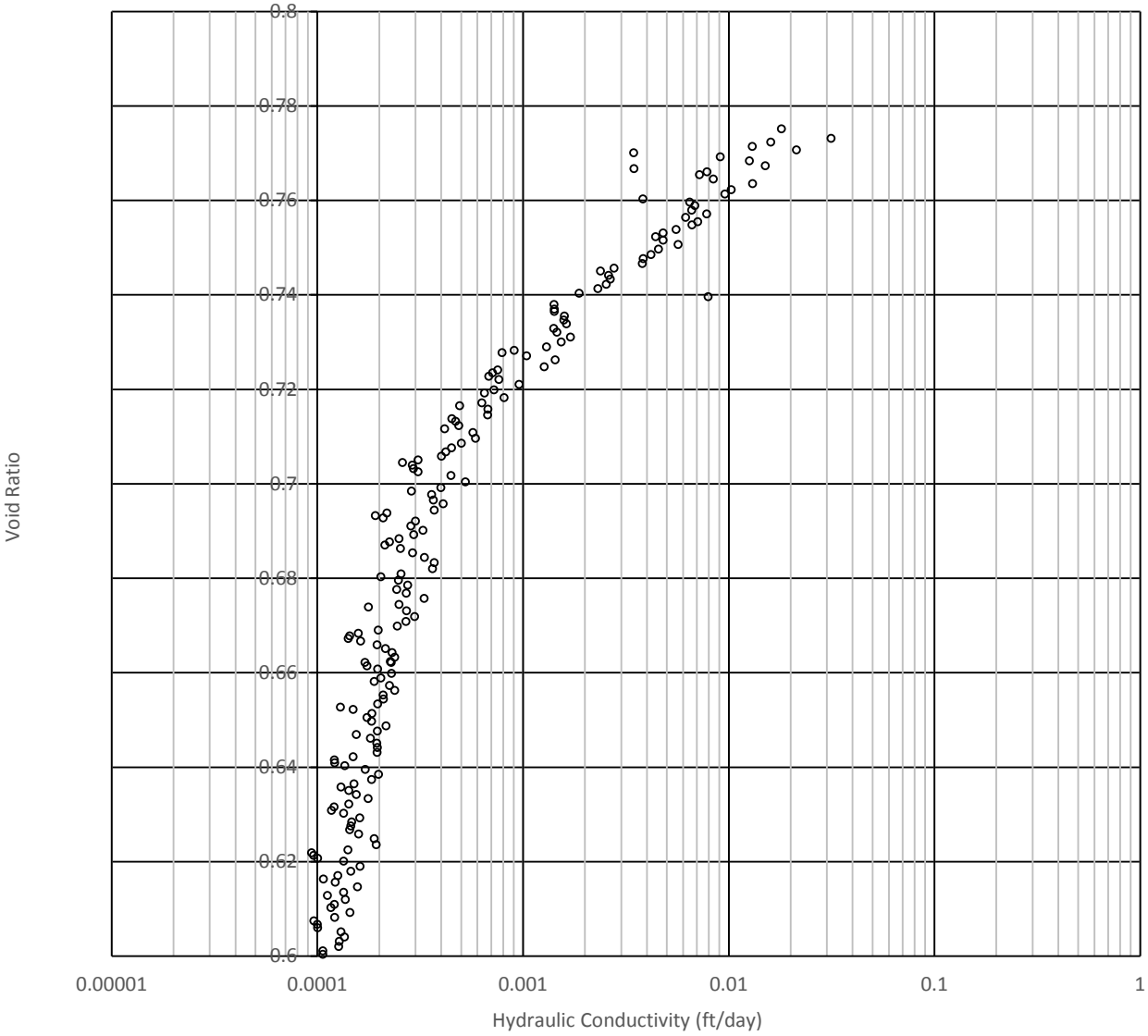
Figure 36

initials MM/DD/YY location\filename.xls



Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
21.7	30	23	48	27	21	CLAY	CL

σ'_{vo}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1150	2100	2550	2100
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
D		Poor	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	5.36
Total Unit Weight (pcf)	117.86
Degree of Saturation (%)	99.46
Void Ratio (e0)	0.778

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

Hayward Baker

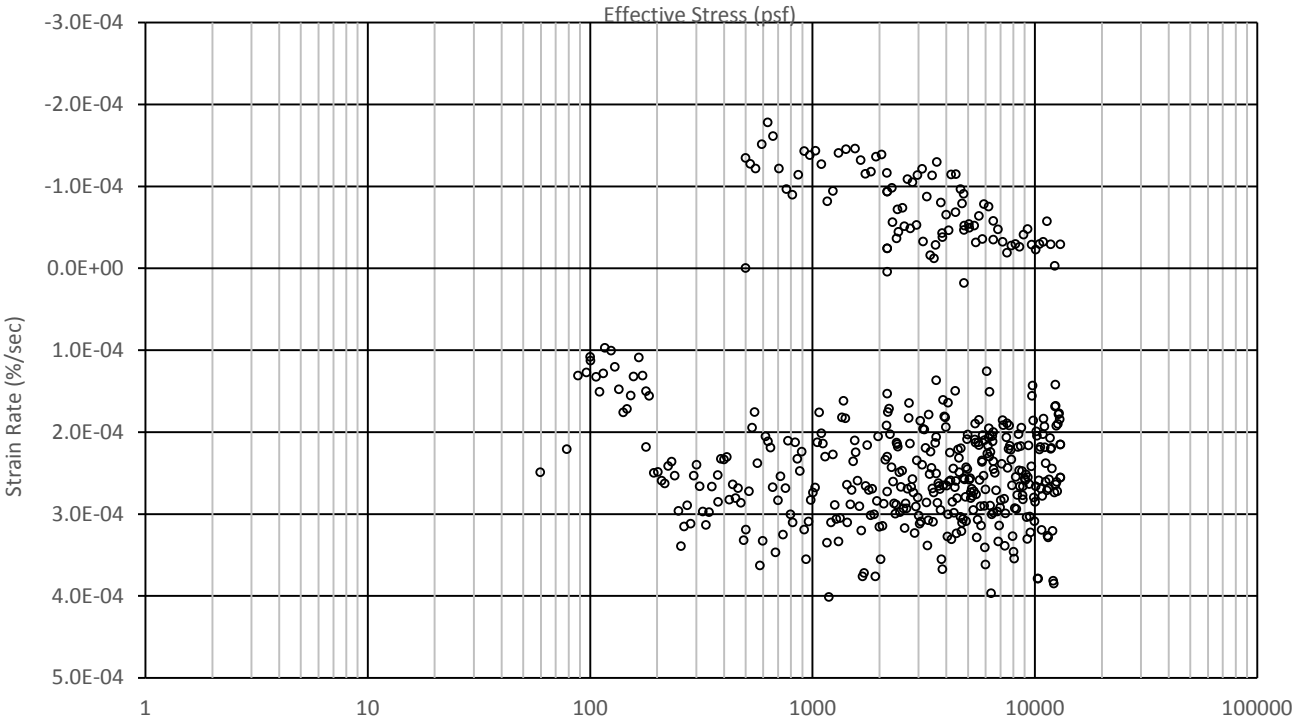
Void ratio versus logarithm of hydraulic conductivity
HB-108 T-1 CRS

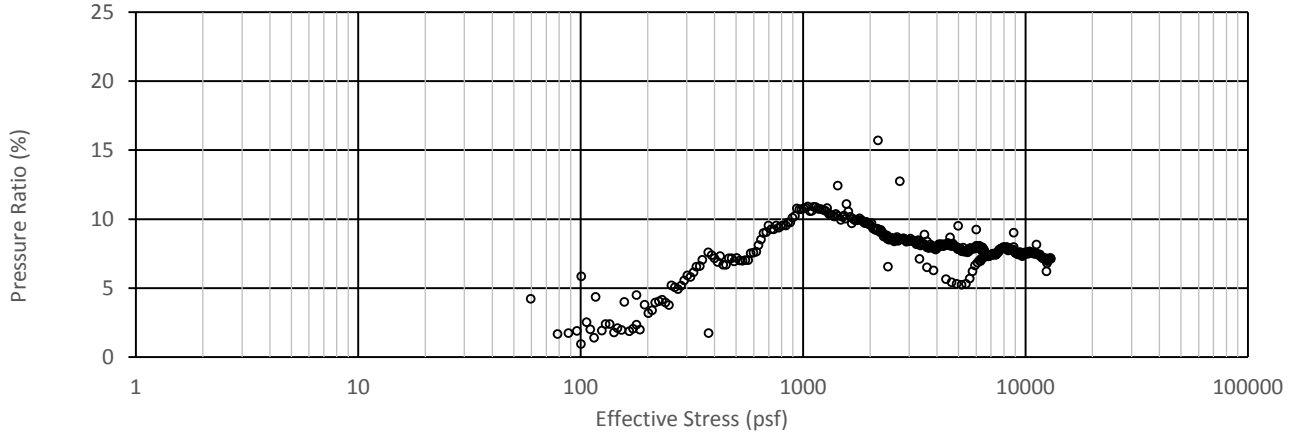
19109-00 12/21/2014

HARTCROWSER

Figure
37

initials MM/DD/YY location\filename.xls





Depth	W.C. (%)		Atterberg Limits			Description	USCS
(ft)	Before	After	LL	PL	PI		
21.7	30	23	48	27	21	CLAY	CL

σ'_{v_o}	Preconsolidation Pressure (psf)		
(psf)	Strain Energy Min/Max		Casagrande
1150	2100	2550	2100
Sample Quality Designation			
Terzaghi et al. (1996)		Lunne et al. (1997)	
D		Poor	

Initial Specimen Properties	
Height (inches)	1.00
Diameter (inches)	2.50
Weight (ounces)	5.36
Total Unit Weight (pcf)	117.86
Degree of Saturation (%)	99.46
Void Ratio (e0)	0.778

Sample Preparation and Comments:
The test specimen was an intact soil sample that was extracted from the sampling tube by cutting and delaminating a section of the sample tube. The test was run at a room temperature between 73 and 76 degrees Fahrenheit.

Hayward Baker

Axial strain, void ratio, and coefficient of consolidation versus logarithm of vertical effective stress for HB-108 T-1 CRS

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Figure 38

initials MM/DD/YY location\filename.xls

Appendix E

Area 300 Design Calculations

Liquefaction Analysis
Settlement Analysis
Bearing Edge and Base Stability

DRAFT

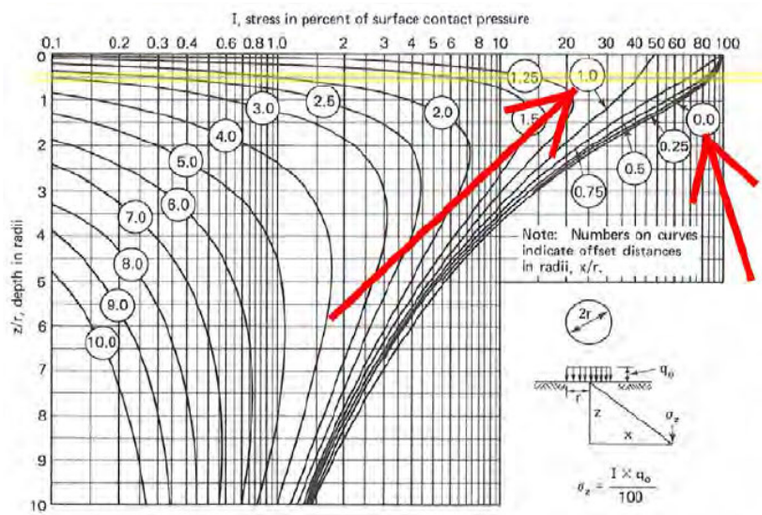
Appendix E
Area 300 Calculations

Tank 1

Design Summary
Tank 1

This table assumes the exploration is directly below the center of the tank.

Tank 1		Orig	Imp.	Improved		Original	
Soil Profile	Depth	Liq Sett.	Liq Sett.	Rapid Sett.	Long Term Sett.	Rapid Sett.	Long Term Sett.
HB-CPT-111	56.8-ft	2.87-in.	1.14-in.	1.96-in.	.-in.	.88-in.	3.93-in.
HB-CPT-112	56.8-ft	1.96-in.	.54-in.	1.95-in.	.07-in.	.85-in.	3.9-in.
HB-CPT-113	59.2-ft	2.65-in.	1.42-in.	3.-in.	.08-in.	1.14-in.	5.99-in.
HB-CPT-115	55.0-ft	3.79-in.	1.92-in.	1.48-in.	.02-in.	1.44-in.	.02-in.
HB-CPT-114(S)B	55.3-ft	2.66-in.	1.49-in.	2.26-in.	.05-in.	1.34-in.	4.49-in.
X-CPT-3	54.8-ft	2.99-in.	1.51-in.	1.93-in.	.09-in.	1.64-in.	3.93-in.
Average:		**2.82-in.**	**1.31-in.**	**2.1-in.**	**.05-in.**	**1.21-in.**	**3.71-in.**



This table calculates Settlement at the edge of the tank.

Tank 1		Orig	Imp.	Improved		Original	
Soil Profile	Depth	Liq Sett.	Liq Sett.	Rapid Sett.	Long Term Sett.	Rapid Sett.	Long Term Sett.
HB-CPT-111	56.8-ft	2.87-in.	1.14-in.	.98-in.	.-in.	.44-in.	1.96-in.
HB-CPT-112	56.8-ft	1.96-in.	.54-in.	.98-in.	.03-in.	.42-in.	1.95-in.
HB-CPT-113	59.2-ft	2.65-in.	1.42-in.	1.5-in.	.04-in.	.57-in.	3.-in.
HB-CPT-115	55.0-ft	3.79-in.	1.92-in.	.74-in.	.01-in.	.72-in.	.01-in.
HB-CPT-114(S)B	55.3-ft	2.66-in.	1.49-in.	2.26-in.	.05-in.	1.34-in.	4.49-in.
X-CPT-3	54.8-ft	2.99-in.	1.51-in.	.97-in.	.04-in.	.82-in.	1.96-in.
Average:		**2.82-in.**	**1.26-in.**	**1.05-in.**	**.02-in.**	**.54-in.**	**1.73-in.**

Note: No reduction to total soil settlement is taken for the liquefaction condition.

Hayward Baker Inc. THINK SAFE	SUBJECT	Tesoro Tank 1			
	Job no.	Bid Design	Design by:	M. Rohrbach	Date: 23-Jan-15
	File	CPT-Liquefaction-Boulanger	Checked		
	Spreadsheet Rev.	23-Dec-14	Reviewed		

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-111

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	17.5-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	29.5	ft.
Elevation of bottom of Soil Profile	-27.26	ft.
Elevation of Installation	25	Work from Existing Grade
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	56.8-ft	
Density of Stone	125	pcf
Poisson's Ra io for Soil, v_s	0.333	
Poisson's Ra io for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	292.50-psf	

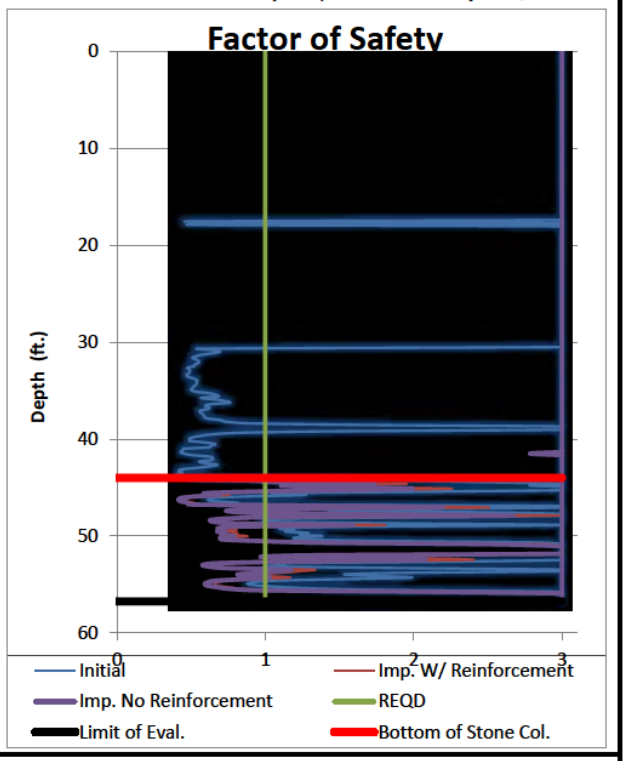
Stone Column Design Information

L = 44-ft			ϕ_c = 50
Column Spacing 8-ft			Ec = 750.0-tsf
Kac = 0.1324743	$=TAN^2(45-(\phi_c/2)$		Gc = 277.8-tsf

SANDY Soils 11% Replacment	SILTY soils: 22% Replacment
SAND: Ar = (Ac/A) 0.11	SILTY SOIL: Ar = (Ac/A) 0.22
Stone Column Diameter 3-ft	Stone Column Diameter 4-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 56.8-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 14.8-ft.

Calculated Min: 2.70-in via T&S, Min never < 0.5 in.

Calculated Max: 2.87-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 7.2-ft.

Calculated Min: 1.30-in via T&S, Min never < 0.5 in.

Calculated Max: 1.39-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 6.2-ft.

Calculated Min: 1.14-in via T&S, Min never < 0.5 in.

Calculated Max: 1.27-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Tank 1 **IMPROVED CASE: CPT-111**

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e = C_1 C_2 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment

= $1-0.5*(q/(q_{bar}-q)$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

q = gDf

Stone Column Length 44-ft

Reduction factor on Priebe: 1

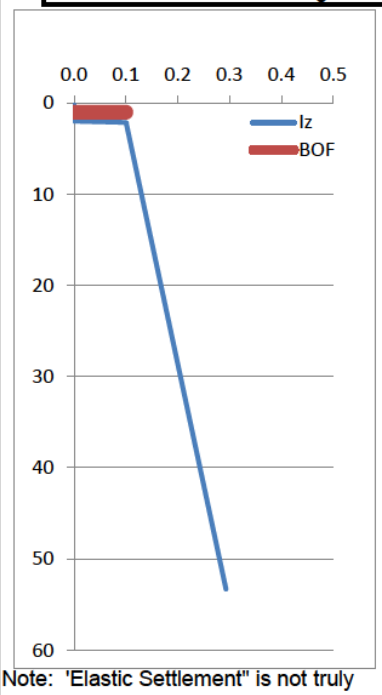
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar}	2882.00-psf	Per GRI page 14
L	212.7-ft	Equivalent Area for 240' Ø tank
B	212.7-ft	Reference GRI page 2 and B=L
fill density	125-pcf	Assigned by Designer
Elevation of Top of Soil Profile	29.5	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and qc: Es=X·qc

x = 3.0

*default Value of X is 3



Note: "Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	250.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.953	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)

I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=0.5B$=	106.3
I_z = 0	at depth	$z=z_2=2.0B$=	425.4

For foundations with L/B ≥ 10

I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=1.0B$=	212.7
I_z = 0	at depth	$z=z_2=4.0B$=	850.8

Interpolation for L/B values between 1 and 10

I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

Σ(Iz/Es)Δz (ft2/lb)= 8.39E-06

Total Elastic Settlement (Flexible foundation)

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0324-ft 0.20-in

Total Rapid Elastic Settlement

$S_e=C_1(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0210-ft 0.13-in

Long Term Settlement 0.07-in

Calculated Consolidation Type Settlement:

Total:	1.77-in	Adjusted for footing Type
Recomp.	0.07-in	Adjusted for footing Type
Virgin Sett.	1.69-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	1.84-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:

Total: 1.96-in

Rapid: 1.96-in

Long Term: 0.00-in

Hayward Baker Inc. THINK SAFE	SUBJECT	Tesoro Tank 1			
	Job no.	Bid Design	Design by:	M. Rohrbach	Date: 23-Jan-15
	File	CPT-Liquefaction-Boulanger	Checked		
	Spreadsheet Rev.	23-Dec-14	Reviewed		

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-112

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	17.4-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	29.39	ft.
Elevation of bottom of Soil Profile	-27.37	ft.
Elevation of Installation	25	Work from Existing Grade
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	56.8-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	306.80-psf	

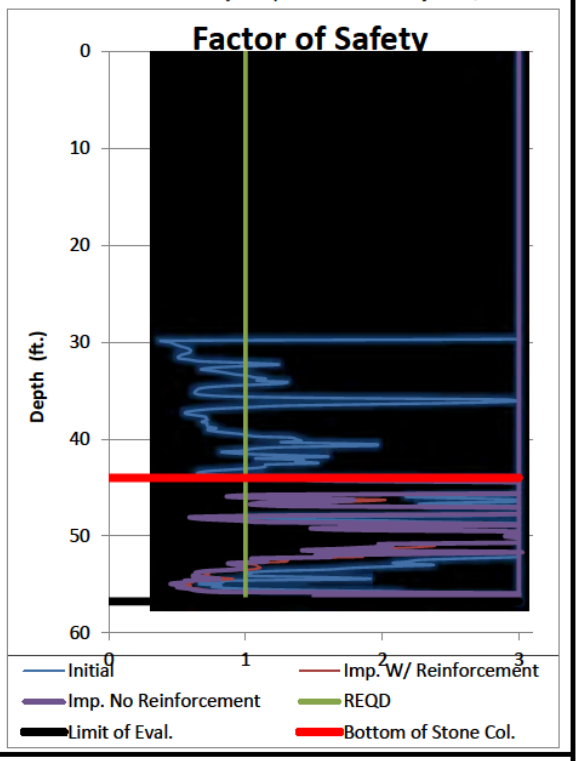
Stone Column Design Information

L =	44-ft		ϕ_c = 50
Column Spacing	8-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 22% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.22
Stone Column Diameter	3-ft	Stone Column Diameter	4-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 56.8-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 9.7-ft.

Calculated Min: 1.72-in via T&S, Min never < 0.5 in.

Calculated Max: 1.96-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 3.8-ft.

Calculated Min: 0.67-in via T&S, Min never < 0.5 in.

Calculated Max: 0.74-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 2.9-ft.

Calculated Min: 0.54-in via T&S, Min never < 0.5 in.

Calculated Max: 0.64-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Tank 1

IMPROVED CASE: CPT-112

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$$S_e = C_1 C_2 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$$

C_1 = a correction factor for the depth of foundation embedment

= 1-0.5*(q/(q_{bar}-q)

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

q = γDf

Stone Column Length 44-ft

Reduction factor on Priebe: 1

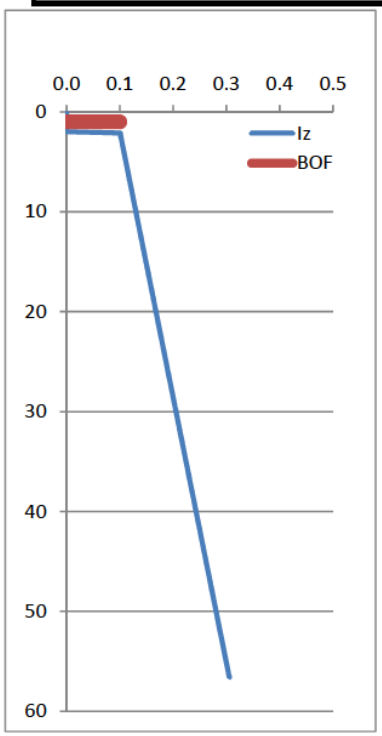
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar}	2882.00-psf	Per GRI page 14
L	212.7-ft	Equivalent Area for 240' Ø tank
B	212.7-ft	Reference GRI page 2 and B=L
fill density	125-pcf	Assigned by Designer
Elevation of Top of Soil Profile	29.39	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and qc: Es=X·qc

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	250.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.953	=1-0.5[q/(q_{bar}-q)]
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1$=0.5B=	106.3
I_z = 0	at depth	$z=z_2$=2.0B=	425.4
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1$=1.0B=	212.7
I_z = 0	at depth	$z=z_2$=4.0B=	850.8
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

Σ(Iz/Es)Δz (ft2/lb)= 8.15E-06

Total Elastic Settlement (Flexible foundation)

$S_e = C_1 C_2 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$= 0.0315-ft 0.19-in

Total Rapid Elastic Settlement

$S_e = C_1 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$= 0.0204-ft 0.13-in

Long Term Settlement 0.07-in

Calculated Consolidation Type Settlement:

Total:	1.76-in	Adjusted for footing Type
Recomp.	0.07-in	Adjusted for footing Type
Virgin Sett.	1.69-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	1.83-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:

Total: 1.95-in

Rapid: 1.95-in

Long Term: -0.01-in

Hayward Baker Inc. THINK SAFE	SUBJECT	Tesoro Tank 1		
	Job no.	Bid Design	Design by: M. Rohrbach	Date: 23-Jan-15
	File	CPT-Liquefaction-Boulanger	Checked	
	Spreadsheet Rev.	23-Dec-14	Reviewed	

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-113

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	16.9-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	28.93	ft.
Elevation of bottom of Soil Profile	-30.29	ft.
Elevation of Installation	25	Work from Existing Grade
Bottom of Tank Eleva ion	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	59.2-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	366.60-psf	

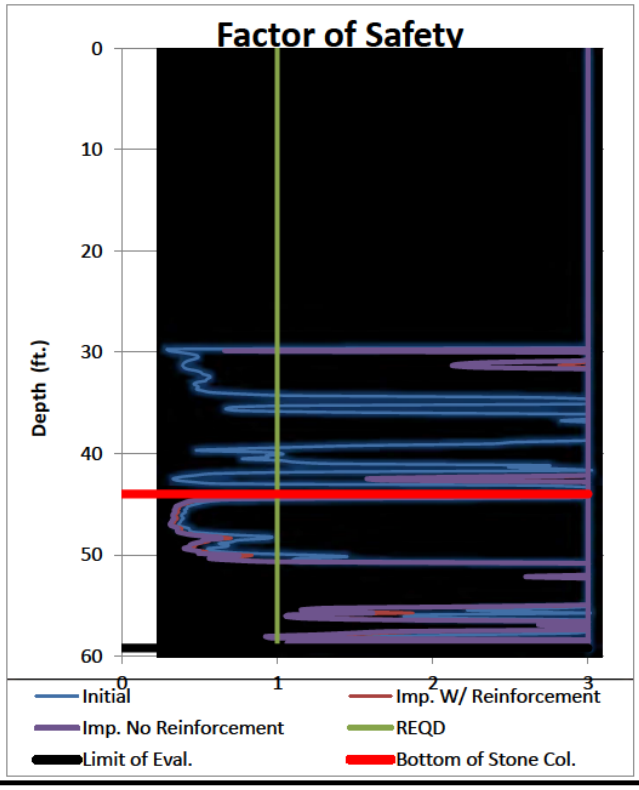
Stone Column Design Information

L = 44-ft			ϕ_c = 50
Column Spacing 8-ft			Ec = 750.0-tsf
Kac = 0.1324743	$=TAN^2(45-(\phi_c/2)$		Gc = 277.8-tsf

SANDY Soils 11% Replacment	SILTY soils: 22% Replacment
SAND: Ar = (Ac/A) 0.11	SILTY SOIL: Ar = (Ac/A) 0 22
Stone Column Diameter 3-ft	Stone Column Diameter 4-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 59.2-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 12.6-ft.

Calculated Min: 2.53-in via T&S, Min never < 0.5 in.

Calculated Max: 2.65-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 6.9-ft.

Calculated Min: 1.50-in via T&S, Min never < 0.5 in.

Calculated Max: 1.56-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 6.4-ft.

Calculated Min: 1.42-in via T&S, Min never < 0.5 in.

Calculated Max: 1.48-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if he calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Tank 1

IMPROVED CASE: CPT-113

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$$S_e = C_1 C_2 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$$

C_1 = a correction factor for the depth of foundation embedment

= $1-0.5*(q/(q_{bar}-q))$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

$q = \gamma Df$

Stone Column Length 44-ft

Reduction factor on Priebe: 1

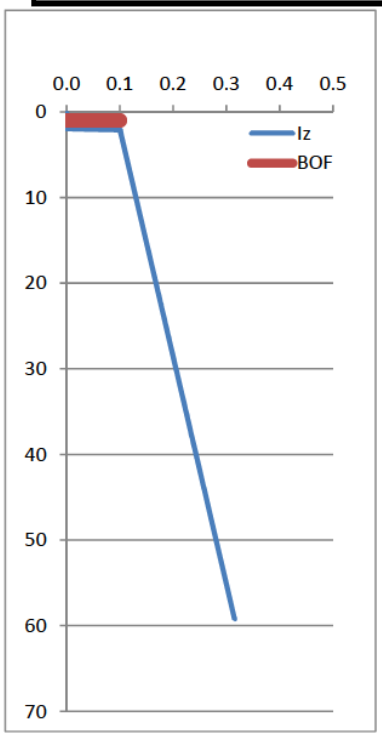
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar}	2882.00-psf	Per GRI page 14
L	212.7-ft	Equivalent Area for 240' ø tank
B	212.7-ft	Reference GRI page 2 and B=L
fill density	125-pcf	Assigned by Designer
Elevation of Top of Soil Profile	28.93	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and qc: Es=X·qc

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	250.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.953	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=0.5B$=	106.3
I_z = 0	at depth	$z=z_2=2.0B$=	425.4
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=1.0B$=	212.7
I_z = 0	at depth	$z=z_2=4.0B$=	850.8
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

$\Sigma(I_z/E_s)\Delta z$ (ft2/lb)= 9.55E-06

Total Elastic Settlement (Flexible foundation)

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0369-ft 0.22-in

Total Rapid Elastic Settlement

$S_e=C_1(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0239-ft 0.15-in

Long Term Settlement 0.08-in

Calculated Consolidation Type Settlement:

Total:	2.74-in	Adjusted for footing Type
Recomp.	0.11-in	Adjusted for footing Type
Virgin Sett.	2.63-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	2.85-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:

Total: 2.96-in

Rapid: 3.00-in

Long Term: -0.04-in

Hayward Baker Inc. THINK SAFE	SUBJEC	Tesoro Tank 1			
	Job no.	Bid Design	Design by:	M. Rohrbach	Date: 23-Jan-15
	File	CPT-Liquefaction-Boulanger	Checked		
	Spreadsheet Rev.	23-Dec-14	Reviewed		

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-115

Project Specific Information

Ear hquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	17.2-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	29.17	ft.
Elevation of bottom of Soil Profile	-25.78	ft.
Elevation of Installation	25	Work from Existing Grade
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	55.0-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (nega ive for cut)	335.40-psf	

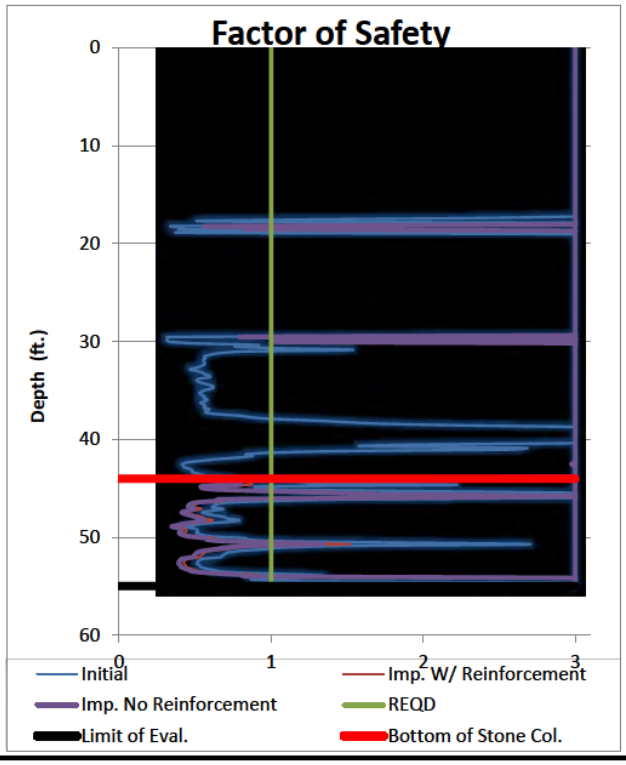
Stone Column Design Information

L =	44-ft		ϕ_c = 50
Column Spacing	8-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 22% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.22
Stone Column Diameter	3-ft	Stone Column Diameter	4-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 55.0-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 18.7-ft.

Calculated Min: 3.64-in via T&S, Min never < 0.5 in.

Calculated Max: 3.79-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 9.5-ft.

Calculated Min: 2.00-in via T&S, Min never < 0.5 in.

Calculated Max: 2.04-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 9.2-ft.

Calculated Min: 1.92-in via T&S, Min never < 0.5 in.

Calculated Max: 1.97-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if he calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Tank 1

IMPROVED CASE: CPT-115

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$$S_e = C_1 C_2 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$$

C_1 = a correction factor for the depth of foundation embedment

= $1-0.5*(q/(q_{bar}-q))$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

$q = \gamma Df$

Stone Column Length 44-ft

Reduction factor on Priebe: 1

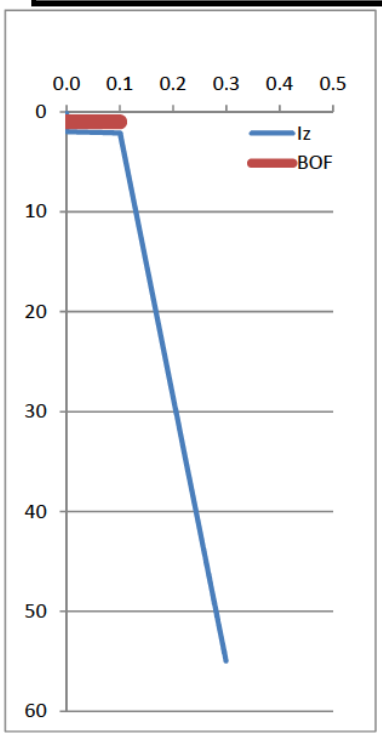
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar}	2882.00-psf	Per GRI page 14
L	212.7-ft	Equivalent Area for 240' ø tank
B	212.7-ft	Reference GRI page 2 and B=L
fill density	125-pcf	Assigned by Designer
Elevation of Top of Soil Profile	29.17	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and qc: Es=X·qc

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	250.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.953	=1-0.5[q/(q_{bar}-q)]
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1$=0.5B=	106.3
I_z = 0	at depth	$z=z_2$=2.0B=	425.4
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1$=1.0B=	212.7
I_z = 0	at depth	$z=z_2$=4.0B=	850.8
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

Σ(Iz/Es)Δz (ft2/lb)= 9.27E-06

Total Elastic Settlement (Flexible foundation)

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0358-ft 0.22-in

Total Rapid Elastic Settlement

$S_e=C_1(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0232-ft 0.14-in

Long Term Settlement 0.08-in

Calculated Consolidation Type Settlement:

Total:	1.29-in	Adjusted for footing Type
Recomp.	0.05-in	Adjusted for footing Type
Virgin Sett.	1.24-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	1.34-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:

Total: 1.51-in

Rapid: 1.48-in

Long Term: 0.02-in

Mathcad – Tank 1
Bearing Checks

Calculations for Tank Edge and Base Stability (Tank 1):

Reference: Stability of Steel Oil Storage Tanks, Duncan, D'Orazio, 1984

GEOMETERY

Tank Diameter: $D := 240ft$

Existing Elev: $ELEVex := 29.26ft$

Tank Bottom Elev: $TankB := 31.75ft$

Fill and LTP Thickness: $Tp := TankB - (ELEVex - 4ft)$ $\quad Tp = 6.49 \cdot ft$

Overburden Thickness: $Tob := TankB - Tp - 12ft$ $\quad Tob = 13.26 \cdot ft$

Saturated Clay above the GW will consolidate quickly

Clay Thickness:

CPT	Clay Thickess	Clay and Silt
111	5'	12.6'
112	8'	12.7'
113	5'	12.2'
114(s)	9.15'	14.53'
115	4'	11'

$Tclay := 9.15ft$ $\quad Tclay\_silt := 15ft$

Stone column replacement ratio: $Ar := 0.11$

SOIL PROPERTIES

Overburden unit weight: $\gamma ob := 96.8pcf$ Post Improvment - GI Sheets REF CPT114B(S)

Tore Vane Strenght of Clay: $TV := 0.8tsf$ Refernce GRI Report Boring B-1,B-2, B-4, B-5, B-6, B-7, B-8, B-10, B-12 and B-14 through B-22.

$TV = 1600 \cdot psf$

Friction angle of stone: $\phi stone := 50°$

Friction angle of silt $\phi silt := 20°$

LTP unit weight: $\gamma p := 130pcf$

SOIL STRENGTH AND STRESSES

Bearing Pressure due to Liquid in Tank: $q := 2882psf$ At full hydrotest load

Effective stress at top of clay: $\sigma ob := (\gamma ob \cdot Tob) + (\gamma p \cdot Tp)$ $\quad \sigma ob = 2127.268 \cdot psf$

Shear Strenght of SC: $Ssc := \sigma ob \cdot \tan(\phi stone)$ $\quad Ssc = 2535.179 \cdot psf$

Cohesive strenght of clay: $Cu := \frac{TV}{2}$ $\quad Cu = 800 \cdot psf$

Improved clay strength: $Su\_clay := Ar \cdot Ssc + (1 - Ar) \cdot Cu$

$Su\_clay = 990.87 \cdot psf$ After equation 10

Improved Clay-Silt Strenght $Su\_clay\_silt := Ar \cdot Ssc + (1 - Ar)(Cu + \sigma ob \cdot \tan(20))$

$Su\_clay\_silt = 5226.416 \cdot psf$ After equation 10

BASE SHEAR STABILITY EVALUATION - AT FULL HYDRO TEST - NO STRENGTH GAIN

$Dc := D + Tp + Tob \qquad Dc = 259.75 \cdot ft$ Equation 1

Bearing pressure at top of clay:

$$Qclay := q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2 + \gamma ob \cdot Tob$$ Equation 2

$$Qclay = 4503.724 \cdot psf$$

FOR CLAY LAYER ONLY

For a clay layer thicker than one sixth of the tank diameter $Nc1 := 6.1$ Equation 4

For a clay layer thinner than one sixth of the tank diamete $Nc2 := 4.1 + \left(\frac{Dc}{3Tclay}\right)$ Equation 5

$$Nc2 = 13.563$$

Place holder for easier coding: $PH := \frac{Dc}{Tclay}$

$Nc := if[(PH < 6), Nc1, Nc2] \qquad Nc = 13.563$

$Qult := Su\_clay \cdot Nc + \gamma ob \cdot Tob \qquad Qult = 14722.397 \cdot psf$

$$FS\_base\_clay := \frac{Su\_clay \cdot Nc}{\left[q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2\right]} \qquad FS\_base\_clay = 4.173$$ Equation 8

FOR COMBINED CLAY-SILT LAYER

For a clay layer thicker than one sixth of the tank diameter $Nc1 := 6.1$ Equation 4

For a clay layer thinner than one sixth of the tank diamete $Nc2 := 4.1 + \left(\frac{Dc}{3Tclay\_silt}\right)$ Equation 5

$$Nc2 = 9.872$$

Place holder for easier coding: $PH := \frac{Dc}{Tclay\_silt}$

$Nc := if[(PH < 6), Nc1, Nc2] \qquad Nc = 9.872$

$Qult := Su\_clay\_silt \cdot Nc + \gamma ob \cdot Tob \qquad Qult = 52879.909 \cdot psf$

$$FS\_base\_clay\_silt := \frac{Su\_clay\_silt \cdot Nc}{\left[q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2\right]} \qquad FS\_base\_clay\_silt = 16.023$$ Equation 8

EDGE SHEAR STABILITY EVALUATION - AT FULL HYDRO TEST - NO STRENGTH GAIN

IMPROVED CLAY

$Qapp := q + \gamma p \cdot Tp \qquad Qapp = 3725.7 \cdot psf$ Equation 11

With of segment that might fail $B := \begin{pmatrix} 10 \\ 20 \\ 40 \\ 60 \\ 80 \\ 100 \\ 120 \end{pmatrix} ft$

$Nc := 5.2 + \frac{B}{D} \qquad Nc = \begin{pmatrix} 5.242 \\ 5.283 \\ 5.367 \\ 5.45 \\ 5.533 \\ 5.617 \\ 5.7 \end{pmatrix}$

$Qall := Su\_clay \cdot Nc + \gamma p \cdot Tp \qquad Qall = \begin{pmatrix} 6037.509 \\ 6078.795 \\ 6161.368 \\ 6243.94 \\ 6326.512 \\ 6409.085 \\ 6491.657 \end{pmatrix} \cdot psf$

$FS\_edge\_clay := \frac{Qall}{Qapp} \qquad FS\_edge\_clay = \begin{pmatrix} 1.621 \\ 1.632 \\ 1.654 \\ 1.676 \\ 1.698 \\ 1.72 \\ 1.742 \end{pmatrix}$

EDGE SHEAR STABILITY EVALUATION - AT FULL HYDRO TEST - NO STRENGTH GAIN

IMPROVED CLAY and SILT

$Qapp := q + \gamma p \cdot Tp$ $\quad Qapp = 3725.7 \cdot psf$ $\quad$ Equation 11

With of segment that might fail $\quad B := \begin{pmatrix} 10 \\ 20 \\ 40 \\ 60 \\ 80 \\ 100 \\ 120 \end{pmatrix} ft$ $\quad Nc := 5.2 + \frac{B}{D}$ $\quad Nc = \begin{pmatrix} 5.242 \\ 5.283 \\ 5.367 \\ 5.45 \\ 5.533 \\ 5.617 \\ 5.7 \end{pmatrix}$

$Qall := Su\_clay\_silt \cdot Nc + \gamma p \cdot Tp \quad Qall = \begin{pmatrix} 28238.831 \\ 28456.598 \\ 28892.133 \\ 29327.668 \\ 29763.202 \\ 30198.737 \\ 30634.272 \end{pmatrix} \cdot psf$

$FS\_edge\_clay\_silt := \frac{Qall}{Qapp} \quad FS\_edge\_clay\_silt = \begin{pmatrix} 7.579 \\ 7.638 \\ 7.755 \\ 7.872 \\ 7.989 \\ 8.106 \\ 8.222 \end{pmatrix}$

SUMMARY OF BASE AND EDGE SHEAR STABILITY EVALUATION - AT FULL HYDRO TEST - NO STRENGTH GAIN

$FS\_base\_clay = 4.173$

$FS\_base\_clay\_silt = 16.023$

$FS\_edge\_clay = \begin{pmatrix} 1.621 \\ 1.632 \\ 1.654 \\ 1.676 \\ 1.698 \\ 1.72 \\ 1.742 \end{pmatrix}$ $\quad FS\_edge\_clay\_silt = \begin{pmatrix} 7.579 \\ 7.638 \\ 7.755 \\ 7.872 \\ 7.989 \\ 8.106 \\ 8.222 \end{pmatrix}$

BASE SHEAR STABILITY EVALUATION - IN SERVICE - NO STRENGTH GAIN

$q := 2500psf$

$Dc := D + Tp + Tob \qquad Dc = 259.75 \cdot ft$ Equation 1

Bearing pressure at top of clay:

$$Qclay := q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2 + \gamma ob \cdot Tob$$ Equation 2

$$Qclay = 4177.606 \cdot psf$$

FOR CLAY LAYER ONLY

For a clay layer thicker than one sixth of the tank diameter $Nc1 := 6.1$ Equation 4

For a clay layer thinner than one sixth of the tank diamete $Nc2 := 4.1 + \left(\frac{Dc}{3Tclay}\right)$ Equation 5

$$Nc2 = 13.563$$

Place holder for easier coding: $PH := \frac{Dc}{Tclay}$

$Nc := if[(PH < 6), Nc1, Nc2] \qquad Nc = 13.563$

$Qult := Su\_clay \cdot Nc + \gamma ob \cdot Tob \qquad Qult = 14722.397 \cdot psf$

$$FS\_base\_clay := \frac{Su\_clay \cdot Nc}{\left[q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2\right]} \qquad FS\_base\_clay = 4.644$$ Equation 8

FOR COMBINED CLAY-SILT LAYER

For a clay layer thicker than one sixth of the tank diameter $Nc1 := 6.1$ Equation 4

For a clay layer thinner than one sixth of the tank diamete $Nc2 := 4.1 + \left(\frac{Dc}{3Tclay\_silt}\right)$ Equation 5

$$Nc2 = 9.872$$

Place holder for easier coding: $PH := \frac{Dc}{Tclay\_silt}$

$Nc := if[(PH < 6), Nc1, Nc2] \qquad Nc = 9.872$

$Qult := Su\_clay\_silt \cdot Nc + \gamma ob \cdot Tob \qquad Qult = 52879.909 \cdot psf$

$$FS\_base\_clay\_silt := \frac{Su\_clay\_silt \cdot Nc}{\left[q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2\right]} \qquad FS\_base\_clay\_silt = 17.828$$ Equation 8

EDGE SHEAR STABILITY EVALUATION - IN SERVICE - NO STRENGTH GAIN

IMPROVED CLAY

$Qapp := q + \gamma p \cdot Tp$ $\quad Qapp = 3343.7 \cdot psf$ Equation 11

With of segment that might fail $\quad B := \begin{pmatrix} 10 \\ 20 \\ 40 \\ 60 \\ 80 \\ 100 \\ 120 \end{pmatrix} ft$

$Nc := 5.2 + \frac{B}{D} \quad Nc = \begin{pmatrix} 5.242 \\ 5.283 \\ 5.367 \\ 5.45 \\ 5.533 \\ 5.617 \\ 5.7 \end{pmatrix}$

$Qall := Su\_clay \cdot Nc + \gamma p \cdot Tp \quad Qall = \begin{pmatrix} 6037.509 \\ 6078.795 \\ 6161.368 \\ 6243.94 \\ 6326.512 \\ 6409.085 \\ 6491.657 \end{pmatrix} \cdot psf$

$FS\_edge\_clay := \frac{Qall}{Qapp} \quad FS\_edge\_clay = \begin{pmatrix} 1.806 \\ 1.818 \\ 1.843 \\ 1.867 \\ 1.892 \\ 1.917 \\ 1.941 \end{pmatrix}$

EDGE SHEAR STABILITY EVALUATION - IN SERVICE - NO STRENGTH GAIN

IMPROVED CLAY and SILT

$Qapp := q + \gamma p \cdot Tp$ $\quad Qapp = 3343.7 \cdot psf$ $\quad$ Equation 11

With of segment that might fail $\quad B := \begin{pmatrix} 10 \\ 20 \\ 40 \\ 60 \\ 80 \\ 100 \\ 120 \end{pmatrix} ft$ $\quad Nc := 5.2 + \frac{B}{D}$ $\quad Nc = \begin{pmatrix} 5.242 \\ 5.283 \\ 5.367 \\ 5.45 \\ 5.533 \\ 5.617 \\ 5.7 \end{pmatrix}$

$Qall := Su\_clay\_silt \cdot Nc + \gamma p \cdot Tp \quad Qall = \begin{pmatrix} 28238.831 \\ 28456.598 \\ 28892.133 \\ 29327.668 \\ 29763.202 \\ 30198.737 \\ 30634.272 \end{pmatrix} \cdot psf$

$FS\_edge\_clay\_silt := \frac{Qall}{Qapp} \quad FS\_edge\_clay\_silt = \begin{pmatrix} 8.445 \\ 8.511 \\ 8.641 \\ 8.771 \\ 8.901 \\ 9.032 \\ 9.162 \end{pmatrix}$

SUMMARY OF BASE AND EDGE SHEAR STABILITY EVALUATION - IN SERVCIE - NO STRENGTH GAIN

$FS\_base\_clay = 4.644$

$FS\_base\_clay\_silt = 17.828$

$FS\_edge\_clay = \begin{pmatrix} 1.806 \\ 1.818 \\ 1.843 \\ 1.867 \\ 1.892 \\ 1.917 \\ 1.941 \end{pmatrix} \quad FS\_edge\_clay\_silt = \begin{pmatrix} 8.445 \\ 8.511 \\ 8.641 \\ 8.771 \\ 8.901 \\ 9.032 \\ 9.162 \end{pmatrix}$

Units

$kips \equiv 1000 \cdot lbf$ $\quad ksi \equiv \frac{1000 \cdot lbf}{in^2}$ $\quad psi \equiv \frac{lbf}{in^2}$ $\quad k \equiv kips$ $\quad N \equiv newton$

$ksf := 1000 \cdot psf$ $\quad ORIGIN \equiv 1$

$tsf := 2 \cdot ksf$ $\quad pcf \equiv \frac{lbf}{ft^3}$ $\quad psf \equiv \frac{lbf}{ft^2}$

$MPa := 10^6 \cdot Pa$

$kPa := 1000 \cdot Pa$

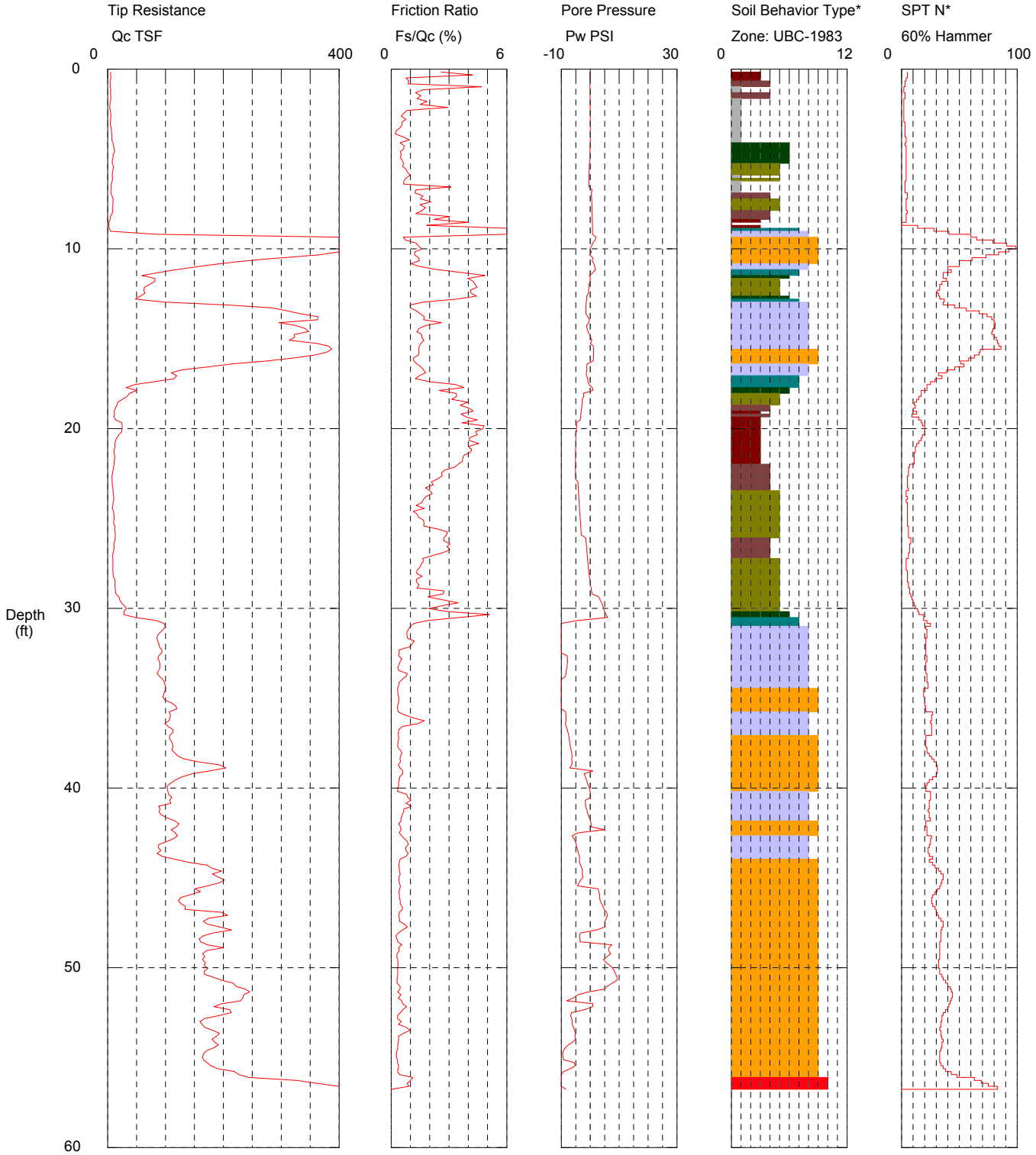
$in^2 = 0.001\,m^2$

$pi := 3.14159$ $\quad kcf \equiv \frac{k}{ft^3}$ $\quad pcf \equiv \frac{lbf}{ft^3}$ $\quad kN \equiv 1000 newton$

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-111
Cone Used: DDG1254

CPT Date/Time: 11/15/2014 10:49:21 AM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 56.76 feet

Depth Increment = 0.164 feet

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

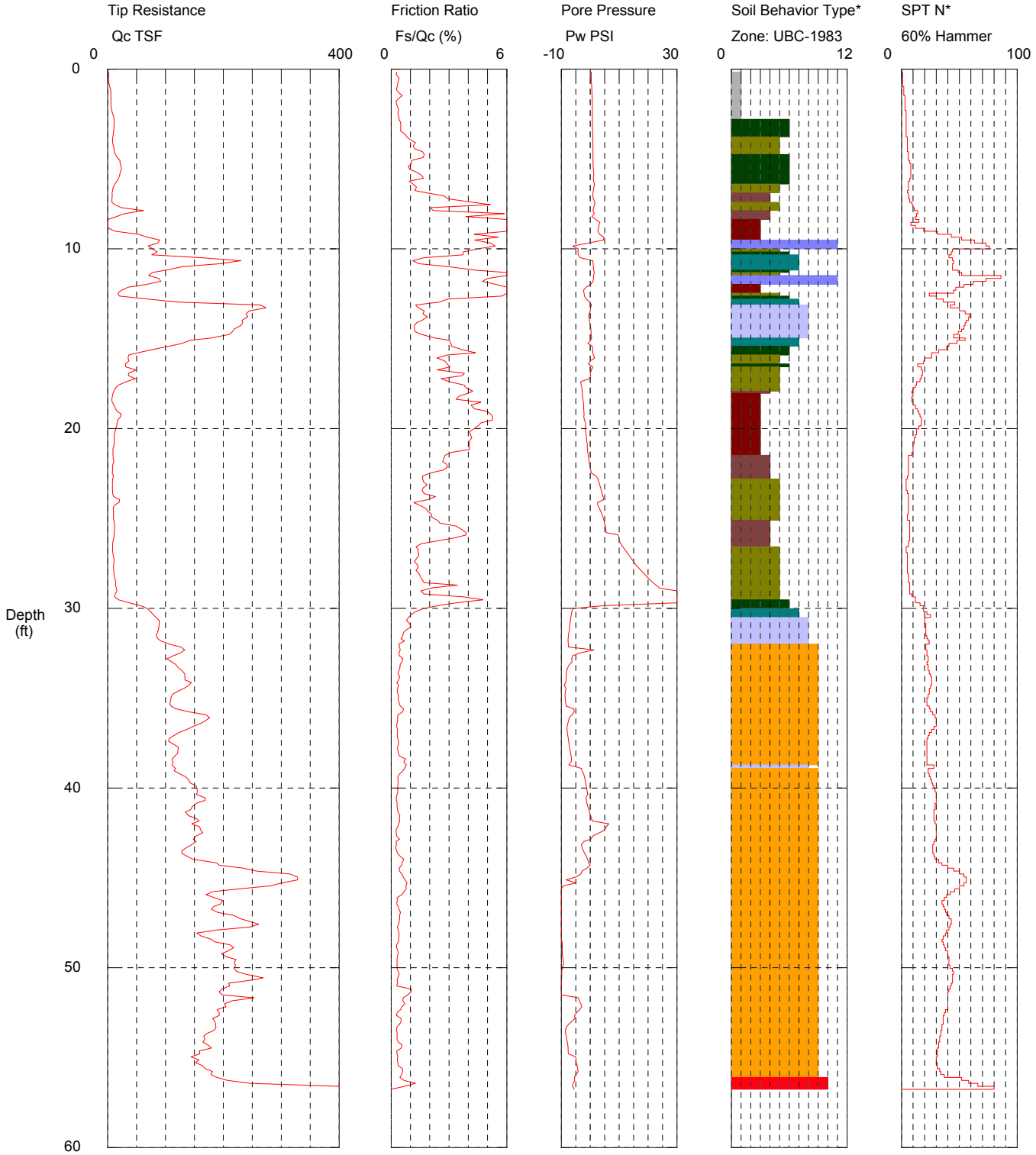
InSitu Engineering Predrilled top 9 feet

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

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-112
Cone Used: DDG1254

CPT Date/Time: 11/14/2014 4:23:51 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 56.76 feet

Depth Increment = 0.164 feet

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

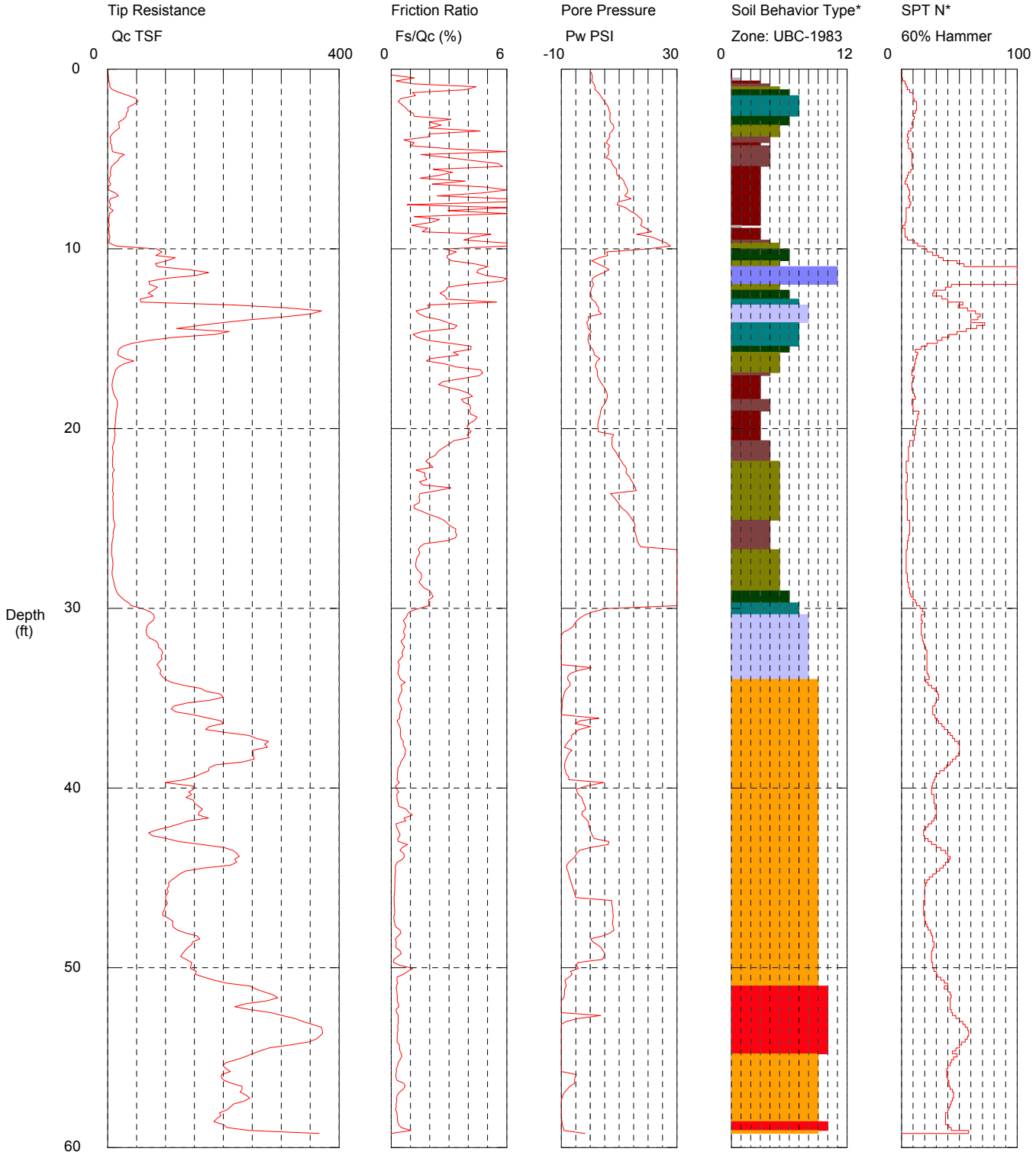
InSitu Engineering

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

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-113
Cone Used: DDG1254

CPT Date/Time: 11/14/2014 2:21:15 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 59.22 feet

Depth Increment = 0.164 feet

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

InSitu Engineering

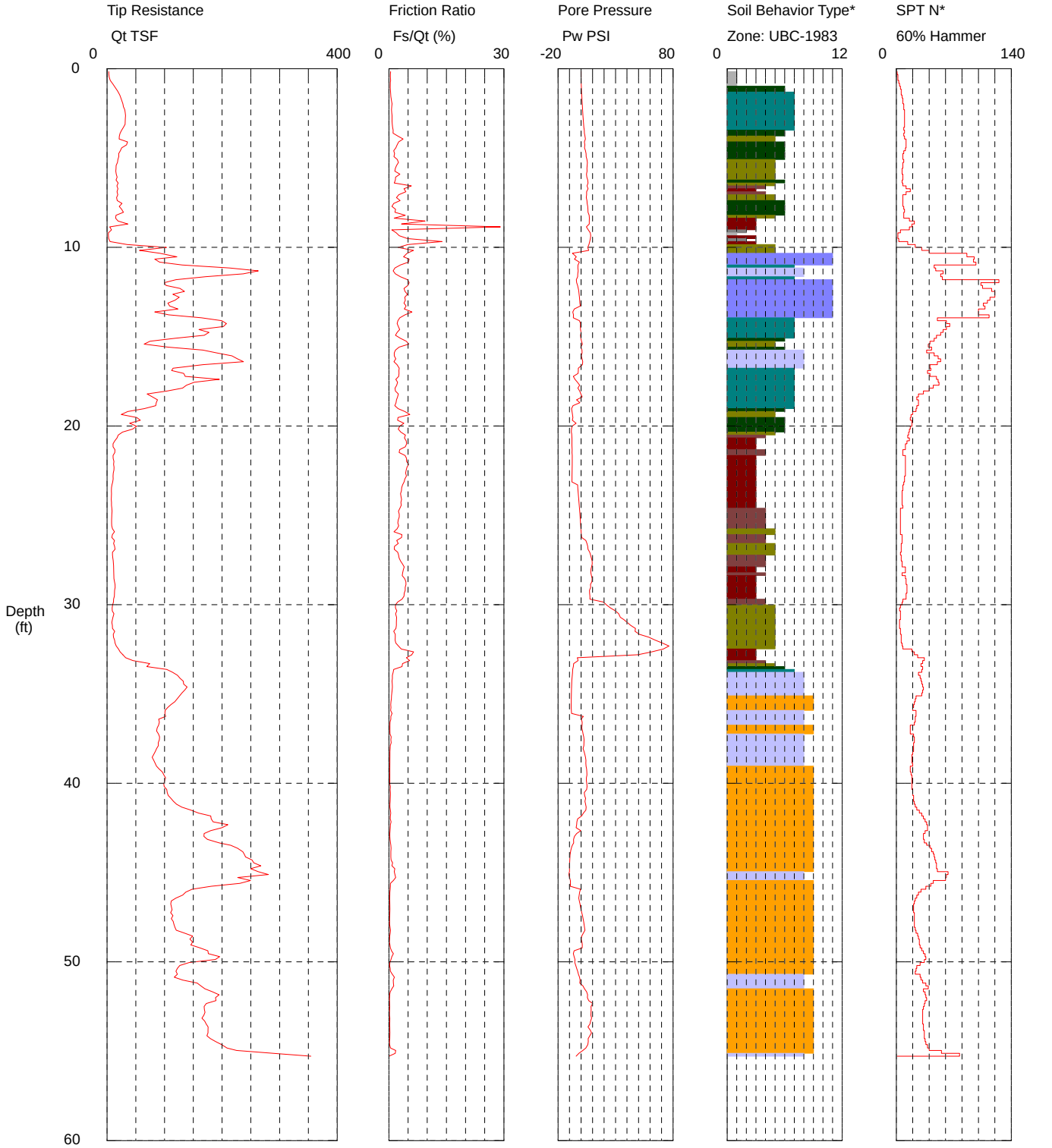
*Soil behavior type and SPT based on data from UBC-1983

EX-0001-007000-PCE

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-114(s)B
Cone Used: DDG1254

CPT Date/Time: 11/13/2014 4:27:24 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 55.28 feet

Depth Increment = 0.164 feet

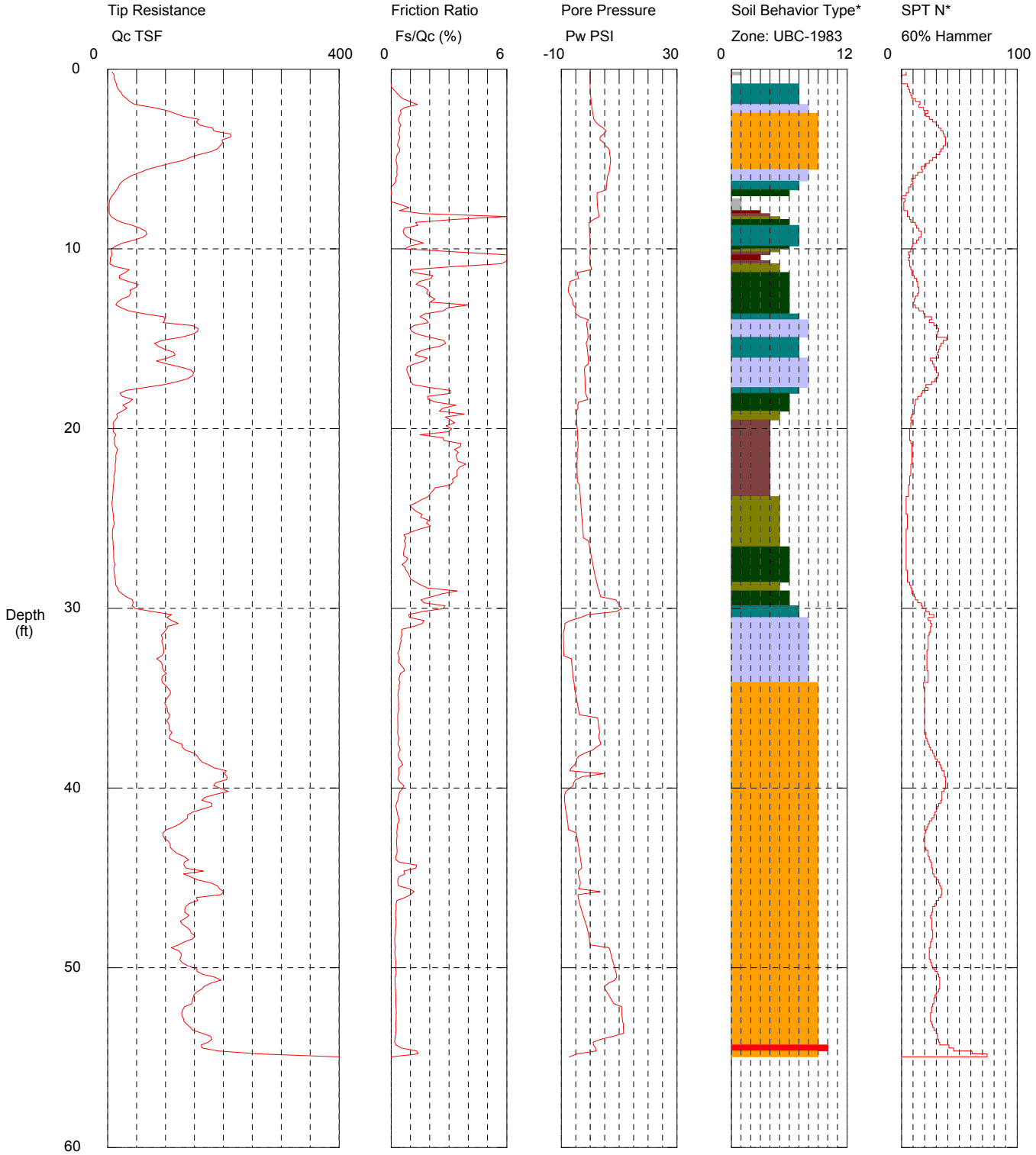
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

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

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-115
Cone Used: DDG1254

CPT Date/Time: 11/15/2014 12:37:58 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 54.95 feet

Depth Increment = 0.164 feet

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

InSitu Engineering Predrilled top 13 feet

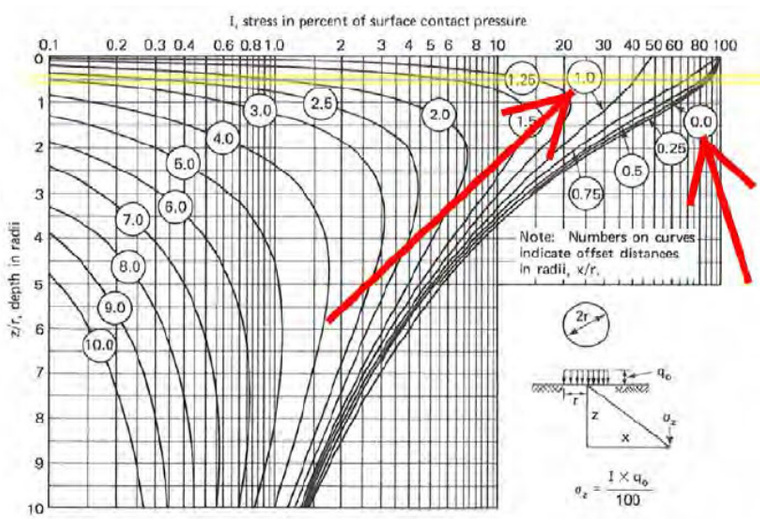
*Soil behavior type and SPT based on data from UBC-1983

Tank 2

Design Summary
Tank 2

This table assumes the exploration is directly below the center of the tank.

Tank 4		Orig	Imp.	Improved		Original	
Soil Profile	Depth	Liq Sett.	Liq Sett.	Rapid Sett.	Long Term Sett.	Rapid Sett.	Long Term Sett.
HB-CPT-141	57.3-ft	2.7-in.	1.84-in.	3.64-in.	.17-in.	1.36-in.	4.8-in.
HB-CPT-145	56.8-ft	3.07-in.	2.26-in.	3.52-in.	.2-in.	1.52-in.	4.66-in.
HB-CPT-144	55.8-ft	2.89-in.	1.74-in.	3.8-in.	.15-in.	1.85-in.	5.27-in.
X-CPT-4	54.3-ft	1.44-in.	.5-in.	4.82-in.	.14-in.	1.54-in.	6.46-in.
HB-CPT-142	54.3-ft	2.22-in.	1.89-in.	3.66-in.	.15-in.	1.6-in.	4.98-in.
HB-CPT-143(s)	57.6-ft	3.38-in.	1.88-in.	3.46-in.	.21-in.	1.7-in.	4.65-in.
Average:		**2.62-in.**	**1.65-in.**	**3.89-in.**	**.16-in.**	**1.58-in.**	**5.23-in.**



This table calculates Settlement at the edge of the tank.

Tank 4		Orig	Imp.	Improved		Original	
Soil Profile	Depth	Liq Sett.	Liq Sett.	Rapid Sett.	Long Term Sett.	Rapid Sett.	Long Term Sett.
HB-CPT-141	57.3-ft	2.7-in.	1.84-in.	1.82-in.	.09-in.	.68-in.	2.4-in.
HB-CPT-145	56.8-ft	3.07-in.	2.26-in.	1.76-in.	.1-in.	.76-in.	2.33-in.
HB-CPT-144	55.8-ft	2.89-in.	1.74-in.	1.9-in.	.08-in.	.93-in.	2.63-in.
X-CPT-4	54.3-ft	1.44-in.	.5-in.	2.41-in.	.07-in.	.77-in.	3.23-in.
HB-CPT-142	54.3-ft	2.22-in.	1.89-in.	3.66-in.	.15-in.	1.6-in.	4.98-in.
HB-CPT-143(s)	57.6-ft	3.38-in.	1.88-in.	3.46-in.	.21-in.	1.7-in.	4.65-in.
Average:		**2.46-in.**	**1.65-in.**	**2.31-in.**	**.1-in.**	**.95-in.**	**3.11-in.**

Note: No reduction to total soil settlement is taken for the liquefaction condition.

Hayward Baker Inc.

THINK SAFE

SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 2		
Job no.	860079	Design by: M. Rohrbach	Date: 14-Apr-15
File	CPT-Liquefaction-Boulanger	Checked	
Spreadsheet Rev.	23-Dec-14	Reviewed	

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-141

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	15.9-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	27.93	ft.
Elevation of bottom of Soil Profile	-29.32	ft.
Elevation of Installation	23.5	
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	57.3-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, ν_s	0.333	
Poisson's Ratio for Stone Column, ν_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	496.60-psf	

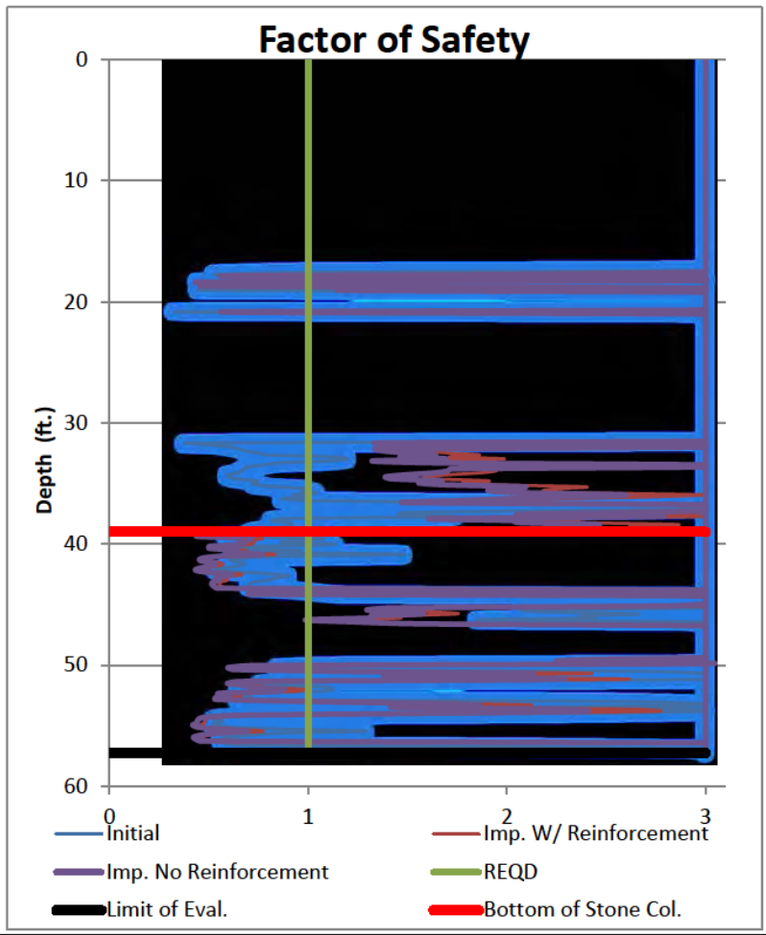
Stone Column Design Information

L = 39-ft			ϕ_c = 50
Column Spacing 8-ft			Ec = 750.0-tsf
Kac = 0.1324743	$=TAN^2(45-(\phi_c/2)$		Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 57.3-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 14.5-ft.

Calculated Min: 2.54-in via T&S, Min never < 0.5 in.

Calculated Max: 2.70-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 10.5-ft.

Calculated Min: 1.91-in via T&S, Min never < 0.5 in.

Calculated Max: 1.95-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 10.0-ft.

Calculated Min: 1.84-in via T&S, Min never < 0.5 in.

Calculated Max: 1.87-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 2 — IMPROVED CASE: CPT-141

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e = C_1 C_2 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment

= $1 - 0.5*(q/(q_{bar} - q))$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

$q = \gamma Df$

Stone Column Length 39-ft

Reduction factor on Priebe: 1

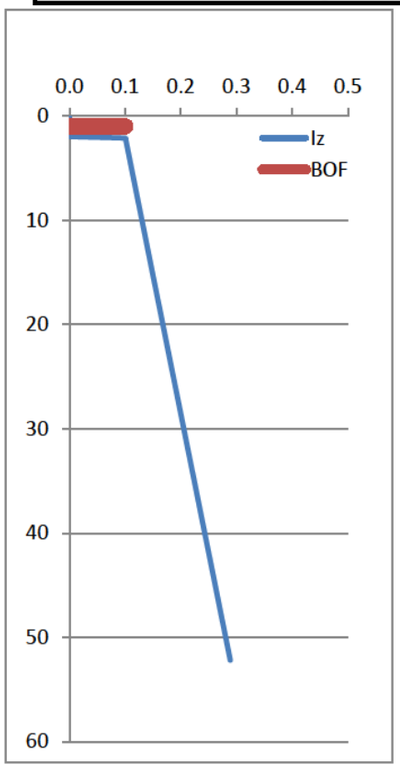
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	
L	212.7-ft	Equivalent Area for 240' Ø tank
B	212.7-ft	
fill density	130-pcf	Assigned by Designer
Elevation of Top of Soil Profile	27.93	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es = X \cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=$**0.5**B=	106.3
I_z = 0	at depth	$z=z_2=$**2.0**B=	425.4
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=$**1.0**B=	212.7
I_z = 0	at depth	$z=z_2=$**4.0**B=	850.8
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

Σ(Iz/Es)Δz (ft2/lb)= 1.81E-05

Total Elastic Settlement (Flexible foundation)

$S_e = C_1 C_2 (q_{bar} - q)\Sigma[(I_z/E_s)\Delta z]$= 0.0694-ft 0.83-in

Total Rapid Elastic Settlement

$S_e = C_1 (q_{bar} - q)\Sigma[(I_z/E_s)\Delta z]$= 0.0450-ft 0.54-in

Long Term Settlement 0.29-in

Calculated Consolidation Type Settlement:

Total:	2.98-in	Adjusted for footing Type
Recomp.	0.12-in	Adjusted for footing Type
Virgin Sett.	2.86-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	3.10-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:

Total: 3.81-in

Rapid: 3.64-in

Long Term: 0.17-in

Hayward Baker Inc.

THINK SAFE

SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 2		
Job no.	860079	Design by: M. Rohrbach	Date: 14-Apr-15
File	CPT-Liquefaction-Boulanger	Checked	
Spreadsheet Rev.	23-Dec-14	Reviewed	

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-142

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	15.6-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	27.55	ft.
Elevation of bottom of Soil Profile	-26.75	ft.
Elevation of Installation	23.5	
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	54.3-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	546.00-psf	

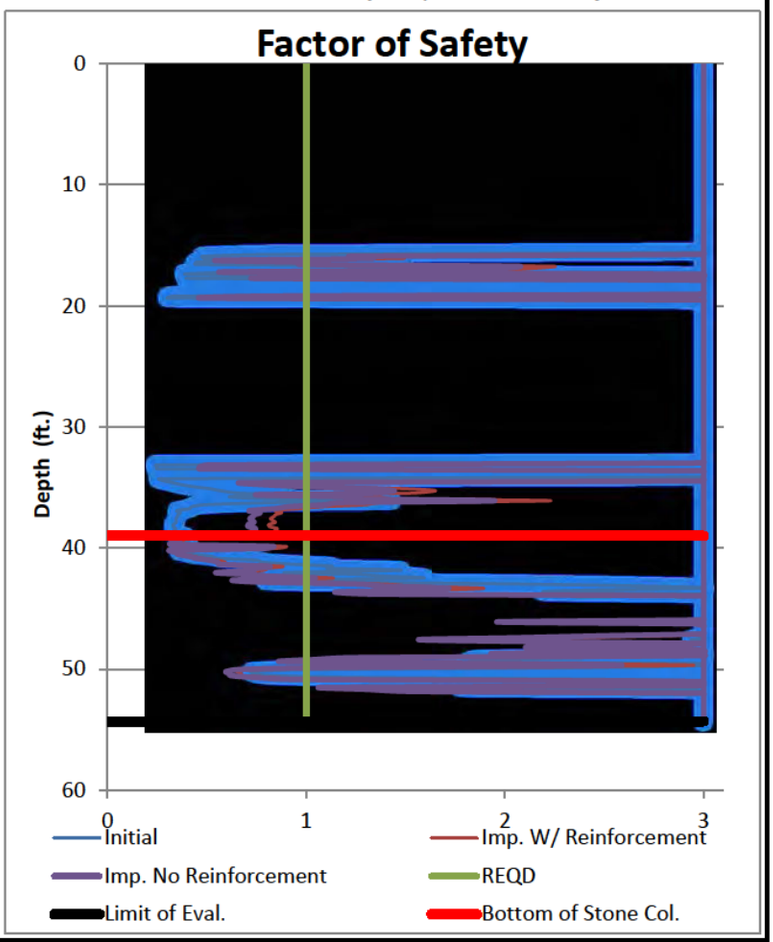
Stone Column Design Information

L =	39-ft		ϕ_c = 50
Column Spacing	8-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 54.3-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 10.3-ft.

Calculated Min: 2.15-in via T&S, Min never < 0.5 in.

Calculated Max: 2.22-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 9.3-ft.

Calculated Min: 1.99-in via T&S, Min never < 0.5 in.

Calculated Max: 2.14-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 8.8-ft.

Calculated Min: 1.89-in via T&S, Min never < 0.5 in.

Calculated Max: 2.11-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 2 — IMPROVED CASE: CPT-142

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e = C_1 C_2 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment

= $1 - 0.5*(q/(q_{bar}-q))$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

$q = \gamma Df$

Stone Column Length 39-ft

Reduction factor on Priebe: 1

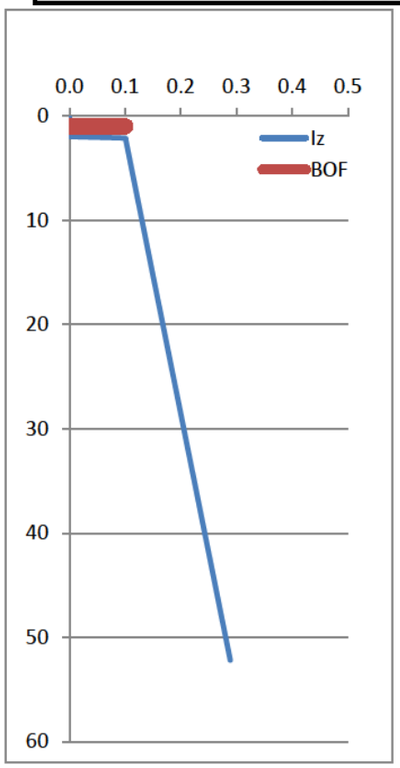
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	
L	212.7-ft	Equivalent Area for 240' Ø tank
B	212.7-ft	
fill density	130-pcf	Assigned by Designer
Elevation of Top of Soil Profile	27.55	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es = X \cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=$**0.5**B=	106.3
I_z = 0	at depth	$z=z_2=$**2.0**B=	425.4
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=$**1.0**B=	212.7
I_z = 0	at depth	$z=z_2=$**4.0**B=	850.8
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

Σ(Iz/Es)Δz (ft2/lb)= 1.69E-05

Total Elastic Settlement (Flexible foundation)

$S_e = C_1 C_2 (q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0649-ft 0.78-in

Total Rapid Elastic Settlement

$S_e = C_1 (q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0421-ft 0.51-in

Long Term Settlement 0.27-in

Calculated Consolidation Type Settlement:

Total:	3.03-in	Adjusted for footing Type
Recomp.	0.12-in	Adjusted for footing Type
Virgin Sett.	2.91-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	3.15-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:

Total: 3.81-in

Rapid: 3.66-in

Long Term: 0.15-in

Hayward Baker Inc.

THINK SAFE

SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 2		
Job no.	860079	Design by: M. Rohrbach	Date: 14-Apr-15
File	CPT-Liquefaction-Boulanger	Checked	
Spreadsheet Rev.	23-Dec-14	Reviewed	

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-143(S)

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	16.6-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	28.58	ft.
Elevation of bottom of Soil Profile	-29.00	ft.
Elevation of Installation	23.5	
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	57.6-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, ν_s	0.333	
Poisson's Ratio for Stone Column, ν_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	412.10-psf	

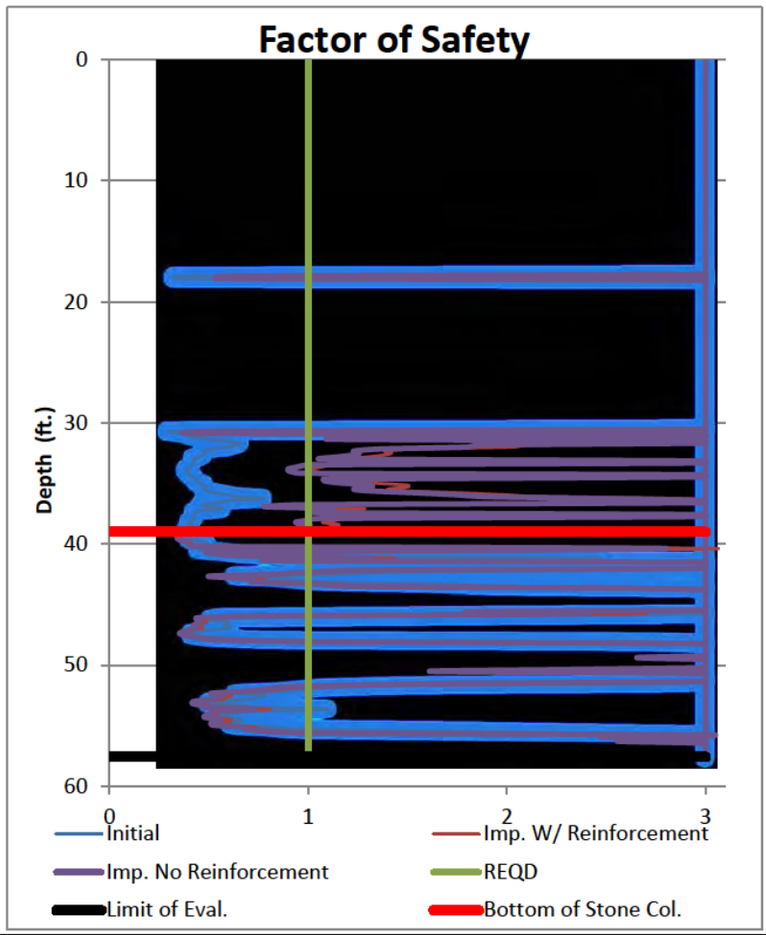
Stone Column Design Information

L = 39-ft			ϕ_c = 50
Column Spacing 8-ft			Ec = 750.0-tsf
Kac = 0.1324743	$=TAN^2(45-(\phi_c/2)$		Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 57.6-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 16.6-ft.

Calculated Min: 3.33-in via T&S, Min never < 0.5 in.

Calculated Max: 3.38-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 11.6-ft.

Calculated Min: 2.29-in via T&S, Min never < 0.5 in.

Calculated Max: 2.57-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 9.6-ft.

Calculated Min: 1.88-in via T&S, Min never < 0.5 in.

Calculated Max: 2.17-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 2 — IMPROVED CASE: CPT-143(S)

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e = C_1 C_2 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment

= $1 - 0.5*(q/(q_{bar} - q))$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

$q = \gamma Df$

Stone Column Length 39-ft

Reduction factor on Priebe: 1

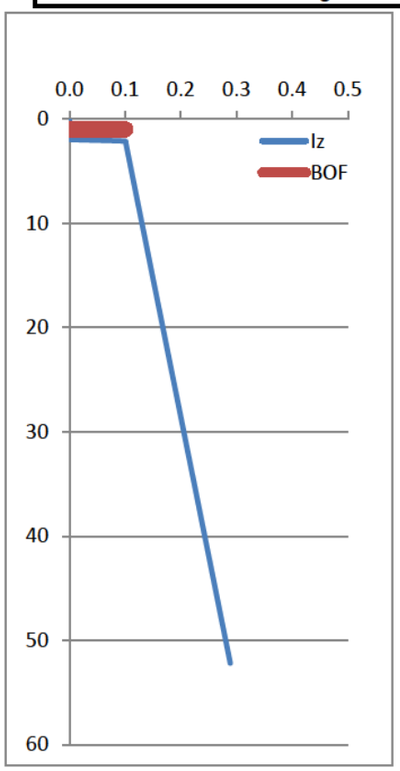
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	
L	212.7-ft	Equivalent Area for 240' Ø tank
B	212.7-ft	
fill density	130-pcf	Assigned by Designer
Elevation of Top of Soil Profile	28.58	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es = X \cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)

I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=$**0.5**B=	106.3
I_z = 0	at depth	$z=z_2=$**2.0**B=	425.4

For foundations with L/B ≥ 10

I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=$**1.0**B=	212.7
I_z = 0	at depth	$z=z_2=$**4.0**B=	850.8

Interpolation for L/B values between 1 and 10

I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

Σ(Iz/Es)Δz (ft2/lb)= 2.01E-05

Total Elastic Settlement (Flexible foundation)

$S_e = C_1 C_2 (q_{bar} - q)\Sigma[(I_z/E_s)\Delta z]$= 0.0771-ft 0.92-in

Total Rapid Elastic Settlement

$S_e = C_1 (q_{bar} - q)\Sigma[(I_z/E_s)\Delta z]$= 0.0500-ft 0.60-in

Long Term Settlement 0.32-in

Calculated Consolidation Type Settlement:

Total:	2.75-in	Adjusted for footing Type
Recomp.	0.11-in	Adjusted for footing Type
Virgin Sett.	2.64-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	2.86-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:

Total: 3.67-in

Rapid: 3.46-in

Long Term: 0.21-in

Hayward Baker Inc.

THINK SAFE

SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 2		
Job no.	860079	Design by: M. Rohrbach	Date: 14-Apr-15
File	CPT-Liquefaction-Boulanger	Checked	
Spreadsheet Rev.	23-Dec-14	Reviewed	

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-144

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	15.4-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	27.4	ft.
Elevation of bottom of Soil Profile	-28.37	ft.
Elevation of Installation	23.5	
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	55.8-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, ν_s	0.333	
Poisson's Ratio for Stone Column, ν_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	565.50-psf	

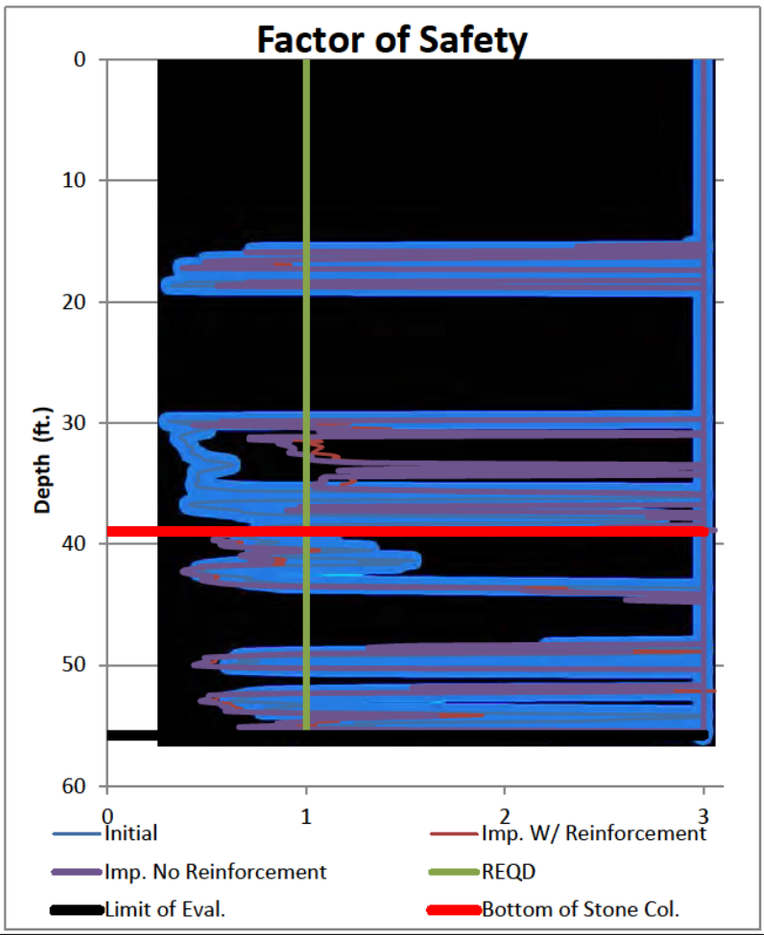
Stone Column Design Information

L =	39-ft		ϕ_c = 50
Column Spacing	8-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 55.8-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 14.8-ft.

Calculated Min: 2.77-in via T&S, Min never < 0.5 in.

Calculated Max: 2.89-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 12.0-ft.

Calculated Min: 2.24-in via T&S, Min never < 0.5 in.

Calculated Max: 2.51-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 9.6-ft.

Calculated Min: 1.74-in via T&S, Min never < 0.5 in.

Calculated Max: 2.07-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 2 — IMPROVED CASE: CPT-144

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e = C_1 C_2 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment

= $1 - 0.5*(q/(q_{bar} - q))$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

$q = \gamma Df$

Stone Column Length 39-ft

Reduction factor on Priebe: 1

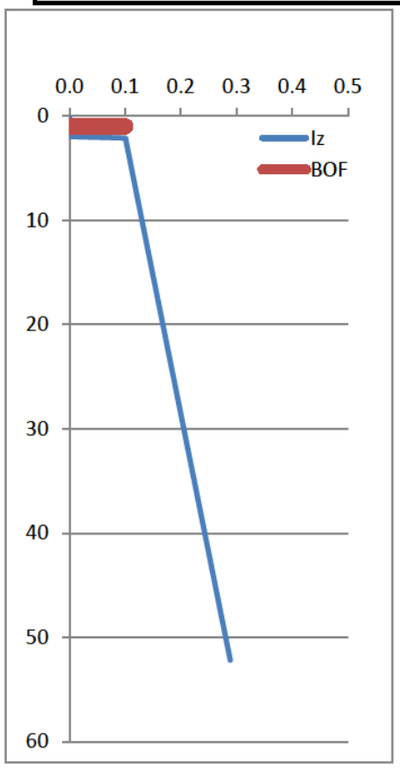
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	
L	212.7-ft	Equivalent Area for 240' Ø tank
B	212.7-ft	
fill density	130-pcf	Assigned by Designer
Elevation of Top of Soil Profile	27.4	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es = X \cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)

I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=$**0.5**B=	106.3
I_z = 0	at depth	$z=z_2=$**2.0**B=	425.4

For foundations with L/B ≥ 10

I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=$**1.0**B=	212.7
I_z = 0	at depth	$z=z_2=$**4.0**B=	850.8

Interpolation for L/B values between 1 and 10

I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

Σ(Iz/Es)Δz (ft2/lb)= 1.74E-05

Total Elastic Settlement (Flexible foundation)

$S_e = C_1 C_2 (q_{bar} - q)\Sigma[(I_z/E_s)\Delta z]$= 0.0666-ft 0.80-in

Total Rapid Elastic Settlement

$S_e = C_1 (q_{bar} - q)\Sigma[(I_z/E_s)\Delta z]$= 0.0433-ft 0.52-in

Long Term Settlement 0.28-in

Calculated Consolidation Type Settlement:

Total:	3.15-in	Adjusted for footing Type
Recomp.	0.13-in	Adjusted for footing Type
Virgin Sett.	3.03-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	3.28-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:

Total: 3.95-in

Rapid: 3.80-in

Long Term: 0.15-in

Hayward Baker Inc.

THINK SAFE

SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 2		
Job no.	860079	Design by: M. Rohrbach	Date: 14-Apr-15
File	CPT-Liquefaction-Boulanger	Checked	
Spreadsheet Rev.	23-Dec-14	Reviewed	

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-145

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	15.9-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	27.86	ft.
Elevation of bottom of Soil Profile	-28.93	ft.
Elevation of Installation	23.5	
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	56.8-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	505.70-psf	

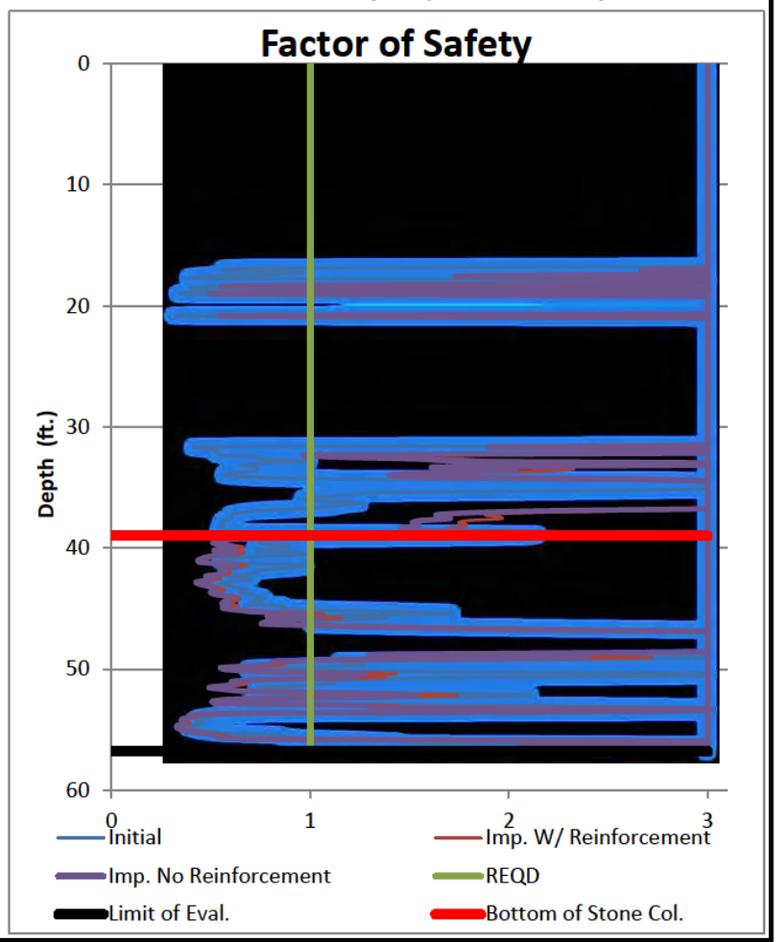
Stone Column Design Information

L = 39-ft			ϕ_c = 50
Column Spacing 8-ft			Ec = 750.0-tsf
Kac = 0.1324743	$=TAN^2(45-(\phi_c/2)$		Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 56.8-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 15.9-ft.

Calculated Min: 2.85-in via T&S, Min never < 0.5 in.

Calculated Max: 3.07-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 12.8-ft.

Calculated Min: 2.36-in via T&S, Min never < 0.5 in.

Calculated Max: 2.48-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 12.3-ft.

Calculated Min: 2.26-in via T&S, Min never < 0.5 in.

Calculated Max: 2.34-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Worksheet Tab: 14-Elastic Sett. (Imp.) Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 2 IMPROVED CASE: CPT-145

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment

= $1-0.5*(q/(q_{bar}-q)$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

$q = \gamma Df$

Stone Column Length 39-ft

Reduction factor on Priebe: 1

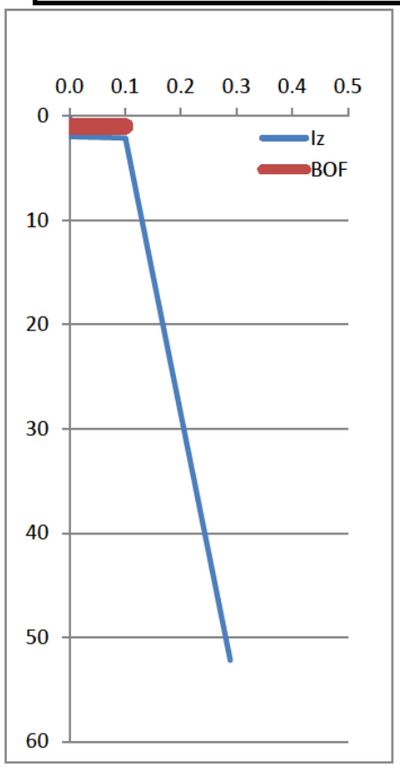
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	
L	212.7-ft	Equivalent Area for 240' Ø tank
B	212.7-ft	
fill density	130-pcf	Assigned by Designer
Elevation of Top of Soil Profile	27.86	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es=X \cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=$**0.5**B=	106.3
I_z = 0	at depth	$z=z_2=$**2.0**B=	425.4
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=$**1.0**B=	212.7
I_z = 0	at depth	$z=z_2=$**4.0**B=	850.8
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

$\Sigma(Iz/Es)\Delta z$ (ft2/lb)= 1.93E-05

Total Elastic Settlement (Flexible foundation)

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0741-ft 0.89-in

Total Rapid Elastic Settlement

$S_e=C_1(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0482-ft 0.58-in

Long Term Settlement 0.31-in

Calculated Consolidation Type Settlement:

Total:	2.83-in	Adjusted for footing Type
Recomp.	0.11-in	Adjusted for footing Type
Virgin Sett.	2.71-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	2.94-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:

Total: 3.72-in

Rapid: 3.52-in

Long Term: 0.20-in

Hayward Baker Inc.

THINK SAFE

SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 2		
Job no.	860079	Design by: M. Rohrbach	Date: 14-Apr-15
File	CPT-Liquefaction-Boulanger	Checked	
Spreadsheet Rev.	23-Dec-14	Reviewed	

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-X-4

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	16.6-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	28.58	ft.
Elevation of bottom of Soil Profile	-25.72	ft.
Elevation of Installation	23.5	
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	54.3-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	412.10-psf	

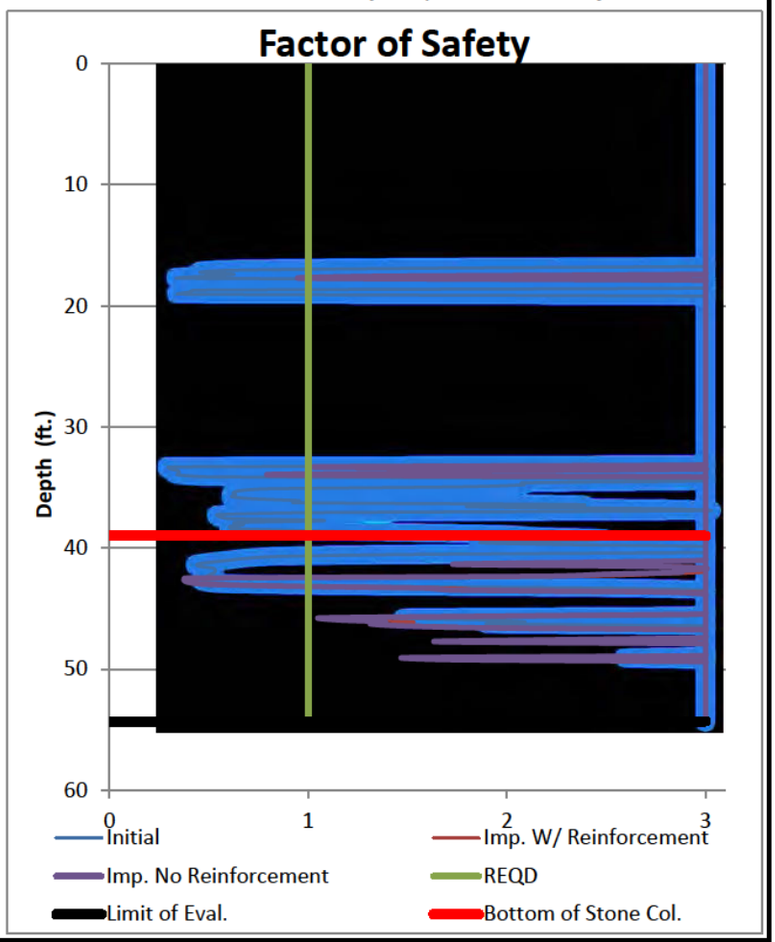
Stone Column Design Information

L =	39-ft		ϕ_c = 50
Column Spacing	8-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 54.3-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 7.4-ft.

Calculated Min: 1.41-in via T&S, Min never < 0.5 in.

Calculated Max: 1.44-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 1.3-ft.

Calculated Min: 0.50-in via T&S, Min never < 0.5 in.

Calculated Max: 0.50-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 1.0-ft.

Calculated Min: 0.50-in via T&S, Min never < 0.5 in.

Calculated Max: 0.50-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 2

IMPROVED CASE: CPT-X-4

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment

= $1-0.5*(q/(q_{bar}-q)$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

$q = \gamma Df$

Stone Column Length 39-ft

Reduction factor on Priebe: 1

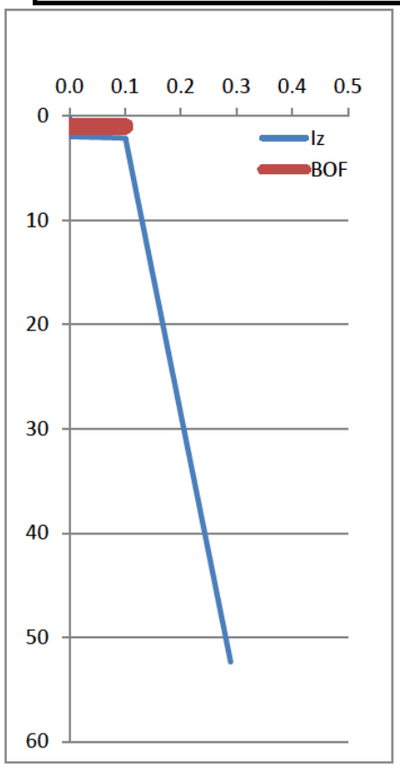
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	
L	212.7-ft	Equivalent Area for 240' Ø tank
B	212.7-ft	
fill density	130-pcf	Assigned by Designer
Elevation of Top of Soil Profile	28.58	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es=X \cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1$=**0.5**B=	106.3
I_z = 0	at depth	$z=z_2$=**2.0**B=	425.4
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1$=**1.0**B=	212.7
I_z = 0	at depth	$z=z_2$=**4.0**B=	850.8
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

Σ(Iz/Es)Δz (ft2/lb)= 1.84E-05

Total Elastic Settlement (Flexible foundation)

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0707-ft 0.85-in

Total Rapid Elastic Settlement

$S_e=C_1(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0459-ft 0.55-in

Long Term Settlement 0.30-in

Calculated Consolidation Type Settlement:

Total:	4.11-in	Adjusted for footing Type
Recomp.	0.16-in	Adjusted for footing Type
Virgin Sett.	3.95-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	4.27-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:

Total: 4.96-in

Rapid: 4.82-in

Long Term: 0.14-in

Mathcad – Tank 2
Bearing Checks

Calculations for Tank Edge and Base Stability (Tank 2):

Reference: Stability of Steel Oil Storage Tanks, Duncan, D'Orazio, 1984

GEOMETERY

Tank Diameter: $D := 240ft$

Existing Elev: $ELEVex := 29.26ft$

Tank Bottom Elev: $TankB := 31.75ft$

Fill and LTP Thickness: $Tp := TankB - (ELEVex - 4ft)$ $\quad Tp = 6.49 \cdot ft$

Overburden Thickness: $Tob := TankB - Tp - 16ft$ $\quad Tob = 9.26 \cdot ft$

Saturated Clay above the GW will consolidate quickly

Clay Thickness:

CPT	Clay Thickess	Clay and Silt
141	8'	16'
142	3'	15'
143	7.6'	14.6'
144	7.3'	13.3'
135	8'	14.8'

$Tclay := 9ft$ $\quad Tclay\_silt := 16ft$

Stone column replacement ratio: $Ar := 0.11$

SOIL PROPERTIES

Overburden unit weight: $\gamma ob := 96.8pcf$ Post Improvment - GI Sheets REF CPT114B(S)

Tore Vane Strenght of Clay: $TV := 0.8tsf$ Refernce GRI Report Boring B-1,B-2, B-4, B-5, B-6, B-7, B-8, B-10, B-12 and B-14 through B-22.

$TV = 1600 \cdot psf$

Friction angle of stone: $\phi stone := 50°$

Friction angle of silt $\phi silt := 20°$

LTP unit weight: $\gamma p := 130pcf$

SOIL STRENGTH AND STRESSES

Bearing Pressure due to Liquid in Tank: $q := 2882psf$ At full hydrotest load

Effective stress at top of clay: $\sigma ob := (\gamma ob \cdot Tob) + (\gamma p \cdot Tp)$ $\quad \sigma ob = 1740.068 \cdot psf$

Shear Strenght of SC: $Ssc := \sigma ob \cdot \tan(\phi stone)$ $\quad Ssc = 2073.732 \cdot psf$

Cohesive strenght of clay: $Cu := \frac{TV}{2}$ $\quad Cu = 800 \cdot psf$

Improved clay strength: $Su\_clay := Ar \cdot Ssc + (1 - Ar) \cdot Cu$

$Su\_clay = 940.111 \cdot psf$ After equation 10

Improved Clay-Silt Strenght $Su\_clay\_silt := Ar \cdot Ssc + (1 - Ar)(Cu + \sigma ob \cdot \tan(20))$

$Su\_clay\_silt = 4404.713 \cdot psf$ After equation 10

BASE SHEAR STABILITY EVALUATION - AT FULL HYDRO TEST - NO STRENGTH GAIN

$Dc := D + Tp + Tob \qquad Dc = 255.75 \cdot ft$ Equation 1

Bearing pressure at top of clay:

$$Qclay := q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2 + \gamma ob \cdot Tob$$ Equation 2

$$Qclay = 4218.04 \cdot psf$$

FOR CLAY LAYER ONLY

For a clay layer thicker than one sixth of the tank diameter $Nc1 := 6.1$ Equation 4

For a clay layer thinner than one sixth of the tank diamete $Nc2 := 4.1 + \left(\frac{Dc}{3Tclay}\right)$ Equation 5

$$Nc2 = 13.572$$

Place holder for easier coding: $PH := \frac{Dc}{Tclay}$

$Nc := if[(PH < 6), Nc1, Nc2] \qquad Nc = 13.572$

$Qult := Su\_clay \cdot Nc + \gamma ob \cdot Tob \qquad Qult = 13655.757 \cdot psf$

$$FS\_base\_clay := \frac{Su\_clay \cdot Nc}{\left[q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2\right]} \qquad FS\_base\_clay = 3.841$$ Equation 8

FOR COMBINED CLAY-SILT LAYER

For a clay layer thicker than one sixth of the tank diameter $Nc1 := 6.1$ Equation 4

For a clay layer thinner than one sixth of the tank diamete $Nc2 := 4.1 + \left(\frac{Dc}{3Tclay\_silt}\right)$ Equation 5

$$Nc2 = 9.428$$

Place holder for easier coding: $PH := \frac{Dc}{Tclay\_silt}$

$Nc := if[(PH < 6), Nc1, Nc2] \qquad Nc = 9.428$

$Qult := Su\_clay\_silt \cdot Nc + \gamma ob \cdot Tob \qquad Qult = 42424.556 \cdot psf$

$$FS\_base\_clay\_silt := \frac{Su\_clay\_silt \cdot Nc}{\left[q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2\right]} \qquad FS\_base\_clay\_silt = 12.502$$ Equation 8

EDGE SHEAR STABILITY EVALUATION - AT FULL HYDRO TEST - NO STRENGTH GAIN

IMPROVED CLAY

$Qapp := q + \gamma p \cdot Tp \qquad Qapp = 3725.7 \cdot psf$ Equation 11

With of segment that might fail $B := \begin{pmatrix} 10 \\ 20 \\ 40 \\ 60 \\ 80 \\ 100 \\ 120 \end{pmatrix} ft$

$Nc := 5.2 + \frac{B}{D} \qquad Nc = \begin{pmatrix} 5.242 \\ 5.283 \\ 5.367 \\ 5.45 \\ 5.533 \\ 5.617 \\ 5.7 \end{pmatrix}$

$Qall := Su\_clay \cdot Nc + \gamma p \cdot Tp \qquad Qall = \begin{pmatrix} 5771.446 \\ 5810.617 \\ 5888.96 \\ 5967.303 \\ 6045.645 \\ 6123.988 \\ 6202.33 \end{pmatrix} \cdot psf$

$FS\_edge\_clay := \frac{Qall}{Qapp} \qquad FS\_edge\_clay = \begin{pmatrix} 1.549 \\ 1.56 \\ 1.581 \\ 1.602 \\ 1.623 \\ 1.644 \\ 1.665 \end{pmatrix}$

EDGE SHEAR STABILITY EVALUATION - AT FULL HYDRO TEST - NO STRENGTH GAIN

IMPROVED CLAY and SILT

$Qapp := q + \gamma p \cdot Tp$ $\quad Qapp = 3725.7 \cdot psf$ $\quad$ Equation 11

With of segment that might fail $\quad B := \begin{pmatrix} 10 \\ 20 \\ 40 \\ 60 \\ 80 \\ 100 \\ 120 \end{pmatrix} ft$ $\quad Nc := 5.2 + \frac{B}{D}$ $\quad Nc = \begin{pmatrix} 5.242 \\ 5.283 \\ 5.367 \\ 5.45 \\ 5.533 \\ 5.617 \\ 5.7 \end{pmatrix}$

$Qall := Su\_clay\_silt \cdot Nc + \gamma p \cdot Tp \quad Qall = \begin{pmatrix} 23931.739 \\ 24115.269 \\ 24482.328 \\ 24849.388 \\ 25216.447 \\ 25583.507 \\ 25950.566 \end{pmatrix} \cdot psf$

$FS\_edge\_clay\_silt := \frac{Qall}{Qapp} \quad FS\_edge\_clay\_silt = \begin{pmatrix} 6.423 \\ 6.473 \\ 6.571 \\ 6.67 \\ 6.768 \\ 6.867 \\ 6.965 \end{pmatrix}$

SUMMARY OF BASE AND EDGE SHEAR STABILITY EVALUATION - AT FULL HYDRO TEST - NO STRENGTH GAIN

$FS\_base\_clay = 3.841$

$FS\_base\_clay\_silt = 12.502$

$FS\_edge\_clay = \begin{pmatrix} 1.549 \\ 1.56 \\ 1.581 \\ 1.602 \\ 1.623 \\ 1.644 \\ 1.665 \end{pmatrix}$ $\quad FS\_edge\_clay\_silt = \begin{pmatrix} 6.423 \\ 6.473 \\ 6.571 \\ 6.67 \\ 6.768 \\ 6.867 \\ 6.965 \end{pmatrix}$

BASE SHEAR STABILITY EVALUATION - IN SERVICE - NO STRENGTH GAIN

$q := 2500psf$

$Dc := D + Tp + Tob \qquad Dc = 255.75 \cdot ft$ Equation 1

Bearing pressure at top of clay:

$$Qclay := q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2 + \gamma ob \cdot Tob$$ Equation 2

$$Qclay = 3881.641 \cdot psf$$

FOR CLAY LAYER ONLY

For a clay layer thicker than one sixth of the tank diameter $Nc1 := 6.1$ Equation 4

For a clay layer thinner than one sixth of the tank diamete $Nc2 := 4.1 + \left(\frac{Dc}{3Tclay}\right)$ Equation 5

$$Nc2 = 13.572$$

Place holder for easier coding: $PH := \frac{Dc}{Tclay}$

$Nc := if[(PH < 6), Nc1, Nc2] \qquad Nc = 13.572$

$Qult := Su\_clay \cdot Nc + \gamma ob \cdot Tob \qquad Qult = 13655.757 \cdot psf$

$$FS\_base\_clay := \frac{Su\_clay \cdot Nc}{\left[q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2\right]} \qquad FS\_base\_clay = 4.274$$ Equation 8

FOR COMBINED CLAY-SILT LAYER

For a clay layer thicker than one sixth of the tank diameter $Nc1 := 6.1$ Equation 4

For a clay layer thinner than one sixth of the tank diamete $Nc2 := 4.1 + \left(\frac{Dc}{3Tclay\_silt}\right)$ Equation 5

$$Nc2 = 9.428$$

Place holder for easier coding: $PH := \frac{Dc}{Tclay\_silt}$

$Nc := if[(PH < 6), Nc1, Nc2] \qquad Nc = 9.428$

$Qult := Su\_clay\_silt \cdot Nc + \gamma ob \cdot Tob \qquad Qult = 42424.556 \cdot psf$

$$FS\_base\_clay\_silt := \frac{Su\_clay\_silt \cdot Nc}{\left[q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2\right]} \qquad FS\_base\_clay\_silt = 13.911$$ Equation 8

EDGE SHEAR STABILITY EVALUATION - IN SERVICE - NO STRENGTH GAIN

IMPROVED CLAY

$Qapp := q + \gamma p \cdot Tp$ $\quad Qapp = 3343.7 \cdot psf$ Equation 11

With of segment that might fail $\quad B := \begin{pmatrix} 10 \\ 20 \\ 40 \\ 60 \\ 80 \\ 100 \\ 120 \end{pmatrix} ft$

$Nc := 5.2 + \frac{B}{D}$ $\quad Nc = \begin{pmatrix} 5.242 \\ 5.283 \\ 5.367 \\ 5.45 \\ 5.533 \\ 5.617 \\ 5.7 \end{pmatrix}$

$Qall := Su\_clay \cdot Nc + \gamma p \cdot Tp$ $\quad Qall = \begin{pmatrix} 5771.446 \\ 5810.617 \\ 5888.96 \\ 5967.303 \\ 6045.645 \\ 6123.988 \\ 6202.33 \end{pmatrix} \cdot psf$

$FS\_edge\_clay := \frac{Qall}{Qapp}$ $\quad FS\_edge\_clay = \begin{pmatrix} 1.726 \\ 1.738 \\ 1.761 \\ 1.785 \\ 1.808 \\ 1.832 \\ 1.855 \end{pmatrix}$

EDGE SHEAR STABILITY EVALUATION - IN SERVICE - NO STRENGTH GAIN

IMPROVED CLAY and SILT

$Qapp := q + \gamma p \cdot Tp$ $\quad Qapp = 3343.7 \cdot psf$ $\quad$ Equation 11

With of segment that might fail $\quad B := \begin{pmatrix} 10 \\ 20 \\ 40 \\ 60 \\ 80 \\ 100 \\ 120 \end{pmatrix} ft$ $\quad Nc := 5.2 + \frac{B}{D}$ $\quad Nc = \begin{pmatrix} 5.242 \\ 5.283 \\ 5.367 \\ 5.45 \\ 5.533 \\ 5.617 \\ 5.7 \end{pmatrix}$

$$Qall := Su\_clay\_silt \cdot Nc + \gamma p \cdot Tp \quad Qall = \begin{pmatrix} 23931.739 \\ 24115.269 \\ 24482.328 \\ 24849.388 \\ 25216.447 \\ 25583.507 \\ 25950.566 \end{pmatrix} \cdot psf$$

$$FS\_edge\_clay\_silt := \frac{Qall}{Qapp} \quad FS\_edge\_clay\_silt = \begin{pmatrix} 7.157 \\ 7.212 \\ 7.322 \\ 7.432 \\ 7.541 \\ 7.651 \\ 7.761 \end{pmatrix}$$

SUMMARY OF BASE AND EDGE SHEAR STABILITY EVALUATION - IN SERVCIE - NO STRENGTH GAIN

$FS\_base\_clay = 4.274$

$FS\_base\_clay\_silt = 13.911$

$$FS\_edge\_clay = \begin{pmatrix} 1.726 \\ 1.738 \\ 1.761 \\ 1.785 \\ 1.808 \\ 1.832 \\ 1.855 \end{pmatrix} \qquad FS\_edge\_clay\_silt = \begin{pmatrix} 7.157 \\ 7.212 \\ 7.322 \\ 7.432 \\ 7.541 \\ 7.651 \\ 7.761 \end{pmatrix}$$

Units

$kips \equiv 1000 \cdot lbf$

$ksf := 1000 \cdot psf$

$tsf := 2 \cdot ksf$

$kPa := 1000 \cdot Pa$

$pi := 3.14159$

$ksi \equiv \frac{1000 \cdot lbf}{in^2}$

$pcf \equiv \frac{lbf}{ft^3}$

$in^2 = 0.001\, m^2$

$psi \equiv \frac{lbf}{in^2}$

$psf \equiv \frac{lbf}{ft^2}$

$pcf \equiv \frac{lbf}{ft^3}$

$kcf \equiv \frac{k}{ft^3}$

$k \equiv kips$

$N \equiv newton$

$ORIGIN \equiv 1$

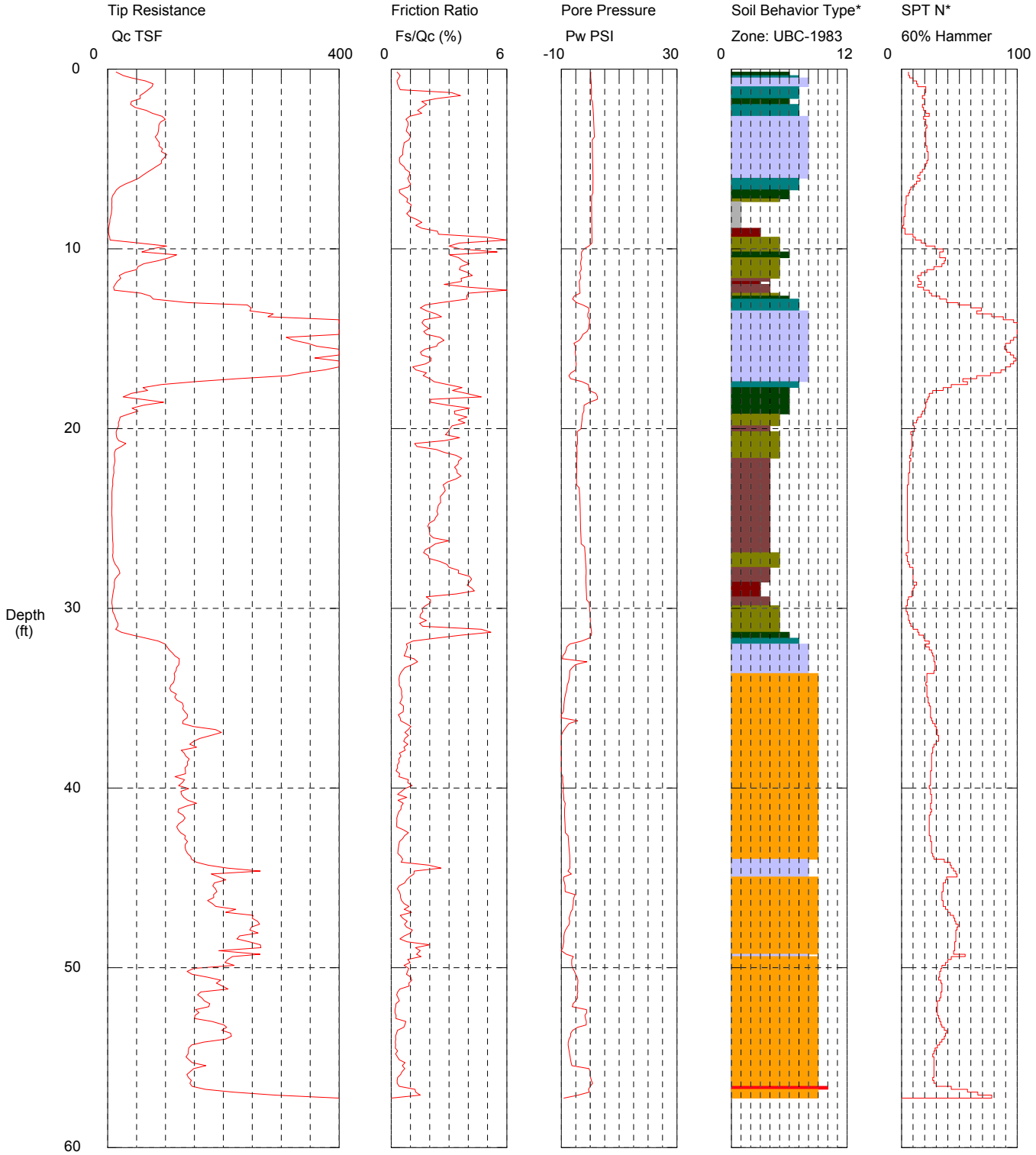
$MPa := 10^6 \cdot Pa$

$kN \equiv 1000 newton$

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-141
Cone Used: DDG1254

CPT Date/Time: 11/20/2014 1:01:57 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 57.25 feet

Depth Increment = 0.197 feet

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

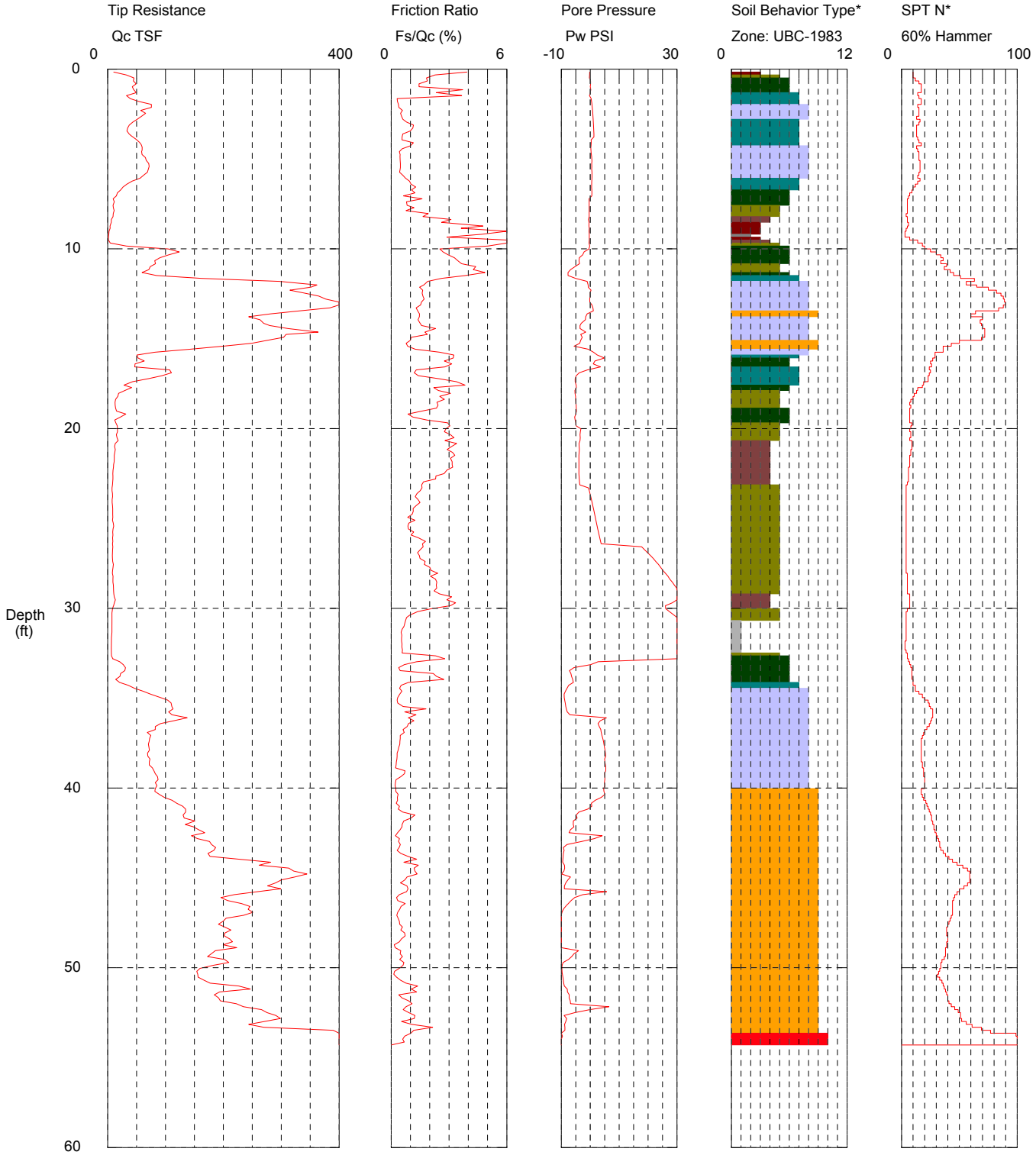
InSitu Engineering Predrilled top 10 feet

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

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-142
Cone Used: DDG1254

CPT Date/Time: 11/20/2014 8:31:51 AM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 54.30 feet

Depth Increment = 0.164 feet

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

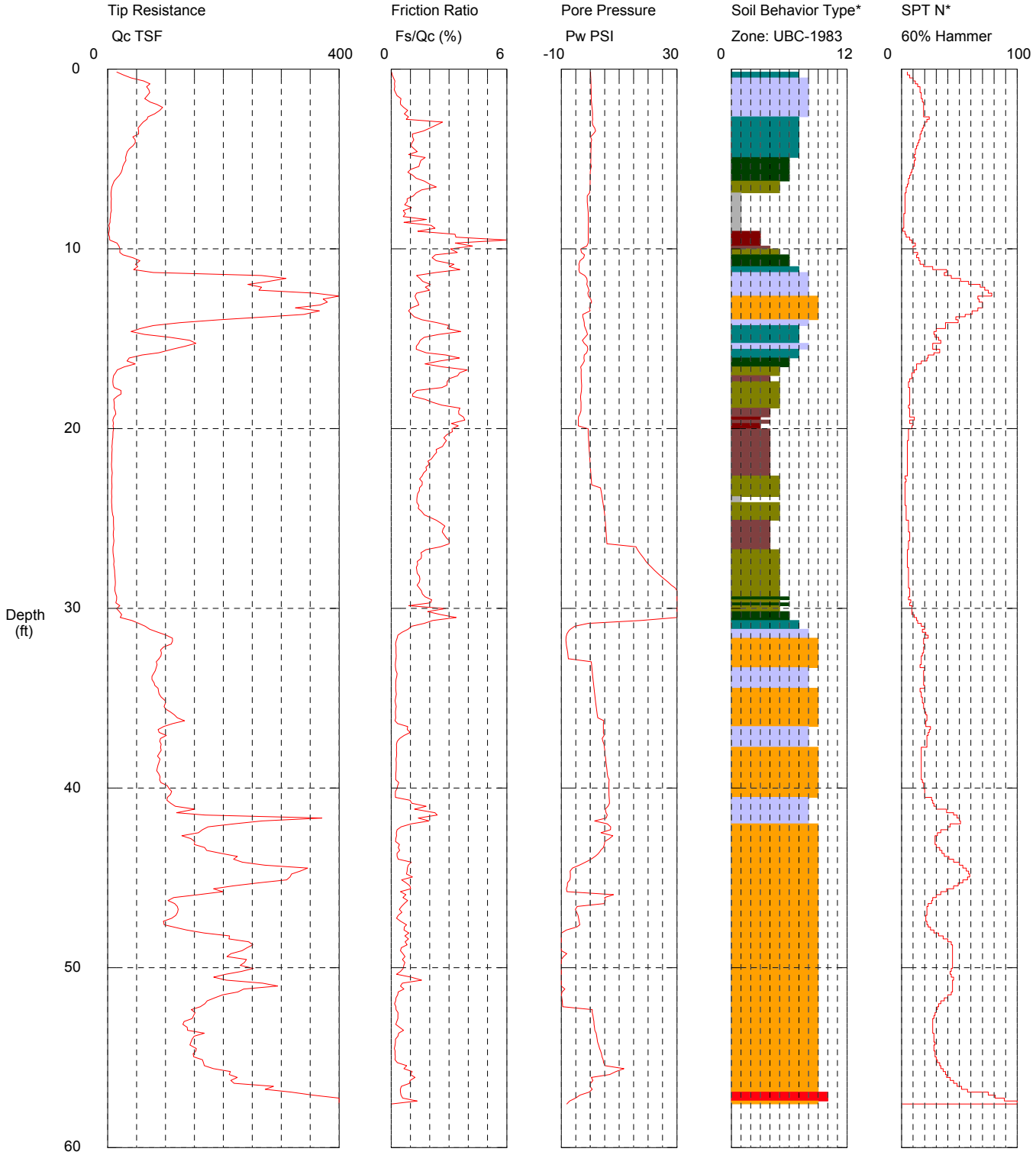
InSitu Engineering Predrilled top 10 feet

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

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-143(S)
Cone Used: DDG1254

CPT Date/Time: 11/20/2014 10:53:36 AM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 57.58 feet

Depth Increment = 0.164 feet

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

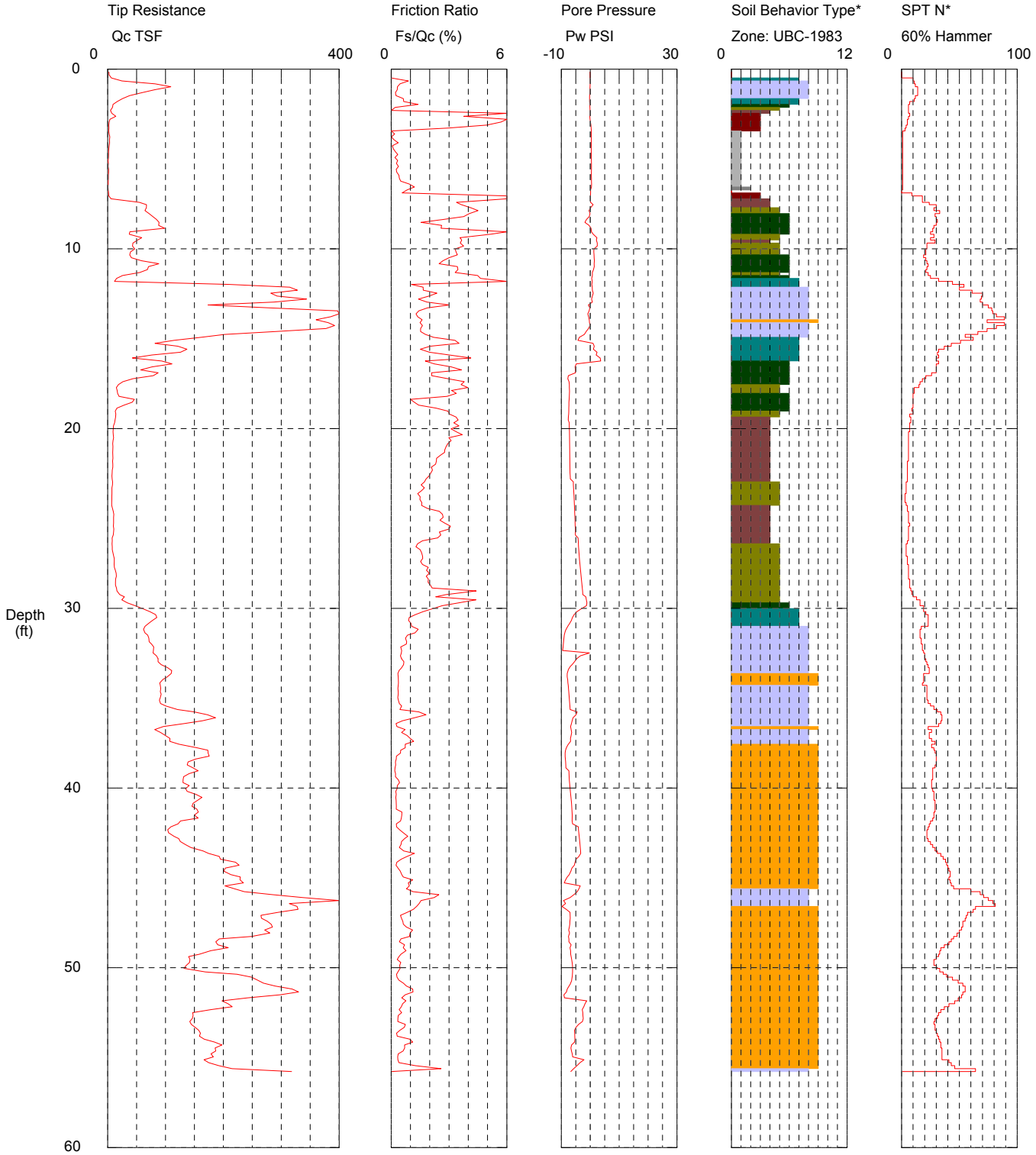
InSitu Engineering Predrilled top 10 feet

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

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-144
Cone Used: DDG1254

CPT Date/Time: 11/20/2014 3:48:24 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 55.77 feet

Depth Increment = 0.164 feet

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

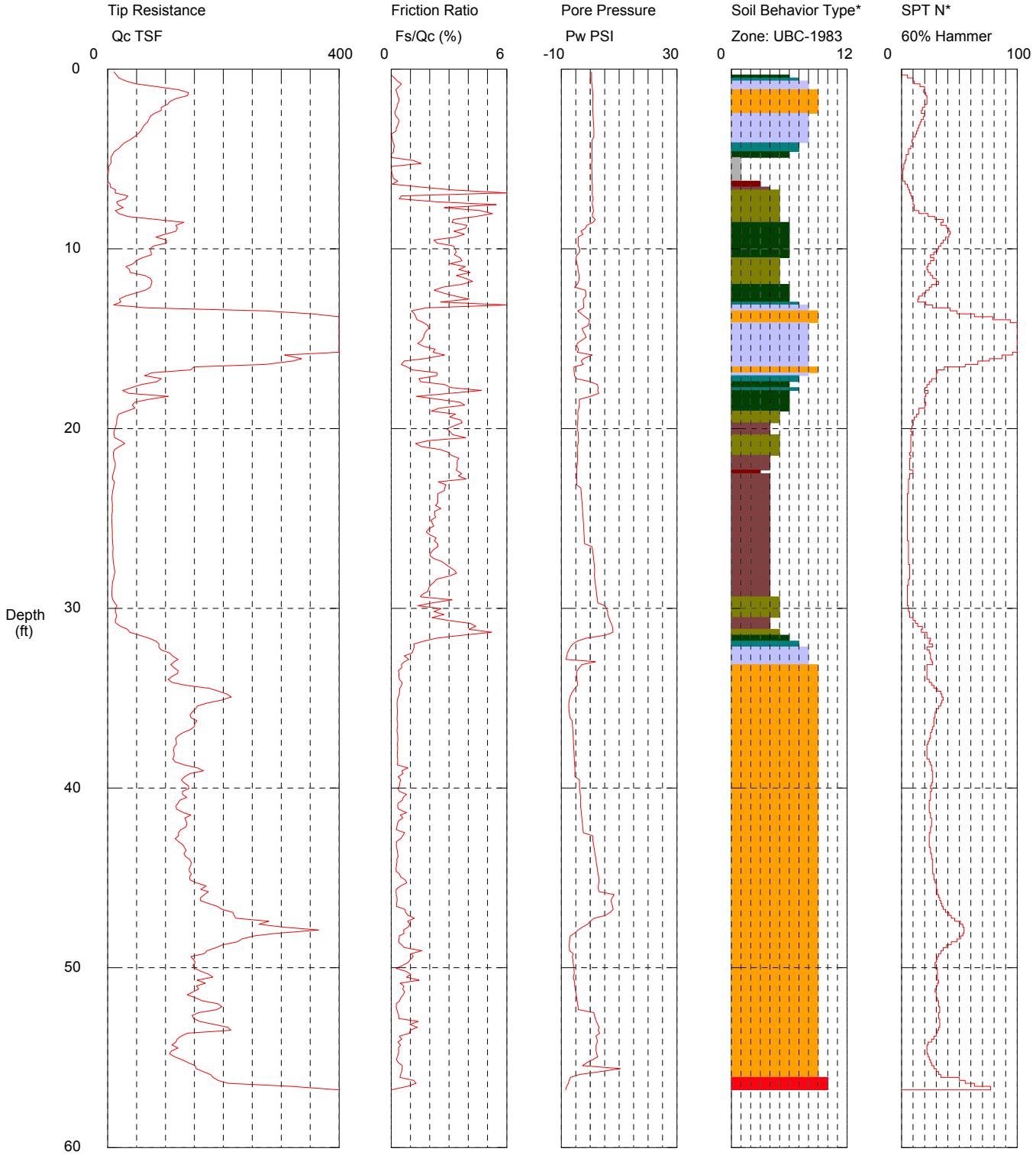
InSitu Engineering Predrilled top 7 feet

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

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-145
Cone Used: DDG1254

CPT Date/Time: 11/20/2014 2:26:23 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 56.79 feet

Depth Increment = 0.164 feet

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

InSitu Engineering Predrilled top 9 feet

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

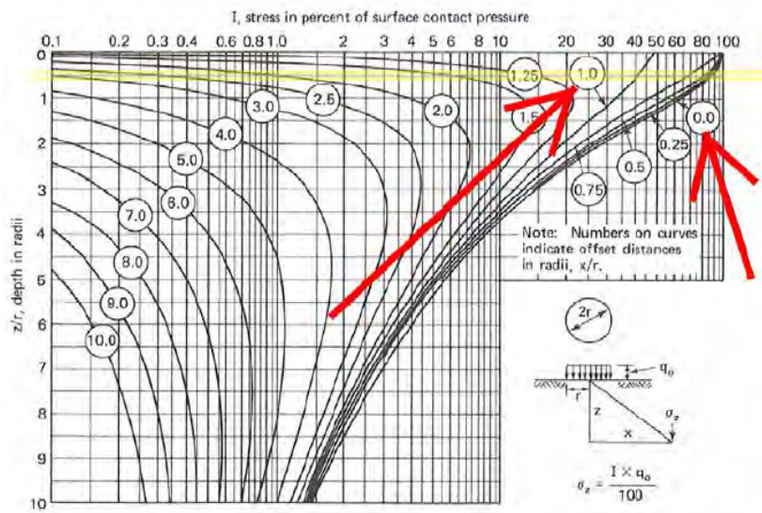
EX-0001-007032-PCE

Tank 3

Design Summary
Tank 3

This table assumes the exploration is directly below the center of the tank.

Tank 2		Orig	Imp.	Improved		Original	
Soil Profile	Depth	Liq Sett.	Liq Sett.	Rapid Sett.	Long Term Sett.	Rapid Sett.	Long Term Sett.
HB-CPT-121	53.8-ft	5.01-in.	1.26-in.	2.21-in.	.16-in.	1.13-in.	2.92-in.
HB-CPT-124	34.5-ft	2.66-in.	.5-in.	1.03-in.	.11-in.	.58-in.	1.31-in.
HB-CPT-126	57.1-ft	3.8-in.	1.92-in.	1.16-in.	.2-in.	1.43-in.	1.68-in.
HB-CPT-125	57.4-ft	6.22-in.	2.34-in.	1.57-in.	.23-in.	1.2-in.	2.01-in.
HB-CPT-122	57.1-ft	4.26-in.	2.17-in.	3.01-in.	.21-in.	1.59-in.	4.-in.
HB-CPT-123(s)	56.8-ft	6.27-in.	2.32-in.	.68-in.	.2-in.	1.03-in.	.88-in.
Average:		**4.7-in.**	**2.-in.**	**1.79-in.**	**.18-in.**	**1.19-in.**	**2.38-in.**



This table calculates Settlement at the edge of the tank.

Tank 2		Orig	Imp.	Improved		Original	
Soil Profile	Depth	Liq Sett.	Liq Sett.	Rapid Sett.	Long Term Sett.	Rapid Sett.	Long Term Sett.
HB-CPT-121	53.8-ft	5.01-in.	1.26-in.	1.1-in.	.08-in.	.57-in.	1.46-in.
HB-CPT-124	34.5-ft	2.66-in.	.5-in.	.52-in.	.06-in.	.29-in.	.65-in.
HB-CPT-126	57.1-ft	3.8-in.	1.92-in.	.58-in.	.1-in.	.71-in.	.84-in.
HB-CPT-125	57.4-ft	6.22-in.	2.34-in.	.78-in.	.11-in.	.6-in.	1.-in.
HB-CPT-122	57.1-ft	4.26-in.	2.17-in.	1.5-in.	.11-in.	.8-in.	2.-in.
HB-CPT-123(s)	56.8-ft	4.26-in.	2.17-in.	.68-in.	.2-in.	1.03-in.	.88-in.
Average:		**4.39-in.**	**1.64-in.**	**.9-in.**	**.09-in.**	**.59-in.**	**1.19-in.**

Note: No reduction to total soil settlement is taken for the liquefaction condition.

Worksheet Tab: 11- Boulanger (improved)

Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Hayward Baker Inc. THINK SAFE	SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 3			
	Job no.	860079	Design by:	M. Rohrbach	Date: 14-Apr-15
	File	CPT-Liquefaction-Boulanger	Checked		
	Spreadsheet Rev.	23-Dec-14	Reviewed		

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-121

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	17.7-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	29.73	ft.
Elevation of bottom of Soil Profile	-24.08	ft.
Elevation of Installation	25	Work from Existing Grade
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	53.8-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	262.60-psf	

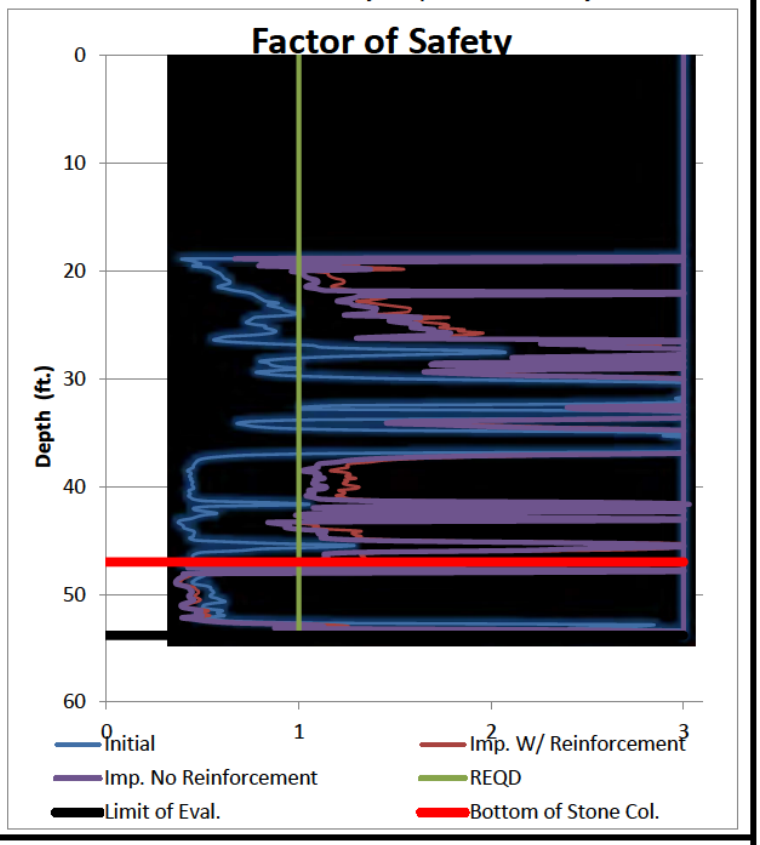
Stone Column Design Information

L =	47-ft		ϕ_c = 50
Column Spacing	8-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 53.8-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 25.4-ft.

Calculated Min: 4.83-in via T&S, Min never < 0.5 in.

Calculated Max: 5.01-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 6.9-ft.

Calculated Min: 1.48-in via T&S, Min never < 0.5 in.

Calculated Max: 2.08-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 5.9-ft.

Calculated Min: 1.26-in via T&S, Min never < 0.5 in.

Calculated Max: 2.13-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 3 — IMPROVED CASE: CPT-121

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment
= $1-0.5*(q/(q_{bar}-q)$

C_2 = a correction factor top account for creep ion soil
= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

q = gDf

Stone Column Length 47-ft

Reduction factor on Priebe: 1

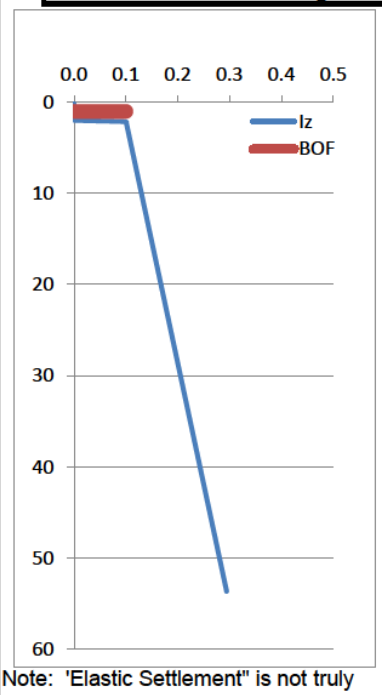
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	
L	212.7-ft	
B	212.7-ft	
fill density	130-pcf	
Elevation of Top of Soil Profile	29.73	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es=X\cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=0.5B$=	106.3
I_z = 0	at depth	$z=z_2=2.0B$=	425.4
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=1.0B$=	212.7
I_z = 0	at depth	$z=z_2=4.0B$=	850.8
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

Σ(Iz/Es)Δz (ft2/lb)= 1.44E-05

Total Elastic Settlement (Flexible foundation)

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0553-ft 0.66-in

Total Rapid Elastic Settlement

$S_e=C_1(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0359-ft 0.43-in

Long Term Settlement 0.23-in

Calculated Consolidation Type Settlement:

Total:	1.71-in	Adjusted for footing Type
Recomp.	0.07-in	Adjusted for footing Type
Virgin Sett.	1.64-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	1.78-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:
Total: 2.37-in
Rapid: 2.21-in
Long Term: 0.16-in

Worksheet Tab: 11- Boulanger (improved)

Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Hayward Baker Inc. THINK SAFE	SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 3			
	Job no.	860079	Design by:	M. Rohrbach	Date: 14-Apr-15
	File	CPT-Liquefaction-Boulanger	Checked		
	Spreadsheet Rev.	23-Dec-14	Reviewed		

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-122

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	17.3-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	29.33	ft.
Elevation of bottom of Soil Profile	-27.76	ft.
Elevation of Installation	25	Work from Existing Grade
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	57.1-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	314.60-psf	

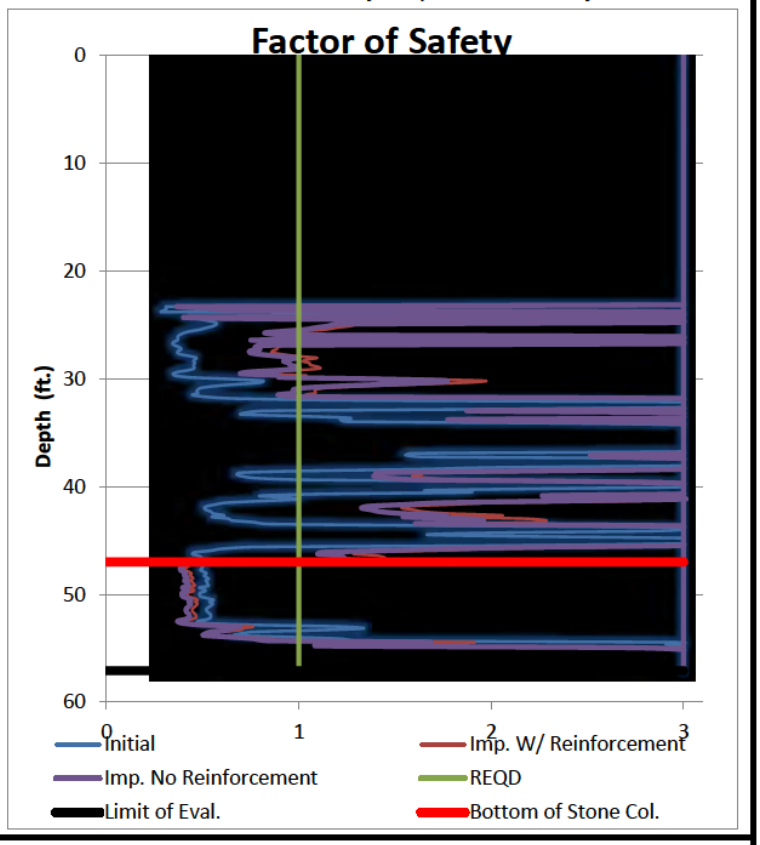
Stone Column Design Information

L =	47-ft		ϕ_c = 50
Column Spacing	8-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 57.1-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 20.3-ft.

Calculated Min: 4.14-in via T&S, Min never < 0.5 in.

Calculated Max: 4.26-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 12.6-ft.

Calculated Min: 2.73-in via T&S, Min never < 0.5 in.

Calculated Max: 2.88-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 10.0-ft.

Calculated Min: 2.17-in via T&S, Min never < 0.5 in.

Calculated Max: 2.41-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 3 — IMPROVED CASE: CPT-122

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e = C_1 C_2 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment
= $1 - 0.5*(q/(q_{bar} - q))$

C_2 = a correction factor top account for creep ion soil
= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

q = gDf

Stone Column Length 47-ft

Reduction factor on Priebe: 1

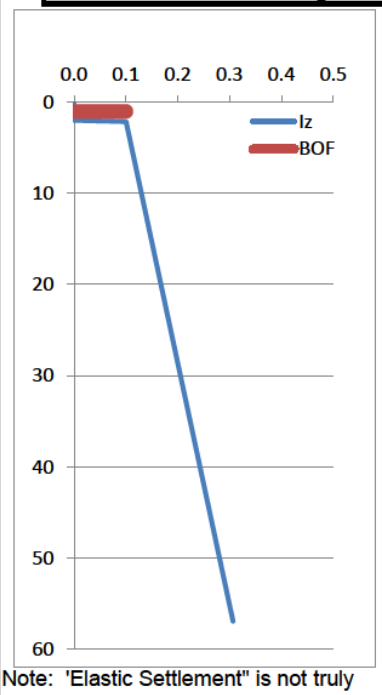
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	
L	212.7-ft	
B	212.7-ft	
fill density	130-pcf	
Elevation of Top of Soil Profile	29.33	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es = X \cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=0.5B$=	106.3
I_z = 0	at depth	$z=z_2=2.0B$=	425.4
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=1.0B$=	212.7
I_z = 0	at depth	$z=z_2=4.0B$=	850.8
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

$\Sigma(Iz/Es)\Delta z$ (ft2/lb)= **1.94E-05**

Total Elastic Settlement (Flexible foundation)

$S_e = C_1 C_2 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$= 0.0744-ft 0.89-in

Total Rapid Elastic Settlement

$S_e = C_1 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$= 0.0483-ft 0.58-in

Long Term Settlement 0.31-in

Calculated Consolidation Type Settlement:

Total:	2.33-in	Adjusted for footing Type
Recomp.	0.10-in	Adjusted for footing Type
Virgin Sett.	2.23-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	2.43-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:
Total: 3.22-in
Rapid: 3.01-in
Long Term: 0.21-in

Worksheet Tab: 11- Boulanger (improved)

Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Hayward Baker Inc. THINK SAFE	SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 3			
	Job no.	860079	Design by:	M. Rohrbach	Date: 14-Apr-15
	File	CPT-Liquefaction-Boulanger	Checked		
	Spreadsheet Rev.	23-Dec-14	Reviewed		

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-123(s)

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	16.7-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	28.7	ft.
Elevation of bottom of Soil Profile	-28.06	ft.
Elevation of Installation	25	Work from Existing Grade
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	56.8-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	396.50-psf	

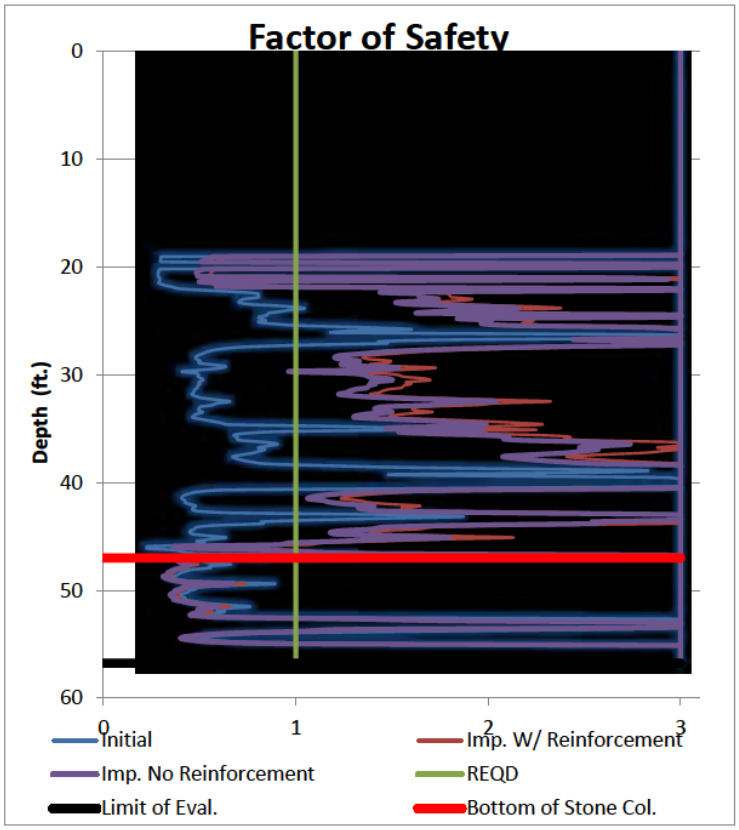
Stone Column Design Information

L =	47-ft		ϕ_c = 50
Column Spacing	8-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 56.8-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 28.2-ft.

Calculated Min: 6.18-in via T&S, Min never < 0.5 in.

Calculated Max: 6.27-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 10.0-ft.

Calculated Min: 2.48-in via T&S, Min never < 0.5 in.

Calculated Max: 2.68-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 9.2-ft.

Calculated Min: 2.32-in via T&S, Min never < 0.5 in.

Calculated Max: 2.43-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 3 — IMPROVED CASE: CPT-123(s)

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e = C_1 C_2 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment
= $1 - 0.5*(q/(q_{bar} - q))$

C_2 = a correction factor top account for creep ion soil
= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

q = gDf

Stone Column Length 47-ft

Reduction factor on Priebe: 1

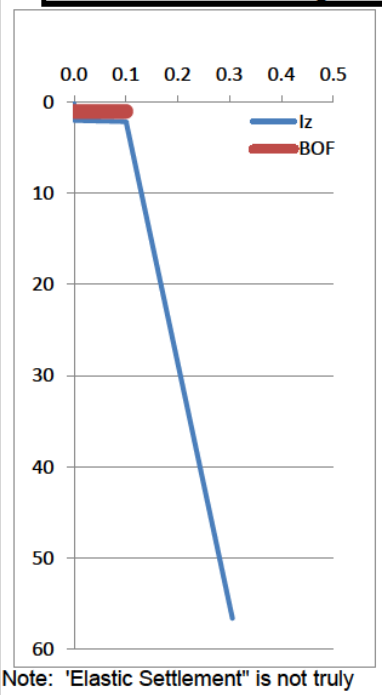
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	
L	212.7-ft	
B	212.7-ft	
fill density	130-pcf	
Elevation of Top of Soil Profile	28.7	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es = X \cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=0.5B$=	106.3
I_z = 0	at depth	$z=z_2=2.0B$=	425.4
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=1.0B$=	212.7
I_z = 0	at depth	$z=z_2=4.0B$=	850.8
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

$\Sigma(Iz/Es)\Delta z$ (ft2/lb)= **1.30E-05**

Total Elastic Settlement (Flexible foundation)

$S_e = C_1 C_2 (q_{bar} - q)\Sigma[(I_z/E_s)\Delta z]$= 0.0500-ft 0.60-in

Total Rapid Elastic Settlement

$S_e = C_1 (q_{bar} - q)\Sigma[(I_z/E_s)\Delta z]$= 0.0325-ft 0.39-in

Long Term Settlement 0.21-in

Calculated Consolidation Type Settlement:

Total:	0.28-in	Adjusted for footing Type
Recomp.	0.01-in	Adjusted for footing Type
Virgin Sett.	0.27-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	0.29-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:

Total: 0.88-in
Rapid: 0.68-in
Long Term: 0.20-in

Worksheet Tab: 11- Boulanger (improved)

Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Hayward Baker Inc. THINK SAFE	SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 3			
	Job no.	860079	Design by:	M. Rohrbach	Date: 14-Apr-15
	File	CPT-Liquefaction-Boulanger	Checked		
	Spreadsheet Rev.	23-Dec-14	Reviewed		

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-124

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	17.7-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	29.73	ft.
Elevation of bottom of Soil Profile	-4.72	ft.
Elevation of Installation	25	Work from Existing Grade
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	34.5-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	262.60-psf	

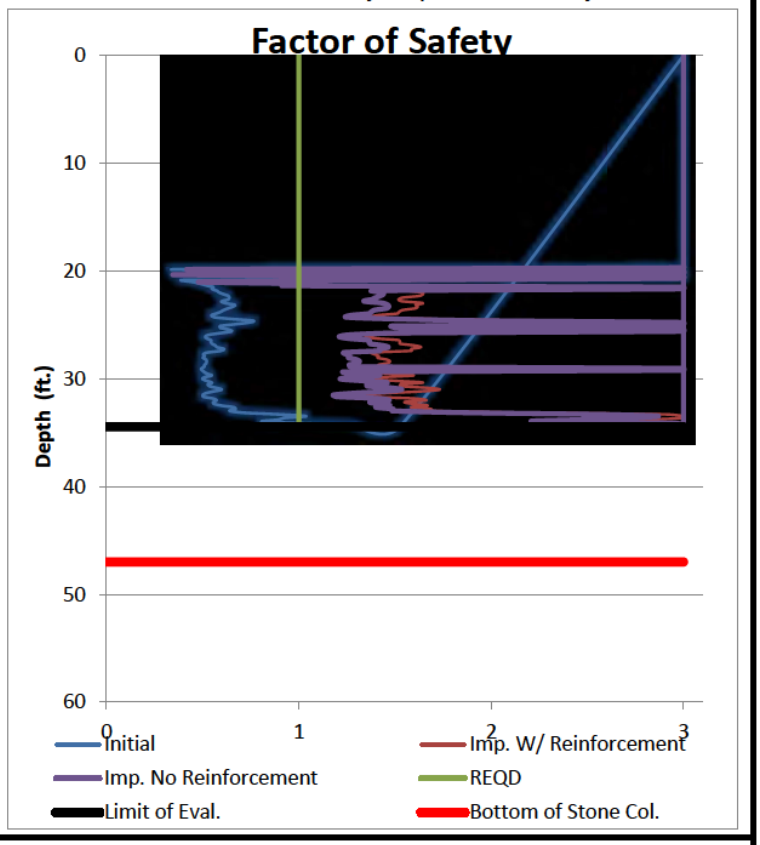
Stone Column Design Information

L =	47-ft		ϕ_c = 50
Column Spacing	8-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 34.5-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 13.5-ft.

Calculated Min: 2.62-in via T&S, Min never < 0.5 in.

Calculated Max: 2.66-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: .9-ft.

Calculated Min: 0.50-in via T&S, Min never < 0.5 in.

Calculated Max: 0.50-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: .5-ft.

Calculated Min: 0.50-in via T&S, Min never < 0.5 in.

Calculated Max: 0.50-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Worksheet Tab: 14-Elastic Sett. (Imp.) Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 3 IMPROVED CASE: CPT-124

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment

= $1-0.5*(q/(q_{bar}-q)$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

q = gDf

Stone Column Length 47-ft

Reduction factor on Priebe: 1

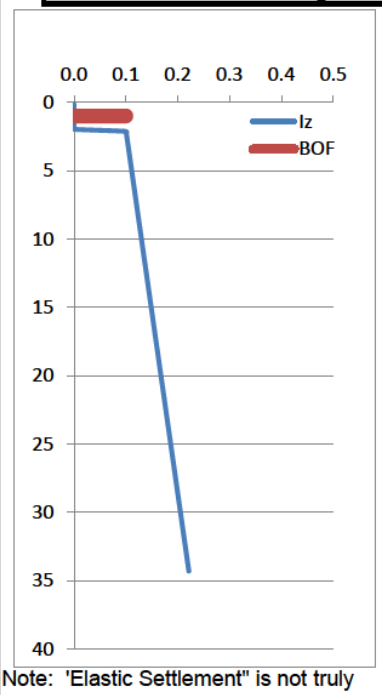
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	
L	212.7-ft	
B	212.7-ft	
fill density	130-pcf	
Elevation of Top of Soil Profile	29.73	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es=X\cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=0.5B$=	106.3
I_z = 0	at depth	$z=z_2=2.0B$=	425.4
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=1.0B$=	212.7
I_z = 0	at depth	$z=z_2=4.0B$=	850.8
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

Σ(Iz/Es)Δz (ft2/lb)= 8.70E-06

Total Elastic Settlement (Flexible foundation)

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0334-ft 0.40-in

Total Rapid Elastic Settlement

$S_e=C_1(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0217-ft 0.26-in

Long Term Settlement 0.14-in

Calculated Consolidation Type Settlement:

Total:	0.75-in	Adjusted for footing Type
Recomp.	0.03-in	Adjusted for footing Type
Virgin Sett.	0.72-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	0.77-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:

Total: 1.15-in

Rapid: 1.03-in

Long Term: 0.11-in

Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Hayward Baker Inc. THINK SAFE	SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 3			
	Job no.	860079	Design by:	M. Rohrbach	Date: 14-Apr-15
	File	CPT-Liquefaction-Boulanger	Checked		
	Spreadsheet Rev.	23-Dec-14	Reviewed		

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-125A

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	16.6-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	28.58	ft.
Elevation of bottom of Soil Profile	-28.83	ft.
Elevation of Installation	25	Work from Existing Grade
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	57.4-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	412.10-psf	

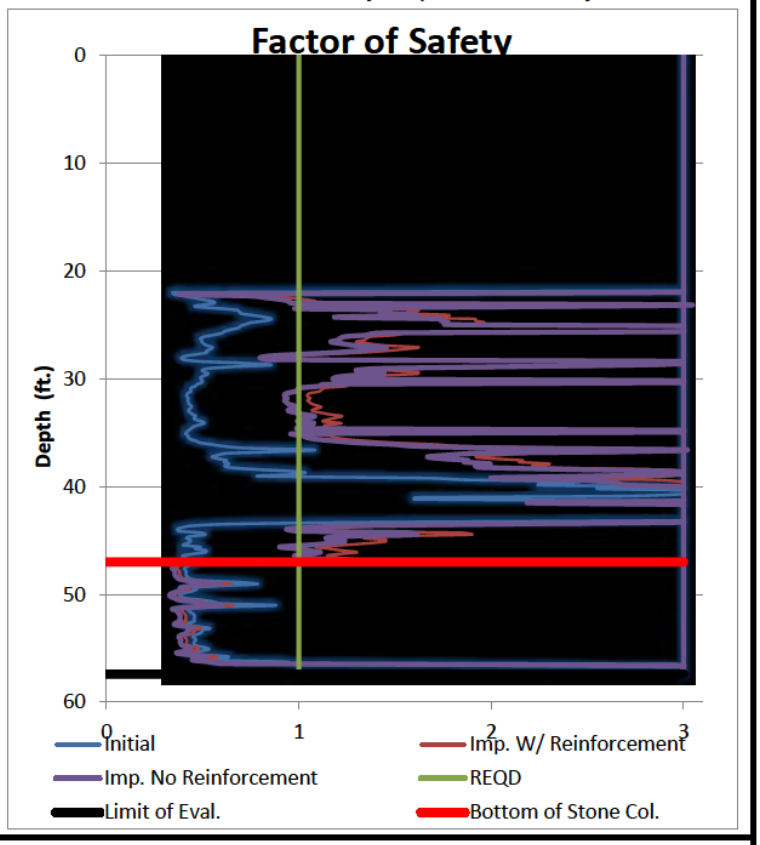
Stone Column Design Information

L =	47-ft		ϕ_c = 50
Column Spacing	8-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 57.4-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 30.0-ft.

Calculated Min: 6.20-in via T&S, Min never < 0.5 in.

Calculated Max: 6.22-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 14.8-ft.

Calculated Min: 3.26-in via T&S, Min never < 0.5 in.

Calculated Max: 3.75-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 10.5-ft.

Calculated Min: 2.34-in via T&S, Min never < 0.5 in.

Calculated Max: 2.86-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Worksheet Tab: 14-Elastic Sett. (Imp.) Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 3 IMPROVED CASE: CPT-125A

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment

= $1-0.5*(q/(q_{bar}-q)$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

q = gDf

Stone Column Length 47-ft

Reduction factor on Priebe: 1

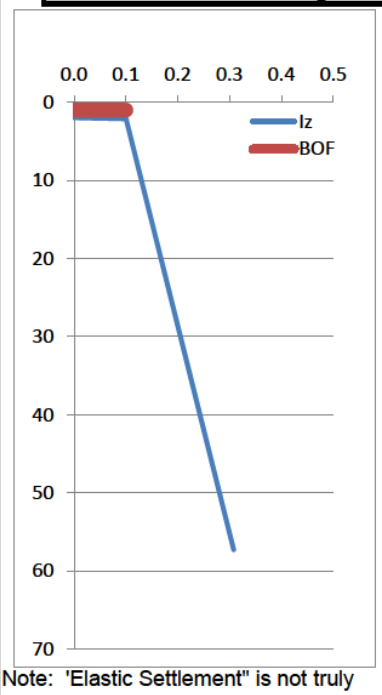
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	
L	212.7-ft	
B	212.7-ft	
fill density	130-pcf	
Elevation of Top of Soil Profile	28.58	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es=X\cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)

I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=0.5B$=	106.3
I_z = 0	at depth	$z=z_2=2.0B$=	425.4

For foundations with L/B ≥ 10

I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=1.0B$=	212.7
I_z = 0	at depth	$z=z_2=4.0B$=	850.8

Interpolation for L/B values between 1 and 10

I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

Σ(Iz/Es)Δz (ft2/lb)= 1.65E-05

Total Elastic Settlement (Flexible foundation)

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0634-ft 0.76-in

Total Rapid Elastic Settlement

$S_e=C_1(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0412-ft 0.49-in

Long Term Settlement 0.27-in

Calculated Consolidation Type Settlement:

Total:	1.03-in	Adjusted for footing Type
Recomp.	0.04-in	Adjusted for footing Type
Virgin Sett.	0.99-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	1.08-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:

Total: 1.79-in

Rapid: 1.57-in

Long Term: 0.23-in

Worksheet Tab: 11- Boulanger (improved)

Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Hayward Baker Inc. THINK SAFE	SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 3			
	Job no.	860079	Design by:	M. Rohrbach	Date: 14-Apr-15
	File	CPT-Liquefaction-Boulanger	Checked		
	Spreadsheet Rev.	23-Dec-14	Reviewed		

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-126

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	16.4-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	28.38	ft.
Elevation of bottom of Soil Profile	-28.71	ft.
Elevation of Installation	25	Work from Existing Grade
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	57.1-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	438.10-psf	

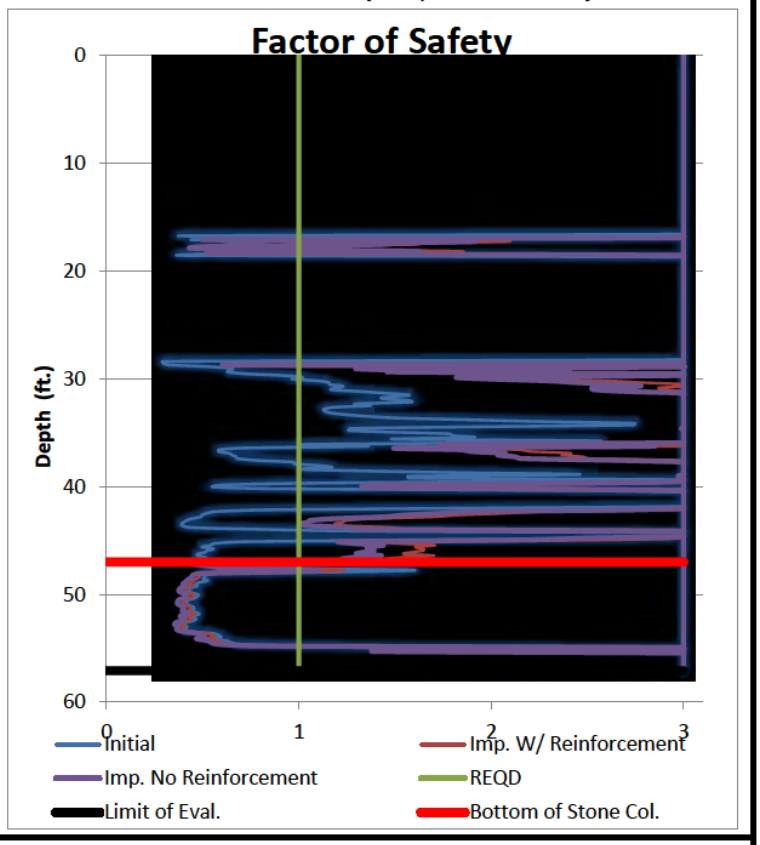
Stone Column Design Information

L =	47-ft		ϕ_c = 50
Column Spacing	8-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 57.1-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 17.1-ft.

Calculated Min: 3.53-in via T&S, Min never < 0.5 in.

Calculated Max: 3.80-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 8.5-ft.

Calculated Min: 1.96-in via T&S, Min never < 0.5 in.

Calculated Max: 2.07-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 8.4-ft.

Calculated Min: 1.92-in via T&S, Min never < 0.5 in.

Calculated Max: 2.08-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 3 — IMPROVED CASE: CPT-126

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment
= $1-0.5*(q/(q_{bar}-q)$

C_2 = a correction factor top account for creep ion soil
= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

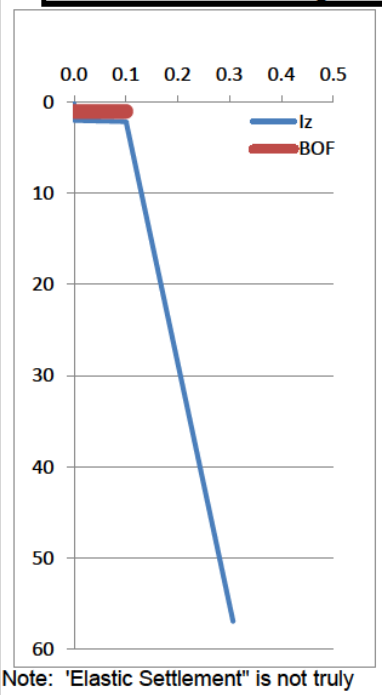
q = gDf

Stone Column Length 47-ft
Reduction factor on Priebe: 1

Foundation Type: Flexible

Correlation value between Es and q_c: $Es=X \cdot q_c$
x = 3.0
*default Value of X is 3

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	
L	212.7-ft	
B	212.7-ft	
fill density	130-pcf	Assigned by Designer
Elevation of Top of Soil Profile	28.38	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1$=0.5B=	106.3
I_z = 0	at depth	$z=z_2$=2.0B=	425.4
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1$=1.0B=	212.7
I_z = 0	at depth	$z=z_2$=4.0B=	850.8
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

$\Sigma(Iz/Es)\Delta z$ (ft2/lb)= **1.42E-05**

Total Elastic Settlement (Flexible foundation)
$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0543-ft 0.65-in

Total Rapid Elastic Settlement
$S_e=C_1(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0353-ft 0.42-in

Long Term Settlement 0.23-in

Calculated Consolidation Type Settlement:

Total:	0.71-in	Adjusted for footing Type
Recomp.	0.03-in	Adjusted for footing Type
Virgin Sett.	0.68-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	0.74-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:
Total: 1.36-in
Rapid: 1.16-in
Long Term: 0.20-in

Mathcad – Tank 3
Bearing Checks

Calculations for Tank Edge and Base Stability (Tank 3):

Reference: Stability of Steel Oil Storage Tanks, Duncan, D'Orazio, 1984

GEOMETERY

Tank Diameter: $D := 240ft$

Existing Elev: $ELEVex := 29.26ft$

Tank Bottom Elev: $TankB := 31.75ft$

Fill and LTP Thickness: $Tp := TankB - (ELEVex - 4ft)$ $\quad Tp = 6.49 \cdot ft$

Overburden Thickness: $Tob := TankB - Tp - 16ft$ $\quad Tob = 9.26 \cdot ft$

Saturated Clay above the GW will consolidate quickly

Clay Thickness:

CPT	Clay Thickess	Clay and Silt
121	10'	10'
122	9'	17.7'
123	7'	19.7'
124(s)	6'	7'
125	4'	16'

$Tclay := 10ft$ $\quad Tclay\_silt := 20ft$

Stone column replacement ratio: $Ar := 0.11$

SOIL PROPERTIES

Overburden unit weight: $\gamma ob := 96.8pcf$ Post Improvment - GI Sheets REF CPT114B(S)

Tore Vane Strenght of Clay: $TV := 0.8tsf$ Refernce GRI Report Boring B-1,B-2, B-4, B-5, B-6, B-7, B-8, B-10, B-12 and B-14 through B-22.

$TV = 1600 \cdot psf$

Friction angle of stone: $\phi stone := 50°$

Friction angle of silt $\phi silt := 20°$

LTP unit weight: $\gamma p := 130pcf$

SOIL STRENGTH AND STRESSES

Bearing Pressure due to Liquid in Tank: $q := 2882psf$ At full hydrotest load

Effective stress at top of clay: $\sigma ob := (\gamma ob \cdot Tob) + (\gamma p \cdot Tp)$ $\quad \sigma ob = 1740.068 \cdot psf$

Shear Strenght of SC: $Ssc := \sigma ob \cdot \tan(\phi stone)$ $\quad Ssc = 2073.732 \cdot psf$

Cohesive strenght of clay: $Cu := \frac{TV}{2}$ $\quad Cu = 800 \cdot psf$

Improved clay strength: $Su\_clay := Ar \cdot Ssc + (1 - Ar) \cdot Cu$

$Su\_clay = 940.111 \cdot psf$ After equation 10

Improved Clay-Silt Strenght $Su\_clay\_silt := Ar \cdot Ssc + (1 - Ar)(Cu + \sigma ob \cdot \tan(20))$

$Su\_clay\_silt = 4404.713 \cdot psf$ After equation 10

BASE SHEAR STABILITY EVALUATION - AT FULL HYDRO TEST - NO STRENGTH GAIN

$Dc := D + Tp + Tob \qquad Dc = 255.75 \cdot ft$ Equation 1

Bearing pressure at top of clay:

$$Qclay := q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2 + \gamma ob \cdot Tob$$ Equation 2

$$Qclay = 4218.04 \cdot psf$$

FOR CLAY LAYER ONLY

For a clay layer thicker than one sixth of the tank diameter $Nc1 := 6.1$ Equation 4

For a clay layer thinner than one sixth of the tank diamete $Nc2 := 4.1 + \left(\frac{Dc}{3Tclay}\right)$ Equation 5

$$Nc2 = 12.625$$

Place holder for easier coding: $PH := \frac{Dc}{Tclay}$

$Nc := if[(PH < 6), Nc1, Nc2] \qquad Nc = 12.625$

$Qult := Su\_clay \cdot Nc + \gamma ob \cdot Tob \qquad Qult = 12765.264 \cdot psf$

$$FS\_base\_clay := \frac{Su\_clay \cdot Nc}{\left[q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2\right]} \qquad FS\_base\_clay = 3.573$$ Equation 8

FOR COMBINED CLAY-SILT LAYER

For a clay layer thicker than one sixth of the tank diameter $Nc1 := 6.1$ Equation 4

For a clay layer thinner than one sixth of the tank diamete $Nc2 := 4.1 + \left(\frac{Dc}{3Tclay\_silt}\right)$ Equation 5

$$Nc2 = 8.362$$

Place holder for easier coding: $PH := \frac{Dc}{Tclay\_silt}$

$Nc := if[(PH < 6), Nc1, Nc2] \qquad Nc = 8.362$

$Qult := Su\_clay\_silt \cdot Nc + \gamma ob \cdot Tob \qquad Qult = 37730.784 \cdot psf$

$$FS\_base\_clay\_silt := \frac{Su\_clay\_silt \cdot Nc}{\left[q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2\right]} \qquad FS\_base\_clay\_silt = 11.089$$ Equation 8

EDGE SHEAR STABILITY EVALUATION - AT FULL HYDRO TEST - NO STRENGTH GAIN

IMPROVED CLAY

$Qapp := q + \gamma p \cdot Tp \qquad Qapp = 3725.7 \cdot psf$ Equation 11

With of segment that might fail $B := \begin{pmatrix} 10 \\ 20 \\ 40 \\ 60 \\ 80 \\ 100 \\ 120 \end{pmatrix} ft$

$Nc := 5.2 + \frac{B}{D} \qquad Nc = \begin{pmatrix} 5.242 \\ 5.283 \\ 5.367 \\ 5.45 \\ 5.533 \\ 5.617 \\ 5.7 \end{pmatrix}$

$Qall := Su\_clay \cdot Nc + \gamma p \cdot Tp \qquad Qall = \begin{pmatrix} 5771.446 \\ 5810.617 \\ 5888.96 \\ 5967.303 \\ 6045.645 \\ 6123.988 \\ 6202.33 \end{pmatrix} \cdot psf$

$FS\_edge\_clay := \frac{Qall}{Qapp} \qquad FS\_edge\_clay = \begin{pmatrix} 1.549 \\ 1.56 \\ 1.581 \\ 1.602 \\ 1.623 \\ 1.644 \\ 1.665 \end{pmatrix}$

EDGE SHEAR STABILITY EVALUATION - AT FULL HYDRO TEST - NO STRENGTH GAIN

IMPROVED CLAY and SILT

$Qapp := q + \gamma p \cdot Tp$ $\quad Qapp = 3725.7 \cdot psf$ $\quad$ Equation 11

With of segment that might fail $\quad B := \begin{pmatrix} 10 \\ 20 \\ 40 \\ 60 \\ 80 \\ 100 \\ 120 \end{pmatrix} ft$ $\quad Nc := 5.2 + \frac{B}{D}$ $\quad Nc = \begin{pmatrix} 5.242 \\ 5.283 \\ 5.367 \\ 5.45 \\ 5.533 \\ 5.617 \\ 5.7 \end{pmatrix}$

$Qall := Su\_clay\_silt \cdot Nc + \gamma p \cdot Tp \quad Qall = \begin{pmatrix} 23931.739 \\ 24115.269 \\ 24482.328 \\ 24849.388 \\ 25216.447 \\ 25583.507 \\ 25950.566 \end{pmatrix} \cdot psf$

$FS\_edge\_clay\_silt := \frac{Qall}{Qapp} \quad FS\_edge\_clay\_silt = \begin{pmatrix} 6.423 \\ 6.473 \\ 6.571 \\ 6.67 \\ 6.768 \\ 6.867 \\ 6.965 \end{pmatrix}$

SUMMARY OF BASE AND EDGE SHEAR STABILITY EVALUATION - AT FULL HYDRO TEST - NO STRENGTH GAIN

$FS\_base\_clay = 3.573$

$FS\_base\_clay\_silt = 11.089$

$FS\_edge\_clay = \begin{pmatrix} 1.549 \\ 1.56 \\ 1.581 \\ 1.602 \\ 1.623 \\ 1.644 \\ 1.665 \end{pmatrix}$ $\quad FS\_edge\_clay\_silt = \begin{pmatrix} 6.423 \\ 6.473 \\ 6.571 \\ 6.67 \\ 6.768 \\ 6.867 \\ 6.965 \end{pmatrix}$

BASE SHEAR STABILITY EVALUATION - IN SERVICE - NO STRENGTH GAIN

$q := 2500psf$

$Dc := D + Tp + Tob \qquad Dc = 255.75 \cdot ft$ Equation 1

Bearing pressure at top of clay:

$$Qclay := q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2 + \gamma ob \cdot Tob$$ Equation 2

$$Qclay = 3881.641 \cdot psf$$

FOR CLAY LAYER ONLY

For a clay layer thicker than one sixth of the tank diameter $Nc1 := 6.1$ Equation 4

For a clay layer thinner than one sixth of the tank diamete $Nc2 := 4.1 + \left(\frac{Dc}{3Tclay}\right)$ Equation 5

$$Nc2 = 12.625$$

Place holder for easier coding: $PH := \frac{Dc}{Tclay}$

$Nc := if[(PH < 6), Nc1, Nc2] \qquad Nc = 12.625$

$Qult := Su\_clay \cdot Nc + \gamma ob \cdot Tob \qquad Qult = 12765.264 \cdot psf$

$$FS\_base\_clay := \frac{Su\_clay \cdot Nc}{\left[q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2\right]} \qquad FS\_base\_clay = 3.976$$ Equation 8

FOR COMBINED CLAY-SILT LAYER

For a clay layer thicker than one sixth of the tank diameter $Nc1 := 6.1$ Equation 4

For a clay layer thinner than one sixth of the tank diamete $Nc2 := 4.1 + \left(\frac{Dc}{3Tclay\_silt}\right)$ Equation 5

$$Nc2 = 8.362$$

Place holder for easier coding: $PH := \frac{Dc}{Tclay\_silt}$

$Nc := if[(PH < 6), Nc1, Nc2] \qquad Nc = 8.362$

$Qult := Su\_clay\_silt \cdot Nc + \gamma ob \cdot Tob \qquad Qult = 37730.784 \cdot psf$

$$FS\_base\_clay\_silt := \frac{Su\_clay\_silt \cdot Nc}{\left[q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2\right]} \qquad FS\_base\_clay\_silt = 12.339$$ Equation 8

EDGE SHEAR STABILITY EVALUATION - IN SERVICE - NO STRENGTH GAIN

IMPROVED CLAY

$Qapp := q + \gamma p \cdot Tp \qquad Qapp = 3343.7 \cdot psf$ Equation 11

With of segment that might fail $B := \begin{pmatrix} 10 \\ 20 \\ 40 \\ 60 \\ 80 \\ 100 \\ 120 \end{pmatrix} ft$

$Nc := 5.2 + \frac{B}{D} \qquad Nc = \begin{pmatrix} 5.242 \\ 5.283 \\ 5.367 \\ 5.45 \\ 5.533 \\ 5.617 \\ 5.7 \end{pmatrix}$

$Qall := Su\_clay \cdot Nc + \gamma p \cdot Tp \qquad Qall = \begin{pmatrix} 5771.446 \\ 5810.617 \\ 5888.96 \\ 5967.303 \\ 6045.645 \\ 6123.988 \\ 6202.33 \end{pmatrix} \cdot psf$

$FS\_edge\_clay := \frac{Qall}{Qapp} \qquad FS\_edge\_clay = \begin{pmatrix} 1.726 \\ 1.738 \\ 1.761 \\ 1.785 \\ 1.808 \\ 1.832 \\ 1.855 \end{pmatrix}$

EDGE SHEAR STABILITY EVALUATION - IN SERVICE - NO STRENGTH GAIN

IMPROVED CLAY and SILT

$Qapp := q + \gamma p \cdot Tp$ $\quad Qapp = 3343.7 \cdot psf$ $\quad$ Equation 11

With of segment that might fail $\quad B := \begin{pmatrix} 10 \\ 20 \\ 40 \\ 60 \\ 80 \\ 100 \\ 120 \end{pmatrix} ft$ $\quad Nc := 5.2 + \frac{B}{D}$ $\quad Nc = \begin{pmatrix} 5.242 \\ 5.283 \\ 5.367 \\ 5.45 \\ 5.533 \\ 5.617 \\ 5.7 \end{pmatrix}$

$$Qall := Su\_clay\_silt \cdot Nc + \gamma p \cdot Tp \quad Qall = \begin{pmatrix} 23931.739 \\ 24115.269 \\ 24482.328 \\ 24849.388 \\ 25216.447 \\ 25583.507 \\ 25950.566 \end{pmatrix} \cdot psf$$

$$FS\_edge\_clay\_silt := \frac{Qall}{Qapp} \quad FS\_edge\_clay\_silt = \begin{pmatrix} 7.157 \\ 7.212 \\ 7.322 \\ 7.432 \\ 7.541 \\ 7.651 \\ 7.761 \end{pmatrix}$$

SUMMARY OF BASE AND EDGE SHEAR STABILITY EVALUATION - IN SERVCIE - NO STRENGTH GAIN

$FS\_base\_clay = 3.976$

$FS\_base\_clay\_silt = 12.339$

$$FS\_edge\_clay = \begin{pmatrix} 1.726 \\ 1.738 \\ 1.761 \\ 1.785 \\ 1.808 \\ 1.832 \\ 1.855 \end{pmatrix} \qquad FS\_edge\_clay\_silt = \begin{pmatrix} 7.157 \\ 7.212 \\ 7.322 \\ 7.432 \\ 7.541 \\ 7.651 \\ 7.761 \end{pmatrix}$$

Units

$kips \equiv 1000 \cdot lbf$

$ksf := 1000 \cdot psf$

$tsf := 2 \cdot ksf$

$kPa := 1000 \cdot Pa$

$pi := 3.14159$

$ksi \equiv \frac{1000 \cdot lbf}{in^2}$

$pcf \equiv \frac{lbf}{ft^3}$

$in^2 = 0.001\, m^2$

$psi \equiv \frac{lbf}{in^2}$

$psf \equiv \frac{lbf}{ft^2}$

$pcf \equiv \frac{lbf}{ft^3}$

$kcf \equiv \frac{k}{ft^3}$

$k \equiv kips$

$N \equiv newton$

$ORIGIN \equiv 1$

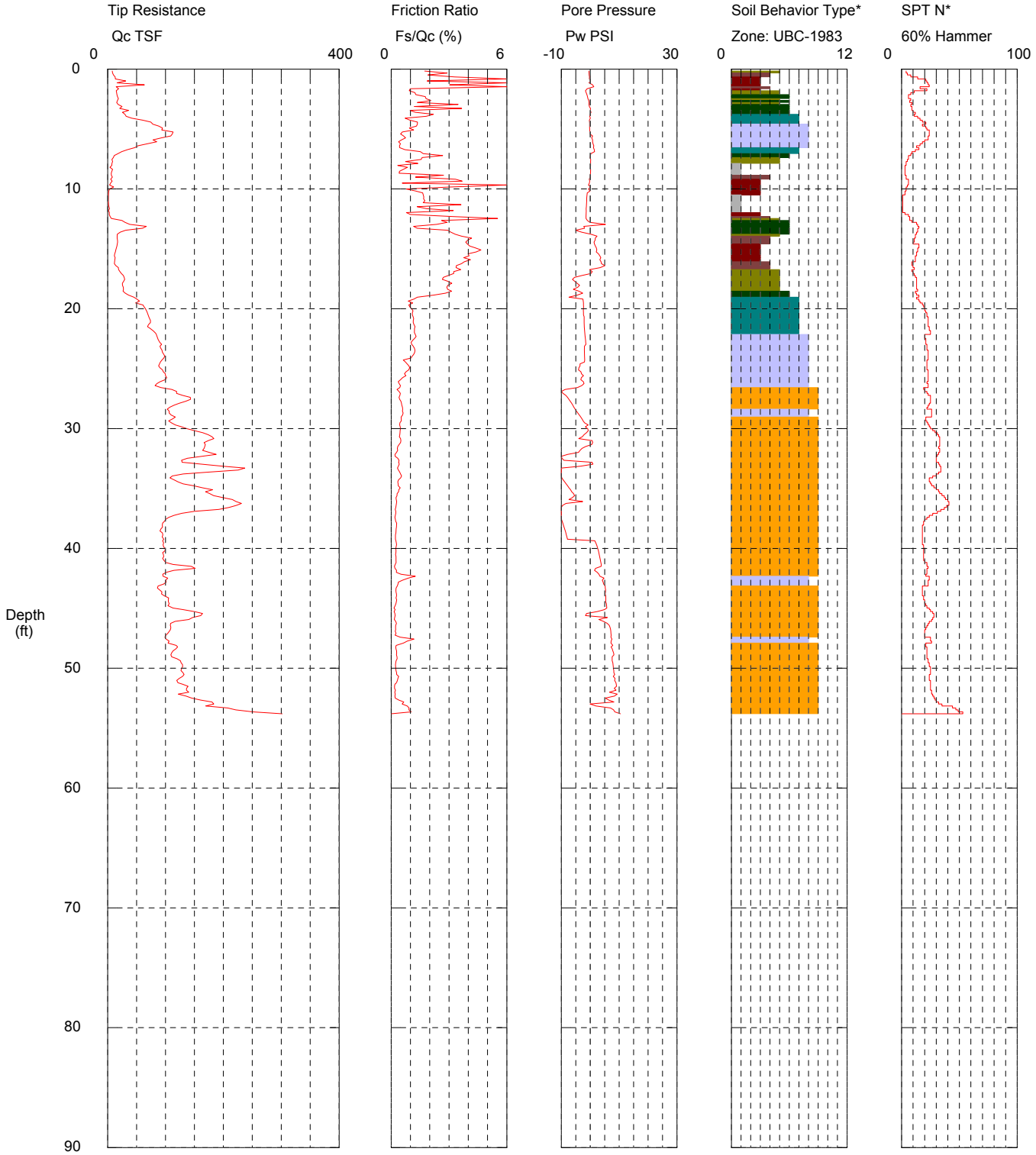
$MPa := 10^6 \cdot Pa$

$kN \equiv 1000 newton$

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-121
Cone Used: DDG1254

CPT Date/Time: 11/16/2014 11:14:47 AM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 53.81 feet

Depth Increment = 0.164 feet

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

InSitu Engineering

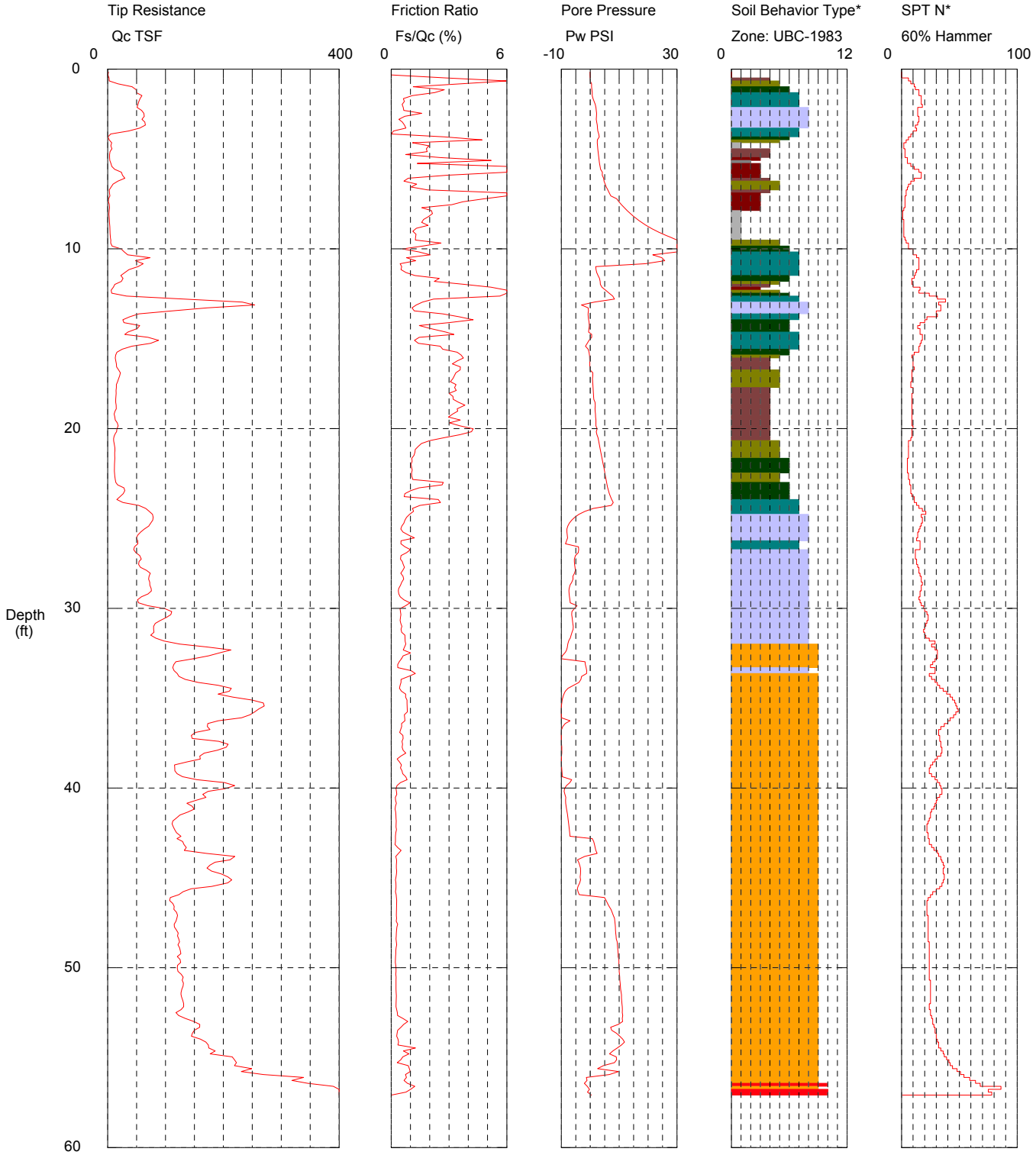
*Soil behavior type and SPT based on data from UBC-1983

EX-0001-007058-PCE

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-122
Cone Used: DDG1254

CPT Date/Time: 11/15/2014 2:36:42 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 57.09 feet

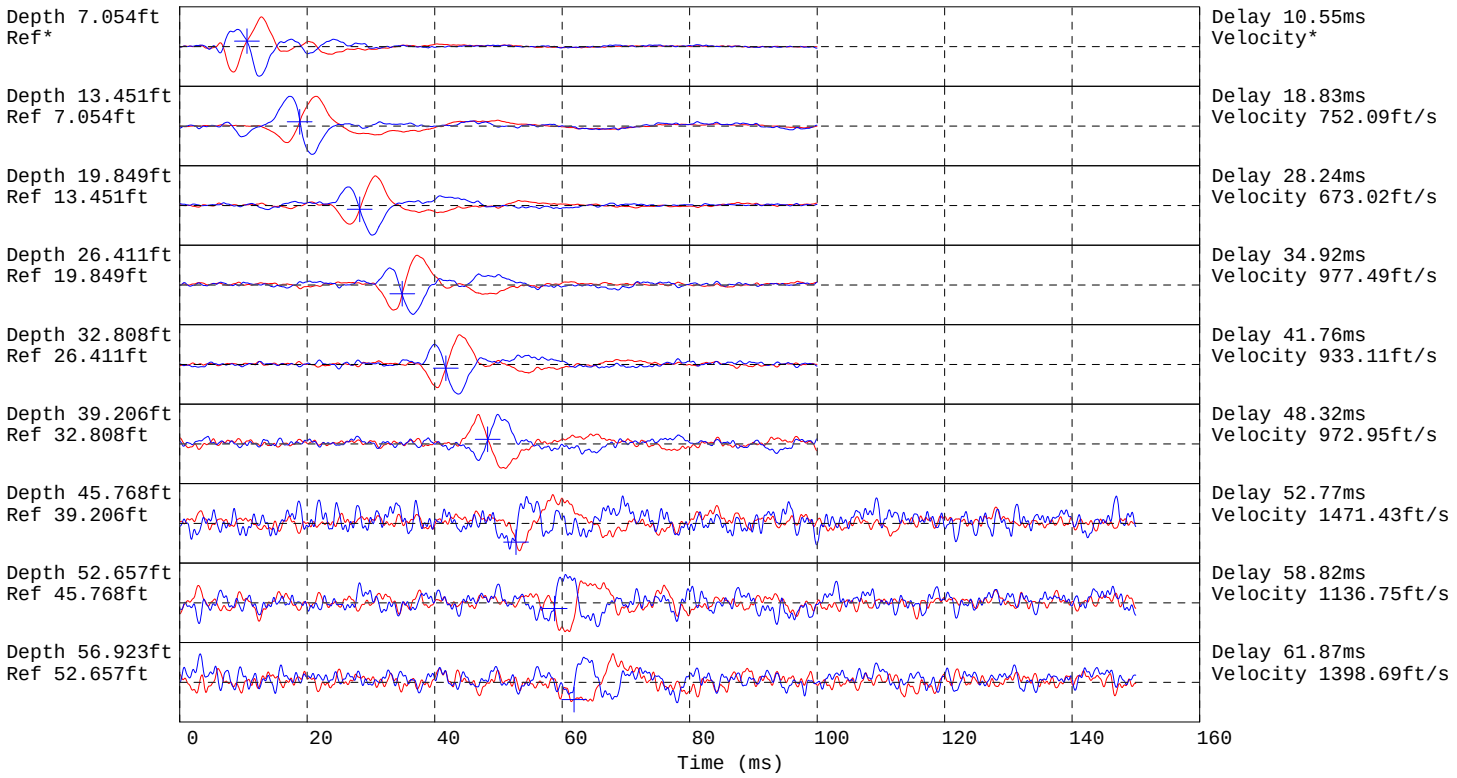
Depth Increment = 0.164 feet

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

InSitu Engineering Predrilled top 13 feet

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

Tesoro Distribution Terminal
HB-CPT-123(s) Shear Wave Analysis

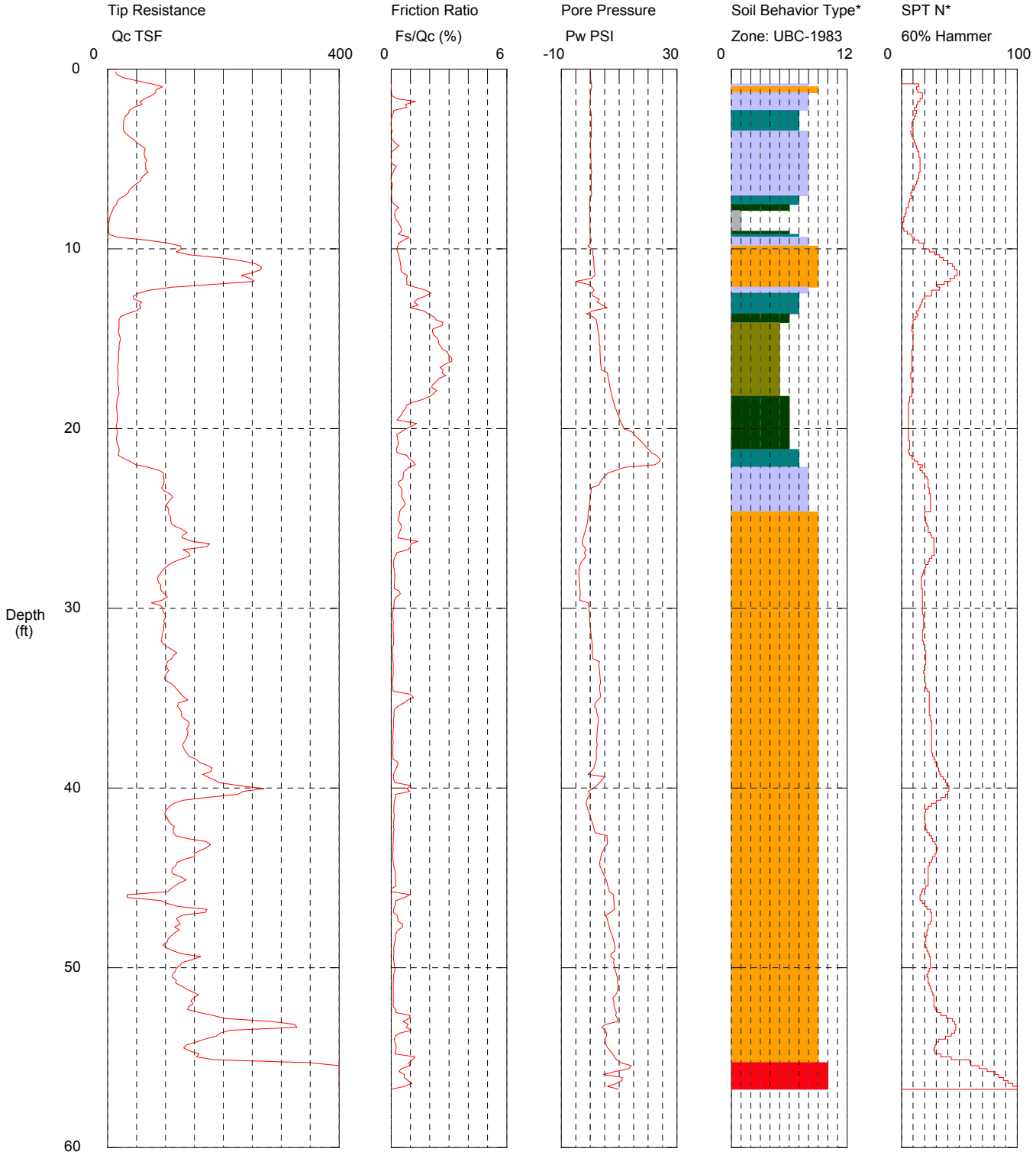


Hammer to Rod String Distance 0.7 (m)
* = Not Determined

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-123(S)
Cone Used: DDG1254

CPT Date/Time: 11/21/2014 2:03:10 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 56.76 feet

Depth Increment = 0.164 feet

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

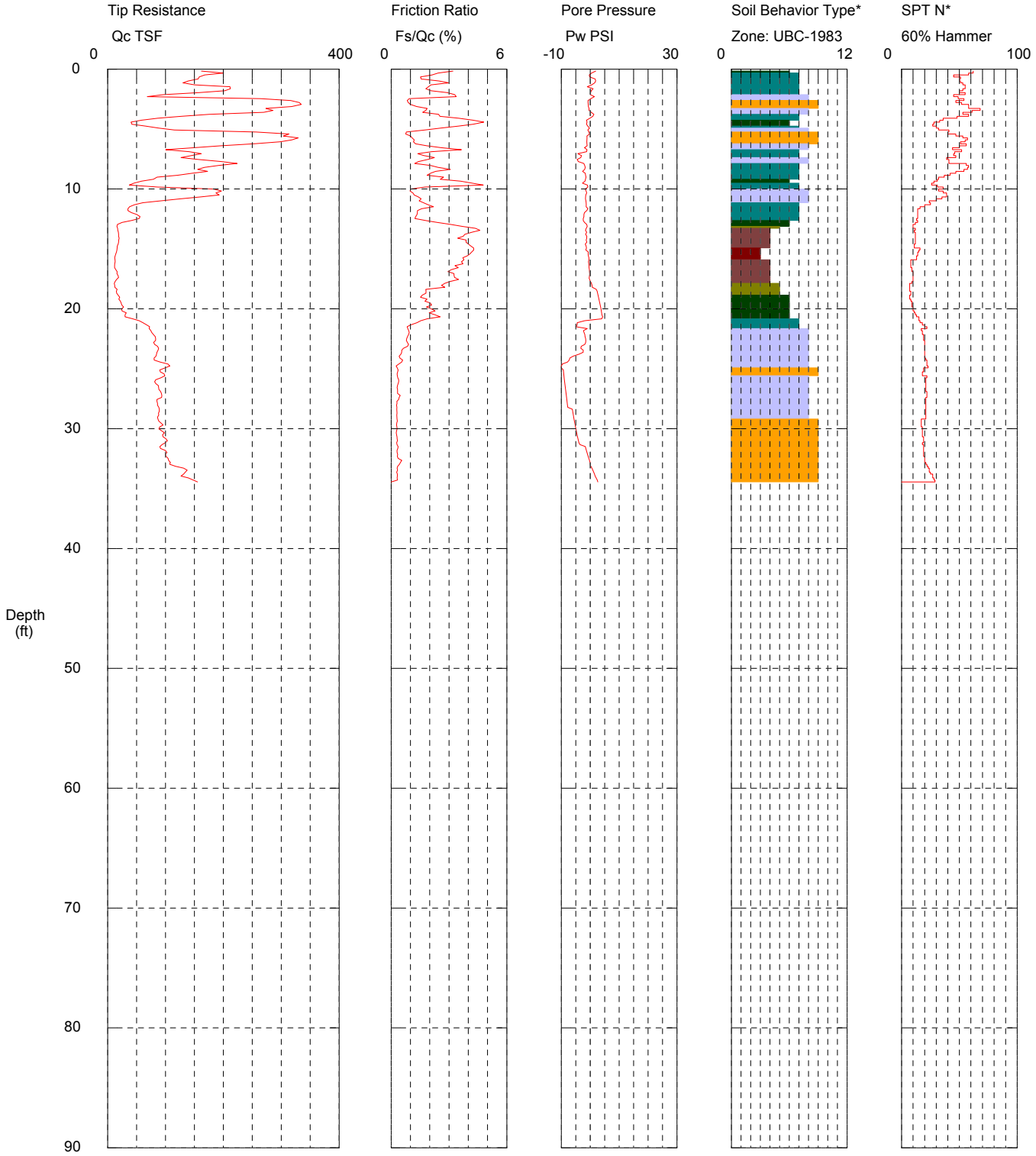
InSitu Engineering Predrilled top 10 feet

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

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-124
Cone Used: DDG1254

CPT Date/Time: 11/16/2014 3:40:09 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 34.45 feet

Depth Increment = 0.164 feet

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

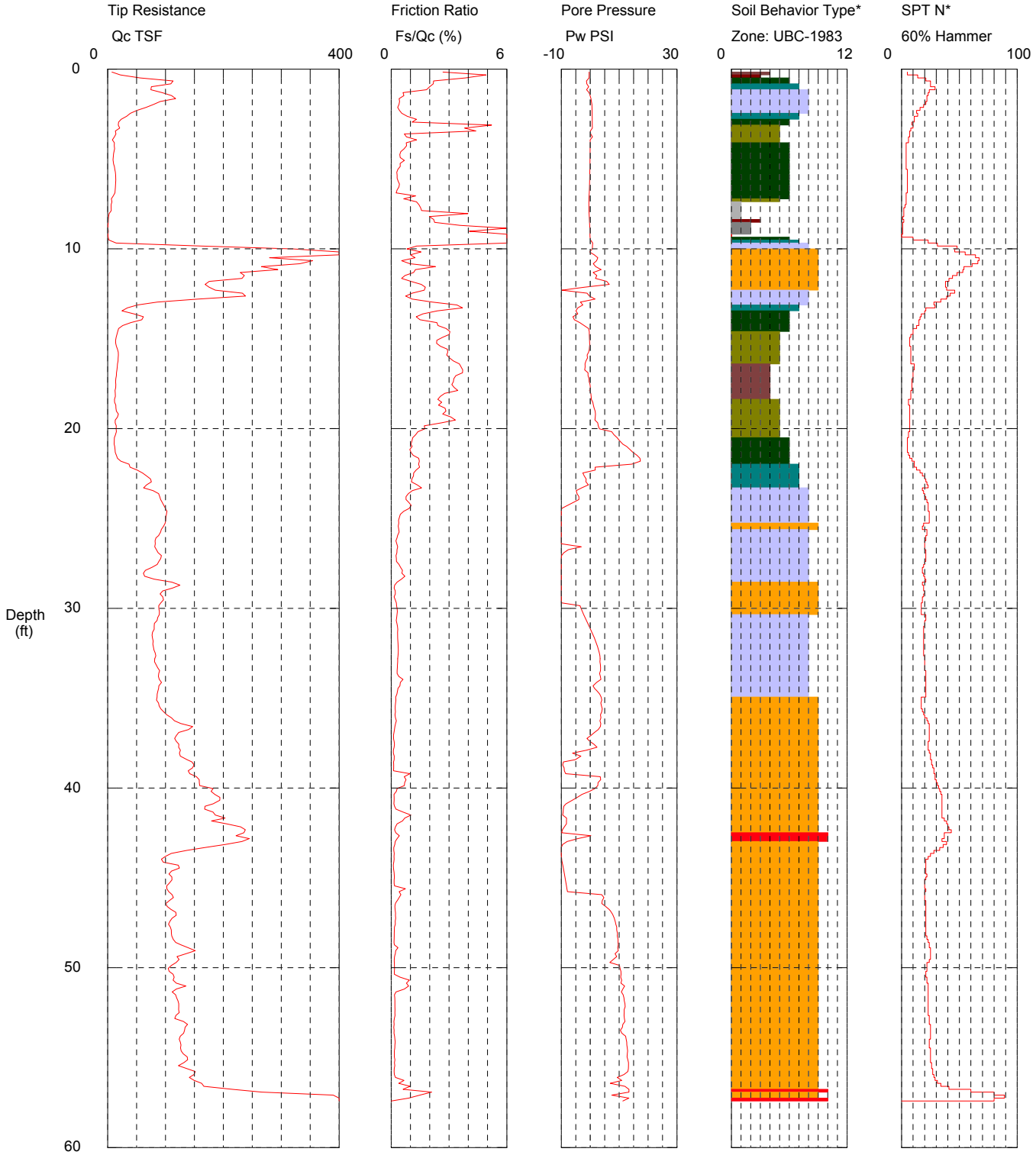
InSitu Engineering Bent rod during push. Terminated early due to bent rod.

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

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-125A
Cone Used: DDG1254

CPT Date/Time: 11/21/2014 9:19:14 AM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 57.41 feet

Depth Increment = 0.164 feet

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

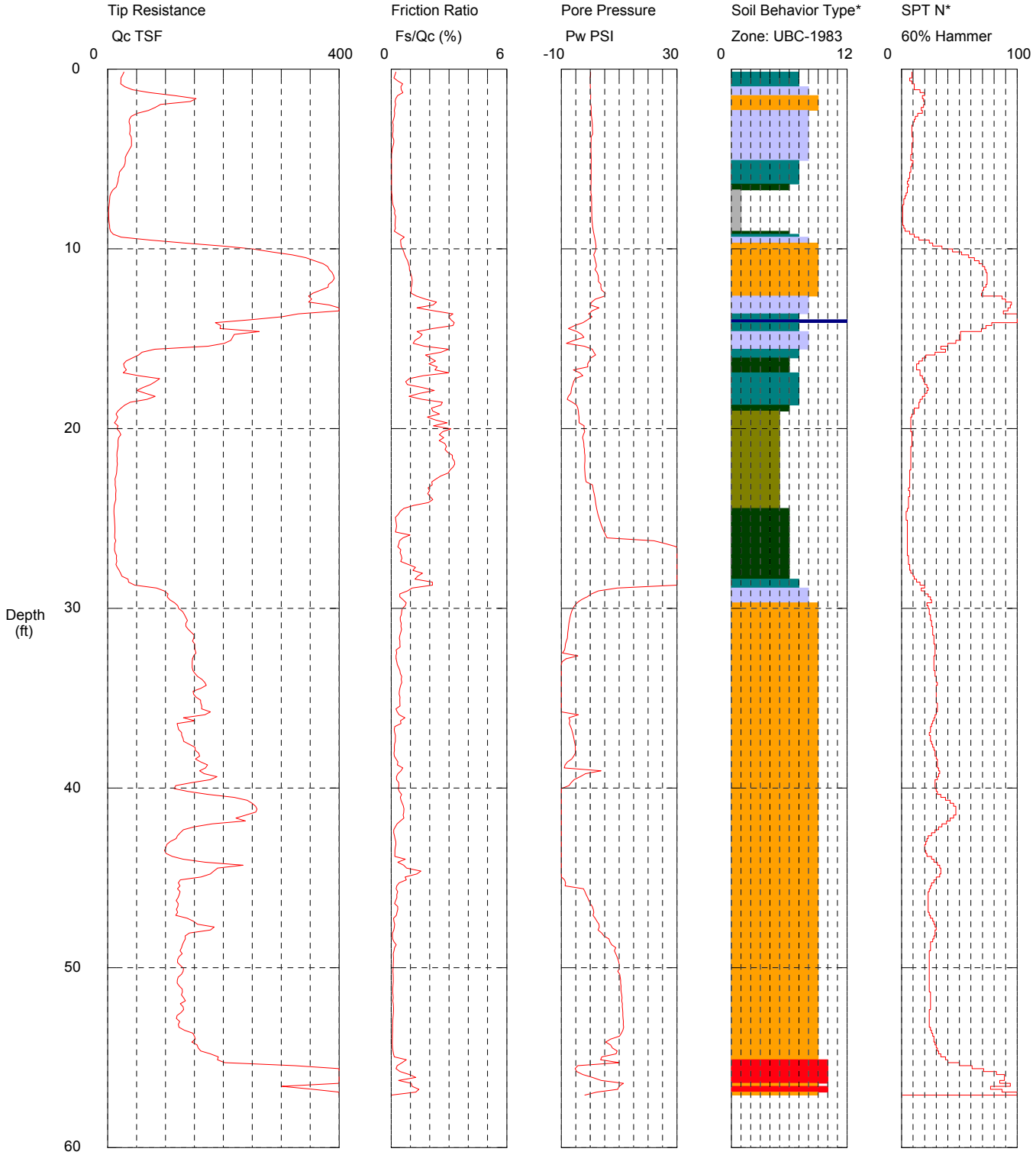
InSitu Engineering Predrilled top 10 feet

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

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-126
Cone Used: DDG1254

CPT Date/Time: 11/21/2014 10:52:24 AM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 57.09 feet

Depth Increment = 0.164 feet

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

InSitu Engineering Predrilled top 10 feet

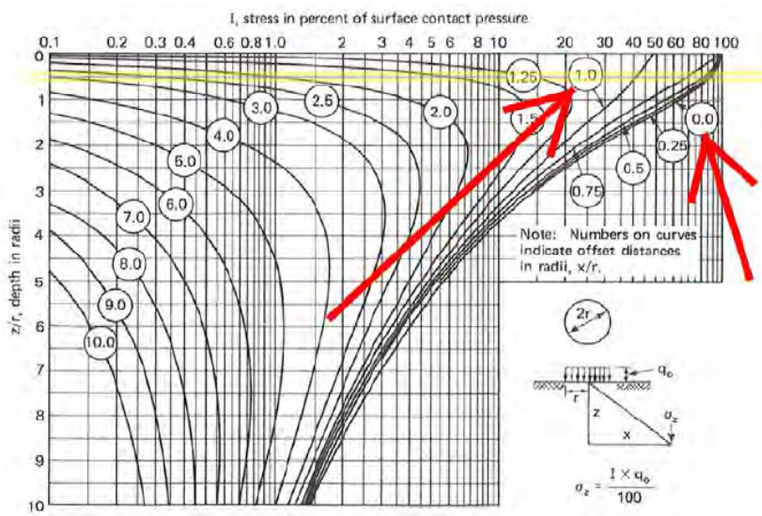
*Soil behavior type and SPT based on data from UBC-1983

Tank 4

Design Summary
Tank 4

This table assumes the exploration is directly below the center of the tank.

Tank 5		Orig	Imp.	Improved		Original	
Soil Profile	Depth	Liq Sett.	Liq Sett.	Rapid Sett.	Long Term Sett.	Rapid Sett.	Long Term Sett.
HB-CPT-151	55.5-ft	4.03-in.	2.77-in.	2.79-in.	.16-in.	1.6-in.	3.89-in.
HB-CPT-154	52.4-ft	3.05-in.	1.38-in.	1.34-in.	.14-in.	.78-in.	1.7-in.
HB-CPT-156	47.2-ft	1.9-in.	.5-in.	2.86-in.	.11-in.	.98-in.	3.8-in.
HB-CPT-155	56.1-ft	2.2-in.	1.05-in.	3.49-in.	.04-in.	.8-in.	4.66-in.
HB-CPT-152	55.9-ft	2.78-in.	.99-in.	4.95-in.	.1-in.	1.7-in.	6.72-in.
HB-CPT-153(s)		SHORT					
Average:		**2.79-in.**	**1.34-in.**	**3.08-in.**	**.11-in.**	**1.17-in.**	**4.15-in.**



This table calculates Settlement at the edge of the tank.

Tank 5		Orig	Imp.	Improved		Original	
Soil Profile	Depth	Liq Sett.	Liq Sett.	Rapid Sett.	Long Term Sett.	Rapid Sett.	Long Term Sett.
HB-CPT-151	55.5-ft	4.03-in.	2.77-in.	1.39-in.	.08-in.	.8-in.	1.94-in.
HB-CPT-154	52.4-ft	3.05-in.	1.38-in.	.67-in.	.07-in.	.39-in.	.85-in.
HB-CPT-156	47.2-ft	1.9-in.	.5-in.	1.43-in.	.05-in.	.49-in.	1.9-in.
HB-CPT-155	56.1-ft	2.2-in.	1.05-in.	1.75-in.	.02-in.	.4-in.	2.33-in.
HB-CPT-152	55.9-ft	2.78-in.	.99-in.	2.47-in.	.05-in.	.85-in.	3.36-in.
HB-CPT-153(s)		SHORT					
Average:		2.79-in.	1.34-in.	1.54-in.	.05-in.	.59-in.	2.08-in.
Note: No reduction to total soil settlement is taken for the liquefaction condition.							

Worksheet Tab: 11- Boulanger (improved)

Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Hayward Baker Inc. THINK SAFE	SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 4			
	Job no.	860079	Design by:	M. Rohrbach	Date: 15-Apr-15
	File	CPT-Liquefaction-Boulanger	Checked		
	Spreadsheet Rev.	23-Dec-14	Reviewed		

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-151

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	16.4-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	28.42	ft.
Elevation of bottom of Soil Profile	-27.03	ft.
Elevation of Installation	22	
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	55.5-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	432.90-psf	

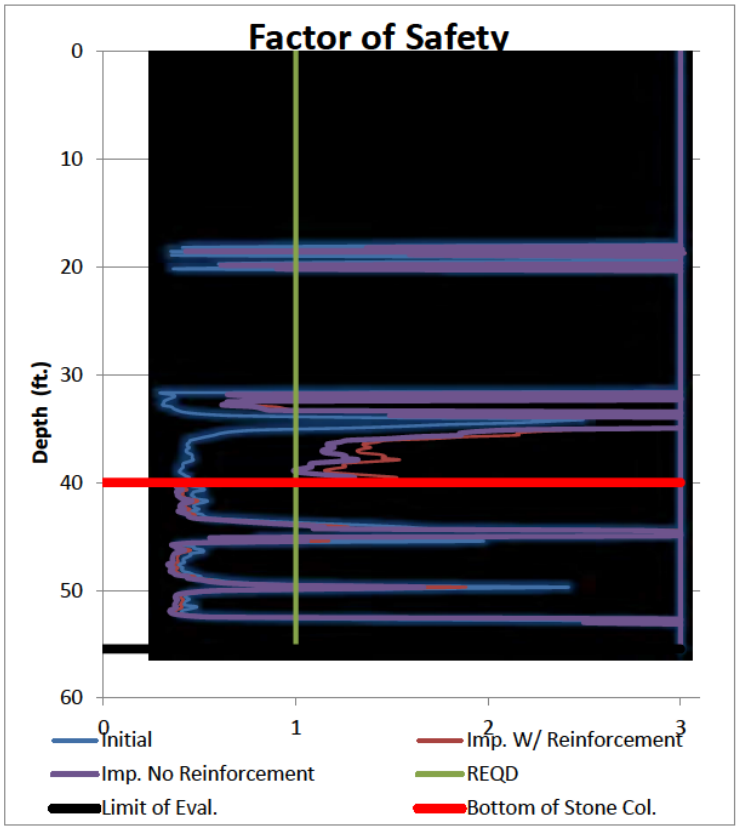
Stone Column Design Information

L =	40-ft		ϕ_c = 50
Column Spacing	8-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 55.5-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 18.8-ft.

Calculated Min: 3.98-in via T&S, Min never < 0.5 in.

Calculated Max: 4.03-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 13.0-ft.

Calculated Min: 2.88-in via T&S, Min never < 0.5 in.

Calculated Max: 3.22-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 12.5-ft.

Calculated Min: 2.77-in via T&S, Min never < 0.5 in.

Calculated Max: 2.91-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Worksheet Tab: 14-Elastic Sett. (Imp.) Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 4 IMPROVED CASE: CPT-151

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment
= $1-0.5*(q/(q_{bar}-q)$

C_2 = a correction factor top account for creep ion soil
= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

q = gDf

Stone Column Length 40-ft

Reduction factor on Priebe: 1

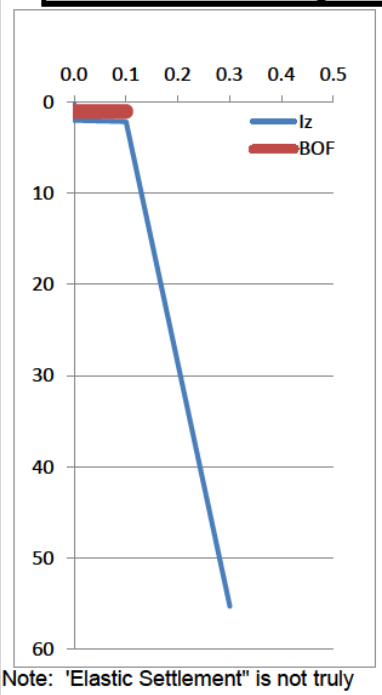
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	
L	212.7-ft	
B	212.7-ft	
fill density	130-pcf	
Elevation of Top of Soil Profile	28.42	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es=X\cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1$=**0.5**B=	106.3
I_z = 0	at depth	$z=z_2$=**2.0**B=	425.4
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1$=**1.0**B=	212.7
I_z = 0	at depth	$z=z_2$=**4.0**B=	850.8
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

$\Sigma(Iz/Es)\Delta z$ (ft2/lb)= 1.53E-05

Total Elastic Settlement (Flexible foundation)

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0589-ft 0.71-in

Total Rapid Elastic Settlement

$S_e=C_1(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0382-ft 0.46-in

Long Term Settlement 0.25-in

Calculated Consolidation Type Settlement:

Total:	2.24-in	Adjusted for footing Type
Recomp.	0.09-in	Adjusted for footing Type
Virgin Sett.	2.15-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	2.33-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:
Total: 2.95-in
Rapid: 2.79-in
Long Term: 0.16-in

Worksheet Tab: 11- Boulanger (improved)

Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Hayward Baker Inc. THINK SAFE	SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 4			
	Job no.	860079	Design by:	M. Rohrbach	Date: 15-Apr-15
	File	CPT-Liquefaction-Boulanger	Checked		
	Spreadsheet Rev.	23-Dec-14	Reviewed		

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-152

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	15.2-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	27.23	ft.
Elevation of bottom of Soil Profile	-28.71	ft.
Elevation of Installation	22	
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	55.9-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	587.60-psf	

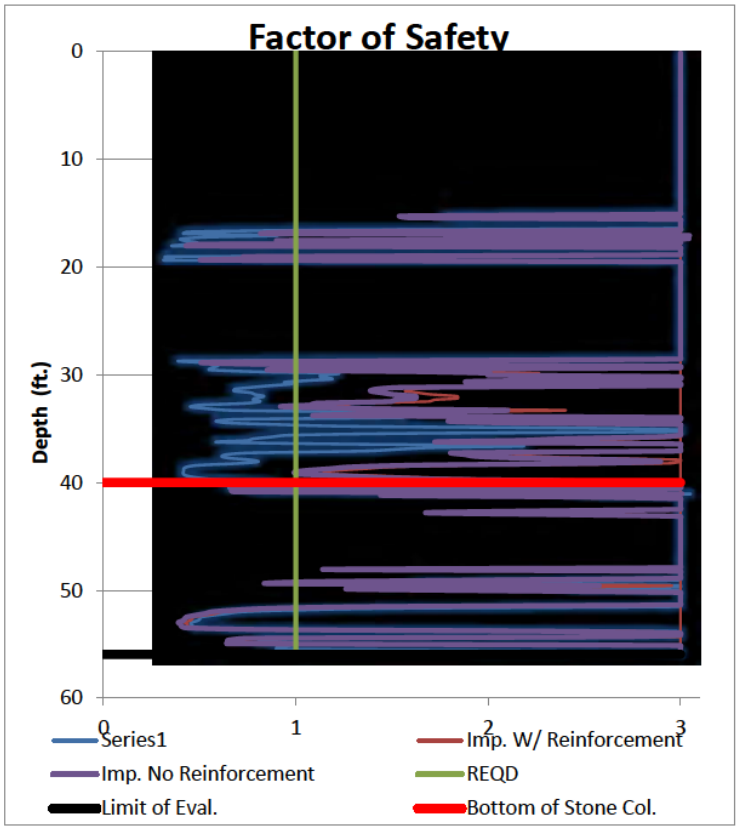
Stone Column Design Information

L =	40-ft		ϕ_c = 50
Column Spacing	8-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 55.9-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 13.9-ft.

Calculated Min: 2.64-in via T&S, Min never < 0.5 in.

Calculated Max: 2.78-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 5.6-ft.

Calculated Min: 1.18-in via T&S, Min never < 0.5 in.

Calculated Max: 1.32-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 4.8-ft.

Calculated Min: 0.99-in via T&S, Min never < 0.5 in.

Calculated Max: 1.26-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Worksheet Tab: 14-Elastic Sett. (Imp.) Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 4 IMPROVED CASE: CPT-152

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment

= $1-0.5*(q/(q_{bar}-q)$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

q = gDf

Stone Column Length 40-ft

Reduction factor on Priebe: 1

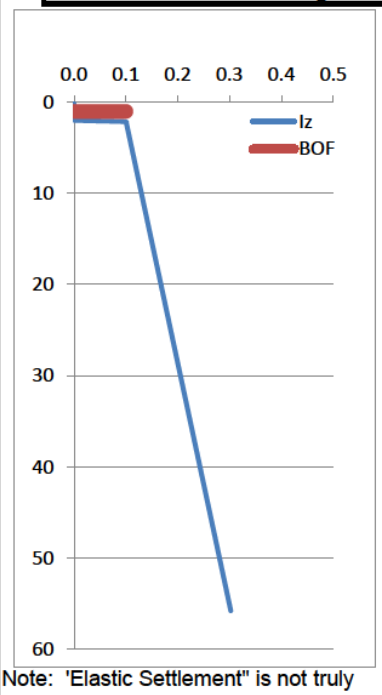
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	
L	212.7-ft	
B	212.7-ft	
fill density	130-pcf	
Elevation of Top of Soil Profile	27.23	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es=X\cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1$=0.5B=	106.3
I_z = 0	at depth	$z=z_2$=2.0B=	425.4
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1$=1.0B=	212.7
I_z = 0	at depth	$z=z_2$=4.0B=	850.8
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

Σ(Iz/Es)Δz (ft2/lb)= 1.70E-05

Total Elastic Settlement (Flexible foundation)

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0652-ft 0.78-in

Total Rapid Elastic Settlement

$S_e=C_1(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0424-ft 0.51-in

Long Term Settlement 0.27-in

Calculated Consolidation Type Settlement:

Total:	4.26-in	Adjusted for footing Type
Recomp.	0.17-in	Adjusted for footing Type
Virgin Sett.	4.09-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	4.44-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:

Total: 5.04-in

Rapid: 4.95-in

Long Term: 0.10-in

Worksheet Tab: 11- Boulanger (improved)

Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Hayward Baker Inc. THINK SAFE	SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 4			
	Job no.	860079	Design by:	M. Rohrbach	Date: 15-Apr-15
	File	CPT-Liquefaction-Boulanger	Checked		
	Spreadsheet Rev.	23-Dec-14	Reviewed		

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-154

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	13.4-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	25.39	ft.
Elevation of bottom of Soil Profile	-26.97	ft.
Elevation of Installation	22	
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	52.4-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	826.80-psf	

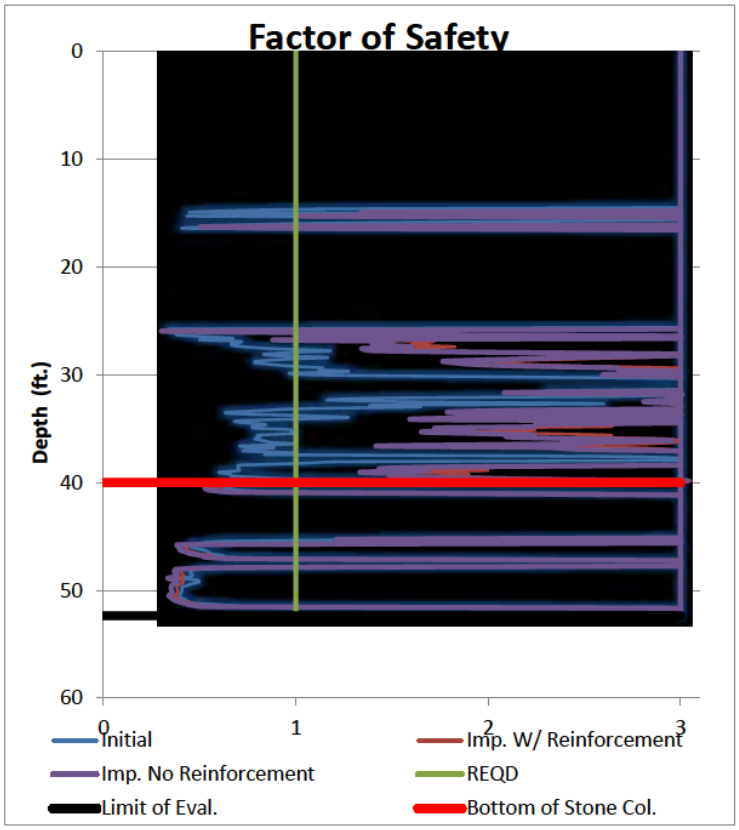
Stone Column Design Information

L =	40-ft		ϕ_c = 50
Column Spacing	8-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 52.4-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 15.8-ft.

Calculated Min: 2.84-in via T&S, Min never < 0.5 in.

Calculated Max: 3.05-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 7.0-ft.

Calculated Min: 1.49-in via T&S, Min never < 0.5 in.

Calculated Max: 1.51-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 6.6-ft.

Calculated Min: 1.38-in via T&S, Min never < 0.5 in.

Calculated Max: 1.48-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 4 — IMPROVED CASE: CPT-154

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e = C_1 C_2 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment
= $1 - 0.5*(q/(q_{bar} - q))$

C_2 = a correction factor top account for creep ion soil
= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

q = gDf

Stone Column Length 40-ft

Reduction factor on Priebe: 1

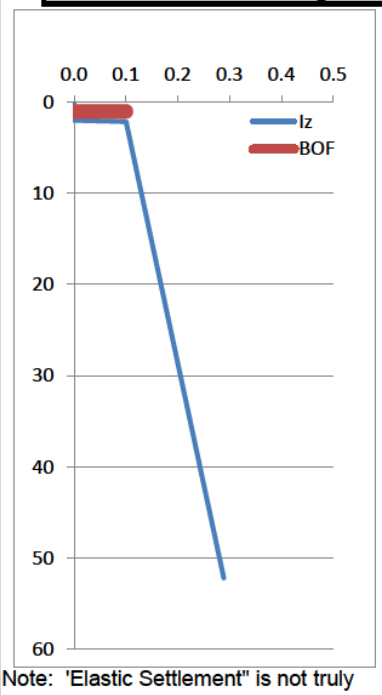
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	
L	212.7-ft	
B	212.7-ft	
fill density	130-pcf	
Elevation of Top of Soil Profile	25.39	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es = X \cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1$=0.5B=	106.3
I_z = 0	at depth	$z=z_2$=2.0B=	425.4
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1$=1.0B=	212.7
I_z = 0	at depth	$z=z_2$=4.0B=	850.8
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

$\Sigma(Iz/Es)\Delta z$ (ft2/lb)= **1.15E-05**

Total Elastic Settlement (Flexible foundation)

$S_e = C_1 C_2 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$= 0.0443-ft 0.53-in

Total Rapid Elastic Settlement

$S_e = C_1 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$= 0.0288-ft 0.35-in

Long Term Settlement 0.18-in

Calculated Consolidation Type Settlement:

Total:	0.95-in	Adjusted for footing Type
Recomp.	0.04-in	Adjusted for footing Type
Virgin Sett.	0.91-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	0.99-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:
Total: 1.48-in
Rapid: 1.34-in
Long Term: 0.14-in

Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Hayward Baker Inc.
THINK SAFE

SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 4		
Job no.	860079	Design by: M. Rohrbach	Date: 15-Apr-15
File	CPT-Liquefaction-Boulanger	Checked	
Spreadsheet Rev.	23-Dec-14	Reviewed	

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-155

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	14.9-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	26.9	ft.
Elevation of bottom of Soil Profile	-29.20	ft.
Elevation of Installation	22	
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	56.1-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	630.50-psf	

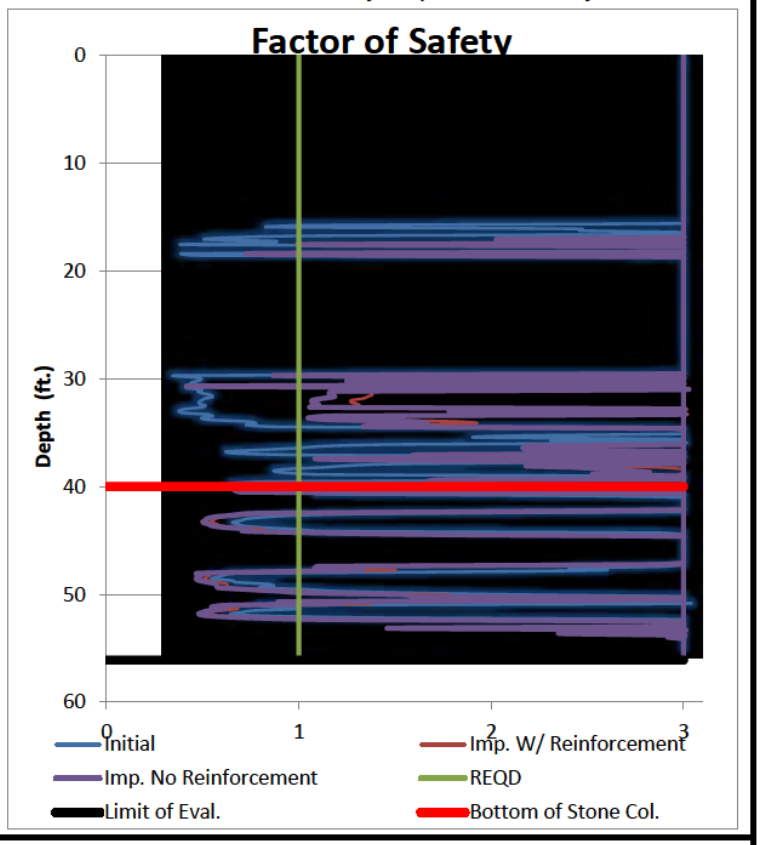
Stone Column Design Information

L =	40-ft		ϕ_c = 50
Column Spacing	8-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 56.1-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 12.0-ft.

Calculated Min: 2.08-in via T&S, Min never < 0.5 in.

Calculated Max: 2.20-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 6.6-ft.

Calculated Min: 1.18-in via T&S, Min never < 0.5 in.

Calculated Max: 1.46-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 5.9-ft.

Calculated Min: 1.05-in via T&S, Min never < 0.5 in.

Calculated Max: 1.24-in via T&S

Special Notes / Comments

Notes:
1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Worksheet Tab: 14-Elastic Sett. (Imp.) Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 4 IMPROVED CASE: CPT-155

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment

= $1-0.5*(q/(q_{bar}-q)$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

q = gDf

Stone Column Length 40-ft

Reduction factor on Priebe: 1

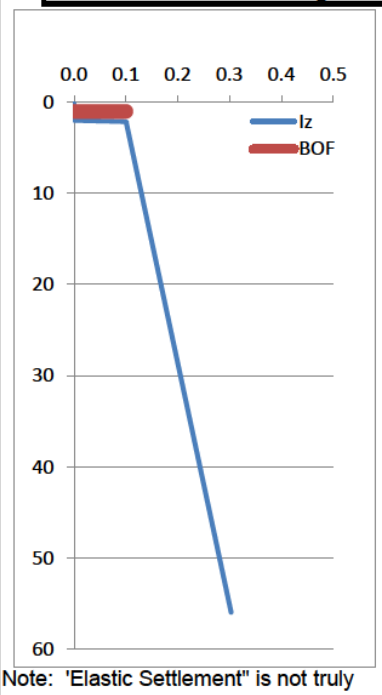
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	
L	212.7-ft	
B	212.7-ft	
fill density	130-pcf	
Elevation of Top of Soil Profile	26.9	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es=X\cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1$=**0.5**B=	106.3
I_z = 0	at depth	$z=z_2$=**2.0**B=	425.4
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1$=**1.0**B=	212.7
I_z = 0	at depth	$z=z_2$=**4.0**B=	850.8
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

Σ(Iz/Es)Δz (ft2/lb)= 1.01E-05

Total Elastic Settlement (Flexible foundation)

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0387-ft 0.46-in

Total Rapid Elastic Settlement

$S_e=C_1(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0251-ft 0.30-in

Long Term Settlement 0.16-in

Calculated Consolidation Type Settlement:

Total:	3.07-in	Adjusted for footing Type
Recomp.	0.12-in	Adjusted for footing Type
Virgin Sett.	2.95-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	3.19-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:

Total: 3.53-in

Rapid: 3.49-in

Long Term: 0.04-in

Worksheet Tab: 11- Boulanger (improved)

Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Hayward Baker Inc. THINK SAFE	SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 4			
	Job no.	860079	Design by:	M. Rohrbach	Date: 15-Apr-15
	File	CPT-Liquefaction-Boulanger	Checked		
	Spreadsheet Rev.	23-Dec-14	Reviewed		

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-156

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	13.0-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	25.01	ft.
Elevation of bottom of Soil Profile	-22.23	ft.
Elevation of Installation	22	
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	47.2-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	876.20-psf	

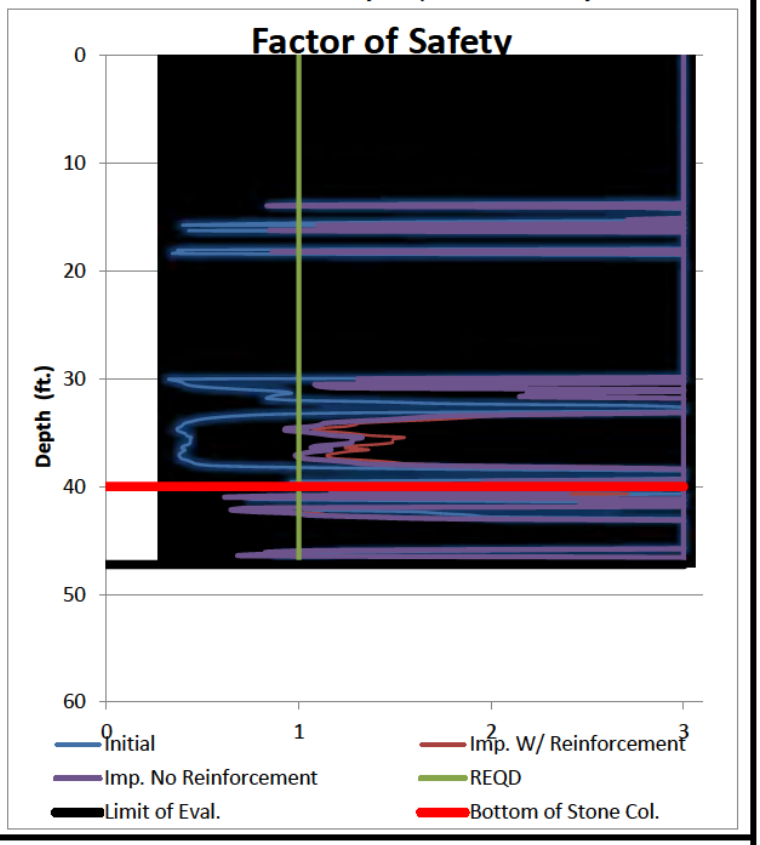
Stone Column Design Information

L =	40-ft		ϕ_c = 50
Column Spacing	8-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 47.2-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 9.2-ft.

Calculated Min: 1.83-in via T&S, Min never < 0.5 in.

Calculated Max: 1.90-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 2.6-ft.

Calculated Min: 0.50-in via T&S, Min never < 0.5 in.

Calculated Max: 0.82-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 1.5-ft.

Calculated Min: 0.50-in via T&S, Min never < 0.5 in.

Calculated Max: 0.53-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Worksheet Tab: 14-Elastic Sett. (Imp.) Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 4 IMPROVED CASE: CPT-156

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

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= $1-0.5*(q/(q_{bar}-q)$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

q = gDf

Stone Column Length 40-ft

Reduction factor on Priebe: 1

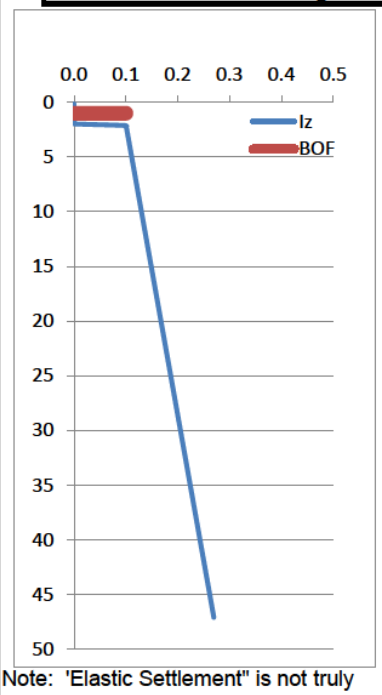
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	
L	212.7-ft	
B	212.7-ft	
fill density	130-pcf	
Elevation of Top of Soil Profile	25.01	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es=X\cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=0.5B$=	106.3
I_z = 0	at depth	$z=z_2=2.0B$=	425.4
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=1.0B$=	212.7
I_z = 0	at depth	$z=z_2=4.0B$=	850.8
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

$\Sigma(Iz/Es)\Delta z$ (ft2/lb)= 1.25E-05

Total Elastic Settlement (Flexible foundation)

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0478-ft 0.57-in

Total Rapid Elastic Settlement

$S_e=C_1(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0310-ft 0.37-in

Long Term Settlement 0.20-in

Calculated Consolidation Type Settlement:

Total:	2.39-in	Adjusted for footing Type
Recomp.	0.09-in	Adjusted for footing Type
Virgin Sett.	2.30-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	2.49-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:

Total: 2.96-in

Rapid: 2.86-in

Long Term: 0.11-in

Tank 4
Stability Check Layer Thickness

Calculations for Tank Edge and Base Stability (Tank 4):

Reference: Stability of Steel Oil Storage Tanks, Duncan, D'Orazio, 1984

GEOMETERY

Tank Diameter: $D := 240ft$

Existing Elev: $ELEVex := 29.26ft$

Tank Bottom Elev: $TankB := 31.75ft$

Fill and LTP Thickness: $Tp := TankB - (ELEVex - 4ft)$ $Tp = 6.49 \cdot ft$

Overburden Thickness: $Tob := TankB - Tp - 16ft$ $Tob = 9.26 \cdot ft$

Saturated Clay above the GW will consolidate quickly

Clay Thickness:

CPT	Clay Thickess	Clay and Silt
151	4'	12'
152	2'	12'
153	6'	11.5'
154	6'	12.2'
155	10.5'	16'
156	6.5	17.3

$Tclay := 10.5ft$ $Tclay\_silt := 17.3ft$

Stone column replacement ratio: $Ar := 0.11$

SOIL PROPERTIES

Overburden unit weight: $\gamma ob := 96.8pcf$ Post Improvment - GI Sheets REF CPT114B(S)

Tore Vane Strenght of Clay: $TV := 0.8tsf$ Refernce GRI Report Boring B-1,B-2, B-4, B-5, B-6, B-7, B-8, B-10, B-12 and B-14 through B-22.

$TV = 1600 \cdot psf$

Friction angle of stone: $\phi stone := 50°$

Friction angle of silt $\phi silt := 20°$

LTP unit weight: $\gamma p := 130pcf$

SOIL STRENGTH AND STRESSES

Bearing Pressure due to Liquid in Tank: $q := 2882psf$ At full hydrotest load

Effective stress at top of clay: $\sigma ob := (\gamma ob \cdot Tob) + (\gamma p \cdot Tp)$ $\sigma ob = 1740.068 \cdot psf$

Shear Strenght of SC: $Ssc := \sigma ob \cdot \tan(\phi stone)$ $Ssc = 2073.732 \cdot psf$

Cohesive strenght of clay: $Cu := \frac{TV}{2}$ $Cu = 800 \cdot psf$

Improved clay strength: $Su\_clay := Ar \cdot Ssc + (1 - Ar) \cdot Cu$

$Su\_clay = 940.111 \cdot psf$ After equation 10

Improved Clay-Silt Strenght $Su\_clay\_silt := Ar \cdot Ssc + (1 - Ar)(Cu + \sigma ob \cdot \tan(20))$

$Su\_clay\_silt = 4404.713 \cdot psf$ After equation 10

BASE SHEAR STABILITY EVALUATION - AT FULL HYDRO TEST - NO STRENGTH GAIN

$Dc := D + Tp + Tob \qquad Dc = 255.75 \cdot ft$ Equation 1

Bearing pressure at top of clay:

$$Qclay := q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2 + \gamma ob \cdot Tob$$ Equation 2

$$Qclay = 4218.04 \cdot psf$$

FOR CLAY LAYER ONLY

For a clay layer thicker than one sixth of the tank diameter $Nc1 := 6.1$ Equation 4

For a clay layer thinner than one sixth of the tank diamete $Nc2 := 4.1 + \left(\frac{Dc}{3Tclay}\right)$ Equation 5

$$Nc2 = 12.219$$

Place holder for easier coding: $PH := \frac{Dc}{Tclay}$

$Nc := if[(PH < 6), Nc1, Nc2] \qquad Nc = 12.219$

$Qult := Su\_clay \cdot Nc + \gamma ob \cdot Tob \qquad Qult = 12383.624 \cdot psf$

$$FS\_base\_clay := \frac{Su\_clay \cdot Nc}{\left[q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2\right]} \qquad FS\_base\_clay = 3.458$$ Equation 8

FOR COMBINED CLAY-SILT LAYER

For a clay layer thicker than one sixth of the tank diameter $Nc1 := 6.1$ Equation 4

For a clay layer thinner than one sixth of the tank diamete $Nc2 := 4.1 + \left(\frac{Dc}{3Tclay\_silt}\right)$ Equation 5

$$Nc2 = 9.028$$

Place holder for easier coding: $PH := \frac{Dc}{Tclay\_silt}$

$Nc := if[(PH < 6), Nc1, Nc2] \qquad Nc = 9.028$

$Qult := Su\_clay\_silt \cdot Nc + \gamma ob \cdot Tob \qquad Qult = 40661 \cdot psf$

$$FS\_base\_clay\_silt := \frac{Su\_clay\_silt \cdot Nc}{\left[q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2\right]} \qquad FS\_base\_clay\_silt = 11.971$$ Equation 8

EDGE SHEAR STABILITY EVALUATION - AT FULL HYDRO TEST - NO STRENGTH GAIN

IMPROVED CLAY

$Qapp := q + \gamma p \cdot Tp \qquad Qapp = 3725.7 \cdot psf$ Equation 11

With of segment that might fail $B := \begin{pmatrix} 10 \\ 20 \\ 40 \\ 60 \\ 80 \\ 100 \\ 120 \end{pmatrix} ft$

$Nc := 5.2 + \frac{B}{D} \qquad Nc = \begin{pmatrix} 5.242 \\ 5.283 \\ 5.367 \\ 5.45 \\ 5.533 \\ 5.617 \\ 5.7 \end{pmatrix}$

$Qall := Su\_clay \cdot Nc + \gamma p \cdot Tp \qquad Qall = \begin{pmatrix} 5771.446 \\ 5810.617 \\ 5888.96 \\ 5967.303 \\ 6045.645 \\ 6123.988 \\ 6202.33 \end{pmatrix} \cdot psf$

$FS\_edge\_clay := \frac{Qall}{Qapp} \qquad FS\_edge\_clay = \begin{pmatrix} 1.549 \\ 1.56 \\ 1.581 \\ 1.602 \\ 1.623 \\ 1.644 \\ 1.665 \end{pmatrix}$

EDGE SHEAR STABILITY EVALUATION - AT FULL HYDRO TEST - NO STRENGTH GAIN

IMPROVED CLAY and SILT

$Qapp := q + \gamma p \cdot Tp$ $\quad Qapp = 3725.7 \cdot psf$ $\quad$ Equation 11

With of segment that might fail $\quad B := \begin{pmatrix} 10 \\ 20 \\ 40 \\ 60 \\ 80 \\ 100 \\ 120 \end{pmatrix} ft$ $\quad Nc := 5.2 + \frac{B}{D}$ $\quad Nc = \begin{pmatrix} 5.242 \\ 5.283 \\ 5.367 \\ 5.45 \\ 5.533 \\ 5.617 \\ 5.7 \end{pmatrix}$

$Qall := Su\_clay\_silt \cdot Nc + \gamma p \cdot Tp \quad Qall = \begin{pmatrix} 23931.739 \\ 24115.269 \\ 24482.328 \\ 24849.388 \\ 25216.447 \\ 25583.507 \\ 25950.566 \end{pmatrix} \cdot psf$

$FS\_edge\_clay\_silt := \frac{Qall}{Qapp} \quad FS\_edge\_clay\_silt = \begin{pmatrix} 6.423 \\ 6.473 \\ 6.571 \\ 6.67 \\ 6.768 \\ 6.867 \\ 6.965 \end{pmatrix}$

SUMMARY OF BASE AND EDGE SHEAR STABILITY EVALUATION - AT FULL HYDRO TEST - NO STRENGTH GAIN

$FS\_base\_clay = 3.458$

$FS\_base\_clay\_silt = 11.971$

$FS\_edge\_clay = \begin{pmatrix} 1.549 \\ 1.56 \\ 1.581 \\ 1.602 \\ 1.623 \\ 1.644 \\ 1.665 \end{pmatrix}$ $\quad FS\_edge\_clay\_silt = \begin{pmatrix} 6.423 \\ 6.473 \\ 6.571 \\ 6.67 \\ 6.768 \\ 6.867 \\ 6.965 \end{pmatrix}$

BASE SHEAR STABILITY EVALUATION - IN SERVICE - NO STRENGTH GAIN

$q := 2500psf$

$Dc := D + Tp + Tob \qquad Dc = 255.75 \cdot ft$ Equation 1

Bearing pressure at top of clay:

$$Qclay := q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2 + \gamma ob \cdot Tob$$ Equation 2

$$Qclay = 3881.641 \cdot psf$$

FOR CLAY LAYER ONLY

For a clay layer thicker than one sixth of the tank diameter $Nc1 := 6.1$ Equation 4

For a clay layer thinner than one sixth of the tank diamete $Nc2 := 4.1 + \left(\frac{Dc}{3Tclay}\right)$ Equation 5

$$Nc2 = 12.219$$

Place holder for easier coding: $PH := \frac{Dc}{Tclay}$

$Nc := if[(PH < 6), Nc1, Nc2] \qquad Nc = 12.219$

$Qult := Su\_clay \cdot Nc + \gamma ob \cdot Tob \qquad Qult = 12383.624 \cdot psf$

$$FS\_base\_clay := \frac{Su\_clay \cdot Nc}{\left[q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2\right]} \qquad FS\_base\_clay = 3.848$$ Equation 8

FOR COMBINED CLAY-SILT LAYER

For a clay layer thicker than one sixth of the tank diameter $Nc1 := 6.1$ Equation 4

For a clay layer thinner than one sixth of the tank diamete $Nc2 := 4.1 + \left(\frac{Dc}{3Tclay\_silt}\right)$ Equation 5

$$Nc2 = 9.028$$

Place holder for easier coding: $PH := \frac{Dc}{Tclay\_silt}$

$Nc := if[(PH < 6), Nc1, Nc2] \qquad Nc = 9.028$

$Qult := Su\_clay\_silt \cdot Nc + \gamma ob \cdot Tob \qquad Qult = 40661 \cdot psf$

$$FS\_base\_clay\_silt := \frac{Su\_clay\_silt \cdot Nc}{\left[q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2\right]} \qquad FS\_base\_clay\_silt = 13.32$$ Equation 8

EDGE SHEAR STABILITY EVALUATION - IN SERVICE - NO STRENGTH GAIN

IMPROVED CLAY

$Qapp := q + \gamma p \cdot Tp \qquad Qapp = 3343.7 \cdot psf$ Equation 11

With of segment that might fail $B := \begin{pmatrix} 10 \\ 20 \\ 40 \\ 60 \\ 80 \\ 100 \\ 120 \end{pmatrix} ft$

$Nc := 5.2 + \frac{B}{D} \qquad Nc = \begin{pmatrix} 5.242 \\ 5.283 \\ 5.367 \\ 5.45 \\ 5.533 \\ 5.617 \\ 5.7 \end{pmatrix}$

$Qall := Su\_clay \cdot Nc + \gamma p \cdot Tp \qquad Qall = \begin{pmatrix} 5771.446 \\ 5810.617 \\ 5888.96 \\ 5967.303 \\ 6045.645 \\ 6123.988 \\ 6202.33 \end{pmatrix} \cdot psf$

$FS\_edge\_clay := \frac{Qall}{Qapp} \qquad FS\_edge\_clay = \begin{pmatrix} 1.726 \\ 1.738 \\ 1.761 \\ 1.785 \\ 1.808 \\ 1.832 \\ 1.855 \end{pmatrix}$

EDGE SHEAR STABILITY EVALUATION - IN SERVICE - NO STRENGTH GAIN

IMPROVED CLAY and SILT

$Qapp := q + \gamma p \cdot Tp$ $\quad Qapp = 3343.7 \cdot psf$ $\quad$ Equation 11

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$$FS\_edge\_clay\_silt := \frac{Qall}{Qapp} \quad FS\_edge\_clay\_silt = \begin{pmatrix} 7.157 \\ 7.212 \\ 7.322 \\ 7.432 \\ 7.541 \\ 7.651 \\ 7.761 \end{pmatrix}$$

SUMMARY OF BASE AND EDGE SHEAR STABILITY EVALUATION - IN SERVCIE - NO STRENGTH GAIN

$FS\_base\_clay = 3.848$

$FS\_base\_clay\_silt = 13.32$

$$FS\_edge\_clay = \begin{pmatrix} 1.726 \\ 1.738 \\ 1.761 \\ 1.785 \\ 1.808 \\ 1.832 \\ 1.855 \end{pmatrix} \qquad FS\_edge\_clay\_silt = \begin{pmatrix} 7.157 \\ 7.212 \\ 7.322 \\ 7.432 \\ 7.541 \\ 7.651 \\ 7.761 \end{pmatrix}$$

Units

$kips \equiv 1000 \cdot lbf$

$ksf := 1000 \cdot psf$

$tsf := 2 \cdot ksf$

$kPa := 1000 \cdot Pa$

$pi := 3.14159$

$ksi \equiv \frac{1000 \cdot lbf}{in^2}$

$pcf \equiv \frac{lbf}{ft^3}$

$in^2 = 0.001\, m^2$

$psi \equiv \frac{lbf}{in^2}$

$psf \equiv \frac{lbf}{ft^2}$

$pcf \equiv \frac{lbf}{ft^3}$

$kcf \equiv \frac{k}{ft^3}$

$k \equiv kips$

$N \equiv newton$

$ORIGIN \equiv 1$

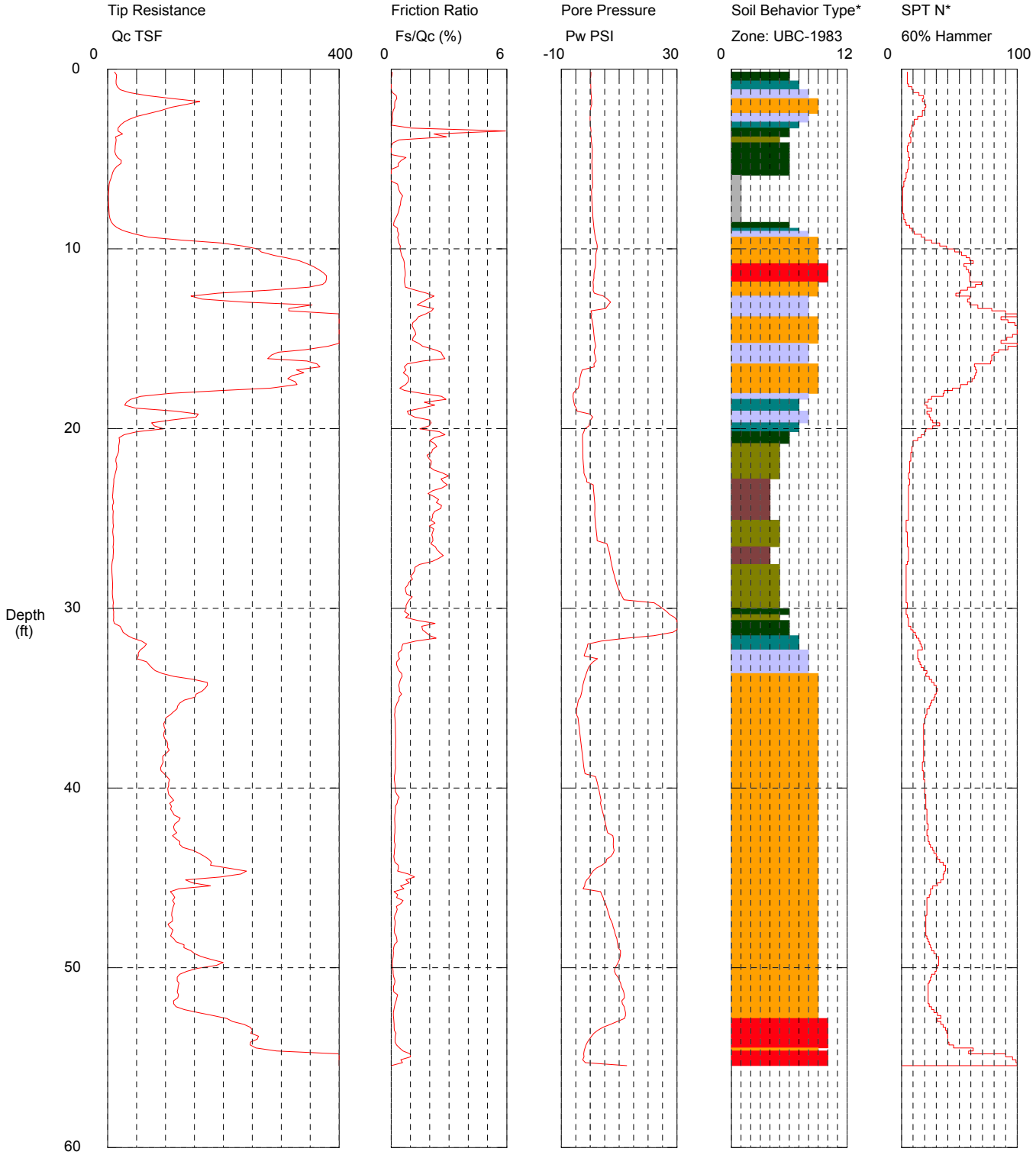
$MPa := 10^6 \cdot Pa$

$kN \equiv 1000 newton$

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-151
Cone Used: DDG1254

CPT Date/Time: 11/21/2014 4:04:51 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 55.45 feet

Depth Increment = 0.164 feet

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

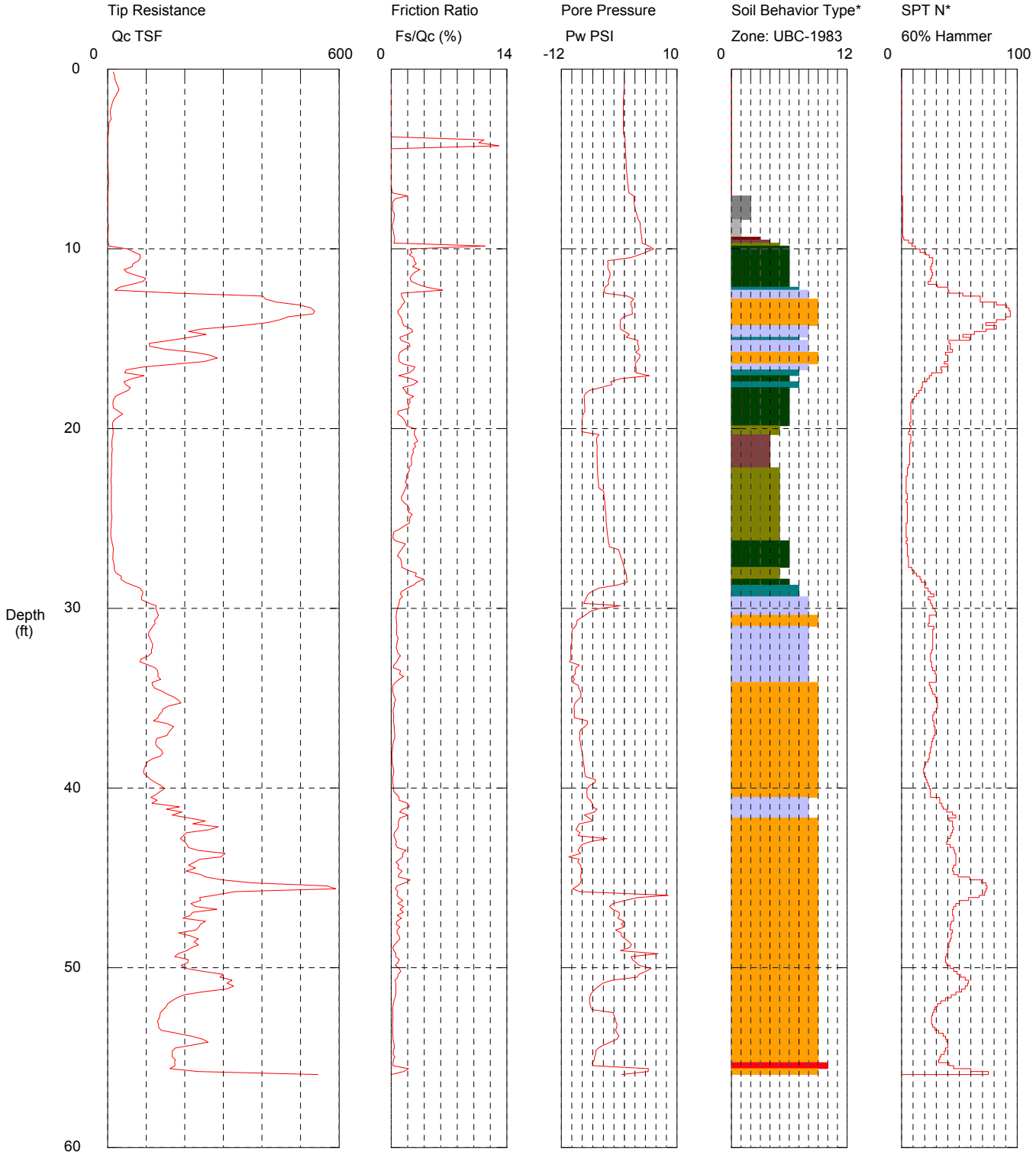
InSitu Engineering Predrilled top 9 feet

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

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-152A
Cone Used: DDG1254

CPT Date/Time: 11/22/2014 2:10:05 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 55.94 feet

Depth Increment = 0.164 feet

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

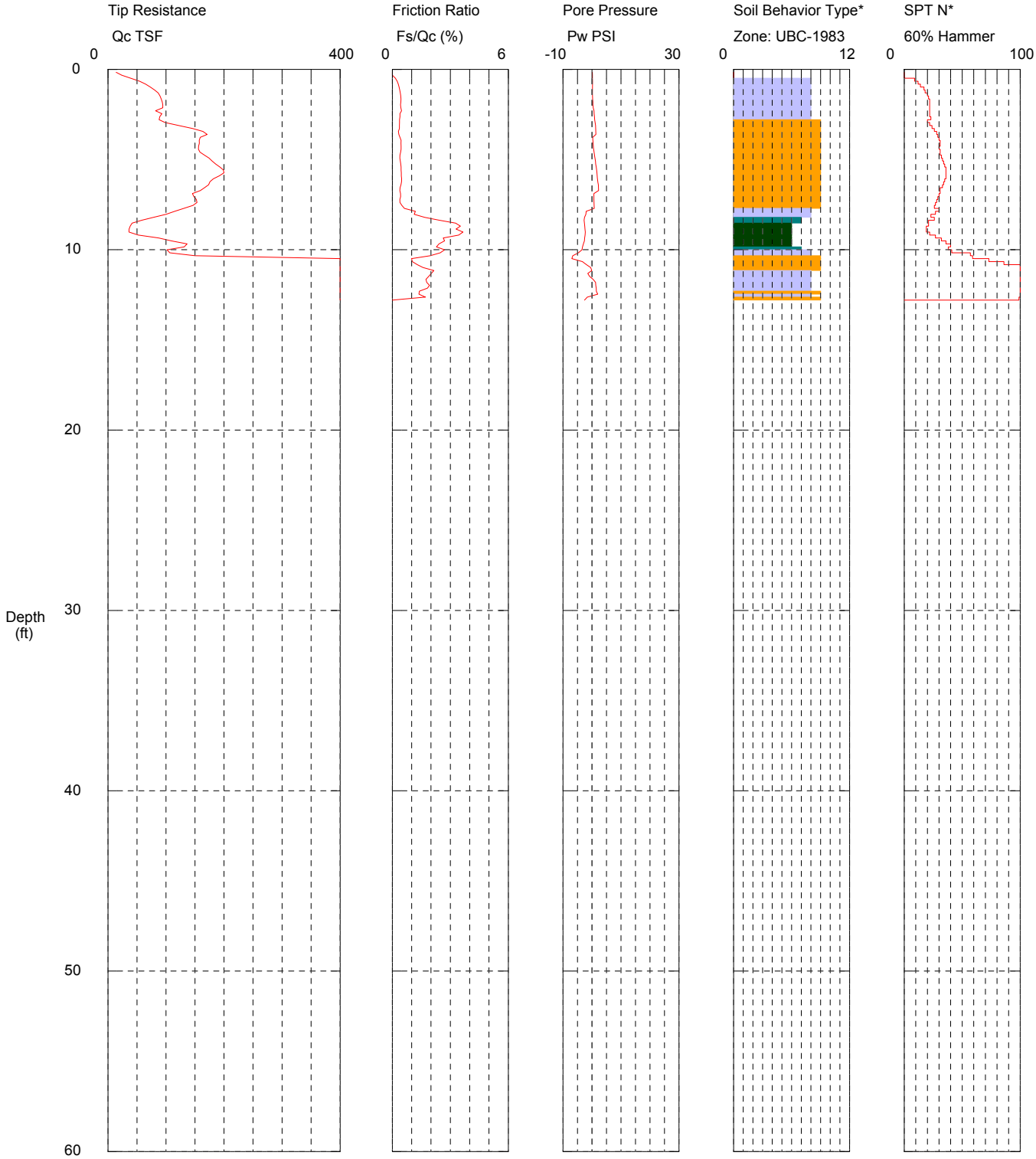
InSitu Engineering Predrilled top 13 feet

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

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-153(s)
Cone Used: DDG1254

CPT Date/Time: 11/12/2014 12:16:48 PM
Location: Port of Vancouver -Tesoro Distribution Center
Job Number:



Maximum Depth = 12.80 feet

Depth Increment = 0.164 feet

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

InSitu Engineering

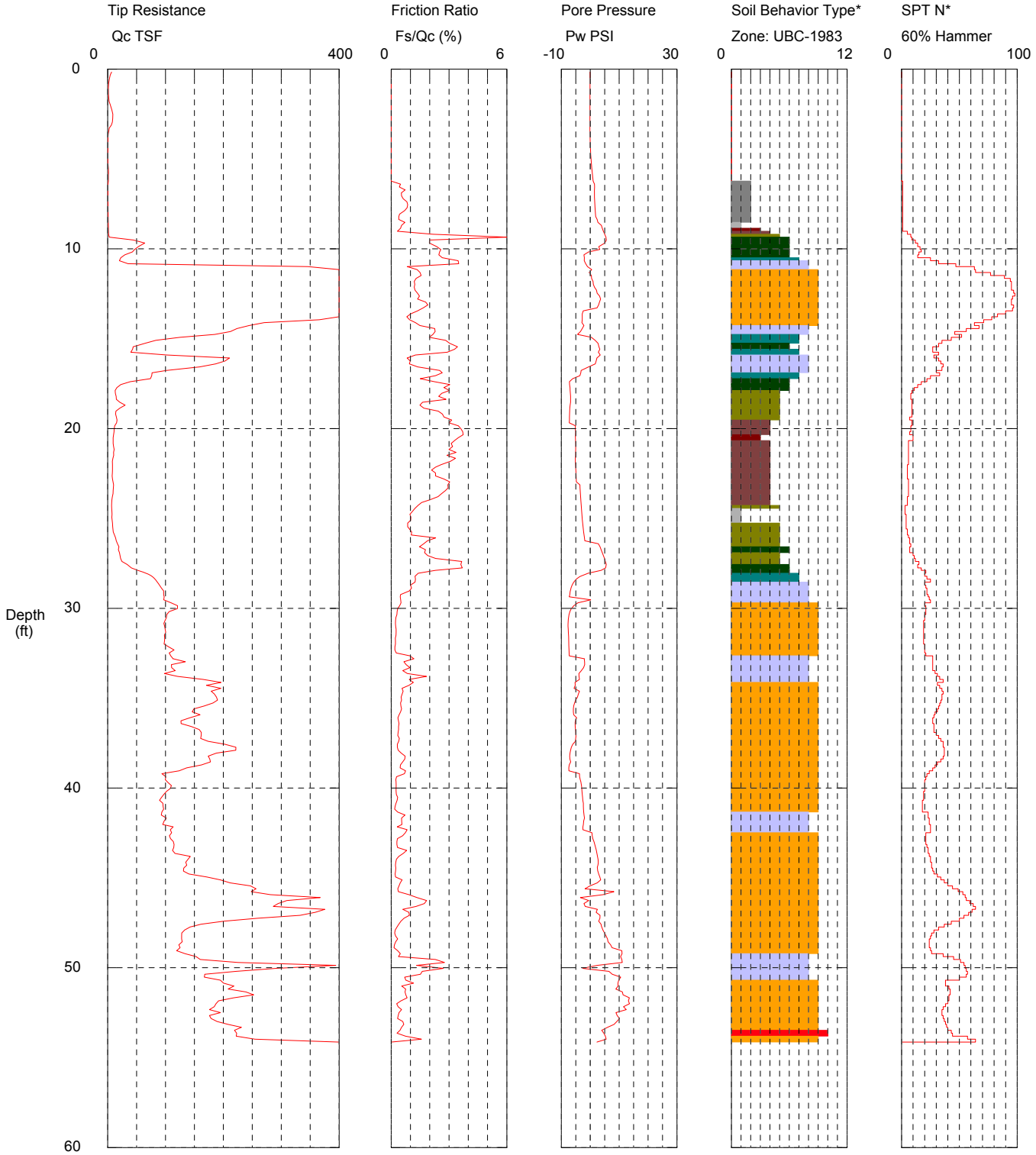
*Soil behavior type and SPT based on data from UBC-1983

EX-0001-007090-PCE

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-153(S)A
Cone Used: DDG1254

CPT Date/Time: 11/22/2014 3:40:54 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 54.13 feet

Depth Increment = 0.164 feet

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

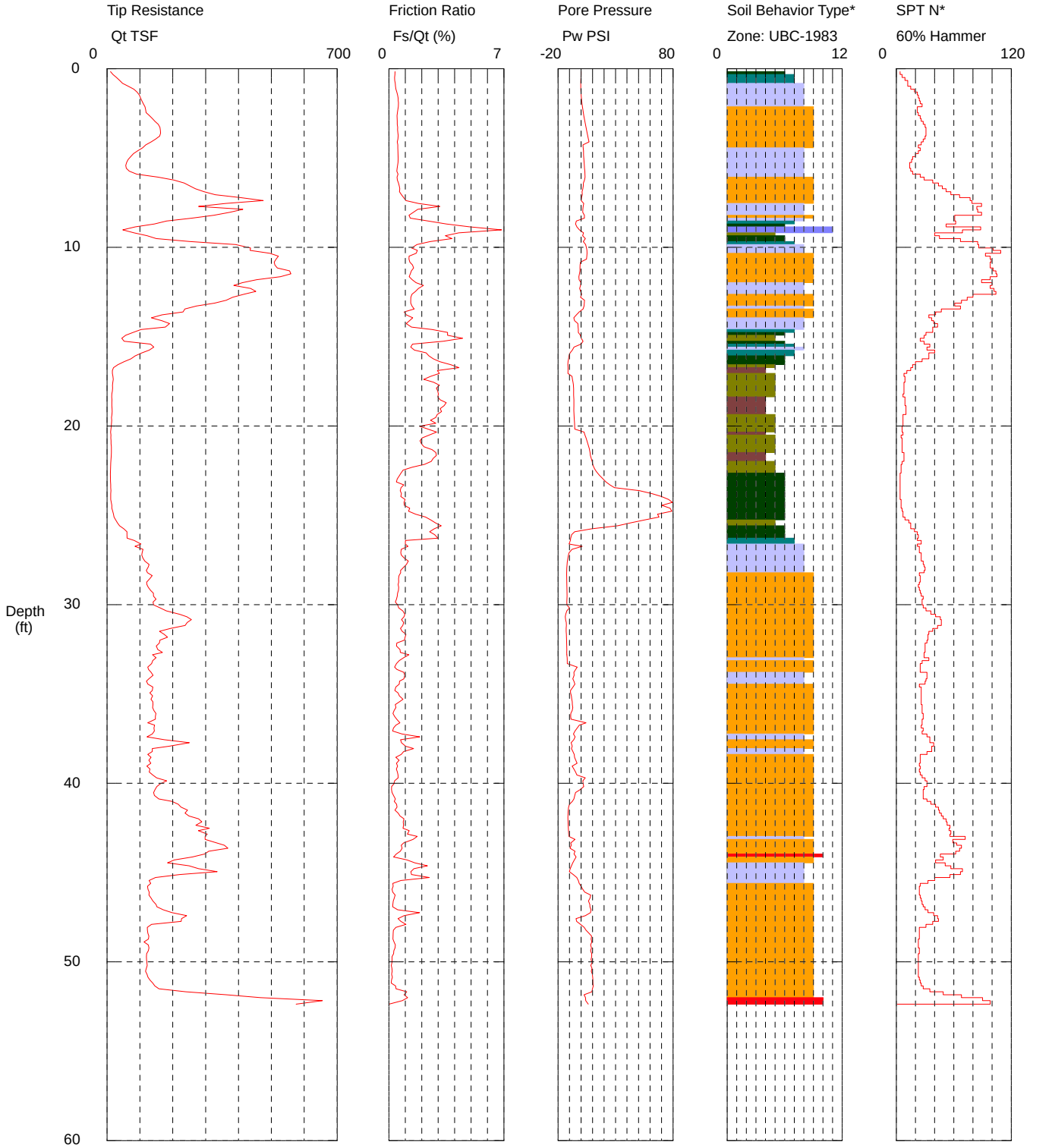
InSitu Engineering Predrilled top 11 feet

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

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-154
Cone Used: DDG1254

CPT Date/Time: 11/12/2014 8:31:19 AM
Location: Port of Vancouver
Job Number: 860058



Maximum Depth = 52.36 feet

Depth Increment = 0.164 feet

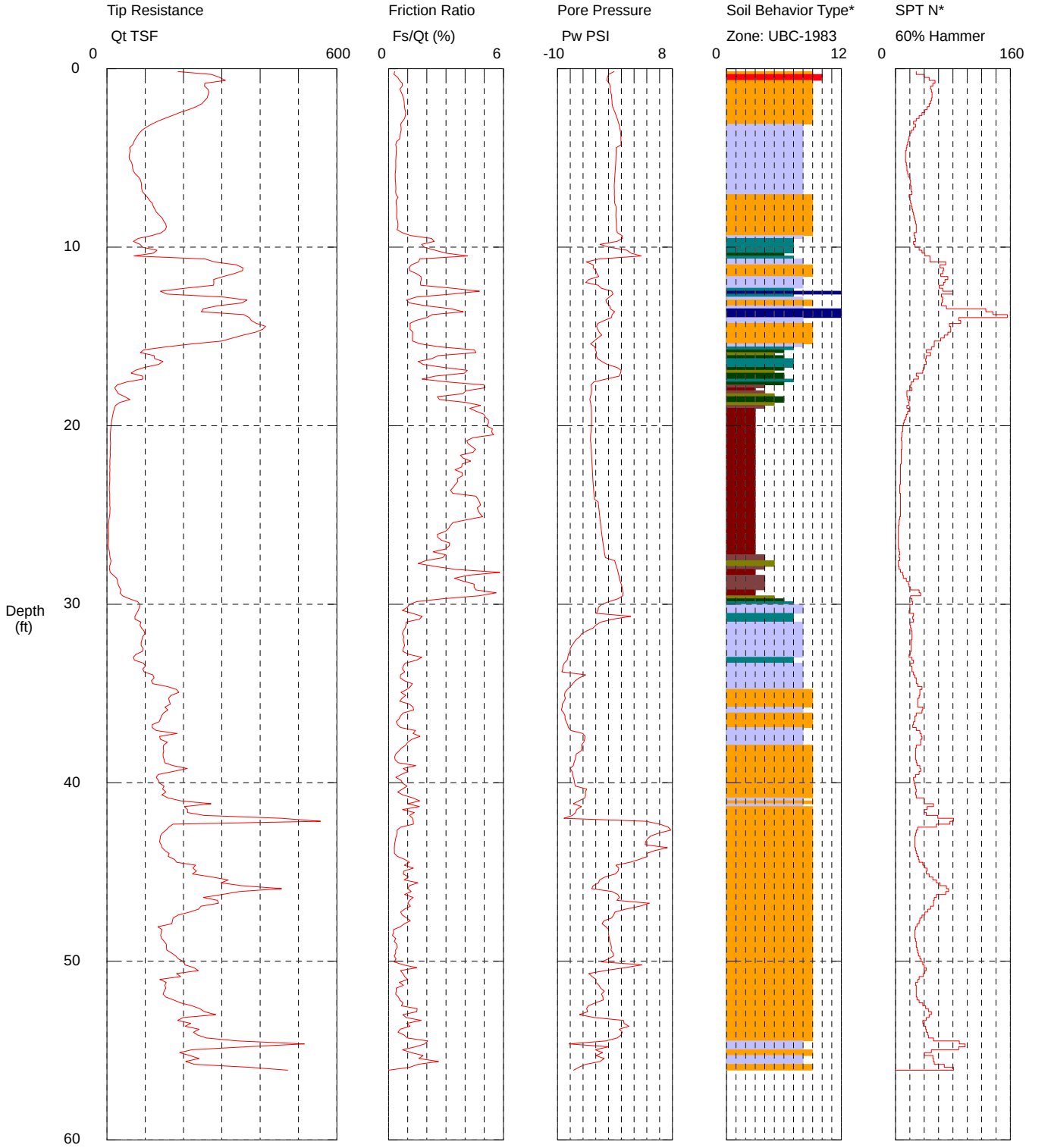
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

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

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-155
Cone Used: DDG1263

CPT Date/Time: 11/10/2014 2:57:41 PM
Location: Port of Vancouver
Job Number: 860058



Maximum Depth = 56.10 feet

Depth Increment = 0.164 feet

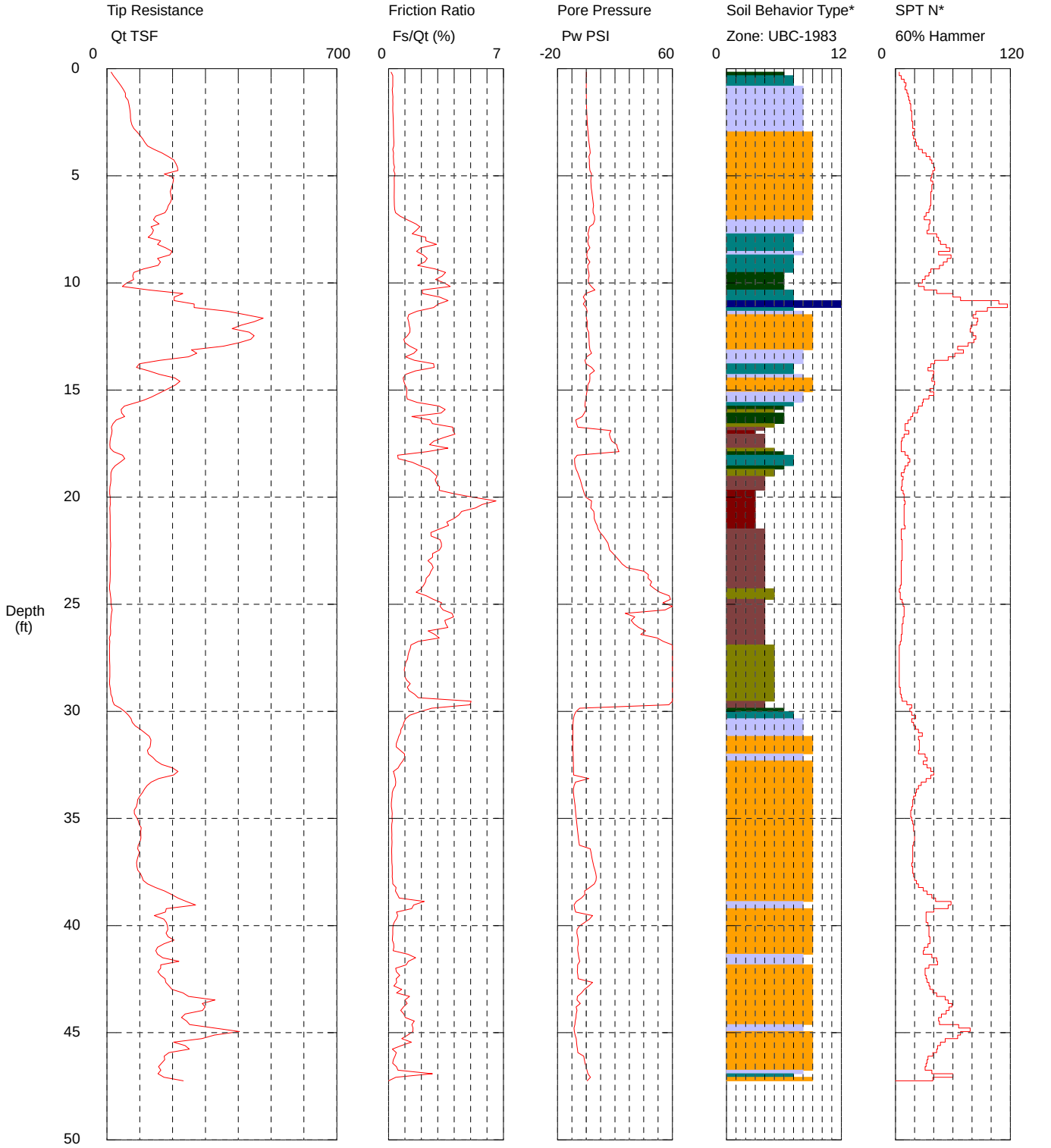
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

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

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-156
Cone Used: DDG1254

CPT Date/Time: 11/11/2014 5:27:30 PM
Location: Port of Vancouver
Job Number: 860058



Maximum Depth = 47.24 feet

Depth Increment = 0.164 feet

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

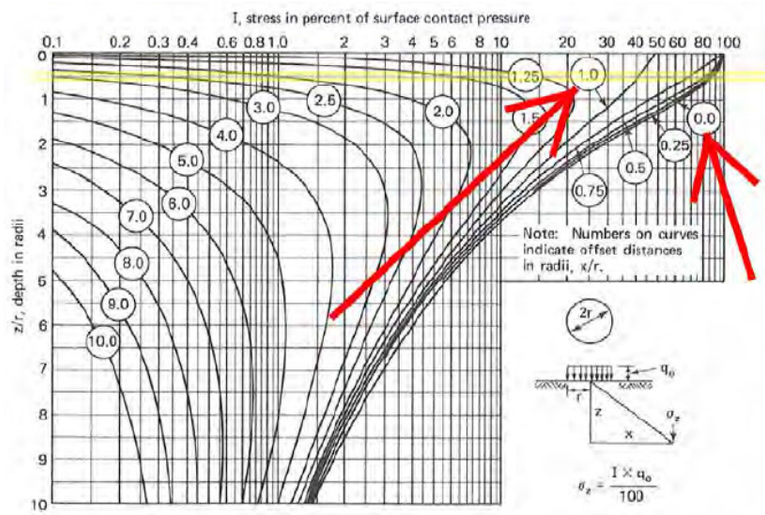
*Soil behavior type and SPT based on data from UBC-1983

Tank 5

Design Summary
Tank 5

This table assumes the exploration is directly below the center of the tank.

Tank 3		Orig	Imp.	Improved		Original	
Soil Profile	Depth	Liq Sett.	Liq Sett.	Rapid Sett.	Long Term Sett.	Rapid Sett.	Long Term Sett.
HB-CPT-131	53.6-ft	3.76-in.	1.77-in.	2.18-in.	.16-in.	.83-in.	2.75-in.
HB-CPT-135	53.0-ft	3.75-in.	2.16-in.	1.98-in.	.21-in.	1.08-in.	2.5-in.
X-CPT-1	54.0-ft	4.18-in.	1.42-in.	1.65-in.	.19-in.	1.04-in.	2.09-in.
HB-CPT-133	53.0-ft	3.41-in.	1.51-in.	2.48-in.	.09-in.	.71-in.	3.23-in.
HB-CPT-132	53.8-ft	4.01-in.	2.19-in.	2.94-in.	.1-in.	.83-in.	3.86-in.
HB-CPT-134(s)	53.5-ft	2.6-in.	1.07-in.	1.14-in.	.19-in.	.84-in.	1.39-in.
Average:		**3.62-in.**	**1.81-in.**	**2.24-in.**	**.15-in.**	**.9-in.**	**2.89-in.**



This table calculates Settlement at the edge of the tank.

Tank 3		Orig	Imp.	Improved		Original	
Soil Profile	Depth	Liq Sett.	Liq Sett.	Rapid Sett.	Long Term Sett.	Rapid Sett.	Long Term Sett.
HB-CPT-131	53.6-ft	3.76-in.	1.77-in.	1.09-in.	.08-in.	.42-in.	1.37-in.
HB-CPT-135	53.0-ft	3.75-in.	2.16-in.	.99-in.	.11-in.	.54-in.	1.25-in.
X-CPT-1	54.0-ft	4.18-in.	1.42-in.	.82-in.	.1-in.	.52-in.	1.05-in.
HB-CPT-133	53.0-ft	3.41-in.	1.51-in.	1.24-in.	.04-in.	.35-in.	1.61-in.
HB-CPT-132	53.8-ft	4.01-in.	2.19-in.	2.94-in.	.1-in.	.83-in.	3.86-in.
HB-CPT-134(s)	53.5-ft	2.6-in.	1.07-in.	1.14-in.	.19-in.	.84-in.	1.39-in.
Average:		**3.82-in.**	**1.81-in.**	**1.42-in.**	**.09-in.**	**.53-in.**	**1.83-in.**

Note: No reduction to total soil settlement is taken for the liquefaction condition.

Worksheet Tab: 11- Boulanger (improved)

Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Hayward Baker Inc. THINK SAFE	SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 5			
	Job no.	860079	Design by:	M. Rohrbach	Date: 14-Apr-15
	File	CPT-Liquefaction-Boulanger	Checked		
	Spreadsheet Rev.	23-Dec-14	Reviewed		

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-131

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	16.7-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	28.69	ft.
Elevation of bottom of Soil Profile	-24.95	ft.
Elevation of Installation	24.5	Work from Existing Grade
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	53.6-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	397.80-psf	

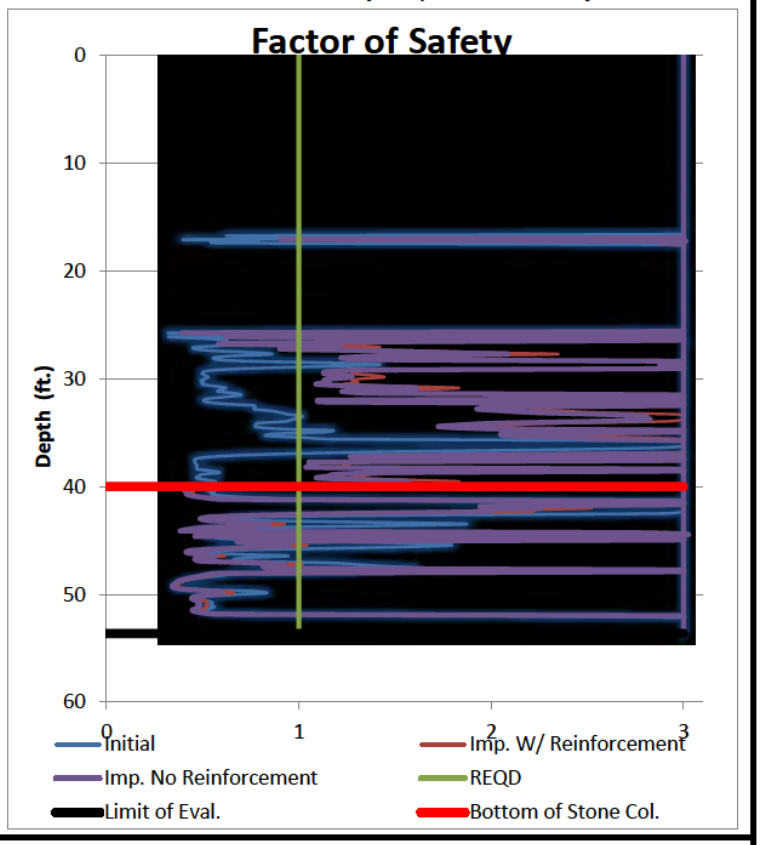
Stone Column Design Information

L =	40-ft		ϕ_c = 50
Column Spacing	8-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 53.6-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 20.2-ft.

Calculated Min: 3.60-in via T&S, Min never < 0.5 in.

Calculated Max: 3.76-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 10.5-ft.

Calculated Min: 1.90-in via T&S, Min never < 0.5 in.

Calculated Max: 2.17-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 9.8-ft.

Calculated Min: 1.77-in via T&S, Min never < 0.5 in.

Calculated Max: 1.94-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Worksheet Tab: 14-Elastic Sett. (Imp.) Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 5 IMPROVED CASE: CPT-131

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment

= $1-0.5*(q/(q_{bar}-q)$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

q = gDf

Stone Column Length 40-ft

Reduction factor on Priebe: 1

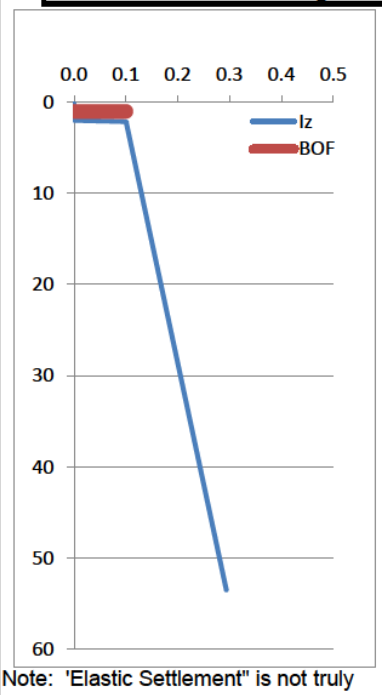
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	
L	212.7-ft	
B	212.7-ft	
fill density	130-pcf	
Elevation of Top of Soil Profile	28.69	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es=X\cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1$=0.5B=	106.3
I_z = 0	at depth	$z=z_2$=2.0B=	425.4
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1$=1.0B=	212.7
I_z = 0	at depth	$z=z_2$=4.0B=	850.8
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

Σ(Iz/Es)Δz (ft2/lb)= 1.42E-05

Total Elastic Settlement (Flexible foundation)

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0545-ft 0.65-in

Total Rapid Elastic Settlement

$S_e=C_1(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0354-ft 0.42-in

Long Term Settlement 0.23-in

Calculated Consolidation Type Settlement:

Total:	1.69-in	Adjusted for footing Type
Recomp.	0.07-in	Adjusted for footing Type
Virgin Sett.	1.62-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	1.76-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:

Total: 2.34-in

Rapid: 2.18-in

Long Term: 0.16-in

Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Hayward Baker Inc.
THINK SAFE

SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 5		
Job no.	860079	Design by: M. Rohrbach	Date: 14-Apr-15
File	CPT-Liquefaction-Boulanger	Checked	
Spreadsheet Rev.	23-Dec-14	Reviewed	

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-132

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	16.7-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	28.65	ft.
Elevation of bottom of Soil Profile	-25.16	ft.
Elevation of Installation	24.5	Work from Existing Grade
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	53.8-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	403.00-psf	

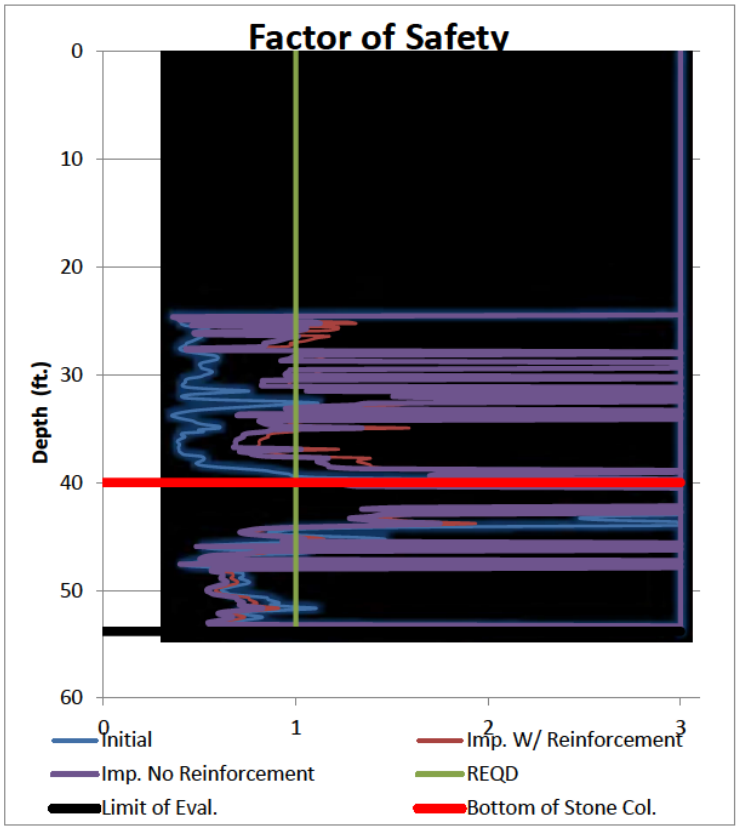
Stone Column Design Information

L =	40-ft		ϕ_c = 50
Column Spacing	8-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 53.8-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 22.5-ft.

Calculated Min: 3.91-in via T&S, Min never < 0.5 in.

Calculated Max: 4.01-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 15.3-ft.

Calculated Min: 2.68-in via T&S, Min never < 0.5 in.

Calculated Max: 2.91-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 12.7-ft.

Calculated Min: 2.19-in via T&S, Min never < 0.5 in.

Calculated Max: 2.47-in via T&S

Special Notes / Comments

Notes:
1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 5 — IMPROVED CASE: CPT-132

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e = C_1 C_2 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment
= $1 - 0.5*(q/(q_{bar} - q))$

C_2 = a correction factor top account for creep ion soil
= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

q = gDf

Stone Column Length 40-ft

Reduction factor on Priebe: 1

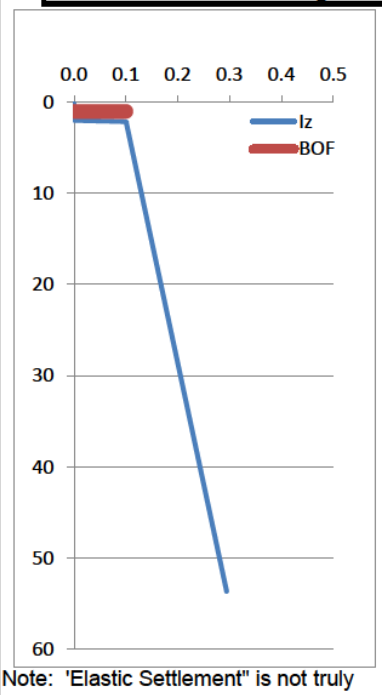
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	
L	212.7-ft	
B	212.7-ft	
fill density	130-pcf	
Elevation of Top of Soil Profile	28.65	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es = X \cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=0.5B$=	106.3
I_z = 0	at depth	$z=z_2=2.0B$=	425.4
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=1.0B$=	212.7
I_z = 0	at depth	$z=z_2=4.0B$=	850.8
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

$\Sigma(Iz/Es)\Delta z$ (ft2/lb)= **1.21E-05**

Total Elastic Settlement (Flexible foundation)

$S_e = C_1 C_2 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$= 0.0463-ft 0.56-in

Total Rapid Elastic Settlement

$S_e = C_1 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$= 0.0301-ft 0.36-in

Long Term Settlement 0.20-in

Calculated Consolidation Type Settlement:

Total:	2.48-in	Adjusted for footing Type
Recomp.	0.10-in	Adjusted for footing Type
Virgin Sett.	2.39-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	2.58-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:
Total: 3.04-in
Rapid: 2.94-in
Long Term: 0.10-in

Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Hayward Baker Inc.
THINK SAFE

SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 5		
Job no.	860079	Design by: M. Rohrbach	Date: 14-Apr-15
File	CPT-Liquefaction-Boulanger	Checked	
Spreadsheet Rev.	23-Dec-14	Reviewed	

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-133

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	16.6-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	28.64	ft.
Elevation of bottom of Soil Profile	-24.35	ft.
Elevation of Installation	24.5	Work from Existing Grade
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	53.0-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	404.30-psf	

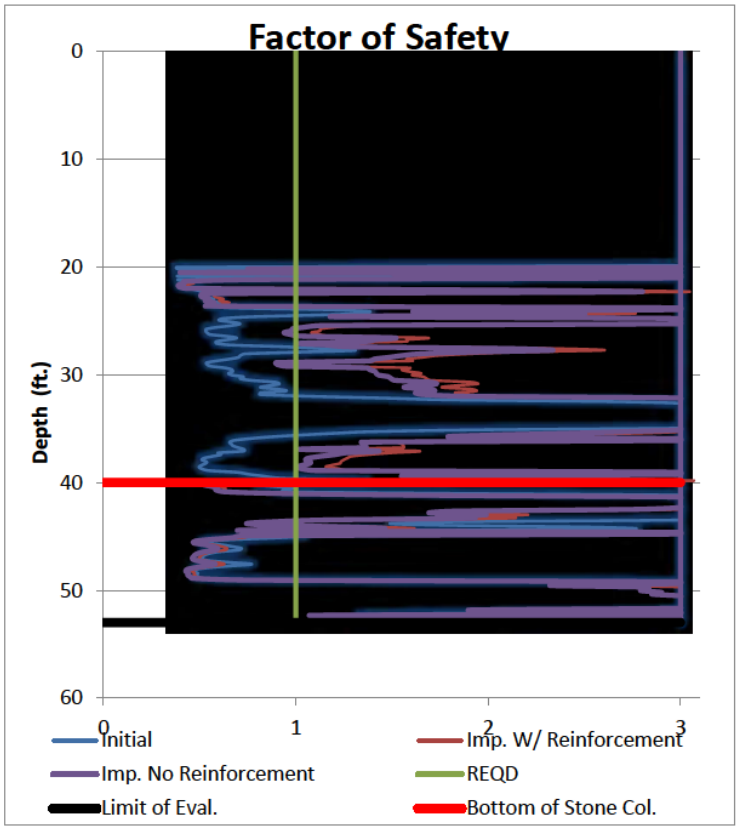
Stone Column Design Information

L =	40-ft		ϕ_c = 50
Column Spacing	8-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 53.0-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 19.6-ft.

Calculated Min: 3.30-in via T&S, Min never < 0.5 in.

Calculated Max: 3.41-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 9.9-ft.

Calculated Min: 1.73-in via T&S, Min never < 0.5 in.

Calculated Max: 1.98-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 8.7-ft.

Calculated Min: 1.51-in via T&S, Min never < 0.5 in.

Calculated Max: 1.75-in via T&S

Special Notes / Comments

Notes:
1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 5 — IMPROVED CASE: CPT-133

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment
= $1-0.5*(q/(q_{bar}-q)$

C_2 = a correction factor top account for creep ion soil
= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

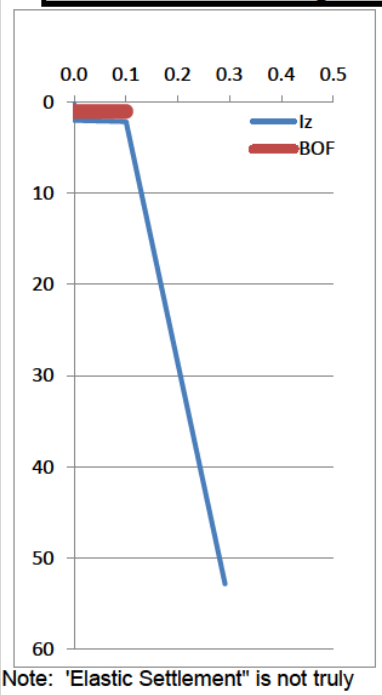
q = gDf

Stone Column Length 40-ft
Reduction factor on Priebe: 1

Foundation Type: Flexible

Correlation value between Es and q_c: $Es=X \cdot q_c$
x = 3.0
*default Value of X is 3

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	
L	212.7-ft	
B	212.7-ft	
fill density	130-pcf	
Elevation of Top of Soil Profile	28.64	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1$=0.5B=	106.3
I_z = 0	at depth	$z=z_2$=2.0B=	425.4
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1$=1.0B=	212.7
I_z = 0	at depth	$z=z_2$=4.0B=	850.8
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

$\Sigma(Iz/Es)\Delta z$ (ft2/lb)= **1.07E-05**

Total Elastic Settlement (Flexible foundation)
$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0410-ft 0.49-in

Total Rapid Elastic Settlement
$S_e=C_1(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0266-ft 0.32-in

Long Term Settlement 0.17-in

Calculated Consolidation Type Settlement:

Total:	2.08-in	Adjusted for footing Type
Recomp.	0.08-in	Adjusted for footing Type
Virgin Sett.	1.99-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	2.16-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:
Total: 2.57-in
Rapid: 2.48-in
Long Term: 0.09-in

Worksheet Tab: 11- Boulanger (improved)

Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Hayward Baker Inc. THINK SAFE	SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 5			
	Job no.	860079	Design by:	M. Rohrbach	Date: 14-Apr-15
	File	CPT-Liquefaction-Boulanger	Checked		
	Spreadsheet Rev.	23-Dec-14	Reviewed		

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-134(S)

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	16.5-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	28.53	ft.
Elevation of bottom of Soil Profile	-24.95	ft.
Elevation of Installation	24.5	Work from Existing Grade
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	53.5-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	418.60-psf	

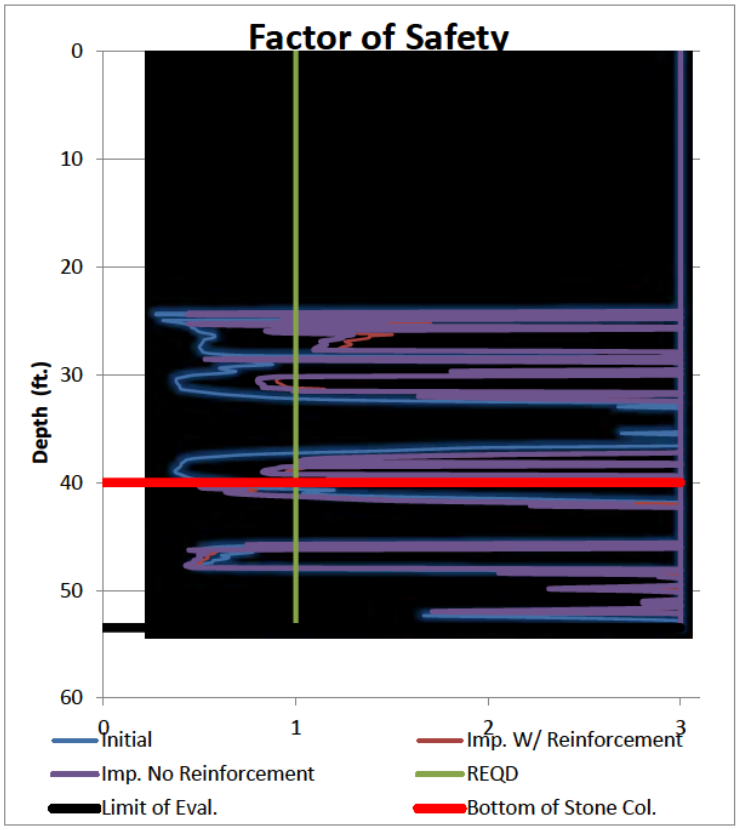
Stone Column Design Information

L =	40-ft		ϕ_c = 50
Column Spacing	8-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 53.5-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 13.0-ft.

Calculated Min: 2.51-in via T&S, Min never < 0.5 in.

Calculated Max: 2.60-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 6.9-ft.

Calculated Min: 1.40-in via T&S, Min never < 0.5 in.

Calculated Max: 1.56-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 5.4-ft.

Calculated Min: 1.07-in via T&S, Min never < 0.5 in.

Calculated Max: 1.27-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 5 — IMPROVED CASE: CPT-134(S)

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e = C_1 C_2 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment
= $1 - 0.5*(q/(q_{bar} - q))$

C_2 = a correction factor top account for creep ion soil
= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

q = gDf

Stone Column Length 40-ft

Reduction factor on Priebe: 1

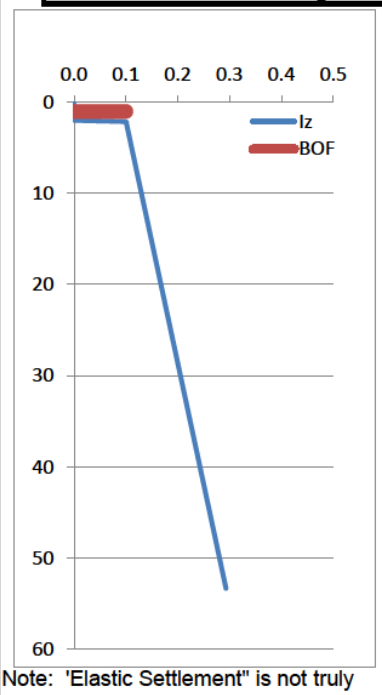
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	
L	212.7-ft	
B	212.7-ft	
fill density	130-pcf	
Elevation of Top of Soil Profile	28.53	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es = X \cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=0.5B$=	106.3
I_z = 0	at depth	$z=z_2=2.0B$=	425.4
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=1.0B$=	212.7
I_z = 0	at depth	$z=z_2=4.0B$=	850.8
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

$\Sigma(Iz/Es)\Delta z$ (ft2/lb)= **1.38E-05**

Total Elastic Settlement (Flexible foundation)

$S_e = C_1 C_2 (q_{bar} - q)\Sigma[(I_z/E_s)\Delta z]$= 0.0529-ft 0.63-in

Total Rapid Elastic Settlement

$S_e = C_1 (q_{bar} - q)\Sigma[(I_z/E_s)\Delta z]$= 0.0343-ft 0.41-in

Long Term Settlement 0.22-in

Calculated Consolidation Type Settlement:

Total:	0.70-in	Adjusted for footing Type
Recomp.	0.03-in	Adjusted for footing Type
Virgin Sett.	0.68-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	0.73-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:
Total: 1.33-in
Rapid: 1.14-in
Long Term: 0.19-in

Worksheet Tab: 11- Boulanger (improved)

Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Hayward Baker Inc. THINK SAFE	SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 5			
	Job no.	860079	Design by:	M. Rohrbach	Date: 14-Apr-15
	File	CPT-Liquefaction-Boulanger	Checked		
	Spreadsheet Rev.	23-Dec-14	Reviewed		

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-135

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	16.6-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	28.58	ft.
Elevation of bottom of Soil Profile	-24.41	ft.
Elevation of Installation	24.5	Work from Existing Grade
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	53.0-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	412.10-psf	

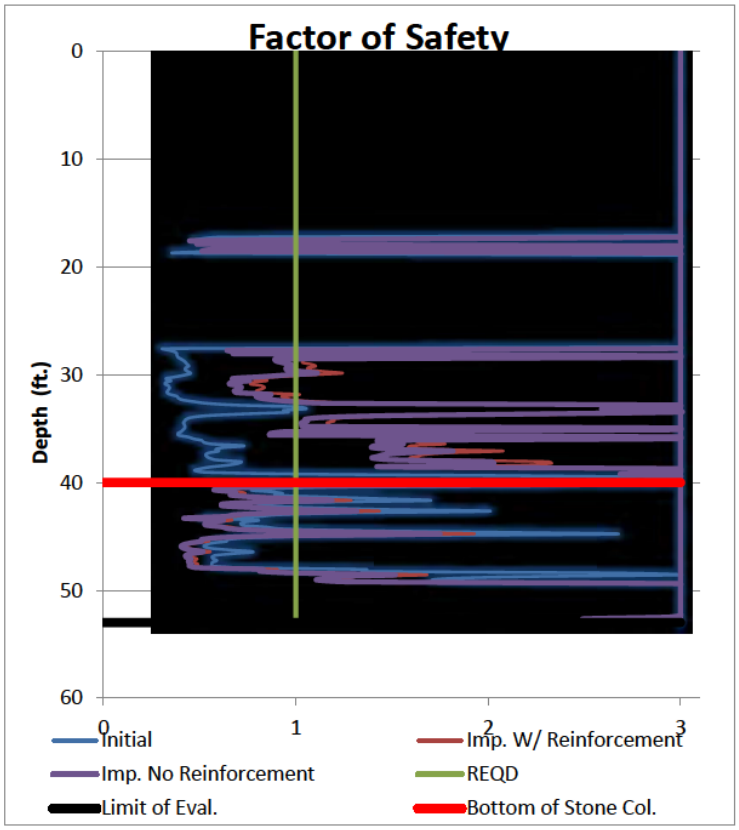
Stone Column Design Information

L =	40-ft		ϕ_c = 50
Column Spacing	8-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 53.0-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 18.2-ft.

Calculated Min: 3.66-in via T&S, Min never < 0.5 in.

Calculated Max: 3.75-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 13.1-ft.

Calculated Min: 2.68-in via T&S, Min never < 0.5 in.

Calculated Max: 2.85-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 10.7-ft.

Calculated Min: 2.16-in via T&S, Min never < 0.5 in.

Calculated Max: 2.43-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Worksheet Tab: 14-Elastic Sett. (Imp.) Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 5 IMPROVED CASE: CPT-135

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment
= $1-0.5*(q/(q_{bar}-q)$

C_2 = a correction factor top account for creep ion soil
= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

q = gDf

Stone Column Length 40-ft

Reduction factor on Priebe: 1

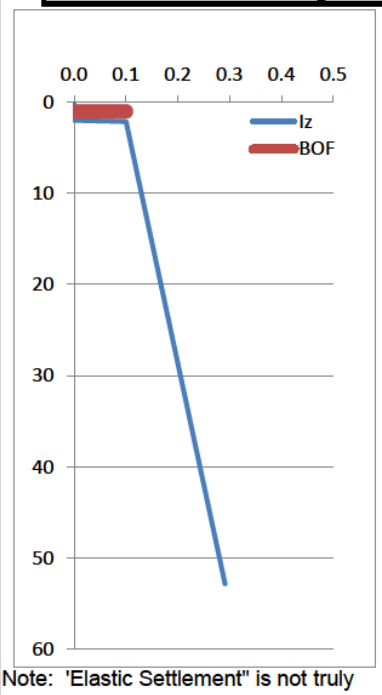
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	
L	212.7-ft	
B	212.7-ft	
fill density	130-pcf	
Elevation of Top of Soil Profile	28.58	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es=X\cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1$=**0.5**B=	106.3
I_z = 0	at depth	$z=z_2$=**2.0**B=	425.4
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1$=**1.0**B=	212.7
I_z = 0	at depth	$z=z_2$=**4.0**B=	850.8
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

$\Sigma(Iz/Es)\Delta z$ (ft2/lb)= 1.68E-05

Total Elastic Settlement (Flexible foundation)

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0644-ft 0.77-in

Total Rapid Elastic Settlement

$S_e=C_1(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0418-ft 0.50-in

Long Term Settlement 0.27-in

Calculated Consolidation Type Settlement:

Total:	1.42-in	Adjusted for footing Type
Recomp.	0.06-in	Adjusted for footing Type
Virgin Sett.	1.37-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	1.48-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:
Total: 2.19-in
Rapid: 1.98-in
Long Term: 0.21-in

Worksheet Tab: 11- Boulanger (improved)

Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Hayward Baker Inc. THINK SAFE	SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 5			
	Job no.	860079	Design by:	M. Rohrbach	Date: 14-Apr-15
	File	CPT-Liquefaction-Boulanger	Checked		
	Spreadsheet Rev.	23-Dec-14	Reviewed		

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-X-CPT-1

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	16.5-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	28.5	ft.
Elevation of bottom of Soil Profile	-25.47	ft.
Elevation of Installation	24.5	Work from Existing Grade
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	54.0-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	422.50-psf	

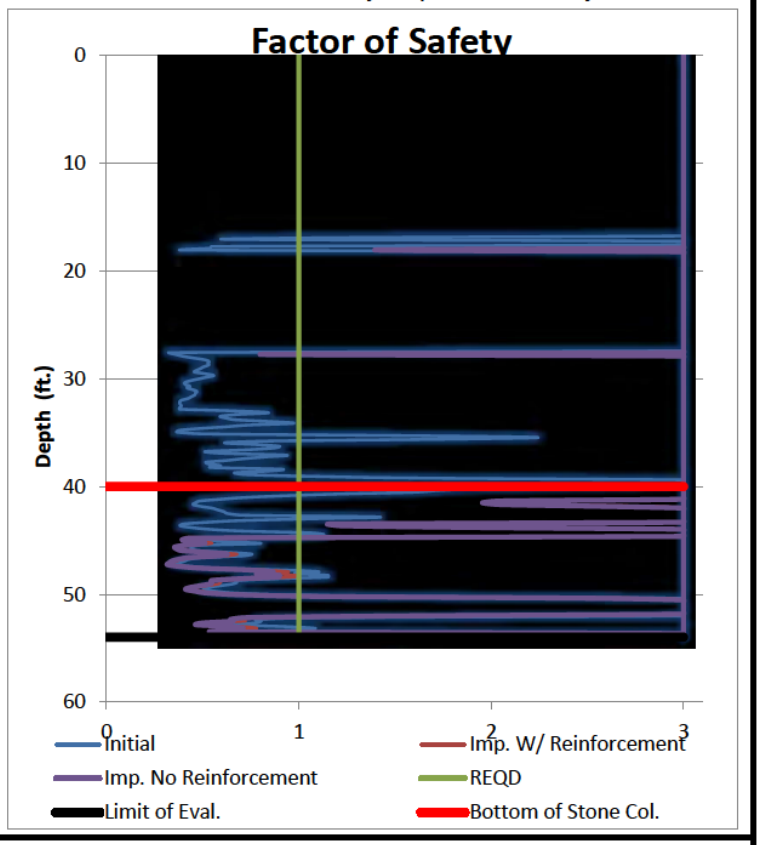
Stone Column Design Information

L =	40-ft		ϕ_c = 50
Column Spacing	8-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 54.0-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 20.8-ft.

Calculated Min: 4.06-in via T&S, Min never < 0.5 in.

Calculated Max: 4.18-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 7.1-ft.

Calculated Min: 1.42-in via T&S, Min never < 0.5 in.

Calculated Max: 1.45-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 7.1-ft.

Calculated Min: 1.42-in via T&S, Min never < 0.5 in.

Calculated Max: 1.44-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Worksheet Tab: 14-Elastic Sett. (Imp.) Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 5 IMPROVED CASE: CPT-X-CPT-1

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment
= $1-0.5*(q/(q_{bar}-q)$

C_2 = a correction factor top account for creep ion soil
= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

q = gDf

Stone Column Length 40-ft

Reduction factor on Priebe: 1

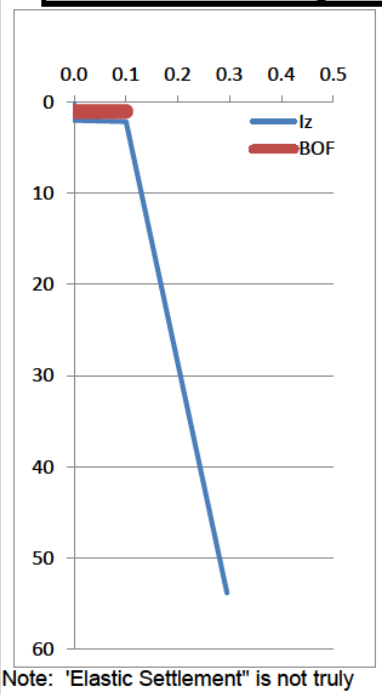
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	
L	212.7-ft	
B	212.7-ft	
fill density	130-pcf	
Elevation of Top of Soil Profile	28.5	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es=X\cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1$=0.5B=	106.3
I_z = 0	at depth	$z=z_2$=2.0B=	425.4
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1$=1.0B=	212.7
I_z = 0	at depth	$z=z_2$=4.0B=	850.8
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

Σ(Iz/Es)Δz (ft2/lb)= 1.52E-05

Total Elastic Settlement (Flexible foundation)

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0584-ft 0.70-in

Total Rapid Elastic Settlement

$S_e=C_1(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0379-ft 0.46-in

Long Term Settlement 0.24-in

Calculated Consolidation Type Settlement:

Total:	1.14-in	Adjusted for footing Type
Recomp.	0.05-in	Adjusted for footing Type
Virgin Sett.	1.10-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	1.19-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:

Total: 1.84-in

Rapid: 1.65-in

Long Term: 0.19-in

Tank 5
Stability Check Layer Thickness

Calculations for Tank Edge and Base Stability (Tank 5):

Reference: Stability of Steel Oil Storage Tanks, Duncan, D'Orazio, 1984

GEOMETERY

Tank Diameter: $D := 240ft$

Existing Elev: $ELEVex := 29.26ft$

Tank Bottom Elev: $TankB := 31.75ft$

Fill and LTP Thickness: $Tp := TankB - (ELEVex - 4ft)$ $Tp = 6.49 \cdot ft$

Overburden Thickness: $Tob := TankB - Tp - 16ft$ $Tob = 9.26 \cdot ft$

Saturated Clay above the GW will consolidate quickly

Clay Thickness:

CPT	Clay Thickess	Clay and Silt
131	6'	13'
132	9'	16'
133	6.5'	13.1'
134(s)	5'	11'
135	5'	14'

$Tclay := 9ft$ $Tclay\_silt := 16ft$

Stone column replacement ratio: $Ar := 0.11$

SOIL PROPERTIES

Overburden unit weight: $\gamma ob := 96.8pcf$ Post Improvment - GI Sheets REF CPT114B(S)

Tore Vane Strenght of Clay: $TV := 0.8tsf$ Refernce GRI Report Boring B-1,B-2, B-4, B-5, B-6, B-7, B-8, B-10, B-12 and B-14 through B-22.

$TV = 1600 \cdot psf$

Friction angle of stone: $\phi stone := 50°$

Friction angle of silt $\phi silt := 20°$

LTP unit weight: $\gamma p := 130pcf$

SOIL STRENGTH AND STRESSES

Bearing Pressure due to Liquid in Tank: $q := 2882psf$ At full hydrotest load

Effective stress at top of clay: $\sigma ob := (\gamma ob \cdot Tob) + (\gamma p \cdot Tp)$ $\sigma ob = 1740.068 \cdot psf$

Shear Strenght of SC: $Ssc := \sigma ob \cdot \tan(\phi stone)$ $Ssc = 2073.732 \cdot psf$

Cohesive strenght of clay: $Cu := \frac{TV}{2}$ $Cu = 800 \cdot psf$

Improved clay strength: $Su\_clay := Ar \cdot Ssc + (1 - Ar) \cdot Cu$

$Su\_clay = 940.111 \cdot psf$ After equation 10

Improved Clay-Silt Strenght $Su\_clay\_silt := Ar \cdot Ssc + (1 - Ar)(Cu + \sigma ob \cdot \tan(20))$

$Su\_clay\_silt = 4404.713 \cdot psf$ After equation 10

BASE SHEAR STABILITY EVALUATION - AT FULL HYDRO TEST - NO STRENGTH GAIN

$Dc := D + Tp + Tob \qquad Dc = 255.75 \cdot ft$ Equation 1

Bearing pressure at top of clay:

$$Qclay := q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2 + \gamma ob \cdot Tob$$ Equation 2

$$Qclay = 4218.04 \cdot psf$$

FOR CLAY LAYER ONLY

For a clay layer thicker than one sixth of the tank diameter $Nc1 := 6.1$ Equation 4

For a clay layer thinner than one sixth of the tank diamete $Nc2 := 4.1 + \left(\frac{Dc}{3Tclay}\right)$ Equation 5

$$Nc2 = 13.572$$

Place holder for easier coding: $PH := \frac{Dc}{Tclay}$

$Nc := if[(PH < 6), Nc1, Nc2] \qquad Nc = 13.572$

$Qult := Su\_clay \cdot Nc + \gamma ob \cdot Tob \qquad Qult = 13655.757 \cdot psf$

$$FS\_base\_clay := \frac{Su\_clay \cdot Nc}{\left[q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2\right]} \qquad FS\_base\_clay = 3.841$$ Equation 8

FOR COMBINED CLAY-SILT LAYER

For a clay layer thicker than one sixth of the tank diameter $Nc1 := 6.1$ Equation 4

For a clay layer thinner than one sixth of the tank diamete $Nc2 := 4.1 + \left(\frac{Dc}{3Tclay\_silt}\right)$ Equation 5

$$Nc2 = 9.428$$

Place holder for easier coding: $PH := \frac{Dc}{Tclay\_silt}$

$Nc := if[(PH < 6), Nc1, Nc2] \qquad Nc = 9.428$

$Qult := Su\_clay\_silt \cdot Nc + \gamma ob \cdot Tob \qquad Qult = 42424.556 \cdot psf$

$$FS\_base\_clay\_silt := \frac{Su\_clay\_silt \cdot Nc}{\left[q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2\right]} \qquad FS\_base\_clay\_silt = 12.502$$ Equation 8

EDGE SHEAR STABILITY EVALUATION - AT FULL HYDRO TEST - NO STRENGTH GAIN

IMPROVED CLAY

$Qapp := q + \gamma p \cdot Tp \qquad Qapp = 3725.7 \cdot psf$ Equation 11

With of segment that might fail $B := \begin{pmatrix} 10 \\ 20 \\ 40 \\ 60 \\ 80 \\ 100 \\ 120 \end{pmatrix} ft$

$Nc := 5.2 + \frac{B}{D} \qquad Nc = \begin{pmatrix} 5.242 \\ 5.283 \\ 5.367 \\ 5.45 \\ 5.533 \\ 5.617 \\ 5.7 \end{pmatrix}$

$Qall := Su\_clay \cdot Nc + \gamma p \cdot Tp \qquad Qall = \begin{pmatrix} 5771.446 \\ 5810.617 \\ 5888.96 \\ 5967.303 \\ 6045.645 \\ 6123.988 \\ 6202.33 \end{pmatrix} \cdot psf$

$FS\_edge\_clay := \frac{Qall}{Qapp} \qquad FS\_edge\_clay = \begin{pmatrix} 1.549 \\ 1.56 \\ 1.581 \\ 1.602 \\ 1.623 \\ 1.644 \\ 1.665 \end{pmatrix}$

EDGE SHEAR STABILITY EVALUATION - AT FULL HYDRO TEST - NO STRENGTH GAIN

IMPROVED CLAY and SILT

$Qapp := q + \gamma p \cdot Tp$ $\quad Qapp = 3725.7 \cdot psf$ $\quad$ Equation 11

With of segment that might fail $\quad B := \begin{pmatrix} 10 \\ 20 \\ 40 \\ 60 \\ 80 \\ 100 \\ 120 \end{pmatrix} ft$ $\quad Nc := 5.2 + \frac{B}{D}$ $\quad Nc = \begin{pmatrix} 5.242 \\ 5.283 \\ 5.367 \\ 5.45 \\ 5.533 \\ 5.617 \\ 5.7 \end{pmatrix}$

$Qall := Su\_clay\_silt \cdot Nc + \gamma p \cdot Tp \quad Qall = \begin{pmatrix} 23931.739 \\ 24115.269 \\ 24482.328 \\ 24849.388 \\ 25216.447 \\ 25583.507 \\ 25950.566 \end{pmatrix} \cdot psf$

$FS\_edge\_clay\_silt := \frac{Qall}{Qapp}$ $\quad FS\_edge\_clay\_silt = \begin{pmatrix} 6.423 \\ 6.473 \\ 6.571 \\ 6.67 \\ 6.768 \\ 6.867 \\ 6.965 \end{pmatrix}$

SUMMARY OF BASE AND EDGE SHEAR STABILITY EVALUATION - AT FULL HYDRO TEST - NO STRENGTH GAIN

$FS\_base\_clay = 3.841$

$FS\_base\_clay\_silt = 12.502$

$FS\_edge\_clay = \begin{pmatrix} 1.549 \\ 1.56 \\ 1.581 \\ 1.602 \\ 1.623 \\ 1.644 \\ 1.665 \end{pmatrix}$ $\quad FS\_edge\_clay\_silt = \begin{pmatrix} 6.423 \\ 6.473 \\ 6.571 \\ 6.67 \\ 6.768 \\ 6.867 \\ 6.965 \end{pmatrix}$

BASE SHEAR STABILITY EVALUATION - IN SERVICE - NO STRENGTH GAIN

$q := 2500psf$

$Dc := D + Tp + Tob \qquad Dc = 255.75 \cdot ft$ Equation 1

Bearing pressure at top of clay:

$$Qclay := q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2 + \gamma ob \cdot Tob$$ Equation 2

$$Qclay = 3881.641 \cdot psf$$

FOR CLAY LAYER ONLY

For a clay layer thicker than one sixth of the tank diameter $Nc1 := 6.1$ Equation 4

For a clay layer thinner than one sixth of the tank diamete $Nc2 := 4.1 + \left(\frac{Dc}{3Tclay}\right)$ Equation 5

$$Nc2 = 13.572$$

Place holder for easier coding: $PH := \frac{Dc}{Tclay}$

$Nc := if[(PH < 6), Nc1, Nc2] \qquad Nc = 13.572$

$Qult := Su\_clay \cdot Nc + \gamma ob \cdot Tob \qquad Qult = 13655.757 \cdot psf$

$$FS\_base\_clay := \frac{Su\_clay \cdot Nc}{\left[q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2\right]} \qquad FS\_base\_clay = 4.274$$ Equation 8

FOR COMBINED CLAY-SILT LAYER

For a clay layer thicker than one sixth of the tank diameter $Nc1 := 6.1$ Equation 4

For a clay layer thinner than one sixth of the tank diamete $Nc2 := 4.1 + \left(\frac{Dc}{3Tclay\_silt}\right)$ Equation 5

$$Nc2 = 9.428$$

Place holder for easier coding: $PH := \frac{Dc}{Tclay\_silt}$

$Nc := if[(PH < 6), Nc1, Nc2] \qquad Nc = 9.428$

$Qult := Su\_clay\_silt \cdot Nc + \gamma ob \cdot Tob \qquad Qult = 42424.556 \cdot psf$

$$FS\_base\_clay\_silt := \frac{Su\_clay\_silt \cdot Nc}{\left[q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2\right]} \qquad FS\_base\_clay\_silt = 13.911$$ Equation 8

EDGE SHEAR STABILITY EVALUATION - IN SERVICE - NO STRENGTH GAIN

IMPROVED CLAY

$Qapp := q + \gamma p \cdot Tp$ $\quad Qapp = 3343.7 \cdot psf$ Equation 11

With of segment that might fail $\quad B := \begin{pmatrix} 10 \\ 20 \\ 40 \\ 60 \\ 80 \\ 100 \\ 120 \end{pmatrix} ft$

$Nc := 5.2 + \frac{B}{D}$ $\quad Nc = \begin{pmatrix} 5.242 \\ 5.283 \\ 5.367 \\ 5.45 \\ 5.533 \\ 5.617 \\ 5.7 \end{pmatrix}$

$Qall := Su\_clay \cdot Nc + \gamma p \cdot Tp$ $\quad Qall = \begin{pmatrix} 5771.446 \\ 5810.617 \\ 5888.96 \\ 5967.303 \\ 6045.645 \\ 6123.988 \\ 6202.33 \end{pmatrix} \cdot psf$

$FS\_edge\_clay := \frac{Qall}{Qapp}$ $\quad FS\_edge\_clay = \begin{pmatrix} 1.726 \\ 1.738 \\ 1.761 \\ 1.785 \\ 1.808 \\ 1.832 \\ 1.855 \end{pmatrix}$

EDGE SHEAR STABILITY EVALUATION - IN SERVICE - NO STRENGTH GAIN

IMPROVED CLAY and SILT

$Qapp := q + \gamma p \cdot Tp$ $\quad Qapp = 3343.7 \cdot psf$ $\quad$ Equation 11

With of segment that might fail $\quad B := \begin{pmatrix} 10 \\ 20 \\ 40 \\ 60 \\ 80 \\ 100 \\ 120 \end{pmatrix} ft$ $\quad Nc := 5.2 + \frac{B}{D}$ $\quad Nc = \begin{pmatrix} 5.242 \\ 5.283 \\ 5.367 \\ 5.45 \\ 5.533 \\ 5.617 \\ 5.7 \end{pmatrix}$

$Qall := Su\_clay\_silt \cdot Nc + \gamma p \cdot Tp \quad Qall = \begin{pmatrix} 23931.739 \\ 24115.269 \\ 24482.328 \\ 24849.388 \\ 25216.447 \\ 25583.507 \\ 25950.566 \end{pmatrix} \cdot psf$

$FS\_edge\_clay\_silt := \frac{Qall}{Qapp} \quad FS\_edge\_clay\_silt = \begin{pmatrix} 7.157 \\ 7.212 \\ 7.322 \\ 7.432 \\ 7.541 \\ 7.651 \\ 7.761 \end{pmatrix}$

SUMMARY OF BASE AND EDGE SHEAR STABILITY EVALUATION - IN SERVCIE - NO STRENGTH GAIN

$FS\_base\_clay = 4.274$

$FS\_base\_clay\_silt = 13.911$

$FS\_edge\_clay = \begin{pmatrix} 1.726 \\ 1.738 \\ 1.761 \\ 1.785 \\ 1.808 \\ 1.832 \\ 1.855 \end{pmatrix} \quad FS\_edge\_clay\_silt = \begin{pmatrix} 7.157 \\ 7.212 \\ 7.322 \\ 7.432 \\ 7.541 \\ 7.651 \\ 7.761 \end{pmatrix}$

Units

$kips \equiv 1000 \cdot lbf$

$ksf := 1000 \cdot psf$

$tsf := 2 \cdot ksf$

$kPa := 1000 \cdot Pa$

$pi := 3.14159$

$ksi \equiv \frac{1000 \cdot lbf}{in^2}$

$pcf \equiv \frac{lbf}{ft^3}$

$in^2 = 0.001\, m^2$

$psi \equiv \frac{lbf}{in^2}$

$psf \equiv \frac{lbf}{ft^2}$

$pcf \equiv \frac{lbf}{ft^3}$

$kcf \equiv \frac{k}{ft^3}$

$k \equiv kips$

$N \equiv newton$

$ORIGIN \equiv 1$

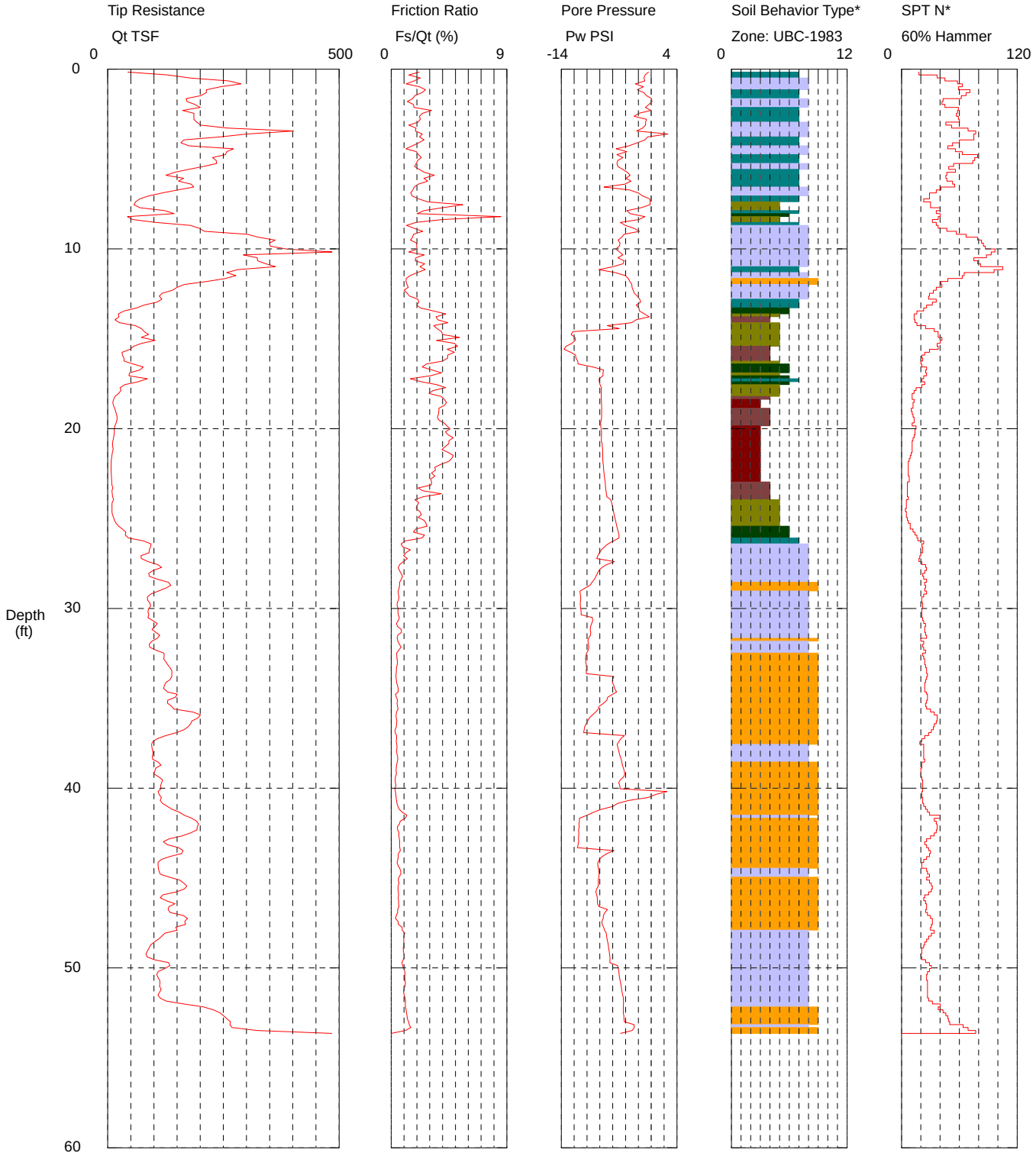
$MPa := 10^6 \cdot Pa$

$kN \equiv 1000 newton$

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-131
Cone Used: DDG1263

CPT Date/Time: 11/12/2014 2:42:41 PM
Location: Port of Vancouver
Job Number: 860058



Maximum Depth = 53.64 feet

Depth Increment = 0.164 feet

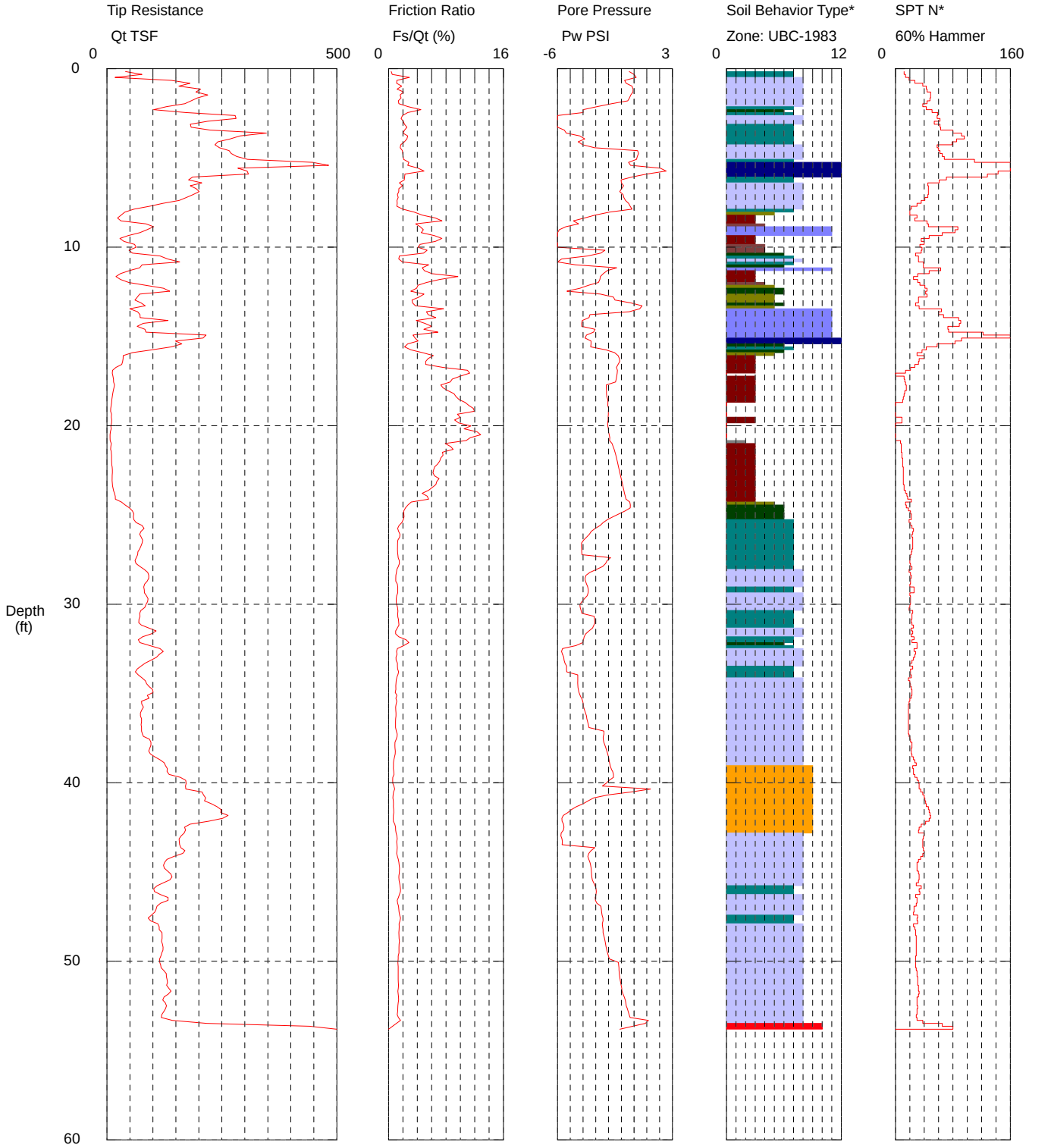
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

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

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-132
Cone Used: DDG1263

CPT Date/Time: 11/12/2014 3:27:24 PM
Location: Port of Vancouver
Job Number: 860058



Maximum Depth = 53.81 feet

Depth Increment = 0.164 feet

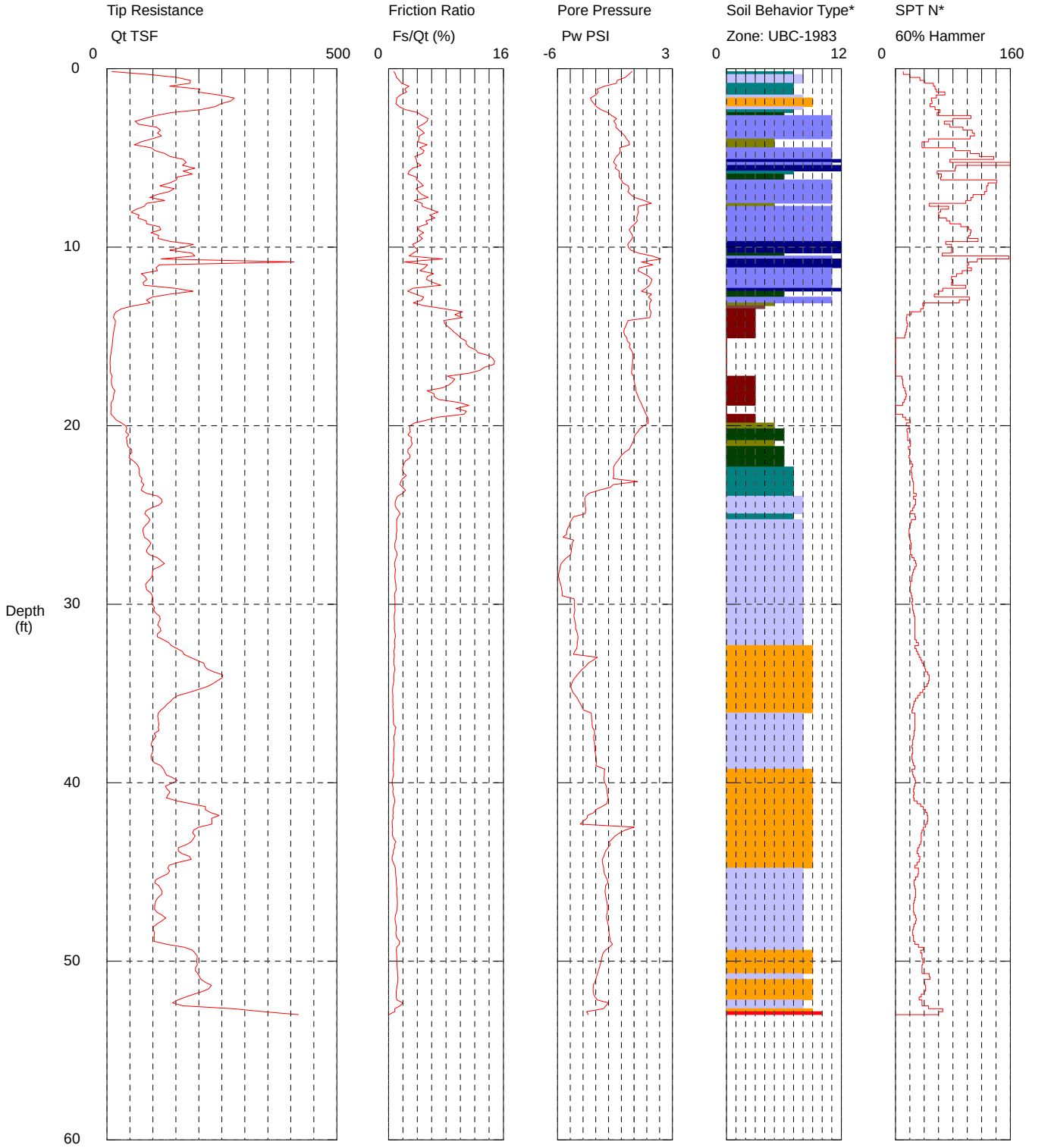
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

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

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-133
Cone Used: DDG1263

CPT Date/Time: 11/12/2014 4:12:29 PM
Location: Port of Vancouver
Job Number: 860058



Maximum Depth = 52.99 feet

Depth Increment = 0.164 feet

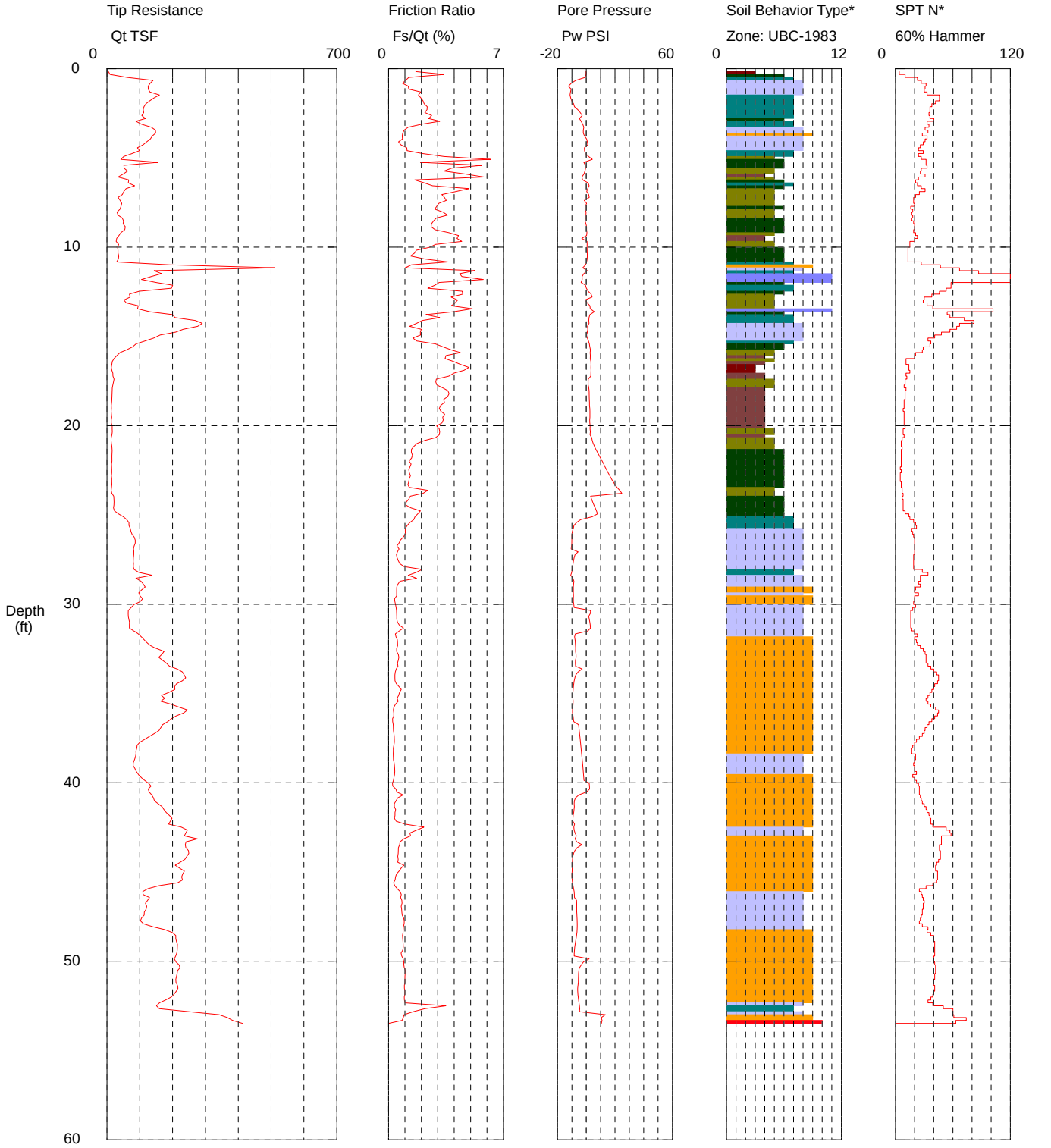
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

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

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-134(s)
Cone Used: DDG1263

CPT Date/Time: 11/11/2014 7:48:33 AM
Location: Port of Vancouver
Job Number: 860058



Maximum Depth = 53.48 feet

Depth Increment = 0.164 feet

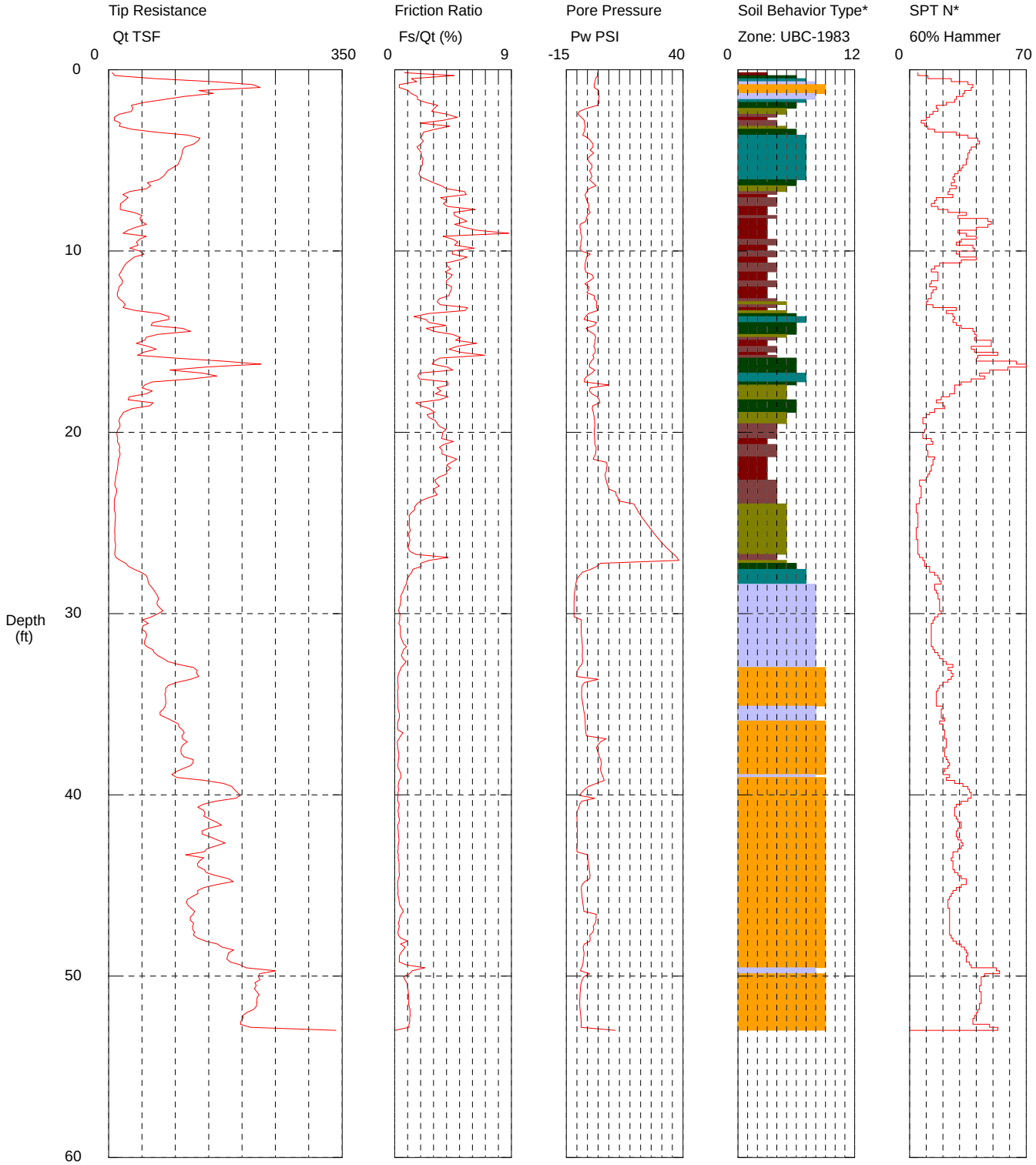
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

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

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-135
Cone Used: DDG1263

CPT Date/Time: 11/12/2014 1:57:34 PM
Location: Port of Vancouver
Job Number: 860058



Maximum Depth = 52.99 feet

Depth Increment = 0.164 feet

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

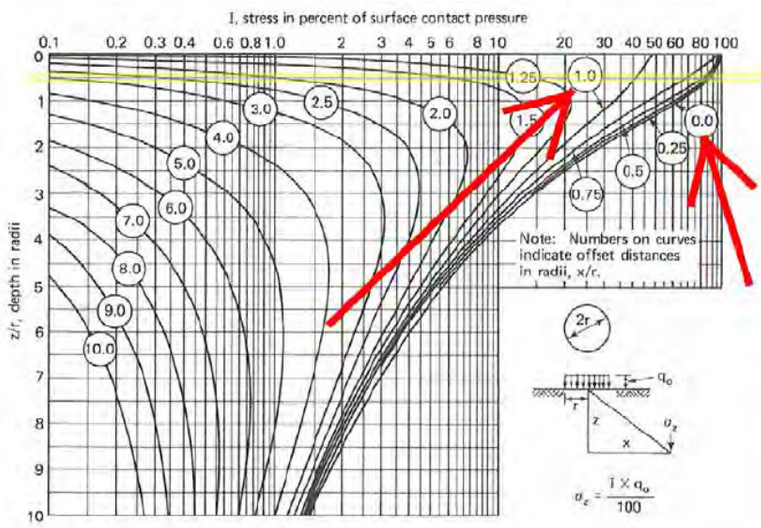
*Soil behavior type and SPT based on data from UBC-1983

Tank 6

Design Summary
Tank 6

This table assumes the exploration is directly below the center of the tank.

Tank 6		Orig	Imp.	Improved		Original	
Soil Profile	Depth	Liq Sett.	Liq Sett.	Rapid Sett.	Long Term Sett.	Rapid Sett.	Long Term Sett.
HB-CPT-161	54.3-ft	5.14-in.	2.1-in.	2.87-in.	.14-in.	2.14-in.	4.25-in.
HB-CPT-165	54.8-ft	4.52-in.	2.43-in.	2.52-in.	.17-in.	1.71-in.	3.58-in.
HB-CPT-164	53.5-ft	4.55-in.	2.33-in.	1.54-in.	.15-in.	1.55-in.	2.29-in.
HB-CPT-166	55.5-ft	2.58-in.	.76-in.	3.96-in.	.03-in.	.9-in.	5.31-in.
HB-CPT-162	54.6-ft	4.09-in.	1.94-in.	3.18-in.	.05-in.	.78-in.	4.22-in.
HB-CPT-163(s)	53.8-ft	3.75-in.	1.95-in.	1.29-in.	.18-in.	.78-in.	1.6-in.
Average:		**4.1-in.**	**1.92-in.**	**2.82-in.**	**.11-in.**	**1.41-in.**	**3.93-in.**



This table calculates Settlement at the edge of the tank.

Tank 6		Orig	Imp.	Improved		Original	
Soil Profile	Depth	Liq Sett.	Liq Sett.	Rapid Sett.	Long Term Sett.	Rapid Sett.	Long Term Sett.
HB-CPT-161	54.3-ft	5.14-in.	2.1-in.	1.43-in.	.07-in.	1.07-in.	2.13-in.
HB-CPT-165	54.8-ft	4.52-in.	2.43-in.	1.26-in.	.08-in.	.85-in.	1.79-in.
HB-CPT-164	53.5-ft	4.55-in.	2.33-in.	.77-in.	.08-in.	.77-in.	1.14-in.
HB-CPT-166	55.5-ft	2.58-in.	.76-in.	1.98-in.	.02-in.	.45-in.	2.65-in.
HB-CPT-162	54.6-ft	4.09-in.	1.94-in.	1.59-in.	.03-in.	.39-in.	2.11-in.
HB-CPT-163(s)	53.8-ft	3.75-in.	1.95-in.	.65-in.	.09-in.	.39-in.	.8-in.
Average:		4.18-in.	1.91-in.	1.41-in.	.05-in.	.71-in.	1.97-in.

Note: No reduction to total soil settlement is taken for the liquefaction condition.

4.1-in.	1.92-in.	1.28-in.	.06-in.	.65-in.	1.77-in.

Hayward Baker Inc. THINK SAFE	SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 6			
	Job no.	860058	Design by:	M. Rohrbach	Date: 14-Apr-15
	File	CPT-Liquefaction-Boulanger	Checked		
	Spreadsheet Rev.	23-Dec-14	Reviewed		

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-161

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	17.3-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	29.26	ft.
Elevation of bottom of Soil Profile	-25.04	ft.
Elevation of Installation	24	
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	54.3-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	323.70-psf	

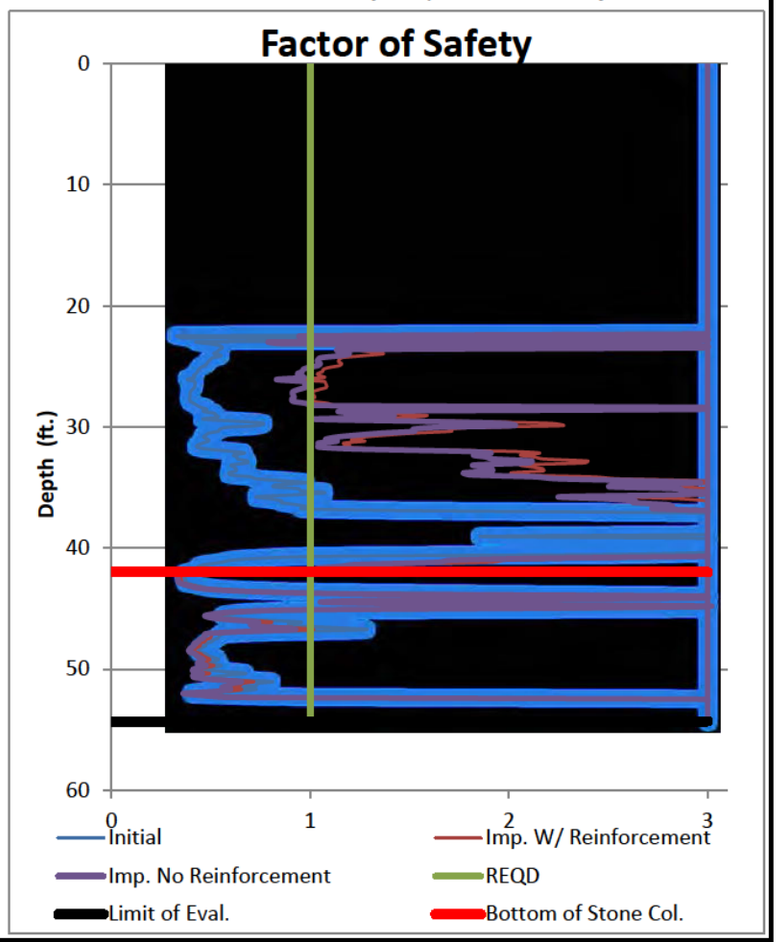
Stone Column Design Information

L =	42-ft		ϕ_c = 50
Column Spacing	8-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 54.3-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 23.8-ft.

Calculated Min: 5.08-in via T&S, Min never < 0.5 in.

Calculated Max: 5.14-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 12.6-ft.

Calculated Min: 2.90-in via T&S, Min never < 0.5 in.

Calculated Max: 3.19-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 9.4-ft.

Calculated Min: 2.10-in via T&S, Min never < 0.5 in.

Calculated Max: 2.53-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 6 — IMPROVED CASE: CPT-161

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e = C_1 C_2 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment

= $1 - 0.5*(q/(q_{bar}-q))$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

$q = \gamma Df$

Stone Column Length 42-ft

Reduction factor on Priebe: 1

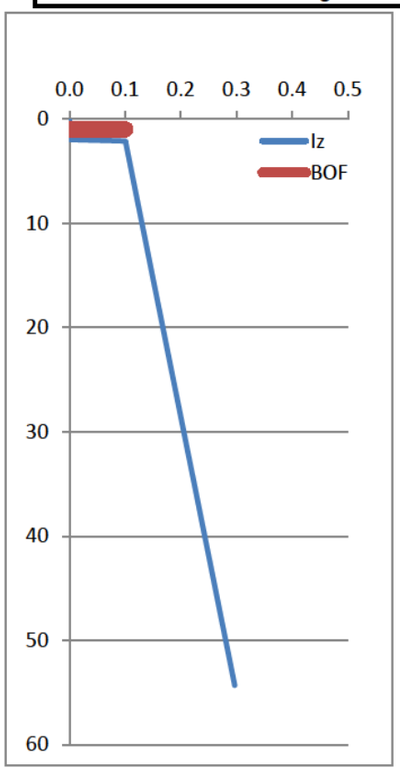
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	Per R&M Calcs.
L	212.7-ft	Equivalent Area for 240' Ø tank
B	212.7-ft	Reference GRI page 2 and B=L
fill density	130-pcf	Assigned by Designer
Elevation of Top of Soil Profile	29.26	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es = X \cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)

I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=0.5B$=	106.3
I_z = 0	at depth	$z=z_2=2.0B$=	425.4

For foundations with L/B ≥ 10

I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=1.0B$=	212.7
I_z = 0	at depth	$z=z_2=4.0B$=	850.8

Interpolation for L/B values between 1 and 10

I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

Σ(Iz/Es)Δz (ft2/lb)= 1.49E-05

Total Elastic Settlement (Flexible foundation)

$S_e = C_1 C_2 (q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0571-ft 0.69-in

Total Rapid Elastic Settlement

$S_e = C_1 (q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0371-ft 0.45-in

Long Term Settlement 0.24-in

Calculated Consolidation Type Settlement:

Total:	2.32-in	Adjusted for footing Type
Recomp.	0.10-in	Adjusted for footing Type
Virgin Sett.	2.23-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	2.42-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:

Total: 3.01-in

Rapid: 2.87-in

Long Term: 0.14-in

Hayward Baker Inc.

THINK SAFE

SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 6		
Job no.	860058	Design by: M. Rohrbach	Date: 14-Apr-15
File	CPT-Liquefaction-Boulanger	Checked	
Spreadsheet Rev.	23-Dec-14	Reviewed	

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-162

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	15.9-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	27.91	ft.
Elevation of bottom of Soil Profile	-26.72	ft.
Elevation of Installation	24	
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	54.6-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	499.20-psf	

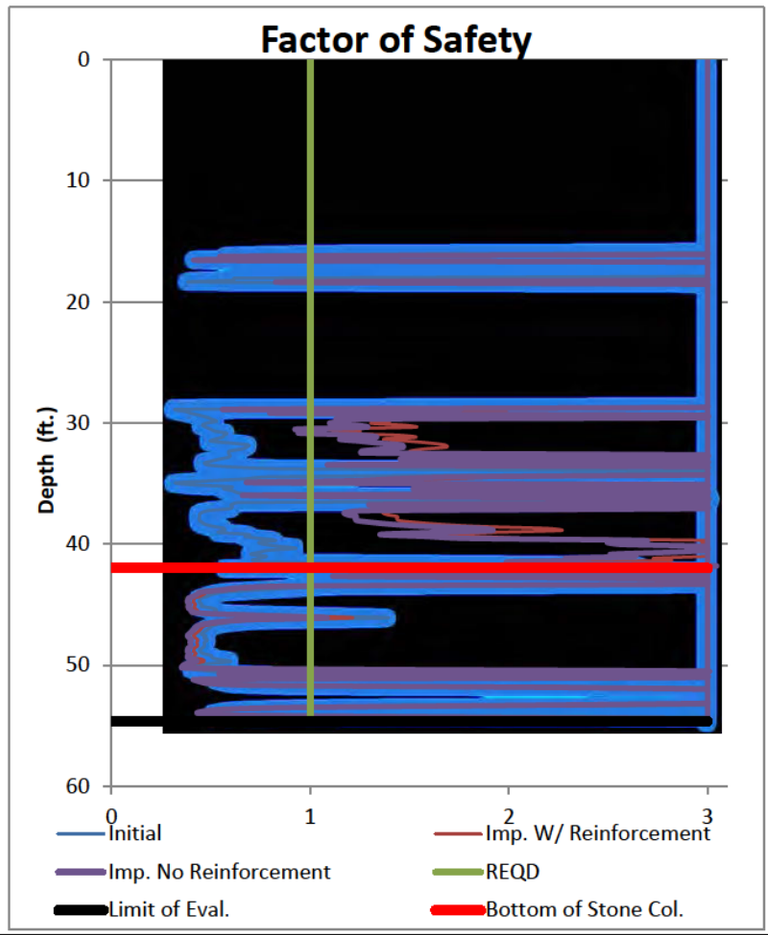
Stone Column Design Information

L =	42-ft		ϕ_c = 50
Column Spacing	8-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 54.6-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 21.5-ft.

Calculated Min: 4.07-in via T&S, Min never < 0.5 in.

Calculated Max: 4.09-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 10.0-ft.

Calculated Min: 2.05-in via T&S, Min never < 0.5 in.

Calculated Max: 2.37-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 9.5-ft.

Calculated Min: 1.94-in via T&S, Min never < 0.5 in.

Calculated Max: 2.01-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 6 — IMPROVED CASE: CPT-162

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e = C_1 C_2 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment

= $1 - 0.5*(q/(q_{bar}-q))$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

$q = \gamma Df$

Stone Column Length 42-ft

Reduction factor on Priebe: 1

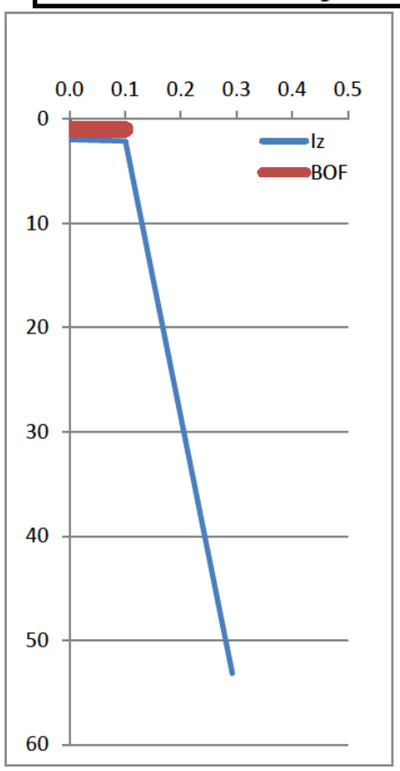
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	Per R&M Calcs.
L	212.7-ft	Equivalent Area for 240' Ø tank
B	212.7-ft	Reference GRI page 2 and B=L
fill density	130-pcf	Assigned by Designer
Elevation of Top of Soil Profile	27.91	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es = X \cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=$**0.5**B=	106.3
I_z = 0	at depth	$z=z_2=$**2.0**B=	425.4
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=$**1.0**B=	212.7
I_z = 0	at depth	$z=z_2=$**4.0**B=	850.8
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

Σ(Iz/Es)Δz (ft2/lb)= 1.02E-05

Total Elastic Settlement (Flexible foundation)

$S_e = C_1 C_2 (q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0392-ft 0.47-in

Total Rapid Elastic Settlement

$S_e = C_1 (q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0254-ft 0.31-in

Long Term Settlement 0.16-in

Calculated Consolidation Type Settlement:

Total:	2.77-in	Adjusted for footing Type
Recomp.	0.11-in	Adjusted for footing Type
Virgin Sett.	2.66-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	2.87-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:

Total: 3.24-in

Rapid: 3.18-in

Long Term: 0.05-in

Hayward Baker Inc.

THINK SAFE

SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 6		
Job no.	860058	Design by: M. Rohrbach	Date: 14-Apr-15
File	CPT-Liquefaction-Boulanger	Checked	
Spreadsheet Rev.	23-Dec-14	Reviewed	

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-163(s)

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	15.8-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	27.81	ft.
Elevation of bottom of Soil Profile	-26.00	ft.
Elevation of Installation	24	
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	53.8-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	512.20-psf	

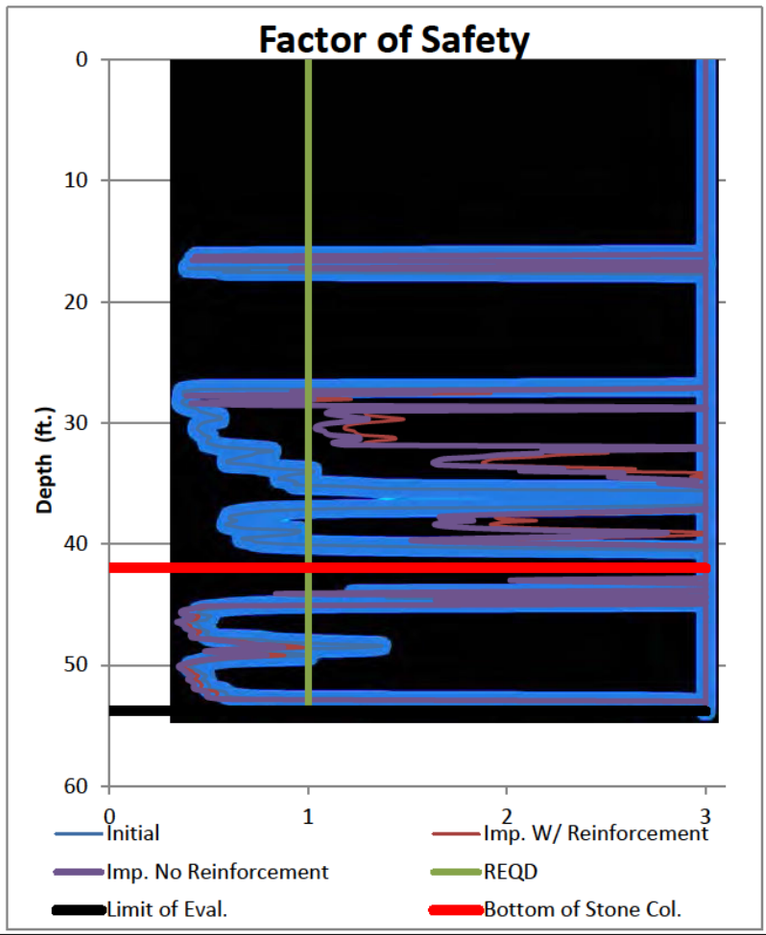
Stone Column Design Information

L = 42-ft			ϕ_c = 50
Column Spacing 8-ft			Ec = 750.0-tsf
Kac = 0.1324743	$=TAN^2(45-(\phi_c/2)$		Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 53.8-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 18.9-ft.

Calculated Min: 3.65-in via T&S, Min never < 0.5 in.

Calculated Max: 3.75-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 9.7-ft.

Calculated Min: 2.03-in via T&S, Min never < 0.5 in.

Calculated Max: 2.25-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 9.4-ft.

Calculated Min: 1.95-in via T&S, Min never < 0.5 in.

Calculated Max: 2.15-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 6 — IMPROVED CASE: CPT-163(s)

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e = C_1 C_2 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment

= $1 - 0.5*(q/(q_{bar}-q))$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

$q = \gamma Df$

Stone Column Length 42-ft

Reduction factor on Priebe: 1

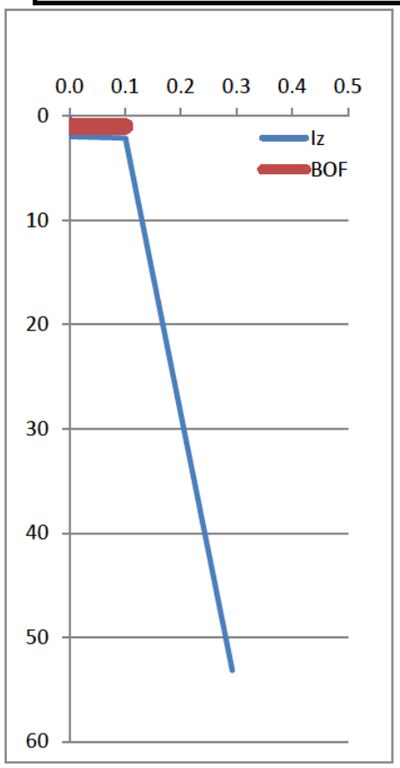
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	Per R&M Calcs.
L	212.7-ft	Equivalent Area for 240' Ø tank
B	212.7-ft	Reference GRI page 2 and B=L
fill density	130-pcf	Assigned by Designer
Elevation of Top of Soil Profile	27.81	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es = X \cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=$**0.5**B=	106.3
I_z = 0	at depth	$z=z_2=$**2.0**B=	425.4
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=$**1.0**B=	212.7
I_z = 0	at depth	$z=z_2=$**4.0**B=	850.8
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

Σ(Iz/Es)Δz (ft2/lb)= 1.28E-05

Total Elastic Settlement (Flexible foundation)

$S_e = C_1 C_2 (q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0489-ft 0.59-in

Total Rapid Elastic Settlement

$S_e = C_1 (q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0318-ft 0.38-in

Long Term Settlement 0.21-in

Calculated Consolidation Type Settlement:

Total:	0.88-in	Adjusted for footing Type
Recomp.	0.03-in	Adjusted for footing Type
Virgin Sett.	0.84-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	0.91-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:

Total: 1.47-in

Rapid: 1.29-in

Long Term: 0.18-in

Hayward Baker Inc.

THINK SAFE

SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 6		
Job no.	860058	Design by: M. Rohrbach	Date: 14-Apr-15
File	CPT-Liquefaction-Boulanger	Checked	
Spreadsheet Rev.	23-Dec-14	Reviewed	

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-164B

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	16.0-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	28	ft.
Elevation of bottom of Soil Profile	-25.48	ft.
Elevation of Installation	24	
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	53.5-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, ν_s	0.333	
Poisson's Ratio for Stone Column, ν_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	487.50-psf	

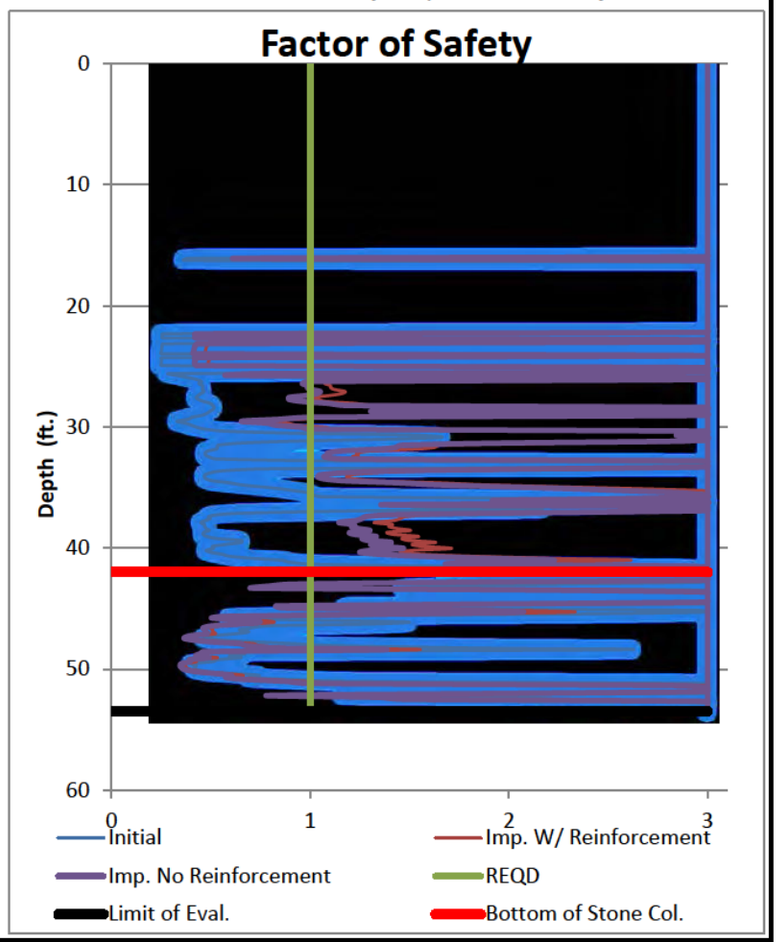
Stone Column Design Information

L =	42-ft		ϕ_c = 50
Column Spacing	8-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 53.5-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 20.3-ft.

Calculated Min: 4.45-in via T&S, Min never < 0.5 in.

Calculated Max: 4.55-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 11.6-ft.

Calculated Min: 2.70-in via T&S, Min never < 0.5 in.

Calculated Max: 3.09-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 9.8-ft.

Calculated Min: 2.33-in via T&S, Min never < 0.5 in.

Calculated Max: 2.55-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 6 — IMPROVED CASE: CPT-164B

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment

= $1-0.5*(q/(q_{bar}-q)$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

$q = \gamma Df$

Stone Column Length 42-ft

Reduction factor on Priebe: 1

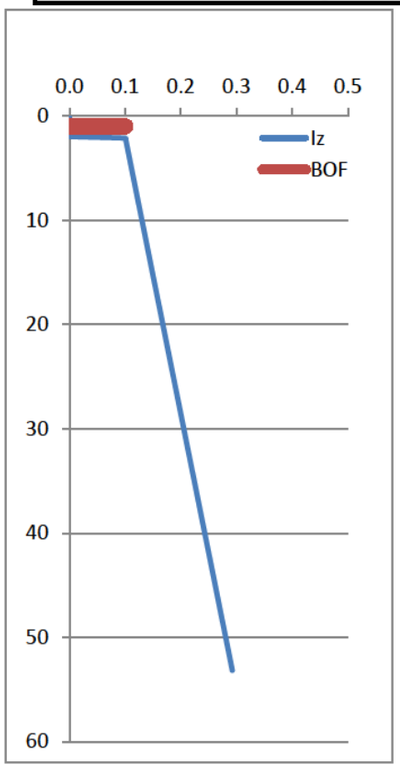
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	Per R&M Calcs.
L	212.7-ft	Equivalent Area for 240' Ø tank
B	212.7-ft	Reference GRI page 2 and B=L
fill density	130-pcf	Assigned by Designer
Elevation of Top of Soil Profile	28	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es=X \cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=0.5B$=	106.3
I_z = 0	at depth	$z=z_2=2.0B$=	425.4
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=1.0B$=	212.7
I_z = 0	at depth	$z=z_2=4.0B$=	850.8
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

$\Sigma(Iz/Es)\Delta z$ (ft2/lb)= 1.26E-05

Total Elastic Settlement (Flexible foundation)

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0484-ft 0.58-in

Total Rapid Elastic Settlement

$S_e=C_1(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0314-ft 0.38-in

Long Term Settlement 0.20-in

Calculated Consolidation Type Settlement:

Total:	1.11-in	Adjusted for footing Type
Recomp.	0.05-in	Adjusted for footing Type
Virgin Sett.	1.07-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	1.16-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:

Total: 1.69-in

Rapid: 1.54-in

Long Term: 0.15-in

Hayward Baker Inc.

THINK SAFE

SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 6		
Job no.	860058	Design by: M. Rohrbach	Date: 14-Apr-15
File	CPT-Liquefaction-Boulanger	Checked	
Spreadsheet Rev.	23-Dec-14	Reviewed	

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-165(A)

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	16.7-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	28.71	ft.
Elevation of bottom of Soil Profile	-26.08	ft.
Elevation of Installation	24	
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	54.8-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	395.20-psf	

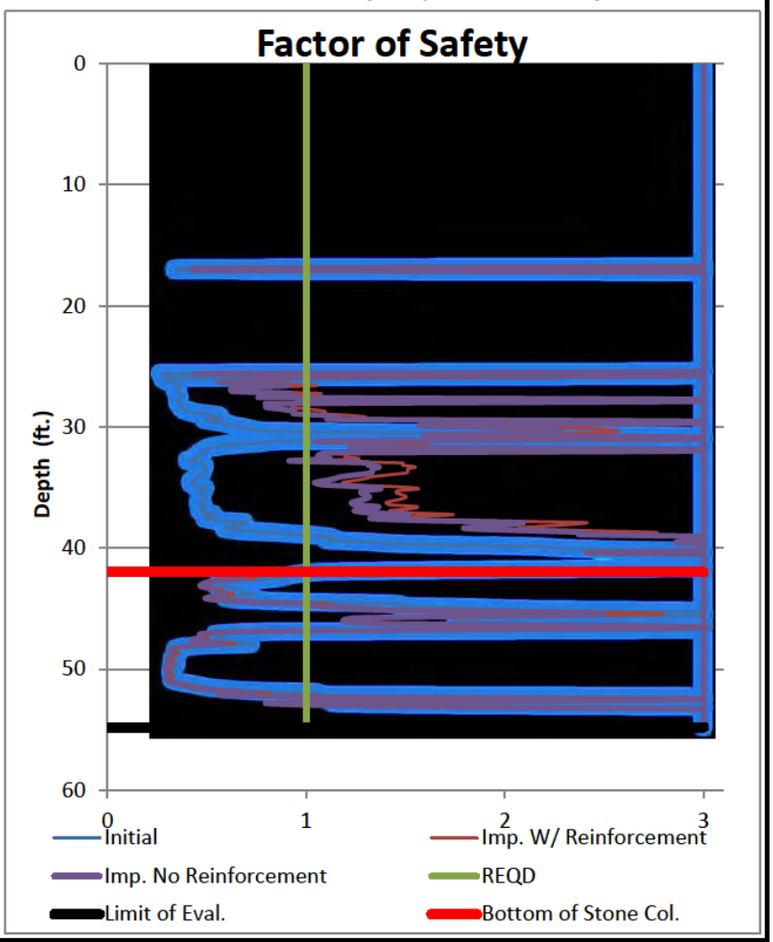
Stone Column Design Information

L = 42-ft			ϕ_c = 50
Column Spacing 8-ft			Ec = 750.0-tsf
Kac = 0.1324743	$=TAN^2(45-(\phi_c/2)$		Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 54.8-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 20.5-ft.

Calculated Min: 4.44-in via T&S, Min never < 0.5 in.

Calculated Max: 4.52-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 11.5-ft.

Calculated Min: 2.66-in via T&S, Min never < 0.5 in.

Calculated Max: 3.09-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 10.5-ft.

Calculated Min: 2.43-in via T&S, Min never < 0.5 in.

Calculated Max: 2.64-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 6 — IMPROVED CASE: CPT-165(A)

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e = C_1 C_2 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment

= $1 - 0.5*(q/(q_{bar}-q))$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

$q = \gamma Df$

Stone Column Length 42-ft

Reduction factor on Priebe: 1

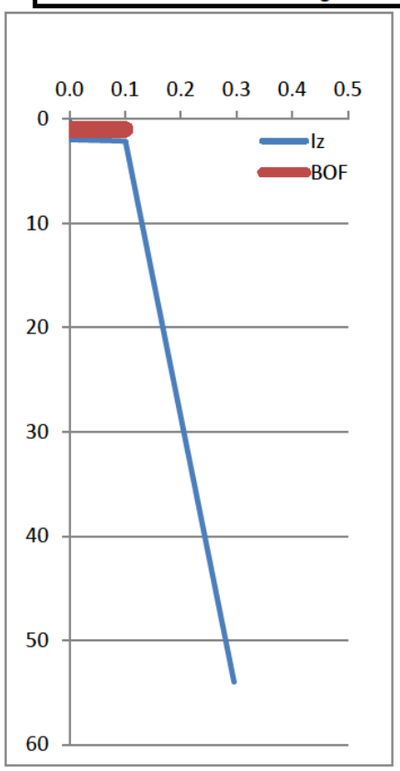
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	Per R&M Calcs.
L	212.7-ft	Equivalent Area for 240' Ø tank
B	212.7-ft	Reference GRI page 2 and B=L
fill density	130-pcf	Assigned by Designer
Elevation of Top of Soil Profile	28.71	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es = X \cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)

I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=$**0.5**B=	106.3
I_z = 0	at depth	$z=z_2=$**2.0**B=	425.4

For foundations with L/B ≥ 10

I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=$**1.0**B=	212.7
I_z = 0	at depth	$z=z_2=$**4.0**B=	850.8

Interpolation for L/B values between 1 and 10

I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

Σ(Iz/Es)Δz (ft2/lb)= 1.51E-05

Total Elastic Settlement (Flexible foundation)

$S_e = C_1 C_2 (q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0579-ft 0.70-in

Total Rapid Elastic Settlement

$S_e = C_1 (q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0376-ft 0.45-in

Long Term Settlement 0.25-in

Calculated Consolidation Type Settlement:

Total:	1.99-in	Adjusted for footing Type
Recomp.	0.08-in	Adjusted for footing Type
Virgin Sett.	1.91-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	2.07-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:

Total: 2.69-in

Rapid: 2.52-in

Long Term: 0.17-in

Hayward Baker Inc.

THINK SAFE

SUBJEC	Tesoro Savage Petroleum Terminal LLC Tank 6		
Job no.	860058	Design by: M. Rohrbach	Date: 14-Apr-15
File	CPT-Liquefaction-Boulanger	Checked	
Spreadsheet Rev.	23-Dec-14	Reviewed	

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-166A

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	15.4-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	27.44	ft.
Elevation of bottom of Soil Profile	-28.01	ft.
Elevation of Installation	24	
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	55.5-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	560.30-psf	

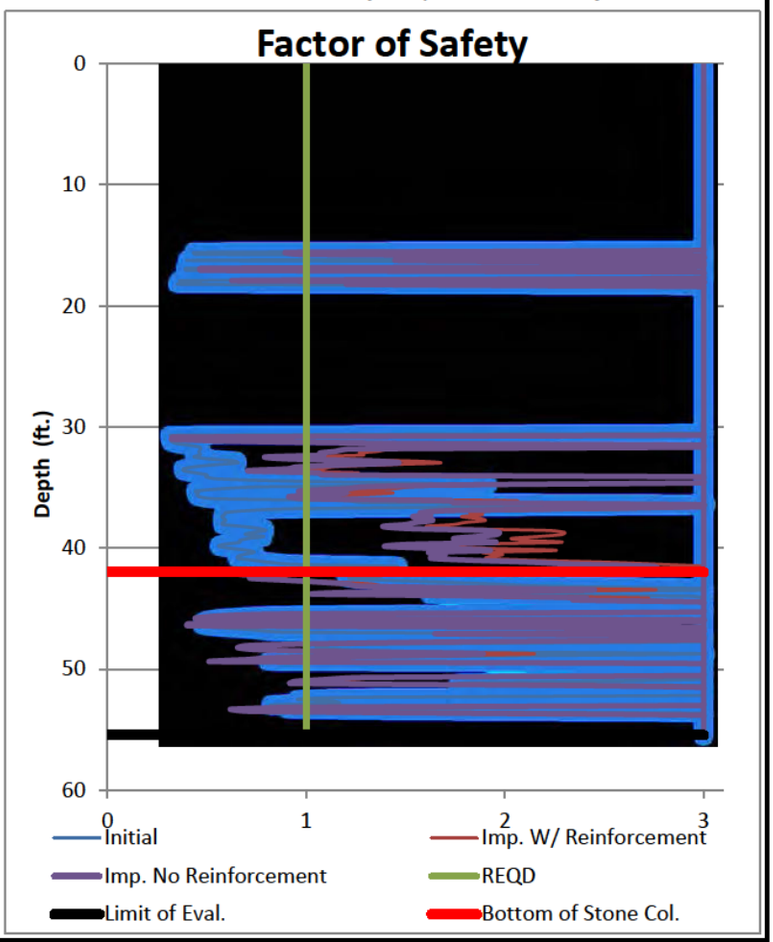
Stone Column Design Information

L =	42-ft		ϕ_c = 50
Column Spacing	8-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 11% Replacment		SILTY soils: 11% Replacment	
SAND: Ar = (Ac/A)	0.11	SILTY SOIL: Ar = (Ac/A)	0.11
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 55.5-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 13.3-ft.

Calculated Min: 2.45-in via T&S, Min never < 0.5 in.

Calculated Max: 2.58-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 6.0-ft.

Calculated Min: 1.07-in via T&S, Min never < 0.5 in.

Calculated Max: 1.34-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 4.1-ft.

Calculated Min: 0.76-in via T&S, Min never < 0.5 in.

Calculated Max: 0.95-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Tank 6 — IMPROVED CASE: CPT-166A

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e = C_1 C_2 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment

= $1 - 0.5*(q/(q_{bar}-q))$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

$q = \gamma Df$

Stone Column Length 42-ft

Reduction factor on Priebe: 1

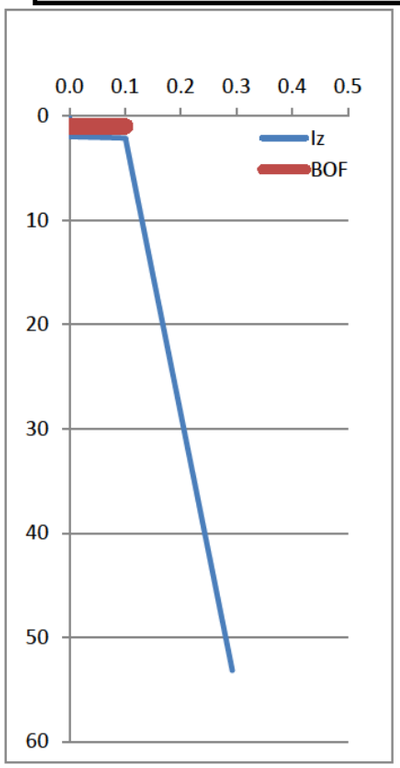
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	2882.00-psf	Per R&M Calcs.
L	212.7-ft	Equivalent Area for 240' Ø tank
B	212.7-ft	Reference GRI page 2 and B=L
fill density	130-pcf	Assigned by Designer
Elevation of Top of Soil Profile	27.44	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es = X \cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.950	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=$**0.5**B=	106.3
I_z = 0	at depth	$z=z_2=$**2.0**B=	425.4
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=$**1.0**B=	212.7
I_z = 0	at depth	$z=z_2=$**4.0**B=	850.8
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	106.3
I_z = 0	2.00	at $z=z_2$=	425.4

Σ(Iz/Es)Δz (ft2/lb)= 1.07E-05

Total Elastic Settlement (Flexible foundation)

$S_e = C_1 C_2 (q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0412-ft 0.49-in

Total Rapid Elastic Settlement

$S_e = C_1 (q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0268-ft 0.32-in

Long Term Settlement 0.17-in

Calculated Consolidation Type Settlement:

Total:	3.50-in	Adjusted for footing Type
Recomp.	0.14-in	Adjusted for footing Type
Virgin Sett.	3.36-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	3.64-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:

Total: 3.99-in

Rapid: 3.96-in

Long Term: 0.03-in

Mathcad – Tank 6
Bearing Checks

Calculations for Tank Edge and Base Stability (Tank 6):

Reference: Stability of Steel Oil Storage Tanks, Duncan, D'Orazio, 1984

GEOMETERY

Tank Diameter: $D := 240ft$

Existing Elev: $ELEVex := 29.26ft$

Tank Bottom Elev: $TankB := 31.75ft$

Fill and LTP Thickness: $Tp := TankB - (ELEVex - 4ft)$ $\quad Tp = 6.49 \cdot ft$

Overburden Thickness: $Tob := TankB - Tp - 16ft$ $\quad Tob = 9.26 \cdot ft$

Saturated Clay above the GW will consolidate quickly

Clay Thickness:

CPT	Clay Thickess	Clay and Silt
161	6'	10'
162	9'	14'
163	1'	13'
164	0.5'	10'
165	5'	11'
166	11'	17'

$Tclay := 11ft$ $\quad Tclay\_silt := 17ft$

Stone column replacement ratio: $Ar := 0.11$

SOIL PROPERTIES

Overburden unit weight: $\gamma ob := 96.8pcf$ Post Improvment - GI Sheets REF CPT114B(S)

Tore Vane Strenght of Clay: $TV := 0.8tsf$ Refernce GRI Report Boring B-1,B-2, B-4, B-5, B-6, B-7, B-8, B-10, B-12 and B-14 through B-22.

$TV = 1600 \cdot psf$

Friction angle of stone: $\phi stone := 50°$

Friction angle of silt $\phi silt := 20°$

LTP unit weight: $\gamma p := 130pcf$

SOIL STRENGTH AND STRESSES

Bearing Pressure due to Liquid in Tank: $q := 2882psf$ At full hydrotest load

Effective stress at top of clay: $\sigma ob := (\gamma ob \cdot Tob) + (\gamma p \cdot Tp)$ $\quad \sigma ob = 1740.068 \cdot psf$

Shear Strenght of SC: $Ssc := \sigma ob \cdot \tan(\phi stone)$ $\quad Ssc = 2073.732 \cdot psf$

Cohesive strenght of clay: $Cu := \frac{TV}{2}$ $\quad Cu = 800 \cdot psf$

Improved clay strength: $Su\_clay := Ar \cdot Ssc + (1 - Ar) \cdot Cu$

$Su\_clay = 940.111 \cdot psf$ After equation 10

Improved Clay-Silt Strenght $Su\_clay\_silt := Ar \cdot Ssc + (1 - Ar)(Cu + \sigma ob \cdot \tan(20))$

$Su\_clay\_silt = 4404.713 \cdot psf$ After equation 10

BASE SHEAR STABILITY EVALUATION - AT FULL HYDRO TEST - NO STRENGTH GAIN

$Dc := D + Tp + Tob \qquad Dc = 255.75 \cdot ft$ Equation 1

Bearing pressure at top of clay:

$$Qclay := q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2 + \gamma ob \cdot Tob$$ Equation 2

$$Qclay = 4218.04 \cdot psf$$

FOR CLAY LAYER ONLY

For a clay layer thicker than one sixth of the tank diameter $Nc1 := 6.1$ Equation 4

For a clay layer thinner than one sixth of the tank diamete $Nc2 := 4.1 + \left(\frac{Dc}{3Tclay}\right)$ Equation 5

$$Nc2 = 11.85$$

Place holder for easier coding: $PH := \frac{Dc}{Tclay}$

$Nc := if[(PH < 6), Nc1, Nc2] \qquad Nc = 11.85$

$Qult := Su\_clay \cdot Nc + \gamma ob \cdot Tob \qquad Qult = 12036.678 \cdot psf$

$$FS\_base\_clay := \frac{Su\_clay \cdot Nc}{\left[q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2\right]} \qquad FS\_base\_clay = 3.354$$ Equation 8

FOR COMBINED CLAY-SILT LAYER

For a clay layer thicker than one sixth of the tank diameter $Nc1 := 6.1$ Equation 4

For a clay layer thinner than one sixth of the tank diamete $Nc2 := 4.1 + \left(\frac{Dc}{3Tclay\_silt}\right)$ Equation 5

$$Nc2 = 9.115$$

Place holder for easier coding: $PH := \frac{Dc}{Tclay\_silt}$

$Nc := if[(PH < 6), Nc1, Nc2] \qquad Nc = 9.115$

$Qult := Su\_clay\_silt \cdot Nc + \gamma ob \cdot Tob \qquad Qult = 41044.035 \cdot psf$

$$FS\_base\_clay\_silt := \frac{Su\_clay\_silt \cdot Nc}{\left[q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2\right]} \qquad FS\_base\_clay\_silt = 12.087$$ Equation 8

EDGE SHEAR STABILITY EVALUATION - AT FULL HYDRO TEST - NO STRENGTH GAIN

IMPROVED CLAY

$Qapp := q + \gamma p \cdot Tp \qquad Qapp = 3725.7 \cdot psf$ Equation 11

With of segment that might fail $B := \begin{pmatrix} 10 \\ 20 \\ 40 \\ 60 \\ 80 \\ 100 \\ 120 \end{pmatrix} ft$

$Nc := 5.2 + \frac{B}{D} \qquad Nc = \begin{pmatrix} 5.242 \\ 5.283 \\ 5.367 \\ 5.45 \\ 5.533 \\ 5.617 \\ 5.7 \end{pmatrix}$

$Qall := Su\_clay \cdot Nc + \gamma p \cdot Tp \qquad Qall = \begin{pmatrix} 5771.446 \\ 5810.617 \\ 5888.96 \\ 5967.303 \\ 6045.645 \\ 6123.988 \\ 6202.33 \end{pmatrix} \cdot psf$

$FS\_edge\_clay := \frac{Qall}{Qapp} \qquad FS\_edge\_clay = \begin{pmatrix} 1.549 \\ 1.56 \\ 1.581 \\ 1.602 \\ 1.623 \\ 1.644 \\ 1.665 \end{pmatrix}$

EDGE SHEAR STABILITY EVALUATION - AT FULL HYDRO TEST - NO STRENGTH GAIN

IMPROVED CLAY and SILT

$Qapp := q + \gamma p \cdot Tp$ $\quad Qapp = 3725.7 \cdot psf$ $\quad$ Equation 11

With of segment that might fail $\quad B := \begin{pmatrix} 10 \\ 20 \\ 40 \\ 60 \\ 80 \\ 100 \\ 120 \end{pmatrix} ft$ $\quad Nc := 5.2 + \frac{B}{D}$ $\quad Nc = \begin{pmatrix} 5.242 \\ 5.283 \\ 5.367 \\ 5.45 \\ 5.533 \\ 5.617 \\ 5.7 \end{pmatrix}$

$Qall := Su\_clay\_silt \cdot Nc + \gamma p \cdot Tp \quad Qall = \begin{pmatrix} 23931.739 \\ 24115.269 \\ 24482.328 \\ 24849.388 \\ 25216.447 \\ 25583.507 \\ 25950.566 \end{pmatrix} \cdot psf$

$FS\_edge\_clay\_silt := \frac{Qall}{Qapp}$ $\quad FS\_edge\_clay\_silt = \begin{pmatrix} 6.423 \\ 6.473 \\ 6.571 \\ 6.67 \\ 6.768 \\ 6.867 \\ 6.965 \end{pmatrix}$

SUMMARY OF BASE AND EDGE SHEAR STABILITY EVALUATION - AT FULL HYDRO TEST - NO STRENGTH GAIN

$FS\_base\_clay = 3.354$

$FS\_base\_clay\_silt = 12.087$

$FS\_edge\_clay = \begin{pmatrix} 1.549 \\ 1.56 \\ 1.581 \\ 1.602 \\ 1.623 \\ 1.644 \\ 1.665 \end{pmatrix}$ $\quad FS\_edge\_clay\_silt = \begin{pmatrix} 6.423 \\ 6.473 \\ 6.571 \\ 6.67 \\ 6.768 \\ 6.867 \\ 6.965 \end{pmatrix}$

BASE SHEAR STABILITY EVALUATION - IN SERVICE - NO STRENGTH GAIN

$q := 2500psf$

$Dc := D + Tp + Tob \qquad Dc = 255.75 \cdot ft$ Equation 1

Bearing pressure at top of clay:

$$Qclay := q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2 + \gamma ob \cdot Tob$$ Equation 2

$$Qclay = 3881.641 \cdot psf$$

FOR CLAY LAYER ONLY

For a clay layer thicker than one sixth of the tank diameter $Nc1 := 6.1$ Equation 4

For a clay layer thinner than one sixth of the tank diamete $Nc2 := 4.1 + \left(\frac{Dc}{3Tclay}\right)$ Equation 5

$$Nc2 = 11.85$$

Place holder for easier coding: $PH := \frac{Dc}{Tclay}$

$Nc := if[(PH < 6), Nc1, Nc2] \qquad Nc = 11.85$

$Qult := Su\_clay \cdot Nc + \gamma ob \cdot Tob \qquad Qult = 12036.678 \cdot psf$

$$FS\_base\_clay := \frac{Su\_clay \cdot Nc}{\left[q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2\right]} \qquad FS\_base\_clay = 3.732$$ Equation 8

FOR COMBINED CLAY-SILT LAYER

For a clay layer thicker than one sixth of the tank diameter $Nc1 := 6.1$ Equation 4

For a clay layer thinner than one sixth of the tank diamete $Nc2 := 4.1 + \left(\frac{Dc}{3Tclay\_silt}\right)$ Equation 5

$$Nc2 = 9.115$$

Place holder for easier coding: $PH := \frac{Dc}{Tclay\_silt}$

$Nc := if[(PH < 6), Nc1, Nc2] \qquad Nc = 9.115$

$Qult := Su\_clay\_silt \cdot Nc + \gamma ob \cdot Tob \qquad Qult = 41044.035 \cdot psf$

$$FS\_base\_clay\_silt := \frac{Su\_clay\_silt \cdot Nc}{\left[q \cdot \left(\frac{D}{Dc}\right)^2 + \gamma p \cdot Tp \cdot \left[\frac{(D + Tp)}{Dc}\right]^2\right]} \qquad FS\_base\_clay\_silt = 13.449$$ Equation 8

EDGE SHEAR STABILITY EVALUATION - IN SERVICE - NO STRENGTH GAIN

IMPROVED CLAY

$Qapp := q + \gamma p \cdot Tp \qquad Qapp = 3343.7 \cdot psf$ Equation 11

With of segment that might fail $B := \begin{pmatrix} 10 \\ 20 \\ 40 \\ 60 \\ 80 \\ 100 \\ 120 \end{pmatrix} ft$

$Nc := 5.2 + \frac{B}{D} \qquad Nc = \begin{pmatrix} 5.242 \\ 5.283 \\ 5.367 \\ 5.45 \\ 5.533 \\ 5.617 \\ 5.7 \end{pmatrix}$

$Qall := Su\_clay \cdot Nc + \gamma p \cdot Tp \qquad Qall = \begin{pmatrix} 5771.446 \\ 5810.617 \\ 5888.96 \\ 5967.303 \\ 6045.645 \\ 6123.988 \\ 6202.33 \end{pmatrix} \cdot psf$

$FS\_edge\_clay := \frac{Qall}{Qapp} \qquad FS\_edge\_clay = \begin{pmatrix} 1.726 \\ 1.738 \\ 1.761 \\ 1.785 \\ 1.808 \\ 1.832 \\ 1.855 \end{pmatrix}$

EDGE SHEAR STABILITY EVALUATION - IN SERVICE - NO STRENGTH GAIN

IMPROVED CLAY and SILT

$Qapp := q + \gamma p \cdot Tp$ $\quad Qapp = 3343.7 \cdot psf$ $\quad$ Equation 11

With of segment that might fail $\quad B := \begin{pmatrix} 10 \\ 20 \\ 40 \\ 60 \\ 80 \\ 100 \\ 120 \end{pmatrix} ft$ $\quad Nc := 5.2 + \frac{B}{D}$ $\quad Nc = \begin{pmatrix} 5.242 \\ 5.283 \\ 5.367 \\ 5.45 \\ 5.533 \\ 5.617 \\ 5.7 \end{pmatrix}$

$$Qall := Su\_clay\_silt \cdot Nc + \gamma p \cdot Tp \quad Qall = \begin{pmatrix} 23931.739 \\ 24115.269 \\ 24482.328 \\ 24849.388 \\ 25216.447 \\ 25583.507 \\ 25950.566 \end{pmatrix} \cdot psf$$

$$FS\_edge\_clay\_silt := \frac{Qall}{Qapp} \quad FS\_edge\_clay\_silt = \begin{pmatrix} 7.157 \\ 7.212 \\ 7.322 \\ 7.432 \\ 7.541 \\ 7.651 \\ 7.761 \end{pmatrix}$$

SUMMARY OF BASE AND EDGE SHEAR STABILITY EVALUATION - IN SERVCIE - NO STRENGTH GAIN

$FS\_base\_clay = 3.732$

$FS\_base\_clay\_silt = 13.449$

$$FS\_edge\_clay = \begin{pmatrix} 1.726 \\ 1.738 \\ 1.761 \\ 1.785 \\ 1.808 \\ 1.832 \\ 1.855 \end{pmatrix} \qquad FS\_edge\_clay\_silt = \begin{pmatrix} 7.157 \\ 7.212 \\ 7.322 \\ 7.432 \\ 7.541 \\ 7.651 \\ 7.761 \end{pmatrix}$$

Units

$kips \equiv 1000 \cdot lbf$

$ksf := 1000 \cdot psf$

$tsf := 2 \cdot ksf$

$kPa := 1000 \cdot Pa$

$pi := 3.14159$

$ksi \equiv \frac{1000 \cdot lbf}{in^2}$

$pcf \equiv \frac{lbf}{ft^3}$

$in^2 = 0.001\, m^2$

$psi \equiv \frac{lbf}{in^2}$

$psf \equiv \frac{lbf}{ft^2}$

$pcf \equiv \frac{lbf}{ft^3}$

$kcf \equiv \frac{k}{ft^3}$

$k \equiv kips$

$N \equiv newton$

$ORIGIN \equiv 1$

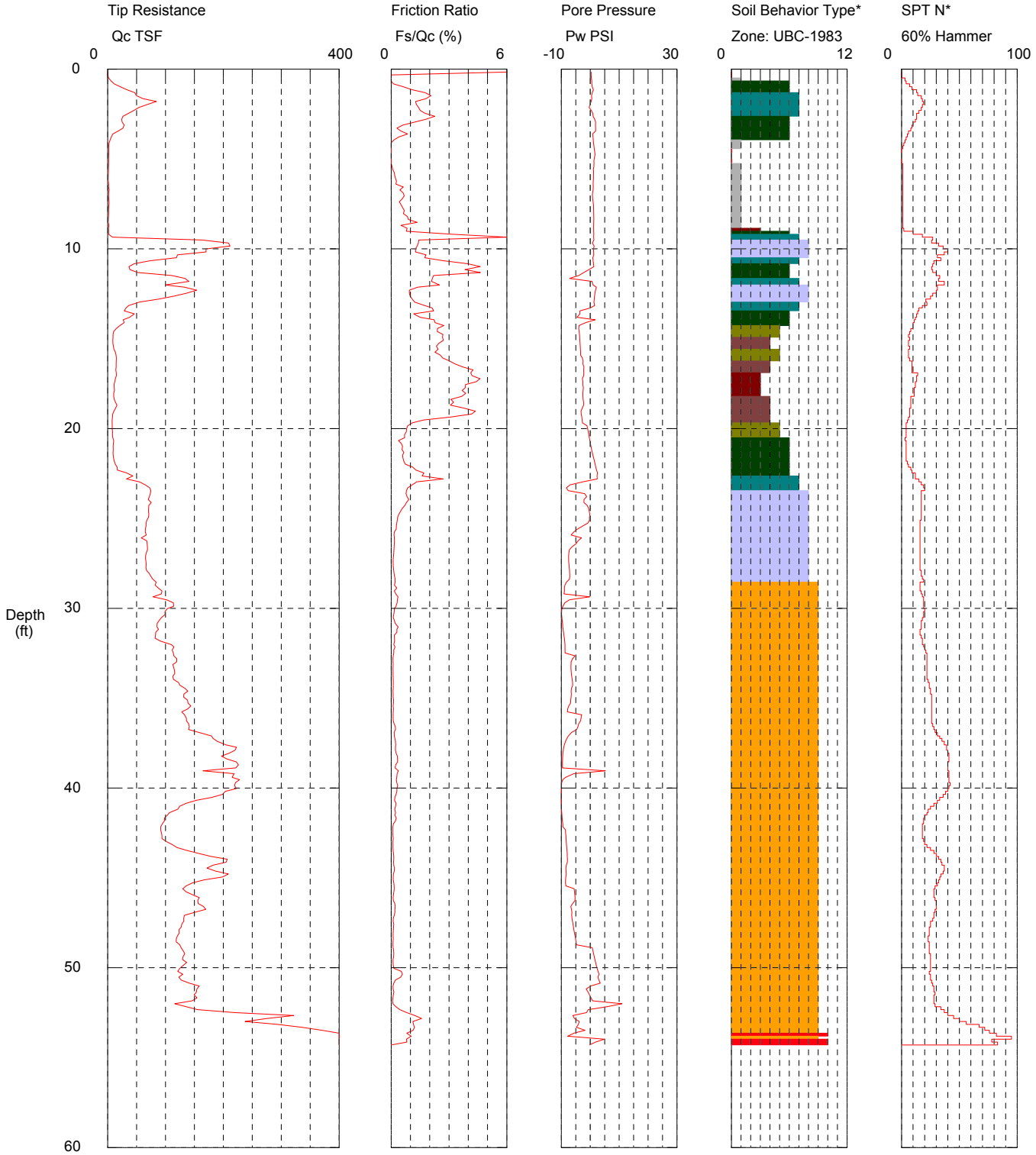
$MPa := 10^6 \cdot Pa$

$kN \equiv 1000 newton$

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-161
Cone Used: DDG1254

CPT Date/Time: 11/22/2014 10:28:12 AM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 54.30 feet

Depth Increment = 0.164 feet

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

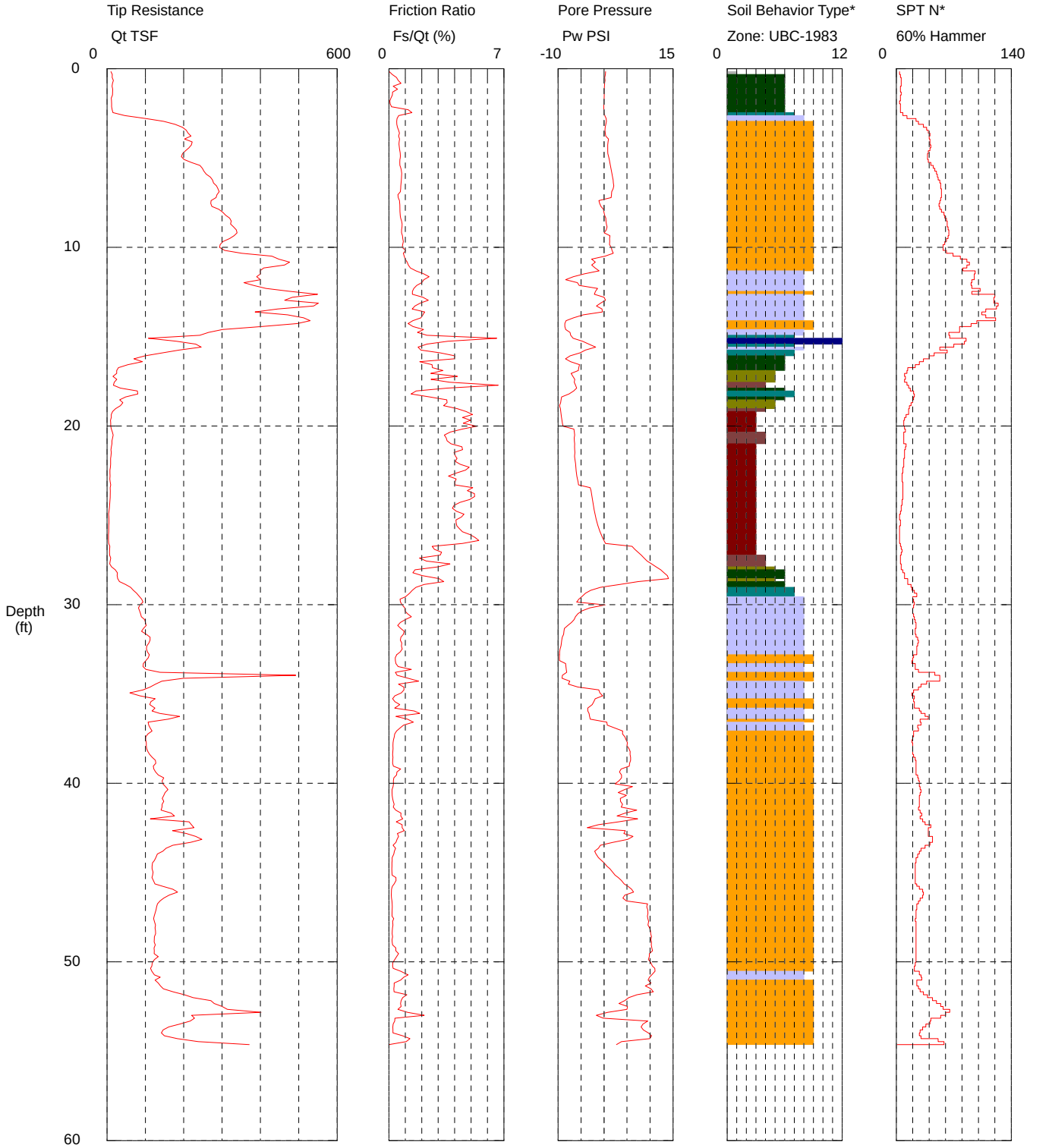
InSitu Engineering Predrilled top 10 feet

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

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-162
Cone Used: DDG1254

CPT Date/Time: 11/12/2014 9:20:32 AM
Location: Port of Vancouver
Job Number: 860058



Maximum Depth = 54.63 feet

Depth Increment = 0.164 feet

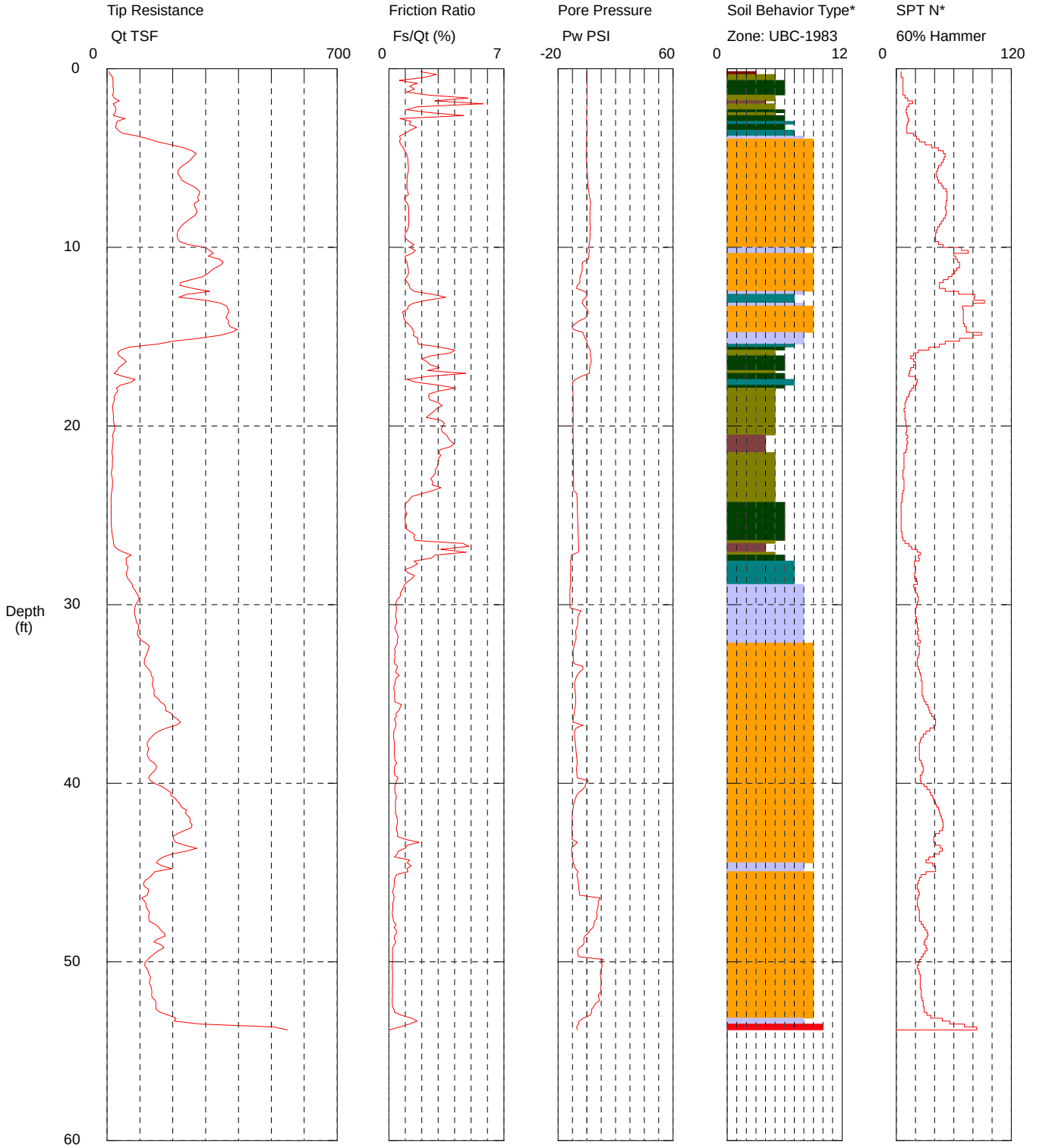
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

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

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-163(s)
Cone Used: DDG1263

CPT Date/Time: 11/11/2014 12:48:55 PM
Location: Port of Vancouver
Job Number: 860058



Maximum Depth = 53.81 feet

Depth Increment = 0.164 feet

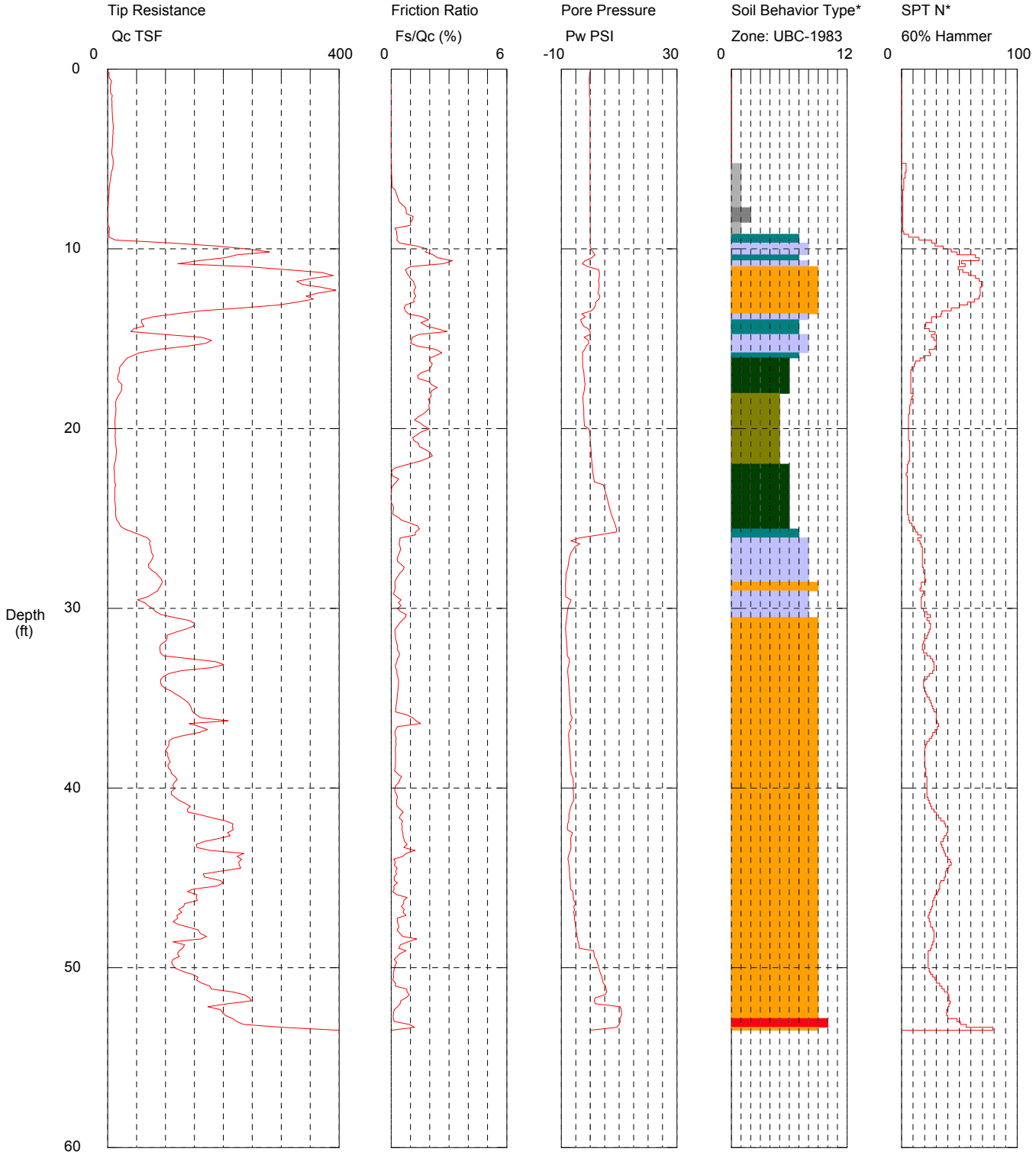
1 sensitive fine grained
2 organic material
3 clay
4 silty clay to clay
5 clayey silt to silty clay
6 sandy silt to clayey silt
7 silty sand to sandy silt
8 sand to silty sand
9 sand
10 gravelly sand to sand
11 very stiff fine grained (*)
12 sand to clayey sand (*)

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

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-164B
Cone Used: DDG1254

CPT Date/Time: 11/22/2014 12:53:45 PM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 53.48 feet

Depth Increment = 0.164 feet

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

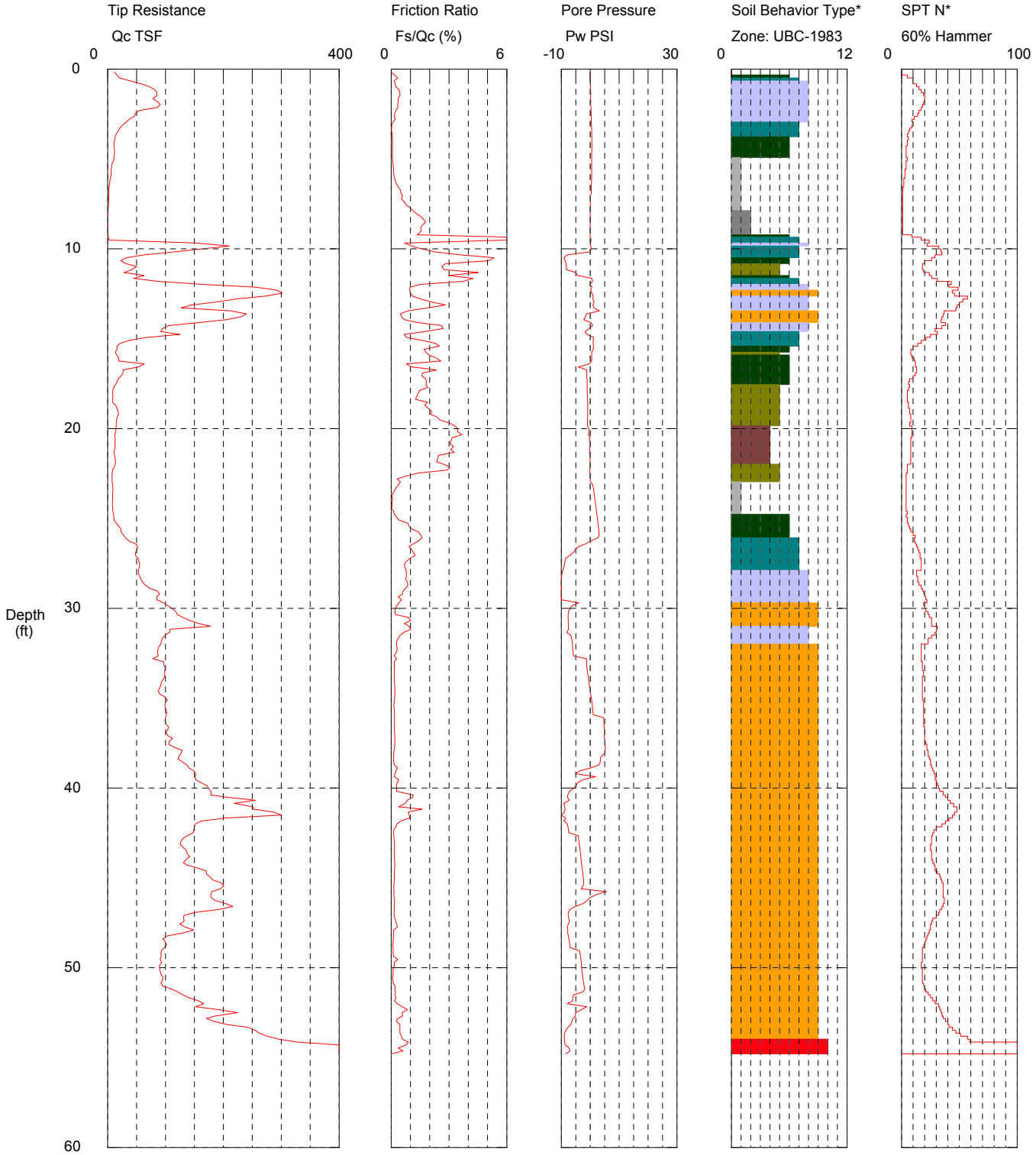
InSitu Engineering Pre-drilled top 10 feet.

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

Hayward Baker

Operator: Walsh
Sounding: HB-CPT-165A
Cone Used: DDG1254

CPT Date/Time: 11/22/2014 11:38:50 AM
Location: Port of Vancouver
Job Number: 8650058



Maximum Depth = 54.79 feet

Depth Increment = 0.164 feet

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

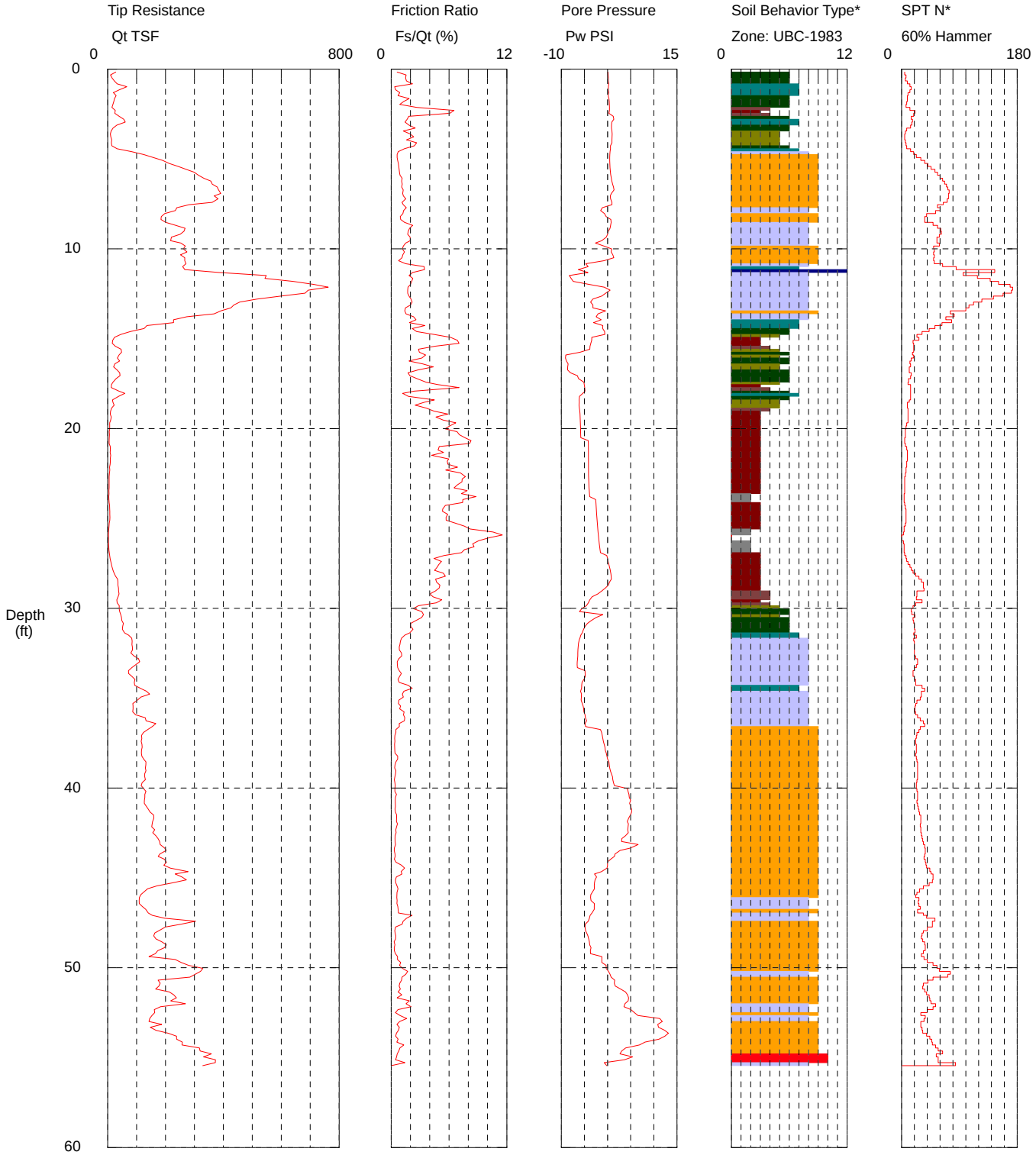
InSitu Engineering Pre-drilled top 10 feet.

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

Hayward Baker

Operator: Pemble
Sounding: HB-CPT-166A
Cone Used: DDG1254

CPT Date/Time: 11/12/2014 10:36:17 AM
Location: Port of Vancouver
Job Number: 860058



Maximum Depth = 55.45 feet

Depth Increment = 0.164 feet

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

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

Tank Pipeline

Hayward Baker Inc. THINK SAFE	SUBJEC	Tesoro Savage Petroleum Terminal LLC Pipeline			
	Job no.	860058	Design by:	M. Rohrbach	Date: 14-Apr-15
	File	CPT-Liquefaction-Boulanger	Checked		
	Spreadsheet Rev.	23-Dec-14	Reviewed		

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-171A

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	16.2-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	28.22	ft.
Elevation of bottom of Soil Profile	-26.08	ft.
Elevation of Installation	28.22	Work from Existing Grade
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	54.3-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	458.90-psf	

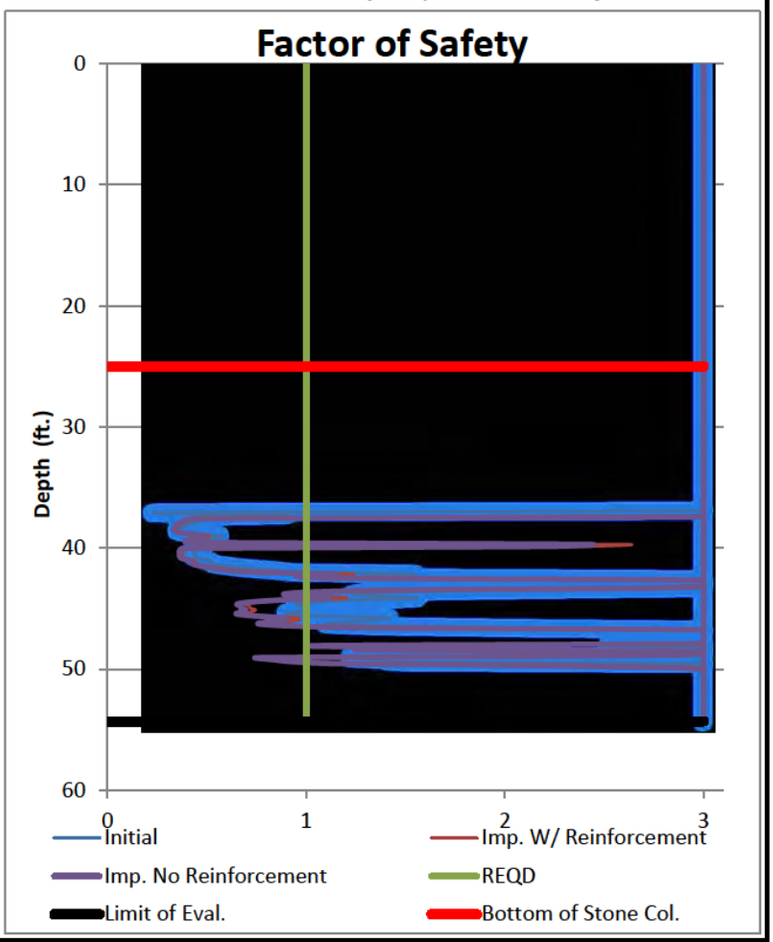
Stone Column Design Information

L =	25-ft		ϕ_c = 50
Column Spacing	7.7-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 12% Replacment		SILTY soils: 12% Replacment	
SAND: Ar = (Ac/A)	0.12	SILTY SOIL: Ar = (Ac/A)	0.12
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 54.3-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 6.1-ft.

Calculated Min: 1.15-in via T&S, Min never < 0.5 in.

Calculated Max: 1.23-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 7.5-ft.

Calculated Min: 1.42-in via T&S, Min never < 0.5 in.

Calculated Max: 1.49-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 7.0-ft.

Calculated Min: 1.34-in via T&S, Min never < 0.5 in.

Calculated Max: 1.43-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Worksheet Tab: 14-Elastic Sett. (Imp.) Evaluation of CPT data (includes- Boulanger, 2014) Rev 12/23/14

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Pipeline

IMPROVED CASE: CPT-171A

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment

= $1-0.5*(q/(q_{bar}-q)$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

$q = \gamma Df$

Stone Column Length 25-ft

Reduction factor on Priebe: 1

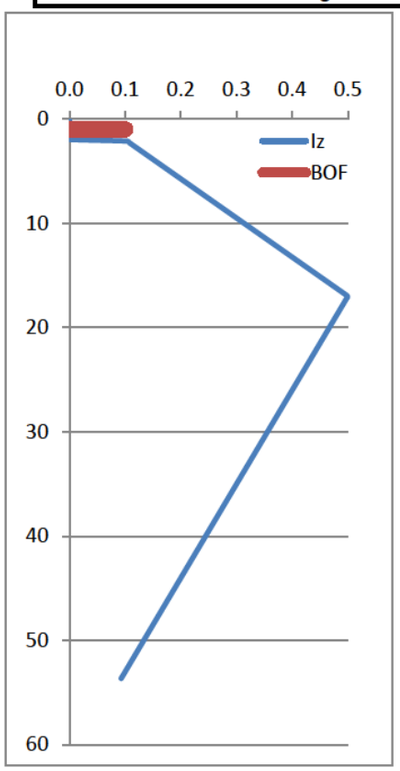
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	1000.00-psf	
L	30.0-ft	
B	30.0-ft	
fill density	130-pcf	Assigned by Designer
Elevation of Top of Soil Profile	28.22	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es=X \cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.824	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)

I_z = 0.1	at depth	z=0= 0.0
I_z = 0.5	at depth	$z=z_1$=**0.5**B= 15.0
I_z = 0	at depth	$z=z_2$=**2.0**B= 60.0

For foundations with L/B ≥ 10

I_z = 0.2	at depth	z=0= 0.0
I_z = 0.5	at depth	$z=z_1$=**1.0**B= 30.0
I_z = 0	at depth	$z=z_2$=**4.0**B= 120.0

Interpolation for L/B values between 1 and 10

I_z = 0.10	0	at z= 0.0
I_z = 0.5	0.50	at $z=z_1$= 15.0
I_z = 0	2.00	at $z=z_2$= 60.0

Σ(Iz/Es)Δz (ft2/lb)= 6.39E-05

Total Elastic Settlement (Flexible foundation)

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0600-ft 0.72-in

Total Rapid Elastic Settlement

$S_e=C_1(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0390-ft 0.47-in

Long Term Settlement 0.25-in

Calculated Consolidation Type Settlement:

Total:	0.35-in	Adjusted for footing Type
Recomp.	0.04-in	Adjusted for footing Type
Virgin Sett.	0.31-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	0.36-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.56-in	

Summary of Settlement:

Total: 1.63-in

Rapid: 1.39-in

Long Term: 0.24-in

Hayward Baker Inc. THINK SAFE	SUBJEC	Tesoro Savage Petroleum Terminal LLC Pipeline			
	Job no.	860058	Design by:	M. Rohrbach	Date: 14-Apr-15
	File	CPT-Liquefaction-Boulanger	Checked		
	Spreadsheet Rev.	23-Dec-14	Reviewed		

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-172

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	16.2-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	28.22	ft.
Elevation of bottom of Soil Profile	-30.67	ft.
Elevation of Installation	28.22	Work from Existing Grade
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	58.9-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, ν_s	0.333	
Poisson's Ratio for Stone Column, ν_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	458.90-psf	

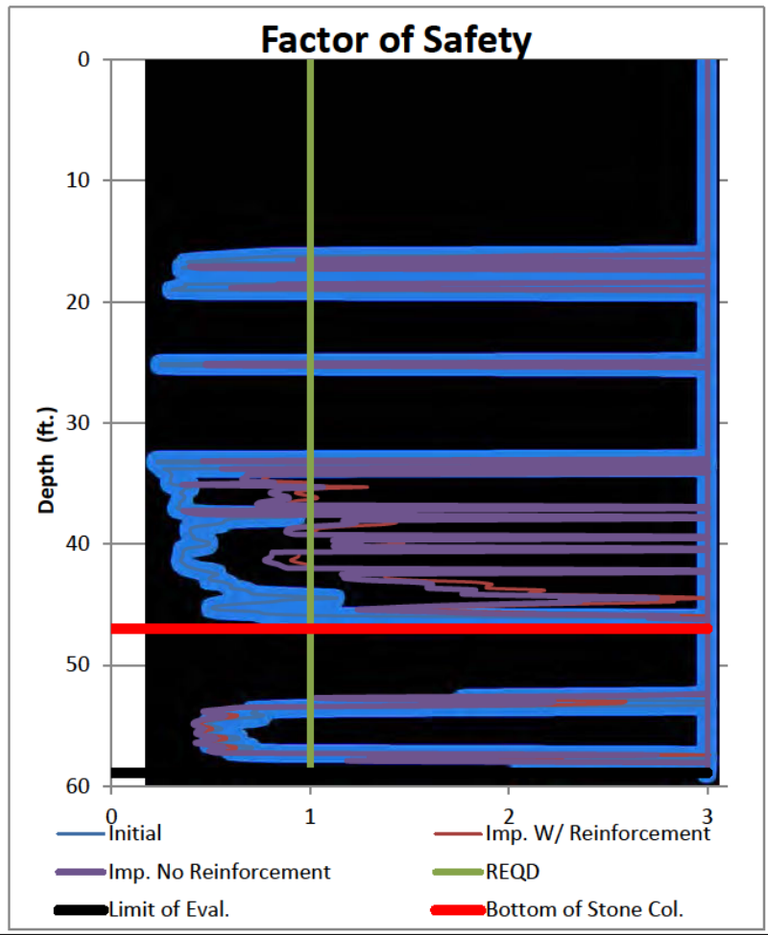
Stone Column Design Information

L =	47-ft		ϕ_c = 50
Column Spacing	7.7-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 12% Replacment		SILTY soils: 12% Replacment	
SAND: Ar = (Ac/A)	0.12	SILTY SOIL: Ar = (Ac/A)	0.12
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 58.9-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 18.0-ft.

Calculated Min: 3.88-in via T&S, Min never < 0.5 in.

Calculated Max: 3.91-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 10.7-ft.

Calculated Min: 2.50-in via T&S, Min never < 0.5 in.

Calculated Max: 2.69-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 9.0-ft.

Calculated Min: 2.13-in via T&S, Min never < 0.5 in.

Calculated Max: 2.28-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Pipeline — IMPROVED CASE: CPT-172

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e = C_1 C_2 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment

= $1 - 0.5*(q/(q_{bar}-q))$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

$q = \gamma Df$

Stone Column Length 47-ft

Reduction factor on Priebe: 1

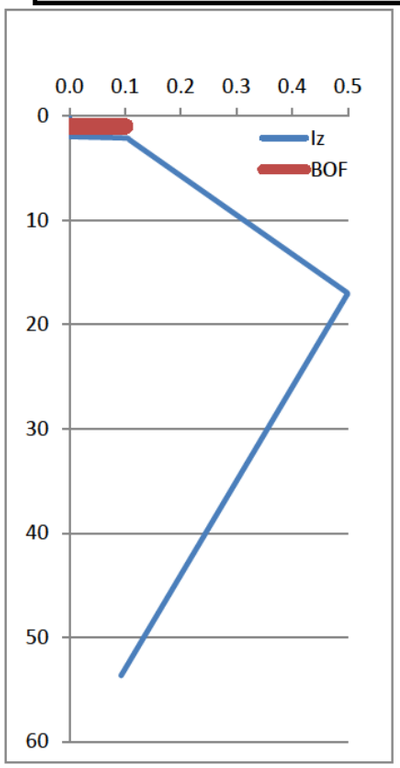
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	1000.00-psf	
L	30.0-ft	
B	30.0-ft	
fill density	130-pcf	Assigned by Designer
Elevation of Top of Soil Profile	28.22	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es = X \cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.824	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=$**0.5**B=	15.0
I_z = 0	at depth	$z=z_2=$**2.0**B=	60.0
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=$**1.0**B=	30.0
I_z = 0	at depth	$z=z_2=$**4.0**B=	120.0
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	15.0
I_z = 0	2.00	at $z=z_2$=	60.0

Σ(Iz/Es)Δz (ft2/lb)= 1.79E-05

Total Elastic Settlement (Flexible foundation)

$S_e = C_1 C_2 (q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0168-ft 0.20-in

Total Rapid Elastic Settlement

$S_e = C_1 (q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0109-ft 0.13-in

Long Term Settlement 0.07-in

Calculated Consolidation Type Settlement:

Total:	0.10-in	Adjusted for footing Type
Recomp.	0.01-in	Adjusted for footing Type
Virgin Sett.	0.10-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	0.11-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:

Total: 0.30-in

Rapid: 0.24-in

Long Term: 0.06-in

Hayward Baker Inc. THINK SAFE	SUBJEC	Tesoro Savage Petroleum Terminal LLC Pipeline			
	Job no.	860058	Design by:	M. Rohrbach	Date: 14-Apr-15
	File	CPT-Liquefaction-Boulanger	Checked		
	Spreadsheet Rev.	23-Dec-14	Reviewed		

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-173

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	15.9-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	27.93	ft.
Elevation of bottom of Soil Profile	-29.65	ft.
Elevation of Installation	23.93	
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	57.6-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	496.60-psf	

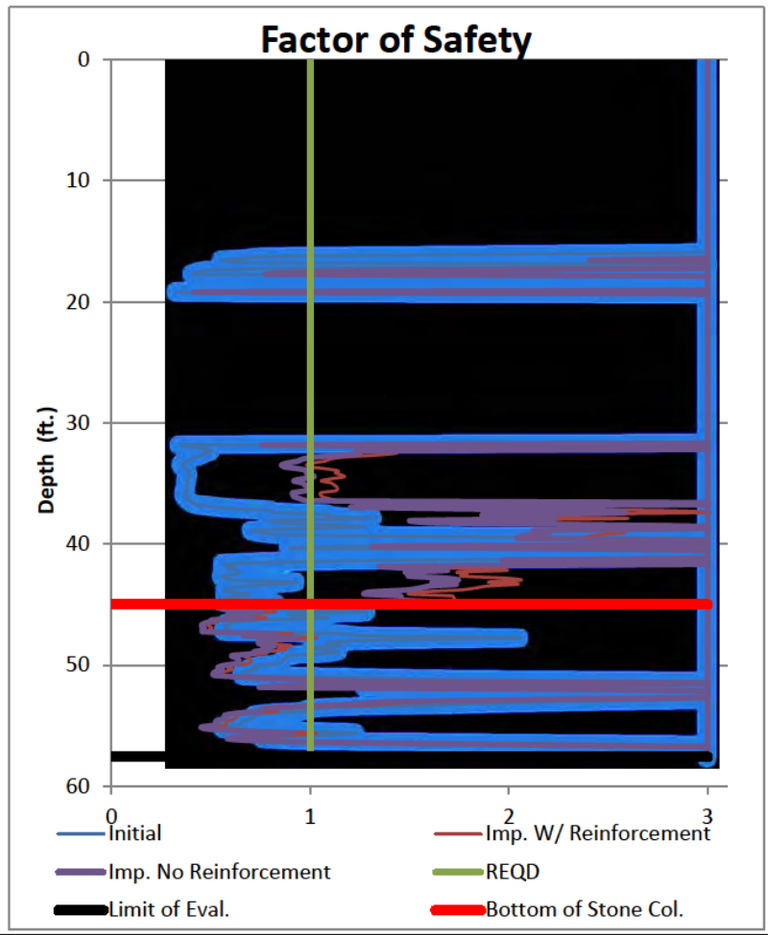
Stone Column Design Information

L =	45-ft		ϕ_c = 50
Column Spacing	7.7-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 12% Replacment		SILTY soils: 12% Replacment	
SAND: Ar = (Ac/A)	0.12	SILTY SOIL: Ar = (Ac/A)	0.12
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 57.6-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 17.9-ft.

Calculated Min: 3.39-in via T&S, Min never < 0.5 in.

Calculated Max: 3.63-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 13.5-ft.

Calculated Min: 2.69-in via T&S, Min never < 0.5 in.

Calculated Max: 2.83-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 9.7-ft.

Calculated Min: 1.89-in via T&S, Min never < 0.5 in.

Calculated Max: 2.23-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Pipeline

IMPROVED CASE: CPT-173

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$$

C_1 = a correction factor for the depth of foundation embedment

= $1-0.5*(q/(q_{bar}-q)$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

$q = \gamma Df$

Stone Column Length 45-ft

Reduction factor on Priebe: 1

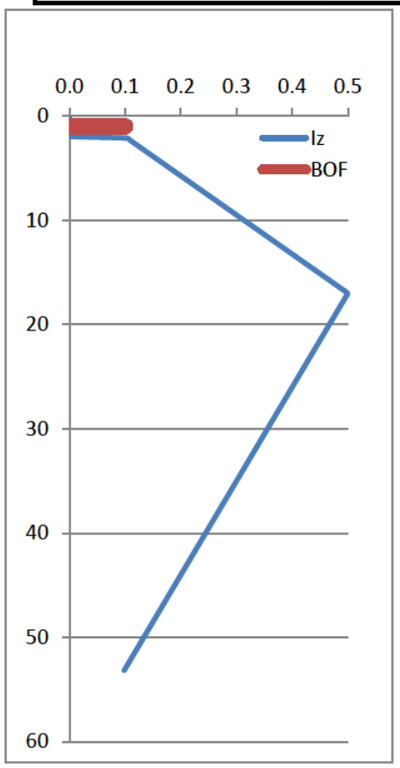
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	1000.00-psf	
L	30.0-ft	
B	30.0-ft	
fill density	130-pcf	Assigned by Designer
Elevation of Top of Soil Profile	27.93	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es=X \cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.824	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1$=**0.5**B=	15.0
I_z = 0	at depth	$z=z_2$=**2.0**B=	60.0
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1$=**1.0**B=	30.0
I_z = 0	at depth	$z=z_2$=**4.0**B=	120.0
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	15.0
I_z = 0	2.00	at $z=z_2$=	60.0

Σ(Iz/Es)Δz (ft2/lb)= 2.98E-05

Total Elastic Settlement (Flexible foundation)

$S_e=C_1C_2(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0280-ft 0.34-in

Total Rapid Elastic Settlement

$S_e=C_1(q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0182-ft 0.22-in

Long Term Settlement 0.12-in

Calculated Consolidation Type Settlement:

Total:	0.41-in	Adjusted for footing Type
Recomp.	0.04-in	Adjusted for footing Type
Virgin Sett.	0.36-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	0.45-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.46-in	

Summary of Settlement:

Total: 1.21-in

Rapid: 1.13-in

Long Term: 0.08-in

Hayward Baker Inc.

THINK SAFE

SUBJEC	Tesoro Savage Petroleum Terminal LLC Pipeline		
Job no.	860058	Design by: M. Rohrbach	Date: 14-Apr-15
File	CPT-Liquefaction-Boulanger	Checked	
Spreadsheet Rev.	23-Dec-14	Reviewed	

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-174

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	16.3-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	28.27	ft.
Elevation of bottom of Soil Profile	-28.16	ft.
Elevation of Installation	24.27	
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	56.4-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, ν_s	0.333	
Poisson's Ratio for Stone Column, ν_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	452.40-psf	

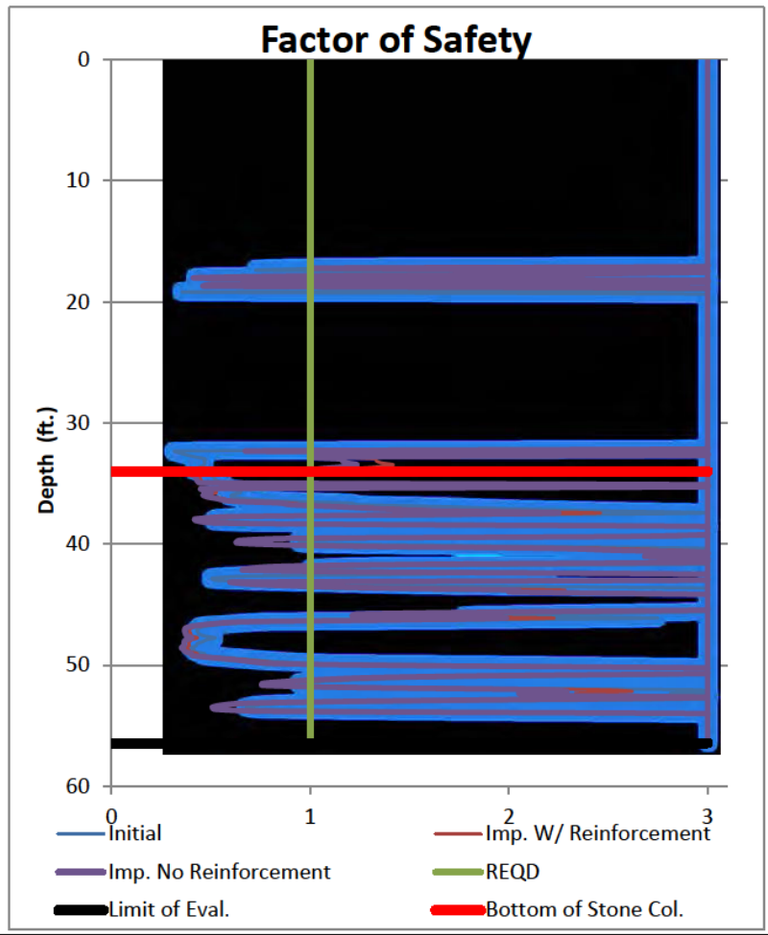
Stone Column Design Information

L = 34-ft			ϕ_c = 50
Column Spacing 7.7-ft			Ec = 750.0-tsf
Kac = 0.1324743	$=TAN^2(45-(\phi_c/2)$		Gc = 277.8-tsf

SANDY Soils 12% Replacment	SILTY soils: 12% Replacment
SAND: Ar = (Ac/A) 0.12	SILTY SOIL: Ar = (Ac/A) 0.12
Stone Column Diameter 3-ft	Stone Column Diameter 3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 56.4-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 11.4-ft.

Calculated Min: 2.15-in via T&S, Min never < 0.5 in.

Calculated Max: 2.23-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 10.5-ft.

Calculated Min: 2.02-in via T&S, Min never < 0.5 in.

Calculated Max: 2.23-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 10.3-ft.

Calculated Min: 1.98-in via T&S, Min never < 0.5 in.

Calculated Max: 2.04-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Pipeline — IMPROVED CASE: CPT-174

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e = C_1 C_2 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment

= $1 - 0.5*(q/(q_{bar}-q))$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

$q = \gamma Df$

Stone Column Length 34-ft

Reduction factor on Priebe: 1

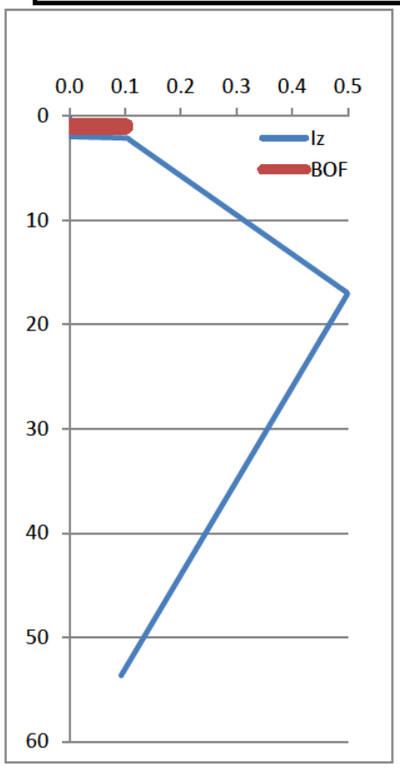
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	1000.00-psf	
L	30.0-ft	
B	30.0-ft	
fill density	130-pcf	Assigned by Designer
Elevation of Top of Soil Profile	28.27	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es = X \cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.824	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=0.5B$=	15.0
I_z = 0	at depth	$z=z_2=2.0B$=	60.0
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=1.0B$=	30.0
I_z = 0	at depth	$z=z_2=4.0B$=	120.0
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	15.0
I_z = 0	2.00	at $z=z_2$=	60.0

$\Sigma(Iz/Es)\Delta z$ (ft2/lb)= 3.05E-05

Total Elastic Settlement (Flexible foundation)

$S_e = C_1 C_2 (q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0286-ft 0.34-in

Total Rapid Elastic Settlement

$S_e = C_1 (q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0186-ft 0.22-in

Long Term Settlement 0.12-in

Calculated Consolidation Type Settlement:

Total:	0.17-in	Adjusted for footing Type
Recomp.	0.03-in	Adjusted for footing Type
Virgin Sett.	0.14-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	0.20-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.43-in	

Summary of Settlement:

Total: 0.94-in

Rapid: 0.85-in

Long Term: 0.09-in

Hayward Baker Inc.

THINK SAFE

SUBJEC	Tesoro Savage Petroleum Terminal LLC Pipeline		
Job no.	860058	Design by: M. Rohrbach	Date: 14-Apr-15
File	CPT-Liquefaction-Boulanger	Checked	
Spreadsheet Rev.	23-Dec-14	Reviewed	

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-175E

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	16.2-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	28.22	ft.
Elevation of bottom of Soil Profile	-26.90	ft.
Elevation of Installation	24.22	
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	55.1-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	458.90-psf	

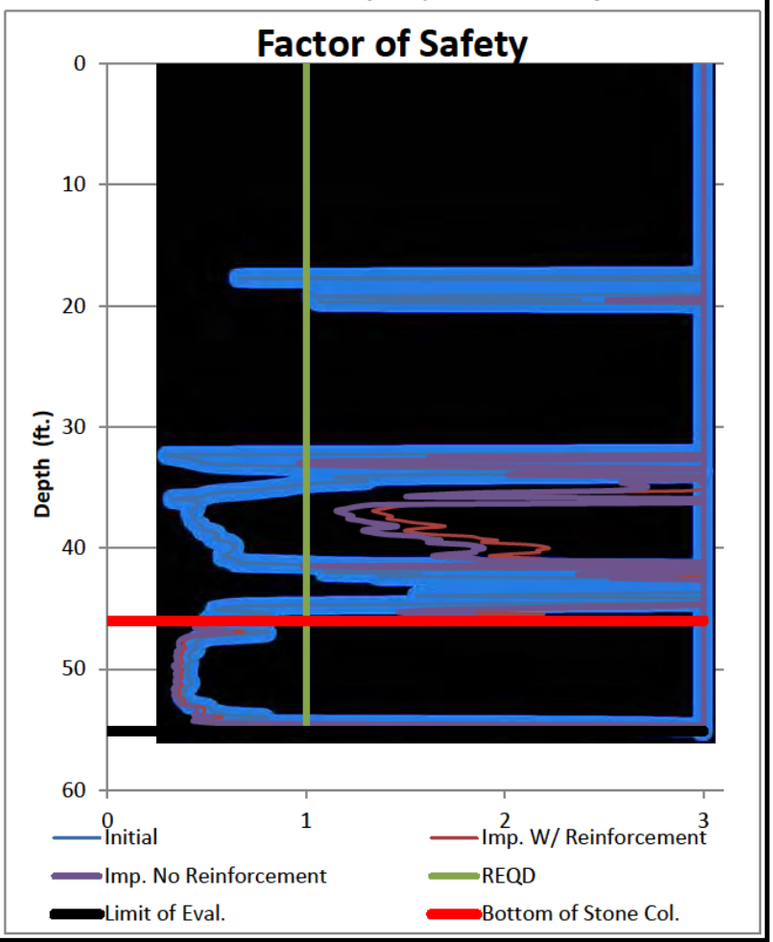
Stone Column Design Information

L = 46-ft			ϕ_c = 50
Column Spacing 7.7-ft			Ec = 750.0-tsf
Kac = 0.1324743	$=TAN^2(45-(\phi_c/2)$		Gc = 277.8-tsf

SANDY Soils 12% Replacment	SILTY soils: 12% Replacment
SAND: Ar = (Ac/A) 0.12	SILTY SOIL: Ar = (Ac/A) 0.12
Stone Column Diameter 3-ft	Stone Column Diameter 3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 55.1-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 17.7-ft.

Calculated Min: 3.55-in via T&S, Min never < 0.5 in.

Calculated Max: 3.63-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 8.8-ft.

Calculated Min: 1.95-in via T&S, Min never < 0.5 in.

Calculated Max: 2.10-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 8.5-ft.

Calculated Min: 1.90-in via T&S, Min never < 0.5 in.

Calculated Max: 1.93-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Pipeline — IMPROVED CASE: CPT-175E

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e = C_1 C_2 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment

= $1 - 0.5*(q/(q_{bar}-q))$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

$q = \gamma Df$

Stone Column Length 46-ft

Reduction factor on Priebe: 1

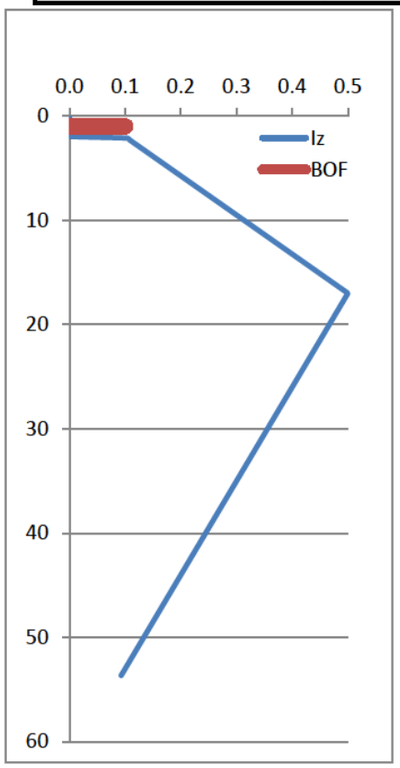
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	1000.00-psf	
L	30.0-ft	
B	30.0-ft	
fill density	130-pcf	Assigned by Designer
Elevation of Top of Soil Profile	28.22	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es = X \cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.824	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=0.5B$=	15.0
I_z = 0	at depth	$z=z_2=2.0B$=	60.0
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=1.0B$=	30.0
I_z = 0	at depth	$z=z_2=4.0B$=	120.0
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	15.0
I_z = 0	2.00	at $z=z_2$=	60.0

$\Sigma(Iz/Es)\Delta z$ (ft2/lb)= 3.19E-05

Total Elastic Settlement (Flexible foundation)

$S_e = C_1 C_2 (q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0299-ft 0.36-in

Total Rapid Elastic Settlement

$S_e = C_1 (q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0194-ft 0.23-in

Long Term Settlement 0.13-in

Calculated Consolidation Type Settlement:

Total:	0.36-in	Adjusted for footing Type
Recomp.	0.03-in	Adjusted for footing Type
Virgin Sett.	0.33-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	0.40-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.28-in	

Summary of Settlement:

Total: 1.00-in

Rapid: 0.91-in

Long Term: 0.10-in

Hayward Baker Inc. THINK SAFE	SUBJEC	Tesoro Savage Petroleum Terminal LLC Pipeline			
	Job no.	860058	Design by:	M. Rohrbach	Date: 14-Apr-15
	File	CPT-Liquefaction-Boulanger	Checked		
	Spreadsheet Rev.	23-Dec-14	Reviewed		

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-176

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	16.6-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	28.59	ft.
Elevation of bottom of Soil Profile	-26.36	ft.
Elevation of Installation	24.59	
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	55.0-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	410.80-psf	

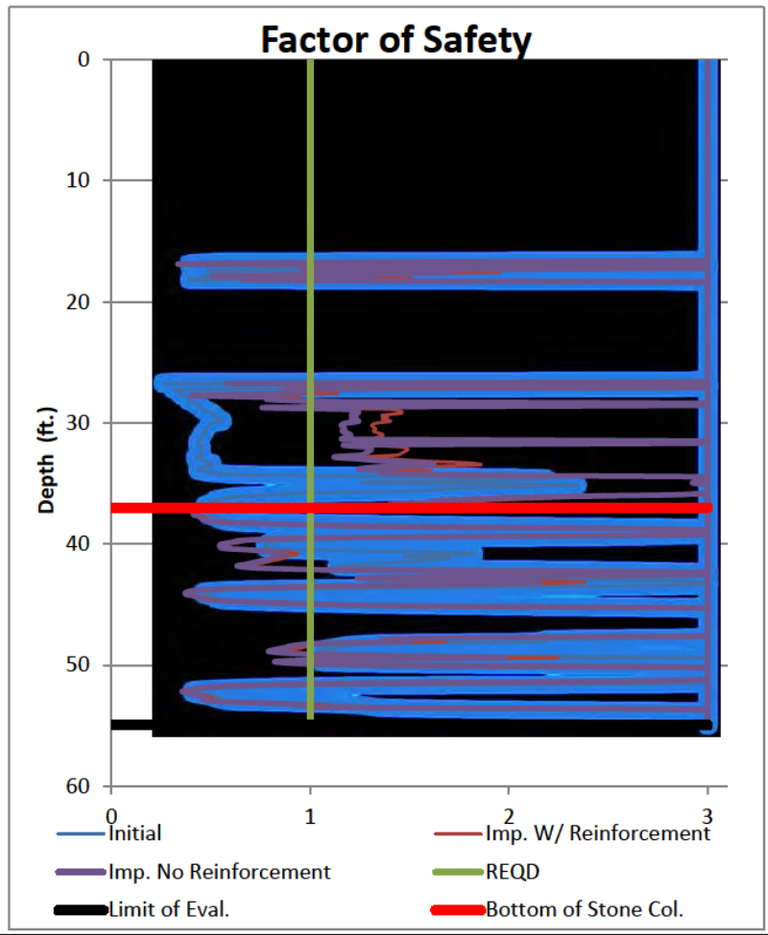
Stone Column Design Information

L =	37-ft		ϕ_c = 50
Column Spacing	7.7-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 12% Replacment		SILTY soils: 12% Replacment	
SAND: Ar = (Ac/A)	0.12	SILTY SOIL: Ar = (Ac/A)	0.12
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 55.0-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 14.4-ft.

Calculated Min: 3.02-in via T&S, Min never < 0.5 in.

Calculated Max: 3.17-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 10.5-ft.

Calculated Min: 2.12-in via T&S, Min never < 0.5 in.

Calculated Max: 2.55-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 8.8-ft.

Calculated Min: 1.78-in via T&S, Min never < 0.5 in.

Calculated Max: 1.96-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Pipeline — IMPROVED CASE: CPT-176

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e = C_1 C_2 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment

= $1 - 0.5*(q/(q_{bar} - q))$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

$q = \gamma Df$

Stone Column Length 37-ft

Reduction factor on Priebe: 1

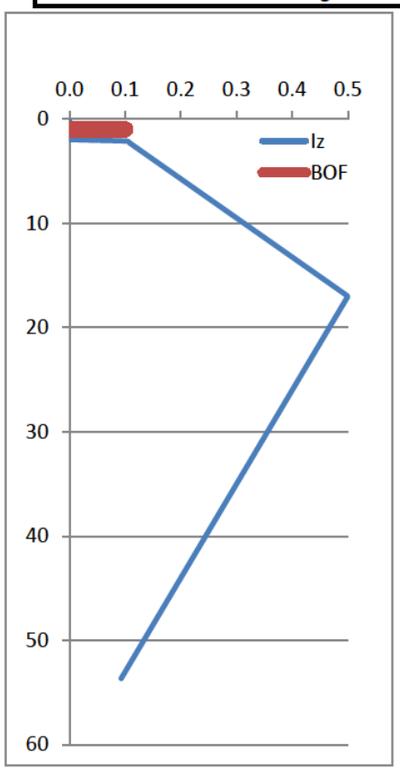
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	1000.00-psf	
L	30.0-ft	
B	30.0-ft	
fill density	130-pcf	Assigned by Designer
Elevation of Top of Soil Profile	28.59	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es = X \cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.824	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=0.5B$=	15.0
I_z = 0	at depth	$z=z_2=2.0B$=	60.0
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=1.0B$=	30.0
I_z = 0	at depth	$z=z_2=4.0B$=	120.0
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	15.0
I_z = 0	2.00	at $z=z_2$=	60.0

$\Sigma(Iz/Es)\Delta z$ (ft2/lb)= 2.77E-05

Total Elastic Settlement (Flexible foundation)

$S_e = C_1 C_2 (q_{bar} - q)\Sigma[(I_z/E_s)\Delta z]$= 0.0260-ft 0.31-in

Total Rapid Elastic Settlement

$S_e = C_1 (q_{bar} - q)\Sigma[(I_z/E_s)\Delta z]$= 0.0169-ft 0.20-in

Long Term Settlement 0.11-in

Calculated Consolidation Type Settlement:

Total:	0.84-in	Adjusted for footing Type
Recomp.	0.06-in	Adjusted for footing Type
Virgin Sett.	0.79-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	0.90-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.00-in	

Summary of Settlement:

Total: 1.15-in

Rapid: 1.10-in

Long Term: 0.05-in

Hayward Baker Inc. THINK SAFE	SUBJEC	Tesoro Savage Petroleum Terminal LLC Pipeline			
	Job no.	860058	Design by:	M. Rohrbach	Date: 14-Apr-15
	File	CPT-Liquefaction-Boulanger	Checked		
	Spreadsheet Rev.	23-Dec-14	Reviewed		

Liquefaction Susceptibility Analysis based on CPT values (After Boulanger 2014)

Exploration Evaluated: CPT-177

Project Specific Information

Earthquake Magnitude	9	(M) - per contract
PGA	0.37	g - per contract
Depth to Groundwater	16.7-ft	Per GRI report page 10
Use moist or dry density for calculation of overburden stress?		Moist
Maximum NF Value for Interpolation of soil density	50	
Elevation of top of Soil profile	28.66	ft.
Elevation of bottom of Soil Profile	-25.64	ft.
Elevation of Installation	24.66	
Bottom of Tank Elevation	31.75	Per Drawing 0300-FD-004 REVJJ
Thickness of soil profile to analyze	54.3-ft	
Density of Stone	125	pcf
Poisson's Ratio for Soil, v_s	0.333	
Poisson's Ratio for Stone Column, v_c	0.35	
Critical Facility (Hospital/Life line Br.)?	NO	
Load from embankment cut/fill (negative for cut)	401.70-psf	

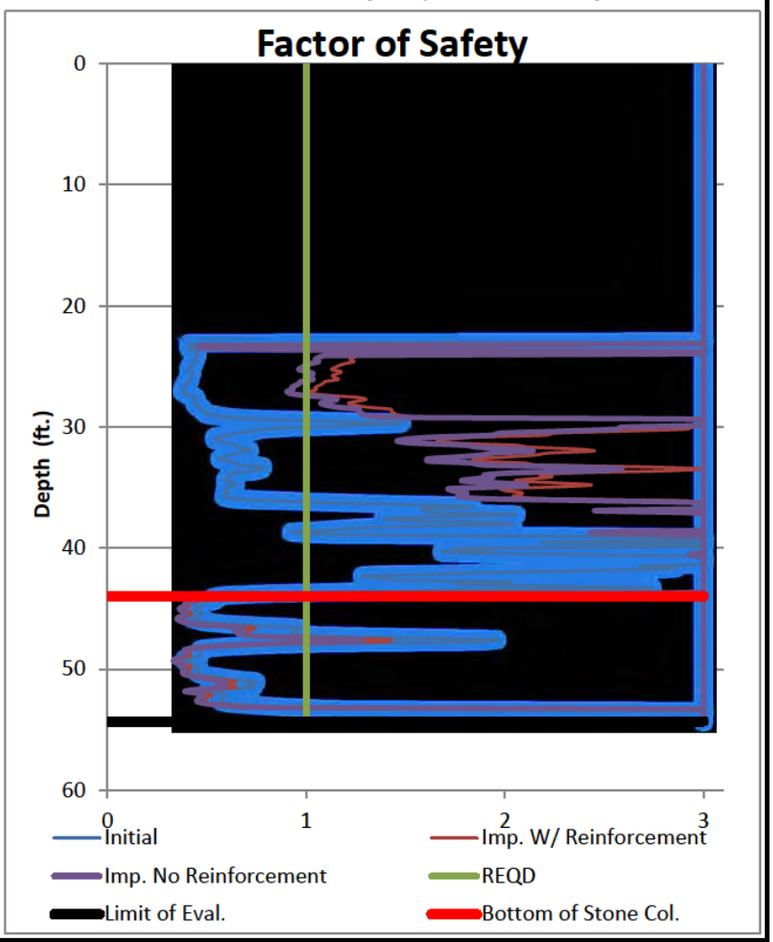
Stone Column Design Information

L =	44-ft		ϕ_c = 50
Column Spacing	7.7-ft		Ec = 750.0-tsf
Kac =	0.1324743	$=TAN^2(45-(\phi_c/2)$	Gc = 277.8-tsf

SANDY Soils 12% Replacment		SILTY soils: 12% Replacment	
SAND: Ar = (Ac/A)	0.12	SILTY SOIL: Ar = (Ac/A)	0.12
Stone Column Diameter	3-ft	Stone Column Diameter	3-ft

Consider Soil Reinforcment Via: **Rayamajhi's Rrd, (ASCE-2013)**

Factor of Safety Required for the Project: 1



Amount of Seismic Caused Settlement

Thickness of profile 54.3-ft

Minimum Sett. occurs at F.S.= 1.0

Maximum Sett. occurs at F.S.= 1.3

Without Ground Improvement:

Evaluated Thickness of Liquefiable Layer: 21.5-ft.

Calculated Min: 4.32-in via T&S, Min never < 0.5 in.

Calculated Max: 4.45-in via T&S

With Ground Improvement (NO Reinforcement)

Thickness of Liquefiable Layer: 11.0-ft.

Calculated Min: 2.38-in via T&S, Min never < 0.5 in.

Calculated Max: 2.62-in via T&S

With Ground Improvement (w/ Reinforcement Method)

Thickness of Liquefiable Layer: 9.0-ft.

Calculated Min: 1.93-in via T&S, Min never < 0.5 in.

Calculated Max: 2.21-in via T&S

Special Notes / Comments

Notes:

1-To simplify presentation of results if the calculated factor of safety is 3.0 or greater it is plotted as 3.0.

Elastic Settlement: Tesoro Savage Petroleum Terminal LLC Pipeline — IMPROVED CASE: CPT-177

Analysis method after Schmertman & Hartman (1978) as presented on page 247 of Principals of Foundation Engineering, 4th edition, Braja M. Das, 1999

$S_e = C_1 C_2 (q_{bar} - q) \Sigma[(I_z/E_s)\Delta z]$

C_1 = a correction factor for the depth of foundation embedment

= $1 - 0.5*(q/(q_{bar}-q))$

C_2 = a correction factor top account for creep ion soil

= 1+0.2log(time in years/0.1)

q_{bar} = stress at the level of the foundation

$q = \gamma Df$

Stone Column Length 44-ft

Reduction factor on Priebe: 1

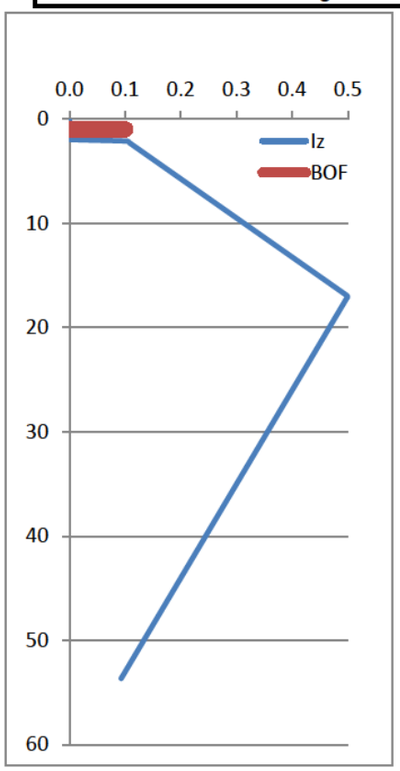
Foundation Type: Flexible

Parameter	Value	Source
Poisson's Ratio for Soil, vs	0.333	Assigned by Designer
q_{bar} (Bearing Pressure)	1000.00-psf	
L	30.0-ft	
B	30.0-ft	
fill density	130-pcf	Assigned by Designer
Elevation of Top of Soil Profile	28.66	Per Site Survey
Elevation of bottom of footing	31.75	Per Drawing 0300-FD-004 REVJJ

Correlation value between Es and q_c: $Es = X \cdot q_c$

x = 3.0

*default Value of X is 3



Note: 'Elastic Settlement" is not truly "Elastic" in the strict definition of the word. The term is used because of the "elastic" mathematical model used.

Calculated Values

Parameter	Value	Source
Footing Depth	2.0-ft	
q	260.00-psf	= γ_{FILL} x (footing depth)
L/B	1.0	=L/B ratio
t	50.0-yrs.	=settlement time
C_1	0.824	$=1-0.5[q/(q_{bar}-q)]$
C_2	1.540	= 1+0.2log(t/0.1)

I_z - Strain Influence factor (where z is depth below footing)

For square or circular footings (L/B=1.0)			
I_z = 0.1	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=0.5B$=	15.0
I_z = 0	at depth	$z=z_2=2.0B$=	60.0
For foundations with L/B ≥ 10			
I_z = 0.2	at depth	z=0=	0.0
I_z = 0.5	at depth	$z=z_1=1.0B$=	30.0
I_z = 0	at depth	$z=z_2=4.0B$=	120.0
Interpolation for L/B values between 1 and 10			
I_z = 0.10	0	at z=	0.0
I_z = 0.5	0.50	at $z=z_1$=	15.0
I_z = 0	2.00	at $z=z_2$=	60.0

$\Sigma(Iz/Es)\Delta z$ (ft2/lb)= 2.64E-05

Total Elastic Settlement (Flexible foundation)

$S_e = C_1 C_2 (q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0248-ft 0.30-in

Total Rapid Elastic Settlement

$S_e = C_1 (q_{bar}-q)\Sigma[(I_z/E_s)\Delta z]$= 0.0161-ft 0.19-in

Long Term Settlement 0.11-in

Calculated Consolidation Type Settlement:

Total:	0.12-in	Adjusted for footing Type
Recomp.	0.01-in	Adjusted for footing Type
Virgin Sett.	0.10-in	Adjusted for footing Type
Rapid Sett (within SC Zone)	0.13-in	Adjusted for footing Type
Adjustment for " c - ϕ " soils:	0.10-in	

Summary of Settlement:

Total: 0.52-in

Rapid: 0.42-in

Long Term: 0.10-in

Appendix F

Area 400 Design Calculations

DRAFT

Appendix F
Area 400 Calculations

Liquifaction Analysis
Pipe-rack
(pre- and post-improvment)

HAYWARD BAKER
KELLER

POV Tesoro CPT 191
(B&I 2014)

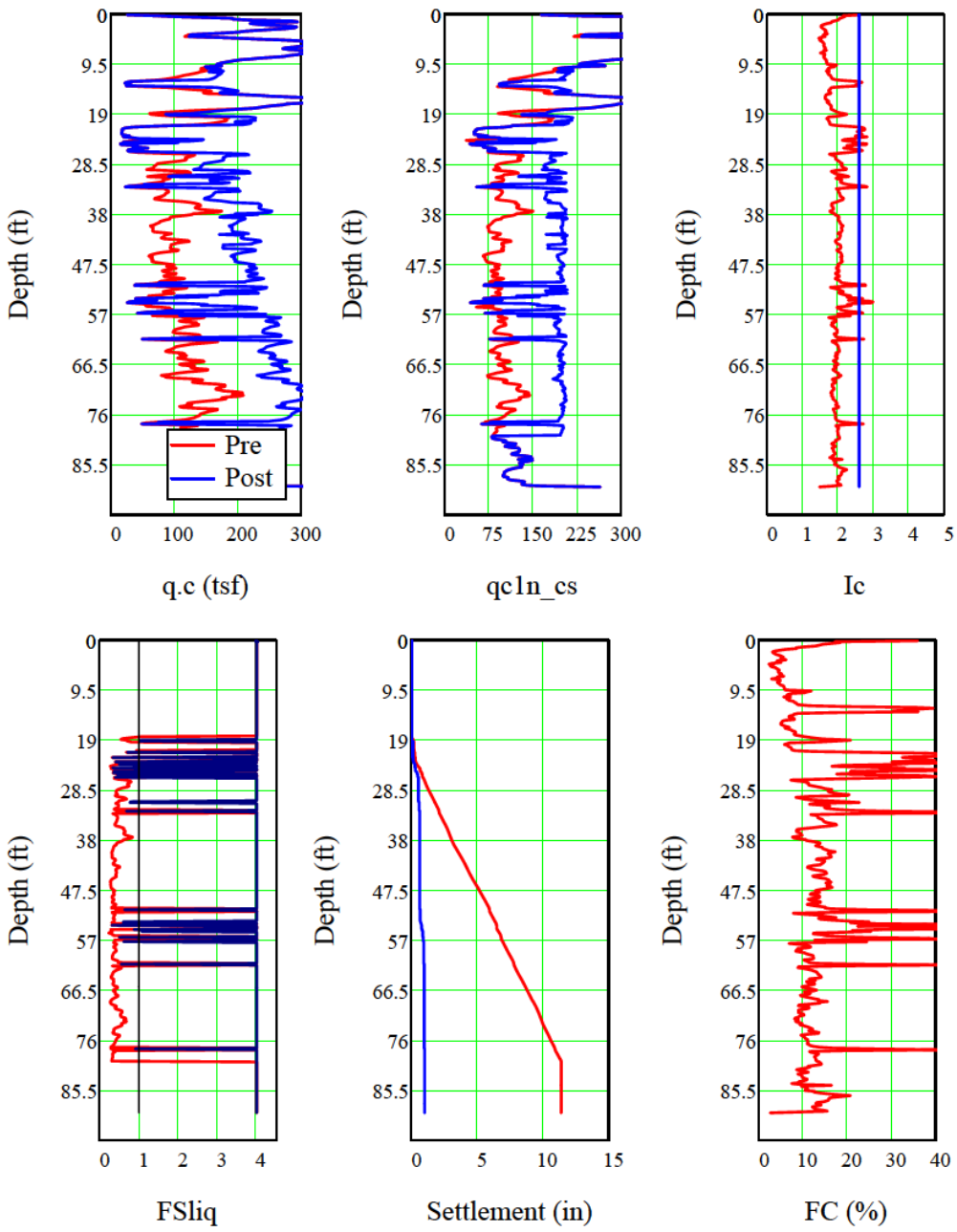
By:TVN
2/24/2015

$M_w = 9$ | Pre_Treatment_LQ_Induced_Settlement = 11.4·in | TYPE = "Rectangular Grid"

$PGA := a_{max} = 0.37 \cdot g$ | Post_Treatment_LQ_Induced_Settlement = 1·in | Spacing = "8 feet spacing"

$DESIGN\_GW := z_{wliq} = 17 \cdot ft$ | $IN\_SITU\_GW := z_{wnow} = 17 \cdot ft$ | MT = "Tokimatsu&Seed"

DOT = 80 ft

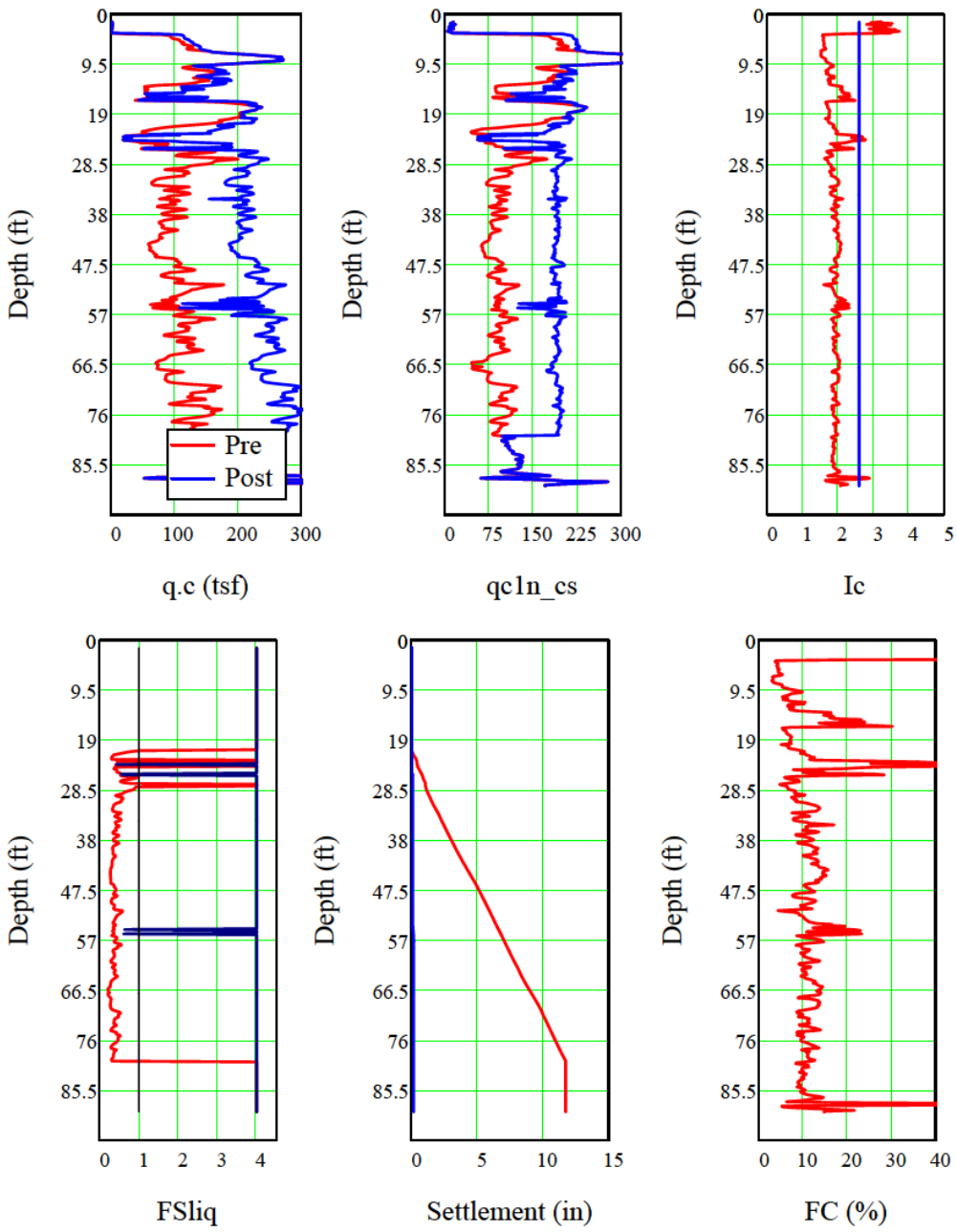


$M_w = 9$ Pre_Treatment_LQ_Induced_Settlement = 11.7·in TYPE = "Rectangular Grid"

$PGA := a_{max} = 0.37 \cdot g$ Post_Treatment_LQ_Induced_Settlement = 0.2·in Spacing = "8 feet spacing"

$DESIGN\_GW := z_{wliq} = 17 \cdot ft$ $IN\_SITU\_GW := z_{wnow} = 17 \cdot ft$ MT = "Tokimatsu&Seed"

DOT = 80 ft



HAYWARD BAKER
KELLER

POV Tesoro CPT 193
(B&I 2014)

By:TVN
2/24/2015

$M_w = 9$

Pre_Treatment_LQ_Induced_Settlement = 10.6·in

TYPE = "Rectangular Grid"

$PGA := a_{max} = 0.37 \cdot g$

Post_Treatment_LQ_Induced_Settlement = 0.3·in

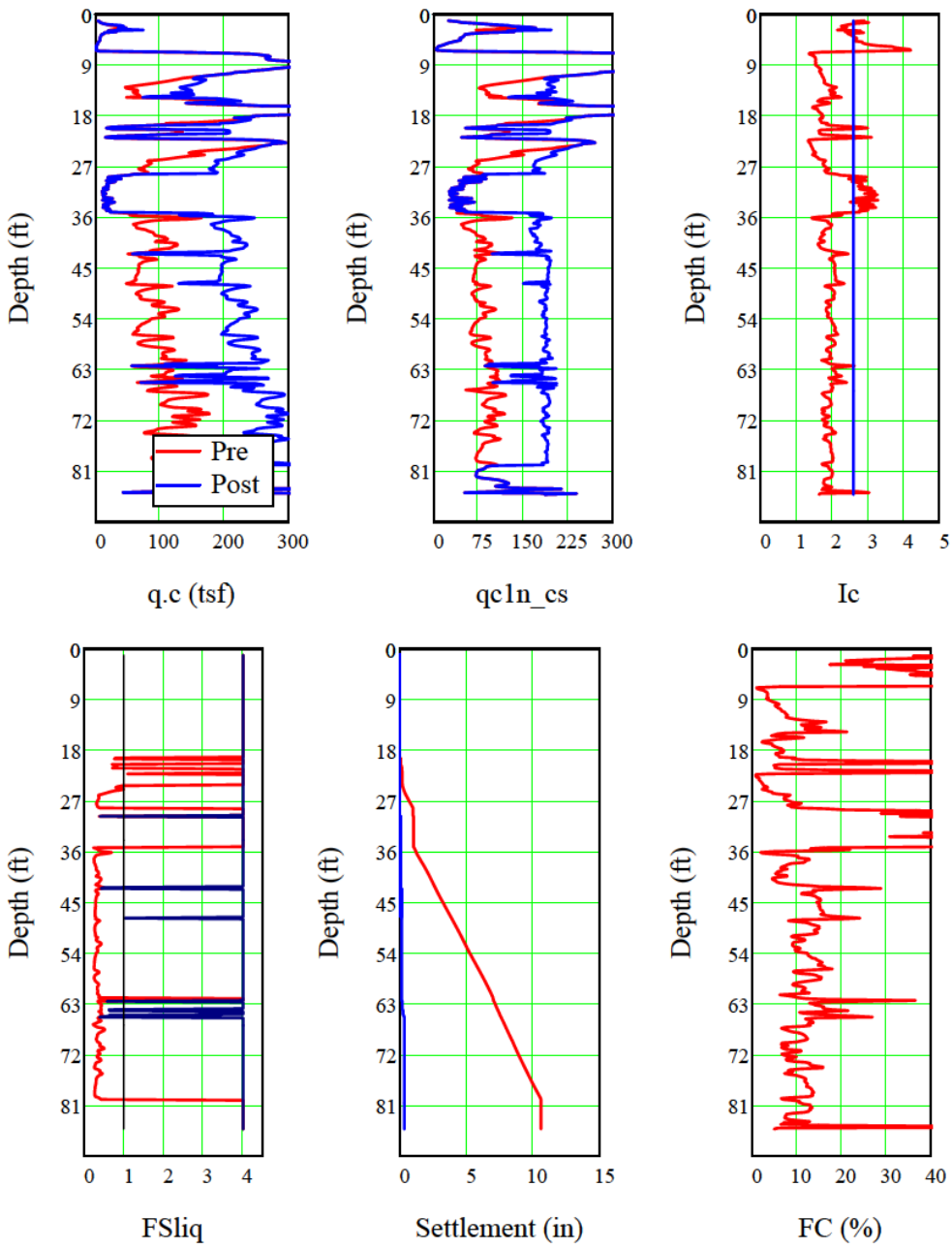
Spacing = "8 feet spacing"

$DESIGN\_GW := z_{wliq} = 17 \cdot ft$

$IN\_SITU\_GW := z_{wnow} = 17 \cdot ft$

MT = "Tokimatsu&Seed"

DOT = 80 ft

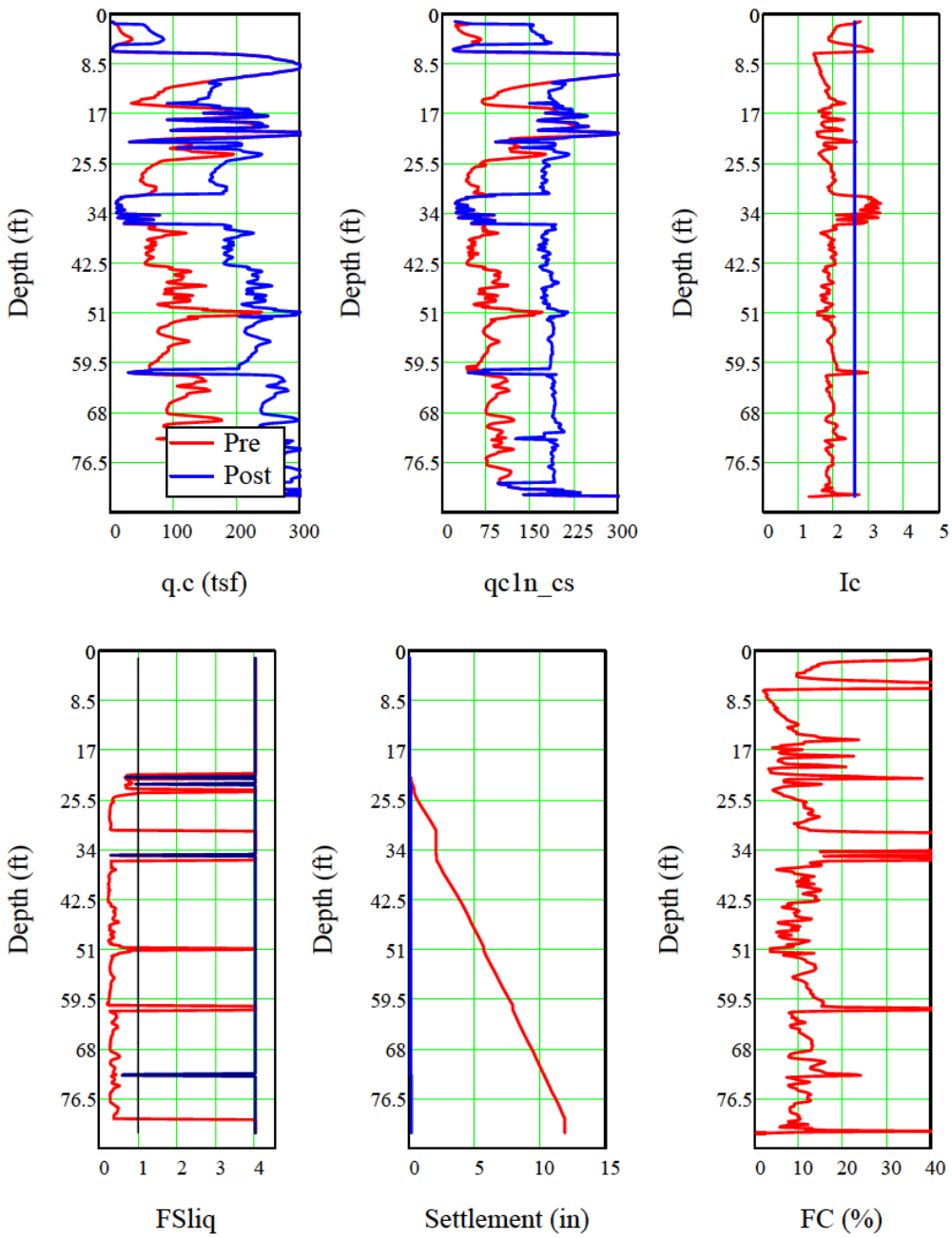


$M_w = 9$ Pre_Treatment_LQ_Induced_Settlement = 11.9·in TYPE = "Rectangular Grid"

$PGA := a_{max} = 0.37 \cdot g$ Post_Treatment_LQ_Induced_Settlement = 0.2·in Spacing = "8 feet spacing"

$DESIGN\_GW := z_{wliq} = 17 \cdot ft$ $IN\_SITU\_GW := z_{wnow} = 17 \cdot ft$ MT = "Tokimatsu&Seed"

DOT = 80 ft



HAYWARD BAKER
KELLER

POV Tesoro CPT 195
(B&I 2014)

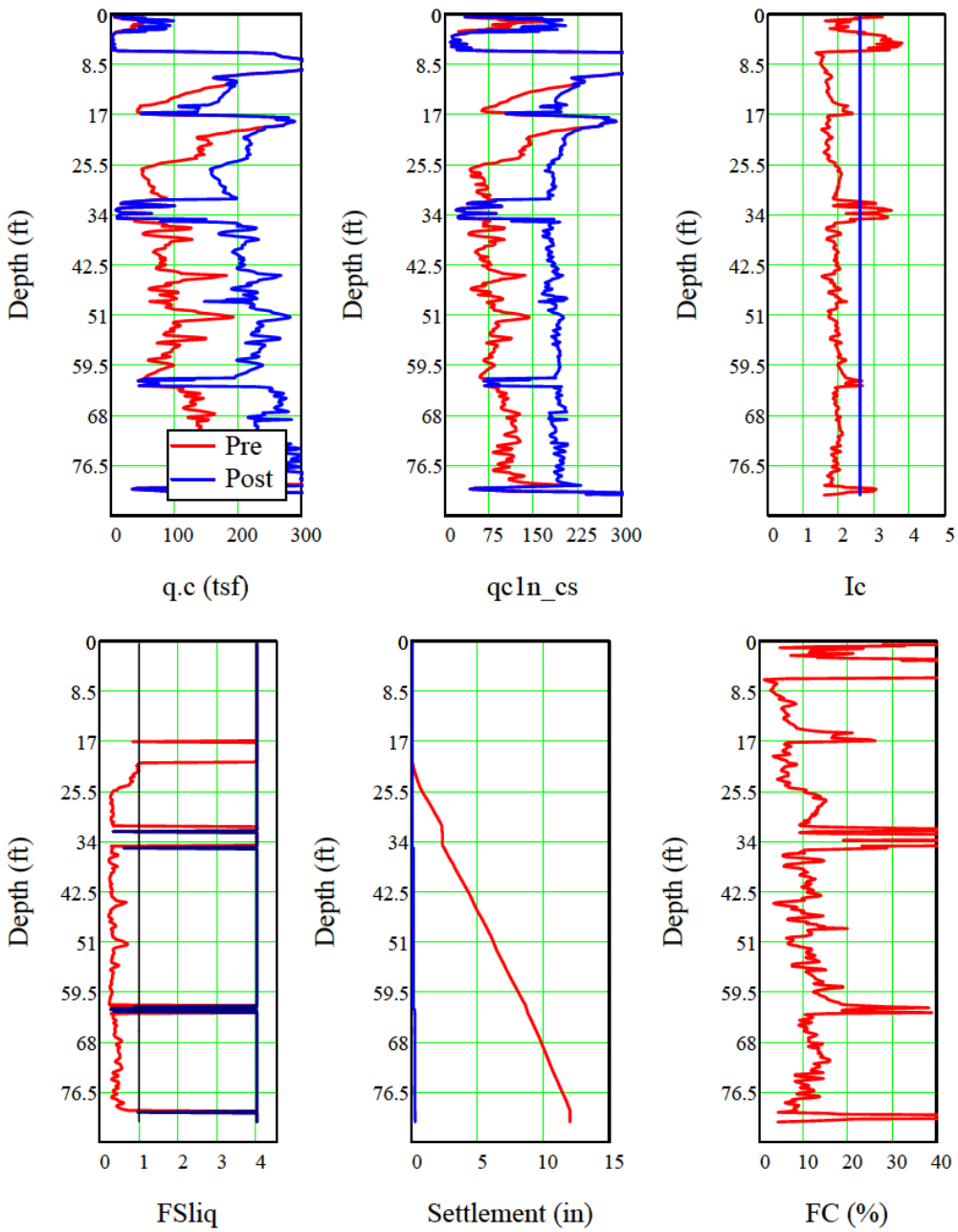
By:TVN
2/24/2015

$M_w = 9$ | Pre_Treatment_LQ_Induced_Settlement = 12·in | TYPE = "Rectangular Grid"

$PGA := a_{max} = 0.37 \cdot g$ | Post_Treatment_LQ_Induced_Settlement = 0.3·in | Spacing = "8 feet spacing"

$DESIGN\_GW := z_{wliq} = 17 \cdot ft$ | $IN\_SITU\_GW := z_{wnow} = 17 \cdot ft$ | MT = "Tokimatsu&Seed"

DOT = 80 ft



POV Tesoro CPT 196
(B&I 2014)

By:TVN
2/24/2015

$M_w = 9$

$PGA := a_{max} = 0.37 \cdot g$

$DESIGN\_GW := z_{wliq} = 17 \cdot ft$

$IN\_SITU\_GW := z_{wnow} = 17 \cdot ft$

$Pre\_Treatment\_LQ\_Induced\_Settlement = 10.8 \cdot in$

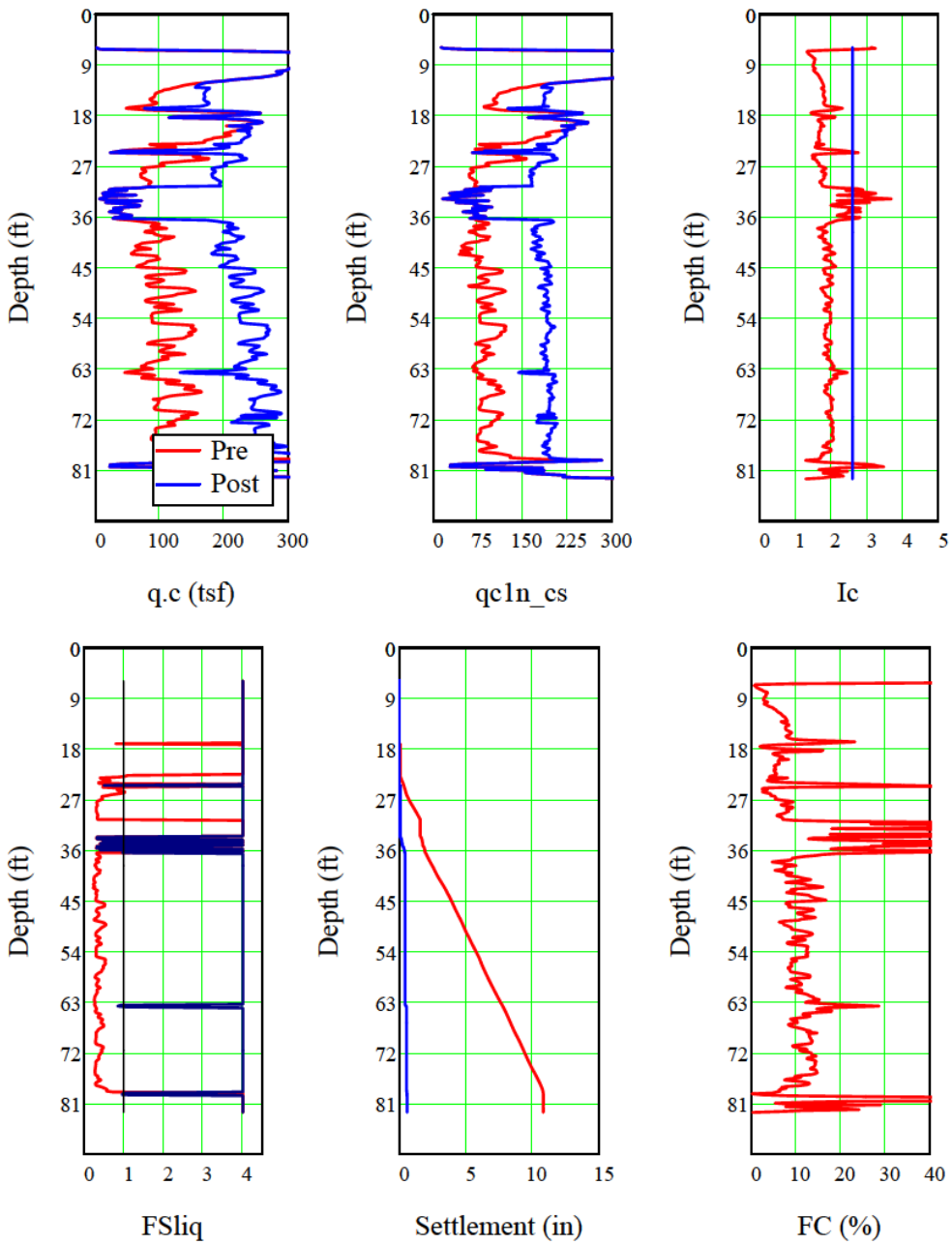
$Post\_Treatment\_LQ\_Induced\_Settlement = 0.6 \cdot in$

$DOT = 80\,ft$

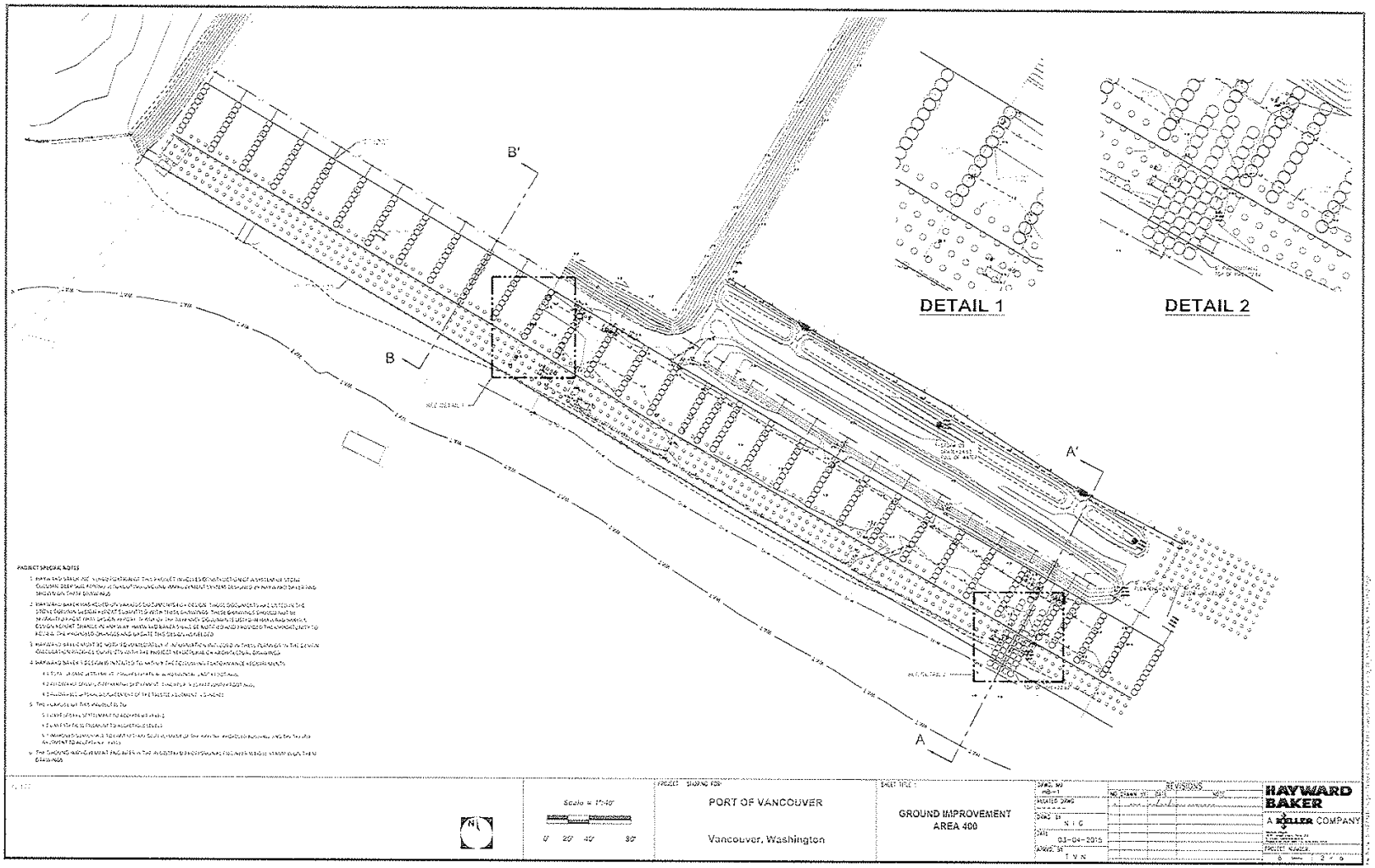
TYPE = "Rectangular Grid"

Spacing = "8 feet spacing"

MT = "Tokimatsu&Seed"

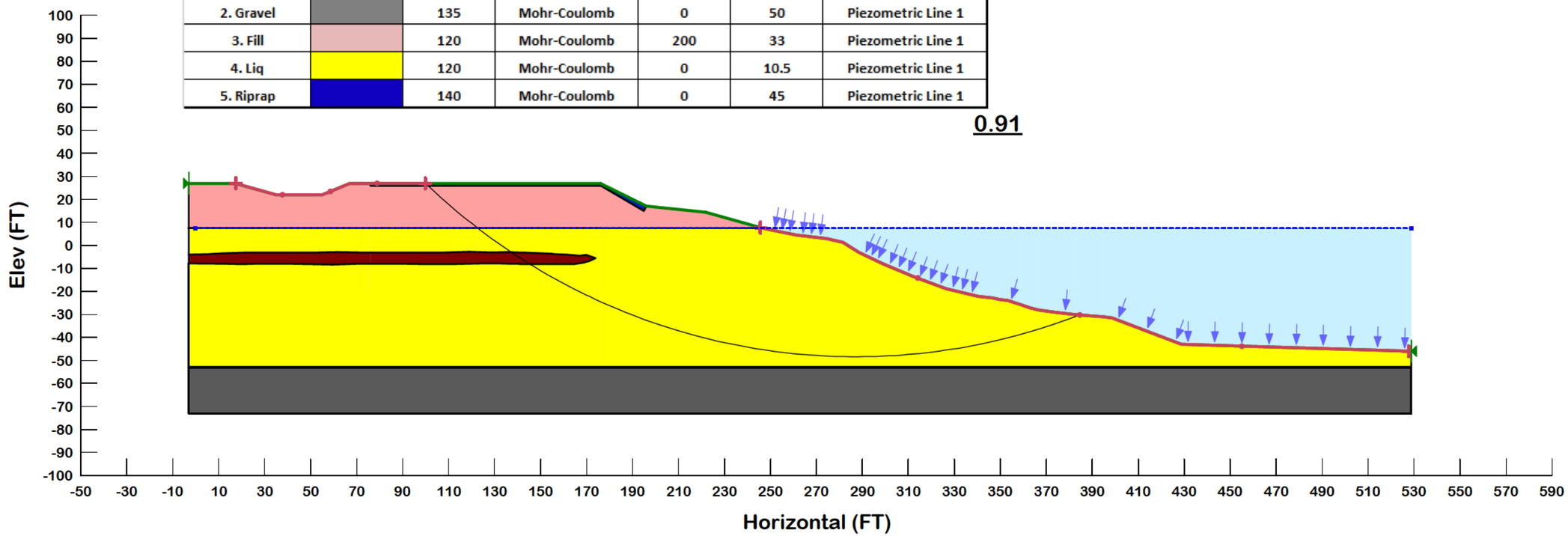


Lateral Spreading Analysis


B'
B
A'
A
DETAIL 1
DETAIL 2
PROJECT SPECIAL NOTES
Scale = 1"=40'
0' 20' 40' 80'
PROJECT STAMPING FOR:
PORT OF VANCOUVER
Vancouver, Washington
SHEET TITLE:
GROUND IMPROVEMENT
AREA 400
DRAWN BY
N I C
DATE
03-04-2015
REVISIONS
HAYWARD BAKER
A KELLER COMPANY

POV SHORELINE LIQUEFIED SLOPE CONDITION - WITHOUT GROUND IMPROVEMENT
ky=0.0

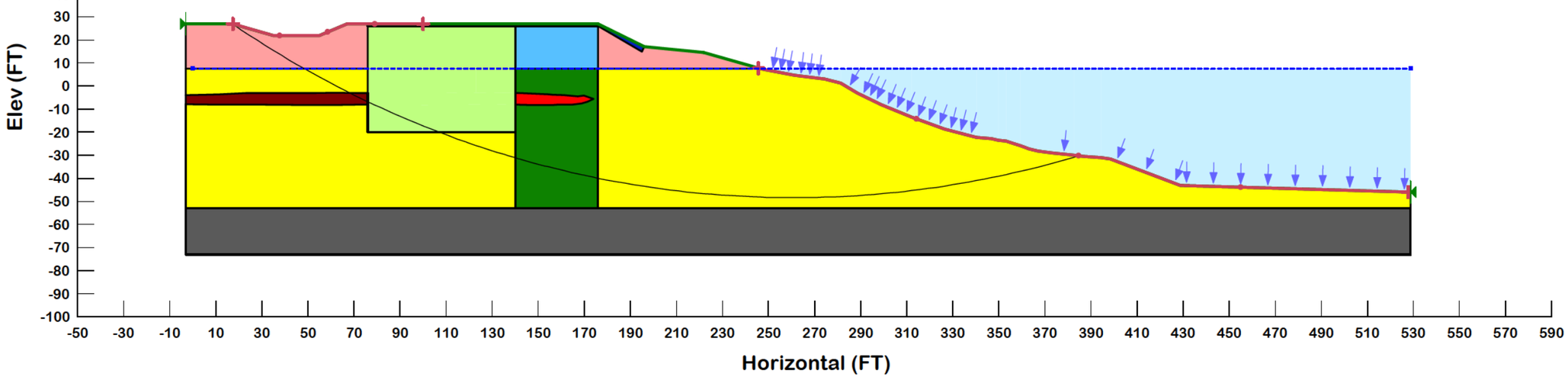
Material Name	Color	Unit Weight [pcf]	Strength Type	Cohesion [psf]	Phi [deg]	Water Surface
1. Silt_1		115	Mohr-Coulomb	300	25	Piezometric Line 1
2. Gravel		135	Mohr-Coulomb	0	50	Piezometric Line 1
3. Fill		120	Mohr-Coulomb	200	33	Piezometric Line 1
4. Liq		120	Mohr-Coulomb	0	10.5	Piezometric Line 1
5. Riprap		140	Mohr-Coulomb	0	45	Piezometric Line 1



POV SHORELINE LIQUEFIED SLOPE CONDITION - WITH GROUND IMPROVEMENT
PSEUDO-STATIC (ky=0.140)

Material Name	Color	Unit Weight [pcf]	Strength Type	Cohesion [psf]	Phi [deg]	Water Surface
1. Silt_1		115	Mohr-Coulomb	300	25	Piezometric Line 1
2. Gravel		135	Mohr-Coulomb	0	50	Piezometric Line 1
3. Fill		120	Mohr-Coulomb	200	33	Piezometric Line 1
4. Liq		120	Mohr-Coulomb	0	10.5	Piezometric Line 1
5. Riprap		140	Mohr-Coulomb	0	45	Piezometric Line 1
6. 20% DSM		100	Mohr-Coulomb	2500	0	Piezometric Line 1
7. Upper SC		120	Mohr-Coulomb	0	39	Piezometric Line 1
8. Lower SC		120	Mohr-Coulomb	0	39	Piezometric Line 1
9. Silt 2		115	Mohr-Coulomb	300	33	Piezometric Line 1

1.0



Pseudo_Static_Cir_liq_DSM_pipeline

Report generated using GeoStudio 2012. Copyright © 1991-2013 GEO-SLOPE International Ltd.

File Information

Title: POV Lateral Spreading GI Program
Created By: Nguyen, Thang
Last Edited By: Rohrbach, Mark
Revision Number: 648
File Version: 8.2
Tool Version: 8.12.3.7901
Date: 3/12/2015
Time: 12:57:26 PM
File Name: Base_Slope_GI_Sensitivity_Below_Riprap.gsz
Directory: C:\Users\MARohrbach\Documents\HBI Projects\Port of Vancouver TESORO Terminal\Area 400\
Last Solved Date: 3/12/2015
Last Solved Time: 12:57:29 PM

Project Settings

Length(L) Units: feet
Time(t) Units: Seconds
Force(F) Units: lbf
Pressure(p) Units: psf
Strength Units: psf
Unit Weight of Water: 62.4 pcf
View: 2D
Element Thickness: 1

Analysis Settings

Pseudo_Static_Cir_liq_DSM_pipeline

Kind: SLOPE/W
Parent: POV Existing
Method: Morgenstern-Price
Settings
- Side Function
  - Interslice force function option: Half-Sine
- Lambda
  - Lambda 1: -1
  - Lambda 2: -0.8
  - Lambda 3: -0.6
  - Lambda 4: -0.4
  - Lambda 5: -0.2
  - Lambda 6: 0
  - Lambda 7: 0.2
  - Lambda 8: 0.4
  - Lambda 9: 0.6
  - Lambda 10: 0.8
  - Lambda 11: 1
- PWP Conditions Source: Piezometric Line
- Apply Phreatic Correction: No
- Use Staged Rapid Drawdown: No

Slip Surface

Direction of movement: Left to Right
Use Passive Mode: No
Slip Surface Option: Entry and Exit
Critical slip surfaces saved: 1
Optimize Critical Slip Surface Location: No
Tension Crack
Tension Crack Option: (none)
F of S Distribution
F of S Calculation Option: Constant
Advanced
Number of Slices: 100
F of S Tolerance: 0.01
Minimum Slip Surface Depth: 0.1 ft
Optimization Maximum Iterations: 2,000
Optimization Convergence Tolerance: 1e-007
Starting Optimization Points: 8
Ending Optimization Points: 16
Complete Passes per Insertion: 1
Driving Side Maximum Convex Angle: 5 °
Resisting Side Maximum Convex Angle: 1 °

Materials

Interbeded Silt (ML/SM/SC)

Model: Mohr-Coulomb
Unit Weight: 115 pcf
Cohesion': 300 psf
Phi': 22 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

GW/GP

Model: Mohr-Coulomb
Unit Weight: 135 pcf
Cohesion': 0 psf
Phi': 50 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Unsaturated Top Soil (SM/SC)

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion': 200 psf
Phi': 33 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Liq_Sand

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion': 0 psf
Phi': 10.5 °
Phi-B: 0 °
Pore Water Pressure

Piezometric Line: 1

RipRap

Model: Mohr-Coulomb
Unit Weight: 140 pcf
Cohesion': 0 psf
Phi': 45 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Asphalt

Model: Mohr-Coulomb
Unit Weight: 130 pcf
Cohesion': 3,000 psf
Phi': 0 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Lower SC

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion': 0 psf
Phi': 41 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Upper SC

Model: Mohr-Coulomb
Unit Weight: 120 pcf
Cohesion': 0 psf
Phi': 41 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Improved Interbeded Silt (ML/SM/SC)

Model: Mohr-Coulomb
Unit Weight: 115 pcf
Cohesion': 300 psf
Phi': 33 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

CDSM

Model: Mohr-Coulomb
Unit Weight: 100 pcf
Cohesion': 2,300 psf
Phi': 0 °
Phi-B: 0 °
Pore Water Pressure
Piezometric Line: 1

Slip Surface Entry and Exit

Left Projection: Range
Left-Zone Left Coordinate: (17.3414, 26.90517) ft
Left-Zone Right Coordinate: (100, 27) ft
Left-Zone Increment: 4
Right Projection: Range
Right-Zone Left Coordinate: (245.6837, 7.73134) ft
Right-Zone Right Coordinate: (527.646, -45.96538) ft
Right-Zone Increment: 4
Radius Increments: 30

Slip Surface Limits

Left Coordinate: (-3, 27) ft
Right Coordinate: (528.8, -46) ft

Piezometric Lines

Piezometric Line 1

Coordinates

	X (ft)	Y (ft)
Coordinate 1	0	7.5
Coordinate 2	528.8	7.5

Seismic Coefficients

Horz Seismic Coef.: 0.135
Ignore seismic load in strength: No

Points

	X (ft)	Y (ft)
Point 1	-3	27
Point 2	17	27
Point 3	35	22
Point 4	55	22
Point 5	76	27
Point 6	76	26
Point 7	176	27
Point 8	176	26
Point 9	196	17
Point 10	195	15
Point 11	221.8	14.5
Point 12	246.5	7.5
Point 13	249	7
Point 14	261.2	4.6
Point 15	274.4	3
Point 16	281.4	1.3
Point 17	288.5	-3
Point 18	299	-8
Point 19	312.1	-13.5

Point 20	326.4	-18.7
Point 21	340.3	-22.2
Point 22	346.7	-22.8
Point 23	349.8	-23.5
Point 24	353.2	-23.9
Point 25	355.9	-24.7
Point 26	359.2	-25.9
Point 27	363.3	-27.2
Point 28	367	-28.2
Point 29	373.7	-29
Point 30	382.6	-30
Point 31	386.9	-30.5
Point 32	394.4	-31
Point 33	398.5	-31.5
Point 34	4,708.6	-32.9
Point 35	428.8	-43
Point 36	528.8	-46
Point 37	528.8	-53
Point 38	528.8	-73
Point 39	-3	-73
Point 40	-3	-53
Point 41	-3	-3.9
Point 42	-3	-7.8
Point 43	5.6	-3.8
Point 44	11.5	-3.5
Point 45	11.9	-8
Point 46	20	-8.1
	23	-3

Point 47		
Point 48	26.4	-8
Point 49	33.7	-3.1
Point 50	39.8	-8
Point 51	44.4	-3.1
Point 52	50.2	-8.2
Point 53	53.3	-3
Point 54	59.7	-8.3
Point 55	61.8	-2.9
Point 56	70.7	-3
Point 57	70.9	-8
Point 58	77.5	-3
Point 59	87.8	-3
Point 60	89.6	-8
Point 61	101.2	-3.1
Point 62	100.7	-8.2
Point 63	108.8	-3
Point 64	110.8	-8.1
Point 65	119	-2.7
Point 66	118.7	-8
Point 67	125	-7.8
Point 68	129.1	-3
Point 69	131.9	-7.8
Point 70	135.3	-2.9
Point 71	136.8	-8
Point 72	141.9	-3
Point 73	141.5	-8.1
	147.3	-8.2

Point 74		
Point 75	152.7	-8.2
Point 76	154.6	-3.6
Point 77	158.2	-8.1
Point 78	161.2	-4
Point 79	164.4	-8.1
Point 80	167.1	-4.4
Point 81	168.8	-7.5
Point 82	169.8	-4.2
Point 83	171.2	-6.8
Point 84	173.9	-5.5
Point 85	67	27
Point 86	-3	7.5
Point 87	90	-38
Point 88	176	-53
Point 89	76	-3
Point 90	76	-8
Point 91	76	7.5
Point 92	176	7.5
Point 93	123	26
Point 94	123	7.5
Point 95	123	-20
Point 96	140	26
Point 97	140	7.5
Point 98	140	-53
Point 99	76	-20
Point 100	90	-53
	156	-3.8

Point 101		
Point 102	156	-8.2
Point 103	176	-73
Point 104	123	-2.8
Point 105	140	-20
Point 106	140	-3
Point 107	140	-8

Regions

	Material	Points	Area (ft²)
Region 1	Unsaturated Top Soil (SM/SC)	86,1,2,3,4,85,5,6,91	1,365.5
Region 2	Asphalt	5,7,8,96,93,6	100
Region 3	CDSM	6,91,94,93	869.5
Region 4	CDSM	93,94,97,96	314.5
Region 5	Upper SC	96,97,92,8	666
Region 6	RipRap	8,10,9,7	34.5
Region 7	Liq_Sand	86,41,43,44,47,49,51,53,55,56,89,91	844.7
Region 8	Interbeded Silt (ML/SM/SC)	41,42,45,46,48,50,52,54,57,90,89,56,55,53,51,49,47,44,43	384.13
Region 9	Unsaturated Top Soil (SM/SC)	8,92,12,11,9,10	554.8
Region 10	CDSM	94,91,89,58,59,61,63,65,104	492.02
Region 11	CDSM	97,94,104,68,70,106	177.34
Region 12	CDSM	107,71,69,67,66,64,62,60,90,99,95,105	767.48
Region 13	Lower SC	106,97,92,88,98,105,107,73,74,75,102,77,79,81,83,84,82,80,78,101,76,72	2,040.4
Region 14	Improved Interbeded Silt (ML/SM/SC)	106,72,76,101,78,80,82,84,83,81,79,77,102,75,74,73,107	137.57
Region 15	CDSM	89,90,60,62,64,66,67,69,71,107,106,70,68,104,65,63,61,59,58	323.15
	GW/GP	40,100,98,88,103,39	3,580

Region 16			
Region 17	GW/GP	88,37,38,103	7,056
Region 18	Liq_Sand	42,45,46,48,50,52,54,57,90,99,95,105,98,100,40	5,662.7
Region 19	Liq_Sand	12,13,14,15,16,17,18,19,20,21,22,23,24,25,26,27,28,29,30,31,32,33,35,36,37,88,92	11,484

Current Slip Surface

Slip Surface: 93
F of S: 0.97
Volume: 14,632.278 ft³
Weight: 1,701,873.3 lbs
Resisting Moment: 1.5833692e+008 lbs-ft
Activating Moment: 1.6401366e+008 lbs-ft
Resisting Force: 350,530.93 lbs
Activating Force: 363,211.26 lbs
F of S Rank: 1
Exit: (384.61691, -30.234525) ft
Entry: (17.341399, 26.905167) ft
Radius: 431.49681 ft
Center: (260.84436, 383.12954) ft

Slip Slices

	X (ft)	Y (ft)	PWP (psf)	Base Normal Stress (psf)	Frictional Strength (psf)	Cohesive Strength (psf)
Slice 1	19.107259	25.710842	-1,136.3566	-36.659595	-23.807019	200
Slice 2	22.638979	23.347373	-988.8761	82.654981	53.676773	200
Slice 3	26.1707	21.033778	-844.50774	198.90846	129.17267	200
Slice 4	29.70242	18.769106	-703.19219	312.19659	202.74284	200
Slice 5	33.23414	16.55245	-564.87286	422.61052	274.44648	200
Slice 6	36.686965	14.430394	-432.45658	567.59049	368.59758	200
Slice 7	40.060895	12.400121	-305.76753	747.94838	485.72336	200
Slice 8	43.434826	10.411448	-181.67436	926.77934	601.85754	200
Slice 9	46.808756	8.4637137	-60.135736	1,104.2022	717.07732	200
Slice 10	50.121791	6.5900037	56.783768	1,604.4892	286.85024	0
Slice 11	53.37393	4.7883219	169.20871	1,788.1265	300.04868	0
Slice 12	57	2.8246316	291.74299	2,071.6757	329.89102	0
Slice 13	61	0.70747594	423.8535	2,453.3063	376.13685	0
Slice 14	65	-1.3564083	552.63988	2,826.4278	421.42167	0
Slice 15	67.595597	-2.6734765	634.82493	3,039.6561	445.70912	0
Slice 16	69.445597	-3.5926474	692.1812	2,891.773	888.69279	300
Slice 17	73.35	-5.4920027	810.70097	3,085.9862	919.27487	300
Slice 18	76.75	-7.124319	912.55751	2,395.887	0	2,300

Slice 19	78.057995	-7.7390582	950.91723	2,467.2742	0	2,300
Slice 20	80.146659	-8.7058785	1,011.2468	2,579.2164	0	2,300
Slice 21	83.207995	-10.103211	1,098.4404	2,740.3397	0	2,300
Slice 22	86.269332	-11.471846	1,183.8432	2,897.4953	0	2,300
Slice 23	88.7	-12.54059	1,250.5328	3,019.8231	0	2,300
Slice 24	91.45	-13.721818	1,324.2414	3,154.7505	0	2,300
Slice 25	95.15	-15.280877	1,421.5267	3,332.4385	0	2,300
Slice 26	98.85	-16.799627	1,516.2967	3,505.3027	0	2,300
Slice 27	100.95	-17.648724	1,569.2804	3,601.8678	0	2,300
Slice 28	102.64351	-18.317319	1,611.0007	3,677.9956	0	2,300
Slice 29	105.53054	-19.443087	1,681.2486	3,806.2144	0	2,300
Slice 30	107.88703	-20.346106	1,737.597	4,119.7665	441.50903	0
Slice 31	109.8	-21.065904	1,782.5124	4,189.3678	446.08429	0
Slice 32	112.775	-22.161144	1,850.8554	4,295.9277	453.16736	0
Slice 33	116.725	-23.582539	1,939.5504	4,435.6162	462.61844	0
Slice 34	118.85	-24.334657	1,986.4826	4,510.2477	467.75222	0
Slice 35	121	-25.072102	2,032.4992	4,584.039	472.89995	0
Slice 36	124	-26.091677	2,096.1207	4,687.0253	480.19579	0
Slice 37	127.05	-27.094685	2,158.7084	4,789.7205	487.62926	0
Slice 38	130.5	-28.206924	2,228.112	4,905.2511	496.1784	0
Slice 39	133.6	-29.175205	2,288.5328	5,007.5524	503.9405	0
Slice 40	136.05	-29.926448	2,335.4103	5,088.0579	510.17308	0
Slice 41	138.4	-30.626535	2,379.0958	5,164.2173	516.19175	0
Slice 42	140.75	-31.317357	2,422.2031	6,111.0986	3,206.7079	0
Slice 43	141.7	-31.592142	2,439.3496	6,155.5649	3,230.4566	0
Slice 44	144.6	-32.404915	2,490.0667	6,291.0195	3,304.1178	0
Slice 45	150	-33.877915	2,581.9819	6,544.4863	3,444.5525	0
	153.65	-34.839242	2,641.9687	6,716.1374	3,541.6208	0

Slice 46						
Slice 47	155.3	-35.259468	2,668.1908	6,793.511	3,586.0861	0
Slice 48	157.1	-35.708518	2,696.2115	6,877.7623	3,634.9666	0
Slice 49	159.7	-36.342667	2,735.7824	6,998.5203	3,705.5416	0
Slice 50	162.8	-37.07767	2,781.6466	7,141.9941	3,790.3923	0
Slice 51	165.75	-37.756005	2,823.9747	7,279.1247	3,872.8028	0
Slice 52	167.95	-38.248408	2,854.7007	7,381.2217	3,934.8447	0
Slice 53	169.3	-38.544343	2,873.167	7,443.4302	3,972.8691	0
Slice 54	170.5	-38.802776	2,889.2932	7,500.4445	4,008.4127	0
Slice 55	172.55	-39.234875	2,916.2562	7,600.3863	4,071.8522	0
Slice 56	174.95	-39.730373	2,947.1753	7,713.7631	4,143.5315	0
Slice 57	177.9	-40.315828	2,983.7077	7,196.3612	780.76917	0
Slice 58	181.7	-41.042512	3,029.0527	7,098.3819	754.20558	0
Slice 59	185.5	-41.733966	3,072.1995	6,997.5473	727.5202	0
Slice 60	189.3	-42.390363	3,113.1586	6,893.6903	700.68013	0
Slice 61	193.1	-43.011863	3,151.9403	6,786.6393	673.65164	0
Slice 62	195.5	-43.390511	3,175.5679	6,693.5502	652.01948	0
Slice 63	197.84286	-43.7391	3,197.3198	6,680.8325	645.63091	0
Slice 64	201.52857	-44.266846	3,230.2512	6,732.317	649.06954	0
Slice 65	205.21429	-44.762194	3,261.1609	6,781.8788	652.52649	0
Slice 66	208.9	-45.225257	3,290.056	6,829.3875	655.97632	0
Slice 67	212.58571	-45.656139	3,316.9431	6,874.7026	659.39174	0
Slice 68	216.27143	-46.054938	3,341.8281	6,917.6729	662.74367	0
Slice 69	219.95714	-46.421741	3,364.7167	6,958.1383	666.00133	0
Slice 70	223.56429	-46.750158	3,385.2099	6,956.8129	661.9575	0
Slice 71	227.09286	-47.041588	3,403.3951	6,913.0788	650.48143	0
Slice 72	230.62143	-47.303894	3,419.763	6,865.8344	638.69158	0
	234.15	-47.537131	3,434.317	6,814.9177	626.55731	0

Slice 73						
Slice 74	237.67857	-47.741345	3,447.0599	6,760.1665	614.048	0
Slice 75	241.20714	-47.916577	3,457.9944	6,701.4192	601.13326	0
Slice 76	244.73571	-48.062864	3,467.1227	6,638.516	587.783	0
Slice 77	247.75	-48.166725	3,473.6037	6,611.6668	581.60561	0
Slice 78	251.03333	-48.250921	3,478.8574	6,624.9492	583.09364	0
Slice 79	255.1	-48.324237	3,483.4324	6,637.1959	584.51551	0
Slice 80	259.16667	-48.359215	3,485.615	6,643.6763	585.31206	0
Slice 81	262.85	-48.359451	3,485.6298	6,646.021	585.74388	0
Slice 82	266.15	-48.331492	3,483.8851	6,654.7462	587.68436	0
Slice 83	269.45	-48.278288	3,480.5652	6,659.3591	589.15462	0
Slice 84	272.75	-48.19983	3,475.6694	6,659.7091	590.12688	0
Slice 85	276.15	-48.092173	3,468.9516	6,655.6397	590.61772	0
Slice 86	279.65	-47.953717	3,460.3119	6,624.1836	586.38896	0
Slice 87	283.175	-47.785391	3,449.8084	6,602.5447	584.32513	0
Slice 88	286.725	-47.586753	3,437.4134	6,501.3735	567.87144	0
Slice 89	290.25	-47.36056	3,423.2989	6,376.543	547.35145	0
Slice 90	293.75	-47.107177	3,407.4879	6,279.027	532.20831	0
Slice 91	297.25	-46.825155	3,389.8896	6,173.6682	515.94287	0
Slice 92	300.6375	-46.525317	3,371.1798	6,053.8914	497.21121	0
Slice 93	303.9125	-46.209395	3,351.4663	5,950.1177	481.63157	0
Slice 94	307.1875	-45.86824	3,330.1782	5,839.5967	465.09323	0
Slice 95	310.4625	-45.50179	3,307.3117	5,722.285	447.58885	0
Slice 96	313.8875	-45.090819	3,281.6671	5,580.3689	426.03919	0
Slice 97	317.4625	-44.632812	3,253.0874	5,448.3341	406.86492	0
Slice 98	321.0375	-44.144402	3,222.6107	5,308.6229	386.61952	0
Slice 99	324.6125	-43.625486	3,190.2303	5,161.2772	365.31195	0
	328.1375	-43.084061	3,156.4454	4,985.2829	338.95499	0

Slice 100						
Slice 101	331.6125	-42.520859	3,121.3016	4,852.8306	320.91993	0
Slice 102	335.0875	-41.928503	3,084.3386	4,714.0747	302.05373	0
Slice 103	338.5625	-41.306869	3,045.5486	4,569.1188	282.37705	0
Slice 104	341.9	-40.682711	3,006.6012	4,397.8246	257.84802	0
Slice 105	345.1	-40.058149	2,967.6285	4,289.5233	244.99871	0
Slice 106	348.25	-39.41897	2,927.7437	4,196.2839	235.11004	0
Slice 107	351.5	-38.733027	2,884.9409	4,034.636	213.0834	0
Slice 108	354.55	-38.067367	2,843.4037	3,934.0483	202.13902	0
Slice 109	357.55	-37.387579	2,800.9849	3,777.5791	181.00104	0
Slice 110	361.25	-36.517644	2,746.701	3,552.795	149.40068	0
Slice 111	365.15	-35.566284	2,687.3361	3,323.4921	117.90455	0
Slice 112	368.675	-34.673111	2,631.6021	3,122.8198	91.041822	0
Slice 113	372.025	-33.794139	2,576.7543	2,967.1014	72.346567	0
Slice 114	375.925	-32.731748	2,510.4611	2,781.3572	50.207617	0
Slice 115	380.375	-31.474558	2,432.0124	2,565.6103	24.760914	0
Slice 116	383.60846	-30.533807	2,373.3095	2,405.4602	5.9587876	0

ESTIMATION OF PERMANENT SEISMIC DISPLACEMENT USING THE BRAY AND RATHJE (1998) PROCEDURE.

Project name	Van. Energy
Project location	Port of Vancouver
Project number	NA
Date	4/14/2015
Designed by	TVN
Reviewed by	LS

HAYWARD BAKER
Geotechnical Construction

KELLER

Shoreline

INPUT PARAMETERS:

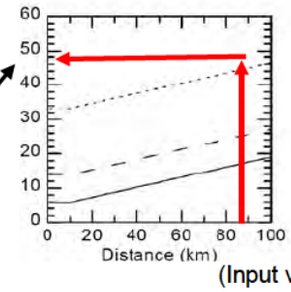
See Slope /W analysis

Yield Acceleration, ky (g):	0.14		Normalized MHEA Sigma:	0	
Vertical Thickness, h (m):	24.35	per CPTs	Mean Period Sigma:	0	
Shear Wave Vel., Vs (m/s):	155		Significant Duration Sigma:	0	
Earthquake Magnitude, M:	9.00	*	Normalized Displacement Sigma:	0	
Earthquake Accel., Rock (g):	0.37	*			
Earthquake Distance, r (km):	86	*	Allowable screen displacement (cm):	5.08	*

Per GRI Report Table 1B

CALCULATIONS:

Site Period (s):	0.628
NRF Factor:	1.023
Mean Period (s):	0.695
Duration, D05-95 (s):	50.000
Ts/Tm:	0.9048
MHEA/MHA*NRF:	0.579
MHEA, kmax:	0.2192
ky/kmax:	0.6387
Normalized Disp. (cm/sec):	0.4460



After Abrahamson and Silva maximum value for D05-95 when r=86 km is 50s.

(Input values marked with asterisks are used for calculation of seismic coefficient for screen)

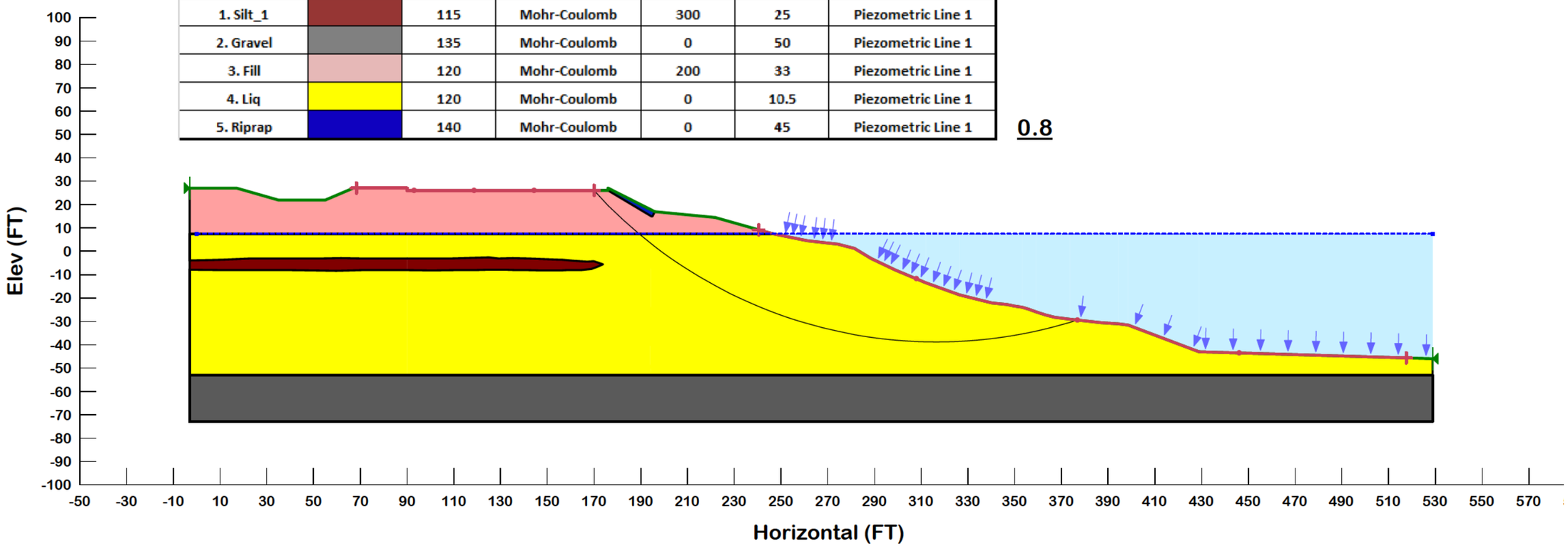
Estimated Displacement (cm):	4.888	**Median f_{eq} for Screen Procedure:**	0.718
Estimated Displacement (in):	1.924	**Seismic Coefficient for Screen Procedure:**	0.266

Abutment Area

POV SHORELINE LIQUEFIED SLOPE CONDITION-AT TRESTLE ABUTMENT
POST EARTHQUAKE (ky=0.0)

Material Name	Color	Unit Weight [pcf]	Strength Type	Cohesion [psf]	Phi [deg]	Water Surface
1. Silt_1		115	Mohr-Coulomb	300	25	Piezometric Line 1
2. Gravel		135	Mohr-Coulomb	0	50	Piezometric Line 1
3. Fill		120	Mohr-Coulomb	200	33	Piezometric Line 1
4. Liq		120	Mohr-Coulomb	0	10.5	Piezometric Line 1
5. Riprap		140	Mohr-Coulomb	0	45	Piezometric Line 1

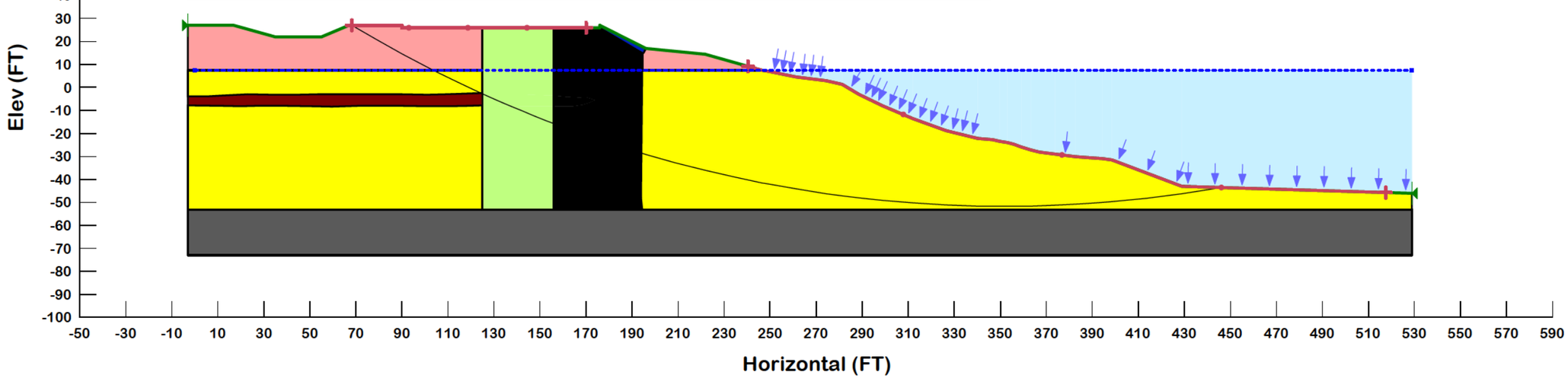
0.8



POV SHORELINE LIQUEFIED SLOPE CONDITION-AT TRESTLE ABUTMENT
PSEUDO-STATIC (ky=0.185) WITH GROUND IMPROVEMENT

Material Name	Color	Unit Weight [pcf]	Strength Type	Cohesion [psf]	Phi [deg]	Water Surface
1. Silt_1		115	Mohr-Coulomb	300	25	Piezometric Line 1
2. Gravel		135	Mohr-Coulomb	0	50	Piezometric Line 1
3. Fill		120	Mohr-Coulomb	200	33	Piezometric Line 1
4. Liq		120	Mohr-Coulomb	0	10.5	Piezometric Line 1
5. Riprap		140	Mohr-Coulomb	0	45	Piezometric Line 1
6. 40% JG		100	Mohr-Coulomb	2210	0	Piezometric Line 1
7. Full Cover JG		100	Mohr-Coulomb	5000	0	Piezometric Line 1

1.0



ESTIMATION OF PERMANENT SEISMIC DISPLACEMENT USING THE BRAY AND RATHJE (1998) PROCEDURE.

Project name	Tersoro Terminal
Project location	Port of Vancouver
Project number	NA
Date	4/16/2015
Designed by	TVN
Reviewed by	LS

HAYWARD BAKER
Geotechnical Construction

KELLER

Trestle

INPUT PARAMETERS:

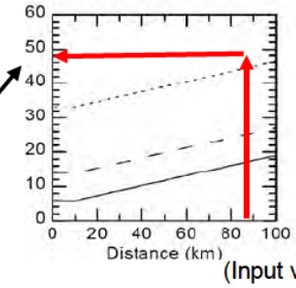
See Slope /W analysis

Yield Acceleration, ky (g):	0.185		Normalized MHEA Sigma:	0	
Vertical Thickness, h (m):	24.384	per CPTs	Mean Period Sigma:	0	
*Shear Wave Vel., Vs (m/s):	185		Significant Duration Sigma:	0	
Earthquake Magnitude, M:	9.00	*	Normalized Displacement Sigma:	0	
Earthquake Accel., Rock (g):	0.37	*			
Earthquake Distance, r (km):	86	*	Allowable screen displacement (cm):	5.08	*

Per GRI Report Table 1B

CALCULATIONS:

Site Period (s):	0.527
NRF Factor:	1.023
Mean Period (s):	0.695
Duration, D05-95 (s):	50.000
Ts/Tm:	0.7591
MHEA/MHA*NRF:	0.665
MHEA, kmax:	0.2515
ky/kmax:	0.7356
Normalized Disp. (cm/sec):	0.2053



After Abrahamson and Silva maximum value for D05-95 when r=86 km is 50s.

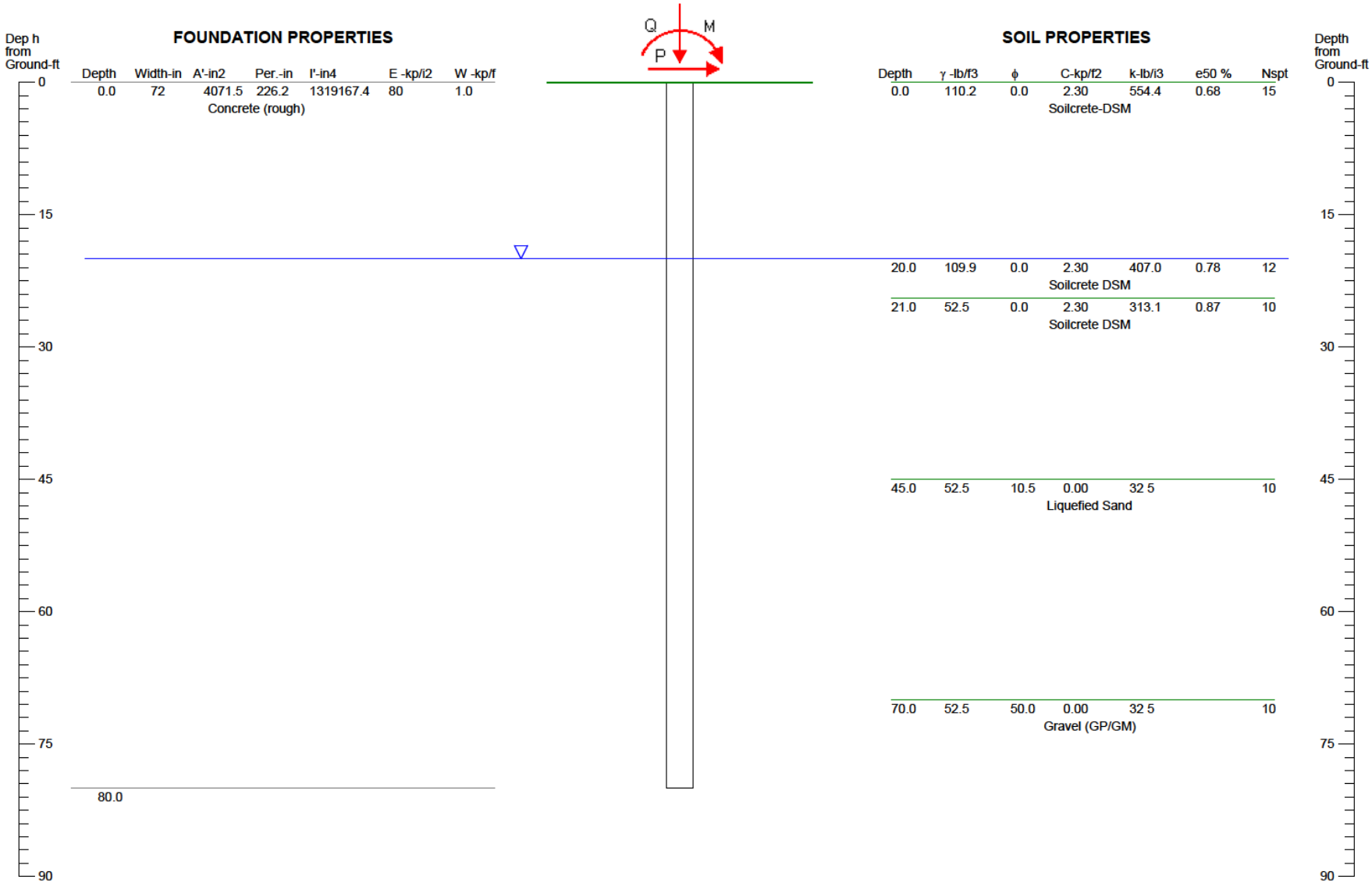
(Input values marked with asterisks are used for calculation of seismic coefficient for screen)

Estimated Displacement (cm):	2.582	**Median f_{eq} for Screen Procedure:**	0.718
Estimated Displacement (in):	1.016	**Seismic Coefficient for Screen Procedure:**	0.266

AllPile Analysis

FOUNDATION PROFILE & SOIL CONDITIONS

Diameter more than 24in (61cm).
For bell section, select "Belled"
in Diameter Varia ion of Pile
Section Screen.



Batter Angle=0 (Pile diameter not to scale) Surface Angle=0

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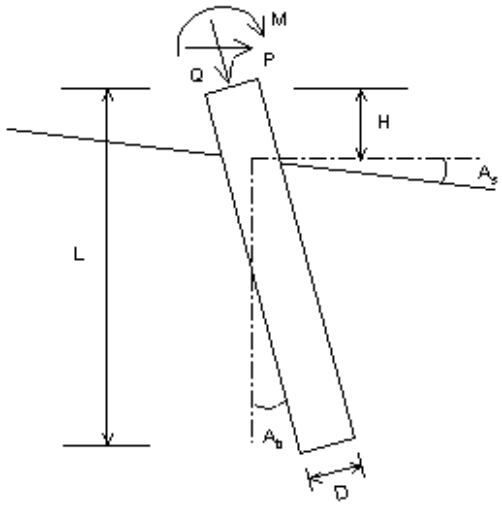
Figure 1

VERTICAL ANALYSIS

Figure 2



Drilled Shaft (dia >24 in. or 61 cm)

Loads:
Load Factor for Vertical Loads= 1.0
Load Factor for Lateral Loads= 1.0
Loads Supported by Pile Cap= 0 %
Shear Condition: Static

(with Load Factor)
Vertical Load, Q= 100.0 -kp

Profile:
Pile Length, L= 80.0 -ft
Top Height, H= 0 -ft
Slope Angle, As= 0
Batter Angle, Ab= 0

* Zero Friction *
Zero Friction Start: 0 -ft End: 20 -ft
* Negative Friction *
Negative Friction Start: 20 -ft End: 70 -ft with Factor: 1

Soil Data:

Depth -ft	Gamma -lb/f3	Phi	C -kp/f2	K -lb/i3	e50 or Dr %	Nspt
0	110.2	0.0	2.30	554.4	0.68	15
20	109.9	0.0	2.30	407.0	0.78	12
21	52.5	0.0	2.30	313.1	0.87	10
45	52.5	10.5	0.00	32.5	34.78	10
70	52.5	50.0	0.00	32.5	34.78	10

Pile Data:

Depth -ft	Width -in	Area -in2	Per. -in	I -in4	E -kp/i2	Weight -kp/f
0.0	72	4071.5	226.2	1319167.4	80	1.0
80.0						

Vertical Capacity:
Weight above Ground= 0.00 Total Weight= -25.79-kp *Soil Weight is not included
Side Resistance (Down)= -932.577-kp Side Resistance (Up)= 1308.823-kp
Tip Resistance (Down)= 5037.396-kp Tip Resistance (Up)= 0.000-kp
Total Ultimate Capacity (Down) Qult= 4104.819-kp Total Ultimate Capacity (Up)= 1283.035-kp
Total Allowable Capacity (Down) Qallow= 2736.546-kp Total Allowable Capacity (Up) Qallow= 1166.395-kp
OK! Qallow > Q

Settlement Calculation:
At Q= 100.00-kp Settlement= 2.25043-in
At Xallow= 2.00-in Q= 44.13777-kp

Note: If the program cannot find a result or the result exceeds the upper limit. The result will be displayed as 99999.

Vertical Load vs. Total Settlement

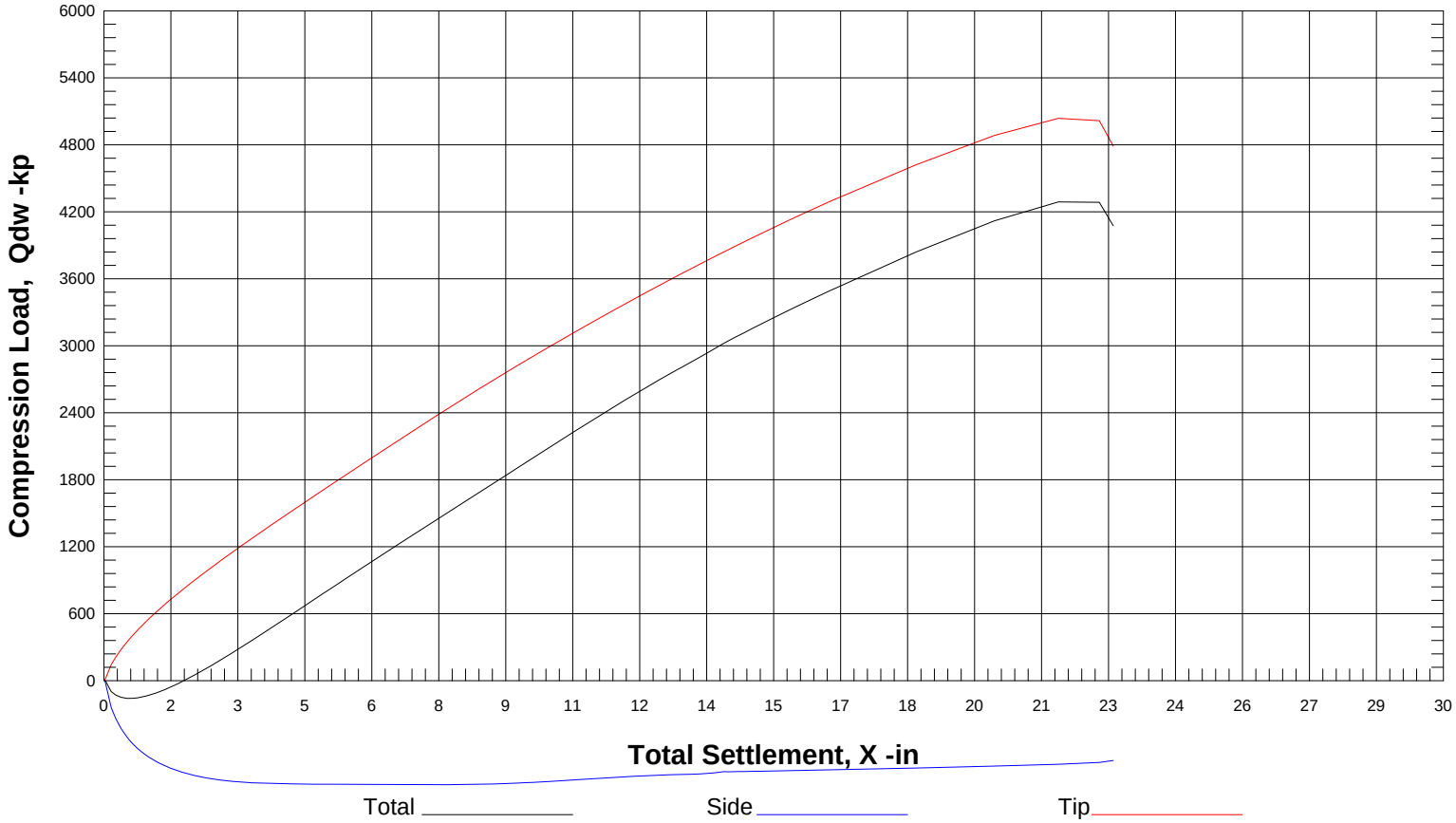


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Figure 4

EX-0001-007214-PCE

SOIL STRESS, SIDE RESISTANCE, & AXIAL FORCE vs DEPTH

Based on Ultimate Load Condition

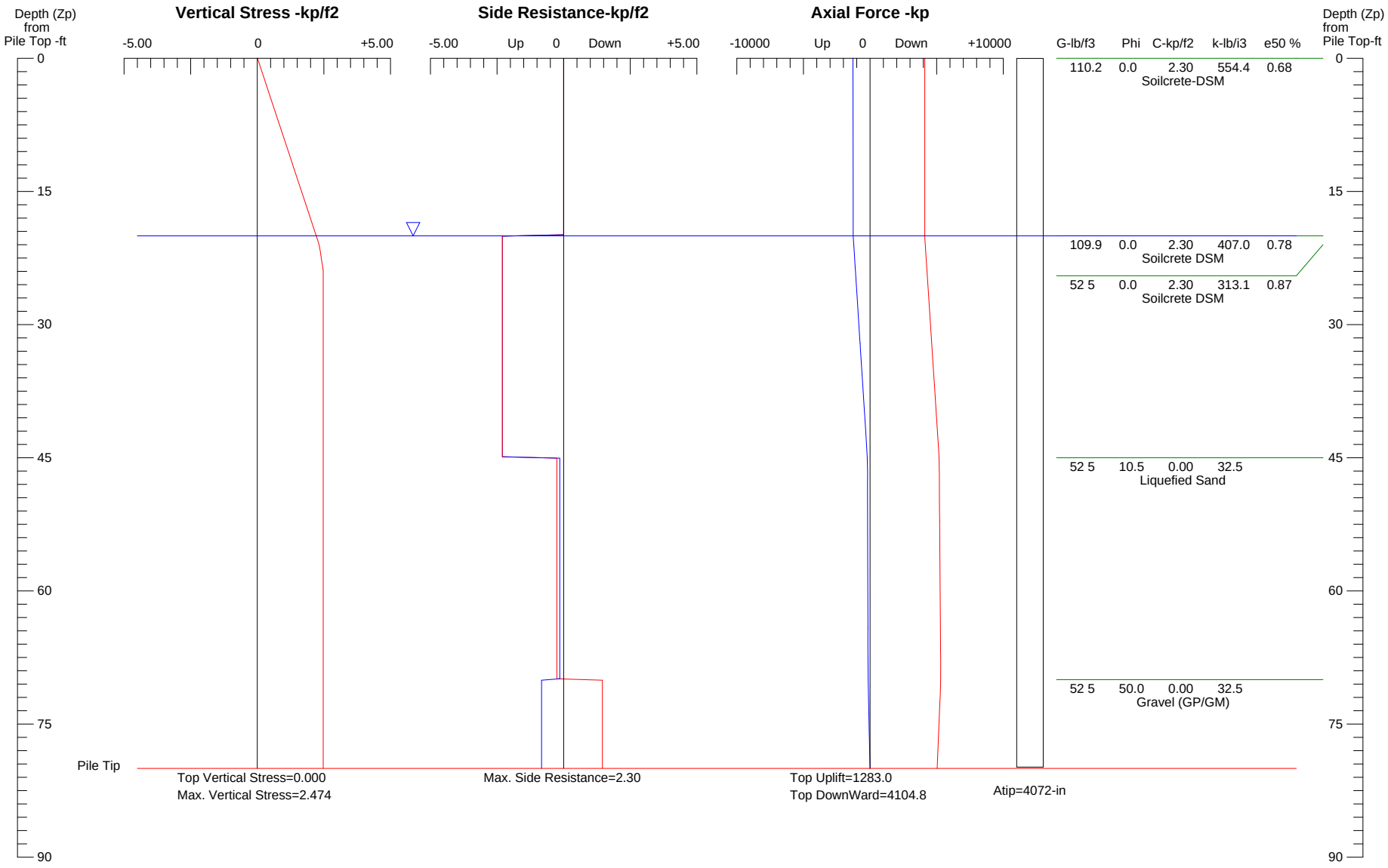

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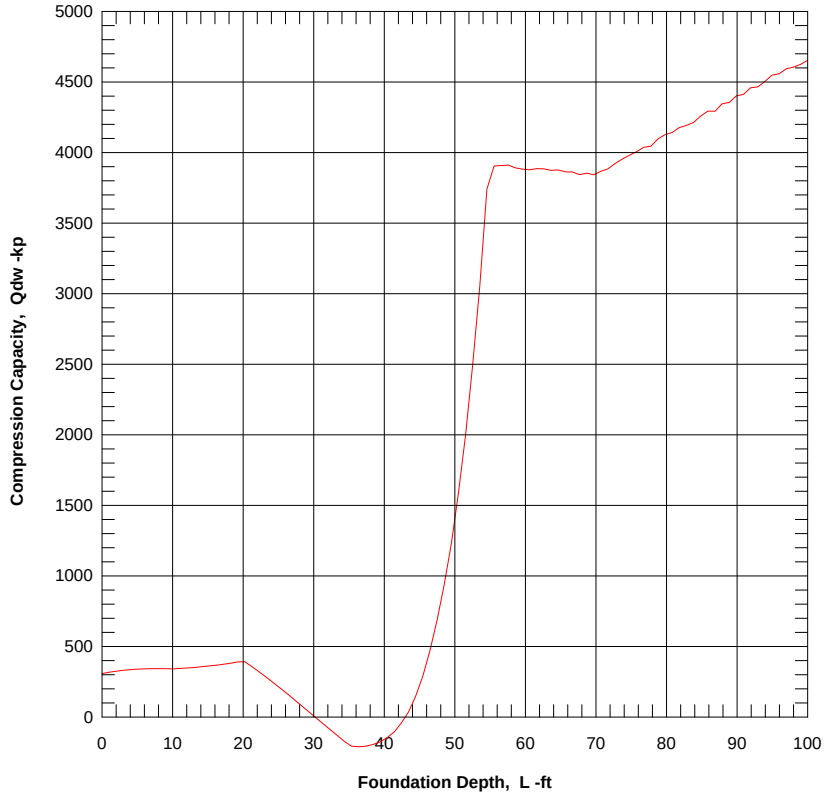
Figure 3

EX-0001-007215-PCE

ULTIMATE CAPACITY vs FOUNDATION DEPTH



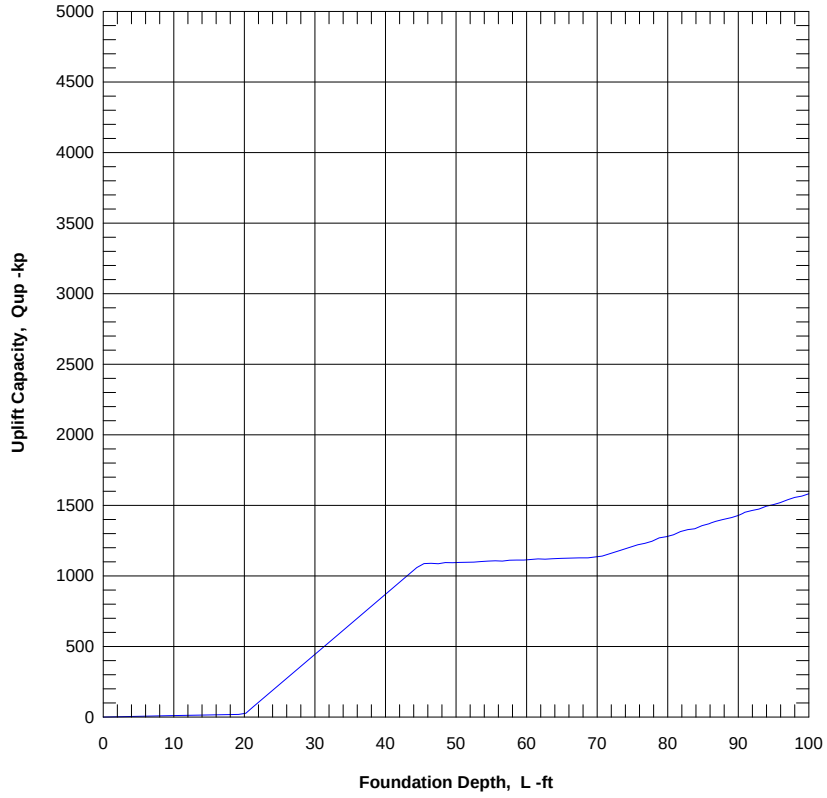


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Figure 5

EX-0001-007216-PCE

Evaluation of Soil Extrusion Between Pannels

HAYWARD BAKER
KELLER

Western Region Office
1780 Lemonwood Drive

Soil Extrusion Check and Composite Undrained Shear Strength of DSM zone

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PROJECT : Port of Tesoro

JOB NUMBER: TBD

PREPARED BY: TVN

DATE: 03/11/2015

NOTES:

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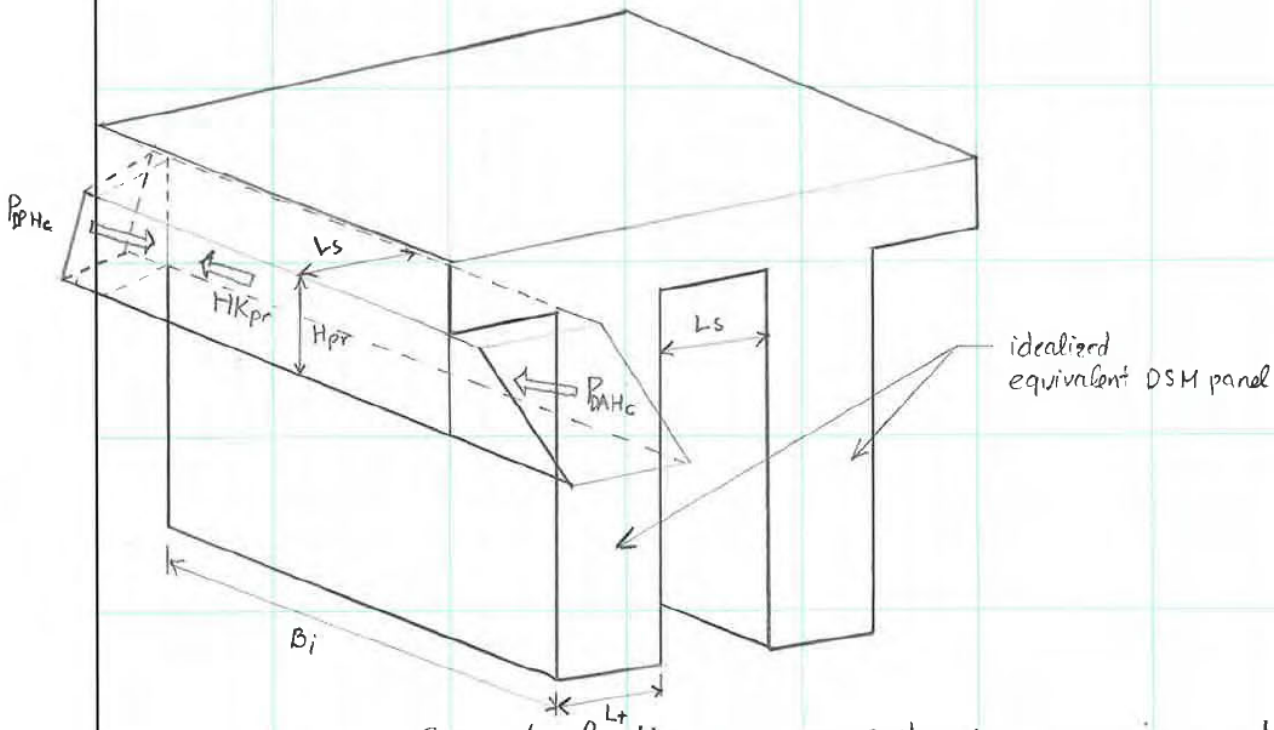
HAYWARD BAKER
A Keller Company

JOB NO.	BY THANG VAN NGUYEN, P.E.	SHEET NO. 2
PROJECT Soil Extrusion between panels Check		DATE 03/11/2015



Conceptual diagram of extrusion of unimproved soil

B_i: width of improved ground (ft)

C_{uc}: undrained shear strength of soft soil (liquefied soil) at assumed prism (psf).

H_{pr}: Height of assumed prism (ft)

HK_{pr}: total seismic inertia force per unit length of prism (klf)

Ls : Approximated panel to panel spacing (from edge to edge) (ft)

P_{DAHc}: Horizontal component of total dynamic active force per unit length acting on the prism (klf)

P_{DPHc}: Horizontal component of total dynamic passive force per unit length acting on the prism (klf)

P_{RW}: total residual water force per unit length (klf)

KELLER

HAYWARD BAKER
A Keller Company

JOB NO.	BY	SHEET NO. 3
PROJECT		DATE 3/11/2015

The factor of safety against soil extrusion is defined as.

$$F_{se} = \frac{2 \times (L_s + H_{pr}) \times B_i \times C_{uc} + P_{DPH_c} \cdot L_s}{(P_{DAH_c} + P_{RW} + HK_{pr})\, L_s} \quad \text{(Equation 6.32b)}$$

"Kitazume & Terashi et al. 2013"

From the proposed ground improvement layout for Area 400 (see HBI submittal drawings), we have the following.

$L_s \approx 30$ ft

$B_i = 60$ ft

$H_{pr} = 45$ ft.

$C_{uc} = 300$ psf (liquefied soil undrained shear strength - assumed)

$P_{RW} = 0$ (assumed)

$K_Y = 0.135$ (see slope stability result for the shoreline)

$PGA = 0.37g$

$EG \approx +27$ ft ±

$GW \approx +7$ ft

$GW_{depth} = 20$ ft.

$\gamma_{soil} = 120$ pcf (assumed soil unit weight)

$\gamma_w = 62.4$ pcf (unit weight of water)

a) Find HK_{pr}:

(I) $HK_{pr} = K_Y \times W_{prism}$.

$W_{prism} = \gamma_{soil} \times H_{pr} - \gamma_w \times (H_{pr} - GW_{depth})$

$= 120\text{pcf} \times 45\text{ft} - 62.4\text{pcf}\,(45\text{ft} - 20\text{ft})$

$= 3840$ psf $= 3.84$ ksf

or 3.84 klf per unit length of prism.

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PROJECT		DATE

Thus,

$$HK_{pr} = 0.135 \times 3.84\ klf = \boxed{0.51\ klf}$$

b) Find P_{DAHc} :

Since the ground is liquefied and the soil behind improved ground is also going to liquefy during the design earthquake

$$\text{(II)} \quad P_{DAHc} = \frac{1}{2} \times \gamma_s' \times H_{pr}^2 + \cancel{w}^{\,0\,(\text{no surcharge})} \cdot H_c + \cancel{P_{RW}}^{\,0} + P_{DW}$$ (Kitazume & Terashi 2013 Equation 6.56 a)

$$= \frac{1}{2} \times \gamma_s \times H_{pr}^2 + P_{DW}.$$

We first need to find P_{DW}.

$$P_{DW} = \frac{7}{8} k_h (\gamma_w + r_u \gamma_s) H_w^2$$

$H_w = H_{pr} - GW_{depth}$

$k_h = PGA = 0.37$

r_u = excess pore water ration = 1.0 for fully liquefied ground.

$$\Rightarrow P_{DW} = \frac{7}{8} \times 0.37 (62.4\ pcf + 1 \times 120\ pcf) \times (45\ ft - 20\ ft)^2$$

$$= \frac{7}{8} \times 0.37 \times 114\ klf = 36.9\ klf$$

Subtitude $P_{DW} = 36.9\ klf$ back to (II).

$$P_{DAHc} = \frac{1}{2} \times 120\ pcf \times (45\ ft)^2 + 36.9\ klf$$

$$P_{DAHc} = \boxed{158.4\ klf}$$

c) Find P_{DPHc} :

Given the ground infront of DSM panels is improved by stone

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columns to create a roughly 35 ft width non-liquefiable block.

(III) $P_{DPHc} = K_{EP} \times \frac{1}{2} \gamma_{soil} \times (H_{pr})^2 + \cancel{P_{\gamma w}}^{\,0}$ (Kitazume & Terashi 2013 equation 6.57c)

we first need to find K_{EP}

$$K_{EP} = \frac{\cos^2(\phi - \theta)}{\cos\theta \cos(\delta - \theta) \times \left\{1 + \sqrt{\frac{\sin(\phi' + \delta)\sin(\phi' - \theta)}{\cos(\delta + \theta)}}\right.}$$

$\phi' = 39°$ Target post improvement effective friction angle of zone improved by stone columns — shall be verified by CPT and derived from tip resistance using Kulhawy & Mayne's correlation

$$\phi' = \min\{17.6 + 11 \log(q_{t1}), 45°\} \text{ for soil with } I_c \leq 2.2$$

$\delta = \phi'/2 = 39°/2 = 19.5°$

$\theta = \tan^{-1}\left(\frac{\gamma}{\gamma'} \times k_{h0}\right)$

$\gamma = 120$ pcf

$\gamma' = 120 - 62.4 = 57.6$ pcf

$k_{h0} = k_\gamma = 0.14$

$\theta = \tan^{-1}\left(\frac{120}{57.6} \times 0.14\right) = 18°$

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PROJECT		DATE

$$K_{EP} = \frac{\cos^2(39^\circ - 18^\circ)}{\cos(18^\circ)\cos(19.5^\circ - 18^\circ) \times \left\{1 + \sqrt{\frac{\sin(39^\circ + 19.5^\circ) \times \sin(39^\circ - 18^\circ)}{\cos(19.5^\circ + 18^\circ)}}\right\}}$$

$$= \frac{0.89}{0.96 \times \{1 + 0.55\}} = 0.598$$

Subtitude back to (III).

$$P_{DRM_c} = 0.598 \times \frac{1}{2} \times 120\,pcf \times (45\,ft)^2$$

$$= \boxed{72.65\ klf}$$

Now the factor of safety against extrusion is calculated.

$$F_{se} = \frac{2 \times (30\,ft + 45\,ft) \times 60\,ft \times 300\,psf + 72.65\,klf \times 30\,ft}{(158\,klf + 0.51\,klf) \times 30\,ft}$$

$$= \frac{4879\ kip}{4755\ kip} = \boxed{1.02}$$

Conclusion:

Since the factor of safety against extrusion for dynamic condition is 1.0, it is justifiable to model the DSM panel zone having a composite sindrain shear strength by area replacement ratio proportioning

$$S_{u_{DSM}} = A_r \times \frac{UCS}{2} + (1 - A_r) \times 300\,psf = 0.2 \times \frac{150\,psi}{2} + 0.8 \times 300\,psf$$

$= 2400\,psf$ (use 2250 psf for design)

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Appendix G
Construction Drawings

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VANCOUVER ENERGY TERMINAL
Vancouver, Washington

GROUND IMPROVEMENT PROGRAM

PREPARED FOR:

TESORO SAVAGE PETROLEUM TERMINAL, LLC
C/O R&M ENGINEERING CONSULTANTS
5280 SOUTH COMMERCE DRIVE, SUITE E-130
MURRAY, UTAH 84107

ENGINEER OF RECORD

Mark A. Rohrbach, PE, GE, P. Eng
Sr. Engineer
Hayward Baker Inc.

MARK ROHRBACH
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PRELIMINARY

SHEET INDEX

HB-1	TITLE SHEET
HB-2	NOTES AND SCHEDULE
HB-3	NOTES AND SCHEDULE
HB-4	STANDARD DETAILS
HB-5	GENERAL SITE PLAN
HB-6	AREA 300 TANK DESIGNATION
HB-7	AREA 300 PIPE RACK SUPPORT
HB-8	TANK 1 PLAN VIEW
HB-9	TANK 2 PLAN VIEW
HB-10	TANK 3 PLAN VIEW
HB-11	TANK 4 PLAN VIEW
HB-12	TANK 5 PLAN VIEW
HB-13	TANK 6 PLAN VIEW
HB-14	AREA 400 SHORELINE

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VANCOUVER ENERGY
Tesoro Savage Petroleum Terminal LLC

PROJECT: TESORO SAVAGE VANCOUVER ENERGY DISTRIBUTION TERMINAL
PORT OF VANCOUVER, WASHINGTON

DESCRIPTION: TITLE SHEET

DESIGN:	START DATE: 4-14-2015	SCALE: N. T. S	
DRAWN: S P B	PRINT DATE:	PROJECT MANAGER:	
CHECKED:	APPROVED: M. ROHRBACH	SIZE: 24x36	
DRAWING NUMBER		SHEET	REV.
HB-1		1/14	0

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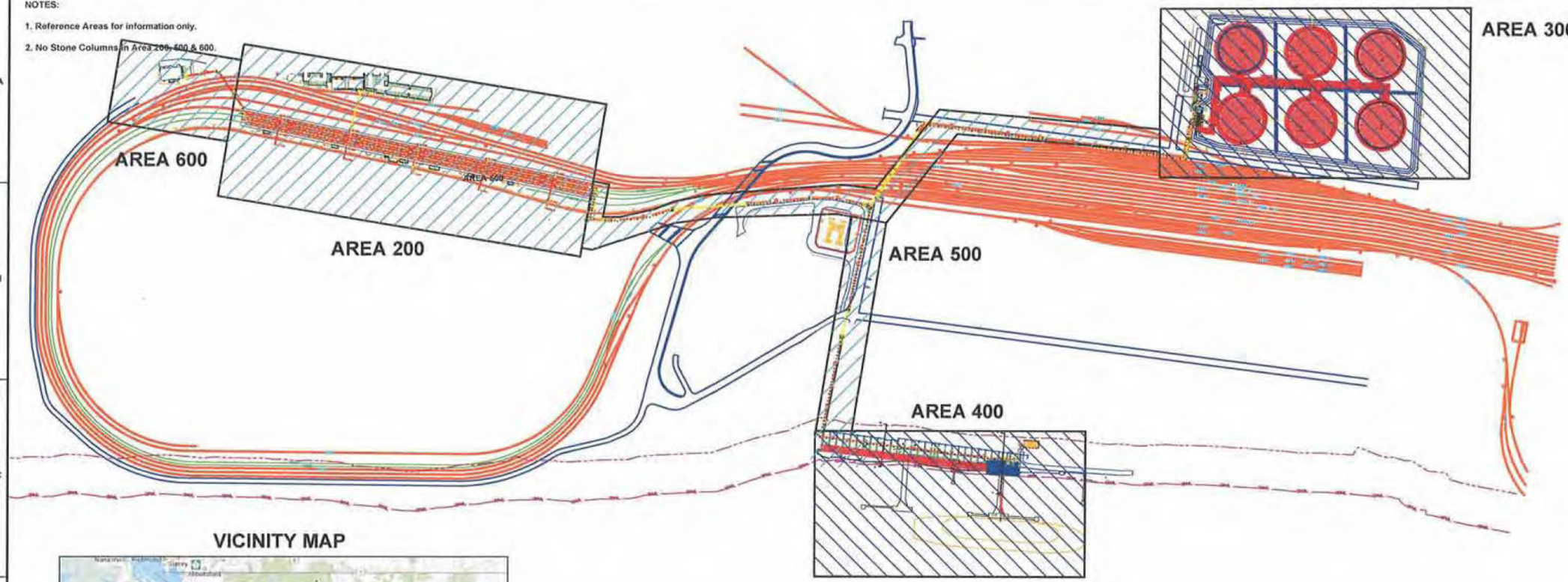
Tank Farm & Shoreline REV1

EX-0001-007226-PCE

NOTES:

1. Reference Areas for information only.
2. No Stone Columns in Area 200, 400 & 600.



VICINITY MAP

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VANCOUVER ENERGY
Tesoro Savage Petroleum Terminal LLC

PROJECT: TESORO SAVAGE VANCOUVER ENERGY DISTRIBUTION TERMINAL
PORT OF VANCOUVER, WASHINGTON

DESCRIPTION: GENERAL SITE PLAN

DESIGN:	START DATE: 4-14-15	SCALE: 1"=230'	
DRAWN: S.P.B.	PRINT DATE:	PROJECT MANAGER:	
CHECKED:	APPROVED: M. ROHRBACH	SIZE: 24x36	
DRAWING NUMBER: HB-2		SHEET 2/14	REV. 0

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Project specific notes

1. Hayward Baker Inc.'s (HBI) portion of this project Involves construction of ground Improvement systems for both Area 300 (Storage) and Area 400 (Shoreline) designed by Hayward Baker and shown on these drawings.
2. Hayward Baker has relied on various documents for design. Those documents are listed In the ground Improvement design report submitted with these drawings. These drawings should not be separated from that design report. If any of the reference documents listed in Hayward Baker's design report change in any way Hayward Baker shall be notified and provided the opportunity to review the proposed changes and update this design as needed.
3. Hayward Baker must be notified immediately if information included in these plans or in the ground improvement calculation package conflicts with the project structural or architectural drawings.
4. Hayward Baker's design requires the tanks be founded on a reinforced concrete ring beam at least 4 feet deep and 18 Inches wide.
5. Hayward Baker's design requires the tanks be supported by a load transfer platform (LTP) a minimum of 4 feet thick, consisting of compacted structural fill with an in-place density of about 130 pcf and an angle of internal friction of at least 36 degrees and the bottom of tank is at Elevation 31.75'.
6. Hayward Baker's design anticipates construction of some piles for dock abutment support. The construction sequencing of these piles and the ground Improvement described herein must be finalized before construction of either element begins.
7. Hayward Baker's design requires isolated reinforced-concrete spread footing to support the transfer pipeline.
8. Hayward Baker's design is intended to satisfy the following performance requirements:

   8.1 for Area 300-Storage
   - Total seismic settlement: 2 inches
   - Allowable seismic differential settlement: 1 inch
   - Hydo-testing performance: In accordance with API acceptance criteria
   - Post-hydro testing total static settlement: 2 inches
   - Post-hydro testing allowable differential settlement: 1 inch in 50 feet

   8.2 for Area 400-Shoreline and Dock Facilities
   - Total seismic settlement: 2 inches vertical
   - Total lateral displacement
     - 2 Inches under pipe-rack foundations
   - 4 inches at Berth 13 trestle abutment
   - Allowable seismic differential settlement: 1 inch in 30 feet

9. The purpose of this project Is to:
   9.1. Limit seismic settlement to acceptable levels
   9.2. Limit static settlement to acceptable levels
   9.3. Improve subsurface soil conditions to an adequate bearing capacity
   9.4. No additional seismic performance requirements.
11. The ground improvement elements' properties used for design are provided in the design report.
12. All ground improvement elements shall be installed within 6 inches of the location shown on these plans. Hayward Baker retains the sole authority to modify ground Improvement element locations due to constructability and site constraints.
13. Practical stone column refusal Is defined as a sustained effort of 250 amps and failure to penetrate 6 Inches In 30 seconds.
14. The ground Improvement design shall be verified using the methods described In the verification testing notes. Any additional testing requested shall be considered extra work.
15. The ground Improvement engineer Is the registered professional engineer whose stamp Is on these drawings.
16. All stone columns must be predrilled using a 24 Inch drill to the bottom of the silt and/or clay layer(s).
17. All stone columns shall be constructed to a diameter of 36 inches.
18. All other ground improvement elements (Jet Grout columns/ Deep Soil Mixed columns) shall be constructed to diameters indicated in the drawings.

General notes

1. Others are to provide a stable and relatively level working surface. The elevation of the working surface shall be
   - For Area 300: Elevation 25' or 4' below existing grade whichever results in the lower elevation.
   - For Area 400: Elevation 25' or 4' below existing grade whichever results In the lower elevation.

   The working surface shall be constructed and managed by others such that Hayward Baker equipment can efficiently access and traverse the site. Hayward Baker Is not responsible for returning the site to Its original grade or condition.
2. Hayward Baker will provide a qualified full time quality control (qc) representative. This representative is titled Hayward Baker superintendent, foreman or Hayward Baker field engineer. Third party testing or Inspection shall be provided by others.
3. Stone columns will be installed to design depth or auger refusal.
4. If obstructions prevent Installation of a stone column to the specified tip elevation:
   a) Should the stone column be more than 75 percent of the design length the stone column is complete.
   b) Should the stone column be less than 75 percent of the design length relocate the stone column and attempt a second Installation to the target tip elevation.
   c) Should the relocated stone column also meet refusal move on to the next production column and contact the design engineer for direction.
   d) Or as directed by the ground improvement engineer.
5. Deep soil mixed and Jet grout columns shall be installed to design depths.
6. Locating, protecting and rerouting/removal of all utilities is the responsibility of others. Hayward Baker is not responsible for damage to existing utilities.
7. After the completion of the ground improvement work, others are responsible for protection of the work. Proper site drainage to prevent ponding of water at the area of the ground Improvement elements and control and coordination of earthwork activities shall be managed such that existing ground Improvement elements are not damaged.
8. The ground Improvement element locations shown on the approved construction drawings are for ground Improvement and site layout. This plan should not be used for foundation layout. Footing locations, sizes and orientation shown on these shop drawings are for information only. Refer to the "for construction" structural package for specific foundation dimensions and locations.
9. In the event that the no-dig zone defined in this drawing set is compromised or ground improvement elements (stone columns, jet grout columns, or deep soil mixed columns) are undermined for any reason it is acceptable to place #57 stone or other approved structural fill and compact the fill to at least 97% of the maximum dry density as determined by ASTM D1557. Compaction must be with an Impact style compactor. This operation shall be monitored by the geotechnical engineer of record and compaction records shall be provided to Hayward Baker. Compaction testing shall occur within 4 feet of the ground improvement elements. Compaction is considered adequate only after Hayward Baker provides written confirmation. For the purposes of this note the no dig zone begins at the elevation of ground improvement element installation.
10. Hayward Baker is not responsible for settlements of foundations/slabs found on unimproved soil or for differential settlements between foundations found on improved soil and foundations/slabs found on unimproved soil.

Stone column supported foundations (tanks, tank supports, and structures)

1. Water shall not pool or collect In foundation excavations.
2. Prior to placing steel and concrete mechanical verification of the Improved foundation sub grade Is required. Compaction shall be performed over the entire foundation sub grade to compact loose soil and stone. If soft areas are encountered, this material should be removed and replaced with #57 stone under the inspection of a qualified geotechnical engineer.
3. A testing agency shall inspect each foundation excavation and approve it prior to placement of structural fill or concrete.
4. All proposed underground utilities within and adjacent to stone column supported foundations shall be field verified by the general contractor and coordinated with Hayward Baker prior to utility trench excavation and utility installation. See the adjacent excavation details in this drawing set.

Design verification notes

Hayward Baker shall construct tests sections to ensure that the necessary ground improvement is being achieved.

A)Stone Column

- Each test section shall include no less than 6 stone columns.
- If a test section Is constructed near the center of a tank or within the proposed shoreline protection zone area It may be Included In the production work. Test sections may not be constructed within 50 feet of a tank perimeter.
- Testing of the test sections shall Include CPT testing and plate load testing. CPT soundings shall extend 3 feet below the tip of the stone columns.
- Acceptance of a test section shall be based on analysis of the CPT data and review of plate load testing results.
- Analysis of the CPT data shall be performed using the same methods used for design.
- Following design verification testing production work below the proposed tanks shall be evaluated by CPT methods at a frequency of
  - For Area 300: Four CPTs per tank and production work below pipeline areas shall be one CPT per 20,000 square feet of ground improvement area.
  - For Area 400: One CPTs per 200 linear feet of transfer pipeline alignment.

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VANCOUVER ENERGY
Tesoro Savage Petroleum Terminal LLC

PROJECT: TESORO SAVAGE VANCOUVER ENERGY DISTRIBUTION TERMINAL PORT OF VANCOUVER, WASHINGTON

DESCRIPTION: NOTES & SCHEDULE

DESIGN:		START DATE:	4-14-15	SCALE:	N. T. S.
DRAWN:	S P B	PRINT DATE:		PROJECT MANAGER:	
CHECKED:		APPROVED:	M. ROHRBACH	SIAAE:	22x34
DRAWING NUMBER				SHEET	REV.
HB-3				3/14	0

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B) Jet Grout Column

- At minimum ,one test section shall be conducted to verify that the proposed equipment, procedures, and mix design can achieve the design geometry and soilcrete strength.
- Each test section shall include at least three columns.
- The methods used to check jet grout column diameter will consist of some/all of the following:
  - Excavation of portions of the completed columns
  - Exploratory core drilling through the extents of the column
  - Or other methods approved by the ground Improvement engineer.
- At minimum three sets of 4 wet samples each collected at random depths shall be procured and cast into cylinders for testing.
- At minimum one set of three dry samples taken through the process of core drilling shall be tested.
- These dry and wet samples shall be tested in unconfined compression to see if they meet the target design strength (an average 150 psi with no break less than 75 psi)

C)Deep Soil Mixed Column

- At minimum, one test section shall be conducted to verify that the proposed equipment, procedures, and mix design can uniformly mix the onsite soils and achieve the design strength.
- The test section shall include at least 3 fully mixed 6 foot diameter columns to the design depth.
- Laboratory testing shall be used to identify initial mix designs used in the field program. Bulk soil samples from the site shall be obtained by Hayward Baker.
- A suite of three mix designs is required.
- At minimum one full-depth core sample from the test elements shall be performed.
- At minimum three sets of 4 wet samples each collected at random depths shall be procured and cast into cylinders for testing.
- At minimum one set of three dry samples taken from the core shall be tested.
- These dry and wet samples shall be tested in unconfined compression to see if they meet the target design strength (an average 150 psi with no less than 10% of breaks below 100 psi)

Tank First Filling Notes:

1. The first filling of each tank must be executed as described below.
2. First filling shall be with water for the purpose of hydro-testing.
3. Settlement monitoring points shall be established as required by API 650 and API 653.
4. After construction of the LTP is complete, a vibrating wire piezometer (VWP) shall be installed below the center of each tank by others. The VWP shall be located in the silt and/or clay layer.
5. At first filling each tank shall be filled as required by API and in accordance with the following anticipated hold times or as directed by Hayward Baker:

% of Full	Anticipated Hold	Settlement monitoring frequency	Authority to resuming filling
25	2 days	Every 4 hours	Hayward Baker
50	4 days	Every 12 hours	Hayward Baker
75	6 days	Daily	Hayward Baker
100	18 days	Days 1 - 6 Daily	Hayward Baker
Days 7-18 every 2 days			

6. VWP readings shall be continuously available to Hayward Baker during first filling and hydro-testing.

STONE COLUMN SCHEDULE

AREA	DIAMETER	LENGTH
Tank 1	3'	40'
Tank 2	3'	43'
Tank 3	3'	36'
Tank 4	3'	35'
Tank 5	3'	36'
Tank 6	3'	38'
Pipeline	3'	See Sheet HB-7
Area 400	3'	See Sheet HB-14

DSM SCHEDULE

AREA	DIAMETER	LENGTH
Piperack Foundation	6'	43'

JET GROUT SCHEDULE

AREA	DIAMETER	LENGTH
Berth 13 Trestle Abutment	6'	78'
Transfer Piperack Foundation	8'	32'

MARK ROHRBACH
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HAYWARD BAKER
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Lynnwood, Washington 98087
206-523-1726 Fax 206-523-1733

VANCOUVER ENERGY
Tesoro Savage Petroleum Terminal LLC

PROJECT: TESORO SAVAGE VANCOUVER ENERGY DISTRIBUTION TERMINAL
PORT OF VANCOUVER, WASHINGTON

DESCRIPTION: NOTES & SCHEDULE

DESIGN:	START DATE: 4-14-15	SCALE: N. T. S.	
DRAWN: S P B	PRINT DATE:	PROJECT MANAGER:	
CHECKED:	APPROVED: M . ROHRBACH	SIAAE: 22x34	
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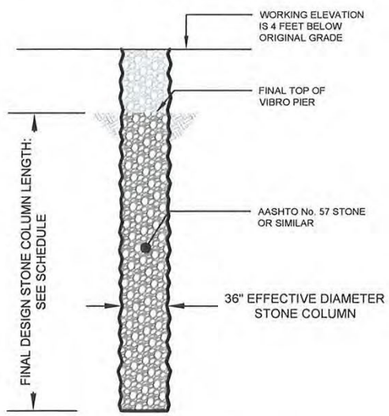
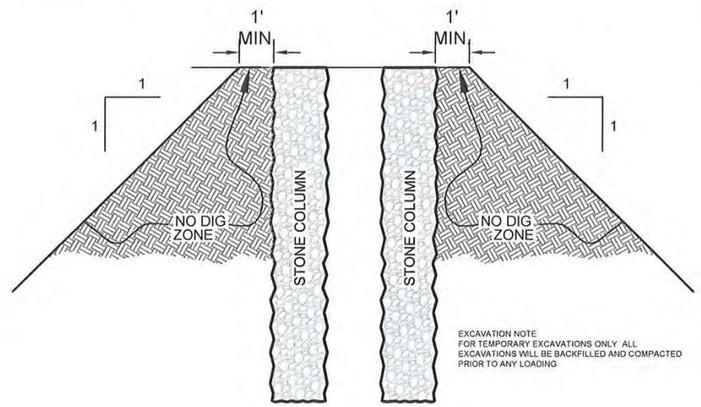


TYPICAL STONE COLUMN
N.T.S.



NO-DIG ZONE: SLAB
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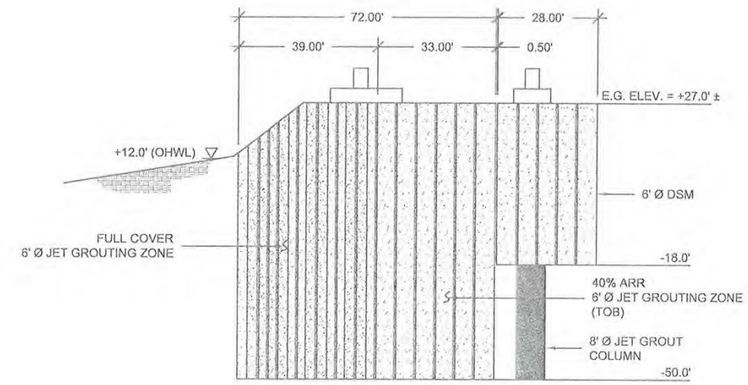

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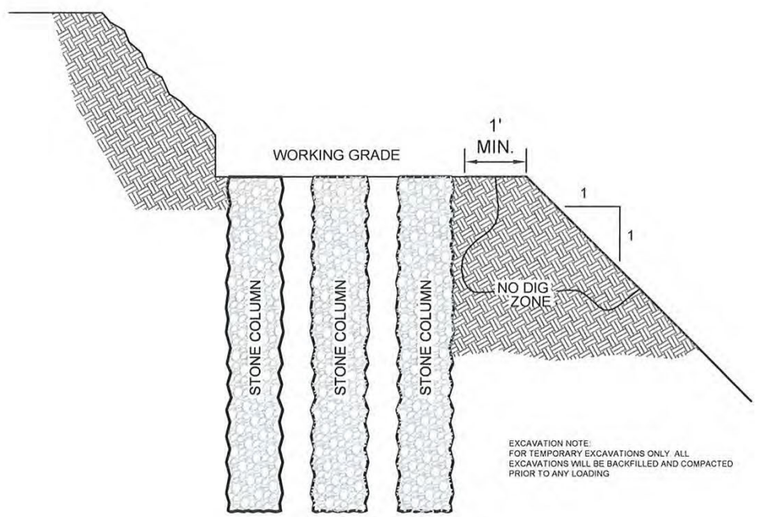
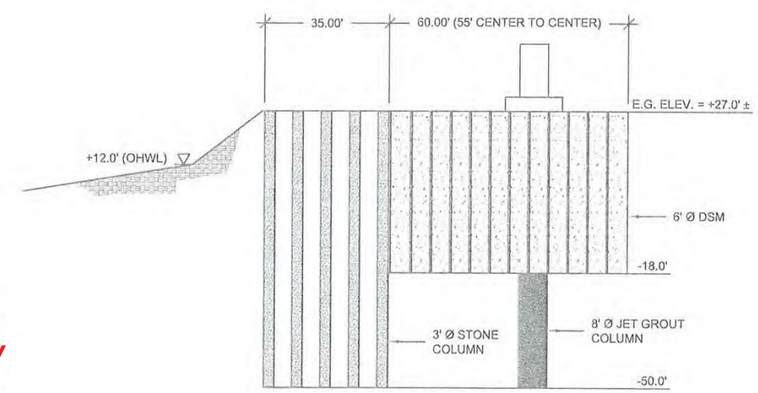


NO-DIG ZONE: SPREAD FOOTING
N.T.S.



SECTION B-B'

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VANCOUVER ENERGY
Tesoro Savage Petroleum Terminal LLC

PROJECT: TESORO SAVAGE VANCOUVER ENERGY DISTRIBUTION TERMINAL
PORT OF VANCOUVER, WASHINGTON

DESCRIPTION: STANDARD DETAILS

DESIGN:	START DATE: 4-14-15	SCALE: N. T. S.	
DRAWN: S P B	PRINT DATE:	PROJECT MANAGER:	
CHECKED:	APPROVED: M. ROHRBACH	SIAAE: 24x36	
DRAWING NUMBER		SHEET	REV.
HB-5		5/4	0

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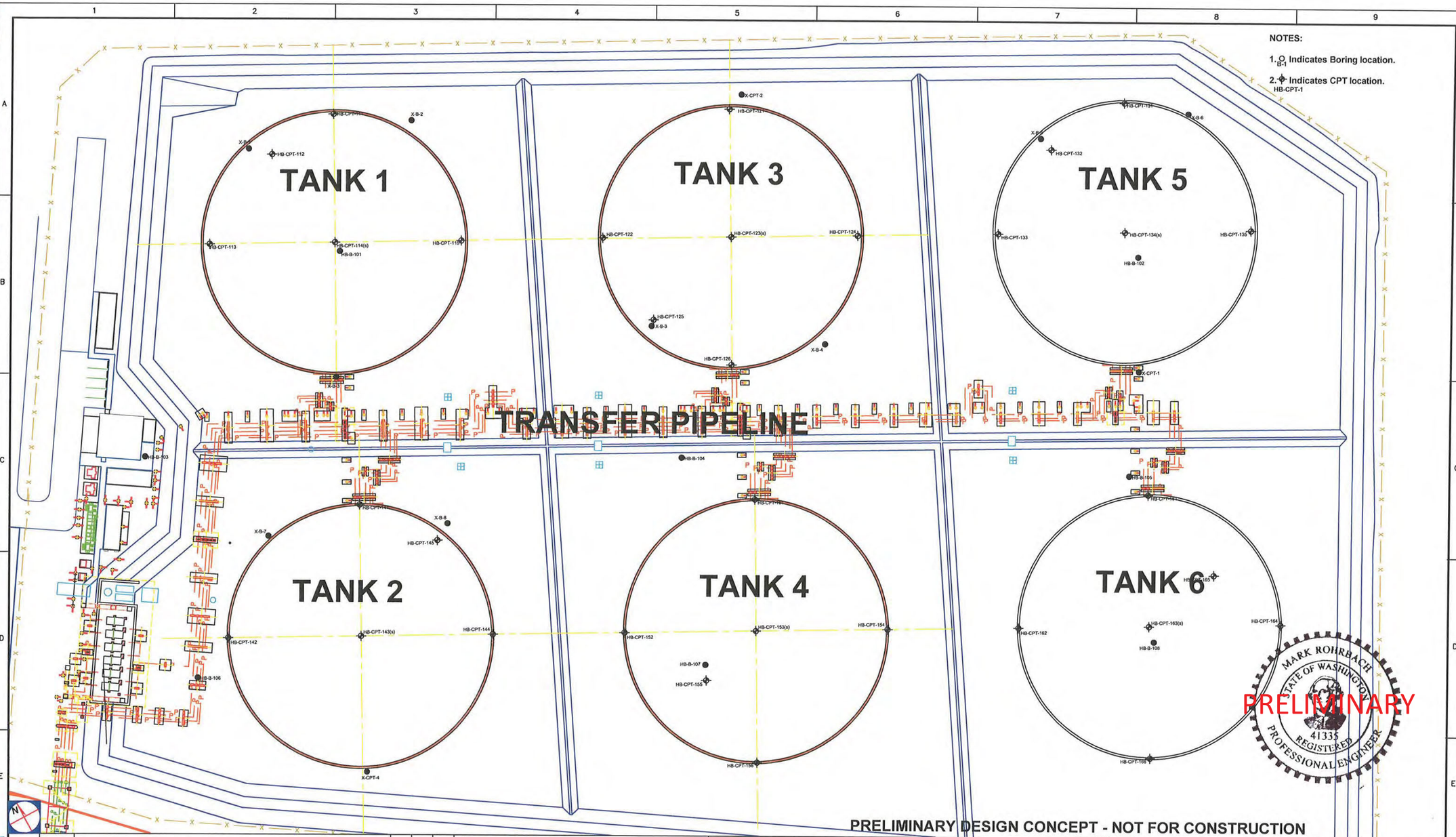

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VANCOUVER ENERGY
Tesoro Savage Petroleum Terminal LLC

PROJECT: TESORO SAVAGE VANCOUVER ENERGY DISTRIBUTION TERMINAL
PORT OF VANCOUVER, WASHINGTON

DESCRIPTION: AREA 300 TANK DESIGNATION

DESIGN:	START DATE: 4-14-15	SCALE: 1"=40'	
DRAWN: S P B	PRINT DATE:	PROJECT MANAGER:	
CHECKED:	APPROVED: M. ROHRBACH	SIAAE: 24x36	
DRAWING NUMBER		SHEET	REV.
HB-6		6/14	0

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Tank Farm & Shoreline REV1

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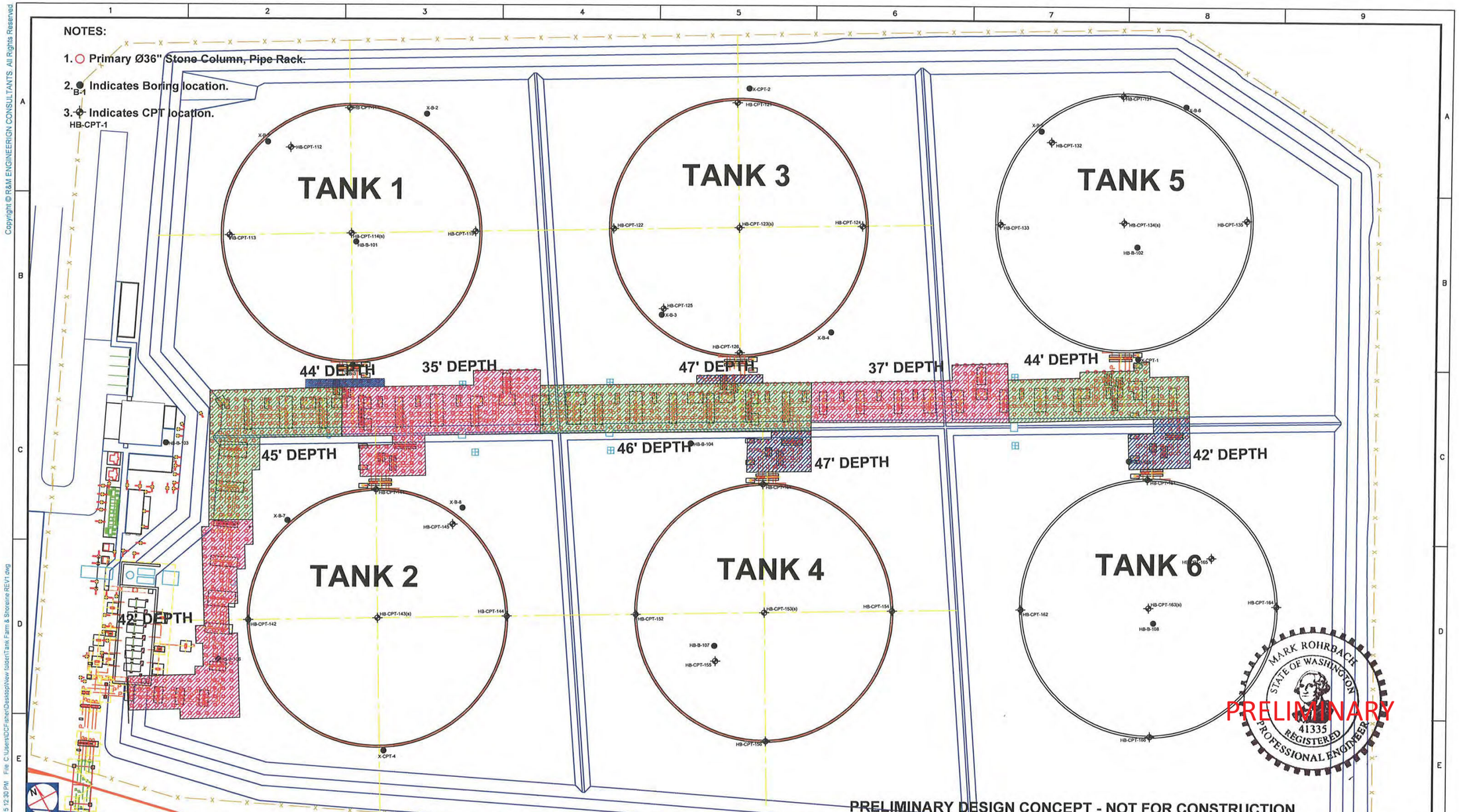
NOTES:

1. ○ Primary Ø36" Stone Column, Pipe Rack.
2. ● B-1 Indicates Boring location.
3. ⊕ HB-CPT-1 Indicates CPT location.



PRELIMINARY DESIGN CONCEPT - NOT FOR CONSTRUCTION

NO.	DATE	REVISION	BY	CK'D	APP

HAYWARD BAKER
A KELLER COMPANY

Seattle Office
1344 "E" Street, Way South
Auburn, Washington 98002
Phone: 253-288-1733

VANCOUVER ENERGY
Tesoro Savage Petroleum Terminal LLC

PROJECT: TESORO SAVAGE VANCOUVER ENERGY DISTRIBUTION TERMINAL
PORT OF VANCOUVER, WASHINGTON

DESCRIPTION: AREA 300 PIPE RACK SUPPORT

DESIGN:	START DATE: 4-14-15	SCALE: 1"=40'	
DRAWN: S P B	PRINT DATE:	PROJECT MANAGER:	
CHECKED:	APPROVED: M. ROHRBACH	SIAAE: 11X17	
DRAWING NUMBER		SHEET	REV.
HB-7		7/14	0

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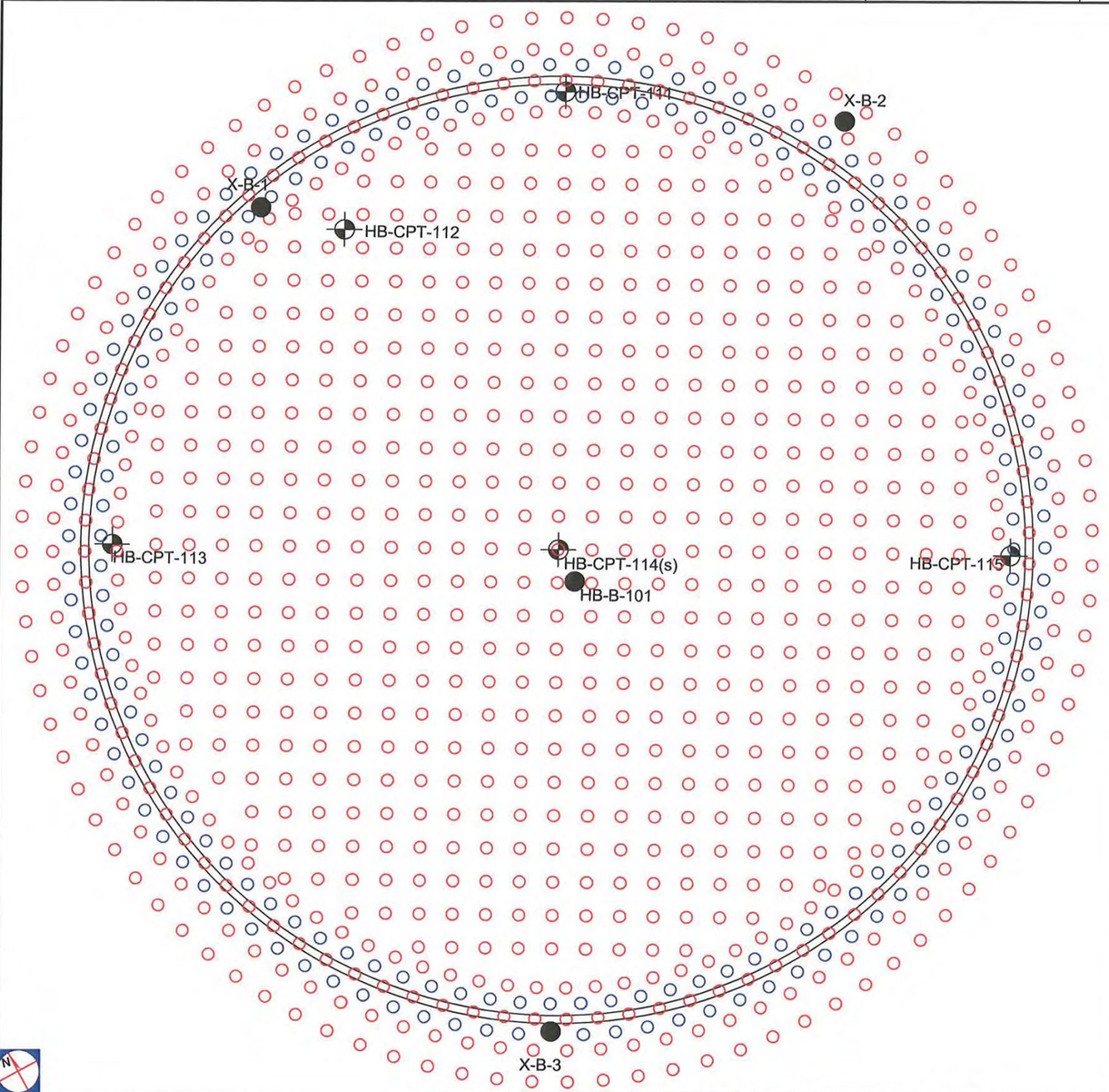

NOTES:

1. Indicates Primary Ø36" Stone Column, 40' Depth.
2. Indicates Secondary Ø36" Stone Column, 30' Depth.
3. B-1 Indicates Pre-Boring location.
4. CPT-1 Indicates Pre-CPT location.
5. Pre-Auger all stone columns.

MARK ROHRBACH
STATE OF WASHINGTON
41335
REGISTERED
PROFESSIONAL ENGINEER

PRELIMINARY

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HAYWARD BAKER
A KELLER COMPANY

VANCOUVER ENERGY
Tesoro Savage Petroleum Terminal LLC

PROJECT: TESORO SAVAGE VANCOUVER ENERGY DISTRIBUTION TERMINAL
PORT OF VANCOUVER, WASHINGTON

DESCRIPTION: TANK 1 PLAN VIEW; STONE COLUMN LAYOUT

DESIGN:	START DATE: 4-14-15	SCALE: 1"=15'	
DRAWN: S P B	PRINT DATE:	PROJECT MANAGER:	
CHECKED:	APPROVED: M . ROHRBACH	SIAAE: 24x36	
DRAWING NUMBER: HB-8		SHEET 8/14	REV. 0

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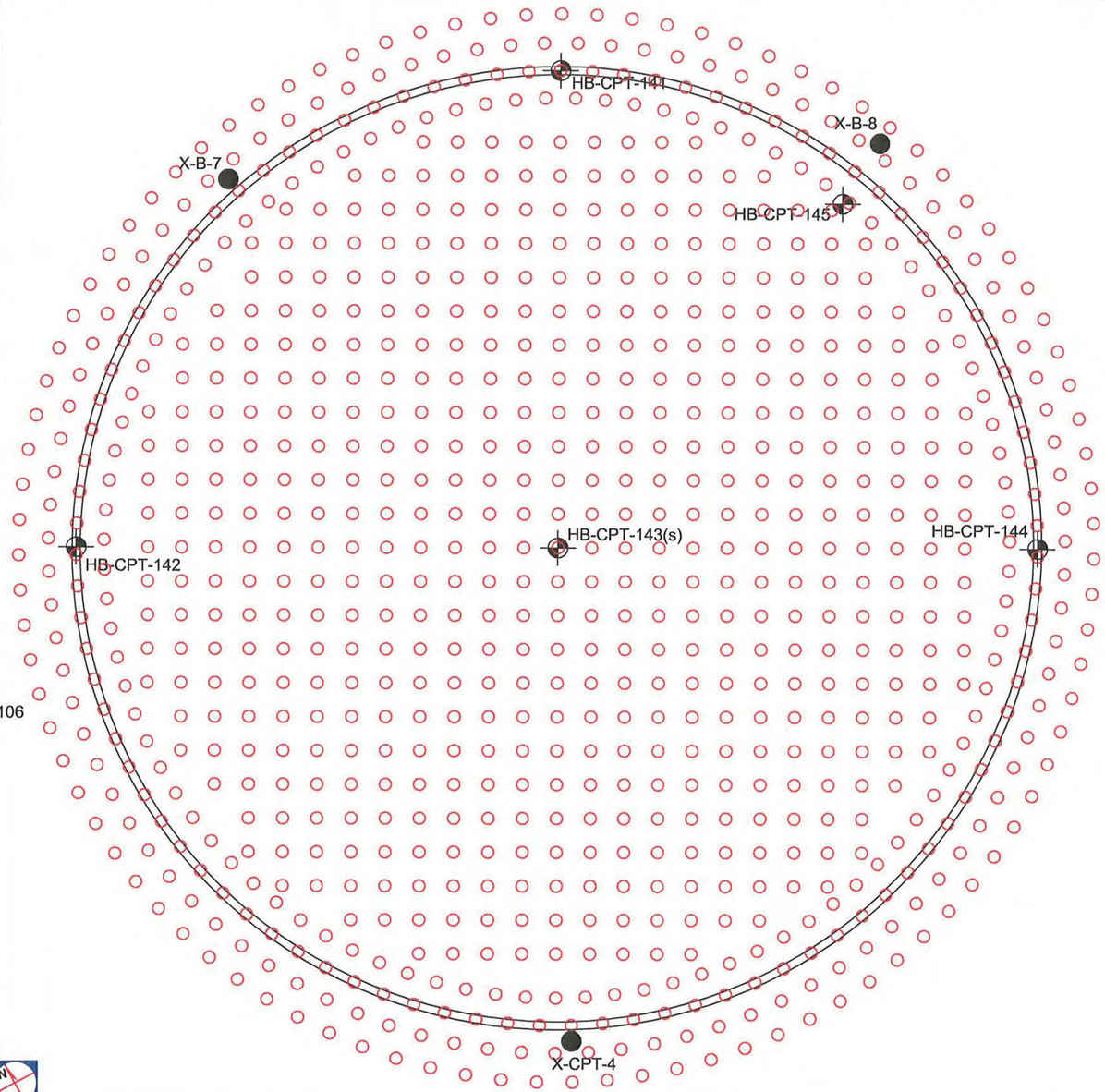

NOTES:

1. ○ Indicates Primary Ø36" Stone Column, 35' Depth.
2. ● B-1 Indicates Pre-Boring location.
3. ⊕ CPT-1 Indicates Pre-CPT location.
4. Pre-Auger all stone columns.

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VANCOUVER ENERGY
Tesoro Savage Petroleum Terminal LLC

PROJECT: TESORO SAVAGE VANCOUVER ENERGY DISTRIBUTION TERMINAL
PORT OF VANCOUVER, WASHINGTON

DESCRIPTION: TANK 2 PLAN VIEW; STONE COLUMN LAYOUT

DESIGN:	START DATE: 4-14-15	SCALE: 1"=15'
DRAWN: S P B	PRINT DATE:	PROJECT MANAGER:
CHECKED:	APPROVED: M. ROHRBACH	SIAAE: 24x36
DRAWING NUMBER		SHEET / REV.
HB-9		9 / 14 / 0

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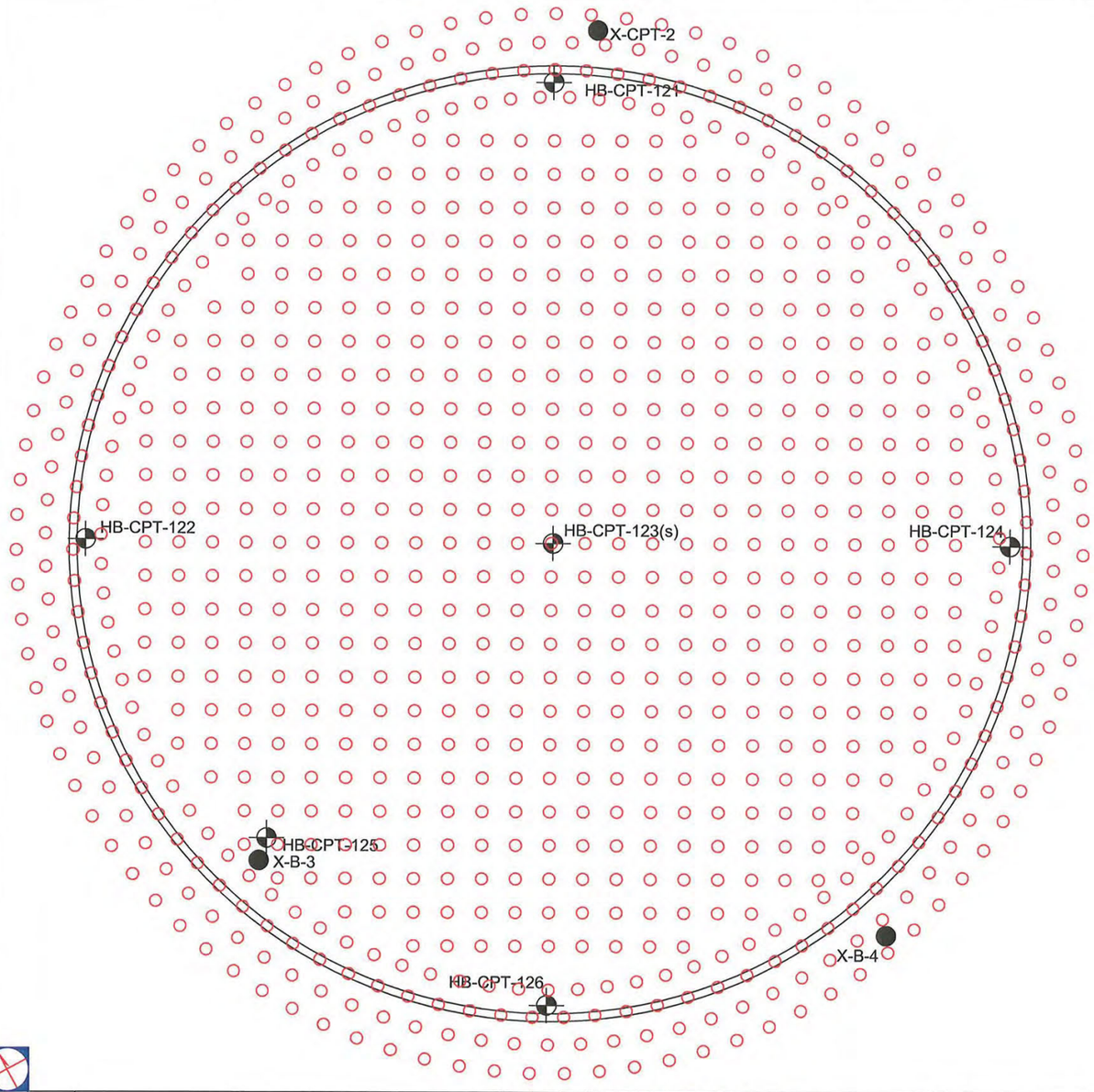

NOTES:

1. Indicates Primary Ø36" Stone Column, 43' Depth.
2. Indicates Pre-Boring location.
3. Indicates Pre-CPT location.
4. Pre-Auger all stone columns.

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VANCOUVER ENERGY
Tesoro Savage Petroleum Terminal LLC

PROJECT: TESORO SAVAGE VANCOUVER ENERGY DISTRIBUTION TERMINAL
PORT OF VANCOUVER, WASHINGTON

DESCRIPTION: TANK 3 PLAN VIEW; STONE COLUMN LAYOUT

DESIGN:	START DATE: 4-14-15	SCALE: 1"=15'	
DRAWN: S P B	PRINT DATE:	PROJECT MANAGER:	
CHECKED:	APPROVED: M. ROHRBACH	SIAAE: 24x36	
DRAWING NUMBER		SHEET	REV.
HB-10		10/14	0

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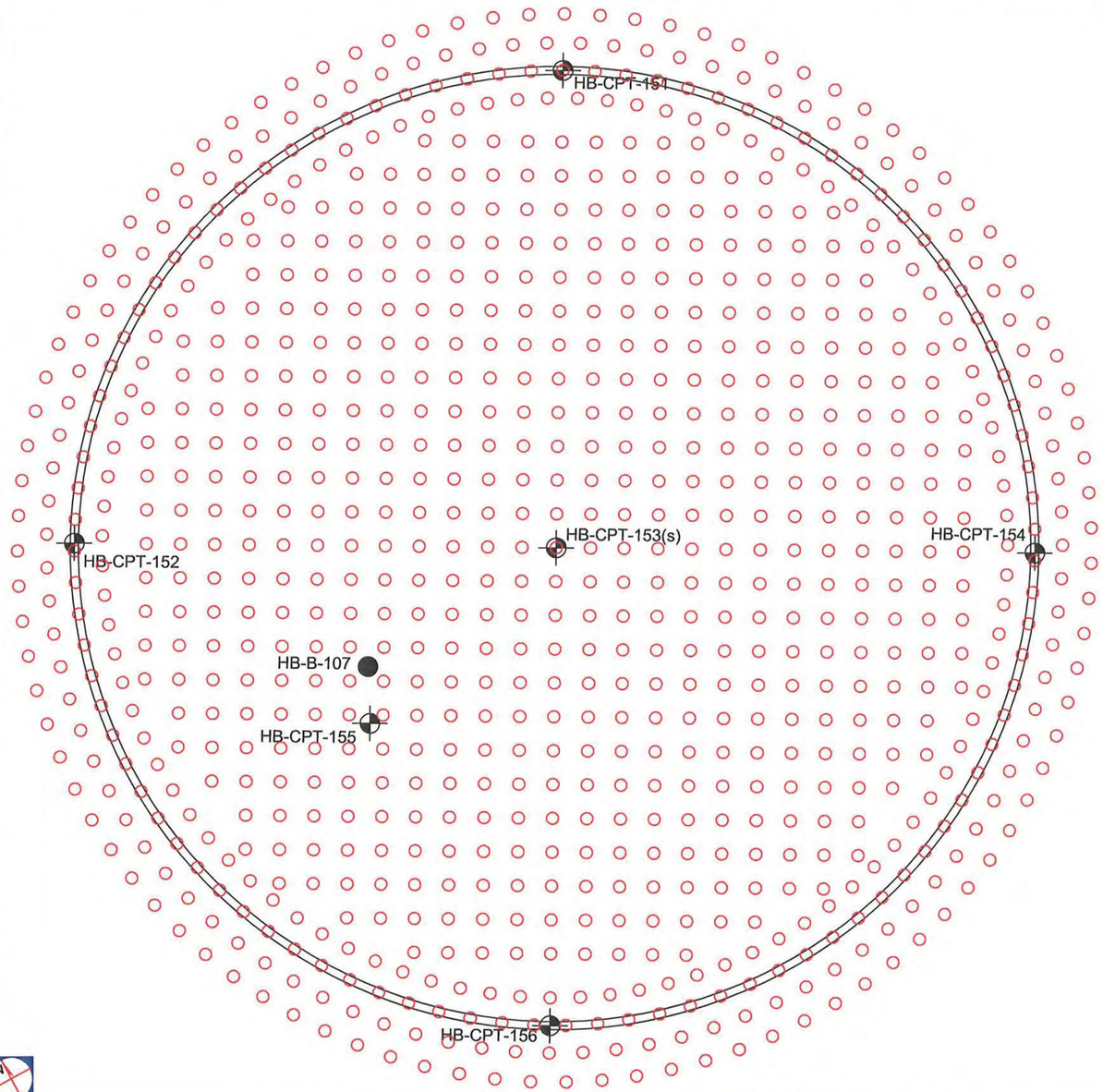
Tank Farm & Shoreline REV1



NOTES:

1. ○ Indicates Primary Ø36" Stone Column, 36' Depth.
2. ● B-1 Indicates Pre-Boring location.
3. ⊕ CPT-1 Indicates Pre-CPT location.
4. Pre-Auger all stone columns.

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VANCOUVER ENERGY
Tesoro Savage Petroleum Terminal LLC

PROJECT: TESORO SAVAGE VANCOUVER ENERGY DISTRIBUTION TERMINAL
PORT OF VANCOUVER, WASHINGTON

DESCRIPTION: TANK 4 PLAN VIEW; STONE COLUMN LAYOUT

DESIGN:	START DATE: 4-14-15	SCALE: 1"=15'	
DRAWN: S P B	PRINT DATE:	PROJECT MANAGER:	
CHECKED:	APPROVED: M. RORHBACH	SIAAE: 24x36	
DRAWING NUMBER: HB-11		SHEET 11/14	REV. 0

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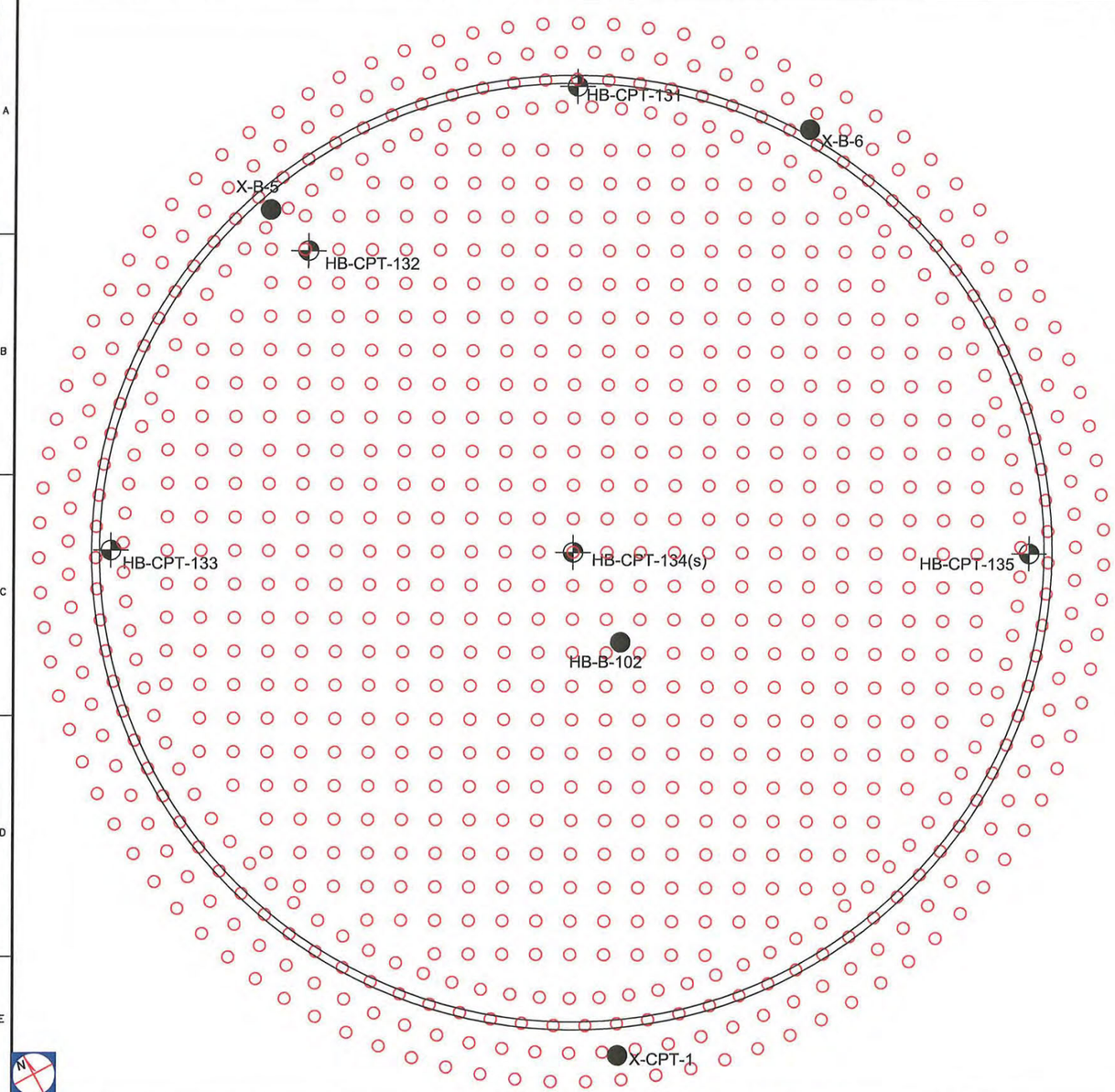

NOTES:

1. ○ Indicates Primary Ø36" Stone Column, 36' Depth.
2. ● B-1 Indicates Pre-Boring location.
3. ⊕ CPT-1 Indicates Pre-CPT location.
4. Pre-Auger all stone columns.

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VANCOUVER ENERGY
Tesoro Savage Petroleum Terminal LLC

PROJECT: TESORO SAVAGE VANCOUVER ENERGY DISTRIBUTION TERMINAL
PORT OF VANCOUVER, WASHINGTON

DESCRIPTION: TANK 5 PLAN VIEW; STONE COLUMN LAYOUT

DESIGN:	START DATE: 4-14-15	SCALE:	1"=15'
DRAWN: S P B	PRINT DATE:	PROJECT MANAGER:	
CHECKED:	APPROVED: M. ROHRBACH	SIAAE:	24x36
DRAWING NUMBER		SHEET	REV.
HB-12		12/14	0

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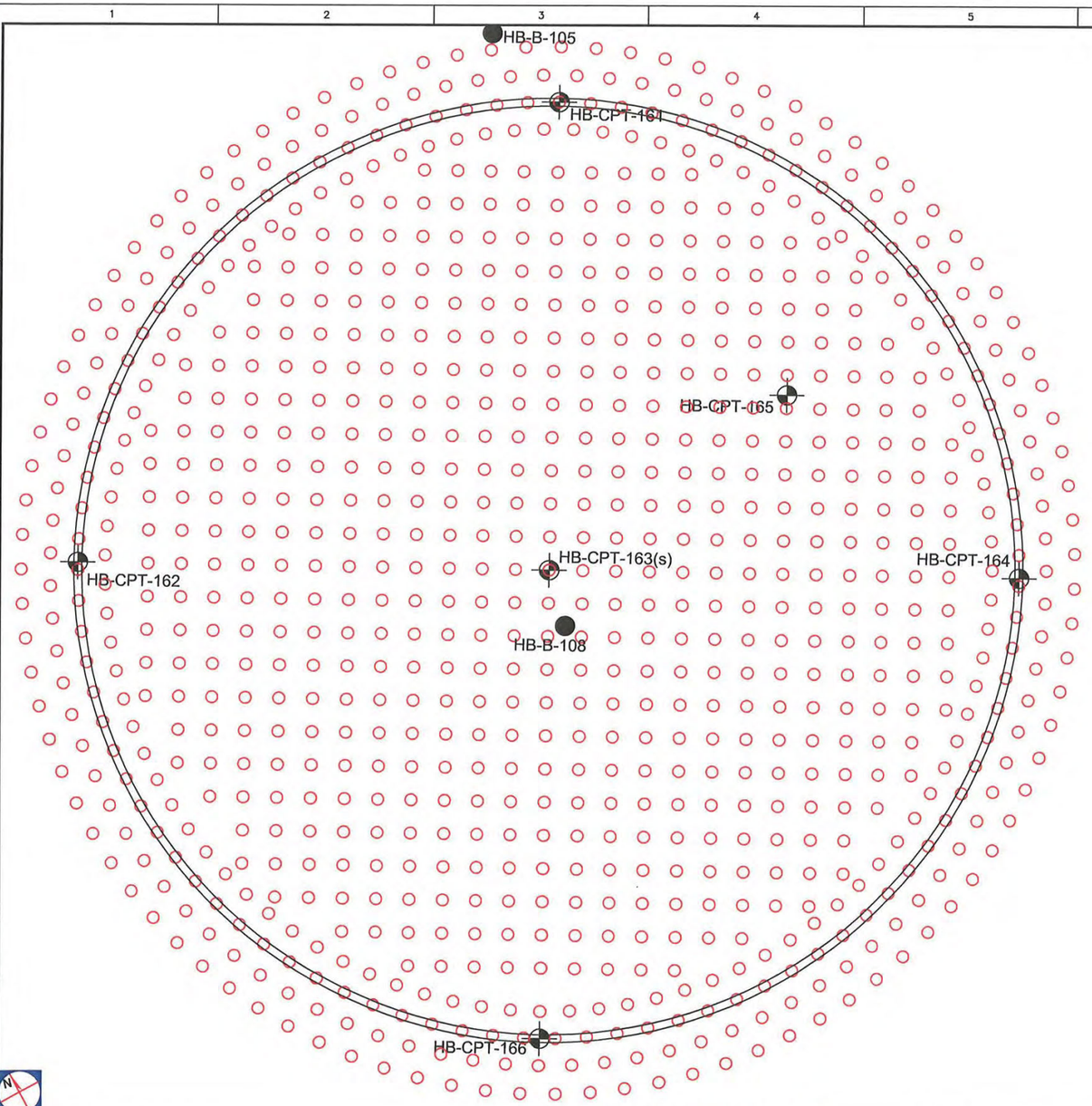

NOTES:

1. ○ Indicates Primary Ø36" Stone Column, 38' Depth.
2. ●B-1 Indicates Pre-Boring location.
3. ⊕CPT-1 Indicates Pre-CPT location.
4. Pre-Auger all stone columns.

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HAYWARD BAKER
A KELLER COMPANY

VANCOUVER ENERGY
Tesoro Savage Petroleum Terminal LLC

PROJECT: TESORO SAVAGE VANCOUVER ENERGY DISTRIBUTION TERMINAL
PORT OF VANCOUVER, WASHINGTON

DESCRIPTION: TANK 6 PLAN VIEW; STONE COLUMN LAYOUT

DESIGN:	START DATE: 4-14-15	SCALE: 1"=15'	
DRAWN: S P B	PRINT DATE:	PROJECT MANAGER:	
CHECKED:	APPROVED: M. ROHRBACH	SIAAE: 24x36	
DRAWING NUMBER		SHEET	REV.
HB-13		13/14	0

Tank Farm & Shoreline REV1

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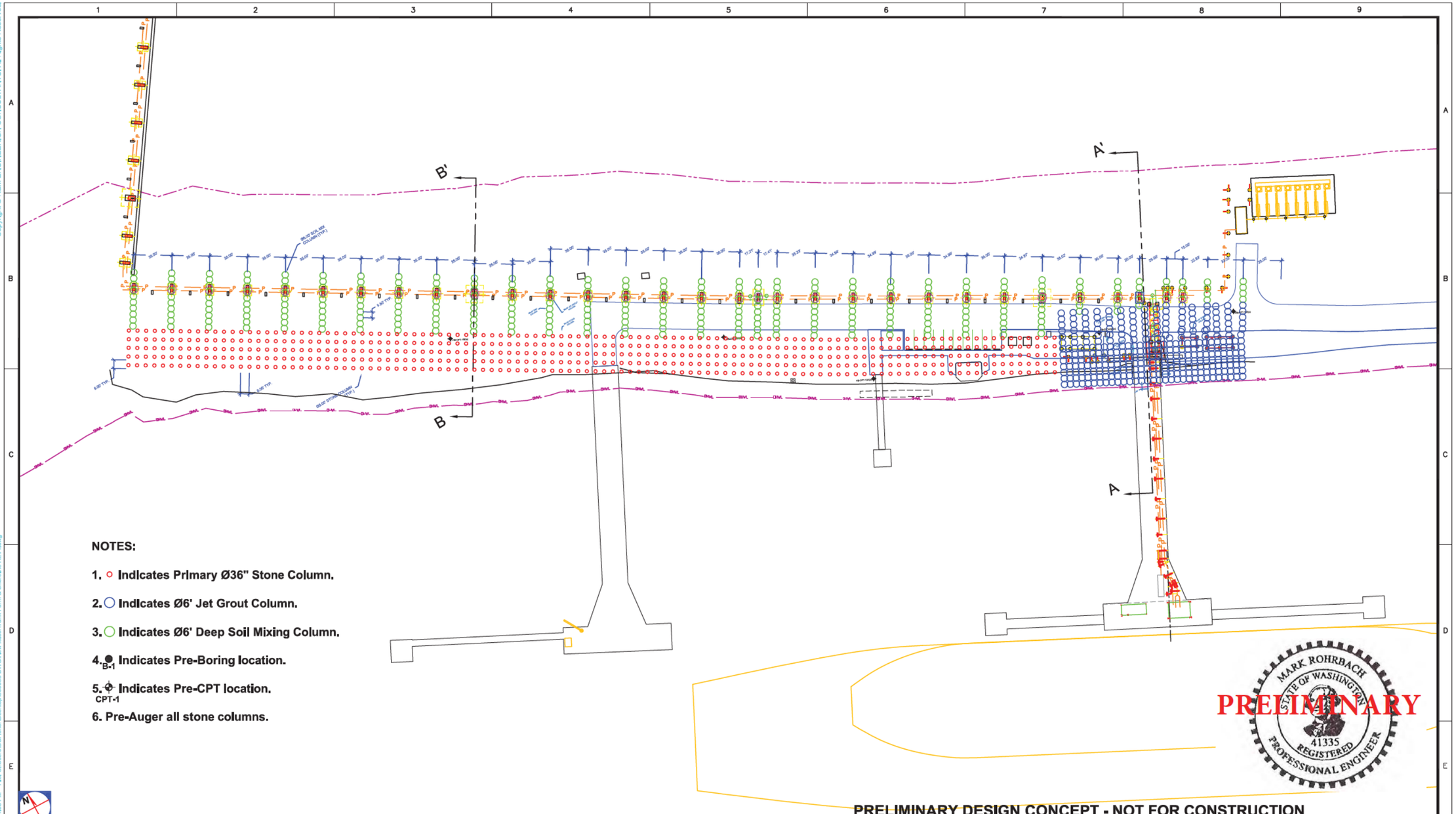

NOTES:

1. ○ Indicates Primary Ø36" Stone Column.
2. ◯ Indicates Ø6' Jet Grout Column.
3. ◯ Indicates Ø6' Deep Soil Mixing Column.
4. ● B-1 Indicates Pre-Boring location.
5. ⌖ CPT-1 Indicates Pre-CPT location.
6. Pre-Auger all stone columns.

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STATE OF WASHINGTON
41335
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PROFESSIONAL ENGINEER

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NO.	DATE	REVISION	BY	CK'D	APP

HAYWARD BAKER
A KELLER COMPANY

Seattle Office
11100 E Marginal Way South
Tukwila, Washington 98168
206-223-1732 Fax 206-223-1733

VANCOUVER ENERGY
Tesoro Savage Petroleum Terminal LLC

PROJECT: TESORO SAVAGE VANCOUVER ENERGY DISTRIBUTION TERMINAL
PORT OF VANCOUVER, WASHINGTON

DESCRIPTION: STONE COLUMN, SOIL MIXING, & JET GROUT LAYOUT AREA 400

DESIGN:	START DATE: 4-14-15	SCALE: 1"=40'	
DRAWN: S P B	PRINT DATE:	PROJECT MANAGER:	
CHECKED:	APPROVED: M. ROHRBACH	SIAAE: 22x34	
DRAWING NUMBER: HB-14		SHEET: 14/14	REV.: 0

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