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10 March 2015

Mr. Stephen Posner
Energy Facility Site Evaluation Council
Washington Utilities and Transportation Commission
P.O. Box 43172
Olympia, WA 98504-3172

Subject: Vancouver Energy
EFSEC Application No. 2013-01, Docket No. EF131590
Response to EFSEC Draft EIS Data Request 4

Dear Mr. Posner:

On behalf of Tesoro Savage Petroleum Terminal LLC (the Applicant) , BergerABAM is providing a response to the Energy Facility Site Evaluation Council's (EFSEC) Draft EIS Data Request 4, dated 9 February 2015.

Please feel free to contact me at 206/431-2373, or irina.makarow@abam.com, if you have any questions about this submittal. We look forward to further coordination with you, your staff, and EFSEC's consultants.

Sincerely,

Irina Makarow
Senior Environmental Project Manager

IM:nb
Attachment

cc: Kelly Flint, Savage Companies
Jay Derr, Van Ness Feldman

Code	Data Request Item	Applicant Response
Seismic Analysis		
SA-01	PDERI Sect. 2.2.2.9, p. 2-20. The oil storage tanks are 240 ft in diameter and 48 ft high. The maximum stored capacity for the oil is 360,000 bbl, which corresponds to a liquid height of ~45 ft. The working capacity is 340,000 bbl (height ~42.5 ft). What design liquid height (H) will be used to compute the impulsive and convective (sloshing) responses of the oil-tank system?	The design liquid level for this tank used the full shell height. Typically, for overfill protection, the client will set the high-high liquid level (HHLL) at a point lower than the top of the shell. In fact, the HHLL is less than that of the lowest rafter section in the tank. This makes the shell design conservative for hydrostatic loading. Per Annex E of API 650, the Hydrodynamic Hoop stress in the shell is calculated on pages 85-90 of 92 in the calculation sheets for the Insulated Tank and pages 84-89 of 91 for the uninsulated tank. These calculations take into account the impulsive and convective sloshing response. The thicknesses for these tanks have already considered the additional hoop stress from seismic loads.
SA-02	PDERI Sect. 2.2.7.3, p. 2-40. If an earthquake causes a break in the oil pipeline, how will it be detected and how will the oil flow through the pipeline be shutdown to minimize the leak volume?	If an earthquake should occur, seismic event switches will be triggered, sending a signal to the Facility PLC system and initiating an emergency shutdown (ESD). As part of the ESD, the transfer pipeline isolation valves will be closed and pumps, if operating, will be shut down. Also, if an earthquake causes a break in the transfer pipeline while a transfer is occurring in the pipeline, the leak detection system consisting of two flow meters located at opposite ends of the transfer pipeline will detect the leak based on a differential flow rate between the two flow meters, and shut down the transfer pumps.
SA-03	PDERI Sect. 2.3.1.2, p. 2-54. A number of building codes are listed for the facility design, but oil storage tanks are designed to the seismic provisions in the 12th edition of API 650 standard, which is not listed. Will the seismic design of the tanks be according to Annex E of this standard? Assuming the answer is yes, then the following questions deal with the application of this standard: a. API 650 Sect. E3. Which Seismic Use Group (SUG I, II, III) will be selected? b. API 650 Sect. E4. Which method (E.4.1 or E.4.2) will be used to determine the ground-motion parameters if the soil is improved to mitigate the liquefaction hazard in Area 300 where the tanks are located? c. API 650 Sect. E.4.6.2. What is the approximate impulsive period of the oil-tank system? d. API 650 Sect. E.6.1.6. Will soil-structure interaction analysis be conducted, and if so how will the results be used to compute the impulsive response?	The seismic design of the tanks will be according to Annex E of API 650. a. The current calculations are based upon a Seismic Use Group I. b. From the perspective of ground improvement design input parameters, API 650 Sections E.4.1 and E.4.2 are the same as ASCE 7 and IBC 2012. HBI suggests API 650 be added to the list of design standards. c. The impulsive period used for each tank is = 16 sec. d. API 650 Section E.6.1.6 allows the use of soil-structure interaction to contribute to damping effects that reduce the design seismic forces in certain situations. One of the requirements to take advantage of this is that the tanks must be mechanically anchored to the foundation, which we are not. Therefore, we did not consider the effects of soil-structure interaction on the effective damping and period of the vibration.
SA-04	PDERI Sect. 2.3.1.2, p. 2-54. The list of codes and standards does not include the newly	a. The "Moderate" design classification has been assigned.

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	released ASCE 61-14 standard, "Seismic Design of Piers and Wharves." However, the September 5, 2014 GRI geotechnical report on the dock facility states this standard be used to design the structures in the Marine Terminal (Area 400)? The following questions deal with the application of this standard: a. What Design Classification will be assigned (High, Moderate, Low)? b. The minimum seismic hazard and performance levels depend on the Design Classification, as indicated in Table 2-1 of the standard, but all three must demonstrate "life safety protection" during the Design Earthquake (DE), as defined in the ASCE 7-05 standard. The Applicant should be aware that the DE peak ground acceleration (PGA) used in the liquefaction evaluation in the ASCE 61-14 standard is 2/3 of the Maximum Considered Earthquake (MCE) PGA that is required for liquefaction assessment in the ASCE 7-10 standard. Thus, the earthquake risk of the Marine Terminal, if designed under the ASCE 61-14 standard, is potentially higher than the risk associated with the onshore structures designed to ASCE 7-10. Furthermore, ASCE 61-14 refers to ASCE 7-05 for the ground-motion data, but that standard has been superseded by the ASCE 7-10 standard. How will the Applicant address these inconsistencies? Will it introduce exceptions to certain provisions in the ASCE 61-14 standard to make it consistent with ASCE 7-10? c. The ASCE 61-14 standard allows earthquake-induced permanent ground deformation, provided it does not compromise the performance goals for the structures and foundations. How will the portion of the pipeline in the terminal (Area 400) be designed to accommodate this deformation?	b. In regard to the statement: "The Applicant should be aware that the DE peak ground acceleration (PGA) used in the liquefaction evaluation in the ASCE 61-14 standard is 2/3 of the Maximum Considered Earthquake (MCE) PGA that is required for liquefaction assessment in the ASCE 7-10 standard. Thus, the earthquake risk of the Marine Terminal, if designed under the ASCE 61-14 standard, is potentially higher than the risk associated with the onshore structures designed to ASCE 7-10.": The soils are susceptible to extensive liquefaction for 2/3 MCE and full MCE; therefore, for the soil conditions at this site, the liquefaction hazards are consistent for the onshore and marine structures. In regard to the statement: "Furthermore, ASCE 61-14 refers to ASCE 7-05 for the ground-motion data, but that standard has been superseded by the ASCE 7-10 standard. How will the Applicant address these inconsistencies?": ASCE 61-14 refers to ASCE 7-05 for the definition of the DE. The current USGS 2008 ground-motion database for this analysis was used to develop both the ASCE 7-05 and ASCE 7-10 design level earthquakes for consistency. c. The pipelines have expansion joints and carefully planned support locations to isolate the pipes on the dock from those on the shore. The joints are designed to accommodate up to 4 inches of differential movement between the dock and the shore.
SA-05	PDERI Appendix A. Figures 27A & 32A present the seismic shear-wave velocities obtained from	GRI collected the initial travel-time versus depth data. The travel-time versus depth data for CPT-1 and CTP-6 is provided in Attachment 1 to this letter. Measurement of shear wave velocity is performed at

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Seismic Analysis		
	the seismic CPT-1 and CTP-6. Provide the travel-time versus depth data from this velocity survey and provide the details on the method used to generate the seismic shear-waves.	selected intervals by striking a beam pressed firmly against the ground. The shear wave velocity is calculated based on the distance between the geophones divided by the incremental travel time between the geophones (corrected for geometry).
SA-06	PDERI Appendix B, Target Bedrock Spectrum. Explain how the risk-targeted probabilistic bedrock spectrum was obtained for periods other than 0.2 sec and 1.0 sec.	The risk targeted probabilistic spectrum can be derived by applying directivity factors and risk coefficients to the USGS 2,475-year PSHA values. The approach presented in Section 21.2.1.1 of ASCE 7-10 was adopted for obtaining the risk coefficients and directivity factors for periods other than 0.2 sec and 1.0 sec.
SA-07	PDERI Appendix B, Estimated Site Response. Provide more details on the derivation of the SAR curves in Figure 9B. a. Were both horizontal components of the 7 time histories listed on p. B-7 input to the D-MOD computer program? b. Were the time histories spectrally matched to the target spectrum or were constant scale factors applied to the original records? c. Is each SAR curve an average of the SAR curves from the site-response analyses? d. Were the time histories of pore-pressure buildup examined to see whether they showed liquefaction, consistent with the results of the simplified liquefaction-evaluation procedures? e. Were uncertainties in the timing of the onset of liquefaction, as well as the uncertainties in the soil properties, considered in the analyses? f. How were the "Site-Specific" spectral acceleration curves, which are only based on relatively few periods (0.0, 0.1, 0.2, 0.3, 0.5, 1.0, 2.0, 3.0, 5.0 sec), derived from the irregularly shaped SAR curves, which were derived for many more periods? Was the ordinate of this rough-looking SAR curve at a given period used or was the SAR curve smoothed first before computing the "Site-Specific" spectrum? The SAR curves in the aforementioned September 5, 2014 GRI geotechnical report appeared to have been smoothed.	a. ASCE 7-10 specifies a minimum of five recorded or simulated horizontal time histories for site response modeling. For the seven time histories summarized on page B-7, one horizontal component of each time histories was used as an input to D-MOD for the analyses. b. Amplitude-scaling was employed to modify the selected time histories to reasonably match the bedrock target spectrum as discussed on page B-10. c. The Spectral Amplification Ratio (SAR) curves presented on Figures 9B and 10B represent the average SAR of the seven time histories used in the analyses. d. The D-MOD pore pressure results were examined and also indicated liquefaction would occur in the zones indicated by the simplified liquefaction-evaluation procedures. e. Uncertainty in the timing of the onset of liquefaction was considered by using seven independent time histories in the site response analysis to account for the uncertainties of ground motion characteristics, including magnitude, intensity, duration, and cycles of loading. Uncertainties in soil properties were considered by comparing the site response results of three different shear wave velocity profiles generated based on measured shear wave data obtained from the site-specific CPT soundings. f. The SAR on the Vancouver Energy-Upland Facility report show the ordinates of all the data points obtained from the D-MOD analysis while the SAR on the Vancouver Energy-Dock Facility report shows the ordinates corresponding to the following selected spectral periods: 0.0 s, 0.1 s, 0.2 s, 0.3 s, 0.5 s, 1.0 s, 2.0 s, 3.0 s, 4.0 s, and 5.0 s. The "Site-Specific" spectral curves were developed by multiplying the SAR and target bedrock spectral values at selected spectral periods. The site-specific curves shown in the report were smoothed for plotting purposes.

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SA-08	PDERI Appendix B, Estimated Site Response. The response spectra in Figures 10B & 11B only extend to 5 sec. However, the fundamental sloshing period for the proposed tanks is likely to be around 10 sec. How will the response spectra at these long periods be derived?	The response spectra in Figures 10B and 11B show 80 percent of Site Class E curve controls the site response for period greater than 2 seconds. Therefore, it is our opinion that the 80 percent Site Class E curve can be extended to derive the spectral values at longer periods as discussed in Section 11.4.5 of ASCE 7-10 (i.e., Sa decays at slope of 1/T).
SA-09	General Questions. How will the seismic design of the various components of the facility proceed up to construction? Will a single structural engineering firm prepare a document providing the seismic design criteria that include the ground motions to be used in the design and the applicable codes and standards for the various structures comprising the facility? This document would also contain any exceptions taken to certain provisions in these codes and standards, if deemed necessary.	The seismic design of Facility components is being considered and prepared by several engineering firms in accordance with each firm's area of expertise. A single structural engineering firm will not prepare a document providing the seismic design criteria that include the ground motions to be used in the design and the applicable codes and standards for the various structures comprising the Facility.
SA-10	Questions on ground improvement. a. When will the details of the proposed ground-improvement schemes for the site become available? b. During and after the ground improvement construction, what in situ testing will be performed to check that the ground-improvement criteria have been satisfied?	a. The details of the proposed ground improvement are anticipated to be submitted to EFSEC in early April 2015. b. Primarily post-improvement verification CPT testing will be used. Based on the results of the verification CPT tests, HBI may also perform seismic CPTs, drilled geotechnical borings, and static plate load tests.

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SA-11	Hayward Baker Inc. Letter Additional exploration either by drilling borings or advancing cone penetrometer test (CPT) probes has been proposed for Areas 300, 400 and 500. The additional exploration in Area 400, i.e. the Terminal along the Columbia River, has been proposed as one boring or cone probe for every 200 feet of pipeline for the approximately 1050 feet of pipeline that runs along the shoreline. Will any of those additional explorations be performed immediately offshore to help better define the subsurface profile from the onshore-to-offshore portions of the terminal? It currently appears that no previous borings or probes were located offshore, although the depth to non-liquefiable soils seems to be increasing in the offshore direction, possibly increasing the estimated deformations from lateral spreading and unstable shoreline slopes.	The design conservatively assumes that the soils below the average high water level are subject to liquefaction. It is also being conservatively assumed that these liquefiable soils extend to the dense gravel interface; GRI used data points from previous investigations and available dock pile-driving logs to estimate the location of this interface. The design, therefore, will protect the structures at the top of the bank in Area 400, as well as the transfer pipelines in Area 500.
SA-12	Hayward Baker Inc. Letter The preliminary concept of ground improvement presented in this document states that the vibro-replacement stone column ground improvement will extend to only 40 feet below the ground surface in the Area 300 Storage occupied by the large tanks, while the top of the non-liquefiable dense gravel layer is up to 60 feet deep there, as indicated in borings and cone penetrometer test (CPT) probes. The GRI geotechnical report states that liquefaction will occur down to the top of the gravel layer throughout the site. For the Area 500 pipelines, the Hayward Baker proposal indicates that the stone columns will extend down to the top of the gravel layer. Is the decision to limit the stone column depth to 40 feet in Area 300 based on an estimate of post-ground improvement settlement that can be tolerated by the tanks and other structures there? Similarly, Hayward Baker has proposed that depths of stone columns in the Area 400 Marine	The reviewer is correct – the ground improvement design is intended to limit the settlement to the requirements identified in API 650 to protect structures (or more conservative settlement criteria as established by the Applicant and described elsewhere). The purpose of the ground improvement program is not to prevent all liquefaction. The purpose of the ground improvement program is to limit the potential for damage related to the occurrence of liquefaction by ensuring the maximum anticipated liquefaction-induced settlement is consistent with the Facility structural and safety requirements. Some minor amount of liquefaction below the bottom of improved ground and above the top of the gravel layer is anticipated and has been accounted for.

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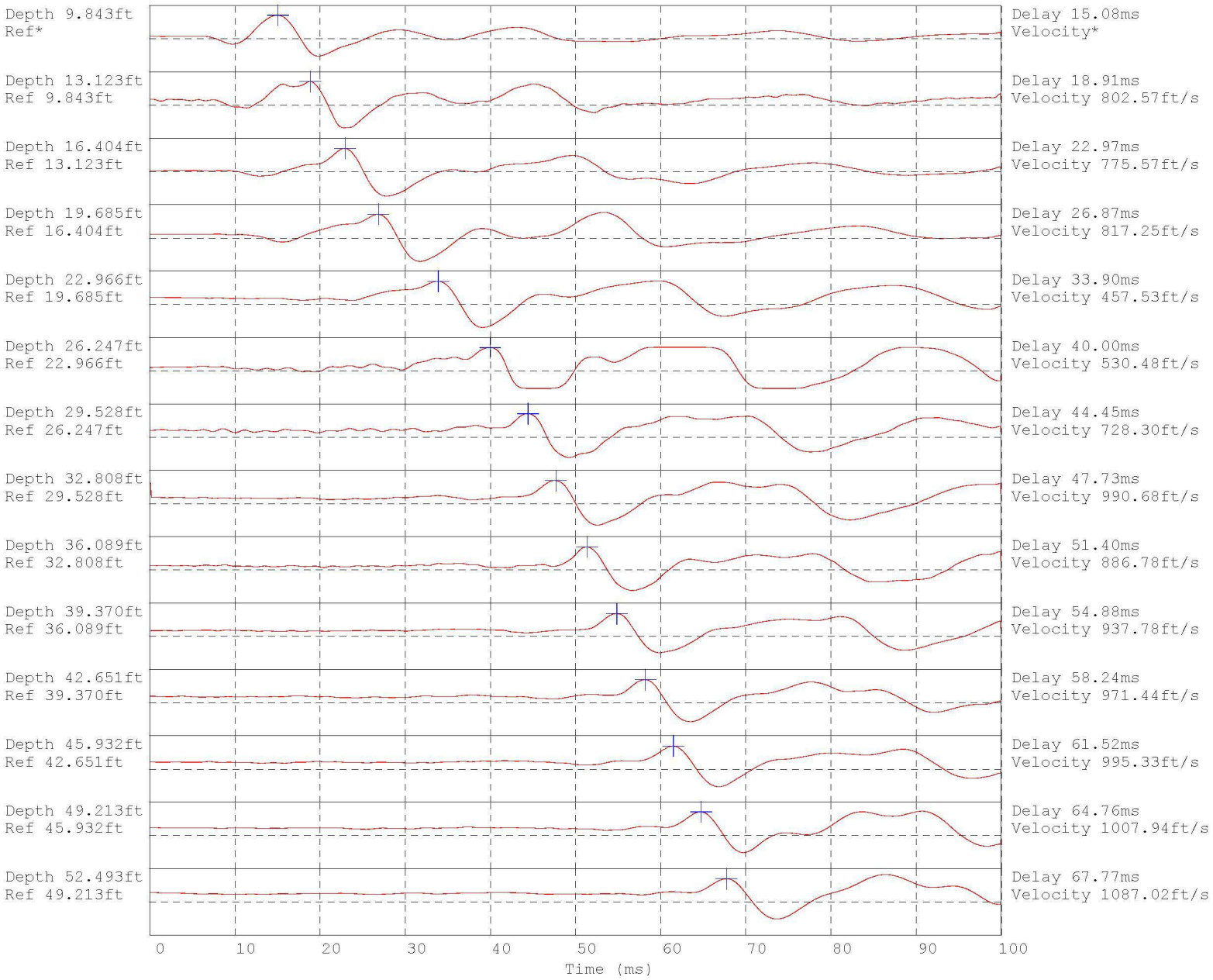
	Terminal be 65 feet, while the depth to the top of the non-liquefiable gravel is up to 87 feet (e.g. at Boring B-23). Is that proposed stone column depth based on estimates of settlement and lateral spread deformation (page 6 of the letter indicates a target post-improvement lateral deformation of only 2-inches) that will be tolerable for the pipelines and other structures in Area 400?	The ground improvement design for Area 400 is based on estimates of settlement and lateral spread deformation (page 6 of the letter indicates a target post-improvement lateral deformation of only 2 inches) that will be tolerable for the pipelines and other structures in Area 400. The Applicant notes, however, that based on the results of the supplemental exploration effort and to avoid impacts below the OHWM, the preliminary design for Area 400 is being refined to account for these deeper liquefiable soils and is anticipated to include stone columns, deep soil mixing, and jet grouting methods.
SA-13	Hayward Baker Inc. Letter The letter indicates that post-treatment verification testing will consist of 1 CPT per 10,000 SF of improved ground area for Area 300, and 1 CPT per 200 feet of pipeline alignment for Area 500, but does not indicate specific verification target for Area 400 Marine Terminal. Can that recommendation be provided, including the recommended areal distribution of CPTs both onshore and, if appropriate, offshore?	Within Area 300, HBI's design calls for about four CPTs per tank and 1 CPT per 20,000 square feet of ground improvement area related to the transfer pipeline. Please note the CPT frequency below the pipeline in Area 300 has been modified to reflect changes in the width of the pipeline corridor. Within Area 400, the ground improvement system has been modified to include soil mixing and jet grout methods. The verification testing requirements will be finalized when the design is finalized and will be consistent with industry standards regarding the type of verification testing and frequency of testing. HBI understands Area 500 is immediately adjacent to Area 400. Any ground improvement works performed in Area 500 will be verified using the same methods to be used for Area 400.

IM:nb
Attachment
10 March 2015

Attachment 1

GRI/CPT-1/W1114

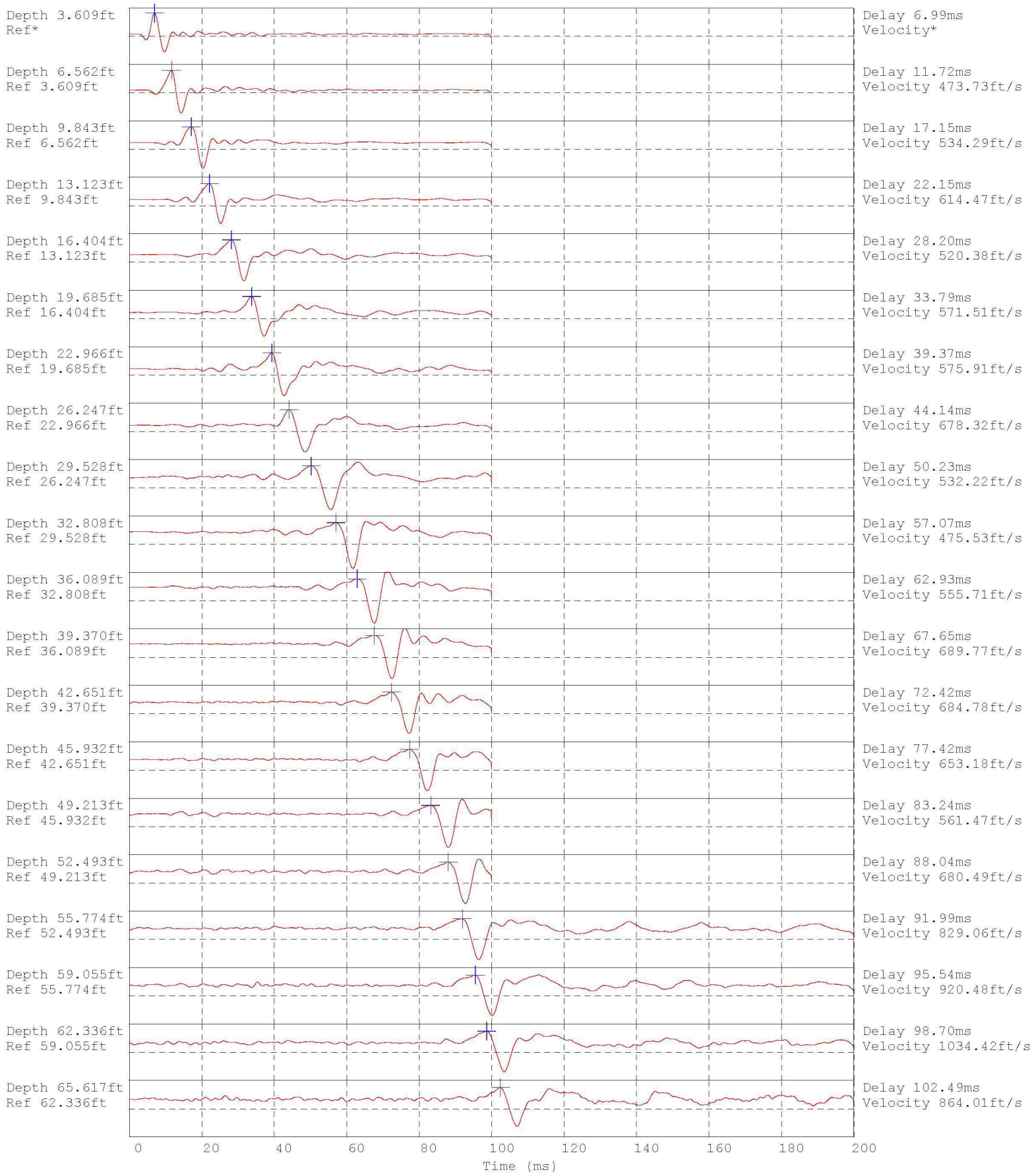


Hammer to Rod String Distance 1.3 (m)
* = Not Determined

Operator: SAV/CM
Sounding: VEI419CPT1

CPT Date/Time: 6/7/2013 1:16:30 PM
Location: CPT-1/NW GATEWAY AVE PT OF VANC

GRI/CPT-6/W1114



Hammer to Rod String Distance 1.3(m)
* = Not Determined

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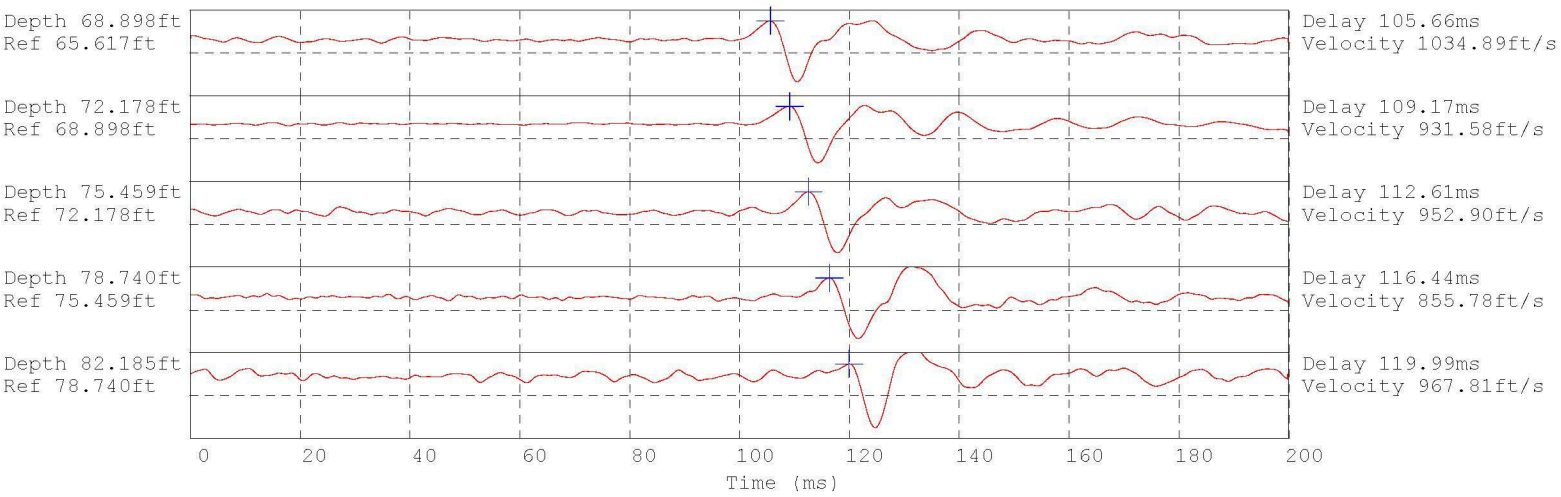
CPT Date/Time: 7/29/2013 12:26:29 PM
Location: CPT-6/NW HARBORSIDE VANC.

GRI/CPT-6/W1114



Hammer to Rod String Distance 1.3 (m)
* = Not Determined

Operator: SAV/CM
Sounding: VEI422CPT6

CPT Date/Time: 7/29/2013 12:26:29 PM
Location: CPT-6/NW HARBORSIDE VANC.