HDR

CORNFORTH CONSULTANTS

Seismic Evaluation of Big Creek Dams No. 1 and 2

Phase 3 – Engineering Evaluation and Corrective Action Alternatives

Newport Big Creek Dams

Newport, Oregon

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ENGINEER
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Acronyms

ac-ft	acre-feet
Alt	Alternative
BC 1	Big Creek Dam No.1
BC 2	Big Creek Dam No.2
BH	Borehole
CIUC	Isotropically Consolidated Triaxial Compression
CPT	Cone Penetration Testing
CSR	Constant Strain Rate
CSTP	Continuous Standard Penetration Testing
CycDSS	Cyclic Direct Simple Shear
DSS	Direct Simple Shear
FLAC	Fast Lagrangian Analysis of Continua
FOS	Factors of Safety
FSV	Field Shear Vane
H	Horizontal
HDR	HDR Engineering, Inc.
LIR	Load-Increment Ratio
MEG	Marine + Earth Geosciences
NEPA	National Environmental Policy Act
NHPA	National Historic Preservation Act
NPDES	National Pollutant Discharge Elimination System
NSHM	National Seismic Hazard Maps
OCR	Overconsolidation Ratio
ODFW	Oregon Department of Fish & Wildlife
PGA	Peak Ground Acceleration
PMF	Probable Maximum Flood
PSHA	Probabilistic Seismic Hazard Analysis

RCC	Roller compacted concrete
SCPT	Seismic Cone Penetration Testing
SCPTu	Seismic Cone Penetration Testing with pore pressure measurements
SHANSEP	Stress History and Normalized Soil Engineering Properties
SPT	Standard Penetration Testing
UC	University of California
USGS	United States Geological Survey
V	Vertical

Executive Summary

HDR Engineering Inc. (HDR) has completed the Phase 3 assessment of the static and seismic stability of Big Creek Dam No. 1 (BC 1) and Big Creek Dam No. 2 (BC 2) for the City of Newport (City). This assessment included 1) an update of the seismic hazard characterization and characteristic earthquake time histories at the site based on the most recent research; 2) additional site characterizations including borings and cone penetration testing, sampling and laboratory testing; 3) analysis and evaluation of the field and laboratory test results; 4) developing a more detailed and comprehensive geologic model of the two dam sites along with generalized profiles and cross-sections for engineering evaluations; 5) an update of the previously completed seepage, static and post-earthquake stability analysis; 6) evaluating the expected seismic response (deformations) of both existing dams to a range of potential earthquakes at the site; 7) developing and evaluating alternatives for corrective actions for BC 1 and BC 2; 8) development of decision level cost estimates for the corrective action concepts; and 9) providing a preliminary environmental permitting overview for the corrective action concepts. The findings from this evaluation are summarized in this report.

Verification of Seismic Response Deficiencies

The static and post-earthquake stability and seismic response analyses presented in this report have confirmed seismic deficiencies at both existing dams (BC 1 and BC 2). The estimated deformation of each dam in response to potential earthquakes suggests a high potential for significant damage and/or failure to occur.

Two methods of evaluation have been used to assess potential deformations including 1) the development of a numerical model based on an industry accepted “Newmark” analysis methodology, and 2) an empirical correlation between seismic loading and observed deformations at a variety of existing dam sites (i.e. case history data) The estimated crest deformations for both dams based on these methods were reasonably similar. The numerical evaluation method results reflect the more rigorous approach and predict larger potential deformations consistent with the unusually long duration of ground shaking that would be associated with a Cascadia earthquake event.

The selection of an appropriate earthquake loading conditions for dam safety evaluations and design represents a critical aspect of the study. The Cascadia Subduction Zone (CSZ) hazard is substantial (Richter Magnitude 9) and the understanding of this magnitude of event, and the corresponding peak ground accelerations, and duration of strong shaking that would result at the Newport dam sites is continuing to evolve throughout the industry. Based on the current standard of practice at both the state and federal levels of jurisdiction in the northwest, ground motions with expected recurrence intervals of up to 4975-years have been used as the basis of our assessment and design presented in this report.

Alternatives for Corrective Actions

Based on the outcome of the stability analysis and evaluation, HDR developed three different alternatives to provide a solution for both dams that would provide adequate dam safety and for a continuous drinking water supply following a significant earthquake event. The repairs for BC 1 would be very costly for the gained benefit as the dam does not hold enough water to pay off the costs of its remediation. A decision was made together with the City to not proceed with any corrective actions for BC 1.

Alternative 1 consists of a raise of BC 2 to include the current water storage from BC 1, recovery of storage in the upper reservoir due to sediment accumulation, and increased storage for future water demands in the city. This alternative presents some challenges as the existing reservoir and outlet works would need to stay operational during construction. The foundation excavation volume for this alternative is very large and sufficient construction material would have to be found to replace the excavated foundation material as well as the new embankment section. Because of the potential for significant deformations of the upstream slope of the dam, a new outlet structure would have to be built through the right abutment of the existing dam. Further, a spillway and fish ladder would need to be constructed. This alternative is doable but does not present the most cost effective and most feasible option.

Alternative 2 consists of a new roller compacted concrete (RCC) dam at a location just downstream of BC 2 where the topography of the valley narrows the most.

Alternative 3 consist of a new embankment (earthen) dam at the same location as Alternative 2.

Both alternatives 2 and 3 are acceptable solutions for corrective actions and represent a “least cost” solution for the project purposes outlined above.

Decision Level Estimates of Probable Costs

Decision level cost estimates were developed for Alternatives 2 and 3. At this time, the costs exclude some important project elements as the extent and dimensions of those elements is unknown at this stage of the project. They also include some significant cost uncertainties and hence are not suitable for establishing project funding. Future preliminary design will be required to provide the basis for a funding level cost estimate. The Preliminary design should include such elements as the spillway for Alternative 3, fish ladder, access road, and pipeline to the water treatment plant.

From a decision making standpoint, the cost estimates show that both Alternatives are similar and that a decision on the preferred dam type and configuration can be based on a number of other considerations such as long term operation and maintenance, owner preference and cost risk uncertainties.. Based on discussions with the City, Alternative 2 is recommended for preliminary design. Should a significant issue be identified with this Alternative during the early stages of preliminary design, Alternative 2 can be pursued as the preferred configuration.

Conclusions and Recommendations

Alternative 2 (RCC dam) provides a number of potential advantages to the City such as a relatively short construction timeline, proven seismic performance of concrete dams, lower cost uncertainty, smaller project impact footprint, and preferred spillway configuration

HDR recommends moving forward with a preliminary design of Alternative 2 (RCC dam). The preliminary design will include both geophysical, and boring characterization of the proposed site, a budget level cost estimate, environmental permit preparation, access road refinement, and additional modeling which is required by the state.

1 Introduction

HDR began working with the City of Newport in 2009 on the design and construction of a new water membrane filtration treatment plant. The water treatment plant is supplied with water stored in two man-made reservoirs in Big Creek, denoted Big Creek Dam No. 1 (BC 1) and Big Creek Dam No. 2 (BC 2). BC 1 reservoir is adjacent to the new treatment plant, and BC 2 reservoir is located approximately 1 mile upstream. These reservoirs were formed by the construction of an earthen dam at each location.

During construction of the new plant, geotechnical explorations were performed for the design of a new intake structure located in the BC 1 reservoir. A single boring drilled in October 2011 by Foundation Engineering, Inc. (FEI) showed foundation material to generally consist of very soft to soft clayey silt and very loose to loose silty sands. The initial boring and engineering evaluation also identified that the loose silty sand soils have a potential for liquefaction during a seismic event and that further dam safety related evaluations were indicated.

BC 1 is 315 feet long with a maximum height of 21 feet. The reservoir normally impounds 190 acre-feet of pool. The dam was designed by CH2M of Corvallis, Oregon and constructed by the City of Newport Public Works Department in 1951. Available design drawings depict the dam as a homogeneous compacted clay dam with embankment slopes of 1 vertical (V) on 3 horizontal (H) upstream and 1V on 2H downstream. Drawings show a 5-foot-thick granular drainage zone at the foundation level of the downstream third of embankment.

BC 2 was originally constructed in 1969 and modified and raised in 1975 and 1976. The dam was to be raised by 17 feet to an overall height of 56 feet and a length of 450 feet. The dam is shown with a central core trench and a downstream drainage system. Foundation materials are described as medium to stiff sandy silts over a weak siltstone. The CH2M-Hill, (CH2M-Hill, Predesign Report for the Raising of Big Creek Dam No. 2, City of Newport, Oregon, 4 Sep 1974), states that a seismic coefficient of 0.1 g was used for a pseudo-static analysis and a bedrock acceleration of 0.18 for a Newmark analysis which was used to estimate potential displacement during a seismic event.

1.1 Project Background

As a result of the potential dam safety-related concerns identified in the initial boring at the site, the City requested HDR perform a seismic evaluation of the embankment dams for both BC 1 and BC 2 reservoirs. This evaluation was completed in 2011 and 2012 and consisted of site investigations to characterize the dams’ earthen and foundation materials, a probabilistic seismic hazard analysis (PSHA), a geologic hazard assessment, and geotechnical analyses to determine the stability of the dams in the event of potential seismic events. The initial site investigation and characterization program consisted of borings, cone penetration testing, seismic refraction geophysical testing, and laboratory testing.

1.2 Previous Report and Results

In February 2013, HDR submitted the “Big Creek Dam No. 1 and No. 2 Preliminary Geotechnical Investigation and Seismic Evaluation” report (February 2013 Report). This is subsequently referred to as the Phase 2 investigation program. The report described the site characterization program, the soils testing program, an evaluation of the results, and the engineering analysis for the two dams. The report included regional and site geology, seismic hazards, preliminary models of subsurface conditions, results of the seepage and stability analysis, and recommendations for the two dams.

The recommendations included the following:

- The seismic safety of BC 1 was estimated to be marginal while a significant safety deficiency was identified at BC 2.
- Additional site characterizations were recommended in order to further refine stratigraphic models of the existing structures, confirm the mineralogical origin of the soils and the corresponding reasons for the low densities, further refine the engineering properties and behavior of the foundation and embankment soils, and reduce uncertainties that occurred with the limited data sampling conducted. The additional data would also be used to support alternative design concepts.
- An update of the time histories was necessary as the U.S. Geological Survey (USGS) guidelines and regulations had changed due to the available research data from the most recent Chile and Japan subduction zone earthquakes. This was necessary to create alternatives that comply with the most recent safety standards and available design criteria.
- Additional laboratory testing was recommended to further examine the soil characteristics of the additional site explorations and refine the soil properties.
- Further engineering analyses were recommended to include the newly analyzed data and use it for computer models to simulate the behavior of the dams in case of a seismic event.
- Based on the findings of the additional analysis, corrective actions would be developed to mitigate the stability problems of the two dams. A range of rehabilitation concepts and methods was recommended for the next phase of the project.

The results presented in this report have subsequently been described as the Phase 2 investigation program.

1.3 Scope of Current Phase

Beginning in July 2014, HDR performed additional (Phase 3) site characterization and further engineering evaluations including concept design/alternative evaluations to reduce the risk of a dam failure for BC 1 and BC 2 in case of a seismic event. The original Phase 3 scope for the project included: additional site explorations, sampling and laboratory testing at both the BC 1 and BC 2 sites; updating the seismic hazard characterization of the site; developing site hydrology that would be used to assess spillway requirements for modified dam configurations; establishing analysis parameters through integrated evaluation of both the field and laboratory test data; updating the previously completed seepage, static and post-earthquake stability analyses; evaluating new seismic response with Newmark Sliding (Rigid) Block analysis based on a more comprehensive geologic model of the site; and developing and evaluating alternatives for corrective actions at both BC 1 and BC 2.

HDR performed initial engineering analysis for existing conditions and for alternative configurations involving corrective actions to mitigate the seismic stability problem for both dams in order to develop opinions on the preferred configuration of corrective actions. During the progress of the work, based on input from the City, HDR modified the approach of the corrective action alternatives to include three potential configurations at or near the BC 2 site that each included the following components of water storage along with remediation of dam safety deficiencies:

Upper Reservoir Storage:	970 acre-feet
Lower Reservoir Storage transfer:	200 acre-feet
Upper Reservoir Sediment Recovery:	100 acre-feet
Future Storage Allowance:	1,000 acre-feet
Total Storage:	**2,270 acre-feet**

The original scope of work also included a risk-based assessment to establish the appropriate level of seismic loading to be included in the design, a review of environmental conditions and clearances that would be needed, consultation with the City Engineer and the State Engineer at the Oregon Water Resources Department for dam safety, and preparation of appropriate reports and decision documents.

As a result of the revised storage and configuration requirements for the project described above the risk-based assessment to establish the appropriate seismic design criteria was removed and a preliminary design criteria of a 4,750-year seismic event was used to configure the alternatives. In addition, the scope of engineering analyses was modified in order to complete the engineering analyses within existing budget limits. The approach to engineering analyses was made in order to include evaluation of the concrete dam alternative by: 1) using a Newmark deformation analysis in lieu of a FLAC analysis for the embankment alternatives, and 2) performing a response spectrum evaluation of the concrete dam configuration.

1.4 Project Team

The Project team for the Phase 2 studies presented in this report included HDR as the principal engineer, with support from Cornforth Consultants (Cornforth), the Geotechnical Earthquake Engineering Department of the University of California, Davis (UC Davis), and Marine + Earth Geosciences (MEG).

Cornforth completed the update to the seismic hazards to the most current USGS standards and also supported the field explorations and index property laboratory testing for the samples.

UC Davis provided support to develop the laboratory testing plan and interpretation of field and laboratory testing data based on their research experience.

MEG provided the laboratory testing for all undisturbed samples.

HDR developed and directed the field and laboratory testing program, provided geologic models of the existing dams along with the engineering evaluation of the dams. Based on the outcome of the engineering analysis, HDR developed concept designs for the Alternatives described in this report along with decision level cost estimates. Three alternatives to mitigate the seismic hazard were identified. HDR also provided a preliminary review of project hydrology, and environmental review which entails a list of the necessary environmental permits associated with the proposed alternatives.

Key HDR personnel for this project included the following:

Verena Winter, P.E.	Project Manager
Keith A. Ferguson, P.E.	Principal Engineer
Scott Anderson, P.E.	Senior Geotechnical Engineer
John Charlton, P.G.	Senior Engineering Geologist
Andrew Little, EIT	Project Engineer
Michael Woodward, EIT	Project Engineer
Richard Hannan, P.E.	Technical Review
Farzad Abedzadeh, PE, PhD	Senior Dam Structural Analyst

2 Phase 3 Site Characterization and Evaluation Results

Additional site characterizations and evaluations were performed during Phase 3 and are summarized below.

2.1 Seismic Hazards and Time Histories

A seismic hazard update in support of this phase was performed based on information from recent large subduction zone earthquakes and newly released probabilistic seismic hazard maps as well as the newly released updated regional seismicity and potential ground motions from USGS's 2014 Probabilistic National Seismic Hazard Maps (NSHM) and supporting documentation. The newer information was compared to the results of the February 2013 report and Cornforth provided additional seismic hazard information and acceleration time history parameters for the site evaluation. The revised seismic hazard analyses and updated information are provided in Appendix A.

2.2 Site Explorations

Subsequent to the initial boring completed at the BC 1 site, field investigations to characterize the site subsurface conditions have occurred during two additional phases. The initial boring at BC 1 occurred in 2010 when the problem was discovered. The results of that boring were included in the previous report from February 2013. The second phase of explorations occurred in December 2011 through January 2012. These investigations consisted of mud rotary and hollow stem auger drilling, cone penetrometer testing, and a surface geophysical survey. The results of Phase 2 were included in the report from February 2013 as well. The third phase of investigations occurred in November/December 2013 and is described in this report. This Phase 3 program consisted of mud rotary drillings and cone penetrometer testing, disturbed and undisturbed sampling, and laboratory testing. A detailed discussion of the Phase 3 program of field investigations is presented in Appendix B.

2.2.1 Boreholes and Cone Penetration Testing Results

The 2013 investigations consisted of additional borings, and cone penetration testing at the BC 1 and BC 2 sites. The drilling work was performed by Western States Drilling and the cone testing was done by Northwest Geophysical Associates, Inc. as a subcontractor to Western States. The borings and cone soundings were necessary to better define the stratigraphy at the site including a better definition of the top of rock, and to collect disturbed and undisturbed soil and rock samples. Continuous Standard Penetration Testing (SPT) was performed in all bore holes. In addition to the SPT data, the procedure also allowed for the collection of disturbed soil samples. Further, undisturbed samples were obtained with 3-inch-diameter thin-walled Shelby tube samples at selected depths in the borings using a fixed piston sampler. The disturbed and undisturbed samples were needed for the second phase of laboratory testing.

The subsurface materials encountered in the BC 1 exploratory bore holes generally consisted of approximately 60 feet of silty sand, clayey silt, and silty clay alluvium overlying Nye Mudstone. The subsurface materials encountered in the BC 2 exploratory bore holes generally consisted of approximately 10 to 15 feet of silty sand and clayey silt alluvium, overlying approximately 30 to 35 feet of silty sand, clayey silt, and silty clay alluvium/colluvium, overlying Nye Mudstone.

Two Seismic Cone Penetration Test (SCPTu) soundings with pore pressure measurements were advanced at the BC 1 site and four were advanced at the BC 2 site. The two SCPTs at BC 1 and two SCPTs at BC 2 were advanced near existing borings to provide a comparison between the SCPT data and SPT data. The SCPT tip resistance, sleeve friction, and pore water pressure was measured at 2-inch increments as the SCPT instrument was pushed at a constant rate of 2 centimeters/second. Shear wave velocity and pore water pressure dissipation measurements were conducted at selected depths at all locations. Each of the four SCPTu explorations at BC 2 showed lower permeabilities at the upper elevations and slightly higher permeability with depth. All SCPTs were terminated at refusal. SCPT data is presented in Appendix B.

2.2.2 Laboratory Testing Results

Laboratory testing of soil samples collected from the 2013 site exploration were taken to MEG in Vancouver, British Columbia and, in conjunction with guidance from Dr. Jason DeJong at the University of California at Davis and HDR, a laboratory test program was developed.

The laboratory testing program was developed using Stress History and Normalized Soil Engineering Properties (SHANSEP) framework, which accounts for the stress history and the anisotropy of the soils due to different modes of shearing that are encountered during slope stability analysis. The three modes are triaxial extension near the toe of the slip surface, triaxial compression at the head of the slip surface, and direct simple shear along the base and transitions of the slip surface.

Radiography (x-ray) of the undisturbed samples was performed to evaluate the suitability of the samples for testing and develop a testing plan for the range of samples taken during the exploration. Consolidation testing consisting of load-increment ratio (LIR) and constant strain rate (CSR) consolidation methods were used to evaluate the sample disturbance and stress history profile with depth. Selected samples were then evaluated in shear by direct simple shear (DSS), isotropically consolidated triaxial compression (CIUC) testing. The SHANSEP method assumes that the behavior of the soil can be represented by the undrained shear strength, S_u, divided (normalized) by the effective overburden pressure, σ'_{v0}, with other parameters to take into account the overconsolidation ratio (OCR) and the shape of the curve, the exponent m. To evaluate the suitability of the SHANSEP framework to represent the behavior of the soil, samples were consolidated to three to four times the estimated pre-consolidation pressure identified in consolidation tests corresponding to an OCR of 1 (the soil is considered normally consolidated at this OCR). Several of the test samples were consolidated to three to four times the pre-consolidation stress and then unloaded to an overburden stress that corresponds to a known OCR, typically an OCR of approximately 4. The plots of these tests can be found in Figure D-1.5 in Appendix D. Individual test results are also found in this Appendix D. The result is a framework with which to evaluate the strength of the soil with depth and OCR.

Cyclic DSS (CycDSS) testing was performed to evaluate strength degradation with cyclic loading. Based on the CycDSS testing the soils appeared to have little to no strength degradation to 100 cycles and Post-CycDSS testing yielded soil strengths nearly the same as samples tested in static DSS. A strength reduction was evaluated by using Figure D-1.8 in Appendix D and the average plasticity index from the soils encountered. A reduction of 20 percent was conservatively used to degrade the strength properties from the peak undrained strength to the post-earthquake undrained strength.

2.3 Engineering Parameters and Assessment

The parameters developed in the laboratory testing program and those calculated and estimated based on SCPTu were used for assessing the existing dams with respect to seismic loading. Permeability values were evaluated from SCPTu dissipation testing and laboratory consolidation testing results. A set of upper and lower bound permeability values were used in the seepage analysis and subsequent stability analysis of the dams. The upper and lower bound values did not result in significantly differing Factors of Safety (FOS) for stability.

Based on the laboratory testing program and the in-situ testing which was calibrated to the laboratory testing data, the slope stability models were updated to use the SHANSEP parameters for the alluvial soils in the foundation. A maximum OCR of 4 was used, neglecting the higher OCR values in some samples that were a result of desiccation and shear stress bias at the toes of the dam where samples were collected and SCPTu testing performed. Figure D-1.4 of Appendix D shows the variation of OCR with depth for the free field environment. The dams themselves increase the overburden stress of the foundation soils and thus reduce the OCR of the underlying soils.

Use of the Field Shear Vane (FSV) and SCPTu was complicated by the drainage conditions within the soils encountered. Intermediate types of soils were encountered exhibiting characteristics of both sand-like and clay-like soils. The drainage conditions complicated the interpretation of both the FSV and SCPTu tests; however the use of dissipation testing as part of the SCPTu soundings assisted in identifying the soils that may be experiencing some degree of drainage conditions during the cone penetration testing. This determination was one of the key Phase 3 exploration program findings and helped to limit the use of the parameters estimated from the in-situ testing. Based on the dissipation and laboratory testing, the SCPTu results were subsequently calibrated with the laboratory testing strengths. This allowed the SCPTu test to validate the SHANSEP framework and parameters. As a result, the Phase 3 program found that with the strength of the foundation materials remaining relatively constant across the entire depth of these materials with appropriate consideration of OCR and overburden pressures.

Results of the engineering parameters evaluation are described in more detail in Appendix D.

2.4 Seismic Deficiency Verification

Based on the Phase 3 exploration, laboratory testing and engineering analyses a significant seismic deficiency was verified at BC 1. Analysis results indicated that this dam would be expected to fail by settlement and overtopping under seismic loading for recurrence intervals of 2,475 and 4,975 years. More frequent events, such as the 475-

and 975-year would likely result in significant damage to the dam, outlet works, water supply pump station, and ability to operate the reservoir. The location and configuration of the critical potential failure surface at BC 1 is very deep, making remediation of the site very challenging and expensive. Given the small amount of storage in the reservoir and the very large anticipated remediation costs, rehabilitation of this dam is judged as non-feasible.

The upper dam, BC 2, also has unacceptable deformations (settlement) during the 2475- and 4,975-year recurrence interval seismic events and would also likely fail due to overtopping and/or seepage through transverse cracks that would develop under these loading conditions. Similar to BC 1, the dam would also likely experience significant damage during earthquakes with more frequent return periods. While the upstream slope for BC 2 may be buttressed by some sediment that has accumulated in the reservoir, analysis results indicate that deformations of the upstream slope of BC 2 would be significant for the larger seismic events resulting in damage or failure of the outlet works, intake structure, and discharge pipeline.

A comparison of the estimates of embankment dam deformations using the Newmark analysis numerical methodology presented in this report with case history data and estimated crest deformations using the empirical methodology from Swaisgood (2003) was made to verify results and conclusions. Using the Swaisgood methodology with the range of estimated peak ground accelerations at the Newport sites for different recurrence interval Cascadia earthquake events indicate that for similar embankment dam case histories in the data base, crest deformations ranged from as little as 1.2 inches for the 475-yr return period peak ground acceleration to over 478 inches for the 4,975-yr. return period peak ground accelerations.

Based on the performance of these similar dams, estimated deformations in the range of 24 to 60 inches have a moderate to high potential for very significant damage or failure. When deformations are estimated to be in this range for these recurrence interval earthquake events, the standard of care within the dam engineering community in the US and internationally would suggest that there is dam safety deficiency and justification to take action to mitigate that deficiency. Estimated deformations of over 60-inches have a high to very high likelihood of complete failure of the dam section and not only is there a deficiency, but justification to take more expedited actions to reduce the risk of failure of the dam.

Swaisgood's estimates of percent settlement are based on the combined thickness of the dam height and the thickness of the underlying loose and/or low density alluvial soils. It should be noted that the case histories only include data up to a PGA of approximately 0.71 g and that extrapolation was necessary to project the regression line to the levels of PGA anticipated for the 2,475 and 4,975-year return period events at the Newport sites. A summary of the estimated deformations from the Newmark analyses along with Swaisgood empirical methodology is provided in Table 1 below. Note that the table cells have been colored to represent the deficiency and action categories described above. The orange cells suggest the deficiency and moderate justification for corrective actions. The red cells suggest a deficiency and justification for more expedited corrective actions. The green cells indicate deformations that are below the level associated with a safety deficiency and need for corrective actions.

Results of engineering analyses and seismic deficiency verification evaluations are presented in more detail in Appendix D.

Table 1: Summary of Estimated Embankment Crest/Downstream Slope Deformations at BC-1 and BC-2

Recurrence Interval Event (years)	Estimated Peak Ground Acceleration (PGA – g's)	Est. Deformations - Empirical (Swaisgood, 2003) (inches)			Est. Deformations – Newmark (inches)		
		Lower Bound	Best Estimate	Upper Bound	Lower Bound	Best Estimate	Upper Bound
BC 1							
2475	0.79	15	33	68	50	>76	90
4975	1.12	218	478	>478	116	>160	184
BC 2							
2475	0.79	15	33	68	32	>48	54
4975	1.12	218	478	>478	56	>96	112

3 Alternatives for Corrective Actions

Based on the results of the Phase 3 explorations, laboratory analysis, and the related engineering assessment, it became apparent that rehabilitation of the lower reservoir, BC 1, is non-feasible from an economic standpoint. The location and depth of the critical potential failure surface through the foundation soil underneath the dam makes mitigation of BC 1 very expensive relative to the amount of storage that is in the reservoir. Consequently, based on discussions with the City, HDR evaluated alternatives to mitigate BC 1 by transferring its current storage capacity to the upstream BC 2 remediation alternatives.

3.1 Alternative Options

The decision to not include BC 1 in the corrective action scenario led to increased storage capacity requirements for BC 2. Additional storage for anticipated sedimentation in the reservoirs and for future storage was also included. Future storage was based on the population projection from the 2008 Water System Master Plan (Civil West Engineering Services, Inc.). The Water System Master Plan indicates a need for a 30 percent increase in water supply by 2030. Table 2 lists theoretical storage capacities for the current reservoirs and for the future solution. The maximum theoretical future storage capacity of 2,270 acre-feet (ac-ft) was used for the configuration level layouts and cost estimates for modifications to BC 2.

Table 2. Reservoir Storage Capacities

Description	Upper Reservoir Storage (ac-ft)	Lower Reservoir Storage (ac-ft)	Sediment Storage Allowance (ac-ft)*	Future Storage Allowance (ac-ft)**	Total Storage Allowance (ac-ft)***
Replace Existing Storage	970	200	100	0	1,270
Minimum Future Storage	970	200	100	380	1,650
Maximum Future Storage	970	200	100	1000	2,270

* Future storage allowance equals an increase of 30 percent of current storage capacities combined

** Indicates estimate of current and future sediment in upper reservoir to be recovered by increased reservoir storage

*** Future storage allowance to be based on approximate minimum and maximum estimates of drought and other supply needs over 20- to 50-year planning horizon. These numbers should be appropriate building blocks for an enlargement project Purpose and Need statement that can be approved under appropriate environmental compliance activity

The project team identified five different alternatives upstream of BC 1 to secure the drinking water source for the City. All alternatives were considered but only three remained feasible and underwent an analysis. All alternatives listed below are conceptual and would require further refinement during the next phase of the project.

Figure 1 shows the five different dam axis considered for the alternatives (All figures are located at the end of this report).

3.1.1 Alternative 1: Raising and Modifying the Existing Dam

Alternative 1 includes raising the existing upper dam (BC 2) to achieve the necessary seismic safety and storage capacity. The new crest of this embankment dam would be downstream of the existing crest as the existing reservoir and dam need to stay in operation during construction. The raised dam would be a continuation from the existing upstream slope at a new 3H:1V (Horizontal:Vertical) slope rising to a total dam height of 111 feet at elevation 131 feet. The new water surface elevation would be at elevation 116 feet for a normal water pool. The new crest would be 20 feet wide and the downstream 3:1 slope would extend into the valley downstream of the existing upper dam.

The dam would have an internal filter and drainage system. The foundation soil of the existing dam would remain in place and the foundation soil for the new portion of the dam would be excavated to bedrock and replaced with suitable compacted dam material.

A new outlet structure consisting of a multi-inlet sloping intake structure and a 36-inch discharge pipe installed in a new tunnel system in the right abutment of the dam and discharging through a control structure into a 20-inch diameter treatment plant pipeline, or 36-inch diameter dam safety discharge to the stream channel. The sloping intake structure would have different inlet ports for water quality purposes so water could be drawn from different elevations of the reservoir. The upstream portion of the outlet pipe would be routed through the right abutment of the dam in a micro-tunnel system creating a seal from the reservoir. This pipe would discharge into an outlet vault within the abutment near the dam axis centerline and then through a 10-foot-diameter access tunnel until it daylights at the control structure The spillway and fish ladder would be routed to the north side of the dam. Figure 2 includes details of this embankment alternative.

Advantages of this alternative include reasonably well-defined foundation geometry, the properties of the existing dam materials have been tested and are well understood, the footprint for the addition would be small compared to a new dam, and a cofferdam and dewatering requirements at the downstream side should not be excessive.

Disadvantages include the possibility that construction of a new outlet and spillway may require the existing dam be taken out of service for a period of time (which may cause water supply issues), only the downstream side of the dam is being seismically stabilized and there would still likely be significant damage to the upstream portion of the embankment during a significant seismic event, and the construction schedule for excavating and embankment construction would be limited due to the short construction season for embankment placement.

This alternative would have significant costs associated with construction of the new outlet works described above.

3.1.2 Alternative 2: New RCC Dam

Alternative 2 includes a new gravity dam structure constructed out of roller compacted concrete (RCC) downstream of the existing upper dam (BC 2) at a location where the valley narrows topographically and offers the possibility of a least cost dam project. The new dam would be located within the existing lower reservoir just downstream of the existing upper dam. This dam would have a height of about 100 feet with the crest at elevation 120 feet. The normal water surface elevation would be at 112 feet. The foundation soil would be excavated and the new dam placed on suitable bedrock. The spillway chute and stilling basin would be over the central portion of the dam. The vertical concrete intake tower would be integrated into the upstream face of the dam and would have intake ports at different levels so water can be drawn from different depths for water quality purposes. From the intake tower a 36 inch outlet pipe would be routed through the base of the dam until it daylights at a gate house and forks into the 20-inch raw water pipe which is connected to the water treatment plant, and into the spillway stilling basin to provide a low level dam safety outlet. Structural details would have to be defined at a later point in time but seismic modeling of the new dam showed the need for a conventional concrete shear key and upstream heal section to provide adequate resistance to cracking and sliding in case of the larger seismic events. The facing, spillway portion, stilling basin, and crest road of the dam would also be conventional concrete. Figure 3 includes details of this RCC alternative.

Advantages of this alternative include a more robust structure that is less susceptible to damage from seismic or hydrologic events, a smaller footprint requiring less excavation than a new embankment dam, smaller quantity of material required for the RCC dam, constructed of material that can generally be placed year around, the ability to incorporate the spillway and outlet work into the RCC structure, little maintenance needs, and this alternative that can be constructed while the existing upstream dam remains in operation.

Disadvantages include the location of the structure in the upstream end of the BC 1 pool that would require a cofferdam and increased dewatering efforts, and foundation conditions that have not been defined which may result in some increase in cost.

3.1.3 Alternative 3: New Embankment Dam

Alternative 3 consists of a new embankment structure at the same proposed location as Alternative 2 (RCC dam). The foundation soil would be excavated to bedrock and suitable embankment earthen material would be placed to construct the dam. The height of the dam would be about 108 feet with the dam crest at elevation 128 feet and a new normal water surface elevation of 112 feet. The downstream and upstream slopes of the dam would be 3H:1V. The dam would have an internal filter and drainage system. The outlet works would be placed in either the lower right or left abutment areas on bedrock and include a multi-port sloping intake structure connected to a concrete encased 36-inch-diameter steel outlet pipe through the dam foundation. The multiple intake ports would be placed for water quality purposes. The 36-inch outlet pipe would daylight at a gate house and fork into the 20-inch raw water pipe going to the water treatment plant, and into the 36-inch pipeline discharging to the stream channel for dam safety purposes.

The spillway channel and access road would be north of the proposed dam. Figure 4 includes details of this embankment alternative.

Advantages of this alternative are limited to the ability to continue operation of the upstream dam during construction, and a dam that is less susceptible to seismic and hydrologic events than the Alternative 1 structure.

Disadvantages include the much larger footprint than Alternatives 1 or 2, the geometry for the rock foundation is unknown, there would be a significant increase in the quantity of foundation excavation required compared to Alternative 2. In addition, the downstream cofferdam and foundation dewatering would be significantly larger than Alternative 2. The construction season for embankment placement would be limited and would take the longest to complete of all the alternatives under consideration. This alternative would have the largest risk exposure to floods and other adverse construction conditions of all alternatives under consideration.

3.1.4 Alternative 4: New Dam Option A

Alternative 4 was considered early in the project as a possible new site location for either an RCC or embankment dam. It was thought to be further downstream of the upper dam (BC 2) located in the lower reservoir about 100 yards downstream of proposed Alternatives 2 and 3. This alternative was eliminated from further consideration as the valley is wider at that particular location and the costs for the dam would be much higher than Alternatives 2 and 3 without providing any other benefits. Figure 1 shows the proposed location of this embankment alternative.

3.1.5 Alternative 5: New Dam Option B

Alternative 5 was similar to alternative 4 as it was considered early in the project as a possible new site location for either an RCC or embankment dam. The location was thought to be where the current access road crosses the lower reservoir as the valley narrows the most at that location. This alternative was not considered further as some of the land that the dam would cover does not belong to the City and is outside the city limits. Acquisition and condemnation of the properties and zoning changes did not seem advantageous in relation with providing a better option than Alternatives 1, 2, or 3. Figure 1 shows the proposed location of this dam alternative.

3.1.6 Alternative 6: No Action

Alternative 6 is the No Action alternative and is still an option that the City has to weigh against the possible risk of loosing the only drinking water source for the City in case of a seismic event.

3.2 Other Related Structures

All alternatives include other related structures that would have to be added to make the dam and water supply functional. The intake tower (for RCC dam alternative) or the sloping intake pipe (for embankment dam alternative) would be equipped with three different ports or gates at different elevations. The reservoir stratifies during the summer months and the lower portion of the lake becomes anaerobic and the upper portion becomes aerobic. This influences the water quality of the lake. Different elevated intake gates allow the treatment plant operators to draw water from different depths of the reservoir to avoid the undesired water during the summer. These gates would need the appropriate size of fish screens to avoid fish getting into the pipeline and therefore into the pumps of the treatment plant. The exact size of those screens would be determined during the next phase as it would depend on regulations and requirements for Oregon Department of Fish and Wildlife (ODFW) and other environmental factors.

All dams require a low level outlet for dam safety that acts as an emergency outlet in case the reservoir has to be drawn down rapidly. This outlet would be part of the outlet works for all alternatives and would be located at the downstream toe of the dam. This outlet would have a stilling basin structure at the end to avoid erosion when the water is being released. The RCC dam has a stilling basin at the toe of the spillway in addition to the dam safety outlet.

The embankment dam options would need a separate spillway as the spillway is not part of the actual dam structure as with the RCC dam alternative. This spillway would have to be refined at a later phase as well. The most likely location would be north of the proposed options around the dam running parallel to the access road.

A new fish ladder may have to be built for all alternatives. The exact requirements for sizing and design of the fish ladder would occur during the next phase of the project as it would depend on permit requirements and regulations by the ODFW. Currently, the location of the fish ladder is anticipated to be right next to the spillway for the embankment dams and to the north side near the access road for the RCC dam.

Presently, there is an access road leading from BC 1 to BC 2 and beyond. This road would have to be realigned as it would be blocked and/or flooded by any of the alternatives discussed. A potential new alignment is shown in Figure 1 but further investigation would be necessary during the next phase of the project.

A new raw water pipeline would have to be constructed starting at the outlets works for the dams and continuing to the existing intake pump station where it would tie into the existing pipeline just downstream of BC 1. Preliminary calculations size the pipe to be 20 inches diameter and constructed of ductile iron. The exact alignment would be determined during the next phase but would likely follow the road.

3.3 Comparison of Alternatives

Each alternative provides opportunities and constraints besides the costs of construction. Items that influence the decision making on an alternative are as follows: constructability, excavation volume, construction materials, foundation conditions, spillway design, intake structure, outlet works, necessary dewatering during construction, seismic and hydraulic resiliency of each dam alternative, environmental impacts and permits, operations and maintenance, and most importantly total costs, including geotechnical explorations, design, construction, permitting and contingency for unexpected events. Table 3 summarizes these items for the three preferred alternatives.

Table 3. Summary of Advantages and Disadvantages of Alternatives 1, 2, 3

Opportunity/ Constraint	Alternative 1 Raising Existing Dam	Alternative 2 New RCC Dam	Alternative 3 New Embankment Dam
Constructability	- Requires modifications to existing spillway - Requires temporary outlet works/coffer dam upstream to provide a continuous, uninterrupted water source during construction - Construction season for an embankment-type dam is limited to summer and early fall. - Source of construction materials for the dam have not been identified and may require a significant distance and processing requirements	- Existing reservoir can be in continuous operation - Downstream cofferdam required - Year-round construction possible - Requires construction of a temporary pipeline from the existing dam outlet to the new outlet during construction - Shortest construction prior and smallest construction risk exposure timeframe of all alternatives.	- Existing reservoir can be in continuous operation - Requires construction of a temporary pipeline from the existing dam outlet to the new outlet during construction - Significant increase in required project footprint - Much larger downstream cofferdam required - Construction season for an embankment type dam is limited to summer and early fall
Excavation Volume	- Moderate foundation excavation required at downstream toe	- Smallest foundation excavation required for dam foundation	- Large foundation excavation required for dam foundation; Several times greater than Alternatives 1 and 2
Construction Material	- Need for large amount of suitable foundation and dam material - Would require an off-site source for filter and drainage materials to be used in the dam	- Need for an appropriate off-site source of aggregate for concrete production	- Need for large amount of suitable foundation and dam material - Would require an off-site source for filter and drainage materials to be used in the dam.
Foundation Conditions	- Foundation conditions reasonably well-defined	- Foundation conditions unknown, and could impact final cost of alternative	- Foundation conditions unknown, and could impact final cost of the alternative
Spillway Design	- New spillway would be constructed into abutment with no stilling basin. Potential for significant erosion damage, if used	- Spillway and Emergency spillway co-located in center of dam with stilling basin. Limited potential for significant erosion and downstream channel degradation.	- New spillway would be constructed into upper right abutment which requires more excavation and cost increase once the design is in place
Intake Structure	- Sloping intake on upstream face of dam, requires lowering the water level significantly which would propose a problem to the continuous water supply - Intake pipe routed through the dam via tunnel in lower right abutment - Sloping intake difficult to operate and maintain	- Intake tower included in dam structure with limited footprint - Intake pipe would be short through the narrow dam compared to Alternatives 1 and 3 - Limited susceptibility to seismic damage	- Sloping intake on upstream face of dam - Intake pipe routed through the dam via tunnel - Sloping intake difficult to operate and maintain

Table 3. Summary of Advantages and Disadvantages of Alternatives 1, 2, 3

Opportunity/ Constraint	Alternative 1 Raising Existing Dam	Alternative 2 New RCC Dam	Alternative 3 New Embankment Dam
Outlet works	- Outlet as a combination of the water supply line to the treatment plant and the dam safety outlet.	- Outlet as a combination of the water supply line to the treatment plant and the dam safety outlet.	- Outlet as a combination of the water supply line to the treatment plant and the dam safety outlet.
Dewatering	- Small downstream cofferdam required for dewatering of area covering the new footprint - Moderate dewatering effort	- Significant downstream cofferdam required (dam located in upper part of reservoir BC 1) - Significant quantity of dewatering may be required	- Cofferdam much larger than Alternative 2 (downstream toe of dam located further downstream in reservoir of BC 1) - Dewatering quantity likely significantly greater than Alternative 2
Seismic Resiliency	- Limited damage due to seismic shaking still probable - Upstream portion of dam still susceptible to significant damage	- Low probability of significant damage resulting from seismic shaking	- Moderate potential for damage resulting from seismic shaking
Hydraulic Resiliency	- Potential for erosion damage during design flow	- Reduced potential for erosion during design flow	- Potential for erosion during design flow similar to Alternative 1
Environmental impacts	- Increase in inundation area - Extensive permitting process - Requires smallest footprint of the three alternatives	- Increase in inundation area - Extensive permitting process - Moderate interruption of existing lower reservoir due to footprint of new dam	- Increase in inundation area - Extensive permitting process - Significant interruption of existing lower reservoir due to footprint of new dam
Maintenance	- Requires annual maintenance to manage vegetation, burrowing animals, erosion, and other potential damage - Maintenance cost similar to Alternative 3	- Structure very resistant to damage and deterioration - Least cost maintenance	- Requires annual maintenance to manage vegetation, burrowing animals, erosion, and other potential damage - Maintenance cost similar to Alternative 1
Total costs	- Most costly due to new outlet works requirement	- Similar to Alternative 3	- Similar to Alternative 2

4 Preliminary Environmental Review

Each alternative would require permits from federal, state, and local agencies. Although the alternatives differ, the necessary work for each alternative would require the same permits and approvals as described in detail in Appendix C. Therefore, the preliminary environmental review does not differentiate permit requirements between alternatives. At this point it is difficult to gauge if one alternative would be more challenging to permit than another. To date, no agencies have been contacted to discuss the project in detail. This section provides an overview of anticipated permitting efforts.

4.1 Major Permits and Timelines

There are several major permits required for this project. Those permits and timelines are described in Table 4. Other permits aside from those listed in this table may be applicable but are not anticipated to be as complicated.

Table 4. Overview of Major Permits and Timelines

Required Permit	Timeline	Submittal Occurs at Engineering Design Level (approximate)
National Environmental Policy Act (NEPA)	12-18 months	15-30%
Clean Water Act Section 404/401 and Oregon Removal-Fill permit Other permits processed concurrently with applications: • Endangered Species Act Section 7 • Magnuson Stevens Fishery Conservation and Management Act (Magnuson Stevens Act) • National Historic Preservation Act (NHPA), Section 106 • Migratory Bird Treaty Act • Oregon Fish Passage • Coastal Zone Management Act	6-18 months	30%
Bald and Golden Eagle Protection Act (if required)	4-6 months	30%
Oregon Water Rights	9-12 months	30%
Clean Water Act Section 402 National Pollutant Discharge Elimination System (NPDES) 1200-C	60 days	100%
City of Newport Conditional Use Permit	30 days	60%
City of Newport Building, Electrical, Plumbing, Mechanical, Sewer/Water Permit	30 days	100%
Oregon State Engineer Design Review and Approval	2 months	100%

4.2 Additional Studies and Potential Costs

The project schedule can be influenced by the permitting process due to approval timelines for certain permits and the potential for unanticipated conditions that may arise and delay the permitting process. This can also delay design as well as construction and increase overall project costs.

Risks associated with complex permitting and stringent permit terms and conditions can result from lack of advance knowledge of the potential impact to sensitive environmental resources or public controversy. Early coordination with the agencies and identification of necessary environmental studies upfront would minimize the risk for permitting process delays. Anticipated environmental studies include completing a cultural resource evaluation and wetland and waters delineation, developing mitigation plans, updating the Emergency Action Plan, and preparing a biological assessment.

Depending on the nature of the project, permitting costs can range from 1 to 6 percent of the overall construction costs.

5 Decision Level Estimates of Probable Costs

The three alternatives presented in Section 3 of this report were further investigated in terms of costs for comparison of feasibility between the three alternatives. The cost estimates were prepared for the purpose of comparing alternatives and not for budgeting purposes. Budgetary costs would be provided during the next phase of the project as part of the preliminary design. These costs would include input from contractor estimating methods for the key units and lump sum items as well as further evaluation of construction material sources and costs.

A number of important budget items are not included in this estimate. The costs for those items would have to be added onto the total costs during the next phase of the project. These items would not make a difference in the outcome of the estimates for comparison purposes between the alternatives as they are similar for each alternative. The items purposely left out include: fish ladder, spillway (for embankment option, spillway is included in the RCC dam), access road to the dam, access road around the reservoir to provide access to the forest land and private properties, and the pipeline from the dam to the water treatment plant. Table 5 summaries the items not included in the cost estimate and the reasoning for exclusion.

Table 5. Excluded Items from Cost Estimate

Excluded Item	Alt 2 – RCC Dam	Alt 3 – Embankment Dam
Spillway	n/a spillway included	Exact alignment of spillway is unknown due to lack of survey and geotechnical information of the area
Fish ladder	Type and requirements of fish ladder are unknown at this point. Environmental assessment is necessary to determine the requirements and size for the fish ladder. It is not possible to set a number to this line item.	
Access Road to Dam	Exact alignment of access road is unknown due to lack of survey and geotechnical information of the area.	
Access Road Around Reservoir	Exact alignment of road unknown due to lack of survey in this area.	
Pipeline to Water Treatment Plant	Exact alignment is unknown due to several options for routing of this pipe and unknown access road alignment.	

5.1 Costs Estimate for Alternative 1 – Upper Dam Embankment Raise

Based on discussions with the City, a cost estimate for Alternative 1 was not completed and has been deferred to be updated at a later date if appropriate and necessary. The reasons for this include: the difficulty with constructability and keeping a continuous drinking water source during construction which makes this alternative less favorable; due to the upstream slope deformation concerns of this dam in a seismic event, replacing the outlet works presents a significant risk to the functionality of the system;

and during the last annual dam inspection in spring of 2015, the State Engineer observed some seepage distress in the pipe inside the dam of the current outlet works. These present concern of the overall stability of the existing dam. Experience on other similar projects suggests that the costs for a new outlet works for Alternative 1 are estimated to be disproportionately higher than for Alternatives 2 and 3 and would make this alternative the most expensive by a relatively wide margin.

5.2 Costs Estimate for Alternative 2 – RCC Dam

A planning level cost estimate for comparison purposes was prepared for Alternative 2 RCC Dam. The estimate includes site preparation, work associated with the dam and other structures associated with the dam (spillway and outlet works) and appropriate cost contingencies for a) design elements not included in the current layout b) permitting, c) engineering during construction, and d) a construction change order/claim contingency percentage. HDR developed a concept design as described in section 3.1.2 for the RCC alternative shown in Figure 3. Based on that concept design, quantities were estimated for each line item and an approximate cost calculated. Table 6 presents a summary of the costs providing a range of costs from a lower bound unit cost to an upper bound unit cost. The items listed in Table 5 were excluded in this cost estimate and need to be added to the construction cost estimate for the next phase. The decision level cost estimate for the RCC dam alternative ranges from $13.7 to $19 million. This number includes the spillway for the dam as an RCC dam has the spillway embedded in the structure.

Table 6. Planning Level Cost Estimate - RCC Dam Alternative 2

Bid Item	Description	Quantity	Unit	Lower Bound Unit Cost	Upper Bound Unit Price	Lower Bound Cost	Upper Bound Cost
Prep Work						**$ 306,225**	**$ 400,257**
1	Clearing and grubbing, stripping topsoil, reclamation of disturbed areas	1.4	Acre	$ 20,000	$ 26,000	$ 28,000	$ 36,400
2	Flood control coffer dam downstream	4,329	CY	$ 25	$ 33	$ 108,225	$ 142,857
3	Temporary pipe from existing dam to downstream of new dam	1,000	LF	$ 170	$ 221	$ 170,000	$ 221,000
Main Dam						**$ 7,853,000**	**$ 10,207,600**
4	Excavation - Foundation General	30,000	CY	$ 8	$ 10	$ 240,000	$ 300,000
5	Embankment - Backfill	15,000	CY	$6	$ 8	$ 90,000	$ 120,000
6	Fill - Roller Compacted Concrete	32,200	CY	$ 80	$ 104	$ 2,576,000	$ 3,348,800
7	Conventional Concrete Reinforced	1,000	CY	$ 750	$ 975	$ 750,000	$ 975,000
8	Conventional Concrete Non-Reinforced	12,100	CY	$ 325	$ 423	$ 3,932,500	$ 5,118,300
9	Construction De-watering	1	LS	$ 125,000	$ 162,500	$ 125,000	$ 162,500
10	Foundation Treatment - Grout Curtain	3,000	LF	$ 16.50	$ 21	$ 49,500	$ 63,000
11	Outlet Works Gates - Slide (Fabrication and Construction)	7,500	LB	$ 12	$ 16	$ 90,000	$ 120,000
Other						**$ 175,000**	**$ 228,600**
12	Intake structure and outlet works	1	EA	$ 100,000	$ 130,000	$ 100,000	$ 130,000
13	fishscreen for intake structure	2,500	LS	$ 12	$ 16	$ 30,000	$ 40,000
14	pipeline thru dam 36"	200	LF	$ 225	$ 293	$ 45,000	$ 58,600
Total Base Construction Cost (BCC)						**$ 8,334,225**	**$ 10,836,457**
15	Design Contingency			25.0%	30.0%	$ 2,083,556	$ 3,250,937
16	Mobilization/Demobilization construction			5.0%	5.0%	$ 416,711	$ 541,823
17	Construction, CO/C Contingency			8.0%	10.0%	$ 666,738	$ 1,083,646
Total Construction Cost						**$ 11,501,231**	**$ 15,712,863**
18	Permitting			3.0%	3.0%	$ 345,037	$ 471,386
19	Design and Site Characterization			7.0%	8.0%	$ 805,086	$ 1,257,029
20	Engineering Support during Construction			9.0%	10.0%	$ 1,035,111	$ 1,571,286
Total Cost (Rounded)						**$ 13,700,000**	**$ 19,000,000**

5.3 Costs Estimate for Alternative 3 – Embankment Dam

A planning level cost estimate for comparison purposes was prepared for Alternative 3 Embankment Dam. As for Alternative 2, the estimate includes site preparation, work associated with the dam, other structures associated with the dam, and appropriate contingencies for a) design costs, b) permitting, c) engineering during construction, and d) a construction change order/claim contingency. HDR developed a concept design as described in section 3.1.3 for the Embankment Alternative shown in Figure 4. Based on that concept design, quantities were determined for each line item and an approximate cost was calculated. Table 7 presents a summary of the costs providing a range of costs. The items listed in Table 5 were excluded in this cost estimate and need to be added to the construction cost estimate for the next phase. The option Embankment dam alternative ranges from $12.9 to $17.8 million. These numbers does not include the spillway for the dam as the spillway is a separate structure for embankment dams.

Table 7. Planning Level Cost Estimate - Embankment Dam Alternative 3

Bid Item	Description	Quantity	Unit	Lower Bound Unit Cost	Upper Bound Unit Price	Lower Bound Cost	Upper Bound Cost
Prep Work						**$ 396,225**	**$ 517,257**
1	Clearing and grubbing, stripping topsoil, reclamation of disturbed areas	5.9	Acre	$20,000	$26,000	$ 118,000	$ 153,400
2	Flood Control coffer dam downstream	4,329	CY	$25	$33	4 108,225	$ 142,857
3	Temporary pipe from existing dam to downstream of new dam	1,000	LF	$170	$221	$ 170,000	$ 221,000
Main Dam						**$ 7,085,140**	**$ 9,161,560**
4	Excavation - Foundation General	124,280	CY	$13	$17	$ 1,615,640	$ 2,112,760
5	Embankment Fill	301,000	CY	$14	$18	$ 4,214,000	$ 5,418,000
6	Embankment Filter Material	15,000	CY	$30	$39	$ 450,000	$ 585,000
7	Construction De-watering	1	LS	$480,000	$624,000	$ 480,000	$ 624,000
8	Foundation Treatment - Grout Curtain	3,000	LF	$17	$21	$ 49,500	$ 63,000
9	Riprap and Bedding	4,200	CY	$30	$39	$ 126,000	$ 163,800
10	Conventional Reinforces Concrete	200	CY	$750	$975	$ 150,000	$ 195,000
Other						**$ 362,500**	**$ 472,600**
11	intake structure and outlet works	1	EA	$175,000	$227,500	$ 175,000	$ 227,500
12	Fish screen for intake structure	2,500	LS	$12	$16	$ 30,000	$ 40,000
13	pipeline thru dam 36"	700	LF	$225	$293	$ 157,500	$ 205,100
Total Base Construction Cost (BCC)						**$ 7,843,865**	**$ 10,151,417**
20	Design Contingency			25.0%	30.0%	$ 1,960,966	$ 3,045,425
21	Mob/Demob construction			5.0%	5.0%	$ 392,193	$ 507,571
22	Construction. CO/C Contingency			8.0%	10.0%	$ 627,509	$ 1,015,142
Total Construction Cost						**$ 10,824,534**	**$ 14,719,555**
23	Permitting			3.0%	3.0%	$ 324,736	$ 441,587
24	Design and Site Characterization			7.0%	8.0%	$ 757,717	$ 1,177,564
25	Engineering Support During Construction			9.0%	10.0%	$ 974,208	$ 1,471,955
Total Cost (Rounded)						**$ 12,900,000**	**$ 17,800,000**

5.4 Comparison Costs Estimates for Alternative 2 & 3

As previously stated, the two cost estimates where prepared for comparing alternatives and assisting in the identification of the preferred alternative to move forward. From a decision making standpoint, the costs for Alternatives 2 and 3 are similar. It should be noted that the RCC dam cost estimate includes the spillway, but the embankment dam does not. The preferred alternative decision needs to be based on advantages and disadvantages of the alternatives presented in Table 3.

Based on the cost estimates, advantages/disadvantages, and overall experience of HDR, we recommend that Alternative 2 be selected for preliminary design. Alternative 3 can be further considered should any future investigations of the site indicate a significant challenge or cost increase to Alternative 2.

6 Conclusions and Recommendations

Phase 3 explorations and engineering analyses have confirmed significant seismic deficiencies with both BC 1 and BC 2 dams. Configuration level analyses and design layouts have provided important information about alternatives to remediate the seismic deficiencies of the Big Creek dams and how to move forward in the future in order to provide the City of Newport with a safe and reliable drinking water source after a seismic event.

6.1 Key Conclusions

Phase 3 of site characterization work provided the basis to update the site model and analysis, and increased the confidence in the findings of the study. The analysis indicated that both existing dams are unsafe due to excessive deformations that would occur during a large seismic event. Some form of remediation is needed to provide appropriate dam safety and water supply security for the City.

Based on the Phase 3 findings, the project purpose was modified to provide all current water storage capacity and an increased water supply meeting master planning requirements at the upper site. Decommissioning of the lower dam and reservoir (BC 1) would be required by the state. The storage from the BC 1 reservoir needs to be recovered. Also increased storage due to sediment accumulation and future water storage capacities needs to be provided with the new modifications.

Several alternatives have been identified that would meet the modified project purpose. The chosen alternatives to proceed include either a new RCC dam or embankment dam at a location immediately downstream of the upper dam (BC 2). Configuration level studies have indicated that both types of dam at this location can be designed and constructed to provide safe and secure water supply for earthquake events that have a minimum recurrence interval of about 5,000 years or higher. Such safety is consistent with state requirements and federal projects with similar potential consequences of dam failure.

6.2 Recommendations

The recommendation to move forward to provide the City with a safe and secure drinking water source is to build a new RCC dam (Alternative 2) at the location just downstream of the existing upper dam (BC 2). Based on the results of the current study, the RCC alternative would provide the most secure and stable option in case of a seismic event. Constructability of an RCC dam is less complicated and takes the least amount of time compared to the embankment option. The footprint of an RCC dam is less and provides fewer disturbances in terms of environmental impact compared to the embankment option. The preliminary costs show the RCC dam is a feasible option compared to the embankment dam.

Preliminary designs that include a comprehensive characterization of the new dam site are needed to update the configuration of the dam, to provide budgetary cost estimates, and to provide information required for permitting of the dam. Such preliminary design would be the objective of the next phase of work.

Information necessary for a preliminary design is geotechnical data of the new proposed site to provide the depth of bedrock and to characterize a foundation concept for the new dam.

The environmental permitting process can be started and prepared for the actual permitting process. A concept for the remediation of Big Creek can be developed at the location of the lower reservoir after the BC 1 dam has been removed. Dialog with ODFW should be started about fish ladder requirements and possible remediation opportunities.

A detailed budgetary cost estimate needs to be prepared that represents actual orders of magnitudes of costs. Based on this preliminary design cost estimate the search for funding and finance options can be explored.

Further, the access road to the dam and around the reservoir would be defined with the help of a comprehensive survey that has to take place to develop a preliminary design. The spillway for the embankment option has to be refined as well with the help of a topographic survey.

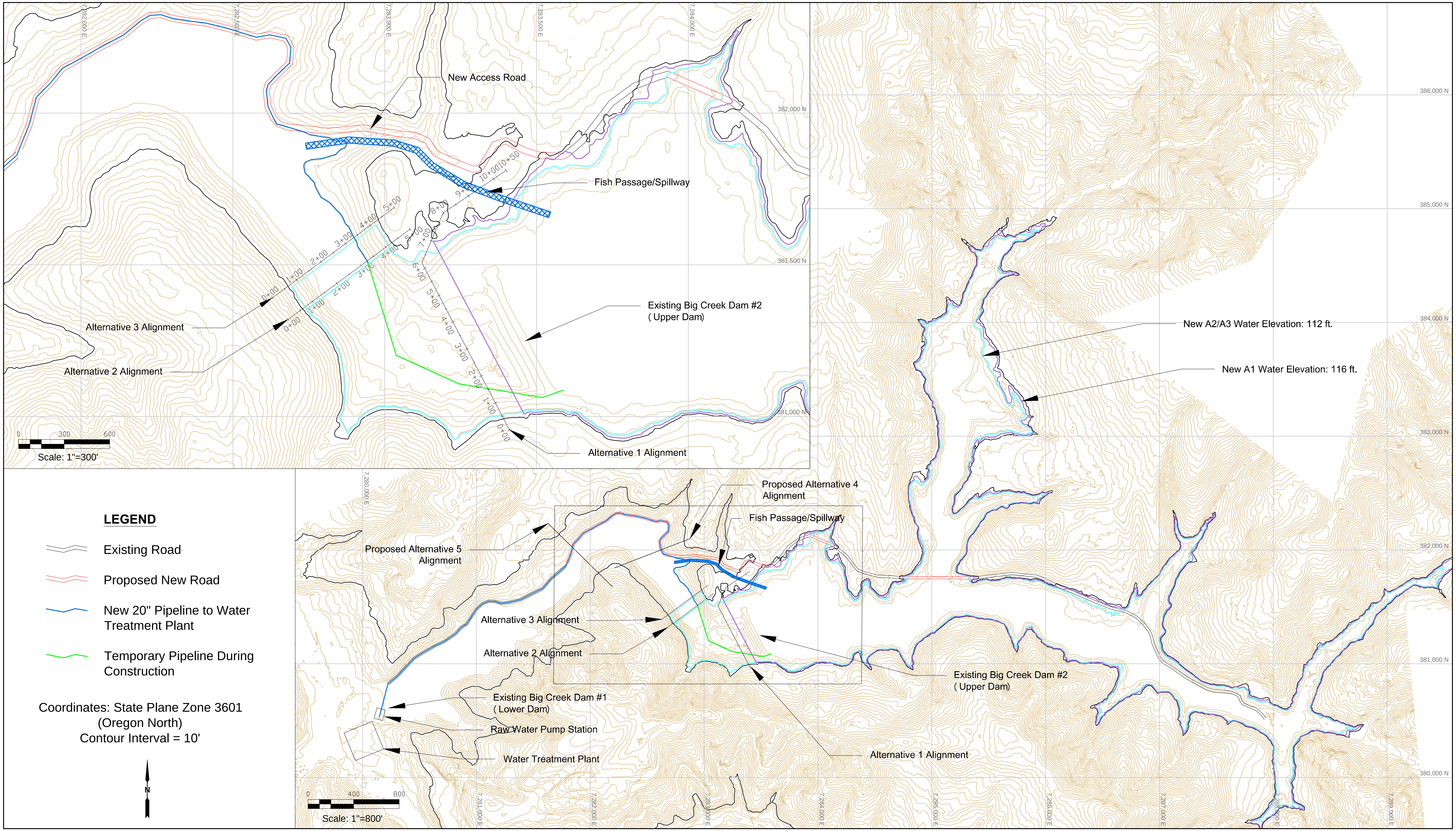
A schedule would need to be developed that presents the next steps of this project.

Some additional modeling analysis for the new dam is necessary during the preliminary design of the dam. This analysis would include two design earthquakes: the biggest crustal and the biggest fault earthquake. Both modeling results would have to be presented to the State to determine the design earthquake requirements for the new dam.

The consequences of a safety related failure of the dam needs to be updated to represent the culvert conditions where Big Creek flows underneath Highway 101 and then into the Ocean. It is likely this culvert would be blocked by debris or damaged in a seismic event. This scenario is not reflected in the current dam breach and inundation limits prepared for consequence evaluations and emergency planning in the Emergency Action Plan report. With the new dam arrangement, a new Emergency Action Plan would also need to be developed once the new dam is in place.

Overall, HDR recommends proceeding with the preliminary design of an RCC dam (Alternative 2) at the identified location. If further explorations show that the foundation soils are not suitable for this option, a refinement of Alternative 3 can be investigated.

Figures



LEGEND

- — Existing Road
- — Proposed New Road
- — New 20" Pipeline to Water Treatment Plant
- — Temporary Pipeline During Construction

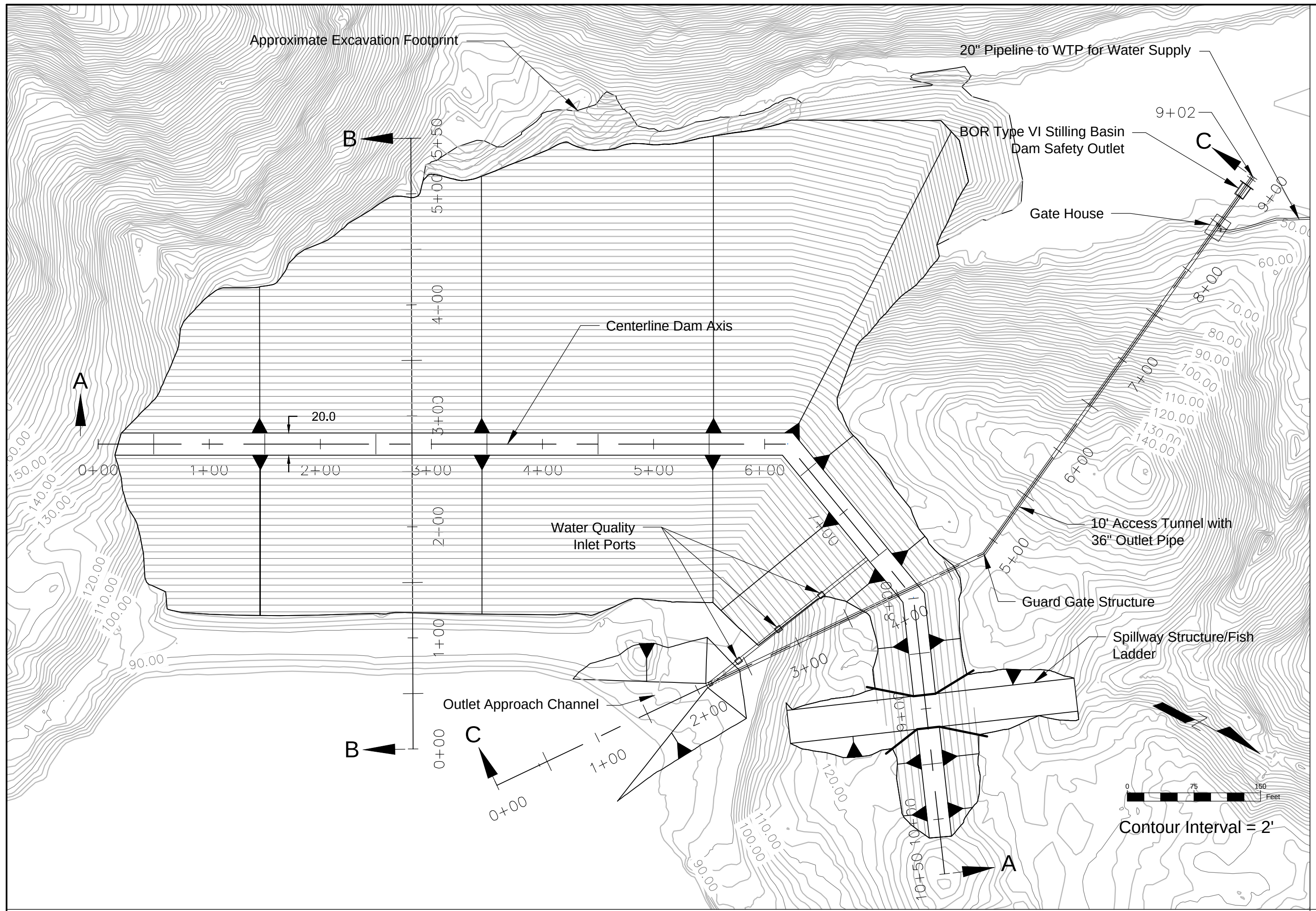
Coordinates: State Plane Zone 3601
(Oregon North)
Contour Interval = 10'

HDR

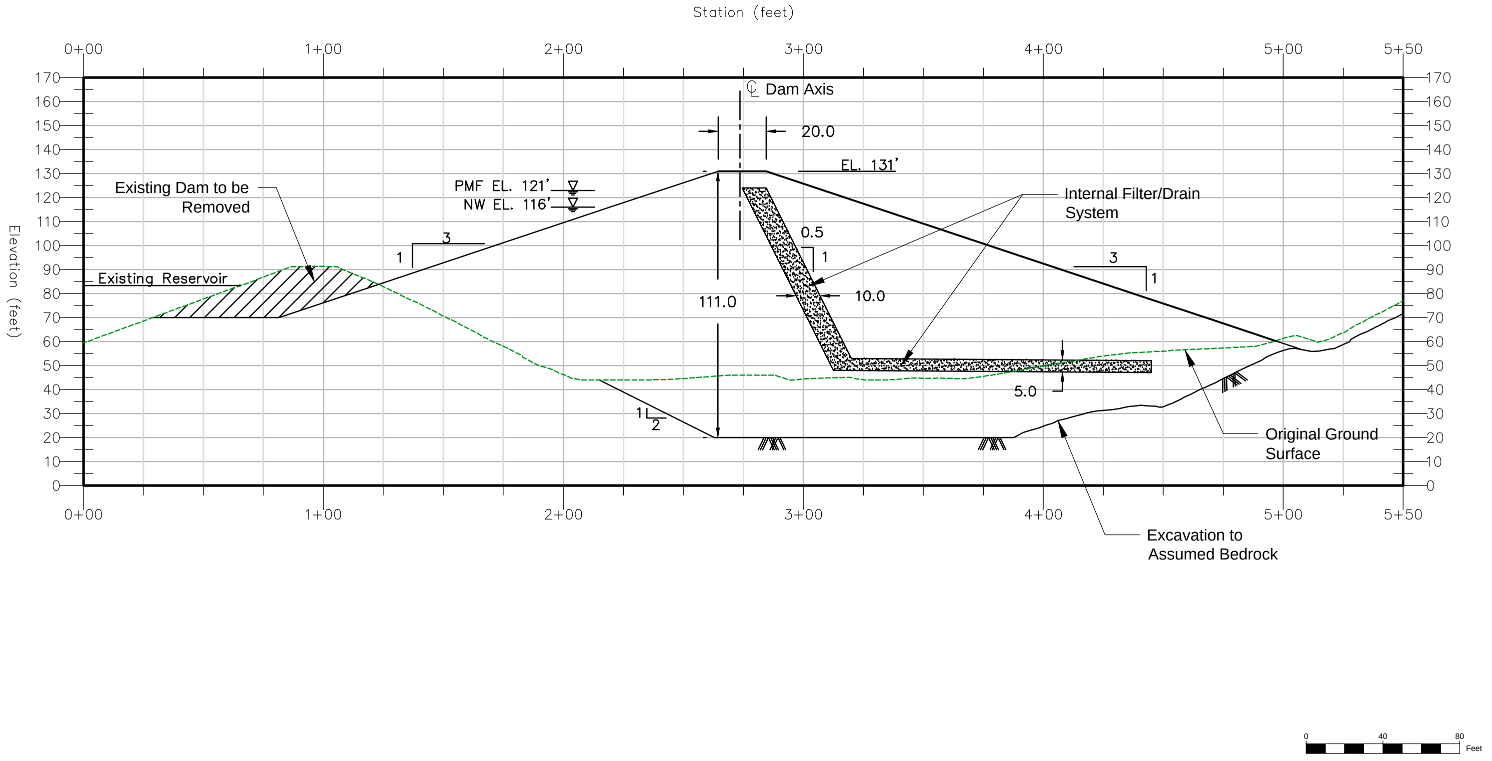
City of Newport
Dam Alternatives Overview

DATE
6-30-2015

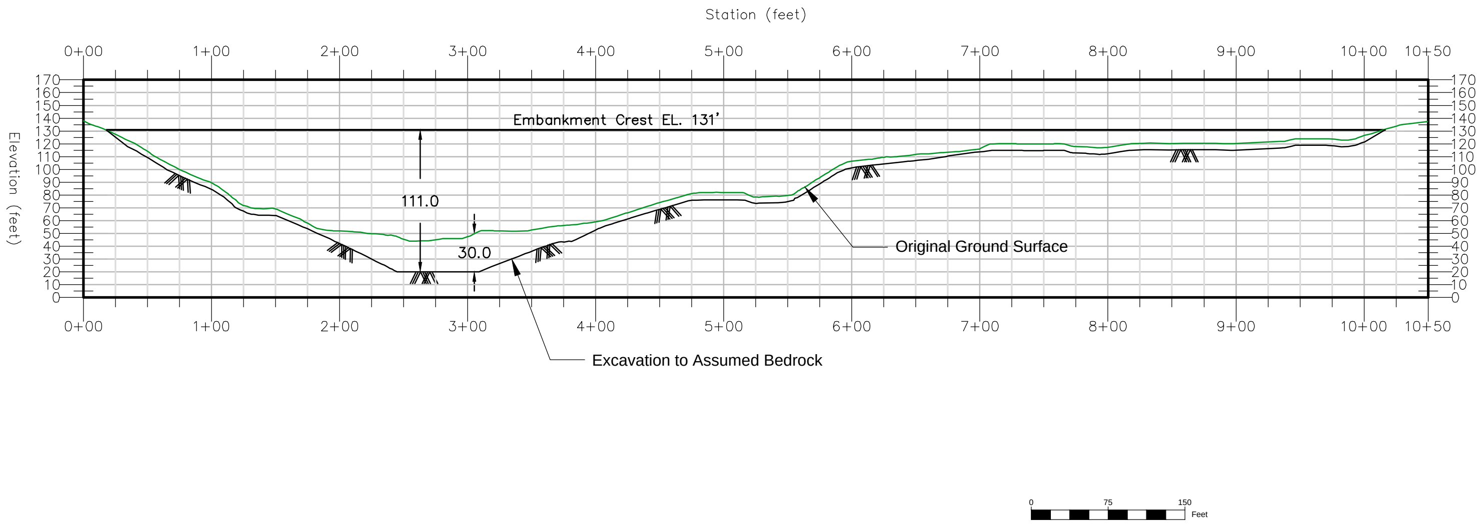
FIGURE
1



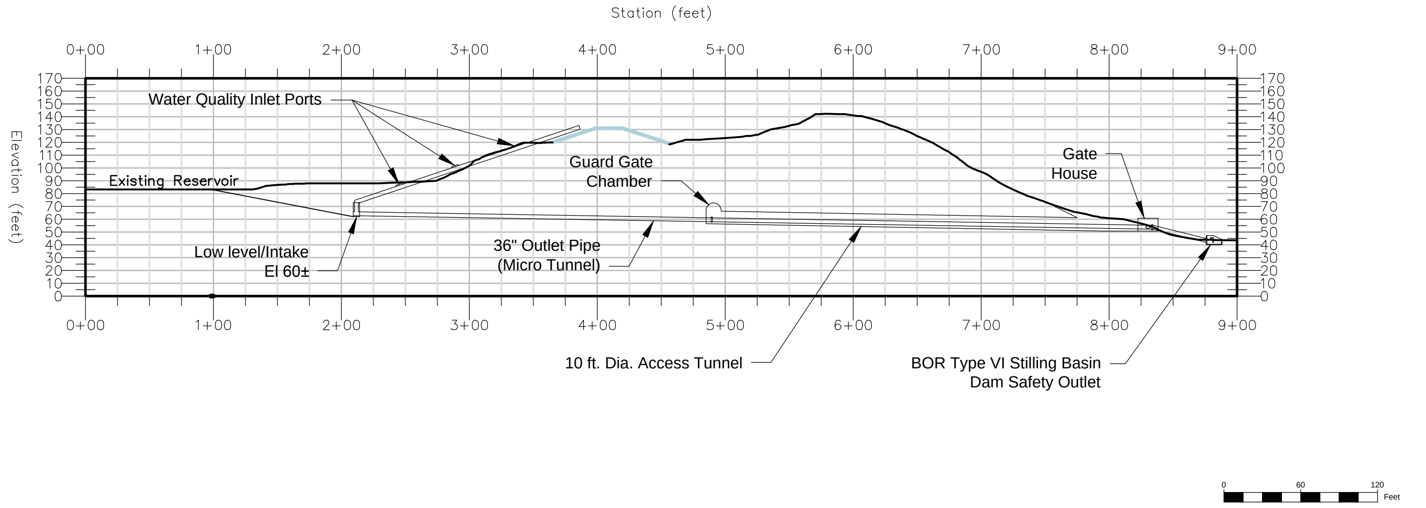
Alternative 1 Embankment Section B–B 2+85



Alternative 1 Embankment Axis Profile A–A



Alternative 1 Embankment Outlet Works C–C

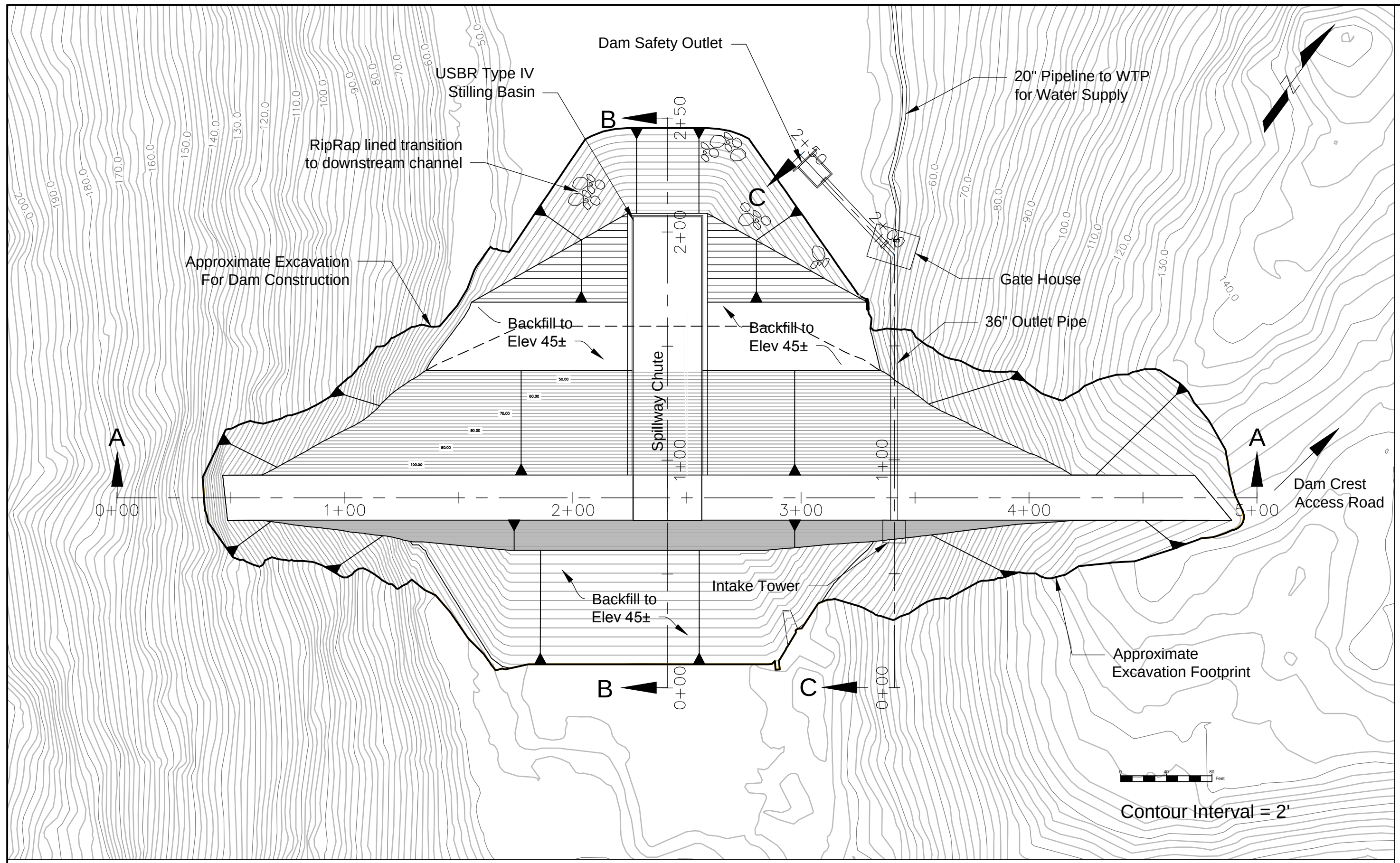


Embankment Volume:	377,000	CY
Excavation Volume:	102,500	CY
Excavation Area:	8.9	AC
Available Storage Volume:	2706	Ac-Ft
Normal Water Elevation:	116	ft
Dam Height:	111	ft

City of Newport
Alternative 1 Upper Dam Embankment Raise

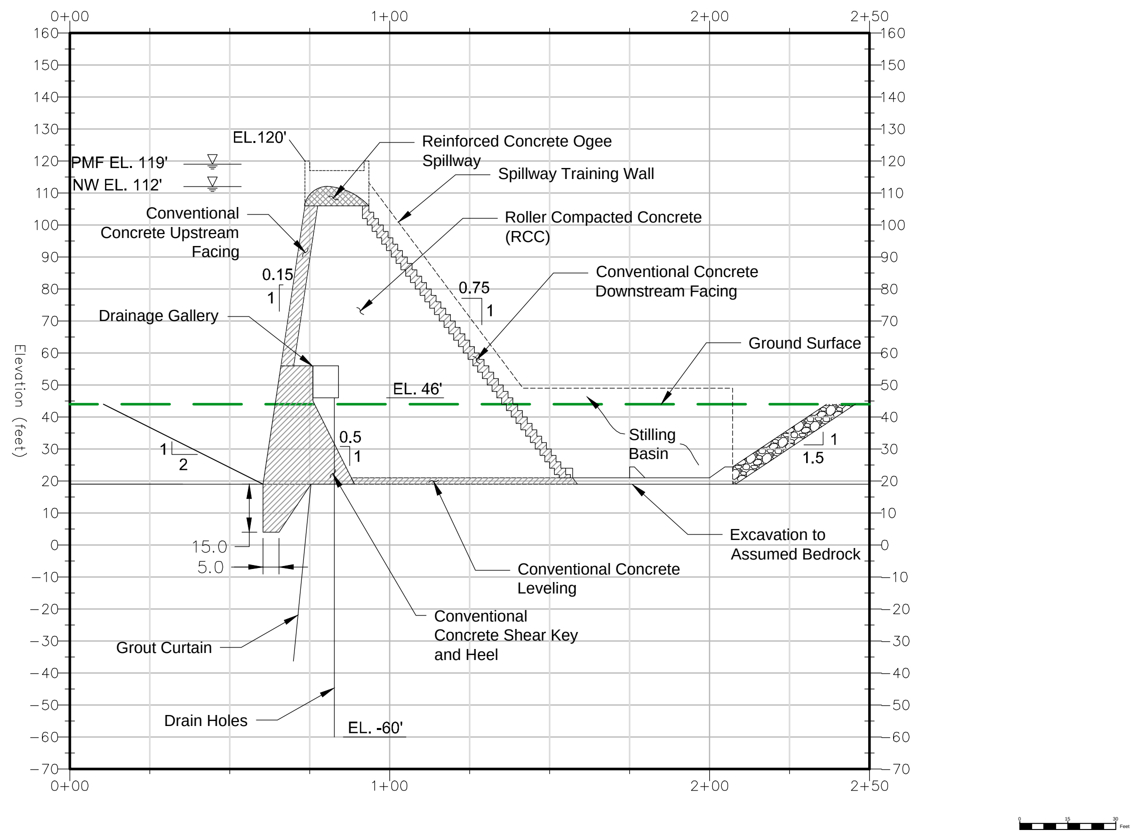
DATE
6-30-2015

FIGURE
2



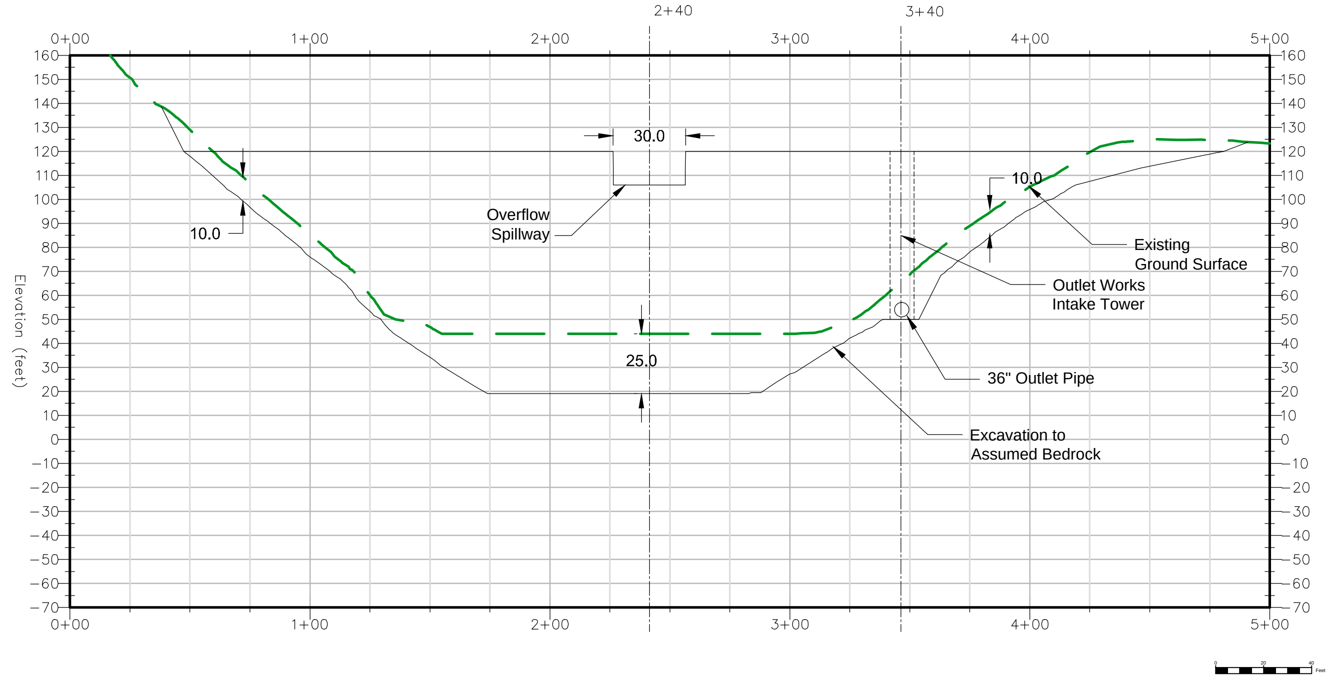
Alternative 2 RCC Dam – Section B-B 2+40

Station (feet)



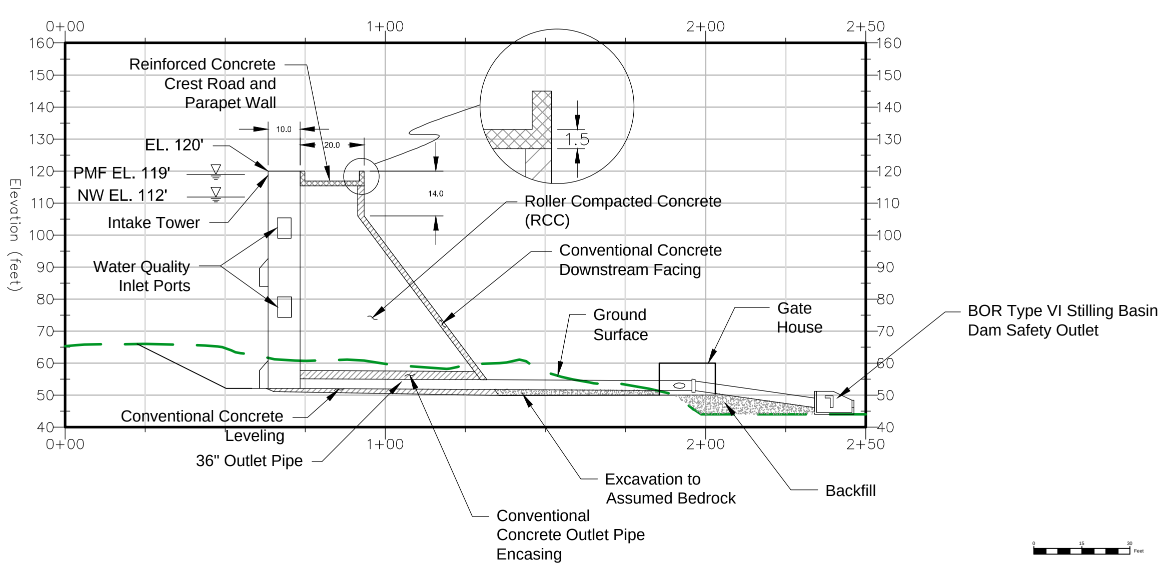
Alternative 2 RCC Dam Axis Profile A-A

Station (feet)



Alternative 2 Outlet Works Section C-C 3+40

Station (feet)

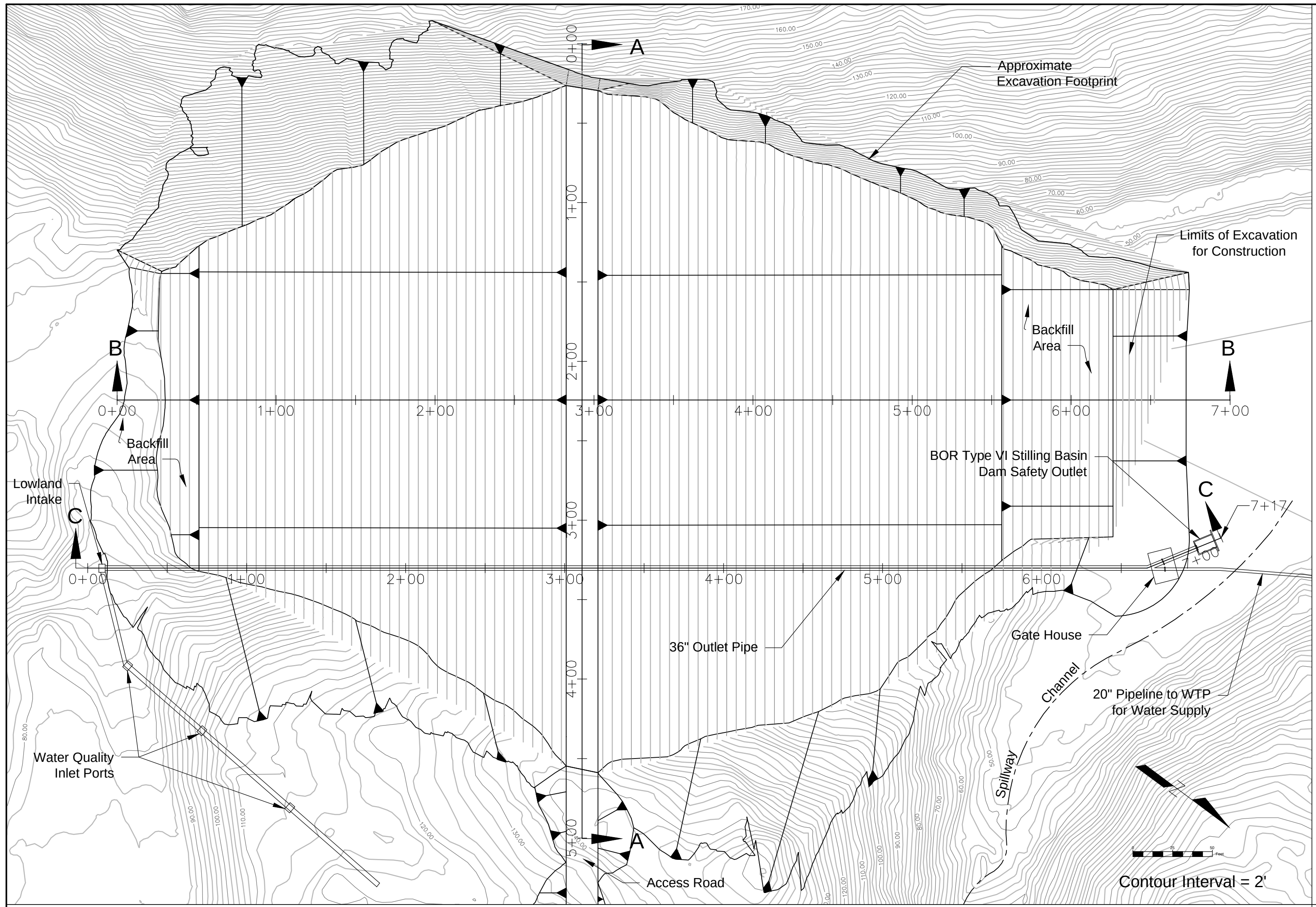


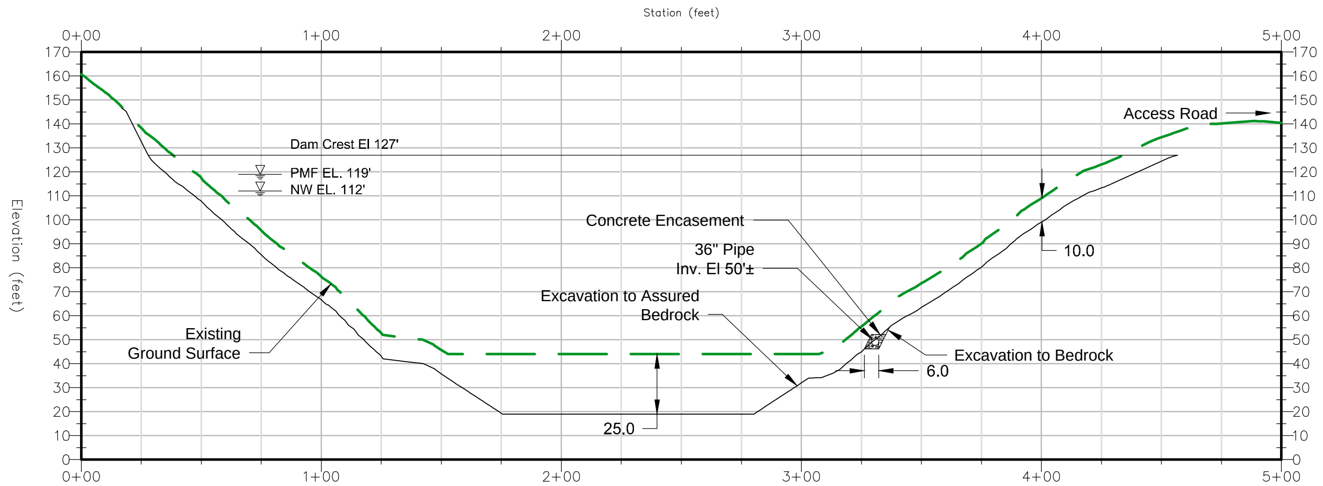
Embankment Volume:	44,723	CY
Excavation Volume:	30,000	CY
Excavation Area:	1.4	AC
Available Storage Volume:	2577	Ac-Ft
Normal Water Elevation:	112	ft
Dam Height:	101	ft

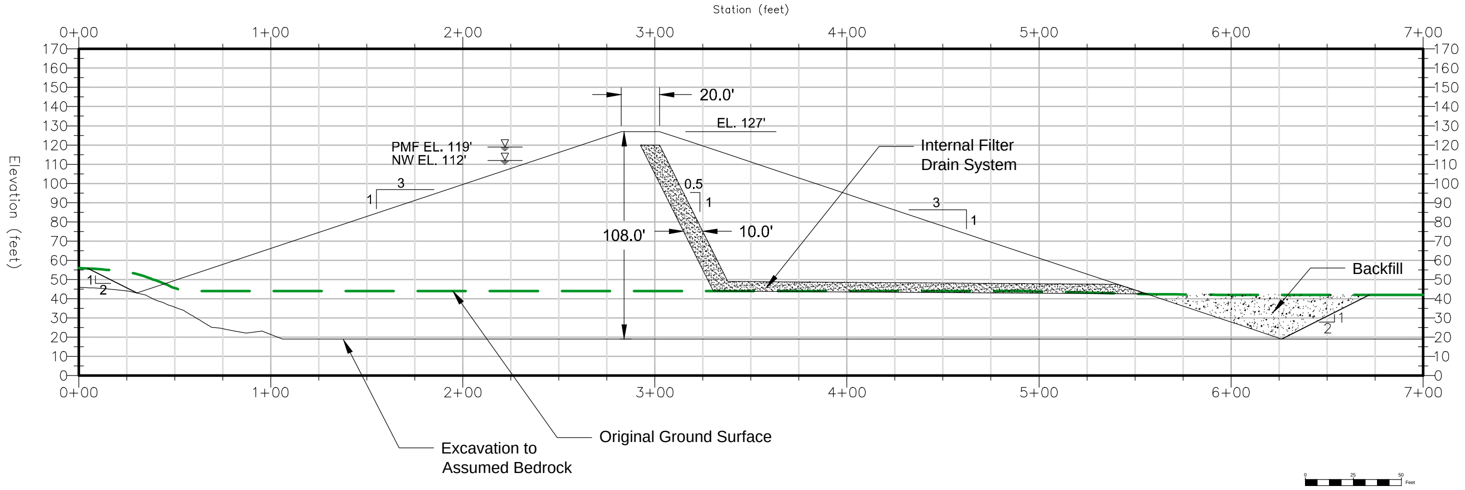
City of Newport
Alternative 2 RCC Dam

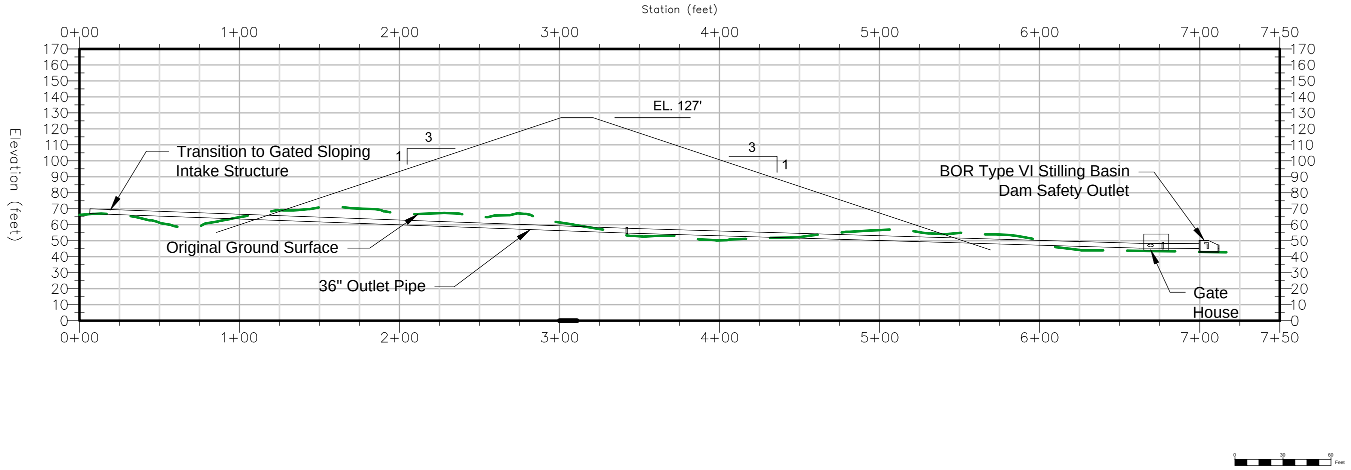
DATE
6-30-2015

FIGURE
3




Alternative A3 Embankment Axis Profile A–A


Alternative A3 Embankment Section B–B 2+40


Alternative A3 Outlet Works Section C–C 3+50

Embankment Volume:	301,000	CY
Excavation Volume:	124,280	CY
Excavation Area:	5.9	AC
Available Storage Volume:	2579	Ac-Ft
Normal Water Elevation:	112	ft
Dam Height:	108	ft

HDR

City of Newport
Alternative 3 Embankmet Dam

DATE
6-30-2015

FIGURE
4

Appendix A.
Seismic Hazards

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December 2, 2014 2384

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Seismic Hazard Update
Big Creek Dam No. 1 and Dam No. 2
Newport, Oregon

Following your authorization, we have performed a seismic hazard update in support of the Phase 3 Engineering Evaluations and Concept Design studies being performed for the Big Creek Dams located near Newport, Oregon. This letter report summarizes an update to the seismic hazard based on information from recent large subduction zone earthquakes and newly released probabilistic seismic hazard maps.

Background

We understand that HDR is currently undertaking Phase 3 engineering evaluations and conceptual design for the seismic performance of Big Creek Dam No. 1 and Dam No. 2. As part of their services, HDR is performing risk analyses, developing a corrective action concept and conducting a preliminary environmental review. This work includes subsurface investigations, evaluation of embankment stability, liquefaction hazard analyses, differential settlement, and surface displacement. As part of this work, HDR has requested an update of the seismic ground motion hazard at the dam sites to incorporate the latest available seismic information.

Seismic Hazard Review

We have reviewed updated information regarding regional seismicity and potential ground motions from USGS's 2014 Probabilistic National Seismic Hazard Maps (NSHM) and supporting documentation. We have compared the newly available information to the results of our prior 2012 seismic hazard analyses (Cornforth Consultants, 2012). In addition, we have provided additional seismic hazard information and acceleration time history parameters for HDR's risk-based site evaluation. The revised seismic hazard analyses and the updated information are provided in the following sections.

National Seismic Hazard Maps.

The USGS maintains probabilistic national seismic hazard maps that are frequently updated. These maps were updated in 2008 and 2010 (Petersen et al., 2008, USGS, 2008a, 2008b, 2010) and most recently in 2014 (Petersen et al., 2014). The 2014 NSHM release includes the spectral acceleration values for peak ground acceleration (PGA), 0.2-second and 1.0-second periods at two exceedance rates (10% probability of exceedance in 50 years, i.e. a 475-year return period and 2% probability of exceedance in 50 years, i.e. a 2,475-year return period).

As part of the update to the seismic hazard report for Big Creek Dams we have reviewed the 2014 NSHM documentation to identify changes that the impact the project site. Generally, the 2014 NSHM update incorporates revised fault source parameters (location, slip rate and magnitude uncertainty, fault dip), new Cascadia Subduction Zone (CSZ) interface earthquake rupture geometries and rates, an updated deep (instraslab) earthquake model, and an increased maximum magnitude 8.0 for crustal and intraslab earthquakes. Newly revised ground motion prediction equations (GMPE) were used for both crustal and subduction zone sources (interface and intraslab). The overall impacts of the updated GMPEs are discussed in the following sections.

Regional Crustal Faults. The USGS provided updated fault source parameters for the 2014 NSHM. Review of the updated fault source information provided no new faults or additional information from the previous report (Cornforth, 2012). Table 1 lists the fault parameters used in the development of the 2014 NSHM for crustal faults sources near the Big Creek Dams.

Table 1. USGS 2014 NSHM Parameters for Faults within 100 km of the Big Creek Dams

Fault Name	Maximum Magnitude	Distance (km)
Yaquina Faults	6.1	3
Waldport Fault	6.4	21
Stonewall Anticline	6.8	35
Daisy Bank Fault	7.3	45
Alvin Canyon Fault	7.2	52
Wecoma Fault	7.3	52
Turner and Mill Creek Faults	6.6	78
Happy Camp Fault	6.6	83

Recent Changes to USGS Probabilistic National Seismic Hazard Map. Figures 1 and 2 show useful comparisons between the most recent 2014 NSHM update and the previous 2010 year map. The peak ground acceleration (PGA) for the 2014 NSHM versus the 2010 map for varying return periods is shown in Figure 1. A graph showing the differences in 2014 versus 2010

NSHM values for the uniform hazard response spectra at 475-year and 2,475-year return periods is shown on Figure 2. The current 2014 map provides spectral accelerations only for the 0.0-second (PGA), 0.2-second and 1.0-second periods and, thus, these are the only spectral accelerations used in the comparison. Review of the 2014 seismic hazard maps indicate that the total uniform hazard for the Big Creek Dam sites has decreased slightly, as shown in Figure 2.

Based on the documentation provided by the USGS for the most recent NSHM, the small reduction to the seismic hazard at the Big Creek sites is generally due to reduced subduction zone intraslab and interface spectral accelerations. The USGS provides graphical representation of the changes due to individual contributors to the overall hazard, from which the following observations were made for the Big Creek Dam Sites:

- The subduction zone interface event PGA and 1.0s spectral accelerations remained about the same as 2008 (increased slightly, up to 0.02g).
- In the short/intermediate periods, at about 0.2s, the spectral accelerations decreased slightly (by approximately 0.05g).
- While contribution to the overall seismic hazard from an intraslab event at the project site is small, the 2014 NSHM documentation indicates that the Intraslab event PGA and spectral acceleration in the vicinity of the Big Creek Dams has decreased by approximately 0.02g.

The revised ground motion parameters provided in this report were developed for a NEHRP BC site class ($V_{S,30}$ = 760 m/s). Given the close proximity of Big Creek Dam No. 1 and Dam No. 2, there is negligible difference in the spectral accelerations and seismic source-to-site distance; therefore, the review of the seismic hazard and selection of ground motions is applicable for both dams.

Ground Motion Sources. Deaggregation of the 2014 NSHM data is not available at this time. However, based on a review of the mapping documentation and fault source parameters, the contributing sources and their associated magnitude and distance from the site have not changed significantly. Therefore, it is likely that the deaggregation plots will not change significantly from the previous iteration provided in the 2012 Seismic Hazard report. The following tables (Tables 2A and 2B) identify the percent contribution from the predominate sources utilizing the most currently available deaggregation data (USGS, 2010) and are reproduced from the 2012 Cornforth seismic hazard report.

Table 2A. Probabilistic Seismic Hazard Deaggregation Contributions at PGA

		Contributions from Principal Sources at PGA (%)			
				Cascadia Subduction Zone	
Return Period	PGA (g)	Gridded (other crustal)	Yaquina Faults	Interface[1]	Intraslab
475-year	0.30	4.4	30.4	59.0	4.4
975-year	0.52	<3	35.8	60.4	<3
2,475-year	0.86	<3	35.2	63.5	<3
4,975-year	1.15	<3	32.8	66.6	<3
9,950-year	1.47	<3	29.8	69.9	<3

[1] CSZ Interface includes Cascadia M8.0-M8.2 floating, M8.3-M8.7 floating and megathrust sources

Table 2B. Probabilistic Seismic Hazard Deaggregation Contributions at 1.0s Period

		Contributions from Principal Sources at 1.0s Spectral Period (%)			
				Cascadia Subduction Zone	
Return Period	1.0s SA (g)	Gridded (other crustal)	Yaquina Faults	Interface[1]	Intraslab
475-year	0.24	<3	20.2	72.4	4.0
975-year	0.42	<3	18.7	78.8	<3
2,475-year	0.71	<3	14.5	84.8	<3
4,975-year	0.97	<3	11.5	88.2	<3
9,950-year	1.27	<3	8.6	91.3	<3

[1] CSZ Interface includes Cascadia M8.0-M8.2 floating, M8.3-M8.7 floating and megathrust sources

Based on the probabilistic seismic hazard deaggregation using 2010 data, the two main constituents to the principal seismic hazard at the Big Creek Dams are earthquake events on the crustal Yaquina Faults and the CSZ interface. The USGS deaggregation results provide earthquake magnitude and distance pairs for sources with contributions greater than 3 percent. For the Big Creek Dam site, the mean computed source distance and magnitude pairs at the 475-year, 975-year, 2,475-year and 4,975-year return periods are shown in Table 3.

Table 3. Mean Magnitude/Distance Pairs for Principal Earthquake Sources

	CSZ Interface (M9)		Yaquina Faults	
Return Period	Magnitude	Distance (km)	Magnitude	Distance (km)
475-year	9.0	25.5	6.1	2.4
975-year	9.0	24.4	6.1	2.4
2,475-year	9.0	23.3	6.1	2.4
4,975-year	9.0	22.7	6.1	2.4

[1] Maginitude and distance pairs determined based on 2010 USGS deaggregation at PGA

Ground Motion Selection

It is our understanding that HDR is using a probability/risk-based approach to determine seismic hazards for the Big Creek Dams, and thus require ground motion parameters for a range of annual exceedance rates (or return periods). Based on the deaggregation results, the Yaquina Faults and subduction zone sources were selected for development of design ground motions using the representative parameters shown in Table 4. The 2010 NSHM data was required to develop the uniform hazard spectra (UHS) due to the limited number of return periods and spectra acceleration data available in the 2014 NSHM update. Based on the comparisons between the 2014 and 2010 maps discussed previously, the use of the 2010 data represents a slightly conservative estimate of the current UHS. The UHS using the 2010 NSHM data for 475-year, 975-year, 2,475-year and 4,975-year return periods are shown in Figure 3. These UHS were utilized for determination of target ground motion response spectra and in the selection of ground motion acceleration time histories as discussed in the following sections.

Table 4. Deaggregated Earthquake Motions

Earthquake Source	Period Range (s)	Geology	Earthquake Magnitude	Distance (km)
Yaquina Faults	0 to 0.6	Rock Site	6.1	2.4
CSZ Interface	0.4 to 2	Rock Site	9.0	23

[1] PGA based on attenuation relationships (see below)

Target Response Spectra. Target response spectra were developed for the two seismic sources identified above using ground motion prediction equations (GMPE) applicable to each source type.

For the Yaquina Faults crustal source, target response spectra for varying return periods were developed from the five GMPE's used by USGS to update and revise the 2014 NSHM. These attenuation relationships and associated weighting factors are: Abrahamson et al., 2013 (0.22 weight); Boore et al., 2013 (0.22 weight); Campbell and Bozorgnia, 2013 (0.22 weight); Chiou & Youngs, 2013 (0.22 weight); and Idriss, 2013 (0.12 weight). The USGS assigned the Idriss GMPE a lower weight due to the lack of detailed modelling features associated with the relationship. The five GMPE's were used to derive the target response spectrum (5% damping ratio) for the Yaquina Faults source earthquake, M=6.1 and R=2.4km. The response spectra from the GMPE's along with the target spectra (weighted average) spectra are shown in Figure 4. The resulting target spectrum closely matches the 2,475-year uniform hazard spectra (UHS) in the 0 to 0.6 second period range using a mean plus one standard deviation motion (Figure 5). For comparison purposes, Figure 5 also includes the target response spectrum that was used to select the acceleration time histories during the 2012 seismic hazard study. Based on this comparison, there is very little change in the target spectra in the short period range (less than 0.2-seconds) and a slightly lower target from the new GMPE's between 0.2 and 0.6-second periods. The peak ground acceleration and deviation from the mean for the 475-year, 975-year, 2,475-year and

4,975-year return periods are shown in Table 5. Additionally, the target response spectra of the crustal source for the four return periods are shown in Figure 6.

For the CSZ source, the four GMPE's selected by the USGS to develop the 2014 NSHM were also used to develop the target response spectra for this project. These GMPE's and associated weights are: Atkinson and Boore 2003-global model (0.10 weight); Zhao et al., 2006 (0.30 weight); Atkinson and Macias, 2009 (0.30 weight); and Addo, et al. (BC Hydro), 2012 (0.30 weight). The USGS retained the older Atkinson and Boore model with a lower weigh to model a gentler decay with distance of intermediate to long-period motions. Newer GMPEs which are strongly influenced by the Tohoku, Japan earthquake exhibit steeper decay, the USGS did not want to discount the possibility of gentler decay for the Pacific Northwest region. The response spectra from the GMPE's using a mean plus one-half standard deviation motion are shown in Figure 7. For the 2,475-year return period the resulting target response spectrum closely matches the UHS in the 0.4 to 2.0 second period range using a mean plus one-half standard deviation motion (Figure 8). The 2012 target response spectrum developed for the previous study is included on Figure 8 for comparison to the 2014 target response spectrum. The updated, 2014 target response spectrum is slightly higher in the short and long period range (up to 0.2 seconds and greater than 1.2 seconds) and slightly lower in the intermediate period range (between 0.2 and 1.2 seconds). The PGA and ground motion deviation/percentile used to match the 475-year, 975-year, 2,475-year and 4,975-year return periods are shown in Table 5. The target response spectra of the Cascadia Subduction Zone interface source for the four return periods are shown in Figure 9.

Table 5. Ground Motions relative to Mean of GMPE's for Varying Return Periods

	Yaquina Faults		**Subduction Zone (M9)**	
Return Period	**PGA**	**Percentile[1]**	**PGA**	**Percentile[1]**
475-year	0.32	31	0.33	31
975-year	0.51	62	0.47	50
2,475-year	0.79	84	0.67	69
4,975-year	1.12	93	0.95	84

[1] Mean motion equal to 50-percentile, mean plus 1 standard deviation equal to 84 percentile

The target response spectra for the Yaquina Faults source and CSZ Interface source are plotted along with the 2,475-year UHS on Figure 10. This depicts graphically how the two sources contribute to the overall total uniform hazard spectrum.

Ground Motion Database Search. A search of ground motion databases was performed to collect available recorded ground motion records and response spectra with similar seismic parameters to the Big Creek Dams site. The search included the PEER Ground Motion Database, the Consortium of Organizations for Strong Motion Observation System (COSMOS) Virtual Data Center and the Japanese Kyoshin Network (K-Net) databases. Crustal and CSZ ground motions were selected based on having similar geologic conditions, earthquake magnitude, closest distance to rupture, peak ground acceleration and the target response spectra.

Ground Motion Time-History Selection. To model the Yaquina Faults source, acceleration time histories that met the magnitude, distance, and geologic site condition (rock site) criteria were further analyzed by comparing their individual response spectra (at 5% damping) with the 2,475-year target spectrum shown in Figure 10. Five acceleration time histories were selected that closely matched the target response spectrum, with particular emphasis in the period range from 0.1 to 0.5 seconds. A summary of the selected ground motions are shown in Table 6. The individual response spectra (geometric mean of the horizontal pair) for the selected time histories along with the 2014 target spectrum are shown on Figure 11. The five selected ground motion time histories were used to create an average response spectrum which is plotted on Figure 12.

Table 6. Selected Ground Motions for Yaquina Faults Earthquake Event

Earthquake	Station	Geology $V_{S,30}$ (m/s)	Magnitude	Closest Distance to Rupture (km)	PGA[1] (g)
Superstition Hills 11/24/1987	Superstition Mtn Camera	Rock 360 m/s	6.54	5.6	0.83
Chi-Chi Taiwan 9/25/1999	TCU079	Rock 360 m/s	6.30	10.1	0.70
Bam, Iran 12/26/2003	Bam	Rock 490 m/s	6.60	1.7	0.72
Baja California 2/7/1987	Cerro Prieto	Rock 660 m/s	5.50	4.5	1.26
Coalinga 7/22/1983	Oil City	Rock 380 m/s	5.77	8.5	0.67

[1] PGA for 2,475-year return period levels (unscaled).

After selecting ground motions for the 2,475-year return period, additional analyses were performed to determine appropriate scaling factors to adjust the ground motion records for 475-year, 975-year and 4,975-year return period levels. The scaled response spectra were compared to the corresponding target response spectrum for the respective return period. The scaling factors were adjusted until a close fit was achieved. The scaling factors for the various return periods are summarized in Table 7.

Table 7. Scaling Factors for Return Period Levels for Yaquina Faults Earthquake Event

		Scaling Factor for Each Return Period Level			
Earthquake	**Station**	**475-year**	**975-year**	**2,475-year**	**4,975-year**
Superstition Hills	Superstition M.	0.40	0.65	1.00	1.37
Chi-Chi Taiwan	TCU079	0.38	0.65	1.00	1.35
Bam Iran	LA –S VA	0.44	0.70	1.00	1.45
Baja California	Cerro Prieto	0.40	0.65	1.00	1.35
Coalinga	Oil City	0.48	0.75	1.00	1.45

For the subduction zone earthquakes, there is a limited database of recorded ground motions. Ground motions from the 1985 Michoacan, Mexico earthquake (M8.1); the 1985 Valparaiso, Chile earthquake (M=7.8); and the 2011 Tohoku, Japan earthquake were evaluated. Additionally, synthetic time histories developed for other projects in the region for CSZ interface earthquakes were also evaluated.

The March 2011, Tohoku Japan earthquake has significantly increased the database of ground motions available for large mega-thrust subduction zone events and has increased the understanding of subduction zone ground motions. The 2014 NSHM and several of the GMPE's incorporated the 2011 Tohoku and 2010 Maule, Chile earthquake ground motions to augment existing data. The Tohoku earthquake was located some distance offshore and the recorded ground motions do not provide the near-source time histories that would correlate well with the expected motions at the Big Creek dam sites. In general, this means that ground motions typically need to be scaled and/or stretched to have good agreement with the target spectrum.

Subduction zone ground motions were selected from two subduction zone events (the 2011 Tohoku, Japan earthquake and the 1985 Michoacan, Mexico earthquake) and a synthetic time history developed for a dam in Northwest Oregon. The geometric mean of the horizontal earthquake response spectra (at 5% damping) were compared to the subduction zone target response spectrum (as shown in Figure 13) for the 2,475-year return period. The response spectra with similar characteristics were selected, scaled and stretched so that spectral accelerations for the medium to longer period ranges (0.4 to 2.0 seconds) were in reasonable agreement with the target response spectrum. Table 8 below shows the parameters of the selected subduction zone earthquake ground motions.

Table 8. Selected Ground Motions for Subduction Zone Interface Earthquake Events

Earthquake	Station	Geology $V_{S,30}$ (m/s)	Magnitude	Closest Distance to Rupture (km)	PGA[1] (g)
Michoacan 9/19/1985	Caleta de Campos	Rock	8.1	38.3	0.65
Tohoku 3/11/2011	Toyosato (MYG007)	Soil and Rock	9.0	151.0	0.71
CSZ Synthetic		Rock	8.5	174.0	0.62

[1] PGA scaled for 2,475-year return period levels.

For each return period (475-year, 975-year, 2,475-year and 4,975-year), the selected acceleration time histories were scaled and stretched to closely match the target response spectra. Raw ground motion spectra were first plotted without scaling or stretching. Ratios of spectral acceleration values in the 0.2 to 2 second period range were determined and applied as scaling factors. Ground motion records were stretched (increased time-step length) to provide a reasonable match with the target response spectra in the 0.2 to 2 second period range. The stretching scaling factors for the 475-year, 975-year, 2,475-year and 4,975-year return periods are shown in Table 9.

Figure 13 shows the geometric mean of response spectra for the horizontal components of the three scaled and stretched ground motions compared to the target response spectrum determined from the subduction zone GMPE's. Figure 14 compares the average response spectrum of the three selected ground motions and the target response spectrum.

Table 9. Scaling Factors for Return Period Levels for Subduction Zone Events

		Scaling Factor for Each Return Period Level			
Earthquake	Stretching Factor	475-year	975-year	2,475-year	4,975-year
Michoacan	1.25	0.75	1.10	1.70	2.2
Tohoku	1.25	0.55	0.80	1.15	1.65
CSZ Synthetic	1.25	2.5	4.0	5.2	8

Recommended Ground Motions for Stability

This seismic hazard update provides additional ground motion acceleration time histories associated with local crustal faults (Yaquina Faults) and subduction zone sources for 475-year, 975-year, 2,475-year and 4,975-year return period levels. Tables 10 and 11 provide summaries of the raw ground motion records recommended for use in seismic stability analyses. Tables 7 and 9 provide the recommended scaling and stretching factors to be applied for the return periods of interest. Digital earthquake records are included in spreadsheet format on a CD located at the end of this report. Both the raw motions and scaled and stretched ground motions for the subduction zone time histories are included in the digital ground motion records.

Table 10. List of Selected Ground Motions for Yaquina Faults Events

Earthquake	Station	Component	Individual PGA (g)	Mean Horiz. PGA (g)
Superstition Hills	Superstition Mtn	FN	0.75	0.83
	Camera	FP	0.91	
	(NGA 727)	Vertical	0.50[1]	
Chi-Chi Taiwan	TCU079	FN	0.73	0.70
	(NGA 3474)	FP	0.68	
		Vertical	0.58	
Bam, Iran	Bam	FN	0.81	0.72
	(NGA 4040)	FP	0.63	
		Vertical	0.97	
Baja California	Cerro Prieto	FN	1.15	1.26
	(NGA 585)	FP	1.38	
		Vertical	0.59	
Coalinga	Oil City	FN	0.87	0.67
	(NGA 407)	FP	0.49	
		Vertical	0.57	

[1] Vertical acceleration time history is not available for this record, vertical acceleration time history is taken as to be 2/3 of the FN acceleration time history

Table 11. List of Selected Ground Motions for Subduction Zone Events

Earthquake	Station	Component	Individual PGA (g)[1]	Mean Horiz. PGA (g)[1]
Michoacan	Caleta de	H1	0.40	0.38
	Campos	H2	0.36	
		Vertical	0.42	
Tohoku	Toyasato	EW	0.66	0.62
	(MYG007)	NS	0.58	
		Vertical	0.25	
CSZ Synthetic		H1	0.12	0.12
		H2	0.12	
		Vertical	0.08	

[1] PGA values shown are raw values and not scaled for return period of interest.

We trust that this report is sufficient for your current requirements. Should you have any questions or comments, please call.

Sincerely,

CORNFORTH CONSULTANTS, INC.

Christopher I Carpenter, P.E.
Associate Engineer

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NOV. 13, 2012
CHRISTOPHER IAN CARPENTER
RENEWS: 12/31/2015

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Limitations in the Use and Interpretation of this Geotechnical Report

Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally accepted engineering principles and practices. This warranty is in lieu of all other warranties, either expressed or implied.

The geotechnical report was prepared for the use of the Owner in the design of the subject facility and should be made available to potential contractors and/or the Contractor for information on factual data only. This report should not be used for contractual purposes as a warranty of interpreted subsurface conditions such as those indicated by the interpretive boring and test pit logs, cross-sections, or discussion of subsurface conditions contained herein.

The analyses, conclusions and recommendations contained in the report are based on site conditions as they presently exist and assume that the exploratory borings, test pits, and/or probes are representative of the subsurface conditions of the site. If, during construction, subsurface conditions are found which are significantly different from those observed in the exploratory borings and test pits, or assumed to exist in the excavations, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. If there is a substantial lapse of time between the submission of this report and the start of work at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, this report should be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

The Summary Boring Logs are our opinion of the subsurface conditions revealed by periodic sampling of the ground as the borings progressed. The soil descriptions and interfaces between strata are interpretive and actual changes may be gradual.

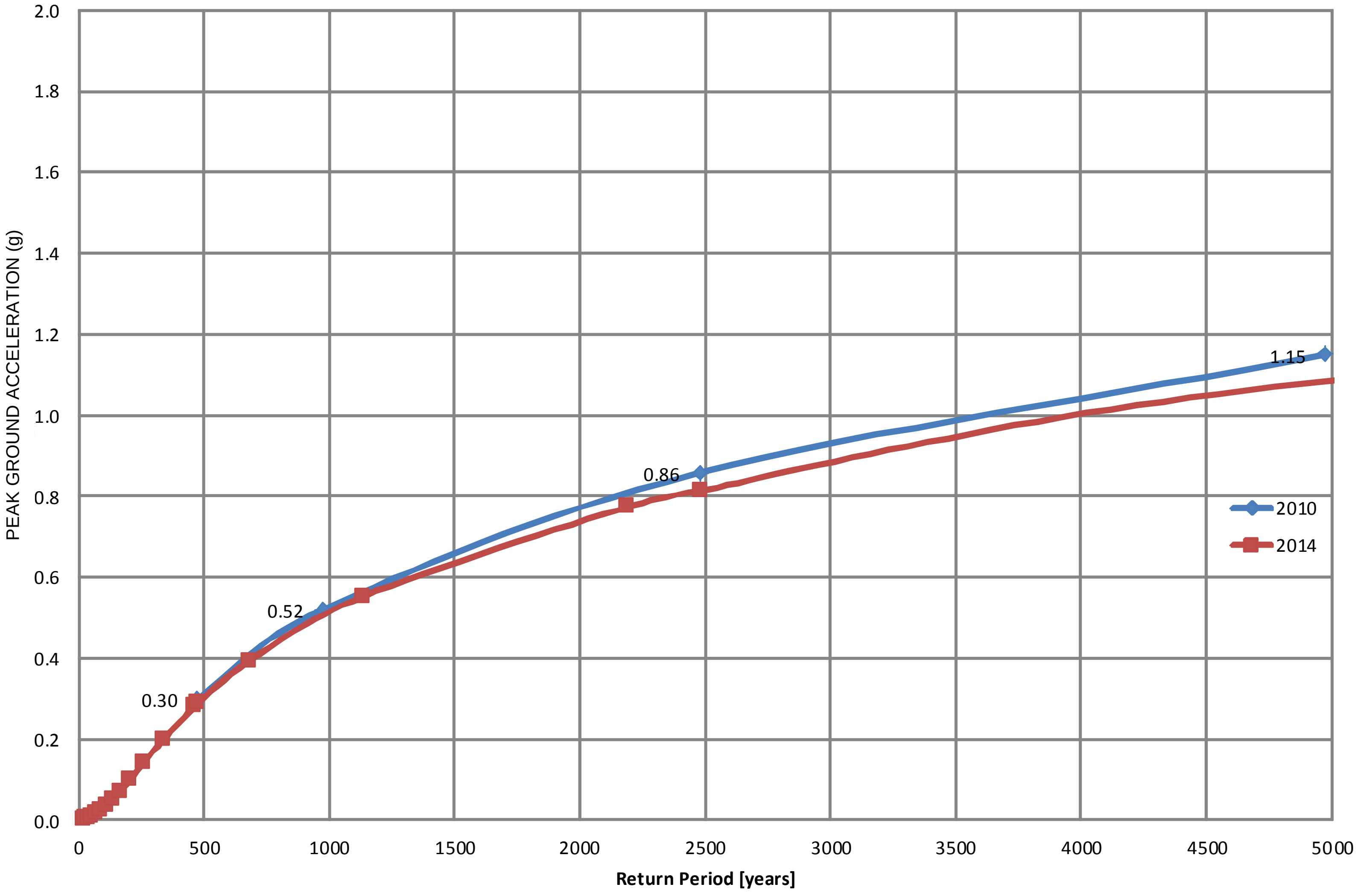
The boring logs and related information depict subsurface conditions only at these specific locations and at the particular time designated on the logs. Soil conditions at other locations may differ from conditions occurring at these boring locations. Also, the passage of time may result in a change in the soil conditions at these boring locations.

Groundwater levels often vary seasonally. Groundwater levels reported on the boring logs or in the body of the report are factual data only for the dates shown.

Unanticipated soil conditions are commonly encountered on construction sites and cannot be fully anticipated by merely taking soil samples, borings or test pits. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. It is recommended that the Owner consider providing a contingency fund to accommodate such potential extra costs.

This firm cannot be responsible for any deviation from the intent of this report including, but not restricted to, any changes to the scheduled time of construction, the nature of the project or the specific construction methods or means indicated in this report; nor can our firm be responsible for any construction activity on sites other than the specific site referred to in this report.

Comparison of 2010 and 2014 USGS NSHM Peak Ground Acceleration ($V_{S,30}$ = 760 m/s) versus Return Period



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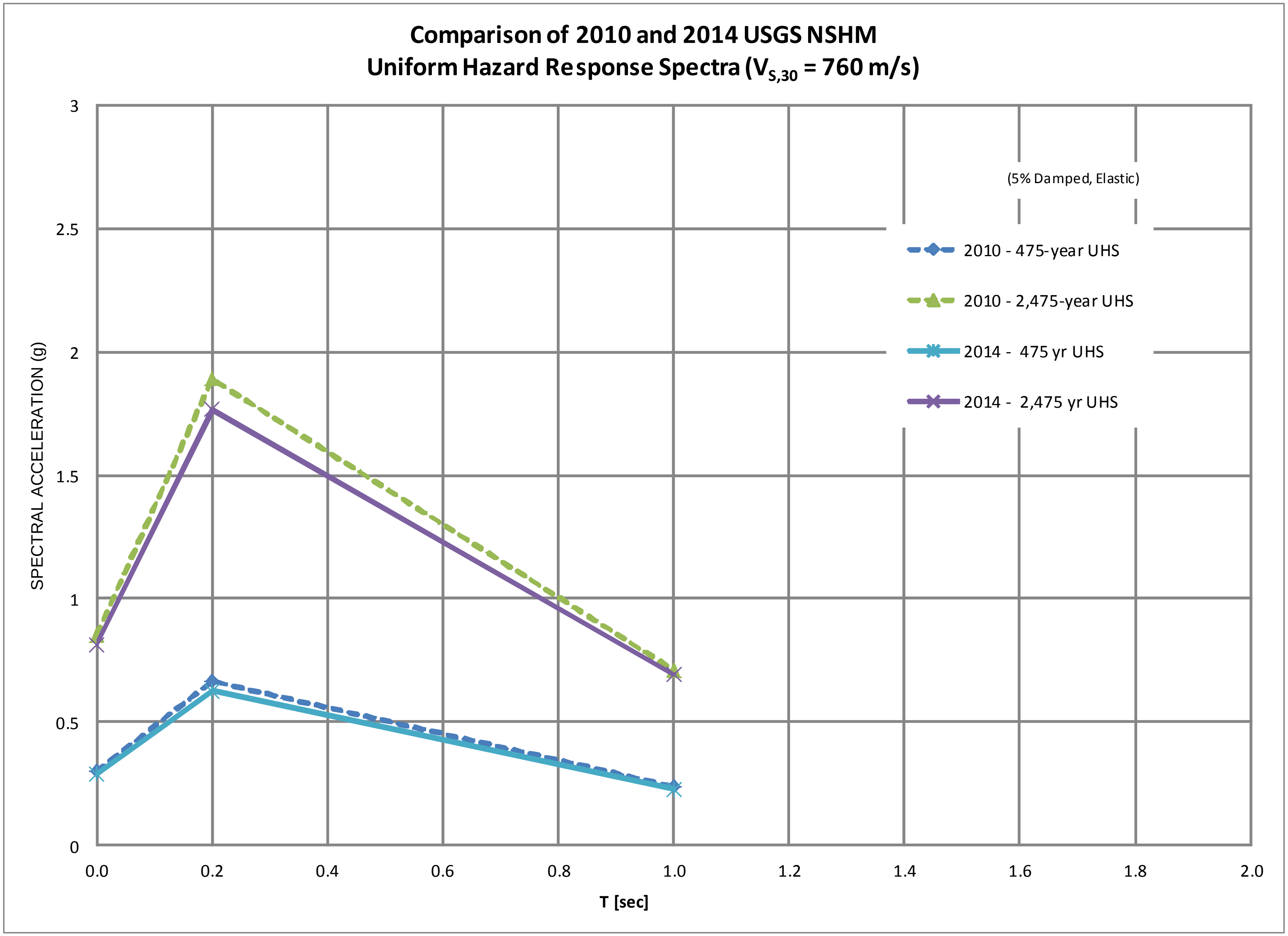
NATIONAL SEISMIC HAZARD MAP
PGA COMPARISON

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

DEC 2014

PROJ. 2.384

FIG. 1



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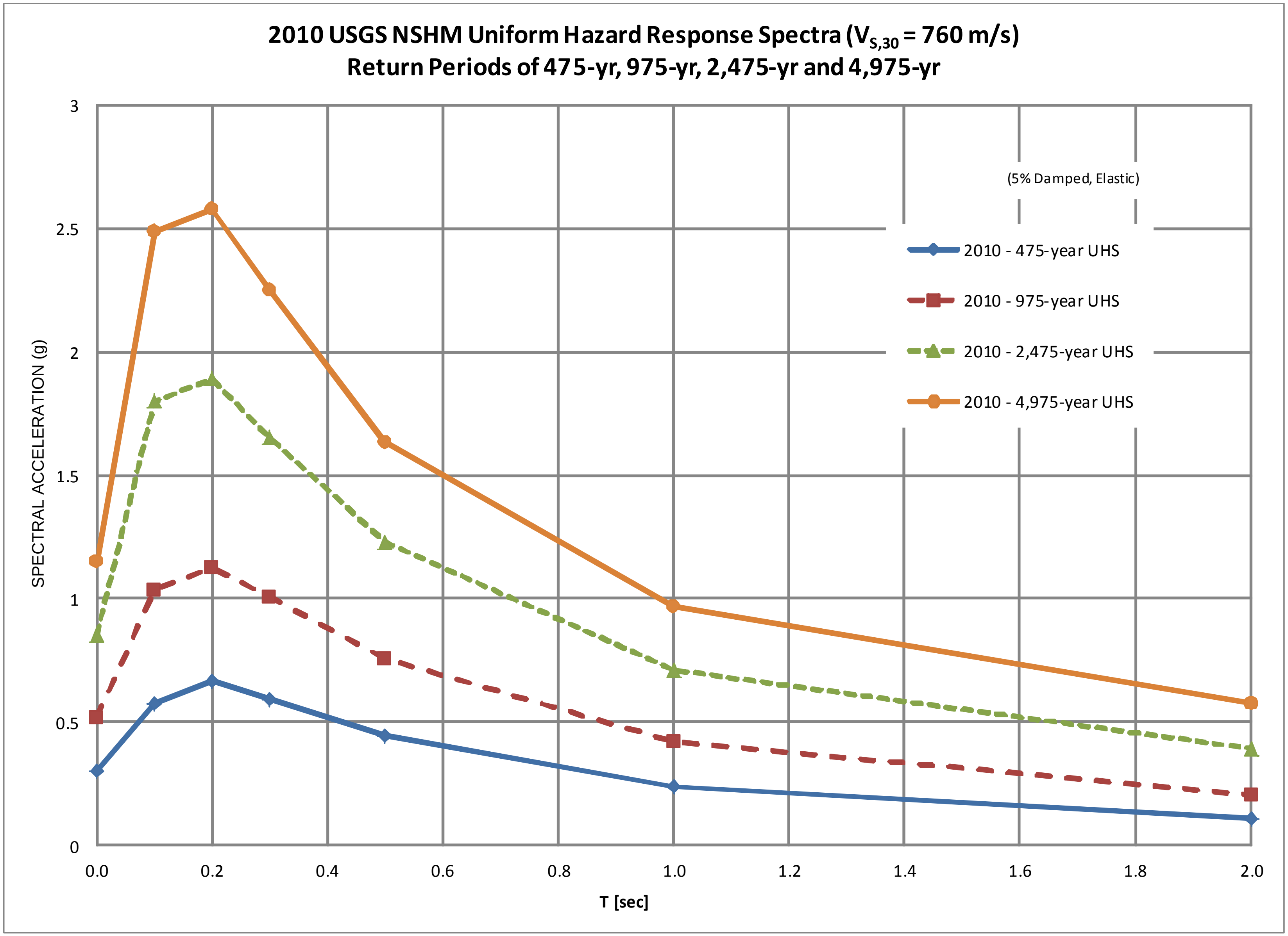
NATIONAL SEISMIC HAZARD MAP
UHS COMPARISON

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

DEC 2014

PROJ. 2.384

FIG. 2



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NATIONAL SEISMIC HAZARD MAP
UNIFORM HAZARD SPECTRA

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

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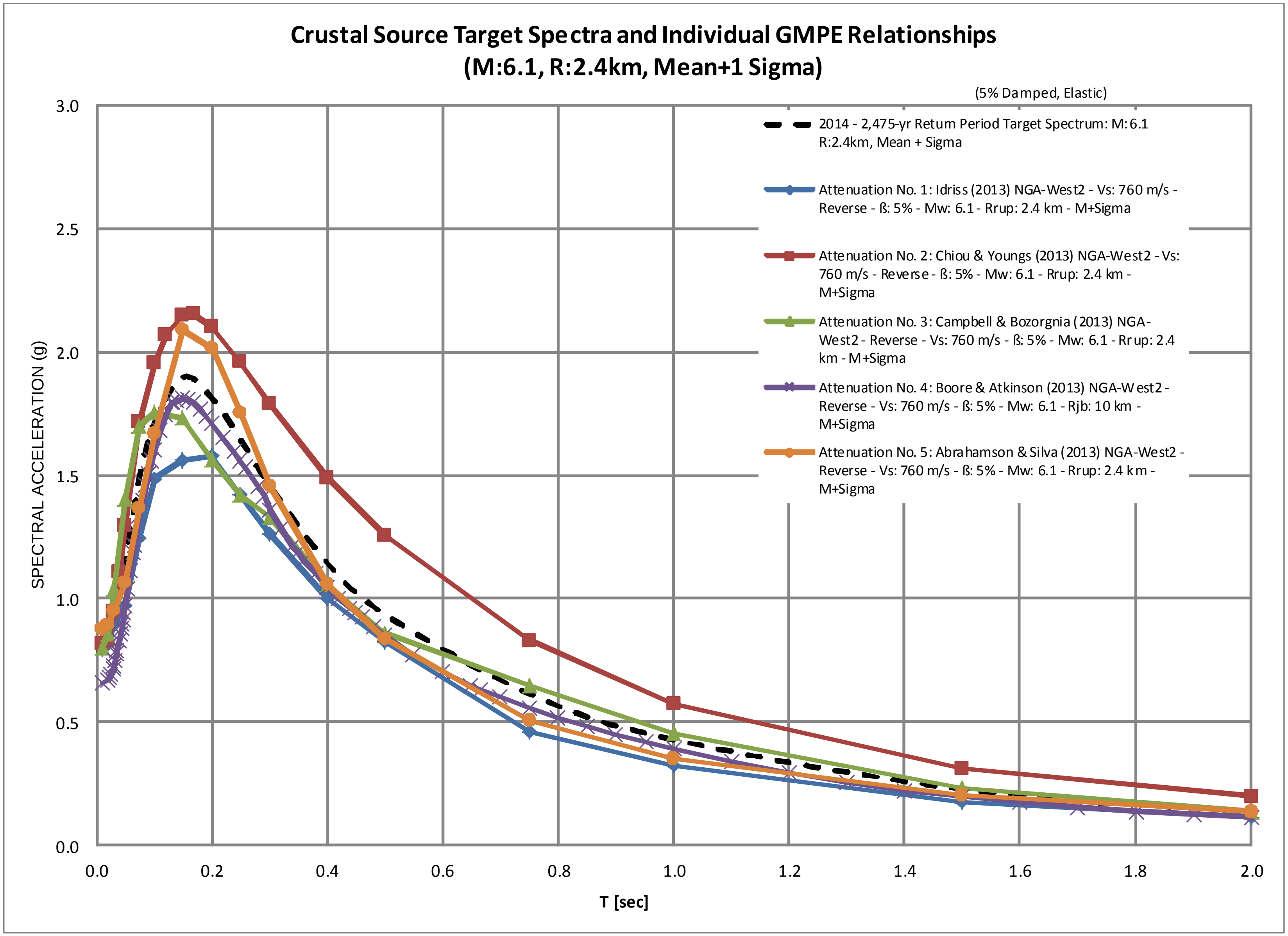
PROJ. 2.384

FIG. 3



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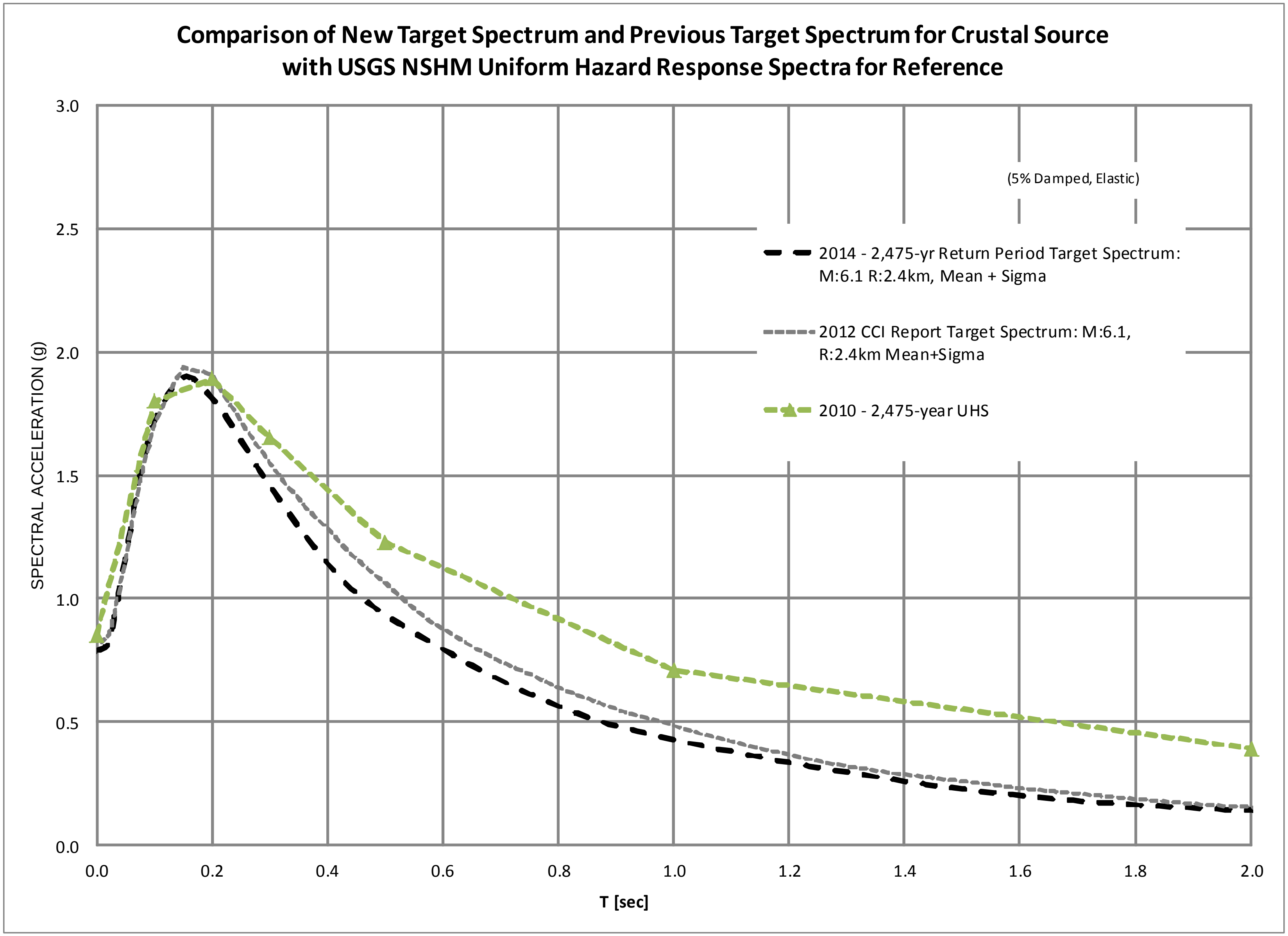
CRUSTAL SOURCE
GMPE RESPONSE SPECTRA

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

DEC 2014

PROJ. 2.384

FIG. 4



Comparison of New Target Spectrum and Previous Target Spectrum for Crustal Source with USGS NSHM Uniform Hazard Response Spectra for Reference

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CRUSTAL SOURCE
TARGET SPECTRA COMPARISON

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

DEC 2014

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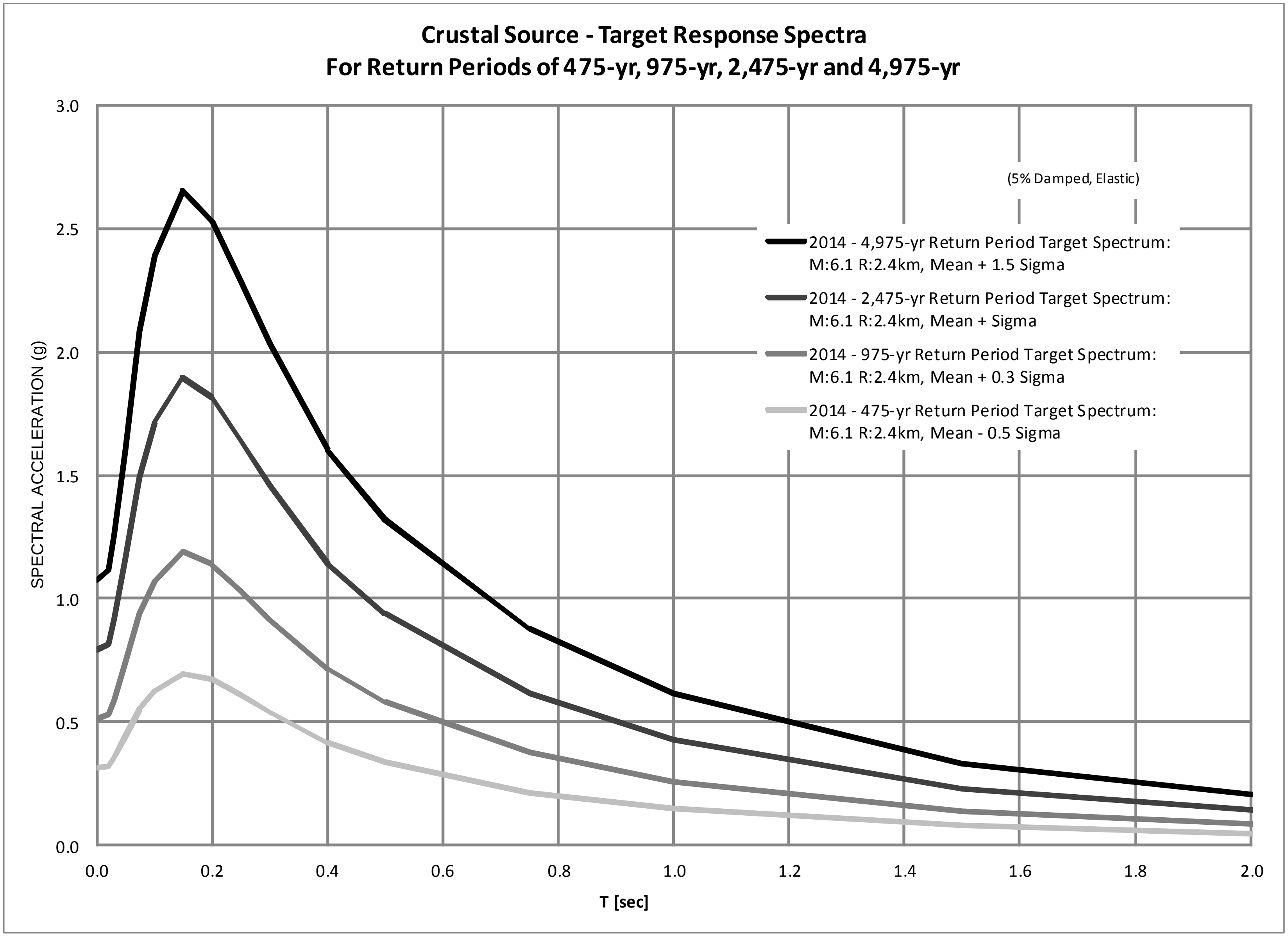
FIG. 5



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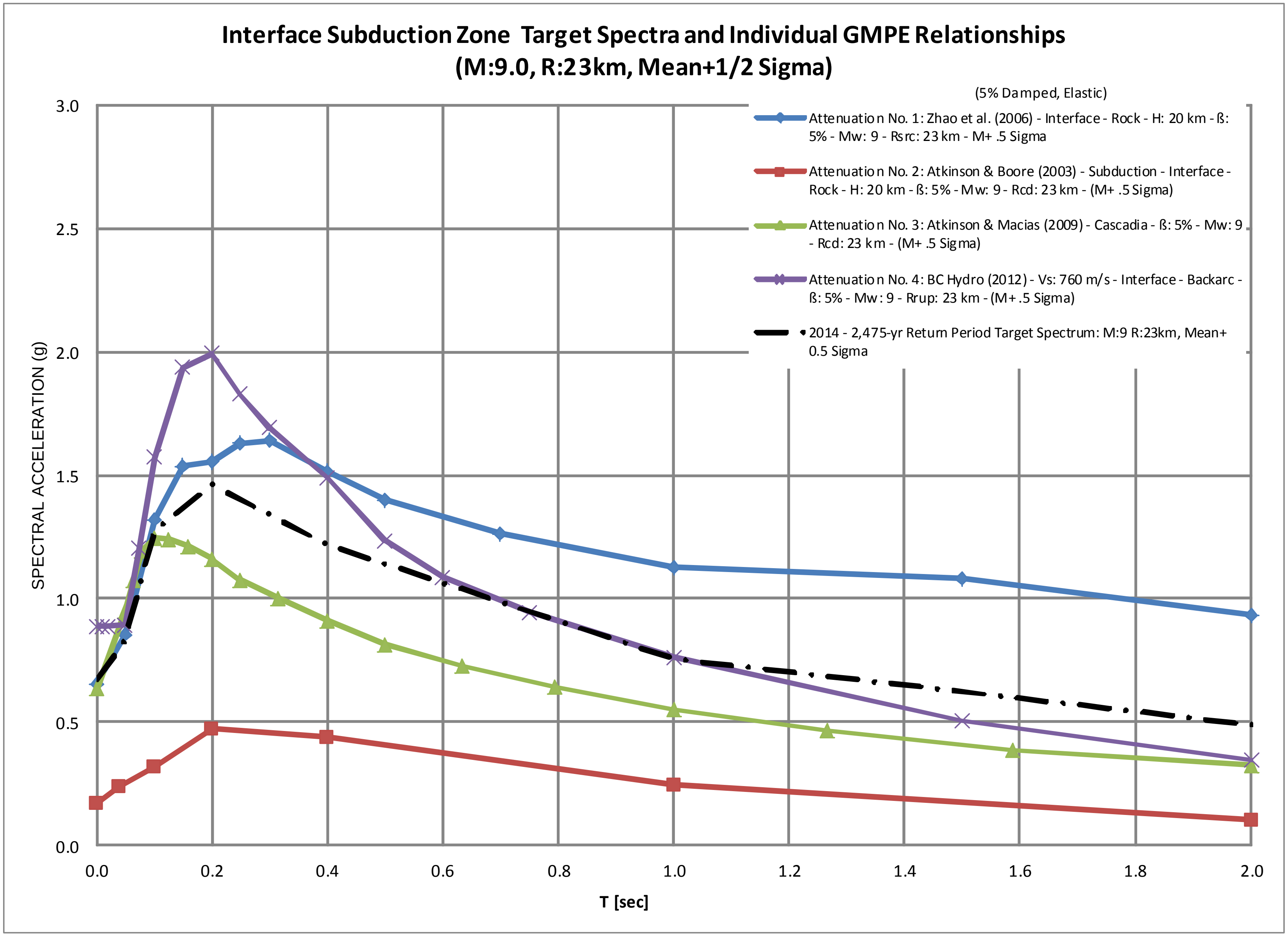
CRUSTAL SOURCE
TARGET SESPONSE SPECTRA

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

DEC 2014

PROJ. 2.384

FIG. 6



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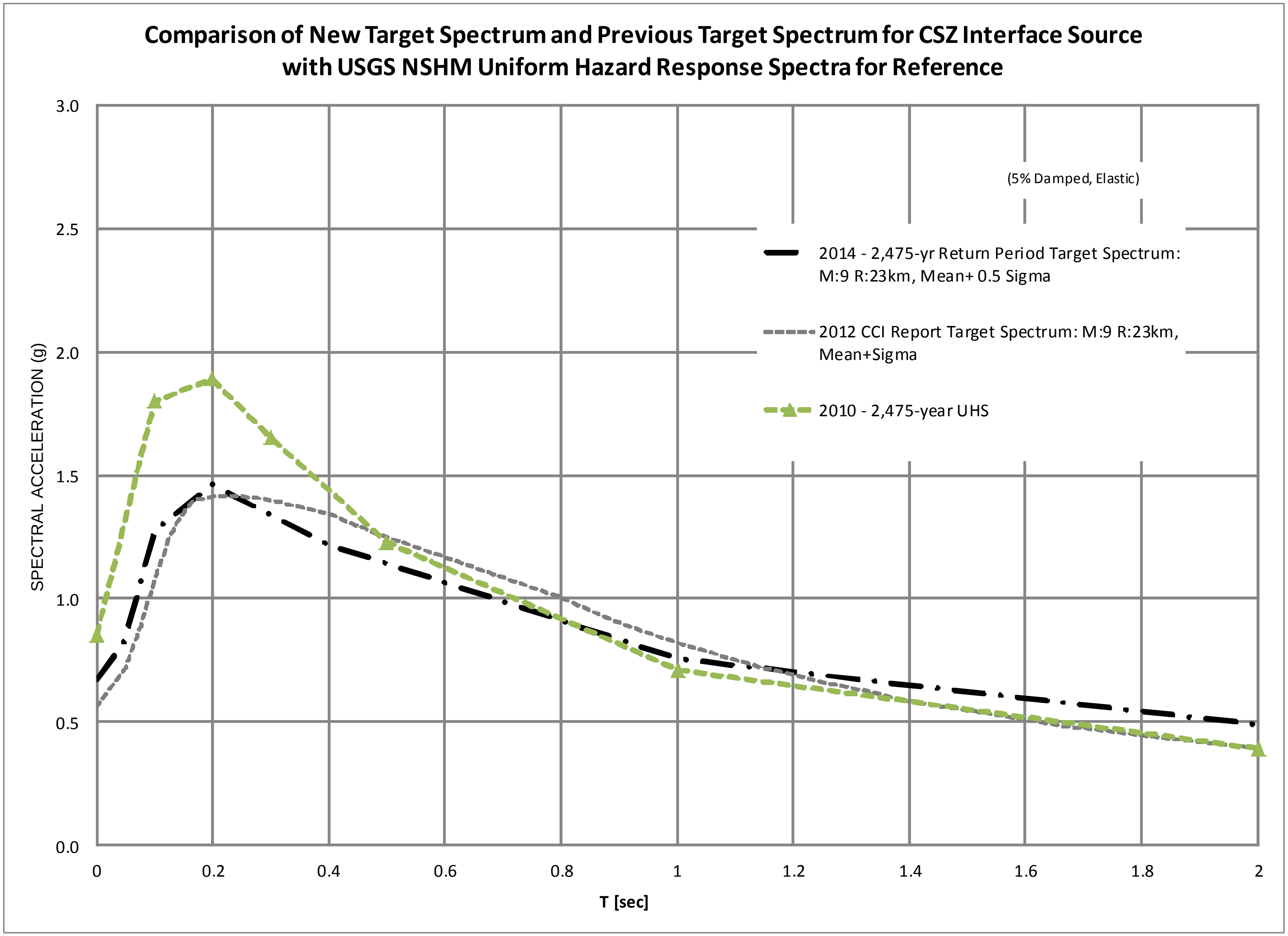
CSZ INTERFACE
GMPE RESPONSE SPECTRA

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

DEC 2014

PROJ. 2.384

FIG. 7



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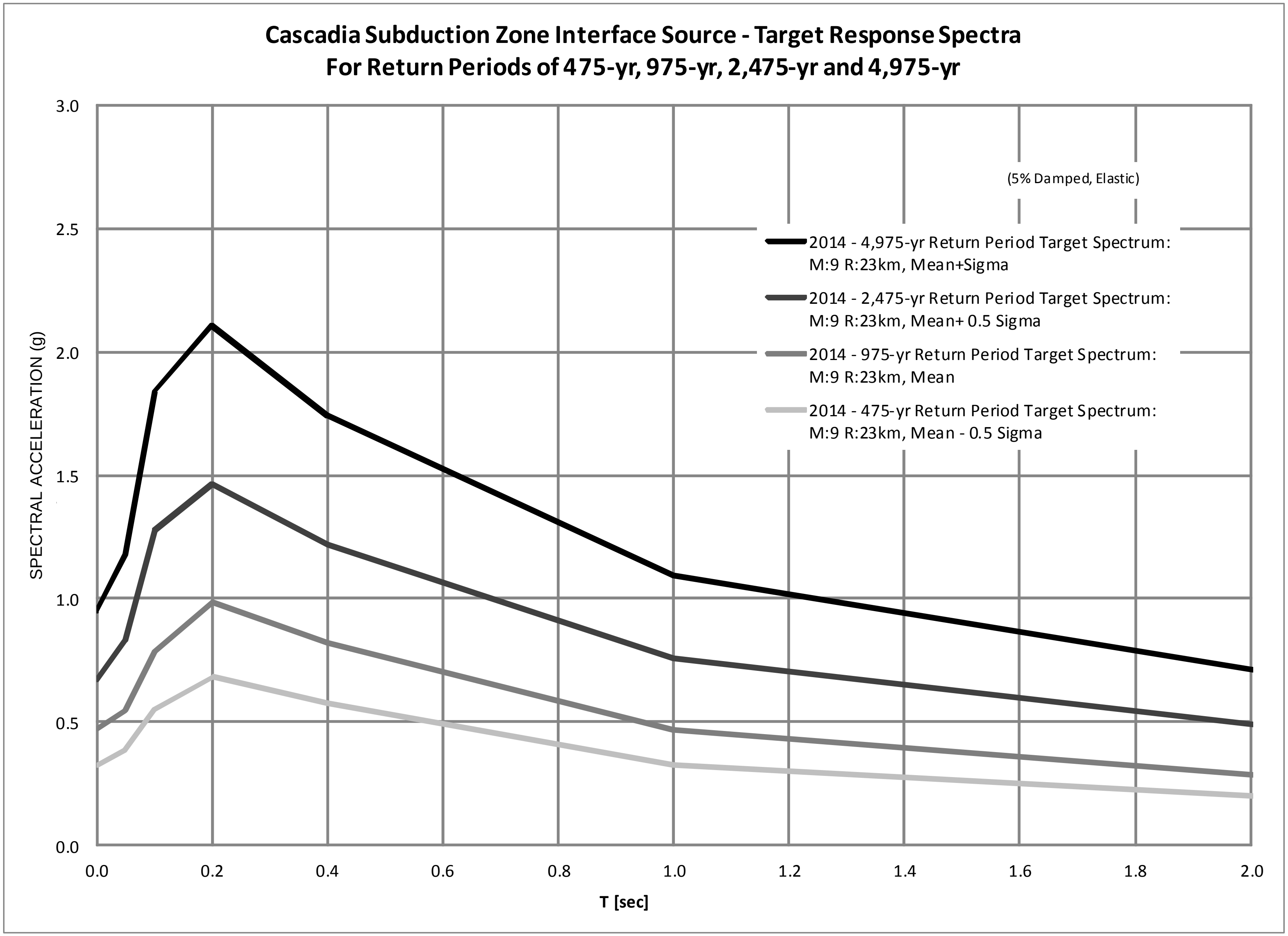
CSZ INTERFACE
GMPE RESPONSE SPECTRA

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

DEC 2014

PROJ. 2.384

FIG. 8



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CSZ INTERFACE
TARGET RESPONSE SPECTRA

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

DEC 2014

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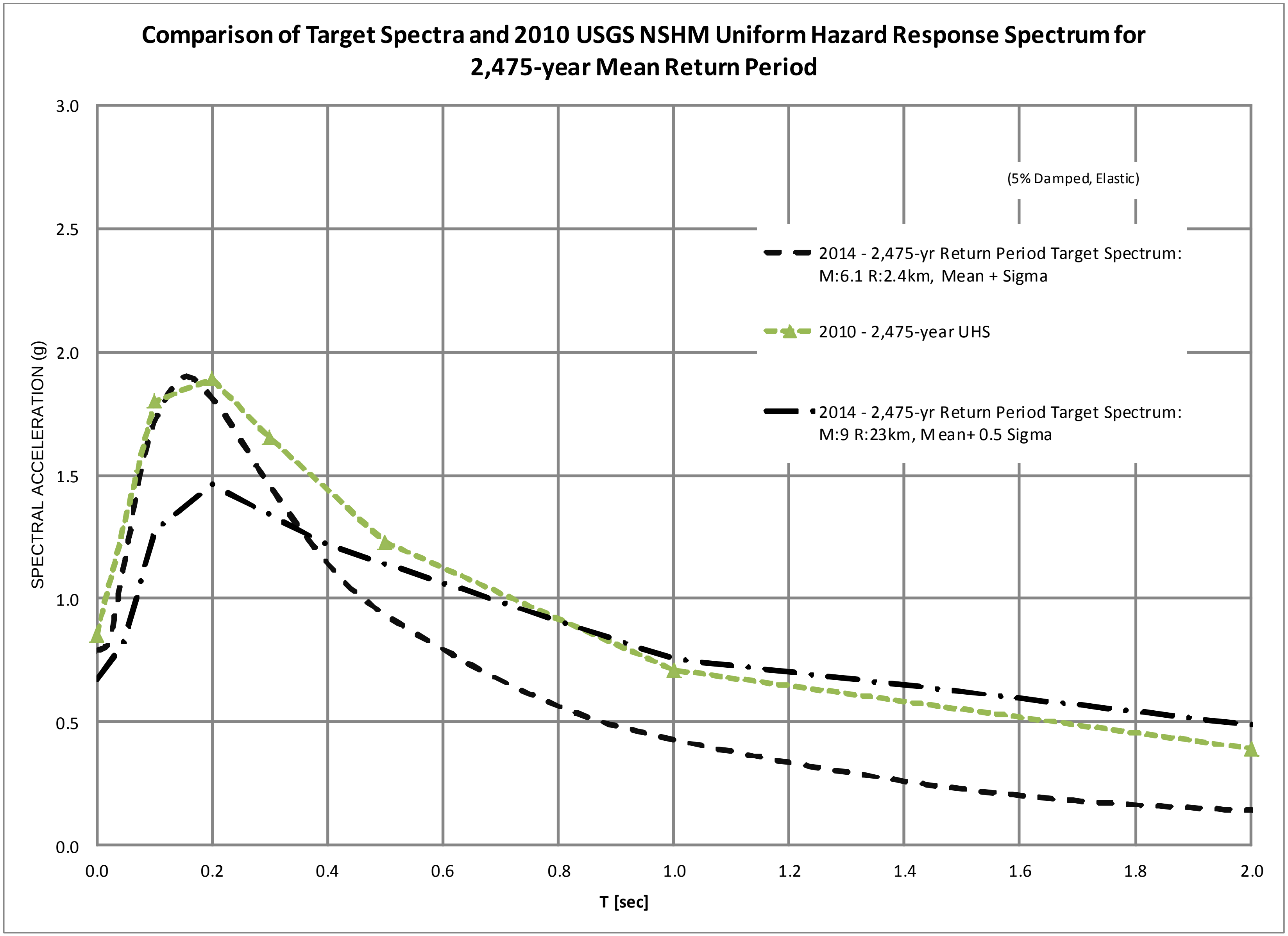
FIG. 9



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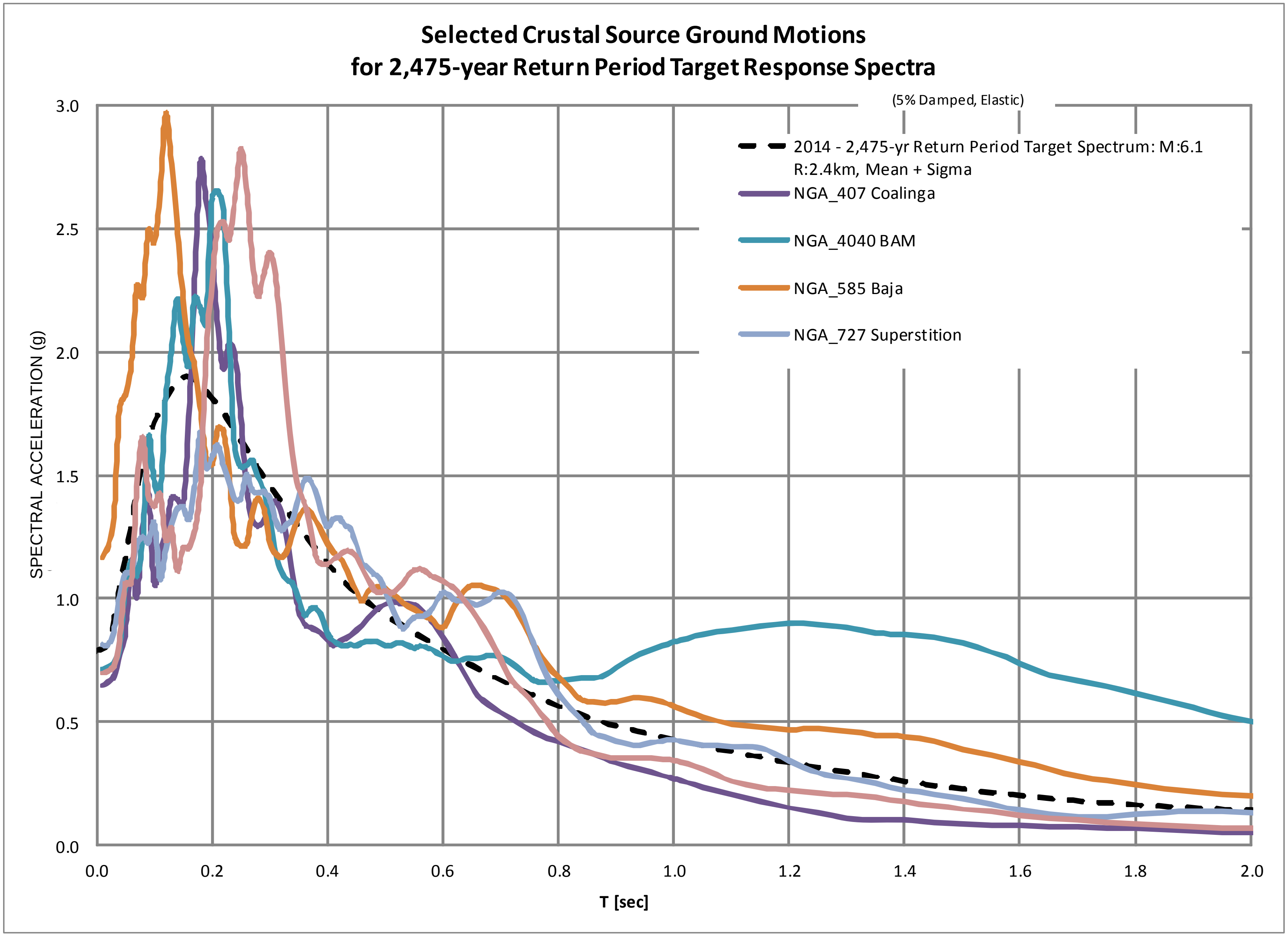
TARGET RESPONSE SPECTRA
COMPARISON WITH UHS

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

DEC 2014

PROJ. 2.384

FIG. 10



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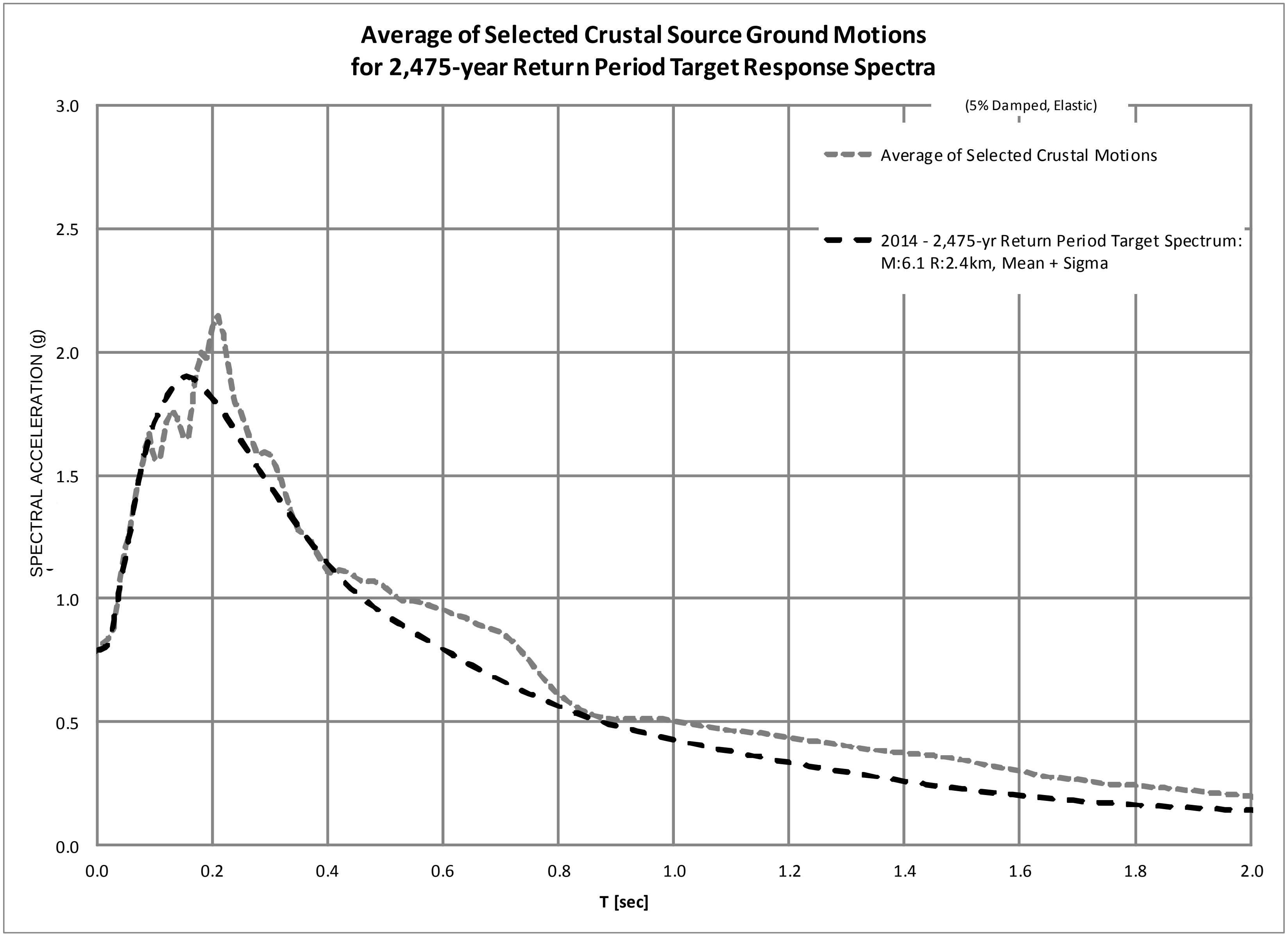
CRUSTAL SOURCE
SELECTED GROUND MOTION SPECTRA

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

DEC 2014

PROJ. 2.384

FIG. 11



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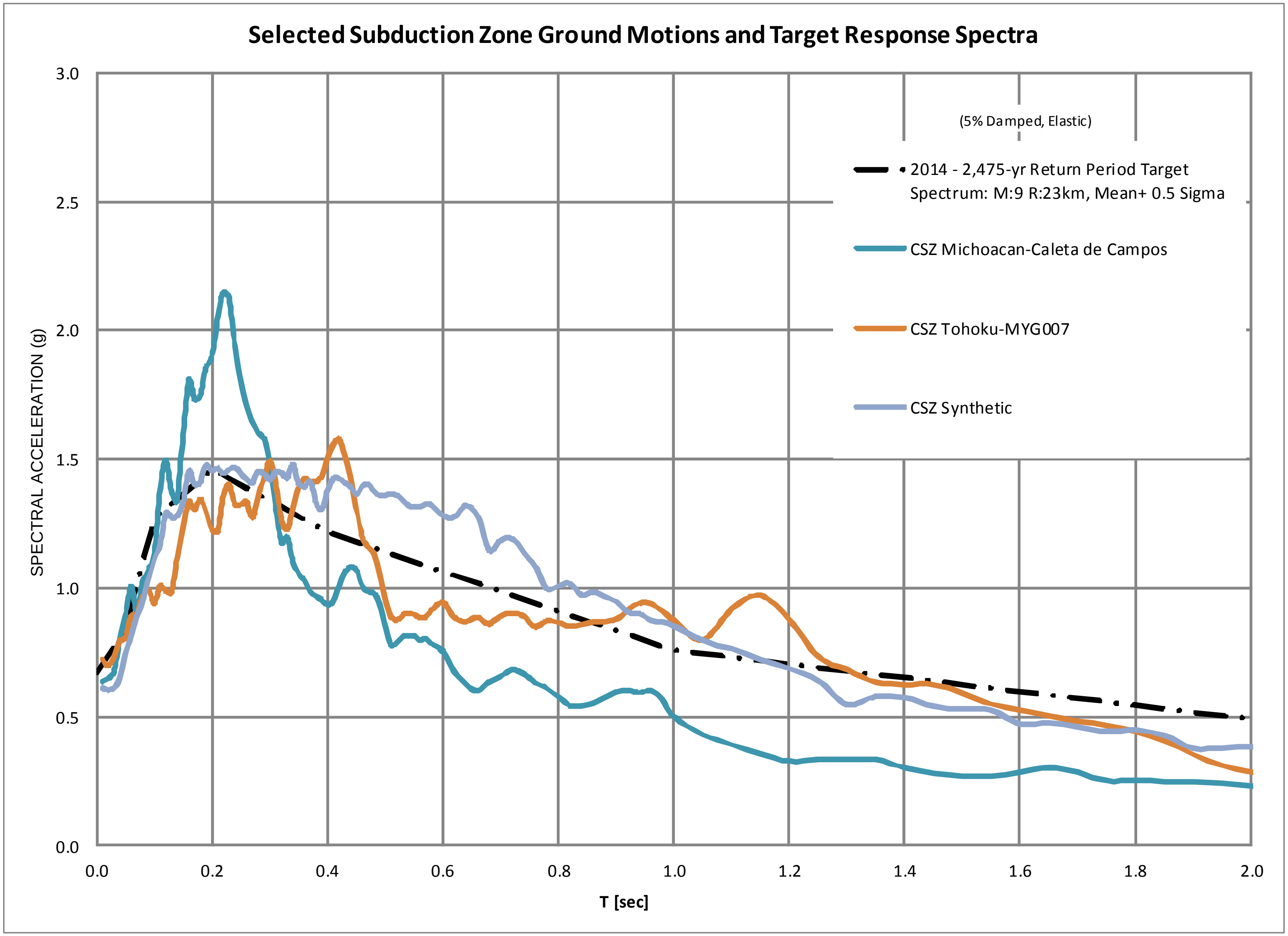
CRUSTAL SOURCE
GROUND MOTION AVERAGE SPECTRA

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

DEC 2014

PROJ. 2.384

FIG. 12



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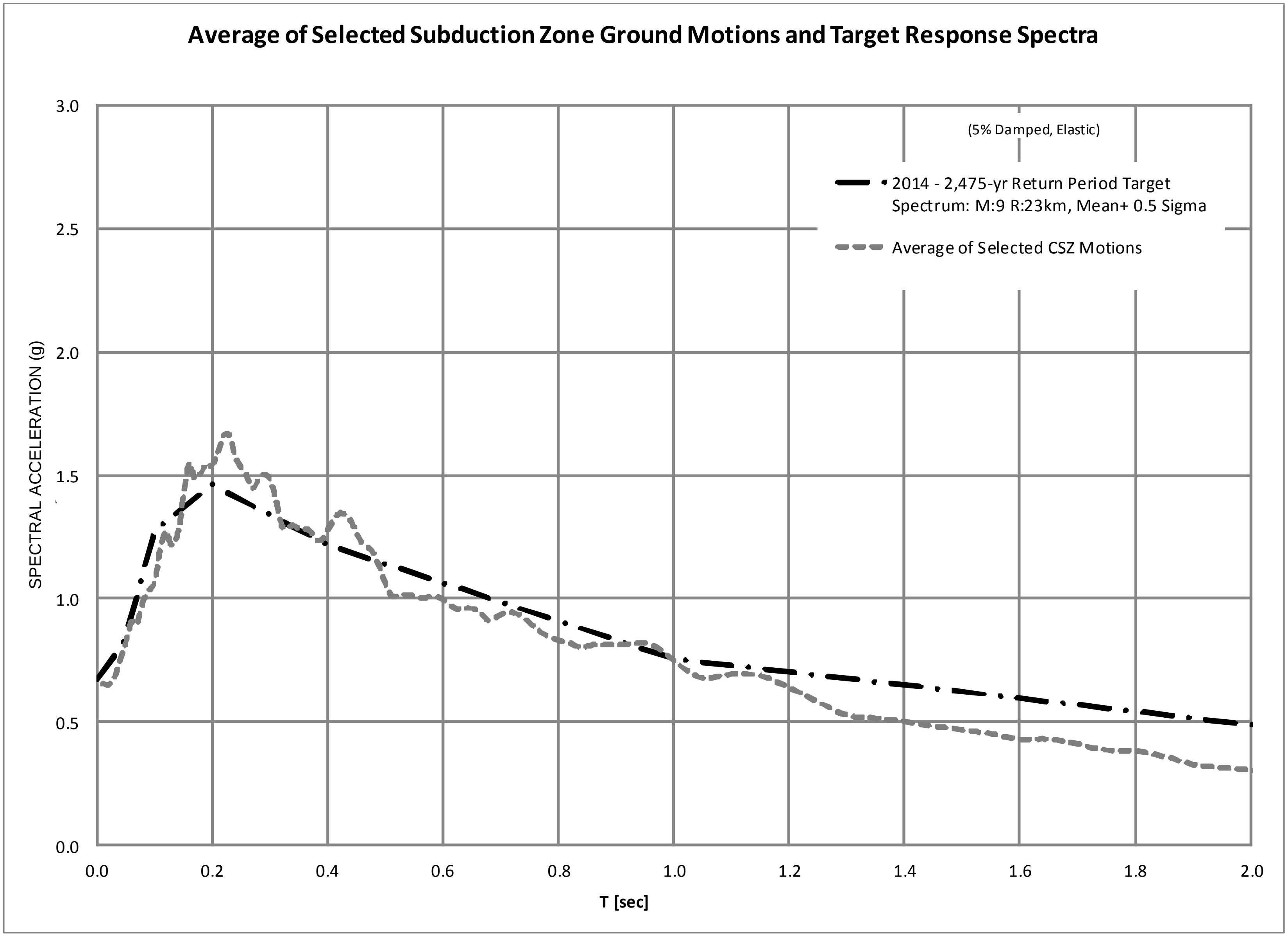
CSZ INTERFACE
SELECTED GROUND MOTION SPECTRA

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

DEC 2014

PROJ. 2.384

FIG. 13



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CSZ INTERFACE
GROUND MOTION AVERAGE SPECTRA

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

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FIG. 14

Appendix B.
Site Characterization

Appendix B

Site Characterization

1.0 Geologic Setting

The regional and site geologic settings are provided in the following sub-sections.

1.1.1 Regional Geology

The Big Creek Dams No. 1 and No. 2 (BC 1 and BC 2, respectively) lie at the western margin of the Oregon Coast Range physiographic province which consists of a moderately high mountain range (Elevations as high as 4,200 feet [ft]) and coastal headlands interspersed with shallow bays, estuaries, beaches, and dunes. The Oregon Coast Range Province is a belt of land uplifted as a result of plate convergence between the Juan de Fuca plate and the North American Plate. The Coast Range overlies the subducted Juan de Fuca Plate and lies east of the Cascadia Subduction Zone (Orr 1992). The region seaward of the location of volcanism is referred to as a forearc basin and the materials deposited in the forearc due to the descending subducting plate is known as an accretionary wedge (Roering 2008). The accretionary wedge is composed of accumulated oceanic sediments and resulting sedimentary rocks, which can be found underlying the project site and surrounding region.

The Coast Range is characterized by gently west dipping, regionally extensive marine sandstone and siltstone. These marine sedimentary rocks, deposited as a result of the accretionary wedge, and are overlying older, Paleocene to Eocene volcanic rocks (Roering 2008). Much of the volcanic formations are the result of pillow basalt formations created when a hot basalt flow rapidly cooled upon meeting the salt water of the ocean (Orr 1992). Synchronous with uplift of sedimentary rocks in the region, large fissures were developed bringing lava flows to the surface, and intrusion of many dikes and sills into the overlying rock throughout the Coast Range. Continued uplift of the Oregon Coast Range has contributed to the development of marine sediment terraces near the coastline (Roering 2008).

1.1.2 Site Geology

The dam sites are located approximately 2 miles north of Yaquina Bay and 0.5 mile inland from Agate Beach. Review of available geologic information indicates the bedrock underlying the dam and reservoir sites is Miocene era Marine sedimentary rock. Snavely, MacLeod, Wagner, and Rau (1976) mapped the bedrock formation as Nye which is generally characterized as a massive, organic-rich mudstone and siltstone, containing sandy siltstone and fine-grained sandstone (Snavely et al. 1976). The bedrock geology of the BC 1 and BC 2 sites is presented in Plate B-1 (Plate B-1 and all figures are located at the end of this report). The marine sedimentary rock is overlain with alluvial streambed material consisting of sands and silts as well as colluvium. The bedrock outcrops at the abutments for both dams, and it appears the alluvial sediment is deepest at the location of the current Big Creek stream channel.

Photo 1: Nye Formation Siltstone in right abutment of BC 1.

2.0 Seismic Setting

This seismic setting discussion of BC 1 and BC 2 is based on the findings of the reports prepared by Cornforth Consultants in 2012 and 2014 titled “Seismic Review and Ground Motion Development” and “Big Creek Dam No. 1 and Dam No. 2 Seismic Hazard Update.” The regional tectonic setting of the project area lies within a zone of active convergence between the Juan de Fuca Oceanic plate and the North American Continental plate. Compressive forces on a global scale are forcing the denser Juan de Fuca plate beneath the lighter North American plate. This process is referred to as “subduction.” Within this regional tectonic setting there are three general types of earthquakes that could generate ground motions at the site. Two are related to the subduction zone (interface and intraplate earthquakes), and the third involves shallow crustal earthquakes within the North American plate. Only the intraplate and crustal earthquakes represent significant hazard to the Big Creek dam sites and are capable of generating significant seismic shaking. Crustal faults are generally located in the upper 20 miles of the earth’s crust and typically have some surface expression related to the movement of the fault. The Cascadia Subduction Zone interface is generally considered to be located at a depth of 50 to 75 miles below the ground surface at the site.

Known active faults in the region have been mapped by the United States Geological Survey (USGS) using information from a number of sources. The location of the faults and information related to them are available through the USGS Earthquake Hazard Program. The Quaternary

Fault Map and associated database is available at http://earthquake.usgs.gov/hazards/qfaults/. Locations of earthquakes along the central Oregon coast during the period 1841 through 2002 are shown on Figure 1 of the Cornforth "Seismic Review and Ground Motion Development" Report (Cornforth 2012). The Quaternary faults and folds of the region are shown on Figure 2 of the Cornforth Report. Quaternary faults are faults that have occurred during the last 2.6 million years and are considered potentially active. Cornforth performed an update to the "Seismic Review and Ground Motion Development Report" based on updated USGS 2014 National Seismic Hazard Maps. The 2014 Cornforth report is included in Appendix A, Seismic Hazards. Two significant sources of seismic hazard were identified for the dam sites.

The first seismic hazard source is the Yaquina Fault which is located approximately 1.9 miles north of the two dams. The Yaquina Fault is a crustal fault approximately 8 miles long. The Yaquina Fault has the potential of producing a magnitude M 6.1 earthquake. Due to the close proximity of the fault to the dams the peak ground acceleration (PGA) at the dam sites is expected to range from 0.32g for a 475-year return period to 1.12g for a 4975-year return period. There have been no recorded earthquake events attributed to this fault, but geologic evidence suggests that the fault is active.

The second seismic hazard source is the Cascadia Subduction Zone (CSZ) located approximately 14 miles off the Oregon coast in the Newport area. The CSZ has the potential of producing a magnitude M 9.0 earthquake, but due to the distance the PGA was determined to be 0.33g with a recurrence interval of 475 years and 0.96g for a 4975-year return period. The CSZ is believed to have generated a magnitude M 9.0 earthquake on January 29, 1700. Geologic evidence suggests that there have been several events related to the CSZ over the last few thousand years, and that the events have been occurring for several million years. Table 5 in Appendix A Seismic Hazards presents the estimates of PGAs for 475-, 975-, 2,475-, and 4,975-year return periods for Yaquina and CSZ earthquakes.

Recent studies of underwater turbidite deposits along the Cascadia margin indicate the CSZ can be subdivided into a northern and southern section with three potential rupture modes: full length, 50 to 70 percent of the southern section, and smaller seismic events for short reaches of the southern section (Goldfinger et al. 2012). For a full length rupture, an average return period for a great earthquake is about 500 to 530 years. The average return period for an earthquake along the southern section of the CSZ based on analysis of the turbidite deposits is approximately 240 years. Therefore, a great earthquake on the full length CSZ could be expected to occur within the next 200 years and a large earthquake of a lesser magnitude on the southern section could occur at any time since it has been 300 years since the last recorded CSZ earthquake

3.0 Field Investigations

Field investigations to characterize the site subsurface conditions have occurred in two additional phases. Phase 2 of field investigations occurred in December 2011 through January 2012. These investigations consisted of three components: geotechnical drilling (mud rotary and hollow stem auger), cone penetrometer testing, and a surface geophysical survey. The third phase (Phase 3) of investigations occurred in November 2013. These investigations consisted of two components: geotechnical drilling (mud rotary) and cone penetrometer testing. The exploration locations are shown on Figure B-1 and Figure B-2 for BC 1 and BC 2, respectively. Summaries of the drilling and cone testing programs completed during both phases of work are provided in Table 1 and Table 2 (located in Section 3.0).

3.1 2011-2012 Field Investigations

3.1.1 Geotechnical Drilling

One boring was drilled at the BC 1 (BC1-B-1) and three borings were drilled at the BC 2 (BC2-B-1 through BC2-B-3) from December 12 through December 15, 2011 and on January 5, 2012 by Western States Drilling. Table 1 provides exploration completion depths to top of the weathered and decomposed Nye Formation based on blow counts that define a stiff clay or silt, or medium dense to dense sand; and observation of some rock structure in the Standard Penetration Test (SPT) sample; and the depth to the top of bedrock based on SPT blow counts of 50/6 inch or greater. Information from all BC 1 and BC 2 drilling, including the subsequent investigations in November 2013, is included in this table.

Table 1: Geotechnical Drilling Summary

Boring ID	Ground Surface El. (NAVD88)	Boring Depth (ft)	Depth to Decomposed Rock (ft)	Depth to Bedrock (ft) (N≥50/6”)
BC1-B-1	47.4	86.5	85	--
BC1-B-2	33.1	69.4	63	66
BC1-B-3(u)	33.0	64.5	61.5	--
BC1-B-4(u)	33.0	70.0	58	67
BC2-B-1	91.6	80.0	67	72
BC2-B-2	91.2	71.5	42	--
BC2-B-3	50.1	41.5	30	--
BC2-B-4	50.0	46.7	39	45
BC2-B-5(u	50.0	48.5	45	--
BC2-B-6(u)	50.0	41.5	30	--

Table Source: Cornforth Consultants Drilling Logs, Northwest Land Surveying, Inc.
NAVD88 = North American Vertical Datum of 1988

Boreholes were advanced using a combination of truck- and track-mounted drill rigs and mud rotary and hollow stem auger drilling methods. The borings were advanced through the existing dams using hollow stem augers to minimize concerns related to hydraulic fracturing of the embankment. The borings were continued using mud rotary techniques beneath the embankment. Boring logs are included in Attachment B 1. Phase 2 Geotechnical Data.

Both disturbed and undisturbed samples were obtained at 5-foot intervals within the embankment dams and 2.5-foot intervals thereafter. Disturbed samples were obtained with an SPT split-spoon sampler in accordance with American Society for Testing and Materials (ASTM) D1586. The hammer energy for the SPT driving system was measured for each drilling rig to obtain the actual energy transfer ratio for the driving system (GeoDesign 2012). The SPT N-value blow counts (as defined in ASTM D1586) were obtained for each sample and recorded on the boring log; the corrected blow counts (i.e., N_{60} corresponding to a 60 percent hammer efficiency) based on the measured energy transfer ratio are also shown on the logs. Undisturbed soil samples were obtained with 3-inch-diameter thin-walled Shelby tube samples at selected depths in accordance with ASTM D1587. HQ wire-line coring methods were used in boring BC2-B-1 to core the siltstone bedrock in accordance with ASTM D2113. HQ coring consists of a 2.5-inch inner diameter triple-walled core barrel advanced in maximum 5-foot runs. Core samples were logged following the rock logging procedures of the United States Department of the Interior Bureau of Reclamation 2001 Engineering Geology Field Manual and photographed.

As shown on Figure B-1 boring BC1-B-1 at BC 1 was drilled along the dam crest approximately 150 feet from the southern end, near the estimated deepest section of the original creek channel. The purpose of this boring was to evaluate the strength and consistency of the fill material within the dam and soils underlying the dam. The boring was drilled to a total depth of 86.5 feet. Highly weathered and decomposed siltstone bedrock was encountered at a depth of 85 feet.

Borings BC2-B-1 and BC2-B-2 were drilled from the crest of BC 2 as shown on Figure B-2. The purpose of these borings was to establish the consistency and depth of the embankment fill, and evaluate the soils underlying the crest of the dam. BC2-B-1 was drilled at the estimated deepest section of the original channel and BC2-B-2 was drilled approximately 140 feet from the northern end of the dam. The total depth of borings BC2-B-1 and BC2-B-2 were 80 and 71.5 feet, respectively.

Boring BC2-B-3 was drilled to a depth of 41.5 feet near the southern end of the dam at the downstream toe approximately 100 feet from the dam centerline. The purpose of this boring was to identify any embankment fill beneath the toe, and to estimate the extent and properties of the alluvial soils that underlie the dam. The top of the weathered and decomposed siltstone was encountered at a depth of 30 feet.

The boreholes were continuously logged during drilling. The boring logs provided in Attachment B 1 were prepared based on a review of the field logs, an examination of the soil samples, and results of the laboratory testing.

3.1.2 Cone Penetrometer Testing

During the 2011-2012 exploration program, four seismic cone penetration test (SCPTu) soundings with pore pressure measurements and shear wave velocity measurements were advanced at BC 1 (BC1-SCPT-1 through BC1-SCPT-4) and three were advanced at BC 2 (BC2-SCPT-1 through BC2-SCPT-3). The locations of the SCPT tests are shown on Figure B-1 and Figure B-2 and the surface elevation and refusal depths are summarized in Table 2. Note that the date in Table 2 also includes cone penetrometer testing completed in 2013 as described in a subsequent section of this report.

The SCPT tip resistance, sleeve friction, and pore water pressure were measured at 2-centimeter (cm) increments as the SCPT instrument was pushed at a constant rate of 2 cm per second (cm/s; ASTM D5778). Shear wave velocity and pore water pressure dissipation measurements were conducted at selected depths in BC1-SCPT-3, BC1-SCPT-4, BC2-SCPT-1, and BC2-SCPT-2. All SCPTs were terminated in decomposed to highly weathered siltstone. BC2-SCPT-2 was advanced approximately 20 feet into the siltstone, whereas the other SCPTs were typically advanced only 5 to 10 feet into the siltstone.

BC1-SCPT-1 and BC1-SCPT-2 were advanced near the downstream toe of BC 1 to a total depth of approximately 50 ft, BC1-SCPT-3 and BC1-SCPT-4 were advanced from the crest of the dam to a total depth of approximately 83 feet. BC1-SCPT-3 was located adjacent to boring BC1-B-1 to provide a basis for correlating the cone data with the soil boring information.

All SCPTs at BC 2 were advanced from the dam crest. BC2-SCPT-1 was located adjacent to boring BC1-B-1 to provide a basis for correlating the cone data with the soil boring information. This exploration extended to a depth of 85 feet. BC2-SCPT-2 was located near the center of the dam, and extended to a depth of 95 feet and BC2-SCPT-3 was located about 80 feet from the northern end of the dam, and extended to a depth of 63 feet.

SCPT data for each sounding, shear wave velocity plots, and pore pressure dissipation plots are included in Attachment B 1.

Table 2: SCPT Summary

SCPT ID	SCPT Elevation	Refusal Depth
BC1-SCPT-1	33.8	50
BC1-SCPT-2	34.3	50
BC1-SCPT-3	47.4	82
BC1-SCPT-4	47.6	82
BC1-SCPT-5	34.6	58.6
BC1-SCPT-6	33.2	71.5
BC2-SCPT-1	91.6	79
BC2-SCPT-2	91.3	57
BC2-SCPT-3	91.0	58
BC2-SCPT-4	49.5	18.3
BC2-SCPT-5	50.2	25.1
BC2-SCPT-6	50.3	30.0
BC2-SCPT-7	50.9	15.4

Table Source: Western States SCPT Drilling Logs
NAVD88 = North American Vertical Datum of 1988

3.1.3 Geophysical Testing

A seismic refraction geophysical survey was conducted at the BC 1 and BC 2 sites on December 20 and 21, 2011 by Northwest Geophysical Associates, Inc. (NGA). The purpose of the survey was to estimate the depth to bedrock and define the bedrock subsurface profile.

The surface seismic refraction survey was performed using a seismograph to record data and sledge hammer to generate seismic compression waves at regular intervals along and at the end of each line. The time required for a seismic wave to travel from a source to a receiver was measured, and the seismic velocity and depth to the underlying soil and rock strata were estimated based on this time period.

The locations of the seismic lines are shown on Figures 2 and 3 of the seismic refraction survey presented in Attachment B 1. A total of three seismic lines were performed; one at BC 1 and two at BC 2. Seismic line 1 (SL-1) was run on the crest of BC 1. SL-2 and SL-3 were run in opposing orientations radiating outward from the downstream toe at BC 2 due to conflicts with the stream, fish ladder, and wetlands.

In general, relatively slow compression or P-wave velocities of 700 to 1,200 feet per second (ft/s) were recorded to a depth of 42 feet at BC 1, suggesting a relatively weak embankment and foundation soil materials below the dam crest. At a depth of about 42 feet, a seismic wave velocity of 3,700 ft/s was measured. The NGA report states that this zone is likely representative of sediments that are saturated to a greater degree than the overlying sediment, perhaps due to a higher organic content. This is the most plausible explanation of this faster velocity zone as boring BC1-B-1 and SCPTu soundings BC1-SCPT-3 and BC1-SCPT-4 encountered siltstone at depths ranging from 82 to 85 feet. In addition, the geophysicist stated that the short seismic line length and the low signal to noise ratio may have limited the ability to detect bedrock at depths of 80 feet and generally affected the overall quality and usefulness of the survey.

Relatively slow P-wave velocities (800 to 1,100 ft/s) were recorded to a depth of 10 feet at BC 2, with faster velocities (4,300 to 5,600 ft/s) recorded below. Again, this is likely representative of sediments that are saturated to a greater degree than the overlying sediment since siltstone bedrock was encountered at a depth of about 30 feet in BC2-B-3 at the downstream toe of the dam. As such, the geophysical survey results were not suitable for estimating the bedrock surface profile at either dam site. Subsequently, the seismic refraction surveys were not used as part of the geotechnical site characterization.

The geophysical data from the 2011-2012 field investigations is included in Attachment B 1.

3.2 2013 Field Investigations

3.2.1 Geotechnical Drilling

Three additional borings were drilled at both the BC 1 dam site (BC1-B-2, BC1-B-3(u), and BC1-B-4(u)) and the BC 2 dam site (BC2-B-4, BC2-B-5(u), and BC2-B-6(u)) from October through November 2013. The drilling work was performed by Western States Drilling. Drilling depths along with the estimates of the depth to the top of bedrock are shown in Table 1.

The boreholes were advanced using a CME 55 track-mounted drill rig using mud rotary drilling techniques. Boring logs for each of the borings are included in Attachment B 2. Phase 3 Geotechnical Data.

Similar to the first drilling program, both disturbed and undisturbed samples were obtained. Disturbed samples were obtained with an SPT split-spoon sampler in accordance with ASTM D1586. Continuous SPT sampling was completed in BC1-B-2 and BC2-B-4. SPT samples were completed at selected intervals to coincide with undisturbed samples obtained in BC1-B-3(u), BC1-B-4(u), BC2-B-5(u), and BC2-B-6(u). The hammer energy for the SPT driving system was measured for the CME 55 track rig to obtain the actual energy transfer ratio for the driving system (GeoDesign 2013). The SPT N-value blow counts (as defined in ASTM D1586) were obtained for each sample and recorded on the boring log. As shown on the boring logs, undisturbed soil samples were obtained with 3-inch-diameter thin-walled Shelby tube samples at selected depths in the borings using a fixed piston sampler that extrudes the sampler into the soil and creates a suction within the tube that enhances the ability to obtain samples in non-cohesive material. Undisturbed sampling was performed in accordance with ASTM D6519. A total of 22 undisturbed samples were collected. Photo 2 and Photo 3 show the sampling tube attached to the fixed piston sampler prior to sampling and the bottom of a retrieved sample after removal of the tube from the sampling apparatus, respectively.

Photo 2: Fixed piston sampler

Photo 3: Undisturbed sample

The depth and sample ID for each undisturbed sample is presented in Table 3.

The boreholes were continuously logged during drilling. The boring logs in Attachment B 2 were prepared based on a review of the field logs, an examination of the soil samples, and results of the laboratory testing.

3.2.2 Cone Penetrometer Testing

Two Seismic Cone Penetration Test (SCPT) soundings with pore pressure measurements were advanced at BC 1 (BC1-SCPT-5 and BC1-SCPT-6) and four were advanced at BC2 (BC2-SCPT-4 through BC2-SCPT-7). The location of the SCPT tests are shown on Figure B-1 and Figure B-2 and summarized in Table 3.

BC1-SCPT-6 and BC1-SCPT-5 were advanced near the downstream toe of BC 1 adjacent to borings BC1-B-2 and BC1-B-3(u), respectively in order to provide a comparison between the SCPT data and SPT data from the adjacent boreholes. In addition, BC2-SCPT-5 and B2-SCPT-6 were advanced near the downstream toe of BC 2 adjacent to borings BC2-B-5(u) and BC2-B-6(u), respectively in order to provide a comparison between SCPT data and SPT data from these adjacent boreholes.

The SCPT tip resistance, sleeve friction, and pore water pressure was measured at 2-inch increments as the SCPT instrument was pushed at a constant rate of 2 cm/s (ASTM D5778). Shear wave velocity and pore water pressure dissipation measurements were conducted at selected depths in BC1-SCPT-5, BC1-SCPT-6, BC2-SCPT-4, BC2-SCPT-5, BC2-SCPT-6, and BC2-SCPT-7. All SCPTs were terminated at refusal. SCPT data is presented in Attachment B 2.

Table 3 presents permeability data from pore pressure dissipation tests.

Table 3: SCPT Pore Pressure Dissipation Test Summary

Dam	SCPT	Depth (m)	Depth (ft)	t50 (seconds)	Permeability k (cm/s)
BC1	SCPT-5	4	13.1	1,531	1.05E-07
		8.15	26.7	481	4.44E-07
		12	39.4	35	1.18E-05
		16	52.5	646	3.07E-07
	SCPT-6	5	16.4	1,610	9.81E-08
		10	32.8	77	4.39E-06
		15	49.2	59	6.12E-06
		20	65.6	24	1.88E-05
BC2	SCPT-4	2	6.6	123	2.44E-06
		4	13.1	71	4.86E-06
		5.75	18.9	15	3.39E-05
	SCPT-5	2	6.6	255	9.82E-07
		4	13.1	29	1.49E-05
		6	19.7	16	3.13E-05
		7.65	25.1	19	2.52E-05
	SCPT-6	3	9.8	126	2.37E-06
		6	19.7	10	5.63E-05
		9	29.5	6	1.07E-04
	SCPT-7	3.05	10.0	239	1.07E-06
		4.7	15.4	9	6.42E-05

The SCPT explorations at the toe of BC 1 present permeability values ranging from 1.88×10^{-5} to 9.81×10^{-8}. The data shows an increase in permeability with depth in SCPT-6 but a similar correlation between permeability and depth cannot be drawn from the data from SCPT-5. The highest permeability of 1.88×10^{-5} is at a depth of 65.6 ft. In BC1-B-4(u) located approximately 15 ft to the north, material at a comparable depth is classified as slightly sandy silt. The lowest permeability at a depth of 16.4 ft is classified on the boring log as sandy silt and having notable wood in that zone.

Each of the four SCPT explorations at BC 2 show lower permeabilities at the upper elevations and slightly higher permeability with depth. The highest permeability value in the BC 2 SCPT explorations is in SCPT-6 at a depth of 29.5 ft. The material encountered in adjacent BC2-B-6(u) at 29.5 ft is sandy silt (MH) to silty sand (SM). The relatively consistent permeabilites ranging from1.49 x 10^{-5} to 3.13 x 10^{-5} in SCPT 5 are in silty sand (SM), silty sand with gravel (SM) to sandy silt (MH).

SCPT data for each sounding are included in Attachment B 2. Dissipation plots for BC-1 SCPT-5 are presented in Attachment B 3. SCPT Pore Pressure Dissipation Plots.

4.0 Laboratory Testing

4.1 2011-2012 Laboratory Testing

NGI conducted laboratory index testing on selected samples from each of the geotechnical borings. Testing consisted of water content, Atterberg limits, gradation analysis, bulk density, and unconfined compressive strength. The results are included in Attachment B 3.

Additional soil testing consisting of unconsolidated undrained triaxial compression, one-dimensional consolidation, and monotonic and cyclic simple shear tests were conducted on selected samples by Fugro Consultants, Inc. in Houston, Texas. The results are included in Appendix D, Engineering Analysis.

Radiocarbon dating of a wood fragment from Boring BC1-B-1 was performed by Beta Analysis, Inc. in Miami, Florida. The laboratory test results from the 2011-2012 investigations are presented in Attachment B 1.

4.2 2013-2014 Laboratory Testing

Cornforth Consultants conducted laboratory index testing on selected samples from each of the geotechnical borings. Testing consisted of water content, Atterberg limits, gradation analysis, bulk density, and unconfined compressive strength. The results are included in Appendix D-Engineering Analysis.

Additional soil testing consisting of triaxial compression, one-dimensional consolidation, constant rate of strain consolidation, direct simple shear, consolidated undrained triaxial compression, and stress controlled cyclic direct simple shear tests were conducted on selected undisturbed samples by MEG Consultants in Vancouver, British Columbia..

Laboratory data interpretation is presented in Appendix D, Engineering Analysis.

5.0 Site Stratigraphy

5.1 BC 1 Site Stratigraphy

Based on the four SPT borings and six SCPTu soundings from the BC 1 site, the following description (model) of site stratigraphy is presented. Clayey silt (MH, defined as elastic silt with high plasticity) embankment fill was encountered to Elevation (EL) 23.5 feet (NAVD88). The embankment fill is underlain by clayey silt, sandy silt, and silty sand alluvium at elevations ranging from EL 25 to about EL -34 feet NAVD88. The alluvial sediments are underlain by decomposed to weathered siltstone bedrock that appears to be continuous across the Big Creek valley. The siltstone bedrock outcrops north and south of the embankment dam abutments. The general subsurface profile along the alignment for BC 1 is shown on Figure B-3.

The following are descriptions of the embankment and foundation materials in accordance with the Unified Soil Classification System (USCS; ASTM D2487) encountered in boring BC1-B-1 drilled from the crest of the dam:

Clayey SILT with some Sand (Dam Fill): The dam fill material was only sampled and tested during the 2011-2012 field investigation with BC1-B-1. Dam fill generally consists of low to medium plasticity clayey silt with some fine sand. The plans for the original dam construction in 1951 indicates 21 feet of clayey silt fill was placed to construct the embankment. This is consistent with the subsurface conditions encountered in boring BC1-B-1 where fill appeared to extend from EL 47.4 (dam crest) to EL 23.9 feet NAVD88 (23.5 feet below the crest of the dam). SPT N-values ranging from 0 to 4 indicate the relative consistency of the fill is very soft to soft. Laboratory testing of selected samples indicate a plasticity index (PI) ranging from 20 to 28, water contents near the liquid limit, and a fines percentage near 50 percent.

High Plasticity SILT (MH) with some Clay and Sand (Alluvium): Alluvial material consisting of high plasticity silt with varying percentages of fine sand, clay, and organics was encountered in all four borings at the BC 1 site. SPT N-values ranged from 0 to 3, indicating the relative consistency of the alluvium is very soft to soft. Laboratory testing on selected samples indicates a PI ranging from 11 to 68. In BC1-B-2 (EL -24 to -30) N-values in MH within 10 feet of the Nye Formation ranged from 3 to 12, indicating the relative consistency of soft to firm. It is possible these zones are actually in residual Nye Formation siltstone and not in alluvium as logged in the field.

Low Plasticity SILT (ML) with some Sand and Clay (Alluvium): Low plasticity silt (ML) was encountered in BC1-B-1 from a depth of 27.5 to 29 feet below the embankment crest and in BC1-B-2 at a depth of 37.5 to 39 feet beneath the toe of the embankment. Atterberg limit testing indicates the silt has a PI ranging from 10 to 14. The N-values recorded in this layer ranged from 0 to 1, indicating the soil is very soft.

Silty SAND (Alluvium): Alluvial material consisting of silty sand with isolated lenses of sandy silt, organic silt, and scattered to numerous organics was encountered at various elevations (depths) within the foundation. N-values ranged from 0 to 8, indicating the relative density is very loose to loose. Scattered organics and wood debris were encountered throughout this layer.

Siltstone (Marine Sedimentary Rock): The borings terminated in decomposed to weathered siltstone of the Nye Formation. In the decomposed condition, the siltstone consists of stiff to hard, clayey silt. The general elevation of the siltstone layer is shown on Figure B-3. The siltstone outcrop north and south of the embankment dam were identified in the field. Photo 3 presents the Nye Formation at the right abutment of the dam.

Based on our evaluation of the available SCPT and geotechnical drilling (SPT and laboratory) data there are no layers in any of the foundation alluvial materials described above that are laterally continuous across the BC 1 site. There is one possible layer of silty sand (SM) observed in borings BC-1-B1 and BC-B1-3(u) that could constitute a layer that is continuous in an upstream and downstream direction (general stream flow direction). However, the SCPT data from BC1-SCPT-5 and BC1-SCPT-3 do not show readings consistent with a silty sand. The depositional environment of an alluvial system that experiences variable flow conditions (flow velocities), variable sediment load (due to slope failures within the drainage basin, ash fall events, increased sediment load during heavy rain events), a migrating channel, and tidal influences during intermittent periods of the depositional history explain the absence of lateral continuity of any layer. This absence of lateral continuity means that there is no distinct layer or zone that has unique geotechnical properties that could influence the stability of the dam.

5.2 BC 2 Site Stratigraphy

The BC 2 embankment fill elevations range approximately from EL 46 to EL 25 feet NAVD88. In general the BC 2 foundation consists of coarser grained material in comparison to the BC 1 foundation. High plasticity silt with some zones of silty Sand with fine to coarse sand and fine gravel is the predominant material in the BC 2 foundation. The foundation material consists of both alluvium and colluvium that are encountered from EL 42 to 20 ft. Embankment material lies between EL 42 to 92 ft. The higher elevation of the valley has prevented or minimized any tidal influence during the depositional history and therefore provided a consistently higher energy depositional environment. Based on the SCPT and geotechnical boring data there are no laterally continuous layers that have unique properties within the BC 2 foundation. Variability in flow energy, sediment load, and channel migration have prevented deposition of a continuous layer of well sorted detritus having properties different from the rest of the foundation. Figure B-4 presents the general profile of the BC 2 and foundation based on SPT borings and SCPT data.

The following are descriptions of the embankment and foundation materials in accordance with the USCS (ASTM D2487) encountered at the BC 2 site:

Fill: Embankment fill material consisting of silty fine sand, sandy silt, and clayey silt, was encountered in BC2-B-1, BC2-B-2, BC2-B-3 drilled from the dam crest and in BC2-B-4. SPT N-values ranged from 0 to 20 and indicate the relative consistency (or density) ranges from very soft to very stiff (very loose to medium dense).

Sandy, Clayey, SILT (Alluvium): The sandy silt (MH) is predominantly very loose to loose with SPT N-values ranging from 2 to 7. The soil is generally highly plastic with PIs ranging from 16 to 38. The percentage of sand sized particles is significant (27 to 49 percent).

Silty SAND with fine gravel (Alluvium): Silty sand (SM) was encountered at various depths in the BC 2 foundation and was very loose to medium dense with SPT N-values ranging from 0 to 11. The fines content of the samples ranged from 21 to 40 percent.

Siltstone (Marine Sedimentary Rock): Decomposed to intensely weathered Nye Formation Siltstone (Clayey silt) was encountered in each boring with the exception of BC2-B-3. The decomposed siltstone had SPT N-values ranging from 8 to >50/6 inch.

6.0 References

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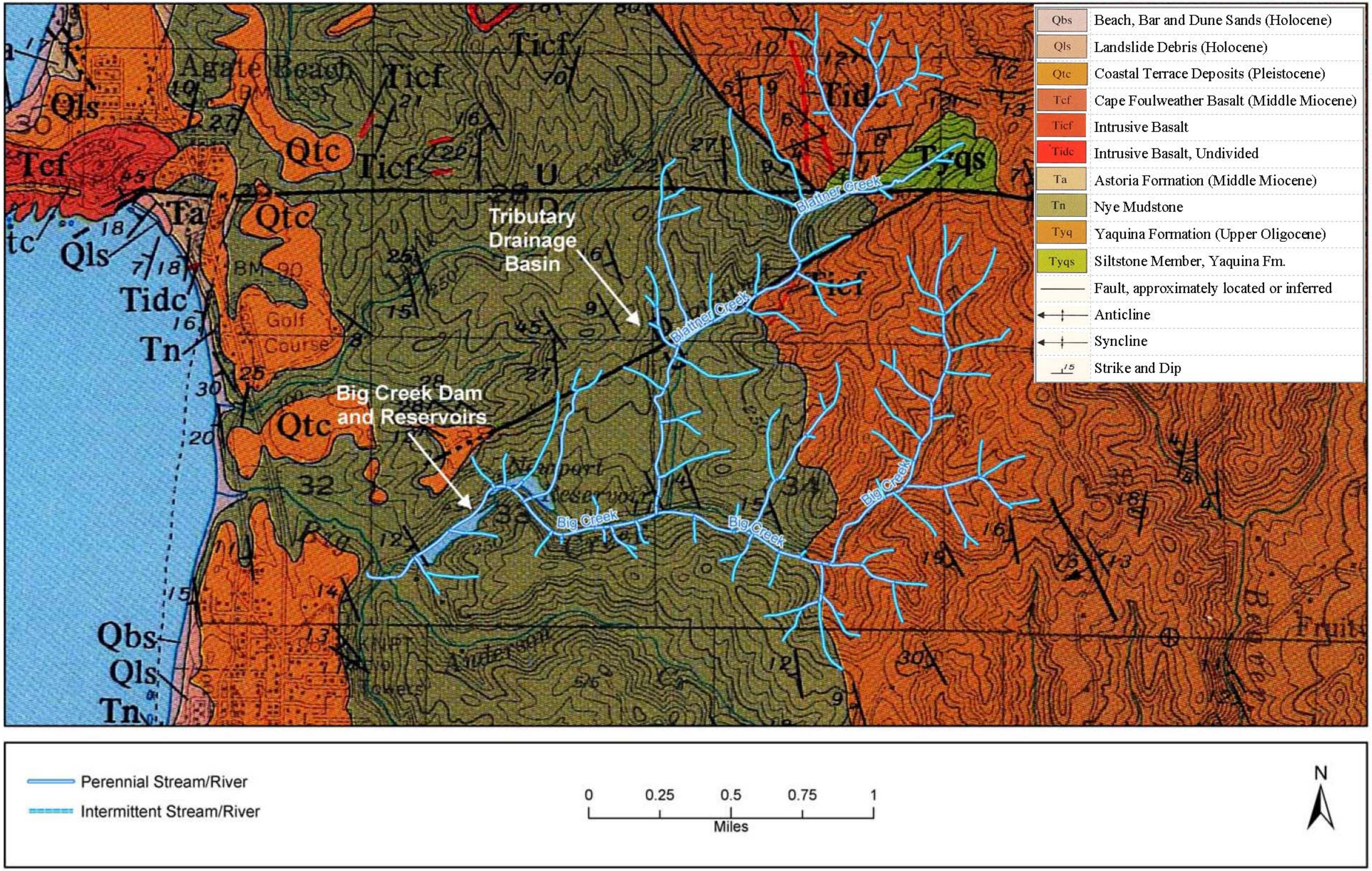
Geologic map of the Yaquina and Toledo Quadrangles, Lincoln County, Oregon: U.S. Geologic Survey, Miscellaneous Investigations Series Map I-867, Scale 1:62500.

Plates and Figures



Source:
Snavely, P. D., Jr., MacLeod, N. S., Wagner, H. C.; Rau, W. W. (1976). *Geologic Map of the Yaquina and Toledo Quadrangles, Lincoln County, Oregon.*U.S. Geological Survey (USGS). USGS IMAP 867

NEWPORT OREGON

HDR

CORNFORTH CONSULTANTS

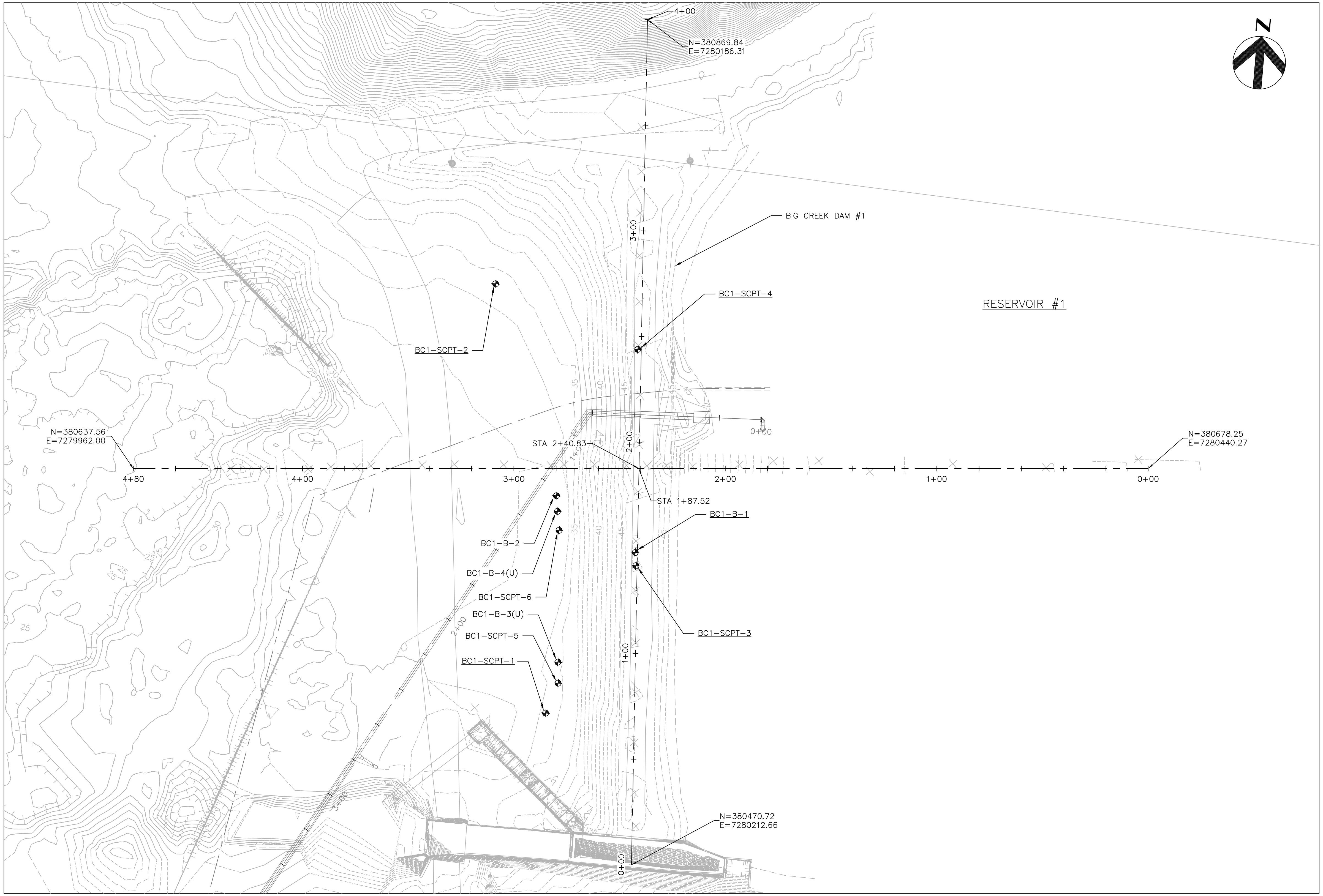
PROJECT NO.	101492
DRAWN:	01/21/2015
DRAWN BY:	NL
CHECKED BY:	MW
FILE NAME:	Big Creek Geologic Map

Big Creek Geologic Map

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

ORIGINATOR:	MW	DRAWING CATEGORY: Map
APPROVED BY:	JC	

Plate
B-1

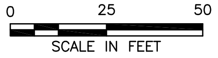


LEGEND

⊕ BC1-B-X	STANDARD PENETRATION TEST (SPT) 2011
⊕ BC1-CPT-X	CONE PENETRATION TEST (CPT) 2011
⊕ BC1-B-X	STANDARD PENETRATION TEST (SPT) 2013
⊕ BC1-B-X(U)	STANDARD PENETRATION TEST (SPT) 2013 UNDISTURBED SAMPLE.
⊕ BC1-SCPT-X	CONE PENETRATION TEST (CPT) 2013

PLAN

SCALE: 1" = 25'



HDR

CITY OF NEWPORT OREGON
SEISMIC EVALUATION OF BIG CREEK
DAMS #1 & 2

DAM BC1 – SITE EXPLORATION LOCATION PLAN

DATE
June 2015

FIGURE
B-1

LEGEND

⊕	BC2-B-X	STANDARD PENETRATION TEST (SPT) 2011
⊕	BC2-CPT-X	CONE PENETRATION TEST (CPT) 2011
⊕	BC2-B-X	STANDARD PENETRATION TEST (SPT) 2013
⊕	BC2-B-X(U)	STANDARD PENETRATION TEST (SPT) 2013 UNDISTURBED SAMPLE.
⊕	BC2-SCPT-X	CONE PENETRATION TEST (CPT) 2013

PLAN
SCALE: 1" = 30'

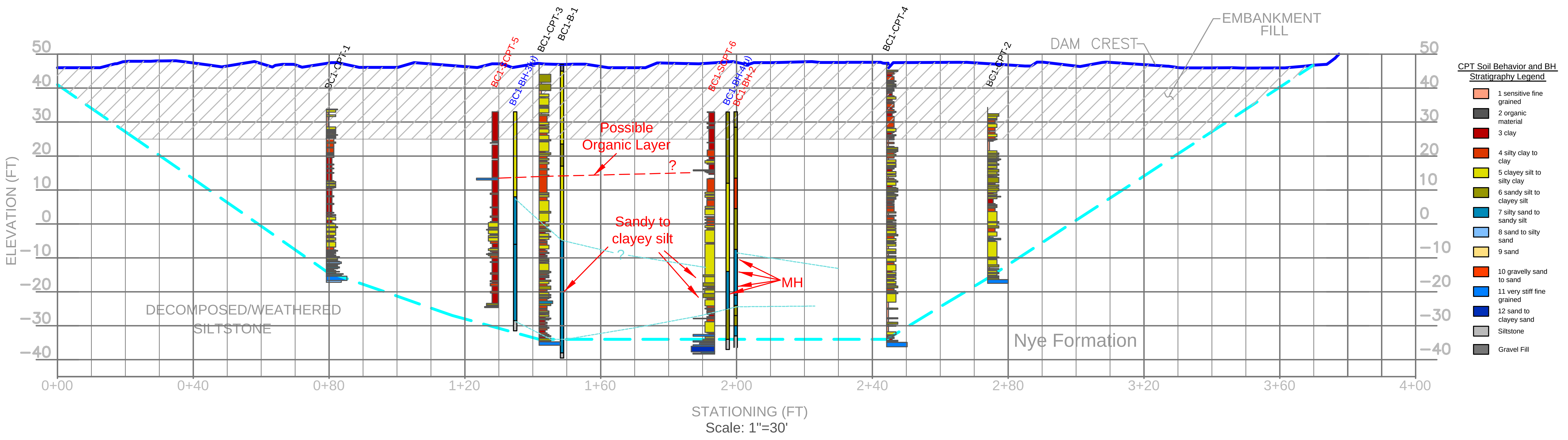
0 30 60
SCALE IN FEET

CITY OF NEWPORT OREGON
SEISMIC EVALUATION OF BIG CREEK
DAM #1 & 2

DAM BC2 – SITE EXPLORATION LOCATION PLAN

DATE
JUNE 2015

FIGURE
B-2



HDR

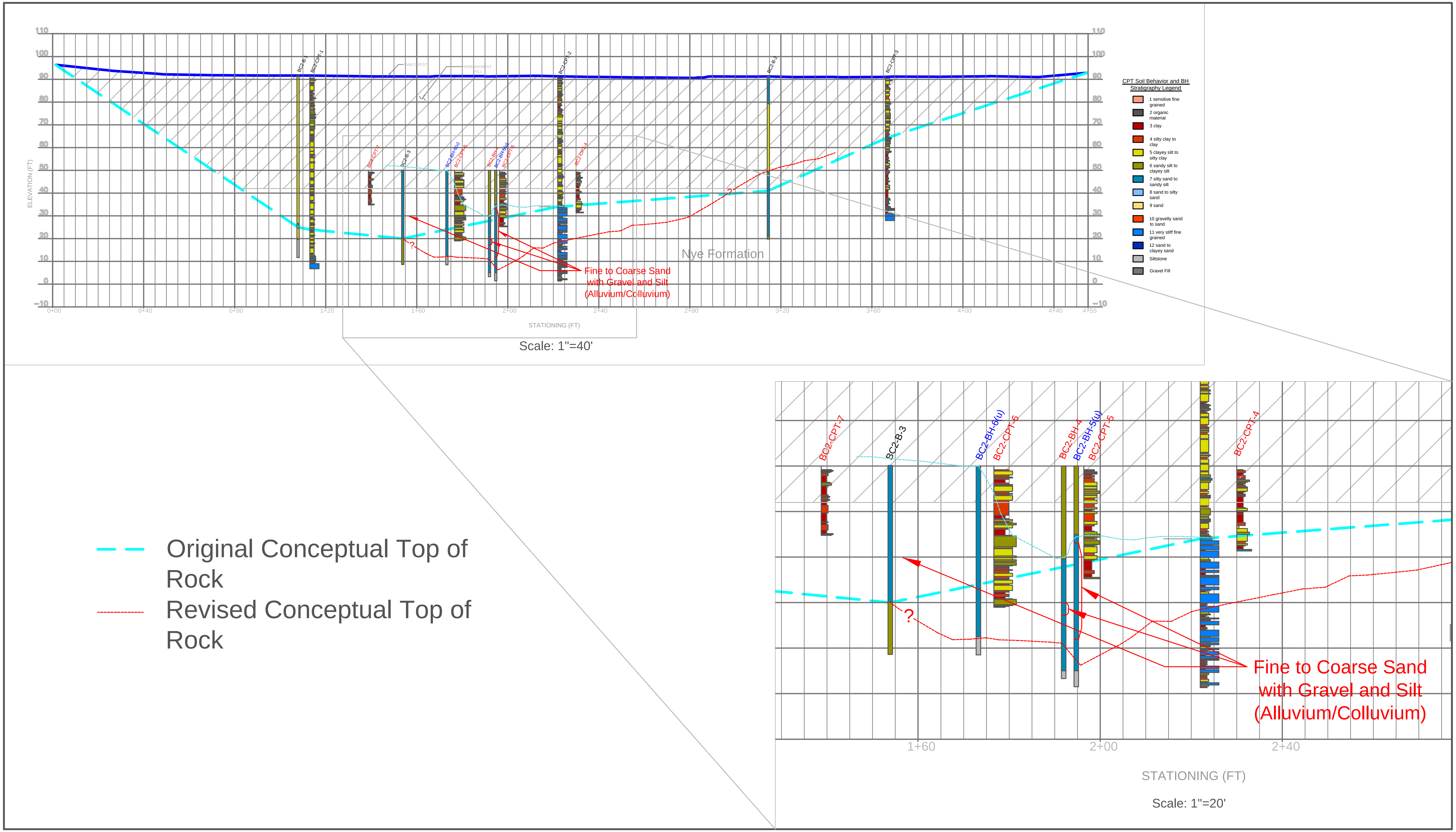
Big Creek 1 Subsurface Profile

DATE

4/8/2015

FIGURE

B - 3



HDR

Big Creek 2 Subsurface Profile

DATE
4/8/2015

FIGURE
B - 4

Attachment B 1. Phase 2 Geotechnical Data

UNIFIED SOIL CLASSIFICATION SYSTEM (ASTM D-2487)

MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES			GROUP SYMBOL	SOIL GROUP NAMES & LEGEND
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	$C_u \geq 4$ AND $1 \leq C_c \leq 3$	GW	WELL-GRADED GRAVEL
			$C_u < 4$ AND/OR $1 > C_c > 3$	GP	POORLY-GRADED GRAVEL
		GRAVELS WITH FINES >12% FINES	FINES CLASSIFY AS ML OR MH	GM	SILTY GRAVEL
			FINES CLASSIFY AS CL OR CH	GC	CLAYEY GRAVEL
	SANDS >50% OF COARSE FRACTION PASSES NO 4. SIEVE	CLEAN SANDS <5% FINES	$C_u \geq 6$ AND $1 \leq C_c \leq 3$	SW	WELL-GRADED SAND
			$C_u < 6$ AND/OR $1 > C_c > 3$	SP	POORLY-GRADED SAND
		SANDS AND FINES >12% FINES	FINES CLASSIFY AS ML OR MH	SM	SILTY SAND
			FINES CLASSIFY AS CL OR CH	SC	CLAYEY SAND
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT<50	INORGANIC	PI>7 AND PLOTS>"A" LINE	CL	LEAN CLAY
			PI>4 AND PLOTS<"A" LINE	ML	SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OL	ORGANIC CLAY OR SILT
	SILTS AND CLAYS LIQUID LIMIT>50	INORGANIC	PI PLOTS >"A" LINE	CH	FAT CLAY
			PI PLOTS <"A" LINE	MH	ELASTIC SILT
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OH	ORGANIC CLAY OR SILT
HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR		PT	PEAT

OTHER SYMBOLS

MATERIALS

- Asphalt
- Aggregate Base
- Boulders & Cobbles
- Topsoil

SAMPLERS

- SPT (2" OD)
- Shelby Tube
- HQ Core

WELL

- Concrete Grout/Fill
- Bentonite/Grout Seal
- Sand Pack + Solid Pipe
- Sand Pack + Slotted Pipe

▽ INITIAL WATER LEVEL MEASUREMENT(WITH DATE)

▼ STABILIZED WATER LEVEL MEASUREMENT(WITH DATE)

GRAIN SIZES

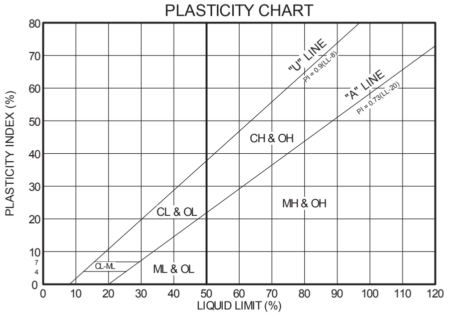
U.S. STANDARD SIEVE: 200, 40, 10, 4, 3/4", 3", 12"

SILTS AND CLAYS	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		

PENETRATION RESISTANCE

SAND & GRAVEL		SILT & CLAY		
RELATIVE DENSITY	BLOWS/FOOT*	CONSISTENCY	BLOWS/FOOT*	UNC. COMP. STRENGTH (KSF)
VERY LOOSE	0 - 4	VERY SOFT	0 - 1	0 - 1/2
LOOSE	5 - 10	SOFT	2 - 4	1/2 - 1
MEDIUM DENSE	11 - 30	FIRM	5 - 8	1 - 2
DENSE	31 - 50	STIFF	9 - 15	2 - 4
VERY DENSE	OVER 50	VERY STIFF	16 - 30	4 - 8
		HARD	OVER 30	OVER 8

* NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).


PLASTICITY CHART

NOTES

AT	ATTERBERG LIMITS
BGS	BELOW GROUND SURFACE
c	COHESION
CD	CONSOLIDATED DRAINED TRIAXIAL
CN	CONSOLIDATION
CR	CORROSIVITY
CU	CONSOLIDATED UNDRAINED TRIAXIAL
DS	DIRECT SHEAR
EI	EXPANSION INDEX
HY	HYDROMETER
LL	LIQUID LIMIT
N_{60}	BLOW COUNT, Corrected for Hammer Energy Only
PI	PLASTICITY INDEX
PR	PERMEABILITY
RV	R-VALUE
SA	SIEVE ANALYSIS
-200	% PASSING NO. 200 SIEVE
TC	CYCLIC TRIAXIAL
UC	UNCONFINED COMPRESSION
UU	UNCONSOLIDATED UNDRAINED TRIAXIAL

INCREASING VISUAL MOISTURE CONTENT

↑
SATURATED
WET
MOIST
DAMP
DRY

COMPONENT PERCENTAGE

PRIMARY	>50%
SECONDARY (-y)	30 - 50%
WITH	15 - 30%
TRACE	0 - 15%

HDR
ONE COMPANY | *Many Solutions*

Boring Legend

City of Newport Big Creek Dams
Preliminary Dam Seismic Evaluation

Date

Figure
B.1-1

LEGEND BIG CREEK DAM SEISMIC.GPJ FOLSOM 3-30-11.GDT 8/24/12

Boring Designation: BC1-B-1

DRILLING LOG

DIVISION

LOCATION City of Newport, OR

SHEET 1 OF 4 SHEETS

1. PROJECT Newport WTP Dam Assessment (Big Creek #1)
2. HOLE NUMBER B-1
LOCATION COORDINATES N380,618.1 E7,280,201.9
3. DRILLING AGENCY Western States
4. NAME OF DRILLER Adonis Pablo
5. DIRECTION OF BORING ☒ VERTICAL ☐ INCLINED
DEG FROM VERTICAL ---
BEARING
6. THICKNESS OF OVERBURDEN N/A
7. DEPTH DRILLED INTO ROCK N/A
8. TOTAL DEPTH OF BORING 86.5ft
9. COORDINATE SYSTEM State Plan Grid 1983
HORIZONTAL
VERTICAL
10. SIZE AND TYPE OF BIT 8" Dia. HSA, 4 7/8" Dia. Mud Rotary
11. MANUFACTURER'S DESIGNATION OF DRILL (Type, Efficiency) CME 75 (Automatic, 78%)
EFFICIENCY MEASURED? ☒
12. TOTAL SAMPLES 29
DISTURBED 25
UNDISTURBED 4
13. TOTAL NUMBER CORE BOXES 0
14. ELEVATION GROUND WATER
INITIAL
STATIC
15. DATE STARTED 12/15/11
DATE COMPLETED 12/15/11
16. ELEVATION TOP OF BORING 47.4 ft
17. TOTAL CORE RECOVERY FOR BORING N/A
18. LOGGED BY Nick Clark, P.E. (HDR)
CHECKED BY

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	$N_{Corr.}$	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Laboratory								REMARKS
										Gravel	Sand	Fines	LL	PI	DD	MC	ASTM Class	
							BASE ROCK crushed, angular gravel (Fill).											
45.4	2.0						Clayey SILT some sand to silty SAND some clay (MH-SM); wet, brown mottled orange, medium plasticity, fine sand, scattered wood fragments, very soft to soft (Fill).											
		▮	10 2 2	4	4				1						64	60		
		▮	0 0 2	2	4				2							44		Soil color is grey below 10.0'
		▮	0 0 0	0	0				3				61	20		65	MH	
		▮	0 0 0	0	0				4									
			350 psi						5	0	52	48	66	28	66	56	SM	
23.9	23.5	▮	0 0 4	4	5		Sandy SILT some clay (ML-MH); wet, gray, low to medium plasticity, fine, very soft (Alluvium).		6									

Boring Designation: BC1-B-1

DRILLING LOG (Cont Sheet)

INSTALLATION	
SHEET 2 OF 4 SHEETS	
PROJECT	Newport WTP Dam Assessment (Big Creek #1)
COORDINATE SYSTEM	State Plan Grid 1983
HORIZONTAL	
VERTICAL	
LOCATION COORDINATES	N 380,618.1 E 7,280,201.9
ELEVATION TOP OF BORING	47.4 ft

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	$N_{Corr.}$	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Laboratory Gravel	Sand	Fines	LL	PI	DD	MC	ASTM Class	REMARKS
							Sandy SILT some clay (ML-MH); wet, gray, low to medium plasticity, fine, very soft (Alluvium). *(continued)*											
			0 0 0	0	0				7	0	38	62	49	14		60	ML	
																		Larger wood fragments from 29.0'
17.4	30.0						Clayey SILT some sand (MH); wet, gray, medium to high plasticity, fine, scattered wood fragments, very soft to soft (Alluvium).											
			300 psi						8				76	29			MH	
			0 0 0	0	0				9				87	41		75	MH	
			0 0 0	0	0				10									
			0 0 0	0	0				11									
			300 psi						12				82	35			MH	
			0 0 0	0	0				13									
			0 0 0	0	0				14	0	47	53	54	14		58	MH	
			1 0 2	2	2				15									
			300 psi						16				68	32			MH	
-4.6	52.0						Silty SAND to sandy SILT (SM); wet, gray, low plasticity, fine sand, scattered wood fragments, very loose to soft (Alluvium).											
			0 0 2	2	2				17	0	47	53	51	6		55	MH	

25.0 27.5 30.0 32.5 35.0 37.5 40.0 42.5 45.0 47.5 50.0 52.5 55.0

Boring Designation: BC1-B-1

DRILLING LOG (Cont Sheet)	INSTALLATION	SHEET 3 OF 4 SHEETS

PROJECT Newport WTP Dam Assessment (Big Creek #1)	COORDINATE SYSTEM State Plan Grid 1983	HORIZONTAL	VERTICAL
LOCATION COORDINATES N 380,618.1 E 7,280,201.9	ELEVATION TOP OF BORING 47.4 ft		

	ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	$N_{Corr.}$	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Laboratory Gravel	Sand	Fines	LL	PI	DD	MC	ASTM Class	REMARKS
				0 0 3	3	3		Silty SAND to sandy SILT (SM); wet, gray, low plasticity, fine sand, scattered wood fragments, very loose to soft (Alluvium). *(continued)*		18	0	60	40					SM	
57.5				1 0 0	0	0				19				42	8		70	ML	
60.0				300 psi						20	0	63	38	51	22	67	52	SM	Medium plasticity silt lense from 60.0'-62.0'
62.5				0 0 3	3	3				21					NP			SM	
65.0				0 0 0	0	0				22	0	74	26	45	3		60	SM	
67.5				0 0 0	0	0				23	0	67	33	45	7		62	SM	
70.0				0 0 2	2	2				24	0	78	22	41	3		82	SM	High moisture contents due to wood fragments
72.5				2 3 5	8	7				25	0	72	28	57	28		78	SM	Erroneous SPT due to wood fragment in tip Medium plasticity silt lense from 72.0'-73.5'
75.0				1 0 2	2	2				26	0	60	40				59	SM	Scattered coarse sand and rounded fine gravel below 70.0'
77.5				2 2 0	2	2				27	0	47	53	53	24		40	OH	Medium plasticity silt lense from 77.5'-79.0'
80.0				0 0 16	16	13				28	0	70	30	46	7		51	SM	Erroneous SPT due to rock in tip
82.5																			
85.0	-37.6	85.0		35			× ×												

Boring Designation: BC1-B-1

DRILLING LOG (Cont Sheet)	INSTALLATION	SHEET 4 OF 4 SHEETS
PROJECT Newport WTP Dam Assessment (Big Creek #1)	COORDINATE SYSTEM State Plan Grid 1983	HORIZONTAL / VERTICAL
LOCATION COORDINATES N 380,618.1 E 7,280,201.9	ELEVATION TOP OF BORING 47.4 ft	

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	$N_{Corr.}$	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Laboratory Gravel	Sand	Fines	LL	PI	DD	MC	ASTM Class	REMARKS
-39.1	86.5	█	35 41	76	59	× ×	SILT (Decomposed Siltstone) (MH); moist, blue/gray, low plasticity, hard (Decomposed Siltstone). *(continued)*		29									

Boring Designation: BC2-B-1

DRILLING LOG

DIVISION

LOCATION: City of Newport, OR

SHEET 1 OF 3 SHEETS

1. PROJECT: Newport WTP Dam Assessment (Big Creek #2)
2. HOLE NUMBER (Section Location): B-1 (See location map)
LOCATION COORDINATES: N381,157.7 E7,283,506.5
3. DRILLING AGENCY: Western States
4. NAME OF DRILLER: Adonis Pablo
5. DIRECTION OF BORING: ☒ VERTICAL ☐ INCLINED
DEG FROM VERTICAL: ---
BEARING
6. THICKNESS OF OVERBURDEN: 70.0_ft
7. DEPTH DRILLED INTO ROCK: 10.0ft
8. TOTAL DEPTH OF BORING: 80.0ft
9. COORDINATE SYSTEM: State Plan Grid 1983
HORIZONTAL
VERTICAL
10. SIZE AND TYPE OF BIT: 8" Diameter HSA
11. MANUFACTURER'S DESIGNATION OF DRILL (Type, Efficiency): CME-75 (Automatic, 78%)
EFFICIENCY MEASURED? ☒
12. TOTAL SAMPLES: 17
DISTURBED: 16
UNDISTURBED: 1
13. TOTAL NUMBER CORE BOXES: 1
14. ELEVATION GROUND WATER
INITIAL: 61.6
STATIC: 36.6
15. DATE STARTED: 12/12/11
DATE COMPLETED: 12/12/11
16. ELEVATION TOP OF BORING: 91.6 ft
17. TOTAL CORE RECOVERY FOR BORING: N/A
18. LOGGED BY: Nick Clark, P.E. (HDR)
CHECKED BY

	ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	$N_{Corr.}$	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Laboratory Gravel	Sand	Fines	LL	PI	DD	MC	ASTM Class	REMARKS
0.0								<u>Clayey SILT</u> (MH); moist to wet, brown mottled orange, medium plasticity, very soft to soft (Fill).											
2.5																			
5.0			▮	0 2 2	4	4			89	1									
7.5																			
10.0			▮	0 0 1	1	2			100	2				56	18		44	MH	
12.5																			
15.0	76.6	15.0	▮	2 3 5	8	11		<u>Clayey SILT some sand</u> (MH); wet, brown mottled orange, low to medium plasticity, fine sand, stiff to very stiff (Fill).	89	3									
17.5																			Harder at 17.0'
20.0			▮	2 6 7	13	15			94	4	0	48	52	50	11		36	MH	
22.5																			
25.0																			

Boring Designation: BC2-B-1

DRILLING LOG (Cont Sheet)	INSTALLATION	SHEET 2 OF 3 SHEETS
PROJECT Newport WTP Dam Assessment (Big Creek #2)	COORDINATE SYSTEM State Plan Grid 1983	HORIZONTAL / VERTICAL
LOCATION COORDINATES N 381,157.7 E 7,283,506.5	ELEVATION TOP OF BORING 91.6 ft	

	ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	$N_{Corr.}$	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Gravel	Sand	Fines	LL	PI	DD	MC	ASTM Class	REMARKS
25.0			▌	3 5 7	12	13		<u>Clayey SILT some sand</u> (MH); wet, brown mottled orange, low to medium plasticity, fine sand, stiff to very stiff (Fill). *(continued)*	88	5									
27.5																			
30.0		▽	▌	1 5 7	12	12			94	6				53	10		41	MH	Water level rose to 30.0' depth after encountering at 55.0' depth
32.5																			
35.0			▌	3 6 8	14	13			89	7									Soil color exhibits grey mottles below 35.0'
37.5																			
40.0			▌	2 6 7	13	11			100	8									
42.5																			
45.0			▌	0 4 8	12	9			100	9	0	27	73	56	14		44	MH	
47.5																			
50.0			▌	2 5 8	13	10			100	10									
52.5																			
55.0	36.6 ▼	55.0	▌																

Boring Designation: BC2-B-1

DRILLING LOG (Cont Sheet)	INSTALLATION	SHEET 3 OF 3 SHEETS
PROJECT Newport WTP Dam Assessment (Big Creek #2)	COORDINATE SYSTEM State Plan Grid 1983	HORIZONTAL | VERTICAL
LOCATION COORDINATES N 381,157.7 E 7,283,506.5	ELEVATION TOP OF BORING 91.6 ft	

ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	$N_{Corr.}$	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Laboratory Gravel	Sand	Fines	LL	PI	DD	MC	ASTM Class	REMARKS
			0 0 0	0	0		Clayey SILT some sand (MH); wet, gray, medium plasticity, fine sand, very soft to medium stiff (Fill). *(continued)*	100	11									
			0 0 0	0	0			100	12									
			300 psi					67	13	0	47	53	53	16	73	46	MH	
			0 0 6	6	4			100	14									
26.6	65.0																	
			0 0 2	2	1		Silty SAND (SM); wet, gray, fine sand, very loose (Fill).	100	15									Harder at 65.5'
24.6	67.0																	
			10 21 27	48	32		Clayey SILT (MH); damp, gray mottled brown, low plasticity, hard (Decomposed Siltstone).	83	16									
			20 25 22	47	30			94	17				50	12			MH	
19.6	72.0																	
							SILTSTONE brown/gray, highly weathered, moderately close joints, very weak, joints are irregular, rough open/closed and healed.	100	R-1									Harder at 72.0'
								93	R-2									
11.6	80.0																	

Boring Designation: BC2-B-2

DRILLING LOG	DIVISION	LOCATION City of Newport, OR	SHEET 1 OF 3 SHEETS

1. PROJECT Newport WTP Dam Assessment (Big Creek #2)	9. COORDINATE SYSTEM: State Plan Grid 1983 ⋮ HORIZONTAL ⋮ VERTICAL 10. SIZE AND TYPE OF BIT: 8" Diameter HSA
2. HOLE NUMBER (Section Location): B-2 (See location map) ⋮ LOCATION COORDINATES: N381,339.3 E7,283,407.9	11. MANUFACTURER'S DESIGNATION OF DRILL (Type, Efficiency) CME 75 (Automatic, 78%) EFFICIENCY MEASURED? ☒
3. DRILLING AGENCY Western States	12. TOTAL SAMPLES: 17 ⋮ DISTURBED: 16 ⋮ UNDISTURBED: 1
4. NAME OF DRILLER Adonis Pablo	13. TOTAL NUMBER CORE BOXES: 0
5. DIRECTION OF BORING ☒ VERTICAL ☐ INCLINED DEG FROM VERTICAL: --- BEARING	14. ELEVATION GROUND WATER INITIAL 71.2 STATIC 53.7 15. DATE STARTED 12/14/11 ⋮ DATE COMPLETED 12/14/11
6. THICKNESS OF OVERBURDEN: N/A	16. ELEVATION TOP OF BORING: 91.2 ft
7. DEPTH DRILLED INTO ROCK: N/A	17. TOTAL CORE RECOVERY FOR BORING: N/A
8. TOTAL DEPTH OF BORING: 71.5ft	18. LOGGED BY: Nick Clark, P.E. (HDR) ⋮ CHECKED BY

	ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	$N_{Corr.}$	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Laboratory Gravel	Sand	Fines	LL	PI	DD	MC	ASTM Class	REMARKS
0.0								<u>Silty SAND some clay</u> (SM); moist to wet, brown mottled orange, low to medium plasticity, fine sand, soft to medium stiff (Fill).											
2.5																			
5.0			■	1 2 2	4	4				1									
7.5																			
10.0			■	1 2 3	5	8				2	0	55	45	52	7		41	SM	
	79.2	12.0						<u>Clayey SILT some sand</u> (MH); wet, brown mottled orange and gray, low to medium plasticity, fine sand, stiff to very stiff (Fill).											Harder at 12.0'
12.5																			
15.0			■	2 5 9	14	19				3									
17.5																			
20.0	▽		■	4 6 9	15	18				4									Water level rose to 20.0' depth after encountering 37.5'
22.5																			
25.0																			

SPK FORM 1836-A
SEP 05

Boring Designation: BC2-B-2

DRILLING LOG (Cont Sheet)	INSTALLATION	SHEET 2 OF 3 SHEETS

PROJECT Newport WTP Dam Assessment (Big Creek #2)	COORDINATE SYSTEM State Plan Grid 1983	HORIZONTAL	VERTICAL
LOCATION COORDINATES N 381,339.3 E 7,283,407.9	ELEVATION TOP OF BORING 91.2 ft		

Depth scale	ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	$N_{Corr.}$	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Laboratory Gravel	Sand	Fines	LL	PI	DD	MC	ASTM Class	REMARKS
25.0				3 7 10	17	18		Clayey SILT some sand (MH); wet, brown mottled orange and gray, low to medium plasticity, fine sand, stiff to very stiff (Fill). *(continued)*		5				55	17		41	MH	
27.5																			
30.0				4 8 10	18	17				6									
32.5																			
35.0				3 4 5	9	8				7									
37.5		▼																	Soft at 37.5'
40.0				0 0 1	1	1				8	0	64	37	52	21		58	SM	Harder at 41.5'
	49.2	42.0						Sandy SILT to Silty SAND some clay (MH-SM); wet, brown mottled gray to gray, medium plasticity, fine sand, trace organics, stiff to very stiff and medium dense (Decomposed Siltstone).											
42.5				1000 psi						9									
45.0				7 9 8	17	14				10									
47.5				0 4 11	15	12				11	0	50	51	50	24		42	MH	Soil color is gray below 47.5'
50.0				4 8 11	19	15				12									
52.5																			
55.0																			

Boring Designation: BC2-B-2

DRILLING LOG (Cont Sheet)

INSTALLATION

SHEET 3 OF 3 SHEETS

PROJECT: Newport WTP Dam Assessment (Big Creek #2)

COORDINATE SYSTEM: State Plan Grid 1983

HORIZONTAL:

VERTICAL:

LOCATION COORDINATES: N 381,339.3 E 7,283,407.9

ELEVATION TOP OF BORING: 91.2 ft

	ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	$N_{Corr.}$	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Gravel	Sand	Fines	LL	PI	DD	MC	ASTM Class	REMARKS
			▮	3 5 10	15	12		Sandy SILT to Silty SAND some clay (MH-SM); wet, brown mottled gray to gray, medium plasticity, fine sand, trace organics, stiff to very stiff and medium dense (Decomposed Siltstone). *(continued)*		13	0	41	59	54	29		33	MH	
57.5			▮	12 11 10	21	17				14									Harder at 58.0'
60.0			▮	2 8 10	18	14				15									Scattered softer lenses up to 6"
62.5																			
65.0			▮	3 5 6	11	8				16	0	79	22	43	10		32	SM	
67.5																			
70.0	21.2 19.7	70.0 71.5	▮	8 15 26	41	30	× ×	SILT trace sand (ML); moist, gray, low plasticity, fine sand, hard (Decomposed Siltstone).		17									

Boring Designation: BC2-B-3

DRILLING LOG

DIVISION

LOCATION City of Newport, OR

SHEET 1 OF 2 SHEETS

1. PROJECT Newport WTP Dam Assessment (Big Creek #2)

9. COORDINATE SYSTEM State Plan Grid 1983 HORIZONTAL VERTICAL

10. SIZE AND TYPE OF BIT 4 7/8" Diameter Mud Rotary

2. HOLE NUMBER (Section Location) B-3 (See location map) LOCATION COORDINATES N381,151.6 E7,283,395.0

11. MANUFACTURER'S DESIGNATION OF DRILL (Type, Efficiency) CME 75 (Automatic, 78%) EFFICIENCY MEASURED? ☒

3. DRILLING AGENCY Western States

12. TOTAL SAMPLES 15 DISTURBED 13 UNDISTURBED 2

4. NAME OF DRILLER Adonis Pablo

13. TOTAL NUMBER CORE BOXES 0

5. DIRECTION OF BORING ☒ VERTICAL ☐ INCLINED DEG FROM VERTICAL --- BEARING

14. ELEVATION GROUND WATER INITIAL STATIC

15. DATE STARTED 1/5/12 DATE COMPLETED 1/5/12

6. THICKNESS OF OVERBURDEN N/A

16. ELEVATION TOP OF BORING 50.1 ft

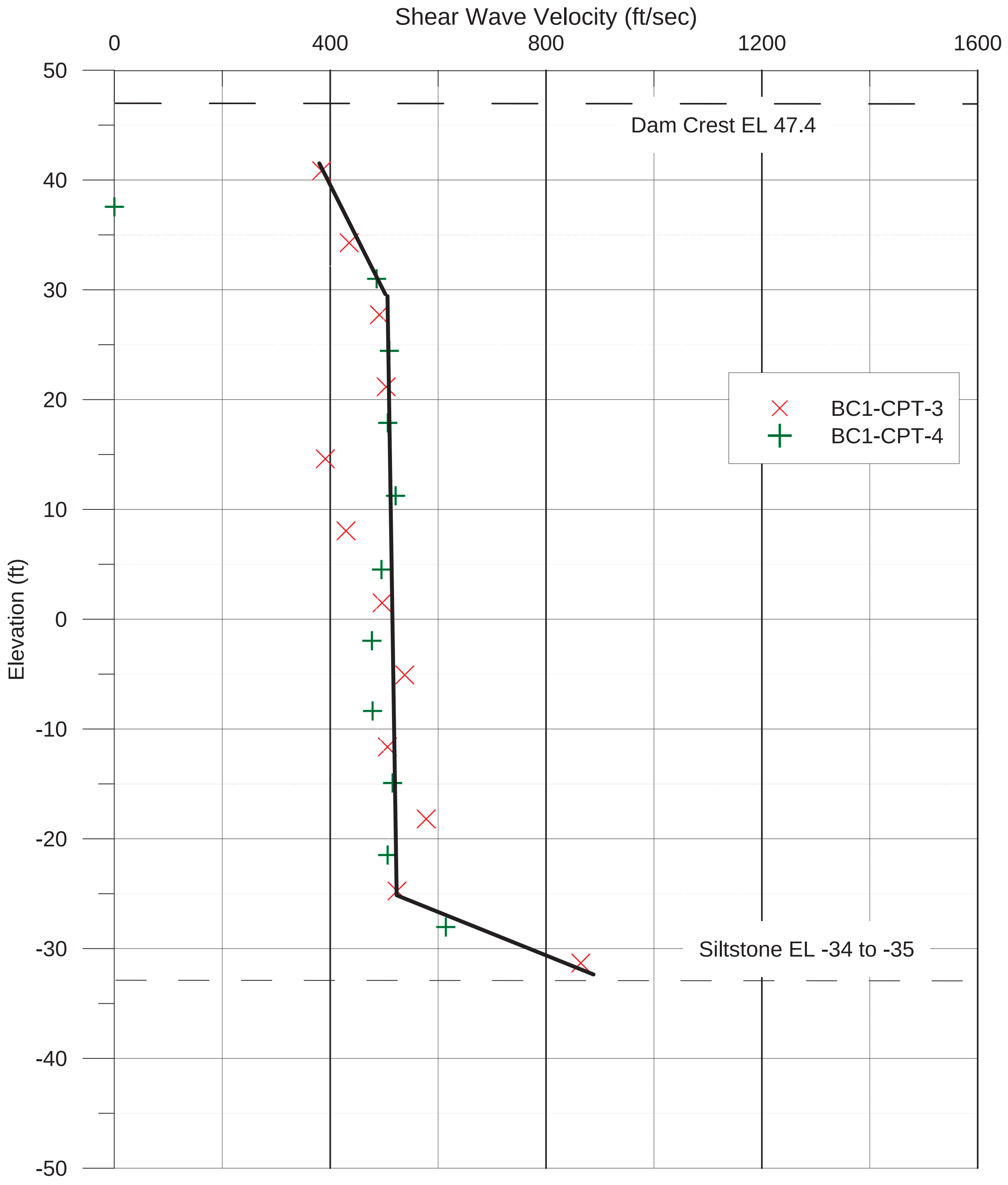
7. DEPTH DRILLED INTO ROCK N/A

17. TOTAL CORE RECOVERY FOR BORING N/A

8. TOTAL DEPTH OF BORING 41.5ft

18. LOGGED BY Nick Clark, P.E. (HDR) CHECKED BY

	ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	$N_{Corr.}$	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Laboratory								REMARKS	
											Gravel	Sand	Fines	LL	PI	DD	MC	ASTM Class		
0.0								<u>Silty SAND to Sandy SILT some clay</u> (ML); brown mottled orange to gray, low plasticity, trace organics, fine sand, scattered gravel fragments, soft to stiff (Fill).												0.0
2.5				3 6 8	14	14				1	0	48	52	48	14			ML		2.5
5.0				4 4 7	11	11				2										5.0
7.5				1 1 3	4	4				3	0	52	48	50	12	71	50	SM		7.5
10.0	40.1	10.0		4 1 3	4	7		<u>SANDY SILT to Silty SAND some clay</u> (MH/SM); wet, gray, nonplastic to medium plasticity, fine to coarse sand, medium stiff to loose (Alluvium).		4										10.0
12.5				1 1 2	3	5				5	0	36	64	52	11		105	MH		12.5
15.0				3 2 3	5	7				6	0	61	39	49	4		57	SM		15.0
17.5				500 psi						7	0	64	37			63	51	SM		17.5
20.0				2 2 2	4	5				8										20.0
22.5				1 3 4	7	9				9	0	69	32	52	19		49	SM		22.5
25.0																				25.0



HDR ENGINEERING, INC.
1001 SW 5th Avenue
Suite 1800
Portland, OR 97204-1134

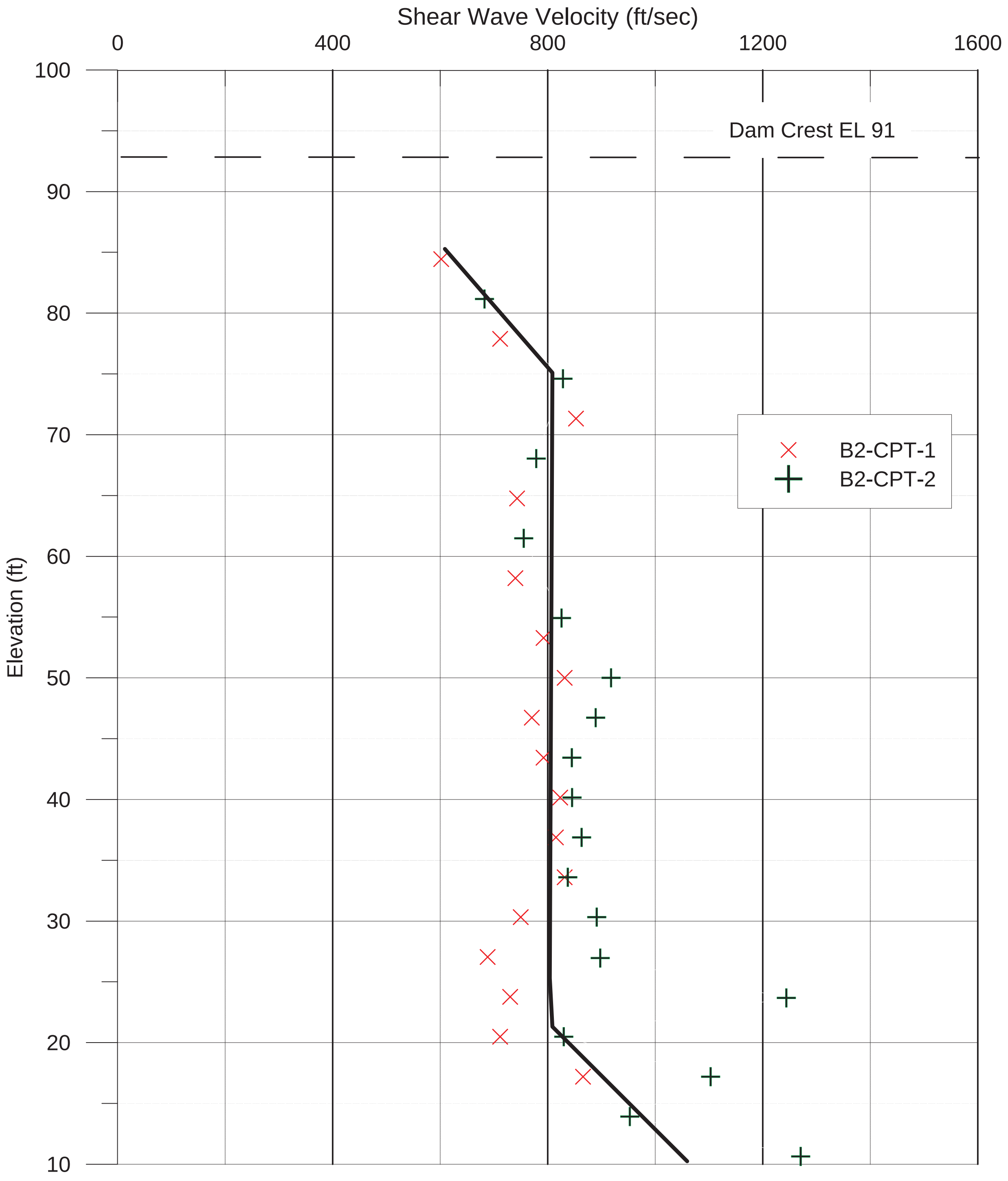
Shear Wave Velocity from CPT Sounding

BC1-CPT-3 and CPT-4
Big Creek Dam #1
Dam Seismic Evaluation

DATE

FIGURE

B.2-1



HDR ENGINEERING, INC.
1001 SW 5th Avenue
Suite 1800
Portland, OR 97204-1134

Shear Wave Velocity from CPT Sounding

BC2-CPT-1 and CPT-2

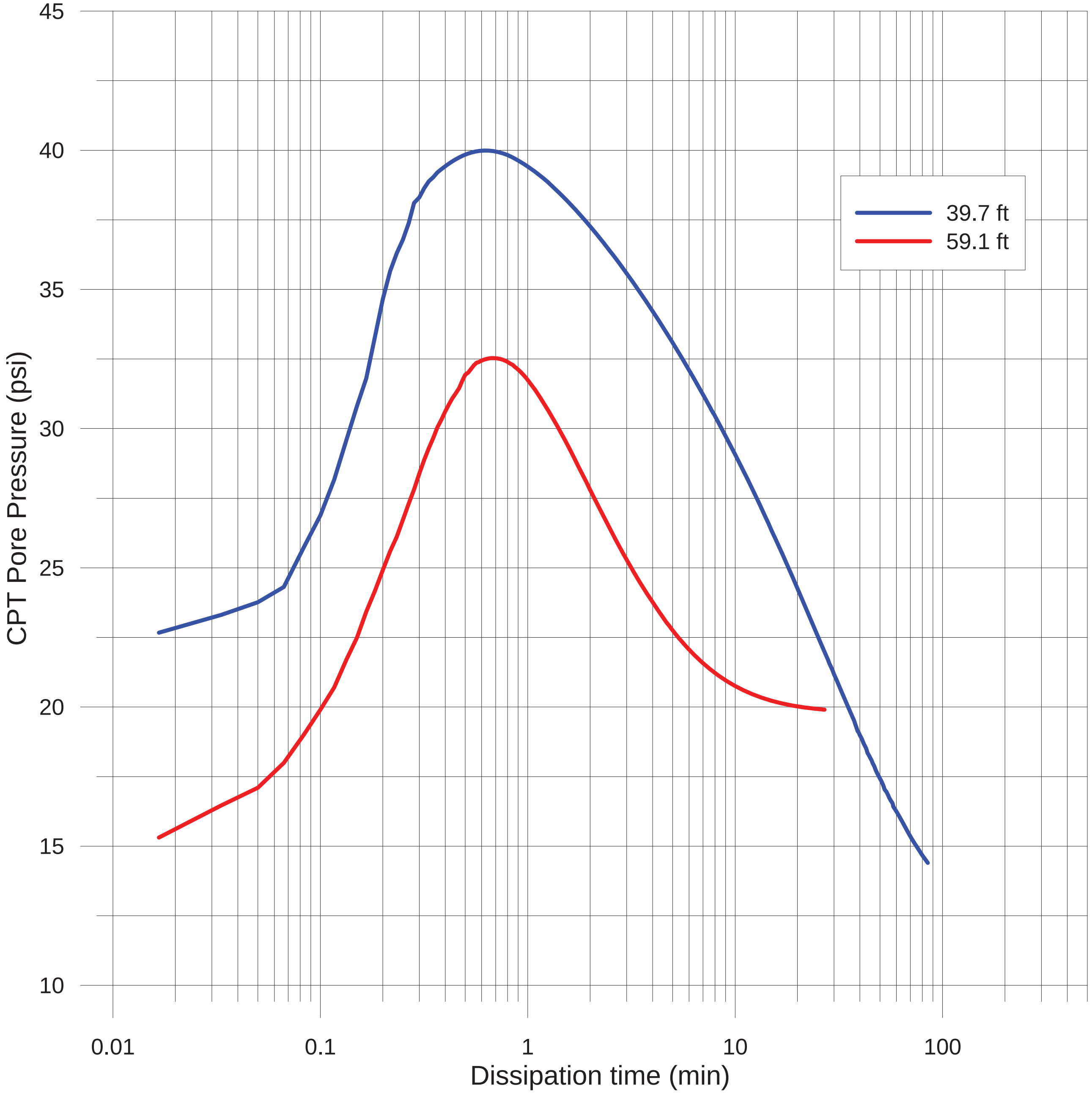
Big Creek Dam #2

Dam Seismic Evaluation

DATE

FIGURE

B.2-2



HDR ENGINEERING, INC.
1001 SW 5th Avenue
Suite 1800
Portland, OR 97204-1134

CPT Pore Pressure Dissipation

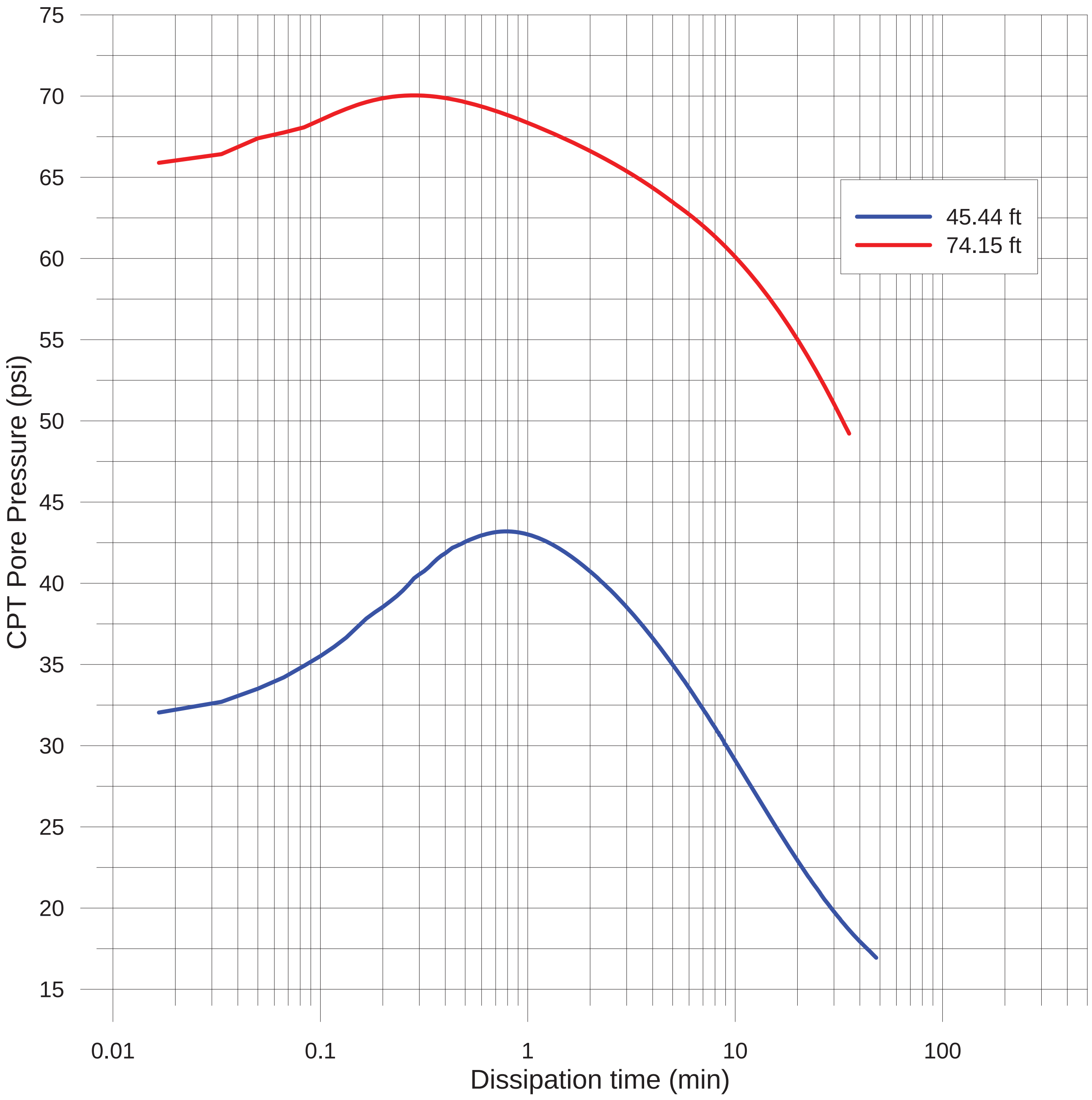
Exploration: BC1-CPT-3

Big Creek Dam #1
Dam Seismic Evaluation

DATE

FIGURE

B.2-3



HDR ENGINEERING, INC.
1001 SW 5th Avenue
Suite 1800
Portland, OR 97204-1134

CPT Pore Pressure Dissipation

Exploration: BC1-CPT-4

Big Creek Dam #1
Dam Seismic Evaluation

DATE

FIGURE

B.2-4

Boring Designation: BC2-B-3

DRILLING LOG (Cont Sheet)	INSTALLATION	SHEET 2 OF 2 SHEETS
PROJECT Newport WTP Dam Assessment (Big Creek #2)	COORDINATE SYSTEM State Plan Grid 1983	HORIZONTAL / VERTICAL
LOCATION COORDINATES N 381,151.6 E 7,283,395.0	ELEVATION TOP OF BORING 50.1 ft	

	ELEV	DEPTH	SAMPLE	Blows/6" or Press.	N_f	$N_{Corr.}$	LEGEND	FIELD CLASSIFICATION OF MATERIALS (Description)	% REC	Samp No.	Laboratory Gravel	Sand	Fines	LL	PI	DD	MC	ASTM Class	REMARKS
25.0			█	0 0 2	2	2		SANDY SILT to Silty SAND some clay (MH/SM); wet, gray, nonplastic to medium plasticity, fine to coarse sand, medium stiff to loose (Alluvium). *(continued)*		10									
27.5				500 psi						11	0	68	32			75	55	SM	
	20.1	30.0																	
30.0			█	3 4 9	13	15		SILT some sand (ML); damp, gray/blue, rounded, low plasticity, fine sand, scattered gravel and wood fragments/organics, stiff (Decomposed Siltstone).		12	0	71	29	51	17		41	SM	
32.5			█	3 5 6	11	12				13									
35.0			█	4 5 5	10	11				14									
37.5																			
40.0			█	6 16 32	48	46				15									
	8.6	41.5																	

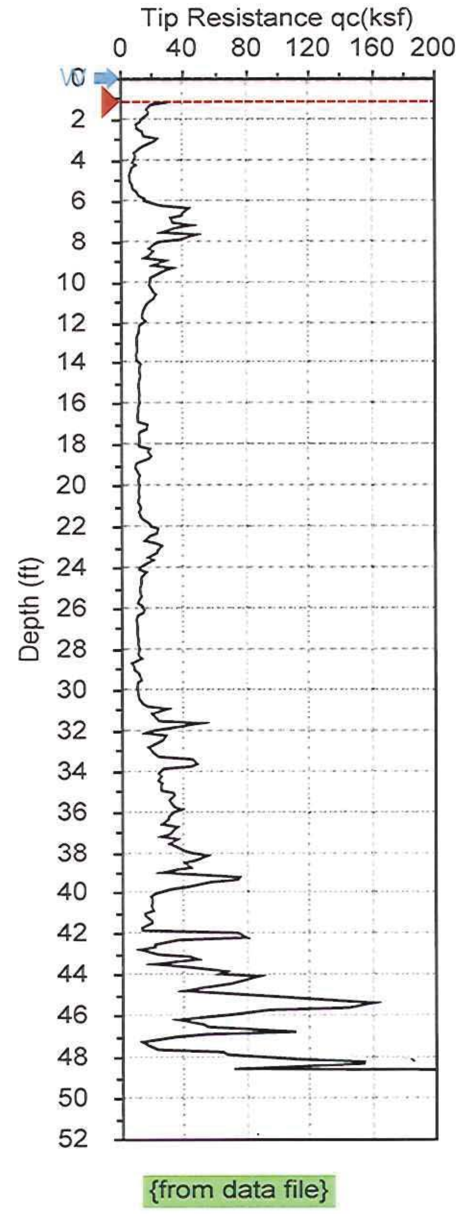
SPK FORM 1836-A
SEP 05

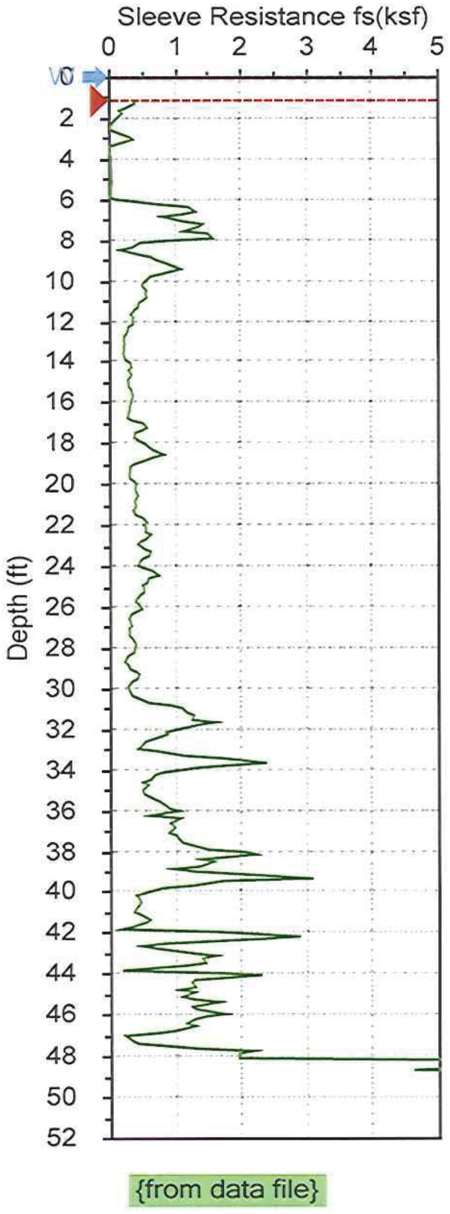
Cone Penetration Test Interpretations

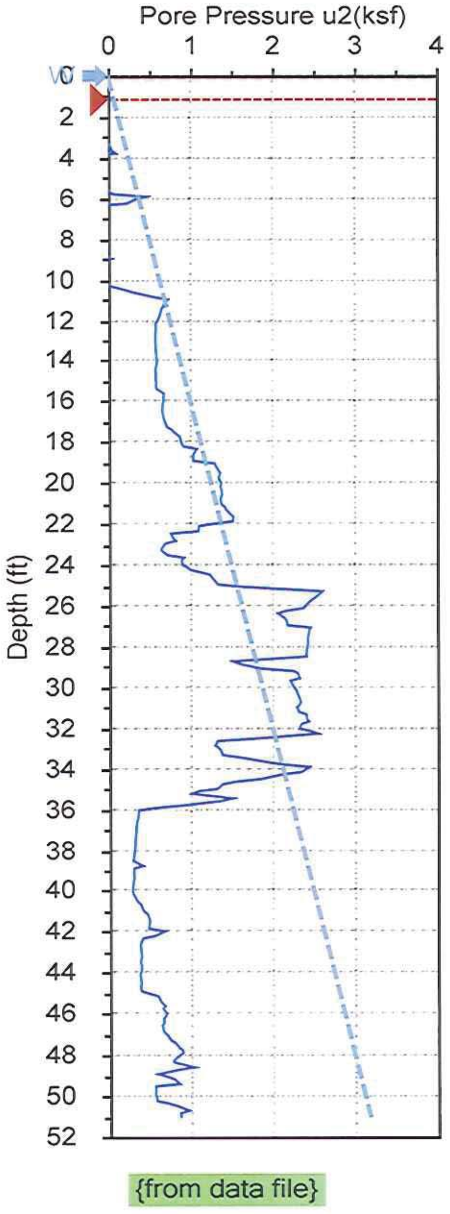
HDR Engineering, inc

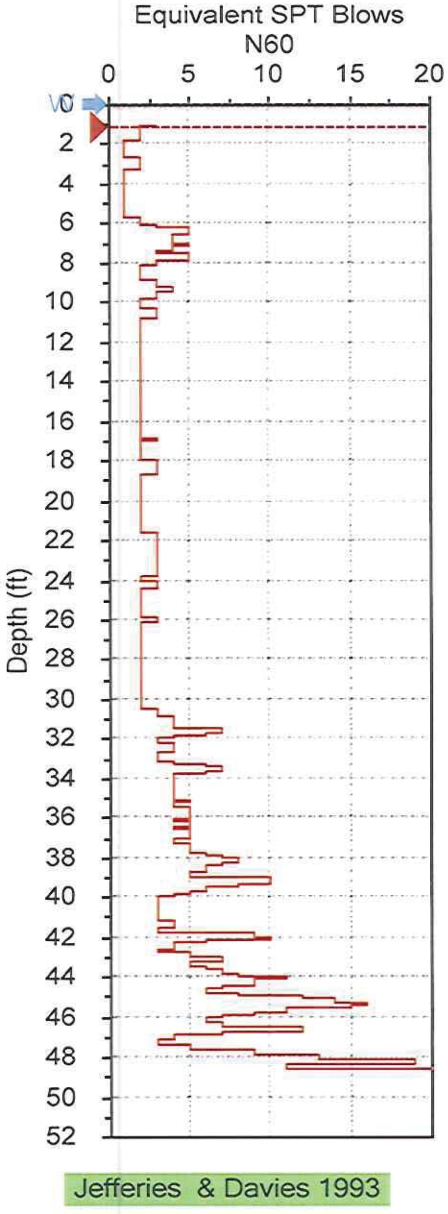
Job Title : Big Creek #1
Job Code :
Client : City of Newport
Address :

Borehole : BC1-CPT-1
Groundwater : 0 ft
Coordinates : X=0 , Y=0 , Z=33.81
Calculated By :
Checked By :







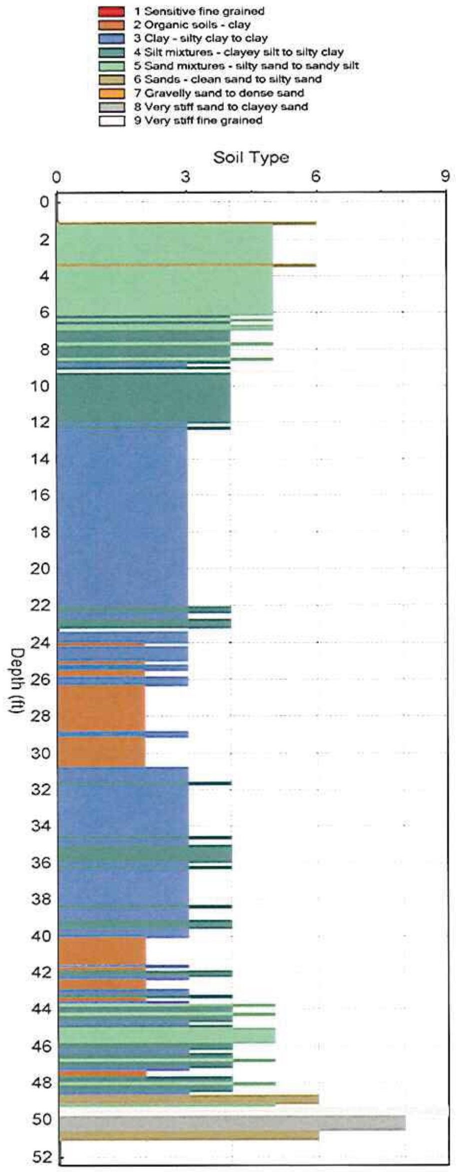


Cone Penetration Test Interpretations

HDR Engineering, inc

Job Title : Big Creek #1
Job Code :
Client : City of Newport
Address :

Borehole : BC1-CPT-1
Groundwater : 0 ft
Coordinates : X=0 , Y=0 , Z=33.81
Calculated By :
Checked By :



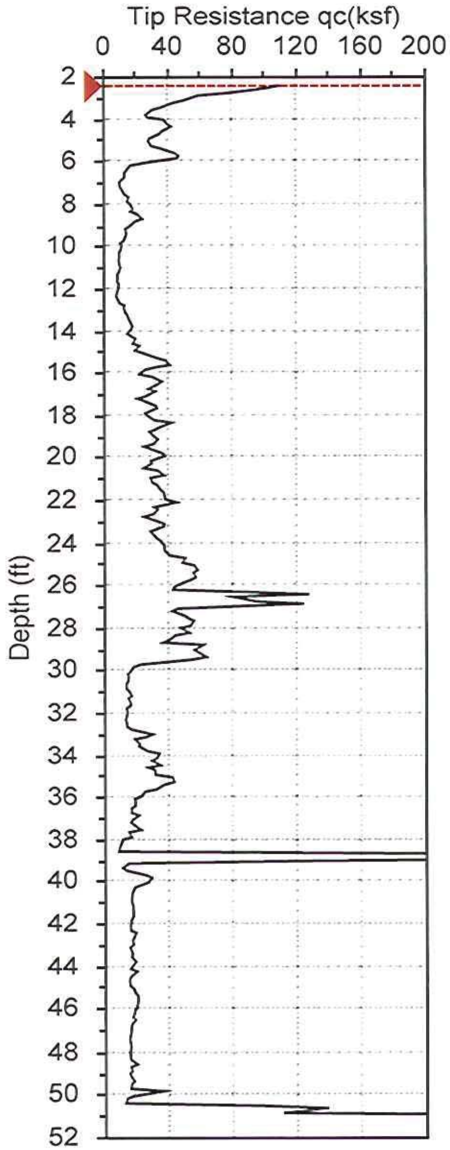
Robertson 1990, Soil Behaviour Type

Cone Penetration Test Interpretations

HDR Engineering, inc

Job Title : Big Creek #1
Job Code :
Client : City of Newport
Address :

Borehole : BC1-CPT-2
Groundwater : 0 ft
Coordinates : X=0 , Y=0 , Z=34.33
Calculated By : B Meyer
Checked By :



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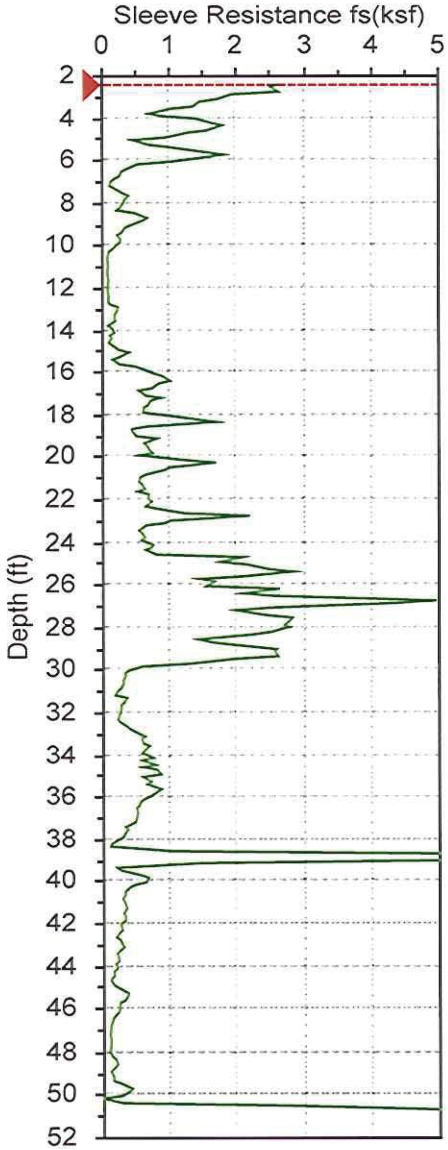

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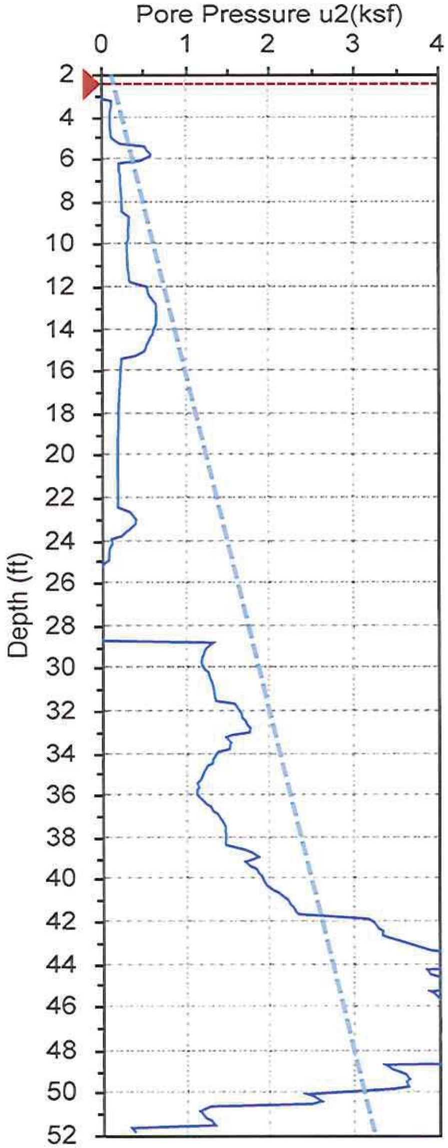

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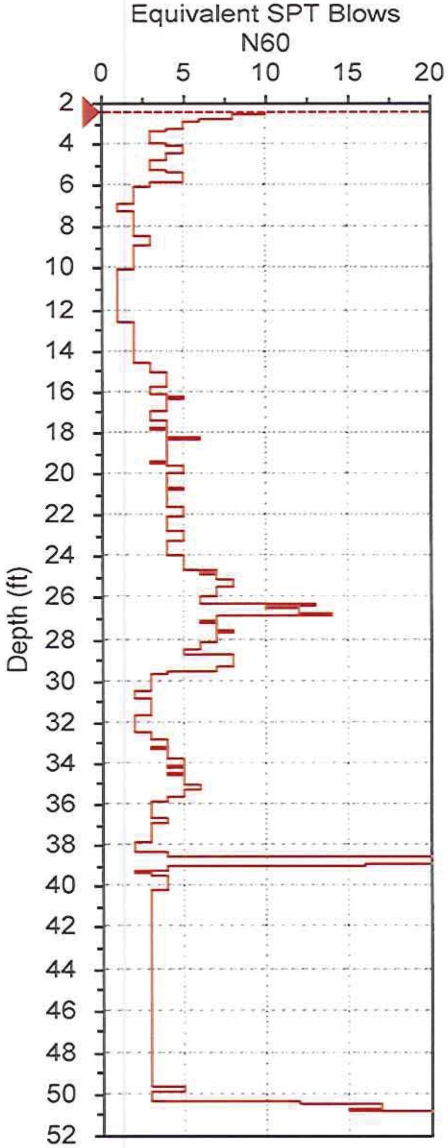
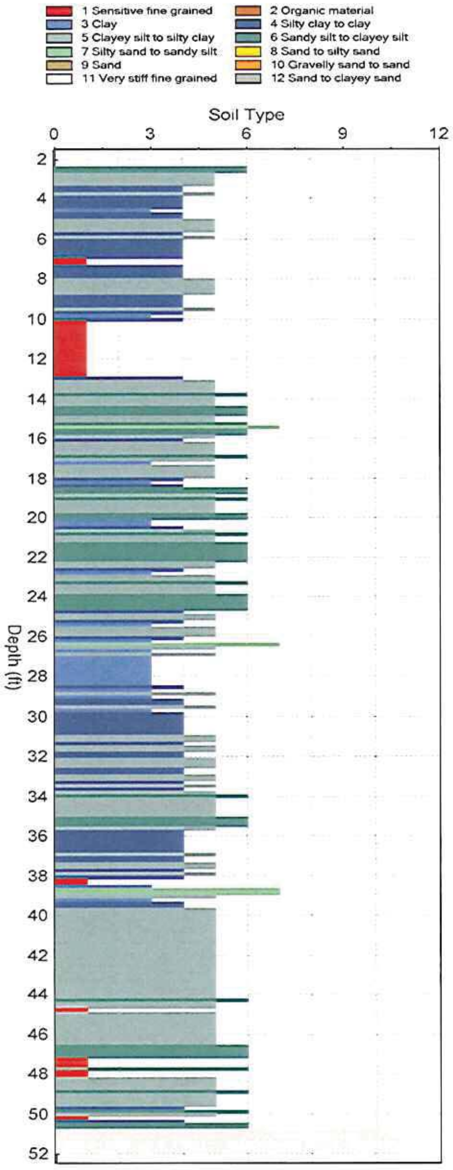


Jefferies & Davies 1993

Cone Penetration Test Interpretations

Job Title : Big Creek #1
Job Code :
Client : City of Newport
Address :

Borehole : BC1-CPT-2
Groundwater : 0 ft
Coordinates : X=0 , Y=0 , Z=34.33
Calculated By : B Meyer
Checked By :



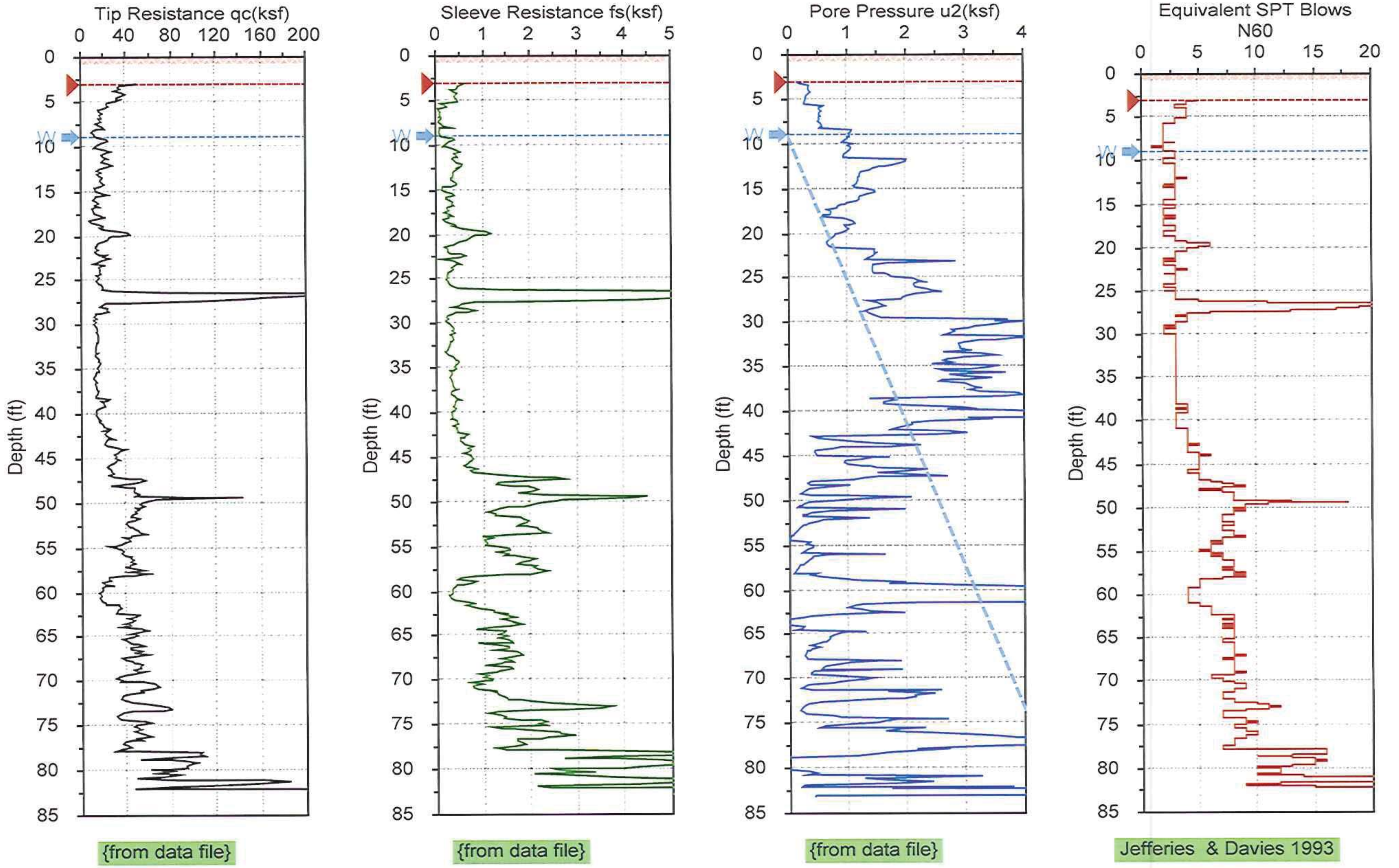
, Soil Behaviour Type

Cone Penetration Test Interpretations

HDR Engineering, inc

Job Title : Big Creek #1
Job Code :
Client : City of Newport
Address :

Borehole : BC1-CPT-3
Groundwater : 9 ft
Coordinates : X=0 , Y=0 , Z=47.4
Calculated By :
Checked By :

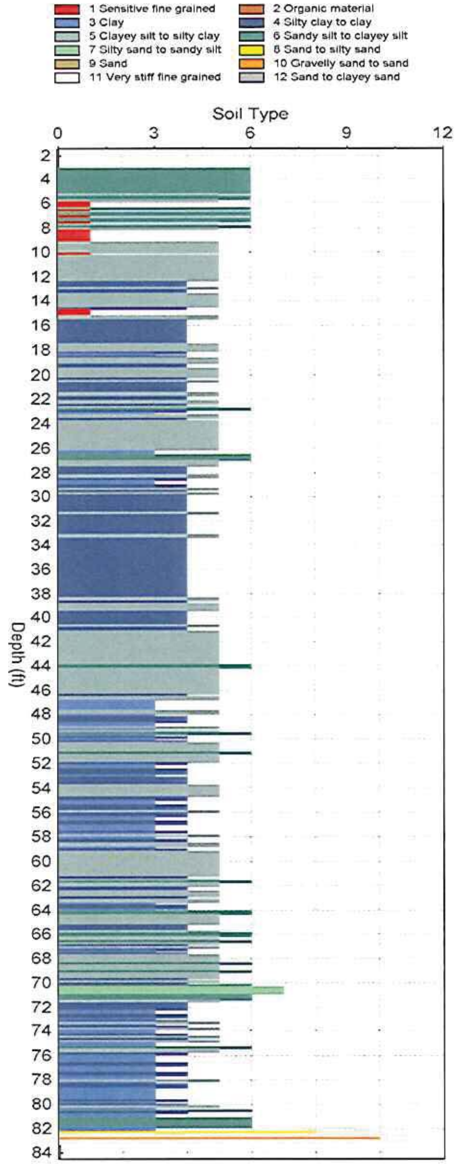


Cone Penetration Test Interpretations

HDR Engineering, inc

Job Title : Big Creek #1
Job Code :
Client : City of Newport
Address :

Borehole : BC1-CPT-3
Groundwater : 9 ft
Coordinates : X=0 , Y=0 , Z=47.4
Calculated By :
Checked By :



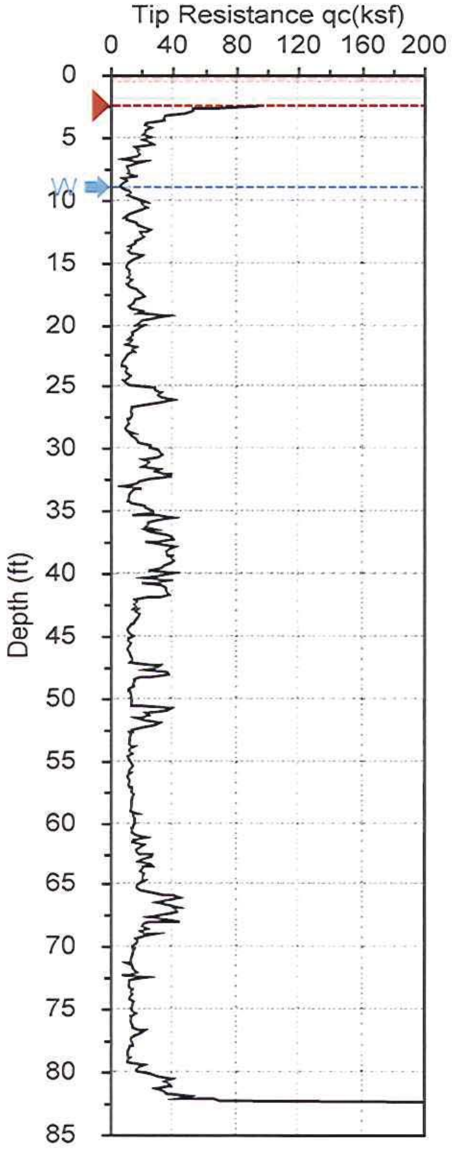
, Soil Behaviour Type

Cone Penetration Test Interpretations

HDR Engineering, inc

Job Title : Big Creek #1
Job Code :
Client : City of Newport
Address :

Borehole : BC1-CPT-4
Groundwater : 9 ft
Coordinates : X=0 , Y=0 , Z=47.6
Calculated By : B Meyer
Checked By :



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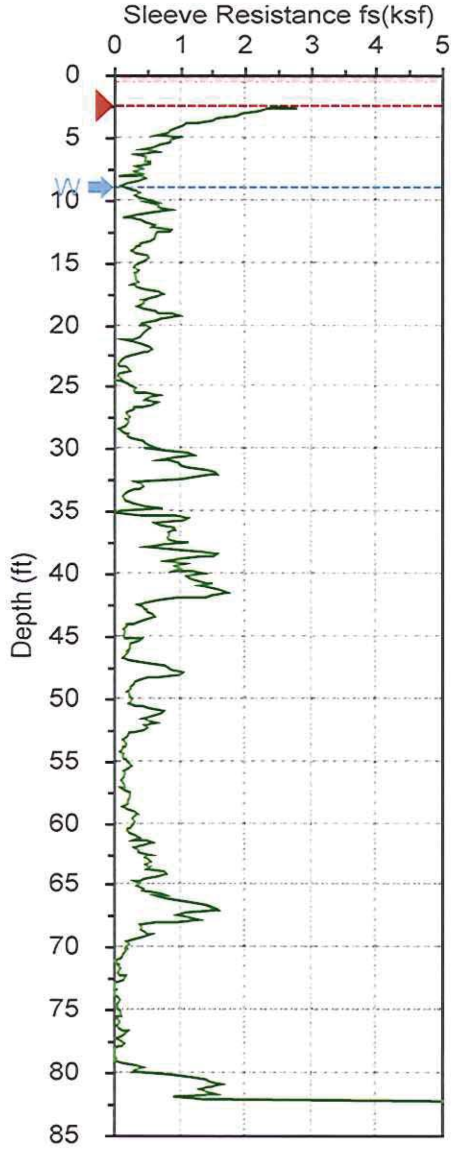

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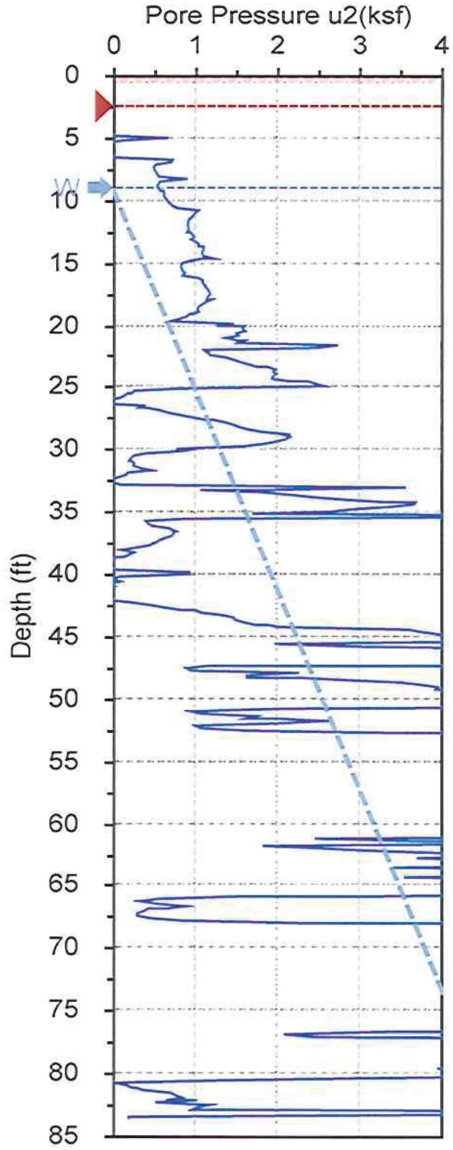

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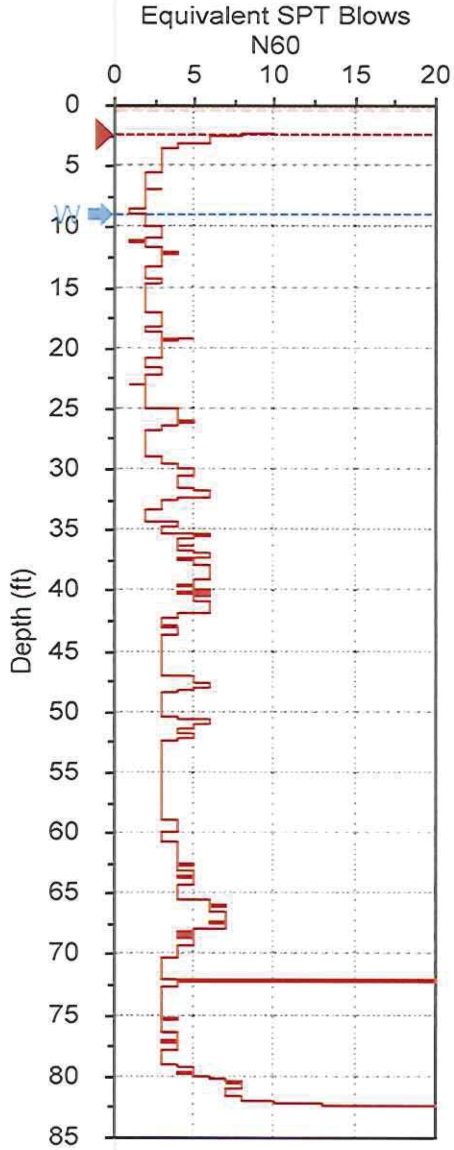


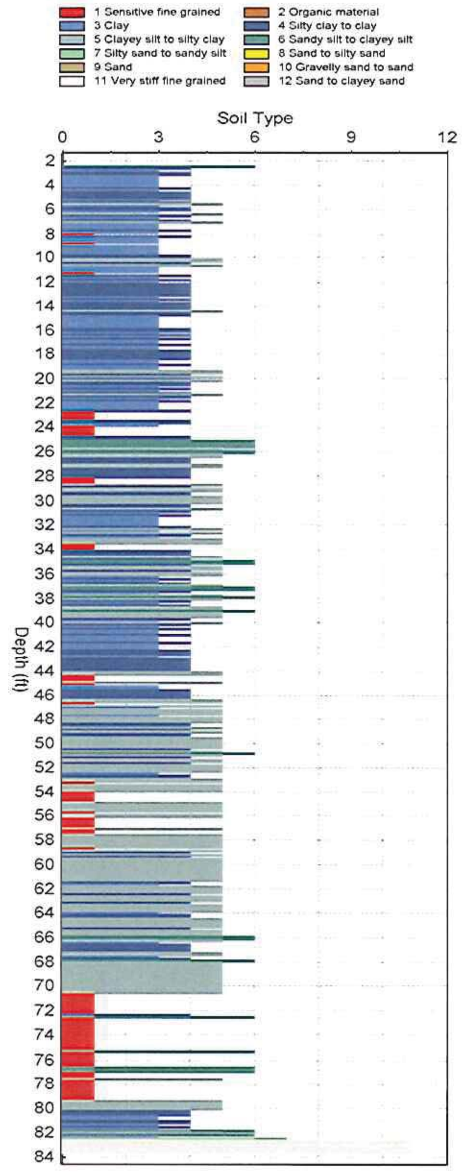
Jefferies & Davies 1993

Cone Penetration Test Interpretations

HDR Engineering, inc

Job Title : Big Creek #1
Job Code :
Client : City of Newport
Address :

Borehole : BC1-CPT-4
Groundwater : 9 ft
Coordinates : X=0 , Y=0 , Z=47.6
Calculated By : B Meyer
Checked By :



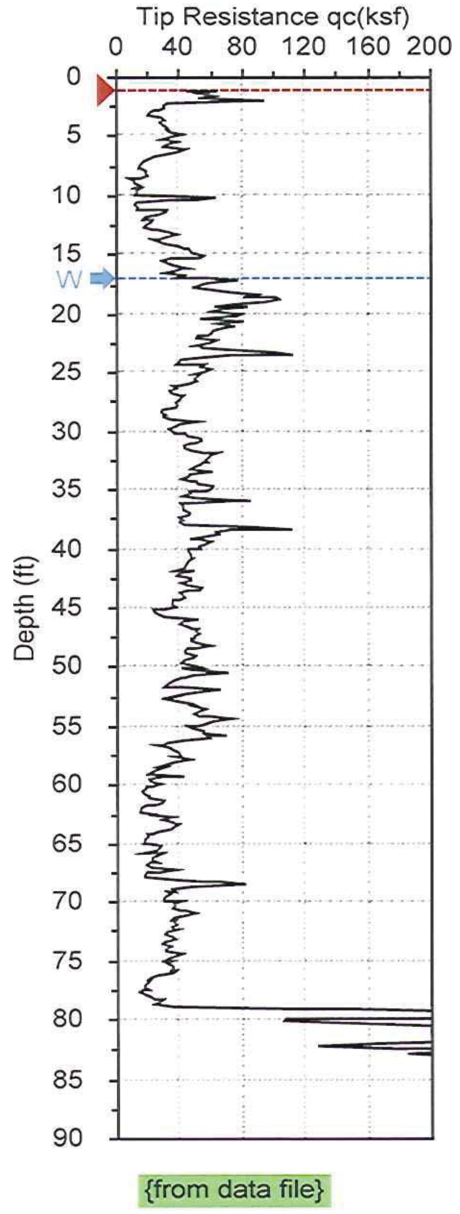
, Soil Behaviour Type

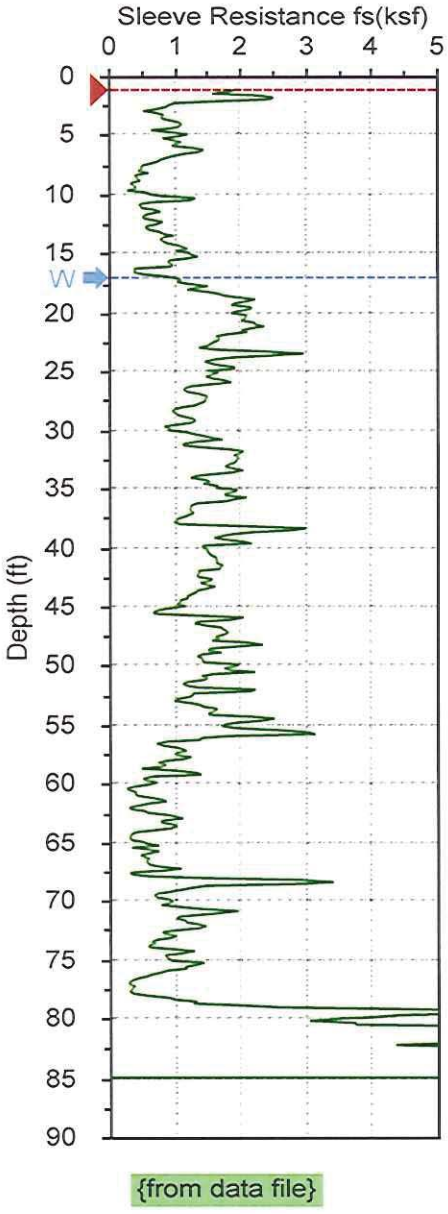
Cone Penetration Test Interpretations

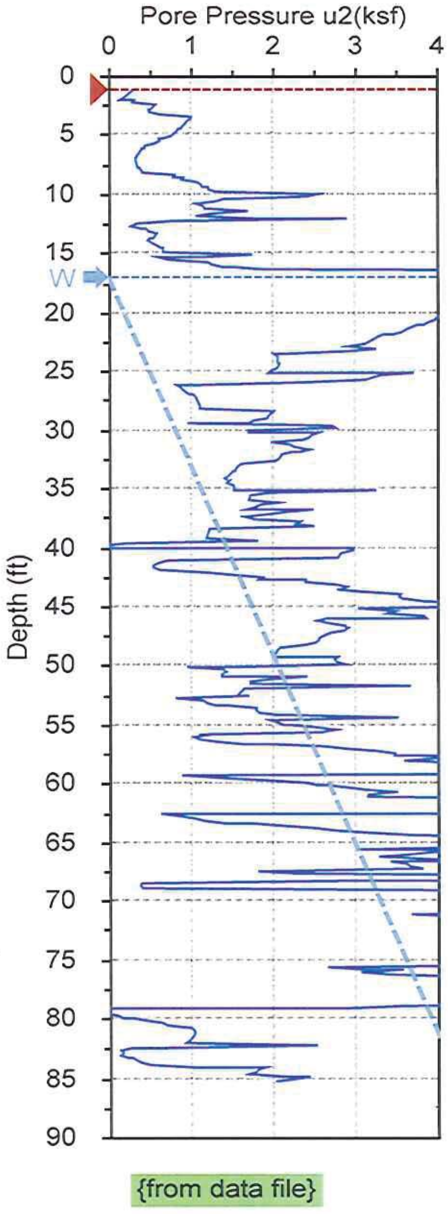
HDR Engineering, inc

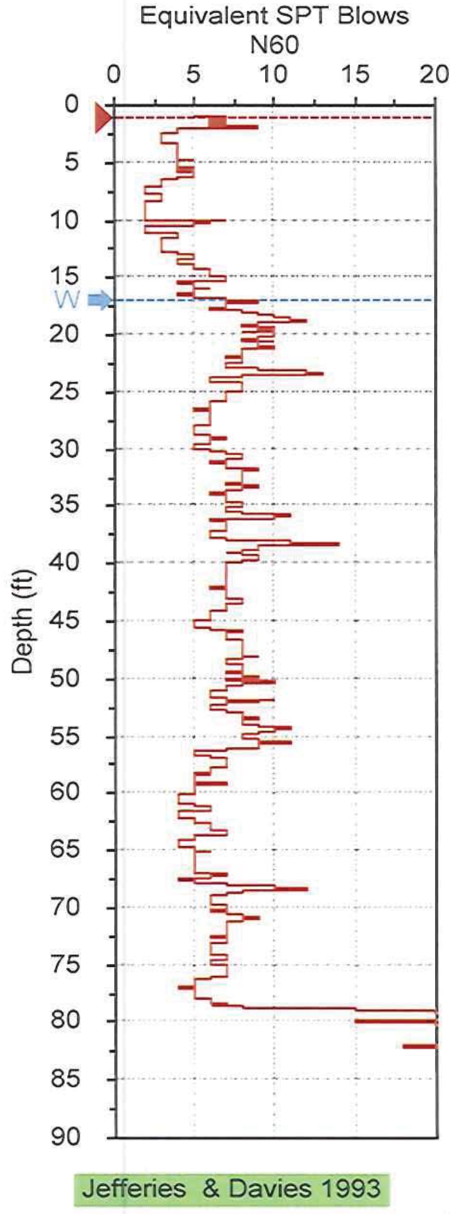
Job Title : Big Creek #~~1~~2
Job Code :
Client : City of Newport
Address :

Borehole : BC2-CPT-1
Groundwater : 17 ft
Coordinates : X=0 , Y=0 , Z=91.6
Calculated By : B Meyer
Checked By :







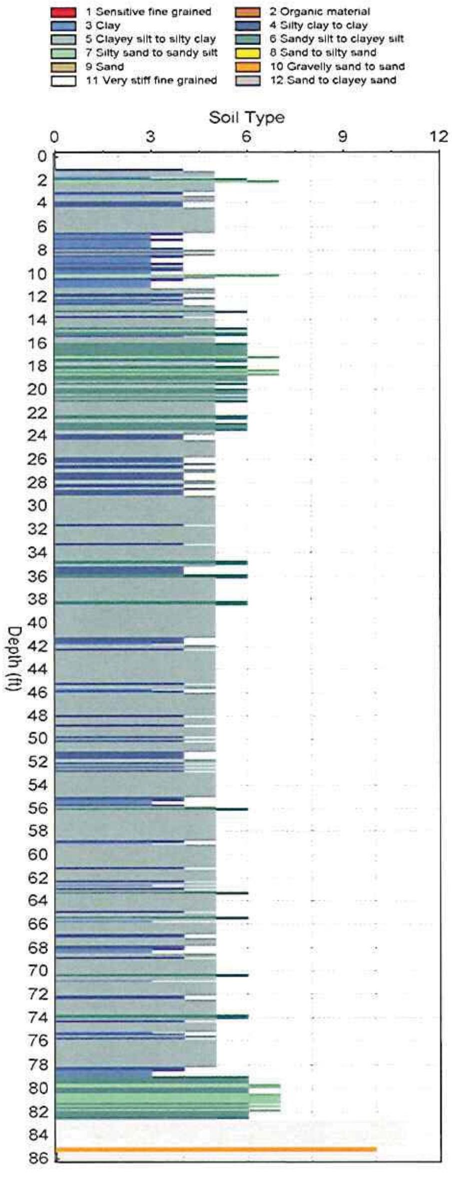


Cone Penetration Test Interpretations

HDR Engineering, inc

Job Title : Big Creek #~~1~~ 2
Job Code :
Client : City of Newport
Address :

Borehole : BC2-CPT-1
Groundwater : 17 ft
Coordinates : X=0 , Y=0 , Z=91.6
Calculated By : B Meyer
Checked By :



, Soil Behaviour Type

Cone Penetration Test Interpretations

HDR Engineering, inc

Job Title : Big Creek #1/2
Job Code :
Client : City of Newport
Address :

Borehole : BC2-CPT-2
Groundwater : 17 ft
Coordinates : X=0 , Y=0 , Z=91.3
Calculated By : B Meyer
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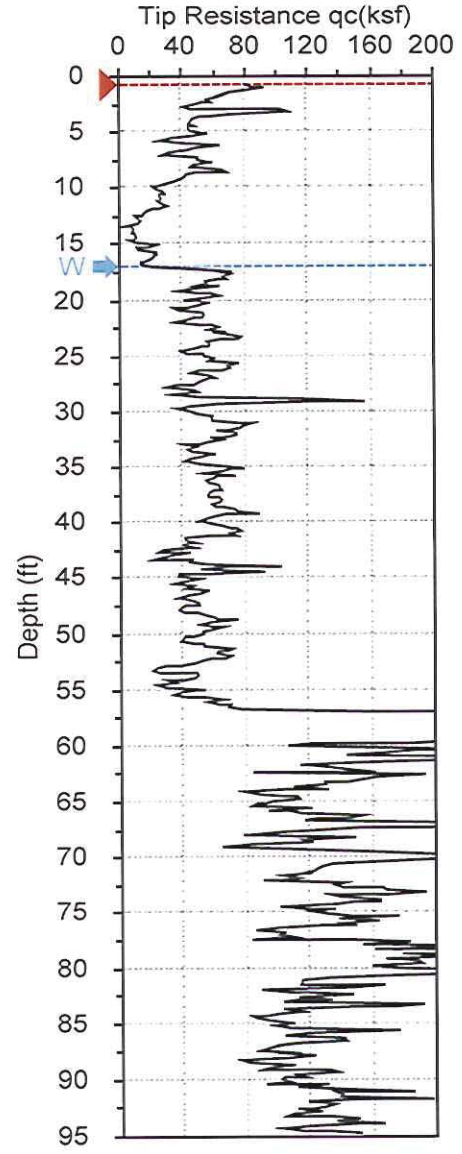

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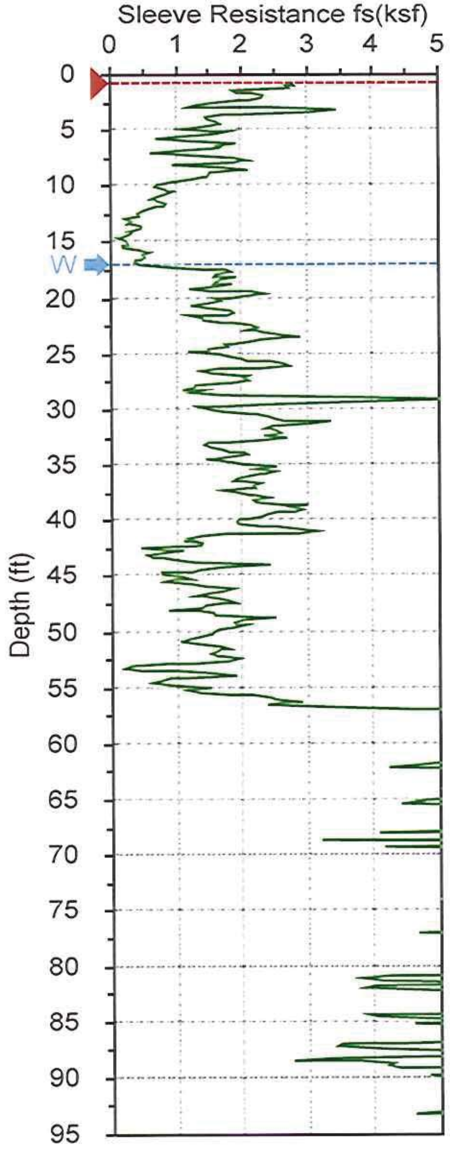

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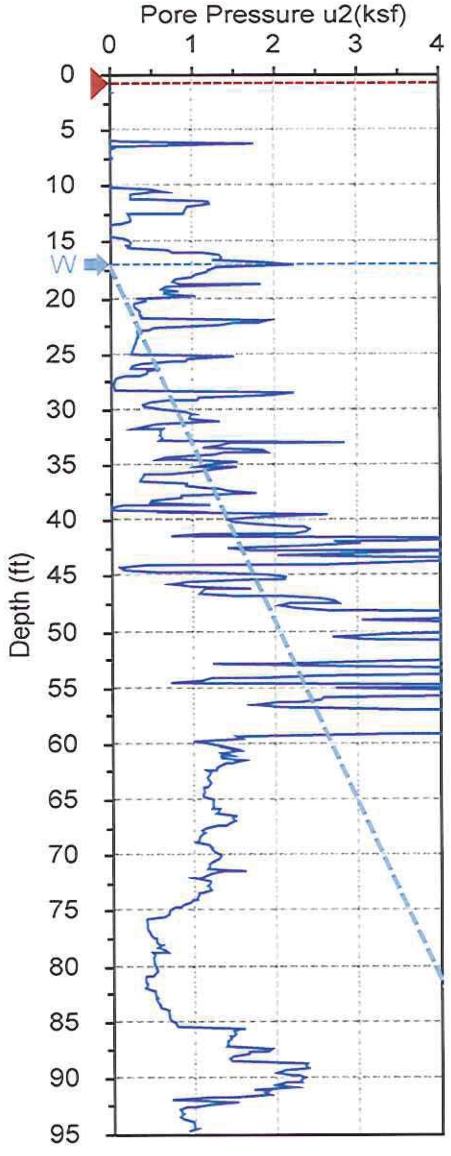

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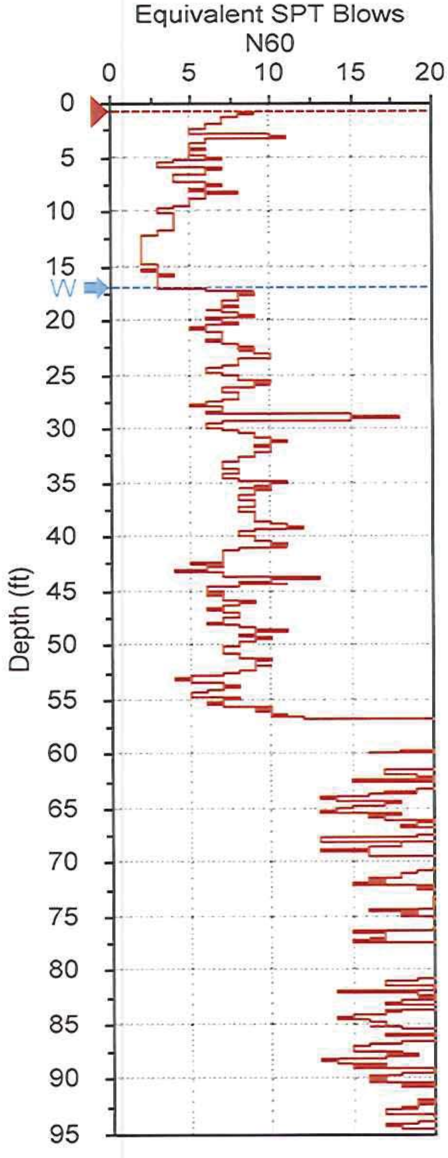


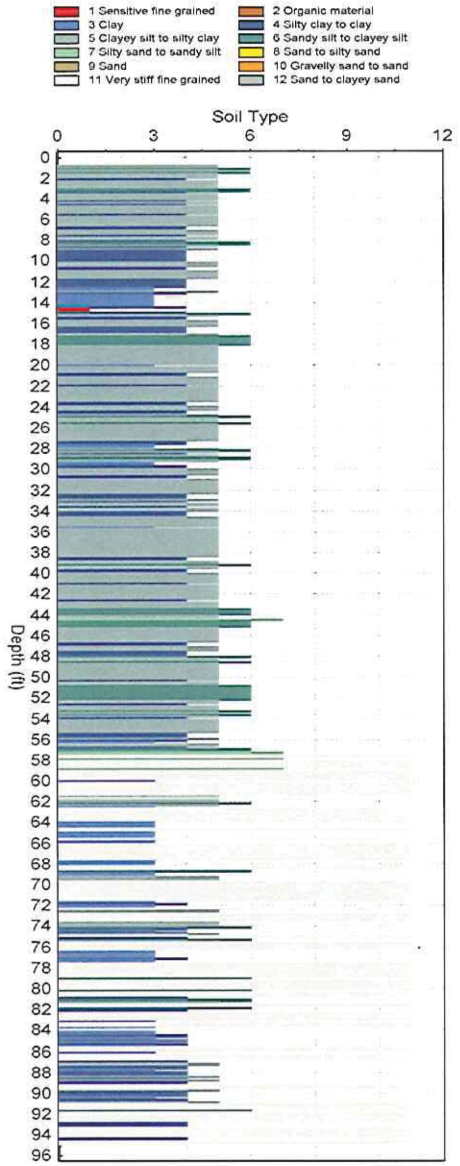
Jefferies & Davies 1993

Cone Penetration Test Interpretations

HDR Engineering, inc

Job Title : Big Creek #~~1~~2
Job Code :
Client : City of Newport
Address :

Borehole : BC2-CPT-2
Groundwater : 17 ft
Coordinates : X=0 , Y=0 , Z=91.3
Calculated By : B Meyer
Checked By :



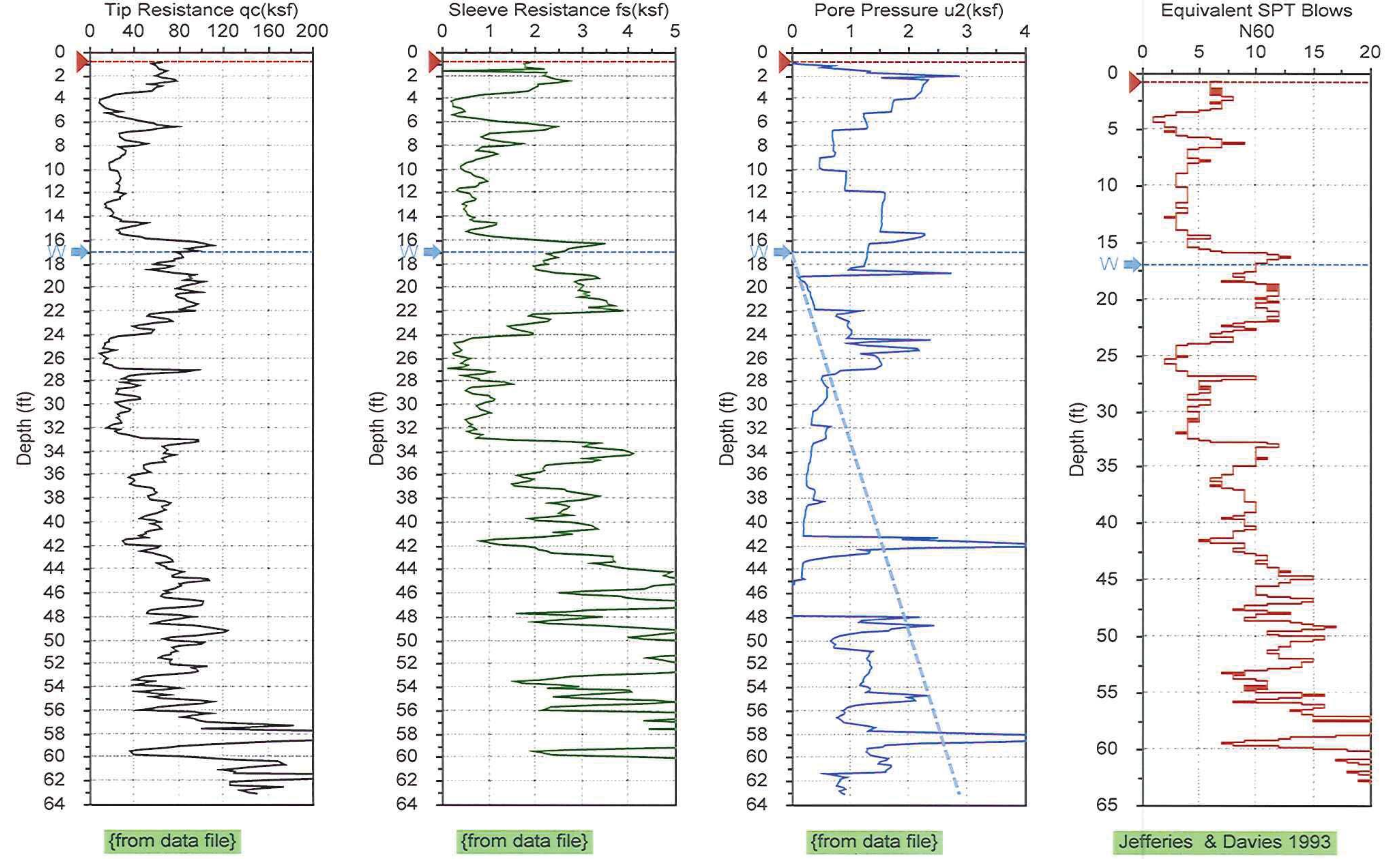
, Soil Behaviour Type

Cone Penetration Test Interpretations

HDR Engineering, inc

Job Title : Big Creek #1/2
Job Code :
Client : City of Newport
Address :

Borehole : BC2-CPT-3
Groundwater : 17 ft
Coordinates : X=0 , Y=0 , Z=91.1
Calculated By : B Meyer
Checked By :

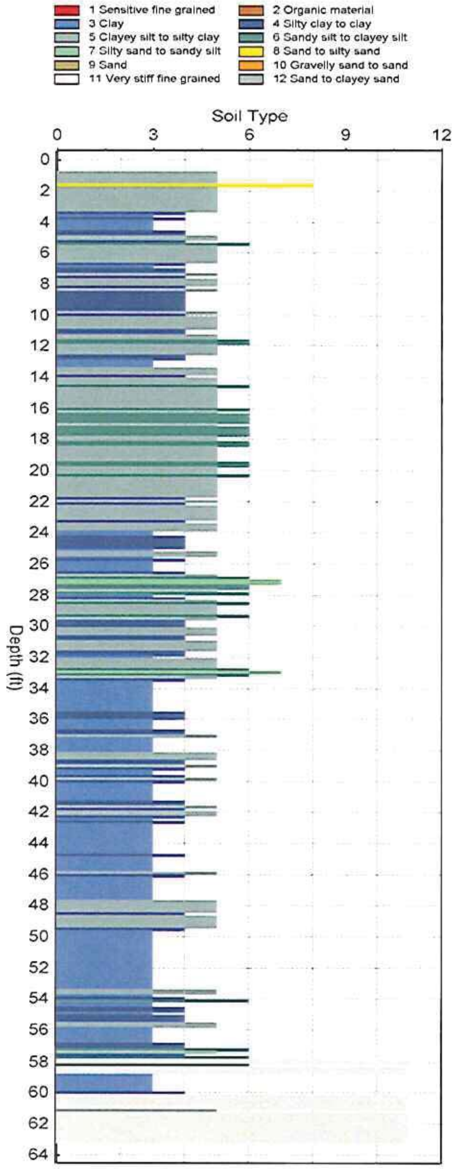


Cone Penetration Test Interpretations

HDR Engineering, inc

Job Title : Big Creek #~~1~~2
Job Code :
Client : City of Newport
Address :

Borehole : BC2-CPT-3
Groundwater : 17 ft
Coordinates : X=0 , Y=0 , Z=91.1
Calculated By : B Meyer
Checked By :



, Soil Behaviour Type

Northwest
Geophysical
Associates, Inc.
A Zonge International Company
Parkside Business Center, Bldg. 1-B
8366 SW Nimbus Ave, Beaverton, OR 97008
Phone: 503-992-6723   FAX: 503-746-7094
www.nga.com

Zonge
Trusted Geophysics

April 2, 2012
Ref: 806

Richard Hannan, PE, RPG, CEG
Geotechnical Engineer
HDR Engineering, Inc.
1001 SW 5th Avenue
Portland, OR 97204-1134

Re: ***Seismic Refraction Survey***
Big Creek Dams #1 and #2
Newport, Oregon

Dear Mr. Hannan,

This letter report presents the results of the seismic refraction survey which Northwest Geophysical Associates, Inc. (NGA) performed at Big Creek Dams #1 and #2 in Newport, Oregon (Figure 1). The fieldwork was performed on December 20 and 21, 2011. The objective of the investigation was to determine depth-to-bedrock beneath the earthen dam structures. A technical description of the seismic refraction method is attached (Appendix A).

This report is a revision of our original report of January 20, 2012 and supersedes that report.

Seismic Line locations are shown on Figures 2 and 3. Interpreted results are presented in this report as Figures 4 and 5.

FIELD SURVEY

Seismic line SL-1 was run on the western edge of the Big Creek Reservoir #1 dam crest as shown on Figure 2. SL-2 and SL-3 were run in opposing orientations radiating outward from the downstream toe of Big Creek Reservoir #2 seen in Figure 3. Limited space along with a flowing stream, fish structure, and wetland prevented orienting our seismic lines parallel to Dam #2. Each seismic line was established in the field using 300-foot tape measures with survey paint and/or pin flags marking each of the geophone locations. Elevations of each geophone along SL-2 and SL-3 were measured with a transit level and stadia rod. Those elevations were then tied approximately to Mean Sea

Level using a Trimble 6000 Series GeoExplorer XH, which is a GPS unit with sub-meter vertical accuracy.

The field investigation was performed using a 24-channel digital seismograph to record the data. A slide-hammer source was used to generate a seismic wave at regular intervals along each seismic line, and also at some distance from the end of each line where space allowed. Seismic line SL-1 used 24 geophones with a geophone spacing of 13 feet to facilitate the line extending across the entire dam crest. Seismic lines SL-2 and SL-3 utilized 18 and 17 geophones respectively, each with a geophone spacing of 10 feet.

DATA PROCESSING

Data were processed using SeisImager software from Geometrics, Inc. and the OYO Corporation. Initial layered earth models were constructed from the raw seismic data based upon the plus-minus or time-term method. At Big Creek Dam #1 a three-layer model best represented the trends seen in the raw P-wave data. At Big Creek Dam #2 a two-layer model provided the best fit to the raw data. Velocity models were then output and compiled in OASIS Montaj for presentation in Figures 4 and 5.

SEISMIC REFRACTION LIMITATIONS

The seismic refraction technique assumes that the velocity increases with depth. Traditional plus-minus or delay time interpretations used here assume a homogeneously layered earth. Those basic assumptions may not be valid at these two sites. Other tomographic interpretation techniques were ineffective with these data sets.

The *hidden layer* problem of seismic refraction occurs when a slow velocity zone or layer occurs beneath a faster velocity layer. That low velocity zone not only is not detected with the refraction interpretation but it leads to an incorrect interpretation of the deeper refractor being shallower than it actually is. We believe this is happening with these seismic data sets discussed below.

This hidden layer issue is mentioned briefly in the attached technical note and described in more detail in several of the references listed in the technical note.

INTERPRETATION AND DISCUSSION

Big Creek Dam #1

Figure 4 presents the results for SL-1 taken along the west edge of the dam crest. In general, based upon our three layer initial model, very slow P-wave velocities (V1=700 ft/sec and V2=1200 ft/sec) were measured extending to depths up to 42 feet beneath the crest of the. V3 (3,700 ft/sec) likely represents more saturated sediments. If a low velocity hidden layer is present, this interface could be considerably shallower.

Borehole BH-1 encountered bedrock at a depth of 85 feet. The seismic refraction survey did not have sufficient depth of exploration to map the bedrock surface. This is due to the short spread length and the low signal to noise ratio due to the seismic noise from the spillway. Even if the bedrock refractor had been reached, the depth calculations would be invalid due to expected velocity reversals within the dam structure.

Big Creek Dam #2

Figure 5 presents the results for SL-2 and SL-3 collected at the downstream toe of the dam. Interpreted profiles from both SL-2 and SL-3 display a low velocity zone (V1 ranging from 800 to 1100 feet/second) that is approximately 10 feet thick approaching Big Creek and the spillway at the center of the dam. We interpret this layer to be either loose fill and/or soft native ground. At SL-2, the low velocity zone thins considerably between shotpoints 2012 and 2018, which would correspond with an area of improved ground evidenced by large gravels and silt that was serving as an access route to the creek. Additionally, this area is near the northwestern edge of the dam area and the velocity high could also be related to the natural geology. Along SL-3, shotpoint 3016 was on an access road with a silty gravel base. The low velocity layer for this line did not thin beneath the road, implying that ground improvement to construct this access road is minimal in thickness.

The higher velocity V2 layer is 4,300 ft/sec at SL-3 and 5,600 ft/sec at SL-2. Based on those modeled velocities, this layer is interpreted to be a weak to moderately strong siltstone that was encountered at drilled boring in the area. However V2 may represent saturated sediments. The depth for this layer ranges from 2 to 13 feet along SL-2 and 6 to 12 feet along SL-3.

Again, if a low velocity hidden layer is present in this region the interpreted interface would be deeper than it actually is.

Discussion

Low P-wave velocities we observed at both dams are consistent with the low S-wave velocities observed in the seismic cone data. It is also consistent with the low SPT blow counts measured during drilling. The variability of the data is consistent with velocity reversals and hidden layer(s) within the section. Due to these velocity reversals, depth-to-bedrock cannot be calculated.

CLOSURE

Northwest Geophysical Associates, Inc. performed this work in a manner consistent with the level of skill ordinarily exercised by members of the profession currently practicing under similar conditions. No warranty, express or implied, beyond exercise of reasonable care and professional diligence, is made. This report is intended for use only in accordance with the purposes of the study described within.

Please feel free to contact us if you have any questions or comments regarding this information, or if you require further assistance. We appreciated the opportunity to work with you on this project.

Sincerely,

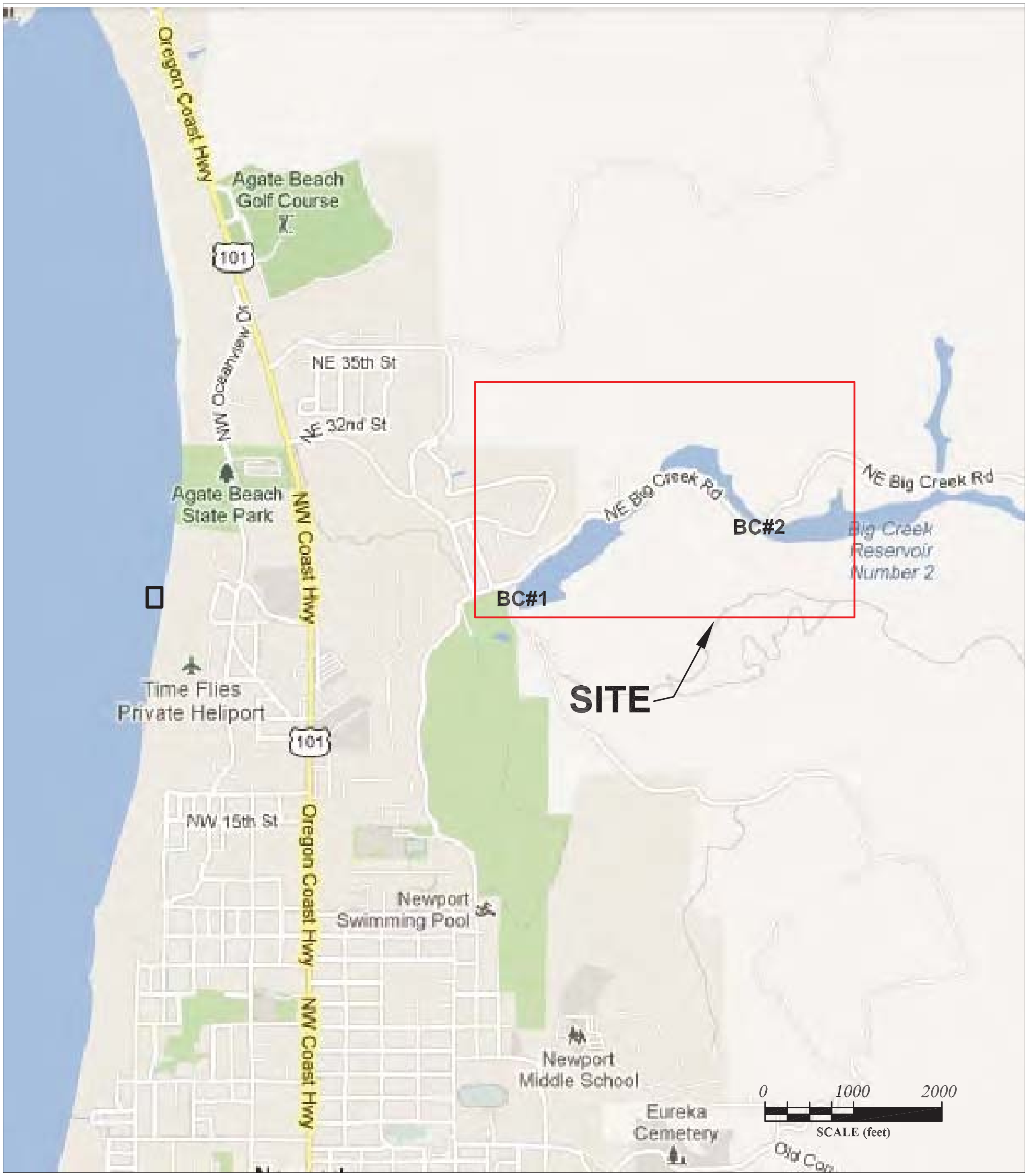
Northwest Geophysical Associates, Inc.

Rowland B. French, Ph.D., R.G.
Senior Geophysicist

Michael Douglas, R.G.
Project Geophysicist

Attachments: Appendix A-Seismic Refraction Technical Note

File: NGA Big Creek Rpt01.doc
NGA Project: 806 / 12005



N

FIGURE 1

SITE LOCATION PLAN

Seismic Refraction Survey
Big Creek Dams #1 and #2
Newport, Oregon

HDR

Prepared by:
Northwest
Geophysical
Associates, Inc.

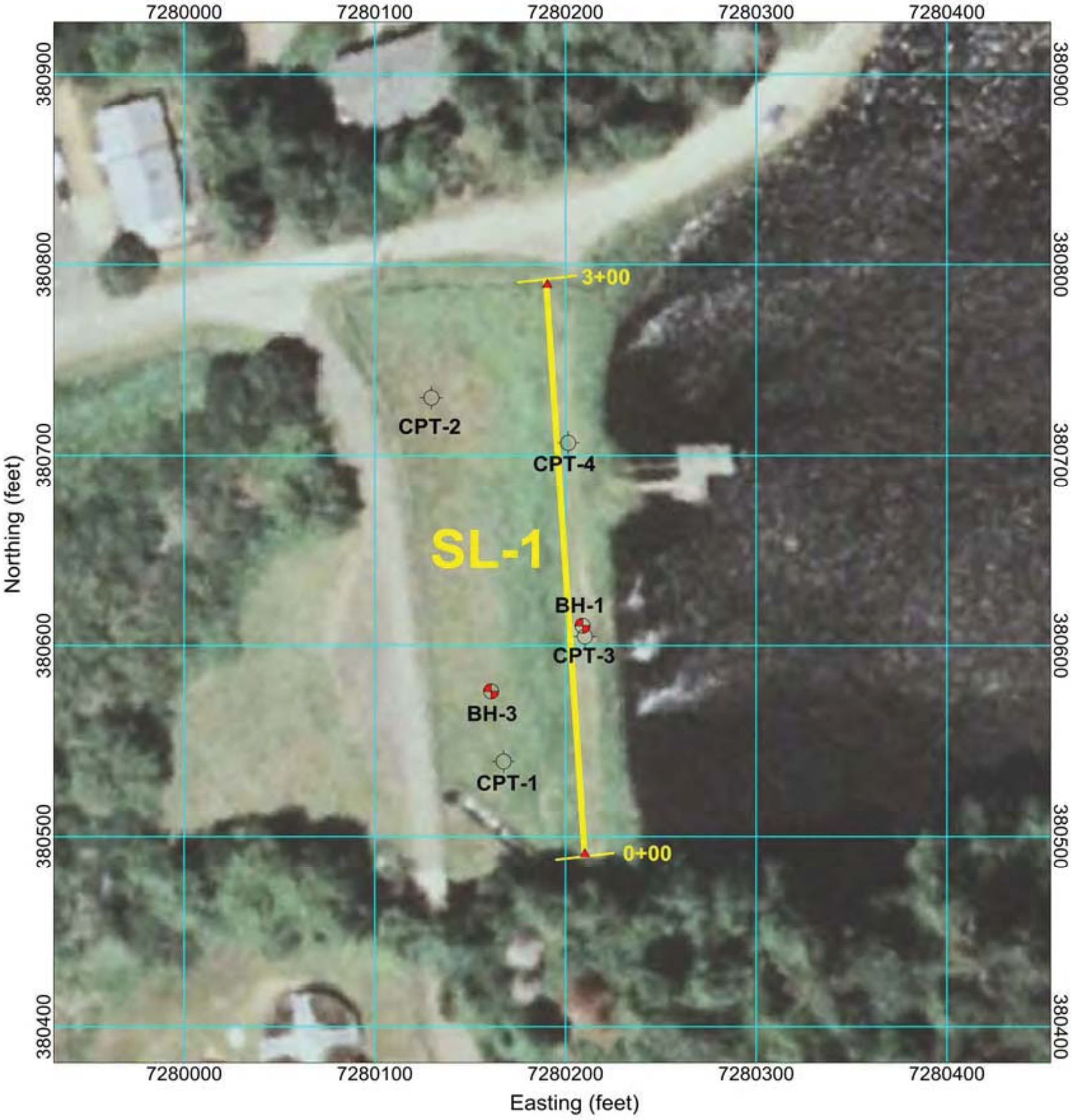
NGA Project #: 806
FILE: FIGURE1.DWG
DATE: 01/2012
REVISION: A

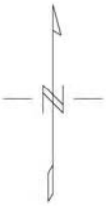
REVISION: A-10-JAN-12

DATE: DECEMBER 2011

FILE: Big_Creek_1.map

NGA PROJECT: 806





Scale 1:1200

US survey foot

NAD83 / Oregon CS83 North zone

Legend

⌖	CPT Boring
●	Drilled Boring

FIGURE 2

Prepared for:

HDR

Prepared by:

Northwest Geophysical Associates, Inc.

BIG CREEK #1
SITE EXPLORATION PLAN

Seismic Refraction Survey
Big Creek Dams #1 and #2
Newport, Oregon

REVISION: A-10-JAN-12

DATE: DECEMBER 2011

FILE: Big_Creek_2.map

NGA PROJECT: 806

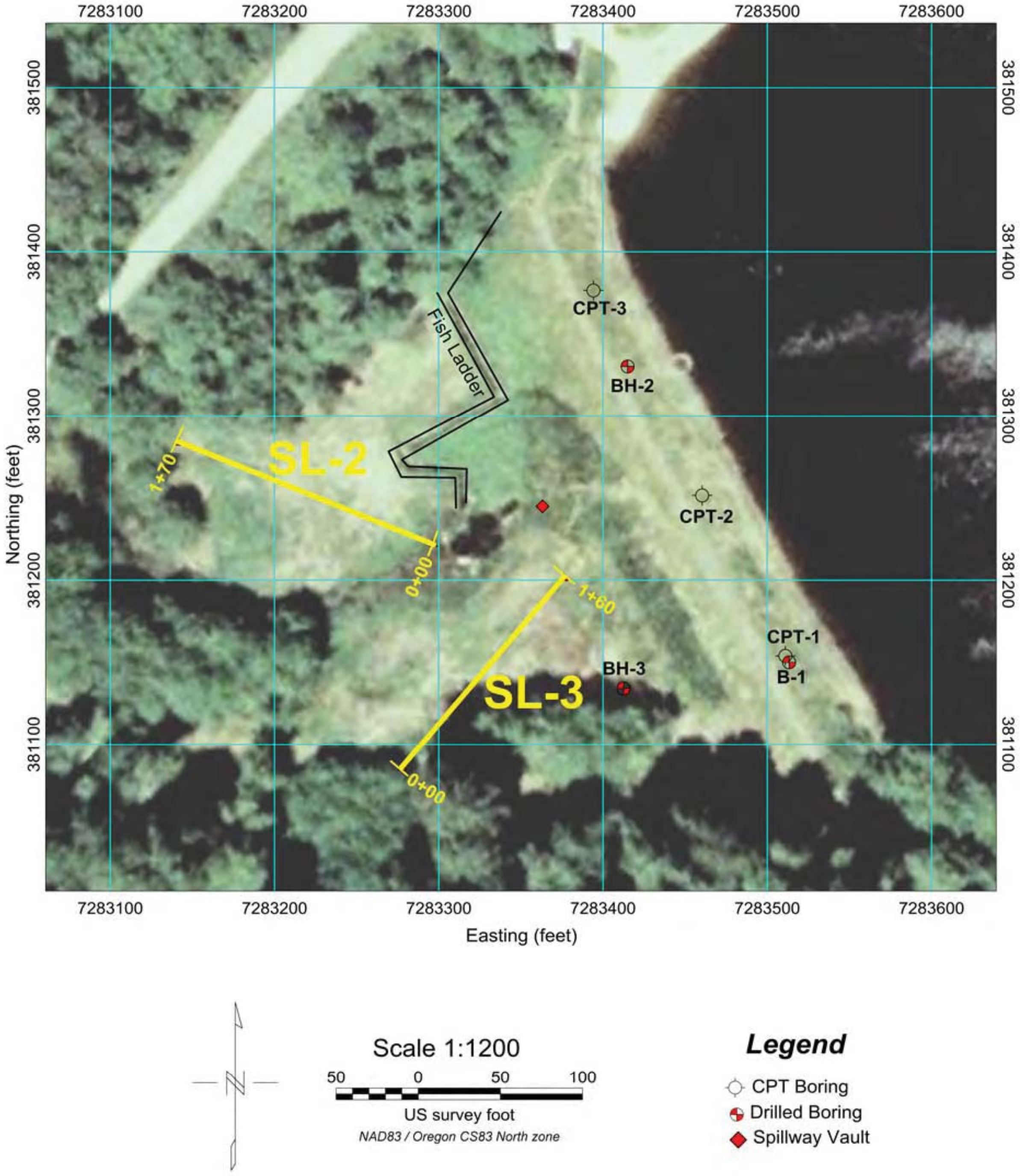


FIGURE 3

Prepared for:

HDR

Prepared by:

Northwest Geophysical Associates, Inc.

BIG CREEK #2
SITE EXPLORATION PLAN

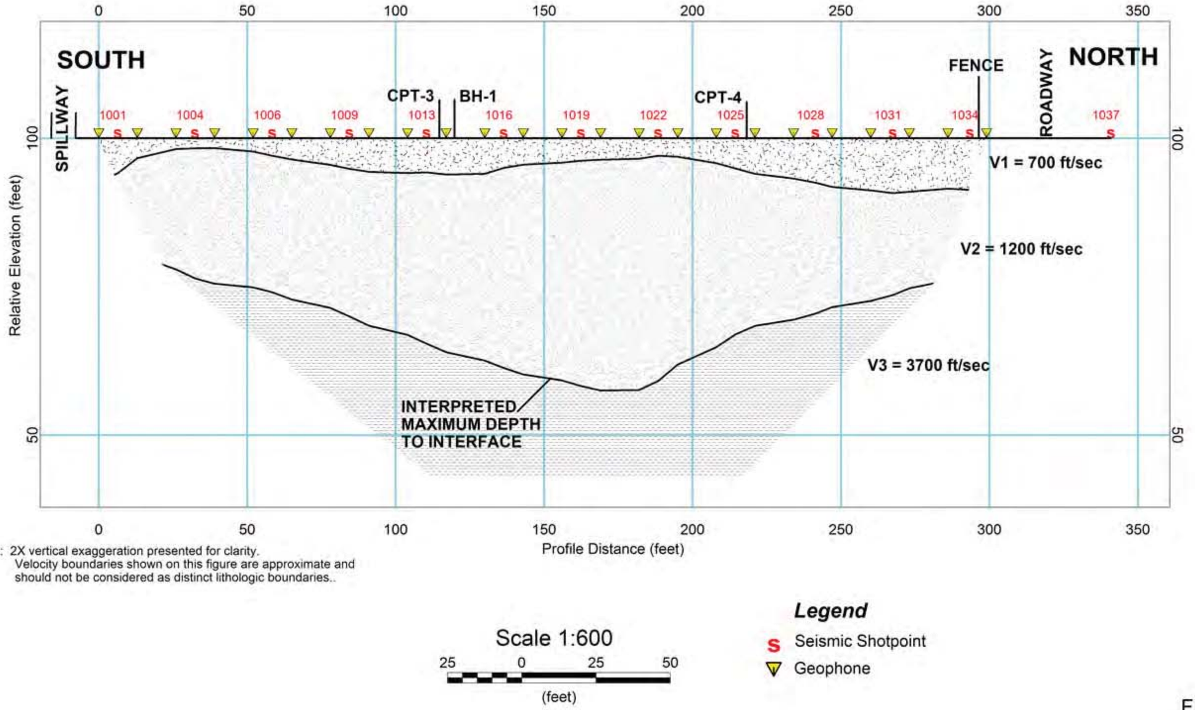
Seismic Refraction Survey
Big Creek Dams #1 and #2
Newport, Oregon

REVISION: B-2-APR-12

DATE: DECEMBER 2011

FILE: Big_Creek_1_profile.map

NGA PROJECT: 806



NOTE: 2X vertical exaggeration presented for clarity.
Velocity boundaries shown on this figure are approximate and should not be considered as distinct lithologic boundaries..

FIGURE 4

BIG CREEK #1
INTERPRETED SEISMIC PROFILE
SL-1

Seismic Refraction Survey
Big Creek Dams #1 and #2
Newport, Oregon

Prepared For:
HDR

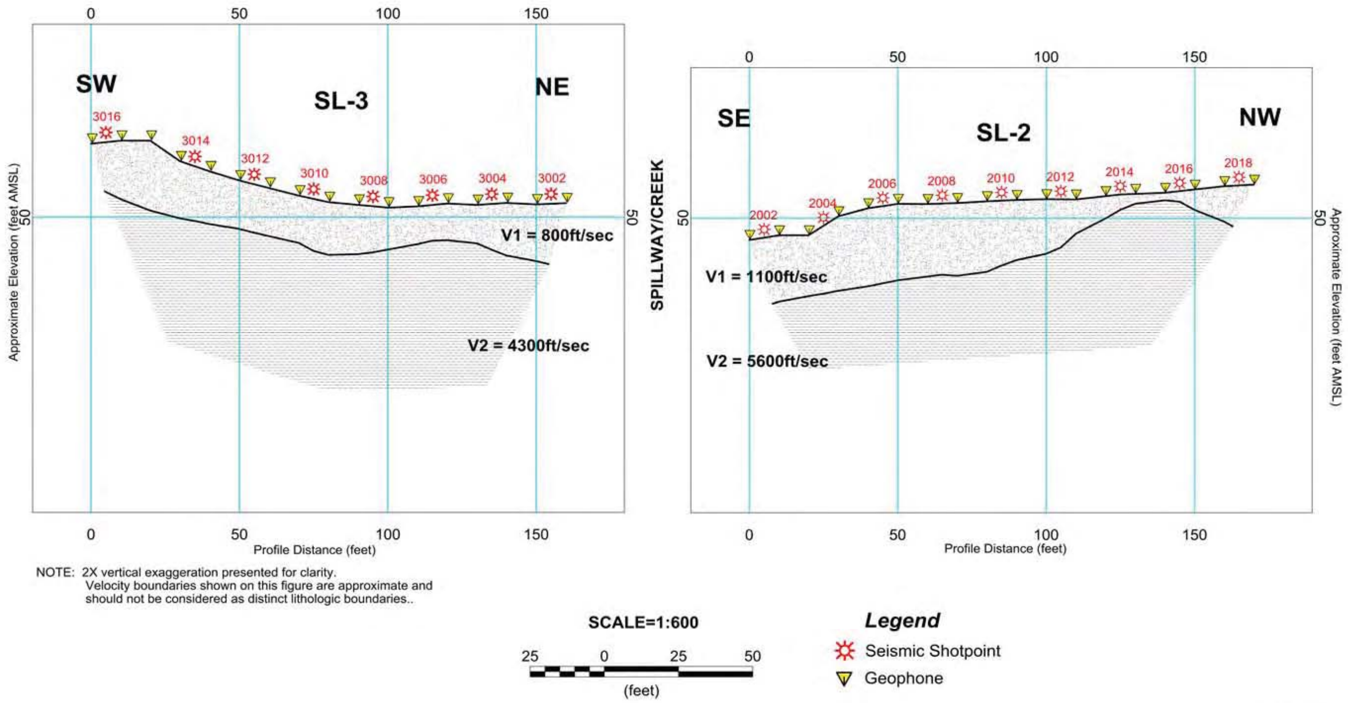
Prepared By:
NGA Northwest Geophysical Associates, Inc.

REVISION: A-10-JAN-12

DATE: DECEMBER 2011

FILE: Big_Creek_2_profile.map

NGA PROJECT: 806



NOTE: 2X vertical exaggeration presented for clarity.
Velocity boundaries shown on this figure are approximate and should not be considered as distinct lithologic boundaries..

FIGURE 5

BIG CREEK #2
INTERPRETED SEISMIC PROFILES
SL-2 & SL-3

Seismic Refraction Survey
Big Creek Dams #1 and #2
Newport, Oregon

Prepared For:
HDR

Prepared By:
NGA Northwest Geophysical Associates, Inc.

BETA

Consistent Accuracy . . .
. . . Delivered On-time

Beta Analytic Inc.
4985 SW 74 Court
Miami, Florida 33155 USA
Tel: 305 667 5167
Fax: 305 663 0964
Beta@radiocarbon.com
www.radiocarbon.com

Darden Hood
President

Ronald Hatfield
Christopher Patrick
Deputy Directors

June 27, 2012

Mr. Nick Clark
HDR Engineering, Inc.
1001 SW 5th Avenue
Suite 1800
Portland, OR 97204
USA

RE: Radiocarbon Dating Result For Sample NEWPORT WTP

Dear Mr. Clark:

Enclosed is the radiocarbon dating result for one sample recently sent to us. It provided plenty of carbon for an accurate measurement and the analysis proceeded normally. The report sheet contains the method used, material type, and applied pretreatments and, where applicable, the two-sigma calendar calibration range.

This report has been both mailed and sent electronically. All results (excluding some inappropriate material types) which are less than about 20,000 years BP and more than about ~250 BP include a calendar calibration page (also digitally available in Windows metafile (.wmf) format upon request). Calibration is calculated using the newest (2004) calibration database with references quoted on the bottom of the page. Multiple probability ranges may appear in some cases, due to short-term variations in the atmospheric 14C contents at certain time periods. Examining the calibration graph will help you understand this phenomenon. Don’t hesitate to contact us if you have questions about calibration.

We analyzed this sample on a sole priority basis. No students or intern researchers who would necessarily be distracted with other obligations and priorities were used in the analysis. We analyzed it with the combined attention of our entire professional staff.

Information pages are also enclosed with the mailed copy of this report. If you have any specific questions about the analysis, please do not hesitate to contact us. Someone is always available to answer your questions.

The cost of the analysis was charged to the VISA card provided. A receipt is enclosed with the mailed report copy. Thank you. As always, if you have any questions or would like to discuss the results, don’t hesitate to contact me.

Sincerely,

Darden Hood
Digital signature on file

BETA ANALYTIC INC.
DR. M.A. TAMERS and MR. D.G. HOOD

4985 S.W. 74 COURT
MIAMI, FLORIDA, USA 33155
PH: 305-667-5167 FAX:305-663-0964
beta@radiocarbon.com

REPORT OF RADIOCARBON DATING ANALYSES

Mr. Nick Clark — Report Date: 6/27/2012

HDR Engineering, Inc. — Material Received: 6/20/2012

Sample Data	Measured Radiocarbon Age	13C/12C Ratio	Conventional Radiocarbon Age(*)
Beta - 324359	4100 +/- 30 BP	-25.7 o/oo	4090 +/- 30 BP

SAMPLE : NEWPORT WTP
ANALYSIS : AMS-Standard delivery
MATERIAL/PRETREATMENT : (wood): acid/alkali/acid
2 SIGMA CALIBRATION : Cal BC 2860 to 2810 (Cal BP 4810 to 4760) AND Cal BC 2750 to 2720 (Cal BP 4700 to 4670)
Cal BC 2700 to 2570 (Cal BP 4650 to 4520) AND Cal BC 2510 to 2500 (Cal BP 4460 to 4450)

Dates are reported as RCYBP (radiocarbon years before present, "present" = AD 1950). By international convention, the modern reference standard was 95% the 14C activity of the National Institute of Standards and Technology (NIST) Oxalic Acid (SRM 4990C) and calculated using the Libby 14C half-life (5568 years). Quoted errors represent 1 relative standard deviation statistics (68% probability) counting errors based on the combined measurements of the sample, background, and modern reference standards. Measured 13C/12C ratios (delta 13C) were calculated relative to the PDB-1 standard.

The Conventional Radiocarbon Age represents the Measured Radiocarbon Age corrected for isotopic fractionation, calculated using the delta 13C. On rare occasion where the Conventional Radiocarbon Age was calculated using an assumed delta 13C, the ratio and the Conventional Radiocarbon Age will be followed by "*". The Conventional Radiocarbon Age is not calendar calibrated. When available, the Calendar Calibrated result is calculated from the Conventional Radiocarbon Age and is listed as the "Two Sigma Calibrated Result" for each sample.

CALIBRATION OF RADIOCARBON AGE TO CALENDAR YEARS

(Variables: C13/C12=-25.7:lab. mult=1)

Laboratory number: **Beta-324359**

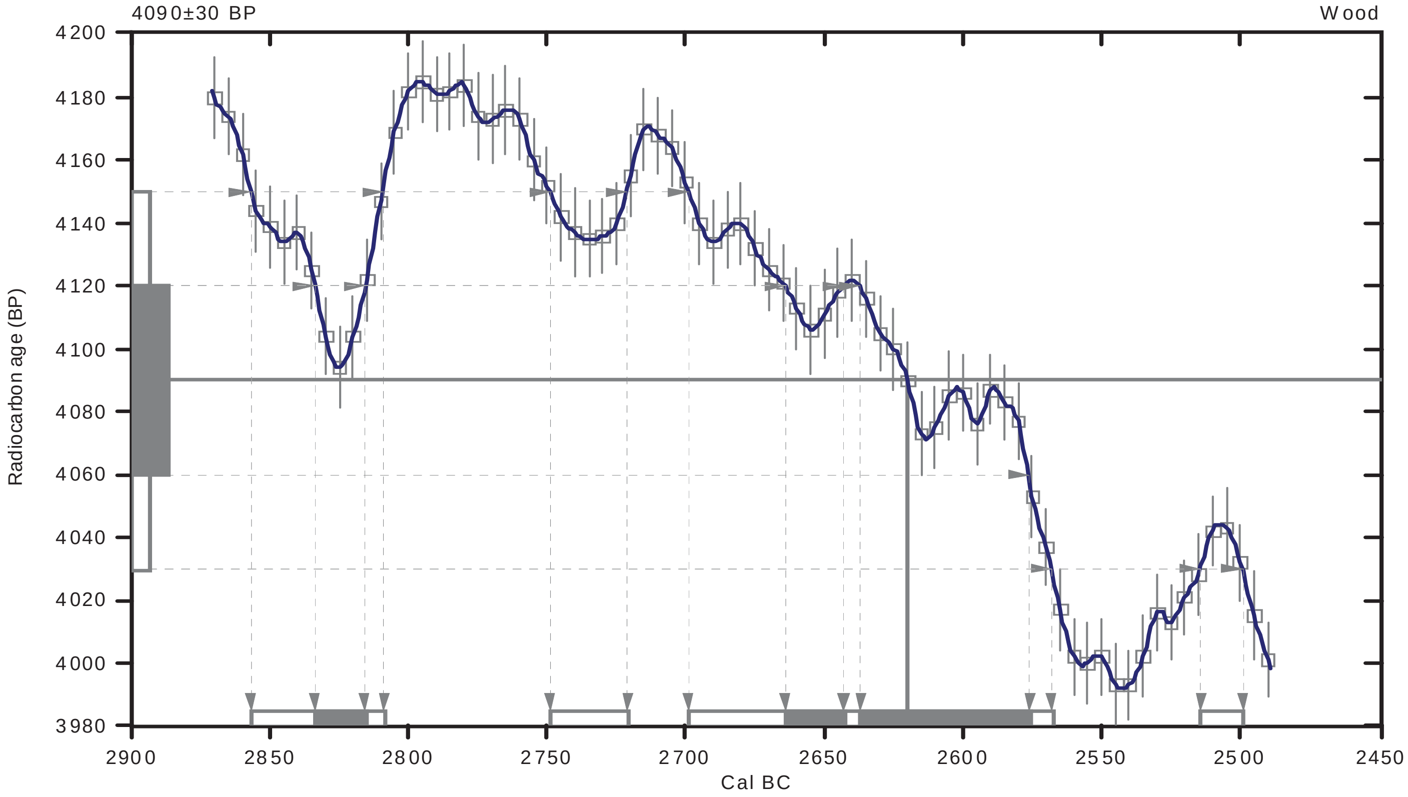
Conventional radiocarbon age: **4090±30 BP**

2 Sigma calibrated results: **Cal BC 2860 to 2810 (Cal BP 4810 to 4760) and**
(95% probability) **Cal BC 2750 to 2720 (Cal BP 4700 to 4670) and**
Cal BC 2700 to 2570 (Cal BP 4650 to 4520) and
Cal BC 2510 to 2500 (Cal BP 4460 to 4450)

Intercept data

Intercept of radiocarbon age with calibration curve: Cal BC 2620 (Cal BP 4570)

1 Sigma calibrated results: Cal BC 2830 to 2820 (Cal BP 4780 to 4770) and
(68% probability) Cal BC 2660 to 2640 (Cal BP 4610 to 4590) and
Cal BC 2640 to 2580 (Cal BP 4590 to 4530)



References:

Database used
INTCAL09
References to INTCAL09 database
Heaton,et.al.,2009, Radiocarbon 51(4):1151-1164, Reimer,et.al, 2009, Radiocarbon 51(4):1111-1150, Stuiver,et.al,1993, Radiocarbon 35(1):137-189, Oeschger,et.al.,1975,Tellus 27:168-192
Mathematics used for calibration scenario
A Simplified Approach to Calibrating C14 Dates
Talma, A. S., Vogel, J. C., 1993, Radiocarbon 35(2):317-322

Table C.1-1 Soil Samples and Laboratory Test Data for BC No. 1

Sample	Depth (ft)	USCS	LL	PL	PI	Moisture Content (%)	Liquidity Index	MC/LL	Dry Density (pcf)	Unconfined Compression (psi)	Wet Density (pcf)	Undrained Shear Strength (ksf)	Fines (%)	Sand (%)
SH-1-1A	5-7					60.3			64.4		103			
SS-1-2	10-11.5					44.0								
SS-1-3	15-16.5	MH	61	41	20	65.0	1.20	1.07						
SS-1-4	20-21.5													
SH-1-5	21.5-23.5	SM	66	38	28	56.1	0.65	0.85	66.1	12.07		0.87	48.1	51.9
SH-1-6	23.5-25													
SS-1-7	27.5-29	ML	49	25	14	59.6	2.47	1.22					61.8	38.2
SH-1-8a	31.7	MH	76	47	29									
SH-1-8a	31.85	MH	121	53	68	105	0.76	0.87	43.0		88	1.03		
SS-1-9	32-33.5	MH	87	46	41	75.1	0.71	0.86						
SS-1-10	35-37													
SS-1-11	37-39													
SH-1-12	40-42		82	47	35									
SS-1-13	42-43.5													

Sample	Depth (ft)	USCS	LL	PL	PI	Moisture Content (%)	Liquidity Index	MC/LL	Dry Density (pcf)	Unconfined Compression (psi)	Wet Density (pcf)	Undrained Shear Strength (ksf)	Fines (%)	Sand (%)
SS-1-14	45-46.5	MH	54	40	14	57.5	1.25	1.06					53	47
SS-1-15	47.5-49													
SS-1-16	50		68	36	32									
SS-1-17	52-53.5	MH	51	45	6	55.8	1.80	1.09					53.4	46.6
SS-1-18	55-56.5	SM											39.6	60.4
SS-1-19	57.5-59	SM	42	34	8	69.5								
SH-1-20	60-62	SM	51	29	22	52.4	1.06	1.03	67.0		102		37.5	62.5
SS-1-21	62-63.5	SM	0		0									
SS-1-22	65-66.5	SM	45	42	3	60.2	6.07	1.34					26.1	73.9
SS-1-23	67.5-69	SM	45	38	7	61.8							32.8	67.2
SS-1-24	70-71.5	SM	41	38	3	81.9	14.63	2.00					21.7	78.3
SS-1-25	72-73.5	SM	57	29	28	78.1	1.75	1.37					28.1	71.9
SS-1-26	75-76.5	SM				59.4							39.6	60.4
SS-1-27	77.5-1.5	CH	53	29	24	39.5	0.44	0.75					51.8	48.2
SS-1-28	80-81.5	SM	46	39	7	51.2	1.74	1.11					29.7	70.3
SS-1-29	85-86.5													

Sample	Depth (ft)	USCS	LL	PL	PI	Moisture Content (%)	Liquidity Index	MC/LL	Dry Density (pcf)	Unconfined Compression (psi)	Wet Density (pcf)	Undrained Shear Strength (ksf)	Fines (%)	Sand (%)
Sieve Analysis														
Sample	Depth (ft)	#8 (%)	No. 10 (%)		No. 16 (%)	#30 (%)			#40 (%)	#50 (%)			No. 100 (%)	No. 200 (%)
SS-1-18	55-56.5	100	99		99	97.0			96.0	93			80	39.6

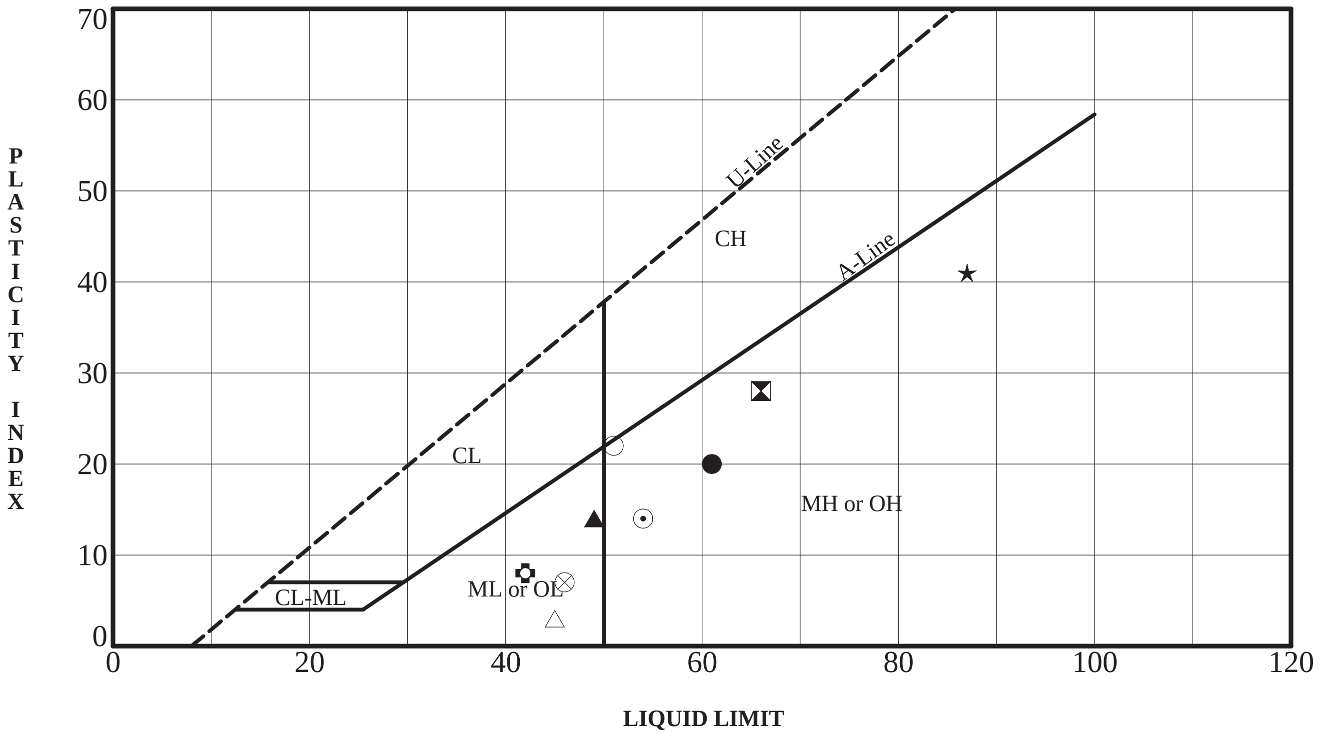
Table C.1-2 Soil Samples and Laboratory Test Data for BC No. 2

Sample	Depth (ft)	USCS	LL	PL	PI	Moisture (%)	Liquidity Index	MC/LL	Dry Density (pcf)	Lab qu (psi)	Wet Density (pcf)	Undrained Shear Strength (ksf)	Fines (%)	Sand (%)
SS-1-1	5-6.5													
SS-1-2	10-11.5	MH	56	38	18	43.9	0.33	0.78						
SS-1-3	15-16.5													
SS-1-4	20-21.5	MH	50	39	11	36.1	-0.26	0.72					51.5	48.5
SS-1-5	25-26.5													
SS-1-6	30-31.5	MH	53	43	10	40.9	-0.21	0.77						
SS-1-7	35-36.5													
SS-1-8	40-41.5													
SS-1-9	45-46.5	MH	56	42	14	44.2	0.16	0.79					72.6	27.4
SS-1-10	50-51.5													
SS-1-11	55-56.5													
SS-1-12	57.5-59													
SH-1-13	60-62	MH	53	37	16	46.2	0.58	0.87	72.6		106		53.1	46.9
SS-1-14	23.5-25													
SS-1-15	65-66.5													

Sample	Depth (ft)	USCS	LL	PL	PI	Moisture (%)	Liquidity Index	MC/LL	Dry Density (pcf)	Lab qu (psi)	Wet Density (pcf)	Undrained Shear Strength (ksf)	Fines (%)	Sand (%)
SS-1-16	67.5-69													
SS-1-17	70-71.5	MH	50	38	12	34.8	-0.27	0.70						
SS-2-1	5-6.5													
SS-2-2	10-11.5	SM	52	45	7	41.4	-0.51	0.80					44.7	55.3
SS-2-3	15-16.5													
SS-2-4	20-21.5													
SS-2-5	25-26.5	MH	55	38	17	41.2	0.19	0.75						
SS-2-6	30-31.5													
SS-2-7	35-36.5													
SS-2-8	40-41.5	SM	52	34	21	58.4	1.16	1.12					36.5	63.5
SH-2-9	42.5-44.5													
SS-2-10	44.5-46													
SS-2-11	47.5-49	OH	50	26	24	42.1	0.67	0.84					50.5	49.5
SS-2-12	50-51.5													
SS-2-13	55-56.5	CH	54	25	29	33.1	0.28	0.61					58.6	41.4

Sample	Depth (ft)	USCS	LL	PL	PI	Moisture (%)	Liquidity Index	MC/LL	Dry Density (pcf)	Lab qu (psi)	Wet Density (pcf)	Undrained Shear Strength (ksf)	Fines (%)	Sand (%)
SS-2-14	57.5-59													
SS-2-15	60-61.5													
SS-2-16	65-66.5	SM	43	33	10	31.7	-0.13	0.74					21.5	78.5
SS-2-17	70-71.5													
SS-3-1	2.5-4	ML	48	34	14									
SS-3-2	5-6.5													
SS-3-3A	7.5-9	SM	50	38	12	49.9	0.99	1.00	71.1	9.37	107	0.67	48.0	52
SS-3-4	10-11.5													
SS-3-5	12.5-14	MH	52	41	11	104.8	5.80	2.02					64.3	35.7
SS-3-6	15-16.5	SM	49	45	4	57.1	3.03	1.17					39.1	60.9
SH-3-7	17.5-19.5	SM				56.4			62.5		98		36.5	63.5
SS-3-8	20-21.5													
SS-3-9	22.5-24	SM	52	33	19	49.4							31.5	68.5
SS-3-10	25-26.5													
SS-3-11	27.5-29													

Sample	Depth (ft)	USCS	LL	PL	PI	Moisture (%)	Liquidity Index	MC/LL	Dry Density (pcf)	Lab qu (psi)	Wet Density (pcf)	Undrained Shear Strength (ksf)	Fines (%)	Sand (%)
SS-3-12	30-31.5	SM				54.8			74.5		115		31.8	68.2
SS-3-12	30-31.5	SM	51	34	17	41.2	0.42	0.81					28.6	71.4
SS-3-13	32.5-34													
SS-3-14	35-36.5													
SS-3-15	40-41.5													



Key Symbol	Boring No.	Depth (Feet)	Liquid Limit	Plasticity Index	Liquidity Index	Water Content (%)	% Passing #200 Sieve	USCS
●	BH-1	15.0	61	20	---	---		
⧗	BH-1	21.5	66	28	---	---		
▲	BH-1	27.5	49	14	---	---		
★	BH-1	32.0	87	41	---	---		
⊙	BH-1	45.0	54	14	---	---		
✿	BH-1	57.5	42	8	---	---		
○	BH-1	60.0	51	22	---	---		
△	BH-1	65.0	45	3	---	---		
⊗	BH-1	80.0	46	7	---	---		

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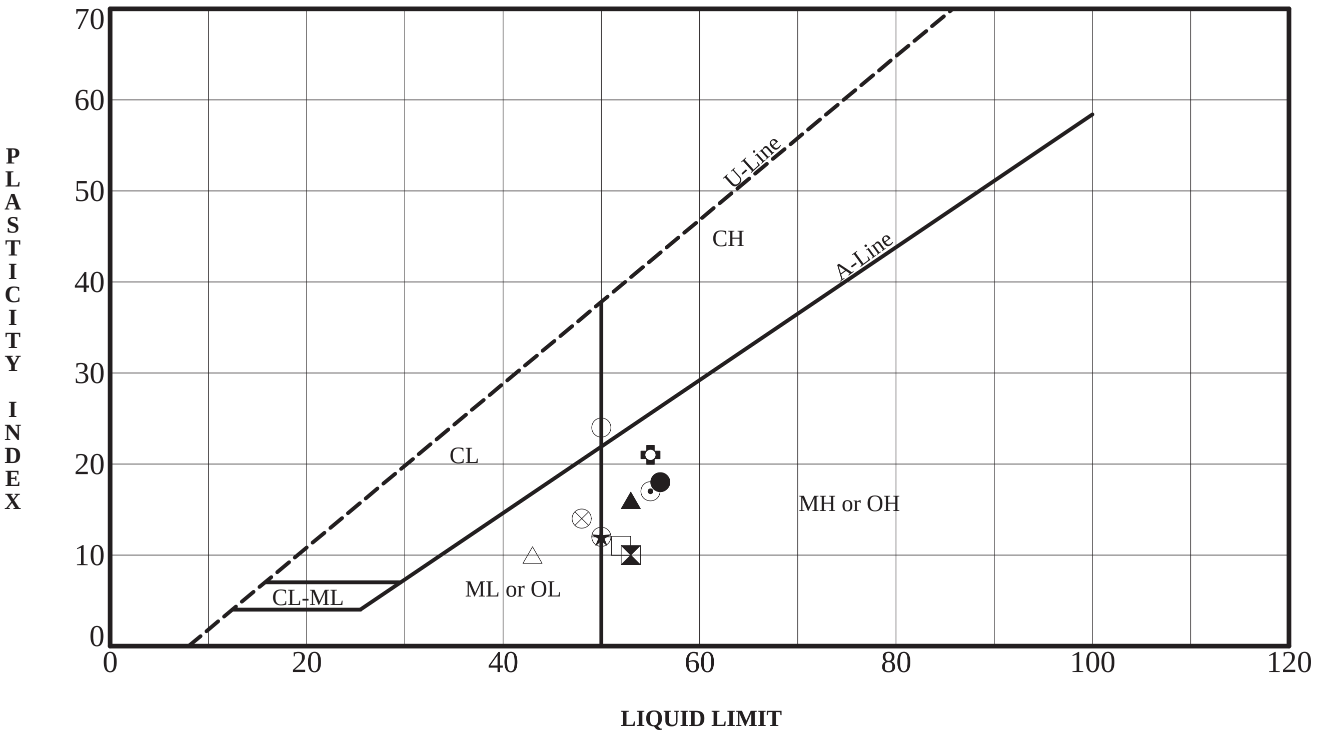
PLASTICITY CHART AND DATA

BC No. 1
Big Creek Dams #1 and #2
Dam Seismic Evaluation

Date
July 2012

Figure
C.1-1



Key Symbol	Boring No.	Depth (Feet)	Liquid Limit	Plasticity Index	Liquidity Index	Water Content (%)	% Passing #200 Sieve	USCS
●	BH-1	10.0	56	18	---	---		
⧗	BH-1	30.0	53	10	---	---		
▲	BH-1	60.0	53	16	---	---		
★	BH-1	70.0	50	12	---	---		
⊙	BH-2	25.0	55	17	---	---		
✤	BH-2	40.0	55	21	---	---		
○	BH-2	47.5	50	24	---	---		
△	BH-2	65.0	43	10	---	---		
⊗	BH-3	2.5	48	14	---	---		
⊕	BH-3	7.5	50	12	---	---		
□	BH-3	12.5	52	11	---	---		

HDR

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PLASTICITY CHART AND DATA

BC No. 2
Big Creek Dams #1 and #2
Dam Seismic Evaluation

Date
July 2012

Figure
C.1-2

ATT NEWPORT WTP - BIG CREEK _2.GPJ FOLSOM 3-30-11.GDT 7/25/12

Northwest Geotech, Inc.

9120 SW Pioneer Court, Suite B • Wilsonville, Oregon 97070 503/682-1880 FAX: 503 / 682-2753

TECHNICAL REPORT

Report To:
Mr. Nick Clark, P.E.
HDR Engineering, Inc.
1001 SW 5th Avenue, Suite 1800
Portland, Oregon 97204

Date: 1/3/12

Lab No: 11-308

Project: Laboratory Testing – Newport WTP

Project No.: 2179.1.1

Report of: Atterberg limits, moisture content, moisture density, and amount of material passing the No. 200 sieve

---

Sample Identification

NGI completed Atterberg limits, moisture content, moisture density, and amount of material passing the No. 200 sieve on soil samples for the subject project. The samples were delivered by a HDR Engineering, Inc. representative on December 23, 2011. Testing was performed in accordance with the standards indicated. Our laboratory test results are summarized on the following pages.

Copies: Addressee

Attachments: Laboratory Test Results

This report shall not be reproduced except in full, without written approval of Northwest Geotech, Inc.

REVIEWED BY: Bridgett Adame

Northwest Geotech, Inc.

9120 SW Pioneer Court, Suite B • Wilsonville, Oregon 97070 503/682-1880 FAX: 503 / 682-2753

TECHNICAL REPORT

Report To: Mr. Nick Clark, P.E.
HDR Engineering, Inc.
1001 SW 5th Avenue, Suite 1800
Portland, Oregon 97204

Date: 1/3/12

Lab No: 11-308

Project: Laboratory Testing – Newport WTP

Project No.: 2179.1.1

Laboratory Testing – Big Creek 1

Atterberg Limits (ASTM D 4318)			
Sample ID	**Liquid Limit**	**Plastic Limit**	**Plasticity Index**
SS-1-3 @ 15 – 16.5 ft.	61	41	20
SS-1-9 @ 32 – 33.5 ft.	87	46	41

Amount of Material Finer than the No. 200 Sieve (ASTM D1140)		
Sample ID	**Moisture Content (%)**	**Percent Passing the No. 200 Sieve**
SS-1-7 @ 27.5 – 29 ft.	59.6	61.8
SS-1-14 @ 45 – 46.5 ft.	57.5	53.0
SS-1-23 @ 67.5 – 69 ft.	61.8	32.8

Moisture Content of Soil and Dry Density of Soil (ASTM D 2216/D2937)		
Sample ID	**Moisture Content (Percent)**	**Dry Density (pcf)**
SS-1-9 @ 32 – 33.5 ft.	75.1	--
SH-1-1A @ 5 – 7 ft.	60.3	64.4

Sieve Analysis of Aggregate – SS-1-18 @ 55 – 56.5 ft. (ASTM C136/ C117)	
Sieve Size	**Percent Passing**
#8	100
#10	99
#16	99
#30	97
#40	96
#50	93
#100	80
#200	39.6

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REVIEWED BY: Bridgett Adame

TECHNICAL REPORT

Report To:
Mr. Nick Clark, P.E.
HDR Engineering, Inc.
1001 SW 5th Avenue, Suite 1800
Portland, Oregon 97204

Date: 1/3/12

Lab No: 11-308

Project: Laboratory Testing – Newport WTP

Project No.: 2179.1.1

Laboratory Testing – Big Creek 2

Atterberg Limits
(ASTM D 4318)

Sample ID	Liquid Limit	Plastic Limit	Plasticity Index
SS-1-2 @ 10 – 11.5 ft.	56	38	18
SS-1-6 @ 30 – 31.5 ft.	53	43	10
SH-1-13 @ 60 – 62 ft.	53	37	16
SS-1-17 @ 70 – 71.5 ft.	50	38	12
SS-2-5 @ 25 – 26.5 ft.	55	38	17
SS-2-8 @ 40 – 41.5 ft.	52	31	21
SS-2-11 @ 47.5 – 49 ft.	50	26	24
SS-2-16 @ 65 – 66.5 ft.	43	33	10

Amount of Material Finer than the No. 200 Sieve
(ASTM D1140)

Sample ID	Moisture Content (%)	Percent Passing the No. 200 Sieve
SH-1-13 @ 60 – 62 ft.	47.9	53.1
SS-2-8 @ 40 – 41.5 ft.	58.4	36.5
SS-2-11 @ 47.5 – 49 ft.	42.1	50.5
SS-2-16 @ 65 – 66.5 ft.	31.7	21.5

Moisture Content of Soil and Dry Density of Soil
(ASTM D 2216/D2937)

Sample ID	Moisture Content (Percent)	Dry Density (pcf)
SS-1-6 @ 30 – 31.5 ft.	40.9	--
SH-1-13 @ 60 – 62 ft.	46.2	72.6
SS-2-5 @ 25 – 26.5 ft.	41.2	--
SS-2-11 @ 47.5 – 49 ft.	42.1	--

Northwest Geotech, Inc.

9120 SW Pioneer Court, Suite B • Wilsonville, Oregon 97070 503/682-1880 FAX: 503 / 682-2753

TECHNICAL REPORT

Report To: Mr. Nick Clark, P.E.
HDR Engineering, Inc.
1001 SW 5th Avenue, Suite 1800
Portland, Oregon 97204

Date: 3/15/12

Lab No: 11-308

Project: Laboratory Testing – Newport WTP

Project No.: 2179.1.1

Report of: Moisture content of soil

Sample Identification

NGI completed moisture content testing on soil samples for the subject project. The samples were delivered by a HDR Engineering, Inc. representative on December 23, 2011. Testing was performed in accordance with the standards indicated. Our laboratory test results are summarized on the following table.

Laboratory Testing

Moisture Content of Soil (ASTM D 2216)	
Sample ID	**Moisture Content (Percent)**
SS1-2 @ 10 – 11.5 ft.	43.9
SS1-17 @ 70 – 71.5 ft.	34.8
SS1-3 @ 15 – 16.5 ft.	65.0

Copies: Addressee

REVIEWED BY: Bridgett Adame

Northwest Geotech, Inc.

9120 SW Pioneer Court, Suite B • Wilsonville, Oregon 97070 503/682-1880 FAX: 503 / 682-2753

TECHNICAL REPORT

Report To:	Mr. Nick Clark, P.E. HDR Engineering, Inc. 1001 SW 5th Avenue, Suite 1800 Portland, Oregon 97204	**Date:** **Lab No:**	1/19/12 12-001
Project:	Laboratory Testing – Newport WTP	**Project No.:**	2179.1.1

Report of: Atterberg limits, moisture content, moisture density, amount of material passing the No. 200 sieve, and unconfined compression of soils

---

Sample Identification

NGI completed Atterberg limits, moisture content, moisture density, amount of material passing the No. 200 sieve, and unconfined compression testing on soil samples for the subject project. The samples were delivered by a HDR Engineering, Inc. representative on January 10, 2012. Testing was performed in accordance with the standards indicated. Our laboratory test results are summarized on the following pages.

Copies: Addressee

Attachments: Laboratory Test Results

This report shall not be reproduced except in full, without written approval of Northwest Geotech, Inc.

SHEET 1 of 5 REVIEWED BY: Bridgett Adame

TECHNICAL REPORT

Report To: Mr. Nick Clark, P.E.
HDR Engineering, Inc.
1001 SW 5th Avenue, Suite 1800
Portland, Oregon 97204

Date: 1/19/12

Lab No: 12-001

Project: Laboratory Testing – Newport WTP

Project No.: 2179.1.1

Laboratory Testing – Big Creek 1

Atterberg Limits
(ASTM D 4318)

Sample ID	Liquid Limit	Plastic Limit	Plasticity Index
SH-1-5 @ 21.5 – 23.5 ft.	66	38	28
SS-1-7 @ 27.5 – 29.0 ft.	49	35	14
SH-1-20 @ 60 – 62 ft.	51	29	22
SS-1-22 @ 65 – 66.5 ft.	45	42	3
SS-1-14 @ 45 – 46.5 ft.	54	40	14
SS-1-19 @ 57.5 – 59 ft.	42	34	8
SS-1-21 @ 62 – 63.5 ft.	NP	NP	NP
SS-1-28 @ 80 – 81.5 ft.	46	39	7

Amount of Material Finer than the No. 200 Sieve
(ASTM D1140)

Sample ID	Moisture Content (%)	Percent Passing the No. 200 Sieve
SH-1-5 @ 21.5 – 23.5 ft.	53.6	48.1
SH-1-20 @ 60 – 62 ft.	48.4	37.5
SS-1-22 @ 65 – 66.5 ft.	60.2	26.1
SS-1-26 @ 75 – 76.5 ft.	59.4	39.6
SS-1-28 @ 80 – 81.5 ft.	51.2	29.7

Moisture Content of Soil and Dry Density of Soil
(ASTM D 2216/D2937)

Sample ID	Moisture Content (Percent)	Dry Density (pcf)
SH-1-5 @ 21.5 – 23.5 ft.	57.6	65.6
SH-1-20 @ 60 – 62 ft.	52.4	67.0

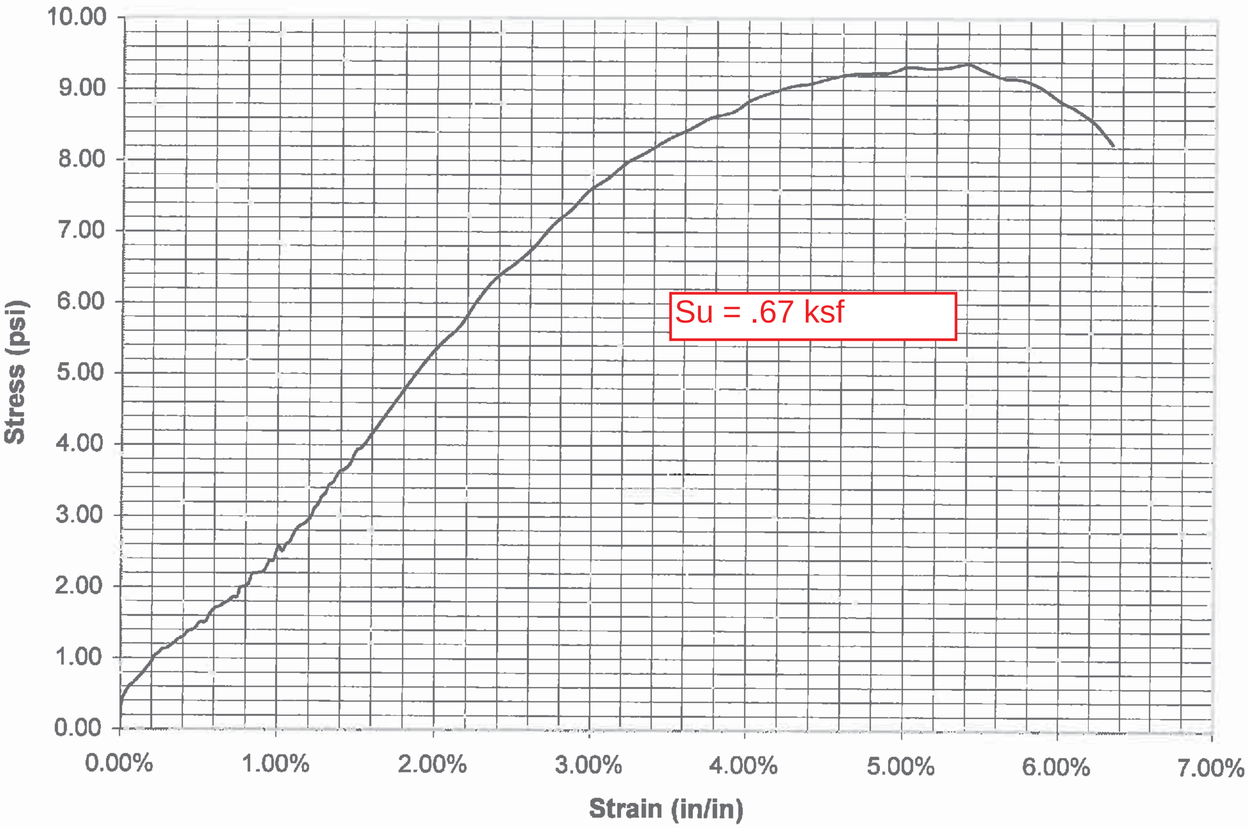
Northwest Geotech, Inc.
9120 SW Pioneer Court, Suite B • Wilsonville, Oregon 97070 503/682-1880 FAX: 503 / 682-2753

TECHNICAL REPORT

Report To:	Mr. Nick Clark, P.E. HDR Engineering, Inc. 1001 SW 5th Avenue, Suite 1800 Portland, Oregon 97204	**Date:** **Lab No:**	1/19/12 12-001
Project:	Laboratory Testing – Newport WTP	**Project No.:**	2179.1.1

Laboratory Testing – Big Creek 1

Unconfined Compressive Strength of Cohesive Soil (ASTM D2166)					
Sample ID	**Moisture Content (percent)**	**Initial Diameter (inches)**	**Initial Height (inches)**	**Unit Weight (pcf)**	**Unconfined Compressive Strength (psi)**
SS-3-3A @ 7.5 – 9 ft.	49.9	2.850	6.020	71.0	9.37



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SHEET 3 of 5 REVIEWED BY: Bridgett Adame

Northwest Geotech, Inc.

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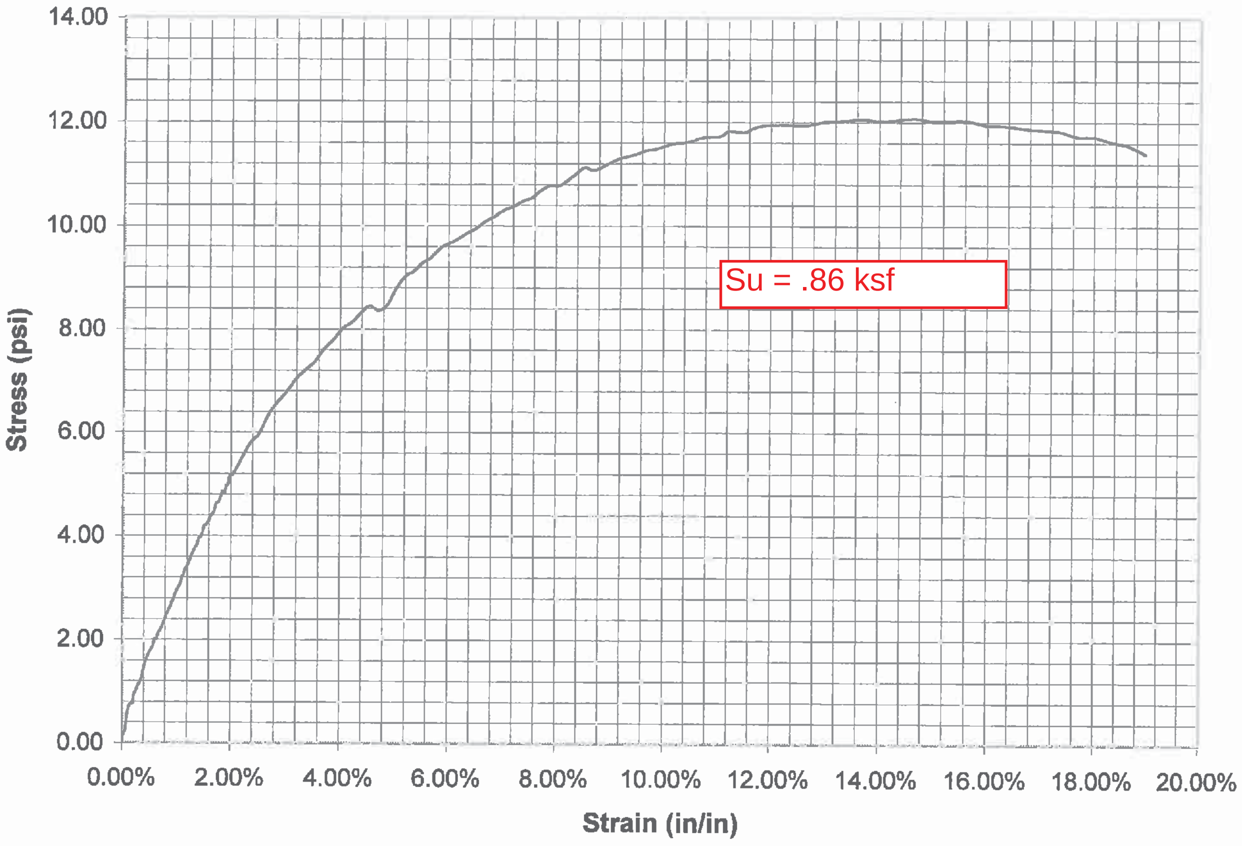
TECHNICAL REPORT

Report To:	Mr. Nick Clark, P.E. HDR Engineering, Inc. 1001 SW 5th Avenue, Suite 1800 Portland, Oregon 97204	**Date:** **Lab No:**	1/19/12 12-001
Project:	Laboratory Testing – Newport WTP	**Project No.:**	2179.1.1

Laboratory Testing – Big Creek 1

Unconfined Compressive Strength of Cohesive Soil (ASTM D2166)

Sample ID	Moisture Content (percent)	Initial Diameter (inches)	Initial Height (inches)	Unit Weight (pcf)	Unconfined Compressive Strength (psi)
SH-1-5 @ 21.5 – 23.5	56.1	2.839	5.949	66.1	12.07



Su = .86 ksf

Northwest Geotech, Inc.

9120 SW Pioneer Court, Suite B • Wilsonville, Oregon 97070 503/682-1880 FAX: 503 / 682-2753

TECHNICAL REPORT

Report To: Mr. Nick Clark, P.E.
HDR Engineering, Inc.
1001 SW 5th Avenue, Suite 1800
Portland, Oregon 97204

Date: 1/19/12

Lab No: 12-001

Project: Laboratory Testing – Newport WTP

Project No.: 2179.1.1

Laboratory Testing – Big Creek 2

Atterberg Limits (ASTM D 4318)

Sample ID	Liquid Limit	Plastic Limit	Plasticity Index
SS-3-1 @ 2.5 – 4 ft.	48	34	14
SS-3-3A @ 7.5 – 9 ft.	50	38	12
SS-3-5 @ 12.5 – 14 ft.	52	41	11

Amount of Material Finer than the No. 200 Sieve (ASTM D1140)

Sample ID	Moisture Content (%)	Percent Passing the No. 200 Sieve
SS-3-3A @ 7.5 – 9 ft.	52.1	48.0
SS-3-5 @ 12.5 – 14 ft.	104.8	64.3
SH-3-7 @ 17.5 – 19.5 ft.	51.1	36.5
SS-3-9 @ 22.5 – 24 ft.	49.4	31.5
SH-3-11 @ 27.5 – 29.5 ft.	54.8	31.8

Moisture Content of Soil and Dry Density of Soil (ASTM D 2216/D2937)

Sample ID	Moisture Content (Percent)	Dry Density (pcf)
SS-3-3A @ 7.5 – 9 ft.	49.6	71.1
SH-3-7 @ 17.5 – 19.5 ft.	56.4	62.5
SH-3-11 @ 27.29.5 ft.	48.5	74.5

REVIEWED BY: Bridgett Adame

Northwest Geotech, Inc.

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TECHNICAL REPORT

Report To:	Mr. Barry Meyer, P.E. HDR Engineering, Inc. 5426 Bay Center Drive, Suite 400 Tampa, FL 33609	**Date:** **Lab No:**	5/4/12 12-050
Project:	Laboratory Testing – Newport WTP	**Project No.:**	2179.1.1

Report of: Atterberg limits and moisture content of soils

Sample Identification

NGI completed Atterberg limits and moisture content testing on soil samples for the subject project. The samples were delivered by a HDR Engineering, Inc. representative. Testing was performed in accordance with the standards indicated. Our laboratory test results are summarized on the following tables.

Laboratory Testing

Atterberg Limits (ASTM D 4318)			
Sample ID	**Liquid Limit**	**Plastic Limit**	**Plasticity Index**
SS-3-9 @ 22.5 – 24 ft. Big Creek # 2	52	33	19
SS-1-23 @ 67.5 – 69 ft. Big Creek #1	45	38	7
BH-1 SH-1-8 @ 31.7 ft.	76	47	29
BH-1 SH-1-12 @ 40 ft.	82	47	35
BH-1 SH-1-16 @ 50 ft.	68	36	32

Moisture Content of Soil (ASTM D 2216)	
Sample ID	**Moisture Content (Percent)**
SS-1-19 @ 57.5 – 59 ft. Big Creek #1	69.5

Copies: Addressee

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TECHNICAL REPORT

Report To: Mr. Richard Hannan, P.E., R.P.G., C.E.G.
HDR Engineering, Inc.
1001 SW 5th Avenue, Suite 1800
Portland, Oregon 97204

Date: 5/16/12

Lab No: 12-062

Project: Laboratory Testing – Newport WTP

Project No.: 2179.1.1

Report of: Atterberg limits and the amount of material finer than the No. 200 sieve

---

Sample Identification

NGI completed Atterberg limits and the amount of material finer than the No. 200 sieve on soil samples for the subject project. The samples were delivered by a HDR Engineering, Inc. representative. Testing was performed in accordance with the standards indicated. Our laboratory test results are summarized on the following tables.

Attachments: Laboratory Test Results

Copies: Addressee

REVIEWED BY: Bridgett Adame

TECHNICAL REPORT

Report To: Mr. Richard Hannan, P.E., R.P.G., C.E.G.
HDR Engineering, Inc.
1001 SW 5th Avenue, Suite 1800
Portland, Oregon 97204

Date: 5/16/12

Lab No: 12-062

Project: Laboratory Testing – Newport WTP

Project No.: 2179.1.1

Laboratory Testing

Atterberg Limits (ASTM D 4318)

Sample ID	Liquid Limit	Plastic Limit	Plasticity Index
BH-1 SS-1-17 @ 52 – 53.5 ft. Big Creek No.1	51	45	6
BH-1 SS-1-24 @ 70 – 71.5 ft. Big Creek No.1	41	38	3
BH-1 SS-1-25 @ 72 – 73.5 ft. Big Creek No.1	57	29	28
BH-1 SS-1-27 @ 77.5 ft. Big Creek No.1	53	29	24
BH-1 SS-1-4 @ 20 – 21.5 ft. Big Creek No.2	50	39	11
BH-1 SS-1-9 @ 45 – 46.5 ft. Big Creek No.2	56	42	14
BH-2 SS-2-2 @ 10 – 11.5 ft. Big Creek No.2	52	45	7
BH-2 SS-2-13 @ 55 – 56.5 ft. Big Creek No.2	54	25	29
BH-3 SS-3-6 @ 15 – 16.5 ft. Big Creek No.2	49	45	4
BH-3 SS-3-12 @ 30 – 31.5 ft. Big Creek No.2	51	34	17

Amount of Material Finer than the No. 200 Sieve (ASTM D1140)

Sample ID	Moisture Content (%)	Percent Passing the No. 200 Sieve
BH-1 SS-1-17 @ 52 – 53.5 ft. Big Creek No.1	55.8	53.4
BH-1 SS-1-24 @ 70 – 71.5 ft. Big Creek No.1	81.9	21.7
BH-1 SS-1-25 @ 72 – 73.5 ft. Big Creek No.1	78.1	28.1
BH-1 SS-1-27 @ 77.5 ft. Big Creek No.1	39.5	51.8
BH-1 SS-1-4 @ 20 – 21.5 ft. Big Creek No.2	36.1	51.5
BH-1 SS-1-9 @ 45 – 46.5 ft. Big Creek No.2	44.2	72.6
BH-2 SS-2-2 @ 10 – 11.5 ft. Big Creek No.2	41.4	44.7
BH-2 SS-2-13 @ 55 – 56.5 ft. Big Creek No.2	33.1	58.6
BH-3 SS-3-6 @ 15 – 16.5 ft. Big Creek No.2	57.1	39.1
BH-3 SS-3-12 @ 30 – 31.5 ft. Big Creek No.2	41.2	28.6

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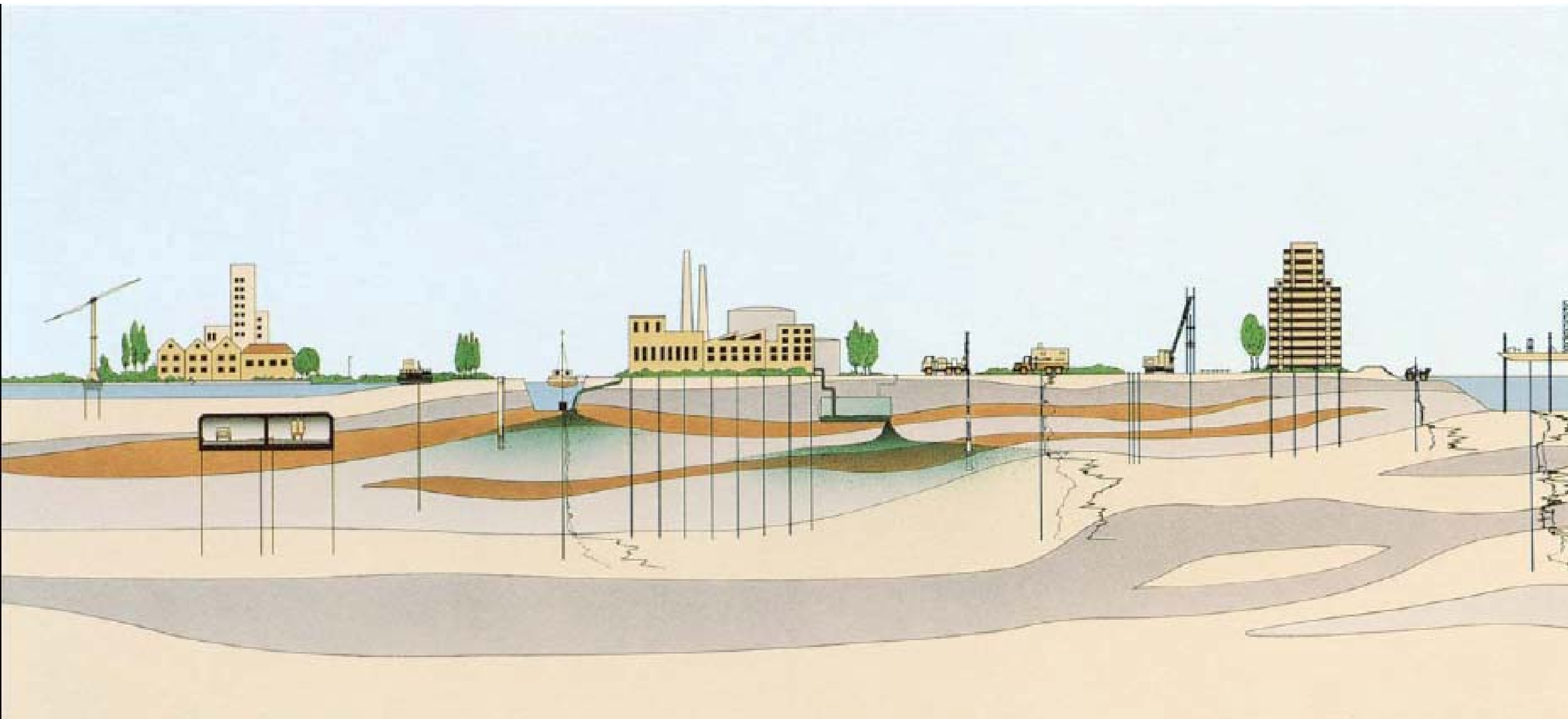
SHEET 2 of 2 REVIEWED BY: Bridgett Adame

FUGRO CONSULTANTS, INC.

LABORATORY DATA REPORT
GEOTECHNICAL PROPERTIES TESTING PROGRAM
Big Creek Dam #1 and Big Creek Dam #2
City of Newport, Oregon

HDR ENGINEERING, INC.
TAMPA, FLORIDA



FUGRO CONSULTANTS, INC.

6100 Hillcroft, Tech Building
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www.fugroconsultants.com

Report No. 04.11110060
June 7, 2012

HDR Engineering, Inc.
5426 Bat Center Drive, Suite 400
Tampa, FL 33609

Attention: Mr. Barry Meyer, PE

Laboratory Data Report

Geotechnical Properties Testing Program

Big Creek Dam #1 and Big Creek Dam #2

City of Newport, Oregon

The Houston Geotechnical Testing Laboratory of Fugro Consultants, Inc. is pleased to present the results of this geotechnical properties testing program for the project referenced above. This report contains a summary of the procedures and results of the tests performed on soil samples provided by HDR Engineering, Inc from March 5, 2012 to May 30, 2012.

We appreciate this opportunity to be of continued service to HDR Engineering, Inc. We look forward to working with you on future projects.

Sincerely,

Fugro Consultants, Inc.

Maurice N. Morvant

Maurice N. Morvant
Assistant Manager, Geotechnical Laboratory

CONTENTS

SECTION 1

INTRODUCTION

General

This report presents the results of the geotechnical testing program performed by the Houston Geotechnical Laboratory (HGL) of Fugro Consultants, Inc. for HDR Engineering, Inc. (HDR) for the Big Creek Dam #1 and Big Creek Dam #2 Project for the City of Newport, Oregon. Mr. Barry Meyer, P.E. of HDR arranged for the samples to be shipped to HGL. HGL received the samples on January 18, 2012. Mr. Meyer also provided the lab test assignments. All tests were performed in general accordance with the appropriate ASTM standards. No interpretation of the test results is made in this report and no soil design parameters have been selected since these tasks are beyond the scope of work.

Report Format:

This report is organized into seven (7) sections. This section provides introductory information, describes the scope of the testing program, and highlights specific test procedures. Section 2 presents X-ray radiograph images of the samples. Index and strength property test results are summarized in Section 3. Sections 4 through 7 present the results and address key points relevant to individual test types.

Purpose and Scope of Testing Program

The purpose of this laboratory testing program is to determine physical and engineering properties of selected samples at depths ranging from ~30 ft to ~50 ft. Purposes of the individual test types are to measure:

- Index properties of selected samples
- Compressibility characteristics of selected samples
- Static shear strength and stress-strain characteristics of selected samples under various stress conditions
- Dynamic properties of selected samples under stress-controlled loading conditions

The following number and type of index and engineering property tests were performed as part of the scope of work:

- Two (2) moisture content tests
- Three (3) liquid limit (LL) and plastic limit (PL) tests
- Two (2) unconsolidated-undrained triaxial tests

- One (1) one-dimensional consolidation test, with incremental loading increments, one unload-reload cycle and one rebound stage from the maximum applied stress increment
- Two (2) direct simple shear tests
- One (1) cyclic simple shear test with a post-cyclic simple shear test

Test Procedures

Tests were performed in accordance with the appropriate ASTM International (ASTM) standards. The following sections summarize essential procedural items.

X-ray Radiography – The sample tubes were X-rayed to facilitate and enhance the sample/specimen selection and processing, as described below. The X-ray procedure used followed that given in ASTM Test Method D4452-06, Standard Practice for X-ray Radiography of Soil Samples, except an Iridium 192 source was used instead of a conventional X-ray tube.

X-ray radiography provides a qualitative measure of the content of the sample, as displayed by the varying shades of gray resulting from variations in density of the soil sample. These shades of gray enable the evaluation of:

- sample quality as noted by signs of voids, fractures, unusual changes in bedding planes or layering, etc.;
- the presence of inclusions in the sample, such as shells and/or calcareous nodules; and
- the presence of naturally occurring fissures, bedding planes, voids, layering, gravel, and silts seams.

The X-ray radiographs were used to:

- identify anomalies that might affect the test results,
- select specimens from the samples for testing.

Specimens were identified for testing by Mr. Meyers. The selected portions of the tubes were then cut into segments with a mechanical hacksaw (18 teeth per inch).

X-ray radiograph images are presented in Section 2.

Index Tests – Water content, liquid and plastic limits, sieve/hydrometer analysis, and specific gravity tests were performed in general accordance with ASTM test methods, as summarized below. The index property tests were performed on trimmings obtained during the preparation of the engineering property test specimen or on an adjacent soil specimen.

The ASTM test methods used in performing these index or physical property tests are listed below, along with any applicable comments.

- *Water Content* – ASTM Test Method D2216-10, Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass
- *Grain Size Analysis* – ASTM Test Method D422-63 (2007), Particle-Size Analysis of Soils
- *Liquid and Plastic Limits* – ASTM Test Method D4318-10, Liquid Limit, Plastic Limit, and Plasticity Index of Soils, Method A

The results of the index properties tests are presented in Section 3.

Unconsolidated-Undrained Triaxial Compression Tests – Unconsolidated-undrained triaxial compression tests were performed in accordance with ASTM Test Method D2850-03a (2007), Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils. The tests were performed using a single specimen from each sample with effective confining pressures ranging from 15 to 19 psi.

Each specimen had a diameter of ~2.8 in. and was trimmed to a height of ~6.0 in. and mounted into the triaxial testing apparatus with a membrane encasing the specimen. The desired confining pressure was then applied, and after a waiting period of about 10 minutes, the specimen was sheared in compression. The chamber pressure was kept constant and specimen drainage was not permitted during the shear stage. The axial loading piston was advanced into the cell at a specific rate-of-strain (1%/min).

During testing, the necessary data (time, axial force, axial deformation, and transducers excitation voltage) were recorded using an automated data-acquisition system. Microsoft® Excel worksheets, along with a Visual Basic code, were used to reduce the data files into engineering units in tabular and graphical format.

The results of the unconsolidated-undrained triaxial tests are presented in Section 4.

Incremental Consolidation Tests – The one-dimensional consolidation test was performed in general accordance with ASTM Test Method D2435/D2435M-11, One-Dimensional Consolidation Properties of Soils Using Incremental Loading, Test Method B. The test had an unload-reload stress cycle (about one log cycle of change in stress) and unloading from the maximum applied stress increment. The duration of the loading increments was determined using Taylor's square root of time fitting method. Loading continued until at least 25% strain was reached.

The test specimen had a diameter of ~2.5 in. and height of ~0.75 in. Deformation data was recorded/plotted using an automated data-acquisition system and was corrected for the deformation of the apparatus, stones, and filter paper.

The results of the incremental consolidation test is presented in Section 5.

Static Direct Simple Shear Tests - Each strain-controlled, undrained static direct simple shear (SDSS) test was performed in general accordance with ASTM Test Method D6528-07, Consolidated Undrained Direct Simple Shear Testing of Cohesive Soils.

The test specimens had a diameter of about 2.60 in. (66.0 mm) and height of about 1.0 in. (25.4 mm). Drainage was allowed on the top and bottom boundaries during consolidation and shearing. The volumes of the test specimens were kept constant during shearing by keeping the specimen's height constant. As a result, undrained conditions (no volume change) were maintained during shearing. It can then be assumed then that the change in vertical stress is equivalent to the change in pore water pressure.

Each specimen was incrementally consolidated for loading to the target stress level, with the maximum and final effective-vertical stress ($\sigma'_{v,c.\ max}$ or σ'_{max} and $\sigma'_{v,c}$ or σ'_{shear}) maintained constant for about 24 hours (curing or simulated aging) or for a sufficient amount of time past the time to reach 90% consolidation (t_{90}). Upon completion of consolidation, we applied monotonic shear loading at a strain rate of 5% per hour. During shearing, the necessary data (time, vertical and horizontal forces, shear deformations, and transducer excitation voltage) were recorded using an automated data acquisition system. An Excel worksheet along with a Visual Basic program was used to process the raw data files.

The test data were not corrected for the effects of the rubber membrane and stack of steel rings. The uncorrected shear stress is slightly higher than the corrected value. Assuming the correction for the membrane and rings is comparable to that of a wire-reinforced rubber membrane, then the correction would be in the range of 0.17 to 0.38 Pascal (Pa) at a shear strain of about 10%. The lower correction is used by the Massachusetts Institute of Technology, and the higher value is used by the Norwegian Geotechnical Institute. Such small corrections are not warranted as both fall well within the normal scatter of soil shear strength measurements.

The results of the SDSS tests are presented in Section 6.

Cyclic Direct Simple Shear Tests - The stress-controlled, undrained cyclic direct simple shear (CyDSS) test was performed using an apparatus similar the device used for the SDSS test except the CyDSS device is heavier and has stiffening elements. The preparation of the trimmed specimen was the same as used for the SDSS tests. The mode of consolidation and pre-shear conditioning was also the same as the procedure used for the SDSS tests.

Upon completion of consolidation, the test specimen was loaded cyclically using an electro-hydraulic closed loop loading system manufactured by MTS Systems Corporation. Specimens were maintained in an undrained (no volume change) state during loading.

The MTS system was programmed to apply a sinusoidal cyclic shear stress at 1.0 Hz. cyclic loading continued until cyclic shear strain reached 4%. A post-cyclic SDSS was performed on the specimen without permitting the specimen to relieve excess pore pressure generated during the cyclic phase. This post-cyclic DSS consists of monotonically loading/shearing the specimen to failure (or shear strain exceeding 20%), as is done for the SDSS tests. The specimen was permitted to relieve excess pore pressure (recover) after the post-cyclic static test and the volume change was recorded.

The data collection system consisted of a National Instruments DAQPad-MIO-16XE-50 unit. This is a high-resolution, multifunction data acquisition (DAQ) unit that communicates through a parallel port with an IBM-compatible computer. The DAQPad-MIO-16XE-50 features a 16-bit analog-to-digital converter (ADC) with 16 single-ended or 8 differential inputs. The horizontal load, vertical load, horizontal displacement, and vertical displacement transducers were connected to the DAQ as differential input signals.

National Instruments Lab View software package was programmed to collect and present data from the CyDSS tests. The system collected 500 data points per channel (horizontal displacement, vertical load, etc.) for each loading cycle. The computer software averaged every 10 sequential readings together and recorded the average value (i.e., the system recorded 50 points per channel per cycle) in two separate files.

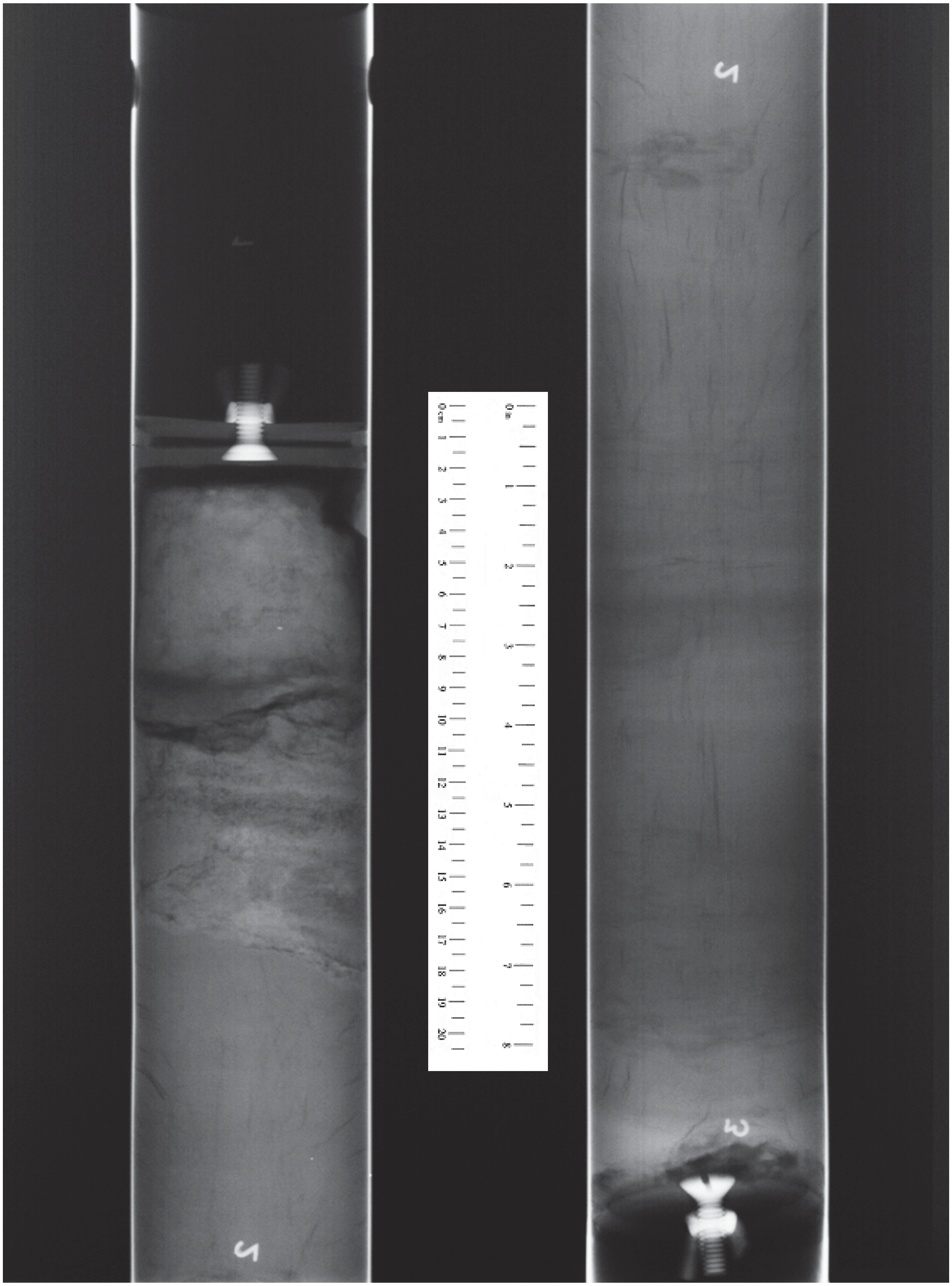
One file in ASCII format contained all the readings (horizontal load and displacement, and vertical load) and was downloaded while the test was conducted. The other file, in Microsoft Excel format, contained maximum and minimum horizontal load, horizontal displacement and vertical load, and average vertical load readings for each cycle. At the end of the test, the files were transferred to another computer for analysis. As with the SDSS tests, the shear strain for the cyclic and post-cyclic direct simple shear tests was also corrected for the recessed height in the top and bottom caps, if applicable.

The results of the CyDSS test, the post-cyclic static and the recovery are presented in Section 7.

SECTION 2

X-RAY RADIOGRAPHY

Photographs of the X-ray radiographs taken of the are presented on Plates 2-1 through 2-3.



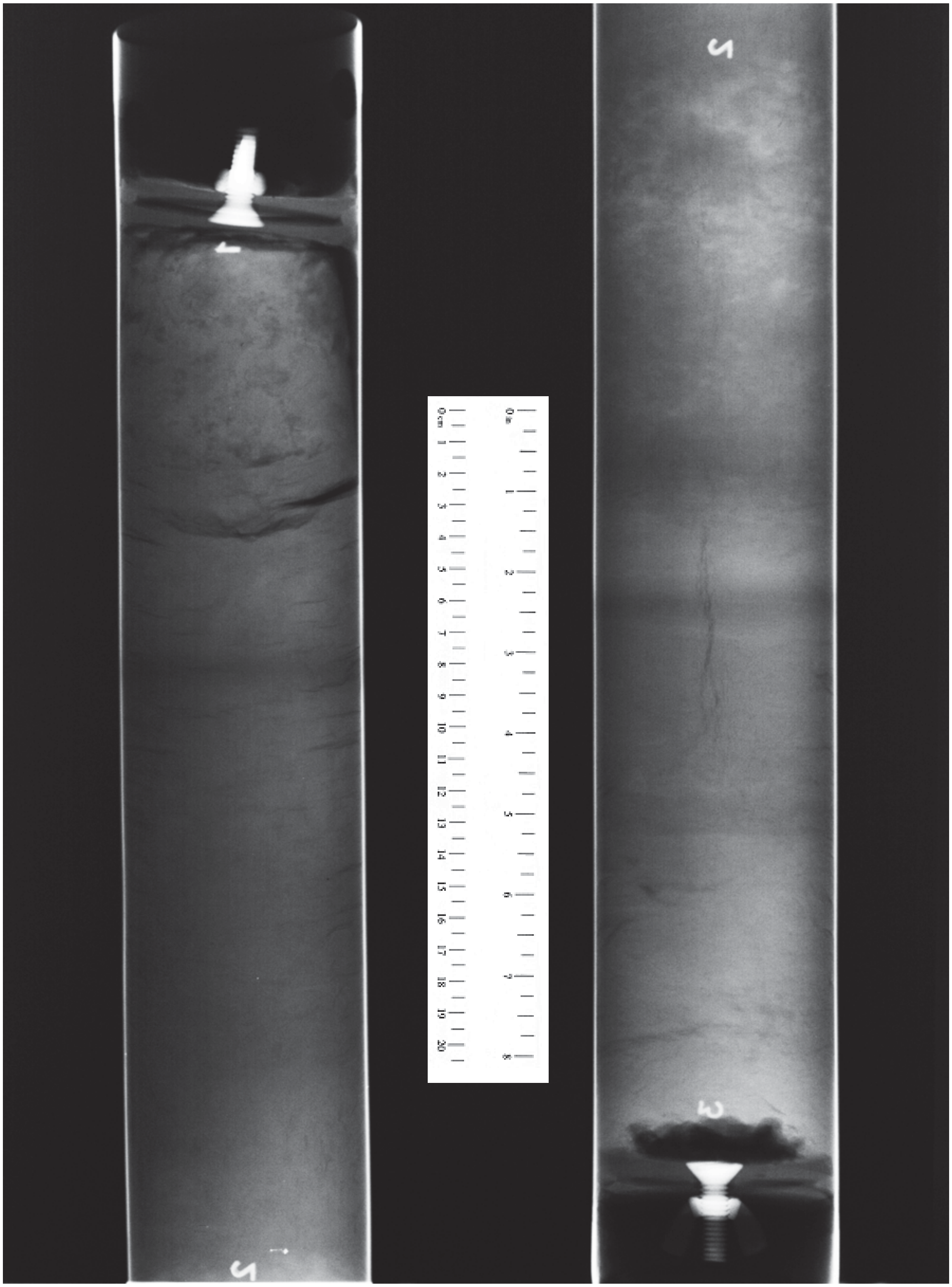
X-RAY RADIOGRAPH

Sample No. SH-1-8, Depth 30-32 ft

Boring: BH-1 BC#1

Big Creek Dam #1 and Big Creek Dam #2

PLATE 2-1



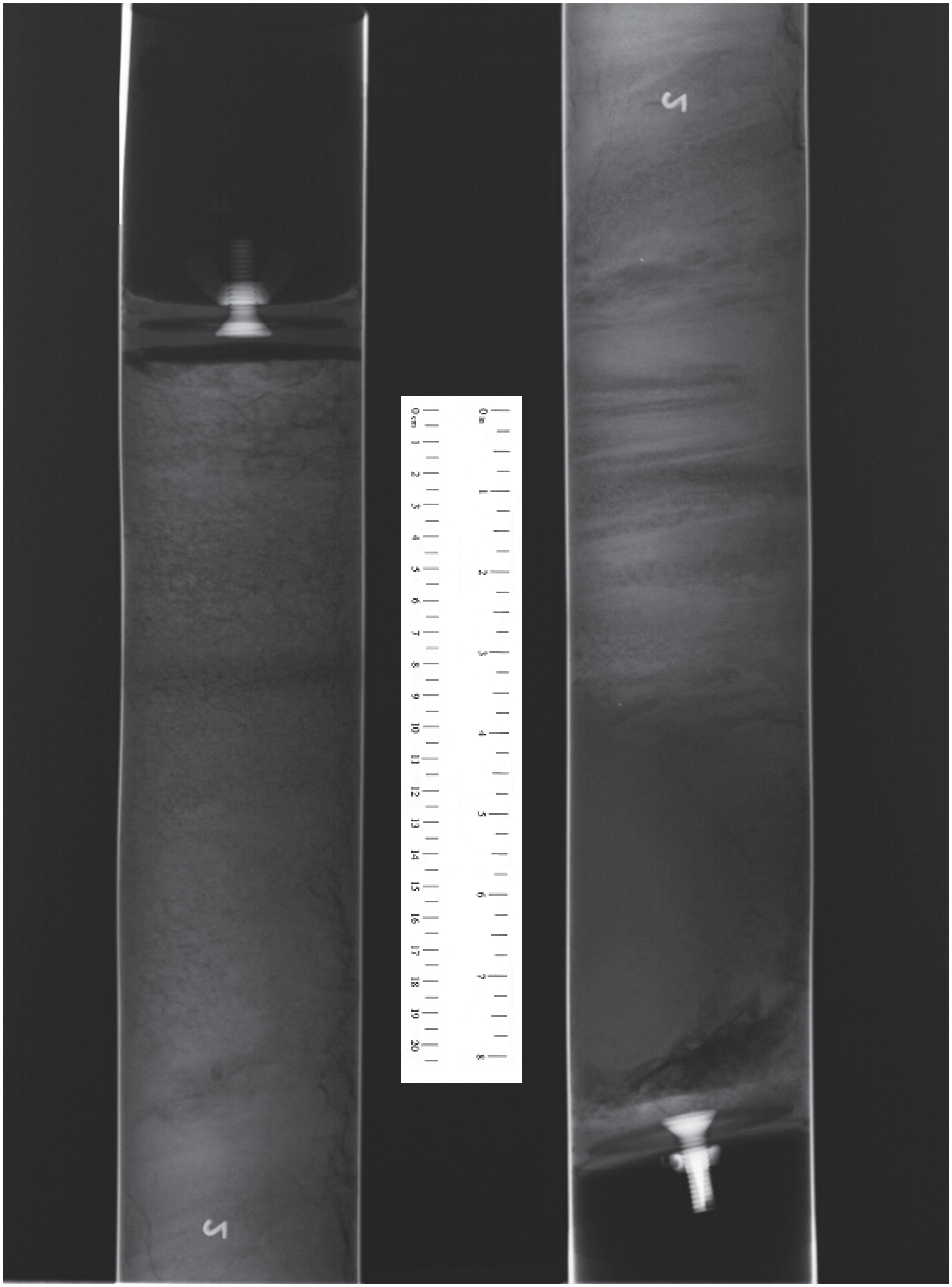
X-RAY RADIOGRAPH

Sample No. SH-1-12, Depth 40.0-42.0 ft

Boring: BH-1 BC#1

Big Creek Dam #1 and Big Creek Dam #2

PLATE 2-2



X-RAY RADIOGRAPH

Sample No. SH-1-16, Depth 50.0-52.0 ft

Boring: BH-1 BC#1

Big Creek Dam #1 and Big Creek Dam #2

PLATE 2-3

SECTION 3

SUMMARY OF INDEX AND STRENGTH PROPERTIES

This section of the report presents the results of index properties and grain size distribution tests of selected samples.

The table on Plate 3-1 presents the test results for moisture content, Atterberg limit, specific gravity, and unconsolidated-undrained triaxial test results. Grain size test results are provided on Plate 3-2.

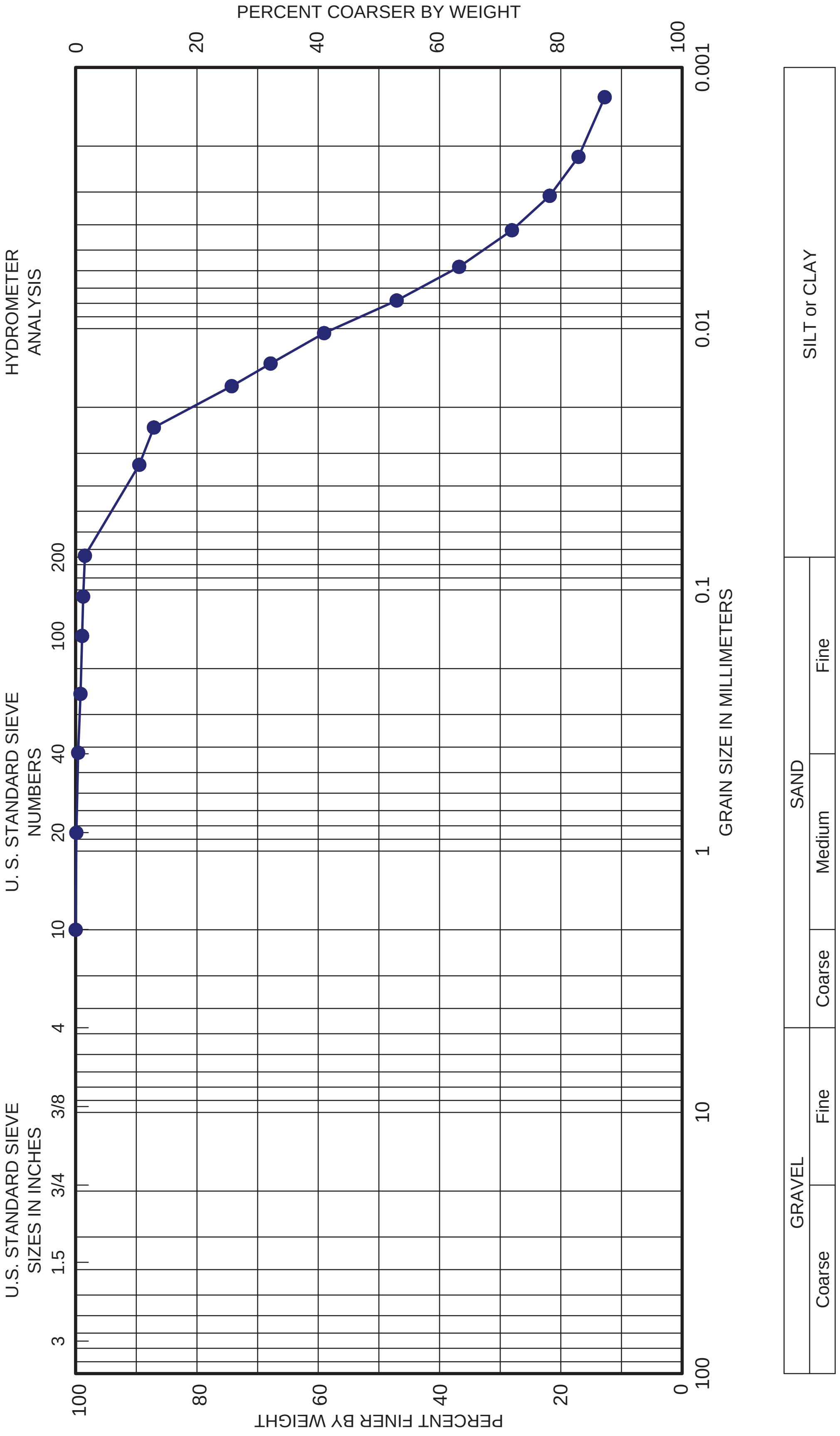
Boring No.	Sample No.	Bottom Depth of Specimen (ft)	Index or Physical Properties:										Peak Deviator Stress (ksf)	TEST TYPE
			Water Content (%)	Liquid Limit LL	Plasticity Index PI	Specific Gravity $G_s^{(1)}$	Total Unit Wt. (pcf)	Dry Unit Wt. (pcf)	Deg. of Sat. (%)	Passing No. 200 Sieve (%)	Finer Than 0.01 mm (%)	Finer Than 0.002 mm (%)		
BH-1 BC#1	SH-1-8d	30.25	61.6	- -	- -	2.72	102.7	63.6	100.5	- -	- -	- -	- -	Cyclic Simple Shear
	SH-1-8uu	31.05	79.4	- -	- -	2.72	95.3	53.1	98.6	- -	- -	- -	0.84	Uncon. Undrained Triaxial
	SH-1-8c	31.55	96.2	- -	- -	2.72	91.4	46.6	99.2	- -	- -	- -	- -	Incremental Consolidation
	SH-1-8b	31.70	91.8	- -	- -	2.72	94.3	49.2	102.0	- -	- -	- -	- -	Static Simple Shear
	SH-1-8a	31.85	107.4	121	68	2.72	88.3	42.6	98.0	- -	- -	- -	- -	Static Simple Shear
	SH-1-8	32.00	- -	- -	- -	- -	- -	- -	- -	98	59	17	- -	Grain Size
	SH-1-12	40.00	124.8	149	102	- -	- -	- -	- -	- -	- -	- -	- -	Moisture Content, Limits
	SH-1-12a	41.15	94.8	- -	- -	2.75	95.4	49.0	104.4	- -	- -	- -	0.80	Uncon. Undrained Triaxial
	BH-1-16	50.90	46.8	56	31	- -	- -	- -	- -	- -	- -	- -	- -	Moisure Content, Limits

Boring No.	Sample No.	Depth of Specimen (ft)	Specimen Visual Description	Consol. Stress $\sigma'_{v,c}$ (ksf)$^{(2)}$	Strain in Consol. $\varepsilon_{a,c}$ (%)	Maximum Shear Stress (ksf)	Strain at Maximum Stress (%)	Effective Vertical Stress at Max. Stress (ksf)	Cyclic Stress Ratio	Number Cycles to Failure	Plate No.
BH-1 BC#1	SH-1-8d	30.25	Silty clay, olive gray with few sand pockets	3.30	10.0	0.72$^{(3)}$	25.0$^{(3)}$	0.41$^{(3)}$	0.267	27	7-1 to 7
	SH-1-8uu	31.05	Silty Clay, gray with sand seams and ferrous stains	- -	- -	- -	- -	- -	- -	- -	4-1
	SH-1-8c	31.55	Silty Clay, dark brown	- -	- -	- -	- -	- -	- -	- -	5-1,2,3
	SH-1-8b	31.70	Silty Clay, olive gray with traces of organic matter	10.0	18.1	3.28	19.7	6.38	- -	- -	6-2,3
	SH-1-8a	31.85	Silty Clay, olive gray with traces of organic matter	2.53	9.8	1.02	18.8	1.70	- -	- -	6-1,3
	SH-1-8	32.00	Silty Clay, dark gray with organic matter	- -	- -	- -	- -	- -	- -	- -	3-2
	SH-1-12	40.00	Clay, greenish gray and gray with organic matter	- -	- -	- -	- -	- -	- -	- -	- -
	SH-1-12a	41.15	Silty Clay, tan and olive gray with shell fragments	- -	- -	- -	- -	- -	- -	- -	4-2
	BH-1-16	50.90	Sandy Clay, brown	- -	- -	- -	- -	- -	- -	- -	4-3

Notes: 1. Specific gravity values are assumed.
2. All static and cyclic simple shear tests specimens had an induced over consolidation ratio of 1.
3. Results are from post cyclic static test.

SUMMARY OF SELECTED INDEX OR PHYSICAL PROPERTIES
Boring BH-1 BC#1
Big Creek Dam #1 and Big Creek Dam #2

PLATE 3-1



SYMBOL	BORING	DEPTH, FT	C_c	C_u	D_{50}	D_{90}	CLASSIFICATION
●	BH-1 BC#1_SH-1-8a	32			0.01	0.03	Silty Clay, dark gray with organic matter

GRAIN SIZE CURVE

PLATE 3-2

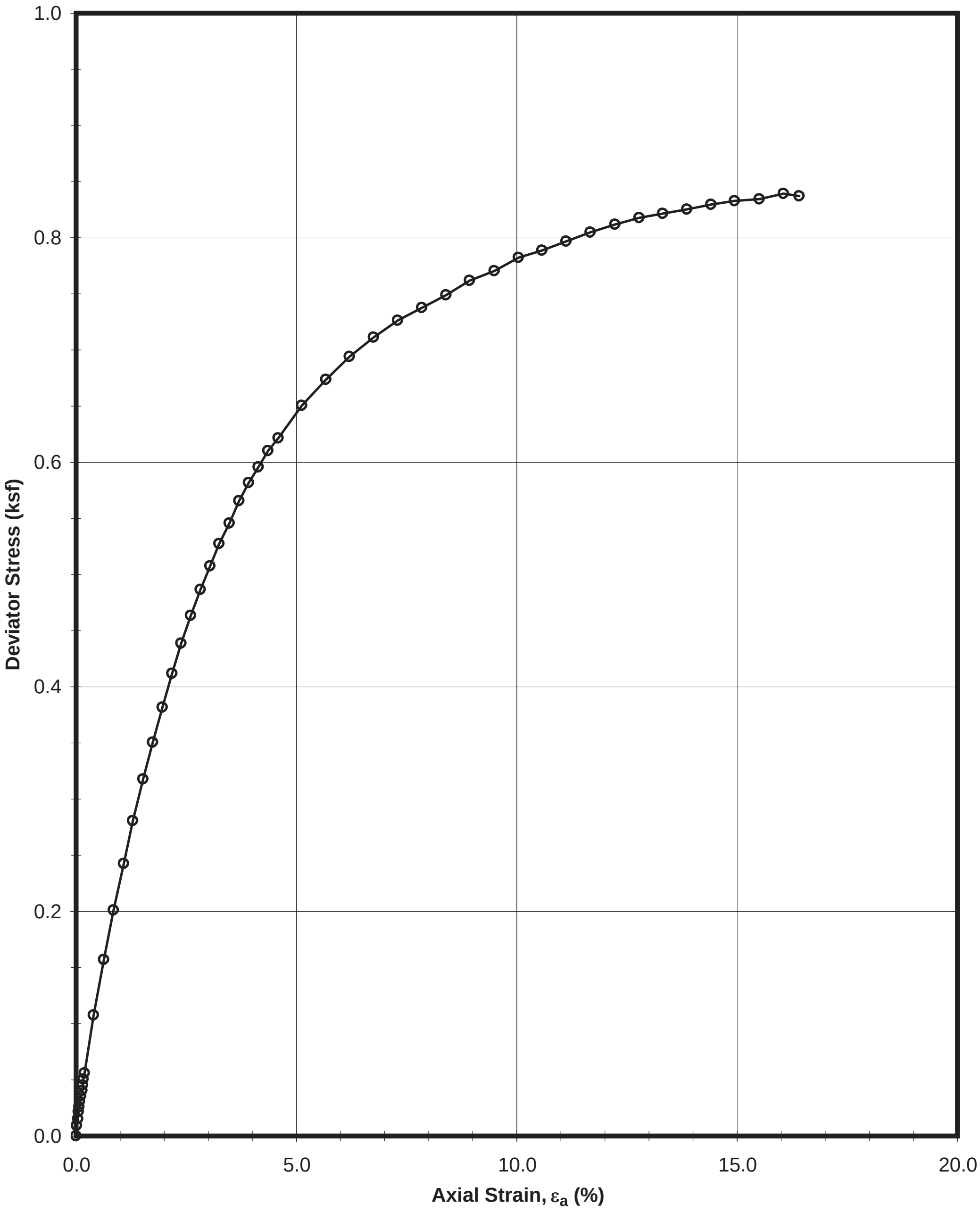
SECTION 4

SUMMARY OF UNCONSOLIDATED-UNDRAINED TRIAXIAL COMPRESSION

This section of the report presents the results of the unconsolidated-undrained triaxial tests of selected samples.

Plots (deviator stress versus axial strain (ε_a)) for each test result are presented on Plates 4-1 and 4-2.

One unconsolidated-undrained triaxial test assignment was not performed because the sample was in an untestable condition. Plate 4-3 displays a photo of the untestable sample.

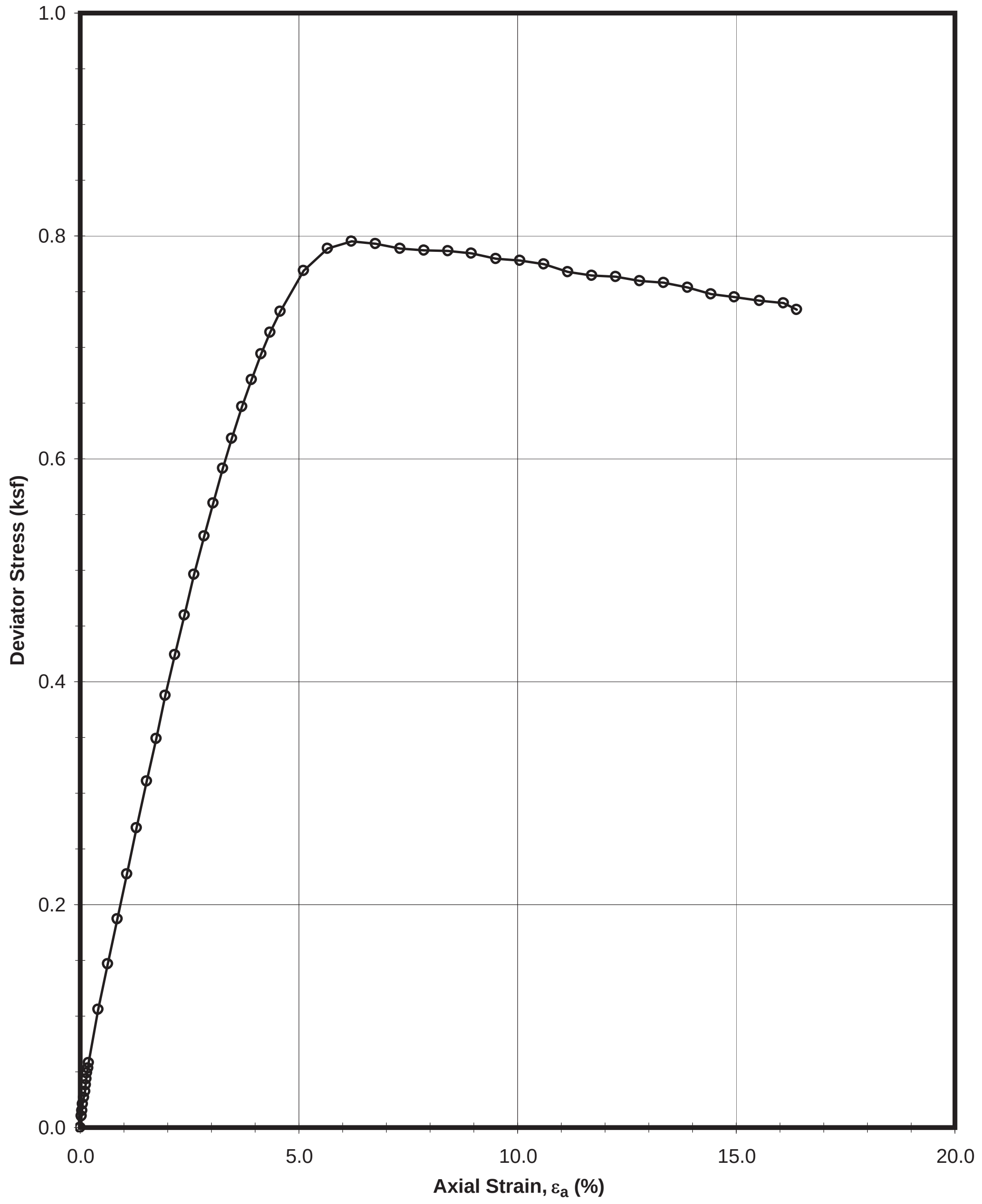


Unconsolidated-Undrained Triaxial Compression Test

Intact Specimen

Boring BH-1 BC#1, Sample SH-1-8, Depth 31.05 ft
Confining Pressure = 15.3 psi
Big Creek Dam #1 and Big Creek Dam #2

PLATE 4-1



Unconsolidated-Undrained Triaxial Compression Test

Intact Specimen

Boring BH-1 BC#1, Sample SH-1-12, Depth 41.15 ft
Confining Pressure = 18.7 psi
Big Creek Dam #1 and Big Creek Dam #2

PLATE 4-2

Unconsolidated-Undrained Triaxial Test Sample

Boring BH-1 BC#1, Sample SH-1-16a

Depth: 50.90 ft.

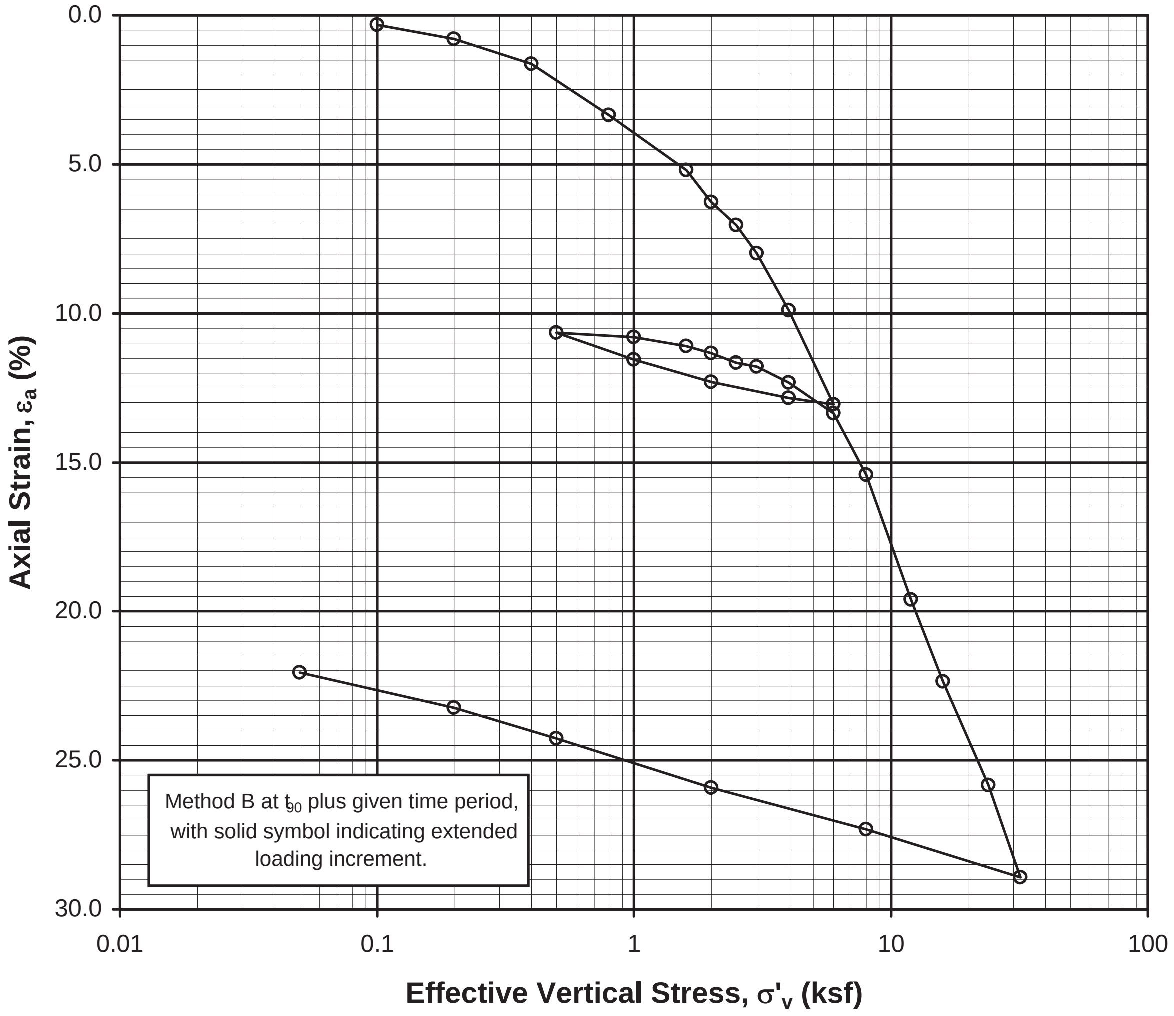
Big Creek Dam #1 and Big Creek Dam #2

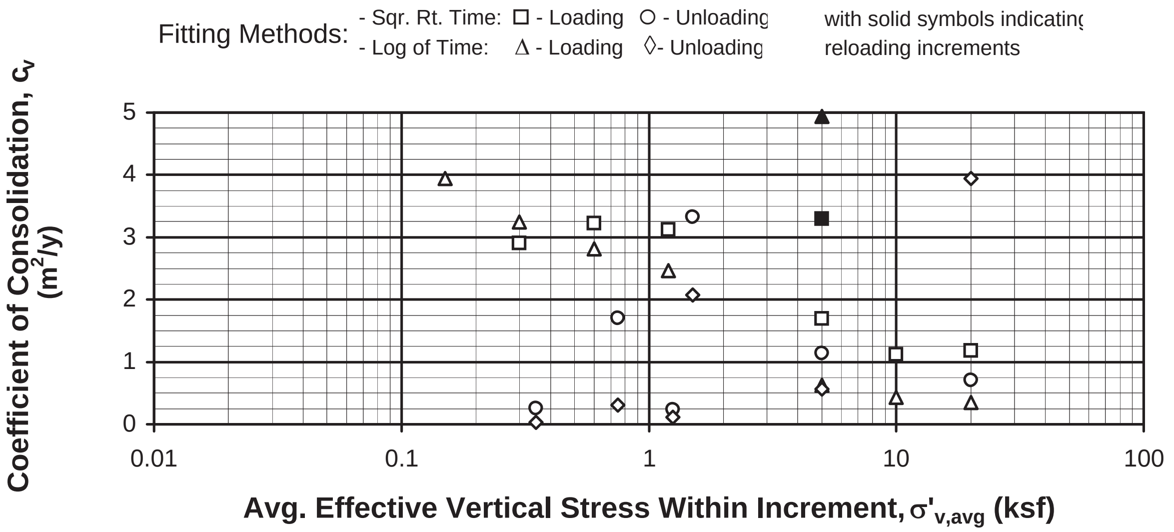
PLATE 4-3

SECTION 5 – INCREMENTAL CONSOLIDATION

This section presents the results of the following individual tests performed for this project:

- One (1) incremental consolidation test. Graphs presenting static direct simple shear results are presented on Plates 5-1 and 5-2. Plate 5-3 provides a test specimen photograph.

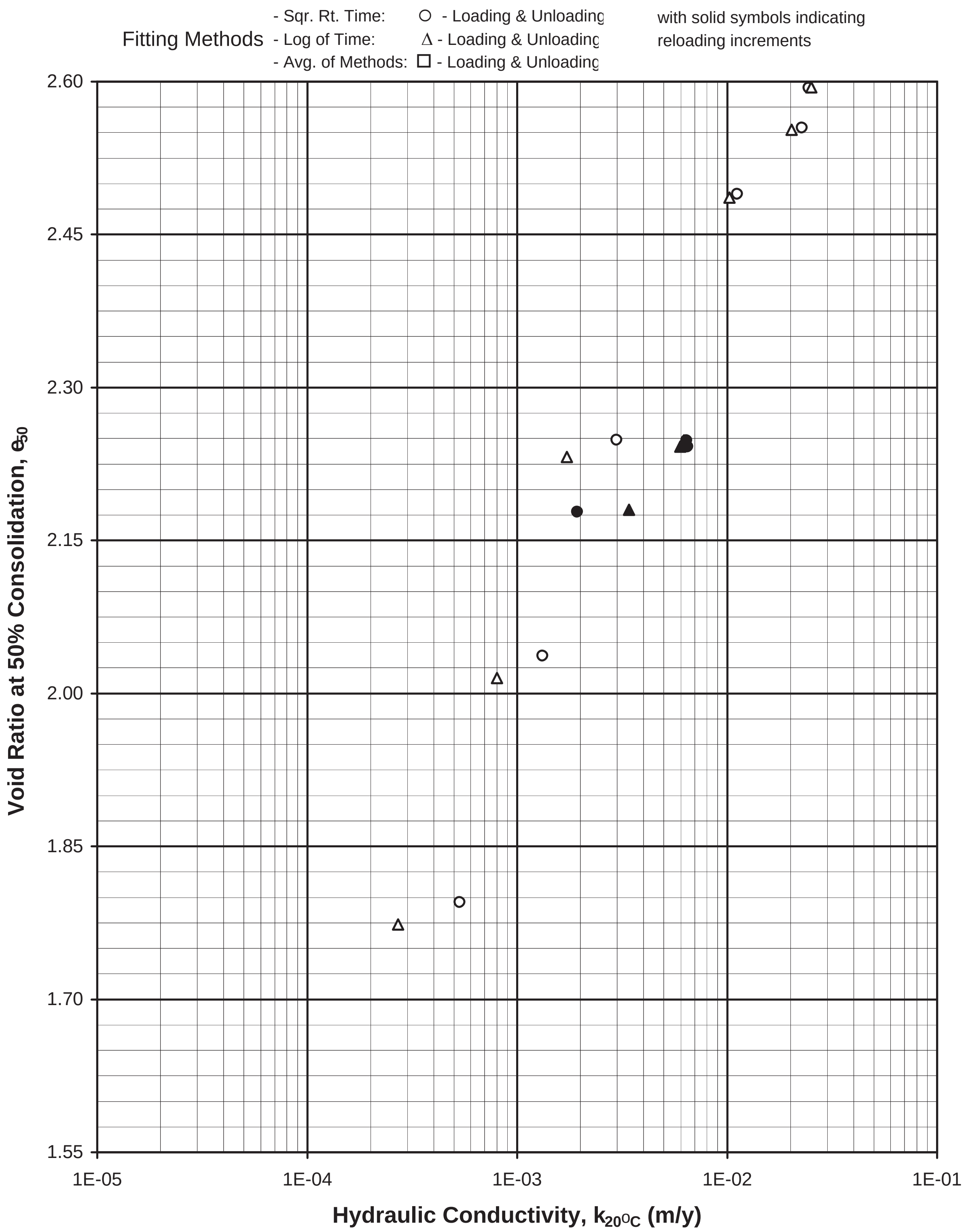




1-D CONSOLIDATION TEST: INC

Sample No. SH-1-8c - Depth 31.55 ft
Boring BH-1 BC#1
Big Creek Dam #1 and Big Creek Dam #2

PLATE 5-1



1-D CONSOLIDATION TEST: INC

Sample No. SH-1-8c - Depth 31.55 ft
Boring BH-1 BC#1
Big Creek Dam #1 and Big Creek Dam #2

PLATE 5-2

Incremental Consolidation Test
Boring BH-1 BC#1, Sample SH-1-8c
Depth: 31.05 – 31.55 ft.
Big Creek Dam #1 and Big Creek Dam #2

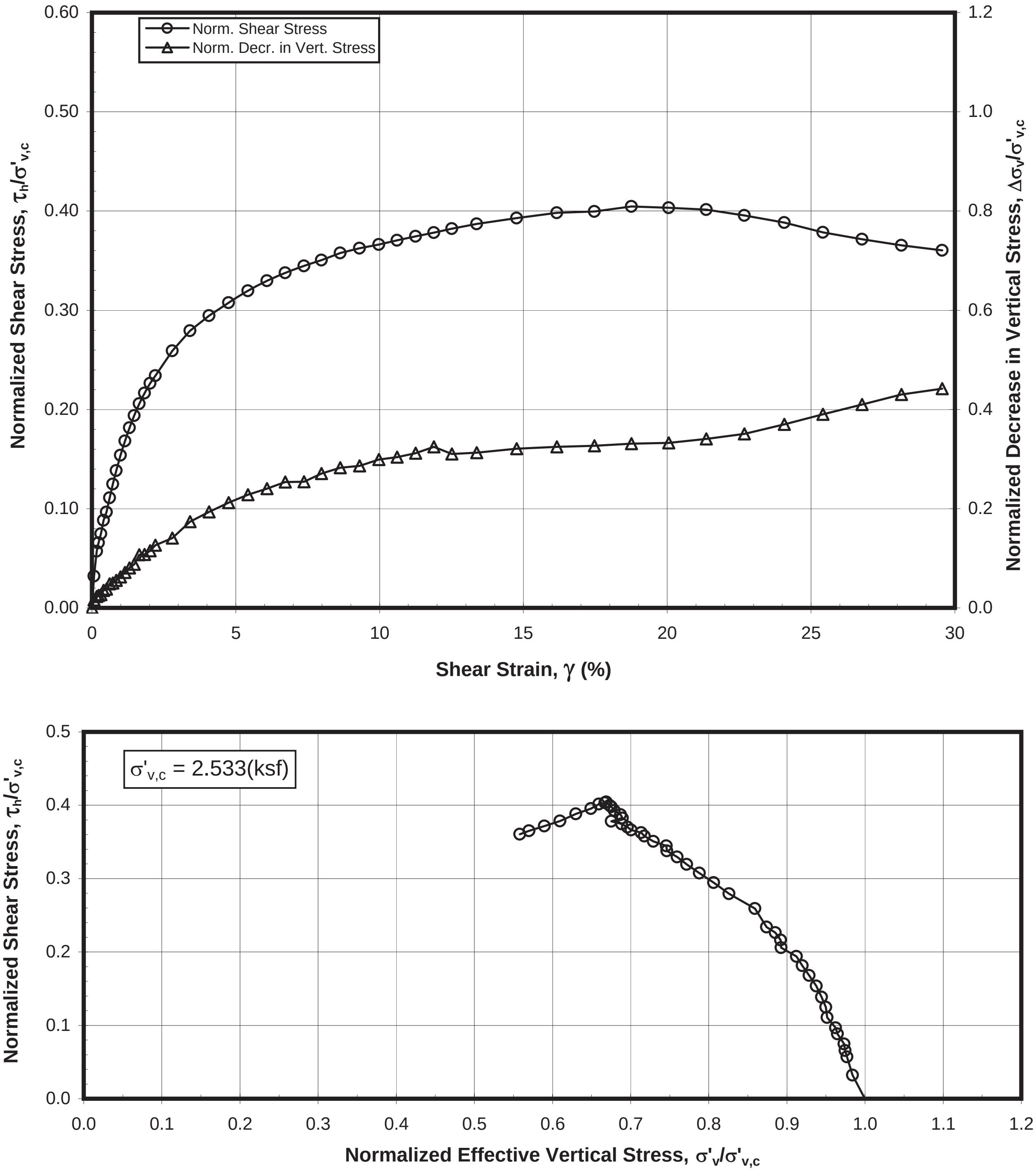
PLATE 5-3

SECTION 6 – STATIC DIRECT SIMPLE SHEAR

This section presents the results of the following individual tests performed for this project:

- Two (2) static direct simple shear tests. Graphs presenting select properties and static direct simple shear results are presented on Plates 6-1 and 6-2. Before and after testing specimen photographs are provided on Plate 6-3.

Tables of the static direct simple shear test results performed on this project are presented on Plate 3-1.



STATIC DSS TEST

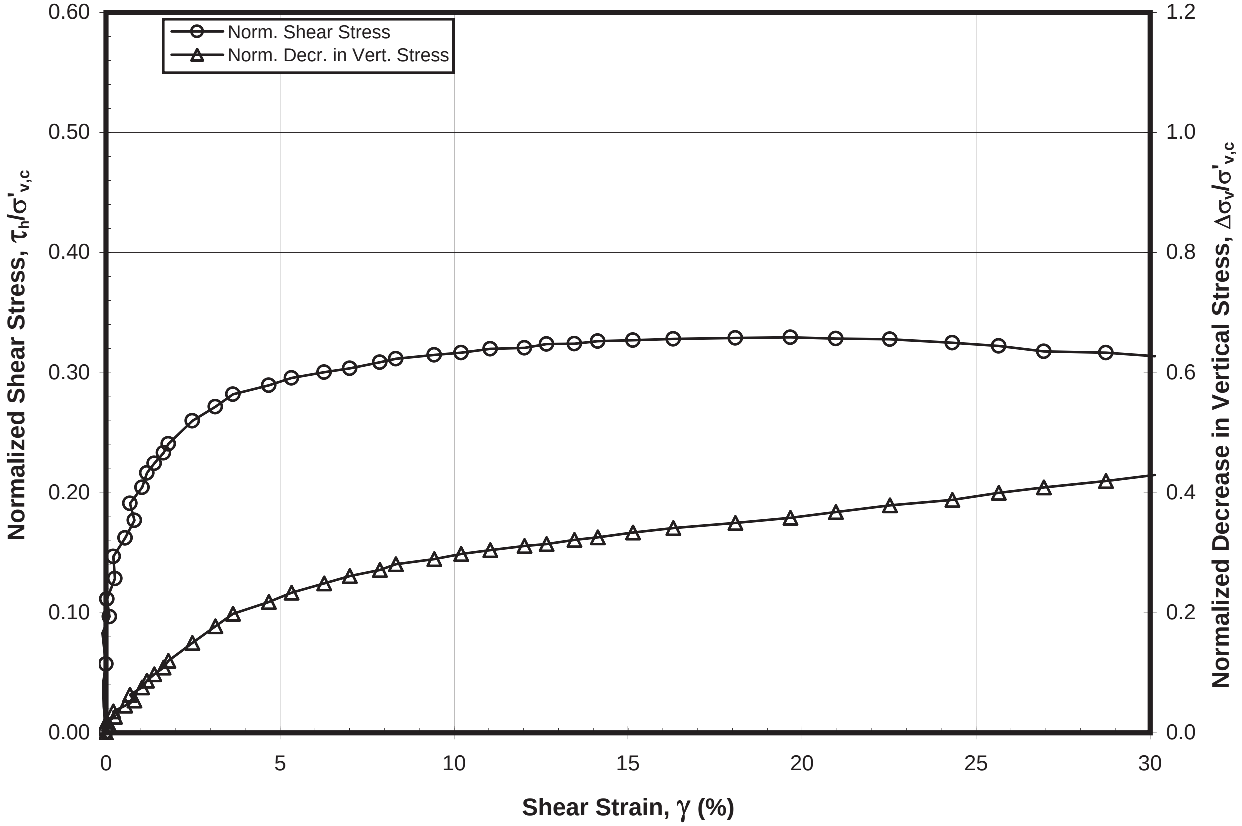
K_o Consolidation - OCR = 1

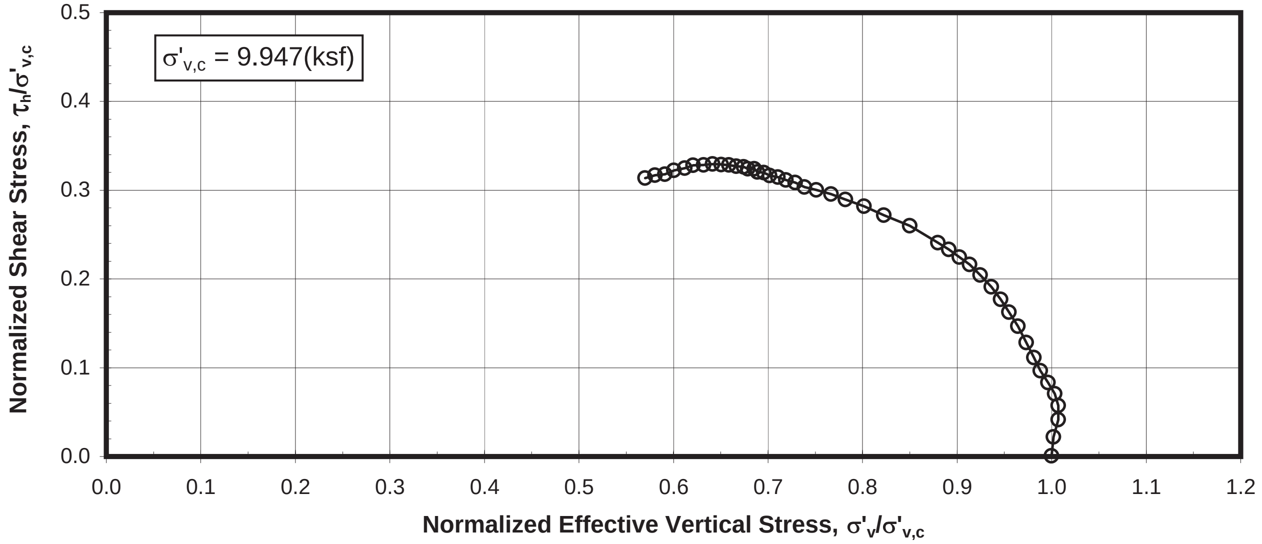
Sample: SH-1-8a - Depth: 31.85 ft

Boring BH-1 BC#1

Big Creek Dam #1 and Big Creek Dam #2

PLATE 6-1





STATIC DSS TEST

K_o Consolidation - OCR = 1

Sample: SH-1-8b - Depth: 31.70 ft

Boring BH-1 BC#1

Big Creek Dam #1 and Big Creek Dam #2

PLATE 6-2

Static Direct Simple Shear Test
Boring BH-1 BC#1, Sample SH-1-8a
Depth: 31.85 ft.
Big Creek Dam #1 and Big Creek Dam #2

Static Direct Simple Shear Test
Boring BH-1 BC#1, Sample SH-1-8B
Depth: 31.70 ft.
Big Creek Dam #1 and Big Creek Dam #2

PLATE 6-3

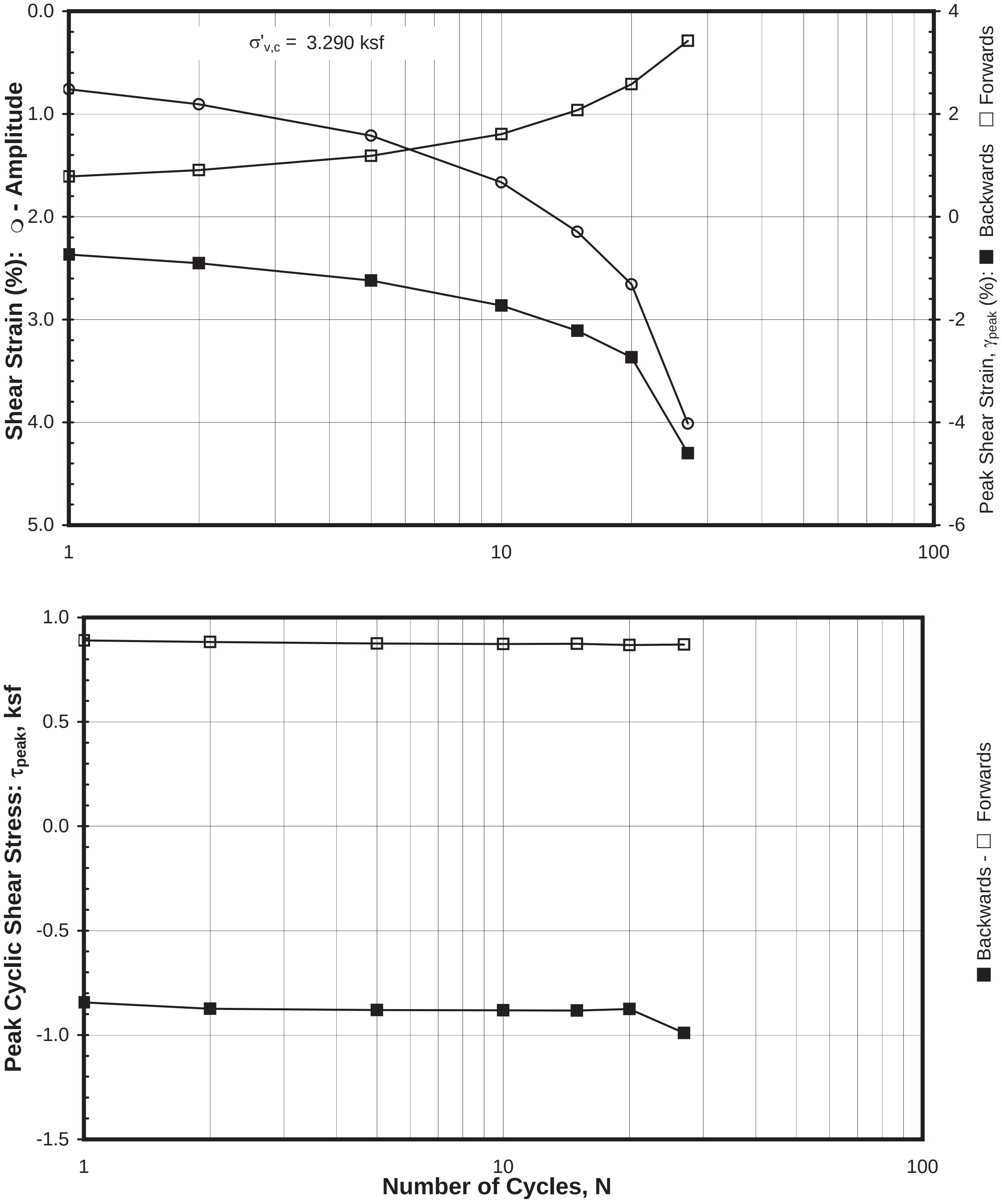
SECTION 7 – CYCLIC DIRECT SIMPLE SHEAR

This section presents the results of the following individual tests performed for this project:

- One (1) cyclic direct simple shear tests. Graphs presenting select properties and static direct simple shear results are presented on Plates 7-1 through 7-4.
- One (1) post-cyclic static direct simple shear test. Graphs presenting select properties and static direct simple shear results are presented on Plates 7-5. Plate 7-6 presents graph of specimen recovery after post-static direct simple shear test.

Plate 7-7 provides before and after-testing specimen photographs.

Tables of the static direct simple shear test results performed on this project are presented on Plate 3-1.



CYCLIC DSS STRENGTH TEST: Without Undrained Bias Shear Stress

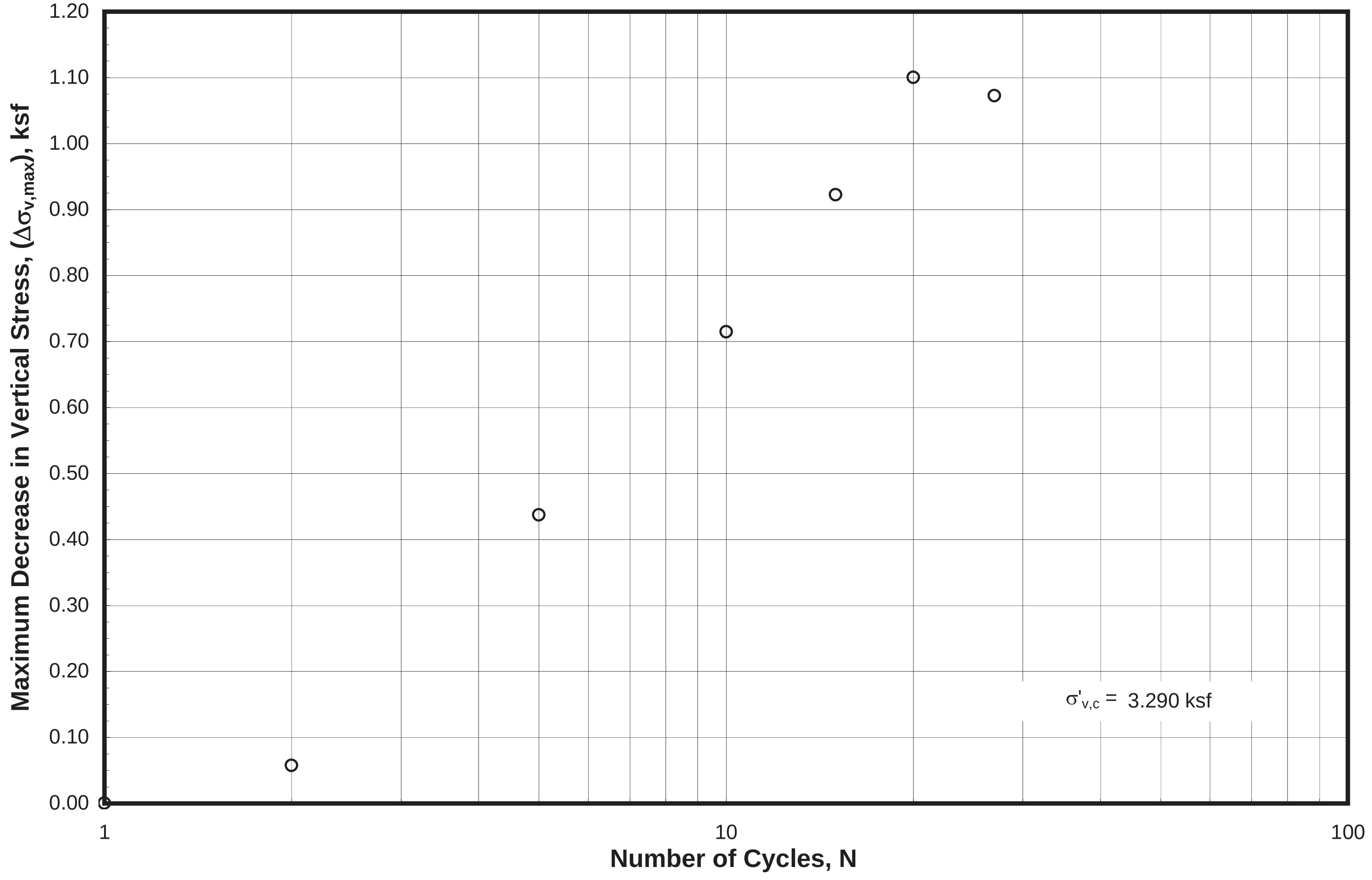
OCR = 1 - Cyclic Rate: 1.0 Hz

Sample: SH-1-8d - Depth: 30.25 ft

Boring BH-1 BC#1

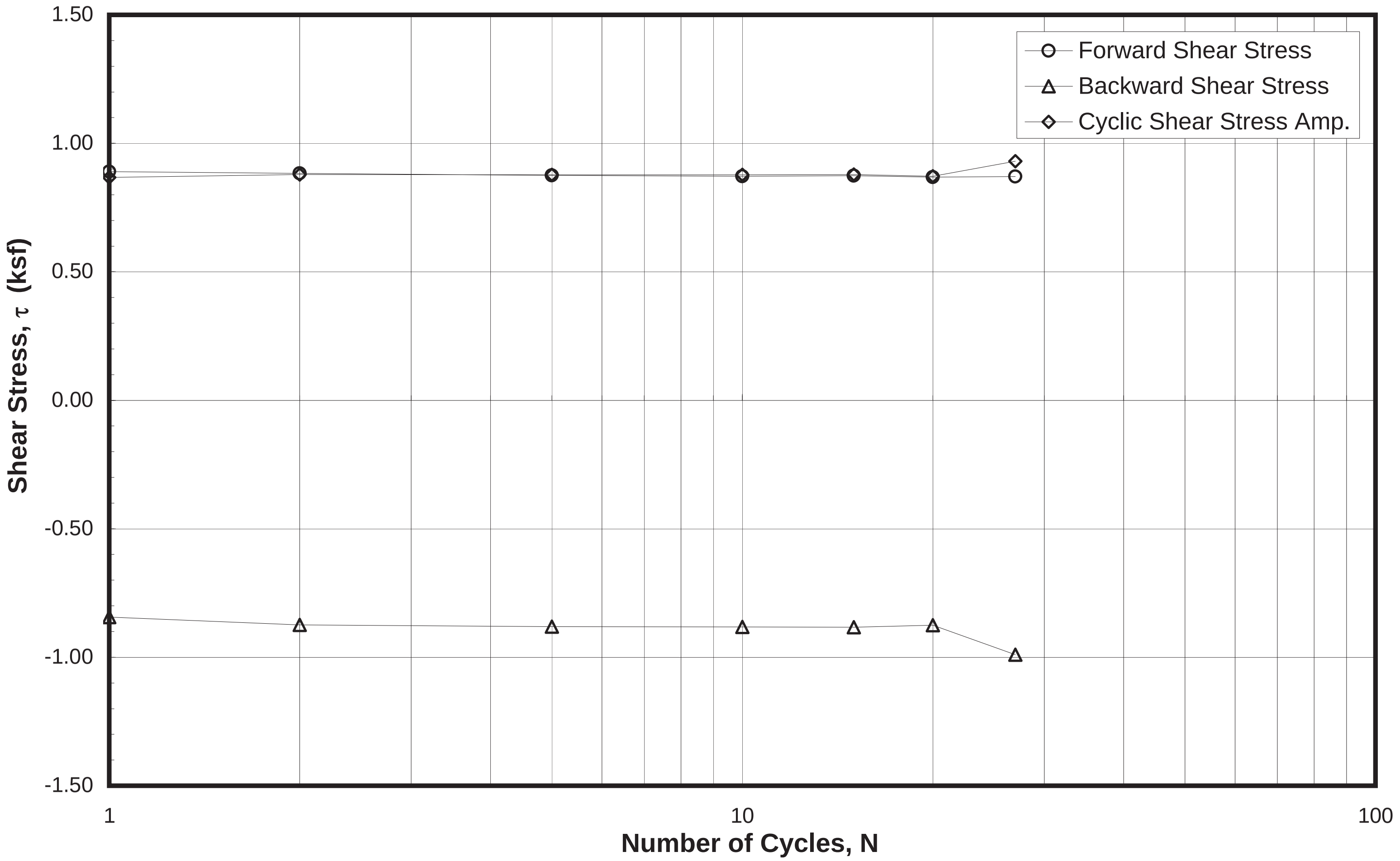
Big Creek Dam #1 and Big Creek Dam #2

PLATE 7-1



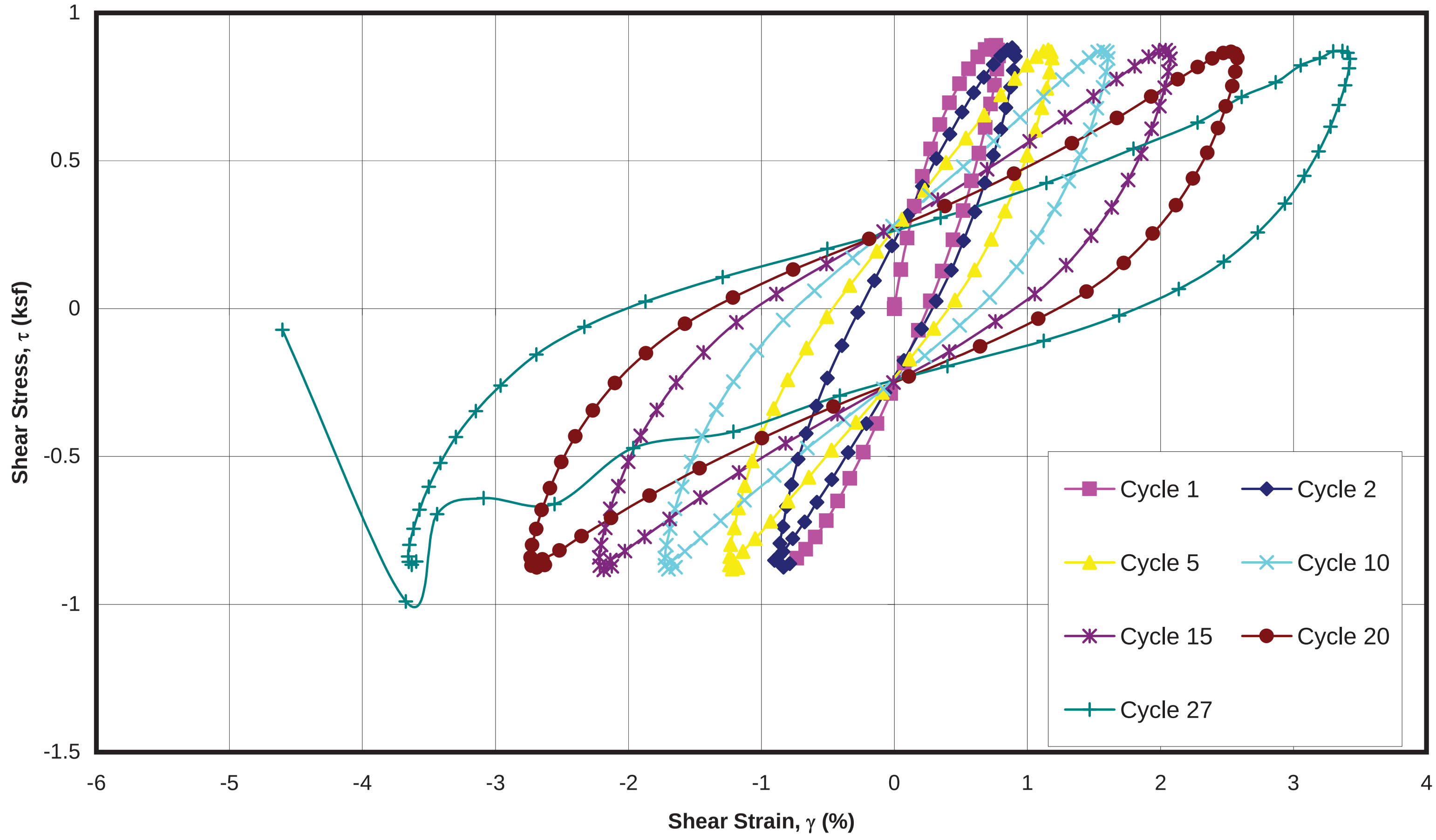
CYCLIC DSS STRENGTH TEST: Without Undrained Bias Shear Stress
OCR = 1 - Cyclic Rate: 1.0 Hz
Sample: SH-1-8d - Depth: 30.25 ft
Boring BH-1 BC#1
Big Creek Dam #1 and Big Creek Dam #2

PLATE 7-2



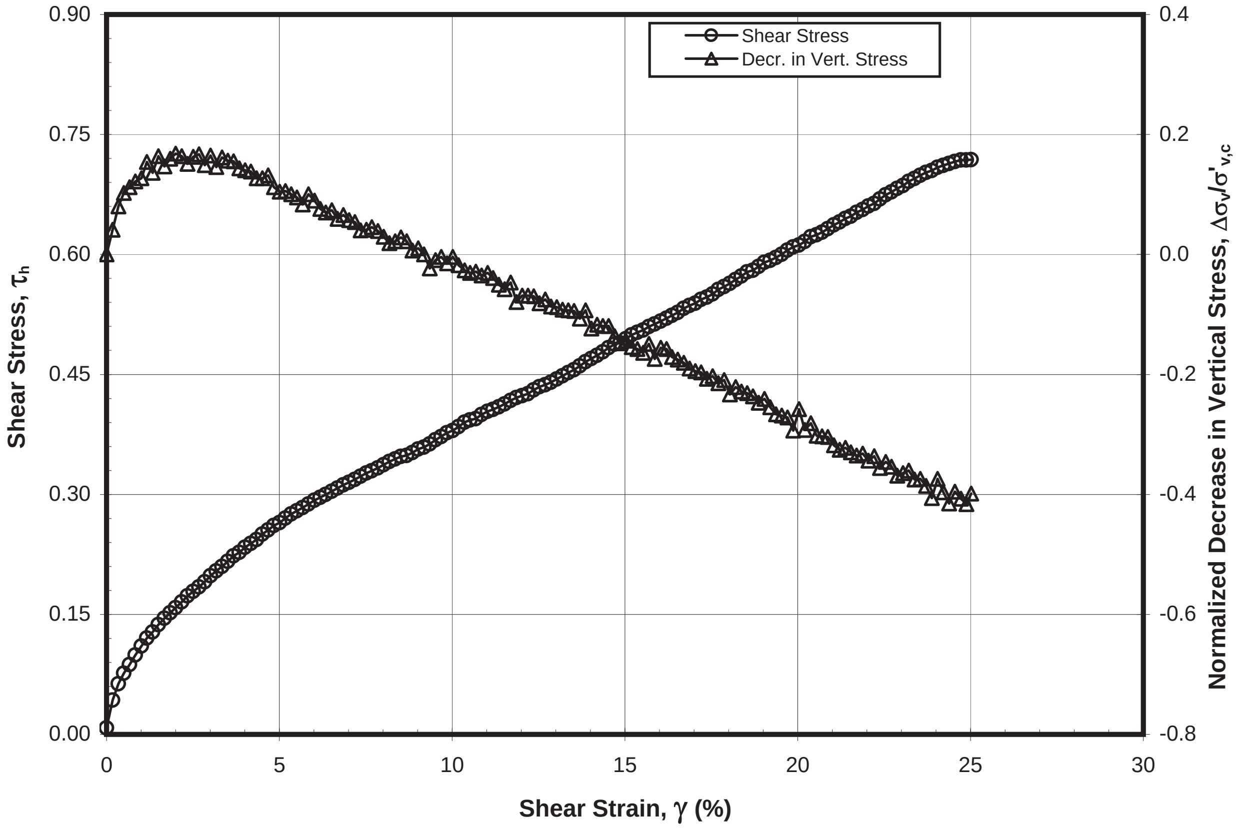
CYCLIC DSS STRENGTH TEST: Without Undrained Bias Shear Stress
OCR = 1 - Cyclic Rate: 1.0 Hz
Sample: SH-1-8d - Depth: 30.25 ft
Boring BH-1 BC#1
Big Creek Dam #1 and Big Creek Dam #2

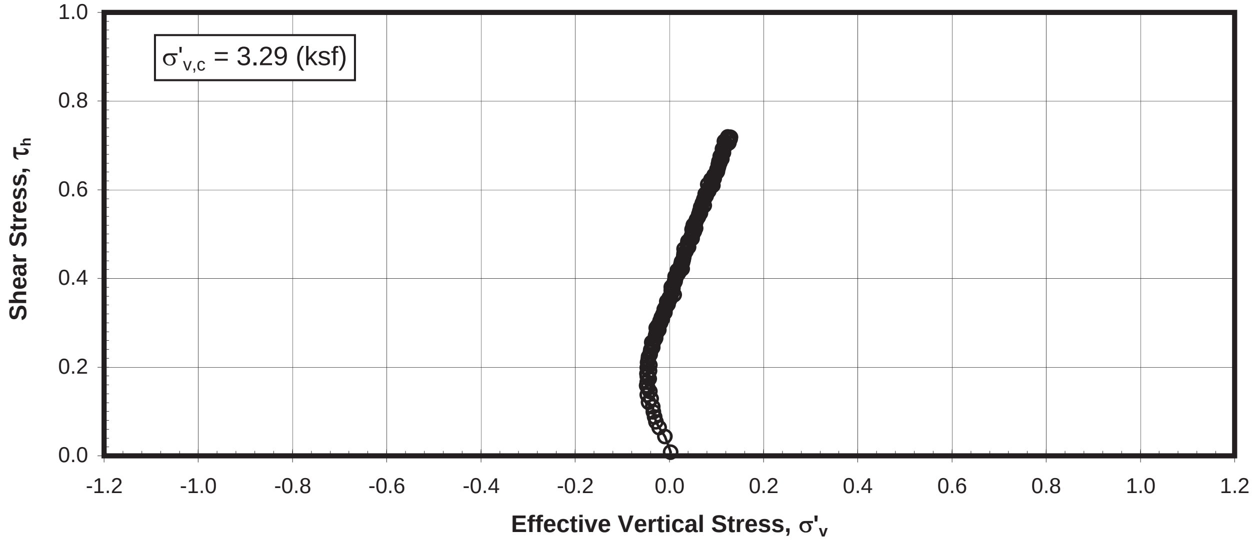
PLATE 7-3



CYCLIC DSS STRENGTH TEST: Without Undrained Bias Shear Stress
OCR = 1 - Cyclic Rate: 1.0 Hz
Sample: SH-1-8d - Depth: 30.25 ft
Boring BH-1 BC#1
Big Creek Dam #1 and Big Creek Dam #2

PLATE 7-4





STATIC DSS POST CYCLIC TEST

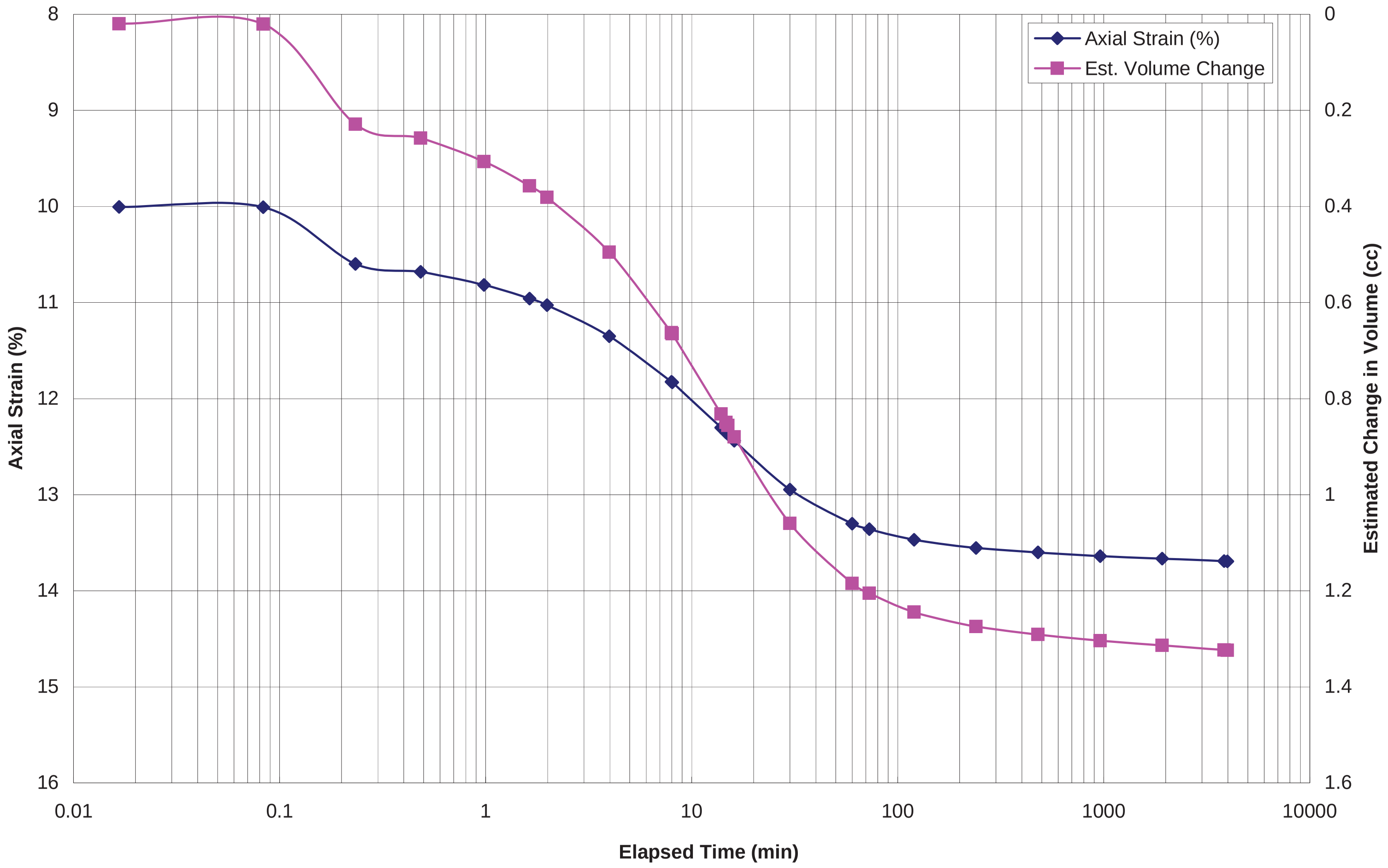
K_o Consolidation - OCR = 1

Sample: SH-1-8d - Depth: 30.25 ft.

Boring BH-1 BC#1

Big Creek Dam #1 and Big Creek Dam #2

PLATE 7-5



CYCLIC DSS STRENGTH TEST: Recovery after Post-Static DSS Test

OCR = 1 - Cyclic Rate: 1.0 Hz

Sample: SH-1-8d - Depth: 30.25 ft

Boring BH-1 BC#1

Big Creek Dam #1 and Big Creek Dam #2

PLATE 7-6

Cyclic Direct Simple Shear Test
Boring BH-1 BC#1, Sample SH-1-8d
Depth: 30.25 ft.
Big Creek Dam #1 and Big Creek Dam #2

PLATE 7-7

End of Report

.

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Beta@radiocarbon.com
www.radiocarbon.com

Darden Hood
President

Ronald Hatfield
Christopher Patrick
Deputy Directors

June 27, 2012

Mr. Nick Clark
HDR Engineering, Inc.
1001 SW 5th Avenue
Suite 1800
Portland, OR 97204
USA

RE: Radiocarbon Dating Result For Sample NEWPORT WTP

Dear Mr. Clark:

Enclosed is the radiocarbon dating result for one sample recently sent to us. It provided plenty of carbon for an accurate measurement and the analysis proceeded normally. The report sheet contains the method used, material type, and applied pretreatments and, where applicable, the two-sigma calendar calibration range.

This report has been both mailed and sent electronically. All results (excluding some inappropriate material types) which are less than about 20,000 years BP and more than about ~250 BP include a calendar calibration page (also digitally available in Windows metafile (.wmf) format upon request). Calibration is calculated using the newest (2004) calibration database with references quoted on the bottom of the page. Multiple probability ranges may appear in some cases, due to short-term variations in the atmospheric 14C contents at certain time periods. Examining the calibration graph will help you understand this phenomenon. Don’t hesitate to contact us if you have questions about calibration.

We analyzed this sample on a sole priority basis. No students or intern researchers who would necessarily be distracted with other obligations and priorities were used in the analysis. We analyzed it with the combined attention of our entire professional staff.

Information pages are also enclosed with the mailed copy of this report. If you have any specific questions about the analysis, please do not hesitate to contact us. Someone is always available to answer your questions.

The cost of the analysis was charged to the VISA card provided. A receipt is enclosed with the mailed report copy. Thank you. As always, if you have any questions or would like to discuss the results, don’t hesitate to contact me.

Sincerely,

Darden Hood
Digital signature on file

BETA ANALYTIC INC.
DR. M.A. TAMERS and MR. D.G. HOOD
4985 S.W. 74 COURT
MIAMI, FLORIDA, USA 33155
PH: 305-667-5167 FAX:305-663-0964
beta@radiocarbon.com

REPORT OF RADIOCARBON DATING ANALYSES

Mr. Nick Clark
HDR Engineering, Inc.

Report Date: 6/27/2012
Material Received: 6/20/2012

Sample Data	Measured Radiocarbon Age	13C/12C Ratio	Conventional Radiocarbon Age(*)
Beta - 324359	4100 +/- 30 BP	-25.7 o/oo	4090 +/- 30 BP

SAMPLE : NEWPORT WTP
ANALYSIS : AMS-Standard delivery
MATERIAL/PRETREATMENT : (wood): acid/alkali/acid
2 SIGMA CALIBRATION : Cal BC 2860 to 2810 (Cal BP 4810 to 4760) AND Cal BC 2750 to 2720 (Cal BP 4700 to 4670)
Cal BC 2700 to 2570 (Cal BP 4650 to 4520) AND Cal BC 2510 to 2500 (Cal BP 4460 to 4450)

Dates are reported as RCYBP (radiocarbon years before present, "present" = AD 1950). By international convention, the modern reference standard was 95% the 14C activity of the National Institute of Standards and Technology (NIST) Oxalic Acid (SRM 4990C) and calculated using the Libby 14C half-life (5568 years). Quoted errors represent 1 relative standard deviation statistics (68% probability) counting errors based on the combined measurements of the sample, background, and modern reference standards. Measured 13C/12C ratios (delta 13C) were calculated relative to the PDB-1 standard.

The Conventional Radiocarbon Age represents the Measured Radiocarbon Age corrected for isotopic fractionation, calculated using the delta 13C. On rare occasion where the Conventional Radiocarbon Age was calculated using an assumed delta 13C, the ratio and the Conventional Radiocarbon Age will be followed by "*". The Conventional Radiocarbon Age is not calendar calibrated. When available, the Calendar Calibrated result is calculated from the Conventional Radiocarbon Age and is listed as the "Two Sigma Calibrated Result" for each sample.

CALIBRATION OF RADIOCARBON AGE TO CALENDAR YEARS

(Variables: C13/C12=-25.7:lab. mult=1)

Laboratory number: **Beta-324359**

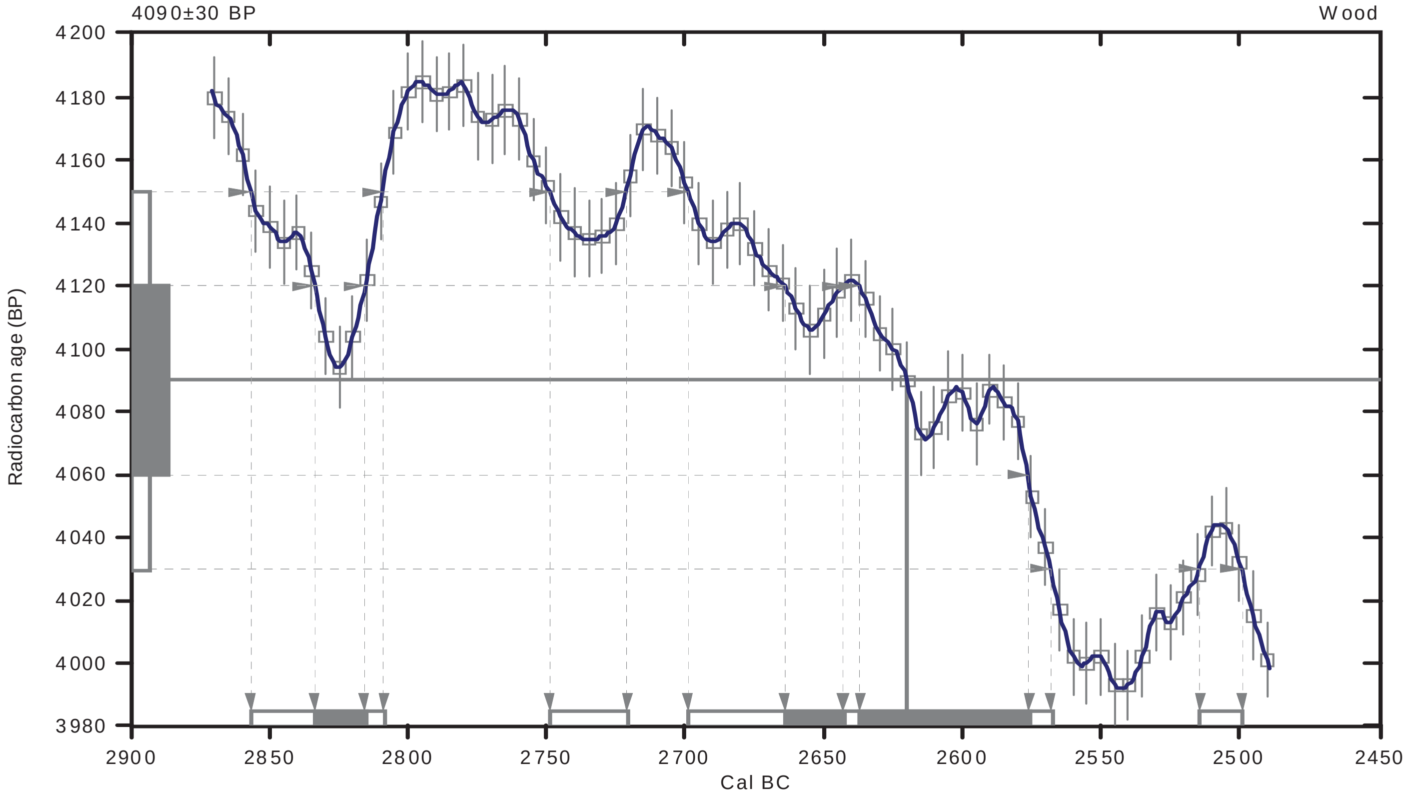
Conventional radiocarbon age: **4090±30 BP**

2 Sigma calibrated results:
(95% probability)
Cal BC 2860 to 2810 (Cal BP 4810 to 4760) and
Cal BC 2750 to 2720 (Cal BP 4700 to 4670) and
Cal BC 2700 to 2570 (Cal BP 4650 to 4520) and
Cal BC 2510 to 2500 (Cal BP 4460 to 4450)

Intercept data

Intercept of radiocarbon age
with calibration curve: Cal BC 2620 (Cal BP 4570)

1 Sigma calibrated results:
(68% probability)
Cal BC 2830 to 2820 (Cal BP 4780 to 4770) and
Cal BC 2660 to 2640 (Cal BP 4610 to 4590) and
Cal BC 2640 to 2580 (Cal BP 4590 to 4530)



References:

Database used
INTCAL09
References to INTCAL09 database
Heaton,et.al.,2009, Radiocarbon 51(4):1151-1164, Reimer,et.al, 2009, Radiocarbon 51(4):1111-1150, Stuiver,et.al,1993, Radiocarbon 35(1):137-189, Oeschger,et.al.,1975,Tellus 27:168-192
Mathematics used for calibration scenario
A Simplified Approach to Calibrating C14 Dates
Talma, A. S., Vogel, J. C., 1993, Radiocarbon 35(2):317-322

Attachment B 2. Phase 3 Geotechnical Data

CORNFORTH
CONSULTANTS

10250 S.W. Greenburg Road, Suite 111
Portland, Oregon 97223
Phone 503-452-1100 Fax 503-452-1528

December 19, 2014 2384

Verena Winter, P.E.
HDR Engineering, Inc.
1001 SW 5th Avenue, Suite 1800
Portland, Oregon 97204

Geotechnical Investigation Report
Big Creek Dam No. 1 and Dam No. 2
Newport, Oregon

Dear Ms. Winter:

Following your authorization, we have completed a geotechnical investigation report of the explorations performed for Big Creek Dam No. 1 and Dam No. 2 in support of the Phase 3 Engineering Evaluations and Concept Design studies being performed for the Big Creek Dams located near Newport, Oregon. This letter report summarizes the exploration program and presents the results.

Introduction

We understand that HDR is currently undertaking Phase 3 engineering evaluations and conceptual design for the seismic performance of Big Creek Dam No. 1 and Dam No. 2. As part of their services, HDR is performing risk analyses, developing a corrective action concept and conducting a preliminary environmental review. This work includes subsurface investigations, evaluation of embankment stability, liquefaction hazard analyses, differential settlement, and surface displacement. As part of this work, HDR has requested a geotechnical investigation report of the explorations performed for Big Creek Dam No. 1 and Dam No. 2.

Geologic Setting

The geology in the vicinity of the Big Creek Dams generally consists of Nye Mudstone overlain by alluvial streambed deposits and poorly-sorted colluvium. The Nye Mudstone consists of sandy siltstone and fine- to medium-grained marine siltstone and sandstone of the Miocene Era. The sedimentary bedrock is overlain by alluvial streambed material consisting of sands and silts and colluvium consisting of sandy, clayey silt with scattered gravels and organics.

Subsurface Investigation

The subsurface exploration program consisted of six mud-rotary borings and six seismic cone penetrometer test (CPT) soundings. A list of the explorations is presented in Table 1. Summary boring logs are provided in Appendix A, and cone penetrometer test logs are provided in Appendix B.

The geotechnical drilling was performed by Western States Soil Conservation, using both a truck-mounted (rubber tire) drill rig and a track-mounted drill rig. The seismic cone penetrometer soundings were performed by Subsurface Technologies using a truck-mounted rig. The borings were completed between October 22 and November 26, 2013. A representative of Cornforth Consultants was onsite with the drill rigs to coordinate the operation, log and sample the subsurface materials, and assist with the installation of instrumentation. Laboratory tests performed on samples collected from the borings included Atterberg limits, moisture contents, and gradations.

Standard Penetration Tests (SPT) were performed at 1.5-foot (continuous) intervals through the overburden materials in BC1-BH-2 and BC2-BH-4. SPT and undisturbed sampling using a piston sampler were performed at select depths through the overburden materials in BC1-BH-3(u), BC1-BH-4(u), BC2-BH-5(u), and BC2-BH-6(u). Sampling locations were chosen in consultation with HDR. The SPT sampler consisted of a 2-inch O.D. split-spoon, with a recessed I.D. (without liners), driven by a 140-lb auto-trip hammer. The piston sampler consisted of 3-inch O.D. galvanized thin-wall sampler. Drilling methods, sampling depths, total drill hole depths, and descriptions of the soil and rock material encountered are provided on the Summary Boring Logs.

Undisturbed sampling was carefully monitored during explorations to quantify sample recovery and potential sample disturbance. A detailed log for each undisturbed samples was kept with information including: sample tube dimensions; piston penetration distance, recovered length of sample and trimmed length of sample. Samples were sealed and protected in wooden storage containers provided by HDR and stored at the project site. HDR collected the samples from storage area at the project site, along with the sampling records and description of materials.

Cone penetrometer soundings were conducted at two locations along the downstream toe of Dam No. 1 (BC1-SCPT-5 and -6) and four locations at or downstream of Dam No. 2 (BC2-SCPT-4, -5, -6 and -7). The sounding included seismic shear wave velocity measurements and pore water pressure dissipation tests. Cone Penetrometer Test Logs are shown in Appendix B.

Table 1: Summary of Explorations

Exploration	Depth (ft)	Approx. Ground Surface Elev. (ft)	Piezometer Screened Depth (ft)
BC1-BH-2	69.4	33	—
BC1-BH-3(u)	64.5	33	51.0 – 61.0
BC1-BH-4(u)	70.0	33	16.0 – 26.0
BC1-SCPT-5	58.6	33	—
BC1-SCPT-6	71.5	33	—
BC2-BH-4	46.7	50	—
BC2-BH-5(u)	48.5	50	35.0 – 45.0
BC2-BH-6(u)	41.5	50	27.5 – 37.5
BC2-SCPT-5	25.1	50	—
BC2-SCPT-6	30.0	50	—
BC2-SCPT-7	15.4	50	—

Field Vane Testing

Where soil conditions were conducive, in-situ vane shear testing was performed in general accordance with ASTM D2573. The results are summarized in Table 2 and noted on the Summary Boring Logs.

Table 2: Summary of In-situ Testing

Boring	Test	Depth (ft)	Shear Strength (psf)
BC1-BH-4(u)	Vane Shear	13.5	1088
BC1-BH-4(u)	Vane Shear	29.5	1985

Instrumentation

Standpipe piezometers were installed in borings BC1-BH-3(u), BC1-BH-4(u), BC2-BH-5(u) and BC2-BH-6(u). The piezometers consist of a 10-foot long, slotted-tip (1-inch diameter, 10-mil machine slot) connected to a 1-inch diameter solid PVC riser pipe. The annular space between the tip and surrounding borehole was backfilled with 10-20 size sand and sealed to the surface with bentonite chips. Details of the piezometer instrumentation are shown of the Summary Boring Logs.

Laboratory Testing

Laboratory testing was performed on selected samples collected during the exploration program to determine the following properties:

- Soil Classification
- Water Content
- Plasticity
- Gradation
- Fines Content (percent passing No. 200 sieve)

All soil and rock samples obtained from the field exploration program were visually examined in the field. The soil and rock classifications, water contents from the SPT samples, and Atterberg limits are shown on the Summary Boring Logs. Laboratory tests were performed in general accordance with ASTM D422, D2216, D4318, and D6913. Atterberg limits , fines content and gradation tests were performed on samples selected in consultation with HDR. Laboratory testing results are summarized in Table 3 and additional laboratory testing data and charts are included in Appendix C.

Table 3: Summary of Laboratory Testing

Boring	Sample	Depth (ft)	PL	LL	PI	Cohesive Index	% Passing #200 Sieve
BC1-BH-2	S-04	6.0-7.5	50	73	23	0.46	
BC1-BH-2	S-08	12.0-13.5	44	65	21	0.48	32
BC1-BH-2	S-12	18.0-19.5		Non-Plastic			
BC1-BH-2	S-13	19.5-21.0	37	70	33	0.89	
BC1-BH-2	S-17	25.5-27.0	44	69	25	0.57	
BC1-BH-2	S-21	31.5-33.0	40	51	11	0.28	60
BC1-BH-2	S-23	34.5-36.0	48	80	32	0.67	
BC1-BH-2	S-25	37.5-39.0	39	49	10	0.26	
BC1-BH-2	S-28	42-43.5		Non-Plastic			48
BC1-BH-2	S-29	43.5-45.0	42	53	11	0.26	
BC1-BH-2	S-31	46.5-48.0	44	58	14	0.32	
BC1-BH-2	S-34	51.0-52.5		Non-Plastic			45
BC1-BH-2	S-36	54-55.5	37	75	38	1.03	53

Boring	Sample	Depth (ft)	PL	LL	PI	Cohesive Index	% Passing #200 Sieve
BC1-BH-2	S-40	60.0-61.5		Non-Plastic			
BC1-BH-3(u)	S-07	30-31.5		Non-Plastic			38
BC1-BH-3(u)	S-11	53-54.5		Non-Plastic			53
BC1-BH-4(u)	S-11	51-52.5	34	62	28	0.82	56
BC2-BH-4	S-02	3.0-4.5	53	69	16	0.30	62
BC2-BH-4	S-06	9.0-10.5	43	60	17	0.40	62
BC2-BH-4	S-08	12.0-13.5		Non-Plastic			
BC2-BH-4	S-14	21.0-22.5	41	74	33	0.80	51
BC2-BH-4	S-16	24.0-25.5		Non-Plastic			27
BC2-BH-4	S-19	28.5-30		Non-Plastic			49
BC2-BH-4	S-20	30.0-.1.5		Non-Plastic			
BC2-BH-4	S-24	36.0-37.5	40	76	36	0.90	50
BC2-BH-5(u)	S-06	22-23.5	40	70	30	0.75	68
BC2-BH-5(u)	S-08	27-28.5	39	76	37	0.95	48
BC2-BH-6(u)	S-06	22-23.5	43	63	20	0.47	62

Subsurface Conditions

General. The subsurface materials encountered in the Big Creek Dam No. 1 exploratory boreholes generally consisted of approximately 60 feet of silty sand, clayey silt, and silty clay alluvium overlying Nye Mudstone. The alluvium contained wood and other organics, mica, and rounded coarse sand and fine gravel. The subsurface materials encountered in the Big Creek Dam No. 2 exploratory boreholes generally consisted of approximately 10 to 15 feet of silty sand and clayey silt alluvium, overlying approximately 30 to 35 feet of silty sand, clayey silt, and silty clay alluvium/colluvium, overlying Nye Mudstone. The alluvium contained organics, mica, and rounded coarse sand. The alluvium/colluvium contained wood and other organics, mica, and rounded and angular coarse sand and fine gravel. Detailed soil and rock descriptions are contained on the Summary Boring Logs in Appendix A.

Groundwater. Groundwater measurements were taken in the four piezometers (see Table 1: Summary of Explorations) upon completion of the subsurface investigation on November 26, 2013 and are shown on the Summary Boring Logs in Appendix A. The piezometers indicated groundwater levels within 3 feet of the ground surface at that time of the measurements.

We trust that this report is sufficient for your current requirements. Should you have any questions or comments, please call.

Sincerely,

CORNFORTH CONSULTANTS, INC.

Zachary Ruby

Zach Ruby
Project Engineer

Christopher I. Carpenter, P.E.
Associate Engineer

REGISTERED PROFESSIONAL ENGINEER
65646PE
OREGON
NOV. 13, 2012
CHRISTOPHER IAN CARPENTER
RENEWS: 12/31/2015

Appendix A – Summary Boring Logs
Appendix B – Cone Penetrometer Logs
Appendix C – Laboratory Testing Summary

Cornforth Consultants, Inc.
Landslide Technology

Limitations in the Use and Interpretation of this Geotechnical Report

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Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally accepted engineering principles and practices. This warranty is in lieu of all other warranties, either expressed or implied.

The geotechnical report was prepared for the use of the Owner in the design of the subject facility and should be made available to potential contractors and/or the Contractor for information on factual data only. This report should not be used for contractual purposes as a warranty of interpreted subsurface conditions such as those indicated by the interpretive boring and test pit logs, cross-sections, or discussion of subsurface conditions contained herein.

The analyses, conclusions and recommendations contained in the report are based on site conditions as they presently exist and assume that the exploratory borings, test pits, and/or probes are representative of the subsurface conditions of the site. If, during construction, subsurface conditions are found which are significantly different from those observed in the exploratory borings and test pits, or assumed to exist in the excavations, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. If there is a substantial lapse of time between the submission of this report and the start of work at the site, or if conditions have changed due to natural causes or construction operations at or adjacent to the site, this report should be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions and time lapse.

The Summary Boring Logs are our opinion of the subsurface conditions revealed by periodic sampling of the ground as the borings progressed. The soil descriptions and interfaces between strata are interpretive and actual changes may be gradual.

The boring logs and related information depict subsurface conditions only at these specific locations and at the particular time designated on the logs. Soil conditions at other locations may differ from conditions occurring at these boring locations. Also, the passage of time may result in a change in the soil conditions at these boring locations.

Groundwater levels often vary seasonally. Groundwater levels reported on the boring logs or in the body of the report are factual data only for the dates shown.

Unanticipated soil conditions are commonly encountered on construction sites and cannot be fully anticipated by merely taking soil samples, borings or test pits. Such unexpected conditions frequently require that additional expenditures be made to attain a properly constructed project. It is recommended that the Owner consider providing a contingency fund to accommodate such potential extra costs.

This firm cannot be responsible for any deviation from the intent of this report including, but not restricted to, any changes to the scheduled time of construction, the nature of the project or the specific construction methods or means indicated in this report; nor can our firm be responsible for any construction activity on sites other than the specific site referred to in this report.

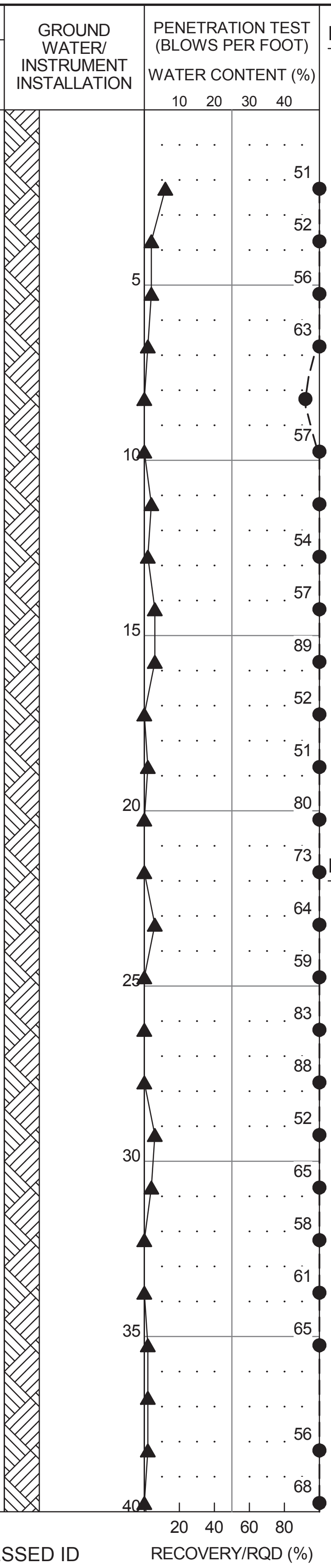
Appendix A - Boring Logs

SURFACE ELEVATION: 33.0 FT.

ELEVATION IN FEET	DEPTH IN FEET	MATERIAL DESCRIPTION
		VERY SOFT, brown, slightly sandy, slightly clayey SILT; numerous orange mottling, scattered mica, occasional organics, moist (FILL)
28.5	4.5	VERY LOOSE, gray, silty SAND to VERY SOFT, gray, slightly sandy, clayey SILT; scattered to numerous organics, scattered mica, wet (ALLUVIUM)
13.5	19.5	VERY SOFT, gray, slightly sandy, silty CLAY to clayey SILT; scattered organics and mica, occasional white mottling, wet (ALLUVIUM)
4.5	28.5	VERY SOFT, gray, slightly sandy, slightly clayey SILT to very silty CLAY; scattered mica, occasional orange and gray mottling, occasional organics, wet (ALLUVIUM)

SAMPLE

NO.	PEN. DATA
S-1	1 2 4
S-2	1 1 1
S-3	0 1 1
S-4	0 0 1
S-5	0 0 0
S-6	0 0 0
S-7	0 1 1
S-8	0 0 1
S-9	1 1 2
S-10	1 1 2
S-11	0 0 0
S-12	1 1 0
S-13	0 0 0
S-14	0 0 0
S-15	0 1 2
S-16	0 0 0
S-17	0 0 0
S-18	0 0 0
S-19	0 1 2
S-20	1 1 1
S-21	0 0 0
S-22	1 0 0
S-23	0 0 1
S-24 *	0 0 1
S-25	0 0 1
S-26	0 0



LEGEND

- ▨ 2-INCH O.D. SPLIT SPOON
- ☒ 3-INCH O.D. SPLIT SPOON
- ▯ 3-INCH O.D. THIN WALL SAMPLER
- ▥ 3-INCH O.D. PITCHER TUBE SAMPLER
- * NO SAMPLE RECOVERY
- ▼ MM/DD/YY GROUND WATER LEVEL AND DATE OBSERVED
- ├─●─┐ LIQUID LIMIT, WATER CONTENT, PLASTIC LIMIT
- ▲ STANDARD PENETRATION TEST (BLOWS/FT.)
- ● WATER CONTENT IN PERCENT
- ▨ CORE RECOVERY IN PERCENT
- ■ RQD IN PERCENT
-] PT-1 PACKER TEST INTERVAL

NOTES

1. MATERIAL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.

HAMMER ASSEMBLY: AUTO TRIP SPT SAMPLER: NO LINER - RECESSED ID

DRILL ROD USED: AWJ BOREHOLE DIAM.: 3 7/8"

DRILLER: WESTERN STATES

DATE START: 10/23/2013 FINISH: 10/24/2013

DRILLING TECHNIQUE: MUD ROTARY

CORNFORTH CONSULTANTS

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Portland, Oregon 97223
Phone 503-452-1100 Fax 503-452-1528

SUMMARY BORING LOG BC1-BH-2 (1 of 2)

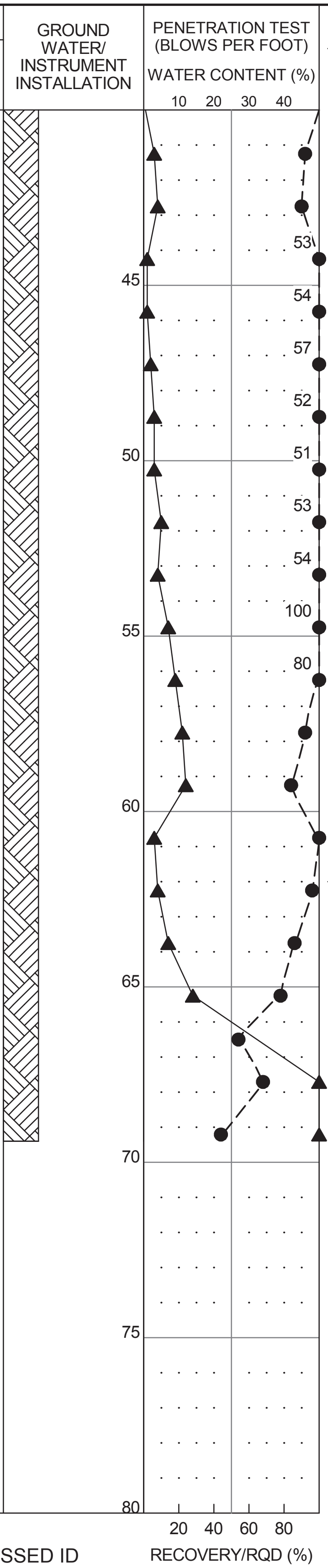
BIG CREEK DAMS
NEWPORT, OR

JAN 2014

PROJ 2328

FIG. A-1

ELEVATION IN FEET	DEPTH IN FEET	MATERIAL DESCRIPTION SURFACE ELEVATION: 33.0 FT.	SAMPLE NO.	SAMPLE PEN. DATA
-7.5	40.5			0
		VERY LOOSE to LOOSE, gray, silty SAND to VERY SOFT gray, slightly sandy, slightly clayey SILT; scattered mica, occasional organics, wet (ALLUVIUM)	S-27	0 2 1
			S-28	1 2 2
			S-29	0 0 1
			S-30	1 1 0
			S-31	0 0 2
			S-32	0 1 2
			S-33	0 1 2
		… scattered rounded course sand from 51.0 to 52.5 feet	S-34	1 2 3
			S-35	1 1 3
-21.0	54			
		LOOSE, gray, slightly clayey, silty SAND; numerous blue-gray and dark brown mottling, scattered organics and mica, occasional rounded coarse sand to fine gravel, wet (ALLUVIUM)	S-36	2 4 3
			S-37	2 3 6
-24.0	57			
		SOFT to MEDIUM STIFF, gray, slightly sandy, slightly clayey SILT; scattered mica and dark brown molttling, occasional organics and pockets of relict rock texture, wet (ALLUVIUM)	S-38	2 5 6
			S-39	3 5 7
			S-40	0 0 3
			S-41	1 2 2
-30.0	63			
		SOFT to MEDIUM STIFF, gray, silty SAND; trace clay, scattered mica, occasional organics, wet (DECOMPOSED NYE MUDSTONE)	S-42	3 2 5
			S-43	3 5 9
-33.0	66			
		VERY DENSE, gray, highly weathered to decomposed SILTSTONE (NYE MUDSTONE)	S-44	12 50
			S-45	50/5"
-36.4	69.4		S-46	50/5"
		Bottom of Boring: 69.4 FT		



LEGEND

▨	2-INCH O.D. SPLIT SPOON
☒	3-INCH O.D. SPLIT SPOON
▯	3-INCH O.D. THIN WALL SAMPLER
▥	3-INCH O.D. PITCHER TUBE SAMPLER
*	NO SAMPLE RECOVERY
▼ MM/DD/YY	GROUND WATER LEVEL AND DATE OBSERVED
├─●─┤	LIQUID LIMIT WATER CONTENT PLASTIC LIMIT
▲	STANDARD PENETRATION TEST (BLOWS/FT.)
●	WATER CONTENT IN PERCENT
▨	CORE RECOVERY IN PERCENT
■	RQD IN PERCENT
] PT-1	PACKER TEST INTERVAL

NOTES

1. MATERIAL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.

HAMMER ASSEMBLY: AUTO TRIP

SPT SAMPLER: NO LINER - RECESSED ID

DRILL ROD USED: AWJ

BOREHOLE DIAM.: 3 7/8"

DRILLER: WESTERN STATES

DATE START: 10/23/2013 FINISH: 10/24/2013

DRILLING TECHNIQUE: MUD ROTARY

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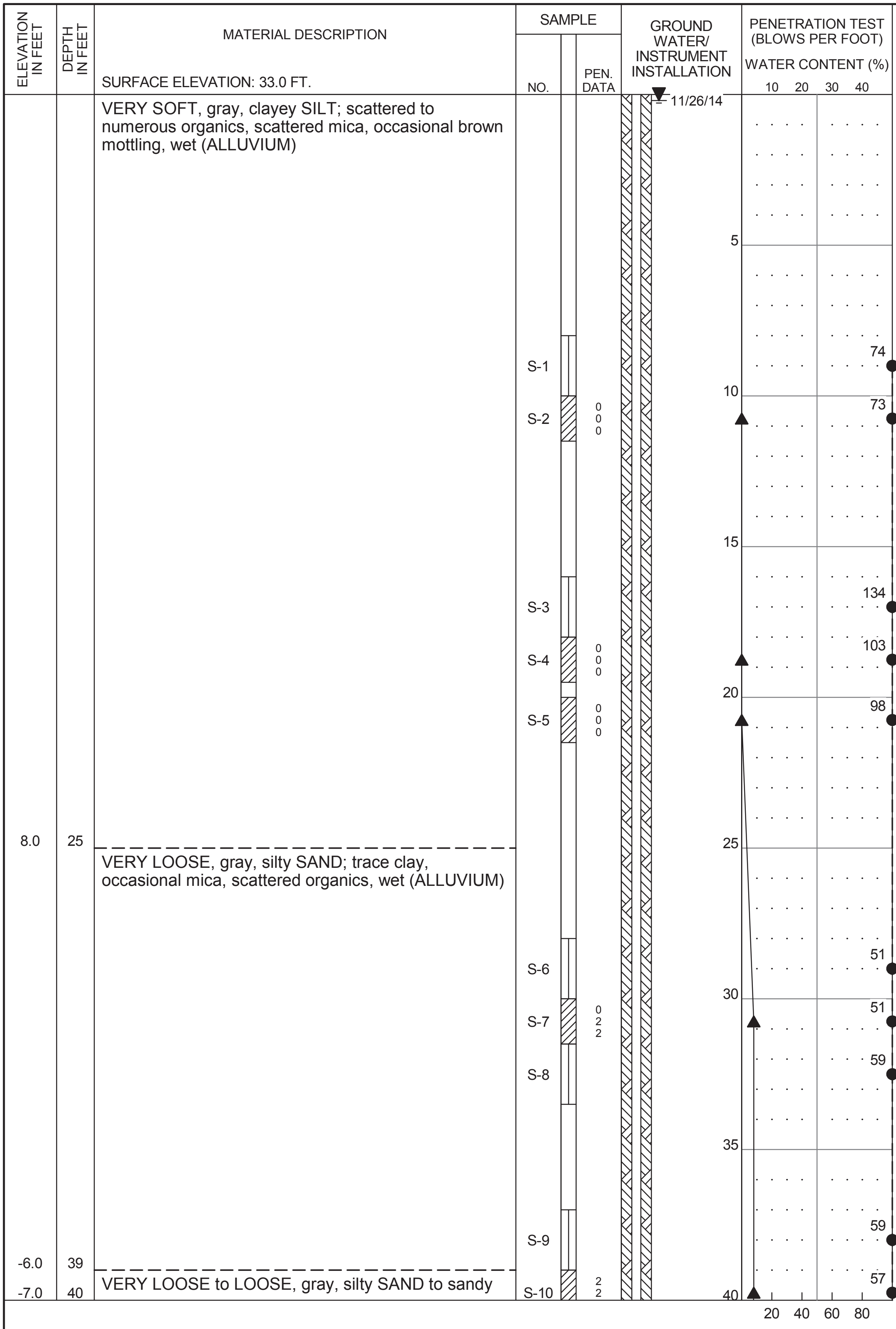
SUMMARY BORING LOG
BC1-BH-2 (2 of 2)

BIG CREEK DAMS
NEWPORT, OR

JAN 2014

PROJ 2328

FIG. A-2



VERY SOFT, gray, clayey SILT; scattered to numerous organics, scattered mica, occasional brown mottling, wet (ALLUVIUM)

VERY LOOSE, gray, silty SAND; trace clay, occasional mica, scattered organics, wet (ALLUVIUM)

VERY LOOSE to LOOSE, gray, silty SAND to sandy

LEGEND

▨	2-INCH O.D. SPLIT SPOON
☒	3-INCH O.D. SPLIT SPOON
▯	3-INCH O.D. THIN WALL SAMPLER
▥	3-INCH O.D. PITCHER TUBE SAMPLER
*	NO SAMPLE RECOVERY
▼ MM/DD/YY	GROUND WATER LEVEL AND DATE OBSERVED
⊢●⊣	LIQUID LIMIT WATER CONTENT PLASTIC LIMIT
▲	STANDARD PENETRATION TEST (BLOWS/FT.)
●	WATER CONTENT IN PERCENT
▨	CORE RECOVERY IN PERCENT
■	RQD IN PERCENT
] PT-1	PACKER TEST INTERVAL

NOTES

1. MATERIAL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.
2. 1-INCH STANDPIPE PIEZOMETER INSTALLED TO A DEPTH OF 61 FEET, SCREENED FROM 51 TO 61 FEET.

HAMMER ASSEMBLY: AUTO TRIP
SPT SAMPLER: NO LINER - RECESSED ID
DRILL ROD USED: NWJ
BOREHOLE DIAM.: 4 7/8"

DRILLER: WESTERN STATES
DATE START: 11/19/2013 FINISH: 11/20/2013
DRILLING TECHNIQUE: MUD ROTARY

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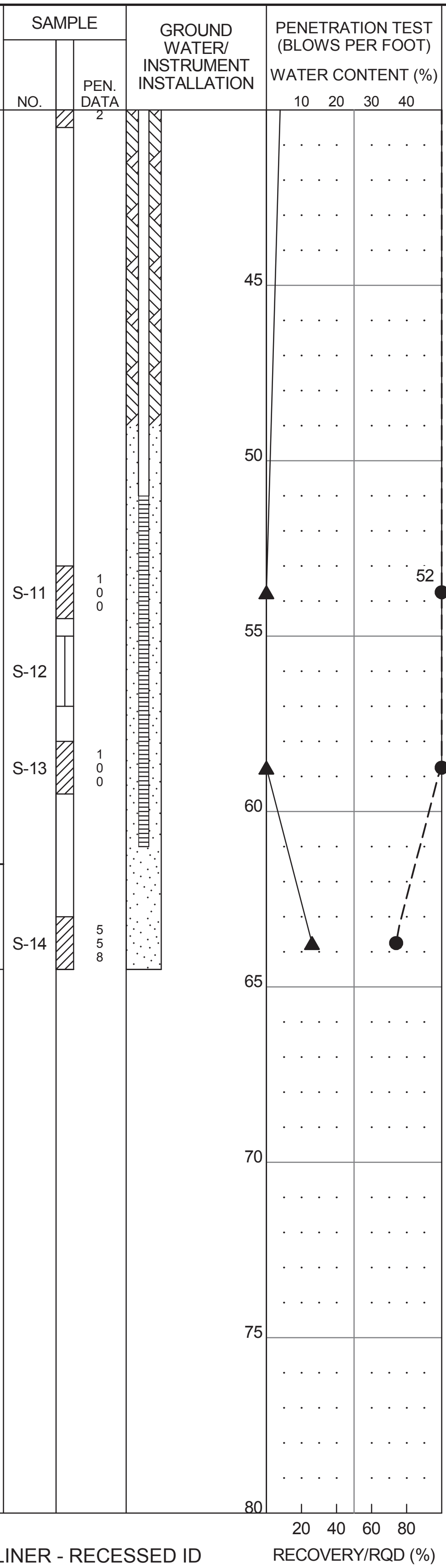
SUMMARY BORING LOG
BC1-BH-3(u) (1 of 2)

BIG CREEK DAMS
NEWPORT, OR

JAN 2014
PROJ 2328
FIG. A-3

ELEVATION IN FEET	DEPTH IN FEET	MATERIAL DESCRIPTION
		SURFACE ELEVATION: 33.0 FT.
		SILT; trace clay, occasional mica and organics, occasional rounded coarse sand to fine gravel, wet (ALLUVIUM)
		… woody from 48.0 to 50.5 feet
-28.5	61.5	MEDIUM DENSE, gray, highly weathered to decomposed SILTSTONE (DECOMPOSED NYE MUDSTONE)
-31.5	64.5	Bottom of Boring: 64.5 FT



LEGEND

- ▨ 2-INCH O.D. SPLIT SPOON
- ☒ 3-INCH O.D. SPLIT SPOON
- ▯ 3-INCH O.D. THIN WALL SAMPLER
- ▥ 3-INCH O.D. PITCHER TUBE SAMPLER
- * NO SAMPLE RECOVERY
- ▼ MM/DD/YY GROUND WATER LEVEL AND DATE OBSERVED
- LIQUID LIMIT, WATER CONTENT, PLASTIC LIMIT
- ▲ STANDARD PENETRATION TEST (BLOWS/FT.)
- ● WATER CONTENT IN PERCENT
- ▨ CORE RECOVERY IN PERCENT
- ■ RQD IN PERCENT
- PT-1 PACKER TEST INTERVAL

NOTES

1. MATERIAL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.
2. 1-INCH STANDPIPE PIEZOMETER INSTALLED TO A DEPTH OF 61 FEET, SCREENED FROM 51 TO 61 FEET.

HAMMER ASSEMBLY: AUTO TRIP
SPT SAMPLER: NO LINER - RECESSED ID
DRILL ROD USED: NWJ
BOREHOLE DIAM.: 4 7/8"

DRILLER: WESTERN STATES
DATE START: 11/19/2013 FINISH: 11/20/2013
DRILLING TECHNIQUE: MUD ROTARY

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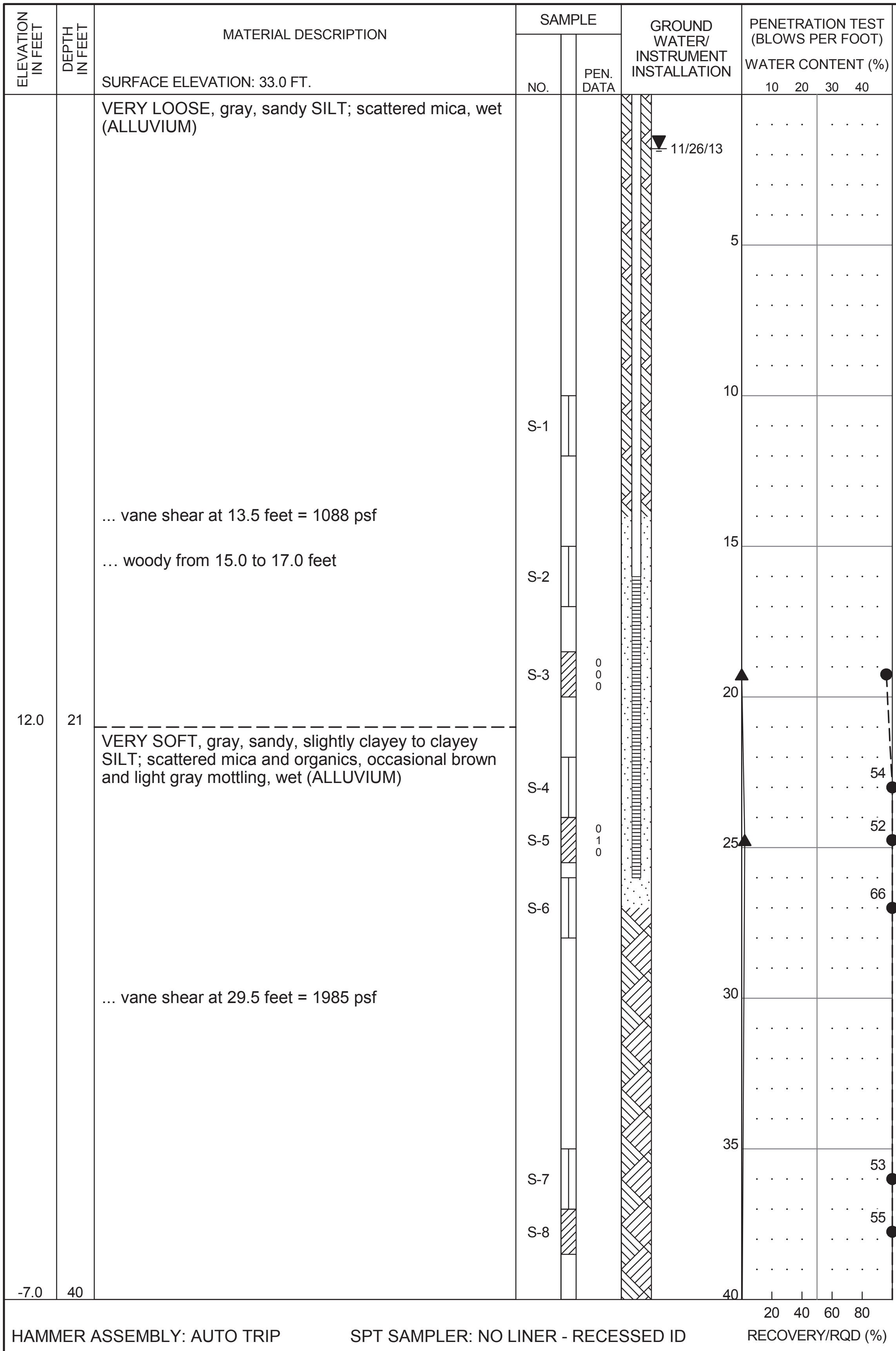
SUMMARY BORING LOG
BC1-BH-3(u) (2 of 2)

BIG CREEK DAMS
NEWPORT, OR

JAN 2014
PROJ 2328
FIG. A-4



SURFACE ELEVATION: 33.0 FT.

VERY LOOSE, gray, sandy SILT; scattered mica, wet (ALLUVIUM)

... vane shear at 13.5 feet = 1088 psf

… woody from 15.0 to 17.0 feet

VERY SOFT, gray, sandy, slightly clayey to clayey SILT; scattered mica and organics, occasional brown and light gray mottling, wet (ALLUVIUM)

... vane shear at 29.5 feet = 1985 psf

LEGEND

- ▨ 2-INCH O.D. SPLIT SPOON
- ☒ 3-INCH O.D. SPLIT SPOON
- ▯ 3-INCH O.D. THIN WALL SAMPLER
- ▥ 3-INCH O.D. PITCHER TUBE SAMPLER
- * NO SAMPLE RECOVERY
- ▼ MM/DD/YY GROUND WATER LEVEL AND DATE OBSERVED
- LIQUID LIMIT / WATER CONTENT / PLASTIC LIMIT
- ▲ STANDARD PENETRATION TEST (BLOWS/FT.)
- ● WATER CONTENT IN PERCENT
- ▨ CORE RECOVERY IN PERCENT
- ■ RQD IN PERCENT
-] PT-1 PACKER TEST INTERVAL

NOTES

1. MATERIAL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.
2. 1-INCH STANDPIPE PIEZOMETER INSTALLED TO A DEPTH OF 26 FEET, SCREENED FROM 16 TO 26 FEET.

HAMMER ASSEMBLY: AUTO TRIP
SPT SAMPLER: NO LINER - RECESSED ID
DRILL ROD USED: NWJ
BOREHOLE DIAM.: 4 7/8"

DRILLER: WESTERN STATES
DATE START: 11/20/2013 FINISH: 11/22/2013
DRILLING TECHNIQUE: MUD ROTARY

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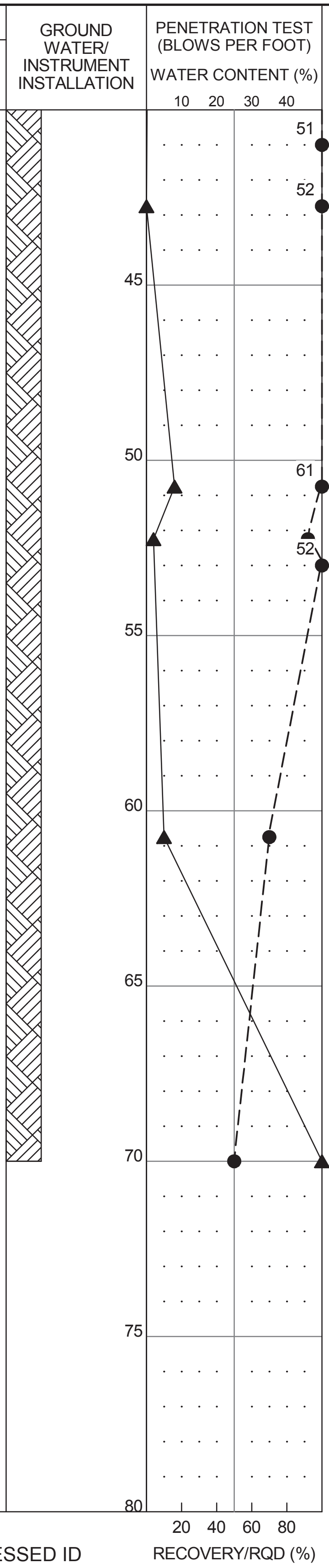
SUMMARY BORING LOG BC1-BH-4(u) (1 of 2)

BIG CREEK DAMS
NEWPORT, OR

JAN 2014
PROJ 2328
FIG. A-5

SURFACE ELEVATION: 33.0 FT.

ELEVATION IN FEET	DEPTH IN FEET	MATERIAL DESCRIPTION	SAMPLE NO.	SAMPLE PEN. DATA
		(continued from previous page)	S-9	
			S-10	0 0 0
-14.0	47	LOOSE, gray, silty SAND; numerous rounded coarse sand to fine gravel, scattered mica, occasional brown mottling and organics, wet (ALLUVIUM)		
			S-11	2 4 4
			S-12	2 1 1
-25.0	58	MEDIUM DENSE, gray, slightly sandy SILT; relict rock texture, numerous angular siltstone fragments, scattered mica, wet (DECOMPOSED NYE MUDSTONE)	S-13	7 5 0
-34.0	67	VERY DENSE, gray, highly weathered SILTSTONE (NYE MUDSTONE)		
-37.0	70	Bottom of Boring: 70 FT	S-14	39 50/6"



LEGEND

- ▨ 2-INCH O.D. SPLIT SPOON
- ☒ 3-INCH O.D. SPLIT SPOON
- ▯ 3-INCH O.D. THIN WALL SAMPLER
- ▥ 3-INCH O.D. PITCHER TUBE SAMPLER
- * NO SAMPLE RECOVERY
- ▼ MM/DD/YY GROUND WATER LEVEL AND DATE OBSERVED
- ├●┤ LIQUID LIMIT, WATER CONTENT, PLASTIC LIMIT
- ▲ STANDARD PENETRATION TEST (BLOWS/FT.)
- ● WATER CONTENT IN PERCENT
- ▨ CORE RECOVERY IN PERCENT
- ■ RQD IN PERCENT
-] PT-1 PACKER TEST INTERVAL

NOTES

1. MATERIAL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.
2. 1-INCH STANDPIPE PIEZOMETER INSTALLED TO A DEPTH OF 26 FEET, SCREENED FROM 16 TO 26 FEET.

HAMMER ASSEMBLY: AUTO TRIP
SPT SAMPLER: NO LINER - RECESSED ID
DRILL ROD USED: NWJ
BOREHOLE DIAM.: 4 7/8"

DRILLER: WESTERN STATES
DATE START: 11/20/2013 FINISH: 11/22/2013
DRILLING TECHNIQUE: MUD ROTARY

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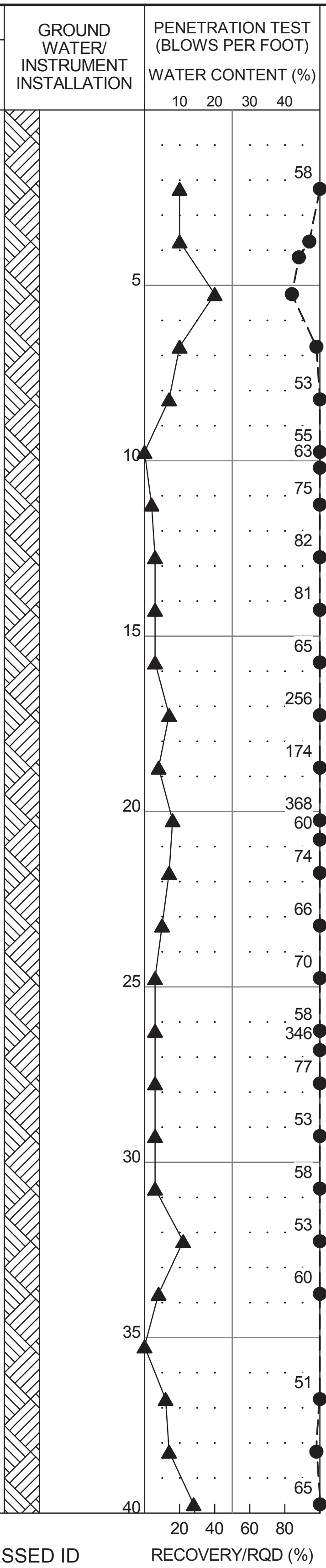
SUMMARY BORING LOG
BC1-BH-4(u) (2 of 2)

BIG CREEK DAMS
NEWPORT, OR

JAN 2014
PROJ 2328
FIG. A-6

ELEVATION IN FEET	DEPTH IN FEET	MATERIAL DESCRIPTION	SAMPLE NO.	PEN. DATA
		SURFACE ELEVATION: 50.0 FT.		
		MEDIUM STIFF to STIFF, brown and gray, slightly sandy, slightly clayey SILT; numerous angular coarse sand to fine gravel and orange mottling, scattered mica, moist (FILL)	S-1	5 5 5
			S-2	3 4 6
			S-3	5 8 12
44.0	6	MEDIUM STIFF to STIFF, brown and gray, slightly sandy, slightly clayey to clayey SILT; numerous orange mottling, scattered mica, occasional organics, moist (ALLUVIUM)	S-4	3 4 6
			S-5	2 3 4
40.4	9.6	VERY SOFT, dark gray, slightly clayey to clayey SILT; numerous gray mottling, wet (ALLUVIUM)	S-6	1 0 0
39.5	10.5	VERY SOFT, slightly sandy, slightly clayey SILT; scattered organics and mica, occasional rounded coarse sand, wet (ALLUVIUM/COLLUVIUM)	S-7	1 1 1
			S-8	0 1 2
			S-9	1 1 2
			S-10	1 2 1
		... woody from 16.0 to 20.0 feet	S-11	3 3 4
			S-12	0 1 3
			S-13	2 3 5
29.6	20.4	VERY LOOSE to LOOSE, gray, silty SAND to VERY SOFT, slightly sandy, silty to very silty CLAY; scattered mica and organics, trace angular sand to fine gravel- sized rock fragments, wet (ALLUVIUM/COLLUVIUM)	S-14	5 3 4
			S-15	2 2 3
			S-16	1 1 2
		… woody debris from 26.4 to 27.0 feet	S-17	2 2 1
			S-18	1 0 3
			S-19	0 1 2
			S-20	1 1 2
		… medium dense with relict rock texture and numerous angular coarse sand to fine gravel from 31.5 to 33.0 feet	S-21	6 5 6
			S-22	3 1 3
			S-23 *	0 0 0
			S-24	3 3 3
			S-25	4 4 3
11.0	39		S-26	6 6



LEGEND

▨ 2-INCH O.D. SPLIT SPOON

☒ 3-INCH O.D. SPLIT SPOON

▯ 3-INCH O.D. THIN WALL SAMPLER

▥ 3-INCH O.D. PITCHER TUBE SAMPLER

* NO SAMPLE RECOVERY

▼ MM/DD/YY GROUND WATER LEVEL AND DATE OBSERVED

LIQUID LIMIT

WATER CONTENT

PLASTIC LIMIT

▲ STANDARD PENETRATION TEST (BLOWS/FT.)

● WATER CONTENT IN PERCENT

▨ CORE RECOVERY IN PERCENT

■ RQD IN PERCENT

PT-1 PACKER TEST INTERVAL

NOTES

1. MATERIAL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.

HAMMER ASSEMBLY: AUTO TRIP

SPT SAMPLER: NO LINER - RECESSED ID

DRILL ROD USED: AWJ

BOREHOLE DIAM.: 3 7/8"

DRILLER: WESTERN STATES
DATE START: 10/24/2013 FINISH: 10/25/2013
DRILLING TECHNIQUE: MUD ROTARY

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SUMMARY BORING LOG
BC2-BH-4 (1 of 2)

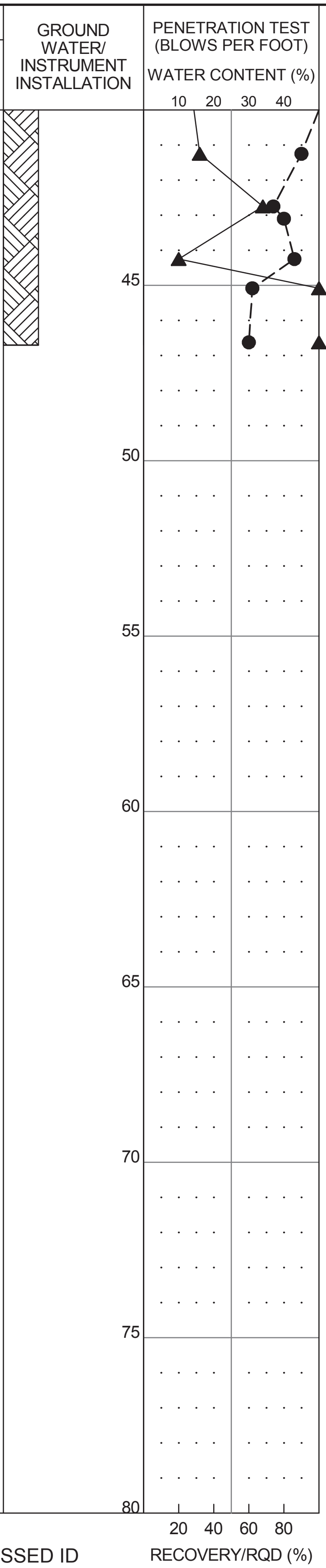
BIG CREEK DAMS
NEWPORT, OR

JAN 2014

PROJ 2328

FIG. A-7

ELEVATION IN FEET	DEPTH IN FEET	MATERIAL DESCRIPTION	SAMPLE NO.	PEN. DATA
		SURFACE ELEVATION: 50.0 FT.		
		MEDIUM DENSE to DENSE, gray, sandy SILT; trace clay, relict rock texture, numerous coarse sand- to fine gravel-sized rock fragments, scattered mica, occasional organics, wet (DECOMPOSED NYE MUDSTONE)		8
			S-27	10 9 7
			S-28	9 17 17
			S-29	6 6 4
5.0	45		S-30	50/2"
		VERY DENSE, gray, highly weathered SILTSTONE (NYE MUDSTONE)		
3.3	46.7		S-31	50/3"
		Bottom of Boring: 46.7 FT		



LEGEND

- ▨ 2-INCH O.D. SPLIT SPOON
- ☒ 3-INCH O.D. SPLIT SPOON
- ▯ 3-INCH O.D. THIN WALL SAMPLER
- ▥ 3-INCH O.D. PITCHER TUBE SAMPLER
- * NO SAMPLE RECOVERY
- MM/DD/YY ▼ GROUND WATER LEVEL AND DATE OBSERVED
- LIQUID LIMIT
- WATER CONTENT
- PLASTIC LIMIT
- ▲ STANDARD PENETRATION TEST (BLOWS/FT.)
- ● WATER CONTENT IN PERCENT
- ▨ CORE RECOVERY IN PERCENT
- ■ RQD IN PERCENT
-] PT-1 PACKER TEST INTERVAL

NOTES

1. MATERIAL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.

HAMMER ASSEMBLY: AUTO TRIP
SPT SAMPLER: NO LINER - RECESSED ID
DRILL ROD USED: AWJ
BOREHOLE DIAM.: 3 7/8"

DRILLER: WESTERN STATES
DATE START: 10/24/2013 FINISH: 10/25/2013
DRILLING TECHNIQUE: MUD ROTARY

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SUMMARY BORING LOG BC2-BH-4 (2 of 2)

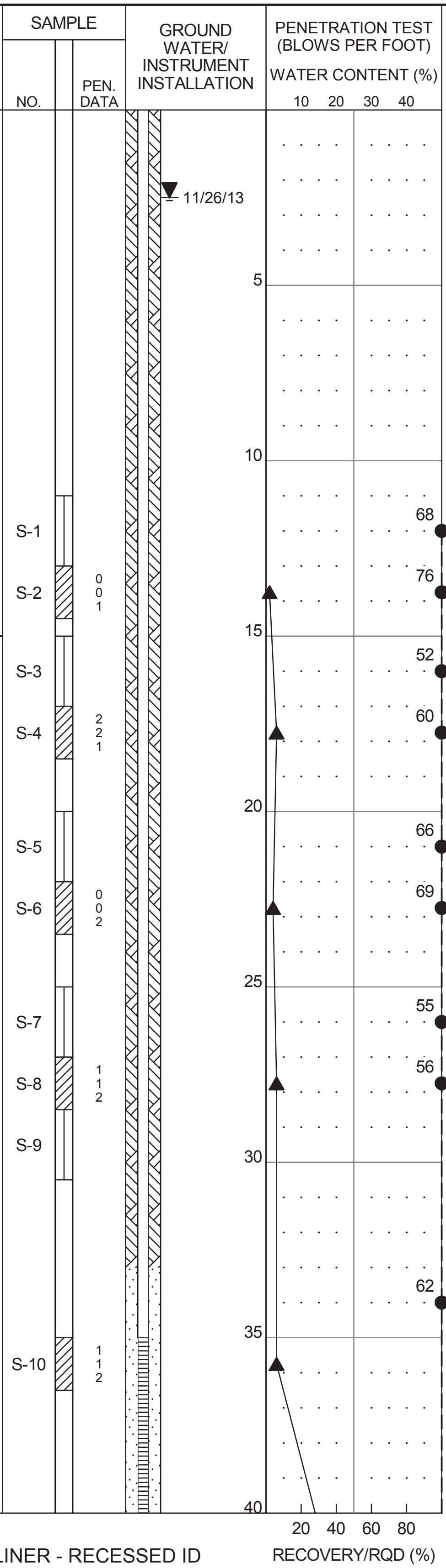
BIG CREEK DAMS
NEWPORT, OR

JAN 2014

PROJ 2328

FIG. A-8

ELEVATION IN FEET	DEPTH IN FEET	MATERIAL DESCRIPTION SURFACE ELEVATION: 50.0 FT.
		VERY LOOSE, gray, slightly silty SAND and VERY SOFT, dark gray, clayey SILT; rounded medium to coarse sand, numerous organics, wet (ALLUVIUM)
35.0	15	VERY LOOSE, gray, slightly silty to silty SAND; trace clay, numerous rounded coarse sand to fine gravel, scattered to numerous organics, scattered mica and brown mottling, wet (ALLUVIUM/COLLUVIUM)
		… stiffer drilling at 37 feet



LEGEND

- ▨ 2-INCH O.D. SPLIT SPOON
- ☒ 3-INCH O.D. SPLIT SPOON
- ▯ 3-INCH O.D. THIN WALL SAMPLER
- ▥ 3-INCH O.D. PITCHER TUBE SAMPLER
- * NO SAMPLE RECOVERY
- ▼ MM/DD/YY GROUND WATER LEVEL AND DATE OBSERVED
- ├●┤ LIQUID LIMIT, WATER CONTENT, PLASTIC LIMIT
- ▲ STANDARD PENETRATION TEST (BLOWS/FT.)
- ● WATER CONTENT IN PERCENT
- ▨ CORE RECOVERY IN PERCENT
- ■ RQD IN PERCENT
-] PT-1 PACKER TEST INTERVAL

NOTES

1. MATERIAL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.
2. 1-INCH STANDPIPE PIEZOMETER INSTALLED TO A DEPTH OF 45 FEET, SCREENED FROM 35 TO 45 FEET.

HAMMER ASSEMBLY: AUTO TRIP
SPT SAMPLER: NO LINER - RECESSED ID
DRILL ROD USED: NWJ
BOREHOLE DIAM.: 4 7/8"

DRILLER: WESTERN STATES
DATE START: 11/25/2013 FINISH: 11/25/2013
DRILLING TECHNIQUE: MUD ROTARY

CORNFORTH CONSULTANTS
10250 S.W. Greenburg Road, Suite 111
Portland, Oregon 97223
Phone 503-452-1100 Fax 503-452-1528

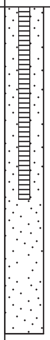
SUMMARY BORING LOG BC2-BH-5(u) (1 of 2)

BIG CREEK DAMS
NEWPORT, OR

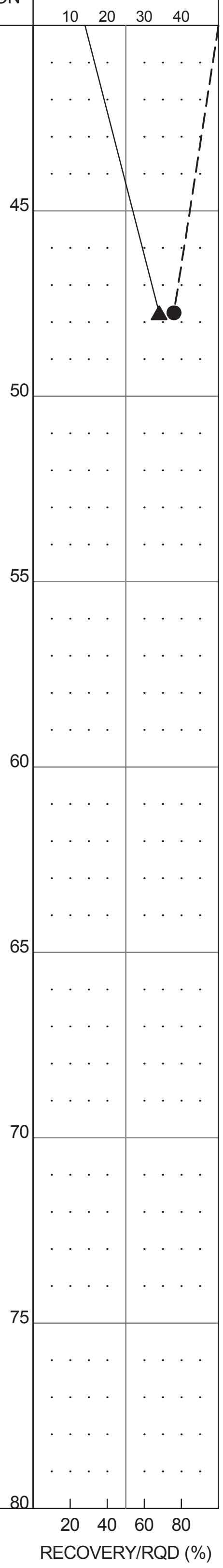
JAN 2014
PROJ 2328
FIG. A-9

ELEVATION IN FEET	DEPTH IN FEET	MATERIAL DESCRIPTION SURFACE ELEVATION: 50.0 FT.	SAMPLE NO.		SAMPLE PEN. DATA
		VERY LOOSE, gray, slightly silty to silty SAND; trace clay, numerous rounded coarse sand to fine gravel, scattered to numerous organics, scattered mica and brown mottling, wet (ALLUVIUM/COLLUVIUM)			
5.0	45	DENSE, gray, highly weathered SILTSTONE (NYE MUDSTONE)			
			S-11	▨	10 11 23
1.5	48.5	Bottom of Boring: 48.5 FT			

GROUND WATER/ INSTRUMENT INSTALLATION



PENETRATION TEST (BLOWS PER FOOT)
WATER CONTENT (%)



RECOVERY/RQD (%)

LEGEND

- ▨ 2-INCH O.D. SPLIT SPOON
- ☒ 3-INCH O.D. SPLIT SPOON
- ▯ 3-INCH O.D. THIN WALL SAMPLER
- ▥ 3-INCH O.D. PITCHER TUBE SAMPLER
- * NO SAMPLE RECOVERY
- ▼ MM/DD/YY GROUND WATER LEVEL AND DATE OBSERVED
- ├─●─┤ LIQUID LIMIT, WATER CONTENT, PLASTIC LIMIT
- ▲ STANDARD PENETRATION TEST (BLOWS/FT.)
- ● WATER CONTENT IN PERCENT
- ▨ CORE RECOVERY IN PERCENT
- ■ RQD IN PERCENT
-] PT-1 PACKER TEST INTERVAL

NOTES

1. MATERIAL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.
2. 1-INCH STANDPIPE PIEZOMETER INSTALLED TO A DEPTH OF 45 FEET, SCREENED FROM 35 TO 45 FEET.

HAMMER ASSEMBLY: AUTO TRIP
SPT SAMPLER: NO LINER - RECESSED ID
DRILL ROD USED: NWJ
BOREHOLE DIAM.: 4 7/8"

DRILLER: WESTERN STATES
DATE START: 11/25/2013 FINISH: 11/25/2013
DRILLING TECHNIQUE: MUD ROTARY

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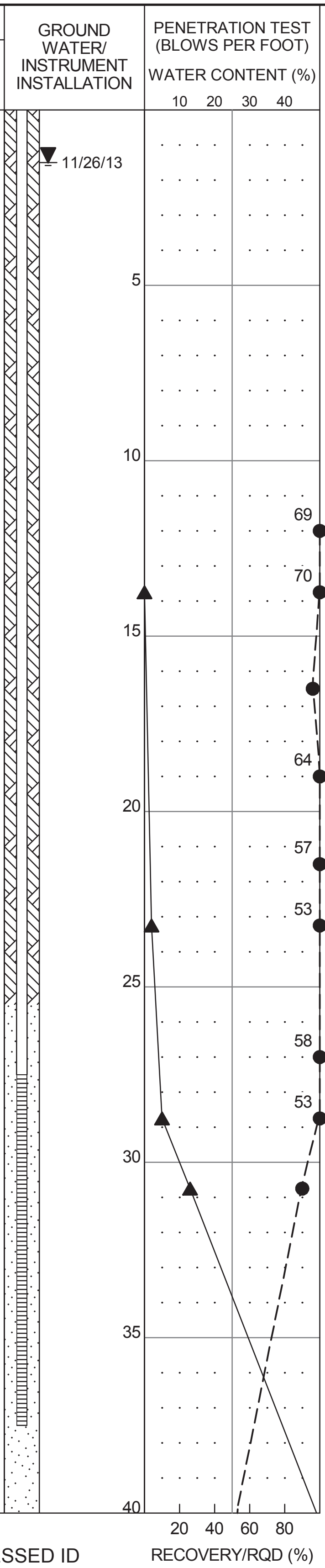
SUMMARY BORING LOG
BC2-BH-5(u) (2 of 2)

BIG CREEK DAMS
NEWPORT, OR

JAN 2014
PROJ 2328
FIG. A-10

ELEVATION IN FEET	DEPTH IN FEET	MATERIAL DESCRIPTION SURFACE ELEVATION: 50.0 FT.
		VERY LOOSE to LOOSE, gray, silty SAND and sandy SILT; scattered to numerous rounded coarse sand to fine gravel, scattered to numerous organics, scattered mica, wet (ALLUVIUM/COLLUVIUM)
20.0	30	MEDIUM DENSE, gray, slightly sandy SILT; trace clay, relict rock texture, wet (DECOMPOSED NYE MUDSTONE)
12.5	37.5	VERY DENSE, gray and brown, highly weathered SILTSTONE (NYE MUDSTONE)

SAMPLE NO.	PEN. DATA
S-1	
S-2	0 0 0
S-3	
S-4	
S-5	
S-6	0 0 2
S-7	
S-8	2 1 4
S-9	3 3 10



LEGEND

- ▨ 2-INCH O.D. SPLIT SPOON
- ☒ 3-INCH O.D. SPLIT SPOON
- ▯ 3-INCH O.D. THIN WALL SAMPLER
- ▥ 3-INCH O.D. PITCHER TUBE SAMPLER
- * NO SAMPLE RECOVERY
- ▼ MM/DD/YY GROUND WATER LEVEL AND DATE OBSERVED
- ├─●─┤ LIQUID LIMIT, WATER CONTENT, PLASTIC LIMIT
- ▲ STANDARD PENETRATION TEST (BLOWS/FT.)
- ● WATER CONTENT IN PERCENT
- ▨ CORE RECOVERY IN PERCENT
- ■ RQD IN PERCENT
-] PT-1 PACKER TEST INTERVAL

NOTES

1. MATERIAL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.
2. 1-INCH STANDPIPE PIEZOMETER INSTALLED TO A DEPTH OF 37.5 FEET, SCREENED FROM 27.5 TO 37.5 FEET.

HAMMER ASSEMBLY: AUTO TRIP
SPT SAMPLER: NO LINER - RECESSED ID
DRILL ROD USED: NWJ
BOREHOLE DIAM.: 4 7/8"

DRILLER: WESTERN STATES
DATE START: 11/26/2013 FINISH: 11/26/2013
DRILLING TECHNIQUE: MUD ROTARY

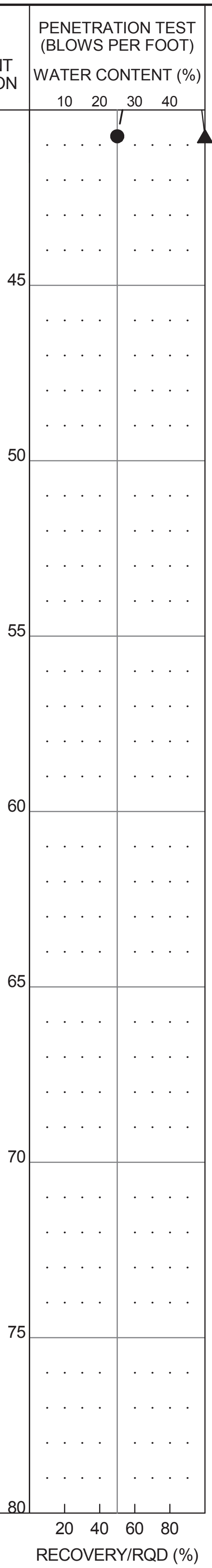
CORNFORTH CONSULTANTS
10250 S.W. Greenburg Road, Suite 111
Portland, Oregon 97223
Phone 503-452-1100 Fax 503-452-1528

SUMMARY BORING LOG
BC2-BH-6(u) (1 of 2)

BIG CREEK DAMS
NEWPORT, OR

JAN 2014
PROJ 2328
FIG. A-11

ELEVATION IN FEET	DEPTH IN FEET	MATERIAL DESCRIPTION	SAMPLE			GROUND WATER/ INSTRUMENT INSTALLATION
		SURFACE ELEVATION: 50.0 FT.	NO.		PEN. DATA	
8.5	41.5	VERY DENSE, gray and brown, highly weathered SILTSTONE (NYE MUDSTONE)	S-10	▨	18 26 26	⁘
		Bottom of Boring: 41.5 FT				



LEGEND

▨ 2-INCH O.D. SPLIT SPOON

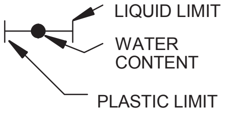
☒ 3-INCH O.D. SPLIT SPOON

▯ 3-INCH O.D. THIN WALL SAMPLER

▥ 3-INCH O.D. PITCHER TUBE SAMPLER

* NO SAMPLE RECOVERY

▼ MM/DD/YY GROUND WATER LEVEL AND DATE OBSERVED



▲ STANDARD PENETRATION TEST (BLOWS/FT.)

● WATER CONTENT IN PERCENT

▨ CORE RECOVERY IN PERCENT

■ RQD IN PERCENT

] PT-1 PACKER TEST INTERVAL

NOTES

1. MATERIAL DESCRIPTIONS AND INTERFACES ARE INTERPRETIVE AND ACTUAL CHANGES MAY BE GRADUAL.
2. 1-INCH STANDPIPE PIEZOMETER INSTALLED TO A DEPTH OF 37.5 FEET, SCREENED FROM 27.5 TO 37.5 FEET.

HAMMER ASSEMBLY: AUTO TRIP
SPT SAMPLER: NO LINER - RECESSED ID
DRILL ROD USED: NWJ
BOREHOLE DIAM.: 4 7/8"

DRILLER: WESTERN STATES
DATE START: 11/26/2013 FINISH: 11/26/2013
DRILLING TECHNIQUE: MUD ROTARY

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SUMMARY BORING LOG BC2-BH-6(u) (2 of 2)

BIG CREEK DAMS
NEWPORT, OR

JAN 2014

PROJ 2328

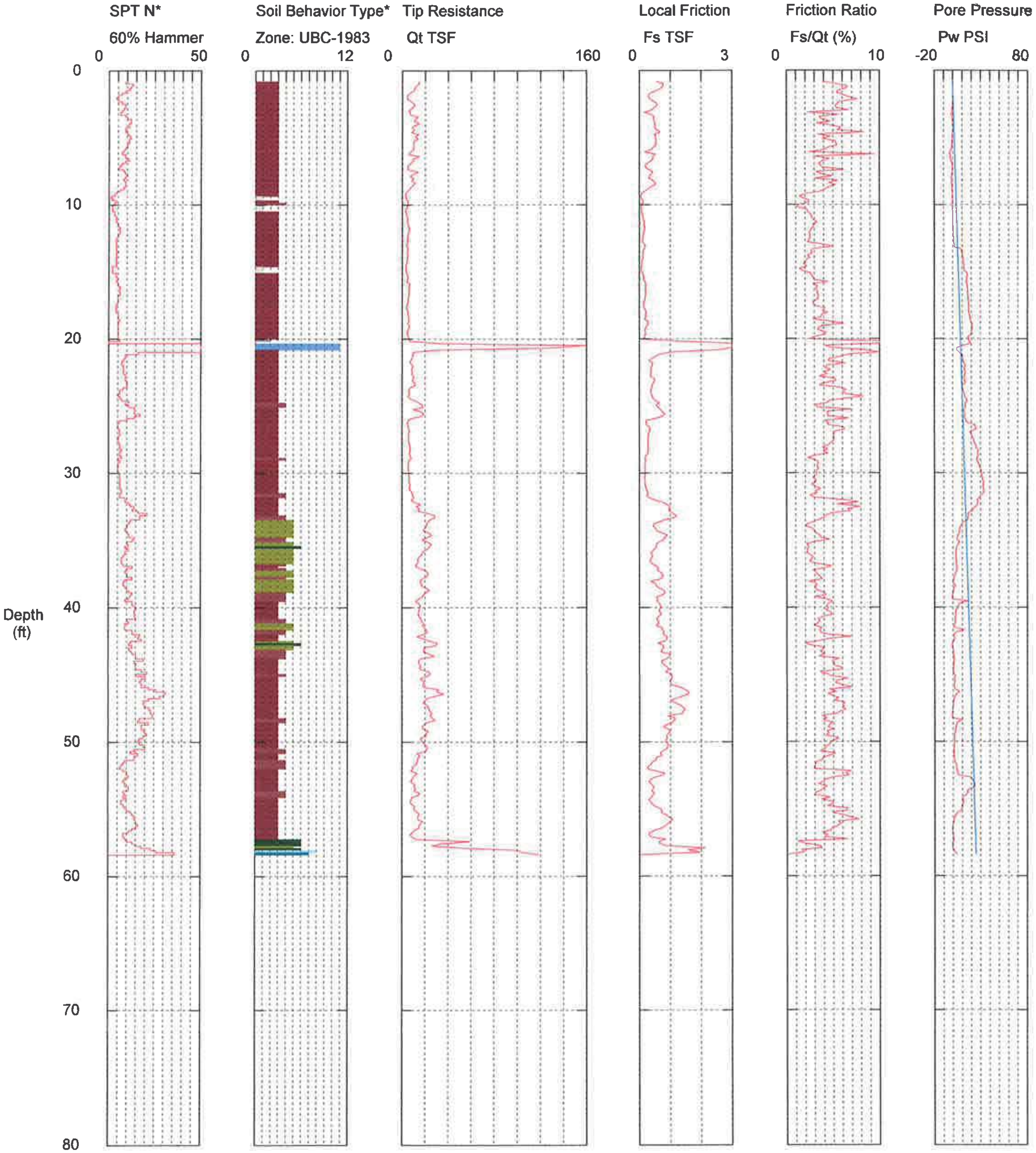
FIG. A-12

Appendix B – Seismic Cone Penetrometer Test Logs

HDR ENG. / BC1-SCPT-5 / BIG CK. DAM NEWPORT

Operator: SAV/CM
Sounding: VEI434BC1SCPT5(483)
Cone Used: DSG0736

CPT Date/Time: 10/23/2013 2:47:53 AM
Location: BC1,SCPT5 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT



Maximum Depth = 58.40 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

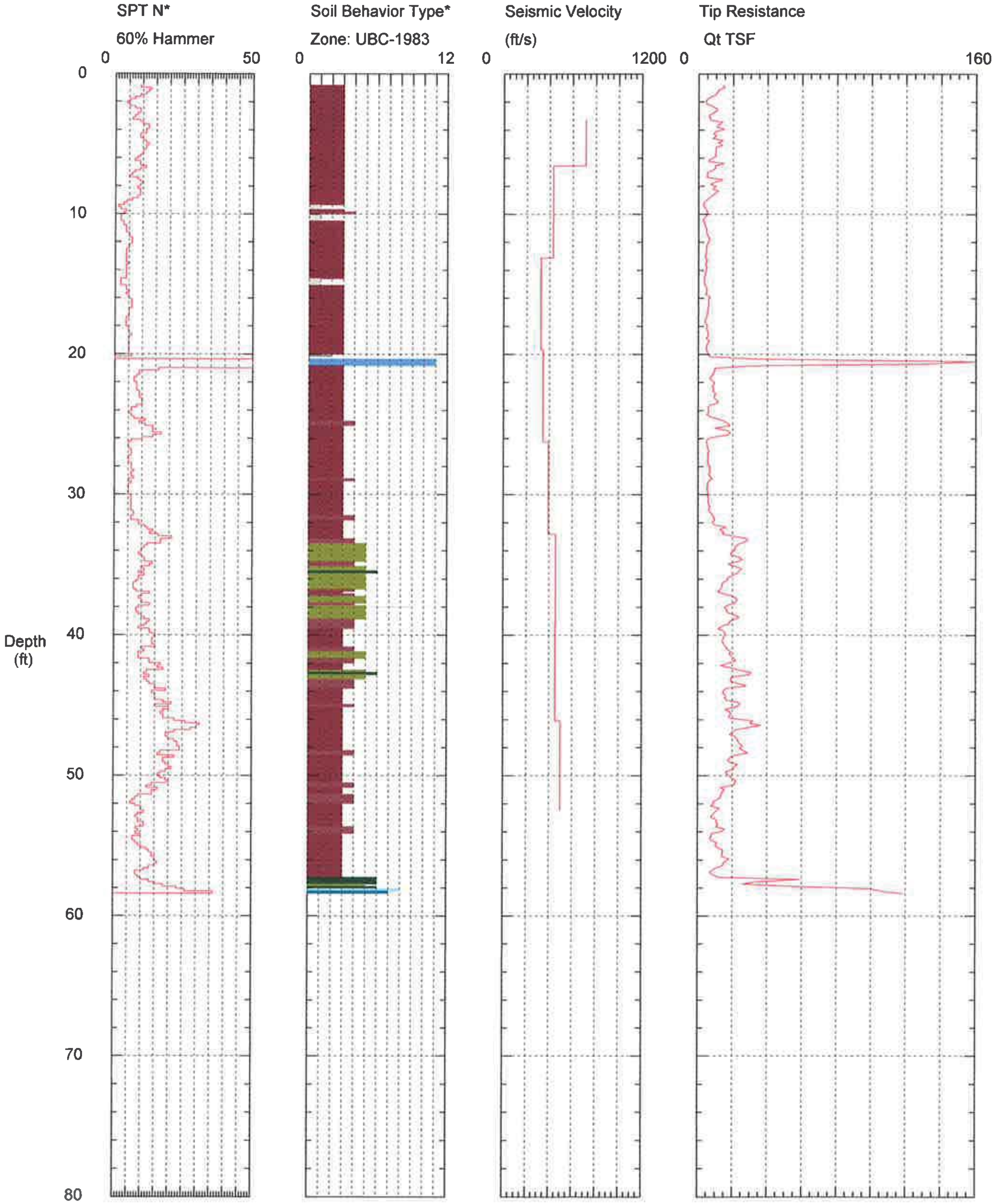
🟥 10 gravelly sand to sand
🟦 11 very stiff fine grained (*)
🟦 12 sand to clayey sand (*)

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

HDR ENG. / BC1-SCPT-5 / BIG CK. DAM NEWPORT

Operator: SAV/CM
Sounding: VEI434BC1SCPT5(483)
Cone Used: DSG0736

CPT Date/Time: 10/23/2013 2:47:53 AM
Location: BC1,SCPT5 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT



Maximum Depth = 58.40 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 (*)

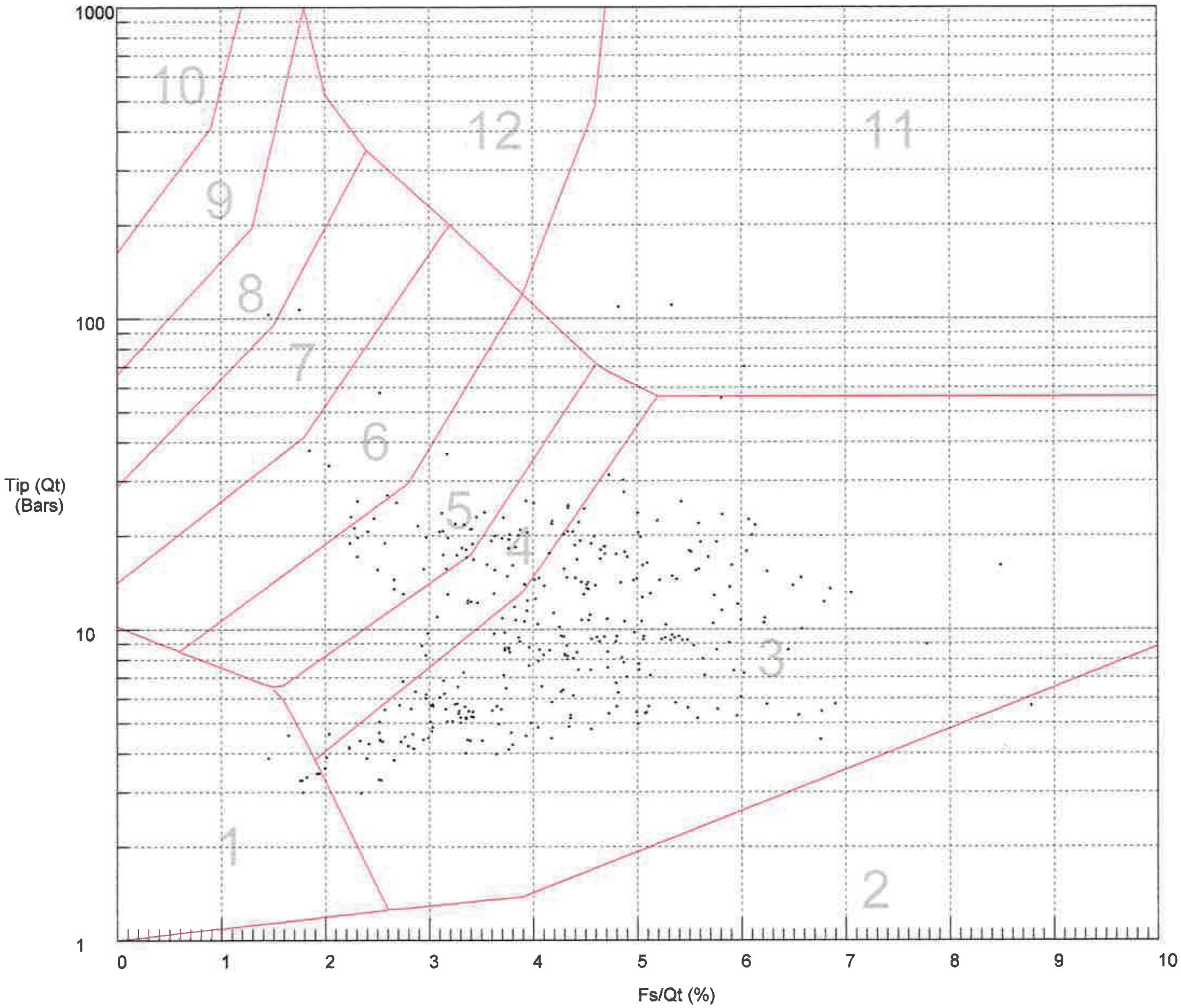
*Soil behavior type and SPT based on data from UBC-1983

HDR ENG. / BC1-SCPT-5 / BIG CK. DAM NEWPORT

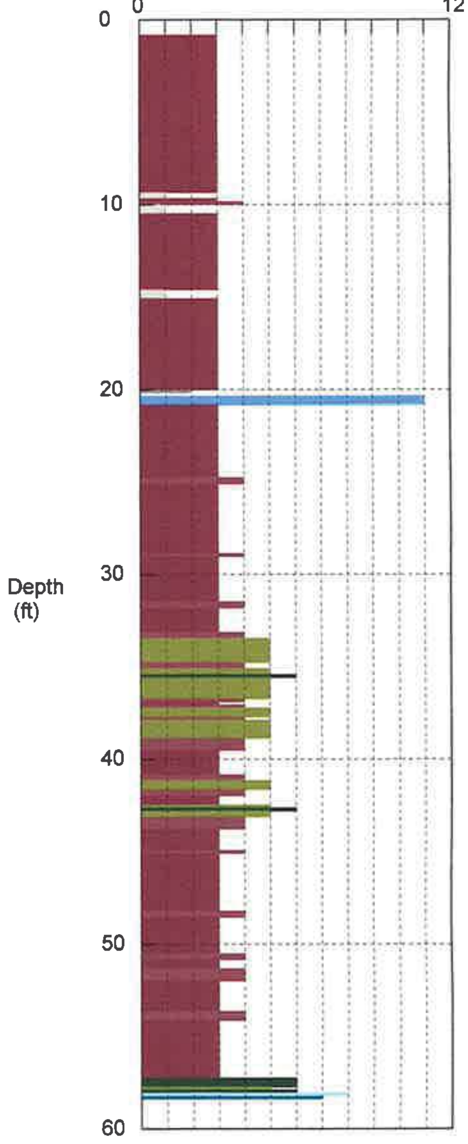
Operator: SAV/CM
Sounding: VEI434BC1SCPT5(483)
Cone Used: DSG0736

CPT Date/Time: 10/23/2013 2:47:53 AM
Location: BC1,SCPT5 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT

Classification Data:
Robertson and Campanella UBC-1983



Soil Behavior Type*
Zone: UBC-1983

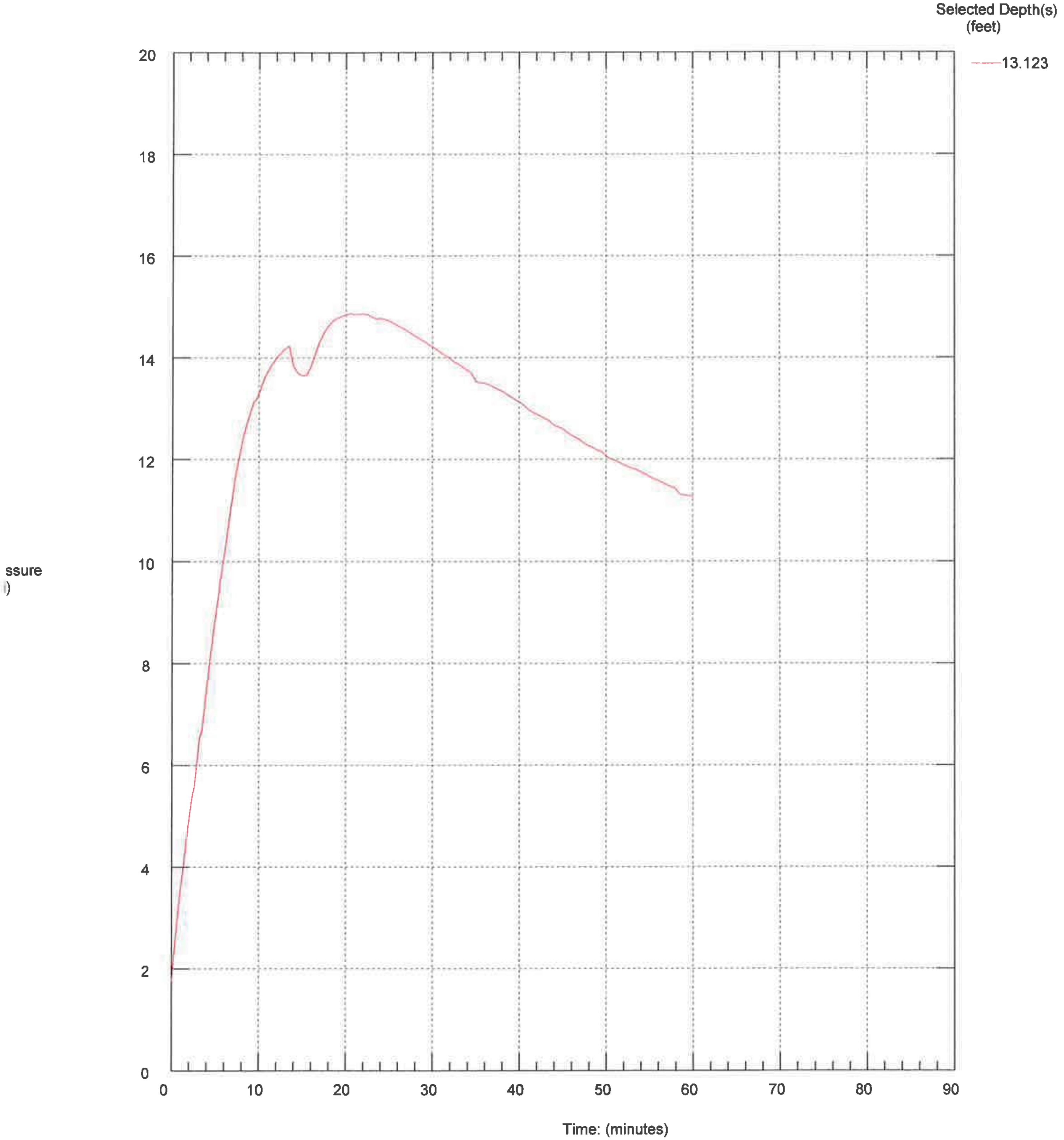


- ⬜ 1 sensitive fine grained
- ⬛ 2 organic material
- 🟥 3 clay
- 🟥 4 silty clay to clay
- 🟩 5 clayey silt to silty clay
- ⬛ 6 sandy silt to clayey silt
- 🟦 7 silty sand to sandy silt
- 🟦 8 sand to silty sand
- 🟧 9 sand
- 🟥 10 gravelly sand to sand
- 🟦 11 very stiff fine grained (*)
- 🟦 12 sand to clayey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

HDR ENG. / BC1-SCPT-5 / BIG CK. DAM NEWPORT

Operator SAV/CM
Sounding: VEI434BC1SCPT5(483)
Cone Used: DSG0736

CPT Date/Time: 10/23/2013 2:47:53 AM
Location: BC1SCPT5 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT

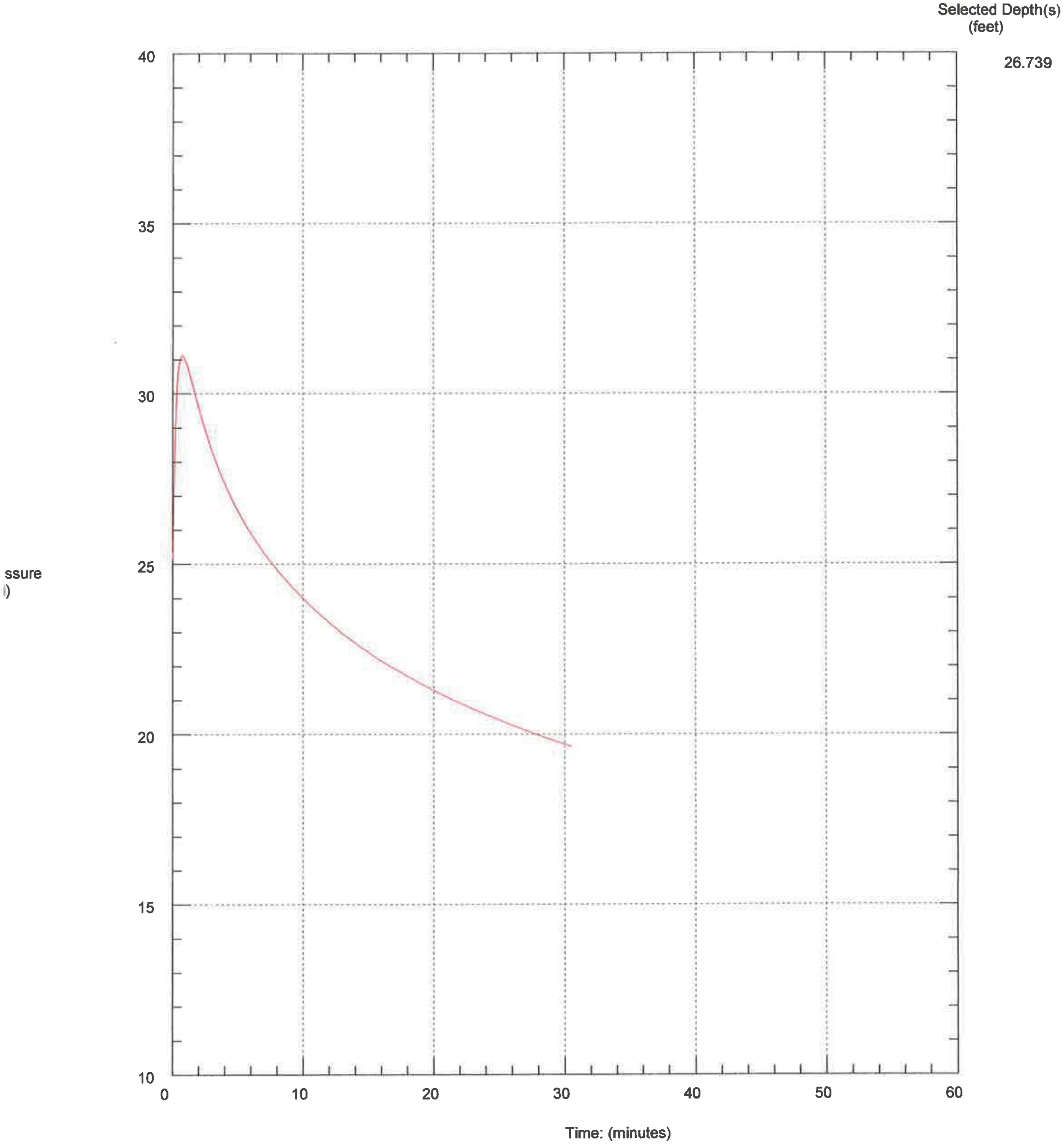


Maximum Pressure = 14.868 psi
Hydrostatic Pressure

HDR ENG. / BC1-SCPT-5 / BIG CK. DAM NEWPORT

Operator SAV/CM
Sounding: VEI434BC1SCPT5(483)
Cone Used: DSG0736

CPT Date/Time: 10/23/2013 2:47:53 AM
Location: BC1SCPT5 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT

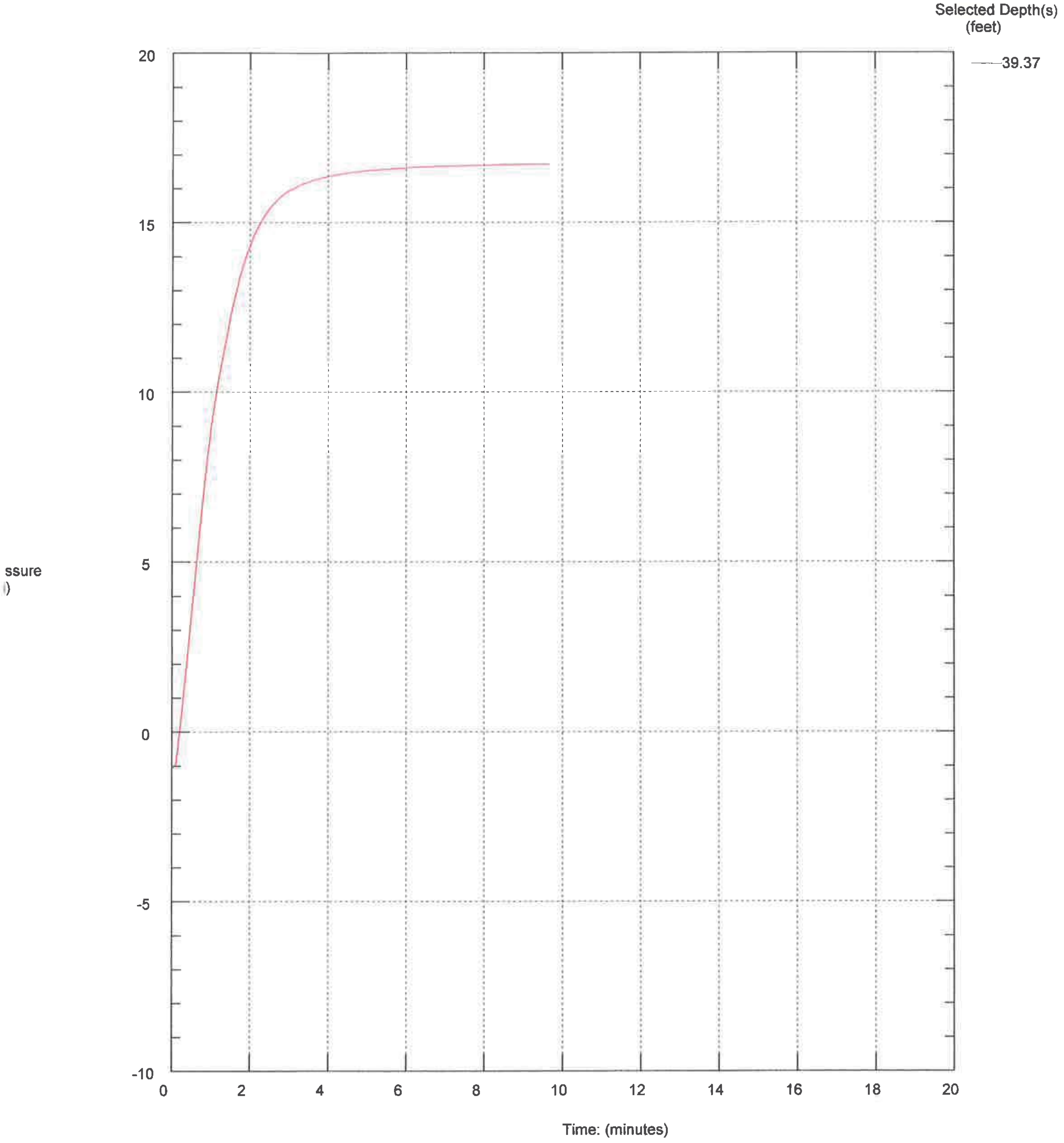


Maximum Pressure = 31.126 psi
Hydrostatic Pressure

HDR ENG. / BC1-SCPT-5 / BIG CK. DAM NEWPORT

Operator SAV/CM
Sounding: VEI434BC1SCPT5(483)
Cone Used: DSG0736

CPT Date/Time: 10/23/2013 2:47:53 AM
Location: BC1SCPT5 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT

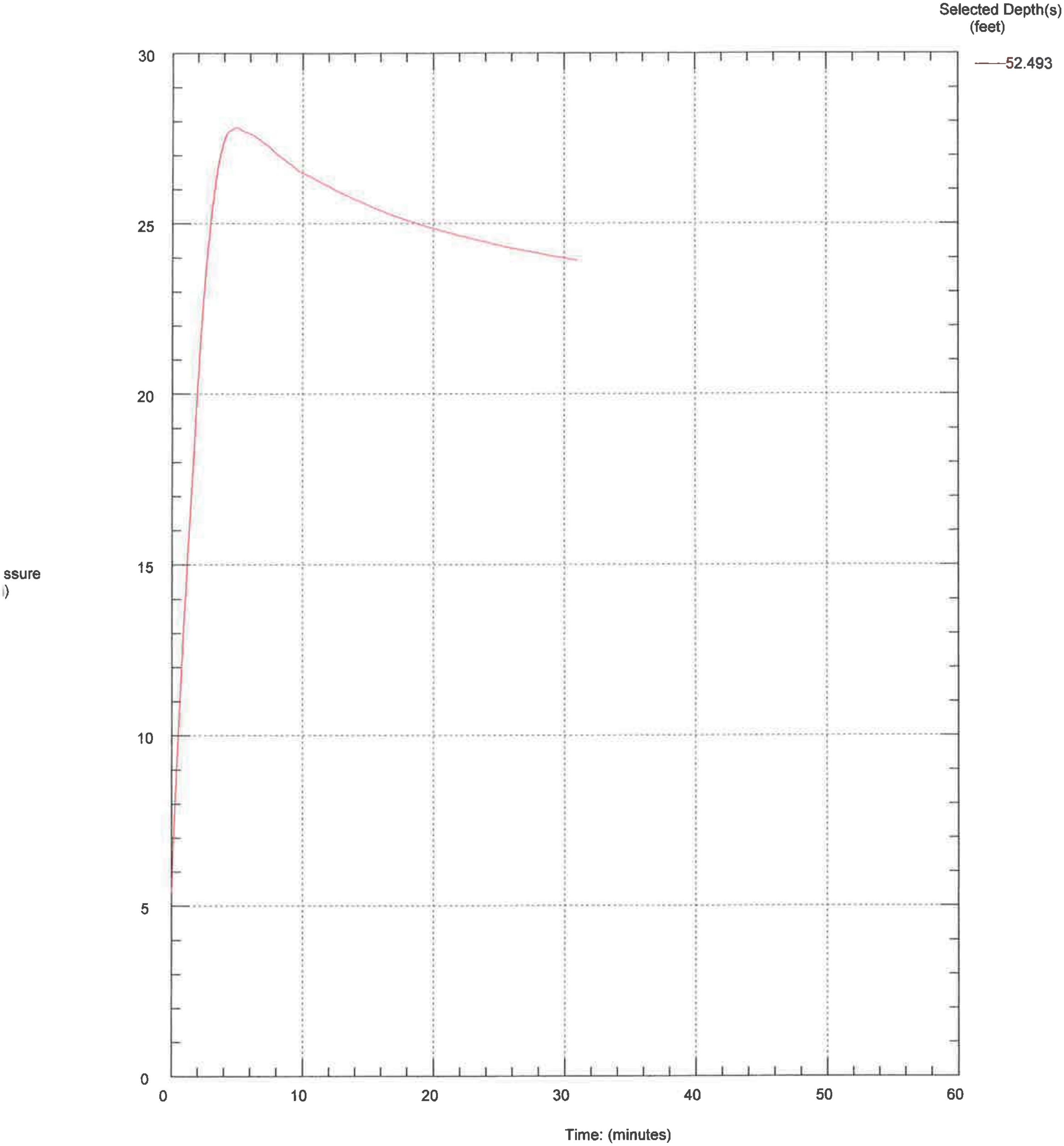


Maximum Pressure = 16.719 psi
Hydrostatic Pressure =

HDR ENG. / BC1-SCPT-5 / BIG CK. DAM NEWPORT

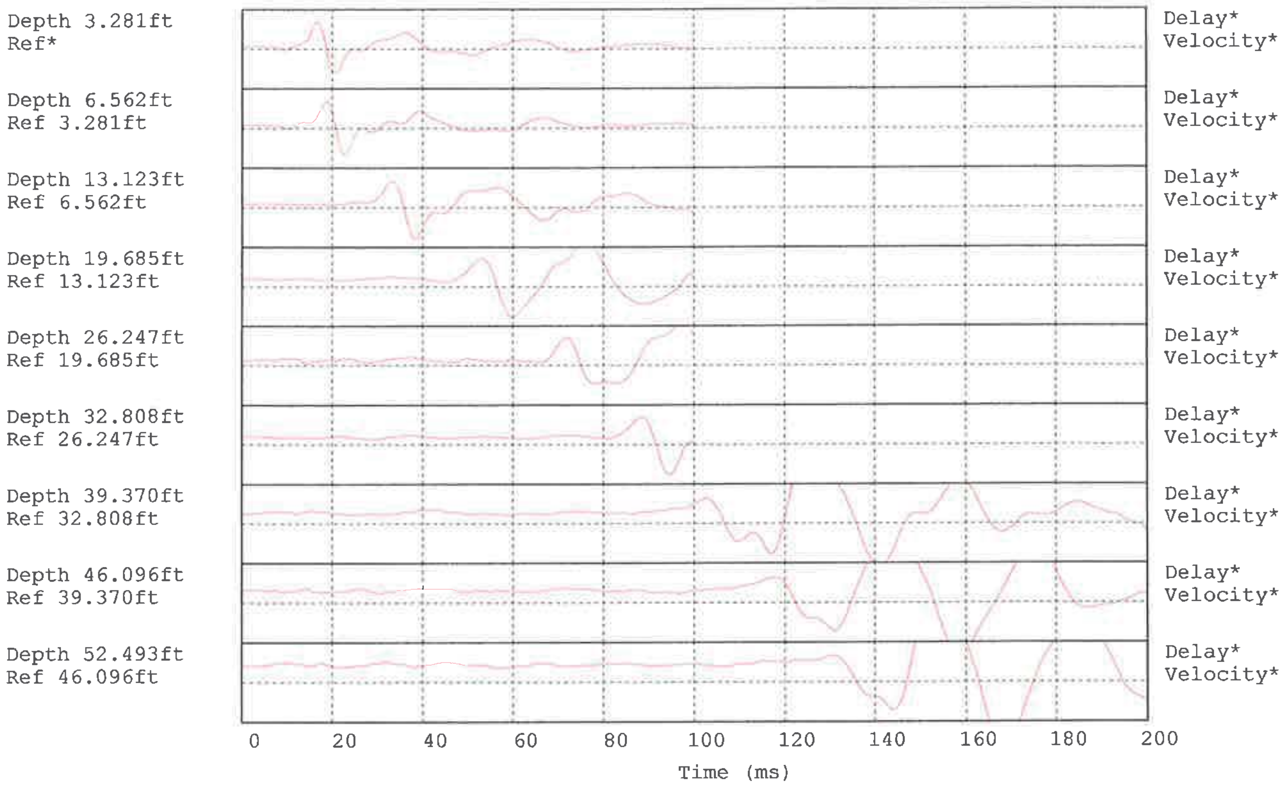
Operator SAV/CM
Sounding: VEI434BC1SCPT5(483)
Cone Used: DSG0736

CPT Date/Time: 10/23/2013 2:47:53 AM
Location: BC1SCPT5 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT



Maximum Pressure = 27.8
Hydrostatic Pressure :

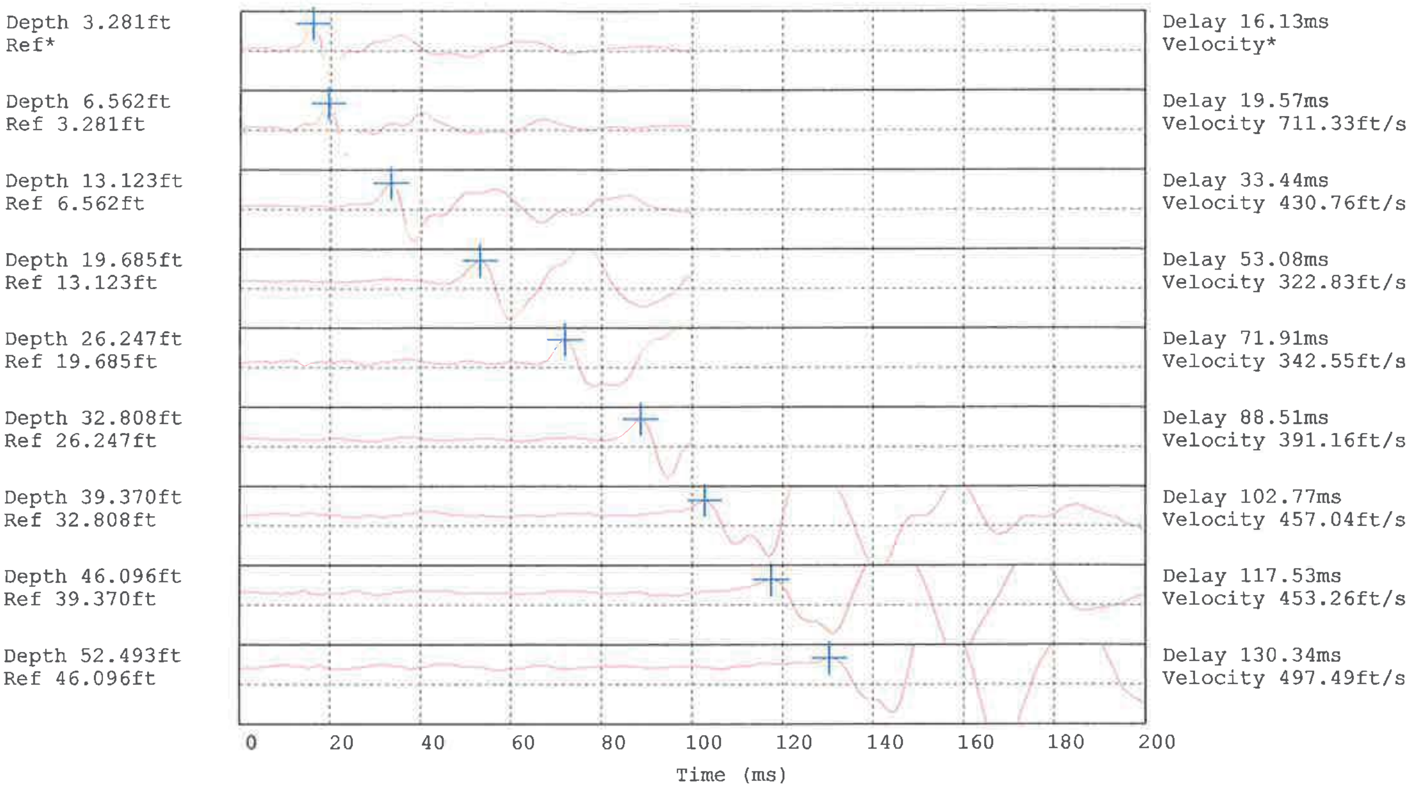
HDR ENG. / BC1SCPT5 / BIG CK. NEWPORT



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

THE SHEAR WAVE PICKS ARE COMPUTER GENERATED VALUES (STAFF REVIEW RECOMMENDED)

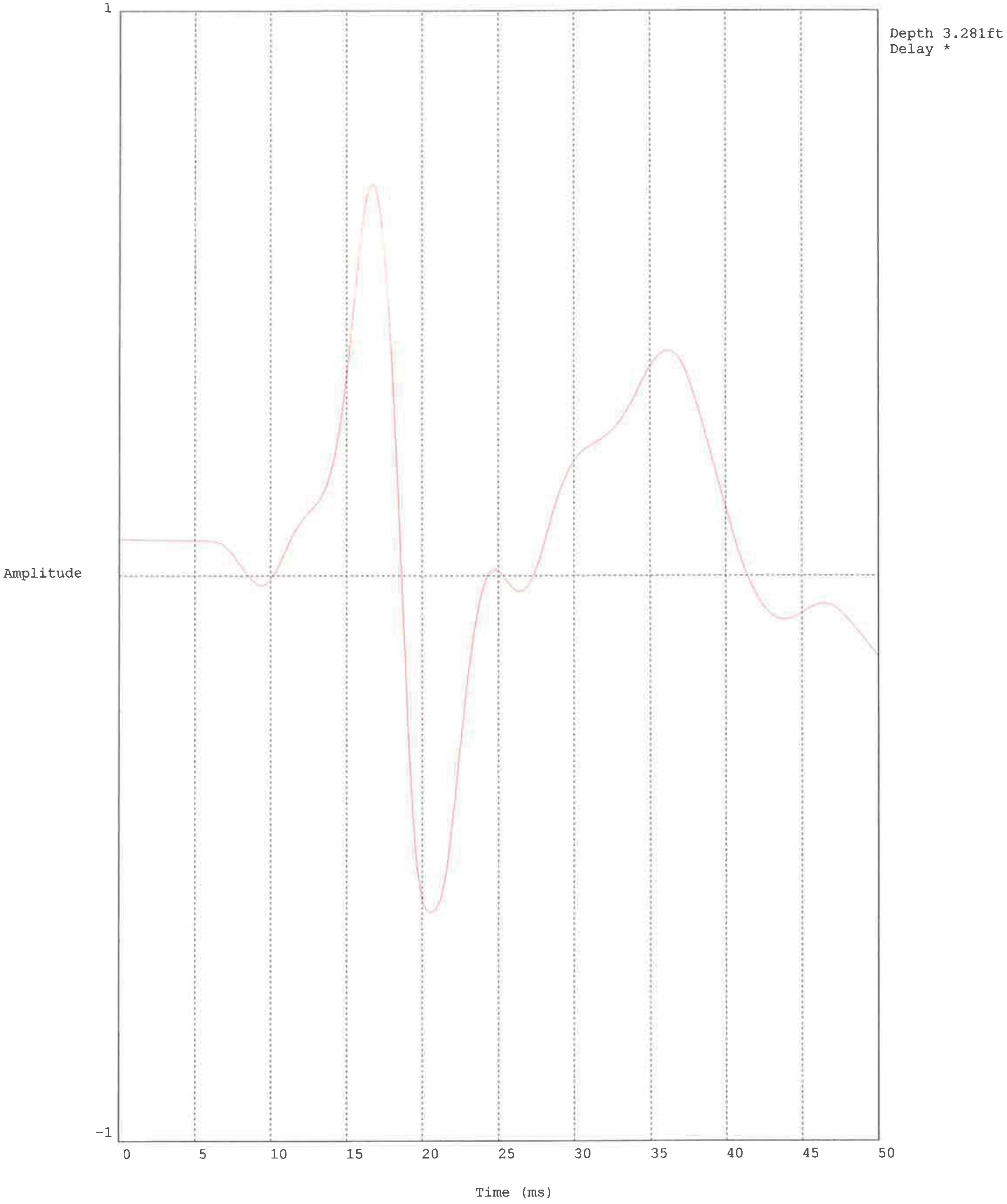
HDR ENG. / BC1SCPT5 / BIG CK. NEWPORT



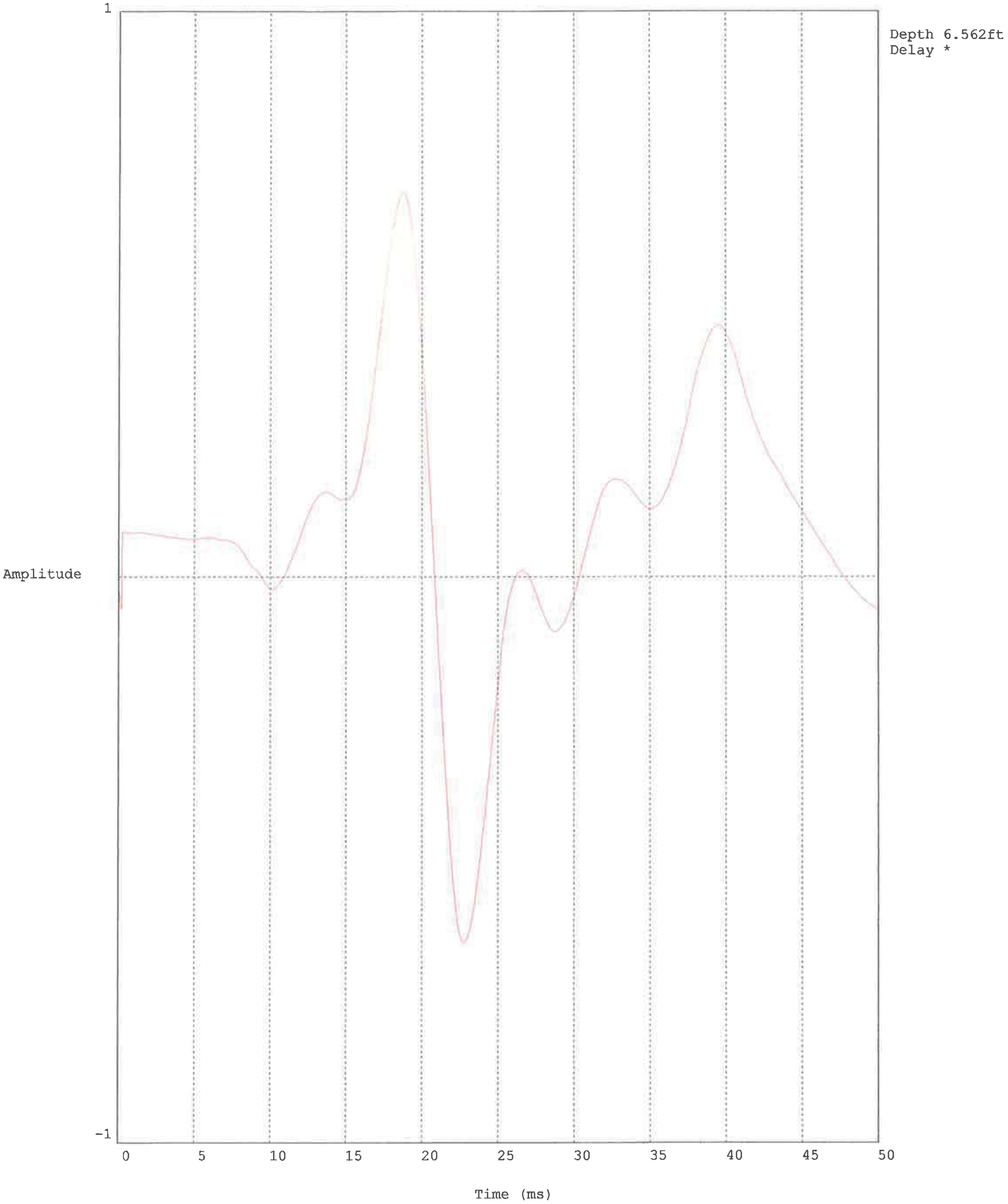
Hammer to Rod String Distance 1.3 (m)
* = Not Determined

THE SHEAR WAVE PICKS ARE COMPUTER GENERATED VALUES (STAFF REVIEW RECOMMENDED)

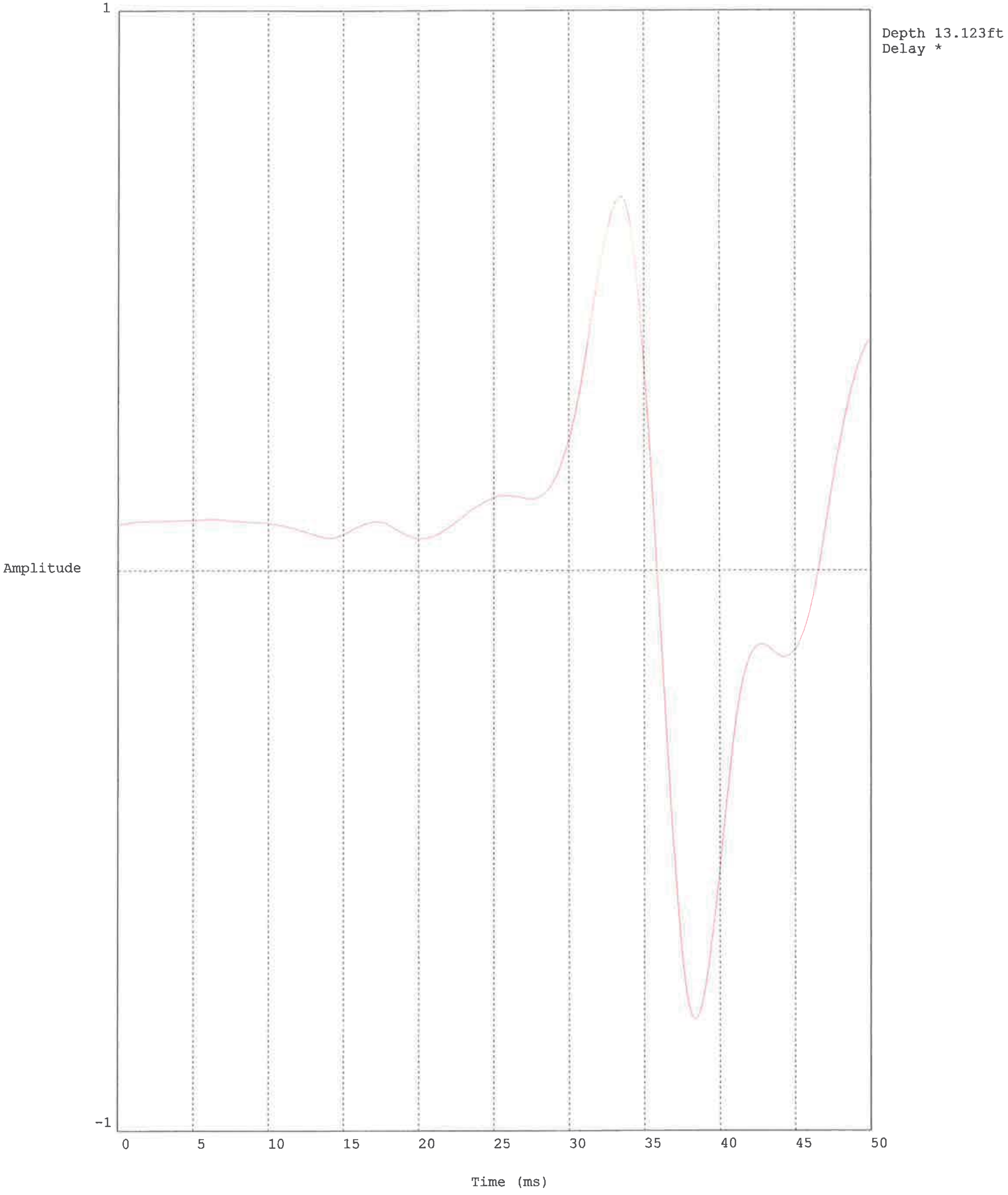
HDR ENG. / BC1SCPT5 / BIG CK. NEWPORT



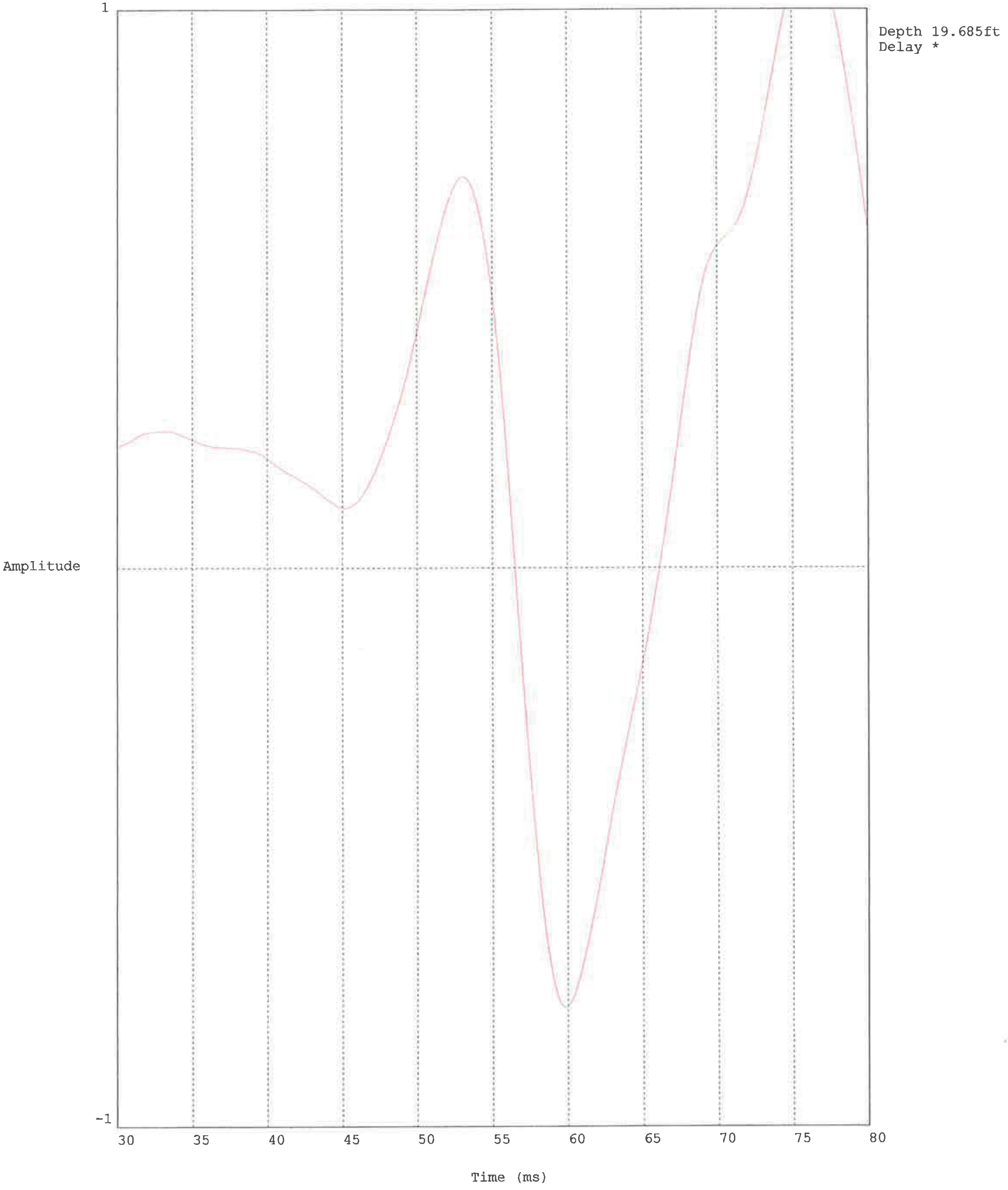
HDR ENG. / BC1SCPT5 / BIG CK. NEWPORT



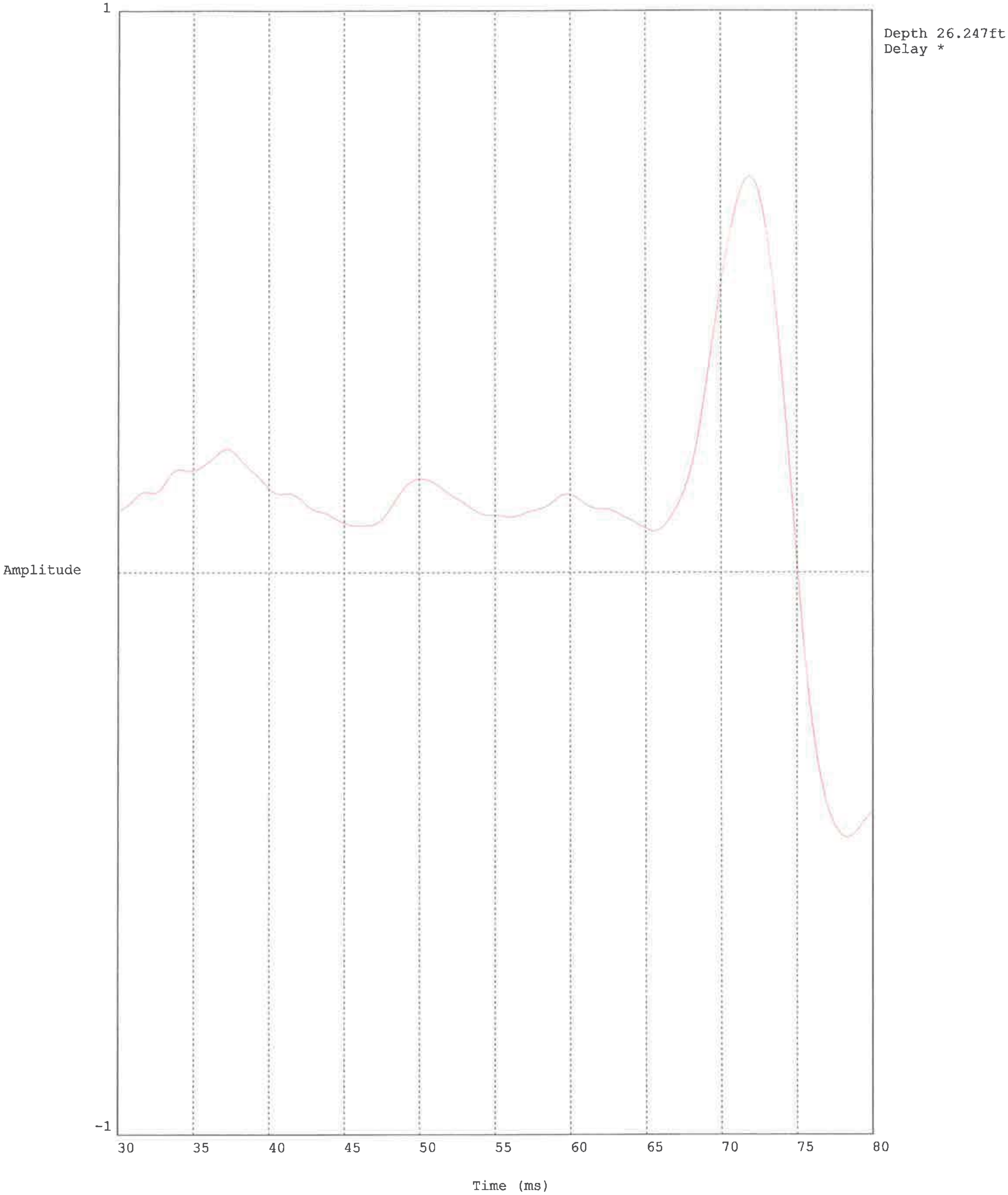
HDR ENG. / BC1SCPT5 / BIG CK. NEWPORT



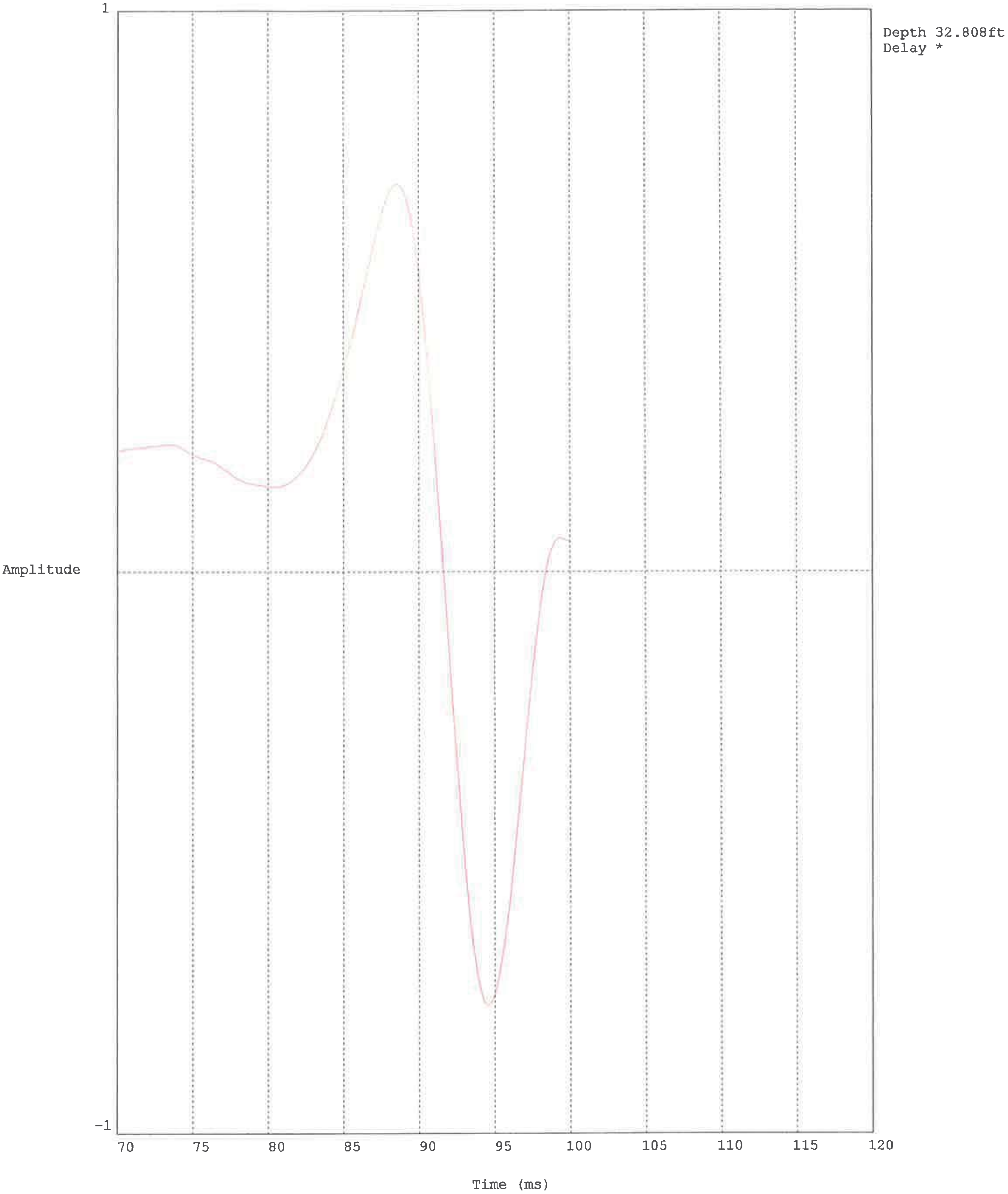
HDR ENG. / BC1SCPT5 / BIG CK. NEWPORT



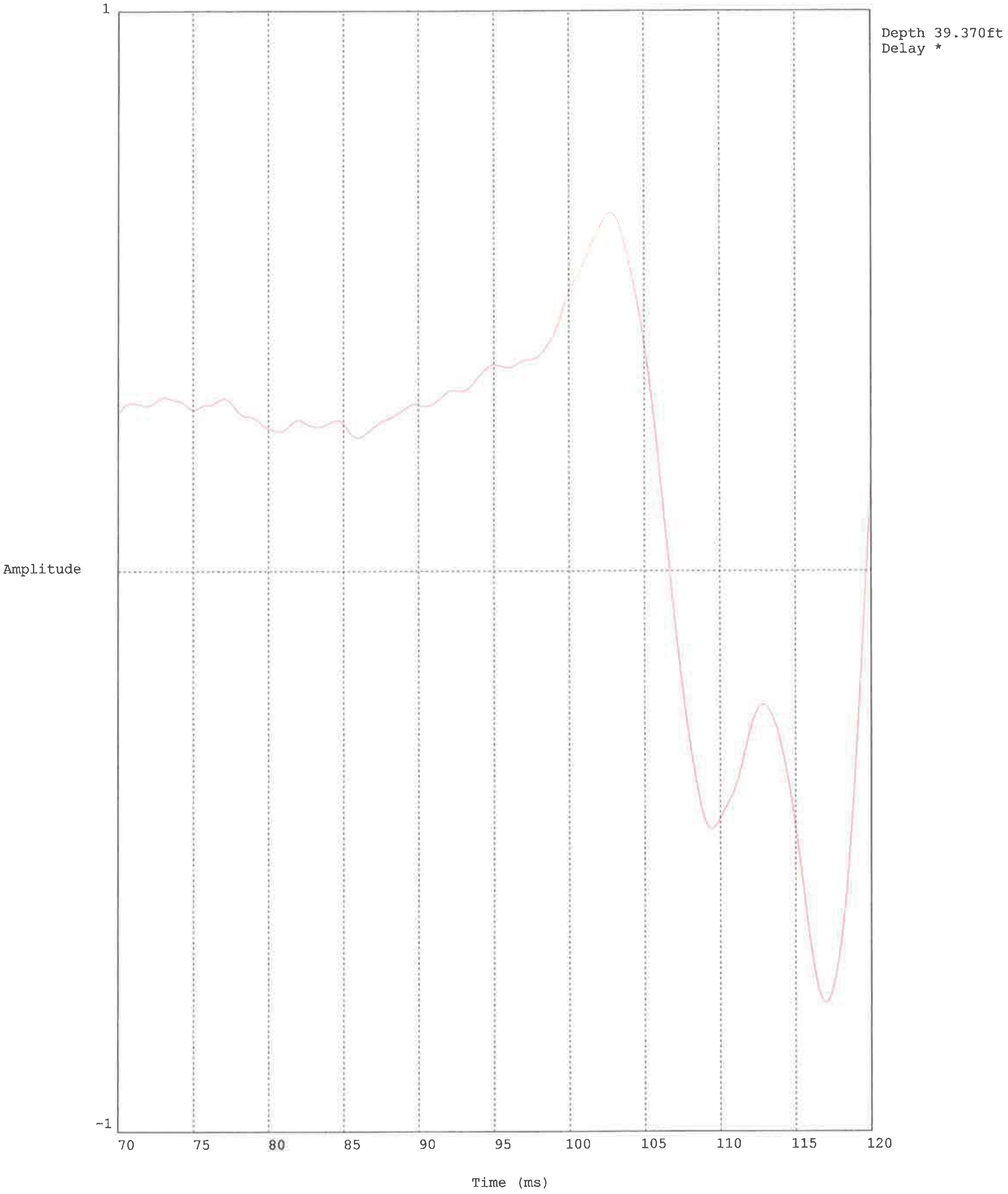
HDR ENG. / BC1SCPT5 / BIG CK. NEWPORT



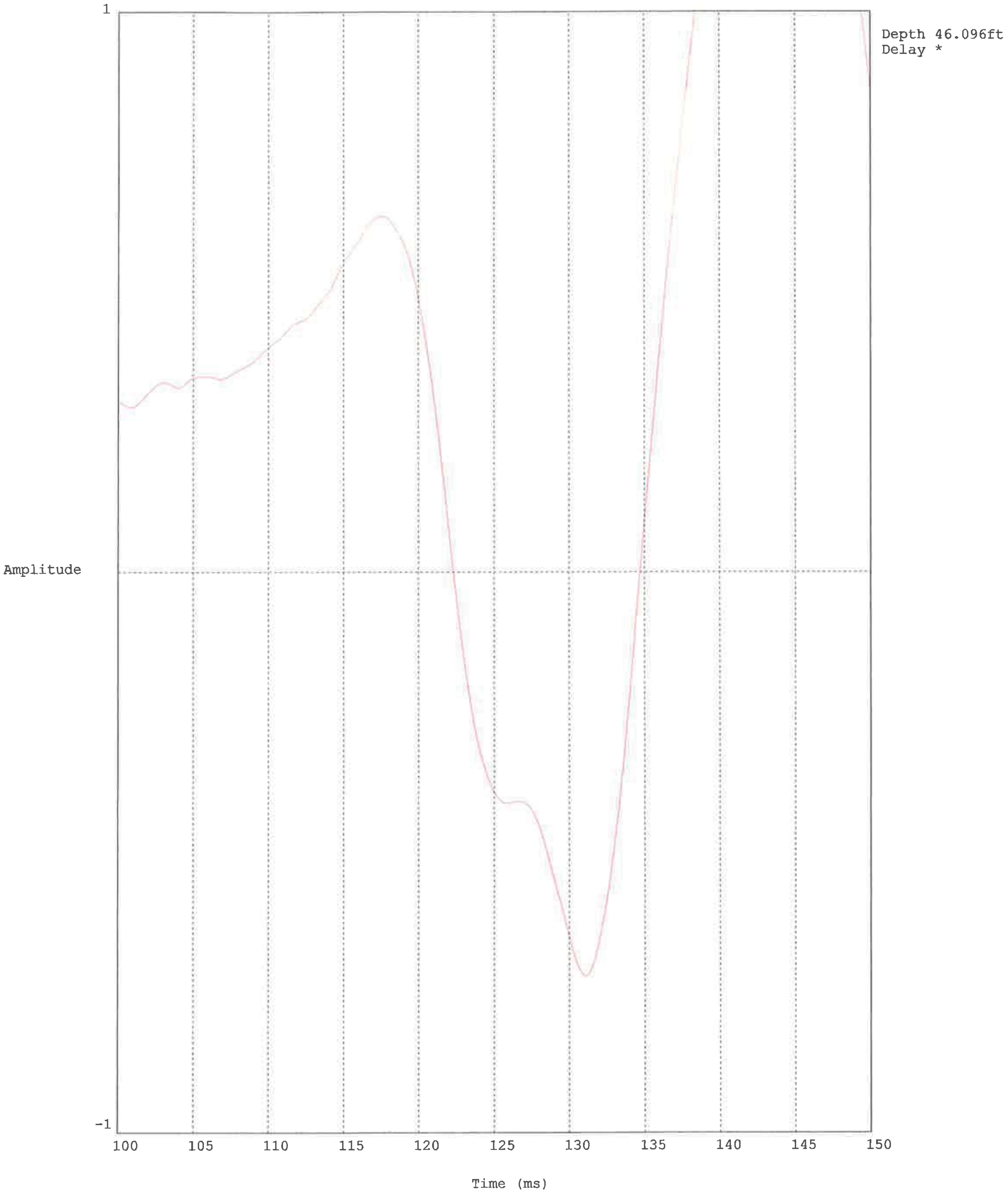
HDR ENG. / BC1SCPT5 / BIG CK. NEWPORT



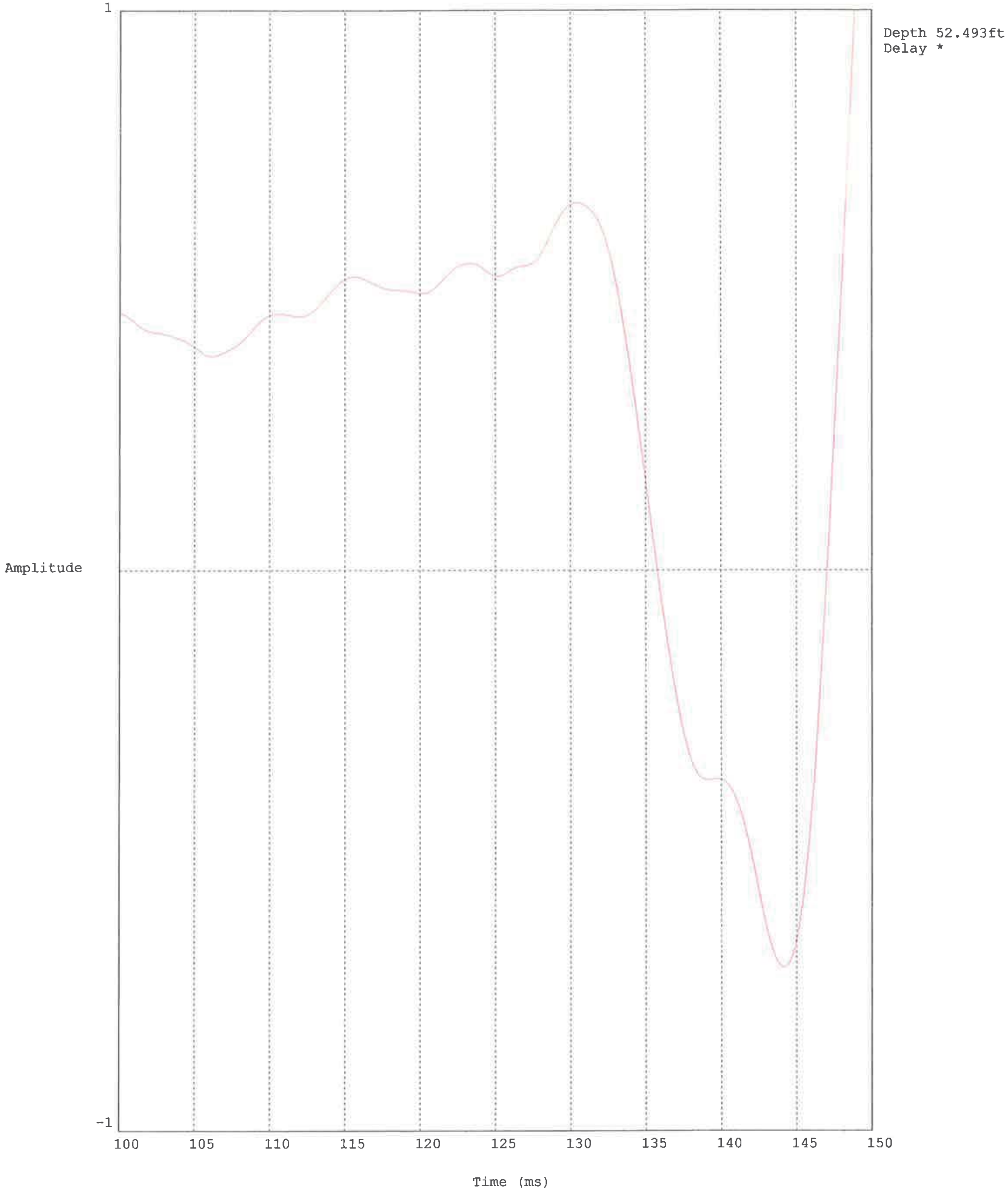
HDR ENG. / BC1SCPT5 / BIG CK. NEWPORT



HDR ENG. / BC1SCPT5 / BIG CK. NEWPORT



HDR ENG. / BC1SCPT5 / BIG CK. NEWPORT



Data File:VEI434BC1SCPT5(483) 10/23/2013 2:47:53 AM
Operator:SAV/CM Location:BC1,SCPT5 / BIG CK. NEWPORT
Cone ID:DSG0736 Job Number:HDR ENG./BIG CK. NEWPORT
Customer: BIG CK. DAM NEWPORT Units:

Depth (ft)	Qt TSF	Fs TSF	Fs/Qt (%)	Pw PSI	SPT N* 60% Hammer	Zone	Soil Behavior Type UBC-1983
0.82	14.22	0.5567	3.916	0.024	9	3	clay
0.98	15.01	0.7755	5.165	0.242	13	3	clay
1.15	11.66	0.7637	6.547	0.215	12	3	clay
1.31	11.09	0.7133	6.431	0.208	11	3	clay
1.48	10.15	0.5678	5.593	0.230	9	3	clay
1.64	7.16	0.3888	5.427	0.213	7	3	clay
1.80	5.05	0.3061	6.057	0.165	5	3	clay
1.97	4.34	0.2909	6.704	0.108	4	3	clay
2.13	4.49	0.3409	7.591	0.098	5	3	clay
2.30	7.80	0.4569	5.859	0.402	7	3	clay
2.46	10.38	0.5673	5.463	0.531	9	3	clay
2.62	10.77	0.5624	5.220	0.105	9	3	clay
2.79	8.37	0.4524	5.403	-0.282	8	3	clay
2.95	6.11	0.4061	6.647	-0.787	7	3	clay
3.12	6.60	0.1523	2.308	-0.349	6	3	clay
3.28	6.93	0.3866	5.581	-2.370	9	3	clay
3.44	13.88	0.4920	3.543	-0.306	10	3	clay
3.61	11.80	0.4400	3.729	-0.246	12	3	clay
3.77	10.37	0.4875	4.702	-0.045	12	3	clay
3.94	14.85	0.4863	3.274	-0.005	11	3	clay
4.10	10.15	0.5102	5.025	0.055	11	3	clay
4.27	8.56	0.5024	5.870	0.033	9	3	clay
4.43	10.25	0.5600	5.464	0.057	9	3	clay
4.59	8.13	0.6742	8.293	-0.093	10	3	clay
4.76	14.08	0.4775	3.391	-0.277	11	3	clay
4.92	11.10	0.5268	4.747	-0.667	12	3	clay
5.09	13.68	0.5275	3.856	-0.419	11	3	clay
5.25	10.13	0.5319	5.250	-0.777	11	3	clay
5.41	10.04	0.5347	5.328	-0.796	10	3	clay
5.58	9.89	0.5362	5.421	-1.143	10	3	clay
5.74	9.84	0.4351	4.420	-1.186	9	3	clay
5.91	9.87	0.3934	3.987	-1.521	8	3	clay
6.07	6.76	0.1701	2.515	-3.374	7	3	clay
6.23	5.47	0.5192	9.486	-3.092	9	3	clay
6.40	14.65	0.4284	2.925	-2.482	9	3	clay
6.56	9.55	0.4422	4.630	-2.521	11	3	clay
6.73	9.82	0.3891	3.961	-1.471	9	3	clay
6.89	7.60	0.2636	3.470	-1.062	7	3	clay
7.05	4.92	0.1773	3.602	-0.459	6	3	clay
7.22	5.24	0.2067	3.946	-0.383	5	3	clay
7.38	4.83	0.2989	6.188	-0.273	8	3	clay
7.55	13.91	0.4437	3.190	-0.203	9	3	clay
7.71	9.95	0.3677	3.697	-1.081	10	3	clay
7.87	6.78	0.3298	4.866	-1.452	9	3	clay
8.04	10.14	0.3037	2.993	-1.452	8	3	clay
8.20	8.19	0.4521	5.519	-1.406	9	3	clay
8.37	11.39	0.5201	4.568	-1.224	9	3	clay
8.53	9.59	0.5021	5.237	-1.179	9	3	clay
8.69	7.64	0.3489	4.563	-0.875	8	3	clay
8.86	6.41	0.2634	4.108	-0.808	6	3	clay
9.02	4.24	0.1514	3.574	-0.493	4	3	clay
9.19	3.12	0.0679	2.179	-0.450	3	3	clay
9.35	2.94	0.0414	1.411	-0.364	1	1	sensitive fine grained
9.51	3.34	0.0578	1.730	-0.287	2	1	sensitive fine grained
9.68	4.51	0.1075	2.384	-0.184	4	3	clay
9.84	5.15	0.1242	2.413	-0.122	3	4	silty clay to clay
10.01	4.79	0.0623	1.301	-0.110	2	1	sensitive fine grained

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

Depth (ft)	Qt TSF	Fs TSF	Fs/Qt (%)	Pw PSI	SPT N* 60% Hammer	Zone	Soil Behavior Type UBC-1983
10.17	4.39	0.0491	1.117	-0.074	2	1	sensitive fine grained
10.33	2.90	0.0642	2.216	-0.038	2	1	sensitive fine grained
10.50	2.97	0.0669	2.252	0.022	3	3	clay
10.66	3.48	0.0876	2.516	0.091	3	3	clay
10.83	3.87	0.1049	2.709	0.163	4	3	clay
10.99	4.57	0.1243	2.720	0.256	4	3	clay
11.15	4.77	0.1402	2.939	0.325	4	3	clay
11.32	4.69	0.1550	3.302	0.409	5	3	clay
11.48	5.29	0.1506	2.848	0.497	5	3	clay
11.65	5.87	0.1714	2.923	0.562	6	3	clay
11.81	6.27	0.1696	2.705	0.622	6	3	clay
11.98	5.79	0.1451	2.506	0.672	6	3	clay
12.14	5.23	0.1298	2.482	0.715	5	3	clay
12.30	4.81	0.1239	2.576	1.227	5	3	clay
12.47	4.57	0.1138	2.489	1.320	4	3	clay
12.63	4.49	0.1123	2.499	1.425	4	3	clay
12.80	4.39	0.0991	2.254	1.535	4	3	clay
12.96	4.10	0.1586	3.864	1.641	4	3	clay
13.12	3.98	0.1966	4.944	1.765	4	3	clay
13.29	5.28	0.1522	2.884	8.507	4	3	clay
13.45	4.51	0.1182	2.621	9.559	5	3	clay
13.62	4.35	0.1202	2.762	10.064	4	3	clay
13.78	4.83	0.1102	2.283	10.470	4	3	clay
13.94	4.60	0.1170	2.544	10.932	4	3	clay
14.11	4.33	0.0936	2.162	11.334	4	3	clay
14.27	4.18	0.0813	1.943	11.664	4	3	clay
14.44	3.64	0.0687	1.887	11.900	4	3	clay
14.60	3.40	0.0740	2.174	12.175	2	1	sensitive fine grained
14.76	3.46	0.0482	1.395	12.364	2	1	sensitive fine grained
14.93	3.41	0.0598	1.756	12.560	2	1	sensitive fine grained
15.09	3.95	0.1015	2.571	15.583	4	3	clay
15.26	4.72	0.1278	2.708	15.638	4	3	clay
15.42	4.98	0.1427	2.863	15.280	5	3	clay
15.58	4.73	0.1391	2.939	14.842	4	3	clay
15.75	4.12	0.1843	4.467	14.466	5	3	clay
15.91	6.89	0.1964	2.850	15.313	5	3	clay
16.08	6.03	0.2017	3.346	15.557	6	3	clay
16.24	6.06	0.1901	3.138	15.952	6	3	clay
16.40	6.01	0.1996	3.320	16.035	6	3	clay
16.57	5.96	0.1957	3.286	16.234	6	3	clay
16.73	5.72	0.1959	3.427	16.282	5	3	clay
16.90	5.44	0.1863	3.425	16.502	5	3	clay
17.06	5.28	0.1773	3.360	16.590	5	3	clay
17.22	5.17	0.1481	2.864	16.743	5	3	clay
17.39	4.75	0.1348	2.840	16.956	4	3	clay
17.55	3.95	0.1250	3.164	16.980	4	3	clay
17.72	3.87	0.1609	4.163	17.143	4	3	clay
17.88	5.10	0.1999	3.921	17.107	4	3	clay
18.04	5.05	0.2240	4.434	17.162	5	3	clay
18.21	5.03	0.1902	3.779	17.901	5	3	clay
18.37	5.56	0.2081	3.741	18.444	5	3	clay
18.54	5.77	0.1621	2.808	18.886	6	3	clay
18.70	6.02	0.2701	4.485	19.197	5	3	clay
18.86	4.46	0.2756	6.173	20.639	5	3	clay
19.03	6.34	0.2488	3.925	20.802	5	3	clay
19.19	5.84	0.2005	3.435	20.622	5	3	clay
19.36	5.04	0.1711	3.392	20.302	5	3	clay
19.52	4.70	0.1996	4.246	20.347	5	3	clay
19.69	4.51	0.1891	4.191	20.725	5	3	clay
19.85	5.39	0.1642	3.046	16.251	5	3	clay
20.01	5.96	0.1494	2.505	17.114	6	2	organic material
20.18	6.49	1.2524	19.308	17.719	0	0	<out of range>

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

Depth (ft)	Qt TSF	Fs TSF	Fs/Qt (%)	Pw PSI	SPT N* 60% Hammer	Zone	Soil Behavior Type UBC-1983
20.34	34.92	4.5478	13.023	19.120	70	11	very stiff fine grained (*)
20.51	178.17	7.4364	4.174	11.393	110	11	very stiff fine grained (*)
20.67	133.13	6.4686	4.859	4.539	109	11	very stiff fine grained (*)
20.83	31.11	2.6004	8.360	4.577	55	3	clay
21.00	9.57	1.0240	10.700	6.974	16	3	clay
21.16	9.71	0.6505	6.701	9.251	9	3	clay
21.33	8.86	0.5141	5.801	10.238	9	3	clay
21.49	8.40	0.3892	4.633	10.513	8	3	clay
21.65	7.02	0.3142	4.474	11.116	7	3	clay
21.82	6.89	0.4391	6.375	11.912	7	3	clay
21.98	9.36	0.4133	4.417	13.720	8	3	clay
22.15	9.20	0.3810	4.143	12.293	8	3	clay
22.31	7.94	0.3757	4.729	12.587	8	3	clay
22.47	8.92	0.3652	4.094	13.472	8	3	clay
22.64	8.41	0.3670	4.361	12.795	9	3	clay
22.80	10.09	0.3543	3.510	12.515	9	3	clay
22.97	9.35	0.4828	5.164	12.393	9	3	clay
23.13	9.22	0.4917	5.333	12.725	10	3	clay
23.29	11.22	0.4554	4.059	13.558	10	3	clay
23.46	11.51	0.4113	3.573	9.834	10	3	clay
23.62	7.19	0.4097	5.696	9.968	8	3	clay
23.79	5.86	0.3742	6.384	10.841	6	3	clay
23.95	5.97	0.3556	5.959	11.652	6	3	clay
24.11	6.20	0.3953	6.378	12.333	5	3	clay
24.28	4.92	0.4053	8.234	12.649	6	3	clay
24.44	6.92	0.4436	6.408	14.325	7	3	clay
24.61	11.29	0.5215	4.620	15.411	11	3	clay
24.77	15.03	0.6553	4.360	13.833	9	4	silty clay to clay
24.93	17.45	0.5370	3.077	13.584	11	4	silty clay to clay
25.10	18.17	0.6082	3.348	13.488	14	3	clay
25.26	9.67	0.6726	6.953	12.075	14	3	clay
25.43	16.57	0.7281	4.395	12.183	14	3	clay
25.59	18.83	0.8355	4.437	13.220	17	3	clay
25.75	17.28	0.7630	4.416	12.807	14	3	clay
25.92	8.21	0.5468	6.661	12.352	10	3	clay
26.08	5.18	0.2145	4.140	14.304	6	3	clay
26.25	4.85	0.2169	4.468	15.579	5	3	clay
26.41	5.23	0.2311	4.422	23.676	5	3	clay
26.57	5.51	0.3319	6.027	24.389	5	3	clay
26.74	5.49	0.3418	6.220	25.166	6	3	clay
26.90	6.37	0.3285	5.154	18.444	6	3	clay
27.07	5.67	0.2910	5.134	19.398	6	3	clay
27.23	5.67	0.2849	5.024	20.390	5	3	clay
27.40	5.65	0.2881	5.102	21.514	5	3	clay
27.56	5.57	0.2836	5.095	22.349	5	3	clay
27.72	5.80	0.2773	4.780	23.320	6	3	clay
27.89	6.38	0.2907	4.553	24.446	6	3	clay
28.05	7.01	0.2734	3.900	25.303	6	3	clay
28.22	6.63	0.2939	4.432	25.740	7	3	clay
28.38	6.92	0.2716	3.926	27.465	6	3	clay
28.54	6.24	0.2784	4.460	26.704	6	3	clay
28.71	7.09	0.2071	2.921	27.498	7	3	clay
28.87	8.15	0.1882	2.310	26.159	5	4	silty clay to clay
29.04	6.13	0.1911	3.116	25.743	6	3	clay
29.20	5.51	0.1877	3.406	27.103	5	3	clay
29.36	5.49	0.1838	3.351	28.204	5	3	clay
29.53	5.32	0.1751	3.294	29.038	5	3	clay
29.69	5.36	0.1833	3.416	29.940	5	3	clay
29.86	5.66	0.1808	3.194	30.748	5	3	clay
30.02	5.80	0.1879	3.238	30.832	6	3	clay
30.18	5.90	0.1793	3.041	31.501	6	3	clay
30.35	6.07	0.1722	2.835	31.741	6	3	clay

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

Depth (ft)	Qt TSF	Fs TSF	Fs/Qt (%)	Pw PSI	SPT N* 60% Hammer	Zone	Soil Behavior Type UBC-1983
30.51	5.86	0.1853	3.164	32.174	6	3	clay
30.68	6.09	0.1888	3.098	33.070	6	3	clay
30.84	6.91	0.1851	2.680	33.037	6	3	clay
31.00	6.37	0.2009	3.154	32.281	6	3	clay
31.17	6.21	0.2266	3.649	32.968	7	3	clay
31.33	7.92	0.2433	3.073	33.917	7	3	clay
31.50	8.67	0.2889	3.333	33.113	6	4	silty clay to clay
31.66	9.31	0.2357	2.533	30.554	6	4	silty clay to clay
31.82	9.86	0.2903	2.943	28.665	9	3	clay
31.99	8.11	0.4910	6.053	26.690	9	3	clay
32.15	9.86	0.7602	7.713	26.804	11	3	clay
32.32	16.18	0.8726	5.395	26.857	12	3	clay
32.48	12.41	0.9772	7.872	25.025	14	3	clay
32.64	14.82	0.9688	6.536	21.373	13	3	clay
32.81	13.86	0.9523	6.869	21.821	16	3	clay
32.97	21.79	1.0592	4.862	16.272	21	3	clay
33.14	28.60	1.2065	4.219	16.820	17	4	silty clay to clay
33.30	28.24	1.1444	4.052	16.071	17	4	silty clay to clay
33.46	24.22	0.8339	3.443	16.121	12	5	clayey silt to silty clay
33.63	22.14	0.6568	2.966	11.286	11	5	clayey silt to silty clay
33.79	21.15	0.4609	2.179	9.750	10	5	clayey silt to silty clay
33.96	21.47	0.4419	2.058	8.272	10	5	clayey silt to silty clay
34.12	19.27	0.5284	2.741	7.354	9	5	clayey silt to silty clay
34.28	18.70	0.5587	2.988	7.124	10	5	clayey silt to silty clay
34.45	23.99	0.7522	3.136	7.036	11	5	clayey silt to silty clay
34.61	25.82	0.9157	3.547	5.455	11	5	clayey silt to silty clay
34.78	22.18	0.8189	3.692	4.515	14	4	silty clay to clay
34.94	17.45	0.8025	4.599	4.058	13	4	silty clay to clay
35.10	21.80	0.6514	2.988	7.402	10	5	clayey silt to silty clay
35.27	25.40	0.5490	2.162	5.866	11	5	clayey silt to silty clay
35.43	24.04	0.5589	2.325	4.427	9	6	sandy silt to clayey silt
35.60	22.64	0.5119	2.261	3.537	11	5	clayey silt to silty clay
35.76	19.62	0.4380	2.233	3.346	9	5	clayey silt to silty clay
35.93	17.12	0.3772	2.203	3.186	9	5	clayey silt to silty clay
36.09	17.07	0.4262	2.496	3.238	8	5	clayey silt to silty clay
36.25	14.53	0.4163	2.865	3.327	7	5	clayey silt to silty clay
36.42	13.74	0.3634	2.645	3.566	7	5	clayey silt to silty clay
36.58	14.01	0.3466	2.473	3.783	7	5	clayey silt to silty clay
36.75	12.99	0.4111	3.164	3.913	8	4	silty clay to clay
36.91	11.46	0.5510	4.809	4.106	13	3	clay
37.07	14.85	0.6184	4.165	4.396	10	4	silty clay to clay
37.24	20.05	0.5690	2.838	4.200	9	5	clayey silt to silty clay
37.40	22.47	0.7144	3.180	3.360	10	5	clayey silt to silty clay
37.57	22.36	0.7487	3.348	2.413	10	5	clayey silt to silty clay
37.73	20.96	0.7696	3.672	1.480	13	4	silty clay to clay
37.89	17.02	0.6508	3.825	0.796	9	5	clayey silt to silty clay
38.06	15.45	0.3730	2.414	0.309	8	5	clayey silt to silty clay
38.22	16.63	0.5292	3.183	0.129	8	5	clayey silt to silty clay
38.39	16.99	0.5434	3.198	3.059	9	5	clayey silt to silty clay
38.55	20.80	0.6320	3.039	1.165	10	5	clayey silt to silty clay
38.71	24.00	0.7888	3.286	0.478	11	5	clayey silt to silty clay
38.88	21.07	0.8238	3.910	0.115	13	4	silty clay to clay
39.04	18.35	0.7794	4.248	-0.373	12	4	silty clay to clay
39.21	17.79	0.5487	3.083	-0.732	11	4	silty clay to clay
39.37	14.17	0.5844	4.126	-1.014	9	4	silty clay to clay
39.53	11.69	0.5779	4.943	15.971	13	3	clay
39.70	14.81	0.6621	4.472	7.201	13	3	clay
39.86	15.76	0.6737	4.274	5.730	15	3	clay
40.03	15.38	0.6877	4.471	5.862	15	3	clay
40.19	14.89	0.6282	4.220	3.571	14	3	clay
40.35	14.23	0.6685	4.699	2.755	14	3	clay
40.52	14.33	0.7007	4.890	2.992	14	3	clay

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

Depth (ft)	Qt TSF	Fs TSF	Fs/Qt (%)	Pw PSI	SPT N* 60% Hammer	Zone	Soil Behavior Type UBC-1983
40.68	15.08	0.5991	3.972	2.621	15	3	clay
40.85	16.43	0.5590	3.401	2.664	10	4	silty clay to clay
41.01	17.14	0.6087	3.551	2.365	11	4	silty clay to clay
41.17	19.00	0.6339	3.336	2.071	9	5	clayey silt to silty clay
41.34	18.17	0.5311	2.923	1.590	9	5	clayey silt to silty clay
41.50	19.50	0.7349	3.769	0.777	9	5	clayey silt to silty clay
41.67	21.25	0.7468	3.514	12.651	13	4	silty clay to clay
41.83	21.81	0.7896	3.620	3.568	13	4	silty clay to clay
41.99	17.91	0.7595	4.241	1.081	17	3	clay
42.16	13.15	0.9068	6.895	1.179	16	3	clay
42.32	19.40	0.8814	4.542	1.803	18	3	clay
42.49	23.97	0.8384	3.498	0.619	12	5	clayey silt to silty clay
42.65	30.75	0.5927	1.928	-0.301	11	6	sandy silt to clayey silt
42.81	29.68	0.7610	2.564	-0.990	13	5	clayey silt to silty clay
42.98	19.53	0.7934	4.062	-0.576	11	5	clayey silt to silty clay
43.14	19.08	0.7231	3.789	0.189	12	4	silty clay to clay
43.31	19.11	0.6937	3.631	0.220	13	4	silty clay to clay
43.47	24.68	1.0083	4.085	0.132	15	4	silty clay to clay
43.64	27.93	0.9596	3.435	0.682	15	4	silty clay to clay
43.80	15.82	0.8906	5.629	1.435	19	3	clay
43.96	14.76	0.7236	4.902	1.313	14	3	clay
44.13	14.52	0.7368	5.074	0.904	15	3	clay
44.29	16.37	0.8187	5.001	0.701	15	3	clay
44.46	15.57	0.9231	5.927	0.517	15	3	clay
44.62	16.53	0.9541	5.772	0.352	18	3	clay
44.78	23.57	0.8381	3.556	0.301	21	3	clay
44.95	24.47	1.0366	4.236	0.796	15	4	silty clay to clay
45.11	21.80	1.0444	4.791	2.832	20	3	clay
45.28	15.89	1.0443	6.573	1.691	18	3	clay
45.44	18.43	0.9968	5.408	2.105	17	3	clay
45.60	19.30	1.0056	5.210	1.703	18	3	clay
45.77	17.84	1.0625	5.956	1.645	18	3	clay
45.93	18.85	1.3190	6.997	1.531	22	3	clay
46.10	32.02	1.4586	4.555	1.995	26	3	clay
46.26	30.02	1.6072	5.354	6.928	31	3	clay
46.42	36.40	1.5898	4.367	4.171	30	3	clay
46.59	28.47	1.4178	4.981	2.805	28	3	clay
46.75	21.74	1.2098	5.565	1.600	22	3	clay
46.92	19.93	1.0129	5.083	0.584	19	3	clay
47.08	18.29	1.1399	6.231	0.148	19	3	clay
47.24	21.68	1.2981	5.989	0.088	20	3	clay
47.41	22.96	1.3998	6.097	-0.115	22	3	clay
47.57	23.31	1.4681	6.297	-0.237	23	3	clay
47.74	24.34	1.4174	5.824	-0.464	23	3	clay
47.90	25.81	1.3855	5.368	-0.600	24	3	clay
48.06	24.18	0.9165	3.790	-0.902	24	3	clay
48.23	26.63	1.1396	4.279	-1.079	17	4	silty clay to clay
48.39	28.87	1.1347	3.930	10.312	16	4	silty clay to clay
48.56	21.66	1.0693	4.937	5.120	22	3	clay
48.72	18.83	0.8727	4.634	4.023	18	3	clay
48.88	17.43	0.7684	4.408	3.477	18	3	clay
49.05	18.65	0.9323	5.000	3.193	19	3	clay
49.21	23.13	0.9965	4.307	2.645	20	3	clay
49.38	20.70	0.9350	4.517	1.959	21	3	clay
49.54	20.94	0.8739	4.172	1.662	19	3	clay
49.70	16.34	0.9093	5.564	1.540	17	3	clay
49.87	16.41	0.8491	5.173	1.645	16	3	clay
50.03	18.53	0.8542	4.609	1.765	17	3	clay
50.20	19.87	0.9263	4.663	1.516	19	3	clay
50.36	22.01	0.9085	4.128	1.294	20	3	clay
50.52	21.47	0.8874	4.134	1.007	14	4	silty clay to clay
50.69	20.71	0.7350	3.549	0.847	12	4	silty clay to clay

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

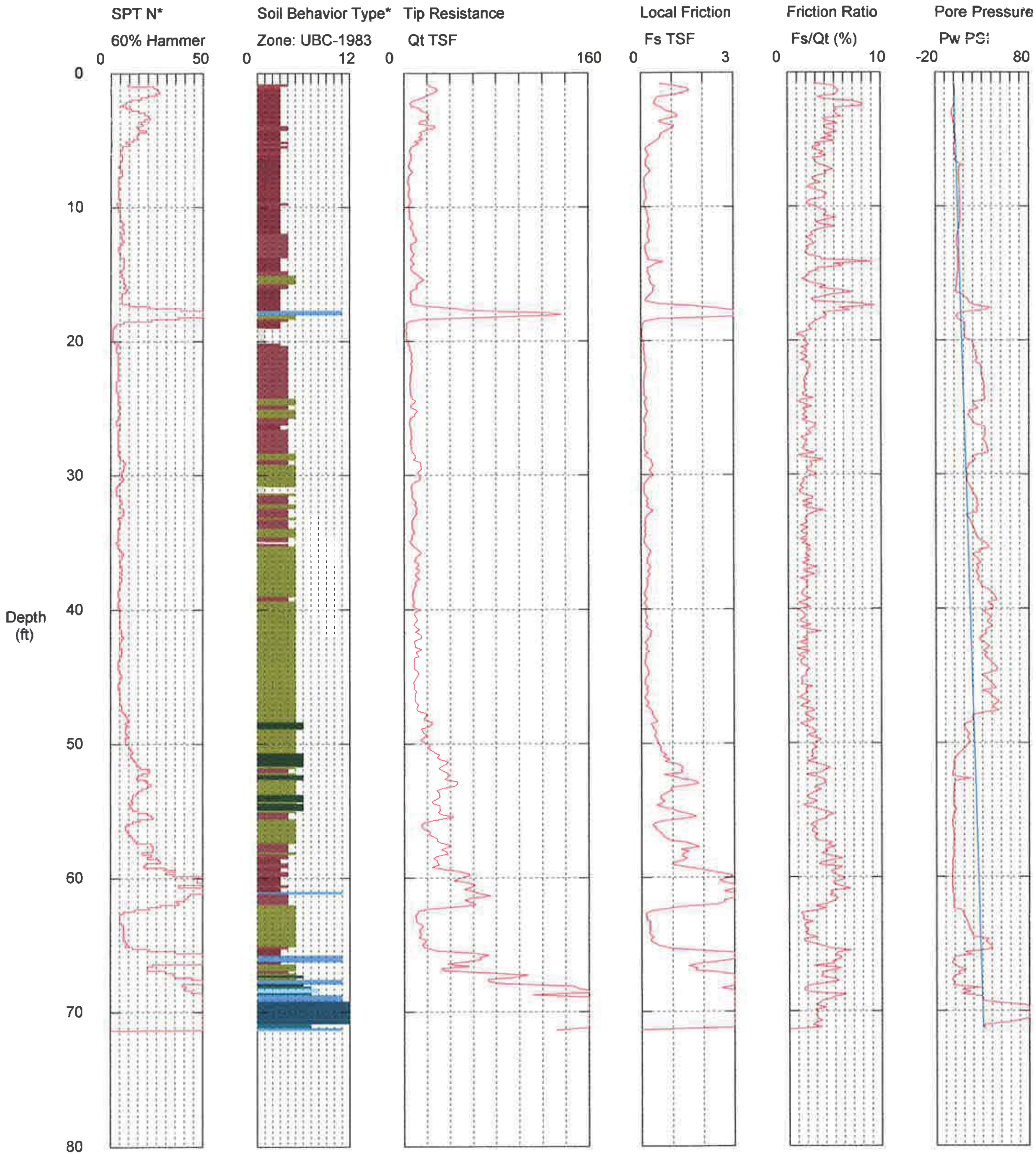
Depth (ft)	Qt TSF	Fs TSF	Fs/Qt (%)	Pw PSI	SPT N* 60% Hammer	Zone	Soil Behavior Type UBC-1983
50.85	12.85	0.6635	5.162	0.713	16	3	clay
51.02	15.52	0.7109	4.580	1.095	14	3	clay
51.18	14.11	0.5421	3.843	1.263	14	3	clay
51.35	13.63	0.5233	3.839	1.473	9	4	silty clay to clay
51.51	13.20	0.4167	3.157	1.894	8	4	silty clay to clay
51.67	12.13	0.3726	3.072	3.453	7	4	silty clay to clay
51.84	9.06	0.2686	2.964	3.697	6	4	silty clay to clay
52.00	9.27	0.2683	2.894	4.025	8	3	clay
52.17	7.62	0.5266	6.914	4.475	10	3	clay
52.33	12.89	0.8119	6.297	5.151	10	3	clay
52.49	11.00	0.7341	6.675	5.393	11	3	clay
52.66	9.71	0.4671	4.812	19.274	9	3	clay
52.82	8.18	0.3349	4.095	20.799	8	3	clay
52.99	7.83	0.3110	3.970	22.435	9	3	clay
53.15	11.02	0.3717	3.375	23.339	9	3	clay
53.31	10.65	0.4551	4.273	23.562	11	3	clay
53.48	11.95	0.4612	3.860	19.388	11	3	clay
53.64	10.63	0.3837	3.611	15.342	8	4	silty clay to clay
53.81	15.71	0.4808	3.062	16.172	9	4	silty clay to clay
53.97	14.35	0.4277	2.980	14.299	8	4	silty clay to clay
54.13	8.11	0.3775	4.651	10.253	10	3	clay
54.30	7.94	0.2963	3.732	10.568	7	3	clay
54.46	7.19	0.3563	4.954	11.152	7	3	clay
54.63	8.07	0.4031	4.994	10.499	8	3	clay
54.79	11.27	0.4909	4.357	9.416	9	3	clay
54.95	10.18	0.6599	6.481	10.121	10	3	clay
55.12	11.37	0.7286	6.410	8.705	12	3	clay
55.28	14.53	0.7088	4.878	4.764	13	3	clay
55.45	14.45	0.8774	6.070	3.186	14	3	clay
55.61	13.89	0.9374	6.751	1.803	14	3	clay
55.77	13.97	1.0840	7.757	1.059	15	3	clay
55.94	18.21	1.0055	5.520	0.787	15	3	clay
56.10	16.00	0.9194	5.745	0.251	16	3	clay
56.27	14.78	0.8134	5.504	-0.320	15	3	clay
56.43	14.73	0.7142	4.850	-0.658	13	3	clay
56.59	10.32	0.4818	4.670	-0.552	11	3	clay
56.76	8.58	0.3269	3.810	-0.349	8	3	clay
56.92	7.04	0.3071	4.361	-0.065	8	3	clay
57.09	9.14	0.3685	4.033	0.182	9	3	clay
57.25	12.98	0.8497	6.548	0.397	10	6	sandy silt to clayey silt
57.41	58.92	0.6516	1.106	0.361	13	6	sandy silt to clayey silt
57.58	33.57	0.6458	1.924	-0.868	15	6	sandy silt to clayey silt
57.74	25.75	0.8860	3.440	-0.084	18	5	clayey silt to silty clay
57.91	55.75	2.1196	3.802	-0.629	23	6	sandy silt to clayey silt
58.07	99.43	1.5655	1.574	0.612	26	8	sand to silty sand
58.23	105.42	1.9582	1.858	1.700	36	7	silty sand to sandy silt
58.40	117.99	-32768	-32768	4.475	0	0	<out of range>

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

HDR ENG. / BC1-SCPT-6 / BIG CK. DAM NEWPORT

Operator: SAV/CM
Sounding: VEI434BC1SCPT6(482)
Cone Used: DSG0736

CPT Date/Time: 10/22/2013 10:53:00 PM
Location: BC1,SCPT6 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT



Maximum Depth = 71.36 feet

Depth Increment = 0.164 feet

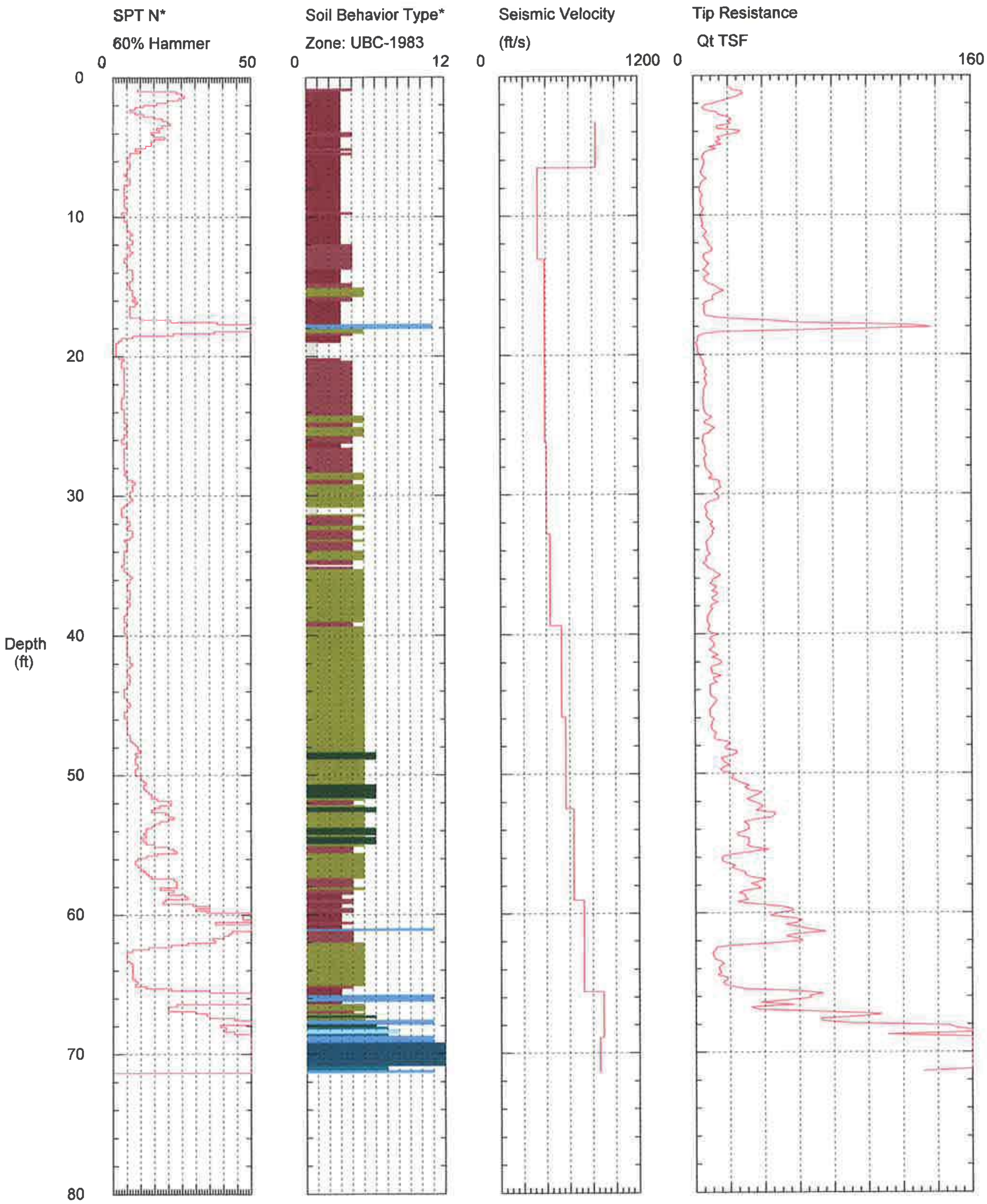
⬜ 1 sensitive fine grained
⬛ 2 organic material
🟥 3 clay
🟫 4 silty clay to clay
🟩 5 clayey silt to silty clay
🟩 6 sandy silt to clayey silt
🟦 7 silty sand to sandy silt
🟦 8 sand to silty sand
🟧 9 sand
🟥 10 gravelly sand to sand
🟦 11 very stiff fine grained (*)
🟦 12 sand to clayey sand (*)

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

HDR ENG. / BC1-SCPT-6 / BIG CK. DAM NEWPORT

Operator: SAV/CM
Sounding: VEI434BC1SCPT6(482)
Cone Used: DSG0736

CPT Date/Time: 10/22/2013 10:53:00 PM
Location: BC1,SCPT6 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT



Maximum Depth = 71.36 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

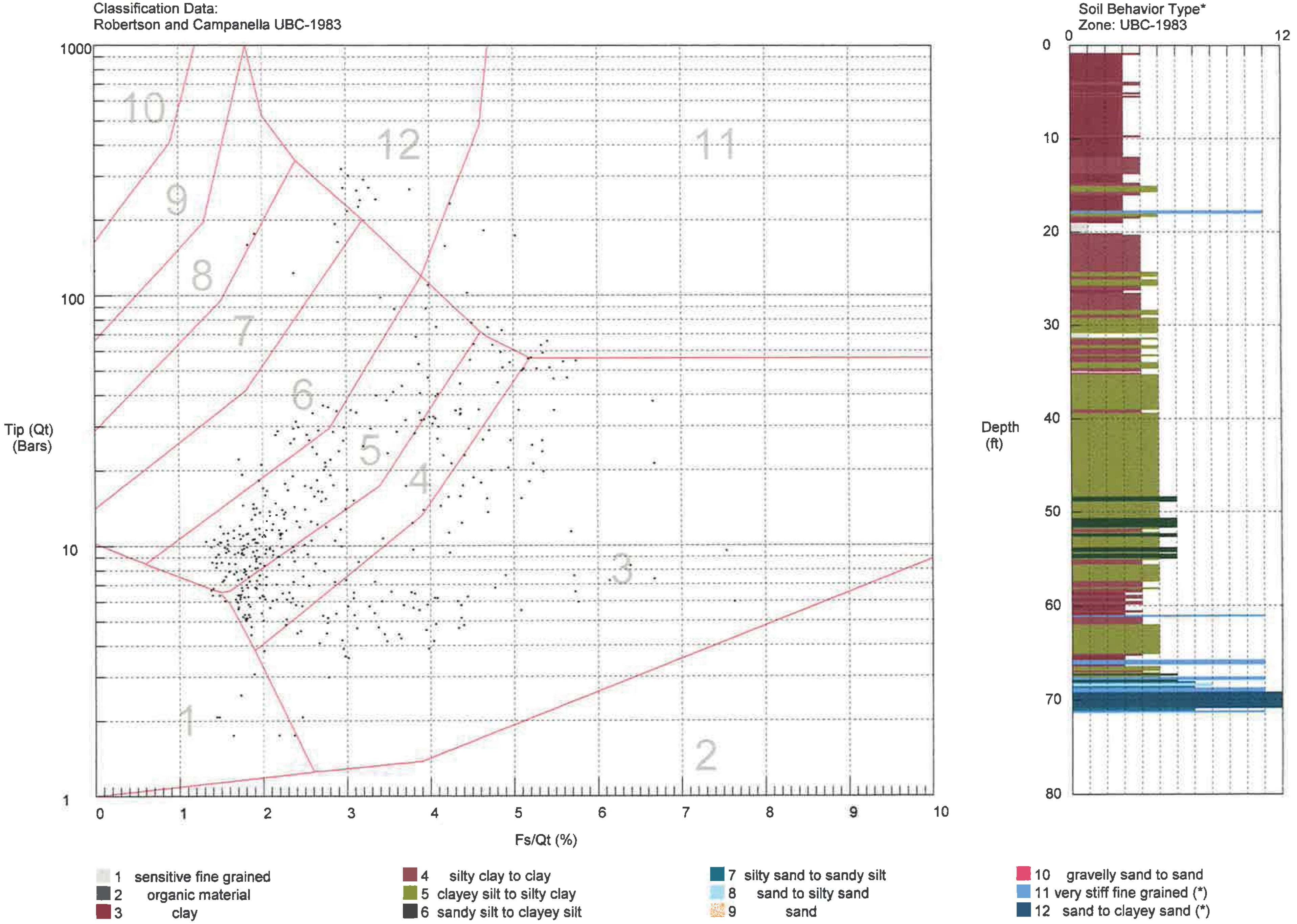
🟥 10 gravelly sand to sand
🟦 11 very stiff fine grained (*)
🟦 12 sand to clayey sand (*)

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

HDR ENG. / BC1-SCPT-6 / BIG CK. DAM NEWPORT

Operator: SAV/CM
Sounding: VEI434BC1SCPT6(482)
Cone Used: DSG0736

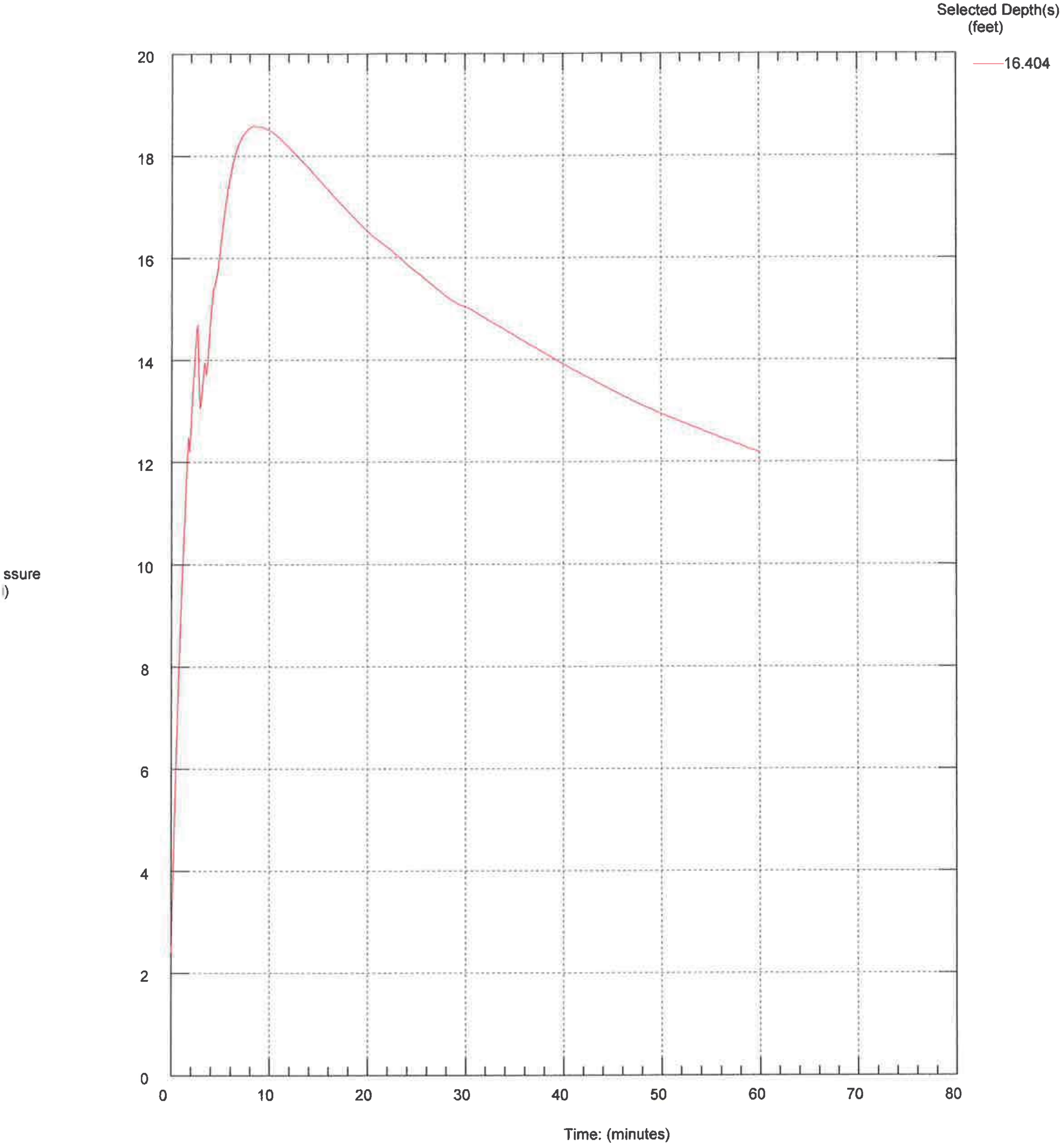
CPT Date/Time: 10/22/2013 10:53:00 PM
Location: BC1,SCPT6 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT



HDR ENG. / BC1-SCPT-6 / BIG CK. DAM NEWPORT

Operator SAV/CM
Sounding: VEI434BC1SCPT6(482)
Cone Used: DSG0736

CPT Date/Time: 10/22/2013 10:53:00 PM
Location: BC1SCPT6 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT

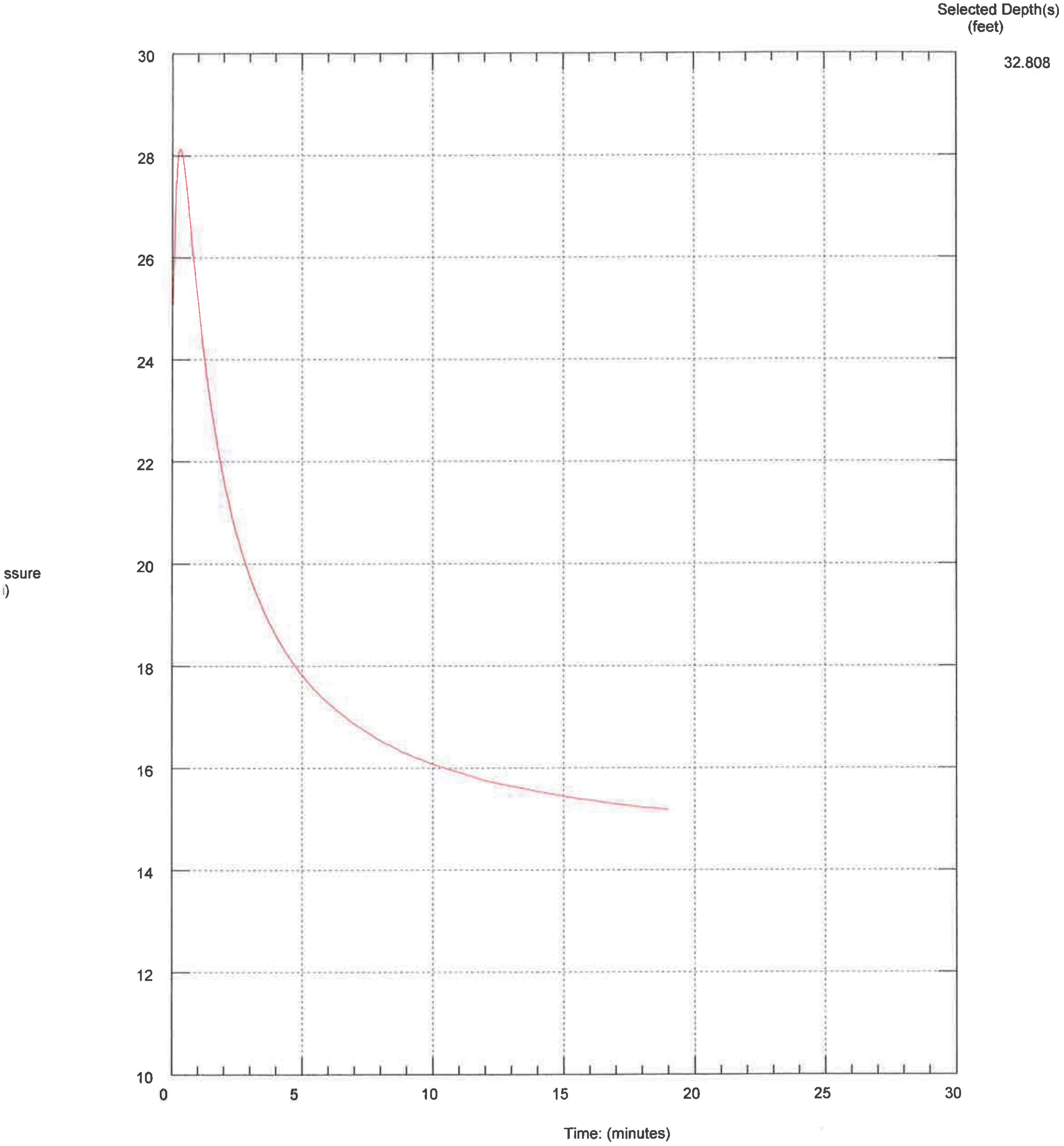


Maximum Pressure = 18.585 psi
Hydrostatic Pressure [illegible] psi

HDR ENG. / BC1-SCPT-6 / BIG CK. DAM NEWPORT

Operator SAV/CM
Sounding: VEI434BC1SCPT6(482)
Cone Used: DSG0736

CPT Date/Time: 10/22/2013 10:53:00 PM
Location: BC1SCPT6 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT

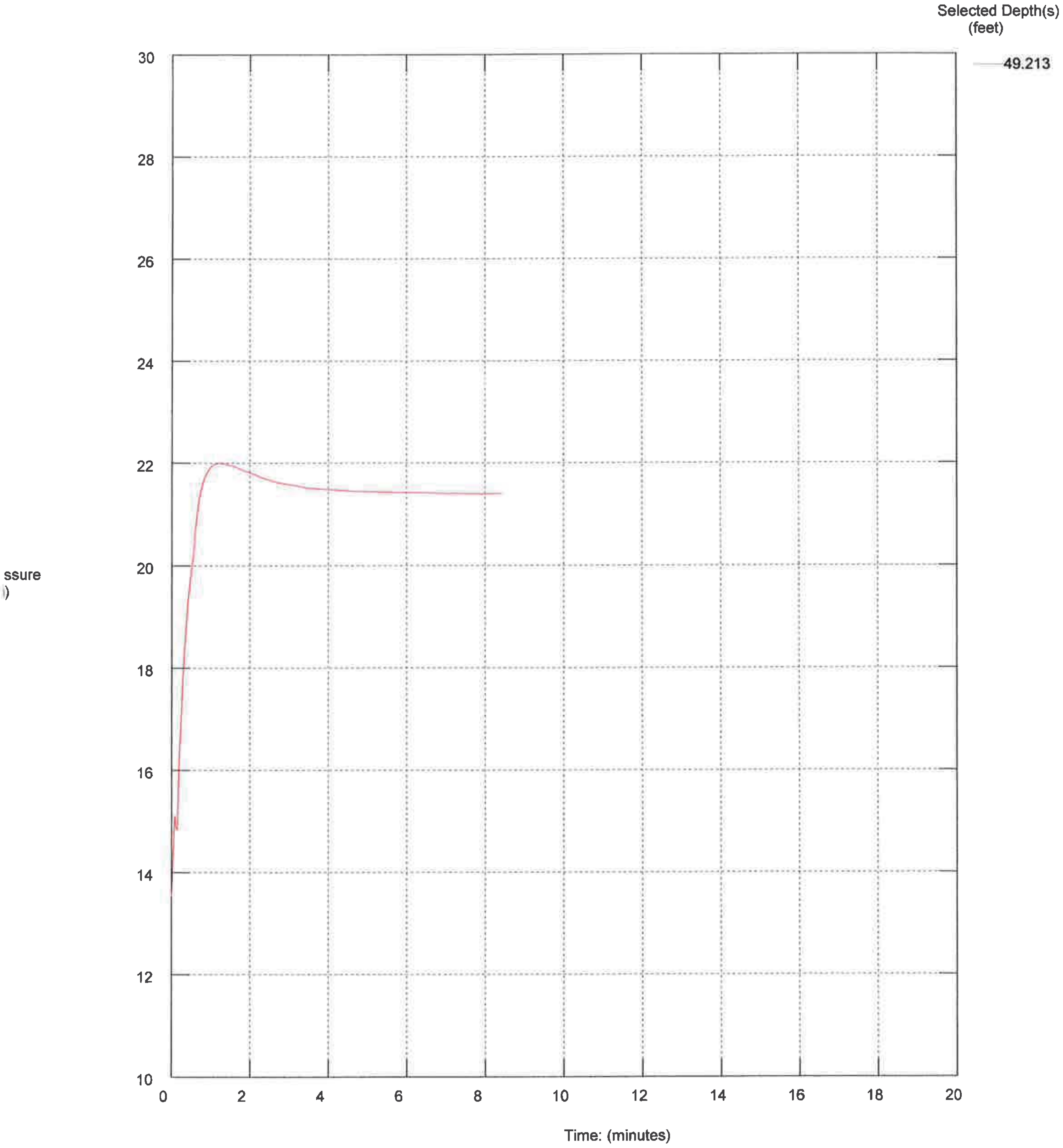


Maximum Pressure = 28.129 psi
Hydrostatic Pressure =

HDR ENG. / BC1-SCPT-6 / BIG CK. DAM NEWPORT

Operator SAV/CM
Sounding: VEI434BC1SCPT6(482)
Cone Used: DSG0736

CPT Date/Time: 10/22/2013 10:53:00 PM
Location: BC1SCPT6 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT

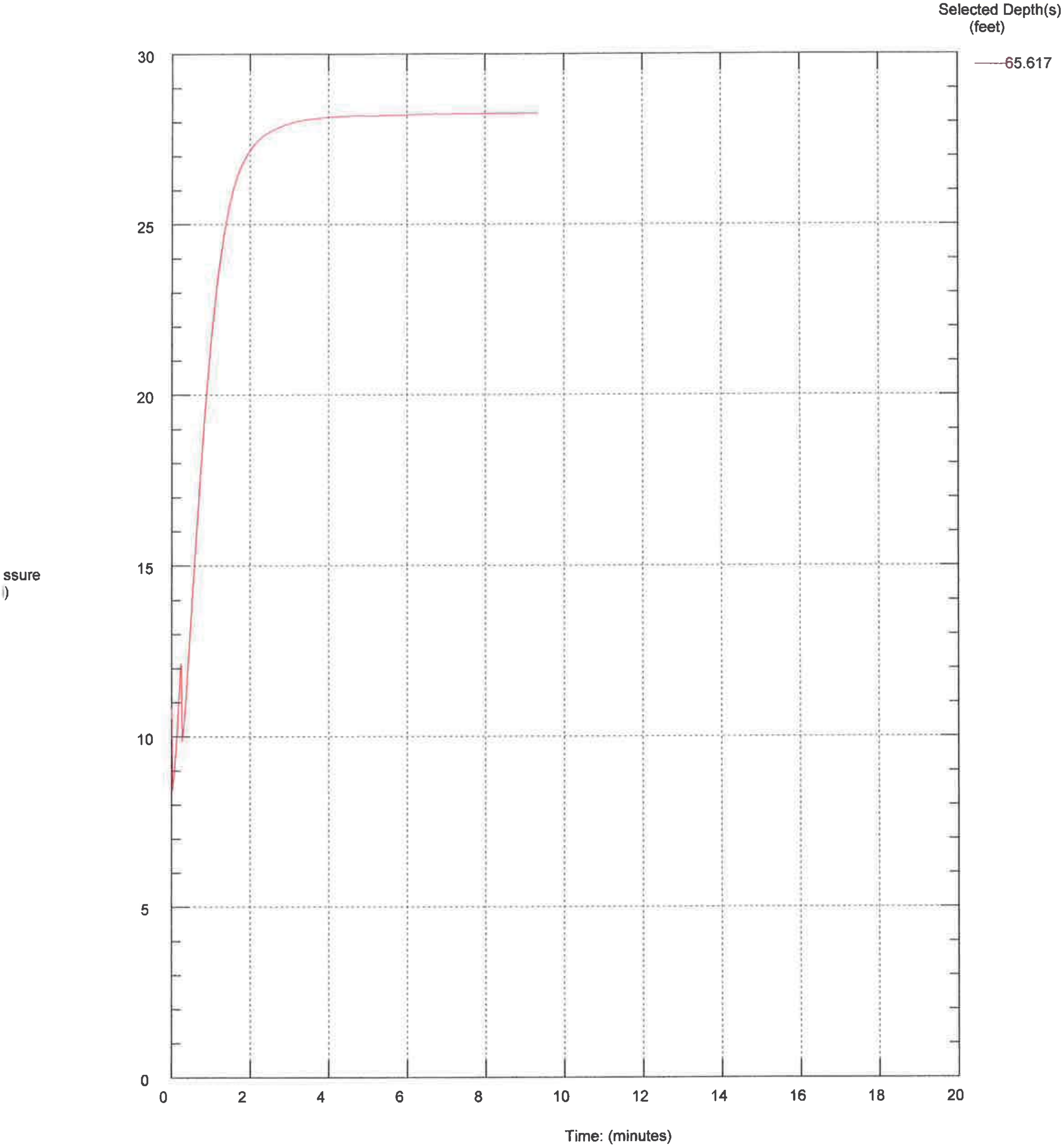


Maximum Pressure = 21.995 psi
Hydrostatic Pressure = [illegible]

HDR ENG. / BC1-SCPT-6 / BIG CK. DAM NEWPORT

Operator SAV/CM
Sounding: VEI434BC1SCPT6(482)
Cone Used: DSG0736

CPT Date/Time: 10/22/2013 10:53:00 PM
Location: BC1SCPT6 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT

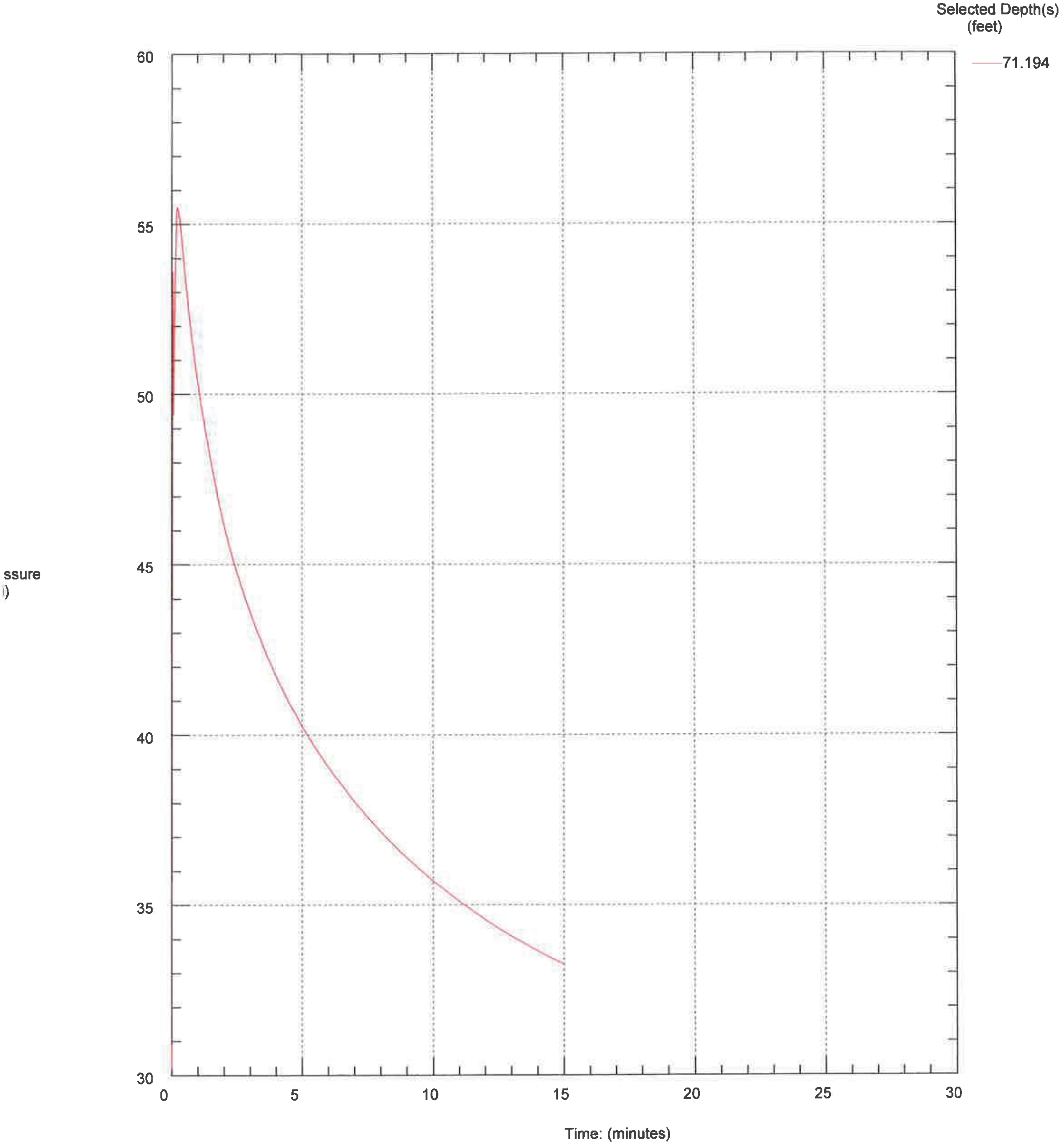


Maximum Pressure = 28.263 psi
Hydrostatic Pressure

HDR ENG. / BC1-SCPT-6 / BIG CK. DAM NEWPORT

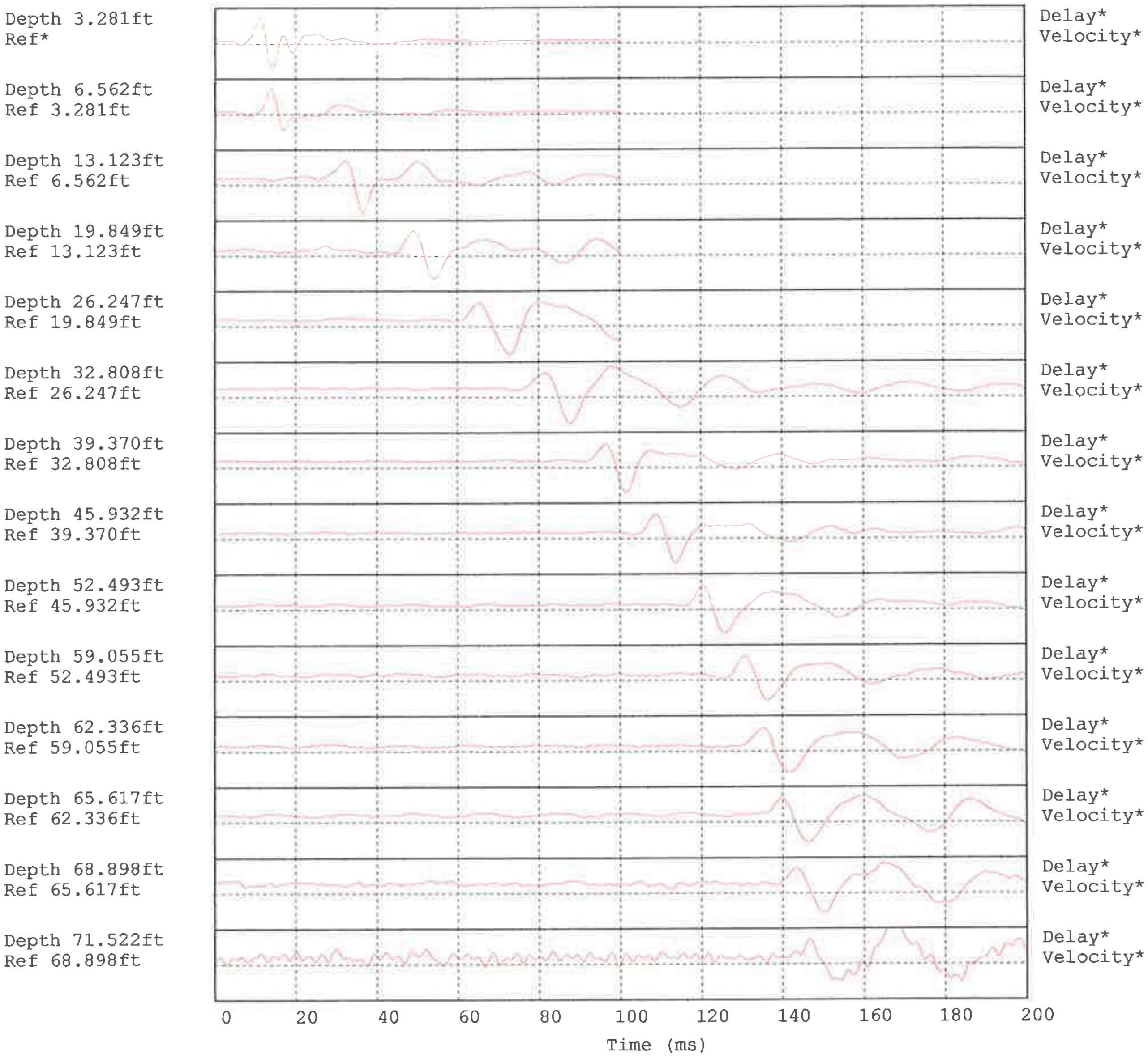
Operator SAV/CM
Sounding: VEI434BC1SCPT6(482)
Cone Used: DSG0736

CPT Date/Time: 10/22/2013 10:53:00 PM
Location: BC1SCPT6 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT



Maximum Pressure = 55.486 psi
Hydrostatic Pressure = [illegible]

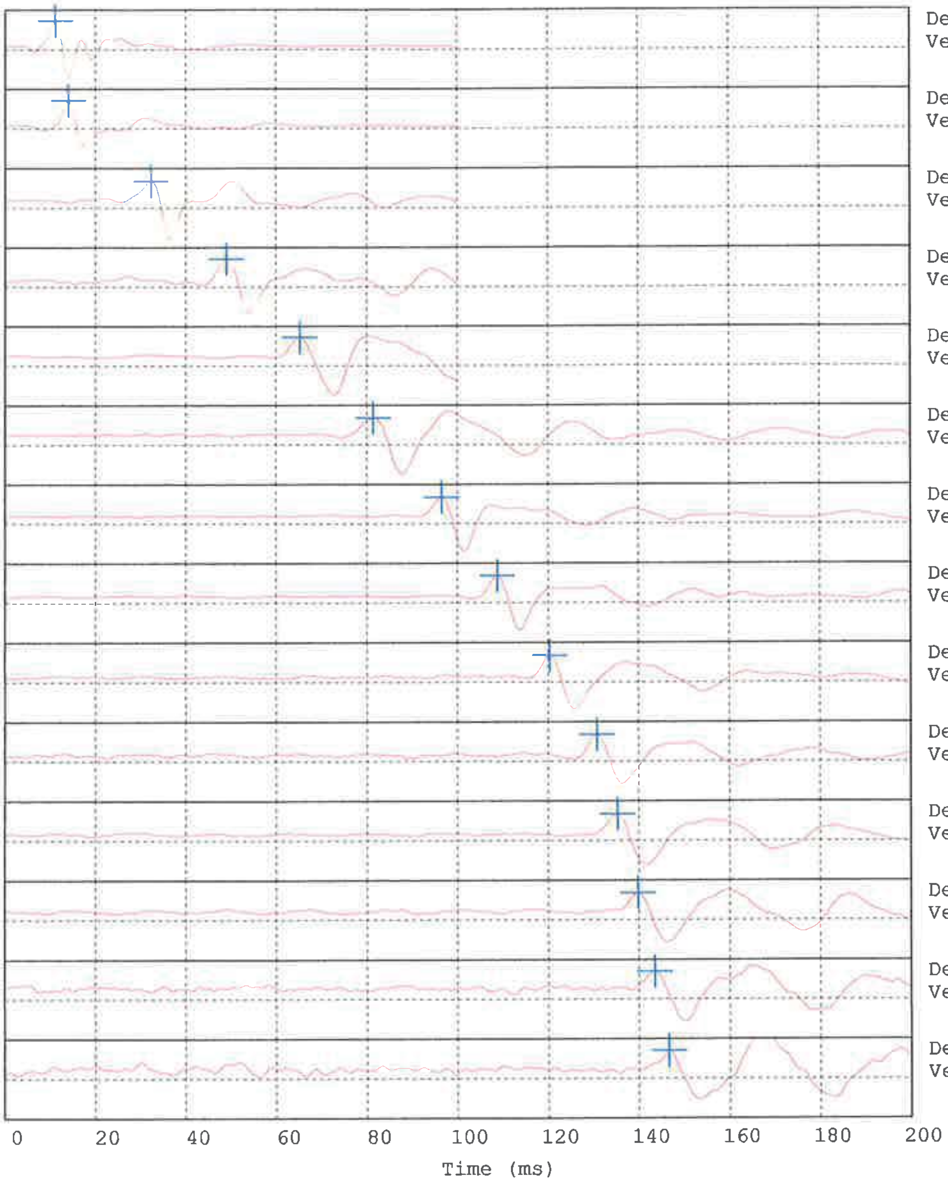
HDR ENG. / BC1,SCPT6 / BIG CK. NEWPORT



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

THE SHEAR WAVE PICKS ARE COMPUTER GENERATED VALUES (STAFF REVIEW RECOMMENDED)

HDR ENG. / BC1,SCPT6 / BIG CK. NEWPORT

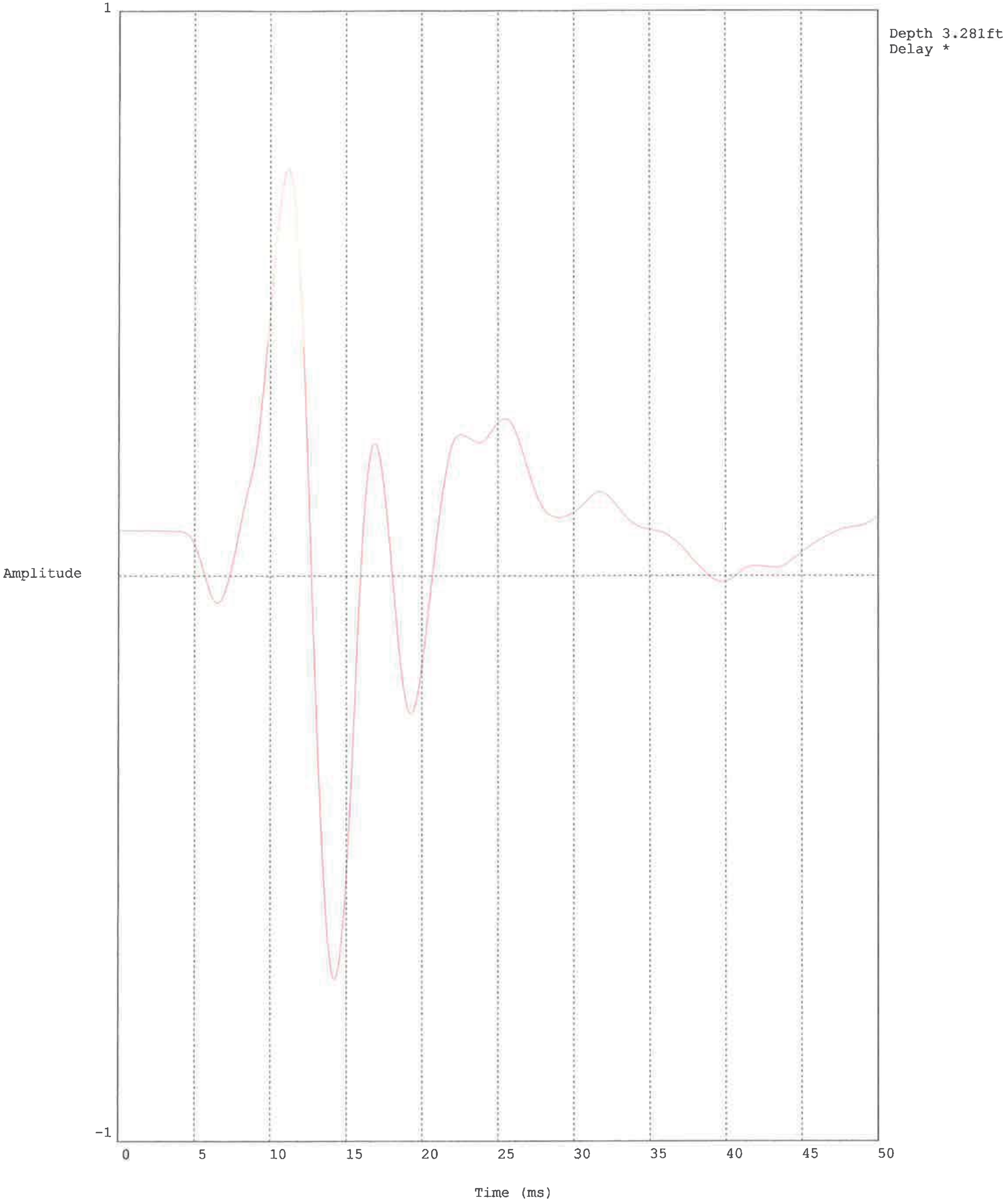


Depth 3.281ft Ref*	Delay 11.21ms Velocity*
Depth 6.562ft Ref 3.281ft	Delay 14.14ms Velocity 834.63ft/s
Depth 13.123ft Ref 6.562ft	Delay 32.30ms Velocity 328.86ft/s
Depth 19.849ft Ref 13.123ft	Delay 49.06ms Velocity 388.09ft/s
Depth 26.247ft Ref 19.849ft	Delay 65.27ms Velocity 387.96ft/s
Depth 32.808ft Ref 26.247ft	Delay 81.44ms Velocity 401.55ft/s
Depth 39.370ft Ref 32.808ft	Delay 96.52ms Velocity 432.17ft/s
Depth 45.932ft Ref 39.370ft	Delay 108.86ms Velocity 528.96ft/s
Depth 52.493ft Ref 45.932ft	Delay 120.42ms Velocity 565.40ft/s
Depth 59.055ft Ref 52.493ft	Delay 130.77ms Velocity 632.07ft/s
Depth 62.336ft Ref 59.055ft	Delay 135.30ms Velocity 722.31ft/s
Depth 65.617ft Ref 62.336ft	Delay 139.83ms Velocity 722.49ft/s
Depth 68.898ft Ref 65.617ft	Delay 143.51ms Velocity 891.77ft/s
Depth 71.522ft Ref 68.898ft	Delay 146.55ms Velocity 859.90ft/s

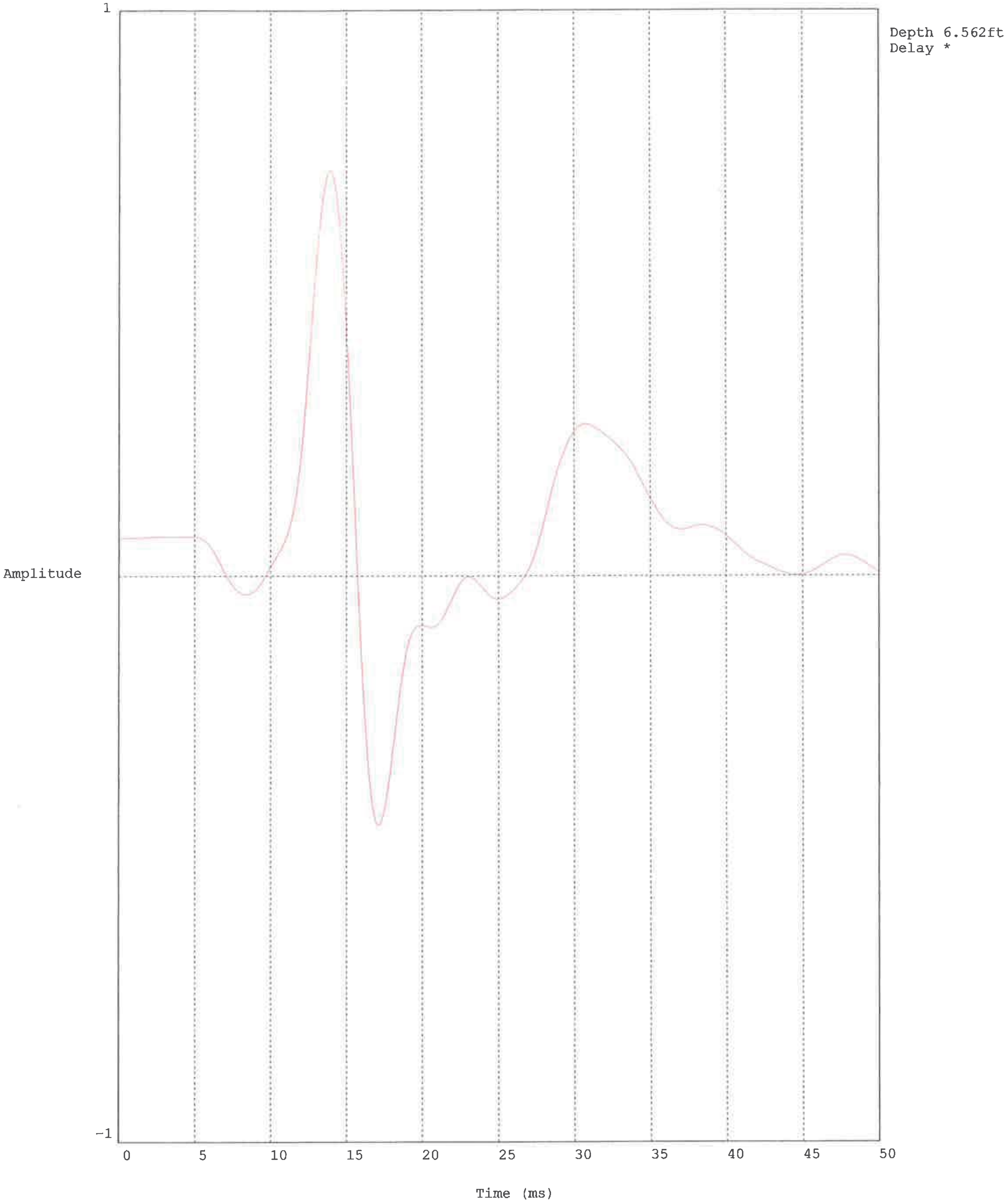
Hammer to Rod String Distance 1.3 (m)
* = Not Determined

THE SHEAR WAVE PICKS ARE COMPUTER GENERATED VALUES (STAFF REVIEW RECOMMENDED)

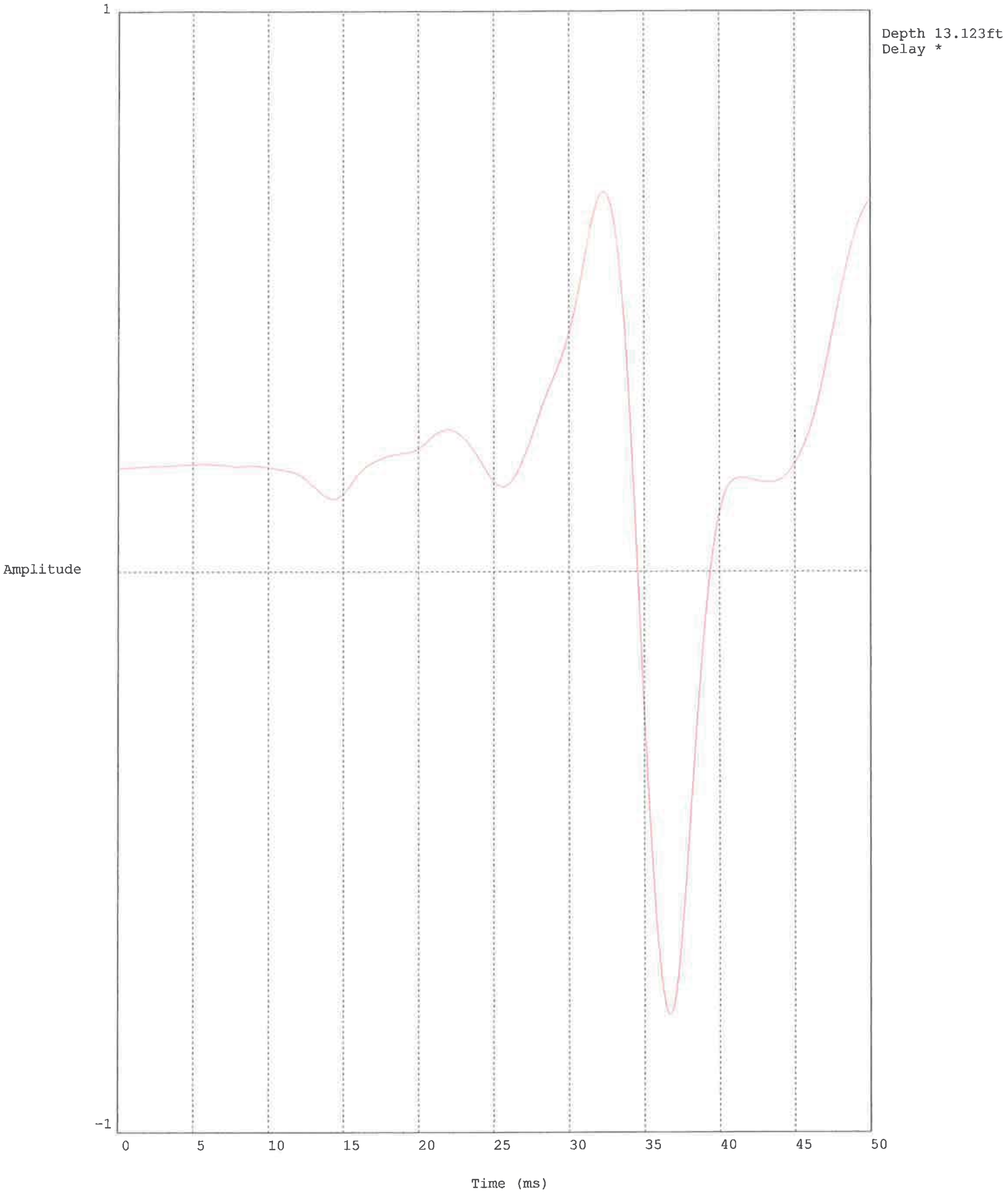
HDR ENG. / BC1,SCPT6 / BIG CK. NEWPORT



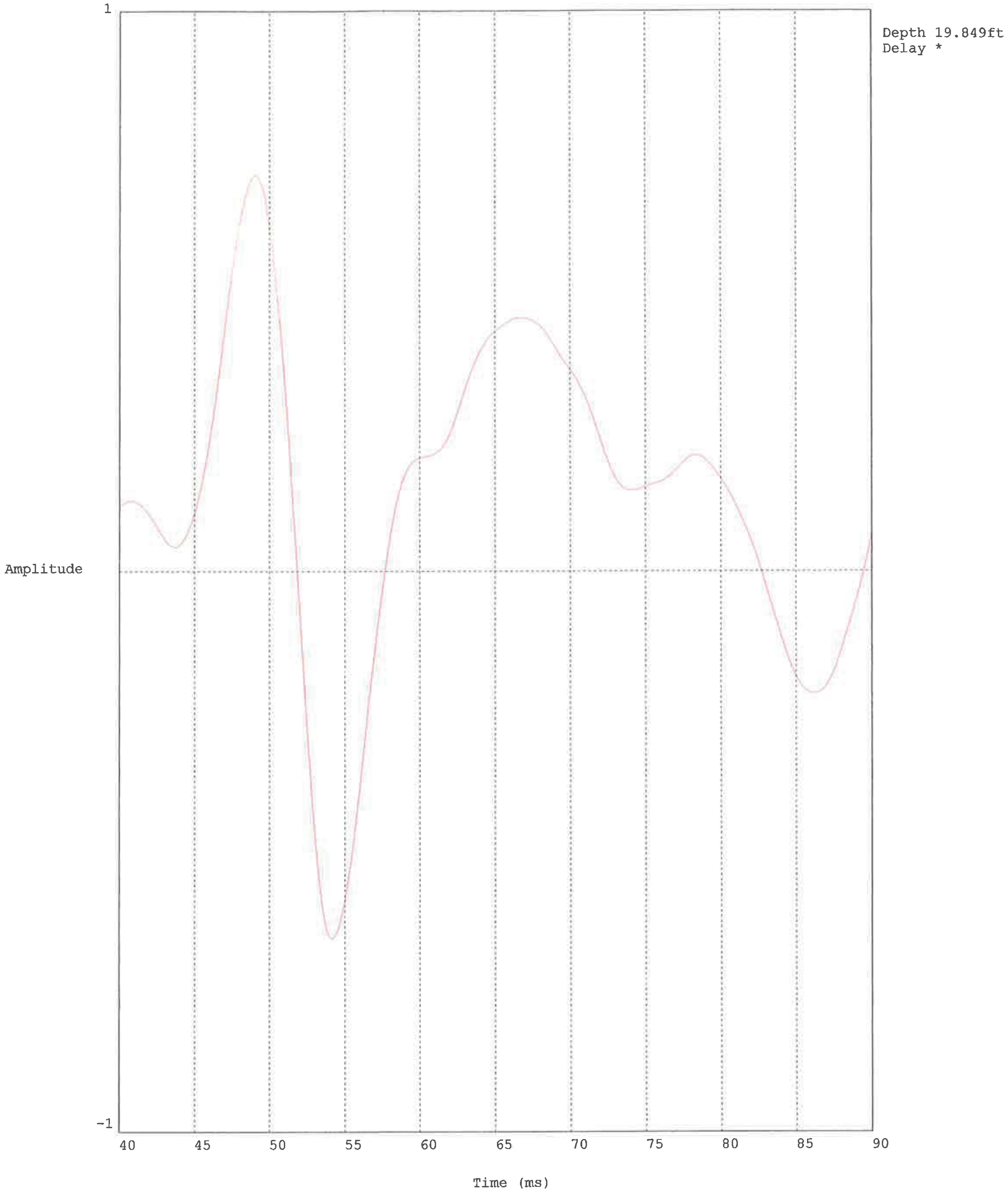
HDR ENG. / BC1,SCPT6 / BIG CK. NEWPORT



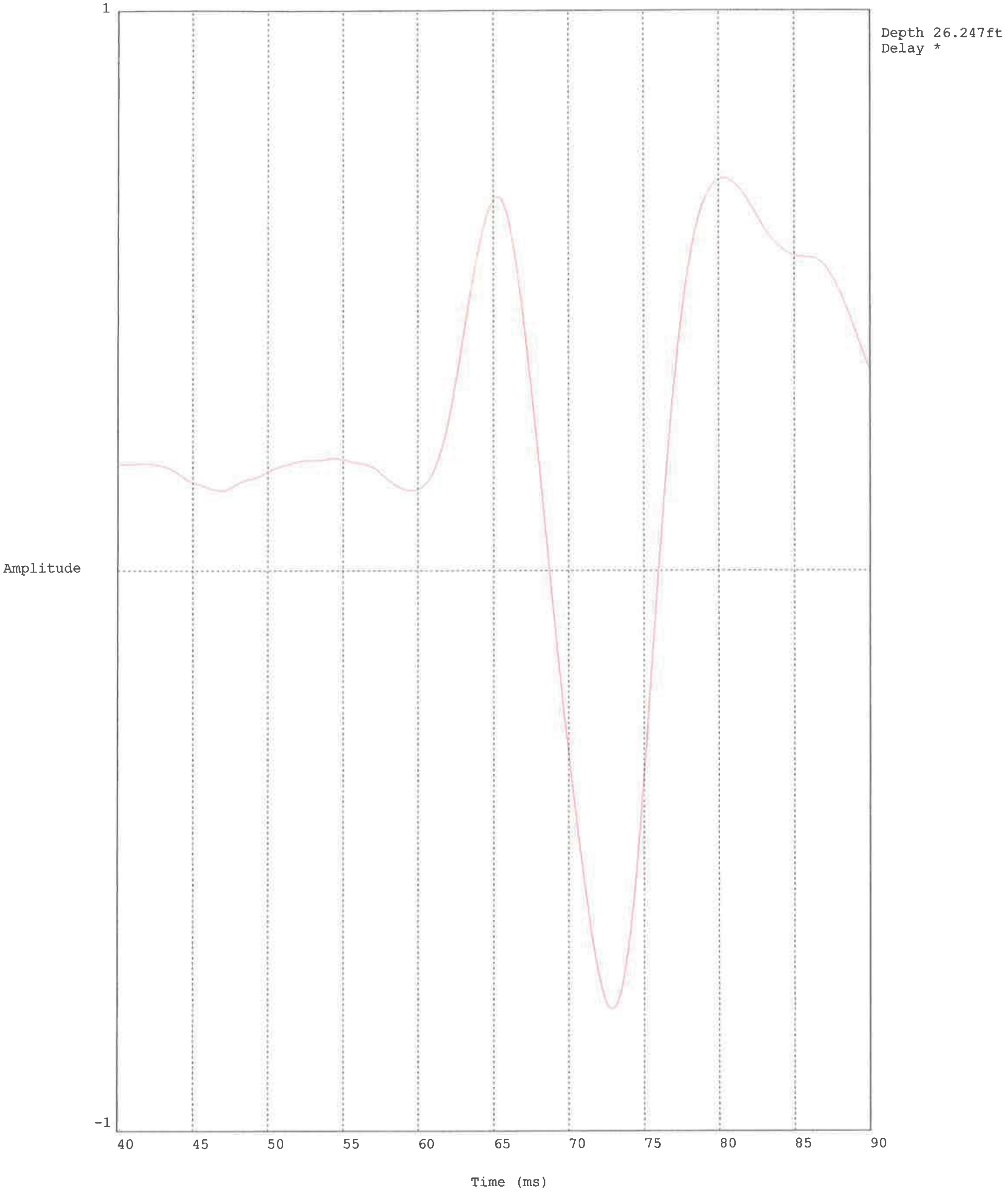
HDR ENG. / BC1,SCPT6 / BIG CK. NEWPORT



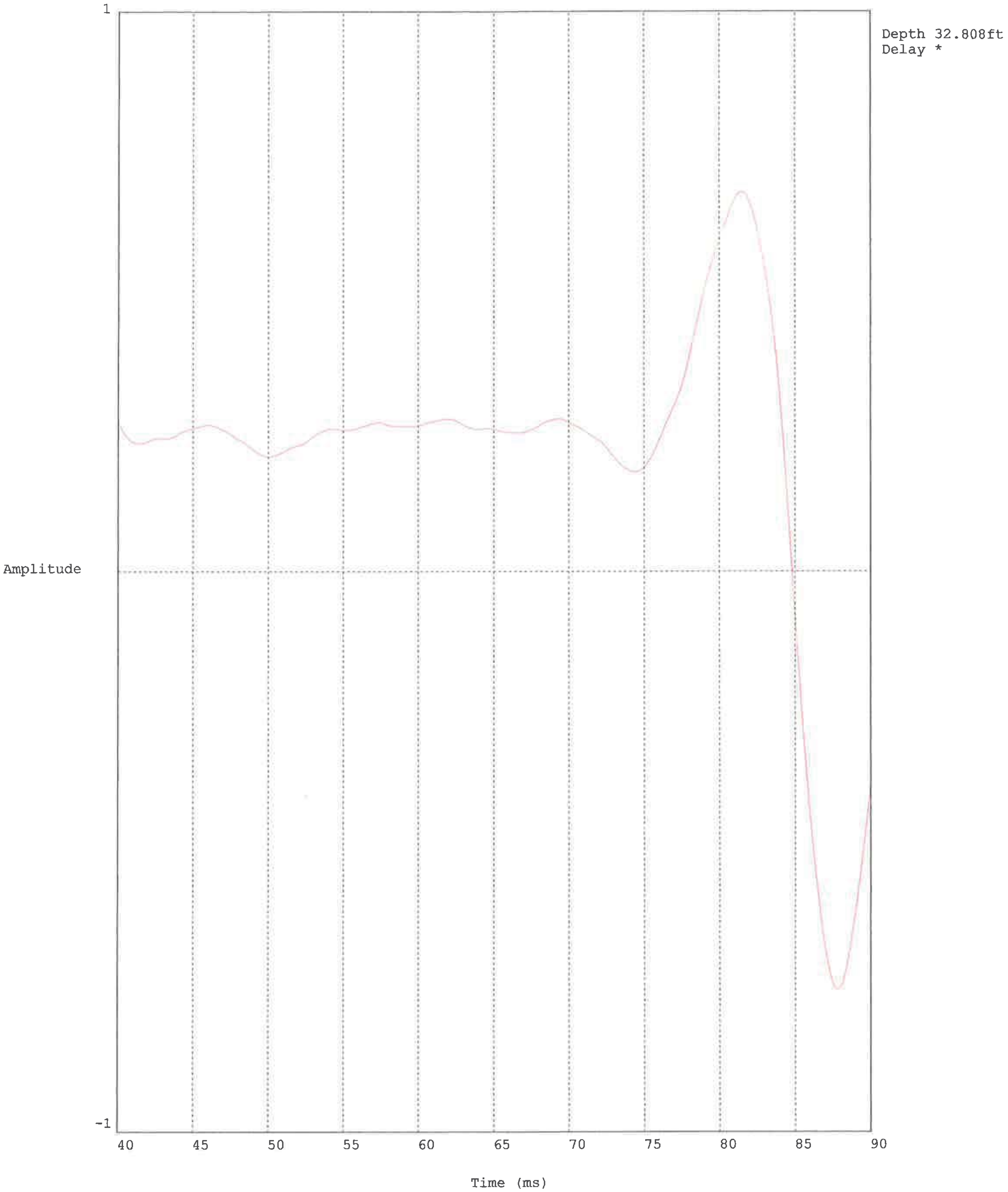
HDR ENG. / BC1,SCPT6 / BIG CK. NEWPORT



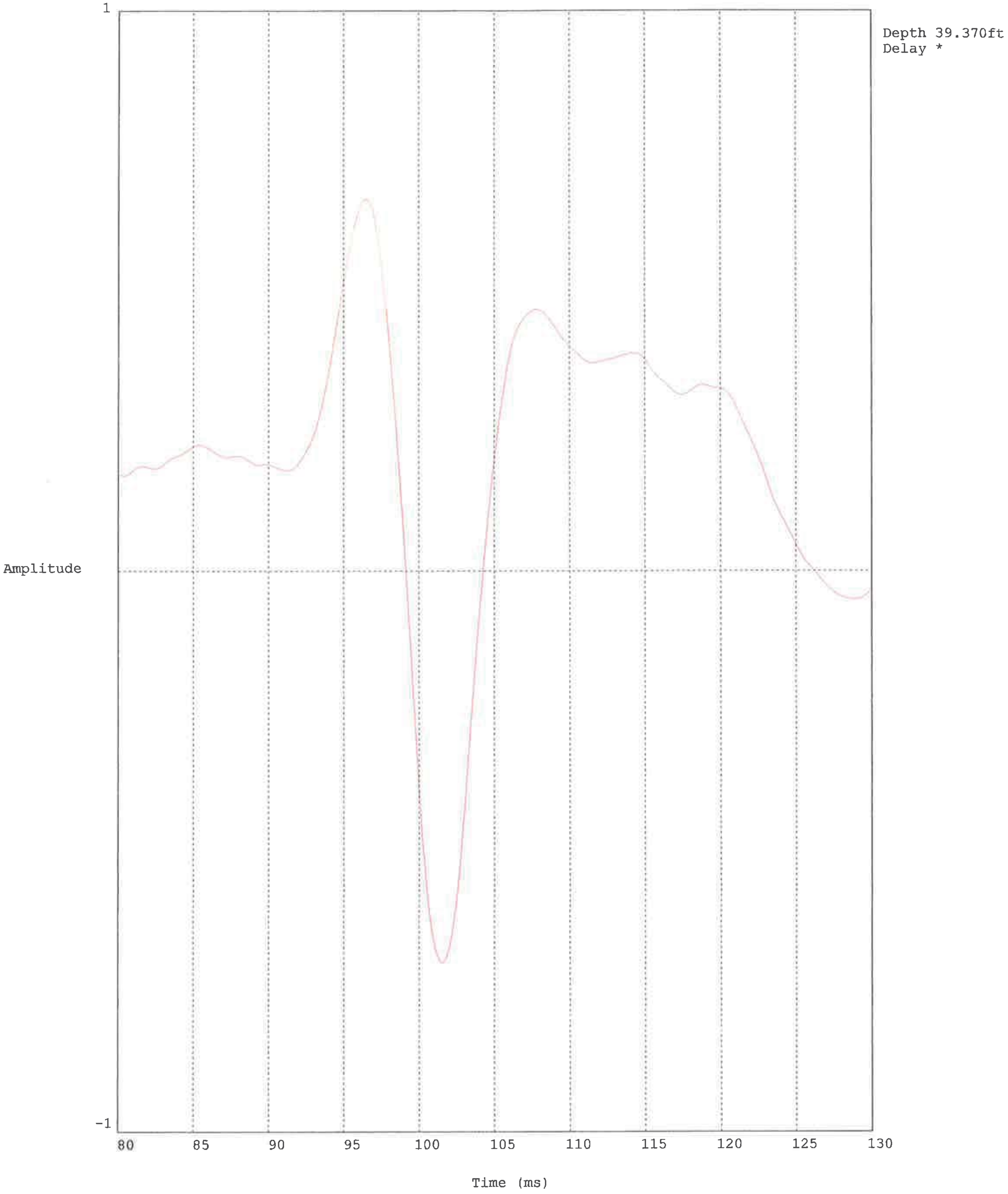
HDR ENG. / BC1,SCPT6 / BIG CK. NEWPORT



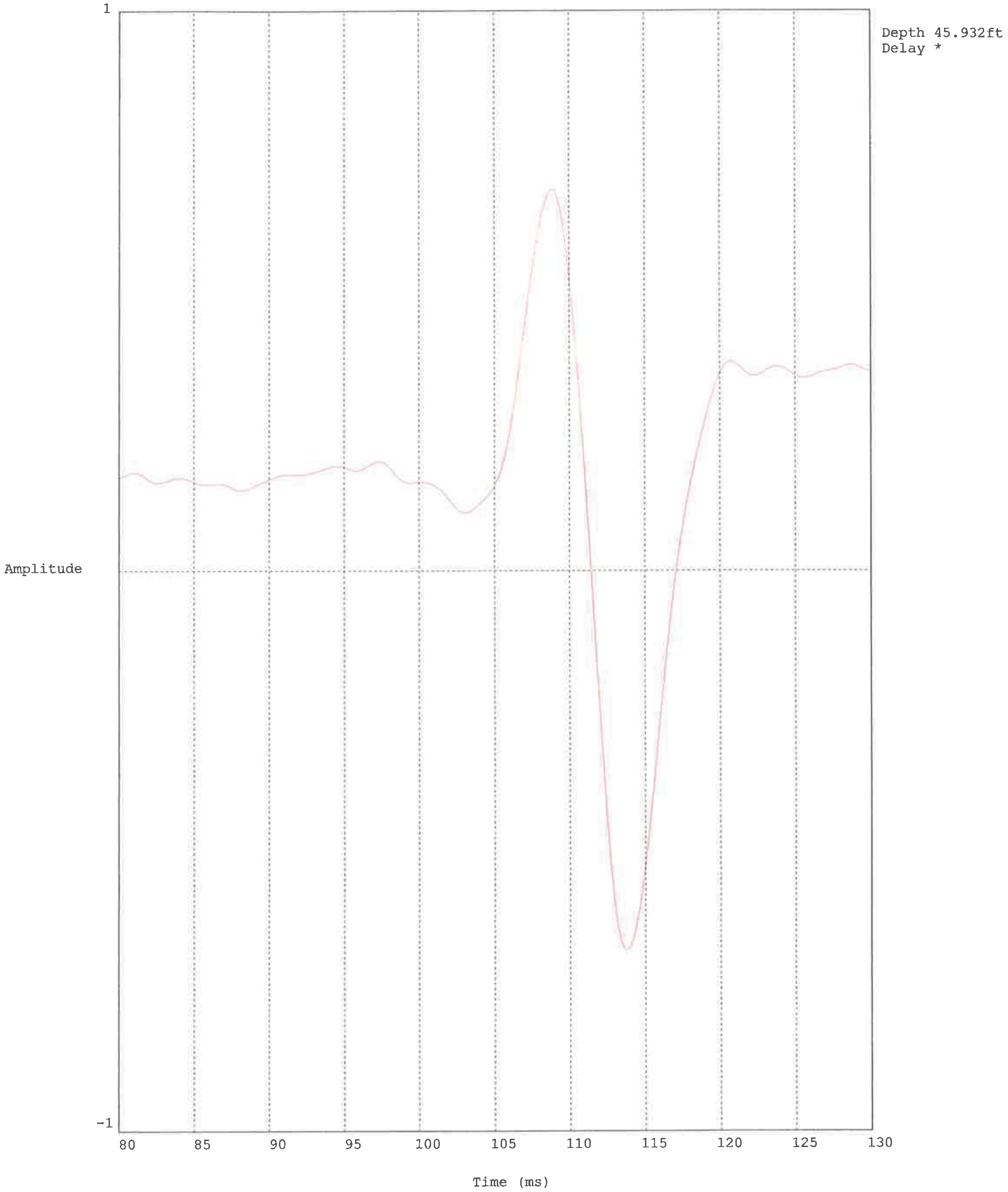
HDR ENG. / BC1,SCPT6 / BIG CK. NEWPORT



HDR ENG. / BC1,SCPT6 / BIG CK. NEWPORT



HDR ENG. / BC1,SCPT6 / BIG CK. NEWPORT



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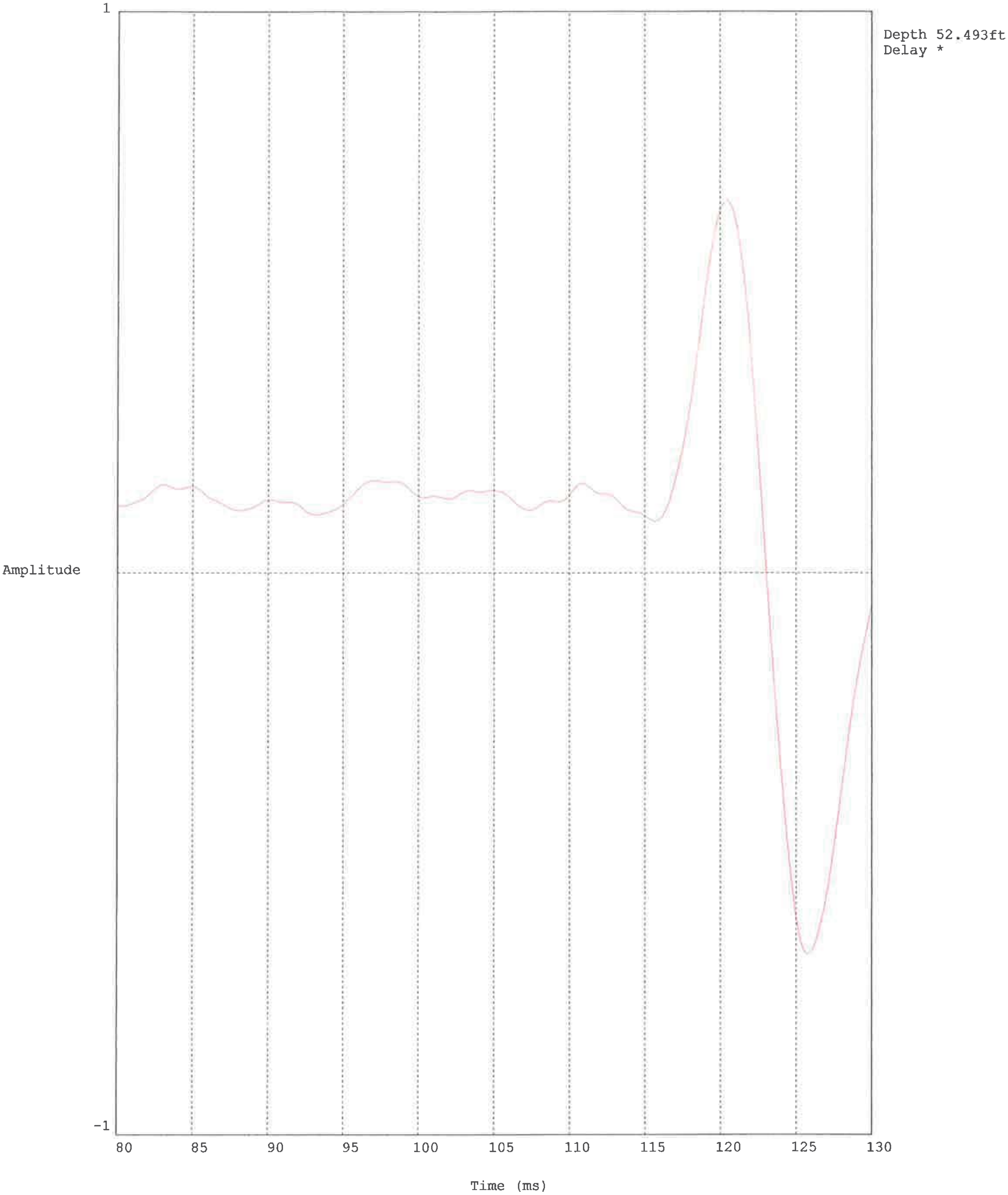

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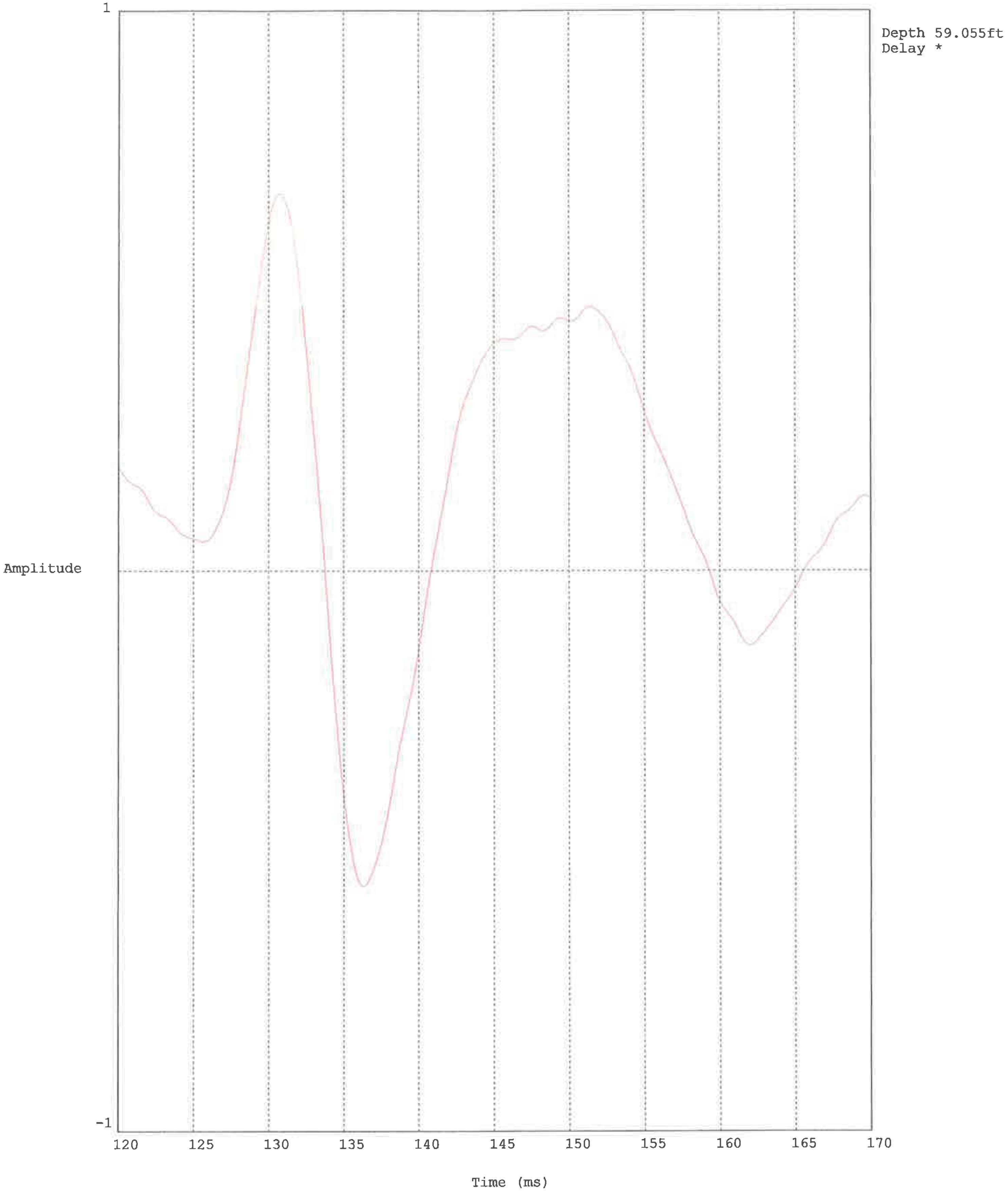

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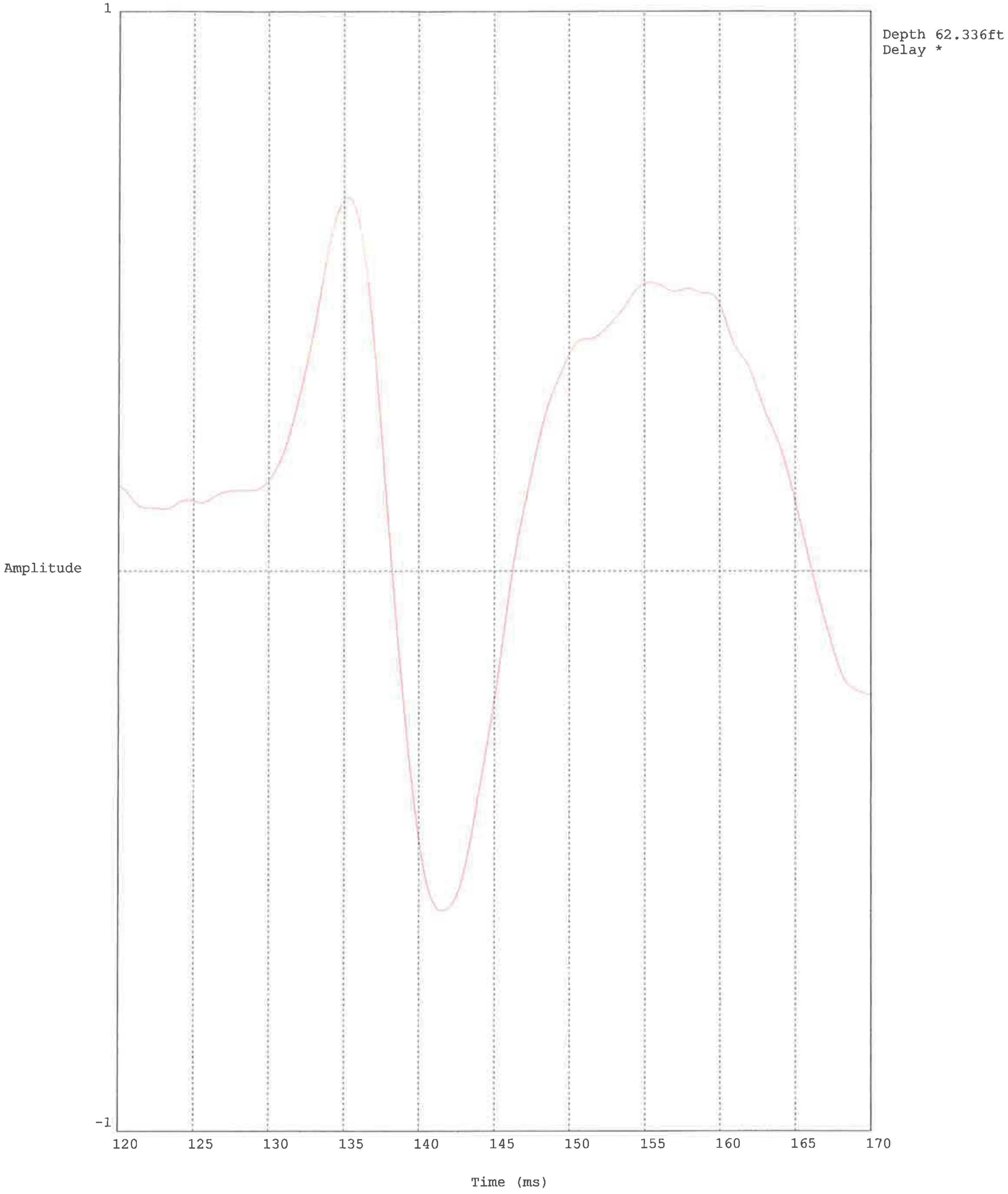
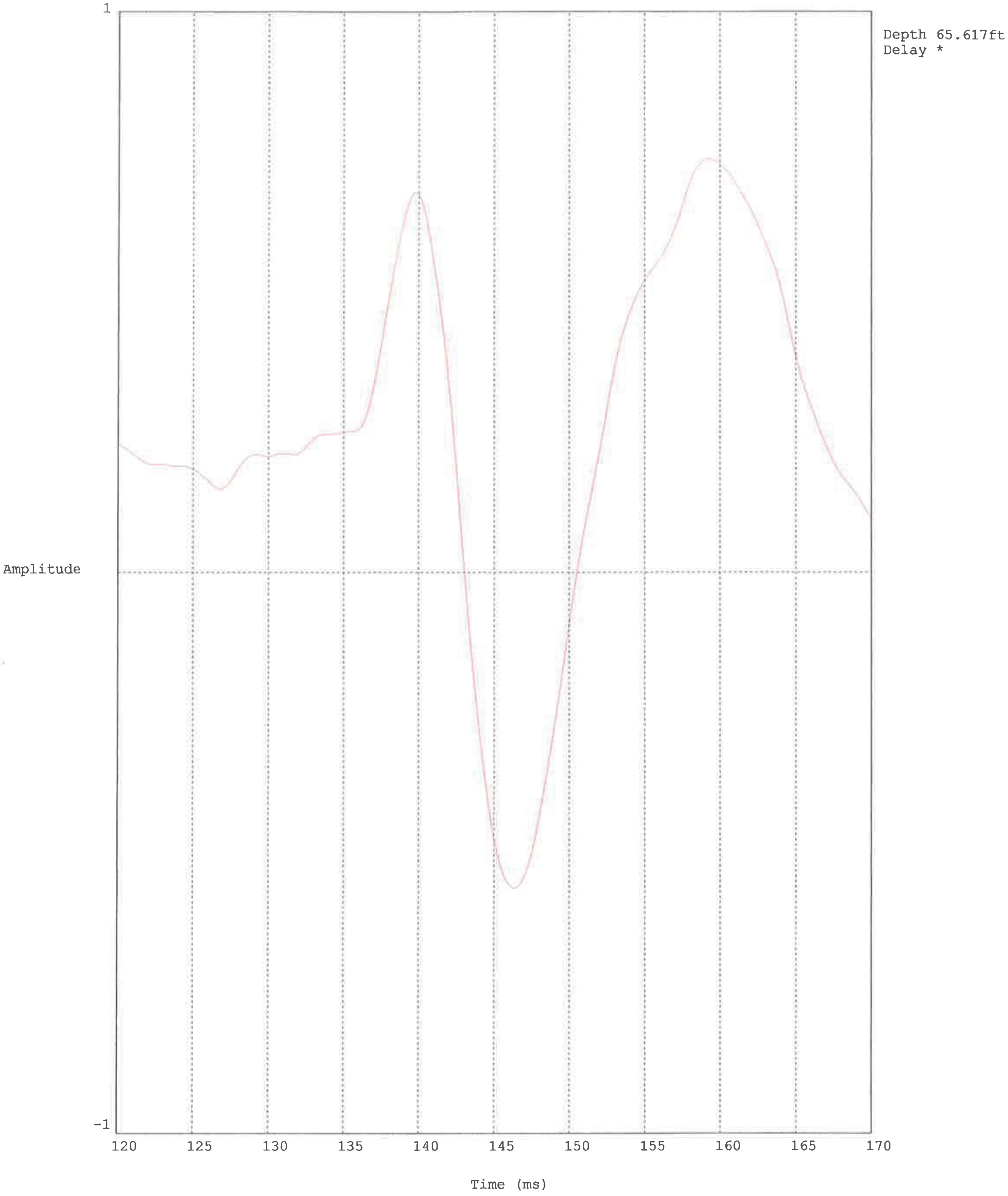
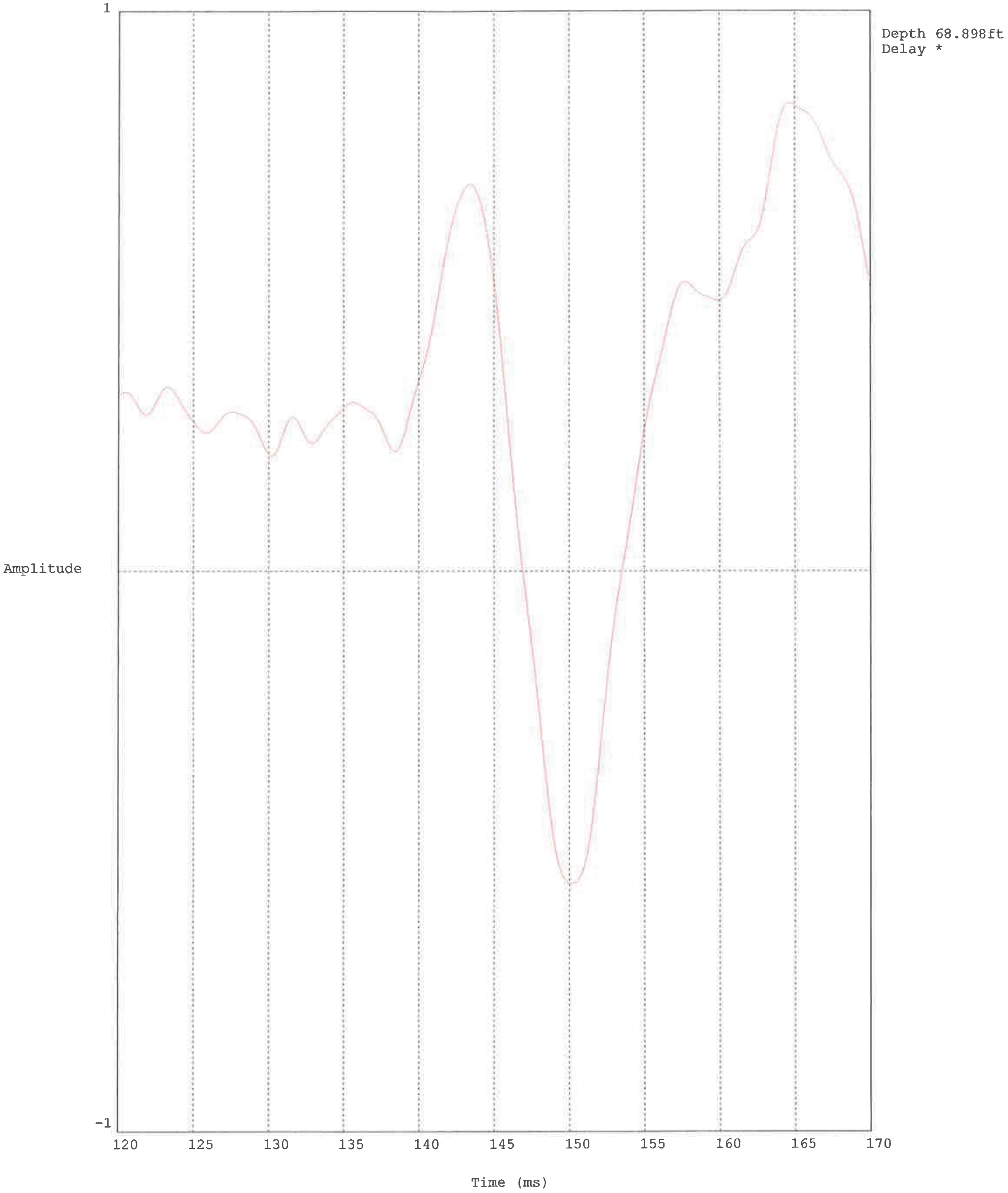


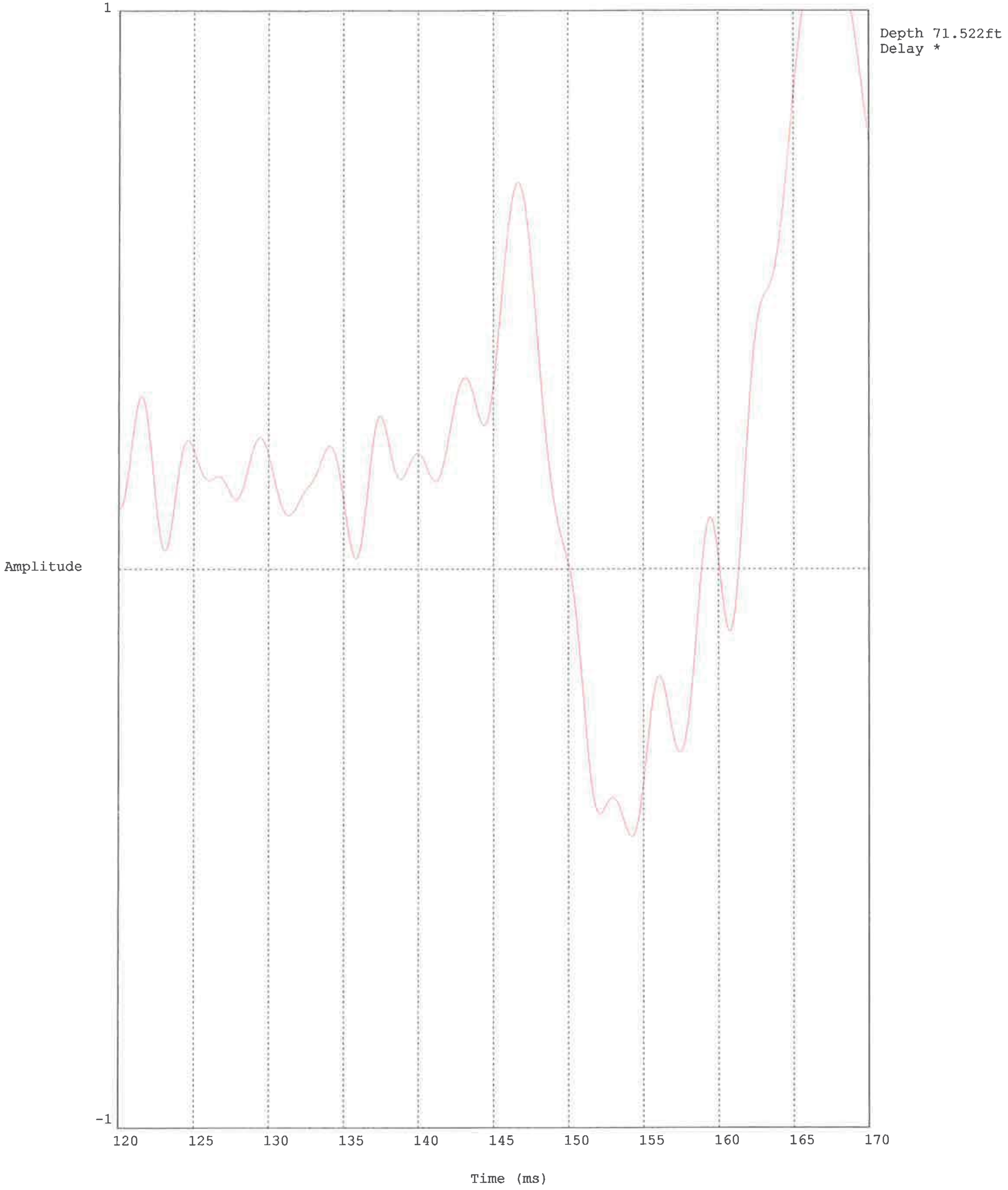
HDR ENG. / BC1,SCPT6 / BIG CK. NEWPORT



HDR ENG. / BC1,SCPT6 / BIG CK. NEWPORT



HDR ENG. / BC1,SCPT6 / BIG CK. NEWPORT



Data File:VEI434BC1SCPT6(482)
Operator:SAV/CM
Cone ID:DSG0736
Customer: BIG CK. DAM NEWPORT

10/22/2013 10:53:00 PM
Location:BC1,SCPT6 / BIG CK. NEWPORT
Job Number:HDR ENG./BIG CK. NEWPORT
Units:

Depth (ft)	Qt TSF	Fs TSF	Fs/Qt (%)	Pw PSI	SPT N* 60% Hammer	Zone	Soil Behavior Type UBC-1983
0.82	21.41	0.6108	2.853	0.194	9	4	silty clay to clay
0.98	22.15	1.0968	4.951	0.375	23	3	clay
1.15	28.49	1.4725	5.168	0.691	25	3	clay
1.31	28.68	1.5709	5.478	1.050	26	3	clay
1.48	25.40	1.3664	5.380	1.143	24	3	clay
1.64	20.51	1.0472	5.106	0.880	20	3	clay
1.80	16.66	0.5498	3.300	0.579	16	3	clay
1.97	11.95	0.4759	3.983	0.366	11	3	clay
2.13	6.66	0.4647	6.977	0.218	8	3	clay
2.30	5.16	0.4181	8.099	0.105	6	3	clay
2.46	6.89	0.5448	7.907	0.265	8	3	clay
2.62	13.98	0.6997	5.006	-0.670	11	3	clay
2.79	14.74	0.7783	5.279	-2.408	15	3	clay
2.95	19.63	0.9829	5.007	-2.370	18	3	clay
3.12	22.14	1.1692	5.280	-2.423	20	3	clay
3.28	20.32	1.1740	5.777	-1.896	21	3	clay
3.44	21.78	0.8278	3.800	-0.572	18	3	clay
3.61	13.88	0.5790	4.170	-1.079	16	3	clay
3.77	13.63	0.7427	5.450	-0.651	17	3	clay
3.94	27.04	1.0751	3.976	1.107	14	4	silty clay to clay
4.10	26.25	1.0296	3.922	0.729	15	4	silty clay to clay
4.27	18.72	0.8901	4.755	0.359	19	3	clay
4.43	13.83	0.7375	5.334	0.265	15	3	clay
4.59	15.76	0.6331	4.017	0.713	14	3	clay
4.76	12.97	0.6253	4.821	0.839	14	3	clay
4.92	16.38	0.6015	3.673	1.036	12	3	clay
5.09	9.26	0.4631	5.001	1.162	8	4	silty clay to clay
5.25	13.06	0.4004	3.065	1.418	10	3	clay
5.41	7.95	0.2847	3.579	-0.062	6	4	silty clay to clay
5.58	6.04	0.1638	2.713	0.026	6	3	clay
5.74	5.41	0.1487	2.748	0.244	5	3	clay
5.91	5.15	0.1995	3.876	0.526	5	3	clay
6.07	6.35	0.2144	3.375	0.684	6	3	clay
6.23	6.24	0.2589	4.150	2.430	6	3	clay
6.40	6.23	0.1700	2.726	2.282	6	3	clay
6.56	5.56	0.1600	2.880	1.002	5	3	clay
6.73	5.05	0.1532	3.033	3.779	5	3	clay
6.89	4.12	0.1459	3.542	8.423	4	3	clay
7.05	4.71	0.2031	4.312	5.477	5	3	clay
7.22	5.91	0.2898	4.906	5.453	6	3	clay
7.38	7.56	0.3000	3.970	5.513	6	3	clay
7.55	5.39	0.2249	4.176	5.068	6	3	clay
7.71	4.45	0.1469	3.298	5.187	4	3	clay
7.87	4.09	0.1083	2.647	5.328	4	3	clay
8.04	3.64	0.1049	2.883	5.414	4	3	clay
8.20	3.45	0.1240	3.596	5.568	4	3	clay
8.37	4.28	0.1094	2.557	5.737	4	3	clay
8.53	5.09	0.1083	2.130	5.704	5	3	clay
8.69	4.98	0.1330	2.673	5.737	4	3	clay
8.86	3.97	0.1451	3.658	5.933	4	3	clay
9.02	4.10	0.1753	4.275	5.811	4	3	clay
9.19	4.10	0.1632	3.983	6.060	4	3	clay
9.35	4.88	0.1892	3.880	6.144	5	3	clay
9.51	5.84	0.1476	2.528	6.075	5	3	clay
9.68	5.46	0.1113	2.038	5.917	3	4	silty clay to clay
9.84	4.70	0.1048	2.232	5.893	4	3	clay
10.01	3.90	0.1219	3.127	6.003	4	3	clay

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

Depth (ft)	Qt TSF	Fs TSF	Fs/Qt (%)	Pw PSI	SPT N* 60% Hammer	Zone	Soil Behavior Type UBC-1983
10.17	4.60	0.1609	3.501	6.237	4	3	clay
10.33	4.92	0.2058	4.184	6.352	5	3	clay
10.50	4.96	0.2044	4.125	6.448	5	3	clay
10.66	5.37	0.2090	3.891	6.450	5	3	clay
10.83	4.80	0.2519	5.249	6.495	5	3	clay
10.99	6.00	0.2451	4.085	6.660	6	3	clay
11.15	7.45	0.2256	3.030	6.366	7	3	clay
11.32	7.14	0.2557	3.582	5.639	6	3	clay
11.48	5.64	0.2896	5.135	5.218	6	3	clay
11.65	6.65	0.2414	3.629	5.293	7	3	clay
11.81	8.18	0.2407	2.942	4.575	7	3	clay
11.98	8.11	0.2033	2.507	4.035	5	4	silty clay to clay
12.14	8.63	0.2368	2.745	3.506	6	4	silty clay to clay
12.30	10.49	0.2061	1.965	3.355	6	4	silty clay to clay
12.47	10.82	0.2660	2.457	3.107	7	4	silty clay to clay
12.63	9.76	0.2605	2.670	3.025	6	4	silty clay to clay
12.80	6.04	0.1721	2.850	2.896	5	4	silty clay to clay
12.96	5.98	0.1223	2.044	3.054	4	4	silty clay to clay
13.12	6.21	0.1409	2.269	3.224	4	4	silty clay to clay
13.29	6.79	0.1366	2.011	4.331	5	4	silty clay to clay
13.45	8.95	0.1683	1.879	3.996	5	4	silty clay to clay
13.62	7.88	0.1761	2.235	4.037	5	4	silty clay to clay
13.78	6.88	0.2428	3.530	4.042	7	3	clay
13.94	5.67	0.2100	3.702	4.286	7	3	clay
14.11	7.97	0.7229	9.075	4.336	7	3	clay
14.27	9.06	0.4609	5.086	4.192	7	3	clay
14.44	6.06	0.3567	5.890	4.133	7	3	clay
14.60	6.16	0.1975	3.207	4.314	6	3	clay
14.76	7.16	0.2363	3.300	4.324	5	4	silty clay to clay
14.93	9.20	0.2089	2.270	4.219	6	4	silty clay to clay
15.09	12.66	0.2888	2.282	3.726	6	5	clayey silt to silty clay
15.26	14.32	0.2941	2.053	3.384	7	5	clayey silt to silty clay
15.42	17.30	0.2687	1.553	3.394	7	5	clayey silt to silty clay
15.58	14.40	0.3048	2.117	2.932	7	5	clayey silt to silty clay
15.75	11.17	0.3178	2.845	2.762	8	4	silty clay to clay
15.91	10.88	0.4436	4.078	2.370	7	4	silty clay to clay
16.08	11.47	0.3884	3.385	2.026	9	3	clay
16.24	7.02	0.4110	5.853	1.758	8	3	clay
16.40	5.36	0.3685	6.875	2.423	6	3	clay
16.57	6.30	0.2971	4.719	9.411	6	3	clay
16.73	5.67	0.2147	3.785	10.288	6	3	clay
16.90	6.25	0.1578	2.526	14.514	6	3	clay
17.06	6.11	0.1701	2.784	15.390	6	3	clay
17.22	7.27	0.5439	7.478	16.385	10	3	clay
17.39	16.43	1.5318	9.324	22.954	21	3	clay
17.55	42.98	2.3768	5.529	38.372	38	3	clay
17.72	58.88	3.9704	6.744	33.286	73	11	very stiff fine grained (*)
17.88	125.36	4.7027	3.751	9.820	103	11	very stiff fine grained (*)
18.04	136.98	5.6196	4.102	3.310	55	5	clayey silt to silty clay
18.21	82.43	3.4157	4.144	2.466	37	5	clayey silt to silty clay
18.37	15.33	0.5400	3.523	3.243	22	4	silty clay to clay
18.54	5.02	0.1500	2.986	7.921	7	3	clay
18.70	2.17	0.0600	2.771	11.532	3	3	clay
18.86	2.16	0.0500	2.310	11.487	2	3	clay
19.03	2.16	0.0500	2.310	11.451	1	1	sensitive fine grained
19.19	1.16	0.0300	2.578	11.446	1	1	sensitive fine grained
19.36	2.16	0.0400	1.848	11.441	1	1	sensitive fine grained
19.52	2.16	0.0200	0.925	11.422	1	1	sensitive fine grained
19.69	2.16	0.0336	1.553	11.424	1	1	sensitive fine grained
19.85	2.16	0.0419	1.937	11.427	1	1	sensitive fine grained
20.01	3.60	0.0616	1.712	19.195	2	1	sensitive fine grained
20.18	3.88	0.0786	2.026	19.776	4	3	clay

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

Depth (ft)	Qt TSF	Fs TSF	Fs/Qt (%)	Pw PSI	SPT N* 60% Hammer	Zone	Soil Behavior Type UBC-1983
20.34	4.49	0.0998	2.222	21.019	3	4	silty clay to clay
20.51	5.47	0.0782	1.430	21.715	3	4	silty clay to clay
20.67	4.96	0.0988	1.992	20.221	3	4	silty clay to clay
20.83	5.94	0.1108	1.865	23.346	4	4	silty clay to clay
21.00	6.97	0.0947	1.360	23.007	4	4	silty clay to clay
21.16	6.39	0.1272	1.993	22.882	4	4	silty clay to clay
21.33	6.29	0.1170	1.859	24.958	4	4	silty clay to clay
21.49	7.34	0.1028	1.401	22.980	4	4	silty clay to clay
21.65	6.04	0.1380	2.287	23.380	4	4	silty clay to clay
21.82	6.49	0.1541	2.375	28.364	4	4	silty clay to clay
21.98	6.86	0.1635	2.382	28.914	4	4	silty clay to clay
22.15	7.10	0.1545	2.177	29.012	4	4	silty clay to clay
22.31	6.36	0.1347	2.118	29.041	4	4	silty clay to clay
22.47	5.71	0.1246	2.182	28.904	4	4	silty clay to clay
22.64	5.70	0.1100	1.930	29.705	4	4	silty clay to clay
22.80	5.51	0.1006	1.824	30.212	4	4	silty clay to clay
22.97	5.47	0.0941	1.720	31.047	3	4	silty clay to clay
23.13	5.14	0.0932	1.812	31.217	3	4	silty clay to clay
23.29	5.28	0.0971	1.840	31.889	3	4	silty clay to clay
23.46	5.48	0.0955	1.741	32.119	3	4	silty clay to clay
23.62	5.39	0.0991	1.840	31.882	3	4	silty clay to clay
23.79	5.46	0.1054	1.929	31.872	4	4	silty clay to clay
23.95	5.85	0.0921	1.574	32.169	4	4	silty clay to clay
24.11	6.29	0.1019	1.622	32.205	4	4	silty clay to clay
24.28	7.10	0.1947	2.745	33.233	4	5	clayey silt to silty clay
24.44	10.26	0.1344	1.310	32.618	4	5	clayey silt to silty clay
24.61	9.28	0.1136	1.225	20.340	4	5	clayey silt to silty clay
24.77	6.20	0.1218	1.965	20.462	5	4	silty clay to clay
24.93	5.94	0.1526	2.570	23.612	5	4	silty clay to clay
25.10	9.93	0.1567	1.578	25.489	4	5	clayey silt to silty clay
25.26	11.96	0.2239	1.872	15.536	5	5	clayey silt to silty clay
25.43	9.54	0.1745	1.829	16.903	5	5	clayey silt to silty clay
25.59	9.14	0.1442	1.577	14.323	4	5	clayey silt to silty clay
25.75	5.54	0.1152	2.078	16.853	4	4	silty clay to clay
25.92	5.30	0.0800	1.508	18.733	3	4	silty clay to clay
26.08	4.98	0.1213	2.433	20.125	3	4	silty clay to clay
26.25	4.93	0.1437	2.917	22.270	5	3	clay
26.41	5.53	0.1621	2.933	31.963	5	3	clay
26.57	6.04	0.1417	2.346	32.956	4	4	silty clay to clay
26.74	6.47	0.1405	2.170	32.757	4	4	silty clay to clay
26.90	6.16	0.1522	2.472	32.592	4	4	silty clay to clay
27.07	6.57	0.1451	2.208	32.977	4	4	silty clay to clay
27.23	6.98	0.0979	1.403	31.518	4	4	silty clay to clay
27.40	6.06	0.1049	1.732	31.028	4	4	silty clay to clay
27.56	5.64	0.1137	2.017	32.355	4	4	silty clay to clay
27.72	6.31	0.1130	1.789	34.496	4	4	silty clay to clay
27.89	6.41	0.1080	1.686	34.972	4	4	silty clay to clay
28.05	6.46	0.1000	1.547	35.663	4	4	silty clay to clay
28.22	7.13	0.1348	1.891	35.959	5	4	silty clay to clay
28.38	8.45	0.2407	2.847	36.595	4	5	clayey silt to silty clay
28.54	9.97	0.1116	1.119	24.104	5	5	clayey silt to silty clay
28.71	10.18	0.1913	1.878	18.802	5	5	clayey silt to silty clay
28.87	8.27	0.3161	3.822	22.748	7	4	silty clay to clay
29.04	14.48	0.3410	2.356	20.589	8	4	silty clay to clay
29.20	14.52	0.3687	2.540	15.576	7	5	clayey silt to silty clay
29.36	14.82	0.3860	2.605	15.292	7	5	clayey silt to silty clay
29.53	15.53	0.3108	2.001	13.991	7	5	clayey silt to silty clay
29.69	13.28	0.2754	2.074	12.333	6	5	clayey silt to silty clay
29.86	11.53	0.3230	2.802	13.118	6	5	clayey silt to silty clay
30.02	13.37	0.4129	3.088	14.627	6	5	clayey silt to silty clay
30.18	14.32	0.2728	1.905	13.641	7	5	clayey silt to silty clay
30.35	13.92	0.2123	1.525	12.924	6	5	clayey silt to silty clay

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

Depth (ft)	Qt TSF	Fs TSF	Fs/Qt (%)	Pw PSI	SPT N* 60% Hammer	Zone	Soil Behavior Type UBC-1983
30.51	9.44	0.1643	1.740	13.603	5	5	clayey silt to silty clay
30.68	7.17	0.1033	1.440	14.878	4	5	clayey silt to silty clay
30.84	6.33	0.0836	1.321	16.499	3	1	sensitive fine grained
31.00	6.38	0.1062	1.664	17.760	3	1	sensitive fine grained
31.17	7.22	0.0878	1.216	18.927	3	1	sensitive fine grained
31.33	7.31	0.0941	1.288	19.769	3	5	clayey silt to silty clay
31.50	6.72	0.1163	1.731	21.536	5	4	silty clay to clay
31.66	7.39	0.1975	2.672	22.906	5	4	silty clay to clay
31.82	8.41	0.2176	2.587	23.935	6	4	silty clay to clay
31.99	10.65	0.1633	1.534	24.147	6	4	silty clay to clay
32.15	9.32	0.2402	2.578	23.870	5	5	clayey silt to silty clay
32.32	9.78	0.1554	1.590	26.207	5	5	clayey silt to silty clay
32.48	11.75	0.2455	2.089	23.679	7	4	silty clay to clay
32.64	10.15	0.3892	3.836	24.193	7	4	silty clay to clay
32.81	10.96	0.3048	2.781	25.071	7	4	silty clay to clay
32.97	10.10	0.2221	2.200	12.630	6	4	silty clay to clay
33.14	8.57	0.1651	1.926	14.261	4	5	clayey silt to silty clay
33.30	7.50	0.1387	1.849	15.739	5	4	silty clay to clay
33.46	6.78	0.1358	2.003	17.076	5	4	silty clay to clay
33.63	6.92	0.1325	1.913	18.501	5	4	silty clay to clay
33.79	7.63	0.1034	1.355	19.577	5	4	silty clay to clay
33.96	7.38	0.1592	2.158	20.326	4	5	clayey silt to silty clay
34.12	7.97	0.1272	1.596	21.672	4	5	clayey silt to silty clay
34.28	9.38	0.1291	1.376	21.916	4	5	clayey silt to silty clay
34.45	6.89	0.1442	2.093	23.057	4	5	clayey silt to silty clay
34.61	6.95	0.1307	1.880	24.269	4	4	silty clay to clay
34.78	7.02	0.0902	1.286	25.358	4	4	silty clay to clay
34.94	4.83	0.0966	2.000	28.969	3	1	sensitive fine grained
35.10	7.09	0.1056	1.488	33.472	4	4	silty clay to clay
35.27	7.57	0.1794	2.371	35.636	4	5	clayey silt to silty clay
35.43	11.00	0.2109	1.917	37.523	5	5	clayey silt to silty clay
35.60	12.44	0.2977	2.394	26.529	6	5	clayey silt to silty clay
35.76	14.87	0.3037	2.042	28.330	7	5	clayey silt to silty clay
35.93	14.00	0.2574	1.839	19.685	6	5	clayey silt to silty clay
36.09	10.54	0.2440	2.314	21.464	6	5	clayey silt to silty clay
36.25	10.35	0.1815	1.753	24.812	5	5	clayey silt to silty clay
36.42	9.06	0.2087	2.304	25.814	5	5	clayey silt to silty clay
36.58	12.58	0.1918	1.524	28.507	5	5	clayey silt to silty clay
36.75	12.44	0.2764	2.221	23.387	6	5	clayey silt to silty clay
36.91	10.06	0.3120	3.101	25.625	6	5	clayey silt to silty clay
37.07	13.58	0.2321	1.709	25.324	6	5	clayey silt to silty clay
37.24	12.65	0.2200	1.739	23.059	6	5	clayey silt to silty clay
37.40	9.98	0.2855	2.862	24.234	5	5	clayey silt to silty clay
37.57	10.97	0.2354	2.147	26.752	6	5	clayey silt to silty clay
37.73	13.72	0.1754	1.278	25.602	6	5	clayey silt to silty clay
37.89	10.28	0.2192	2.133	25.147	5	5	clayey silt to silty clay
38.06	9.45	0.1838	1.946	28.230	5	5	clayey silt to silty clay
38.22	10.69	0.2165	2.026	28.969	5	5	clayey silt to silty clay
38.39	9.07	0.1904	2.099	30.024	5	5	clayey silt to silty clay
38.55	8.56	0.1442	1.685	36.882	4	5	clayey silt to silty clay
38.71	7.73	0.1153	1.491	39.331	4	5	clayey silt to silty clay
38.88	7.44	0.1089	1.464	41.345	4	5	clayey silt to silty clay
39.04	7.45	0.1177	1.580	43.062	5	4	silty clay to clay
39.21	7.78	0.1895	2.436	44.485	5	4	silty clay to clay
39.37	8.72	0.1611	1.848	45.349	4	5	clayey silt to silty clay
39.53	8.45	0.0956	1.131	35.864	4	5	clayey silt to silty clay
39.70	7.62	0.1383	1.816	38.540	4	5	clayey silt to silty clay
39.86	8.57	0.2026	2.364	42.051	5	5	clayey silt to silty clay
40.03	13.80	0.1349	0.977	40.685	5	5	clayey silt to silty clay
40.19	10.07	0.1437	1.427	30.114	5	5	clayey silt to silty clay
40.35	8.77	0.1509	1.720	34.711	5	5	clayey silt to silty clay
40.52	10.20	0.1875	1.838	37.995	5	5	clayey silt to silty clay

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

Depth (ft)	Qt TSF	Fs TSF	Fs/Qt (%)	Pw PSI	SPT N* 60% Hammer	Zone	Soil Behavior Type UBC-1983
40.68	11.26	0.1900	1.687	39.451	5	5	clayey silt to silty clay
40.85	11.53	0.1414	1.227	36.392	5	5	clayey silt to silty clay
41.01	9.54	0.1528	1.601	35.077	5	5	clayey silt to silty clay
41.17	8.49	0.2086	2.456	39.346	5	5	clayey silt to silty clay
41.34	11.21	0.1474	1.315	40.881	5	5	clayey silt to silty clay
41.50	13.54	0.2682	1.981	34.713	6	5	clayey silt to silty clay
41.67	9.82	0.3461	3.526	33.011	6	5	clayey silt to silty clay
41.83	13.90	0.2941	2.115	34.589	6	5	clayey silt to silty clay
41.99	15.58	0.3037	1.949	30.507	7	5	clayey silt to silty clay
42.16	14.75	0.2358	1.599	27.498	6	5	clayey silt to silty clay
42.32	9.66	0.2058	2.131	27.541	5	5	clayey silt to silty clay
42.49	9.31	0.1142	1.226	32.176	5	5	clayey silt to silty clay
42.65	9.94	0.1912	1.923	34.582	5	5	clayey silt to silty clay
42.81	9.80	0.2023	2.064	37.720	6	5	clayey silt to silty clay
42.98	15.37	0.1430	0.930	35.218	6	5	clayey silt to silty clay
43.14	10.96	0.2500	2.282	30.920	6	5	clayey silt to silty clay
43.31	11.19	0.1656	1.480	35.620	6	5	clayey silt to silty clay
43.47	13.12	0.1420	1.082	32.374	5	5	clayey silt to silty clay
43.64	8.90	0.1619	1.818	33.704	5	5	clayey silt to silty clay
43.80	9.05	0.1238	1.368	37.877	4	5	clayey silt to silty clay
43.96	8.98	0.1192	1.327	39.760	4	5	clayey silt to silty clay
44.13	8.91	0.1316	1.477	42.228	4	5	clayey silt to silty clay
44.29	8.83	0.1736	1.968	44.555	4	5	clayey silt to silty clay
44.46	9.55	0.2163	2.266	46.355	5	5	clayey silt to silty clay
44.62	12.08	0.2425	2.008	45.475	5	5	clayey silt to silty clay
44.78	12.55	0.2323	1.852	37.578	6	5	clayey silt to silty clay
44.95	11.89	0.2224	1.871	29.222	6	5	clayey silt to silty clay
45.11	10.50	0.1451	1.382	33.915	5	5	clayey silt to silty clay
45.28	9.46	0.1352	1.430	35.605	5	5	clayey silt to silty clay
45.44	8.38	0.1268	1.513	38.860	4	5	clayey silt to silty clay
45.60	8.94	0.1238	1.385	41.281	4	5	clayey silt to silty clay
45.77	8.73	0.2280	2.612	42.823	4	5	clayey silt to silty clay
45.93	8.99	0.1711	1.903	45.246	5	5	clayey silt to silty clay
46.10	11.11	0.2112	1.900	30.425	5	5	clayey silt to silty clay
46.26	10.06	0.1974	1.963	36.340	5	5	clayey silt to silty clay
46.42	9.86	0.1508	1.530	39.190	5	5	clayey silt to silty clay
46.59	9.60	0.1113	1.160	41.615	5	5	clayey silt to silty clay
46.75	9.56	0.1627	1.702	45.124	5	5	clayey silt to silty clay
46.92	10.37	0.2413	2.328	48.068	5	5	clayey silt to silty clay
47.08	12.43	0.2140	1.722	44.665	6	5	clayey silt to silty clay
47.24	11.69	0.1517	1.298	40.465	6	5	clayey silt to silty clay
47.41	11.01	0.2735	2.485	42.596	6	5	clayey silt to silty clay
47.57	12.37	0.3467	2.803	47.922	7	5	clayey silt to silty clay
47.74	19.29	0.3472	1.800	31.380	8	5	clayey silt to silty clay
47.90	20.31	0.3703	1.824	21.122	9	5	clayey silt to silty clay
48.06	17.62	0.3995	2.267	18.566	9	5	clayey silt to silty clay
48.23	17.56	0.4860	2.768	20.343	10	5	clayey silt to silty clay
48.39	24.40	0.5199	2.130	19.025	8	6	sandy silt to clayey silt
48.56	24.17	0.3050	1.262	12.096	9	6	sandy silt to clayey silt
48.72	21.14	0.3689	1.745	10.332	8	6	sandy silt to clayey silt
48.88	15.08	0.3841	2.547	10.956	8	5	clayey silt to silty clay
49.05	16.11	0.4502	2.794	12.718	8	5	clayey silt to silty clay
49.21	17.12	0.4939	2.884	13.565	9	5	clayey silt to silty clay
49.38	20.29	0.4333	2.136	16.444	9	5	clayey silt to silty clay
49.54	17.03	0.4350	2.554	13.644	8	5	clayey silt to silty clay
49.70	14.42	0.5080	3.522	14.345	8	5	clayey silt to silty clay
49.87	16.05	0.5222	3.253	16.208	8	5	clayey silt to silty clay
50.03	22.19	0.5528	2.491	15.796	10	5	clayey silt to silty clay
50.20	21.77	0.6096	2.800	13.010	10	5	clayey silt to silty clay
50.36	21.09	0.6269	2.972	11.546	11	5	clayey silt to silty clay
50.52	24.54	0.6136	2.500	8.296	12	5	clayey silt to silty clay
50.69	27.46	0.6914	2.518	6.053	11	6	sandy silt to clayey silt

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

Depth (ft)	Qt TSF	Fs TSF	Fs/Qt (%)	Pw PSI	SPT N* 60% Hammer	Zone	Soil Behavior Type UBC-1983
50.85	31.02	0.8151	2.627	3.774	11	6	sandy silt to clayey silt
51.02	28.65	0.6938	2.422	1.724	12	6	sandy silt to clayey silt
51.18	31.13	0.9111	2.927	0.210	13	6	sandy silt to clayey silt
51.35	38.51	0.7534	1.956	-0.564	14	6	sandy silt to clayey silt
51.51	36.64	1.0957	2.990	-1.468	14	6	sandy silt to clayey silt
51.67	33.09	1.2724	3.845	0.213	16	5	clayey silt to silty clay
51.84	29.99	1.3305	4.437	-1.363	21	4	silty clay to clay
52.00	33.29	1.2808	3.848	-1.700	21	4	silty clay to clay
52.17	36.97	1.3283	3.593	-1.975	17	5	clayey silt to silty clay
52.33	39.06	0.7278	1.864	-2.272	15	6	sandy silt to clayey silt
52.49	37.91	1.0457	2.758	-2.475	14	6	sandy silt to clayey silt
52.66	35.15	1.3482	3.836	17.540	19	5	clayey silt to silty clay
52.82	46.16	1.7593	3.811	0.344	20	5	clayey silt to silty clay
52.99	46.11	1.8442	4.000	-0.375	22	5	clayey silt to silty clay
53.15	44.11	1.4005	3.175	-0.866	20	5	clayey silt to silty clay
53.31	34.64	0.9517	2.747	-1.294	17	5	clayey silt to silty clay
53.48	28.19	0.9810	3.480	-1.114	15	5	clayey silt to silty clay
53.64	30.46	0.8397	2.756	-0.964	14	5	clayey silt to silty clay
53.81	30.83	0.7207	2.338	-0.971	12	6	sandy silt to clayey silt
53.97	31.09	0.6249	2.010	-1.122	12	6	sandy silt to clayey silt
54.13	28.30	0.6207	2.193	-1.236	11	6	sandy silt to clayey silt
54.30	24.62	0.7491	3.043	-1.232	12	5	clayey silt to silty clay
54.46	24.99	0.6523	2.611	-1.052	10	6	sandy silt to clayey silt
54.63	31.07	0.4797	1.544	-0.990	11	6	sandy silt to clayey silt
54.79	30.88	0.7395	2.395	-1.198	12	6	sandy silt to clayey silt
54.95	31.49	1.0473	3.326	2.181	15	5	clayey silt to silty clay
55.12	29.82	1.2471	4.183	-0.524	20	4	silty clay to clay
55.28	32.34	1.5508	4.795	-1.000	22	4	silty clay to clay
55.45	42.61	1.7731	4.161	-1.258	23	4	silty clay to clay
55.61	33.89	1.3656	4.029	-2.035	16	5	clayey silt to silty clay
55.77	23.18	0.7450	3.215	-2.521	12	5	clayey silt to silty clay
55.94	16.37	0.4549	2.778	-2.475	9	5	clayey silt to silty clay
56.10	15.23	0.3562	2.339	-2.227	8	5	clayey silt to silty clay
56.27	15.68	0.4635	2.956	-1.952	8	5	clayey silt to silty clay
56.43	18.42	0.4693	2.548	-1.660	9	5	clayey silt to silty clay
56.59	23.26	0.5850	2.515	-1.488	10	5	clayey silt to silty clay
56.76	21.31	0.6658	3.124	-1.351	11	5	clayey silt to silty clay
56.92	24.80	0.6978	2.813	-1.203	12	5	clayey silt to silty clay
57.09	28.86	0.8174	2.832	-1.131	13	5	clayey silt to silty clay
57.25	28.96	0.9329	3.221	-1.126	14	5	clayey silt to silty clay
57.41	31.71	1.4757	4.653	-1.184	22	4	silty clay to clay
57.58	40.41	1.6988	4.204	-1.198	23	4	silty clay to clay
57.74	37.59	1.8570	4.940	-1.440	23	4	silty clay to clay
57.91	32.26	1.2601	3.906	-1.782	23	4	silty clay to clay
58.07	36.20	1.5157	4.187	-1.901	17	5	clayey silt to silty clay
58.23	37.93	1.3458	3.548	-0.959	22	4	silty clay to clay
58.40	28.81	1.2984	4.507	-1.507	20	4	silty clay to clay
58.56	25.31	1.4661	5.793	-1.930	26	3	clay
58.73	28.70	1.3465	4.692	-2.117	27	3	clay
58.89	29.19	1.0415	3.568	-2.523	19	4	silty clay to clay
59.06	30.39	0.9821	3.232	-2.836	18	4	silty clay to clay
59.22	24.43	1.4114	5.778	-2.014	29	3	clay
59.38	35.89	1.8770	5.229	-2.578	35	3	clay
59.55	48.93	2.2434	4.585	-2.695	30	4	silty clay to clay
59.71	56.08	2.6607	4.744	-3.006	34	4	silty clay to clay
59.88	56.27	2.9522	5.246	-2.982	51	3	clay
60.04	47.66	2.8313	5.941	-2.889	47	3	clay
60.20	43.14	2.5128	5.824	-2.906	47	3	clay
60.37	54.92	2.6416	4.810	-2.882	51	3	clay
60.53	61.51	3.0211	4.912	-2.903	37	4	silty clay to clay
60.70	58.74	3.4065	5.799	-2.848	55	3	clay
60.86	51.96	3.4665	6.671	-2.602	54	3	clay

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

Depth (ft)	Qt TSF	Fs TSF	Fs/Qt (%)	Pw PSI	SPT N* 60% Hammer	Zone	Soil Behavior Type UBC-1983
61.02	59.85	2.6686	4.459	-2.451	57	11	very stiff fine grained (*)
61.19	65.31	3.0912	4.733	-2.370	43	4	silty clay to clay
61.35	75.11	3.3062	4.402	-1.598	42	4	silty clay to clay
61.52	58.41	3.0501	5.222	-1.256	40	4	silty clay to clay
61.68	52.40	2.6575	5.072	-1.284	36	4	silty clay to clay
61.84	58.70	2.6967	4.594	-1.289	37	4	silty clay to clay
62.01	61.98	2.4058	3.881	-1.284	27	5	clayey silt to silty clay
62.17	46.28	1.3544	2.927	-1.428	21	5	clayey silt to silty clay
62.34	20.66	0.8325	4.030	-1.354	13	5	clayey silt to silty clay
62.50	11.72	0.3300	2.815	7.562	7	5	clayey silt to silty clay
62.66	11.68	0.1532	1.312	8.877	5	5	clayey silt to silty clay
62.83	9.95	0.1492	1.499	9.958	5	5	clayey silt to silty clay
62.99	10.26	0.1619	1.579	11.238	5	5	clayey silt to silty clay
63.16	10.64	0.1643	1.544	12.348	5	5	clayey silt to silty clay
63.32	11.35	0.2969	2.616	13.374	6	5	clayey silt to silty clay
63.48	14.82	0.2576	1.738	14.651	7	5	clayey silt to silty clay
63.65	16.45	0.2821	1.715	15.052	7	5	clayey silt to silty clay
63.81	14.19	0.2605	1.835	15.782	7	5	clayey silt to silty clay
63.98	13.49	0.2891	2.143	16.691	7	5	clayey silt to silty clay
64.14	14.16	0.2468	1.742	17.898	7	5	clayey silt to silty clay
64.30	14.96	0.2998	2.004	18.738	7	5	clayey silt to silty clay
64.47	12.76	0.3957	3.101	20.359	7	5	clayey silt to silty clay
64.63	18.58	0.3002	1.616	36.545	8	5	clayey silt to silty clay
64.80	18.61	0.3164	1.701	33.740	9	5	clayey silt to silty clay
64.96	16.19	0.4506	2.783	35.739	8	5	clayey silt to silty clay
65.12	16.65	0.6252	3.755	39.140	12	4	silty clay to clay
65.29	21.52	0.9406	4.370	40.503	21	3	clay
65.45	28.59	1.9268	6.741	34.199	35	3	clay
65.62	58.93	3.1119	5.280	10.595	51	3	clay
65.78	73.71	3.7188	5.046	18.647	63	11	very stiff fine grained (*)
65.94	65.92	3.7539	5.695	0.677	66	11	very stiff fine grained (*)
66.11	66.44	3.6671	5.520	-0.576	59	11	very stiff fine grained (*)
66.27	52.81	2.4833	4.703	-2.040	50	3	clay
66.44	37.70	1.8554	4.922	-2.781	23	5	clayey silt to silty clay
66.60	56.65	1.5176	2.679	0.392	20	5	clayey silt to silty clay
66.77	32.15	1.8069	5.620	4.326	20	5	clayey silt to silty clay
66.93	35.23	1.7211	4.885	12.261	30	4	silty clay to clay
67.09	71.85	2.5494	3.548	3.702	34	5	clayey silt to silty clay
67.26	107.49	3.9634	3.687	5.675	35	6	sandy silt to clayey silt
67.42	97.75	3.3999	3.478	-1.009	44	5	clayey silt to silty clay
67.59	72.75	3.6318	4.992	-2.528	78	11	very stiff fine grained (*)
67.75	72.91	3.9274	5.387	-2.418	75	11	very stiff fine grained (*)
67.91	87.86	3.4023	3.872	-0.679	39	6	sandy silt to clayey silt
68.08	146.64	3.1621	2.156	-2.081	41	7	silty sand to sandy silt
68.24	151.12	2.5828	1.709	26.776	40	8	sand to silty sand
68.41	202.53	3.3778	1.668	7.270	44	8	sand to silty sand
68.57	199.47	4.5916	2.302	16.892	55	7	silty sand to sandy silt
68.73	111.05	6.9298	6.240	6.692	158	11	very stiff fine grained (*)
68.90	183.59	9.1585	4.989	29.215	174	11	very stiff fine grained (*)
69.06	249.38	11.2439	4.509	30.614	233	11	very stiff fine grained (*)
69.23	297.89	10.6468	3.574	29.624	133	12	sand to clayey sand (*)
69.39	284.47	9.4111	3.308	49.297	135	12	sand to clayey sand (*)
69.55	263.14	7.6229	2.897	77.776	121	12	sand to clayey sand (*)
69.72	210.86	8.4479	4.006	193.606	131	12	sand to clayey sand (*)
69.88	349.28	9.6916	2.775	233.741	146	12	sand to clayey sand (*)
70.05	351.99	11.1732	3.174	310.308	161	12	sand to clayey sand (*)
70.21	304.96	8.8013	2.886	439.768	151	12	sand to clayey sand (*)
70.37	287.23	8.6326	3.005	261.827	135	12	sand to clayey sand (*)
70.54	250.85	7.5530	3.011	82.552	120	12	sand to clayey sand (*)
70.70	214.74	7.6961	3.584	68.207	113	12	sand to clayey sand (*)
70.87	243.67	7.0945	2.912	54.515	73	7	silty sand to sandy silt
71.03	223.22	5.7765	2.588	31.726	64	7	silty sand to sandy silt

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

```
  Depth        Qt         Fs       Fs/Qt         Pw     SPT N*             Soil Behavior Type
  (ft)        TSF        TSF        (%)         PSI 60% Hammer    Zone                UBC-1983

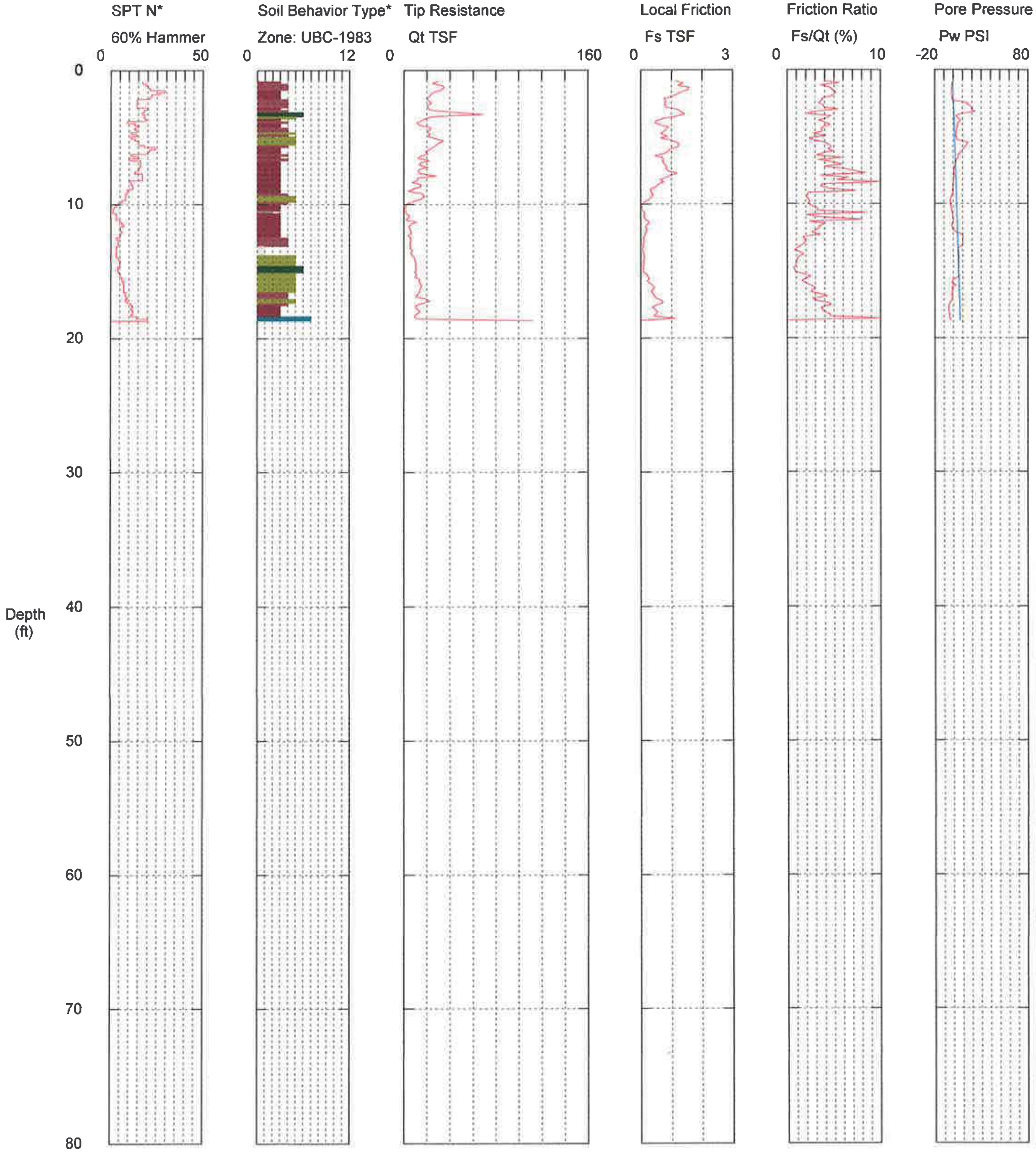
  71.19     248.68     8.8475      3.558      31.568        182       11 very stiff fine grained (*)
  71.36     131.79-32768.0000 -24864.220      33.011          0        0       <out of range>

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

HDR ENG. / BC-2CPT-4 / BIG CK. DAM NEWPORT

Operator: SAV/CM
Sounding: VEI434BC2SCPT4(487)
Cone Used: DSG0736

CPT Date/Time: 10/24/2013 3:55:36 AM
Location: BC2SCPT4 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT



Maximum Depth = 18.70 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

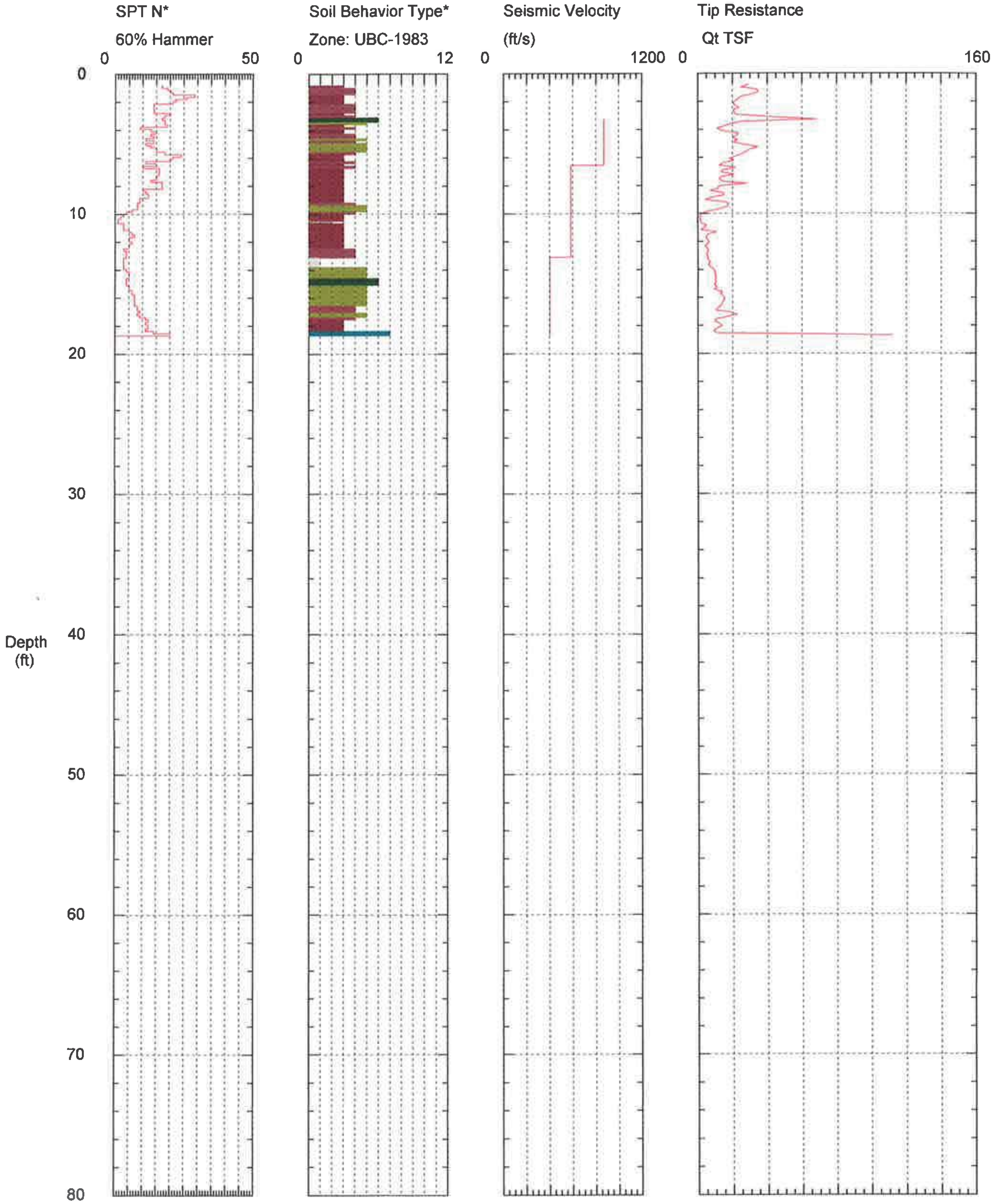
🟥 10 gravelly sand to sand
🟦 11 very stiff fine grained (*)
🟦 12 sand to clayey sand (*)

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

HDR ENG. / BC-2CPT-4 / BIG CK. DAM NEWPORT

Operator: SAV/CM
Sounding: VEI434BC2SCPT4(487)
Cone Used: DSG0736

CPT Date/Time: 10/24/2013 3:55:36 AM
Location: BC2SCPT4 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT



Maximum Depth = 18.70 feet

Depth Increment = 0.164 feet

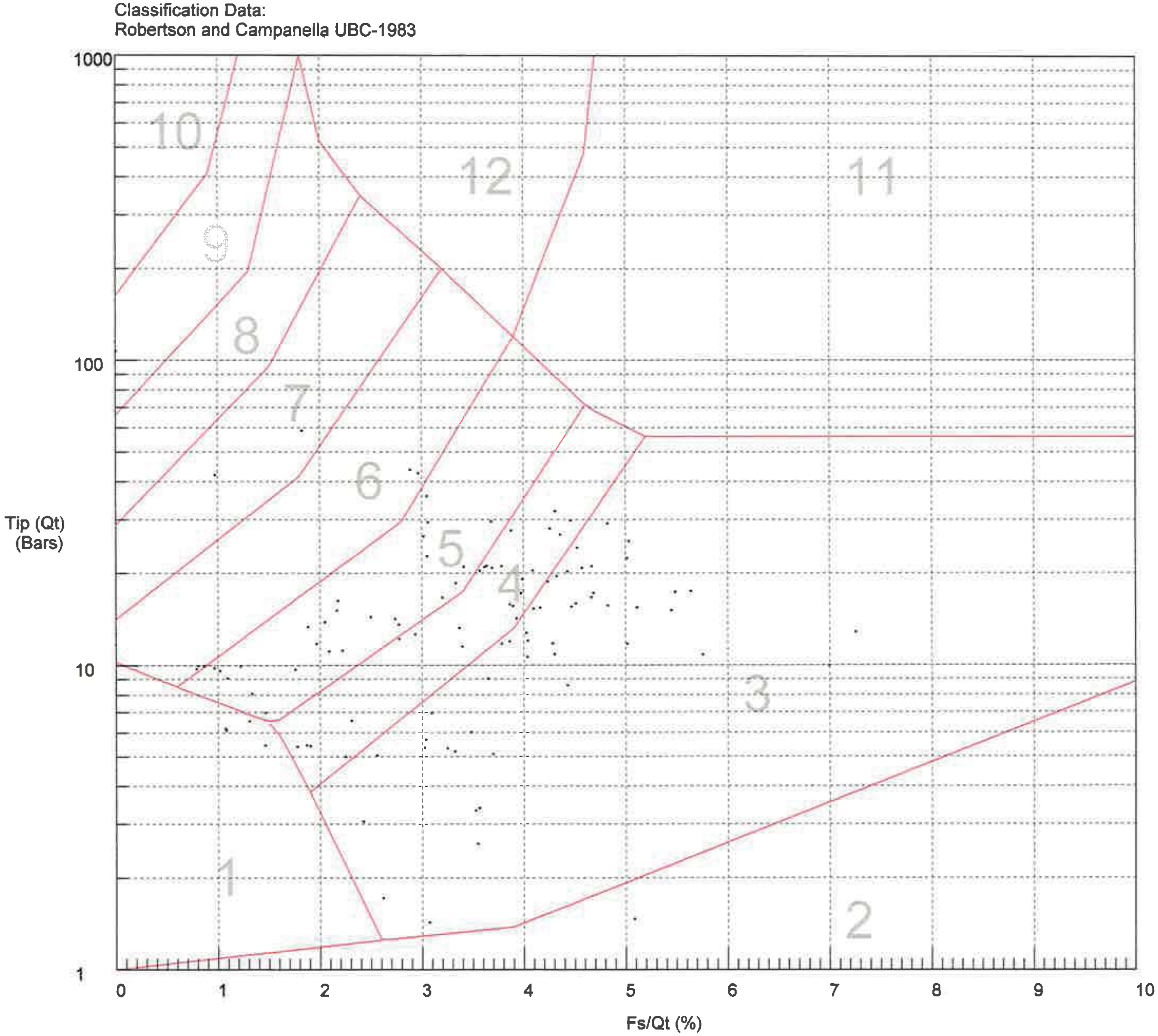
- ⬜ 1 sensitive fine grained
- ⬛ 2 organic material
- 🟥 3 clay
- 🟫 4 silty clay to clay
- 🟩 5 clayey silt to silty clay
- 🟩 6 sandy silt to clayey silt
- 🟦 7 silty sand to sandy silt
- 🟦 8 sand to silty sand
- 🟧 9 sand
- 🟥 10 gravelly sand to sand
- 🟦 11 very stiff fine grained (*)
- 🟦 12 sand to clayey sand (*)

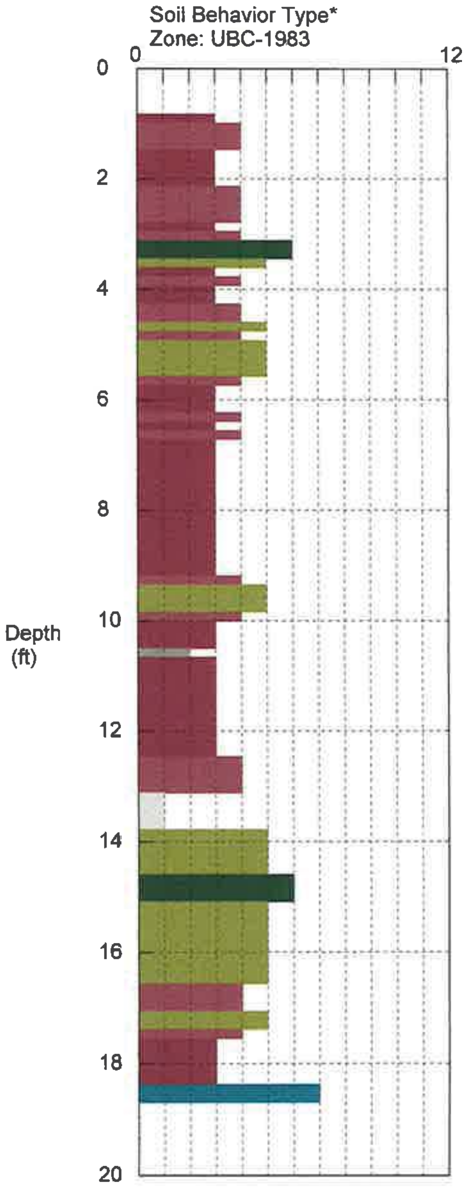
*Soil behavior type and SPT based on data from UBC-1983

HDR ENG. / BC-2CPT-4 / BIG CK. DAM NEWPORT

Operator: SAV/CM
Sounding: VEI434BC2SCPT4(487)
Cone Used: DSG0736

CPT Date/Time: 10/24/2013 3:55:36 AM
Location: BC2SCPT4 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT



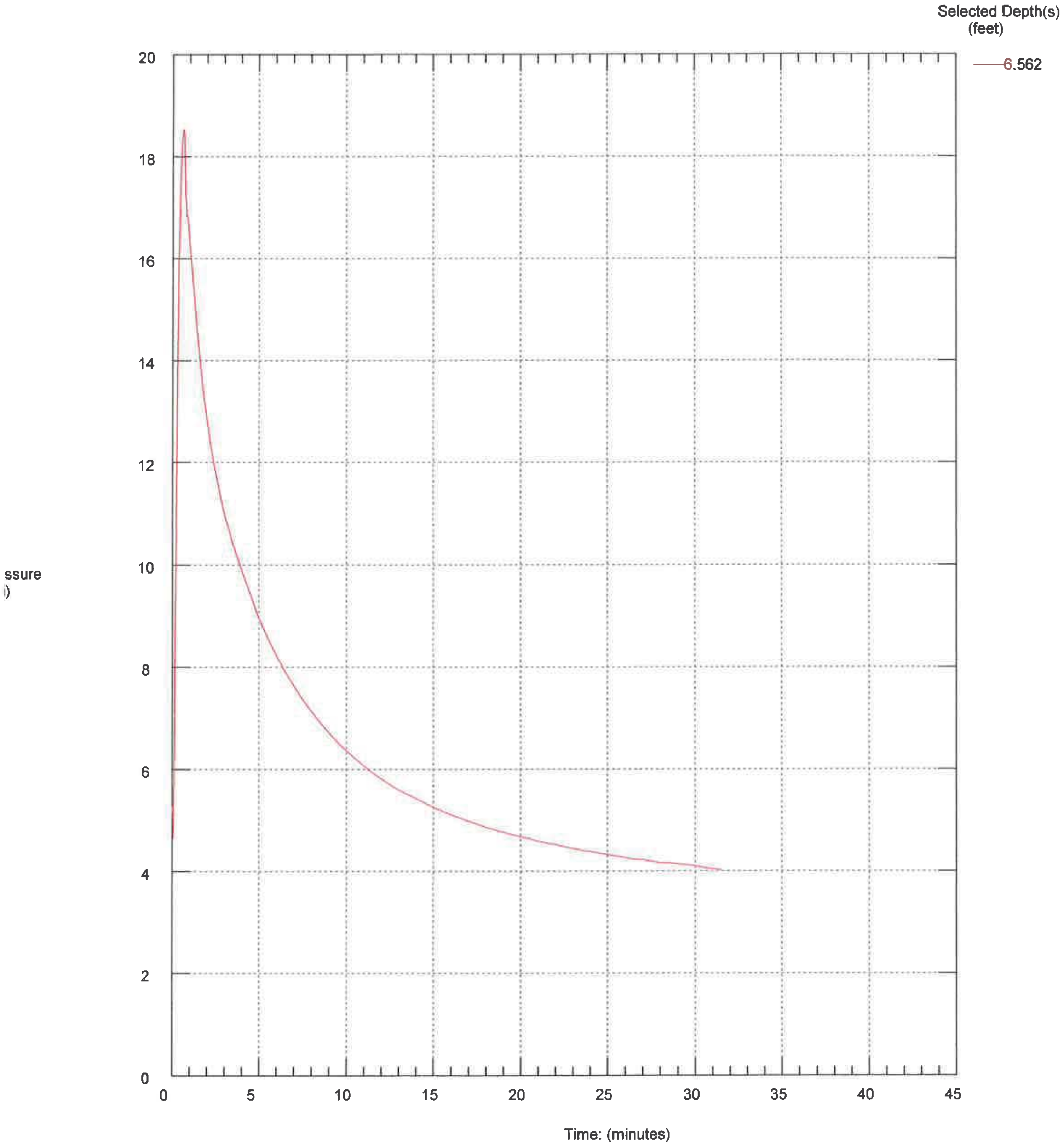


- 1 sensitive fine grained
- 2 organic material
- 3 clay
- 4 silty clay to clay
- 5 clayey silt to silty clay
- 6 sandy silt to clayey silt
- 7 silty sand to sandy silt
- 8 sand to silty sand
- 9 sand
- 10 gravelly sand to sand
- 11 very stiff fine grained (*)
- 12 sand to clayey sand (*)

HDR ENG. / BC-2,SCPT-4 / BIG CK. DAM NEWPORT

Operator SAV/CM
Sounding: VEI434BC2SCPT4(487)
Cone Used: DSG0736

CPT Date/Time: 10/24/2013 3:55:36 AM
Location: BC2SCPT4 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT

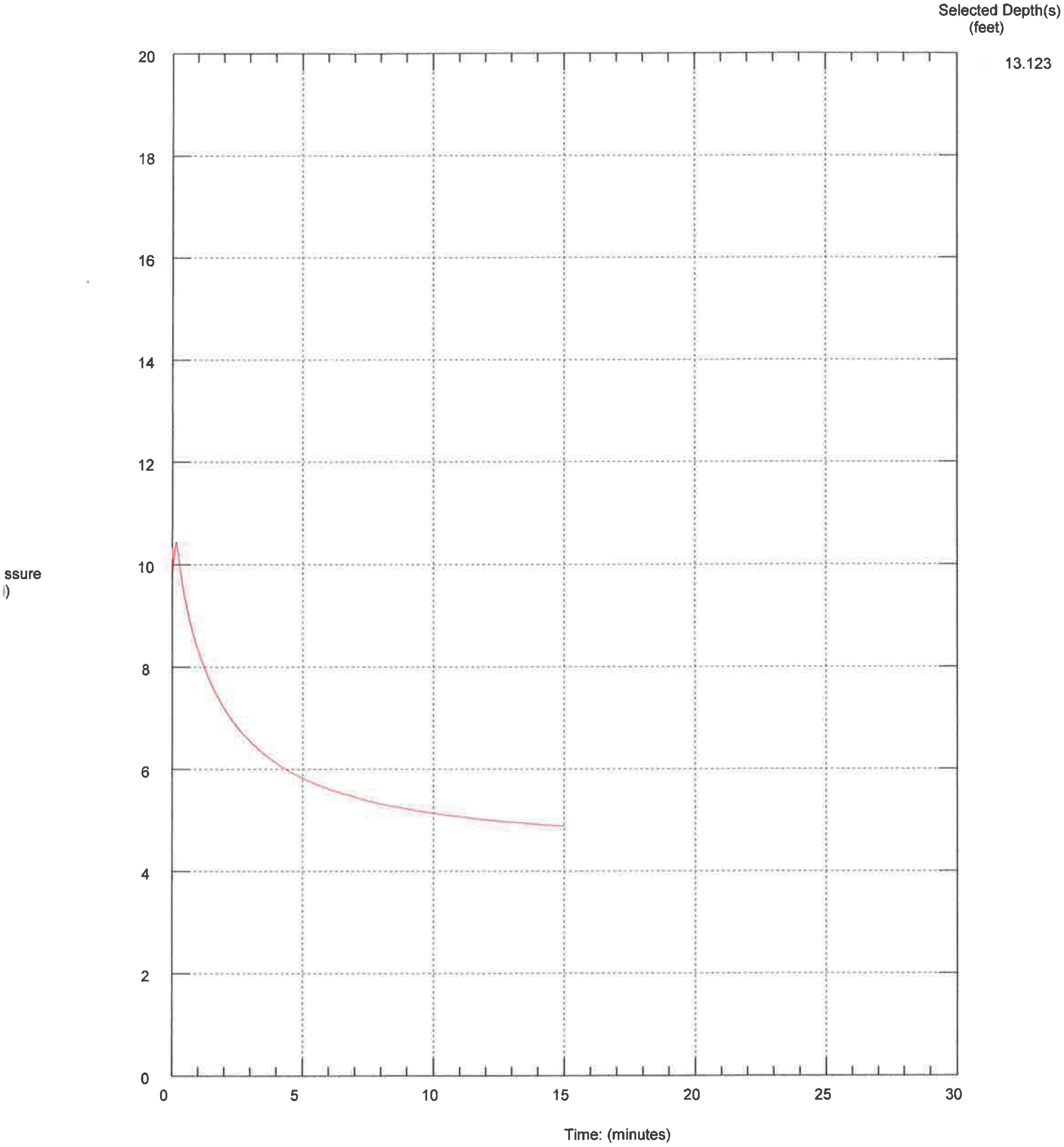


Maximum Pressure = 18.5
Hydrostatic Pressure =

HDR ENG. / BC-2,SCPT-4 / BIG CK. DAM NEWPORT

Operator SAV/CM
Sounding: VEI434BC2SCPT4(487)
Cone Used: DSG0736

CPT Date/Time: 10/24/2013 3:55:36 AM
Location: BC2SCPT4 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT

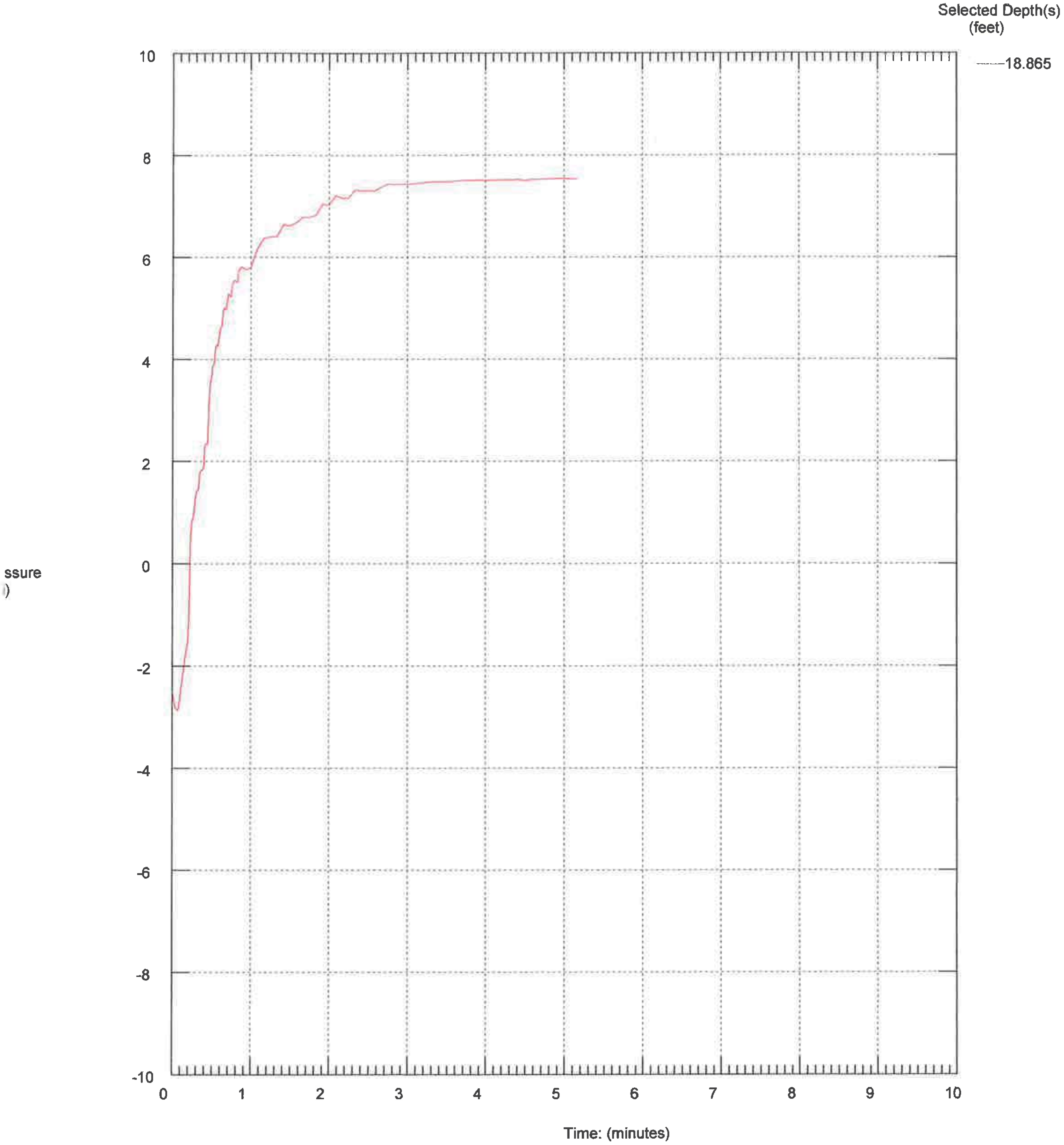


Maximum Pressure = 10.437 psi
Hydrostatic Pressure

HDR ENG. / BC-2,SCPT-4 / BIG CK. DAM NEWPORT

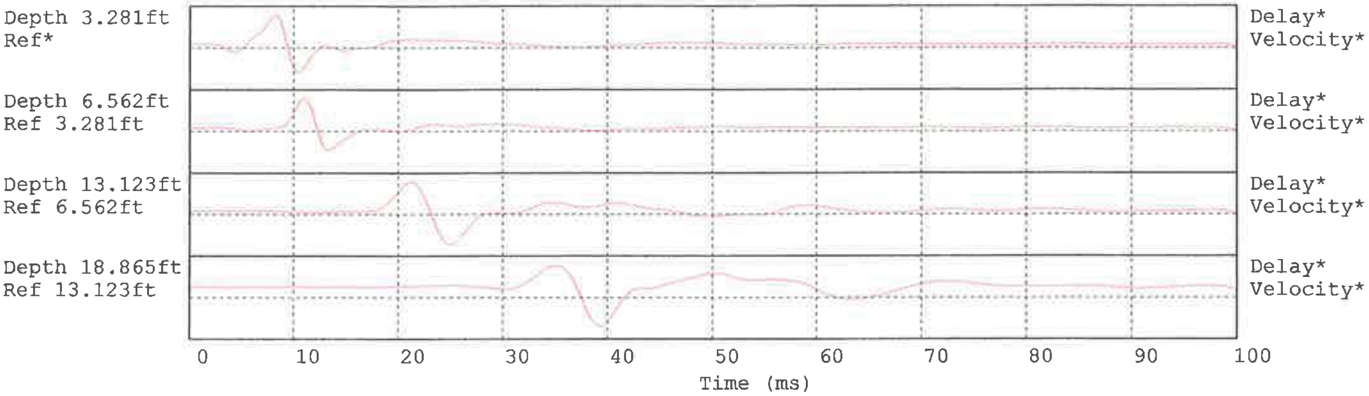
Operator SAV/CM
Sounding: VEI434BC2SCPT4(487)
Cone Used: DSG0736

CPT Date/Time: 10/24/2013 3:55:36 AM
Location: BC2SCPT4 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT



Maximum Pressure = 7.543 psi
Hydrostatic Pressure =

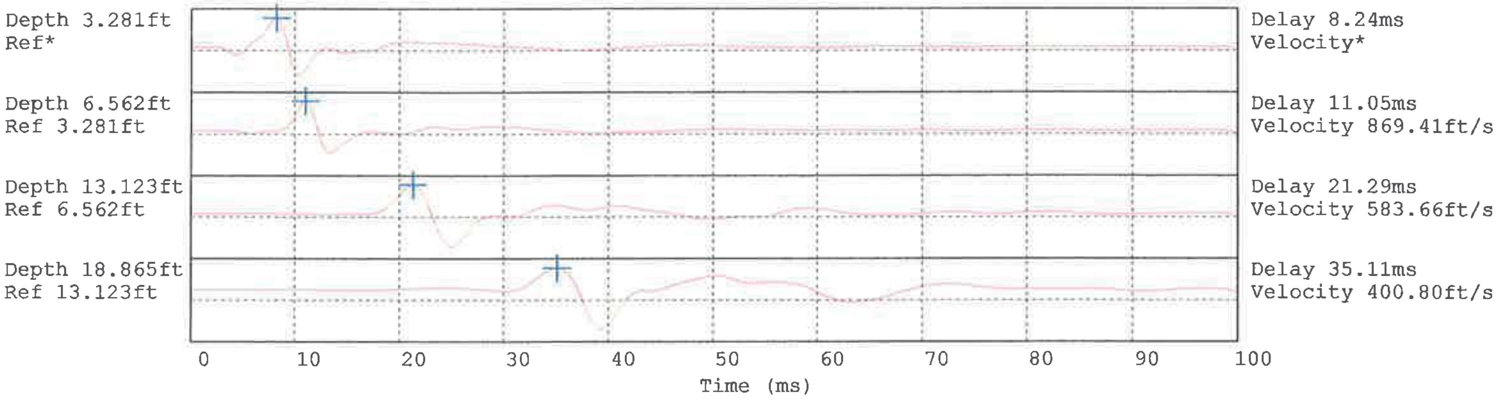
HDR ENG. / BC2,SCPT4 / BIG CK. NEWPORT



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

THE SHEAR WAVE PICKS ARE COMPUTER GENERATED VALUES (STAFF REVIEW RECOMMENDED)

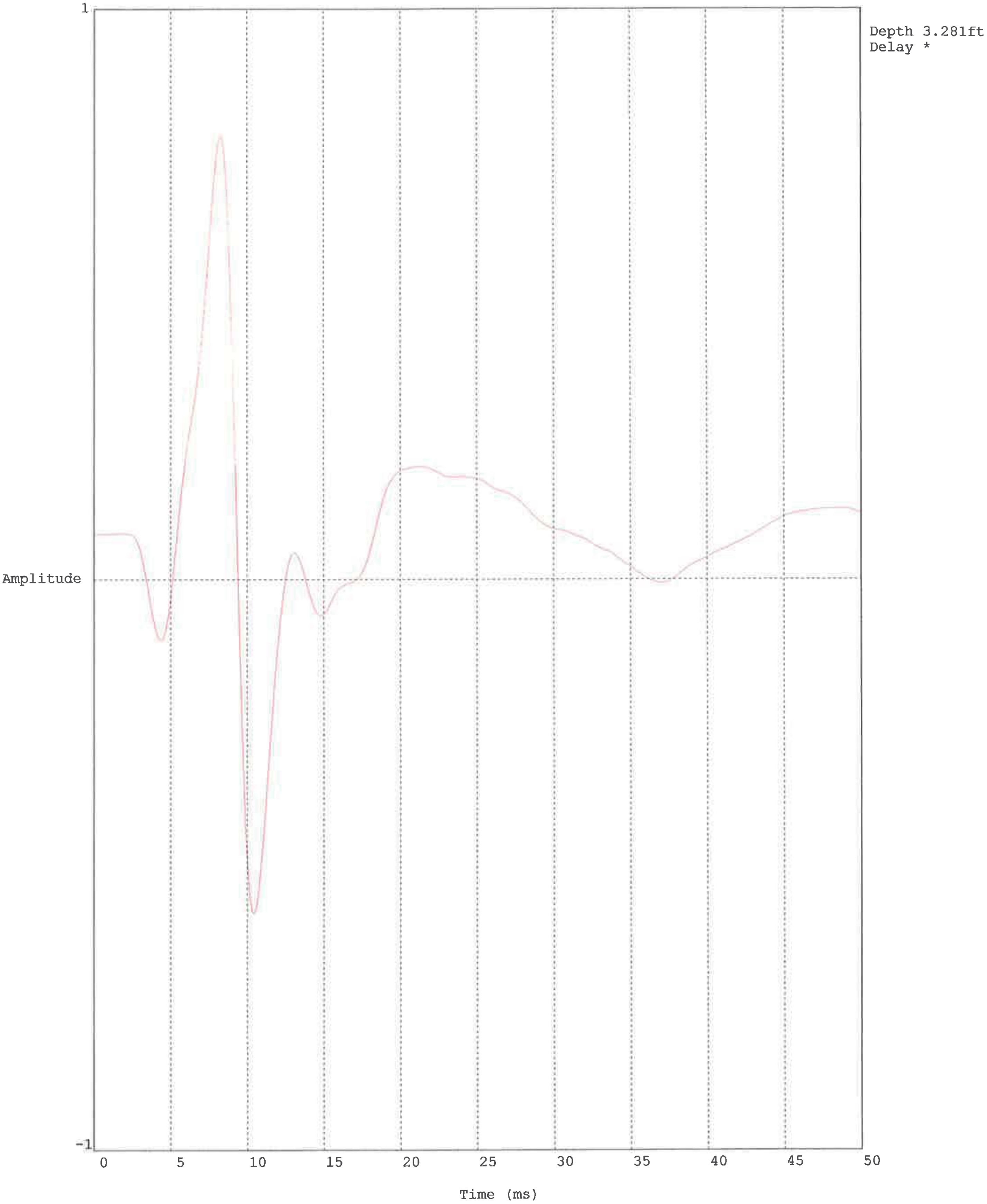
HDR ENG. / BC2,SCPT4 / BIG CK. NEWPORT



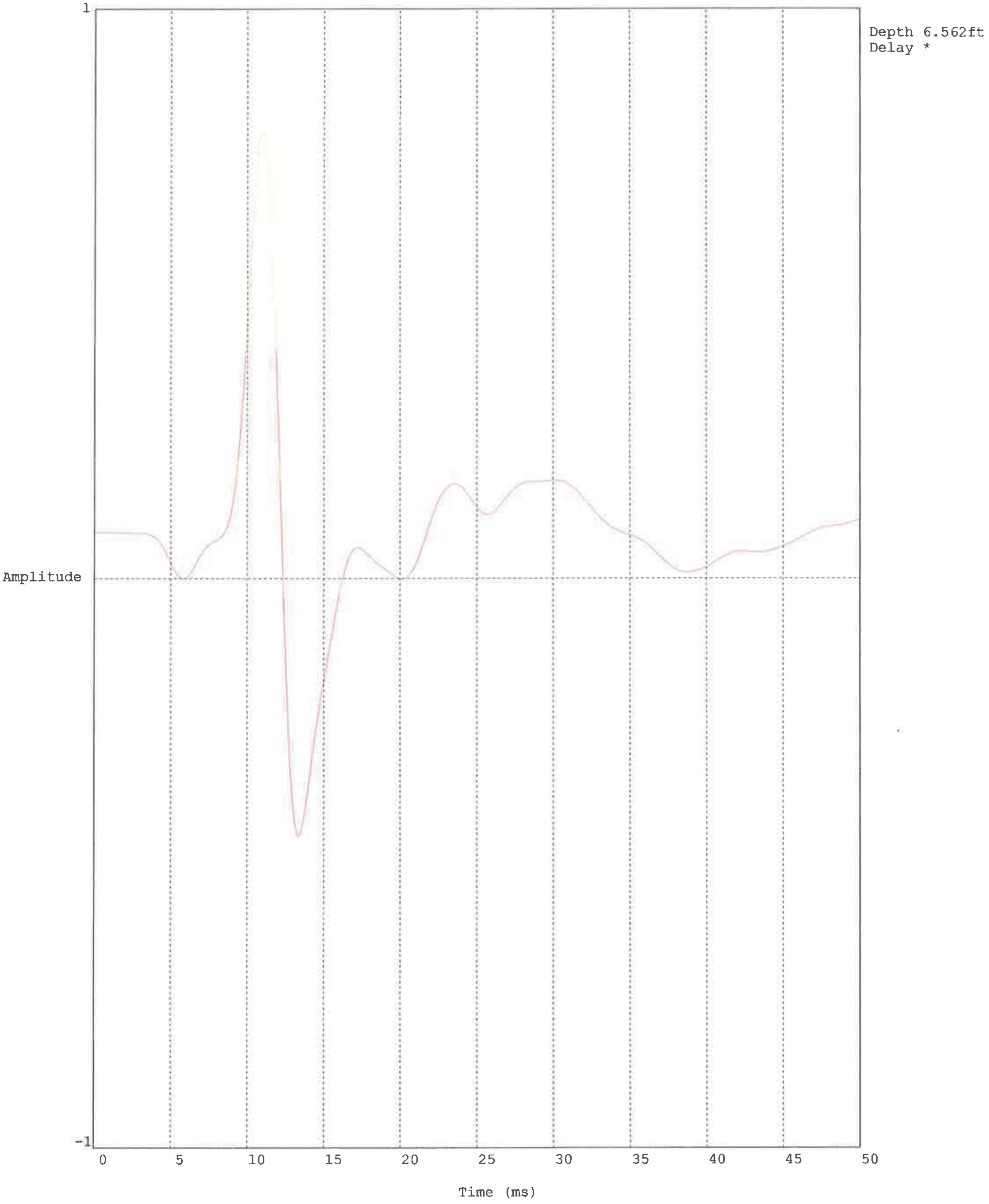
Hammer to Rod String Distance 1.3 (m)
* = Not Determined

THE SHEAR WAVE PICKS ARE COMPUTER GENERATED VALUES (STAFF REVIEW RECOMMENDED)

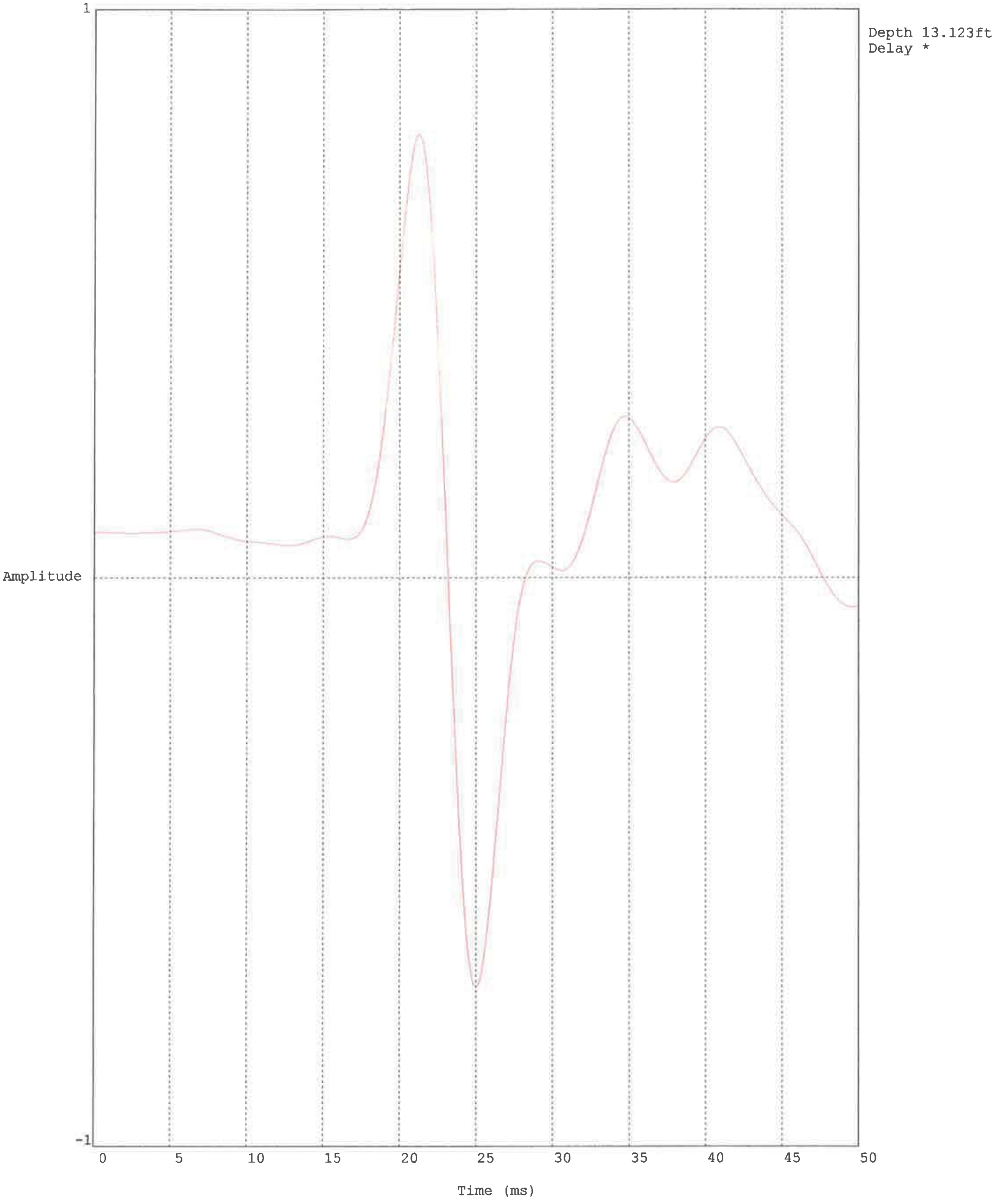
HDR ENG. / BC2,SCPT4 / BIG CK. NEWPORT



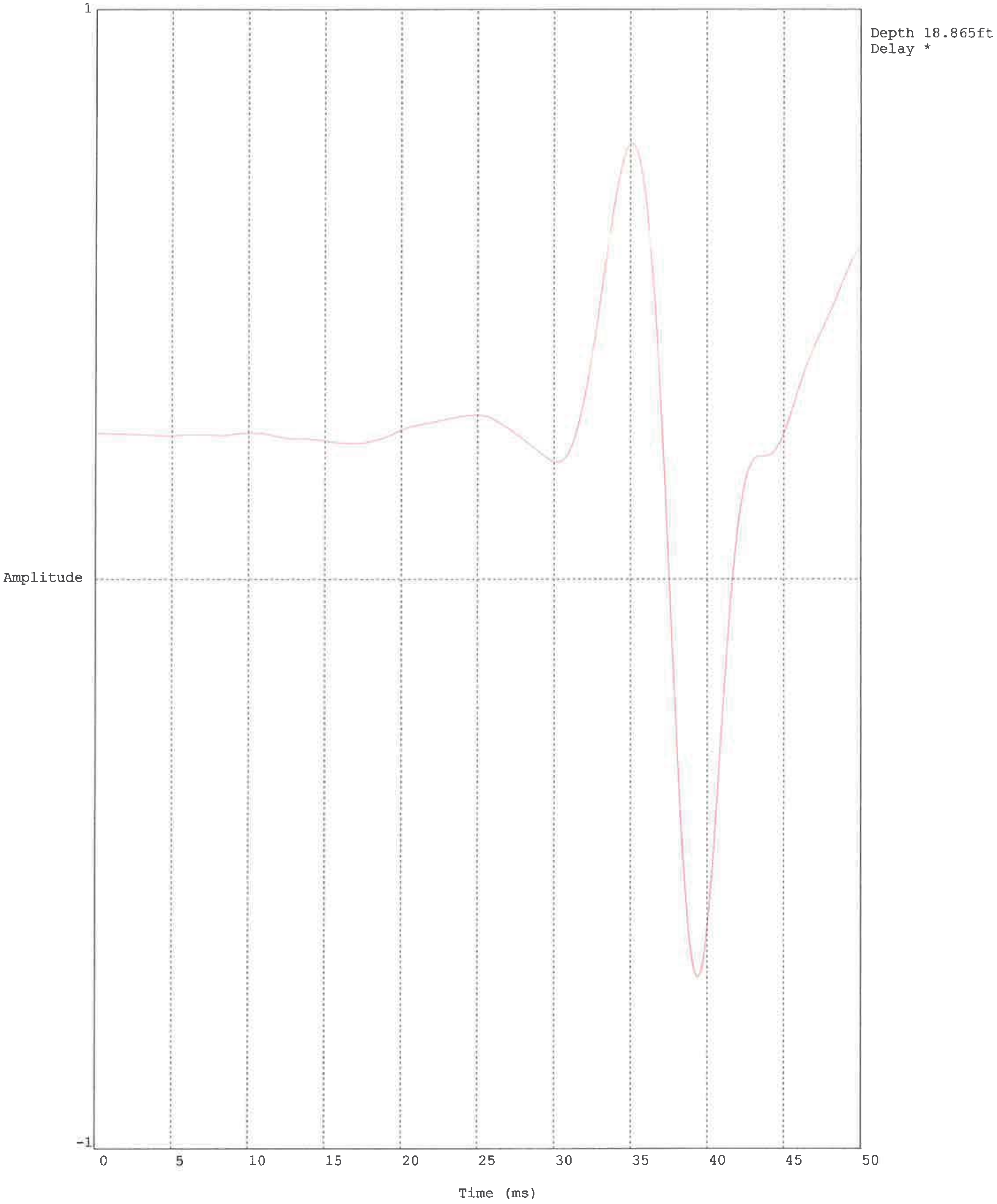
HDR ENG. / BC2,SCPT4 / BIG CK. NEWPORT



HDR ENG. / BC2,SCPT4 / BIG CK. NEWPORT



HDR ENG. / BC2,SCPT4 / BIG CK. NEWPORT



Data File:VEI434BC2SCPT4(487)
Operator:SAV/CM
Cone ID:DSG0736
Customer: BIG CK. DAM NEWPORT

10/24/2013 3:55:36 AM
Location:BC2SCPT4 / BIG CK. NEWPORT
Job Number:HDR ENG./BIG CK. NEWPORT
Units:

Depth (ft)	Qt TSF	Fs TSF	Fs/Qt (%)	Pw PSI	SPT N* 60% Hammer	Zone	Soil Behavior Type UBC-1983
0.82	29.29	1.1533	3.937	0.081	17	3	clay
0.98	24.74	1.3781	5.570	-0.825	19	4	silty clay to clay
1.15	33.95	1.2138	3.575	-1.621	20	4	silty clay to clay
1.31	34.86	1.5803	4.533	-1.519	21	4	silty clay to clay
1.48	31.64	1.5351	4.851	-1.564	29	3	clay
1.64	24.96	1.2955	5.190	-1.825	26	3	clay
1.80	23.47	1.1981	5.105	-1.904	22	3	clay
1.97	21.93	1.0340	4.714	-1.607	21	3	clay
2.13	19.90	0.7534	3.785	-1.153	14	4	silty clay to clay
2.30	22.19	0.8297	3.740	-0.643	14	4	silty clay to clay
2.46	23.74	0.7962	3.354	15.287	14	4	silty clay to clay
2.62	20.39	0.7848	3.849	15.411	14	4	silty clay to clay
2.79	21.08	0.8243	3.910	20.369	20	3	clay
2.95	22.16	1.2091	5.455	19.757	18	4	silty clay to clay
3.12	43.37	1.3234	3.052	23.212	17	6	sandy silt to clayey silt
3.28	68.46	1.4394	2.103	8.509	18	6	sandy silt to clayey silt
3.44	25.53	1.2041	4.716	2.879	18	5	clayey silt to silty clay
3.61	18.60	0.7908	4.253	5.355	19	3	clay
3.77	14.87	0.5034	3.386	8.253	9	4	silty clay to clay
3.94	11.11	0.4561	4.104	5.730	13	3	clay
4.10	13.93	0.6468	4.642	4.322	15	3	clay
4.27	23.27	0.9070	3.898	4.774	13	4	silty clay to clay
4.43	22.87	0.8392	3.670	3.614	14	4	silty clay to clay
4.59	20.15	0.7606	3.775	3.040	11	5	clayey silt to silty clay
4.76	22.98	0.6496	2.827	2.949	14	4	silty clay to clay
4.92	20.89	0.8740	4.185	2.879	11	5	clayey silt to silty clay
5.09	27.49	0.6535	2.377	3.520	13	5	clayey silt to silty clay
5.25	34.55	0.9741	2.820	4.951	15	5	clayey silt to silty clay
5.41	30.27	1.1991	3.962	15.748	15	5	clayey silt to silty clay
5.58	28.20	1.2514	4.437	14.844	18	4	silty clay to clay
5.74	25.66	1.2171	4.744	12.716	24	3	clay
5.91	22.23	0.9735	4.379	10.855	21	3	clay
6.07	18.36	0.9017	4.910	9.181	20	3	clay
6.23	20.88	0.7831	3.750	7.191	11	4	silty clay to clay
6.40	14.63	0.4565	3.121	5.474	15	3	clay
6.56	12.37	0.7219	5.836	5.285	10	4	silty clay to clay
6.73	21.98	0.7288	3.316	3.257	16	3	clay
6.89	14.33	0.7240	5.053	2.502	16	3	clay
7.05	13.57	0.7967	5.873	2.305	16	3	clay
7.22	21.13	0.8438	3.993	1.645	15	3	clay
7.38	13.70	0.8326	6.079	0.804	15	3	clay
7.55	12.58	0.9065	7.208	0.579	13	3	clay
7.71	13.95	1.1775	8.444	0.694	17	3	clay
7.87	28.22	1.0034	3.555	0.909	17	3	clay
8.04	12.22	0.8022	6.565	0.474	17	3	clay
8.20	11.87	0.6339	5.338	0.112	10	3	clay
8.37	7.18	0.7502	10.444	-0.091	11	3	clay
8.53	14.87	0.5682	3.822	-0.096	12	3	clay
8.69	14.76	0.5265	3.567	-0.134	12	3	clay
8.86	7.92	0.3546	4.477	-0.442	9	3	clay
9.02	4.24	0.3111	7.346	-0.603	9	3	clay
9.19	16.19	0.3682	2.274	-0.340	8	4	silty clay to clay
9.35	17.76	0.3811	2.145	-0.914	8	5	clayey silt to silty clay
9.51	17.00	0.3588	2.111	-2.270	8	5	clayey silt to silty clay
9.68	12.54	0.2837	2.263	-2.977	6	5	clayey silt to silty clay
9.84	5.52	0.1344	2.436	-3.147	4	4	silty clay to clay
10.01	2.55	0.0570	2.233	-3.111	3	3	clay

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

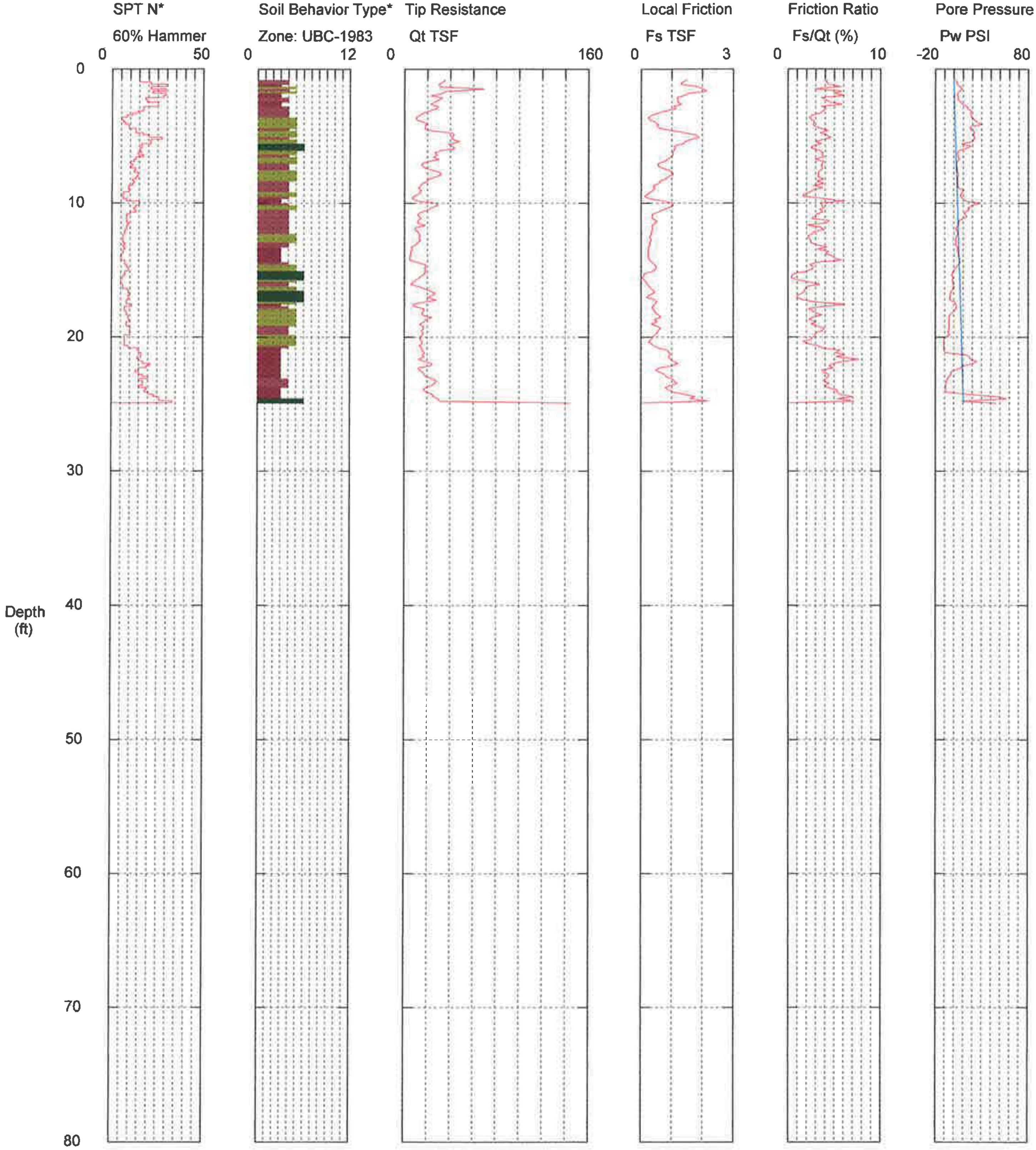
Depth (ft)	Qt TSF	Fs TSF	Fs/Qt (%)	Pw PSI	SPT N* 60% Hammer	Zone	Soil Behavior Type UBC-1983
10.17	1.52	0.0407	2.679	-2.444	2	3	clay
10.33	1.32	0.0432	3.282	-1.904	1	3	clay
10.50	1.65	0.0536	3.259	-1.440	1	2	organic material
10.66	1.63	0.1367	8.368	-1.000	3	3	clay
10.83	4.82	0.0967	2.006	-0.593	3	3	clay
10.99	3.98	0.1343	3.373	-0.421	3	3	clay
11.15	1.82	0.1470	8.072	-0.175	5	3	clay
11.32	10.92	0.2616	2.396	0.306	6	3	clay
11.48	6.12	0.2482	4.056	-1.313	7	3	clay
11.65	4.77	0.1648	3.454	-1.583	5	3	clay
11.81	5.10	0.1777	3.488	-0.543	5	3	clay
11.98	6.42	0.1982	3.085	0.029	6	3	clay
12.14	6.29	0.1648	2.619	3.059	5	3	clay
12.30	4.09	0.1448	3.544	10.140	5	3	clay
12.47	5.44	0.0946	1.741	10.599	3	4	silty clay to clay
12.63	6.16	0.1133	1.838	10.126	4	4	silty clay to clay
12.80	5.50	0.1113	2.022	9.968	4	4	silty clay to clay
12.96	5.34	0.0990	1.855	10.128	4	4	silty clay to clay
13.12	6.06	0.0897	1.480	10.233	3	1	sensitive fine grained
13.29	5.67	0.0600	1.058	4.819	3	1	sensitive fine grained
13.45	7.40	0.0565	0.764	5.455	3	1	sensitive fine grained
13.62	6.33	0.0909	1.436	5.520	3	1	sensitive fine grained
13.78	6.81	0.1205	1.771	5.783	3	5	clayey silt to silty clay
13.94	8.72	0.1076	1.234	6.070	4	5	clayey silt to silty clay
14.11	9.76	0.1075	1.101	6.070	5	5	clayey silt to silty clay
14.27	9.98	0.0953	0.955	5.869	5	5	clayey silt to silty clay
14.44	10.39	0.1029	0.990	5.728	5	5	clayey silt to silty clay
14.60	10.34	0.0970	0.938	5.761	4	6	sandy silt to clayey silt
14.76	10.53	0.0688	0.653	5.747	4	6	sandy silt to clayey silt
14.93	10.37	0.0854	0.824	5.697	4	6	sandy silt to clayey silt
15.09	9.62	0.0864	0.899	5.692	5	5	clayey silt to silty clay
15.26	11.22	0.2087	1.860	6.003	5	5	clayey silt to silty clay
15.42	9.57	0.2382	2.488	6.706	6	5	clayey silt to silty clay
15.58	14.03	0.2794	1.991	1.557	6	5	clayey silt to silty clay
15.75	13.26	0.2068	1.559	-0.658	7	5	clayey silt to silty clay
15.91	14.56	0.3004	2.063	-1.504	7	5	clayey silt to silty clay
16.08	15.50	0.3788	2.444	3.107	7	5	clayey silt to silty clay
16.24	14.97	0.4453	2.975	-1.698	7	5	clayey silt to silty clay
16.40	13.91	0.3895	2.800	-0.835	7	5	clayey silt to silty clay
16.57	13.71	0.3454	2.520	-1.181	8	4	silty clay to clay
16.73	11.95	0.4262	3.567	-1.279	8	4	silty clay to clay
16.90	10.45	0.4537	4.341	-1.141	9	4	silty clay to clay
17.06	19.12	0.5176	2.707	-0.937	8	5	clayey silt to silty clay
17.22	22.66	0.7001	3.089	-2.007	9	5	clayey silt to silty clay
17.39	16.50	0.7236	4.385	-3.681	11	4	silty clay to clay
17.55	10.35	0.4891	4.726	-4.606	12	3	clay
17.72	10.12	0.3709	3.665	-4.699	11	3	clay
17.88	12.88	0.4850	3.765	-4.632	12	3	clay
18.04	13.90	0.5400	3.885	-4.537	12	3	clay
18.21	10.91	0.4973	4.558	-4.362	11	3	clay
18.37	9.23	0.4269	4.626	-4.176	14	7	silty sand to sandy silt
18.54	10.86	1.1200	10.316	-3.944	20	7	silty sand to sandy silt
18.70	111.97	-32767.9800	-29266.280	-2.372	0	0	<out of range>

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

HDR ENG. / BC-2,SCPT-5 / BIG CK. DAM NEWPORT

Operator: SAV/CM
Sounding: VEI434BC2SCPT5(486)
Cone Used: DSG0736

CPT Date/Time: 10/24/2013 1:30:31 AM
Location: BC2,SCPT5 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT



Maximum Depth = 24.93 feet

Depth Increment = 0.164 feet

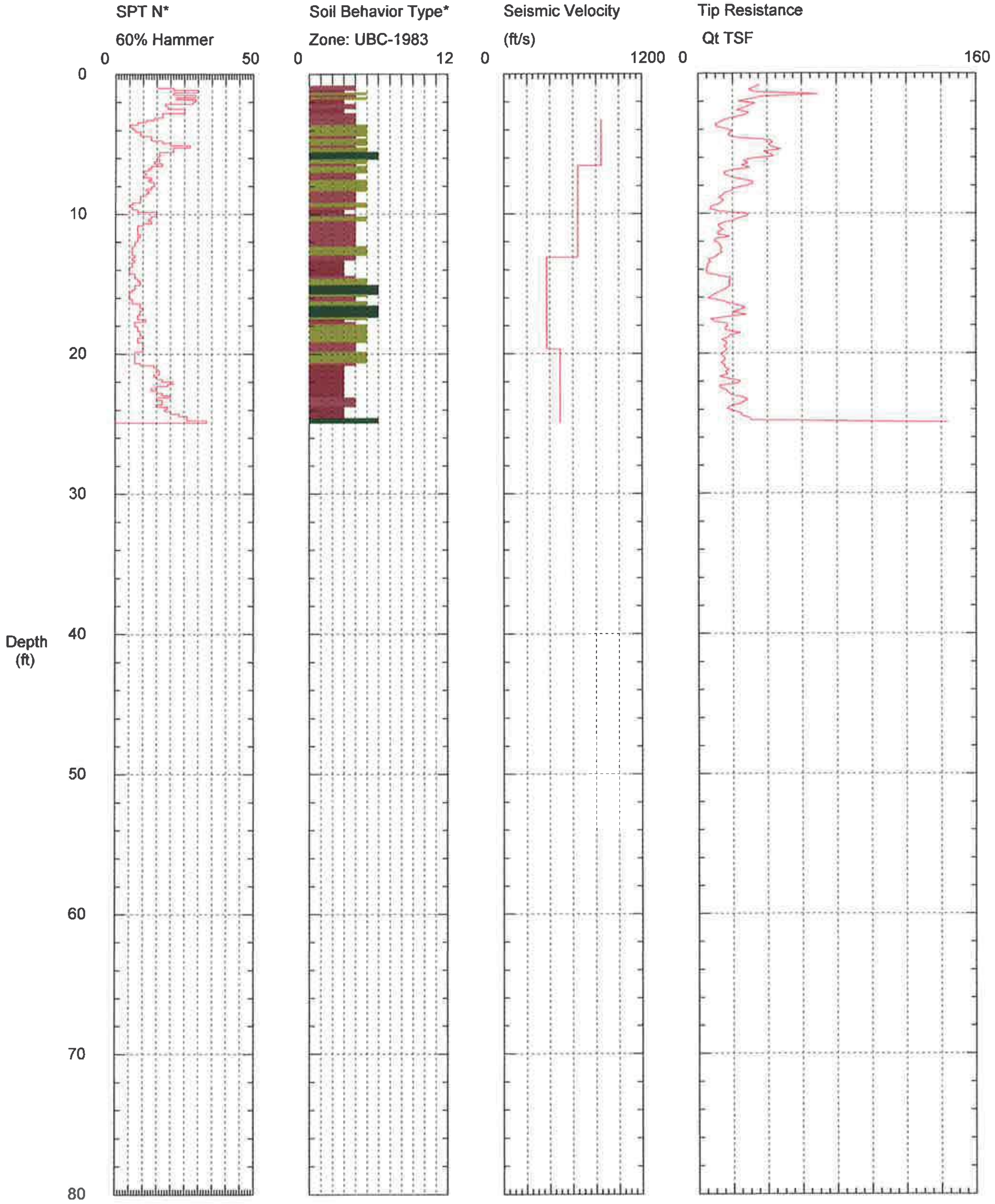
- ■ 1 sensitive fine grained
- ■ 2 organic material
- ■ 3 clay
- ■ 4 silty clay to clay
- ■ 5 clayey silt to silty clay
- ■ 6 sandy silt to clayey silt
- ■ 7 silty sand to sandy silt
- ■ 8 sand to silty sand
- ■ 9 sand
- ■ 10 gravelly sand to sand
- ■ 11 very stiff fine grained (*)
- ■ 12 sand to clayey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

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

HDR ENG. / BC-2,SCPT-5 / BIG CK. DAM NEWPORT

Operator: SAV/CM
Sounding: VEI434BC2SCPT5(486)
Cone Used: DSG0736

CPT Date/Time: 10/24/2013 1:30:31 AM
Location: BC2,SCPT5 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT



Maximum Depth = 24.93 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 (*)

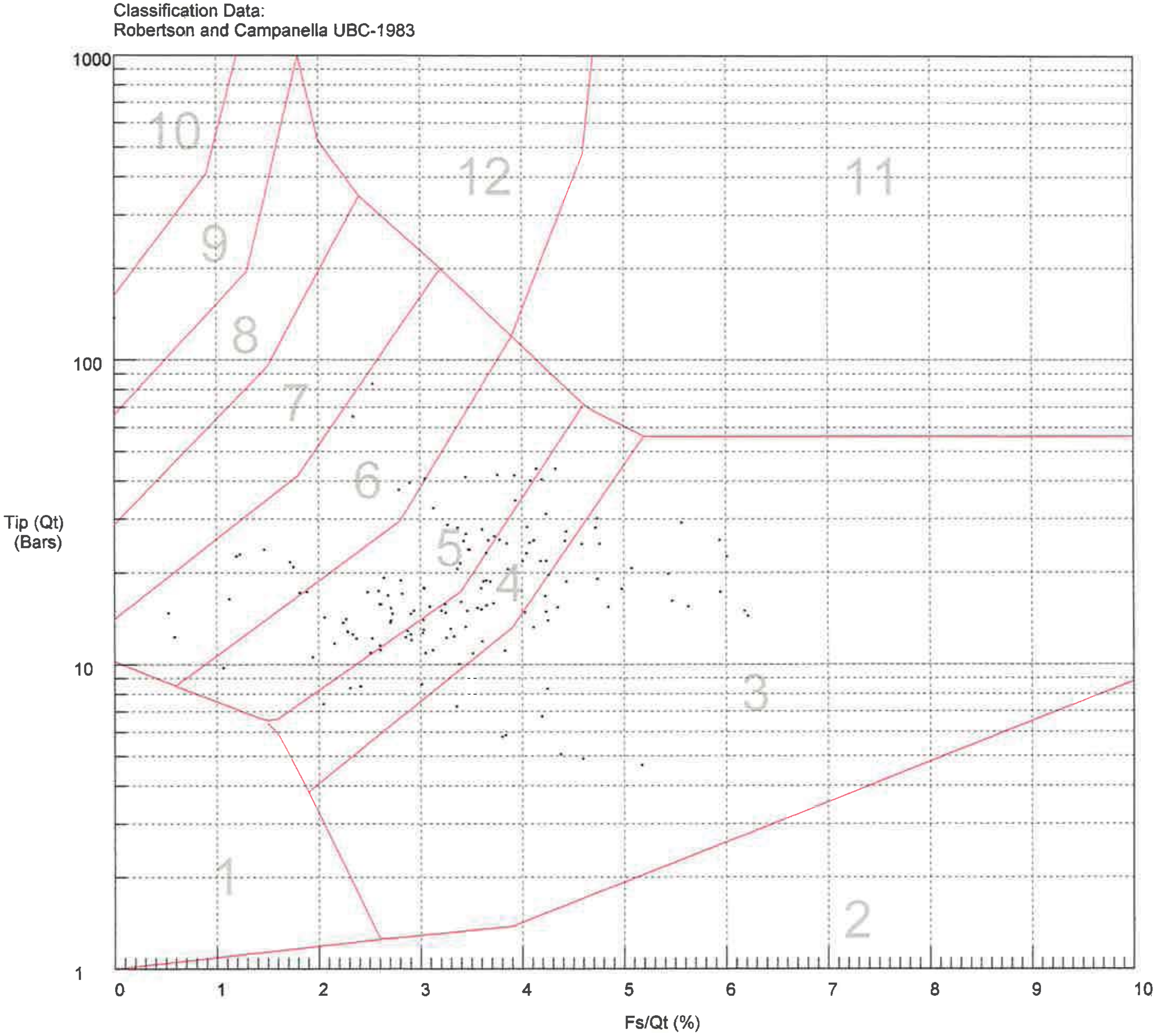
*Soil behavior type and SPT based on data from UBC-1983

HDR ENG. / BC-2,SCPT-5 / BIG CK. DAM NEWPORT

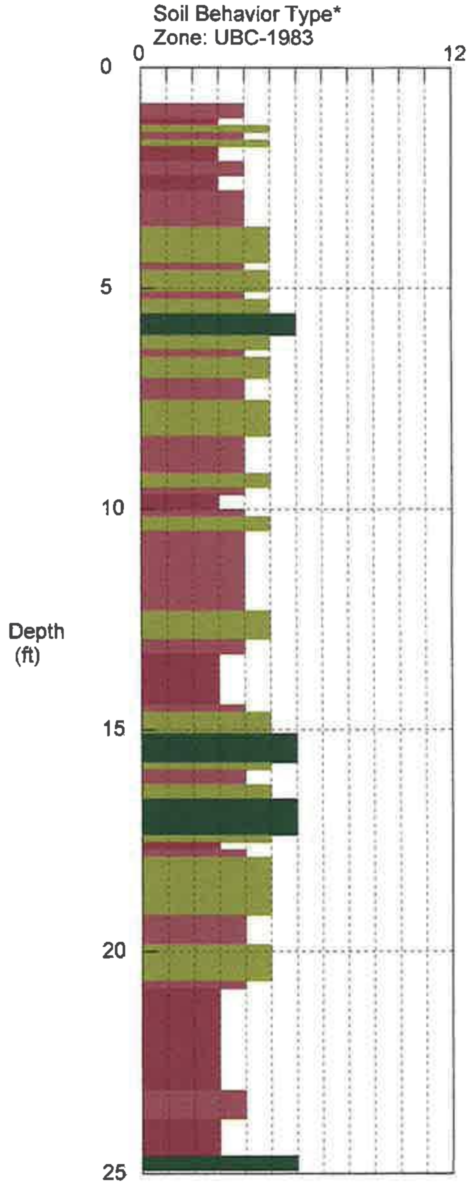
Operator: SAV/CM
Sounding: VEI434BC2SCPT5(486)
Cone Used: DSG0736

CPT Date/Time: 10/24/2013 1:30:31 AM
Location: BC2,SCPT5 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT

Classification Data:
Robertson and Campanella UBC-1983

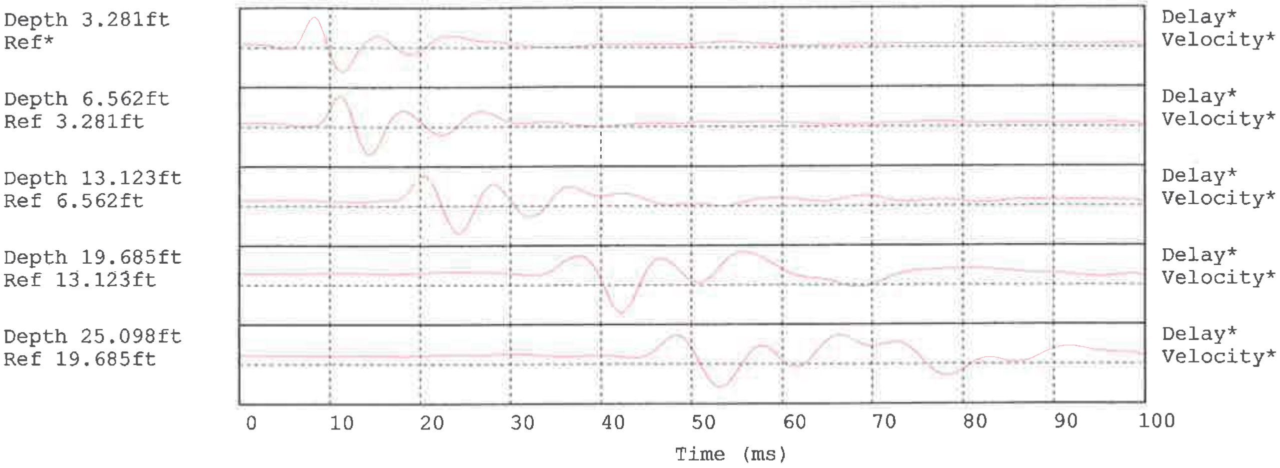


Soil Behavior Type*
Zone: UBC-1983



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

HDR ENG. / BC2,SCPT5 / BIG CK. NEWPORT

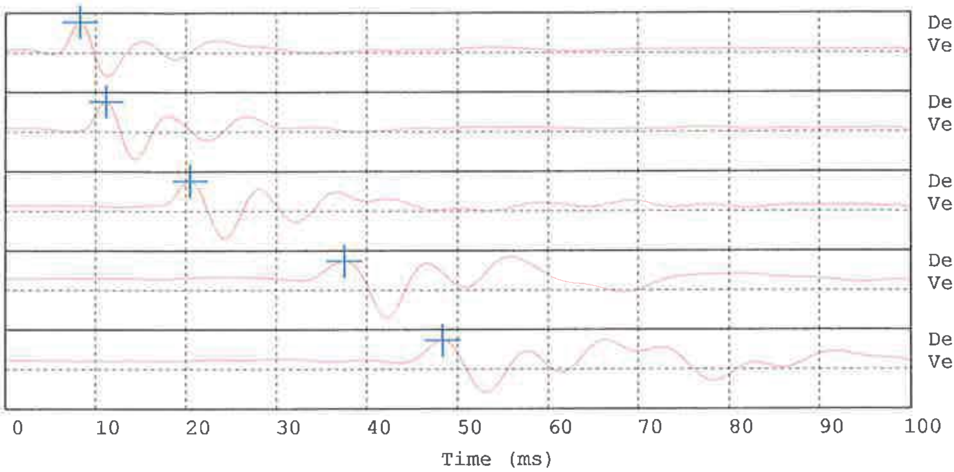


Hammer to Rod String Distance 1.3 (m)
* = Not Determined

THE SHEAR WAVE PICKS ARE COMPUTER GENERATED VALUES (STAFF REVIEW RECOMMENDED)

HDR ENG. / BC2,SCPT5 / BIG CK. NEWPORT

Depth 3.281ft Ref*	Delay 8.28ms Velocity*
Depth 6.562ft Ref 3.281ft	Delay 11.17ms Velocity 845.91ft/s
Depth 13.123ft Ref 6.562ft	Delay 20.47ms Velocity 642.52ft/s
Depth 19.685ft Ref 13.123ft	Delay 37.54ms Velocity 371.59ft/s
Depth 25.098ft Ref 19.685ft	Delay 48.36ms Velocity 491.37ft/s



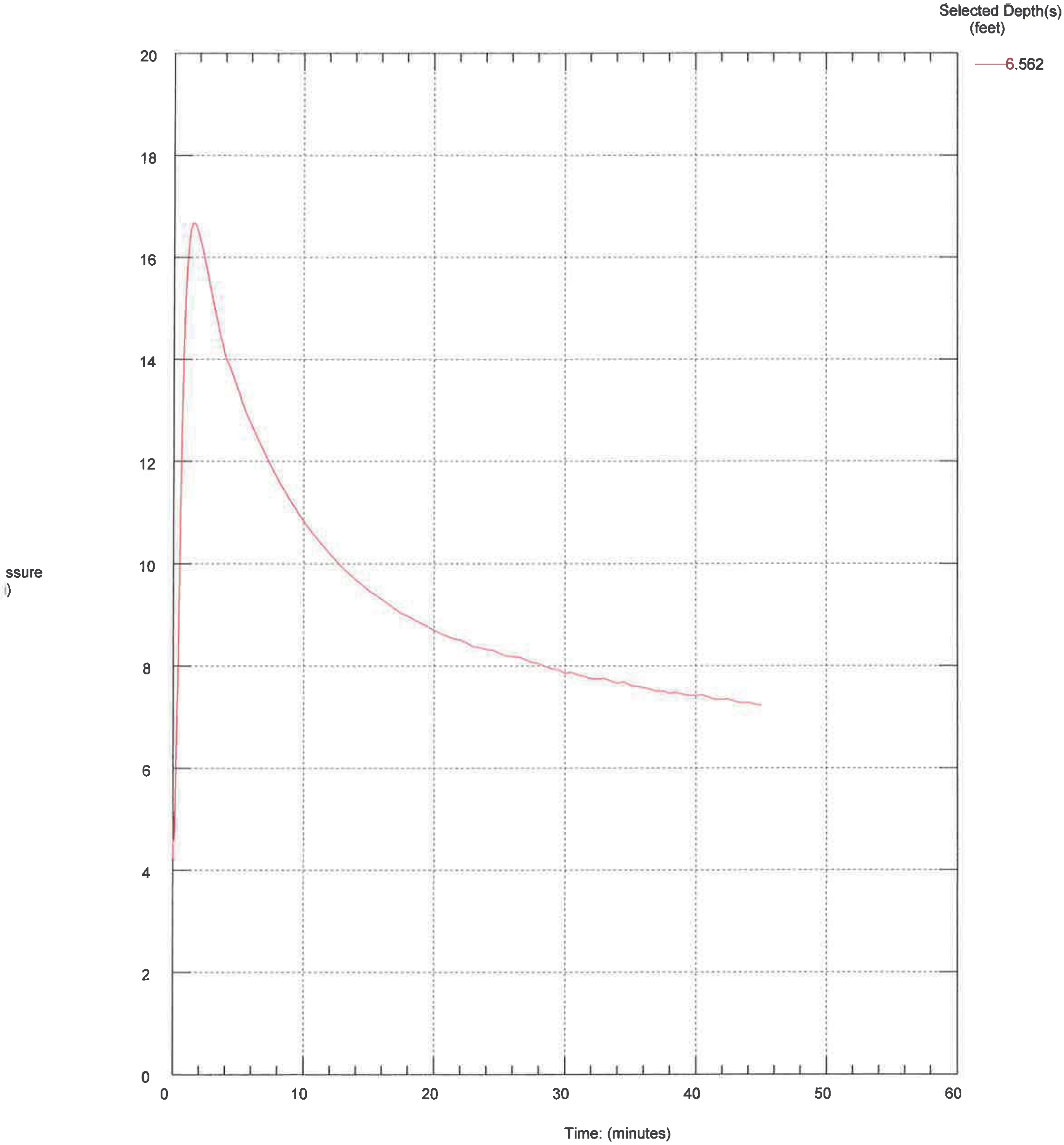
Hammer to Rod String Distance 1.3 (m)
* = Not Determined

THE SHEAR WAVE PICKS ARE COMPUTER GENERATED VALUES (STAFF REVIEW RECOMMENDED)

HDR ENG. / BC-2,CPT-5 / BIG CK. DAM NEWPORT

Operator SAV/CM
Sounding: VEI434BC2SCPT5(486)
Cone Used: DSG0736

CPT Date/Time: 10/24/2013 1:30:31 AM
Location: BC2SCPT5 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT

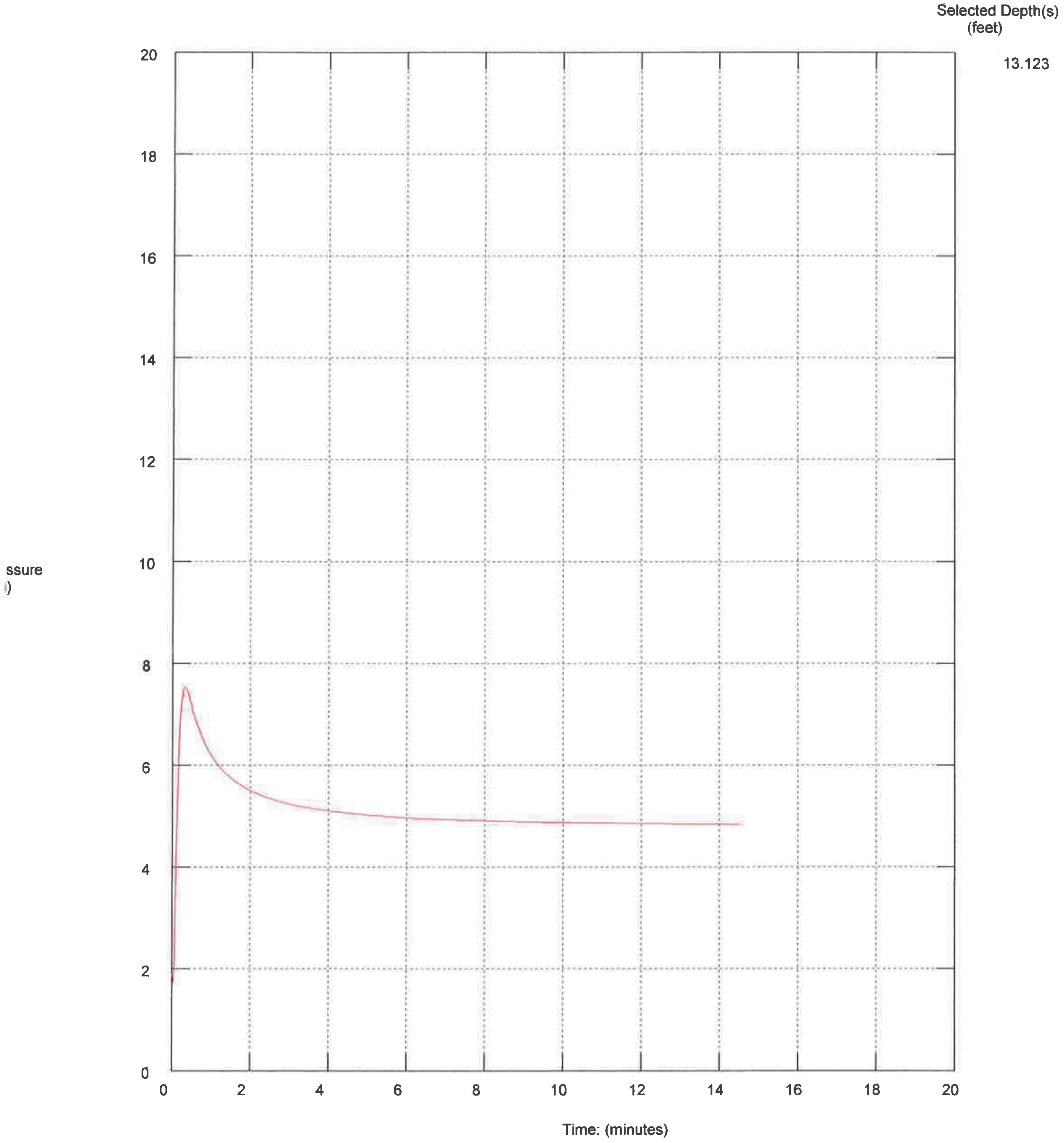


Maximum Pressure = 16.671 psi
Hydrostatic Pressure :

HDR ENG. / BC-2,CPT-5 / BIG CK. DAM NEWPORT

Operator SAV/CM
Sounding: VEI434BC2SCPT5(486)
Cone Used: DSG0736

CPT Date/Time: 10/24/2013 1:30:31 AM
Location: BC2SCPT5 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT

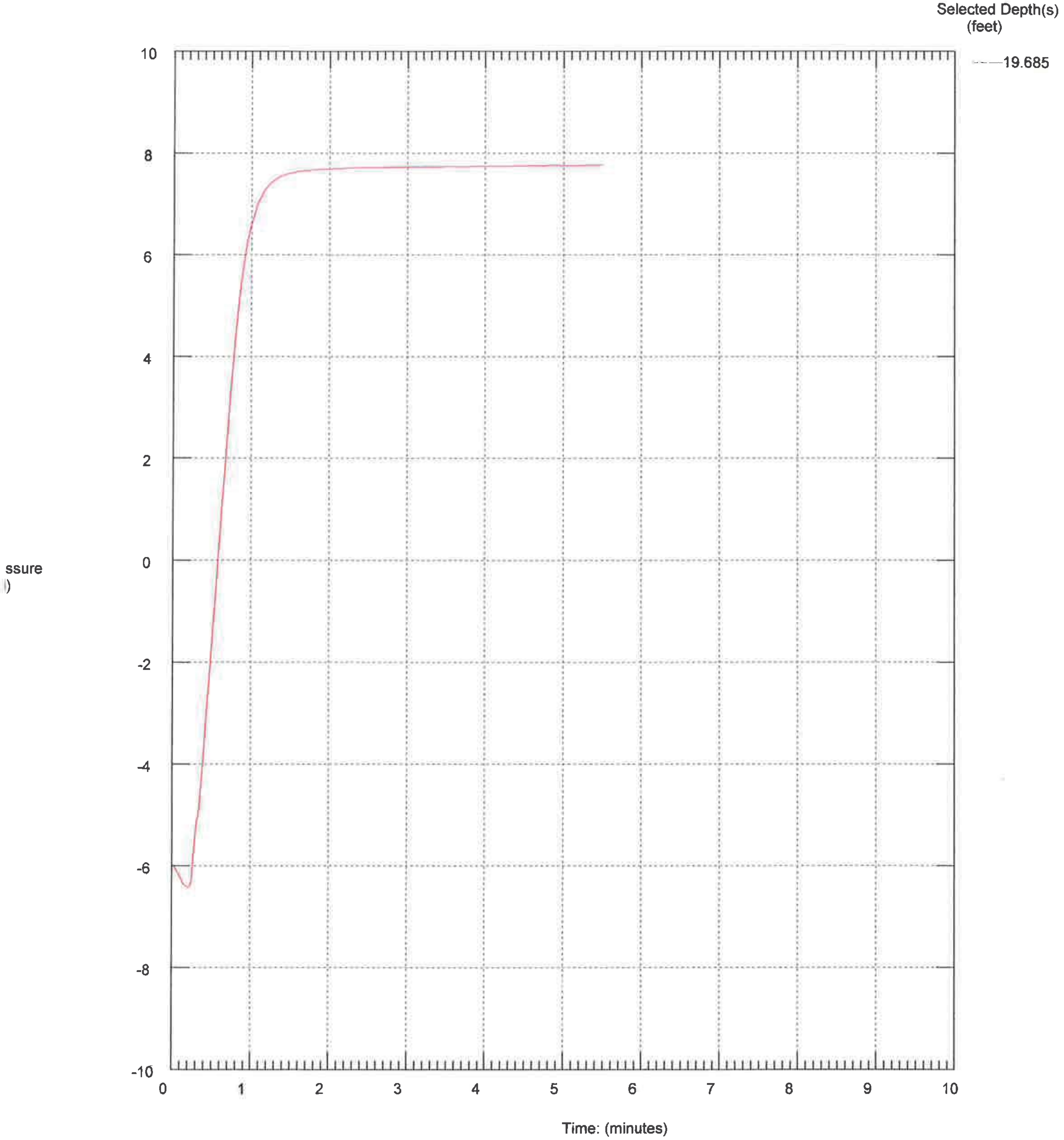


Maximum Pressure = 7.531 psi
Hydrostatic Pressure

HDR ENG. / BC-2,CPT-5 / BIG CK. DAM NEWPORT

Operator SAV/CM
Sounding: VEI434BC2SCPT5(486)
Cone Used: DSG0736

CPT Date/Time: 10/24/2013 1:30:31 AM
Location: BC2SCPT5 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT

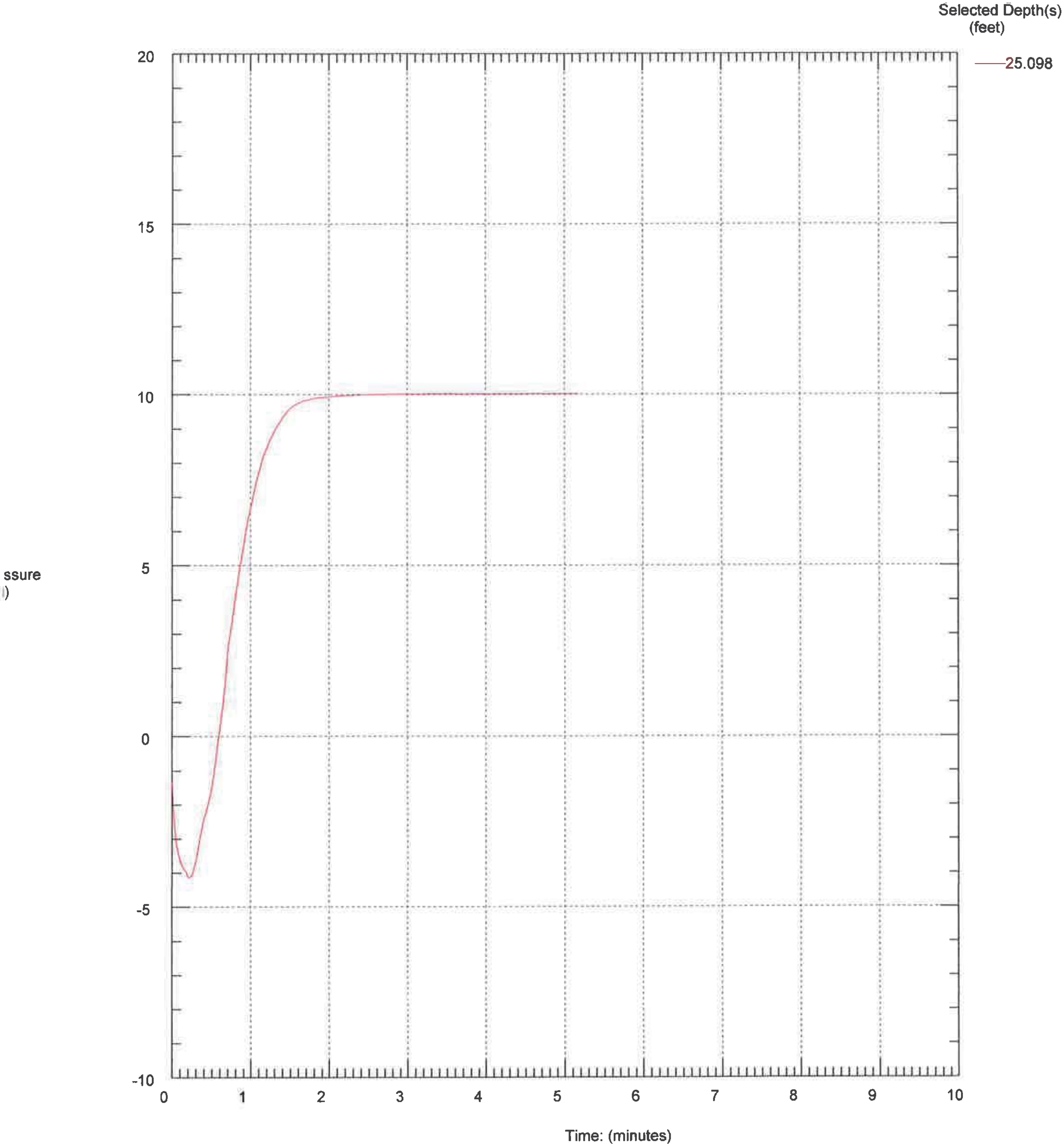


Maximum Pressure = 7.761 psi
Hydrostatic Pressure

HDR ENG. / BC-2,CPT-5 / BIG CK. DAM NEWPORT

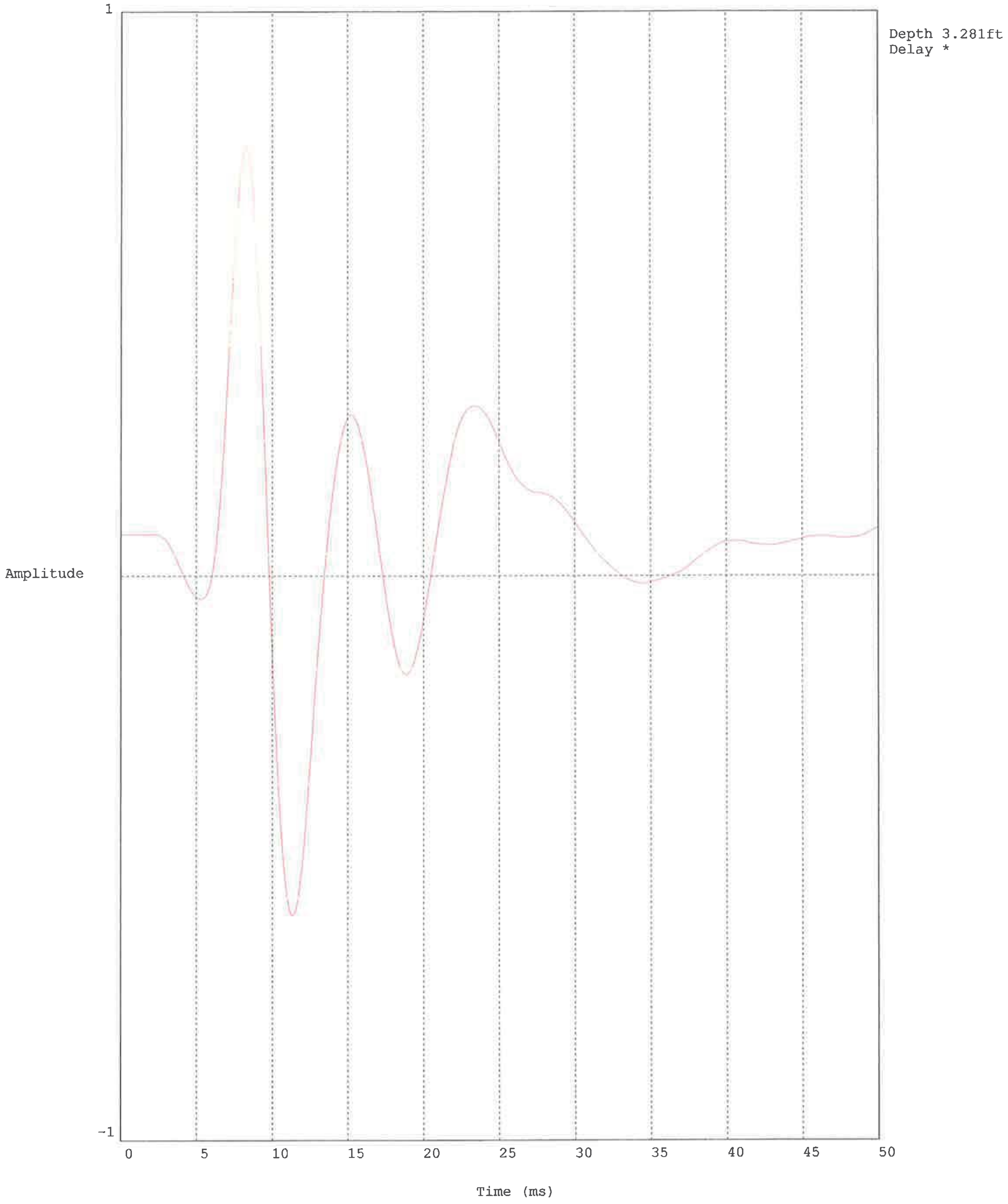
Operator SAV/CM
Sounding: VEI434BC2SCPT5(486)
Cone Used: DSG0736

CPT Date/Time: 10/24/2013 1:30:31 AM
Location: BC2SCPT5 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT

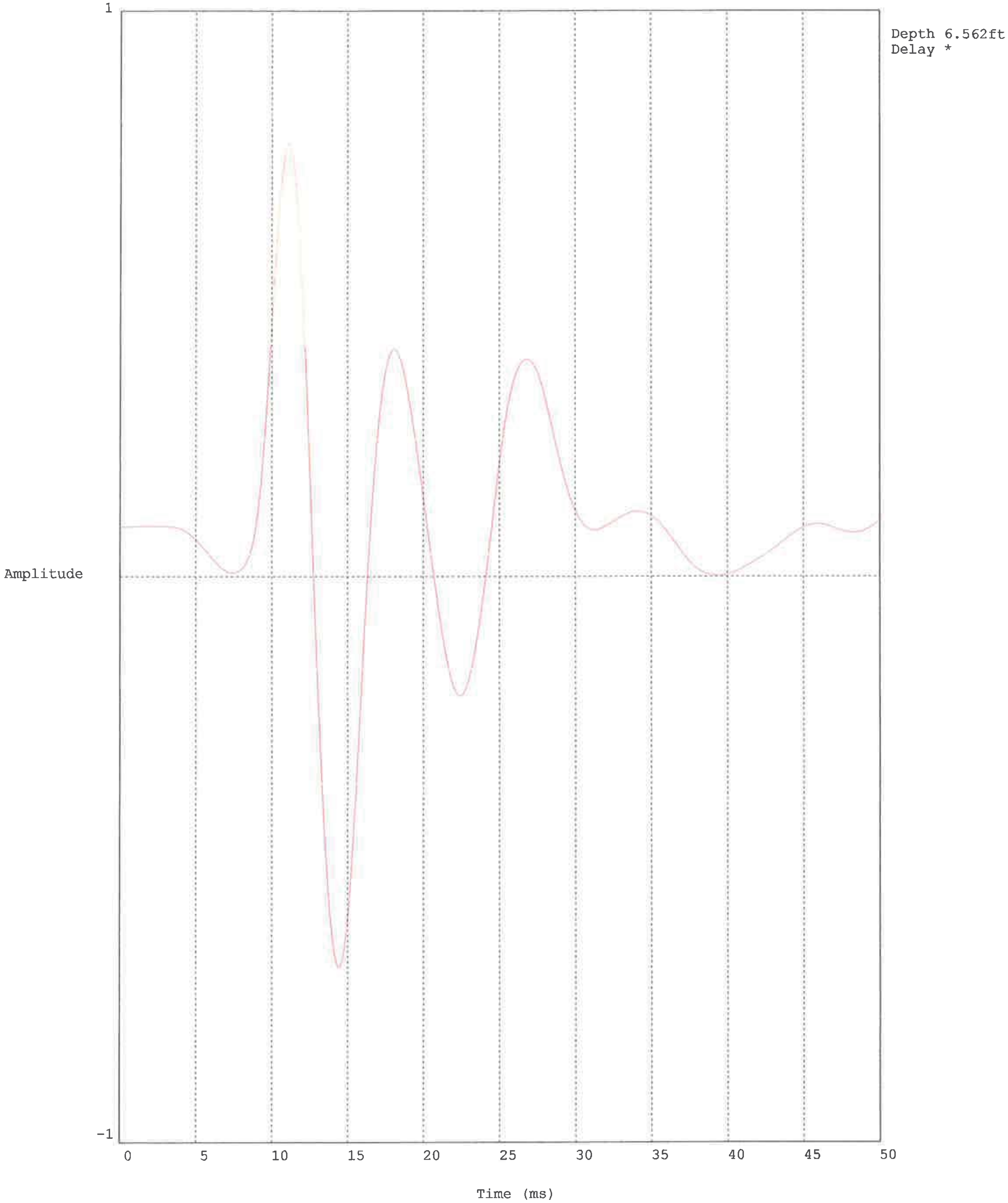


Maximum Pressure = 10.023 psi
Hydrostatic Pressure

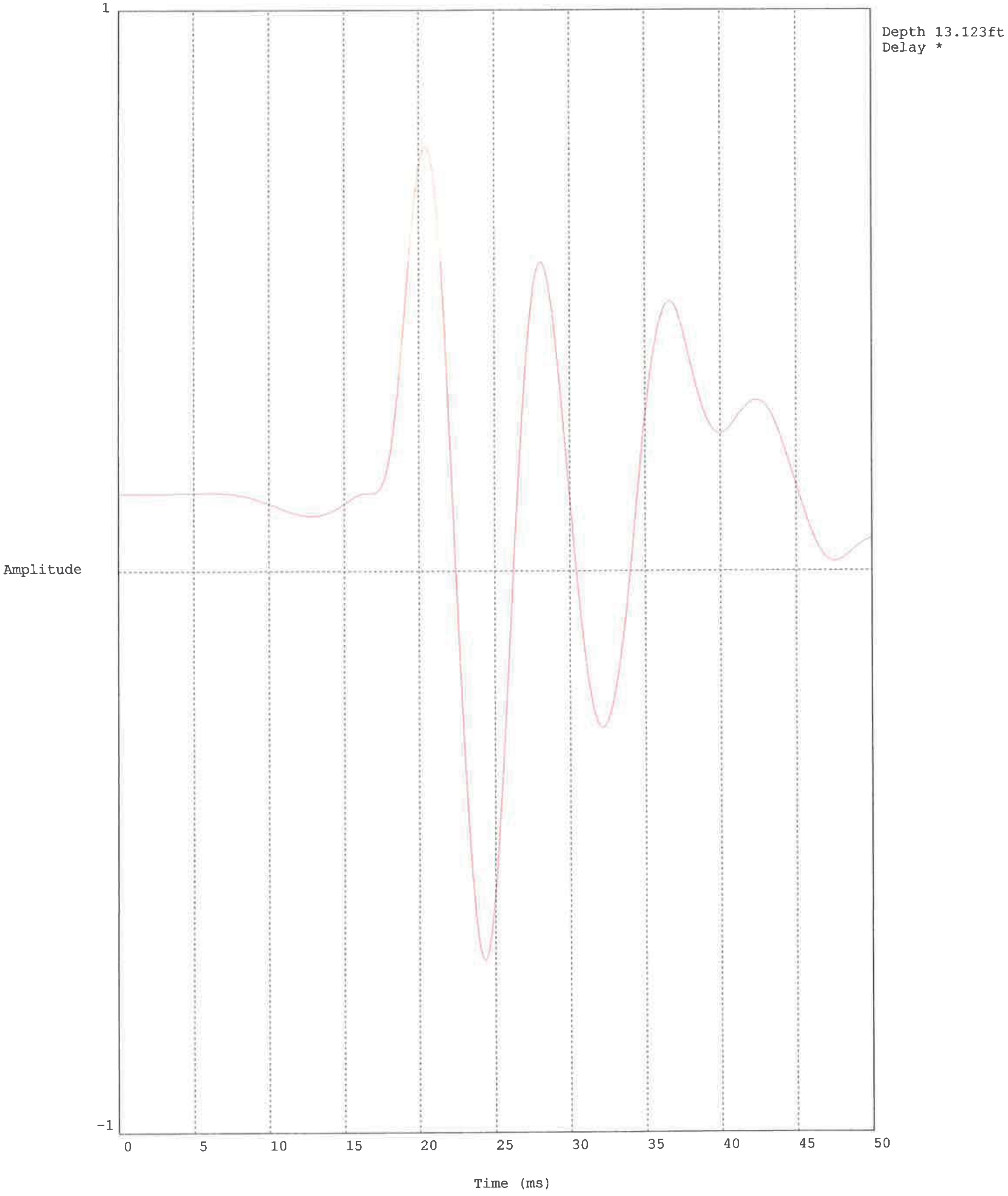
HDR ENG. / BC2,SCPT5 / BIG CK. NEWPORT



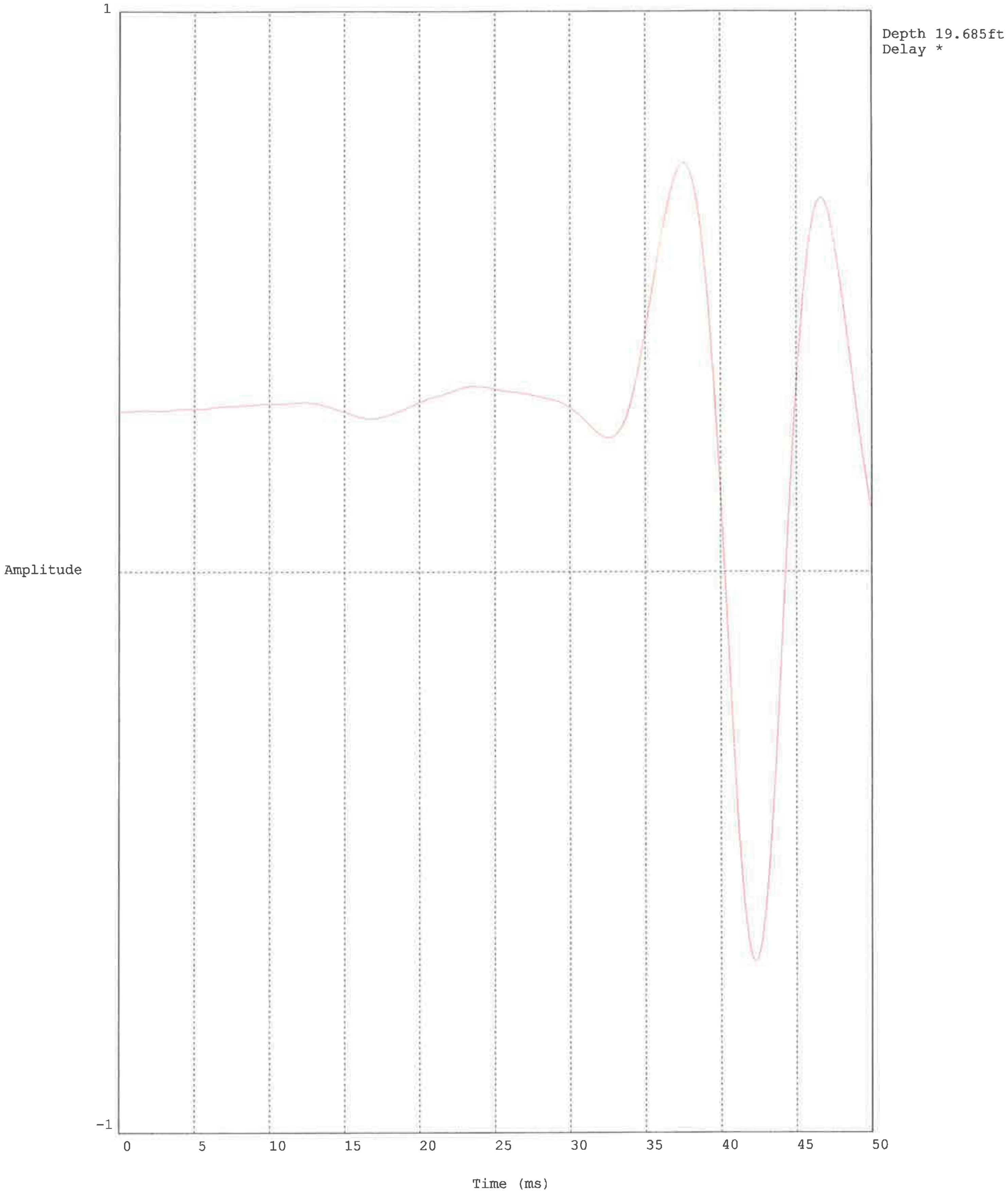
HDR ENG. / BC2,SCPT5 / BIG CK. NEWPORT



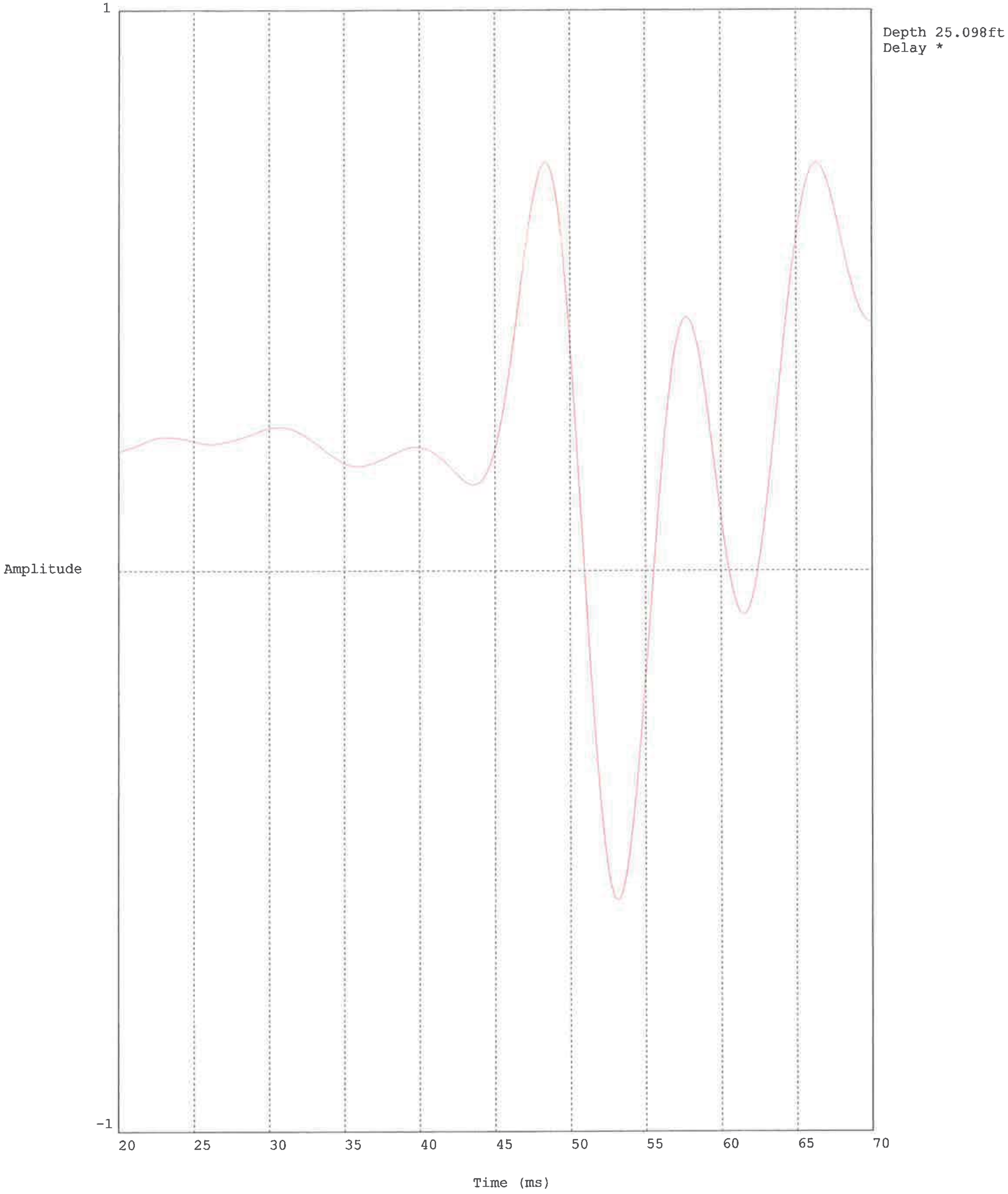
HDR ENG. / BC2,SCPT5 / BIG CK. NEWPORT



HDR ENG. / BC2,SCPT5 / BIG CK. NEWPORT



HDR ENG. / BC2,SCPT5 / BIG CK. NEWPORT



Data File:VEI434BC2SCPT5(486)
Operator:SAV/CM
Cone ID:DSG0736
Customer: BIG CK. DAM NEWPORT

10/24/2013 1:30:31 AM
Location:BC2,SCPT5 / BIG CK. NEWPORT
Job Number:HDR ENG./BIG CK. NEWPORT
Units:

Depth (ft)	Qt TSF	Fs TSF	Fs/Qt (%)	Pw PSI	SPT N* 60% Hammer	Zone	Soil Behavior Type UBC-1983
0.82	35.58	1.5072	4.236	1.461	15	4	silty clay to clay
0.98	32.83	1.3528	4.120	3.511	21	4	silty clay to clay
1.15	29.52	1.2881	4.363	3.597	30	3	clay
1.31	32.11	1.8362	5.718	7.144	21	5	clayey silt to silty clay
1.48	69.36	2.0171	2.908	9.026	29	4	silty clay to clay
1.64	36.58	2.1229	5.804	5.816	22	5	clayey silt to silty clay
1.80	31.76	1.5618	4.918	4.070	29	3	clay
1.97	23.12	1.4041	6.074	4.063	28	3	clay
2.13	33.10	1.1884	3.591	5.216	18	4	silty clay to clay
2.30	29.48	1.2069	4.093	4.410	19	4	silty clay to clay
2.46	26.04	1.1925	4.580	7.837	25	3	clay
2.62	22.45	1.3114	5.841	9.671	25	3	clay
2.79	29.53	1.0737	3.636	14.316	17	4	silty clay to clay
2.95	27.67	1.1364	4.108	14.950	17	4	silty clay to clay
3.12	21.09	0.8042	3.813	19.006	14	4	silty clay to clay
3.28	15.73	0.5483	3.485	17.755	11	4	silty clay to clay
3.44	13.55	0.3605	2.660	18.671	8	4	silty clay to clay
3.61	10.08	0.2338	2.320	19.032	5	5	clayey silt to silty clay
3.77	10.64	0.2647	2.489	22.600	6	5	clayey silt to silty clay
3.94	15.51	0.4435	2.860	24.191	7	5	clayey silt to silty clay
4.10	19.97	0.5478	2.744	30.487	9	5	clayey silt to silty clay
4.27	18.07	0.5190	2.871	19.046	9	5	clayey silt to silty clay
4.43	17.72	0.6247	3.525	16.892	13	4	silty clay to clay
4.59	23.38	1.0146	4.339	21.237	13	5	clayey silt to silty clay
4.76	41.58	1.4433	3.472	21.670	17	5	clayey silt to silty clay
4.92	43.32	1.8005	4.157	20.785	20	5	clayey silt to silty clay
5.09	40.72	1.8835	4.625	22.174	27	4	silty clay to clay
5.25	42.83	1.6365	3.821	21.928	21	5	clayey silt to silty clay
5.41	48.07	1.4235	2.961	19.135	21	5	clayey silt to silty clay
5.58	38.21	1.3861	3.627	8.325	16	6	sandy silt to clayey silt
5.74	41.65	1.0860	2.608	18.410	16	6	sandy silt to clayey silt
5.91	43.95	1.1130	2.532	13.194	15	6	sandy silt to clayey silt
6.07	31.77	1.0739	3.380	5.560	16	5	clayey silt to silty clay
6.23	26.43	1.0077	3.813	3.850	14	5	clayey silt to silty clay
6.40	28.82	1.0567	3.667	4.723	17	4	silty clay to clay
6.56	25.23	0.9745	3.863	4.259	13	5	clayey silt to silty clay
6.73	30.16	0.8727	2.893	6.211	12	5	clayey silt to silty clay
6.89	19.40	0.7502	3.867	3.635	10	5	clayey silt to silty clay
7.05	15.12	0.5530	3.656	2.937	11	4	silty clay to clay
7.22	15.34	0.5505	3.589	2.944	10	4	silty clay to clay
7.38	18.57	0.6857	3.692	3.324	13	4	silty clay to clay
7.55	24.97	0.8953	3.585	3.724	12	5	clayey silt to silty clay
7.71	31.11	1.0174	3.270	4.128	14	5	clayey silt to silty clay
7.87	31.97	1.0520	3.291	4.910	14	5	clayey silt to silty clay
8.04	26.96	0.8727	3.237	4.080	13	5	clayey silt to silty clay
8.20	20.68	0.8023	3.880	3.611	11	5	clayey silt to silty clay
8.37	19.81	0.6130	3.094	4.374	12	4	silty clay to clay
8.53	15.37	0.5933	3.861	3.958	11	4	silty clay to clay
8.69	14.34	0.3985	2.778	4.687	9	4	silty clay to clay
8.86	11.92	0.4424	3.710	4.455	9	4	silty clay to clay
9.02	14.57	0.3964	2.721	11.078	9	4	silty clay to clay
9.19	13.90	0.3215	2.312	8.693	6	5	clayey silt to silty clay
9.35	11.81	0.2015	1.706	7.741	5	5	clayey silt to silty clay
9.51	7.39	0.1198	1.621	7.170	6	4	silty clay to clay
9.68	7.00	0.2833	4.044	9.882	8	3	clay
9.84	11.67	0.7035	6.028	13.175	15	3	clay
10.01	28.07	0.9931	3.538	27.984	15	4	silty clay to clay

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

Depth (ft)	Qt TSF	Fs TSF	Fs/Qt (%)	Pw PSI	SPT N* 60% Hammer	Zone	Soil Behavior Type UBC-1983
10.17	28.82	1.0490	3.640	19.362	13	5	clayey silt to silty clay
10.33	23.30	0.8984	3.856	14.957	12	5	clayey silt to silty clay
10.50	20.74	0.7100	3.423	16.193	13	4	silty clay to clay
10.66	14.74	0.5584	3.789	11.539	10	4	silty clay to clay
10.83	11.59	0.3599	3.105	11.429	8	4	silty clay to clay
10.99	12.45	0.3727	2.994	14.354	8	4	silty clay to clay
11.15	14.51	0.3678	2.535	10.346	8	4	silty clay to clay
11.32	11.42	0.5091	4.458	5.218	8	4	silty clay to clay
11.48	11.48	0.4722	4.115	5.355	9	4	silty clay to clay
11.65	18.19	0.3729	2.051	5.261	8	4	silty clay to clay
11.81	10.22	0.3598	3.521	2.542	8	4	silty clay to clay
11.98	9.35	0.3660	3.915	2.913	7	4	silty clay to clay
12.14	11.88	0.3356	2.825	4.070	7	4	silty clay to clay
12.30	12.96	0.3402	2.625	4.726	6	5	clayey silt to silty clay
12.47	13.44	0.2912	2.167	3.209	6	5	clayey silt to silty clay
12.63	12.97	0.2875	2.217	2.353	6	5	clayey silt to silty clay
12.80	13.75	0.3394	2.469	2.148	6	5	clayey silt to silty clay
12.96	11.44	0.2776	2.426	2.193	7	4	silty clay to clay
13.12	9.83	0.2956	3.007	1.877	6	4	silty clay to clay
13.29	5.77	0.2403	4.166	4.829	7	3	clay
13.45	7.21	0.2283	3.169	3.566	6	3	clay
13.62	5.36	0.2343	4.373	4.267	6	3	clay
13.78	5.58	0.2270	4.069	4.900	5	3	clay
13.94	4.95	0.2333	4.711	5.283	5	3	clay
14.11	4.82	0.2445	5.070	5.709	5	3	clay
14.27	4.86	0.2793	5.741	6.118	7	3	clay
14.44	11.44	0.3617	3.162	6.560	7	4	silty clay to clay
14.60	18.65	0.4499	2.413	5.029	8	5	clayey silt to silty clay
14.76	18.05	0.4928	2.730	2.497	9	5	clayey silt to silty clay
14.93	17.99	0.4735	2.632	0.784	9	5	clayey silt to silty clay
15.09	18.22	0.3781	2.075	-0.906	7	6	sandy silt to clayey silt
15.26	17.74	0.1263	0.712	-1.755	7	6	sandy silt to clayey silt
15.42	15.52	0.0726	0.468	-2.372	6	6	sandy silt to clayey silt
15.58	13.04	0.0452	0.347	-0.361	5	6	sandy silt to clayey silt
15.75	10.02	0.1076	1.074	-1.739	5	5	clayey silt to silty clay
15.91	7.49	0.1716	2.291	-1.126	5	4	silty clay to clay
16.08	5.72	0.1959	3.422	-0.297	6	4	silty clay to clay
16.24	13.34	0.2718	2.038	0.079	6	5	clayey silt to silty clay
16.40	17.65	0.3230	1.830	-1.026	9	5	clayey silt to silty clay
16.57	23.37	0.4292	1.837	-2.597	9	6	sandy silt to clayey silt
16.73	27.00	0.4175	1.546	-3.539	10	6	sandy silt to clayey silt
16.90	24.22	0.2466	1.018	-4.015	9	6	sandy silt to clayey silt
17.06	19.86	0.1837	0.925	-4.499	9	6	sandy silt to clayey silt
17.22	27.90	0.4539	1.627	-4.613	8	6	sandy silt to clayey silt
17.39	17.74	0.5104	2.878	1.320	8	5	clayey silt to silty clay
17.55	7.35	0.4560	6.207	2.387	11	3	clay
17.72	9.75	0.3671	3.764	2.590	7	4	silty clay to clay
17.88	17.05	0.3769	2.211	2.069	7	5	clayey silt to silty clay
18.04	16.96	0.4410	2.601	0.713	8	5	clayey silt to silty clay
18.21	15.52	0.4704	3.032	-2.270	8	5	clayey silt to silty clay
18.37	16.18	0.5928	3.665	-4.747	9	5	clayey silt to silty clay
18.54	24.40	0.6391	2.619	-4.903	9	5	clayey silt to silty clay
18.70	18.60	0.4296	2.310	-4.982	10	5	clayey silt to silty clay
18.86	17.35	0.5250	3.026	-5.211	8	5	clayey silt to silty clay
19.03	13.50	0.3374	2.499	-4.915	8	5	clayey silt to silty clay
19.19	16.25	0.5193	3.196	-5.350	10	4	silty clay to clay
19.36	16.61	0.6489	3.907	-5.575	10	4	silty clay to clay
19.52	14.81	0.5465	3.692	-5.788	10	4	silty clay to clay
19.69	16.64	0.5156	3.099	-5.986	10	4	silty clay to clay
19.85	15.68	0.4487	2.862	-4.613	7	5	clayey silt to silty clay
20.01	13.71	0.3734	2.724	-9.571	7	5	clayey silt to silty clay
20.18	13.72	0.3430	2.501	-10.795	7	5	clayey silt to silty clay

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

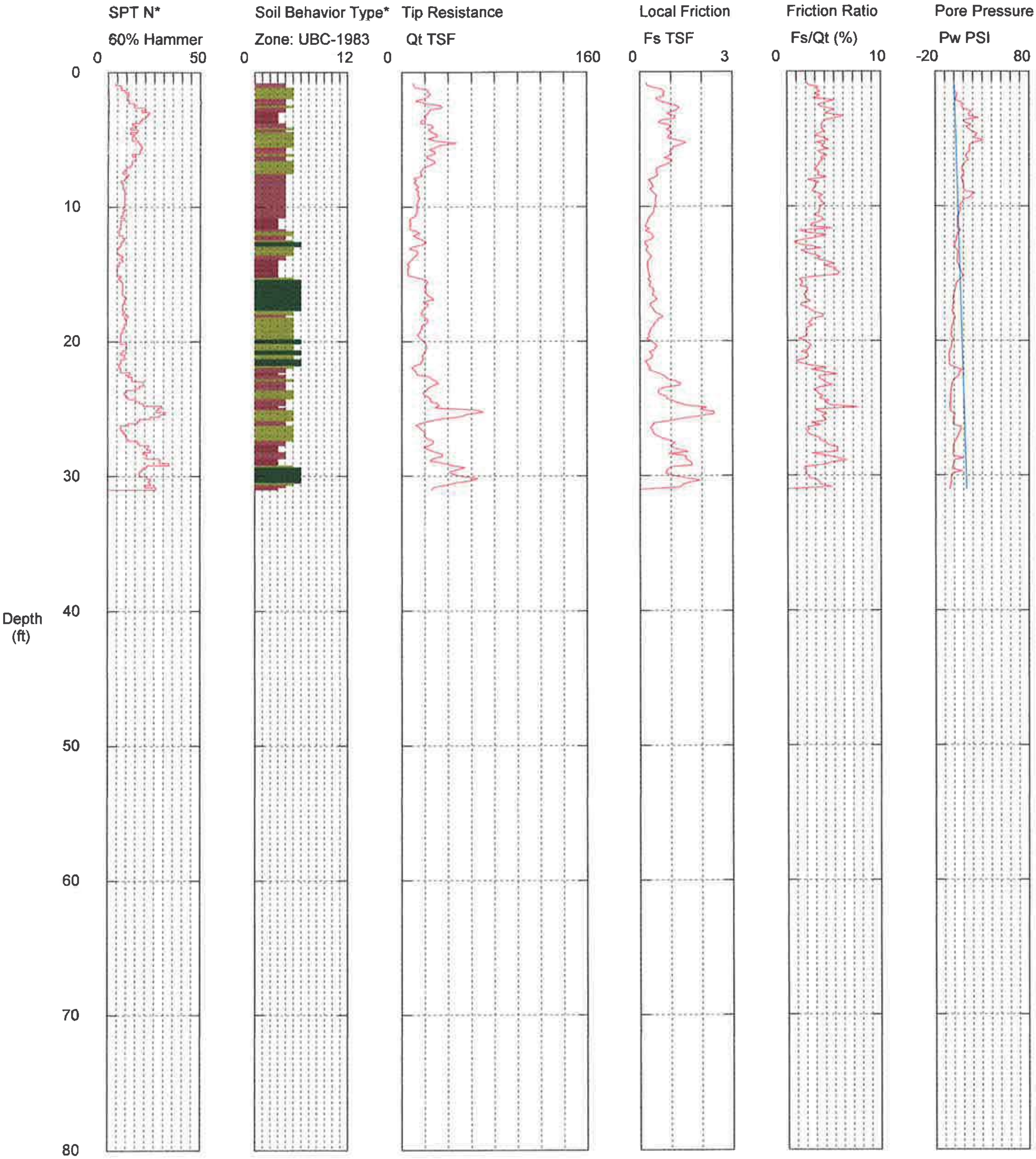
Depth (ft)	Qt TSF	Fs TSF	Fs/Qt (%)	Pw PSI	SPT N* 60% Hammer	Zone	Soil Behavior Type UBC-1983
20.34	15.60	0.2473	1.585	-10.702	7	5	clayey silt to silty clay
20.51	15.42	0.3298	2.139	-10.839	7	5	clayey silt to silty clay
20.67	13.19	0.4265	3.234	-10.793	9	4	silty clay to clay
20.83	15.41	0.5762	3.739	-10.736	14	3	clay
21.00	15.15	0.8576	5.661	-10.671	15	3	clay
21.16	17.92	0.9159	5.111	-8.964	16	3	clay
21.33	15.63	0.9678	6.190	13.675	16	3	clay
21.49	17.23	0.8952	5.195	17.281	14	3	clay
21.65	12.49	0.9550	7.645	17.896	15	3	clay
21.82	17.37	1.0607	6.107	24.860	17	3	clay
21.98	24.46	1.2127	4.959	21.043	21	3	clay
22.15	23.04	1.0183	4.420	9.758	19	3	clay
22.31	12.43	0.6085	4.895	9.076	15	3	clay
22.47	12.92	0.4771	3.693	-0.584	13	3	clay
22.64	16.22	0.6231	3.841	-2.667	15	3	clay
22.80	17.38	0.7723	4.444	-3.312	17	3	clay
22.97	19.05	0.8302	4.359	-5.532	20	3	clay
23.13	25.28	1.0266	4.060	-6.699	15	4	silty clay to clay
23.29	28.37	1.0835	3.820	-7.880	17	4	silty clay to clay
23.46	26.46	1.1860	4.482	-8.614	17	4	silty clay to clay
23.62	23.79	0.9328	3.920	-8.772	15	4	silty clay to clay
23.79	18.32	0.7865	4.294	-8.849	19	3	clay
23.95	16.66	0.8867	5.321	-8.696	18	3	clay
24.11	20.57	1.0912	5.305	-8.289	20	3	clay
24.28	24.95	1.4015	5.617	19.611	23	3	clay
24.44	25.41	1.7701	6.966	49.496	26	3	clay
24.61	29.90	1.5953	5.336	57.055	26	6	sandy silt to clayey silt
24.77	30.84	2.2100	7.166	8.887	33	6	sandy silt to clayey silt
24.93	143.66	-32768.0200	-22809.890	45.808	0	0	<out of range>

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

HDR ENG. / BC-2,CPT-6 / BIG CK. DAM NEWPORT

Operator: SAV/CM
Sounding: VEI434BC2SCPT6(488)
Cone Used: DSG0736

CPT Date/Time: 10/24/2013 8:42:45 PM
Location: BC2,SCPT6 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT



Maximum Depth = 31.00 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

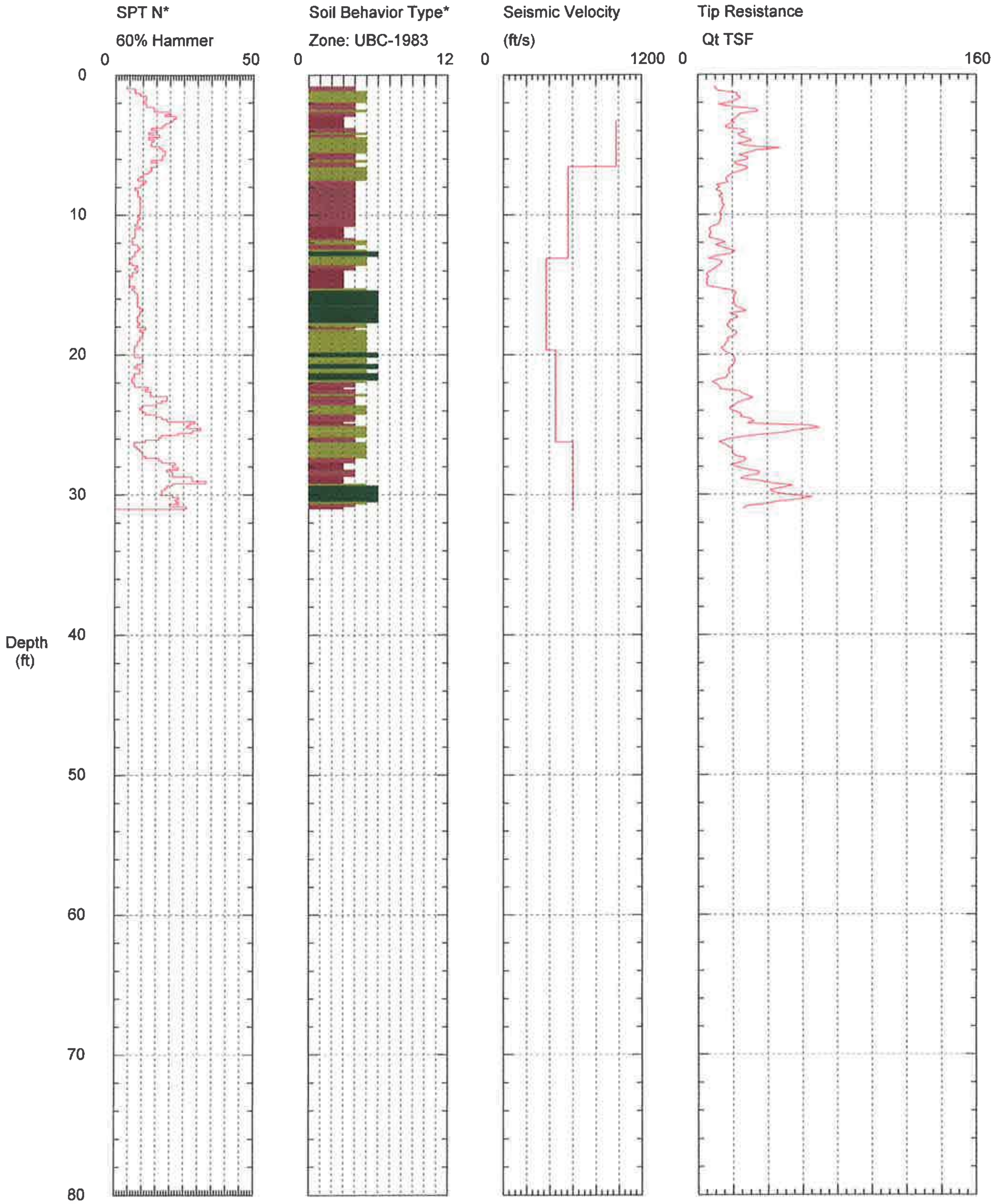
🟥 10 gravelly sand to sand
🟦 11 very stiff fine grained (*)
🟦 12 sand to clayey sand (*)

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

HDR ENG. / BC-2,CPT-6 / BIG CK. DAM NEWPORT

Operator: SAV/CM
Sounding: VEI434BC2SCPT6(488)
Cone Used: DSG0736

CPT Date/Time: 10/24/2013 8:42:45 PM
Location: BC2,SCPT6 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT



Maximum Depth = 31.00 feet

Depth Increment = 0.164 feet

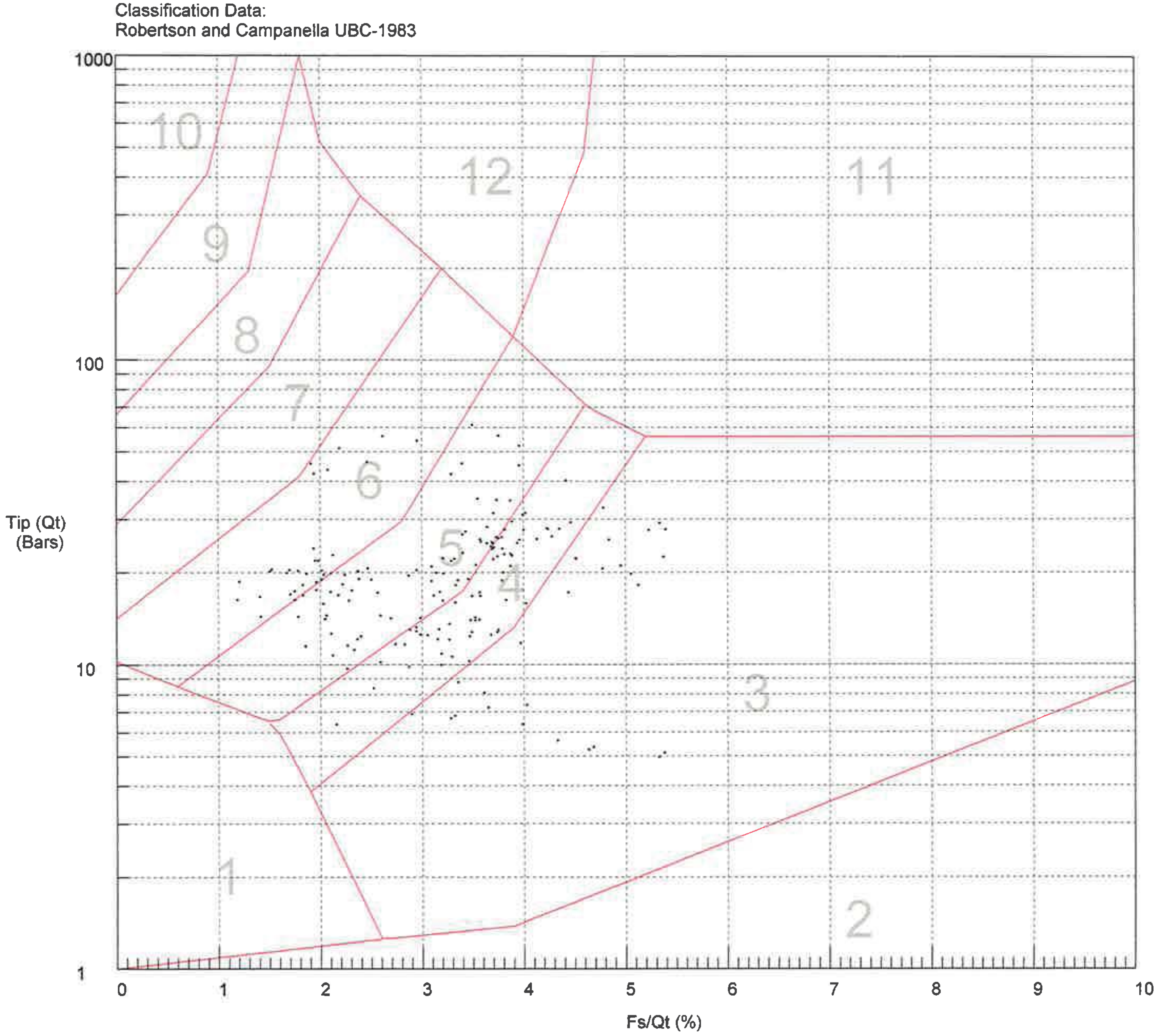
⬜ 1 sensitive fine grained
⬛ 2 organic material
🟥 3 clay
🟫 4 silty clay to clay
🟩 5 clayey silt to silty clay
🟩 6 sandy silt to clayey silt
🟦 7 silty sand to sandy silt
🟦 8 sand to silty sand
🟧 9 sand
🟥 10 gravelly sand to sand
🟦 11 very stiff fine grained (*)
🟦 12 sand to clayey sand (*)

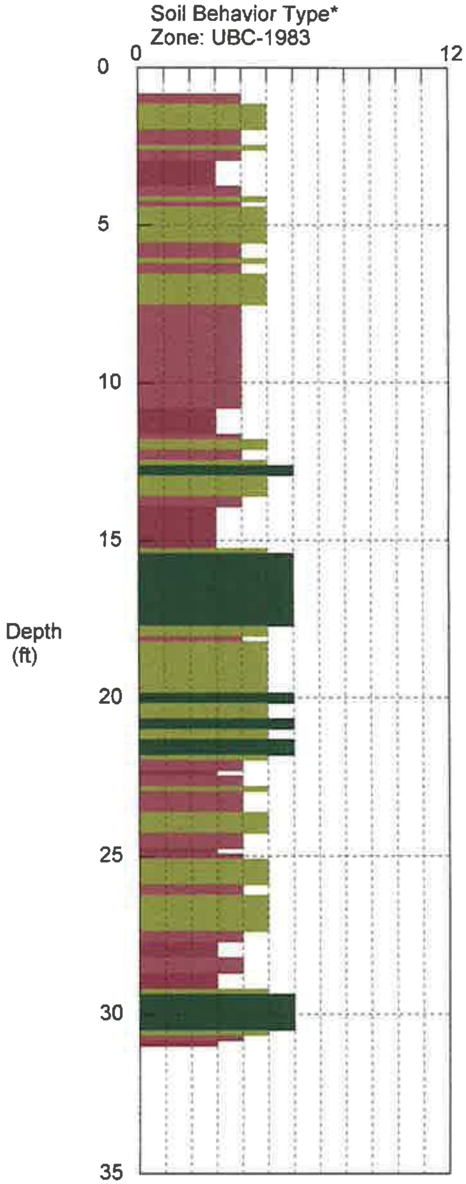
*Soil behavior type and SPT based on data from UBC-1983

HDR ENG. / BC-2,CPT-6 / BIG CK. DAM NEWPORT

Operator: SAV/CM
Sounding: VEI434BC2SCPT6(488)
Cone Used: DSG0736

CPT Date/Time: 10/24/2013 8:42:45 PM
Location: BC2,SCPT6 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT

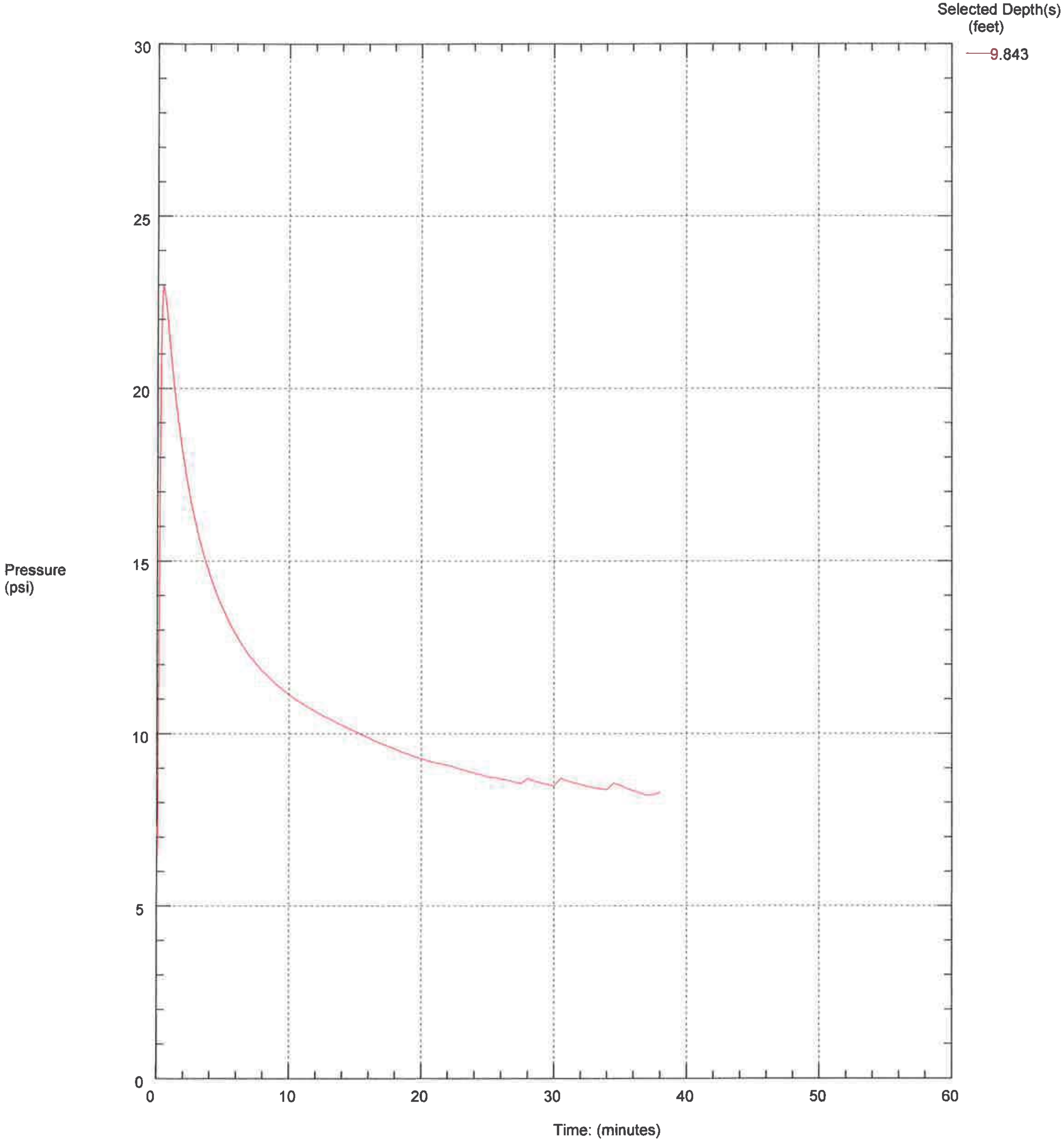




- 1 sensitive fine grained
- 2 organic material
- 3 clay
- 4 silty clay to clay
- 5 clayey silt to silty clay
- 6 sandy silt to clayey silt
- 7 silty sand to sandy silt
- 8 sand to silty sand
- 9 sand
- 10 gravelly sand to sand
- 11 very stiff fine grained (*)
- 12 sand to clayey sand (*)

HDR ENG. / BC-2CPT-6 / BIG CK. DAM NEWPORT

Operator SAV/CM	CPT Date/Time: 10/24/2013 8:42:45 PM
Sounding: VEI434BC2SCPT6(488)	Location: BC2SCPT6 / BIG CK. NEWPORT
Cone Used: DSG0736	Job Number: HDR ENG./BIG CK. NEWPORT

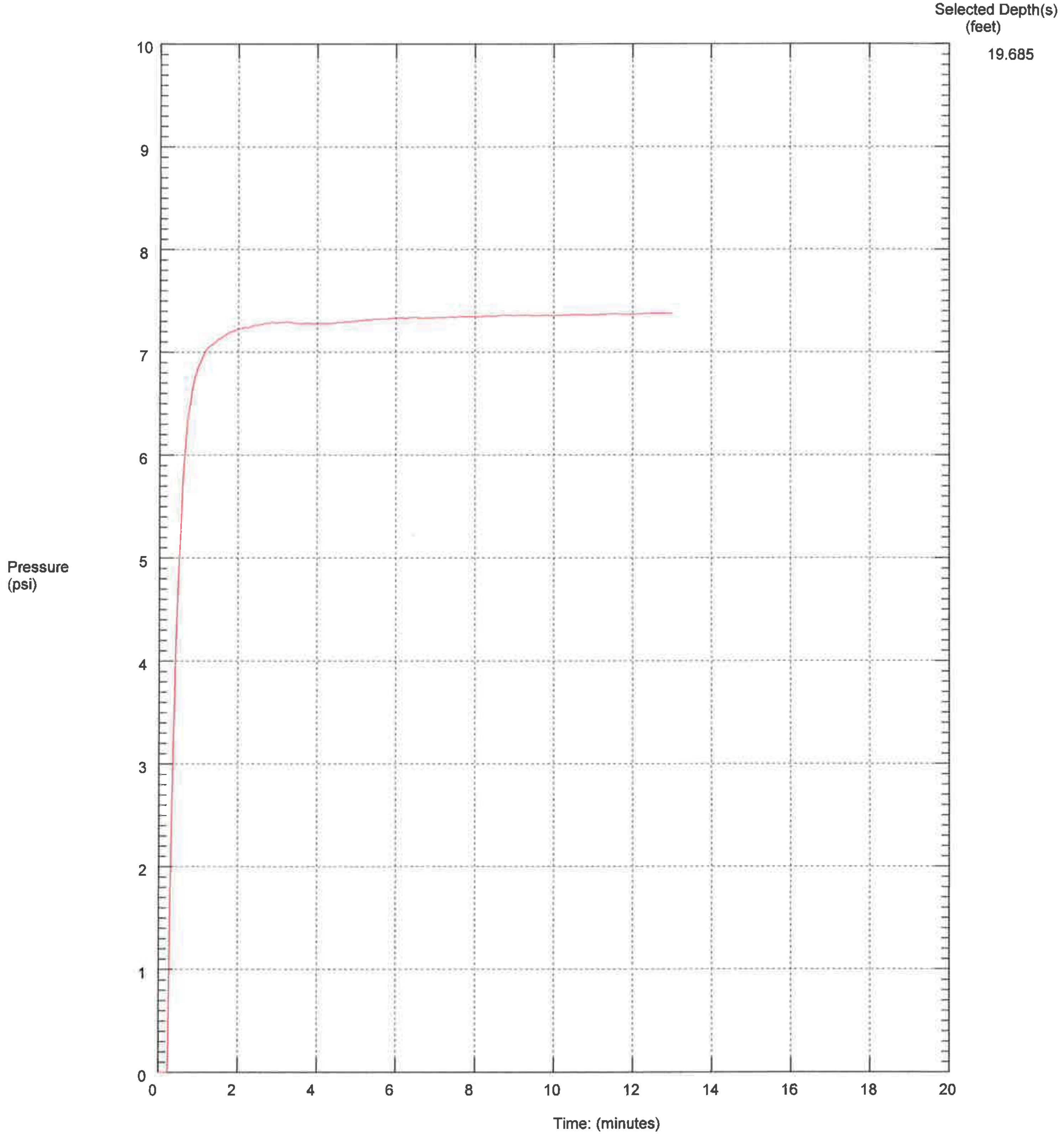


Maximum Pressure = 22.956 psi
Hydrostatic Pressure =

HDR ENG. / BC-2CPT-6 / BIG CK. DAM NEWPORT

Operator SAV/CM
Sounding: VEI434BC2SCPT6(488)
Cone Used: DSG0736

CPT Date/Time: 10/24/2013 8:42:45 PM
Location: BC2SCPT6 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT

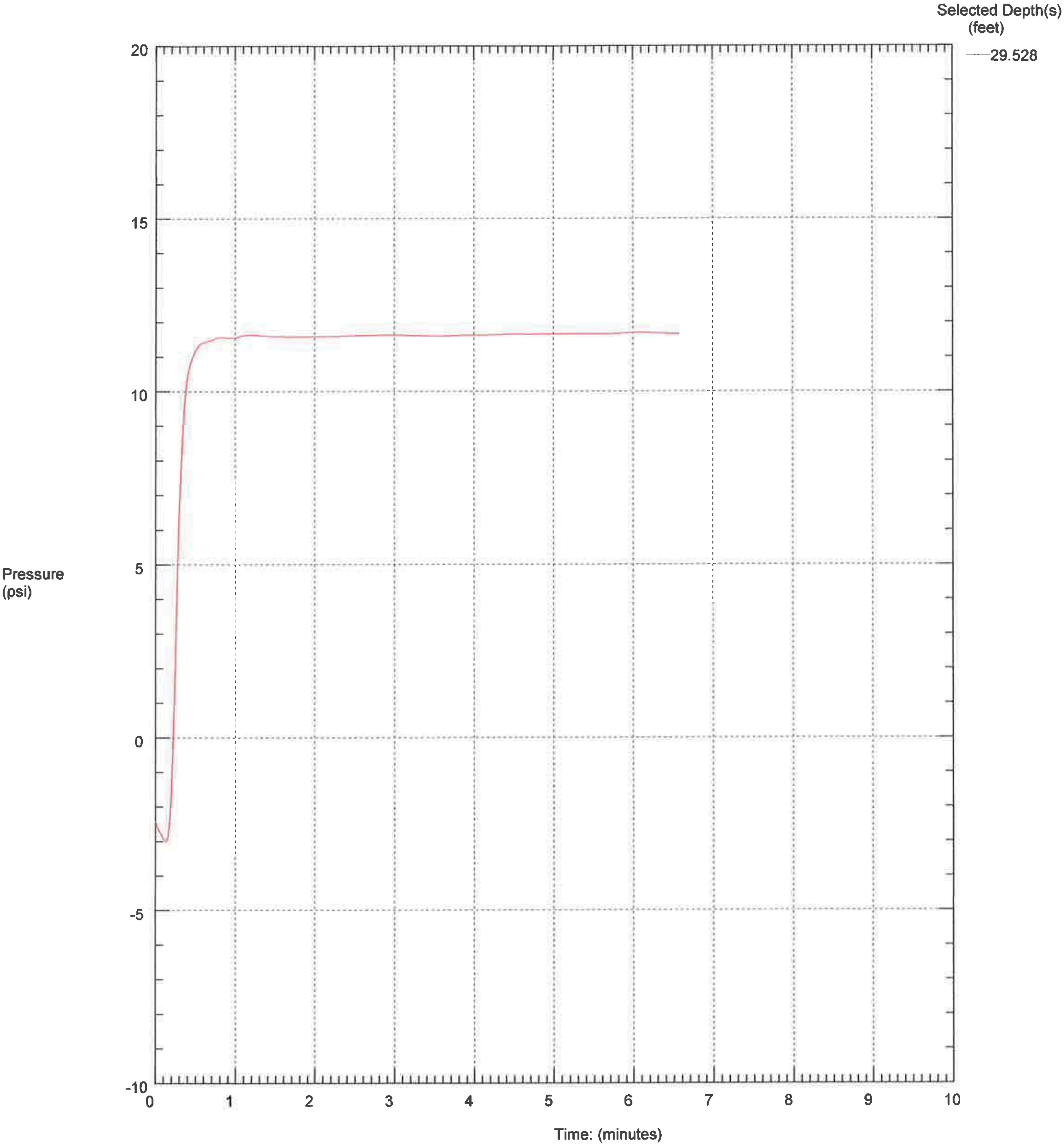


Maximum Pressure = 7.376 psi
Hydrostatic Pressure =

HDR ENG. / BC-2CPT-6 / BIG CK. DAM NEWPORT

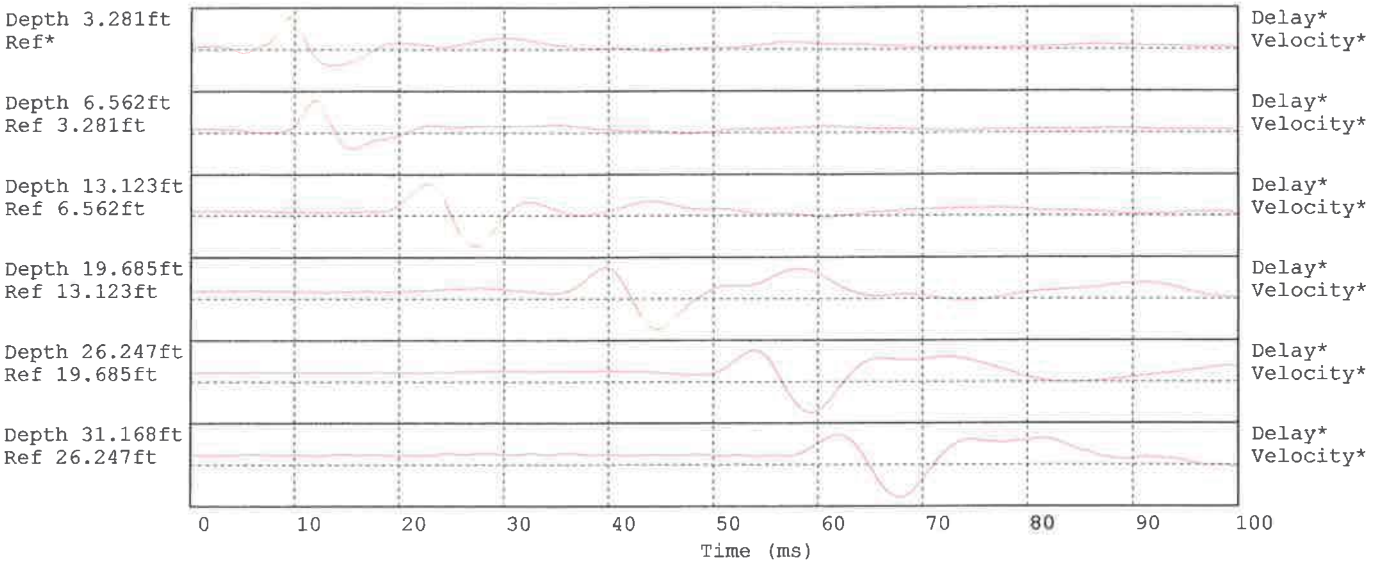
Operator SAV/CM
Sounding: VEI434BC2SCPT6(488)
Cone Used: DSG0736

CPT Date/Time: 10/24/2013 8:42:45 PM
Location: BC2SCPT6 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT



Maximum Pressure = 11.692 psi
Hydrostatic Pressure :

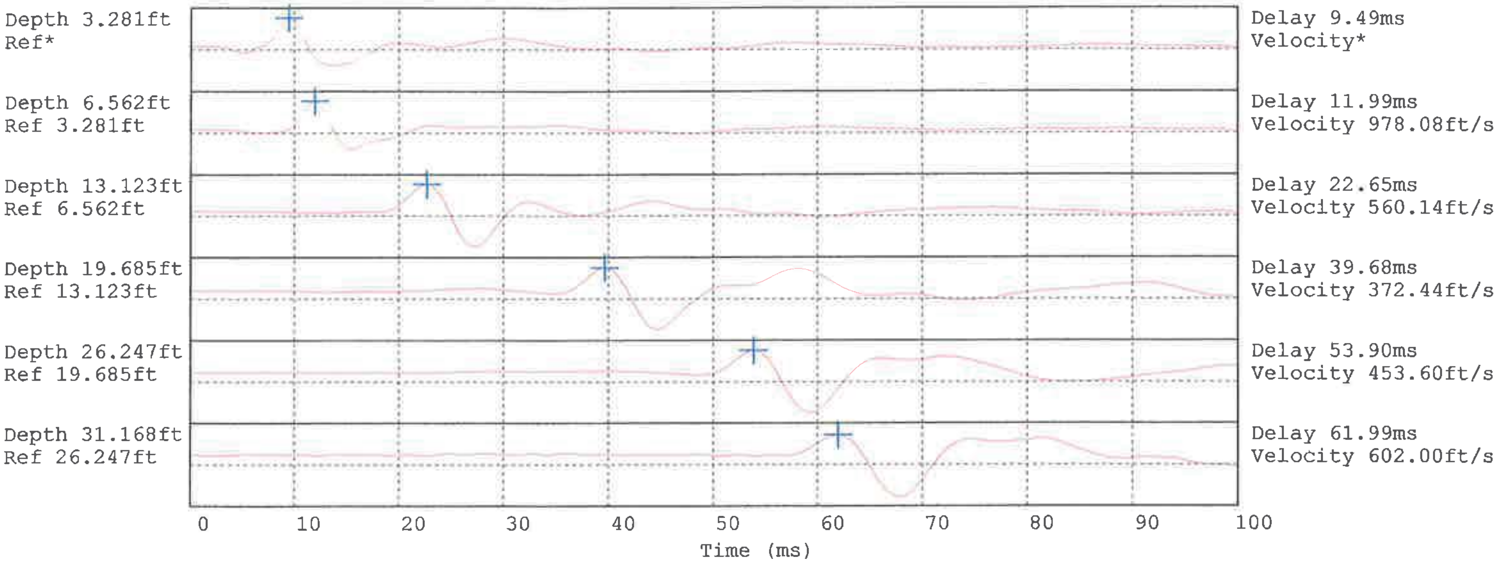
HDR ENG. / BC2SCPT6 / BIG CK. NEWPORT



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

THE SHEAR WAVE PICKS ARE COMPUTER GENERATED VALUES (STAFF REVIEW RECOMMENDED)

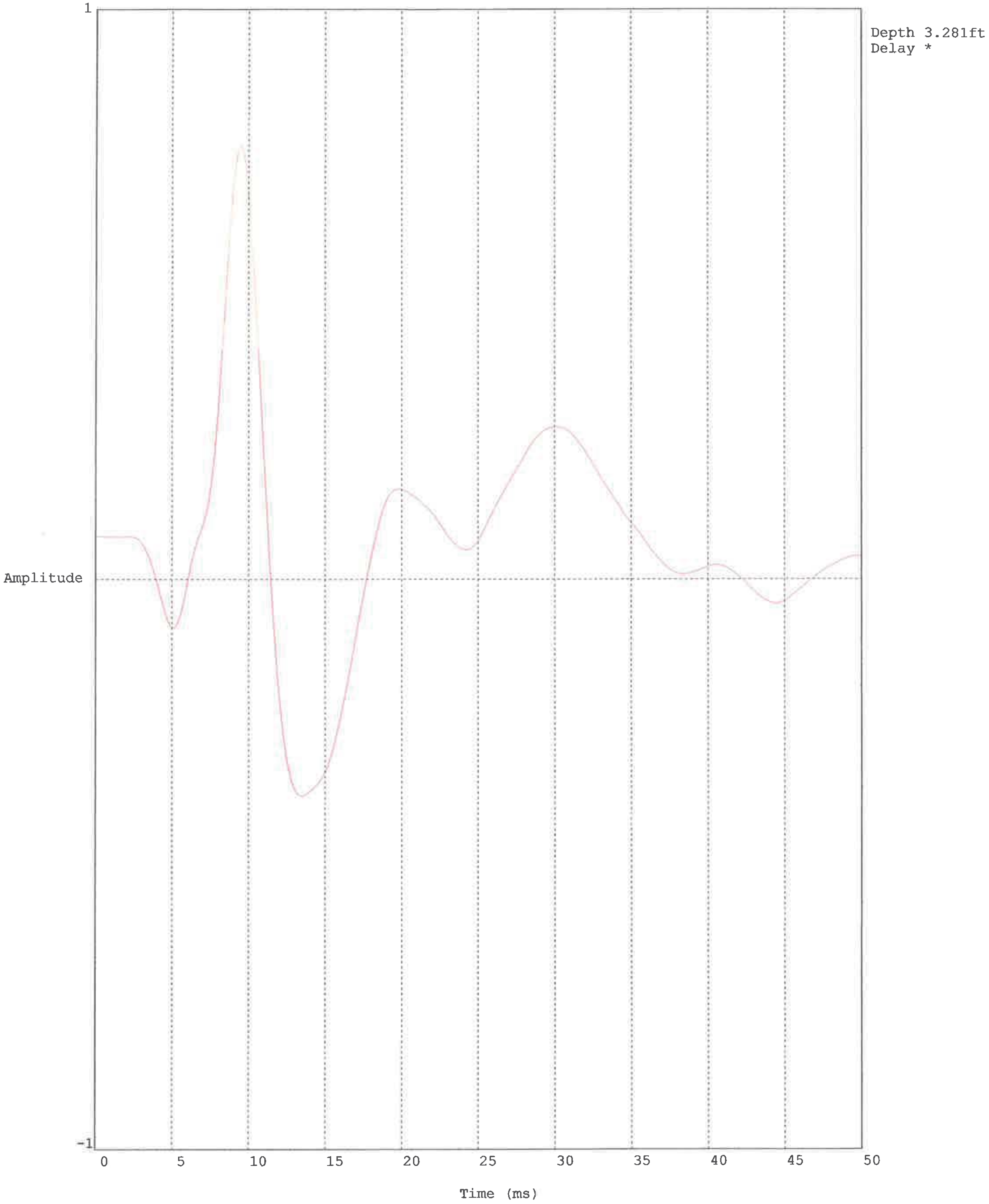
HDR ENG. / BC2SCPT6 / BIG CK. NEWPORT



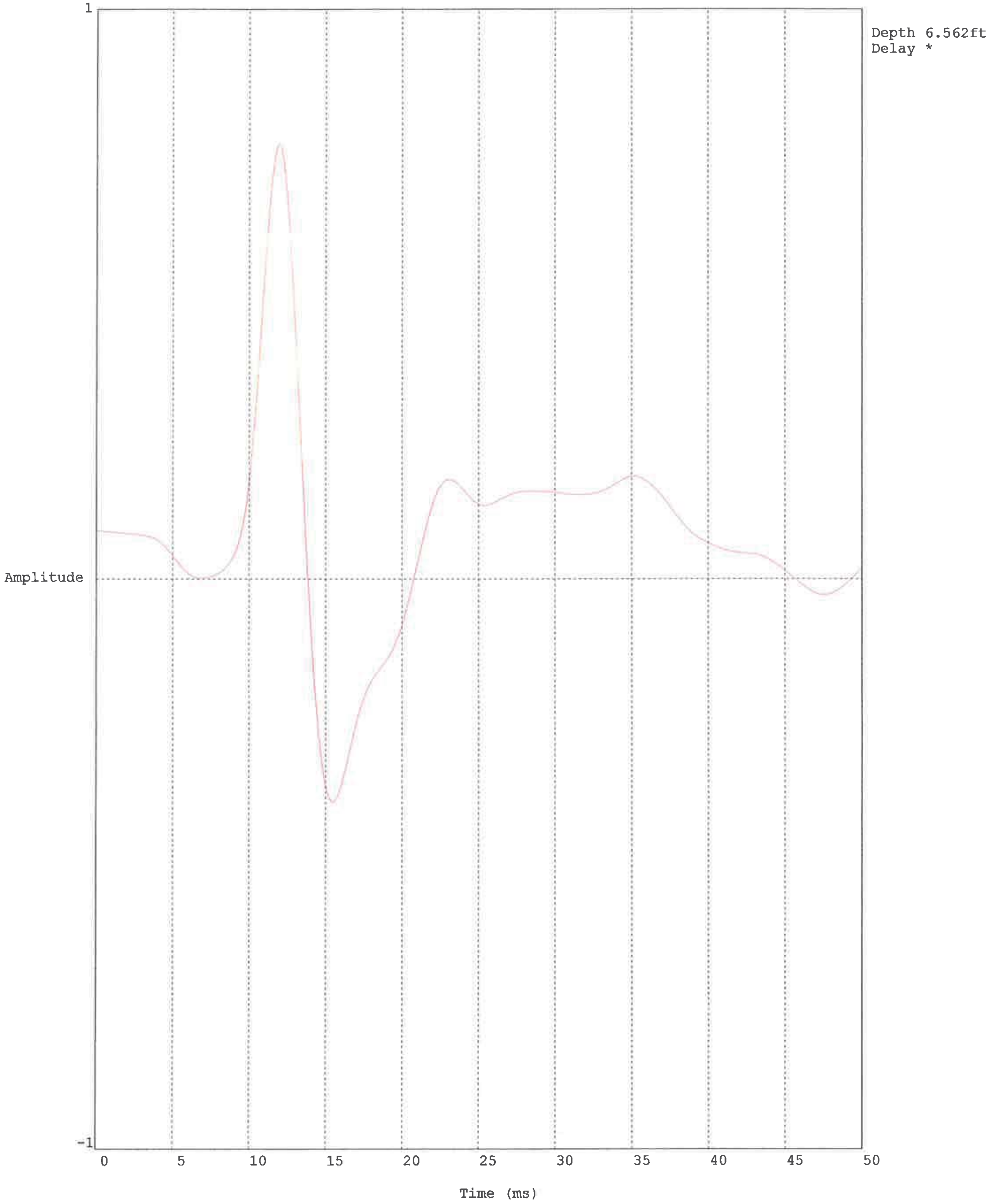
Hammer to Rod String Distance 1.3 (m)
* = Not Determined

THE SHEAR WAVE PICKS ARE COMPUTER GENERATED VALUES (STAFF REVIEW RECOMMENDED)

HDR ENG. / BC2SCPT6 / BIG CK. NEWPORT



HDR ENG. / BC2SCPT6 / BIG CK. NEWPORT



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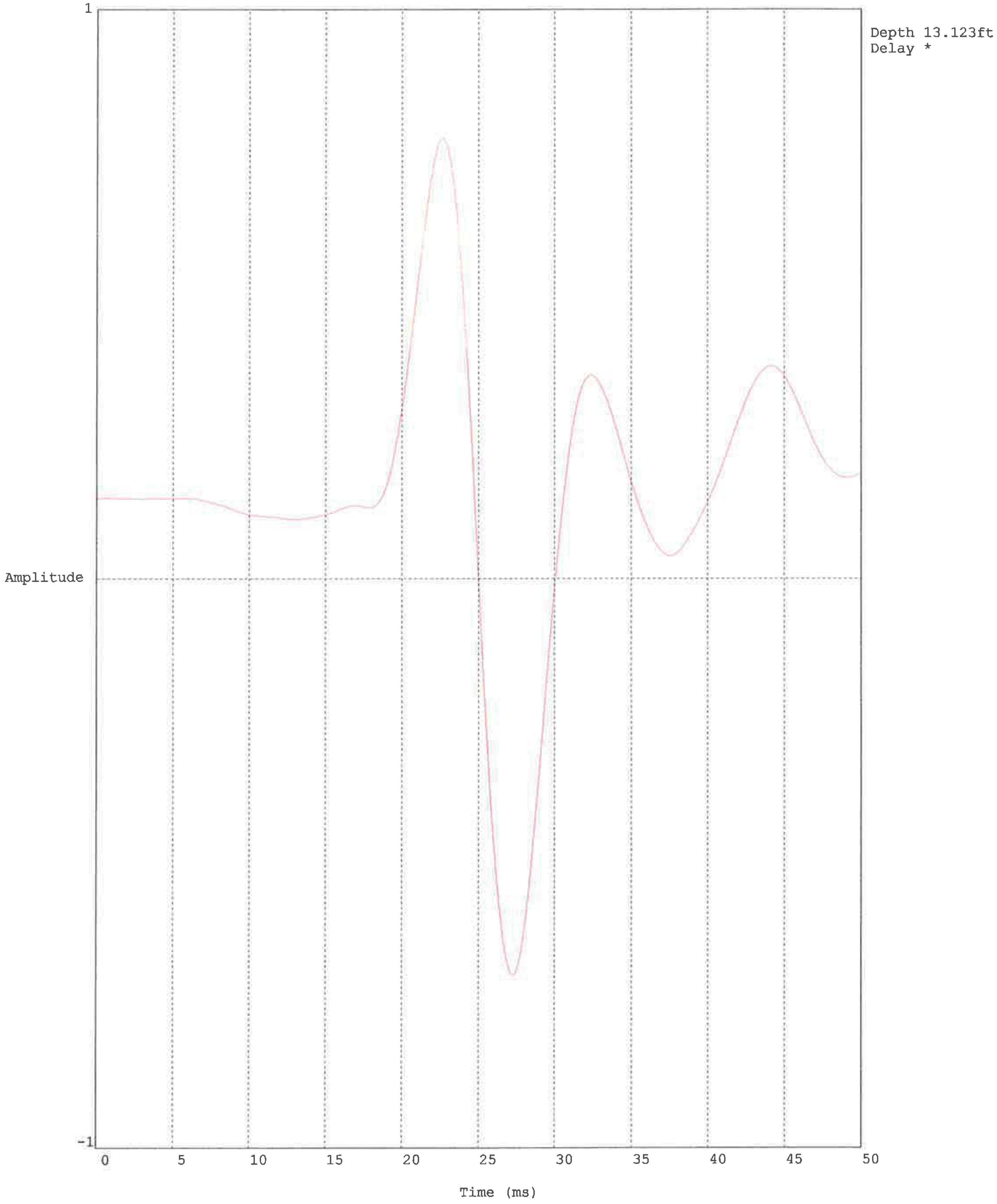

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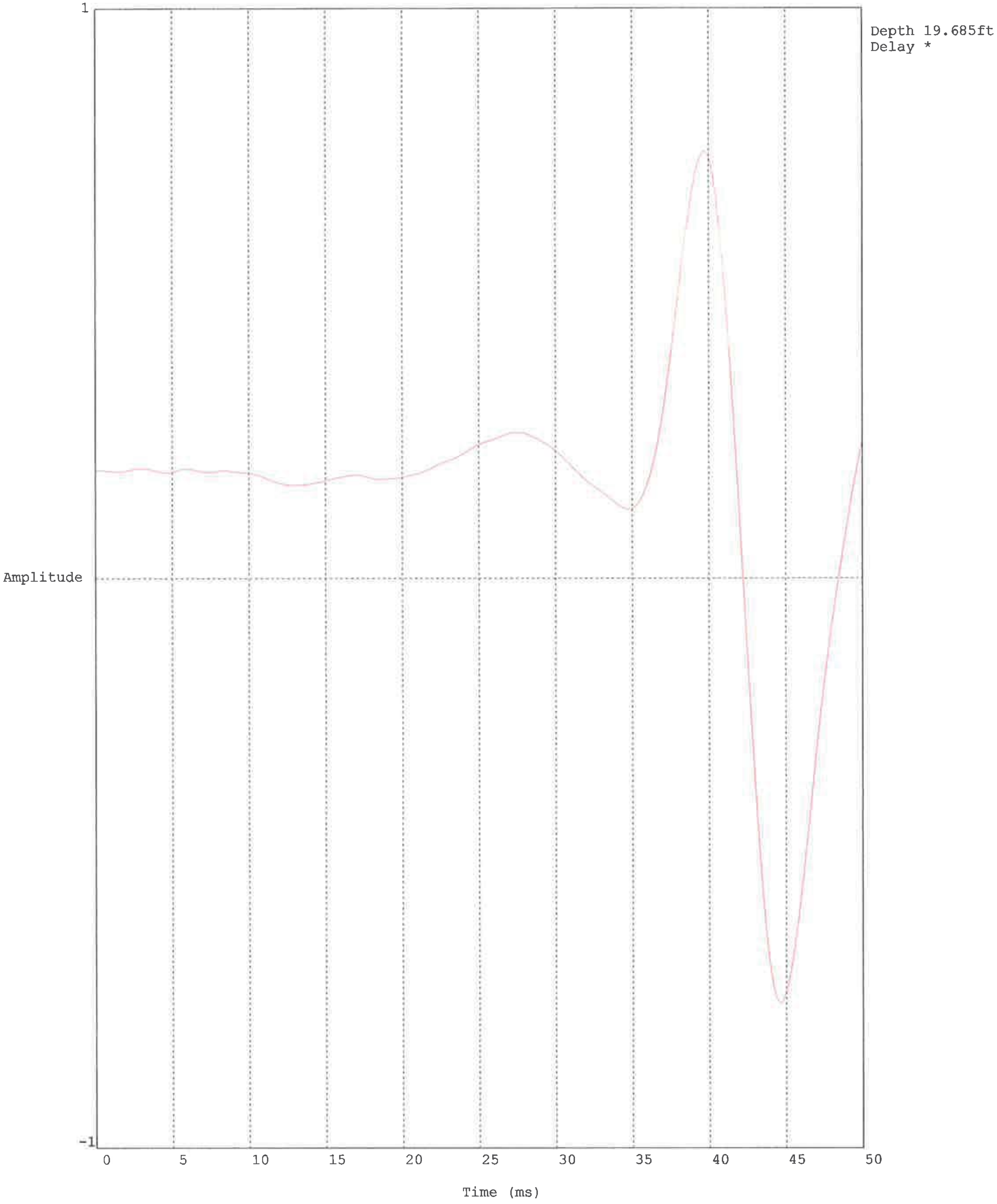

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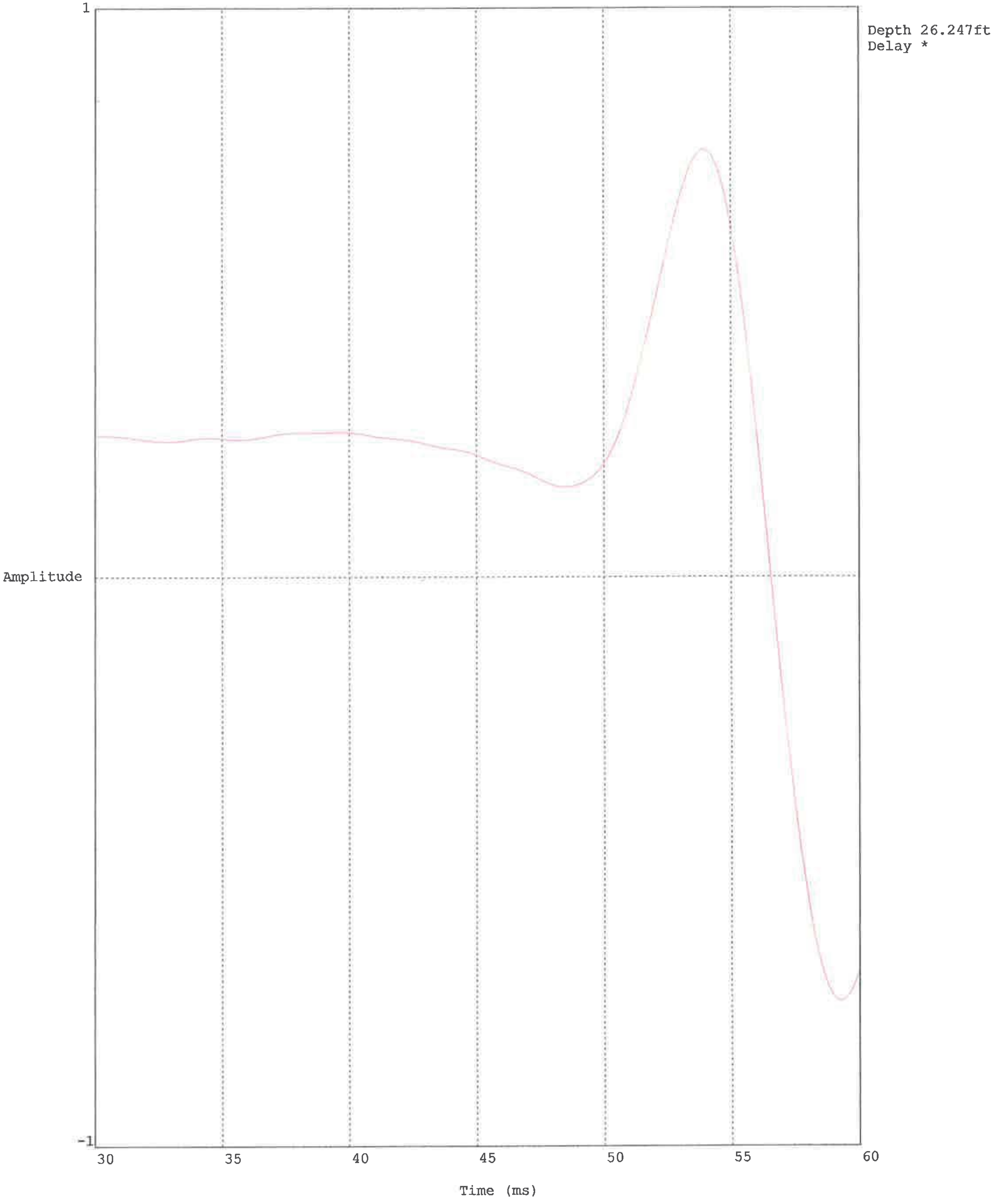

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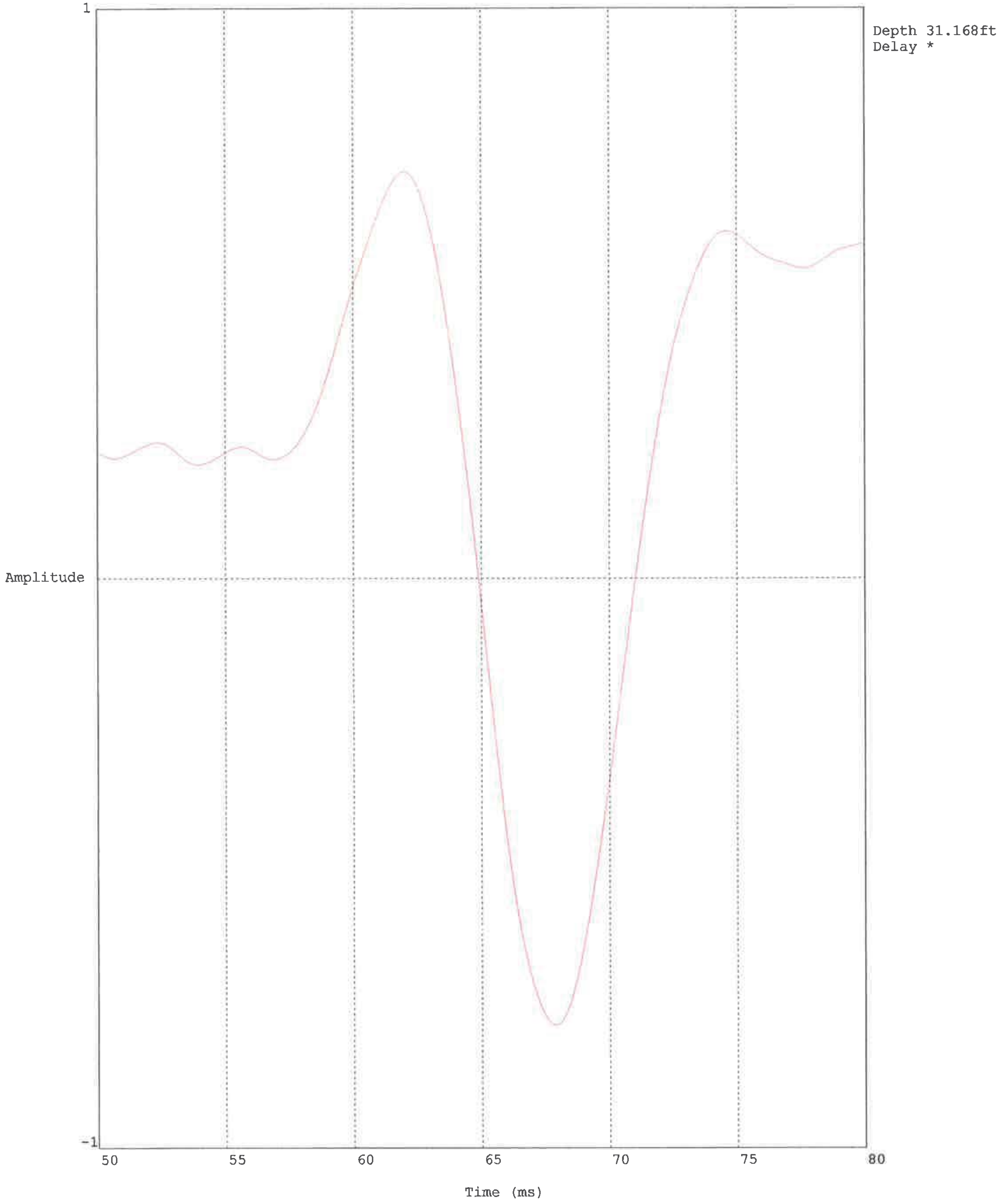

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Data File:VEI434BC2SCPT6(488)            10/24/2013 8:42:45 PM
 Operator:SAV/CM                           Location:BC2,SCPT6 / BIG CK. NEWPORT
  Cone ID:DSG0736                        Job Number:HDR ENG./BIG CK. NEWPORT
 Customer: BIG CK. DAM NEWPORT                  Units:

Depth          Qt         Fs         Fs/Qt        Pw    SPT N*            Soil Behavior Type
 (ft)         TSF        TSF           (%)       PSI 60% Hammer    Zone              UBC-1983

 0.82        9.98     0.2100        2.105     -0.038          4      4     silty clay to clay
 0.98       10.00     0.2200        2.200     -0.031          7      4     silty clay to clay
 1.15       12.00     0.3977        3.314      0.029          7      5  clayey silt to silty clay
 1.31       22.61     0.7125        3.151      0.067          9      5  clayey silt to silty clay
 1.48       22.55     0.7928        3.516      0.172         11      5  clayey silt to silty clay
 1.64       24.90     0.7402        2.972      4.035         11      5  clayey silt to silty clay
 1.80       22.32     0.7803        3.496      2.411         10      5  clayey silt to silty clay
 1.97       18.54     0.5143        2.774      1.985         11      4     silty clay to clay
 2.13       11.79     0.5857        4.966      1.131         11      4     silty clay to clay
 2.30       20.59     0.8473        4.115      7.997         14      4     silty clay to clay
 2.46       33.54     1.1179        3.333      8.959         14      5  clayey silt to silty clay
 2.62       34.68     1.2684        3.658      8.275         20      4     silty clay to clay
 2.79       24.24     1.2037        4.966     11.180         18      4     silty clay to clay
 2.95       23.64     1.0585        4.477     18.601         22      3            clay
 3.12       21.66     0.8765        4.046     15.526         21      3            clay
 3.28       19.25     1.1460        5.952     16.054         20      3            clay
 3.44       21.16     1.1147        5.268     26.008         18      3            clay
 3.61       16.74     0.6662        3.980     10.353         17      3            clay
 3.77       16.05     0.6120        3.814     11.743         13      4     silty clay to clay
 3.94       26.30     0.9591        3.646     18.503         15      4     silty clay to clay
 4.10       27.09     0.9979        3.683     12.881         12      5  clayey silt to silty clay
 4.27       23.26     0.8635        3.712     15.490         16      4     silty clay to clay
 4.43       24.99     0.9941        3.978     18.611         12      5  clayey silt to silty clay
 4.59       30.07     1.0370        3.448     24.609         14      5  clayey silt to silty clay
 4.76       30.83     0.9130        2.962     22.935         13      5  clayey silt to silty clay
 4.92       23.07     0.9016        3.908     21.196         13      5  clayey silt to silty clay
 5.09       28.28     1.2490        4.417     31.042         16      5  clayey silt to silty clay
 5.25       47.30     1.5020        3.175     22.222         17      5  clayey silt to silty clay
 5.41       33.51     1.3219        3.945     16.839         18      5  clayey silt to silty clay
 5.58       29.22     1.0798        3.696     17.013         18      4     silty clay to clay
 5.74       23.95     1.0659        4.450     15.956         17      4     silty clay to clay
 5.91       28.98     0.9650        3.329     17.188         17      4     silty clay to clay
 6.07       28.52     1.0261        3.598     14.354         13      5  clayey silt to silty clay
 6.23       21.56     0.9227        4.280     12.639         15      4     silty clay to clay
 6.40       22.01     0.7971        3.621     13.909         15      4     silty clay to clay
 6.56       27.90     1.0572        3.789     15.339         13      5  clayey silt to silty clay
 6.73       28.60     1.0024        3.504     11.984         13      5  clayey silt to silty clay
 6.89       24.31     0.8263        3.399     10.291         12      5  clayey silt to silty clay
 7.05       19.95     0.6484        3.251      9.050         10      5  clayey silt to silty clay
 7.22       18.11     0.5274        2.913      8.320          9      5  clayey silt to silty clay
 7.38       16.13     0.5425        3.363      9.117          8      5  clayey silt to silty clay
 7.55       16.55     0.5562        3.361     10.781         11      4     silty clay to clay
 7.71       17.44     0.5682        3.258      9.229         10      4     silty clay to clay
 7.87       10.72     0.4497        4.197      8.158          9      4     silty clay to clay
 8.04       12.76     0.2767        2.168      8.727          7      4     silty clay to clay
 8.20       10.57     0.3592        3.398      8.736          8      4     silty clay to clay
 8.37       13.10     0.3937        3.004     10.497          8      4     silty clay to clay
 8.53       14.25     0.4425        3.106     10.269          8      4     silty clay to clay
 8.69       11.98     0.3366        2.809      9.748          9      4     silty clay to clay
 8.86       14.49     0.3930        2.712     11.331          9      4     silty clay to clay
 9.02       13.90     0.4579        3.294     22.696          9      4     silty clay to clay
 9.19       14.21     0.5408        3.806     18.298          9      4     silty clay to clay
 9.35       15.66     0.5225        3.335     17.396          9      4     silty clay to clay
 9.51       14.12     0.5042        3.570     10.618          9      4     silty clay to clay
 9.68       13.92     0.5129        3.683      9.294          9      4     silty clay to clay
 9.84       12.63     0.5079        4.023      7.368          9      4     silty clay to clay
10.01       13.52     0.4768        3.527      5.596          8      4     silty clay to clay

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

Depth (ft)	Qt TSF	Fs TSF	Fs/Qt (%)	Pw PSI	SPT N* 60% Hammer	Zone	Soil Behavior Type UBC-1983
10.17	13.07	0.4573	3.498	6.998	9	4	silty clay to clay
10.33	13.42	0.4607	3.434	6.488	8	4	silty clay to clay
10.50	12.38	0.4295	3.469	5.321	8	4	silty clay to clay
10.66	12.09	0.3466	2.866	4.467	7	4	silty clay to clay
10.83	8.83	0.3197	3.620	3.296	9	3	clay
10.99	6.58	0.2554	3.878	3.408	7	3	clay
11.15	7.28	0.2531	3.475	4.042	7	3	clay
11.32	7.46	0.1991	2.669	3.650	7	3	clay
11.48	6.92	0.1747	2.523	3.468	7	3	clay
11.65	6.54	0.3121	4.774	4.286	6	4	silty clay to clay
11.81	12.79	0.1747	1.366	7.309	6	5	clayey silt to silty clay
11.98	15.76	0.3299	2.094	3.805	6	5	clayey silt to silty clay
12.14	10.24	0.4251	4.152	3.810	8	4	silty clay to clay
12.30	13.13	0.4394	3.346	2.502	9	4	silty clay to clay
12.47	18.17	0.3574	1.967	2.509	8	5	clayey silt to silty clay
12.63	21.49	0.1693	0.788	1.741	7	6	sandy silt to clayey silt
12.80	18.90	0.1784	0.944	1.098	7	6	sandy silt to clayey silt
12.96	10.77	0.2577	2.393	0.512	6	5	clayey silt to silty clay
13.12	6.31	0.2300	3.648	1.566	5	5	clayey silt to silty clay
13.29	13.38	0.1999	1.494	4.116	5	5	clayey silt to silty clay
13.45	13.99	0.2835	2.026	3.810	6	5	clayey silt to silty clay
13.62	12.25	0.3500	2.857	3.597	8	4	silty clay to clay
13.78	10.33	0.3664	3.547	3.661	7	4	silty clay to clay
13.94	8.76	0.2814	3.214	3.245	8	3	clay
14.11	6.34	0.2690	4.245	3.229	6	3	clay
14.27	4.90	0.2454	5.013	4.245	6	3	clay
14.44	6.42	0.2498	3.891	5.520	5	3	clay
14.60	5.13	0.2667	5.198	5.948	5	3	clay
14.76	5.23	0.2691	5.143	6.857	5	3	clay
14.93	5.22	0.2939	5.633	7.976	5	3	clay
15.09	5.64	0.3016	5.352	8.954	7	3	clay
15.26	12.25	0.3344	2.730	9.896	6	5	clayey silt to silty clay
15.42	20.12	0.2612	1.298	5.611	7	6	sandy silt to clayey silt
15.58	22.20	0.3575	1.610	4.166	8	6	sandy silt to clayey silt
15.75	21.02	0.3345	1.591	2.030	8	6	sandy silt to clayey silt
15.91	20.95	0.2853	1.361	1.923	8	6	sandy silt to clayey silt
16.08	20.61	0.4254	2.064	1.447	8	6	sandy silt to clayey silt
16.24	20.69	0.4492	2.171	0.868	8	6	sandy silt to clayey silt
16.40	21.14	0.4372	2.068	0.163	8	6	sandy silt to clayey silt
16.57	21.94	0.3976	1.813	-0.457	9	6	sandy silt to clayey silt
16.73	25.67	0.5044	1.964	-0.770	10	6	sandy silt to clayey silt
16.90	27.75	0.5555	2.002	-0.921	9	6	sandy silt to clayey silt
17.06	18.32	0.4614	2.519	-0.084	9	6	sandy silt to clayey silt
17.22	22.57	0.3413	1.512	-0.772	8	6	sandy silt to clayey silt
17.39	22.70	0.3286	1.448	-0.966	8	6	sandy silt to clayey silt
17.55	18.65	0.4140	2.219	-1.332	8	6	sandy silt to clayey silt
17.72	18.17	0.4546	2.502	-1.490	9	5	clayey silt to silty clay
17.88	17.03	0.5104	2.997	-1.765	8	5	clayey silt to silty clay
18.04	17.51	0.6764	3.862	-1.091	11	4	silty clay to clay
18.21	19.39	0.7368	3.800	1.158	9	5	clayey silt to silty clay
18.37	22.30	0.5714	2.562	0.990	10	5	clayey silt to silty clay
18.54	22.27	0.5764	2.588	-0.631	10	5	clayey silt to silty clay
18.70	20.17	0.4494	2.228	-0.758	9	5	clayey silt to silty clay
18.86	16.95	0.4586	2.705	-1.966	9	5	clayey silt to silty clay
19.03	17.72	0.3576	2.018	-2.533	8	5	clayey silt to silty clay
19.19	16.05	0.3401	2.119	-2.836	8	5	clayey silt to silty clay
19.36	15.78	0.3082	1.953	-2.992	7	5	clayey silt to silty clay
19.52	13.59	0.2870	2.111	-3.092	7	5	clayey silt to silty clay
19.69	14.82	0.3089	2.085	-3.133	7	5	clayey silt to silty clay
19.85	16.53	0.2049	1.240	-0.122	7	6	sandy silt to clayey silt
20.01	19.72	0.3795	1.925	-1.743	7	6	sandy silt to clayey silt
20.18	20.95	0.4604	2.197	-2.573	10	5	clayey silt to silty clay

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

Depth (ft)	Qt TSF	Fs TSF	Fs/Qt (%)	Pw PSI	SPT N* 60% Hammer	Zone	Soil Behavior Type UBC-1983
20.34	21.09	0.5424	2.571	-2.616	10	5	clayey silt to silty clay
20.51	21.39	0.4989	2.332	-3.138	10	5	clayey silt to silty clay
20.67	20.89	0.4628	2.215	-4.295	8	6	sandy silt to clayey silt
20.83	19.38	0.2910	1.502	-4.840	7	6	sandy silt to clayey silt
21.00	18.09	0.3896	2.153	-5.218	9	5	clayey silt to silty clay
21.16	17.62	0.3981	2.259	-5.168	9	5	clayey silt to silty clay
21.33	18.53	0.3514	1.897	-5.080	7	6	sandy silt to clayey silt
21.49	17.15	0.1583	0.923	-5.061	7	6	sandy silt to clayey silt
21.65	16.66	0.2263	1.358	-5.135	6	6	sandy silt to clayey silt
21.82	11.19	0.2512	2.245	-4.888	6	5	clayey silt to silty clay
21.98	8.42	0.3436	4.083	0.732	7	4	silty clay to clay
22.15	11.25	0.2900	2.576	8.251	7	4	silty clay to clay
22.31	12.54	0.4809	3.834	5.589	12	3	clay
22.47	13.08	0.6920	5.293	5.634	11	4	silty clay to clay
22.64	24.03	0.8231	3.425	4.372	13	4	silty clay to clay
22.80	25.76	0.8729	3.389	0.918	13	5	clayey silt to silty clay
22.97	29.58	1.1445	3.869	-1.074	19	4	silty clay to clay
23.13	31.66	1.3003	4.107	-1.806	19	4	silty clay to clay
23.29	26.69	1.2721	4.767	-2.227	17	4	silty clay to clay
23.46	22.58	0.7698	3.409	-2.896	15	4	silty clay to clay
23.62	21.71	0.6133	2.824	-3.312	10	5	clayey silt to silty clay
23.79	18.41	0.5856	3.180	-3.561	9	5	clayey silt to silty clay
23.95	18.87	0.6250	3.312	-3.743	10	5	clayey silt to silty clay
24.11	22.50	0.8540	3.795	-3.803	11	5	clayey silt to silty clay
24.28	25.10	0.8663	3.452	-3.905	15	4	silty clay to clay
24.44	24.66	1.0764	4.366	-4.001	17	4	silty clay to clay
24.61	30.66	1.2403	4.045	-4.109	19	4	silty clay to clay
24.77	32.18	1.4880	4.625	-4.290	29	3	clay
24.93	28.60	2.1490	7.514	-4.470	27	4	silty clay to clay
25.10	65.52	1.9403	2.961	-4.281	26	5	clayey silt to silty clay
25.26	70.11	2.4085	3.436	-3.587	31	5	clayey silt to silty clay
25.43	56.66	2.3711	4.185	0.725	28	5	clayey silt to silty clay
25.59	50.16	1.8599	3.708	-0.067	23	5	clayey silt to silty clay
25.75	34.85	1.3714	3.936	-0.761	17	5	clayey silt to silty clay
25.92	23.63	0.9696	4.104	-0.882	16	4	silty clay to clay
26.08	16.58	0.4282	2.583	-0.959	11	4	silty clay to clay
26.25	12.33	0.4360	3.537	-1.146	7	5	clayey silt to silty clay
26.41	15.55	0.3317	2.133	7.804	7	5	clayey silt to silty clay
26.57	17.53	0.4078	2.327	6.785	8	5	clayey silt to silty clay
26.74	19.89	0.4128	2.075	6.240	9	5	clayey silt to silty clay
26.90	20.01	0.4532	2.265	5.010	10	5	clayey silt to silty clay
27.07	19.93	0.5573	2.797	3.925	10	5	clayey silt to silty clay
27.23	21.44	0.7494	3.496	3.059	11	5	clayey silt to silty clay
27.40	27.19	0.9450	3.475	2.052	16	4	silty clay to clay
27.56	27.38	1.1209	4.094	0.335	17	4	silty clay to clay
27.72	24.00	1.0263	4.276	-0.808	22	3	clay
27.89	18.60	1.0092	5.425	-1.062	21	3	clay
28.05	23.50	1.2385	5.270	-0.871	23	3	clay
28.22	28.58	1.5484	5.418	-0.835	19	4	silty clay to clay
28.38	35.30	0.9152	2.593	-1.002	21	4	silty clay to clay
28.54	34.89	1.5011	4.302	-1.299	21	4	silty clay to clay
28.71	27.22	1.4711	5.404	9.815	28	3	clay
28.87	24.53	1.5587	6.353	0.514	28	3	clay
29.04	35.43	1.6703	4.715	-0.198	33	3	clay
29.20	42.86	1.6869	3.936	-0.832	21	5	clayey silt to silty clay
29.36	54.32	1.0016	1.844	-1.679	19	6	sandy silt to clayey silt
29.53	48.06	0.8872	1.846	-2.423	18	6	sandy silt to clayey silt
29.69	41.36	0.8540	2.065	8.203	17	6	sandy silt to clayey silt
29.86	43.79	0.8403	1.919	-1.839	17	6	sandy silt to clayey silt
30.02	51.76	1.1446	2.211	-2.679	21	6	sandy silt to clayey silt
30.18	65.58	1.5381	2.345	-2.743	23	6	sandy silt to clayey silt
30.35	59.10	1.9291	3.264	-2.997	22	6	sandy silt to clayey silt

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

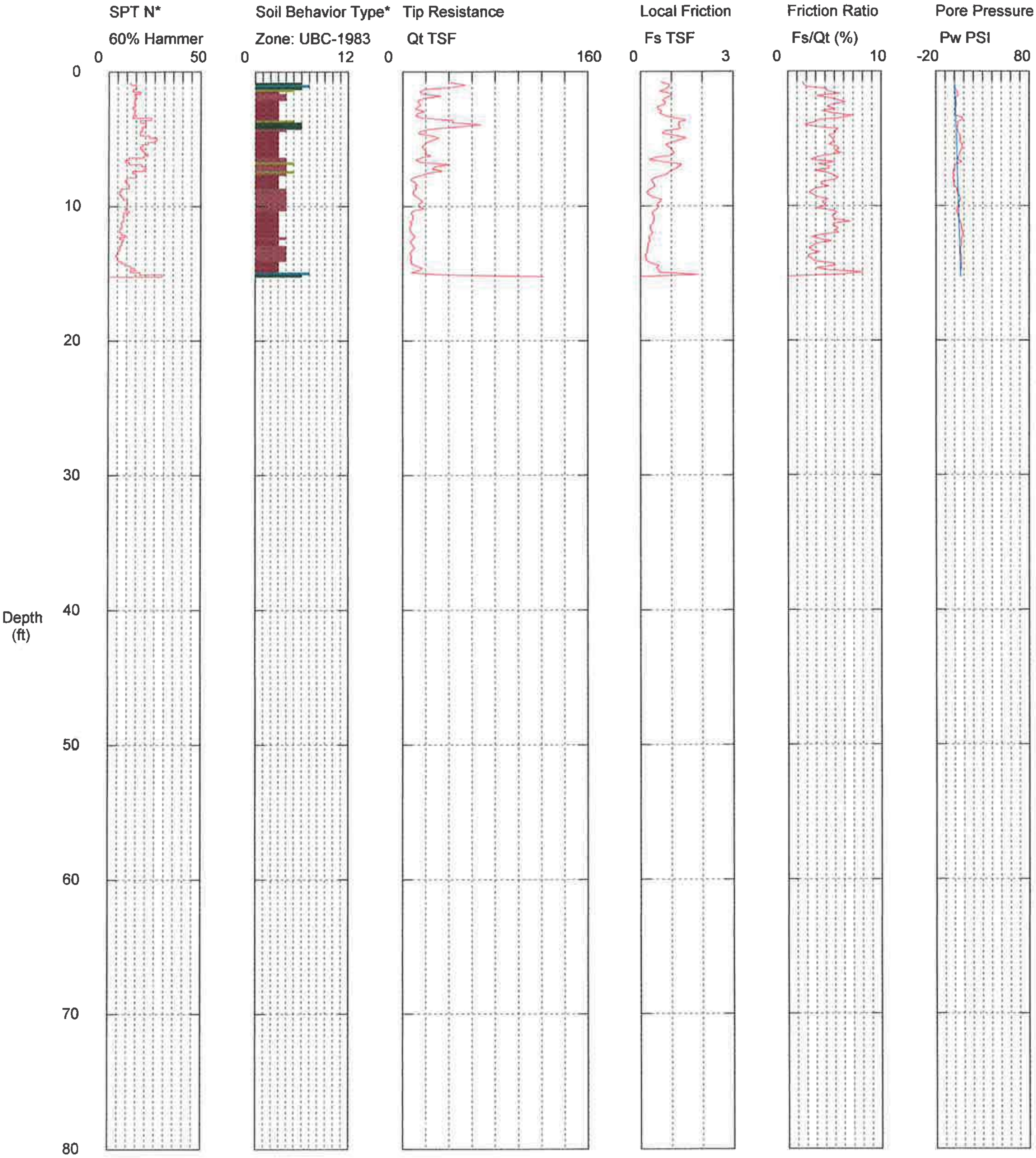
Depth (ft)	Qt TSF	Fs TSF	Fs/Qt (%)	Pw PSI	SPT N* 60% Hammer	Zone	Soil Behavior Type UBC-1983
30.51	46.31	1.5795	3.411	-3.415	23	5	clayey silt to silty clay
30.68	38.21	1.3659	3.575	-3.645	20	4	silty clay to clay
30.84	27.56	1.2969	4.706	-4.070	26	3	clay
31.00	26.10	-32768	-32768	-4.812	0	0	<out of range>

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

HDR ENG. / BC-2,SCPT-7 / BIG CK. DAM NEWPORT

Operator: SAV/CM
Sounding: VEI434BC2SCPT7(489)
Cone Used: DSG0736

CPT Date/Time: 10/24/2013 11:30:31 PM
Location: BC2,SCPT7 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT



Maximum Depth = 15.26 feet

Depth Increment = 0.164 feet

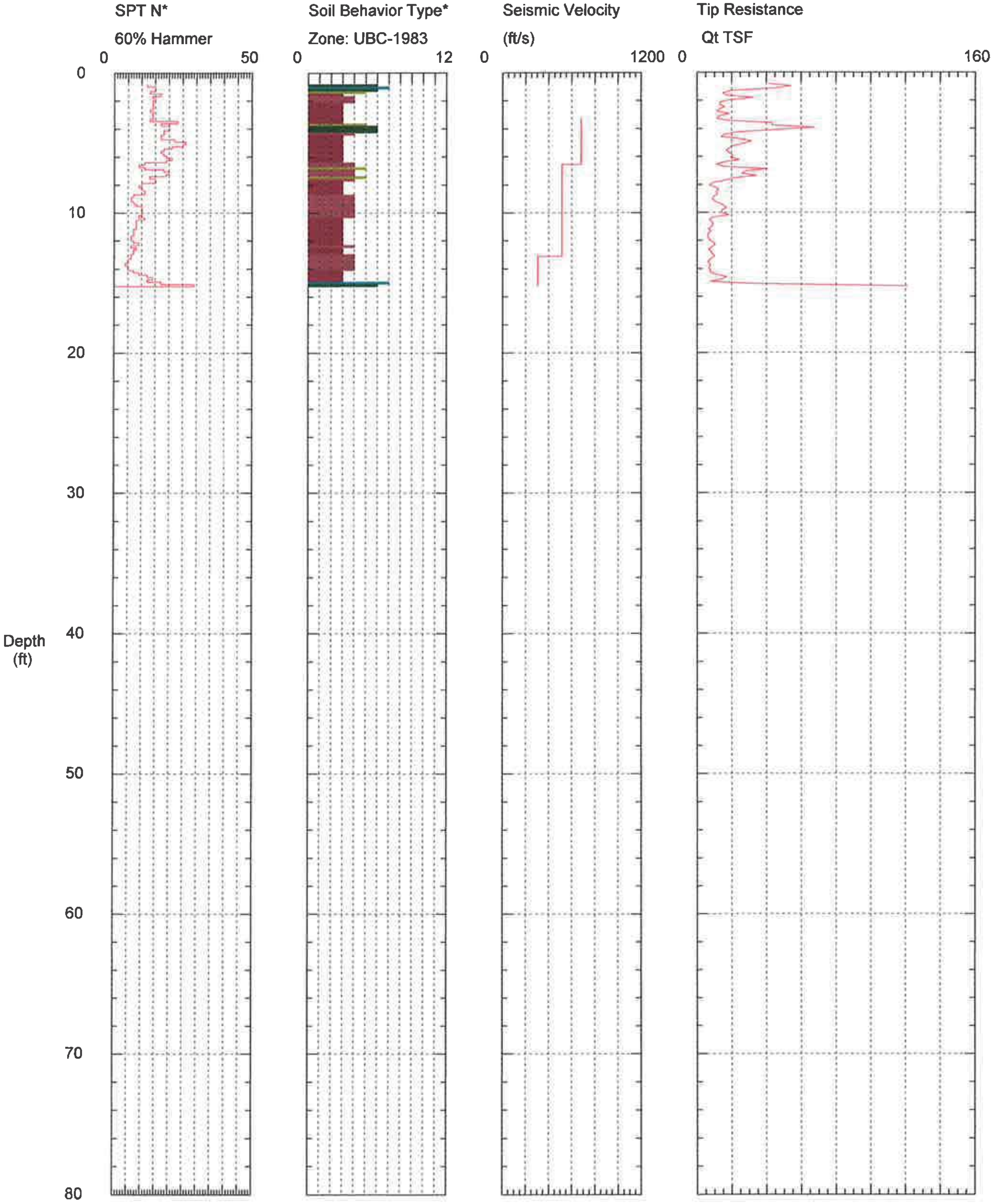
- ⬜ 1 sensitive fine grained
- ⬛ 2 organic material
- 🟥 3 clay
- 🟫 4 silty clay to clay
- 🟩 5 clayey silt to silty clay
- 🟩 6 sandy silt to clayey silt
- 🟦 7 silty sand to sandy silt
- 🟦 8 sand to silty sand
- 🟧 9 sand
- 🟥 10 gravelly sand to sand
- 🟦 11 very stiff fine grained (*)
- 🟦 12 sand to clayey sand (*)

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

HDR ENG. / BC-2,SCPT-7 / BIG CK. DAM NEWPORT

Operator: SAV/CM
Sounding: VEI434BC2SCPT7(489)
Cone Used: DSG0736

CPT Date/Time: 10/24/2013 11:30:31 PM
Location: BC2,SCPT7 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT



Maximum Depth = 15.26 feet

Depth Increment = 0.164 feet

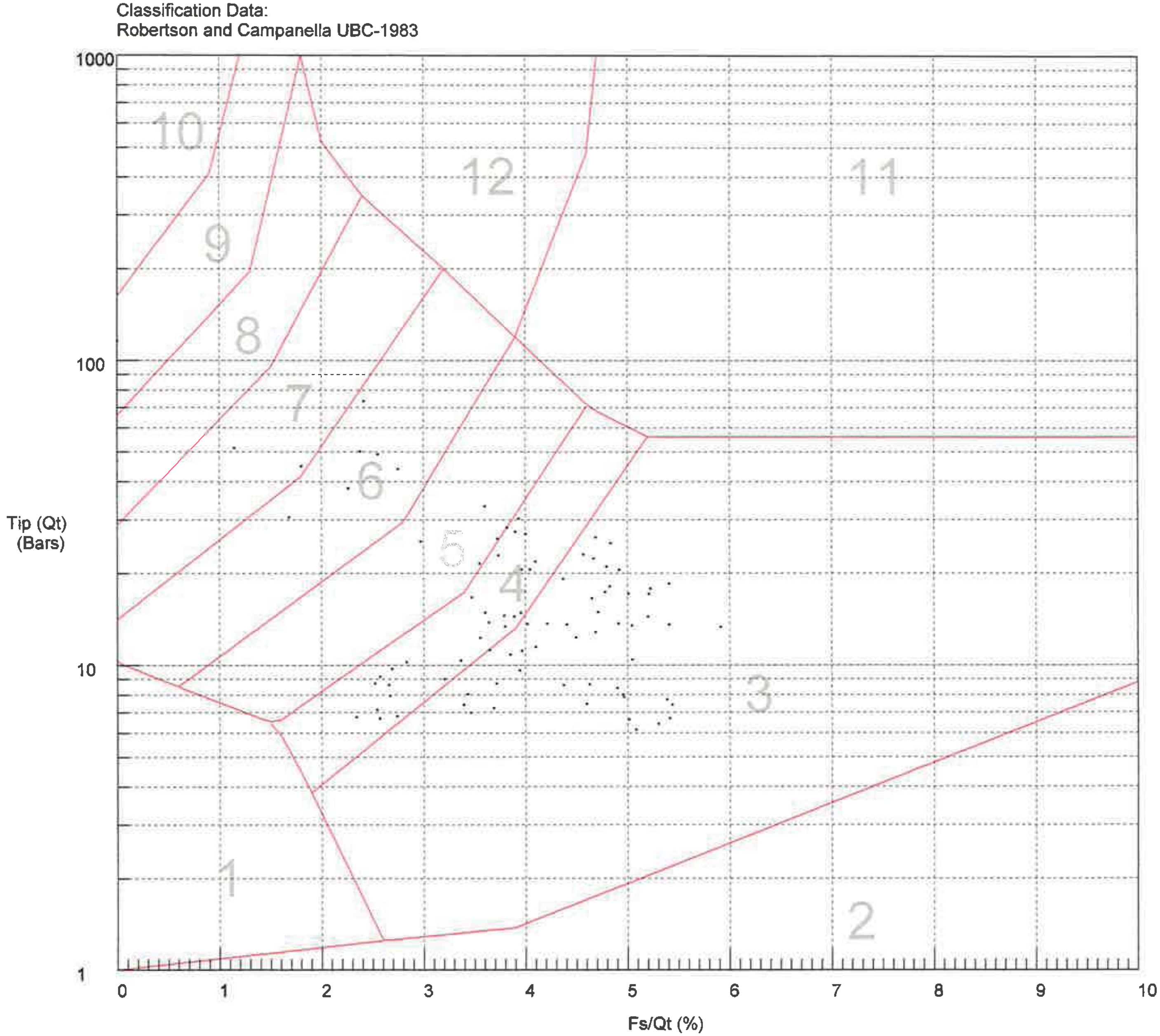
- ▢ 1 sensitive fine grained
- ■ 2 organic material
- ■ 3 clay
- ■ 4 silty clay to clay
- ■ 5 clayey silt to silty clay
- ■ 6 sandy silt to clayey silt
- ■ 7 silty sand to sandy silt
- ■ 8 sand to silty sand
- ▨ 9 sand
- ■ 10 gravelly sand to sand
- ■ 11 very stiff fine grained (*)
- ■ 12 sand to clayey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

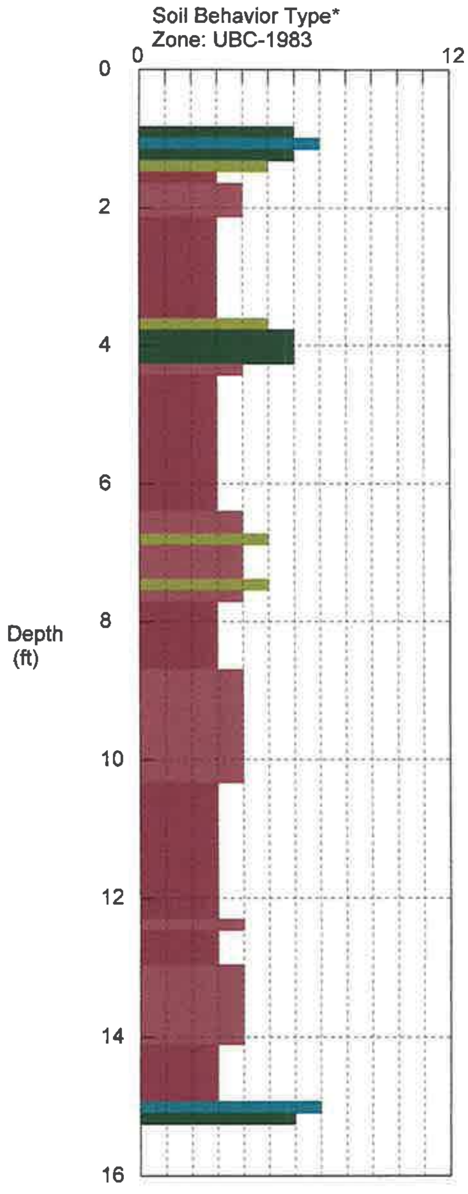
*Soil behavior type and SPT based on data from UBC-1983

HDR ENG. / BC-2,SCPT-7 / BIG CK. DAM NEWPORT

Operator: SAV/CM
Sounding: VEI434BC2SCPT7(489)
Cone Used: DSG0736

CPT Date/Time: 10/24/2013 11:30:31 PM
Location: BC2,SCPT7 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT



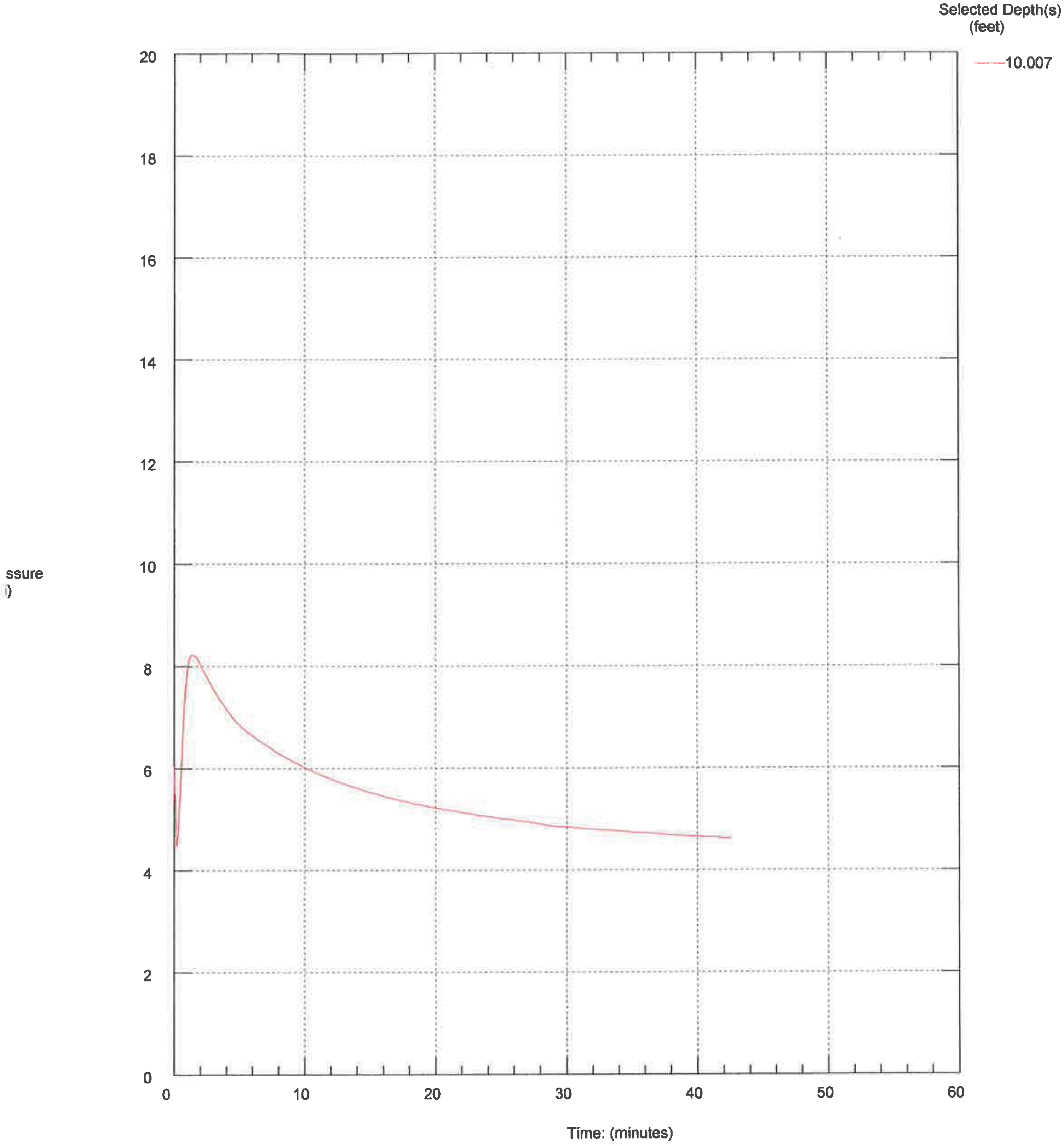


1 sensitive fine grained	4 silty clay to clay	7 silty sand to sandy silt	10 gravelly sand to sand
2 organic material	5 clayey silt to silty clay	8 sand to silty sand	11 very stiff fine grained (*)
3 clay	6 sandy silt to clayey silt	9 sand	12 sand to clayey sand (*)

HDR ENG. / BC-2CPT-7 / BIG CK. DAM NEWPORT

Operator SAV/CM
Sounding: VEI434BC2SCPT7(489)
Cone Used: DSG0736

CPT Date/Time: 10/24/2013 11:30:31 PM
Location: BC2SCPT7 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT

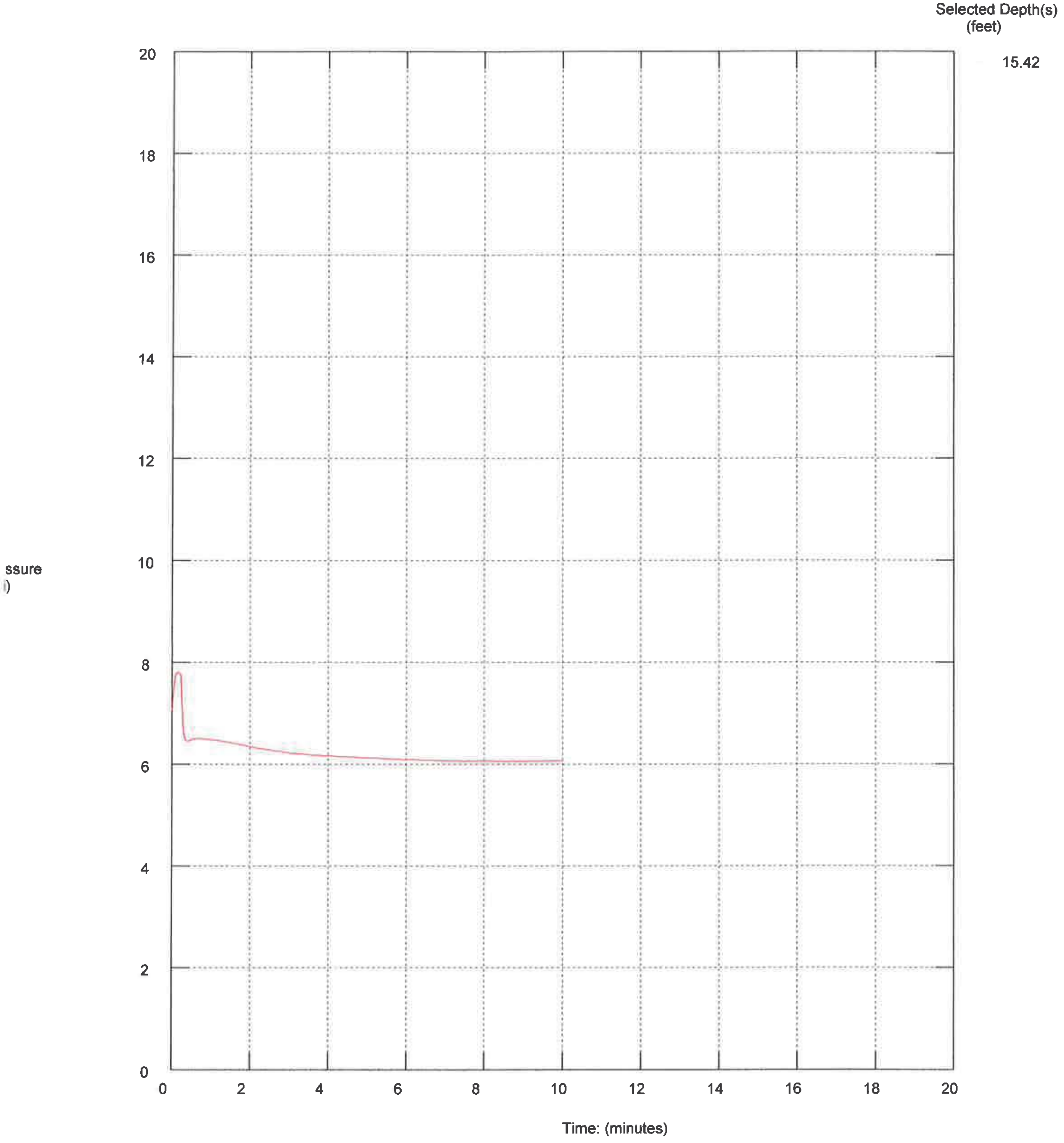


Maximum Pressure = 8.227 psi
Hydrostatic Pressure

HDR ENG. / BC-2CPT-7 / BIG CK. DAM NEWPORT

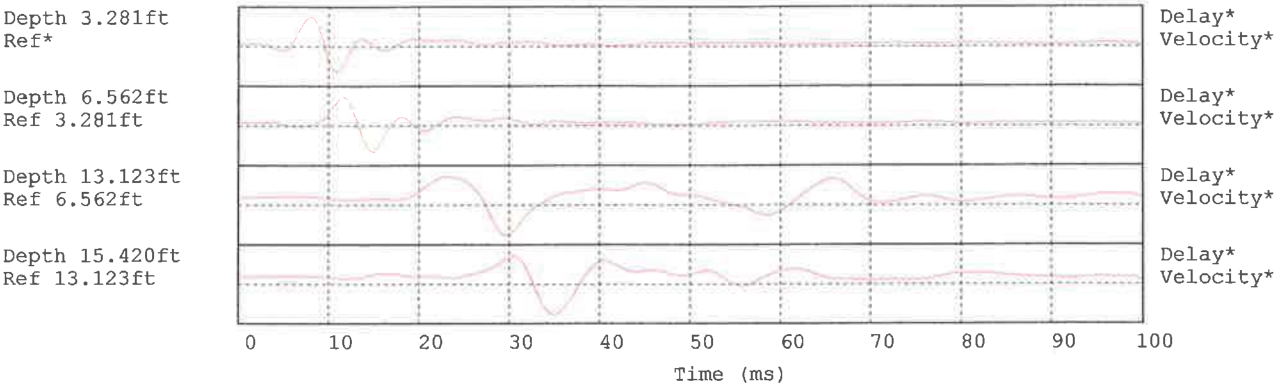
Operator SAV/CM
Sounding: VEI434BC2SCPT7(489)
Cone Used: DSG0736

CPT Date/Time: 10/24/2013 11:30:31 PM
Location: BC2SCPT7 / BIG CK. NEWPORT
Job Number: HDR ENG./BIG CK. NEWPORT



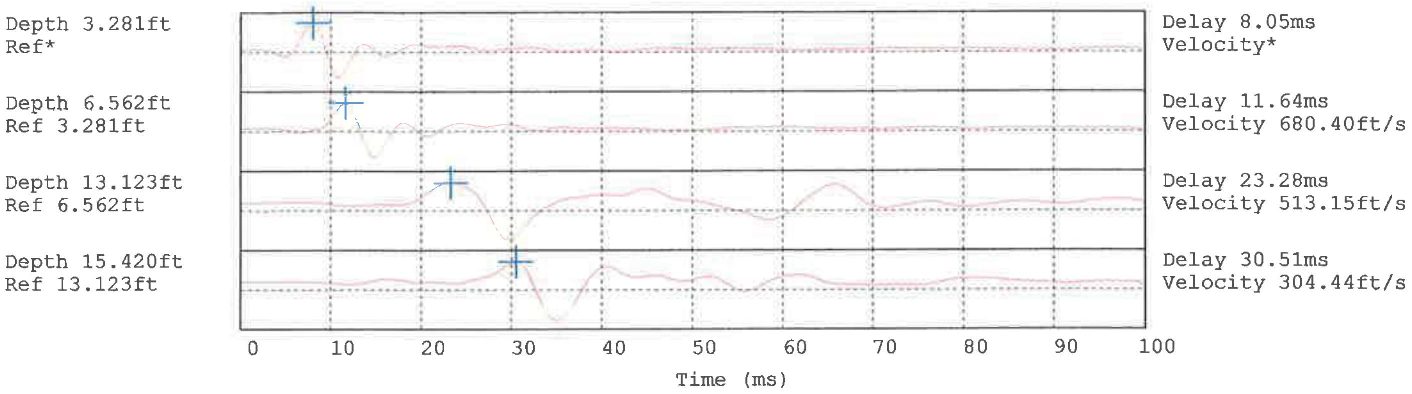
Maximum Pressure = 7.811 psi
Hydrostatic Pressure = (

HDR ENG. / BC2,SCPT7 / BIG CK. NEWPORT



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

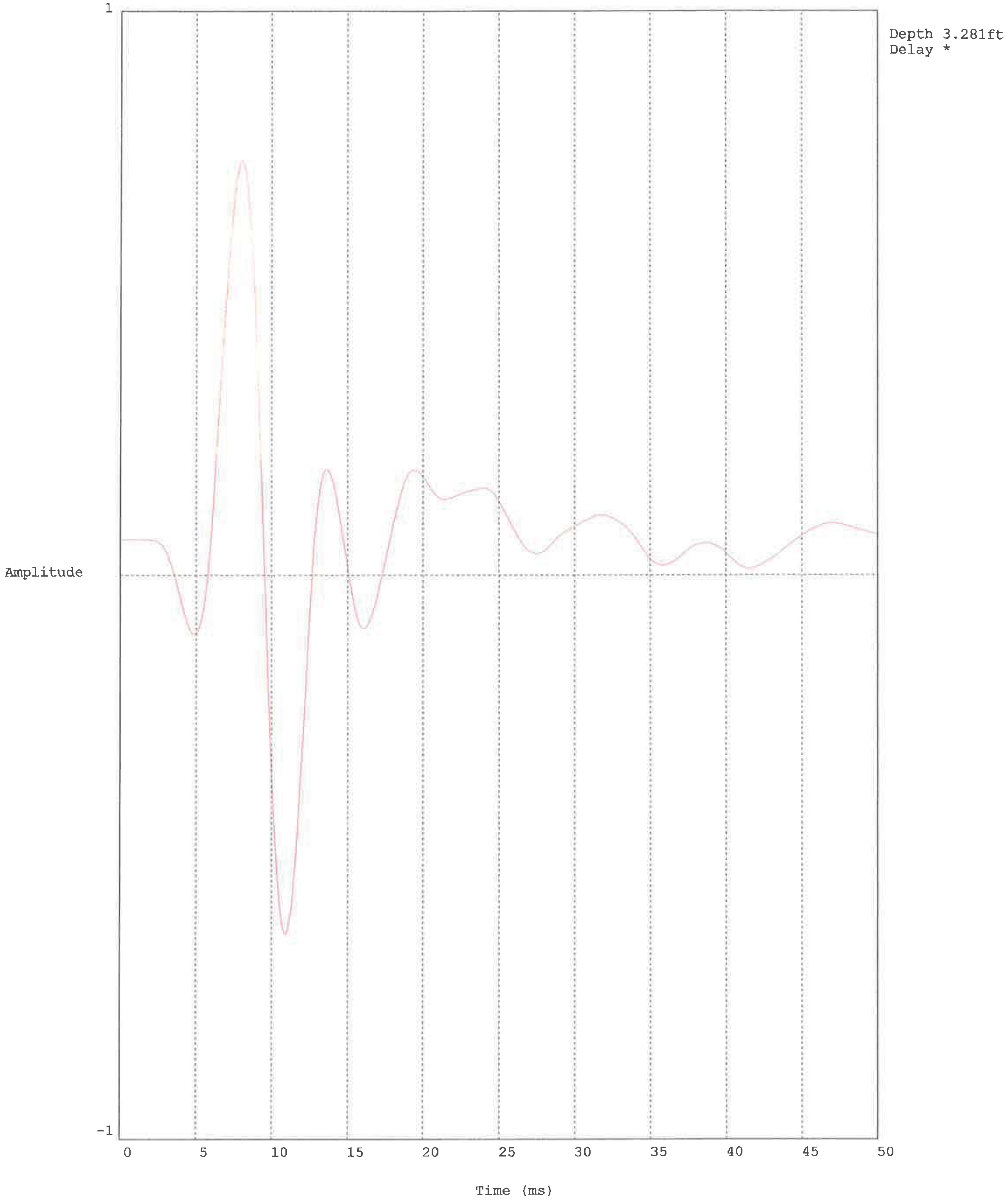
HDR ENG. / BC2,SCPT7 / BIG CK. NEWPORT



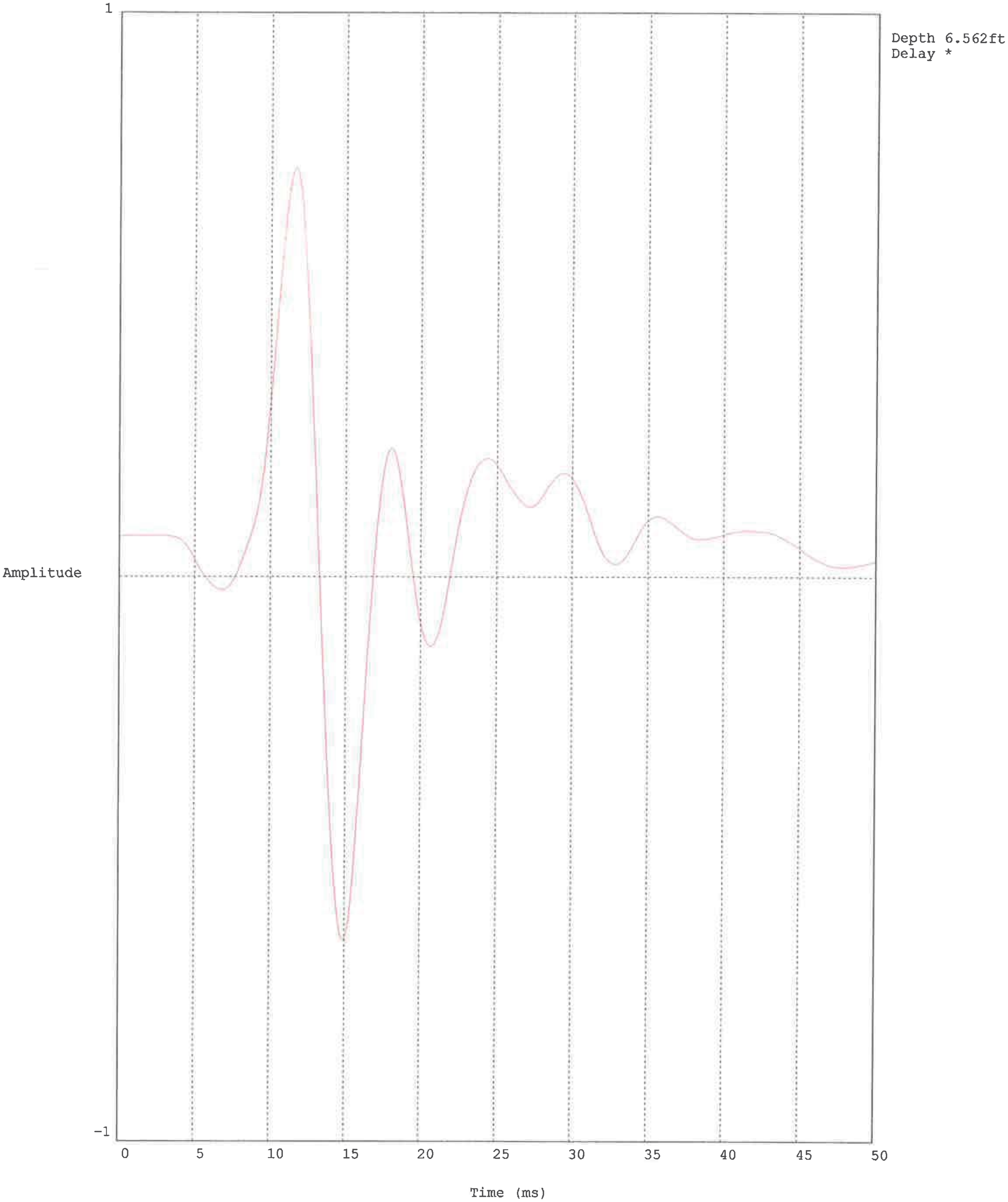
Hammer to Rod String Distance 1.3 (m)
* = Not Determined

THE SHEAR WAVE PICKS ARE COMPUTER GENERATED VALUES (STAFF REVIEW RECOMMENDED)

HDR ENG. / BC2,SCPT7 / BIG CK. NEWPORT



HDR ENG. / BC2,SCPT7 / BIG CK. NEWPORT



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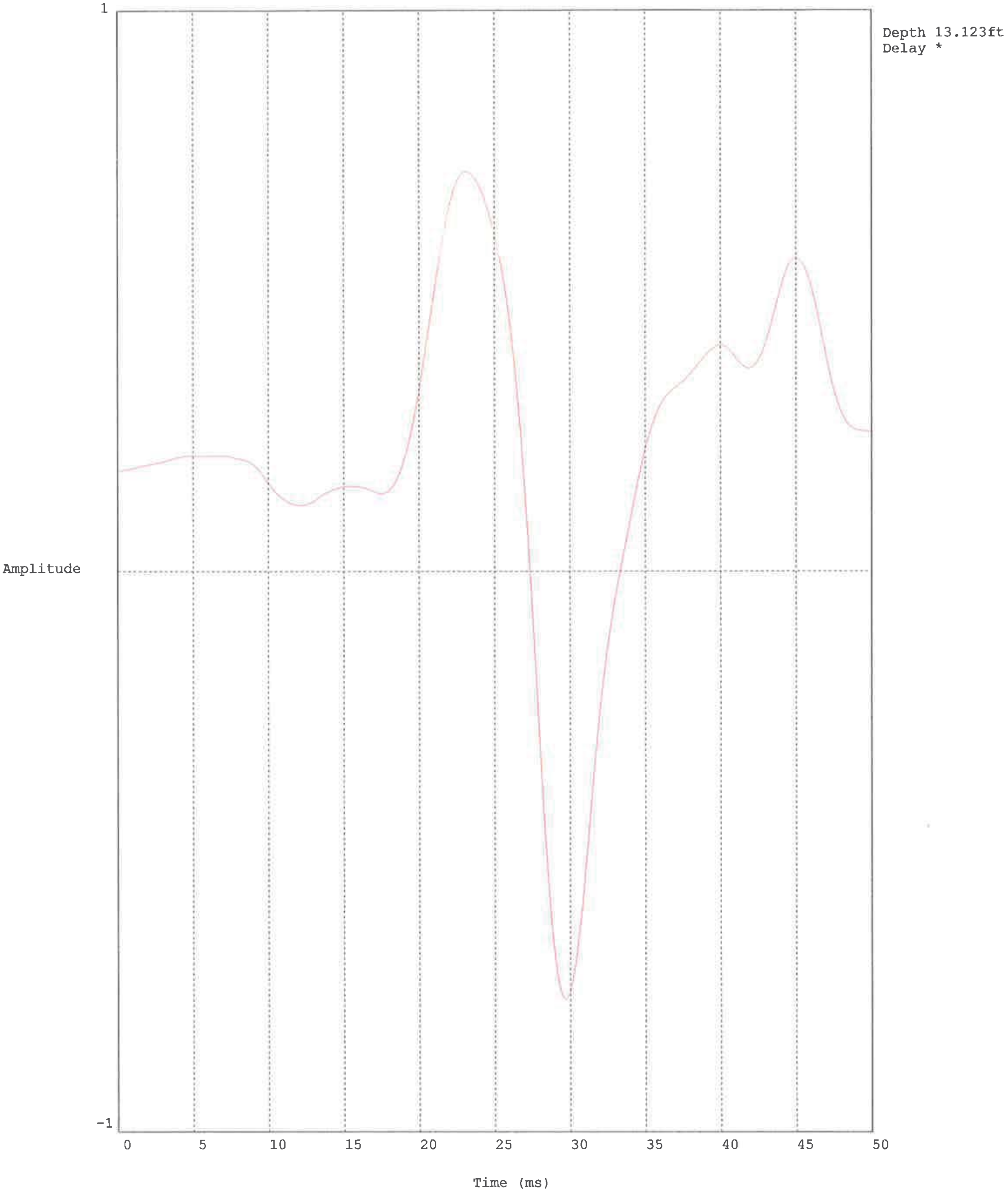
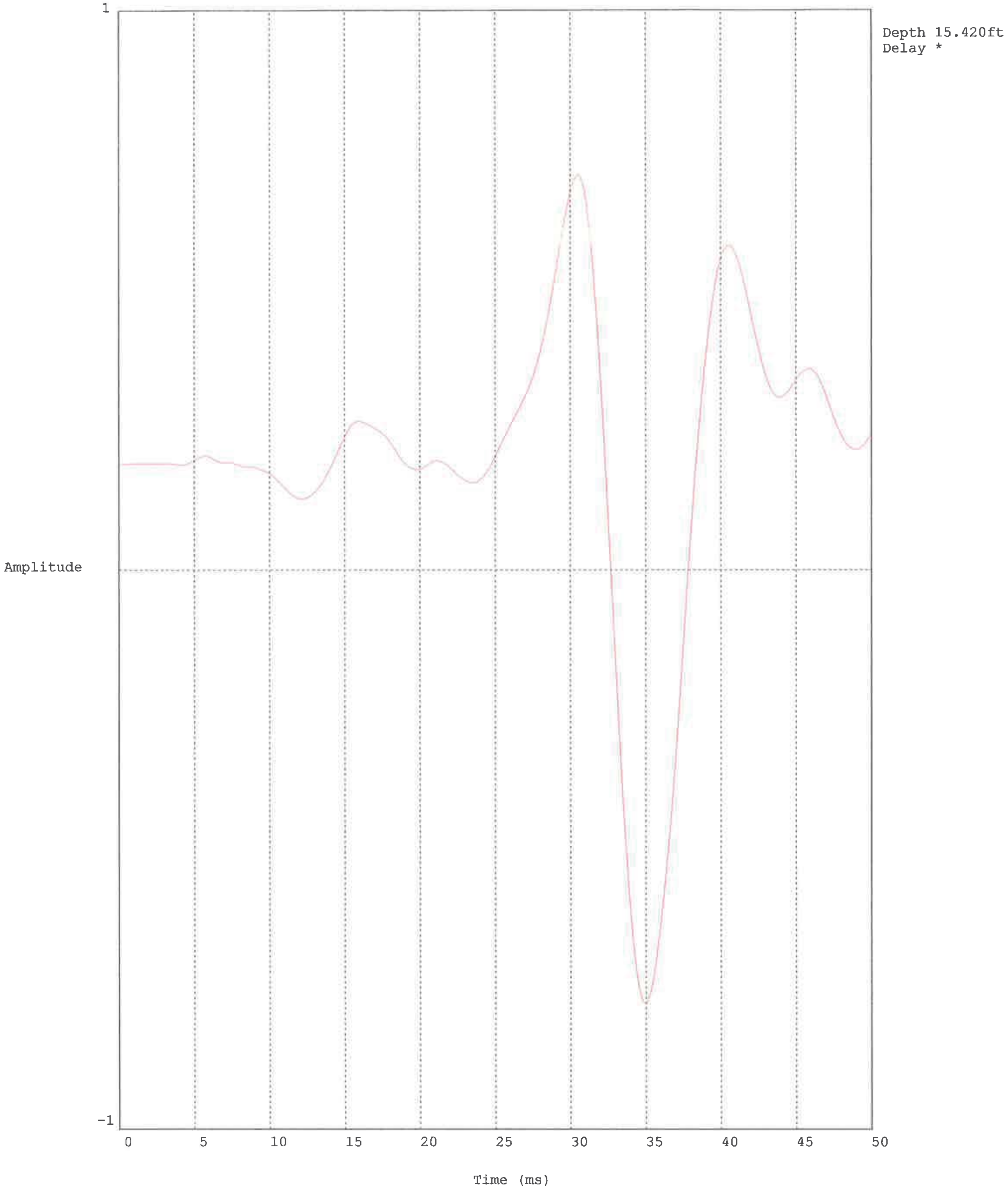


HDR ENG. / BC2,SCPT7 / BIG CK. NEWPORT



Data File:VEI434BC2SCPT7(489)
Operator:SAV/CM
Cone ID:DSG0736
Customer: BIG CK. DAM NEWPORT

10/24/2013 11:30:31 PM
Location:BC2,SCPT7 / BIG CK. NEWPORT
Job Number:HDR ENG./BIG CK. NEWPORT
Units:

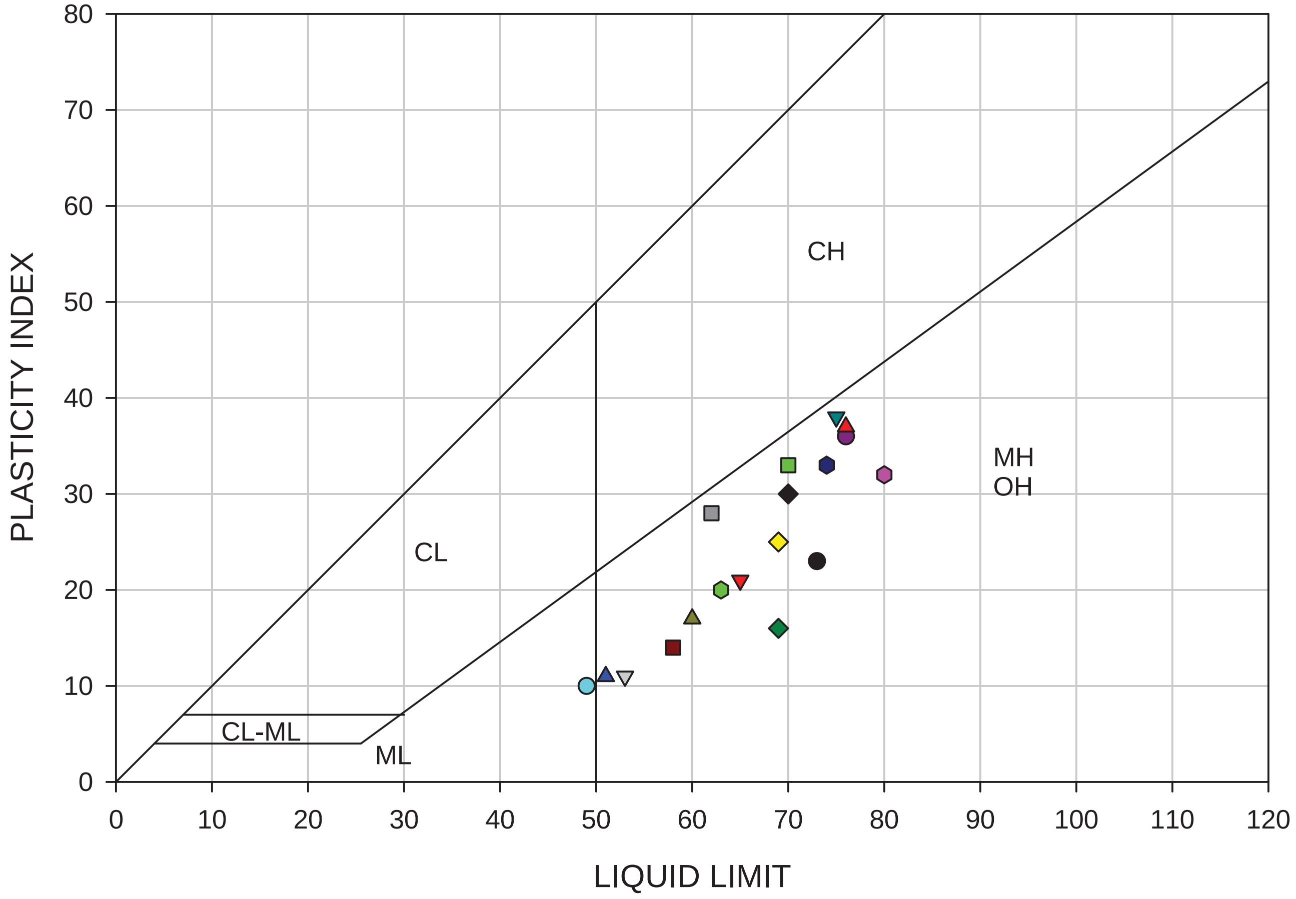
Depth (ft)	Qt TSF	Fs TSF	Fs/Qt (%)	Pw PSI	SPT N* 60% Hammer	Zone	Soil Behavior Type UBC-1983
0.82	41.80	0.6722	1.608	0.122	12	6	sandy silt to clayey silt
0.98	54.10	0.9387	1.735	0.175	15	7	silty sand to sandy silt
1.15	45.25	0.9303	2.056	0.232	15	6	sandy silt to clayey silt
1.31	19.99	0.8257	4.132	0.421	13	5	clayey silt to silty clay
1.48	14.80	0.6225	4.206	2.561	17	3	clay
1.64	17.23	0.9696	5.626	3.374	14	4	silty clay to clay
1.80	32.59	1.0206	3.132	3.965	15	4	silty clay to clay
1.97	18.72	0.8167	4.363	1.208	14	4	silty clay to clay
2.13	13.39	0.7273	5.430	1.425	14	3	clay
2.30	13.24	0.8137	6.148	1.724	14	3	clay
2.46	16.07	0.7683	4.780	1.887	14	3	clay
2.62	13.06	0.5543	4.244	1.392	13	3	clay
2.79	11.11	0.5628	5.064	0.945	14	3	clay
2.95	18.76	0.6910	3.683	0.746	14	3	clay
3.12	12.80	0.6249	4.883	0.581	14	3	clay
3.28	11.46	0.7959	6.943	1.246	13	3	clay
3.44	17.56	1.0512	5.987	8.268	23	3	clay
3.61	43.39	1.4591	3.363	8.679	17	5	clayey silt to silty clay
3.77	43.38	1.2454	2.871	3.489	20	6	sandy silt to clayey silt
3.94	67.66	1.2332	1.823	3.669	20	6	sandy silt to clayey silt
4.10	46.30	1.2641	2.730	3.198	18	6	sandy silt to clayey silt
4.27	24.00	1.2936	5.390	3.453	18	4	silty clay to clay
4.43	15.80	0.7982	5.053	3.458	17	3	clay
4.59	13.96	0.7081	5.072	3.925	17	3	clay
4.76	24.81	1.1007	4.437	5.730	22	3	clay
4.92	31.46	1.4688	4.668	6.240	26	3	clay
5.09	26.21	1.2964	4.946	6.077	25	3	clay
5.25	21.18	1.0448	4.933	6.000	21	3	clay
5.41	18.65	0.8241	4.419	8.282	18	3	clay
5.58	17.00	0.8732	5.135	8.117	17	3	clay
5.74	18.20	0.9996	5.493	8.217	18	3	clay
5.91	20.74	1.0494	5.060	6.787	19	3	clay
6.07	19.19	1.0931	5.697	5.529	21	3	clay
6.23	24.52	1.0260	4.184	5.034	19	3	clay
6.40	16.52	0.5119	3.099	3.539	11	4	silty clay to clay
6.56	11.34	0.2809	2.478	3.470	9	4	silty clay to clay
6.73	15.47	0.7838	5.067	7.796	11	5	clayey silt to silty clay
6.89	40.85	1.3367	3.272	2.284	18	4	silty clay to clay
7.05	28.30	1.2634	4.465	0.395	20	4	silty clay to clay
7.22	25.89	1.1364	4.389	-0.428	19	4	silty clay to clay
7.38	34.51	0.9878	2.863	-1.047	13	5	clayey silt to silty clay
7.55	21.18	0.9122	4.308	-1.282	15	4	silty clay to clay
7.71	16.29	0.7868	4.830	-1.325	15	3	clay
7.87	9.42	0.5089	5.405	-1.320	10	3	clay
8.04	7.02	0.3548	5.057	-1.162	9	3	clay
8.20	10.66	0.3891	3.651	-0.782	10	3	clay
8.37	12.47	0.4448	3.566	-0.450	11	3	clay
8.53	10.88	0.4752	4.367	-0.230	11	3	clay
8.69	11.85	0.3642	3.075	2.604	7	4	silty clay to clay
8.86	9.70	0.2517	2.595	2.545	6	4	silty clay to clay
9.02	8.96	0.2044	2.280	3.477	6	4	silty clay to clay
9.19	10.08	0.2824	2.801	4.398	7	4	silty clay to clay
9.35	13.04	0.4211	3.230	5.874	8	4	silty clay to clay
9.51	15.45	0.6676	4.321	6.065	10	4	silty clay to clay
9.68	17.16	0.6410	3.735	5.252	10	4	silty clay to clay
9.84	14.06	0.5371	3.820	3.520	10	4	silty clay to clay
10.01	14.19	0.5873	4.138	5.960	10	4	silty clay to clay

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

Depth (ft)	Qt TSF	Fs TSF	Fs/Qt (%)	Pw PSI	SPT N* 60% Hammer	Zone	Soil Behavior Type UBC-1983
10.17	18.46	0.5594	3.031	2.719	9	4	silty clay to clay
10.33	9.47	0.4533	4.789	2.176	11	3	clay
10.50	7.05	0.3735	5.301	2.602	8	3	clay
10.66	8.59	0.4157	4.842	3.590	8	3	clay
10.83	9.04	0.4352	4.813	4.379	8	3	clay
10.99	8.77	0.4414	5.036	4.685	8	3	clay
11.15	6.39	0.4259	6.664	5.809	7	3	clay
11.32	8.11	0.3971	4.898	6.510	7	3	clay
11.48	6.51	0.3133	4.815	7.280	7	3	clay
11.65	6.25	0.3349	5.360	7.598	6	3	clay
11.81	6.53	0.3315	5.080	7.863	6	3	clay
11.98	7.37	0.4011	5.445	8.942	7	3	clay
12.14	9.51	0.3430	3.609	9.363	9	3	clay
12.30	10.41	0.2689	2.583	8.117	6	4	silty clay to clay
12.47	8.34	0.2933	3.515	6.957	8	3	clay
12.63	6.45	0.3024	4.692	7.182	7	3	clay
12.80	7.94	0.2421	3.050	7.746	7	3	clay
12.96	8.88	0.2442	2.750	7.306	6	4	silty clay to clay
13.12	10.20	0.2318	2.272	6.163	6	4	silty clay to clay
13.29	8.24	0.2129	2.584	5.874	5	4	silty clay to clay
13.45	6.48	0.2206	3.406	6.443	5	4	silty clay to clay
13.62	6.63	0.1510	2.277	7.060	4	4	silty clay to clay
13.78	7.89	0.1672	2.119	7.242	5	4	silty clay to clay
13.94	6.70	0.1783	2.660	7.280	5	4	silty clay to clay
14.11	7.81	0.2236	2.862	7.447	7	3	clay
14.27	7.41	0.3563	4.808	7.261	9	3	clay
14.44	11.72	0.5979	5.101	7.423	12	3	clay
14.60	16.92	0.5237	3.096	6.907	14	3	clay
14.76	14.16	0.5984	4.225	6.584	12	3	clay
14.93	7.60	0.6160	8.107	6.417	17	7	silty sand to sandy silt
15.09	32.65	1.8580	5.691	6.517	29	6	sandy silt to clayey silt
15.26	121.20	-32768.0100	-27037.160	6.649	0	0	<out of range>

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

Appendix C – Laboratory Testing Data



Boring No.	Sample	Depth (ft)	PL	LL	PI	CI*
● BC1-BH-2	S-4	6.0-7.5	50	73	23	0.46
▼ BC1-BH-2	S-8	12.0-13	44	65	21	0.48
■ BC1-BH-2	S-13	19.5-21	37	70	33	0.89
◇ BC1-BH-2	S-17	25.5-27	44	69	25	0.57
▲ BC1-BH-2	S-21	31.5-33	40	51	11	0.28
⬢ BC1-BH-2	S-23	34.5-36	48	80	32	0.67
○ BC1-BH-2	S-25	37.5-39	39	49	10	0.26
▽ BC1-BH-2	S-29	43.5-45	42	53	11	0.26
■ BC1-BH-2	S-31	46.5-48	44	58	14	0.32
◆ BC2-BH-4	S-2	3.0-4.5	53	69	16	0.30
▲ BC2-BH-4	S-6	9.0-10.5	43	60	17	0.40
⬢ BC2-BH-4	S-14	21.0-22	41	74	33	0.80
● BC2-BH-4	S-24	36.0-37	40	76	36	0.90
▼ BC1-BH-2	S-36	54-55.5	37	75	38	1.03
■ BC1-BH-4(u)	S-11	51-52.5	34	62	28	0.82
◆ BC2-BH-5(u)	S-6	22-23.5	40	70	30	0.75
▲ BC2-BH-5(u)	S-8	27-28.5	39	76	37	0.95
⬡ BC2-BH-6(u)	S-6	22-23.5	43	63	20	0.47

* Cohesive index is PI/PL

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PLASTICITY CHART

BIG CREEK DAMS
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FEB 2014
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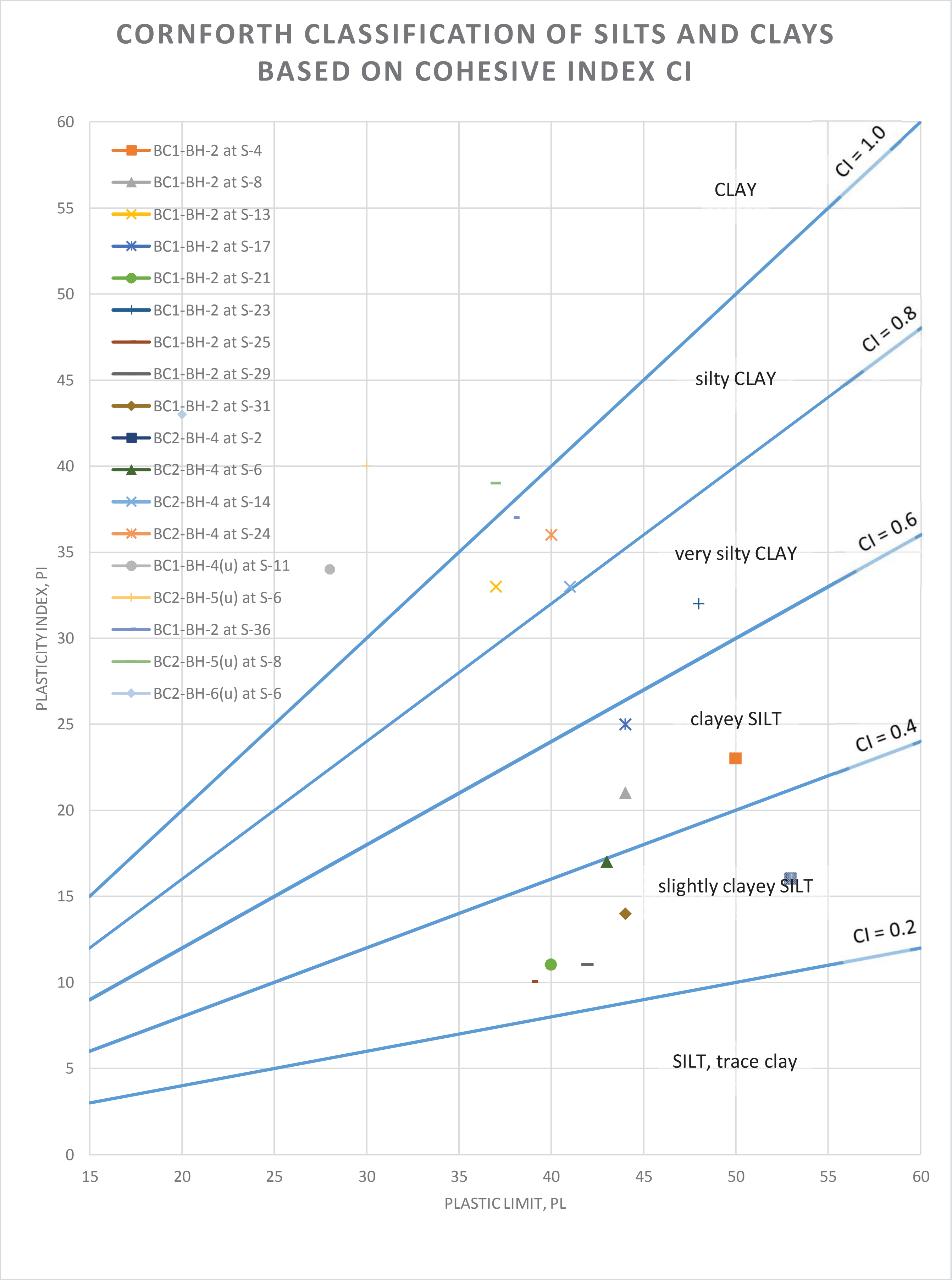
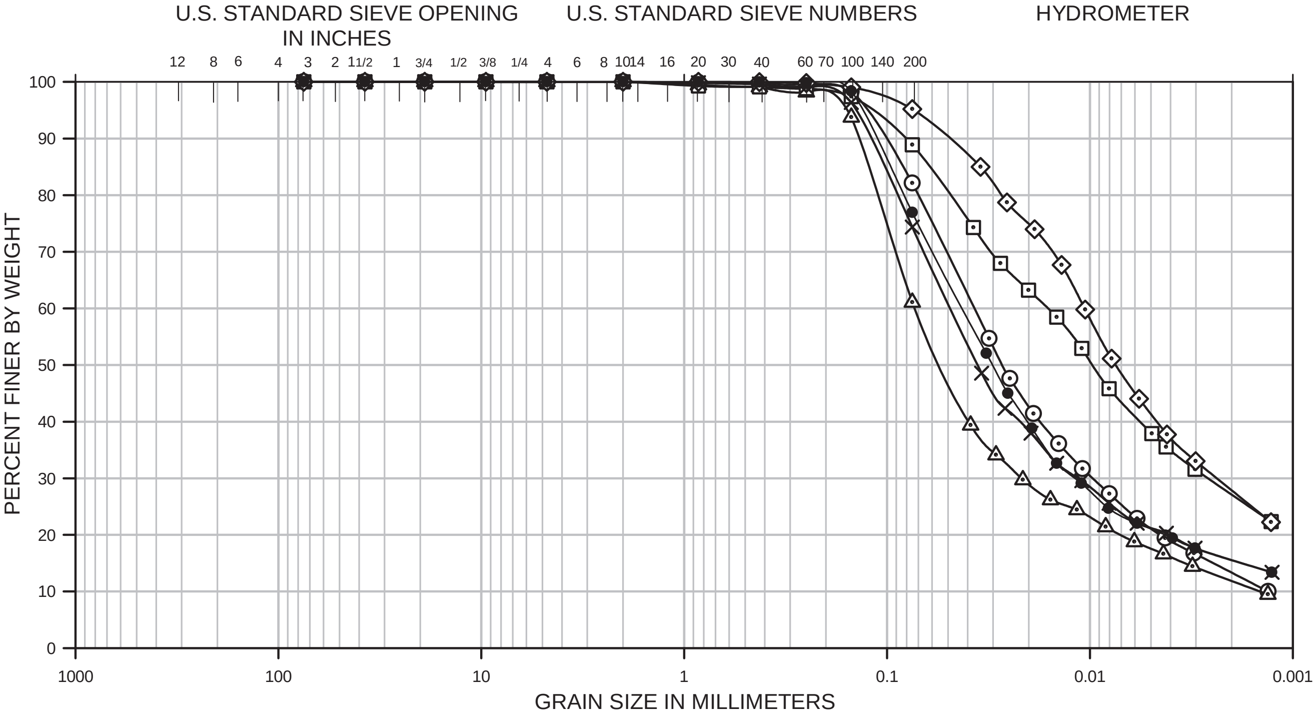
FIG. XX


CORNFORTH CLASSIFICATION OF SILTS AND CLAYS
BASED ON COHESIVE INDEX CI
BC1-BH-2 at S-4
BC1-BH-2 at S-8
BC1-BH-2 at S-13
BC1-BH-2 at S-17
BC1-BH-2 at S-21
BC1-BH-2 at S-23
BC1-BH-2 at S-25
BC1-BH-2 at S-29
BC1-BH-2 at S-31
BC2-BH-4 at S-2
BC2-BH-4 at S-6
BC2-BH-4 at S-14
BC2-BH-4 at S-24
BC1-BH-4(u) at S-11
BC2-BH-5(u) at S-6
BC1-BH-2 at S-36
BC2-BH-5(u) at S-8
BC2-BH-6(u) at S-6
CLAY
silty CLAY
very silty CLAY
clayey SILT
slightly clayey SILT
SILT, trace clay
CI = 1.0
CI = 0.8
CI = 0.6
CI = 0.4
CI = 0.2
PLASTICITY INDEX, PI
PLASTIC LIMIT, PL
60
55
50
45
40
35
30
25
20
15
10
5
0
15
20
25
30
35
40
45
50
55
60



COBBLES	GRAVEL		SAND			FINES	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

Boring No.	Sample No.	Depth	Classification	Nat W%	LL	PL	PI
⊙ BC1-BH-2	S-4	6.0-7.5	Slightly sandy, clayey SILT (ALLUVIUM)	63	73	50	23
△ BC1-BH-2	S-12	18.0-19.5	Silty SAND to sandy SILT (ALLUVIUM)	51	NON-PLASTIC		
⊡ BC1-BH-2	S-13	19.5-21.0	Slightly sandy, silty CLAY (ALLUVIUM)	80	70	37	33
◇ BC1-BH-2	S-17	25.5-27.0	Slightly sandy, clayey SILT (ALLUVIUM)	83	69	44	25
× BC1-BH-2	S-23	34.5-36.0	Sandy, very silty CLAY(ALLUVIUM)	65	80	48	32
● BC1-BH-2	S-25	37.5-39.0	Slightly clayey SILT (ALLUVIUM)	56	49	39	10

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GRADATION GRAPH

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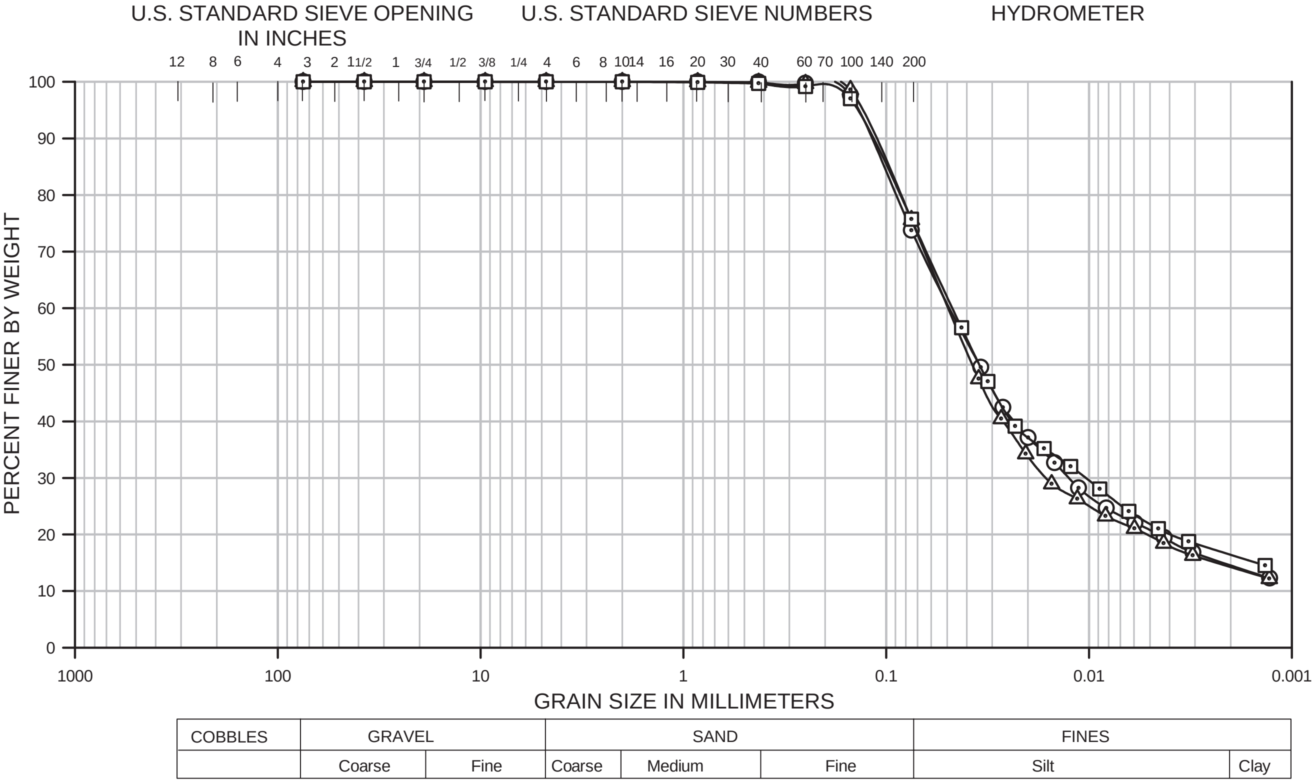
FIG. XX



Boring No.	Sample No.	Depth	Classification	Nat W%	LL	PL	PI
⊙ BC1-BH-2	S-29	43.5-45.0	Slightly sandy, slightly clayey SILT (ALLUVIUM)	53	53	42	11
△ BC1-BH-2	S-31	46.5-48.0	Slightly sandy, slightly clayey SILT (ALLUVIUM)	57	58	44	14
⊡ BC1-BH-2	S-40	60.0-61.5	Slightly sandy, slightly clayey SILT (ALLUVIUM)	50	NON-PLASTIC		

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GRADATION GRAPH

BIG CREEK DAMS
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FIG. XX

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GRADATION GRAPH

BIG CREEK DAMS
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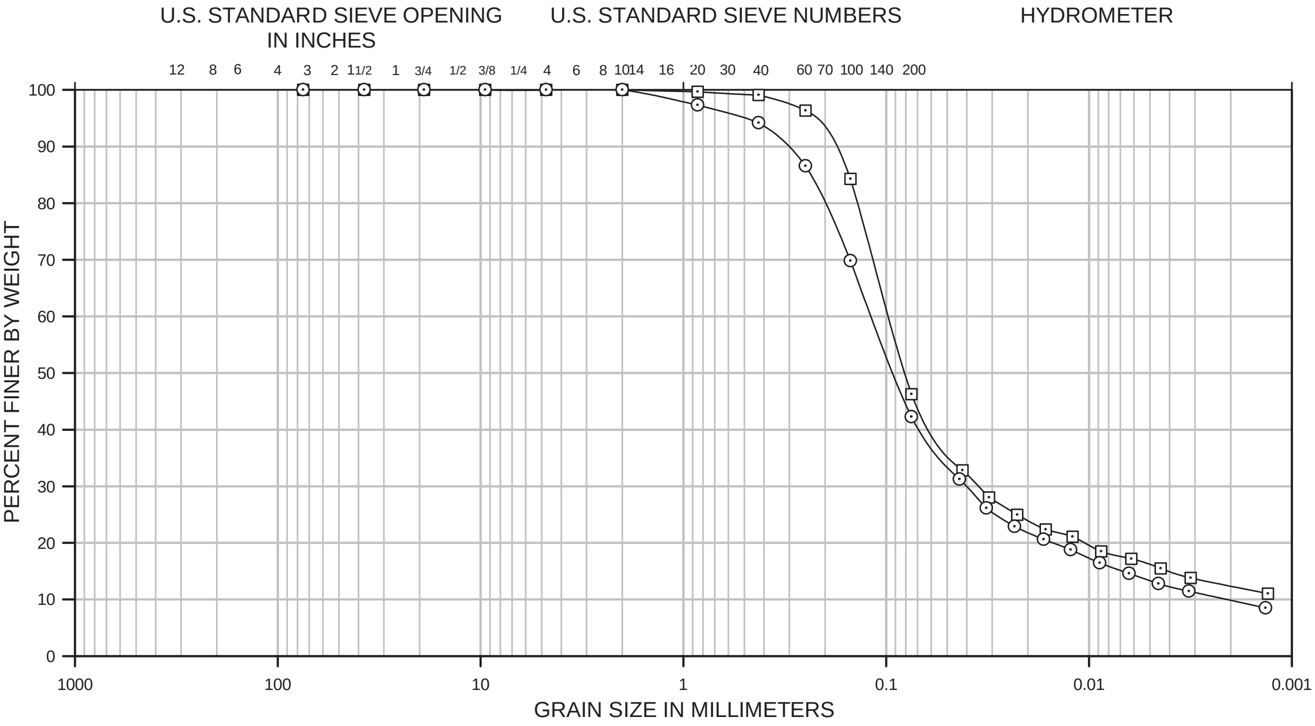
JAN 2014
PROJ. 2328
FIG. XX



COBBLES	GRAVEL		SAND			FINES	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

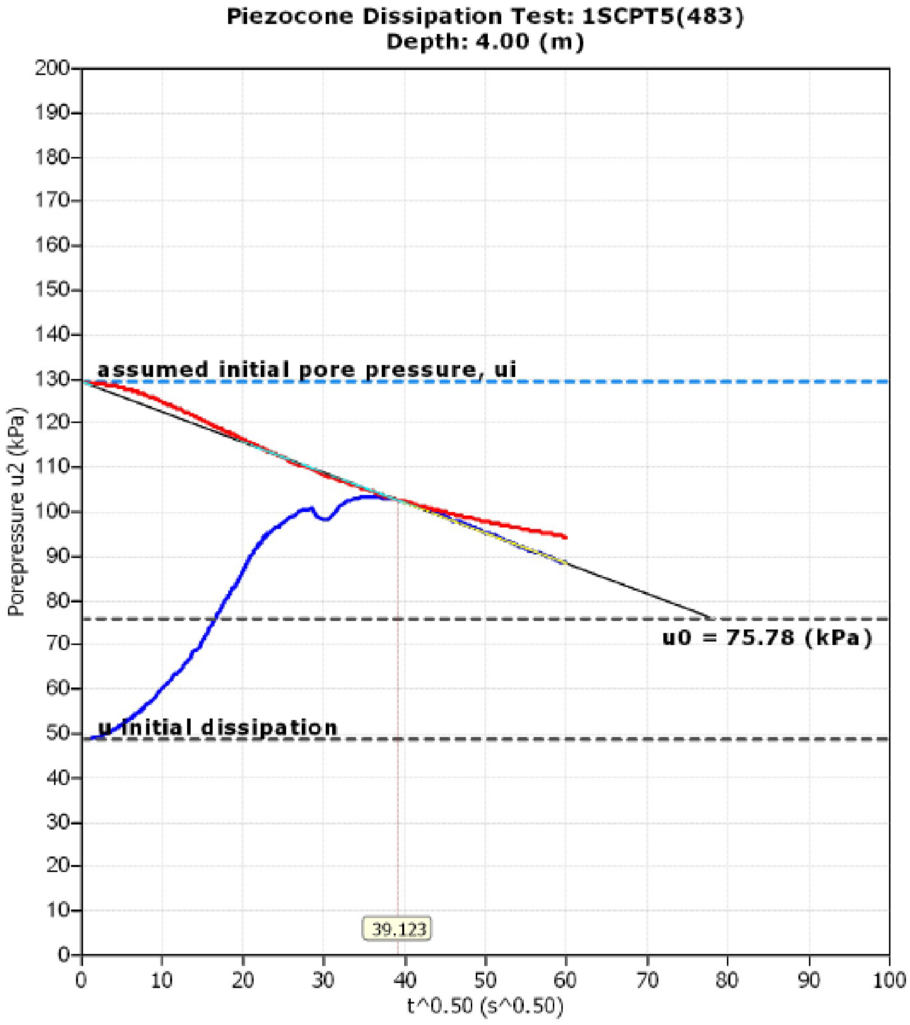
Boring No.	Sample No.	Depth	Classification	Nat W%	LL	PL	PI
─⊡─ BC2-BH-4	S-8	12.0-13.5	SILT and slightly silty SAND (ALLUVIUM/COLLUVIUM)	82	NON-PLASTIC		
─⊙─ BC2-BH-4	S-20	30.0-31.5	Slightly clayey, silty SAND (ALLUVIUM/COLLUVIUM)	58	NON-PLASTIC		

BC1-BH-2		**BC1-BH-2**		**BC1-BH-2**		**BC1-BH-2**	
S-4		**S-12**		**S-13**		**S-17**	
grain size (mm)	% finer by weight	grain size (mm)	% finer by weight	grain size (mm)	% finer by weight	grain size (mm)	% finer by weight
7.50E+01	100.00	7.50E+01	100.00	7.50E+01	100.00	7.50E+01	100.00
3.75E+01	100.00	3.75E+01	100.00	3.75E+01	100.00	3.75E+01	100.00
1.90E+01	100.00	1.90E+01	100.00	1.90E+01	100.00	1.90E+01	100.00
9.50E+00	100.00	9.50E+00	100.00	9.50E+00	100.00	9.50E+00	100.00
4.75E+00	100.00	4.75E+00	100.00	4.75E+00	100.00	4.75E+00	100.00
2.00E+00	100.00	2.00E+00	100.00	2.00E+00	100.00	2.00E+00	100.00
8.50E-01	99.87	8.50E-01	99.48	8.50E-01	99.28	8.50E-01	100.00
4.25E-01	99.67	4.25E-01	99.09	4.25E-01	99.12	4.25E-01	99.95
2.50E-01	99.31	2.50E-01	98.21	2.50E-01	98.80	2.50E-01	99.79
1.50E-01	98.26	1.50E-01	93.75	1.50E-01	97.40	1.50E-01	99.05
7.50E-02	82.11	7.50E-02	61.07	7.50E-02	88.90	7.50E-02	95.20
3.13E-02	54.68	3.87E-02	39.40	3.75E-02	74.29	3.46E-02	85.00
2.48E-02	47.60	2.90E-02	34.12	2.76E-02	67.96	2.56E-02	78.70
1.90E-02	41.41	2.14E-02	29.72	2.01E-02	63.22	1.87E-02	73.98
1.42E-02	36.10	1.56E-02	26.21	1.46E-02	58.47	1.38E-02	67.68
1.09E-02	31.68	1.16E-02	24.45	1.10E-02	52.93	1.06E-02	59.80
8.01E-03	27.26	8.36E-03	21.41	8.03E-03	45.82	7.82E-03	51.14
5.85E-03	22.87	6.04E-03	18.77	4.95E-03	37.91	5.72E-03	44.05
4.25E-03	19.39	4.34E-03	16.64	4.21E-03	35.54	4.17E-03	37.75
3.07E-03	16.74	3.12E-03	14.44	3.03E-03	31.58	3.01E-03	33.03
1.32E-03	9.98	1.33E-03	9.48	1.28E-03	22.35	1.28E-03	22.26

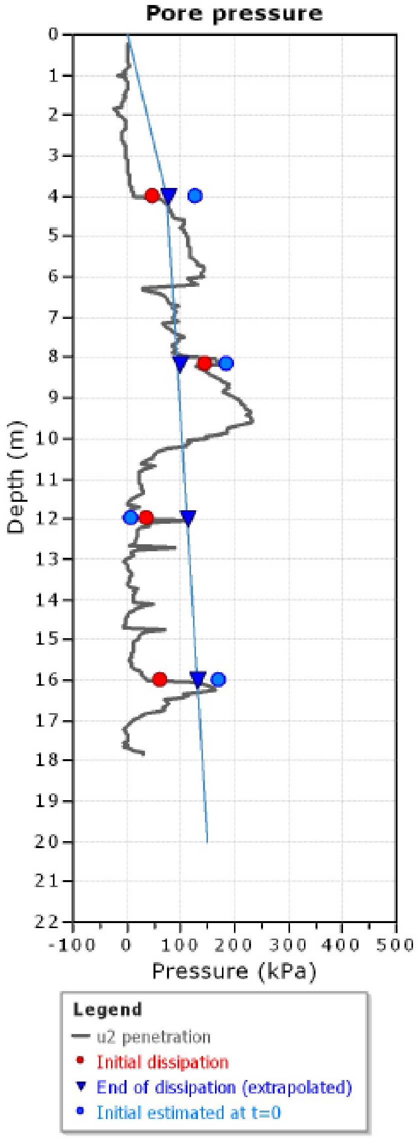
BC1-BH-2		**BC1-BH-2**		**BC1-BH-2**		**BC1-BH-2**	
S-23		**S-25**		**S-29**		**S-31**	
grain size (mm)	% finer by weight	grain size (mm)	% finer by weight	grain size (mm)	% finer by weight	grain size (mm)	% finer by weight
7.50E+01	100.00	7.50E+01	100.00	7.50E+01	100.00	7.50E+01	100.00
3.75E+01	100.00	3.75E+01	100.00	3.75E+01	100.00	3.75E+01	100.00
1.90E+01	100.00	1.90E+01	100.00	1.90E+01	100.00	1.90E+01	100.00
9.50E+00	100.00	9.50E+00	100.00	9.50E+00	100.00	9.50E+00	100.00
4.75E+00	100.00	4.75E+00	100.00	4.75E+00	100.00	4.75E+00	100.00
2.00E+00	100.00	2.00E+00	100.00	2.00E+00	100.00	2.00E+00	100.00
8.50E-01	99.85	8.50E-01	99.98	8.50E-01	99.98	8.50E-01	99.98
4.25E-01	99.62	4.25E-01	99.96	4.25E-01	99.96	4.25E-01	99.87
2.50E-01	99.09	2.50E-01	99.79	2.50E-01	99.73	2.50E-01	99.66
1.50E-01	96.39	1.50E-01	98.37	1.50E-01	97.67	1.50E-01	98.60
7.50E-02	74.37	7.50E-02	76.93	7.50E-02	73.74	7.50E-02	75.63
3.41E-02	48.57	3.24E-02	52.06	3.41E-02	49.55	3.50E-02	47.53
2.62E-02	42.39	2.53E-02	45.00	2.65E-02	42.44	2.71E-02	40.45
1.95E-02	37.97	1.93E-02	38.82	1.99E-02	37.11	2.05E-02	34.26
1.46E-02	32.67	1.46E-02	32.64	1.48E-02	32.66	1.53E-02	28.96
1.10E-02	29.57	1.10E-02	29.11	1.12E-02	28.22	1.14E-02	26.30
8.03E-03	25.60	8.10E-03	24.69	8.20E-03	24.66	8.29E-03	23.27
5.85E-03	22.06	5.85E-03	22.05	5.93E-03	22.05	5.97E-03	21.06
4.20E-03	20.30	3.91E-03	19.40	4.25E-03	19.43	4.28E-03	18.44
3.03E-03	17.64	3.03E-03	17.63	3.07E-03	16.82	3.08E-03	16.29
1.27E-03	13.37	1.27E-03	13.36	1.28E-03	12.25	1.29E-03	12.19

BC1-BH-2		**BC2-BH-4**		**BC2-BH-4**	
S-40		**S-8**		**S-20**	
grain size (mm)	% finer by weight	grain size (mm)	% finer by weight	grain size (mm)	% finer by weight
7.50E+01	100.00	7.50E+01	100.00	7.50E+01	100.00
3.75E+01	100.00	3.75E+01	100.00	3.75E+01	100.00
1.90E+01	100.00	1.90E+01	100.00	1.90E+01	100.00
9.50E+00	100.00	9.50E+00	100.00	9.50E+00	100.00
4.75E+00	100.00	4.75E+00	100.00	4.75E+00	100.00
2.00E+00	100.00	2.00E+00	100.00	2.00E+00	100.00
8.50E-01	99.92	8.50E-01	97.33	8.50E-01	99.67
4.25E-01	99.73	4.25E-01	94.18	4.25E-01	99.09
2.50E-01	99.17	2.50E-01	86.55	2.50E-01	96.36
1.50E-01	97.03	1.50E-01	69.81	1.50E-01	84.28
7.50E-02	75.72	7.50E-02	42.27	7.50E-02	46.28
4.25E-02	56.55	4.34E-02	31.26	4.20E-02	32.82
3.15E-02	47.06	3.21E-02	26.16	3.11E-02	28.03
2.31E-02	39.15	2.33E-02	22.91	2.26E-02	24.98
1.66E-02	35.19	1.67E-02	20.60	1.63E-02	22.37
1.23E-02	32.03	1.23E-02	18.76	1.21E-02	21.09
8.85E-03	28.07	8.85E-03	16.45	8.70E-03	18.47
6.36E-03	24.11	6.34E-03	14.59	6.18E-03	17.19
4.55E-03	21.02	4.54E-03	12.78	4.43E-03	15.49
3.23E-03	18.76	3.22E-03	11.45	3.15E-03	13.80
1.35E-03	14.51	1.35E-03	8.50	1.31E-03	11.03

Attachment B 3. SCPT Pore Pressure Dissipation Plots



Piezocone Dissipation Test: 1SCPT5(483)
Depth: 4.00 (m)



Pore pressure

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OCR vs. Depth Based on Laboratory Data

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

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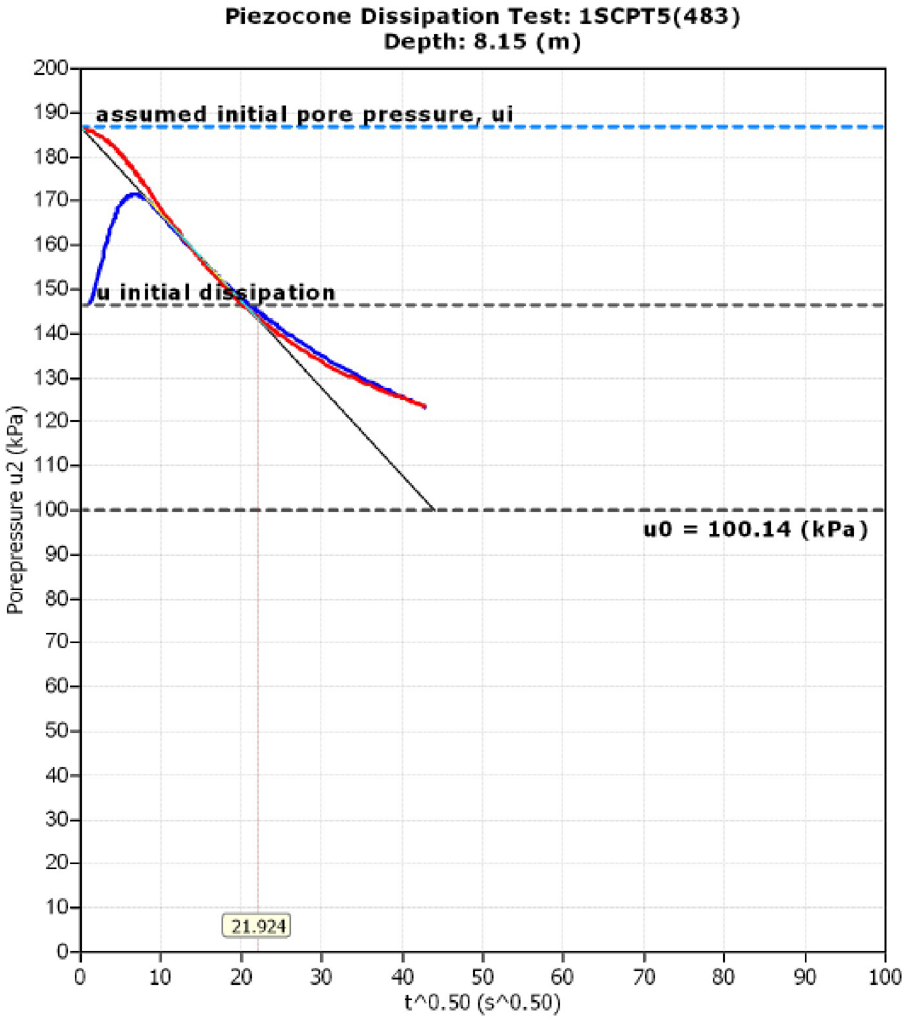
FIGURE

B-3-1



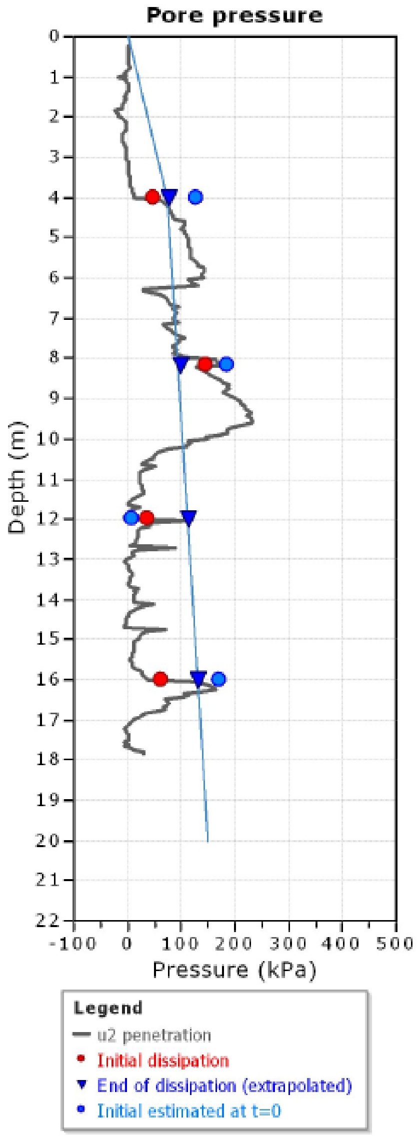


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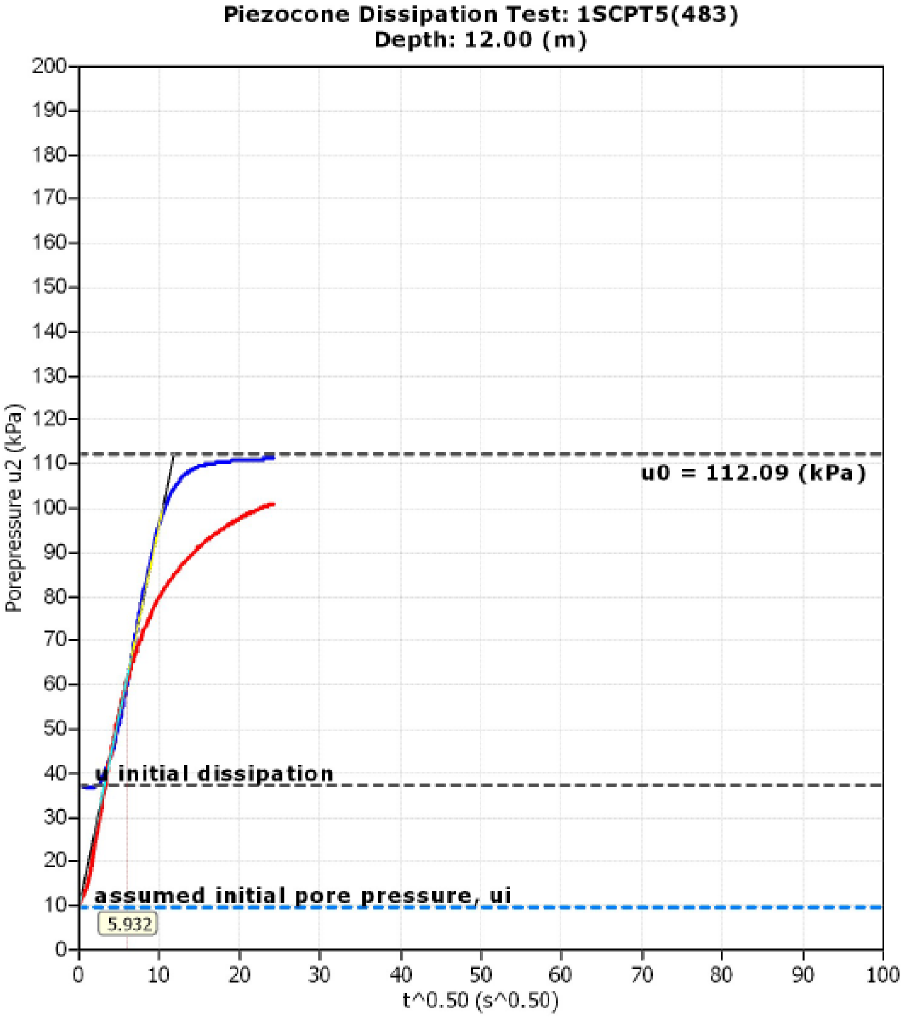
FIGURE
B-3-2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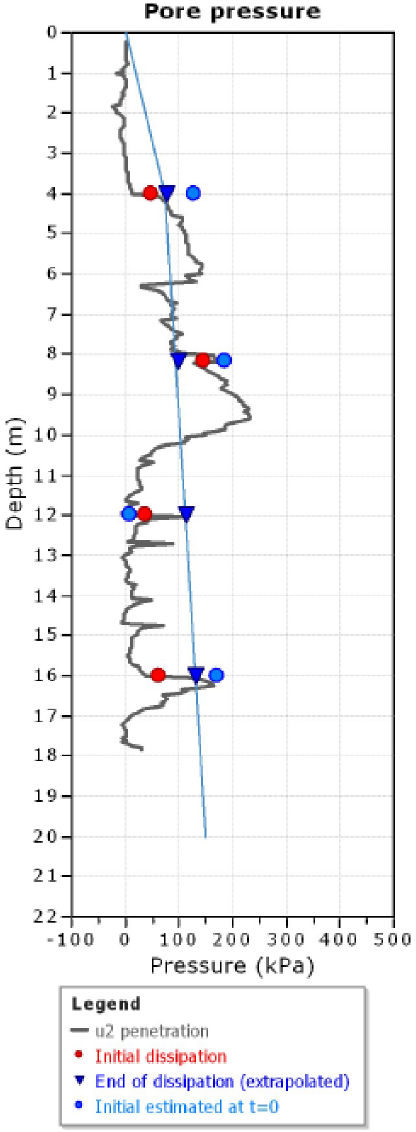
Pore pressure

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OCR vs. Depth Based on Laboratory Data

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

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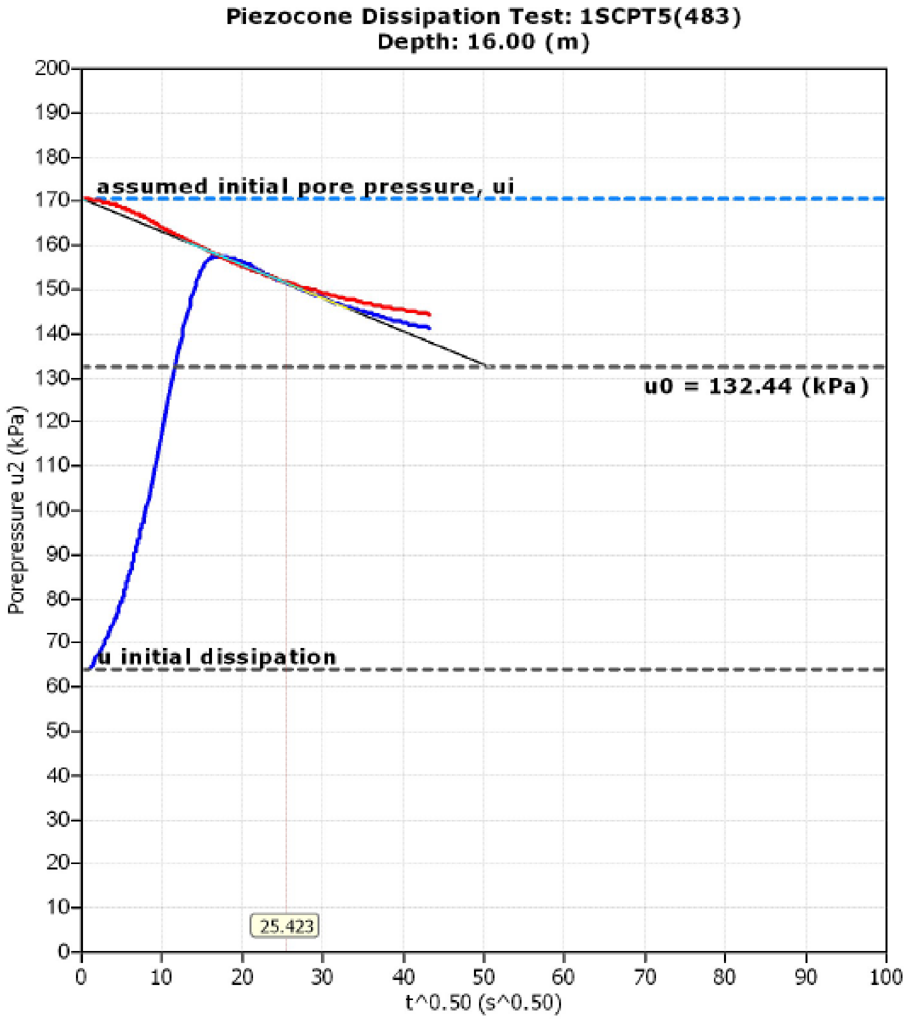
FIGURE
B-3-3



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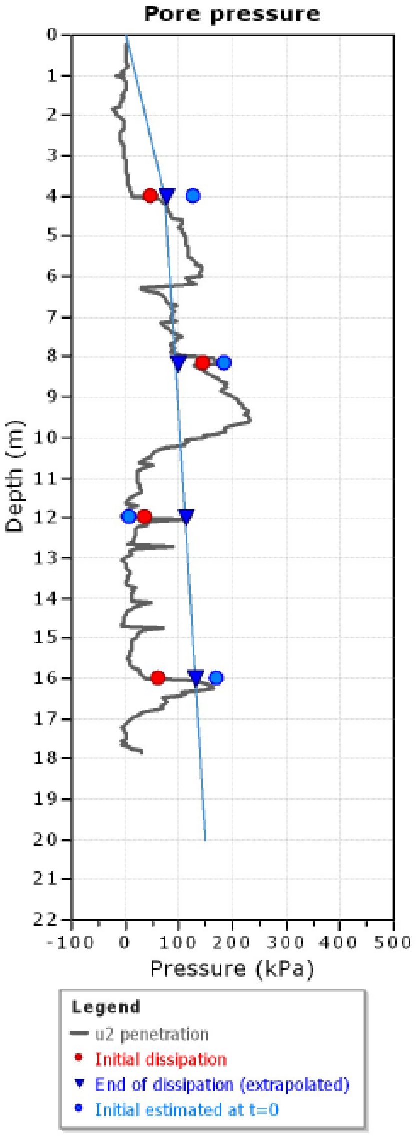

Pore pressure

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OCR vs. Depth Based on Laboratory Data

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

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FIGURE
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Appendix C.
Preliminary Environmental and Permitting Review

Appendix C

Preliminary Environmental and Permitting Review

1.0 Introduction

The City of Newport (City) is currently evaluating potential dam retrofits and replacements due to seismic concerns with Big Creek Dam No. 1 and Dam No. 2 (BC 1 and BC 2, respectively). These dams support reservoirs that provide the only source of drinking water for the City. As part of the overall assessment, HDR Engineering, Inc. (HDR) is evaluating the permits applicable to each alternative. This memorandum outlines the permits and regulatory clearances anticipated for the project, as well as the potential risks and timelines associated with the permit approval processes.

1.1 Proposed Project Alternatives

HDR identified the following three different alternatives currently being evaluated as part of a feasibility study:

- Alternative 1: raises the existing upper dam (BC 2).
- Alternative 2: constructs a new roller compacted concrete (RCC) dam. The RCC dam would be located just downstream of the upper dam, where the valley narrows to its smallest point.
- Alternative 3: constructs a new embankment dam. The location of this dam is the same as Alternative 2 (just downstream of the upper dam).

Common work elements among the alternatives include several new elements: access road, outlets works, pipeline from the outlet works of the dam to the water treatment plant, fish ladder, spillway, and intake structure with fish screens.

2.0 Anticipated Permits and Approvals

Each alternative would require permits from federal, state, and local agencies. Although the alternatives differ, the necessary work for each alternative would require the same permits and approvals described in this section. As such, this memorandum does not differentiate permitting requirements between alternatives. Discussions have not yet occurred with the permitting agencies, and it is difficult to gauge if one alternative would be more challenging to permit than another. However, it is typically easier to obtain permits for modifications to an existing structure (Alternative 1) than to permit corrective action structures (Alternatives 2 and 3). Adding additional storage versus a dam safety purpose only project likely would increase the permitting timeframe and requirements. Early coordination with the regulatory agencies during permitting would help identify key issues, potential conflicts, and mitigation strategies, and streamline the review process. Table 2-1 provides an overview of the permits and timelines, followed by a brief discussion of the major permits required for the project.

Table 2-1: Overview of Major Permits and Timelines

Required Permit	Timeline	Submittal occurs at Engineering Design Level (approximate)
National Environmental Policy Act (NEPA)	12-18 months	15-30%
Clean Water Act Section 404/401 and Oregon Removal-Fill Permit Other permits processed concurrently with applications: Endangered Species Act Section 7 Magnuson Stevens Fishery Conservation and Management Act (Magnuson Stevens Act) National Historic Preservation Act (NHPA), Section 106 Migratory Bird Treaty Act Oregon Fish Passage Coastal Zone Management Act	6-18 months	30%
Bald and Golden Eagle Protection Act (if required)	4-6 months	30%
Oregon Water Rights	9-12 months	30%
Clean Water Act Section 402 National Pollutant Discharge Elimination System (NPDES) 1200-C Permit	60 days	100%
City of Newport Conditional Use Permit	30 days	60%
City of Newport Building, Electrical, Plumbing, Mechanical, Sewer/Water Permit	30 days	100%
Oregon State Engineer Design Review and Approval	2 months	100%

Clean Water Act Section 404/401 and Oregon Removal-Fill permit – Work in the water (including wetlands) would require a Clean Water Act Section 404/401 authorization from the U.S. Army Corps of Engineers (USACE)/Oregon Department of Environmental Quality (DEQ) and approval from the Oregon Department of State Lands (DSL) for a Removal-Fill permit. Given the extent of impacts to Big Creek and possible wetlands along the creek and reservoir, the project is expected to require individual permits from each agency. Although the permit from DSL would take 120 calendar days, the permit from the USACE and DEQ would likely take between 6 and 12 months.

- Mitigation, such as on-site wetland restoration or payment into a wetland mitigation bank for permanent impacts to water (including wetlands), is anticipated. The Tamara Quays mitigation bank is located nearby, and our project is within the service area of the bank. It may be feasible to use this bank for not only wetland impacts but also impacts to waters, although further discussions with the USACE and DSL would be necessary.

Endangered Species Act (ESA) Section 7 Consultation – Big Creek discharges to the Pacific Ocean and contains ESA-listed coho salmon downstream of the water treatment facility. In addition, the surrounding forested areas contain populations of marbled murrelet and northern spotted owl, although the project area is not listed as a critical habitat for these species. Other ESA-listed species also are located in the vicinity and may require evaluation. The presence of these species would likely require evaluation as part of a Biological Assessment (BA). Currently, the federal nexus for the project is presumed to be through the Clean Water Act Section 404 permit and thus a BA would be submitted as part of this permit. Consultation likely would be formal for National Marine Fisheries Service (NMFS) and informal for the United States Fish and Wildlife Service (USFWS), where a 180-day review process is typical.

Oregon Fish Passage – The Oregon Department of Fish and Wildlife (ODFW) requires any project on an artificial obstruction to fish passage that would have a fundamental change in permit status (OAR 635 412 0020) and include a structural modification to the feature that would increase storage (OAR 625 412 005(9)(c)) to evaluate effects to fish passage. Given the current and historical presence of migratory fish species (e.g., coho salmon and steelhead), ODFW would request the project provide fish passage during construction and operation and that work occur during the in-water work window of July 1 through September 15. Each alternative includes a new fish passage structure that would meet this requirement. The fish passage requirements are included as part of the DSL Removal-Fill permit process.

The current feasibility study does not include modification or removal of Dam No. 1, which is located downstream of each alternative. However, if the water released through the new or modified dam structure is altered and affects the flow through the existing fish passage structure, an evaluation of fish passage and possible upgrades may be required at Dam No. 1.

Oregon Water Rights – The City has certified water storage rights associated with the existing storage in each of the Big Creek reservoirs. The proposed storage options would increase the storage volume of both reservoirs for projected long-term water supply needs by the city and recovery of storage due to sediment accumulation. The proposed project would also potentially change the points of water diversion. As such, the City would have to submit separate water rights application to the Oregon Water Resources Department (OWRD) to change the point of diversion, storage volume, and use of stored water from the reservoirs. As part of that process, ODFW would review the application for potential conditions to add to the water right prior to approval. OWRD has a significant backlog for processing water rights applications. An expedited review process can be used through OWRD's "reimbursement authority" program, where the process could take six to nine months. Fees can cost up to $5,000, dependent on volume of storage and whether OWRD considers the process a water rights transfer or new storage.

In addition to the major permits described above, there are several other permits that may be required. Permits anticipated for the project are summarized in Table 2-2, including an overview of the process, timeframes, and risks.

Table 2-2: Anticipated Permits and Approvals

Permit / Approval and Responsible Agency	Triggers and Process	Timeframe and Risks
NEPA Lead Federal Agency	**Trigger:** • Federal permit or approval required; siting on federal lands; receipt of federal grants or funds. • This project would require approval from several federal agencies, including the USACE. **Process:** • Prior to issuing a federal permit or approval, a federal agency must ensure it has complied with NEPA. • The lead federal agency would need to be determined based on the appropriate federal "nexus." At this juncture, the USACE is presumed to be the lead federal agency for the project. This could change if federal funding from another federal agency is issued. • The process to conduct NEPA compliance depends on how the agency implements NEPA in its review process; for example, an agency may require a separate NEPA document preparation track, or may incorporate the review into its internal review process. • The level of environmental review (Categorical Exclusion [CE], Environmental Assessment [EA], or Environmental Impact Statement [EIS]) depends on the potential effects of the project and standards of the lead agency in determining if those effects are significant.	**Timeline:** • Project processed as a CE or EA – 4 to 12 months. • Project processed as an EIS – 12 to 18 months. **Risks**: Potential for significant adverse impacts to sensitive resources can prompt an agency to consider preparing an EIS. An EIS is a lengthy process that would require additional time and effort.

Table 2-2: Anticipated Permits and Approvals

Permit / Approval and Responsible Agency	Triggers and Process	Timeframe and Risks
Clean Water Act Section 404 U.S. Army Corps of Engineers (USACE)	**Trigger**: • Permanent or temporary discharge of fill in waters of the U.S. including wetlands. **Process**: • The type of activity and degree of alteration to the waters of the U.S. determines the level of review. This project would likely require an individual permit. • Submit a Joint Permit Application (JPA) that includes project plans, biological information (i.e., BA), wetland delineation, and other pertinent information. • A Compensatory Mitigation plan may need to be developed prior to completion of the permit application if resources are permanently affected. • Temporary impacts would require development of a restoration plan to be included as part of the Compensatory Mitigation plan. • Fee of $100 is required but rarely requested.	**Timeline**: • Individual permit is a 6- to 18-month process after permit application is deemed complete. • 30-day public notice is required. **Risks**: • If consultation under the ESA is required, the timeline for issuing the USACE approvals would include this consultation. • If extensive coordination is required under the National Historic Preservation Act (NHPA), this would need to occur prior to the USACE permit being issued.
Clean Water Act Section 401 Water Quality Certification DEQ	**Trigger**: • Any federal agency issuing a permit or an approval must comply with Section 401 of the Clean Water Act; DEQ has been delegated the federal jurisdiction to perform Section 401 review for projects in Oregon. For this project, the approval would be processed as part of the USACE Clean Water Act Section 404 permit. **Process**: • Review Clean Water Act Section 404 permit to determine if the project would affect beneficial uses of waters (including wetlands). • Stormwater, erosion, and sediment control plans would be required if more than 1 acre of disturbance.	**Timeline**: • Concurrent with Clean Water Act 404 permit process. • A DEQ certification decision is made within 90 days after an application is deemed complete; however, for complex projects it may take up to one year to receive certification. In practice most certifications are processed in less than one year.

Table 2-2: Anticipated Permits and Approvals

Permit / Approval and Responsible Agency	Triggers and Process	Timeframe and Risks
Federal ESA USFWS and NMFS	**Trigger**: • Any federal agency issuing a permit or an approval must comply with the federal ESA **Process**: • The applicant would conduct appropriate literature and field studies to identify the potential presence of federally-listed species at the project site. Based on preliminary database searches, both aquatic and terrestrial ESA-listed species may be present. • If ESA-listed species or their protected habitat is present at the site or in the area potentially affected, the federal agency issuing the permit must review potential impacts and, if needed, conduct Section 7 consultation with the Service responsible for the species. • Consultation can be “informal” (i.e., not likely to adversely affect), or “formal” (i.e., likely to adversely affect). However, this project is expected to require formal consultation. • Requires preparation of a BA if federal ESA-listed species would be potentially affected by the proposed project.	**Timeline**: • Concurrent with the review process by the federal agency undertaking consultation, but may add time to the agency’s approval timeline. • The informal consultation process takes approximately 135 days. • The formal consultation takes approximately 180 days. **Risks**: Potential adverse impact to a protected species or its habitat can significantly lengthen the overall permit/approval process and require off-setting actions (i.e., mitigation).
Magnuson Stevens Act NMFS	**Trigger**: • Review required for potential impacts to Essential Fish Habitat (EFH) for ocean species and all anadromous fish throughout their migratory range. **Process**: • The applicant would conduct appropriate literature and field studies to identify the potential presence of anadromous fish species at the project site. Based on preliminary database searches, no anadromous fish are present. • Included as part of the BA.	**Timeline**: • Concurrent with ESA Section 7 consultation.

Table 2-2: Anticipated Permits and Approvals

Permit / Approval and Responsible Agency	Triggers and Process	Timeframe and Risks
NHPA Section 106 Oregon State Historic Preservation Office (SHPO)	**Trigger**: • Any Federal Agency issuing a permit or an approval must comply with the federal NHPA. **Process**: • The applicant would conduct appropriate literature and field studies to identify the potential presence of cultural and archeological resources at the project site. • The NHPA requires consideration of potential project-related effects on properties listed, or eligible for listing in the National Register of Historic Places as well as cultural resources. In particular, Section 106 of the NHPA requires federal agencies to consult with SHPO to determine if activities may affect historic properties or cultural resources. • SHPO is also required to consult with local Native American Tribes regarding cultural resources. • If the project is determined to adversely affect a potentially eligible property or cultural resource, preparation of Determinations of Eligibility and Findings of Effect would be required.	**Timeline**: • Section 106 is processed concurrently with either NEPA or Clean Water Act Section 404 permit. **Risks**: Potential for significant adverse impact to Tribal cultural or archeological resources may require preparation of a Memorandum of Understanding with affected Tribes.
Clean Water Act Section 402 – NPDES Permits DEQ	**Trigger**: • Clearing, grading, and excavation that disturbs 1 acre or more of land. **Process**: • Adherence to the Clean Water Act Section 402 requires NPDES stormwater permits from DEQ. • As with Clean Water Act Section 401, a stormwater plan and Erosion and Sediment Control Plan would need to be prepared for these activities.	**Timeline**: • This permit is processed approximately 60 days prior to construction. • There are two public notices with a 30-day public comment period.
Migratory Bird Treaty Act (MBTA) USFWS	**Trigger**: • Under the MBTA, taking, killing, or possessing migratory birds is unlawful, except as authorized under a valid permit. **Process**: • Measures are usually part of the construction specifications and include timing certain activities outside of nesting and mating season, removing trees outside of the nesting season, or conducting individual tree nest clearances.	**Timeline**: • No specific permit is required.

Table 2-2: Anticipated Permits and Approvals

Permit / Approval and Responsible Agency	Triggers and Process	Timeframe and Risks
Bald and Golden Eagle Protection Act USFWS	**Trigger**: • Potential impacts from construction or operation that would harass or harm bald and golden eagles. **Process**: • If an eagle roosting area or nest is within 0.5 mile of the project an analysis of visual and noise effects is required. If no effect would occur, no further documentation is required. • If the analysis determines that the project will affect eagles, a permit from the USFWS would be required. The permit includes a brief project description, effects analysis, and general site plans.	**Timeline**: • This permit does not have a specific regulatory timeline for issuance but is typically issued within 3 to 4 months.
CZMA Oregon Department of Land Conservation and Development (DLCD)	**Trigger**: • Activities and development affecting coastal resources that involve federal activities, federal licenses or permits, and federal assistance programs (funding) require written Coastal Zone Management (CZM) federal consistency determinations by the DLCD. **Process**: • As part of the JPA submittal, the applicant completes Federal Consistency documentation. The USACE and DSL provide the documentation to DLCD for review. • DLCD provides either written concurrence or objects to the consistency determination. • Public review of the consistency documentation occurs as part of the public notice requirements associated with the Clean Water Act Section 404 and Removal-Fill permits.	**Timeline**: • Concurrent with Clean Water Act Section 404 and Removal-Fill permit processes. • Certificate of Consistency is issued approximately 45 to 90 days after permit application is deemed complete. Complex projects can take up to 6 months. **Risks**: Potential for delays due to public comments received during the public notice period.

Table 2-2: Anticipated Permits and Approvals

Permit / Approval and Responsible Agency	Triggers and Process	Timeframe and Risks
Oregon Water Rights OWRD	**Trigger**: • Construction and/or modification of a new or existing dam (change in storage volume or point of diversion associated with dam outlet). **Process**: • Water right transfer and/or new storage water right applications prepared for change in point of diversion, storage of water, and use of stored water. • For municipal agency, application requires general land use information, preliminary plans and specifications (conceptual design level), and appropriate mapping. • OWRD review process includes application completeness review, initial review, public notice period, proposed final order and public notice period, and issuance of final order. • ODFW conducts a review and provides conditions to the water right, as needed prior to issuance of proposed final order. • Engineered plans and specifications must be approved prior to storage of water.	**Timeline**: • Applications can be submitted to OWRD as soon as conceptual level design is available. • 30-day and 45-day public notice/protest periods are part of the process. • Under expedited review process, applications could take 9 to 12 months. **Risks**: Potential for delays due to public protests prior to issuance of final order.
Oregon Fish Passage Approval ODFW	**Trigger**: • ODFW is responsible for reviewing and approving projects that may affect fish passage. Any in-water work, whether temporary or permanent, would require adherence to the fish passage laws and in-water work timing. **Process**: • An application for fish passage is prepared and submitted to ODFW prior to or concurrently with submittal of the DSL Removal-Fill Permit. ODFW would review the project and provide a recommendation to DSL. • An isolation and fish recovery plan would be required with the permit submittal (to both ODFW and DSL) and implemented during construction. • Fish capture and release efforts require a Scientific Sampling Permit from ODFW and NMFS (if federal ESA species are present).	**Timeline**: • Concurrent with DSL Removal-Fill permit process. • Fish passage is implemented through the DSL Removal-Fill permit process.

Table 2-2: Anticipated Permits and Approvals

Permit / Approval and Responsible Agency	Triggers and Process	Timeframe and Risks
Oregon Endangered Species Act ODFW and Oregon Department of Agriculture	**Trigger**: • Potential impacts to state listed wildlife, fish, and plant species as a result of project implementation. **Process**: • State ESA protection is limited to state-owned land, state-leased land, and land over that the state has a recorded easement. Based on known information, the project does not occur on state land, and thus the state ESA does not apply.	**Timeline**: • Concurrent with DSL Removal-Fill permit process. • If no DSL permit is required, additional coordination may occur during the ESA Section 7 consultation. **Risks**: Potential adverse impact to a protected species or its habitat can significantly lengthen the overall permit/approval process.
City of Newport Conditional Use Permit City of Newport Community Development Department	**Trigger**: • The alternatives are within Residential Low Density Single Family (R-1) or Public Structure (P-1) zoning depending on the alternative. This would require a conditional use permit from the City. **Process**: • It is anticipated that the conditional use permit would be processed as a Type II decision. Application including narrative and plans is submitted to the City for approval.	**Timeline**: • Typically processed in 30 days once application is deemed complete. However, state law provides a statutory timeline of 120 days to process the application and given the uniqueness of the project, the process may extend to the 120-day timeline.
City of Newport Building, Electrical, Plumbing, Mechanical, Sewer/Water Permit City of Newport Building Department	**Trigger**: • The dam alternative locations are within Lincoln County. Construction and/or modification of a dam would trigger the need for building, electrical, plumbing, mechanical, sewer/water permits. **Process**: • Application including plans is submitted to the County for approval. • Each permit is a separate process. • These are typically obtained immediately prior to construction as information regarding the contractor is required for the application.	**Timeline**: • Typically processed in 30 days once application is deemed complete.

3.0 Additional Studies

Permitting can pose risks to a project in terms of schedule and cost due to unanticipated complex permit reviews during the project development stage or permit terms and conditions and environmental resource mitigation requirements. Such risks can result in an increase to the cost of construction or operation of the project. Risks associated with complex permitting and stringent permit terms and conditions can result from lack of advance knowledge of the potential impact to sensitive environmental resources or public controversy, including, but not limited to:

- Working in or adjacent to state and U.S. waters, including wetlands;
- Presence of fish, wildlife, and plant species, or their associated habitat;
- Presence of cultural or archeological resources; and
- Public apprehension or opposition to the project.

Identification of the issues that pose risks and avoidance of impacts to the greatest degree to avoid the need for permits are important elements of early project development. To identify these issues and other potential permit requirements, the following studies are recommended:

- Wetland and waters delineation: Although this can be done year-round (weather permitting), it is best to conduct the delineation in late spring or early summer when hydrology is present.
- Coordination with the regulatory agencies to identify potential ESA-listed species: Species-specific surveys are typically not performed but because species are known to be present, a BA would be required for ESA Section 7 consultation.
- Cultural and archeological resources investigations, such as pedestrian surveys and literature review.
- Mitigation would be required for permanent and temporary impacts to water resources. Mitigation site selection or use of a mitigation bank would need to occur prior to submittal of the permits in order to prepare a mitigation plan. Any temporary impacts would require restoration and be included in the mitigation plan.
- An Emergency Action Plan (EAP) was completed for the existing dam in 2009. As part of the approval process with the OWRD, the EAP may require updating.

If early coordination with resource agencies and identification of these issues are completed, HDR anticipates that permits could be obtained in 12 to 18 months from time of submittal of a complete application.

4.0 Potential Costs

Costs for permitting, including initial studies and investigations for wetland/waters delineations, fisheries and aquatic resources, terrestrial resources, botanical and rare plant surveys, historic and archaeological surveys, and permit application development, can range from 1 to 6 percent of the overall construction costs, depending on the project magnitude and scope. This does not include permit application or renewal fees for multi-year projects.

Appendix D.
Engineering Analyses

Appendix D-1. Engineering Properties and Updated Evaluation Existing Dams BC 1 and BC 2

Appendix D-2. Seismic Response Evaluation of RCC Dam Alternative A-2

Appendix D-3. Evaluation of Embankment Dam Alternative A-3

Appendix D-1

Engineering Properties and Updated Evaluation
Existing Dams BC 1 and BC 2

1.0 Introduction

This document summarizes the engineering properties of the embankment and foundation soils/bedrock required to assess seepage conditions and associated water pressures and gradients in the dam and foundation, along with the potential for liquefaction or cyclic strength degradation and the corresponding shear strength values to be used in slope stability and seismic response analyses. Figures D-1.1 and D-1.2 show the generalized cross sections of the Big Creek 1 (BC 1) and Big Creek 2 (BC 2) dams used in the updated evaluation of the existing dams.

2.0 Engineering Property Characterization

Engineering properties of the embankment and foundation soils/bedrock are required to complete an updated assessment of 1) seepage conditions including water pressures and gradients in the dams and dam foundations, and 2) the potential for liquefaction or cyclic strength degradation and corresponding shear strength values to be used in slope stability and seismic response analyses. The updated assessment of these parameters is outlined in the following sections.

2.1 Permeability (K)

An estimate of the steady-state seepage phreatic water surface through the dam and foundation is required for stability and seismic response evaluations. To estimate the location of the phreatic surface, vertical permeability (K_v), horizontal permeability (K_h), and ratio of vertical to horizontal permeability (anisotropy) of the embankment and foundation soils at the two dam sites are required.

Permeability values from the previous analysis were selected based on a variety of published sources of information including values developed through extensive testing for major levee improvements for the Natomas Levee Improvement Program in the Sacramento River basin near Sacramento, California (Board of Senior Consultants [BOSC] 2010). A summary of estimated permeability values for a wide range of soil types adopted for these evaluations were shown in Table 7 of Big Creek Dam No. 1 and No. 2, Preliminary Geotechnical Investigation and Seismic Evaluation for City of Newport, Oregon (HDR 2013). Using a combination of the t_{50} values (time to 50 percent pore pressure dissipation) from the dissipation testing and University of British Columbia (UBC) equation, permeability values were estimated and summarized in Table 7 (HDR 2013) were revised. The estimated permeability of the internal gravel and toe drains remained the same as those presented in the previous, Phase 2, analyses. It was noted that blanket drains were installed in both dams during construction. A review of the available construction documents found that there were no specifications for these materials. Further, blanket drain materials were not sampled and tested during either the Phase 2 or 3 site exploration programs. For the analyses, HDR has assumed the blanket drains were constructed from slightly silty fine sand (with approximately 3 to 7 percent fines).

Laboratory permeability tests were not performed as part of this or previous evaluations. However, as part of the Phase 3 site characterization work, Cone Penetration Tests (CPTs) with pore water pressure measurement capabilities (SCPTu) and associated pore water pressure dissipation testing was performed at various depths throughout the foundation soil profiles at each site. The dissipation testing results are presented in Appendix B Site Characterization. Permeability values estimated by the CPT software (Geologimiki 2014) were compared with those calculated using the UBC (UBC 2006) equation, based on the t_{50} values. These site specific values were also compared to the values previously used for the analysis cross sections, which were developed based on index test results and classification of material type. In general, the calculated permeability values from the recent SCPTu dissipation testing were similar to those developed using correlations between soil types and permeability. The anisotropy of the foundation soils cannot be computed with dissipation testing, so the previous analysis (Phase 2) values were adopted for the current evaluation.

A summary of permeability values and K_v/K_h (anisotropy) ratios used in the updated evaluations are presented in Table D 1.

Table D 1 Permeability Values used in Seepage Analyses, BC 1 and BC 2

Soil Type	Kv (ft/day)		Kv/Kh
	Lower Bound	Upper Bound	
MH	0.0003	0.0030	0.25
SM/ML	0.01	0.10	0.25
SP-SM (blanket drain)	0.40	0.40	1

MH = high plasticity silty soils
ML = low plasticity silty soils
SP-SM = poorly graded sand to silty sand
SM = silty sand soils

2.1.1 Soil Strength Parameters

Shear strength parameters are required for the analysis of existing static (pre-earthquake) and post-earthquake loading conditions. The parameters were estimated for each soil type shown in the representative BC 1 and BC 2 cross sections illustrated on Figure D-1.1 and Figure D-1.2, respectively. Static and post-earthquake strength parameters were developed from interpretation of site characterization information including the results of laboratory testing, Standard Penetration Test (SPT), and Seismic Cone Penetration Test (SCPTu) data, and correlations with soil index properties. The previous analyses performed by HDR (2013) and the CH2MHill preliminary design report (1974) were reviewed to provide context that was used in addition to the current HDR analyses.

The Phase 2 engineering analyses used post-earthquake strengths that were developed using a two-step process. First, a general determination was made on an expected “sand-like” or “clay-like” behavior. For those embankment and foundation materials that are expected to have a “clay-like” behavior, estimates of the peak undrained shear strength (S_u) of the embankment and foundations soils were made based on the results from the SCPTu. Using the estimates of peak strength and results of a single laboratory cyclic simple shear test, an estimate of the amount of strength degradation was made to establish the post-earthquake shear strength input to the stability analysis models. For the foundation materials that were estimated to have a more “sand-like” response to earthquake loads, the post-earthquake residual strength (also referred to as post-earthquake steady state strength) for the potentially liquefiable sand-like soils was estimated using the relationship proposed by Seed and Harder (1990) as shown on Figure D-1.3. Seed (2010) calculated a least squares fit through the Seed and Harder (1990) data. This relationship (red dashed curve) was used to estimate the post-earthquake strength of the sand-like soils (Plasticity Index [PI]<7).

For Phase 2, the strengths assumed for the generalized BC 1 cross section were based on individual stratigraphy at the SCPTu locations, while the strengths for the generalized BC 2 cross section BC 2 were based on a range of soil types, and estimated self weight stresses throughout the full soil depth. The Phase 2 analysis soil properties are summarized in Table D 2, Table D 3, and Table D 4. The Phase 2 undrained strengths accounted for the overburden stress by subdividing the soil into layers and manually inputting the undrained strengths. This methodology used a Mohr-Coulomb material model. As will be subsequently discussed in Section 2.1.2, this approach was modified during Phase 3 to modeling strength as a function of overburden pressure or Stress History and Normalized Soil Engineering Properties (SHANSEP) as used in Phase 3.

As part of the Phase 2 analyses, silty sand layers were assumed to be “sand-like” soils having post-earthquake strengths ranging from 0.08 to 0.29 kips per square foot (ksf). Drained strengths were used for the peak values associated with steady state conditions. The higher plasticity silts were modeled as “clay-like” soils having peak undrained strengths that ranged from 0.50 to 1.93 ksf, and post-earthquake strengths from 0.34 to 1.93 ksf.

Table D 2. Strength Values for Post-earthquake Slope Stability Analysis based on BC1-CPT-3 Embankment and Foundation Conditions at BC 1

Elevation		Interpreted Soil Type	Undrained Shear Strength (ksf)	
From	To		Peak-	Post-earthquake
47	32	Clayey Silt	1.0	0.67
32	20		0.80	0.54
20	0		0.75	0.50
0	-34	Silty Sand	-	0.20

Table D 3. Strength Values for Post-Earthquake Slope Stability Analysis, BC1-CPT-4 Profile, Dam BC 1

Elevation		Interpreted Soil Type	Undrained Shear Strength (ksf)	
From	**To**		**Peak**	**Post-Earthquake**
47	40	Clayey Silt (embankment fill)	1.0	0.67
40	25		0.75	0.50
25	10		0.65	0.44
10	5	Silty Sand (embankment fill)	-	0.2
5	-23	Clayey Silt (foundation alluvium)	0.60	0.40
-23	-34		0.50	0.34

Table D 4. Estimated Undrained Strength Values for Slope Stability Analyses, based on results of CPTu Soundings BC2-CPT-1, BC2-CPT-2, and BC2-CPT-3, Dam BC 2

Elevation		Interpreted Soil Type	Undrained Shear Strength (ksf)	
From	**To**		**Peak**	**Post-Earthquake**
50	47.5	Clay-like Soil	-	-
47.5	45		1.93	1.93
45	42.5		1.52	1.52
42.5	40		0.55	0.36
40	37.5	Sand-like Soil	-	0.25
37.5	35		-	0.18
35	30		-	0.25
30	27.5		-	0.15
27.5	25		-	0.29
25	20		-	0.08

In Phase 2 an evaluation of the SPT $N_{1,60}$ values (SPT blow counts normalized to 1 ton per square foot overburden pressure and an applied hammer efficiency of 60%) and the liquefaction potential of the sand-like soils at both dam sites indicates that SM and ML materials at the dam sites have the potential to liquefy due to an earthquake on either the Yaquina or Cascadia Subduction Zone (CSZ) faults. These materials were estimated to have reasonably good strength under static loading conditions; however, they have the potential to lose signification strength during an earthquake event. The Phase 2 site characterization program also questioned the continuity of these materials at both dam sites. Similarly, previous investigations suggested there is the potential for cyclic softening and loss of strength of some of the “clay-like” MH embankment and foundation soils during and immediately following either earthquake loading condition. It was also recognized that the high plasticity soils (MH) at the site are uncommon materials for which only limited material property characterization research has been performed. These factors led to the need for the Phase 3 investigation program.

The Phase 3 site characterization program was focused on providing a more extensive basis for the site characterization models, generalized cross-sections, and associated engineering parameters to be used to analyze the existing dams. The Phase 3 program included additional sampling, laboratory testing and in-situ testing with SCPTu and Field Shear Vane (FSV). The distinction of whether a material was "clay-like" or "sand-like" in behavior was re-evaluated based on the updated geologic models for both dam sites including estimates of the extent and continuity of the more "sand-like" soils. Professor Jason DeJong, a nationally recognized expert in soil behavior from the University of California at Davis, was brought onto the project team to assist with the characterization of the high and low plasticity silty (MH and ML) soils and silty sand (SM) soils encountered during the exploration programs of Phases 2 and 3.

The Phase 3 exploration program indicated that the subsurface profiles at each of the sites consisted primarily of high plasticity silts (MH) with relatively discontinuous lenses of low plasticity silt (ML) and silty sand materials (SM). The characterization of the engineering properties of the foundation soils focused on the predominant MH materials along with the lenses of SM soils as summarized in the sections below.

2.1.2 Strength Parameter Selection

The Phase 2 exploration program for BC 1 dam, used information from two borings, BC1-B-1 and BC1-B-2 and four CPTu soundings to assess the static and post-earthquake shear strength of the soils used in stability evaluations. The current exploration added two undisturbed sample borings, BC1-BH-3(u) and BC1-BH-4(u), along with two SCPTu soundings with pore water pressure measurement capabilities BC1-SCPT-5 and BC1-SCPT-6. The addition of the undisturbed (u) sample borings provided "high quality" samples for laboratory testing along with additional SCPTu data, including water pressures and water pressure dissipation testing. The undisturbed borings were performed using an Osterberg piston sampler to minimize disturbance effects.

For BC 2 dam, the previous exploration consisted of three borings BC2 B-1, BC2 B-2, and BC2 B-3 along with three CPTu soundings BC2 CPT-1, BC2 CPT-2, and BC2 CPT-3. Two of the borings BC2 B-1 and BC2 B-2 and all of the CPT soundings were drilled from the crest of the dam, while boring BC2 B-3 was drilled near the toe at the right abutment, see Figures B.1 and B.2 in Appendix B Site Characterization.

The Phase 3 exploration program at BC 2 dam was done along the downstream toe of the dam and consisted of one SPT boring (BC2 BH-4), two undisturbed borings (BC2 BH-5(u) and BC2 BH-6(u)), and four SCPT soundings (BC2-SCPT-4, BC2-SCPT-5, BC2-SCPT-6, and BC2-SCPT-7).

<u>Static Shear Strength</u>. Estimated minimum factors of safety (FOS) for static loading conditions (long-term steady state seepage conditions), were performed for both dams using estimates of drained (effective stress) strength parameters (e.g., USACE 2003). The effective stress friction angle for the high plasticity silt (MH) soils were estimated based on laboratory PI determinations (Mitchell 1976) and the soil mineralogy. For an average PI of 30 for the high plasticity silt embankment soils, a drained friction angle of 34 degrees was selected, which was contrasted with the Phase 2 value of 36 degrees. For the silty sand (SM) foundation soils in the borings, the

drained friction angle was estimated using equivalent $N_{1,60}$ values estimated from the CPTu, SCPT, and SPT profiles. For an average $N_{1,60}$ of 4 blows per foot (bpf) and based on engineering judgment, a drained friction angle of 34 degrees also was estimated (Mayne et al 2001). A cohesion of 0.2 and 0.1 ksf was included for both the embankment and foundation soils, respectively, to reflect the expected curvature of the failure envelope in the low effective stress range and minimize the influence of shallow (infinite slope) failure surfaces on the estimates of the location and minimum FOS during stability analyses. A summary of the drained shear strength parameters used for static stability evaluations of both dams is presented in Table D 5.

Table D 5. Strength Values for Pre-Earthquake Static (Steady-State) Slope Stability Analysis

Material Type	Effective Stress Parameters		Total Unit Weight (pounds per cubic foot [pcf])
	c′ (ksf)	ϕ′ (degrees)	
Embankment Fill (MH, ML and SM soils)	0.2	34	110
Foundation Alluvium (MH, ML and SM soils)	0.1	34	100

c’ – effective cohesion, ϕ’ – effective friction angle

Undrained and Post-Earthquake Strength. Undrained and post-earthquake soils strengths were estimated for the Phase 3 engineering evaluations based on laboratory test results and SCPT data interpretation methods. The initial step of the Phase 3 laboratory testing consisted of both constant rate of strain (CRS) and load increment ratio (LIR) consolidation to estimate the stress history with depth of the soil deposits and provide preconsolidation stress values for use in the strength testing. The second step included strength testing by direct simple shear (DSS) and isotropically consolidated triaxial (CIUC) with pore pressure measurements testing protocols. Post-earthquake strength reduction was estimated using cyclic DSS (CycDSS) and index testing correlations based on PI values. SCPT testing results were used to assist in estimating the undrained strengths and provide correlations to other parameters.

The unique nature of volcanic soils present at the dam sites introduced some problems with performance and interpretation of both laboratory and in-situ testing results. Specifically, the results of dissipation testing performed in the SCPT soundings showed that some drainage was occurring during the penetration in some of the foundation soils. Hence a portion of the SCPT results were indicative of partially drained conditions rather than undrained conditions for which correlations to strength and other engineering properties are based. Once the laboratory testing was completed, the cone factor, N_{kt}, was calibrated to the laboratory undrained strength, S_u, to provide a more accurate and complete picture of the strength variation with depth (

Attachment A). This provided a consistent (constant) lower bound to the values of strength with depth. A similar problem was encountered during vane shear testing (VST) performed in boring BC1 BH-4. Similar to the CPT sounding, the VST results were influenced by the drainage conditions of the soils and yielded drained strength parameters. Vane blades were damaged as a result of this condition and the VST was discontinued.

DSS testing was carried out on undisturbed samples taken from various depths throughout the soil profile. CRS and LIR consolidation testing provided the pre-consolidation stress values that were used in the strength testing phase to consolidate the soils to various levels of over-consolidation ratios (OCRs). The soils were tested at various OCRs as part of the stress history and normalized engineering properties (SHANSEP) method (Ladd and Foott 1974). A plot of the estimated OCR of foundation soil samples as a function of depth is presented in Figure D-1.4.

While typically used for normally consolidated clays, the SHANSEP method has been applied effectively to other cohesive and moderately cohesive soils. The equation for characterizing the undrained strength of soils as a function of overburden stress in the SHANSEP method is provided below:

$$\frac{S_u}{\sigma_v'} = S(OCR)^m$$

Figures D-1.5 and D-1.6 show plots of the normalized strength parameter of undrained strength divided by the effective vertical pressure (S_u/σ_v') versus the OCR. The plots of the different samples from BC 1 and BC 2 show that the values of S_u/σ_v' range from about 0.22 to 0.23 (at normal consolidation OCR = 1) with an exponent “m” that adjusts the strength with OCR that ranges from about 0.80 to 0.94. Also plotted are the values of normalized strength versus OCR for samples that were not subject to the SHANSEP testing method. They plot slightly above the lines from the SHANSEP method and follow the general trend of the SHANSEP curves. The SHANSEP curves provide a reasonably conservative boundary to the strengths of the other undisturbed samples.

As previously noted, during Phase 2, the post-earthquake strengths were developed using a process where a general determination was made based on whether the soil was expected to behave as “sand-like” or “clay-like” materials. These evaluations were based on the CPTu results using a typical N_{kt} (cone) factor. However this cone factor was not calibrated to the laboratory testing for the specific materials. Both DeJong (2014) and HDR used the laboratory data to adjust the SCPTu results by adjusting the cone factor N_{kt} to a value of 22, providing a more consistent basis with which to evaluate the SCPTu data and adjust for the range of drainage conditions observed.

The current testing program provided the undrained strengths from the DSS testing and the CycDSS testing was used to evaluate the degradation of the materials due to cyclic loading. The majority of cyclic testing indicated little to no degradation of the materials due to cyclic loading (Figure D-1.7). Based on a correlation with plasticity, a reduction in strength due to cyclic loading of approximately 20 percent was selected for Phase 3 engineering evaluations (Figure D-1.8).

Immediately after completion of the cyclic test (CycDSS), a monotonic simple shear test was performed to evaluate the post-cyclic undrained shear strength. The results of the post-cyclic undrained shear testing are shown in Table D-1.A.4 in Attachment A. Results of these tests generally show little to no strength reduction due to the cyclic loading.

3.0 Summary of Results of Updated Engineering Evaluations of the Existing Dams

The following section outlines the results of updated seepage, slope stability and Newmark deformation analyses of the existing dams.

3.1 Seepage Analyses Results

Generalized cross sections for engineering analyses are shown on Figures D-1.1 and D-1.2 for BC 1 and BC 2, respectively. The upper figures of Figures D-1.1 and D-1.2 relate to the permeability of the materials rather than the strength characterization and indicate the stratigraphy used as related to permeability.

As previously discussed, the values of permeability were selected based on SCPTu dissipation test results and interpreted using Lunne et al. (1997), UBC (2006) and Robertson (2009). From these interpreted values the upper and lower bound estimates were made for materials and an interpreted stratigraphy was developed based on order of magnitude estimates, with the “lower” materials being interpreted as more permeable than the “upper” materials as shown in the above referenced figures. The previously presented Table D 1 lists the permeability values used for the analysis.

Figures D-1.9 and D-1.10 illustrate the approximate phreatic surfaces and head contours for BC 1 and BC 2 dams, respectively. As previously stated, the differences between the upper and lower bound parameters yielded little difference in the location of the phreatic surface, head, and gradient contours.

3.2 Slope Stability Analysis Results

Phase 3 slope stability analyses were performed for the upstream and downstream slopes at both the BC 1 and BC 2 dams. Seepage parameter assumptions made little difference in the phreatic surface of the lower (BC 1) dam and did not result in FOS values that differed between the two seepage parameter cases. The two seepage parameter cases for the upper (BC 2) dam resulted in slight differences, with the upper bound (Case 2) seepage parameters resulting in lower calculated FOS. Figures D-1.1 and D-1.2 provide schematic dam sections for the BC 1 and BC 2 seepage and stability analyses. Results of seepage analyses, head contours and phreatic surfaces are shown in Figures D-1.9 and D-1.10 for BC 1 and BC 2, respectively.

In general, based on the strength parameters estimated from the laboratory testing program, the FOS values calculated indicate the dam is stable under drained, peak undrained and post-earthquake conditions at full reservoir loading. The FOS values for these conditions do not yield the potential for a stability failure due to the ground shaking.

Table D 6. Slope Stability Analysis Results for BC 1

Section	Factor of Safety			
	Case 1[(1)]		Case 2[(1)]	
	DS	US	DS	US
Drained Strength Parameters	3.43	3.75	3.43	3.75
Peak Undrained Strength Parameters	2.28	3.83	2.28	3.83
Post Earthquake Undrained Strength Parameters	1.81	3.07	1.81	3.07

Case 1: Lower Bound Seepage Parameters
Case 2: Upper Bound Seepage Parameters

Table D 7. Slope Stability Analysis Results for BC 2

Strength Envelope	Factor of Safety			
	Case 1[(1)]		Case 2[(1)]	
	DS	US	DS	US
Drained Strength Parameters	1.39	3.67	1.32	3.26
Peak Undrained Strength Parameters	1.35	3.28	1.33	3.26
Post Earthquake Undrained Strength Parameters	1.50	2.62	1.47	2.61

Case 1: Lower Bound Seepage Parameters
Case 2: Upper Bound Seepage Parameters

Graphical results are shown in Figures D-1.11 to D-1.13 for BC 1 and Figures D-1.14 to D-1.16 for BC 2.

4.0 Newmark Deformation Analysis Results

Newmark sliding block analyses were performed for the BC 1 and BC 2 dams in their existing configurations. In addition to the rigid block analyses, both coupled and uncoupled sliding block analyses were performed.

Slope stability analysis using both the peak undrained and post-earthquake strengths were used to evaluate the yield acceleration of each cross section of the dam. The pseudo-static slope stability is performed and the seismic coefficients are varied until the FOS is approximately 1.0 (i.e., indication of the point of anticipated failure). The vertical component of the seismic coefficient was taken as 50 percent of the horizontal component due to phase lag in the vertical wave with respect to the horizontal shear wave.

Table D 8 and Table D 9 list the estimated yield coefficients, k_y (g), for both the upstream and downstream slopes for both the peak- and post-earthquake undrained strengths.

Table D 8. Estimated Yield Coefficients (Accelerations) for BC 1

Strength Envelope	Yield Acceleration	
	Downstream	Uptream
Peak Undrained Strength Parameters	0.095	0.105
Post-Earthquake Undrained Strength Parameters	0.060	0.130

Table D 9. Estimated Yield Coefficients (Accelerations) for BC 2

Strength Envelope	Yield Acceleration	
	Downstream	Upstream
Peak Undrained Strength Parameters	0.165	0.310
Post-Earthquake Undrained Strength Parameters	0.097	0.230

One assumption made for the analysis was that the actual strength of the soil during shaking would shift from the peak undrained strength at the beginning of shaking to the post-earthquake strength some time during or immediately following shaking, depending on the rate of strength reduction, potential pore pressure generation, and characteristics of the ground motion. Displacement curves were generated using yield coefficients that vary between the post-earthquake to the peak undrained strengths to evaluate the range of possible deformations that could result depending on the rate of strength reduction.

A Seismic Hazard Update, Cornforth (2014) provided the ground motions for use in the Newmark analysis. Three ground motions representative of the CSZ events and five ground motions representative of intraslab or local crustal events were used for the analysis. Scaling factors were provided in the 2014 Cornforth report to adjust the motions for return periods other than the 2,475-year event. Return periods of 475-, 975-, 2,475- and 4,975-year events were used in the deformation analysis. Mean horizontal peak ground accelerations (PGAs) for the local crustal faults ranged from 0.67 to 1.26g (acceleration due to gravity) and 0.12 to 0.62g for the CSZ events. Details regarding the development of the ground motions can be found in Appendix A Seismic Hazards (Cornforth 2014). Both the H1 and H2 (the mutually perpendicular horizontal earthquake records) for each event were used, providing a total of 16 earthquake time-histories for each recurrence interval.

Rigid-block analysis, first developed by Newmark (1965), treats a potential slope failure mass block as a rigid mass (no internal deformation) that slides in a perfectly plastic manner on an inclined plane. Thus, the mass experiences no permanent displacement until the base acceleration exceeds the critical (yield) acceleration of the block. When the base acceleration exceeds the critical acceleration, the block begins to move downslope. Displacements are estimated using a two-stage integration procedure: (1) the parts of the acceleration-time history that lie above the critical acceleration are integrated to yield a velocity-time history; (2) the velocity-time history is then integrated to yield the cumulative displacement of the sliding block. Rigid-block analysis yields satisfactory results for relatively thin slope failures in stiff or brittle material having period ratios (T_s/T_m) less than about 0.1, where T_s is the fundamental site period and T_m is the mean shaking period (Rathje et al. 2004). For thicker failure surfaces in softer materials, rigid-block analysis tends not to be conservative.

A decoupled sliding-block analysis is a modification of the traditional Newmark analysis that does not require the potential failure mass to behave as a rigid block but rather models its dynamic response. The decoupled sliding-block analysis computes the dynamic response of the sliding mass without consideration of sliding and then uses the computed response in a rigid sliding-block analysis. The dynamic response of the sliding mass is computed using a one-dimensional, modal analysis in the time domain (Rathje and Bray 1999). The sliding mass is defined by its height, shear-wave velocity, and damping ratio; the shear-wave velocity (V_s) below the sliding mass is also specified (this can be conservatively taken as rock). The modal analysis has a rigid base, but the effects of a visco-elastic base are modeled through additional damping that is assigned based on the V_s of the base and the V_s of the sliding mass (Lee 2004). The dynamic response can be modeled as linear elastic or equivalent linear.

A coupled sliding-block analysis is an extension of a decoupled analysis. The coupled analysis models the interaction of sliding/limited shear stresses on the dynamic response of the sliding mass. Coupled analysis is considered the most rigorous and yields the most accurate estimates of displacement for deeper failures in softer material.

During the analyses the decoupled analyses were performed and generally yielded larger deformations, followed by the coupled and then the rigid block analyses. The values for V_s for the alluvial material were estimated using an average of the shear wave velocities from the SCPT testing. The V_s values of the dam embankment and underlying rock were estimated based on material type. The height of the failure for the analyses was approximately 70 feet and illustrates the distance from the crest of the dam to the alluvium/rock interface, which is where the resulting failure surface obtained from the pseudo-static slope stability analysis is located for both dams BC 1 and BC 2.

In order to calibrate estimates of deformations with the Newmark methodology described above, a initial assessment of potential deformations using an empirical methodology by Swaisgood (2003) was made. The Swaisgood methodology is based on an assessment of the response of a large number of different types of embankment and rockfill dams subjected to broad range of earthquakes and corresponding peak ground accelerations.

In order to improve the applicability of the Swaisgood method to the Newport Dams, the data base of case histories were sorted and a regression analysis of the dam's seismic response as a function of PGA was developed as shown on Figure D-1.17. In addition to the regression analysis showing the best fit line to the data, boundaries representing a reasonable upper and lower bound of expected deformations were added to the figure.

Using the Swaisgood methodology with the range of estimated peak ground accelerations at the Newport sites for different recurrence interval Cascadia earthquake events indicate that for similar embankment dam case histories in the data base, the best estimate of crest deformations ranged from as little as 1.2 inches for the 475-yr return period peak ground acceleration to over 478 inches for the 4,975-yr. return period peak ground accelerations.

Shown along the bottom axis of this figure are the estimated PGA's at the Newport Dam sites based on estimated recurrence intervals of 475-, 975-, 2475, and 4975-years. Starting at this point, estimates of the upper and lower bound along with the most likely or best estimate can be made as illustrated by the red lines on the figure. Using the 2475-year PGA as an example, the empirical methodology suggests a best estimate crest deformation for BC-1 of about 33-inches and lower and upper bounds of 15 and 68-inches, respectively. By coincidence, the estimated deformations at BC-2 are the same using this methodology. The combination of the dam height (DT) and alluvial thickness (AT) is the same at both sites and is about 70 feet.

Results of the Newmark analyses are presented on Figures D-1.18 and D-1.19. The results generally indicate that for the 4,975-year recurrence CSZ event, the dam crest settlement (loss of freeboard) could be over 180 inches for BC 1 and over 110 inches for BC 2. The maximum displacement would occur in the downstream direction. The maximum estimated settlement (freeboard loss) for the 2,475-year recurrence interval CSZ events are approximately 90 inches for BC 1 and over 50 inches for BC 2. As previously noted, the range of estimated displacements present the variation in potential deformations associated with variations in yield acceleration corresponding with the peak undrained (highest yield acceleration and lowest deformations) to the post-earthquake strengths (lowest yield acceleration and highest deformations).

The potential dam crest settlement for the 4,975-year recurrence interval earthquakes actually exceed the available freeboard at both dams and indicate the high potential for overtopping and failure of the dams. BC 1 has the highest failure potential. Earthquakes with an estimated recurrence interval of 2,475 years show a reduced but still significant potential for failure by seepage through transverse cracks that would occur in the dam or by overtopping. Earthquakes with estimated 975- or 475-year recurrence intervals would likely result in acceptable deformations for both BC 1 and BC 2.

Upstream potential deformations are correspondingly less than the downstream deformations, with maximum crest settlement of 105 inches and 30 inches for BC 1 and BC 2, respectively during a 4,750-year CSZ earthquake event.

Table D 10 contains some factors that we would expect to result in both reduced and increased deformations beyond those that can be shown or demonstrated explicitly in the Newmark analyses. It can be seen from the table that there are more factors contributing to an increase in expected deformation over the deformations given in the Swaisgood database, which would tend to indicate that the crest deformations estimated by the empirical Swaisgood method may underestimate the crest deformations and that deformations are likely in the upper range of the results estimated with the reduced residual strength shown on Figures D-1.18 and D-1.19.

Table D 10: Factors Contributing to the Reduction or Increase in Expected Deformations

Factors Contributing to a Reduction in Expected Deformation	Factors Contributing to an Increase in Expected Deformation
BC #2 has a central core that extends deeper into the underlying foundation material which could contribute to a small reduction in expected deformations.	Duration of strong shaking 4 to 10 times longer than typical crustal earthquake duration. This will cause an increase in total deformations over those that would occur for the crustal type events at the site.
	The long duration of strong shaking associated with a Cascadia Subduction Zone (CSZ) earthquake event will likely cause a reduction to residual strength in the foundation soils relatively early in the earthquake time history. Hence a large portion of the embankment deformation will occur while the foundation soils are at residual strength.
	BC#1 has a thick alluvial foundation that is relatively soft
	BC#2 foundation was not taken to bedrock and contains soft alluvial soils under the entire footprint of the dam.

Results of estimates of deformation using the empirical method by Swaisgood have been added between the results for the downstream and upstream slope on both of these figures. A summary of the estimates of deformations for both the Newmark and empirical Swaisgood method is presented Table D 11below

Table D.11

Table D 11: Summary of Estimated Embankment Crest/Downstream Slope Deformations at BC-1 and BC-2

Recurrence Interval Event (years)	Estimated Peak Ground Acceleration (PGA – g's)	Est. Deformations - Empirical (Swaisgood, 2003) (inches)			Est. Deformations – Newmark (inches)		
		Lower Bound	Best Estimate	Upper Bound	Lower Bound	Best Estimate	Upper Bound
BC 1							
2475	0.79	15	33	68	50	>76	90
4975	1.12	218	478	>478	116	>160	184
BC 2							
2475	0.79	15	33	68	32	>48	54
4975	1.12	218	478	>478	56	>96	112

Based on the performance of these similar dams, estimated deformations in the range of 24 to 60 inches have a moderate to high potential for very significant damage or failure. When deformations are estimated to be in this range for these recurrence interval earthquake events, the standard of care within the dam engineering community in the US and internationally would suggest that there is dam safety deficiency and justification to take action to mitigate that deficiency. Estimated deformations of over 60-inches have a high to very high likelihood of complete failure of the dam section and not only is there a deficiency, but justification to take more expedited actions to reduce the risk of failure of the dam.

Note that the cells in Table D 11 have been colored to represent the deficiency and action categories described above. The orange cells suggest a dam safety deficiency and moderate justification for corrective actions. The red cells indicate a dam safety deficiency and justification for more expedited corrective actions. The green cells indicate deformations that are below the level associated with a safety deficiency and need for corrective actions.

As can be seen in the Table D 11information, both the Newmark and Swaisgood deformation estimation methodologies indicate that damaging deformations would likely occur at both dams.

5.0 Conclusions

Based on the Phase 3 exploration, laboratory testing, and engineering analyses, both BC 1 and BC 2 are seismically deficient and would be anticipated to fail under seismic loading for events with recurrence intervals beginning around 2,475 years. More frequent events, such as the 475- and 975-year would likely experience damage that would impact operation of the reservoirs, but would not result in a full breach. It is further noted that estimated deformations of the upstream slope of BC 2 could have significant effects on the outlet works intake structure and discharge pipe. Hence corrective actions are indicated for both dams.

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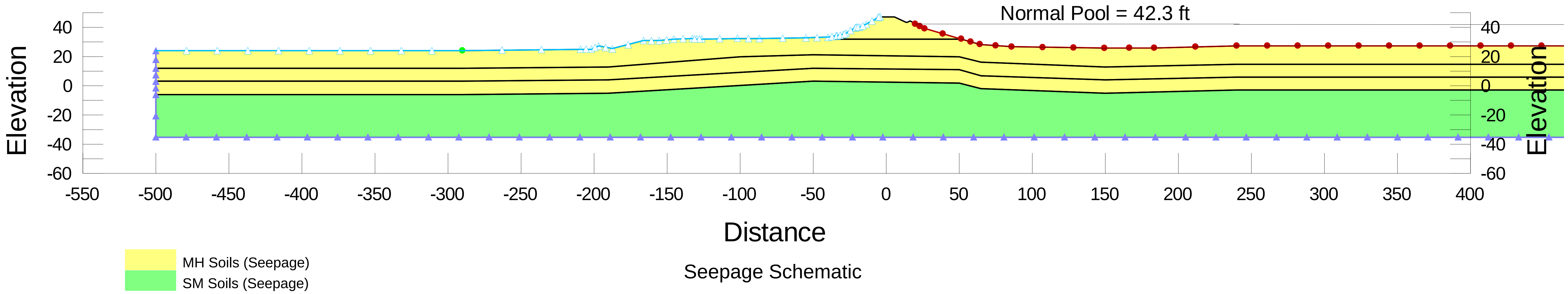
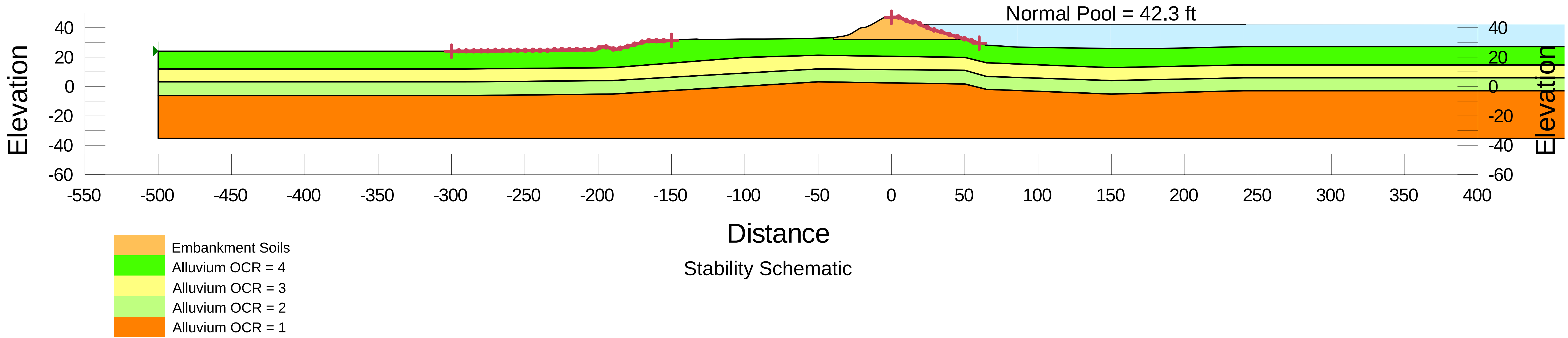
Figures



Seepage Schematic



Stability Schematic

Units—Feet

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BC-1 Seepage and Stability Schematic

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

ORIGINATOR: STA
APPROVED BY: VW
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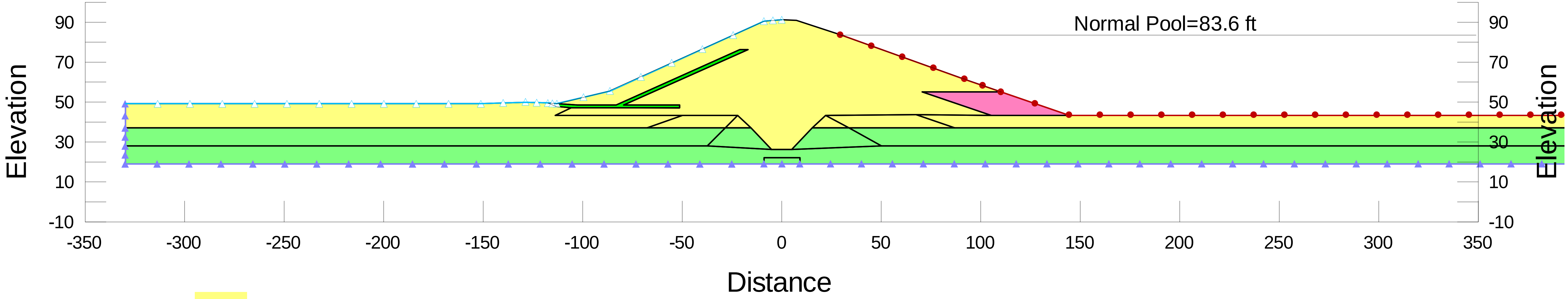
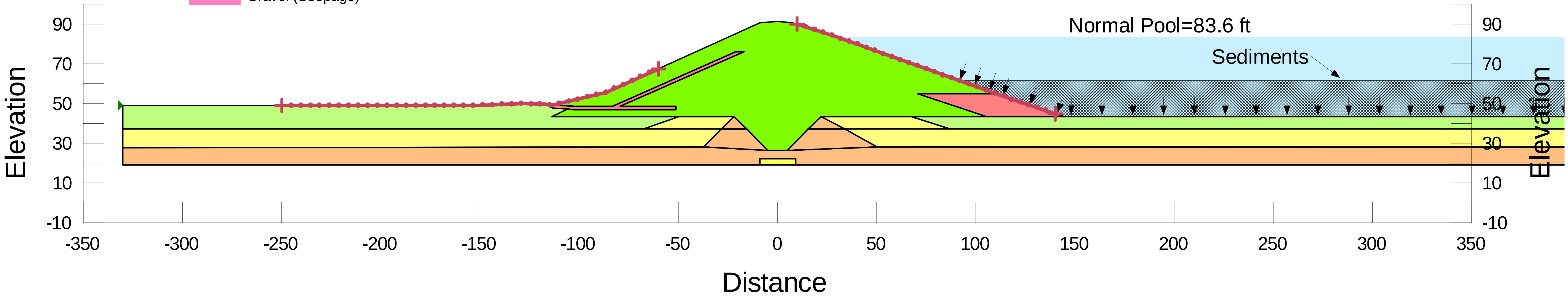
FIGURE
D-1.**1**



🟨 MH Soils (Seepage)
🟩 SM Soils (Seepage)
🟩 Filter and Drainage Blanket
🟪 Gravel (Seepage)

Seepage Schematic



🟩 Embankment Soils
🟩 Alluvium OCR = 4
🟨 Alluvium OCR = 3
🟧 Alluvium OCR = 2
🟨 Alluvium OCR = 1
🟥 Filter/Drainage Blanket and Gravel

Stability Schematic

Units—Feet

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PROJECT NO. 221223	BC-2 Seepage and Stability Schematic		FIGURE
DRAWN: 05/14/15	BIG CREEK DAMS 1 AND 2 NEWPORT, OREGON		D-1.2
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FILE NAME: <file name>	ORIGINATOR: STA	DRAWING CATEGORY: <drawing category>	
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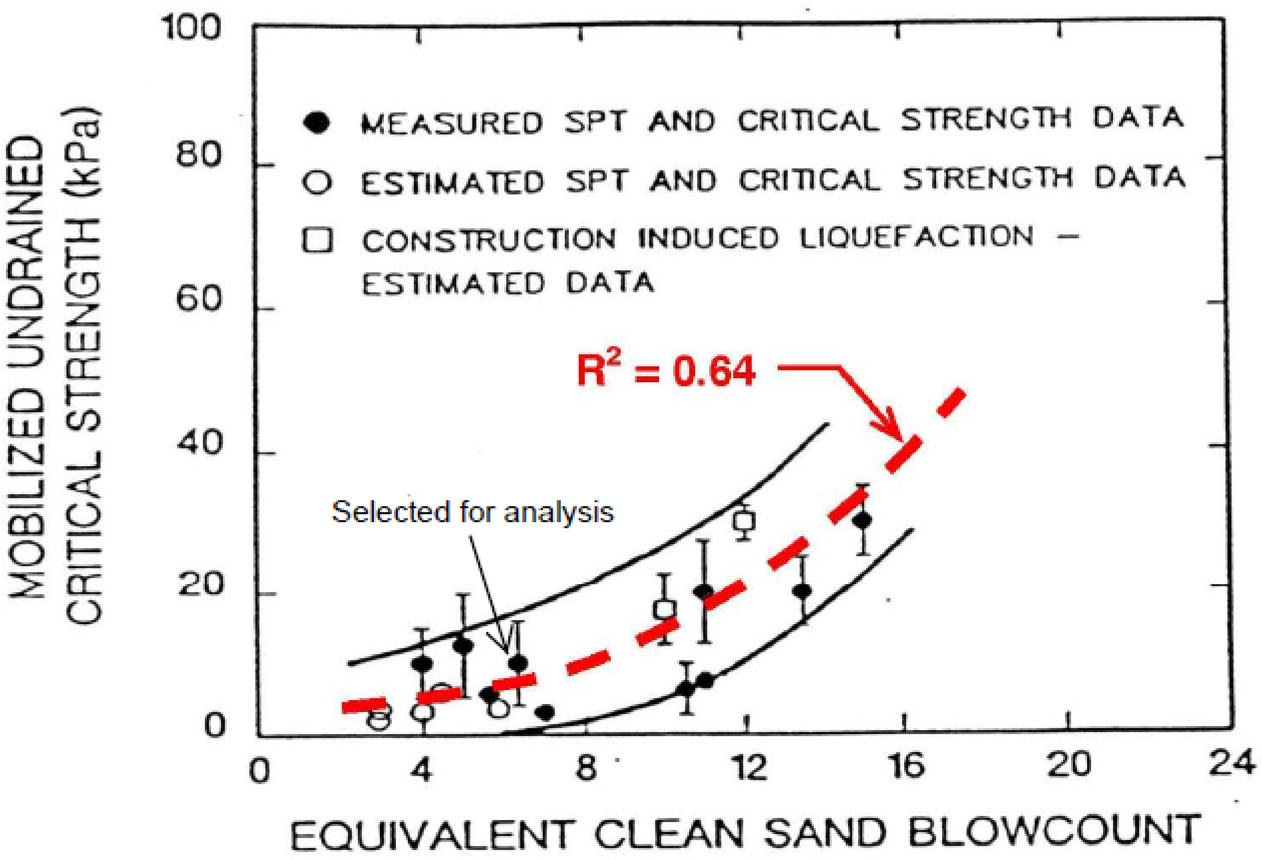

Figure 4-14: Regression of the Full-Scale Field Failure Case History Data of Seed and Harder (1990) Plotted in the Critical State Context as $S_{u,r}$ vs. $N_{1,60,CS}$.

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FILE NAME:	<file name>

Shear Strength Based on Equivalent Clean Sand Blow Count

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

ORIGINATOR: <originated by>
APPROVED BY: <approved by>

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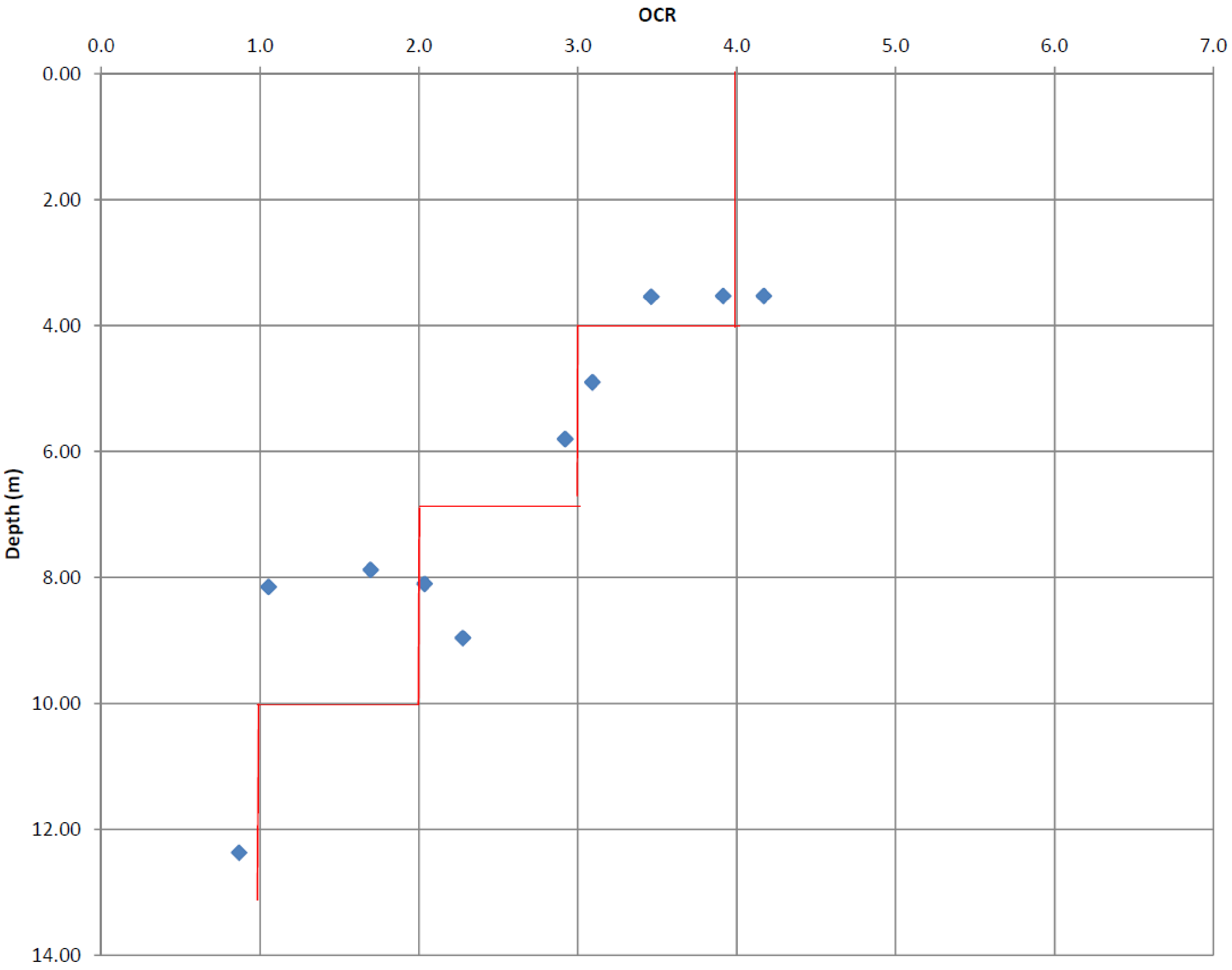
FIGURE
D-1.3



PROJECT NO. 101492	OCR vs. Depth Based on Laboratory Data	FIGURE
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DRAWN BY: <drawn by>		
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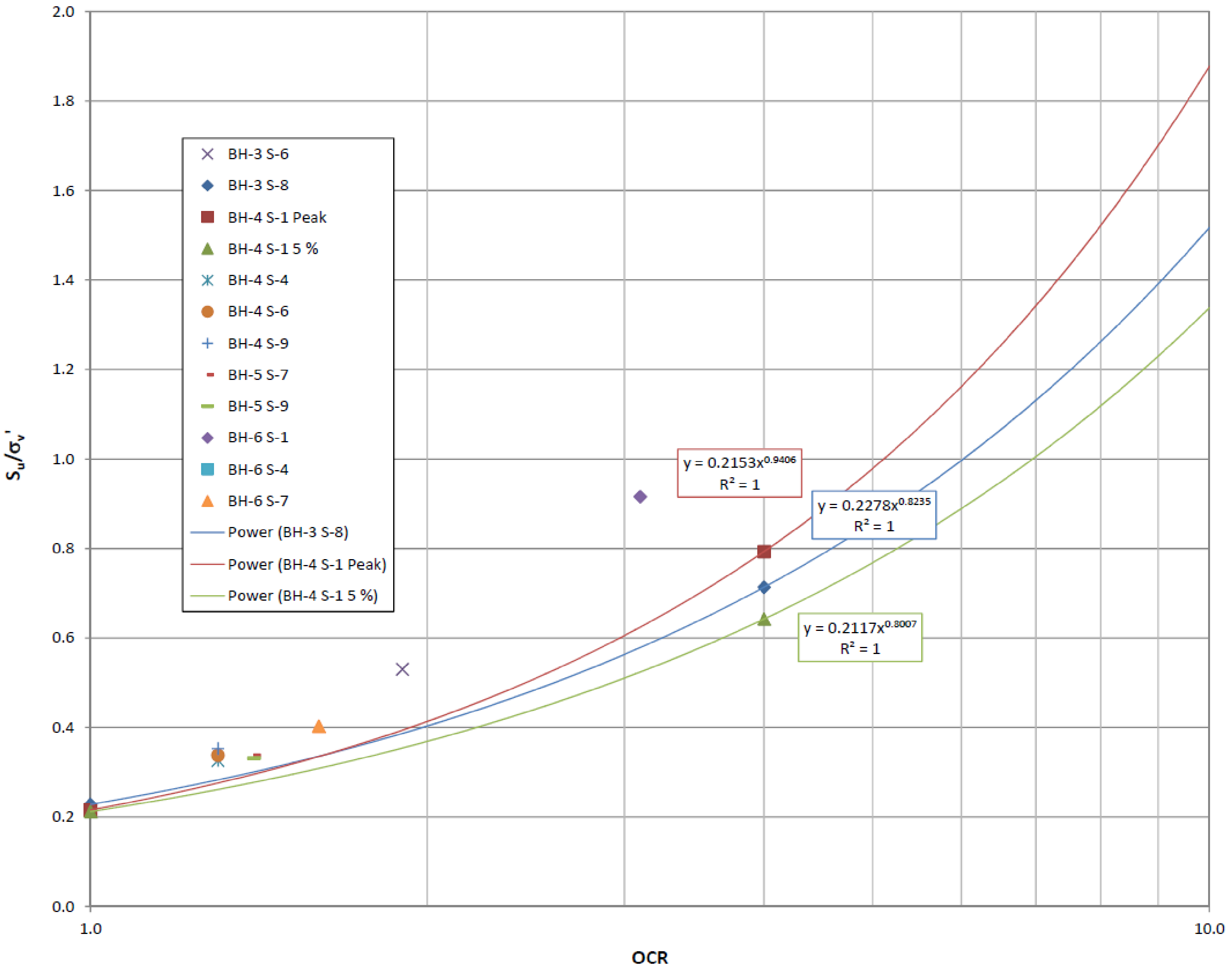

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SHANSEP Curves and Individual Data Points

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

ORIGINATOR:	<originated by>
APPROVED BY:	<approved by>

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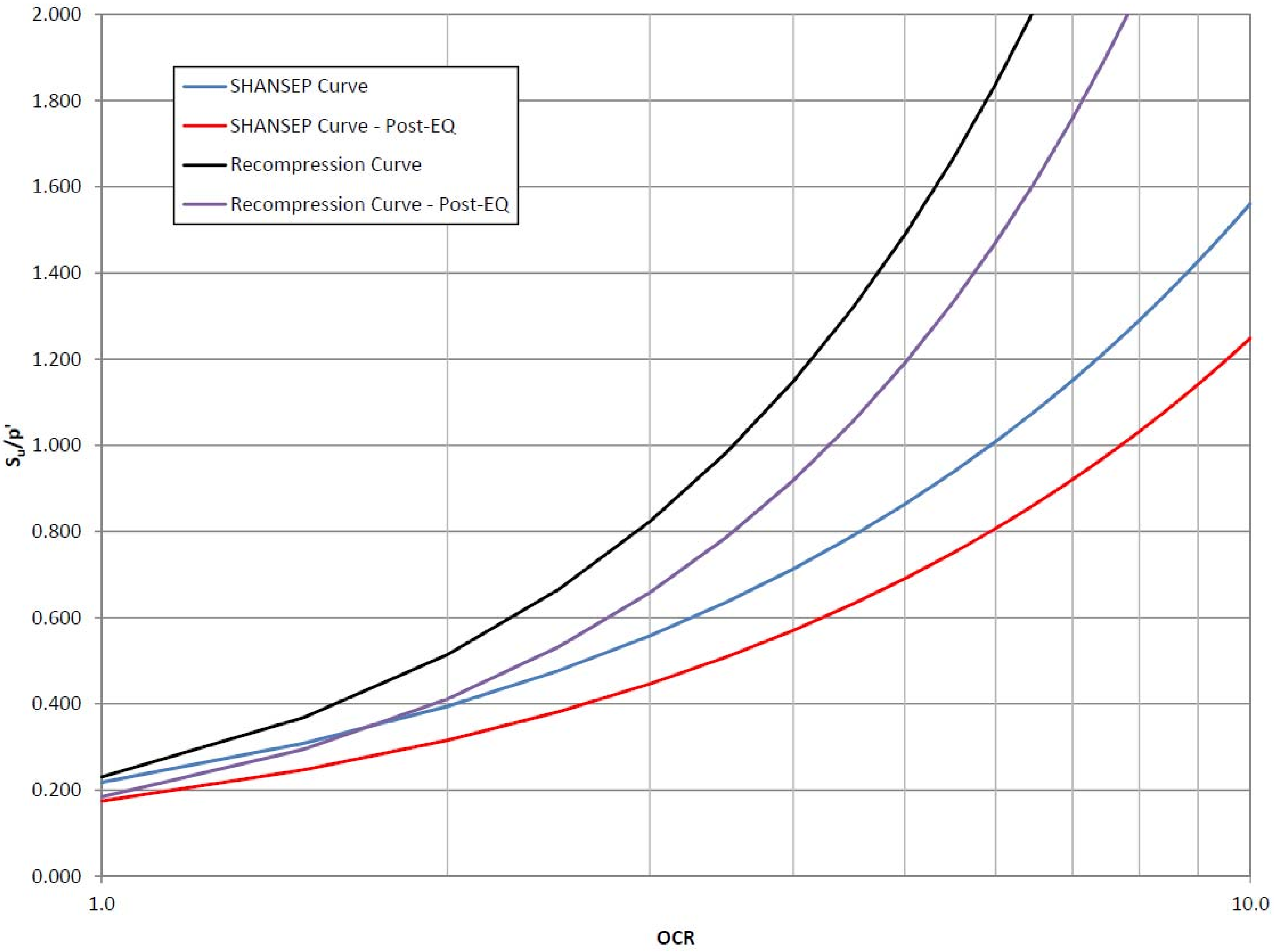
FIGURE
D-1.5



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SHANSEP and Recompression Curves

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

ORIGINATOR: <originated by>
APPROVED BY: <approved by>
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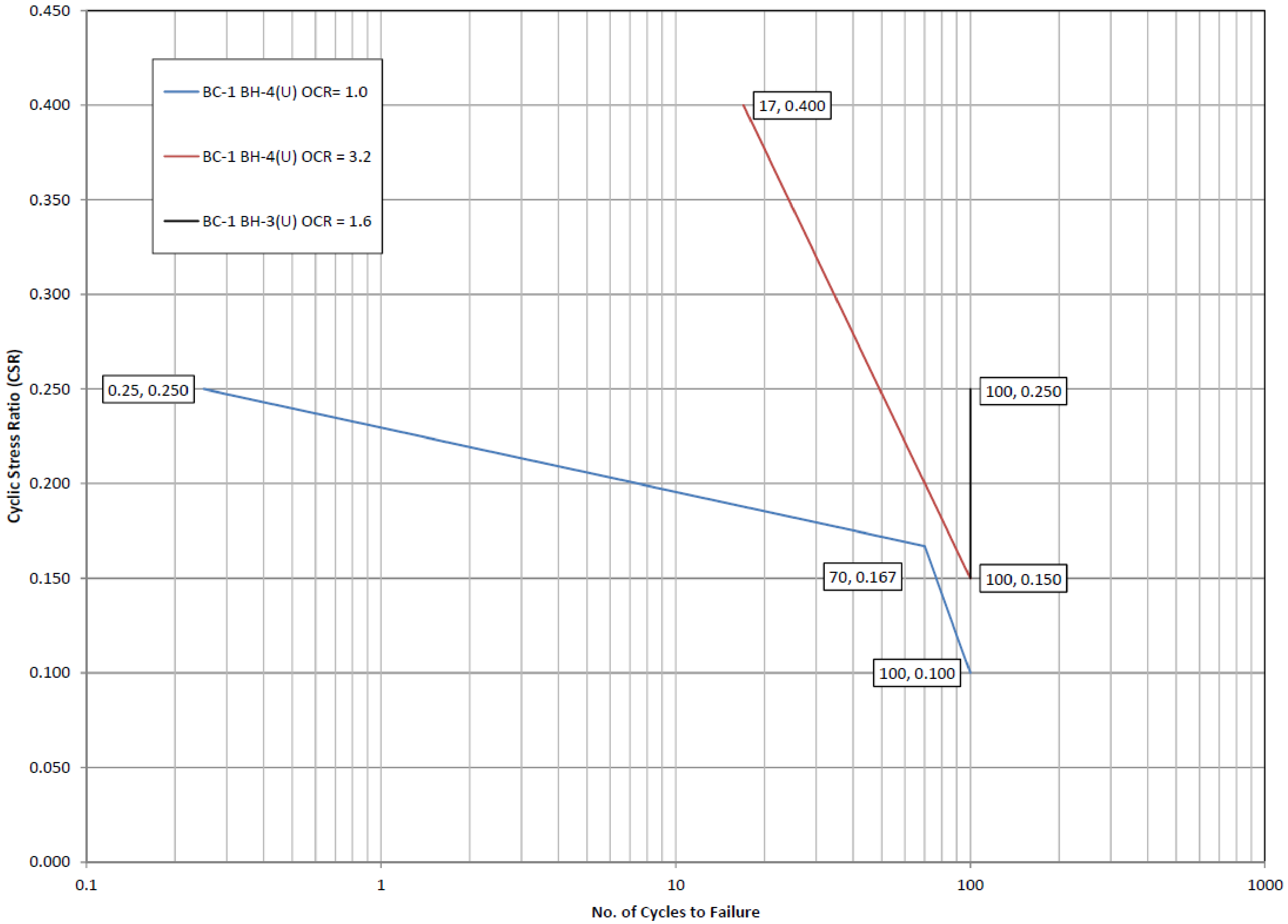
FIGURE
D-1.6



PROJECT NO. 101492	Number of Cycles to Failure		FIGURE
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DRAWN BY: <drawn by>			
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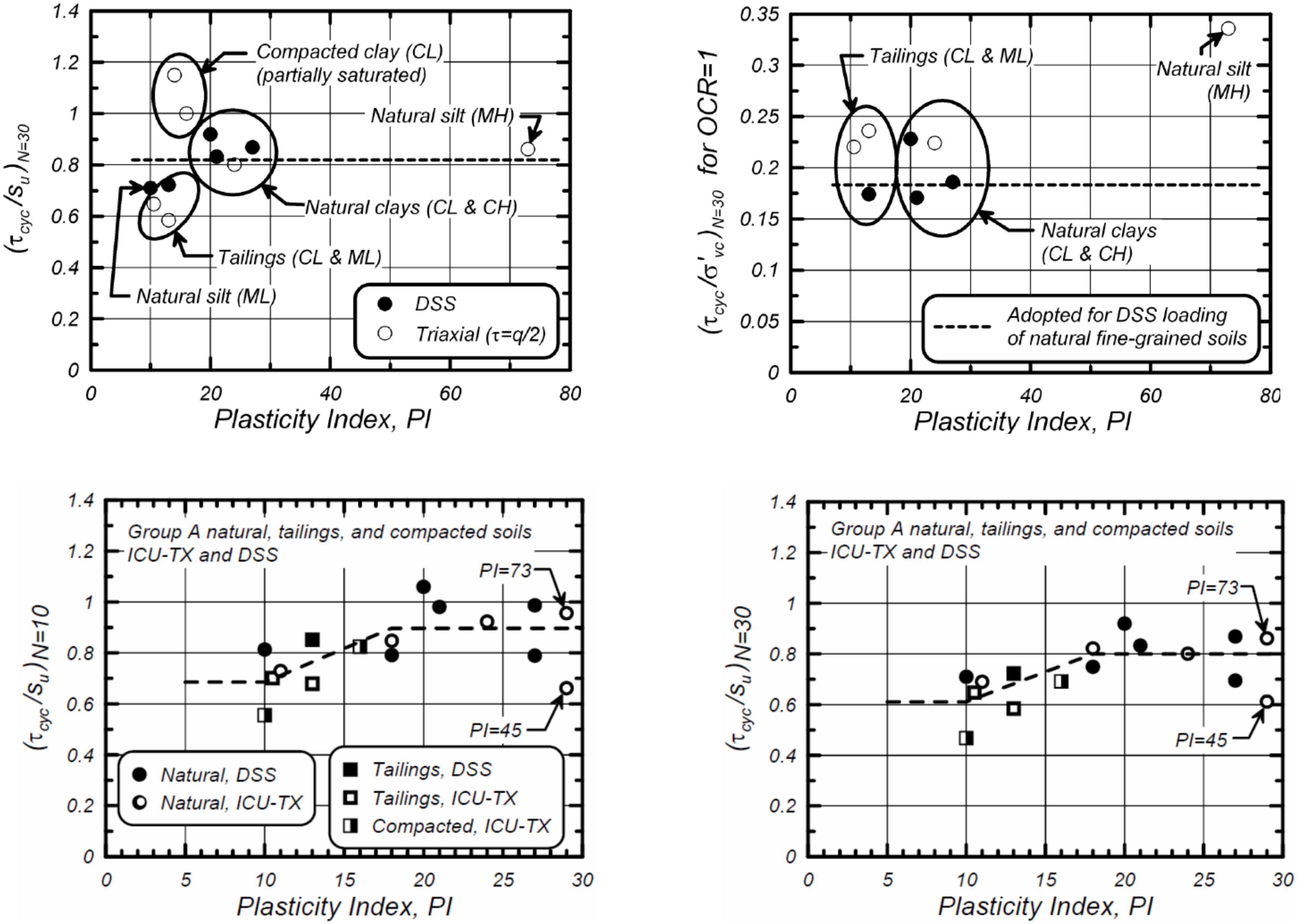

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PROJECT NO. 101492
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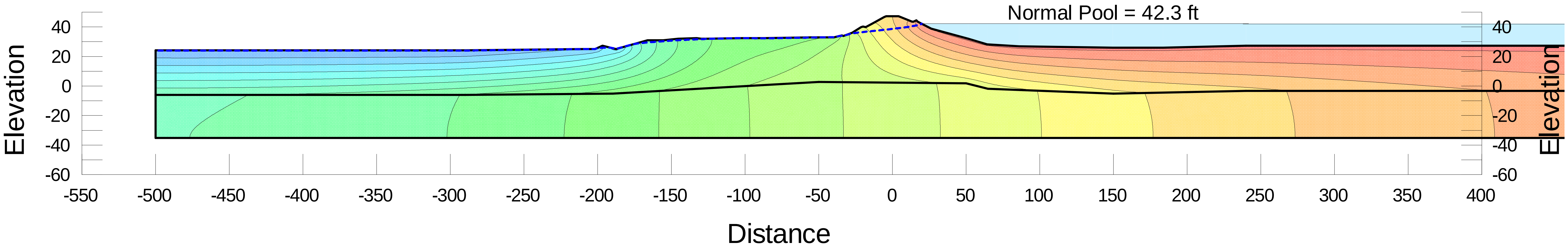
DRAWN BY: <drawn by>
CHECKED BY: <checked by>
FILE NAME: <file name>

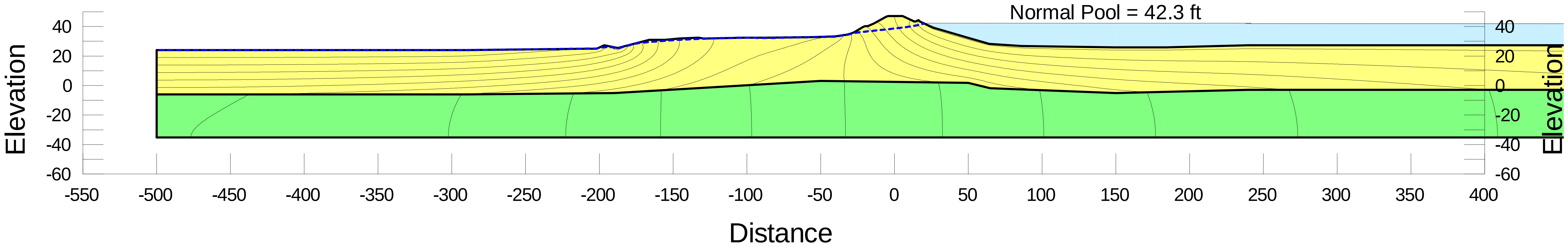
Shear Strength Reduction Based on Plasticity Index

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

ORIGINATOR: <originated by>
APPROVED BY: <approved by>
DRAWING CATEGORY: <drawing category>

FIGURE
D-1.8





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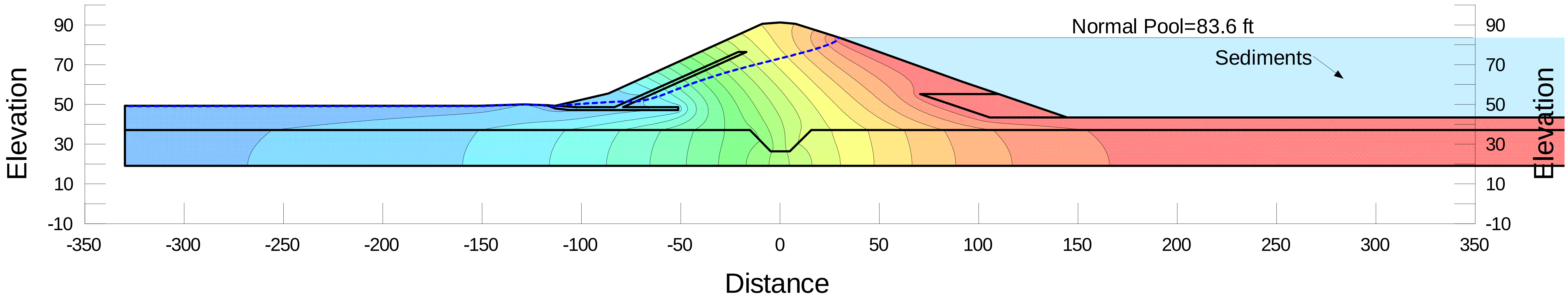
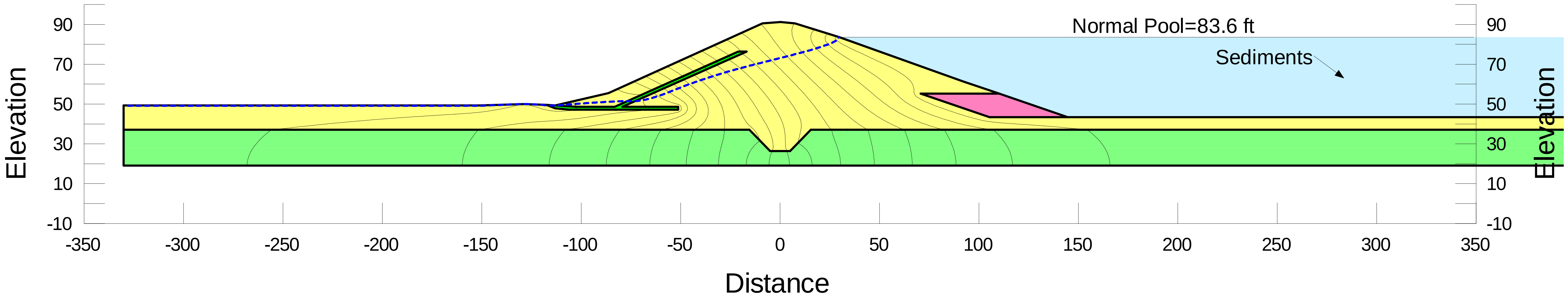
BC –1 Seepage Analysis Results

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

ORIGINATOR: STA
APPROVED BY: VW
DRAWING CATEGORY: <drawing category>

FIGURE
D-1.9





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PROJECT NO.	221223
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BC –2 Seepage Analysis Results

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

ORIGINATOR: STA
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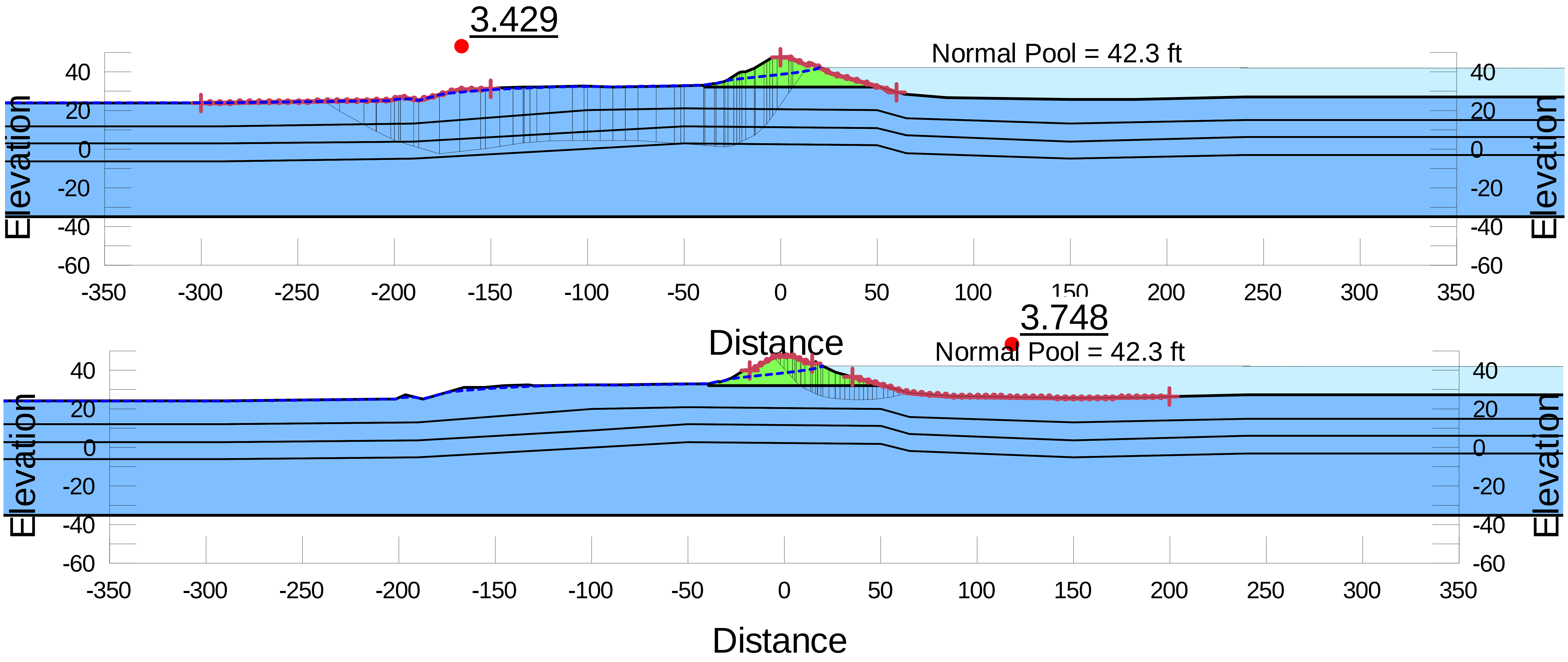
FIGURE
D-1.10



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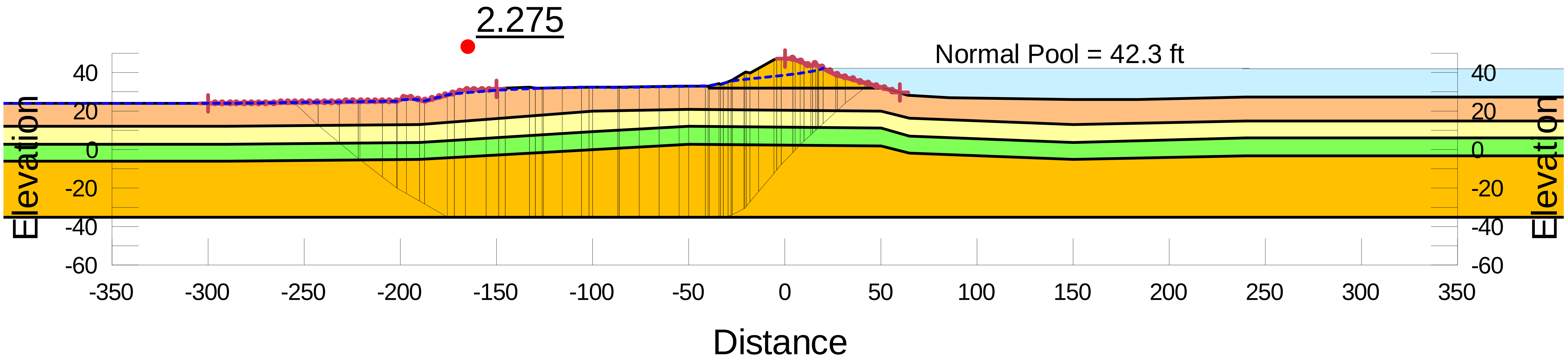
PROJECT NO. 221223
DRAWN: 05/14/15
DRAWN BY: STA
CHECKED BY: KAF
FILE NAME: <file name>

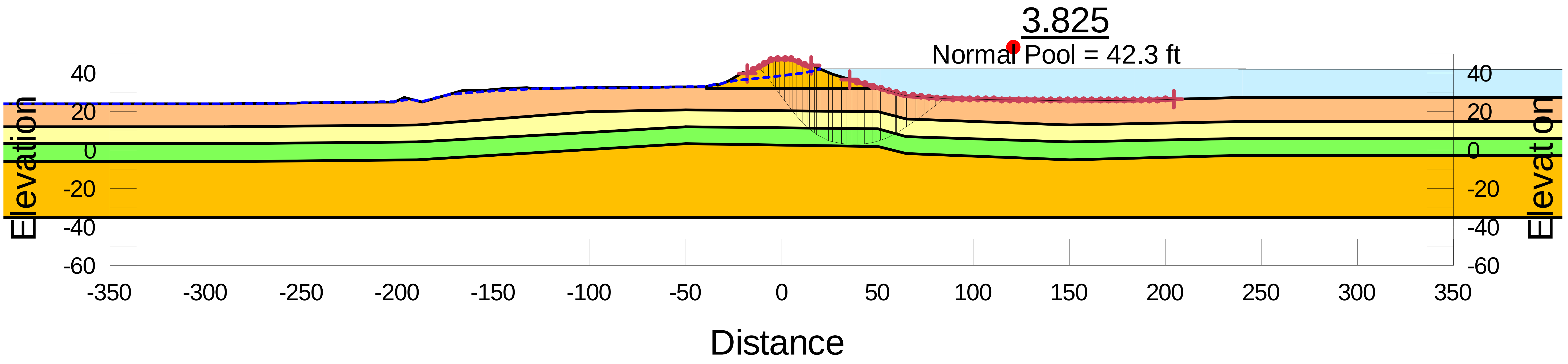
BC –1 Drained Stability Analysis Results

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

ORIGINATOR: STA
APPROVED BY: VW
DRAWING CATEGORY: <drawing category>

FIGURE
D-1.11





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PROJECT NO. 221223
DRAWN: 05/14/15
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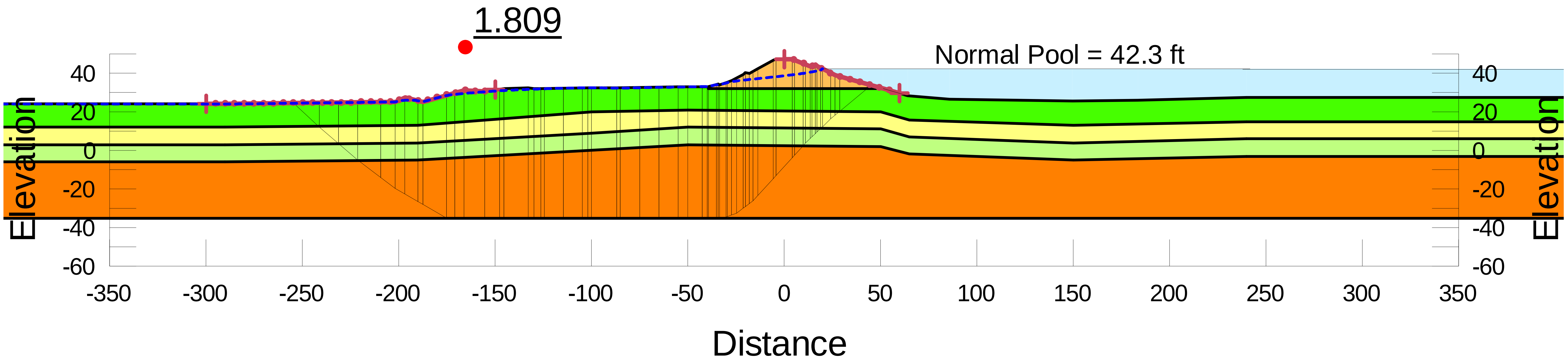
CHECKED BY: KAF
FILE NAME: <file name>

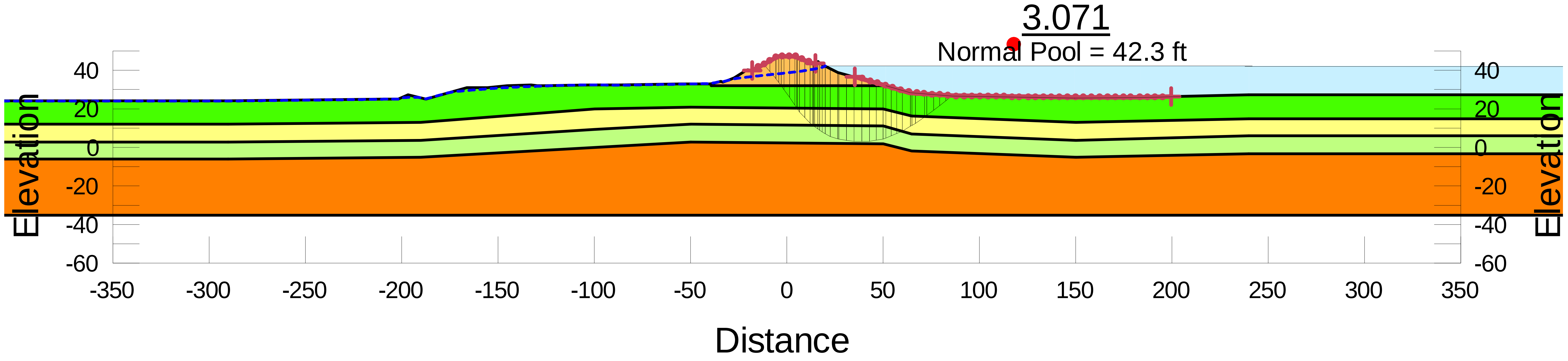
BC –1Undrained Stability Analysis Results

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

ORIGINATOR: STA
APPROVED BY: VW
DRAWING CATEGORY: <drawing category>

FIGURE
D-1.12





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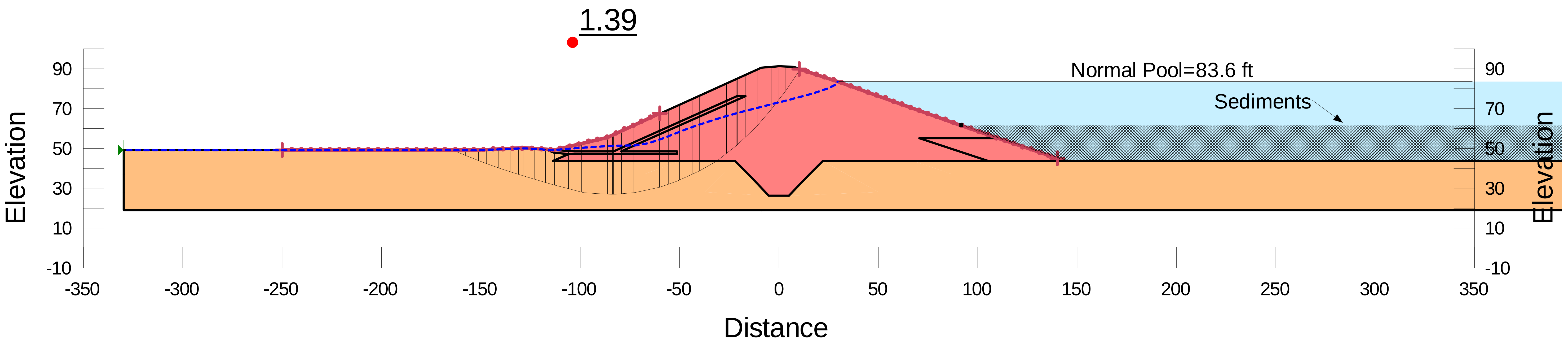
CHECKED BY:	KAF
FILE NAME:	<file name>

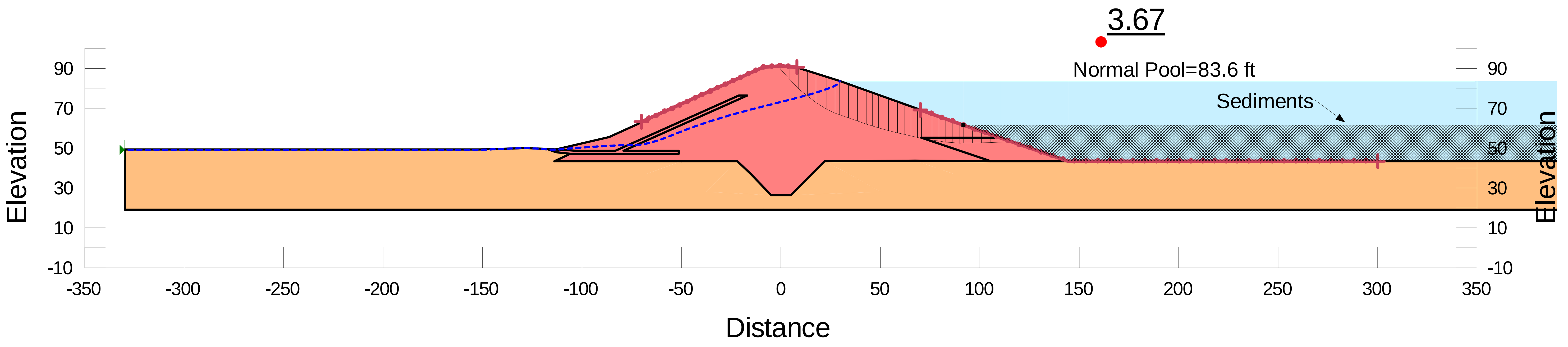
BC –1 Post-EQ Stability Analysis Results

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

ORIGINATOR: STA
APPROVED BY: VW
DRAWING CATEGORY: <drawing category>

FIGURE
D-1.13





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PROJECT NO. 221223
DRAWN: 05/14/15
DRAWN BY: STA
CHECKED BY: KAF
FILE NAME: <file name>

BC –2 Drained Stability Analysis Results

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

ORIGINATOR: STA
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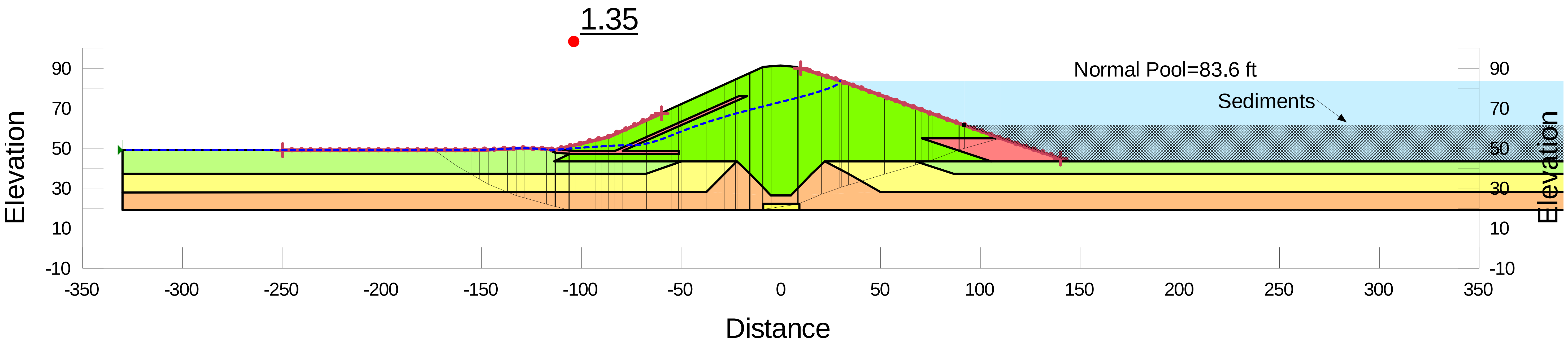
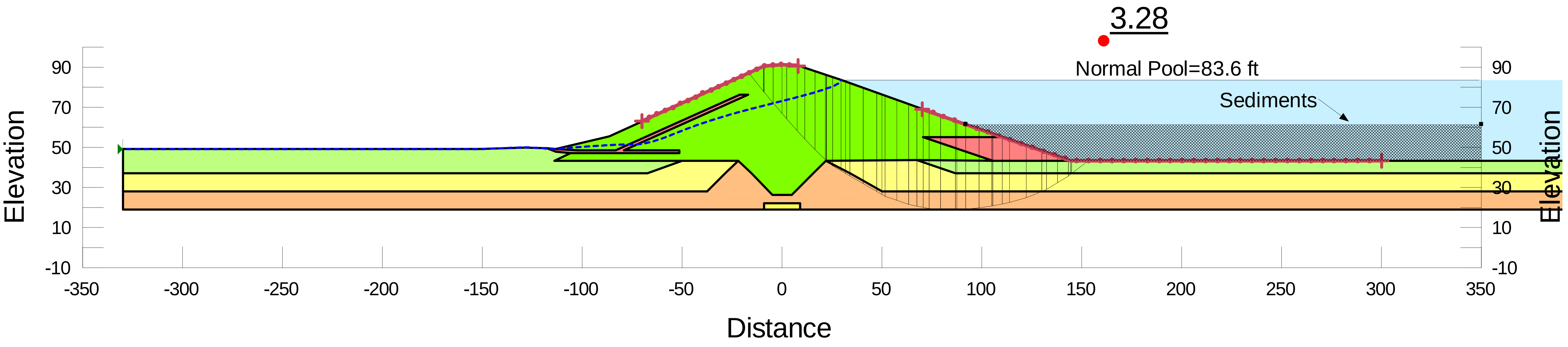
FIGURE
D-1.14





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PROJECT NO. 221223
DRAWN: 05/14/15
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FILE NAME: <file name>

BC –2 Undrained Stability Analysis Re-

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

ORIGINATOR: STA
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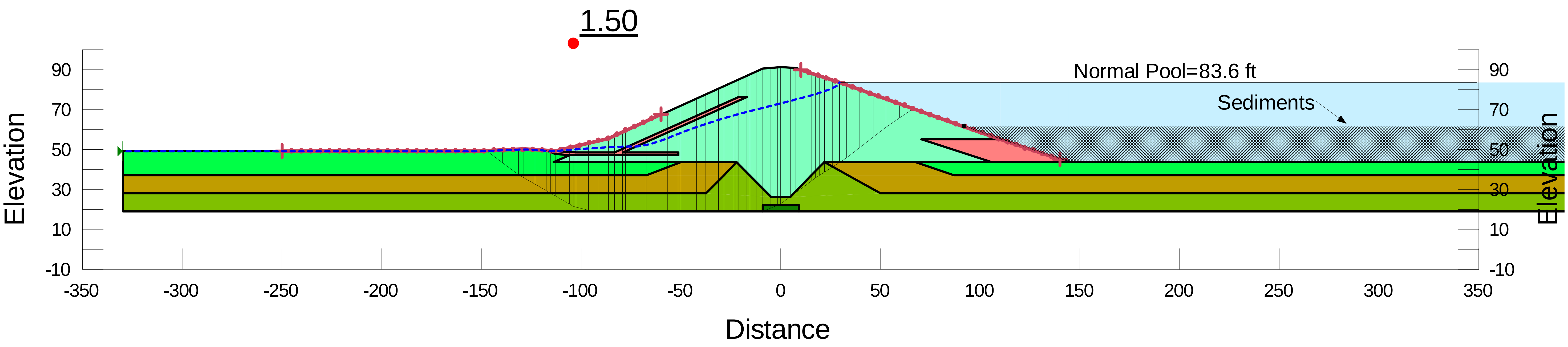
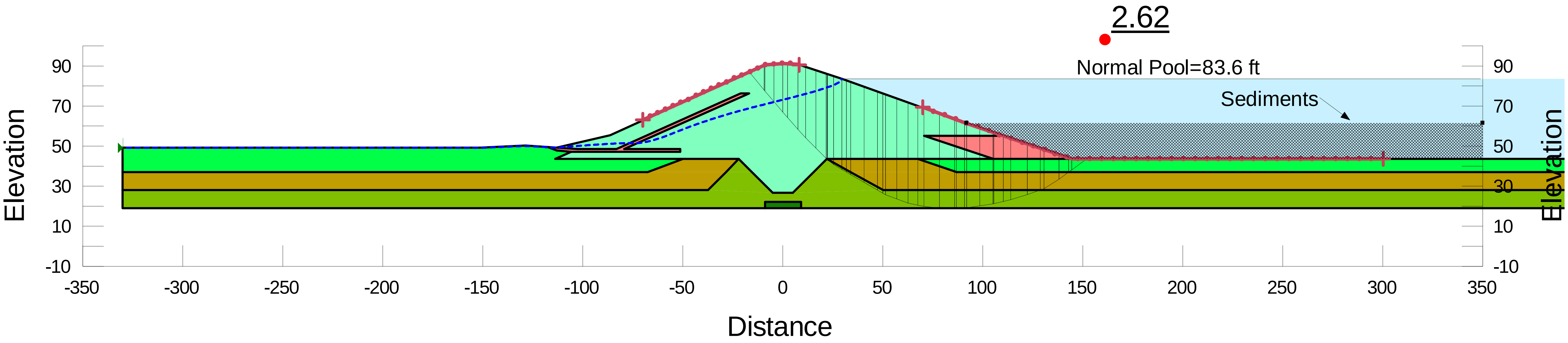
FIGURE
D-1.15





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PROJECT NO. 221223
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BC –2 Post-EQ Stability Analysis Results

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

ORIGINATOR: STA
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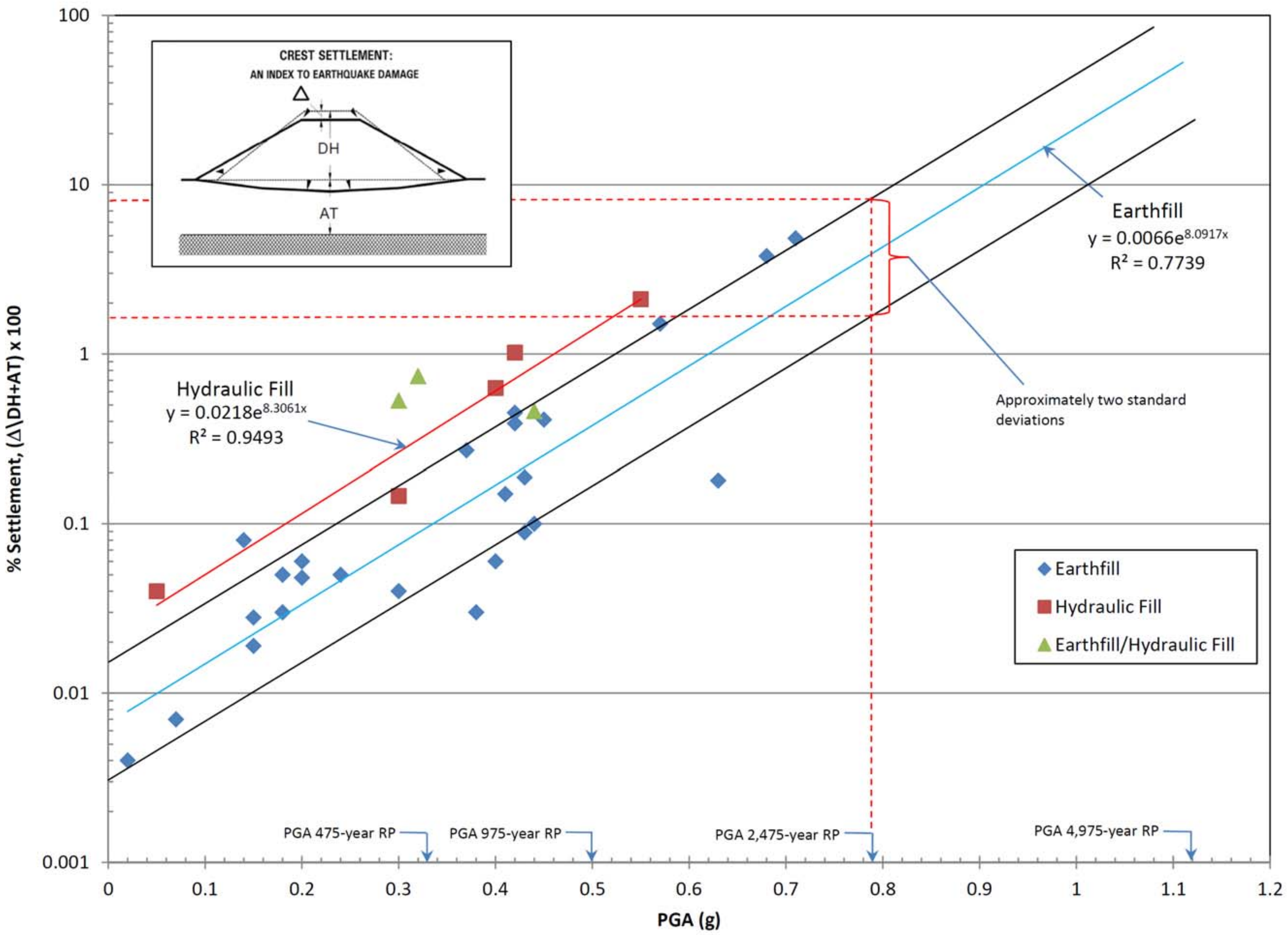
FIGURE
D-1.16



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PROJECT NO.	101492
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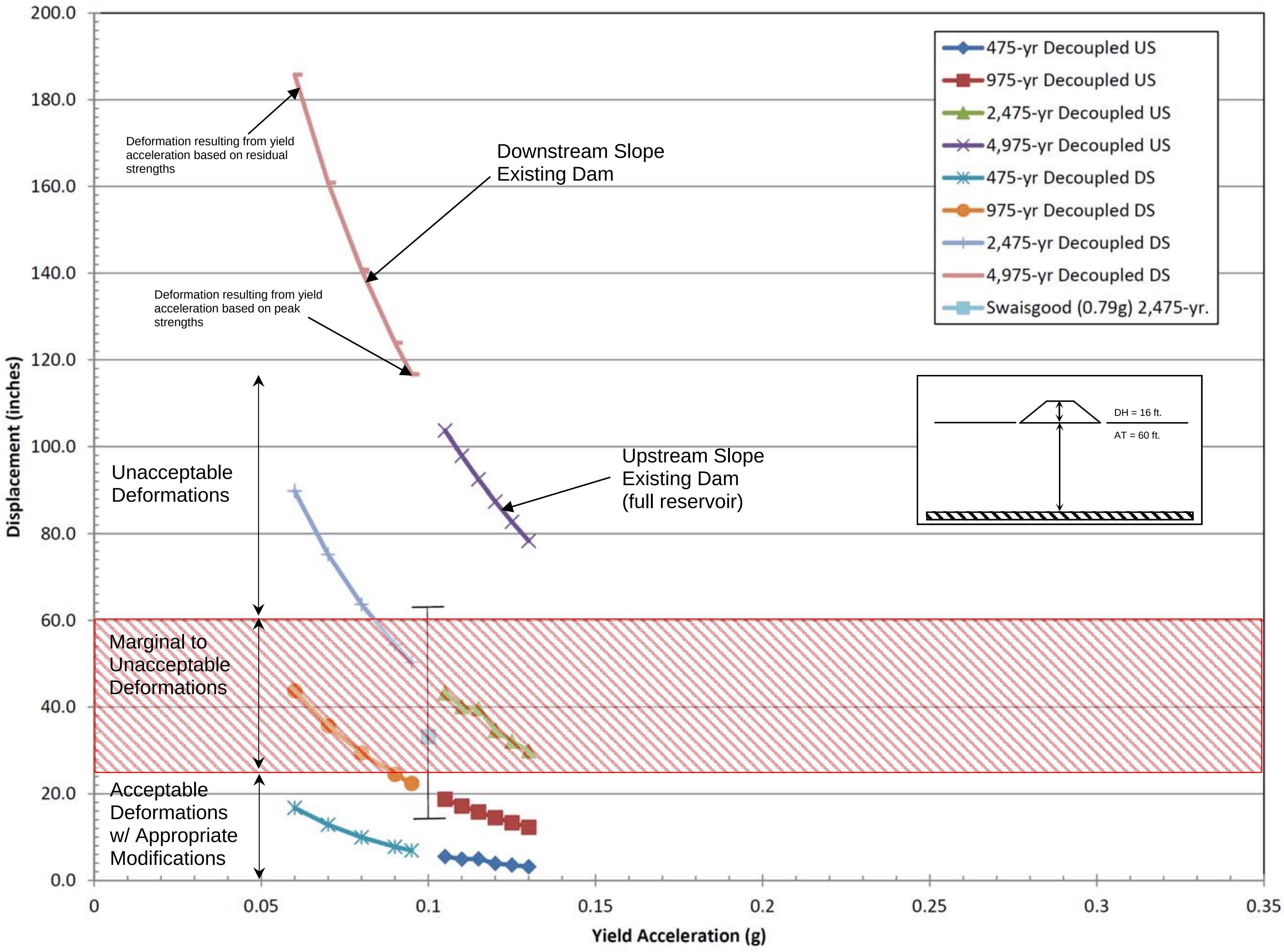
Swaisgood(2003) %Settlement vs. PGA

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

ORIGINATOR: STA
APPROVED BY: VW
DRAWING CATEGORY:

FIGURE
D-1.17



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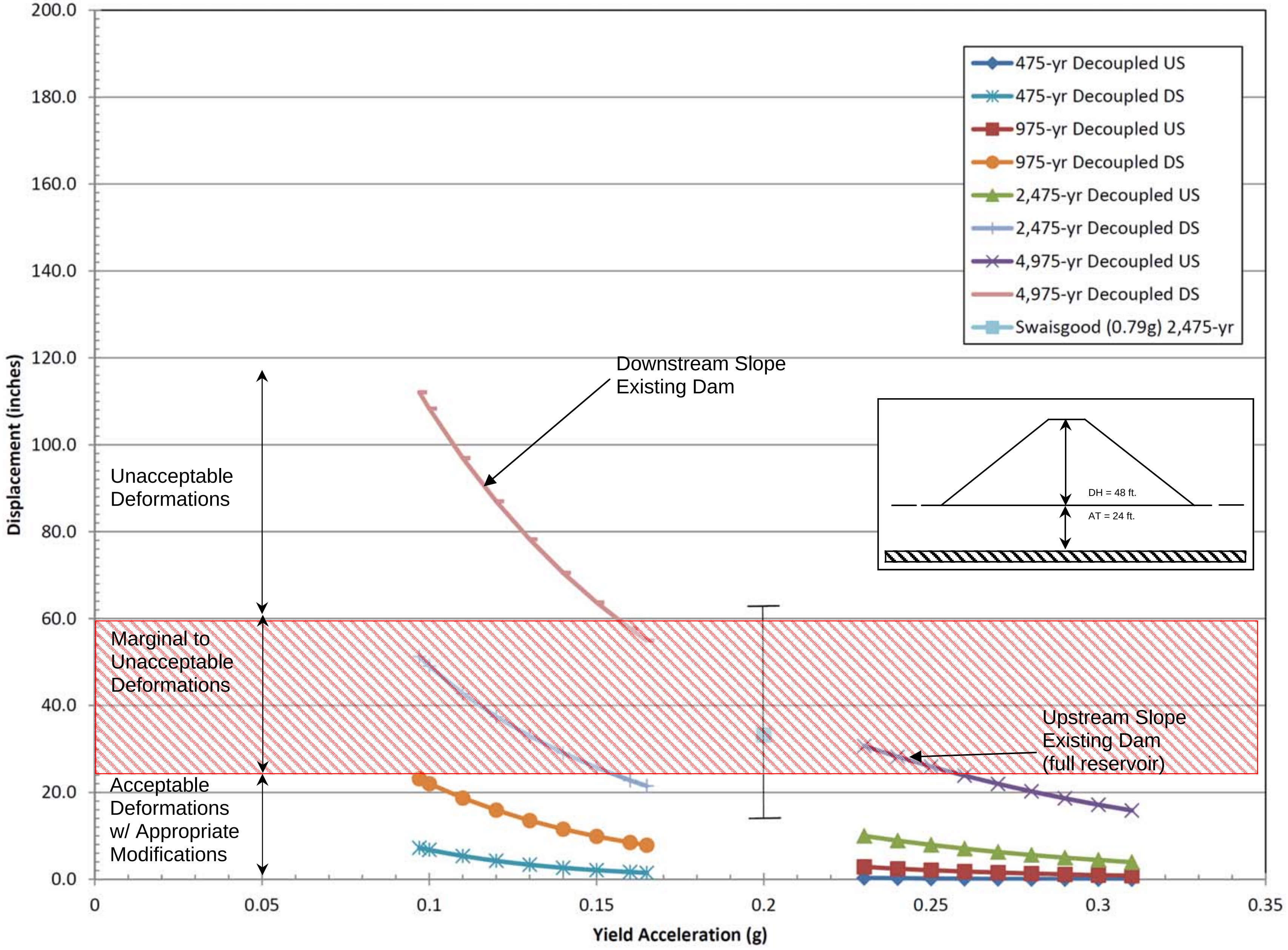
PROJECT NO.	101492
DRAWN:	8-20-2015
DRAWN BY:	STA
CHECKED BY:	KAF
FILE NAME:	

Newmark Displacements CSZ BC-1

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

ORIGINATOR: STA
APPROVED BY: VW
DRAWING CATEGORY:

FIGURE
D-1.18



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PROJECT NO. 101492
DRAWN: 6-30-2015
DRAWN BY: STA
CHECKED BY: KAF
FILE NAME:

Newmark Displacements CSZ BC-2

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

ORIGINATOR: STA
APPROVED BY: VW
DRAWING CATEGORY:

FIGURE
D-1.19

Attachment A. DeJong Letter dated November 4, 2014

Newport Dams - Representative Design Value and Integration of CPT and Laboratory Data

Jason T. DeJong, Ph.D.

2834 Danube Ave • Davis, CA 95616 • 530-902-2878 • jdejong@ucdavis.edu

To: Scott Anderson, HDR Date: 11/4/2014
Fr: Jason DeJong & Chris Krage

Re: Newport Dams - representative design value & integration of CPT and laboratory data

---

Dear Scott,

This document is presented in response to your verbal request for evaluation of representative design values for Newport Dams (Big Creek Upper and Lower) based on integration of CPT and laboratory data. We have completed the following analysis and provide the representative design parameters based on in-situ sCPT tests and undisturbed sampling and lab tests. Justification for assumptions made is provided as necessary.

Assumptions:

- Primary silt layer is classified as a single geologic unit of interest.
- Primary silt layer is given representative design parameters.
- Ground water level is assumed to be at the ground surface.

Site specific calibration of sCPT borings and undisturbed lab tests:

Unit Weight

- Laboratory tests indicate the unit weight (γ) of the silty material from 3 to 13 meters ranges from 14 to 17 kN/m^3. A representative **unit weight of 15.5 kN/m^3** is selected for analysis. Figure 1 shows the plot of unit weight vs. depth and is attached.
- Selection of a representative unit weight necessitates the reevaluation of estimations of vertical stress (σ_{v0}) and vertical effective stress (σ'_{v0}). Original estimations using empirical correlations from cone profiles should be discarded and updated using the representative unit weight and the design ground water elevation.

Over-Consolidation Ratio and Preconsolidation Stress

- The preconsolidation stress profile is estimated by binning laboratory test results by sample quality. In addition, vertical effective stresses (σ'_{v0}) must be estimated using proper unit weights and the ground water table from the time of drilling and sampling. These revisions result in a representative OCR profile that can be fit by a power curve of the form $OCR = \left(\frac{depth}{b} \right)^{1/m}$ where **m = 1.20** and **b = 26** as shown in Figure 2. Note that OCR is nearly normally consolidated (OCR <1.5) below depths of about 12 m.

- Groundwater level at the time of sampling must be known to calculate the in-situ stresses of the samples at the time of sampling. In-lieu of specific ground water level information, the groundwater level is assumed to be hydrostatic and is set to the ground surface.

Undrained Shear Strength from Laboratory Tests

- A representative relationship for undrained shear strength are values for **S of 0.23** and **m of 1.16** according to the relationship $s_u / \sigma'_{v0} = S * \mathrm{OCR}^{\mathrm{m}}$.
- Undrained shear strength was obtained from laboratory tests using both the SHANSEP procedure and recompression procedures. Normally consolidated shear strength ratios $(s_u/\sigma'_{v0})_{NC}$ can be represented by a value of 0.23.
- Using the SHANSEP procedure, values for m (the exponent that captures strength gain with OCR) range from 0.80 to 0.94.
- Recompression to estimated in-situ stresses without the SHANSEP procedure yields a consistently larger m value (m=1.16). Interpretation of SHANSEP results should proceed with caution due to the irregular behavior of this silt, since the SHANSEP procedure was developed for "ordinary clays".
- It was not possible to assess the specimen quality of the samples tested with the DSS device since the reconconsolidate data was not reported. If this data exists, it may be advantageous to analysis it in order to assess the relative quality of the strength data obtained.
- Figure 3 shows the plot of normalized undrained strengths vs. OCR and is attached below.

Interpretation of CPT results

- When laboratory test data is available, a site-specific calibration of CPT measurements should be performed using site specific unit weights and strengths.
- The estimation of preconsolidation stress using the Mayne (2007) relationship with a k factor = 0.33 should not be used. The k factor for this site varies with depth, though a representative **k factor of 0.10** could be reasonably approximated for the silt unit to obtain a continuous profile of the preconsolidation stress.
- Calibration of sCPT results to lab samples reveals that an **N_{kt} factor of 20** is reasonable for this site (Figure 4). This value is towards the upper end of the range of typical values for N_{kt} (e.g. 10-20). It is noted that the cone factor (N_{kt}) does vary considerably with depth, and the selected value is conservative (Figure 5).
- Note that the estimation of N_{kt} from laboratory test data provides a better fit to BC1 sCPT soudings, and is more conservative with respect to BC2 sCPT soundings.
- SHANSEP s_u values were not considered in the $_{Nkt}$ fit since the strength from recompression strength tests yielded higher values.

Additional Notes

- The appearance of constant undrained strengths with depth is an artifact of decreasing OCR with depth, since the OCR exponent m is close to 1. Therefore the decrease in OCR with depth counterbalances the increase in σ'_v with depth until normally consolidated conditions are achieved.
- 2 excel sheets are included in this package.
    - Laboratory_Testing_Data.xls contains reduced forms of the laboratory data and all included plots. Note that changes can be made to the "Summary" sheet to alter the fits for representative properties (e.g. unit weight, OCR profile, normalized strength parameters). Note that when GWT or unit weight is changed in this sheet, the OCR fit values (intercept and exponent) must be updated from the graphical fit. The summary sheet also contains the s_u values obtained using the sCPT results and the identified N_{kt} factor. This plot is linked to summary data in the CPT_Interpretation.xls workbook.
    - CPT_Interpretation.xls contains the reduced form of sCPT tests: BC1_SCPT-5, BC1_SCPT-6, BC2_SCPT-5, and BC2_SCPT-6. The fitting parameters obtained from representative laboratory tests can be altered in sheet BC2_SCPT-6 (all other SCPT sheets reference these cells).

Sincerely,

Jason DeJong & Chris Krage

Attachments:

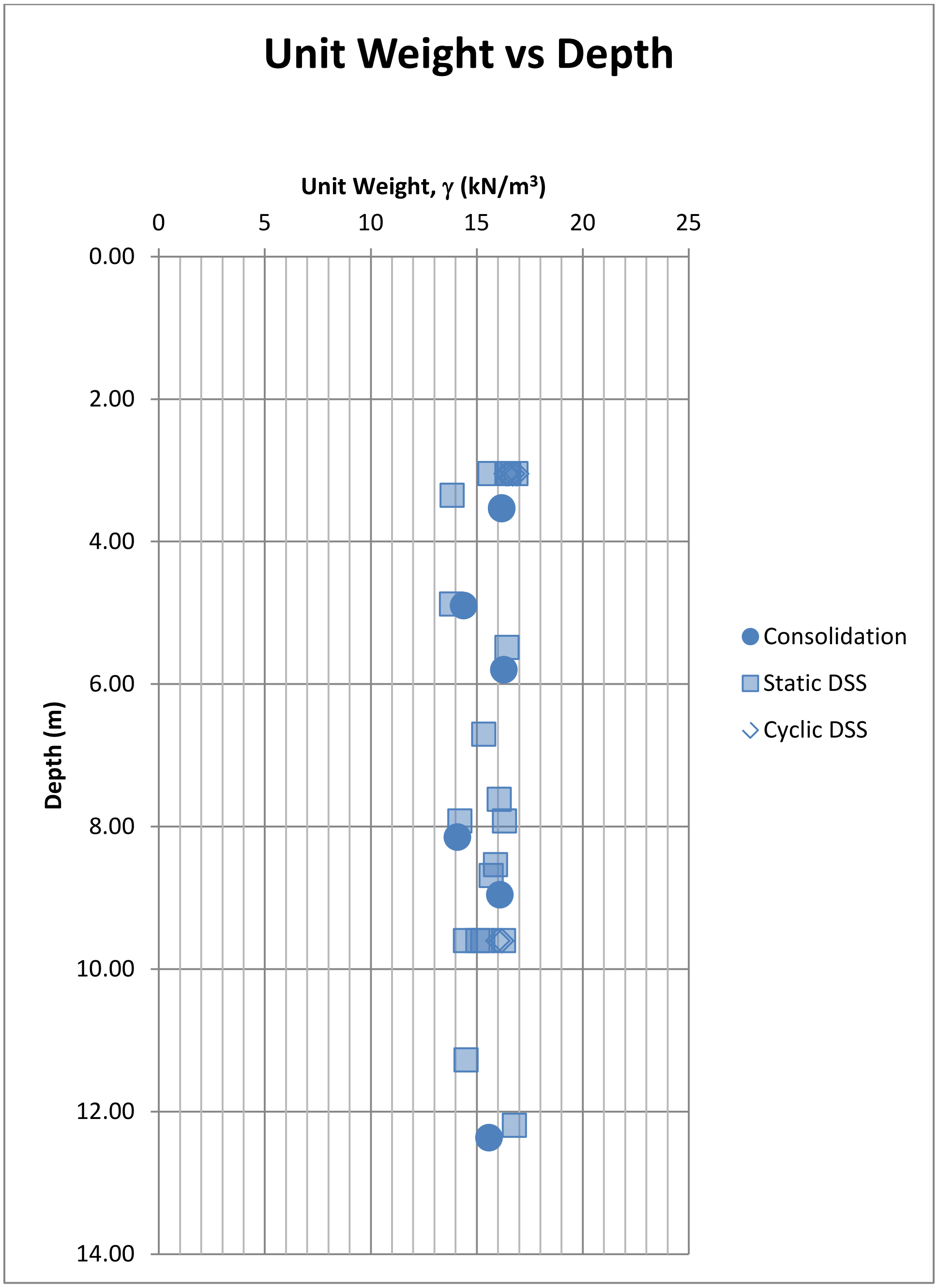


Figure 1 - Unit weight vs. Depth.

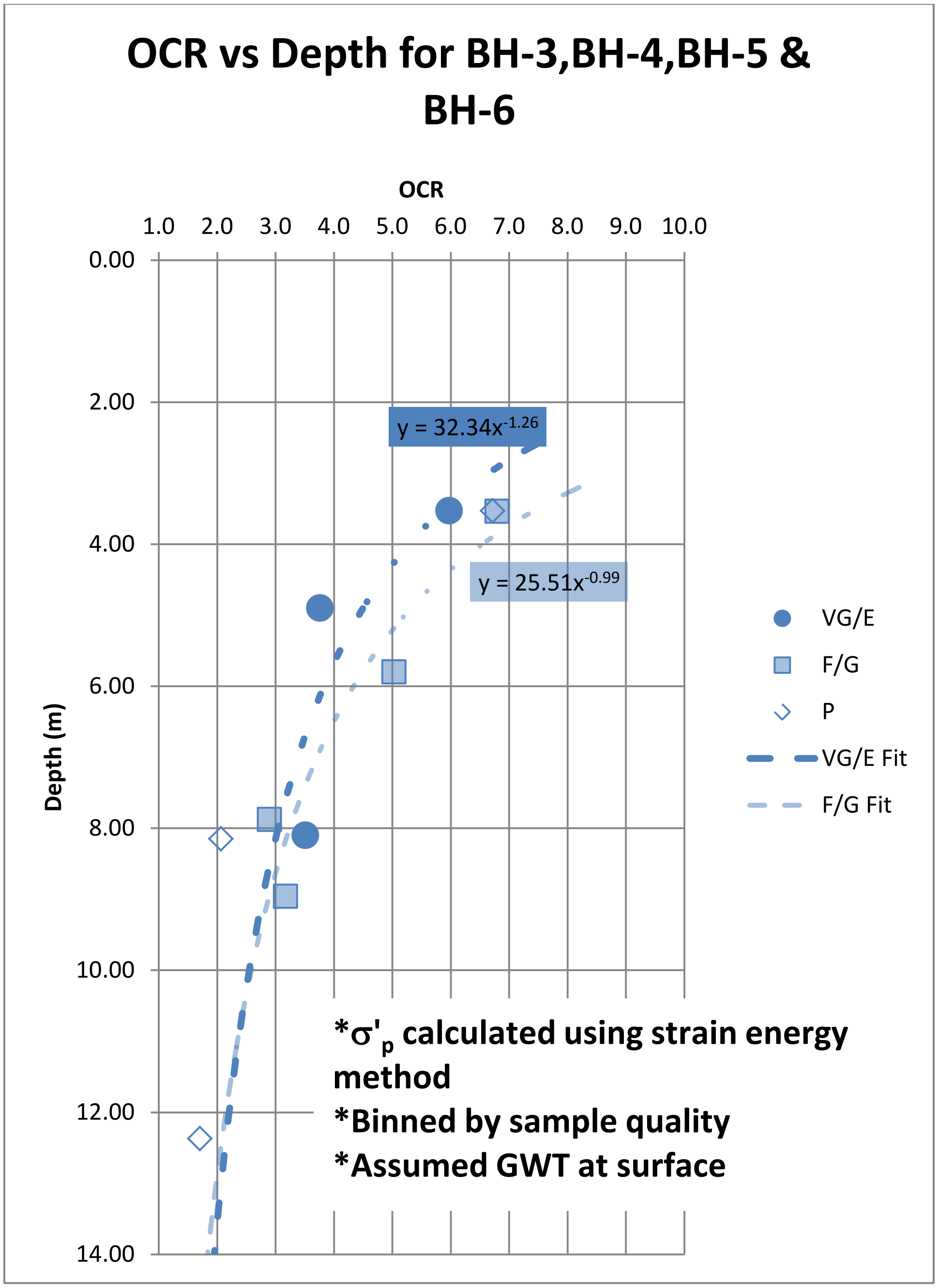


Figure 2 - OCR vs. Depth.

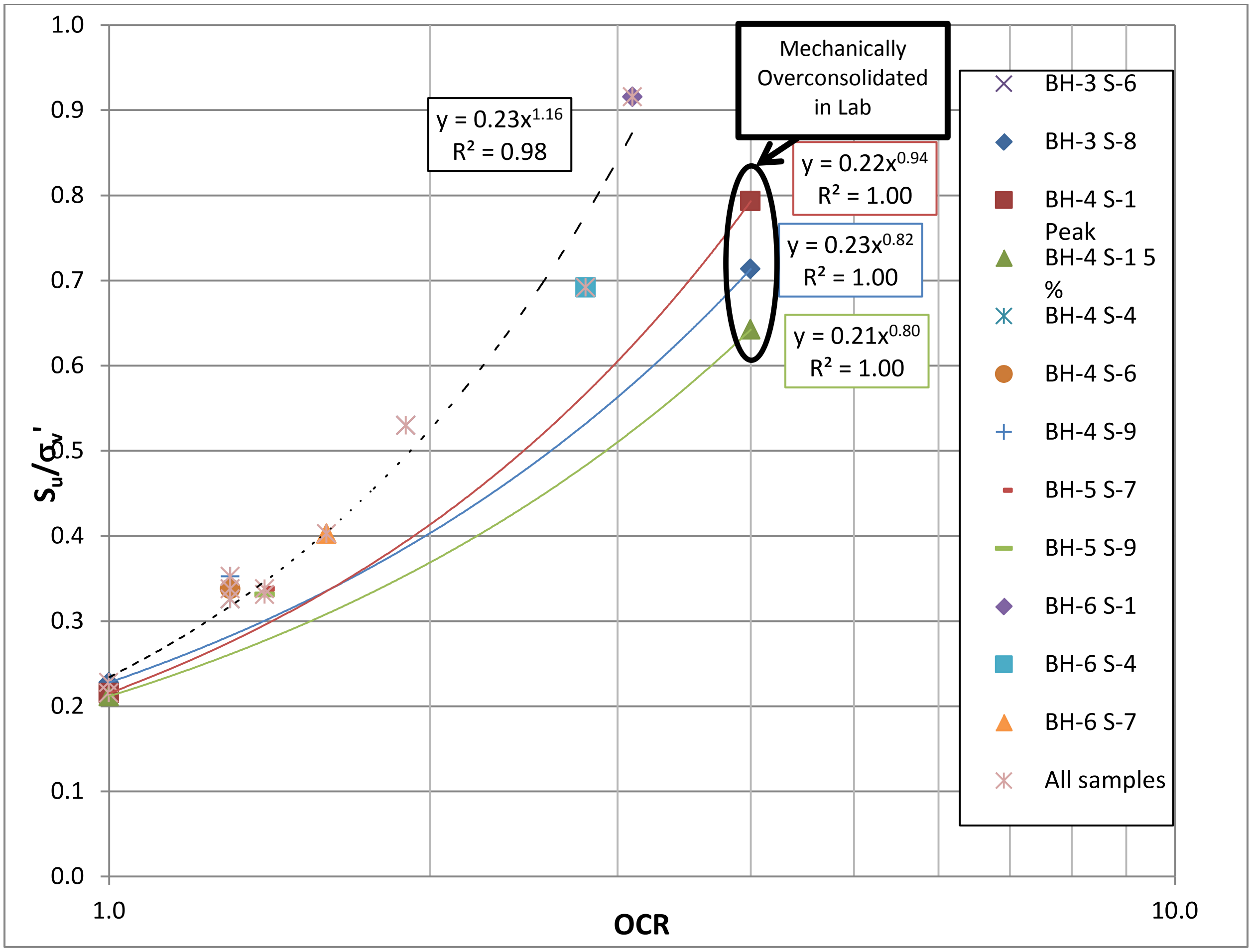


Figure 3 - Undrained Strength vs. OCR.

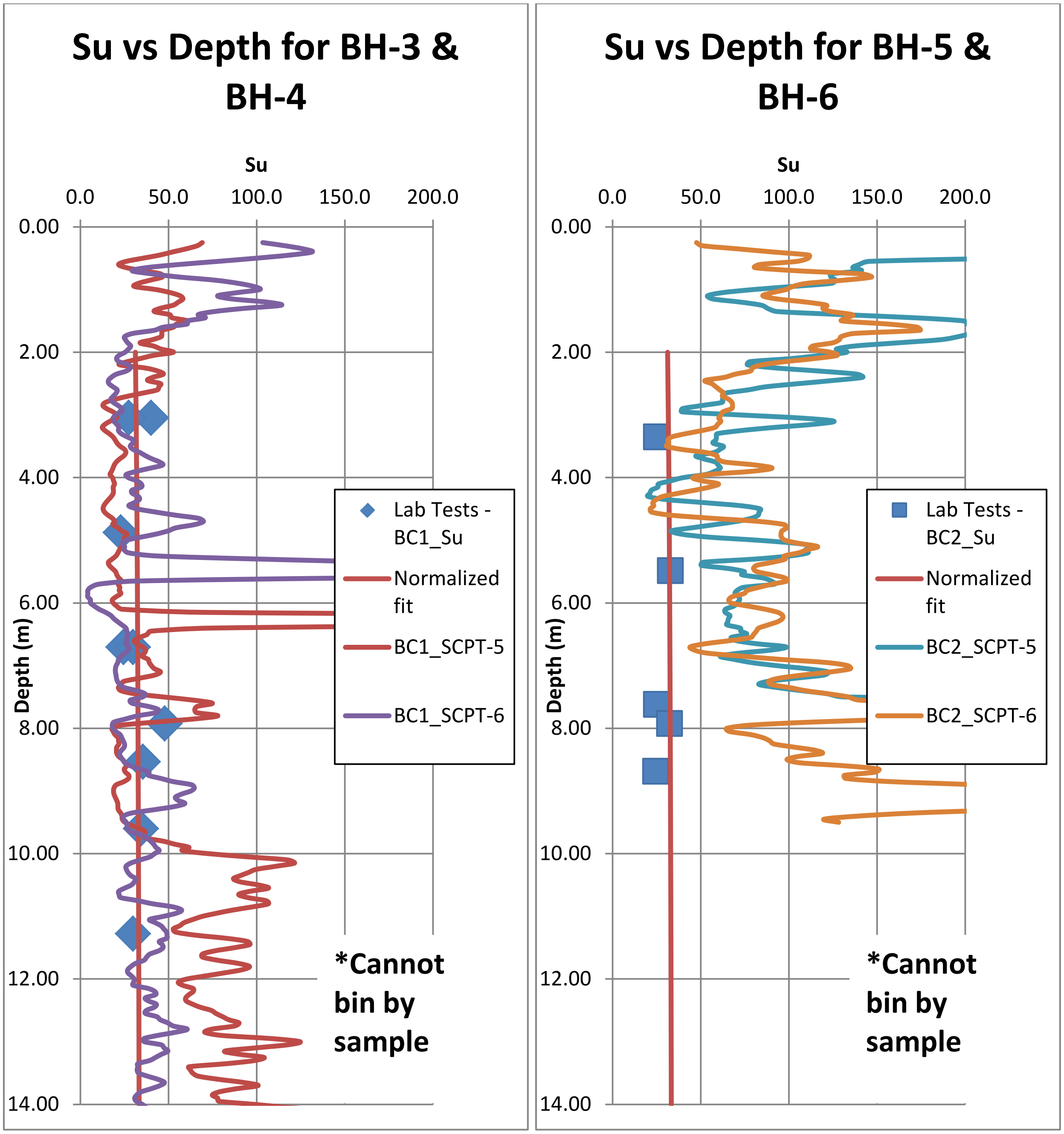


Figure 4 - Su vs Depth from sCPT results using a Nkt factor of 20.

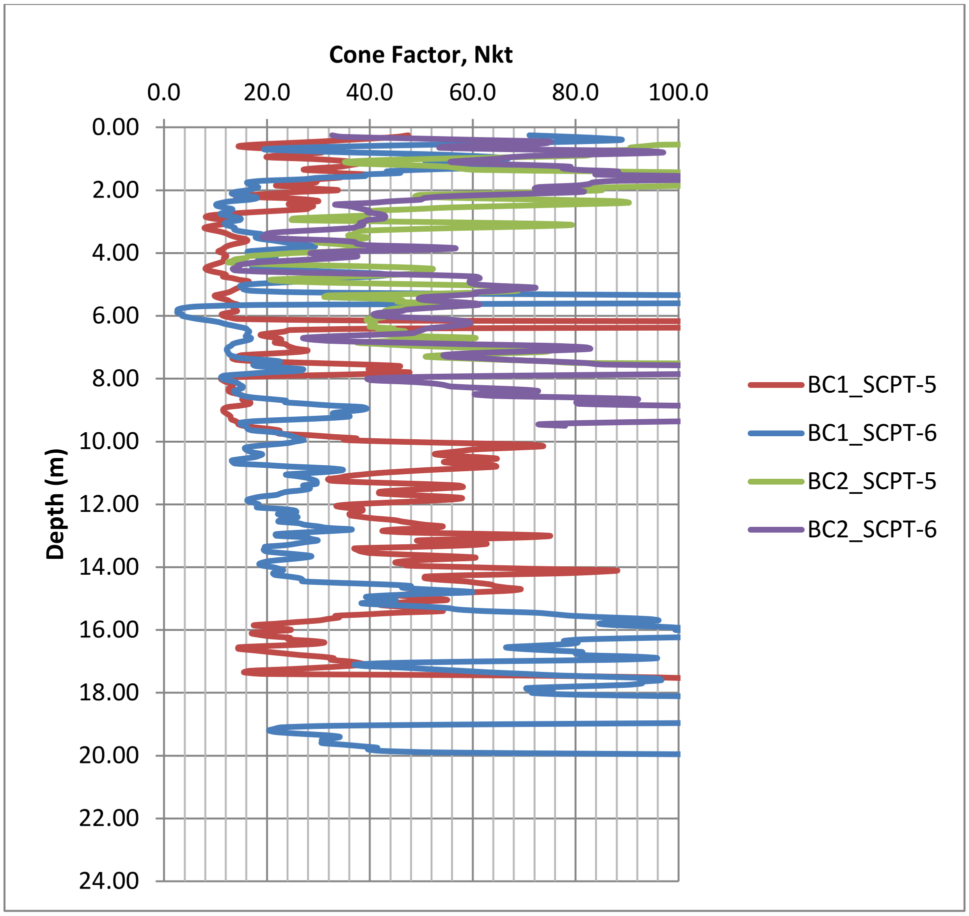
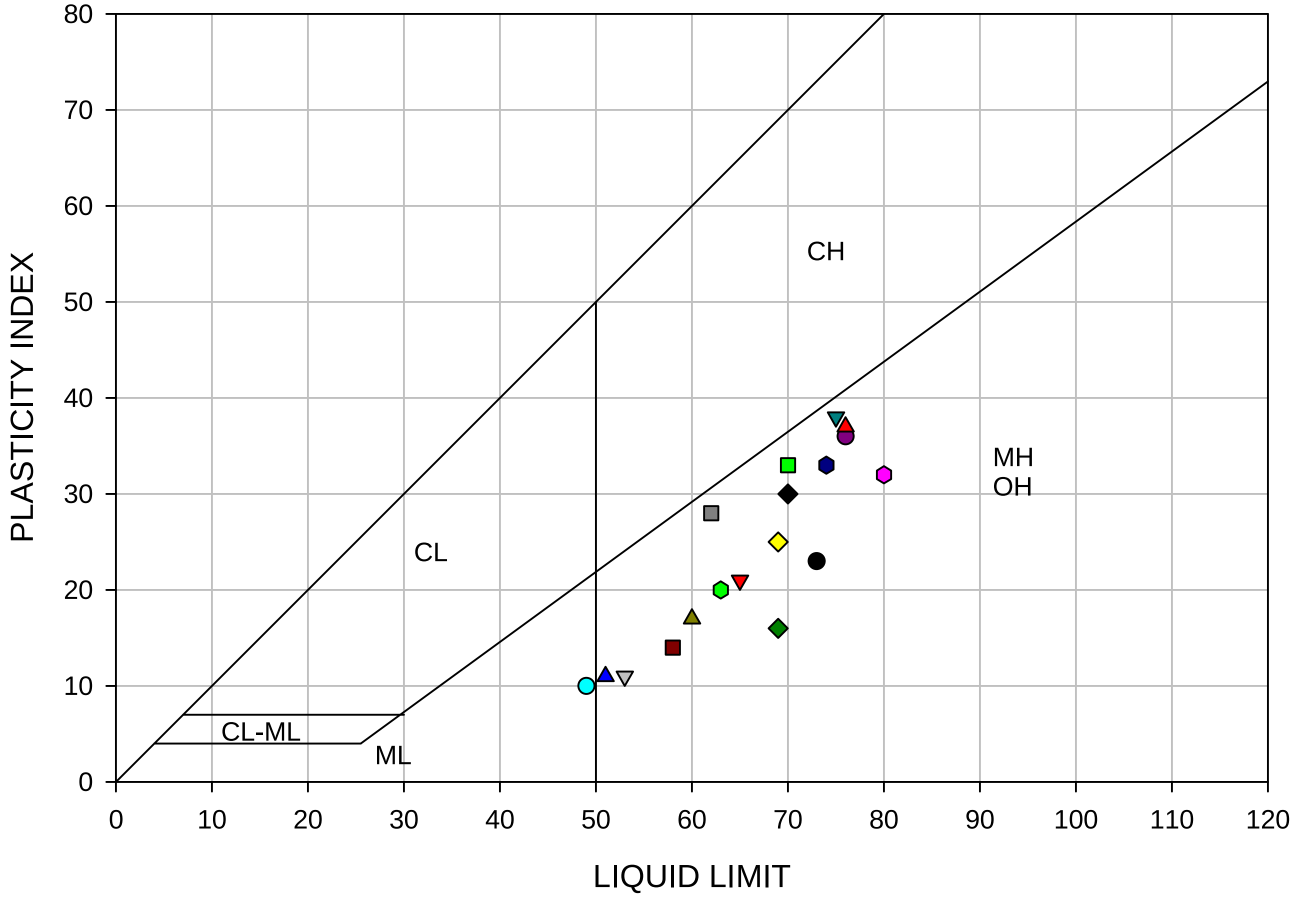


Figure 5 - Nkt vs. Depth calibrated to lab undrained strengths.

Attachment B. Phase 3 Geotechnical Data

Index Testing



Boring No.	Sample	Depth (ft)	PL	LL	PI	CI*
● BC1-BH-2	S-4	6.0-7.5	50	73	23	0.46
▼ BC1-BH-2	S-8	12.0-13	44	65	21	0.48
■ BC1-BH-2	S-13	19.5-21	37	70	33	0.89
◇ BC1-BH-2	S-17	25.5-27	44	69	25	0.57
▲ BC1-BH-2	S-21	31.5-33	40	51	11	0.28
⬢ BC1-BH-2	S-23	34.5-36	48	80	32	0.67
○ BC1-BH-2	S-25	37.5-39	39	49	10	0.26
▽ BC1-BH-2	S-29	43.5-45	42	53	11	0.26
■ BC1-BH-2	S-31	46.5-48	44	58	14	0.32
◆ BC2-BH-4	S-2	3.0-4.5	53	69	16	0.30
▲ BC2-BH-4	S-6	9.0-10.5	43	60	17	0.40
⬢ BC2-BH-4	S-14	21.0-22	41	74	33	0.80
● BC2-BH-4	S-24	36.0-37	40	76	36	0.90
▼ BC1-BH-2	S-36	54-55.5	37	75	38	1.03
■ BC1-BH-4(u)	S-11	51-52.5	34	62	28	0.82
◆ BC2-BH-5(u)	S-6	22-23.5	40	70	30	0.75
▲ BC2-BH-5(u)	S-8	27-28.5	39	76	37	0.95
⬢ BC2-BH-6(u)	S-6	22-23.5	43	63	20	0.47

* Cohesive index is PI/PL

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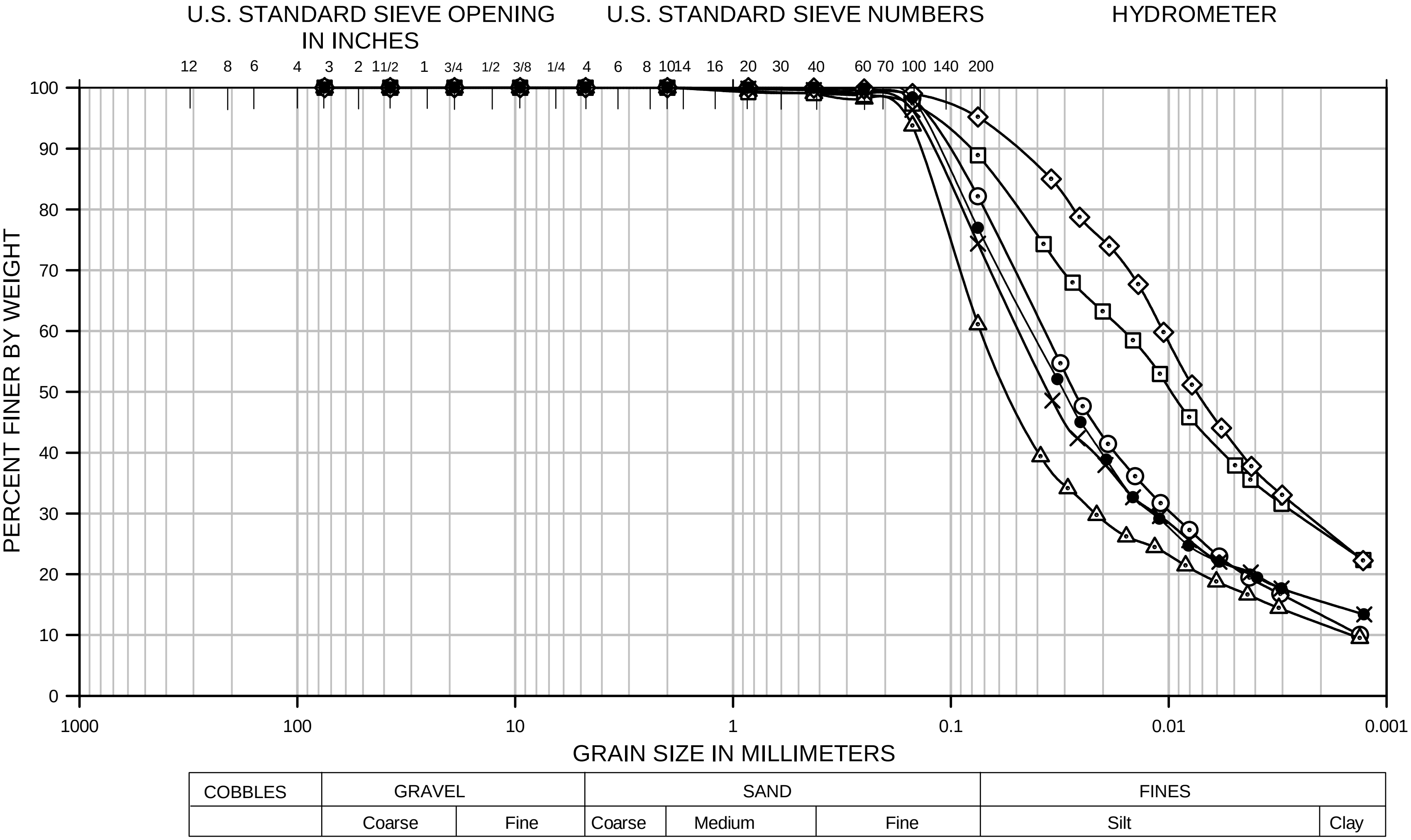
PLASTICITY CHART

BIG CREEK DAMS
NEWPORT, OR

FEB 2014

PROJ. 2328

D-1.A.1



Boring No.	Sample No.	Depth	Classification	Nat W%	LL	PL	PI
⊙ BC1-BH-2	S-4	6.0-7.5	Slightly sandy, clayey SILT (ALLUVIUM)	63	73	50	23
△ BC1-BH-2	S-12	18.0-19.5	Silty SAND to sandy SILT (ALLUVIUM)	51	NON-PLASTIC		
⊡ BC1-BH-2	S-13	19.5-21.0	Slightly sandy, silty CLAY (ALLUVIUM)	80	70	37	33
◈ BC1-BH-2	S-17	25.5-27.0	Slightly sandy, clayey SILT (ALLUVIUM)	83	69	44	25
× BC1-BH-2	S-23	34.5-36.0	Sandy, very silty CLAY(ALLUVIUM)	65	80	48	32
● BC1-BH-2	S-25	37.5-39.0	Slightly clayey SILT (ALLUVIUM)	56	49	39	10

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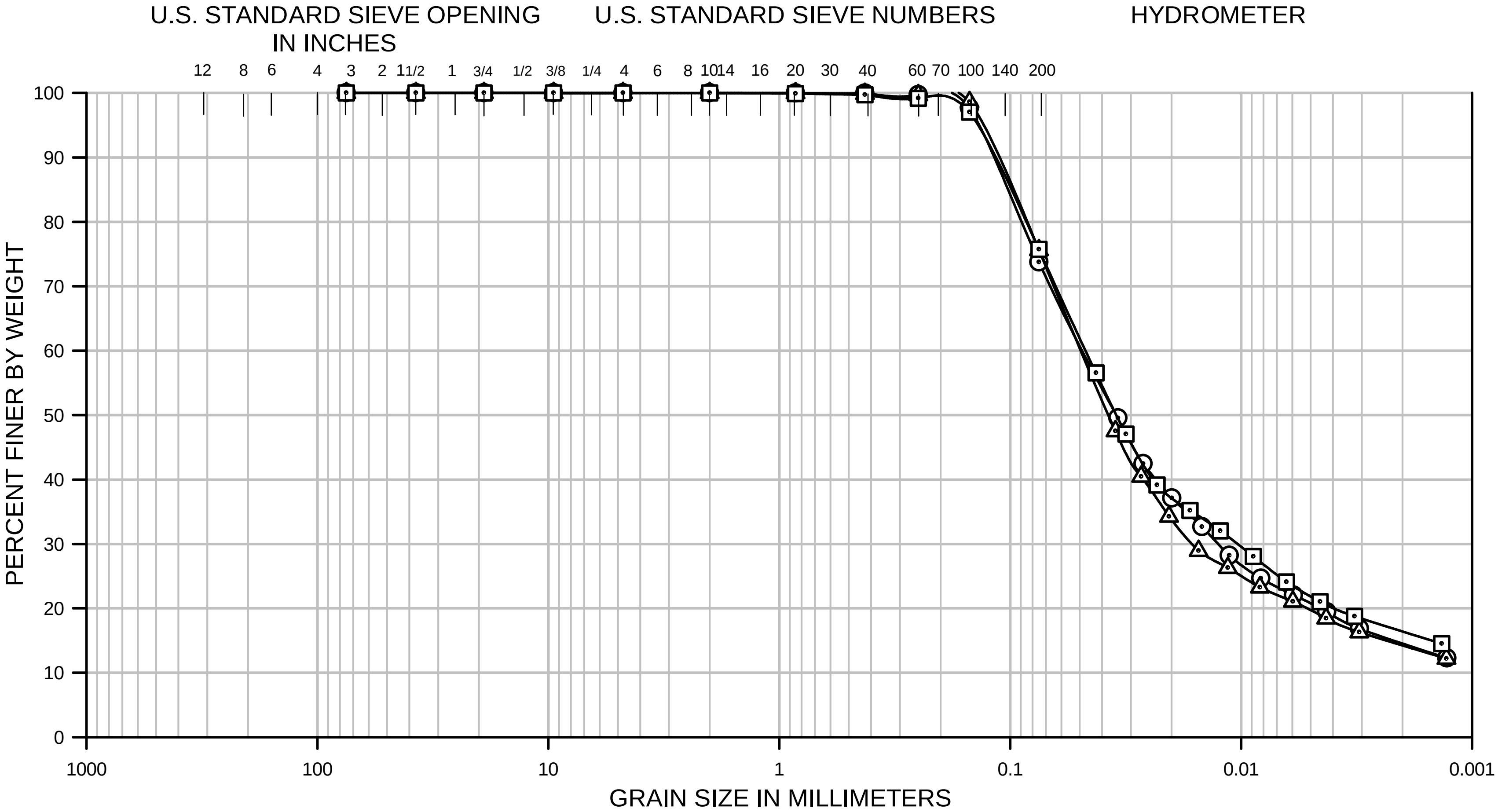
GRADATION GRAPH

BIG CREEK DAMS
NEWPORT, OR

JAN 2014

PROJ. 2328

D-1.A.2



COBBLES	GRAVEL		SAND			FINES	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

Boring No.	Sample No.	Depth	Classification	Nat W%	LL	PL	PI
⊙ BC1-BH-2	S-29	43.5-45.0	Slightly sandy, slightly clayey SILT (ALLUVIUM)	53	53	42	11
△ BC1-BH-2	S-31	46.5-48.0	Slightly sandy, slightly clayey SILT (ALLUVIUM)	57	58	44	14
⊡ BC1-BH-2	S-40	60.0-61.5	Slightly sandy, slightly clayey SILT (ALLUVIUM)	50	NON-PLASTIC		

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GRADATION GRAPH

BIG CREEK DAMS
NEWPORT, OR

D-1.A.3

PROJ. 2328

JAN 2014

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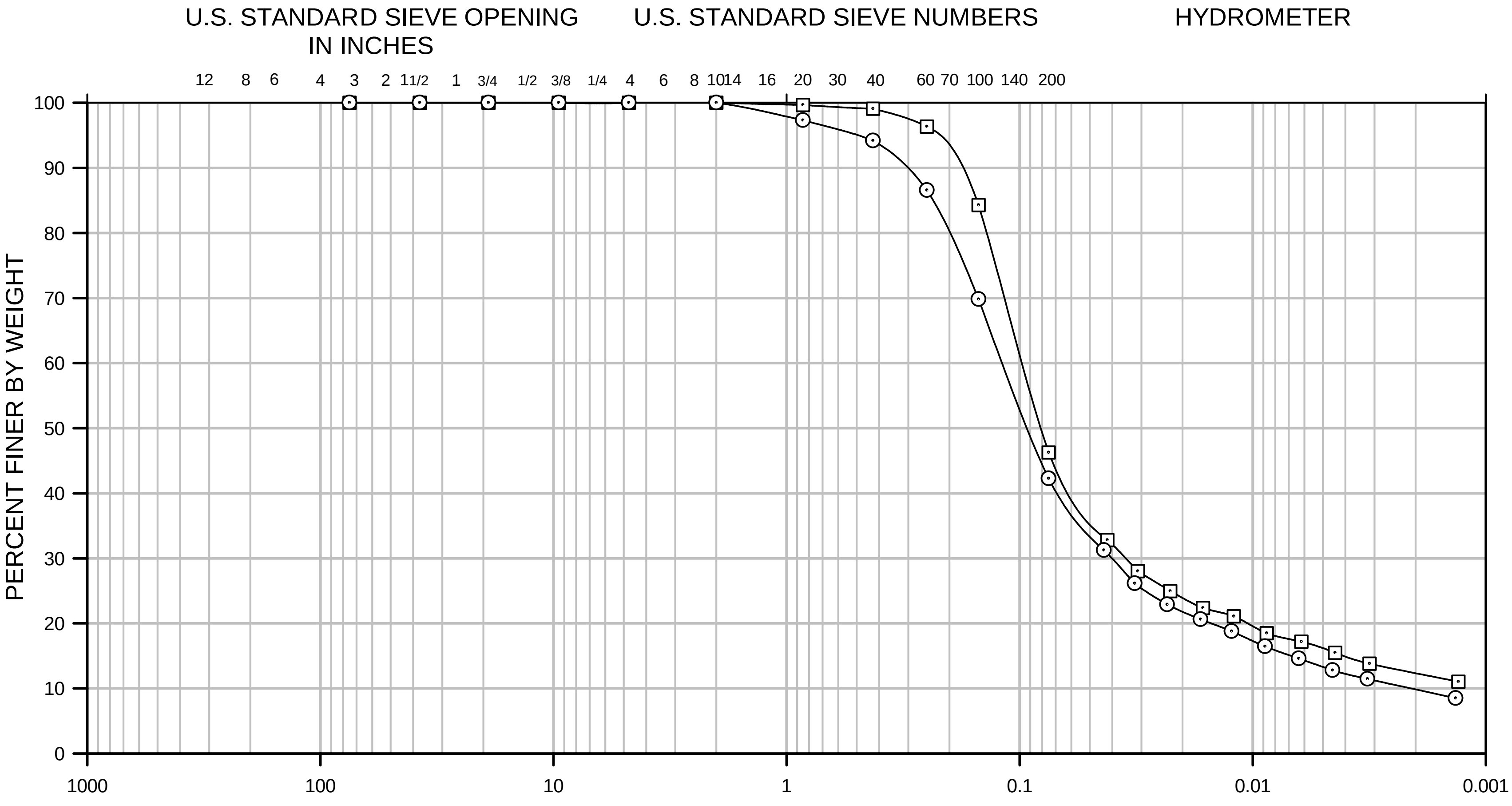
GRADATION GRAPH

BIG CREEK DAMS
NEWPORT, OR

JAN 2014

PROJ. 2328

D-1.A.4



COBBLES	GRAVEL		SAND			FINES	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

Boring No.	Sample No.	Depth	Classification	Nat W%	LL	PL	PI
⊡ BC2-BH-4	S-8	12.0-13.5	SILT and slightly silty SAND (ALLUVIUM/COLLUVIUM)	82	NON-PLASTIC		
⊙ BC2-BH-4	S-20	30.0-31.5	Slightly clayey, silty SAND (ALLUVIUM/COLLUVIUM)	58	NON-PLASTIC		

Consolidation Testing

MEG TECHNICAL SERVICES

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Marine + Earth MEG Geosciences

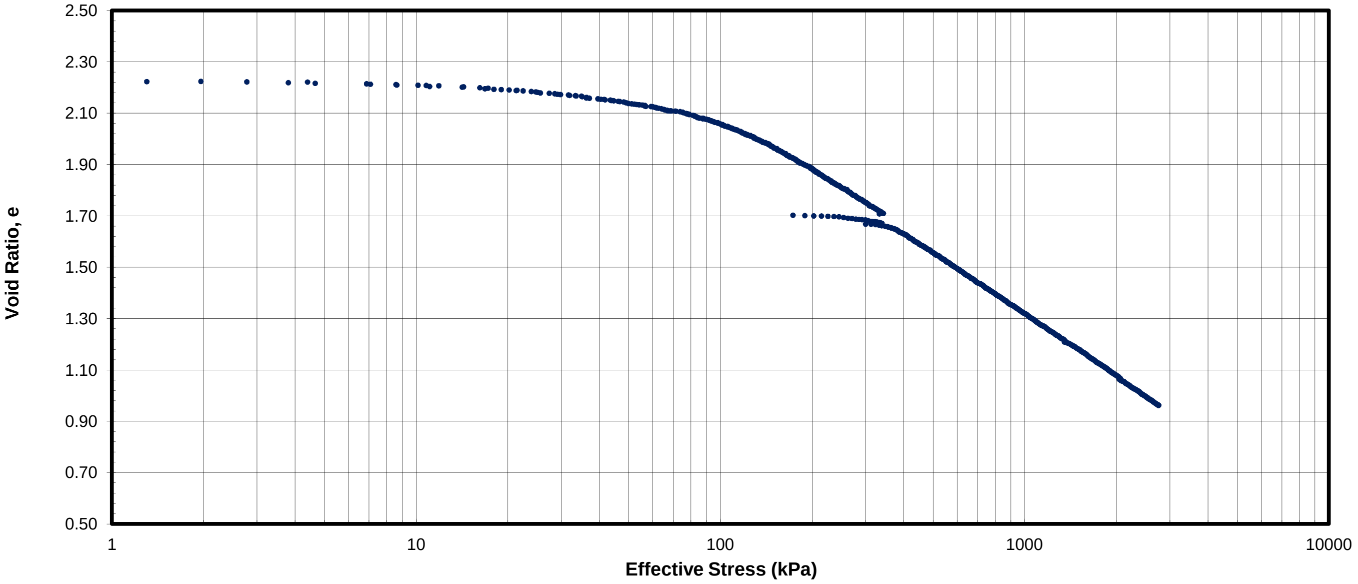
Constant Rate of Strain Consolidation (ASTM D4186)

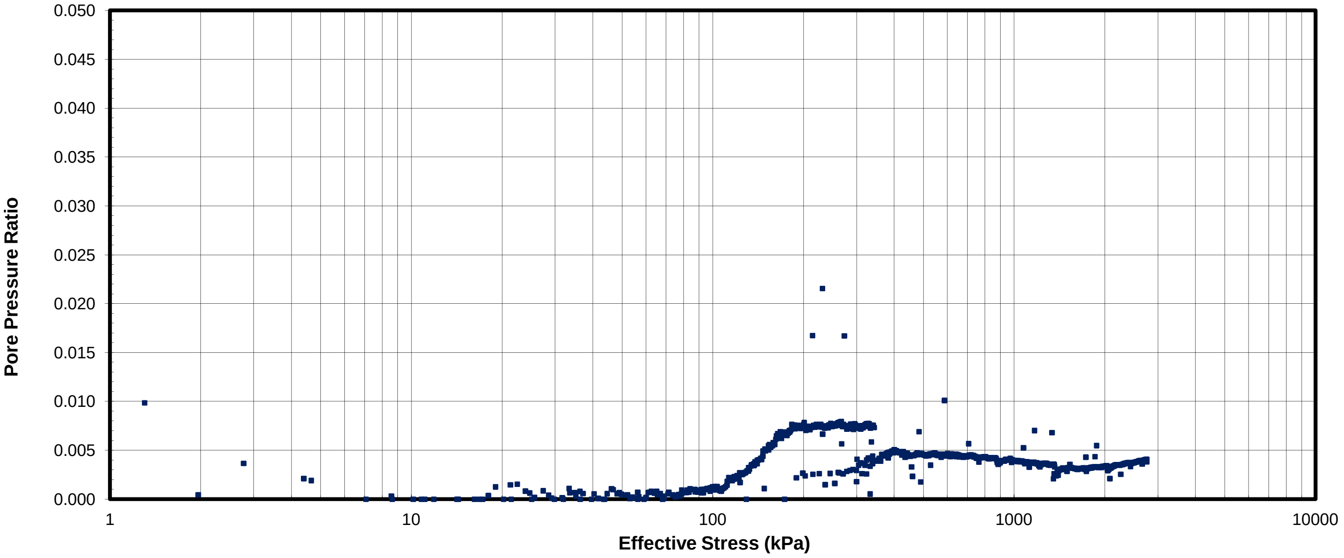
Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 1, 2014
Borehole:	BH3	Station:	CRS 2
Sample No.:	S1	Depth (m):	4.90

Weight of Ring (g):	211.22	**Ring + Wet Weight (g):**	329.07	**Initial Void Ratio, e:**	2.23
Initial Height (mm):	25.41	**Ring + Dry Weight (g):**	275.21	**Height of Soil, Hs (mm):**	7.87
Diameter of Ring (mm):	63.50	**Water Content (%):**	84.2	**Height of Void, Hv (mm):**	17.54
Unit Weight (kN/m^3)	14.37	**Specific Gravity, Gs:**	2.57		

Loading Strain Rate (%/hr):	0.8	**Max Stress (kPa):**	2749	**Backpressure (kPa):**	407
Unloading Strain Rate (%/hr):	N/A	**Max Strain (%):**	39.2		

Comment: rate changed to 1.6% strain per hour at 68.8 kPa





Prepared By:	PC	Checked By:	PS	Approved By:	EP
Date:	May 1, 2014	Date:	May 1, 2014	Date:	May 1, 2014

MEG TECHNICAL SERVICES

(A Division of MEG Consulting Limited)

Constant Rate of Strain Consolidation (ASTM D4186)

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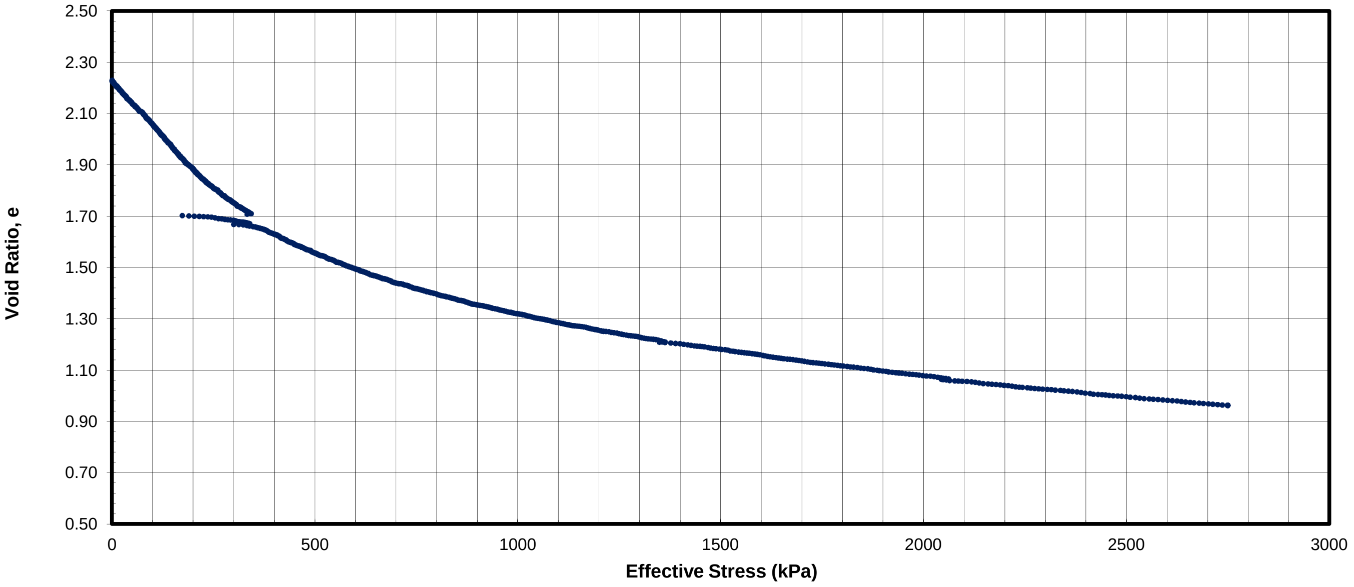
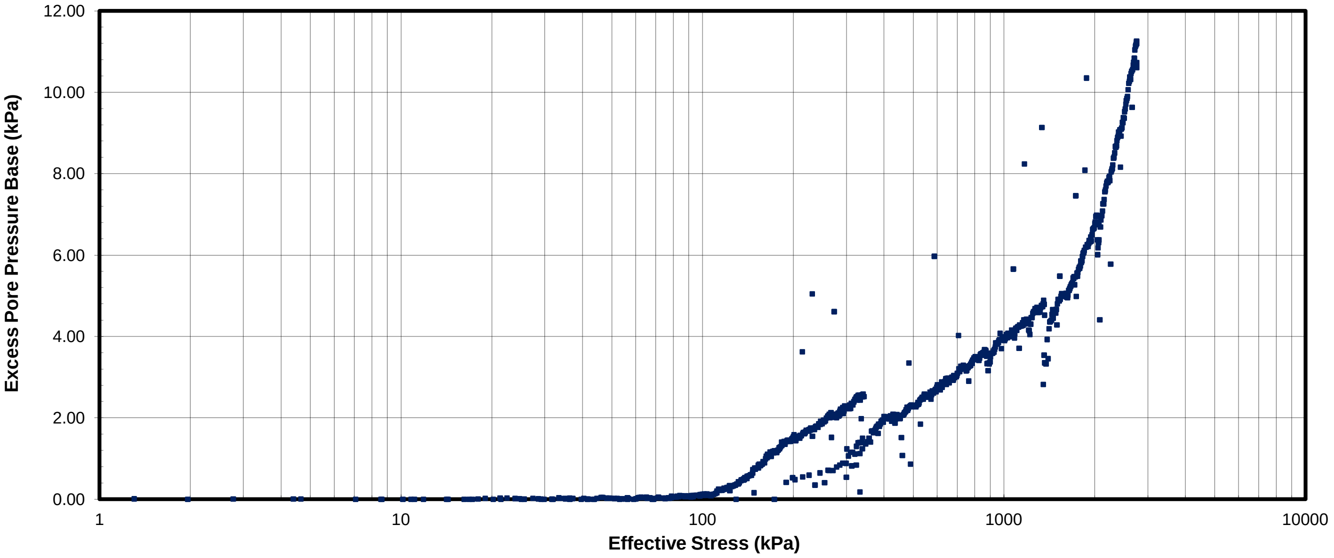
Location:	Oregon, USA	Date:	May 1, 2014
Borehole:	BH3	Station:	CRS 2
Sample No.:	S1	Depth (m):	4.90

Weight of Ring (g):	211.22	**Ring + Wet Weight (g):**	329.07	**Initial Void Ratio, e:**	2.23
Initial Height (mm):	25.41	**Ring + Dry Weight (g):**	275.21	**Height of Soil, Hs (mm):**	7.87
Diameter of Ring (mm):	63.50	**Water Content (%):**	84.2	**Height of Void, Hv (mm):**	17.54
Unit Weight (kN/m^3)	14.37	**Specific Gravity, Gs:**	2.57		

Loading Strain Rate (%/hr):	0.8	**Max Stress (kPa):**	2749	**Backpressure (kPa):**	407
Unloading Strain Rate (%/hr):	N/A	**Max Strain (%):**	39.2		

Comment: rate changed to 1.6% strain per hour at 68.8 kPa





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Constant Rate of Strain Consolidation (ASTM D4186)

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Diameter of Ring (mm):	63.50	**Water Content (%):**	84.2	**Height of Void, Hv (mm):**	17.54
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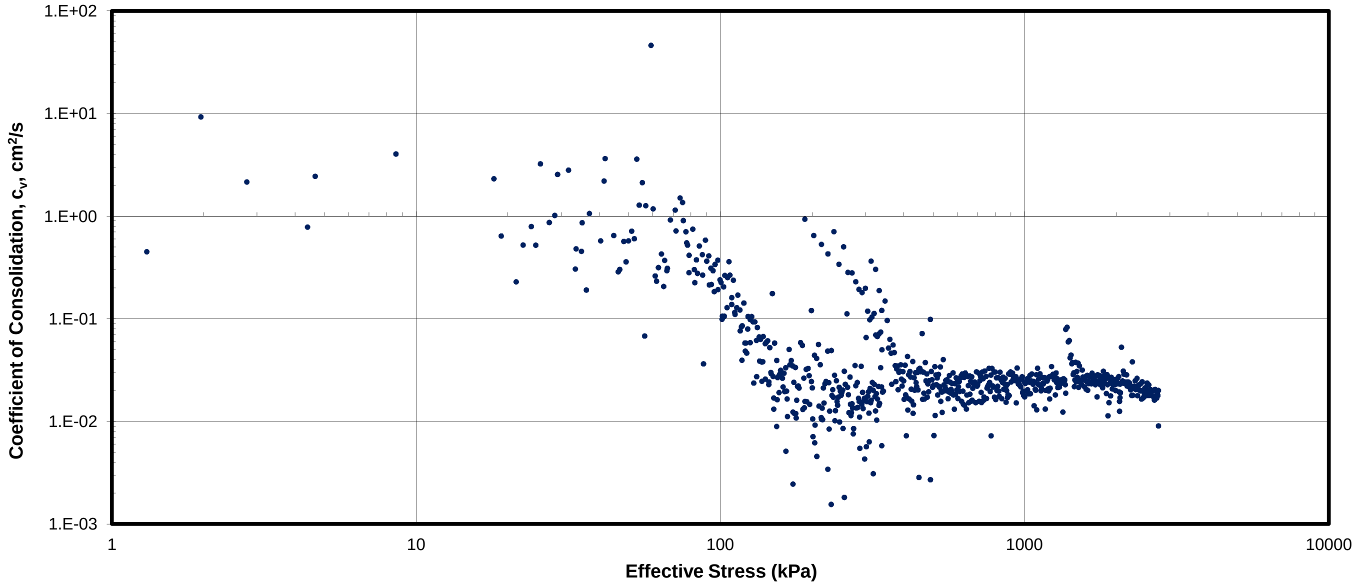
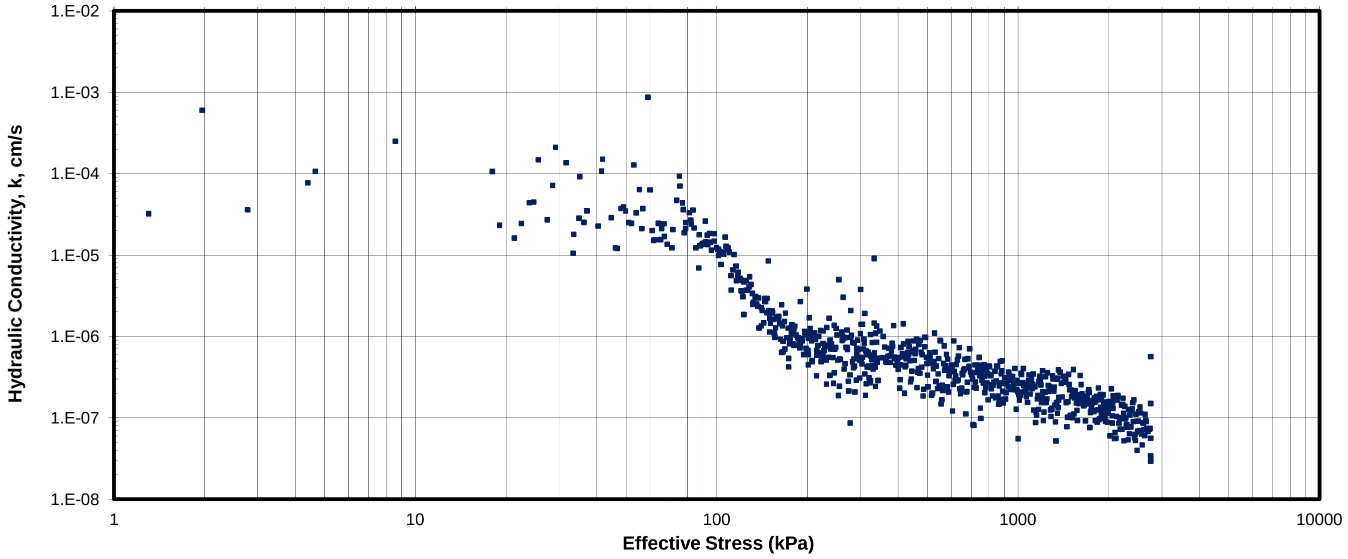
Unloading Strain Rate (%/hr):	N/A	**Max Strain (%):**	39.2		

Comment: **1.6% strain per hour at 68.8 kPa**





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Date:	May 1, 2014	Date:	May 1, 2014	Date:	May 1, 2014

MEG TECHNICAL SERVICES

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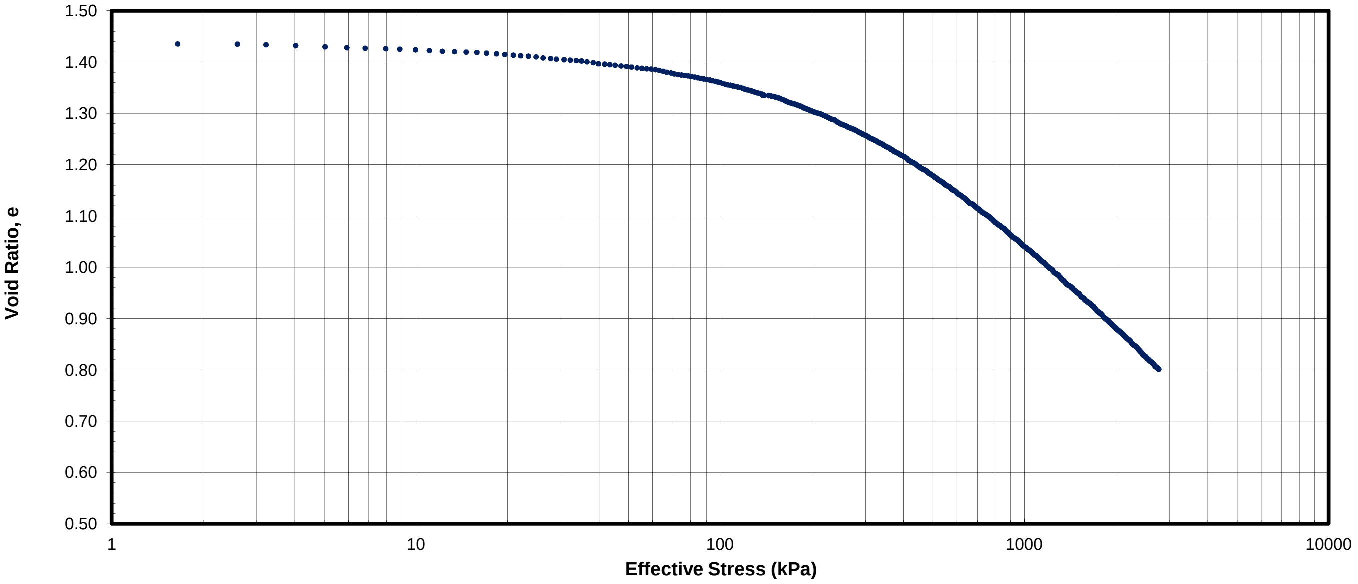
Constant Rate of Strain Consolidation (ASTM D4186)

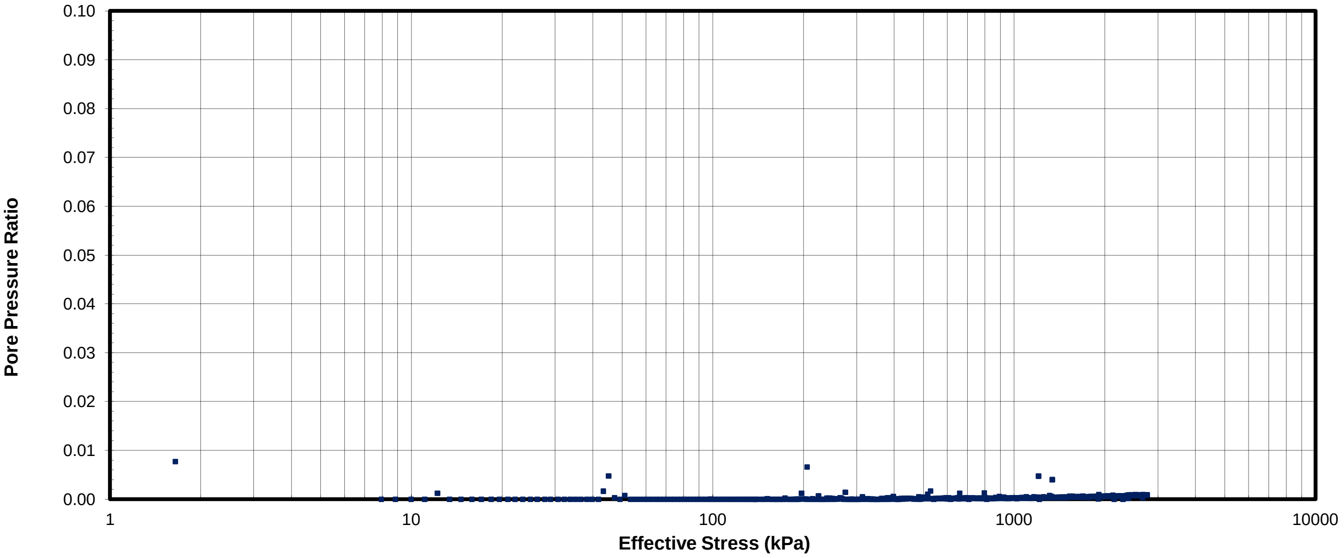
Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 15, 2014
Borehole:	BH3	Station:	1
Sample No.:	S6	Depth (m):	8.96

Weight of Ring (g):	211.23	**Ring + Wet Weight (g):**	343.60	**Initial Void Ratio, e:**	1.49
Initial Height (mm):	25.30	**Ring + Dry Weight (g):**	298.89	**Height of Soil, Hs (mm):**	10.15
Diameter of Ring (mm):	63.50	**Water Content (%):**	51.0	**Height of Void, Hv (mm):**	15.15
Unit Weight (kN/m^3)	16.14	**Specific Gravity, Gs:**	2.73		

Loading Strain Rate (%/hr):	2	**Max Stress (kPa):**	2759	**Backpressure (kPa):**	417
Unloading Strain Rate (%/hr):	N/A	**Max Strain (%):**	27.7		

Comments: test started at 2% per hour and rate was increased to 4% per hour at 6.69% strain (144 kPa)





Prepared By:	PC	Checked By:	PS	Approved By:	
Date:	May 16, 2014	Date:	May 16, 2014	Date:	

MEG TECHNICAL SERVICES

(A Division of MEG Consulting Limited)

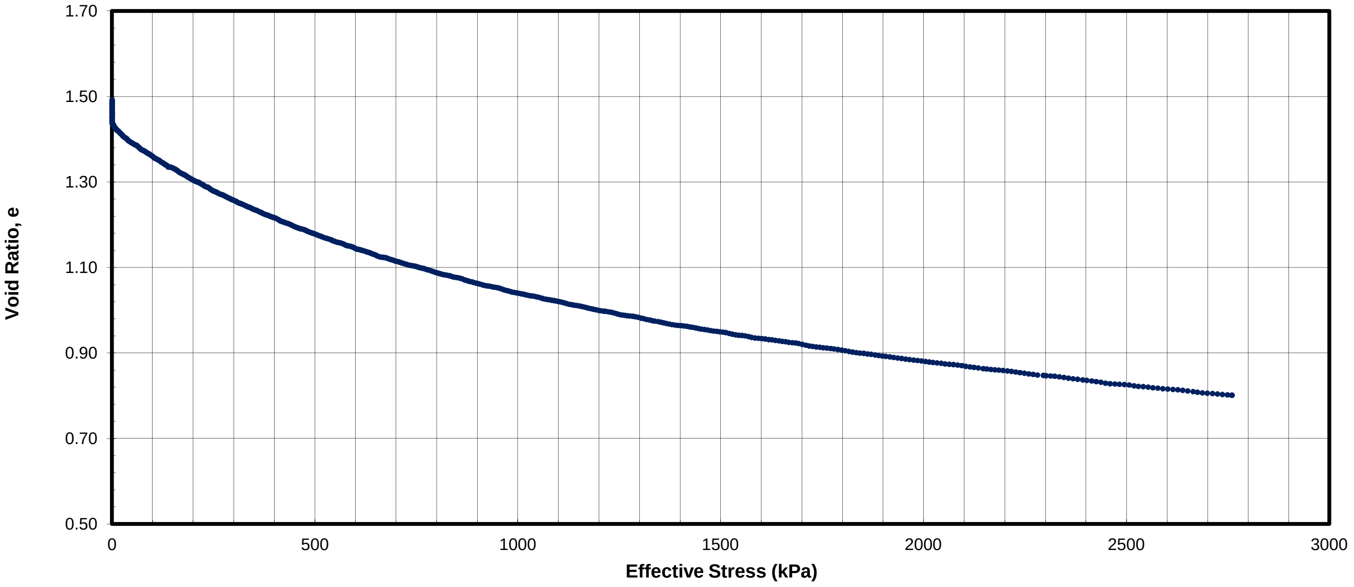
Constant Rate of Strain Consolidation (ASTM D4186)

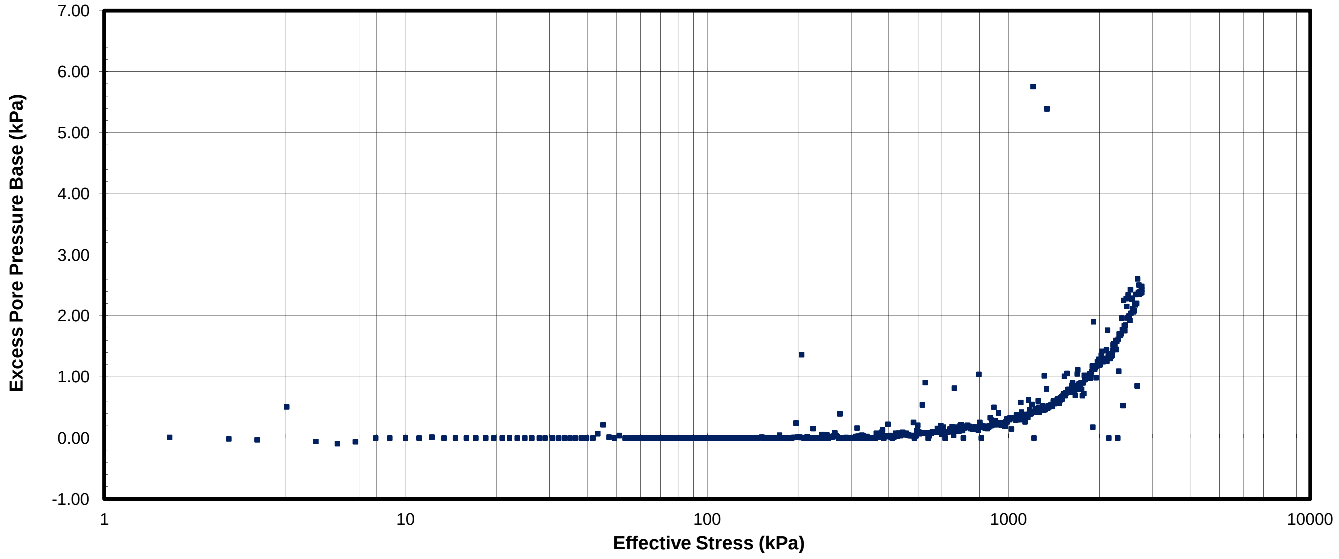
Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 15, 2014
Borehole:	BH3	Station:	1
Sample No.:	S6	Depth (m):	8.96

Weight of Ring (g):	211.23	**Ring + Wet Weight (g):**	343.60	**Initial Void Ratio, e:**	1.49
Initial Height (mm):	25.30	**Ring + Dry Weight (g):**	298.89	**Height of Soil, Hs (mm):**	10.15
Diameter of Ring (mm):	63.50	**Water Content (%):**	51.0	**Height of Void, Hv (mm):**	15.15
Unit Weight (kN/m^3)	16.14	**Specific Gravity, Gs:**	2.73		

Loading Strain Rate (%/hr):	2	**Max Stress (kPa):**	2759	**Backpressure (kPa):**	417
Unloading Strain Rate (%/hr):	N/A	**Max Strain (%):**	27.7		

Comments: test started at 2% per hour and rate was increased to 4% per hour at 6.69% strain (144 kPa)





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Date:	May 16, 2014	Date:	May 16, 2014	Date:	

MEG TECHNICAL SERVICES

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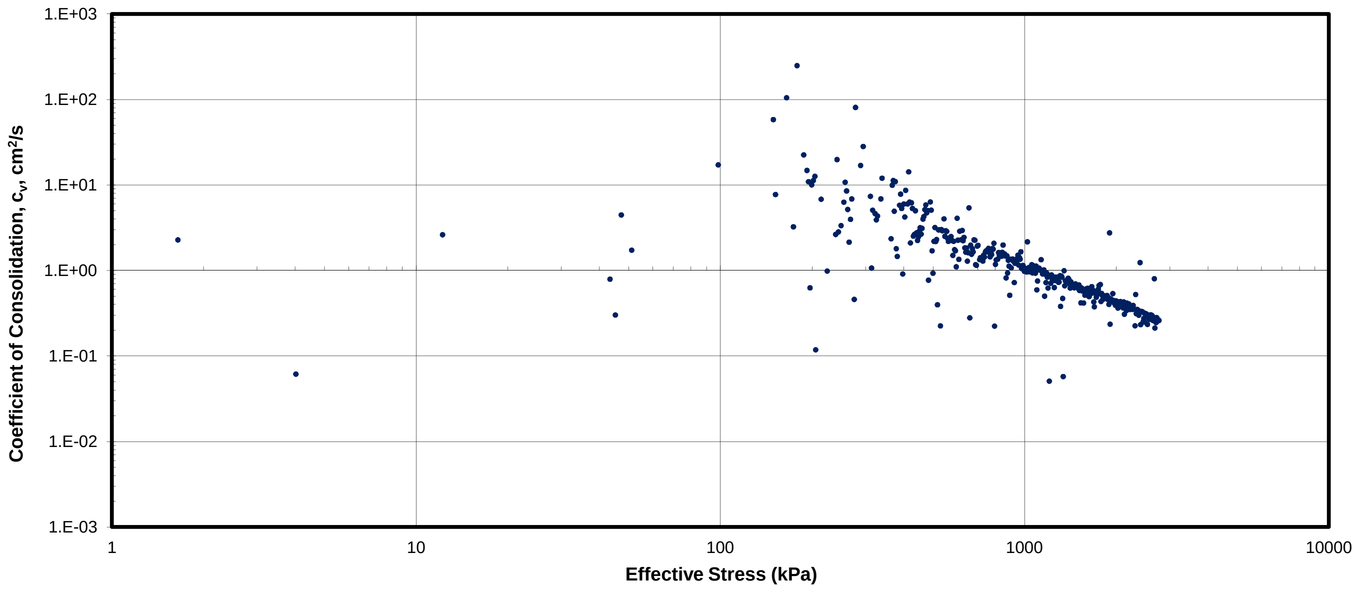
Constant Rate of Strain Consolidation (ASTM D4186)

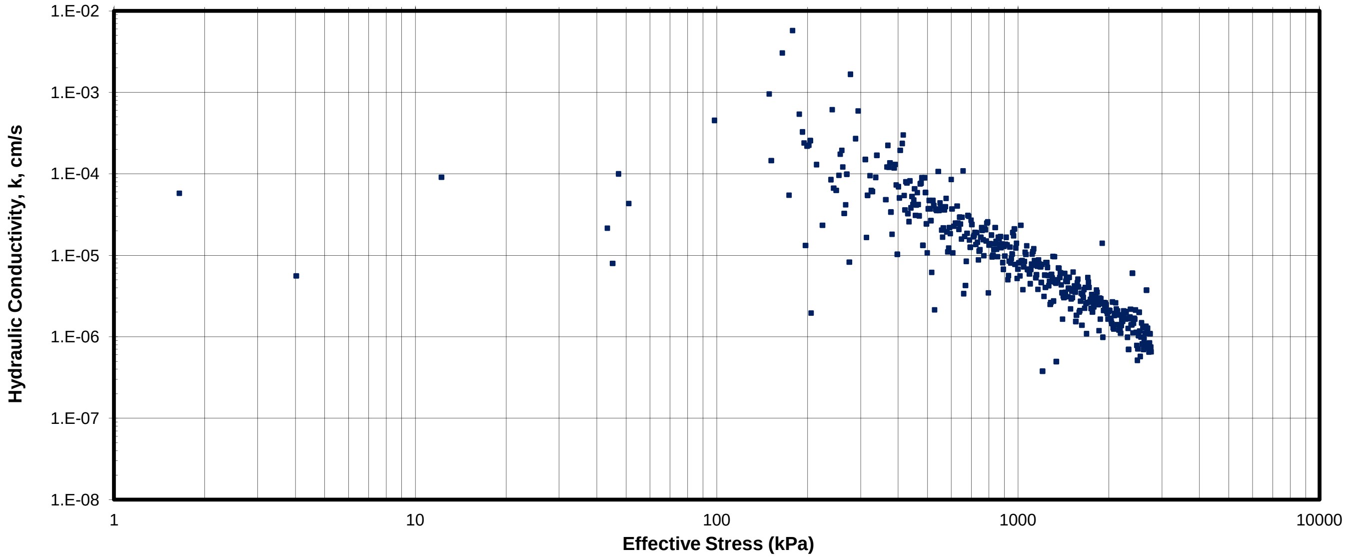
Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 15, 2014
Borehole:	BH3	Station:	1
Sample No.:	S6	Depth (m):	8.96

Weight of Ring (g):	211.23	**Ring + Wet Weight (g):**	343.60	**Initial Void Ratio, e:**	1.49
Initial Height (mm):	25.30	**Ring + Dry Weight (g):**	298.89	**Height of Soil, Hs (mm):**	10.15
Diameter of Ring (mm):	63.50	**Water Content (%):**	51.0	**Height of Void, Hv (mm):**	15.15
Unit Weight (kN/m^3)	16.14	**Specific Gravity, Gs:**	2.73		

Loading Strain Rate (%/hr):	2	**Max Stress (kPa):**	2759	**Backpressure (kPa):**	417
Unloading Strain Rate (%/hr):	N/A	**Max Strain (%):**	27.7		

Comments: test started at 2% per hour and rate was increased to 4% per hour at 6.69% strain (144 kPa)





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Date:	May 16, 2014	Date:	May 16, 2014	Date:	

MEG TECHNICAL SERVICES

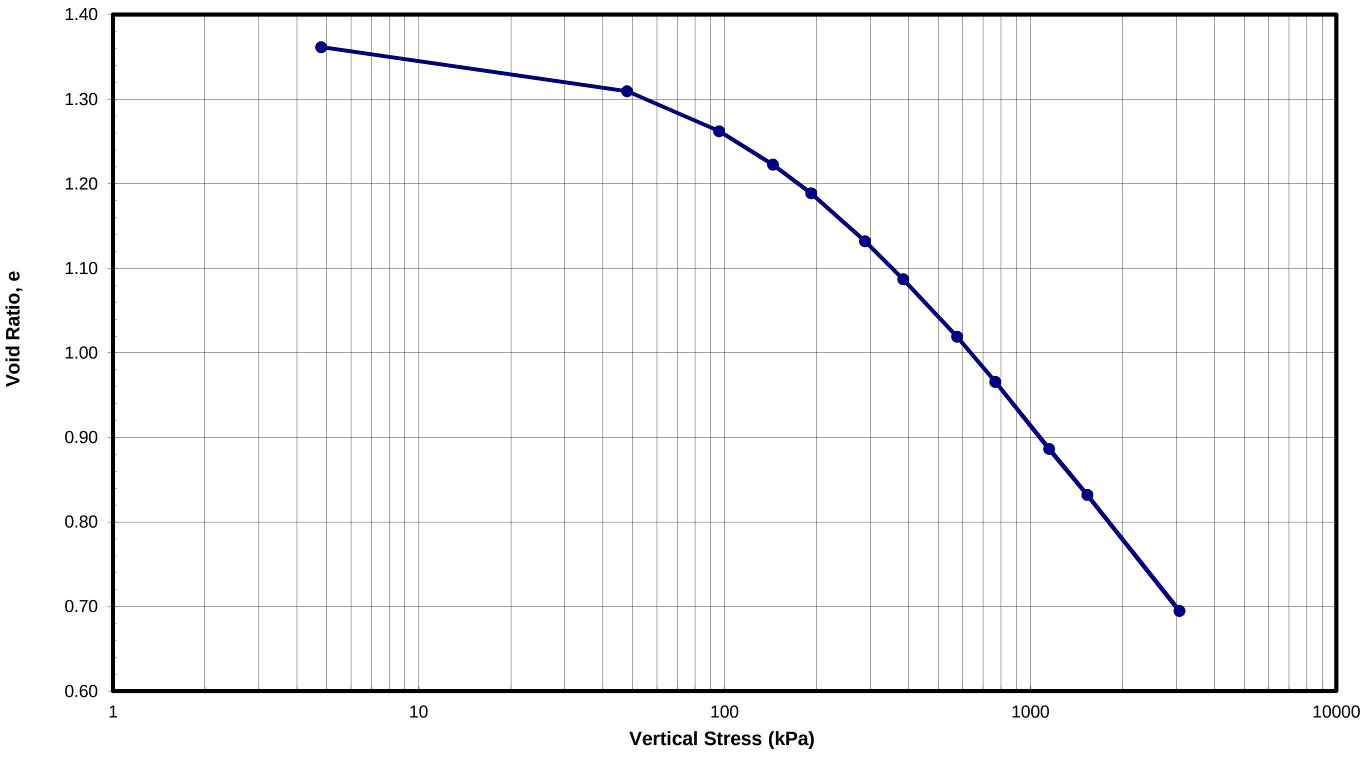
(A Division of MEG Consulting Limited)

One-Dimensional Consolidation (ASTM D 2435)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BH4	Station:	3
Sample No.:	S1	Depth (m):	3.54

Weight of Ring (g):	211.92	**Ring + Wet Weight (g):**	344.73	**Initial Void Ratio, e:**	1.37
Initial Height (mm):	25.40	**Ring + Dry Weight (g):**	301.94	**Height of Soil, Hs (mm):**	10.74
Diameter of Ring (mm):	63.50	**Water Content (%):**	47.5	**Height of Void, Hv (mm):**	14.66
Unit Weight (kN/m^3)	16.20	**Specific Gravity, Gs:**	2.65		

Step No.	**Vertical Stress (kPa)**	**Height of Sample (mm)**	**Vertical Strain (%)**	**Final Void Ratio e_f**	**Change in Void Ratio e**	**Coefficient of Compressibility a_v (m^2/MN)**	**Coefficient of Volume Compressibility m_v (m^2/MN)**
1	5	25.3594	0.1600	1.3616	0.00		
2	48	24.8006	2.3600	1.3096	0.05	1.2076	0.51
3	96	24.2926	4.3600	1.2623	0.05	0.9881	0.42
4	144	23.8684	6.0300	1.2228	0.04	0.8250	0.35
5	192	23.5052	7.4600	1.1890	0.03	0.7065	0.30
6	287	22.8956	9.8600	1.1322	0.06	0.5928	0.25
7	383	22.4130	11.7600	1.0873	0.04	0.4693	0.20
8	575	21.6840	14.6300	1.0194	0.07	0.3545	0.15
9	766	21.1099	16.8900	0.9659	0.05	0.2791	0.12
10	1149	20.2590	20.2400	0.8867	0.08	0.2069	0.09
11	1532	19.6748	22.5400	0.8323	0.05	0.1420	0.06
12	3064	18.2016	28.3400	0.6951	0.14	0.0895	0.04



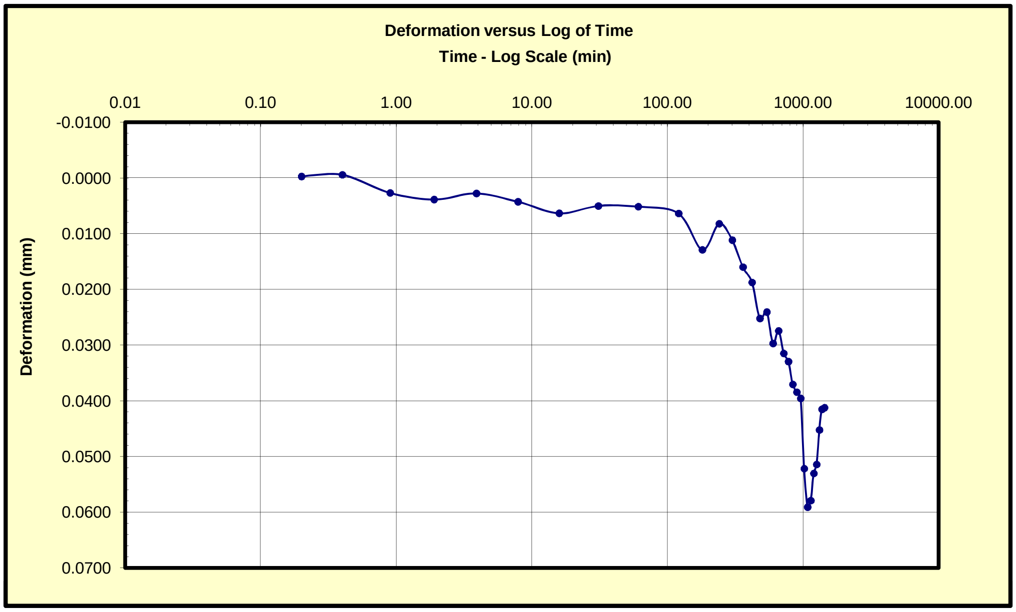
Prepared By:	PC	Checked By:	PS	Approved By:	EP
Date:	May 7, 2014	Date:	May 7, 2014	Date:	May 7, 2014

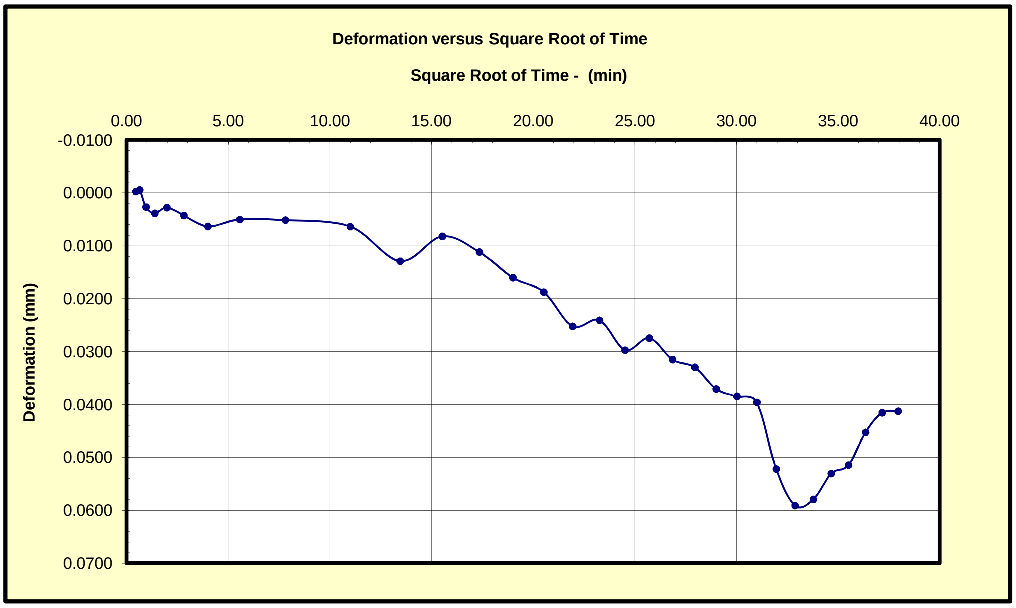
MEG TECHNICAL SERVICES

(A Division of MEG Consulting Limited)

One-Dimensional Consolidation (ASTM D 2435)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BH4	Station:	3
Sample No.:	S1	Depth (m):	3.54
Consolidation Step:	1	Vertical Stress (kPa):	5





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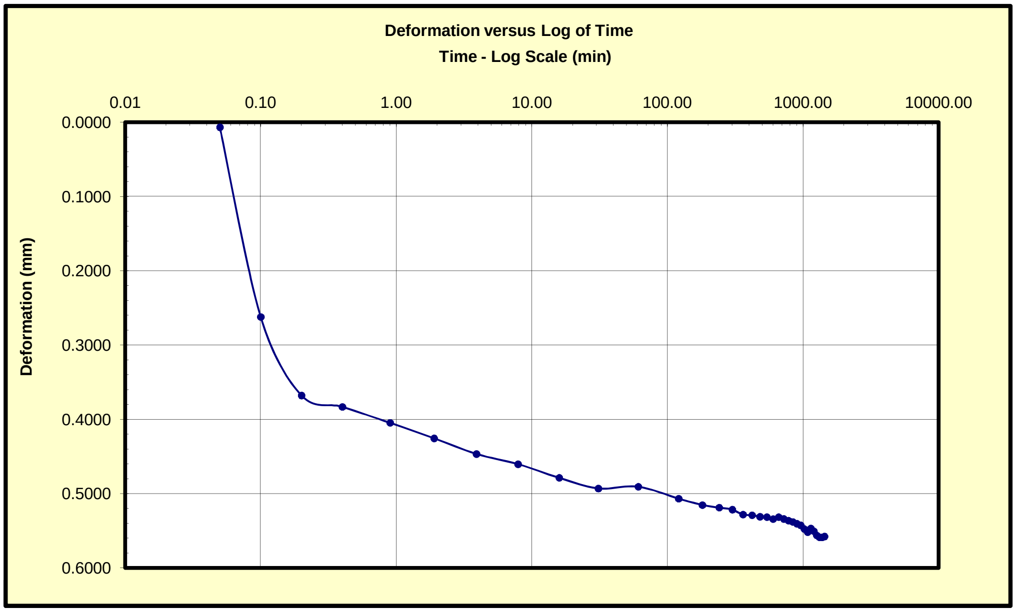
Date:	May 7, 2014	Date:	May 7, 2014	Date:	May 7, 2014

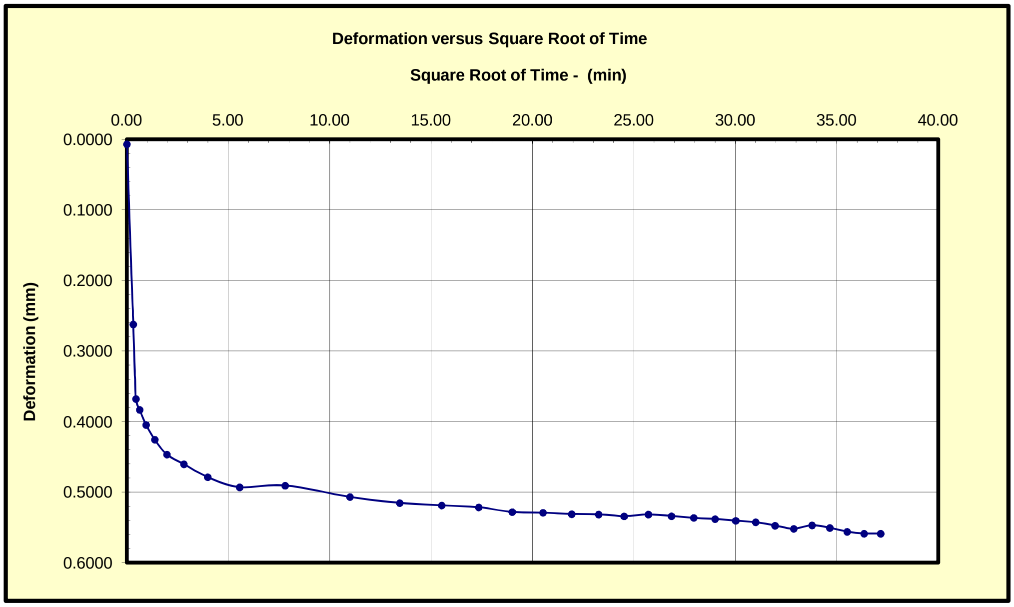
MEG TECHNICAL SERVICES

(A Division of MEG Consulting Limited)

One-Dimensional Consolidation (ASTM D 2435)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BH4	Station:	3
Sample No.:	S1	Depth (m):	3.54
Consolidation Step:	2	Vertical Stress (kPa):	48





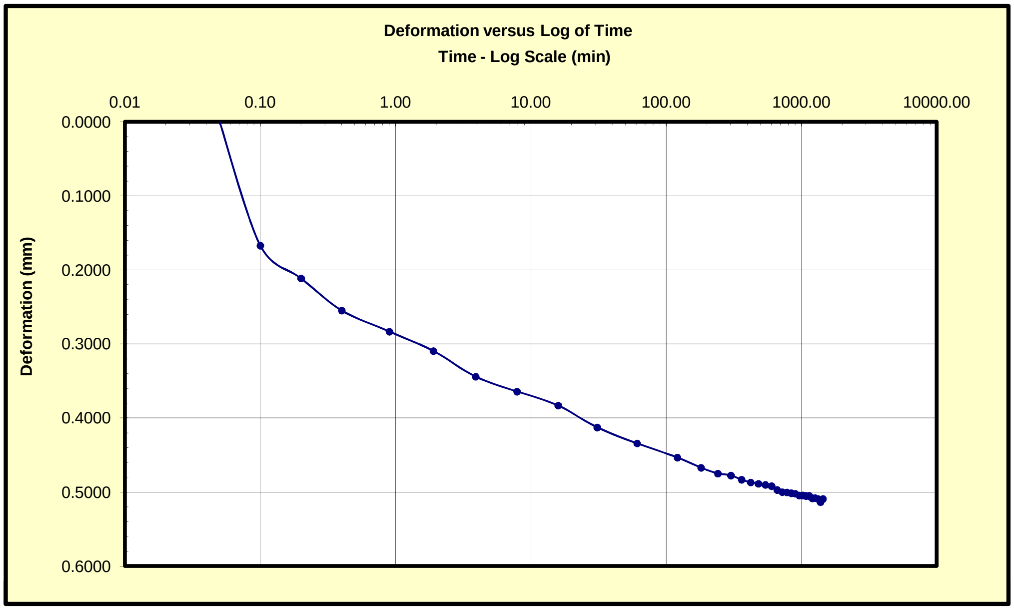
Prepared By:	PC	Checked By:	PS	Approved By:	EP
Date:	May 7, 2014	Date:	May 7, 2014	Date:	May 7, 2014

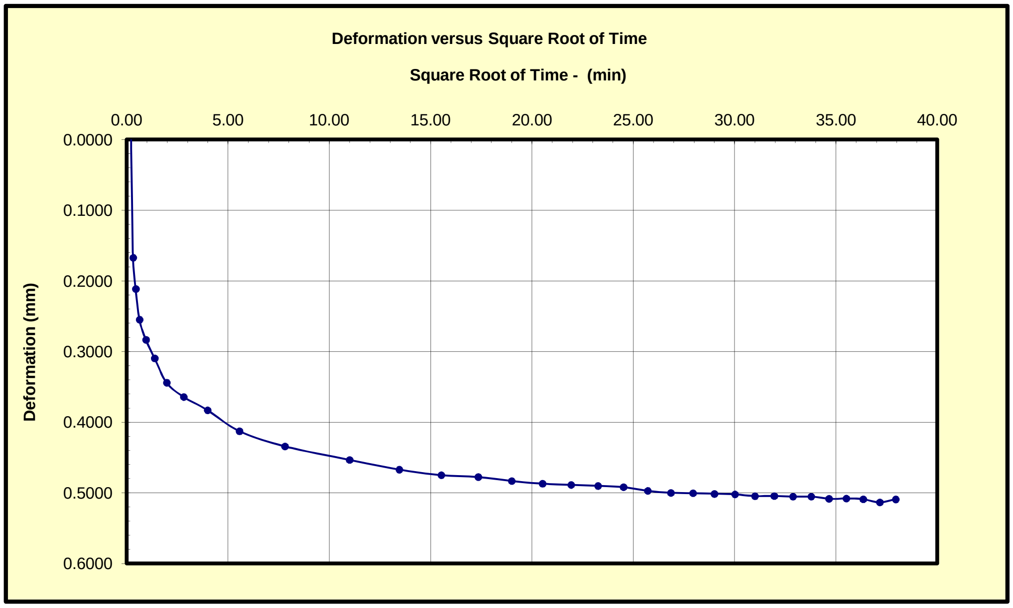
MEG TECHNICAL SERVICES

(A Division of MEG Consulting Limited)

One-Dimensional Consolidation (ASTM D 2435)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BH4	Station:	3
Sample No.:	S1	Depth (m):	3.54
Consolidation Step:	3	Vertical Stress (kPa):	96





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Date:	May 7, 2014	Date:	May 7, 2014	Date:	May 7, 2014

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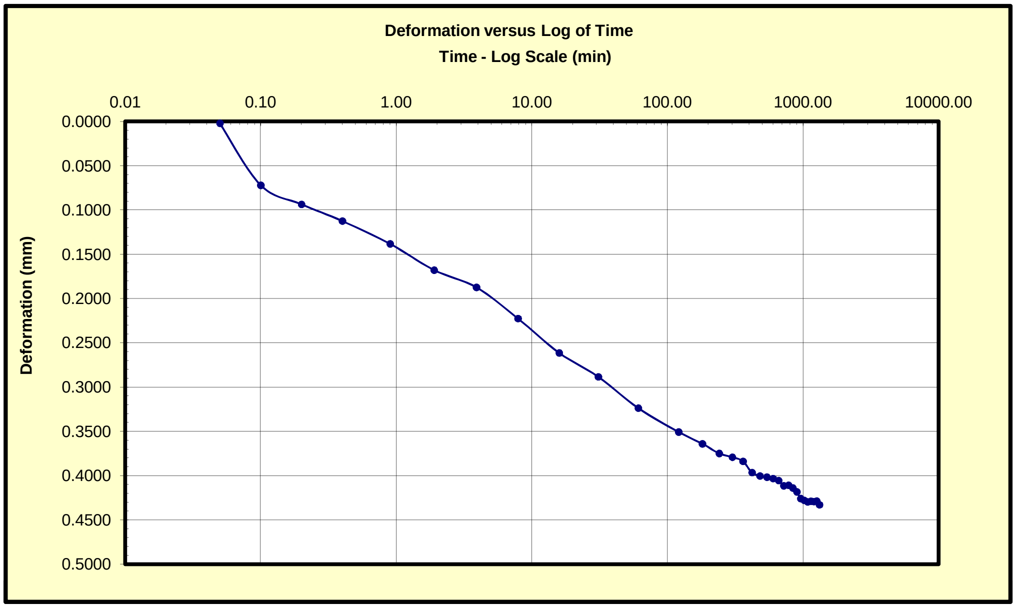
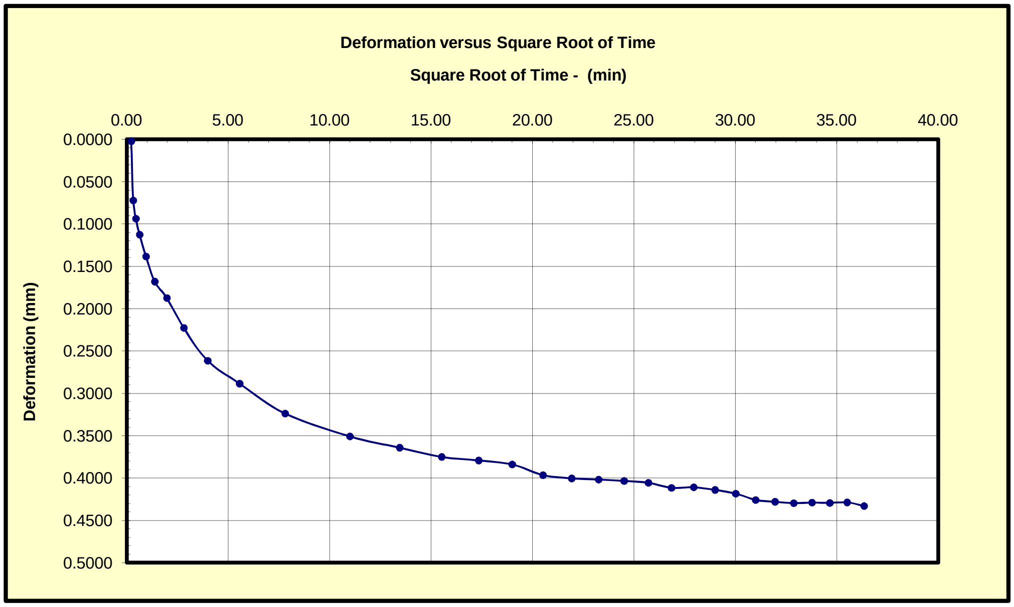
One-Dimensional Consolidation (ASTM D 2435)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BH4	Station:	3
Sample No.:	S1	Depth (m):	3.54
Consolidation Step:	4	Vertical Stress (kPa):	144





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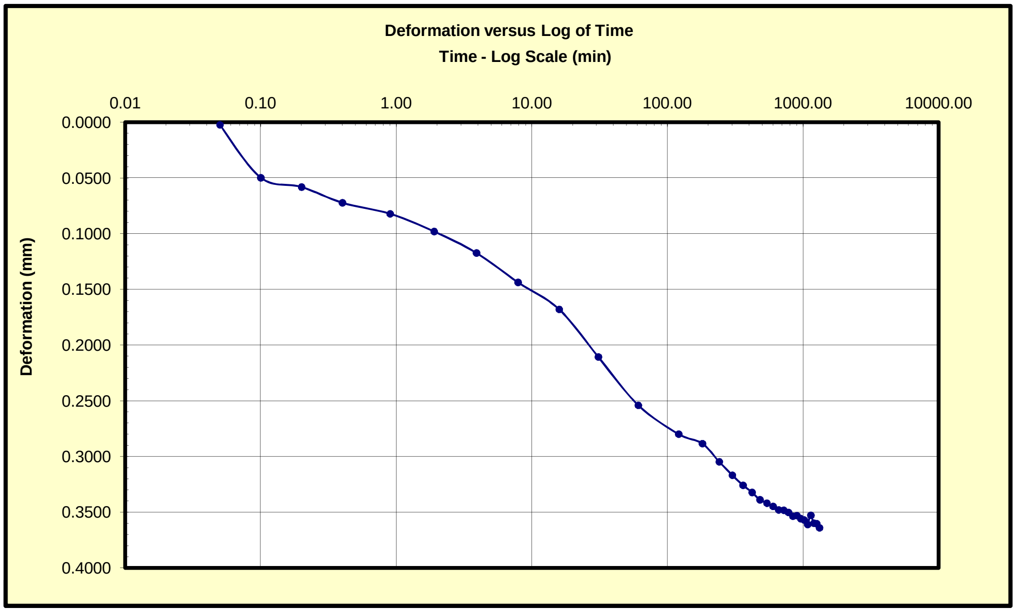
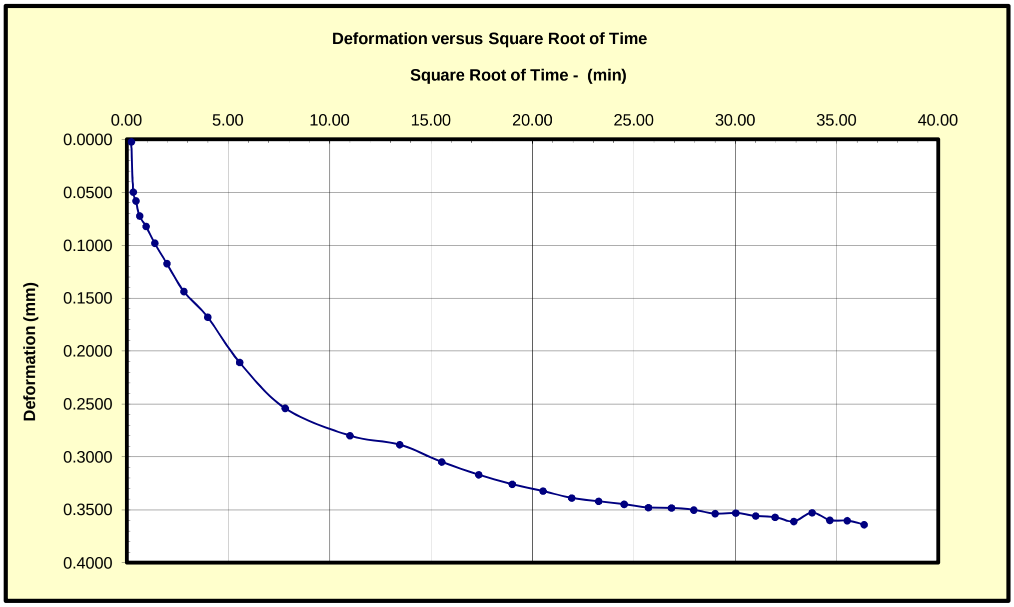
One-Dimensional Consolidation (ASTM D 2435)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BH4	Station:	3
Sample No.:	S1	Depth (m):	3.54
Consolidation Step:	5	Vertical Stress (kPa):	192





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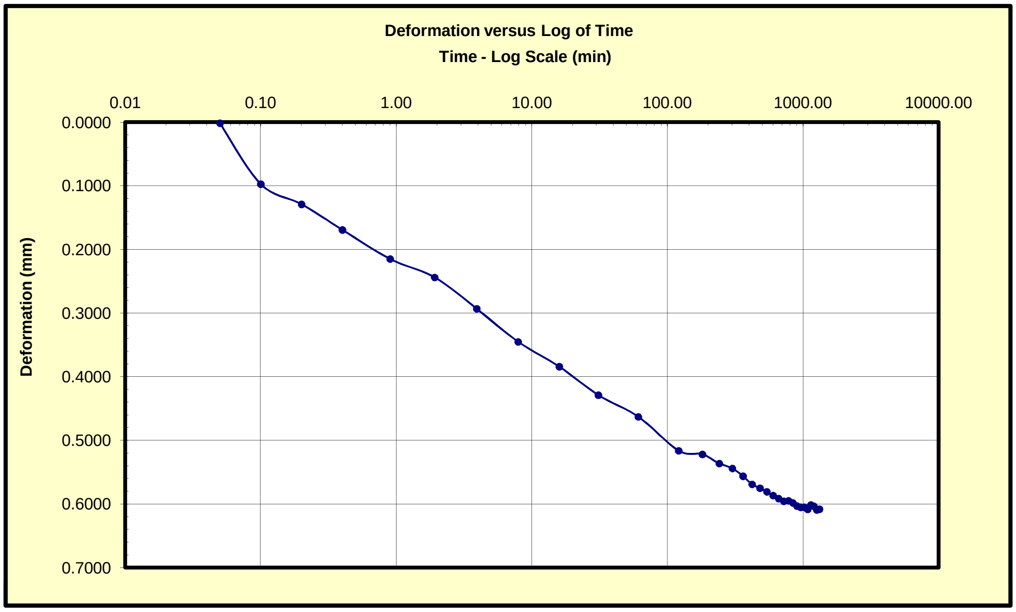
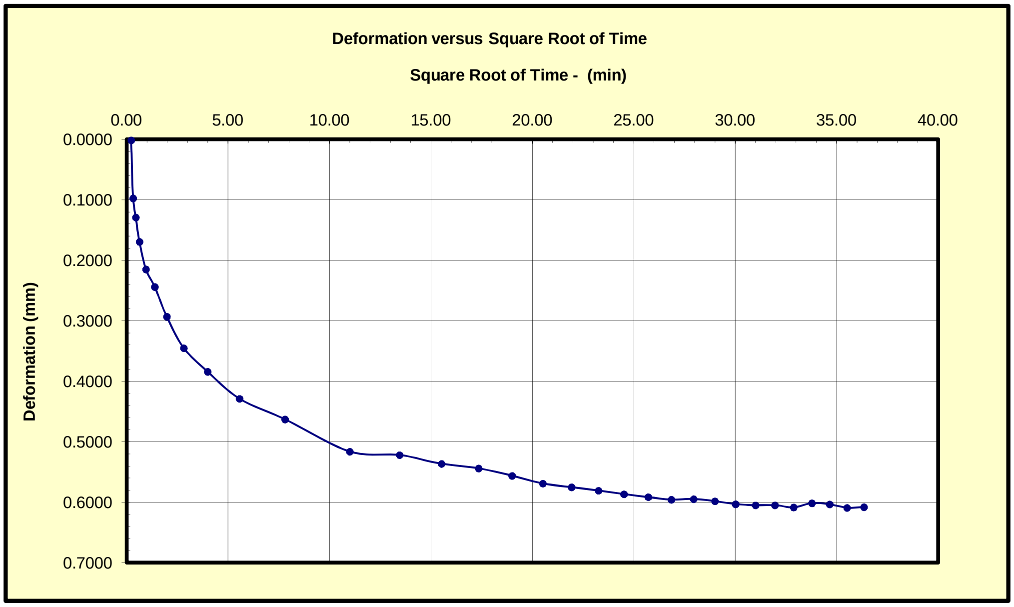
Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BH4	Station:	3
Sample No.:	S1	Depth (m):	3.54
Consolidation Step:	6	Vertical Stress (kPa):	287





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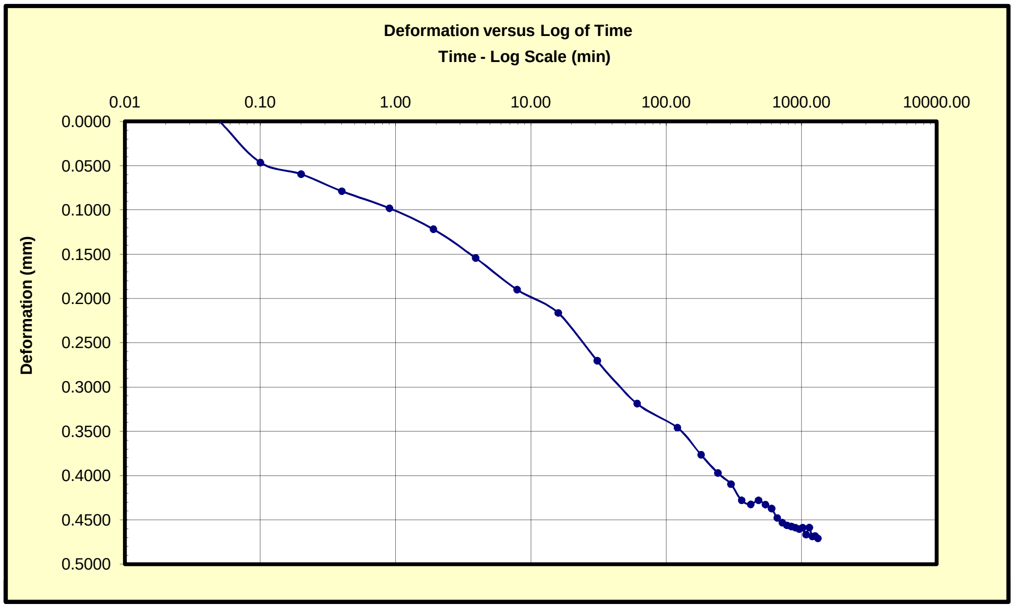
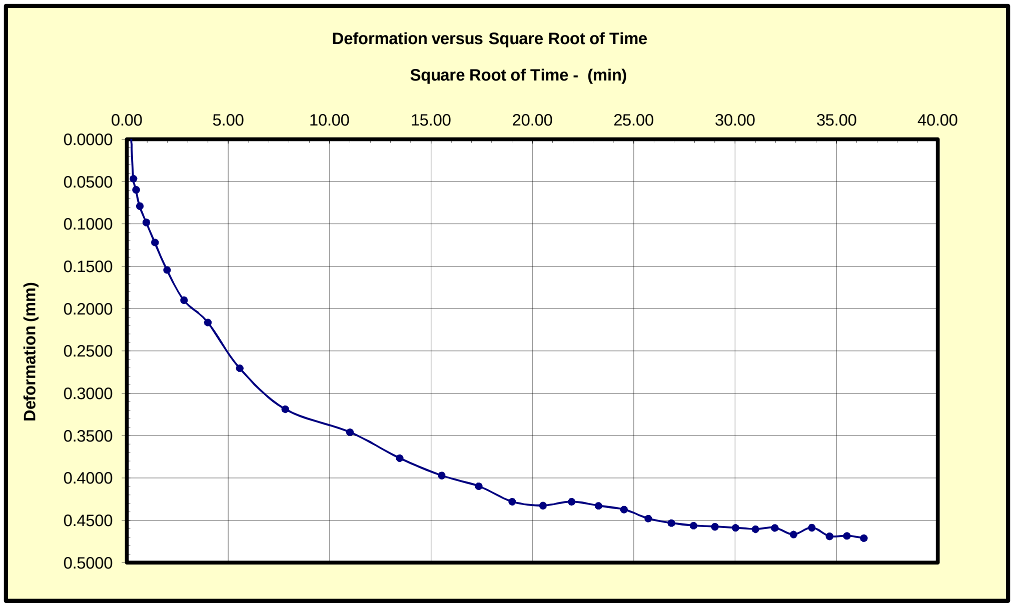
Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BH4	Station:	3
Sample No.:	S1	Depth (m):	3.54
Consolidation Step:	7	Vertical Stress (kPa):	383





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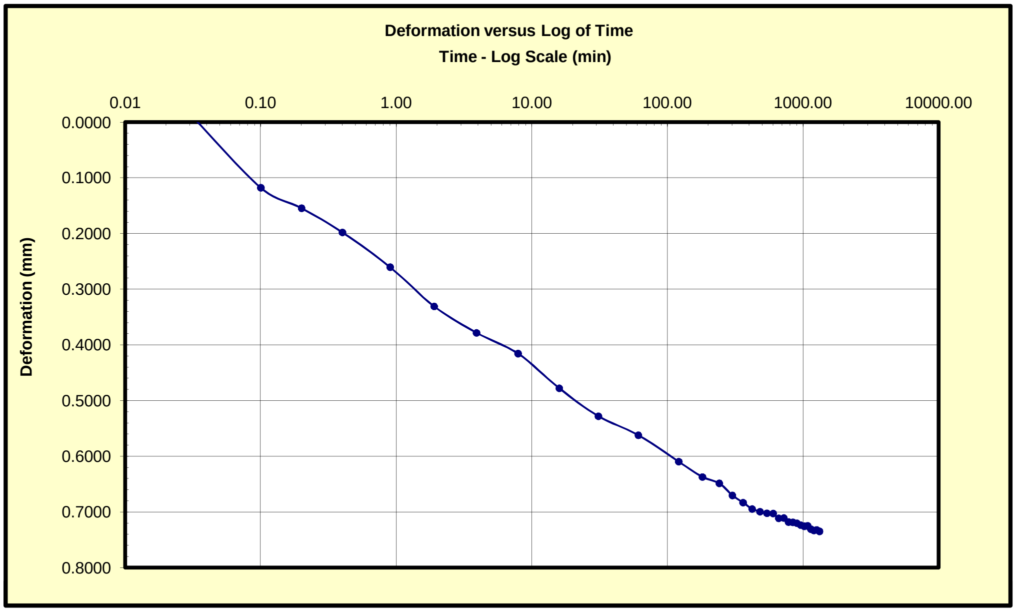
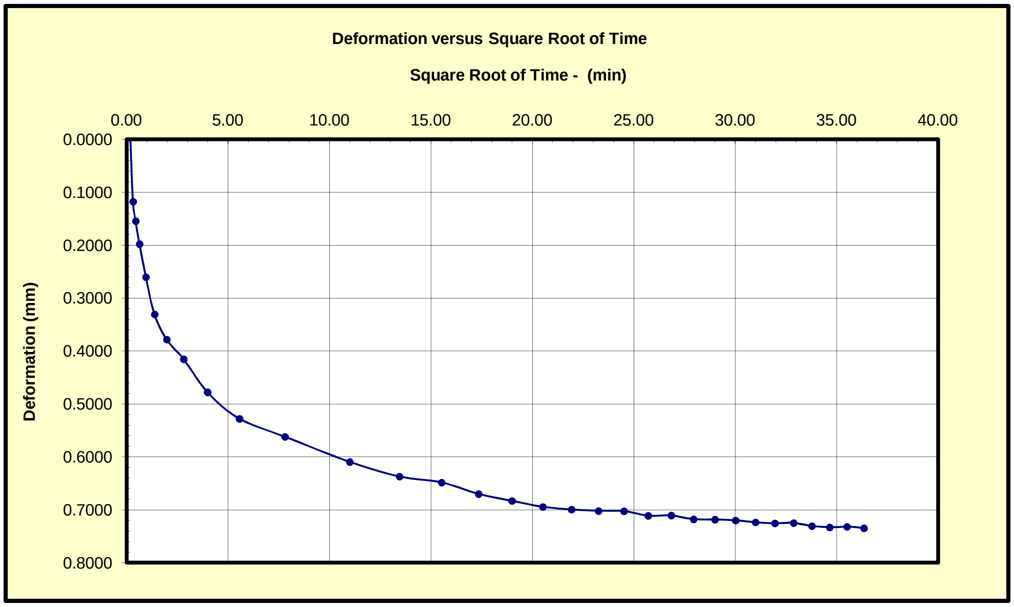
One-Dimensional Consolidation (ASTM D 2435)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BH4	Station:	3
Sample No.:	S1	Depth (m):	3.54
Consolidation Step:	8	Vertical Stress (kPa):	575





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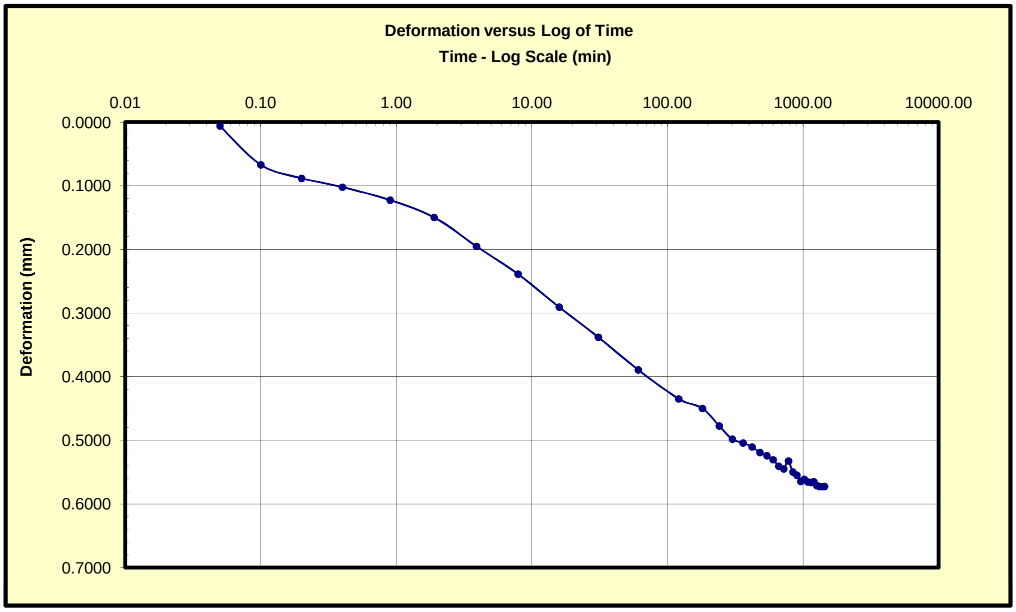
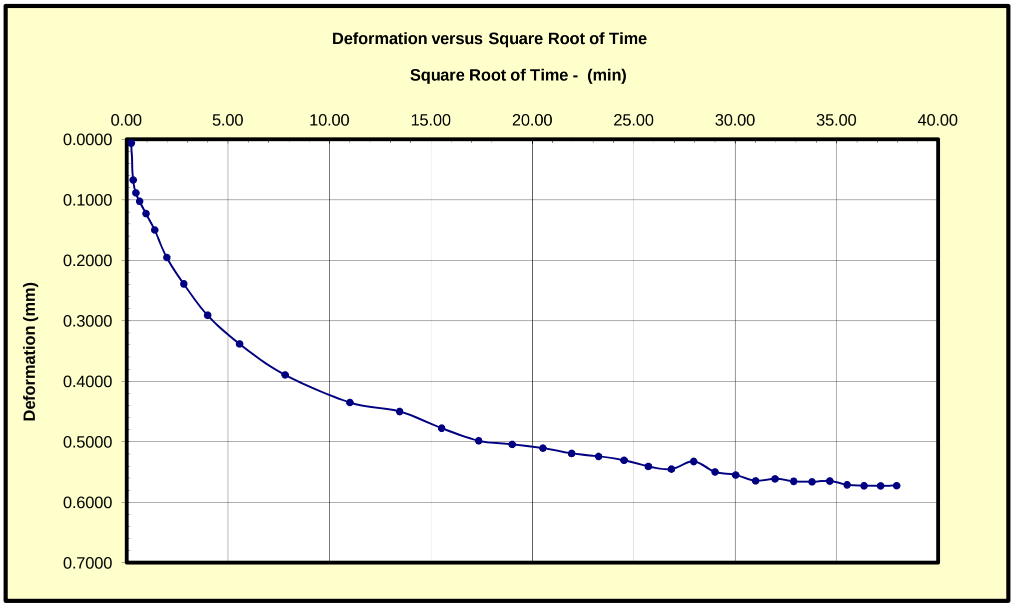
Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BH4	Station:	3
Sample No.:	S1	Depth (m):	3.54
Consolidation Step:	9	Vertical Stress (kPa):	766





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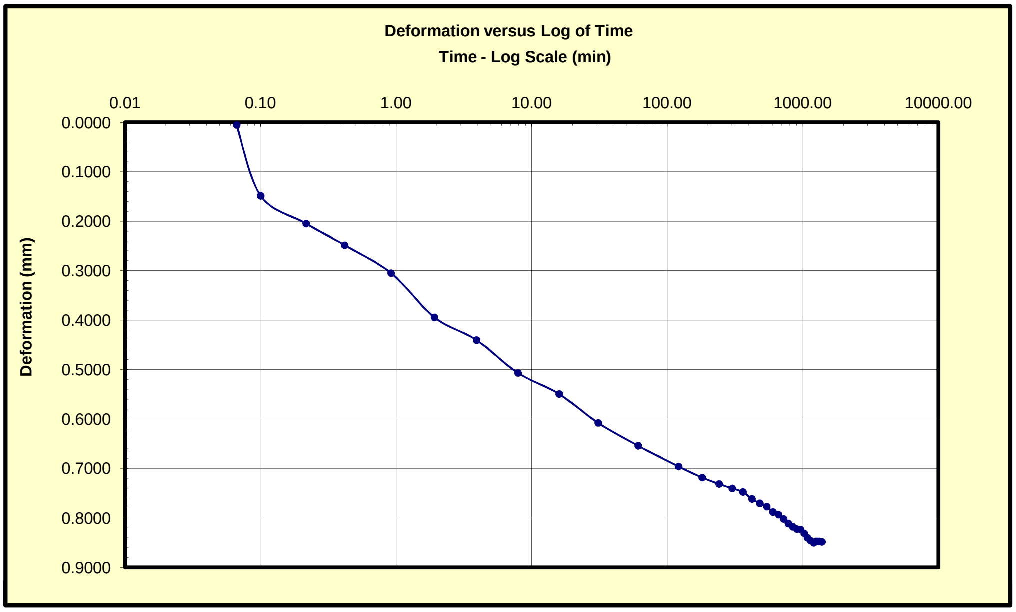
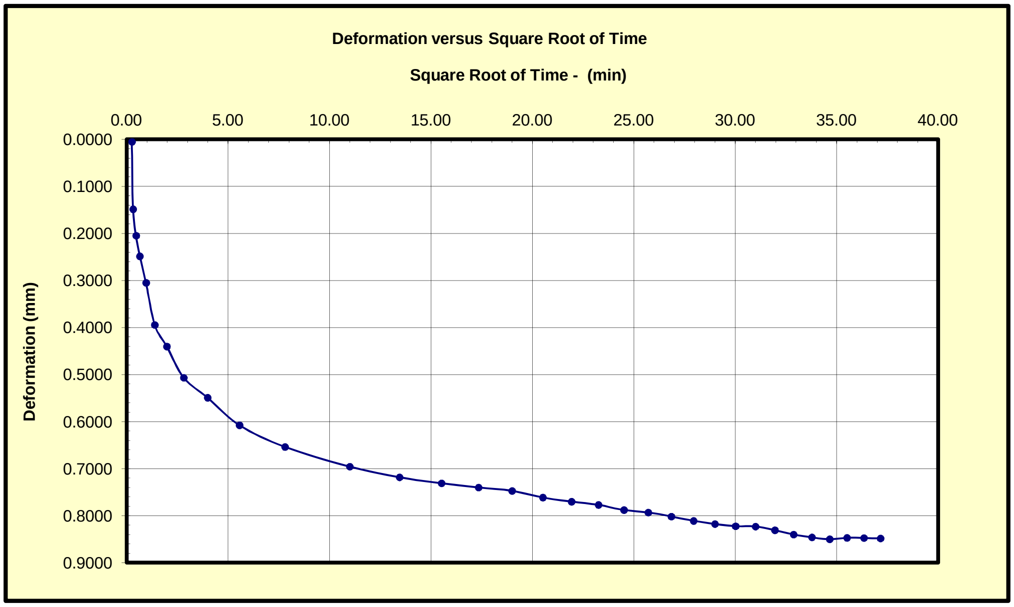
Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BH4	Station:	3
Sample No.:	S1	Depth (m):	3.54
Consolidation Step:	10	Vertical Stress (kPa):	1149





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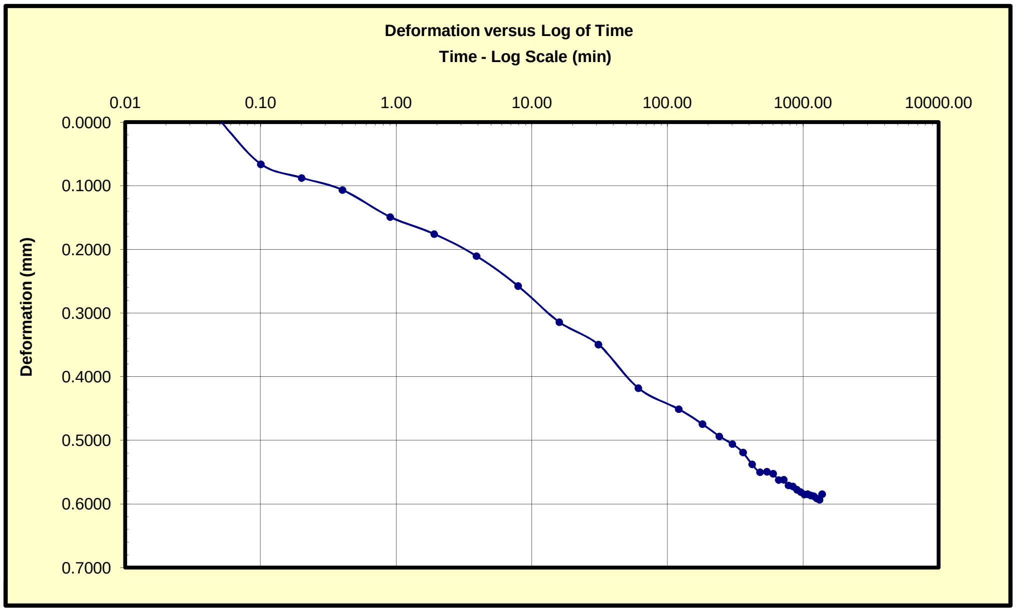
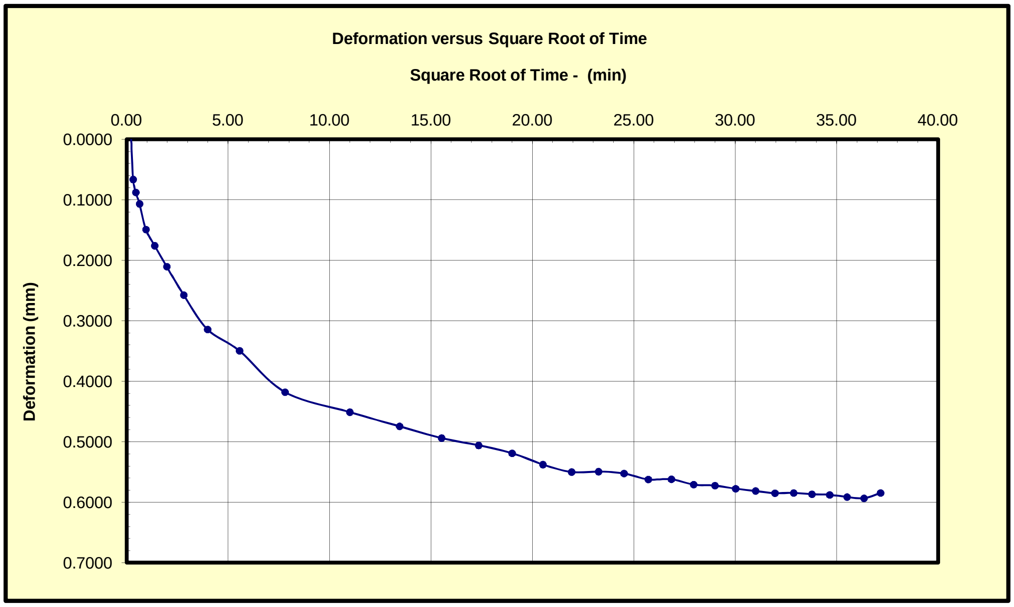
One-Dimensional Consolidation (ASTM D 2435)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BH4	Station:	3
Sample No.:	S1	Depth (m):	3.54
Consolidation Step:	11	Vertical Stress (kPa):	1532





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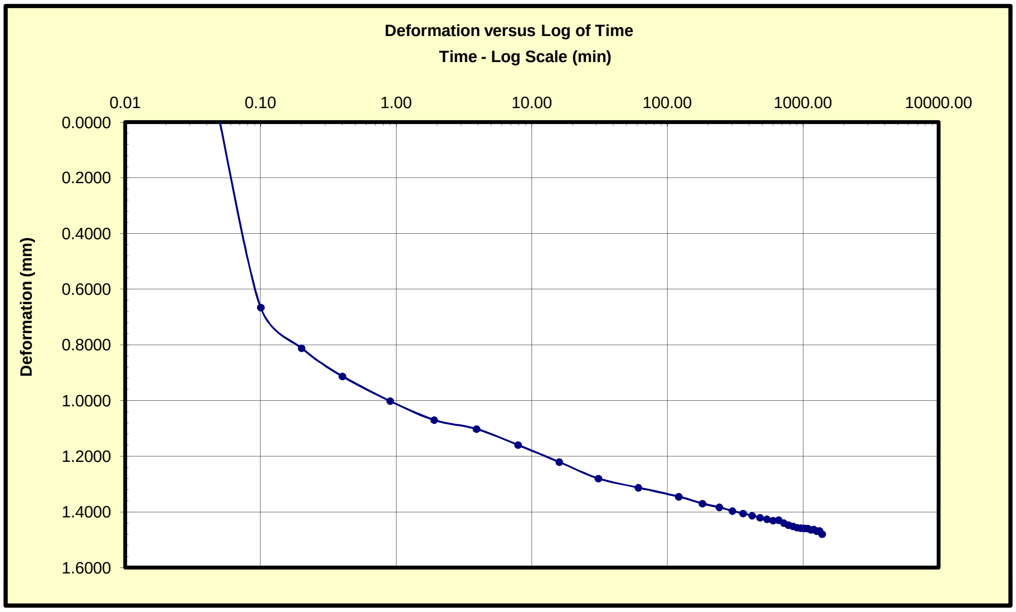
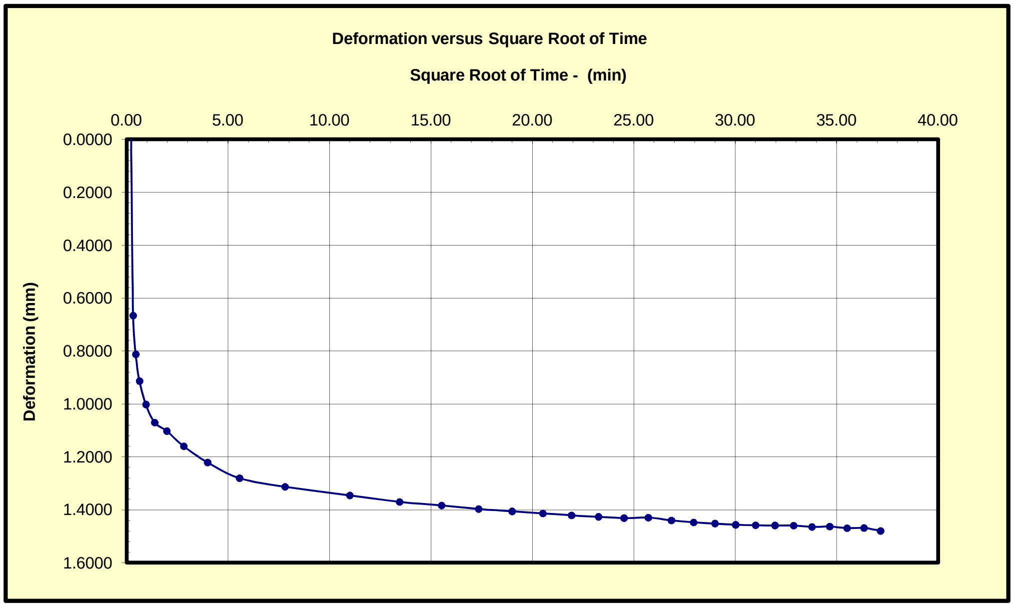
Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BH4	Station:	3
Sample No.:	S1	Depth (m):	3.54
Consolidation Step:	12	Vertical Stress (kPa):	3064





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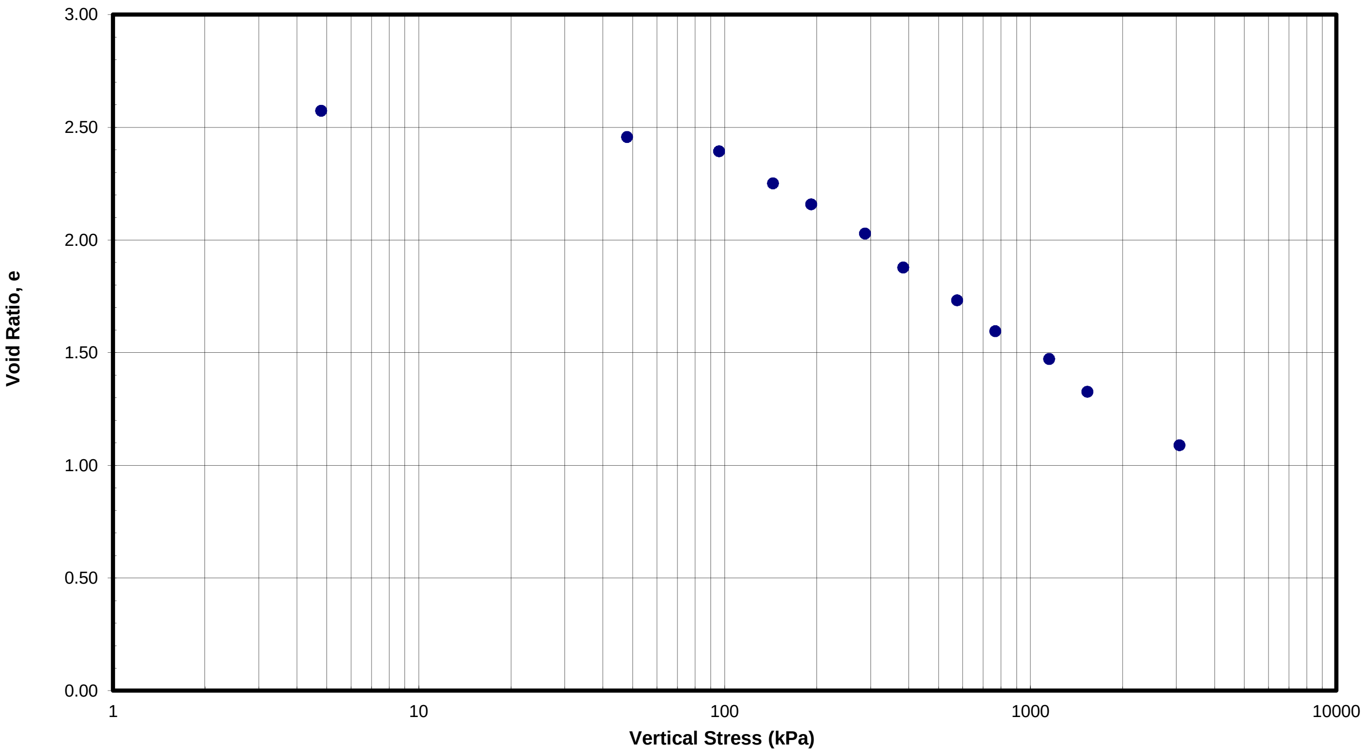
One-Dimensional Consolidation (ASTM D 2435)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 26, 2014
Borehole:	BH4	Station:	3
Sample No.:	S6	Depth (m):	8.15

Weight of Ring (g):	216.34	**Ring + Wet Weight (g):**	332.20	**Initial Void Ratio, e:**	2.59
Initial Height (mm):	25.40	**Ring + Dry Weight (g):**	276.63	**Height of Soil, Hs (mm):**	7.07
Diameter of Ring (mm):	63.50	**Water Content (%):**	92.2	**Height of Void, Hv (mm):**	18.33
Unit Weight (kN/m^3)	14.13	**Specific Gravity, Gs:**	2.69		

Step No.	Vertical Stress (kPa)	Height of Sample (mm)	Vertical Strain (%)	Final Void Ratio e_f	Change in Void Ratio e	Coefficient of Compressibility a_v (m^2/MN)	Coefficient of Volume Compressibility m_v (m^2/MN)
1	5	25.2781	0.4800	2.5743	0.00		
2	48	24.4551	3.7200	2.4579	0.12	2.7004	0.75
3	96	24.0066	5.4857	2.3945	0.06	1.3245	0.37
4	144	23.0006	9.4465	2.2522	0.14	2.9710	0.83
5	192	22.3418	12.0400	2.1591	0.09	1.9454	0.54
6	287	21.4250	15.6496	2.0294	0.13	1.3538	0.38
7	383	20.3581	19.8500	1.8786	0.15	1.5754	0.44
8	575	19.3290	23.9015	1.7331	0.15	0.7598	0.21
9	766	18.3594	27.7190	1.5960	0.14	0.7159	0.20
10	1149	17.4879	31.1501	1.4727	0.12	0.3217	0.09
11	1532	16.4592	35.2000	1.3273	0.15	0.3797	0.11
12	3064	14.7822	41.8025	1.0902	0.24	0.1548	0.04



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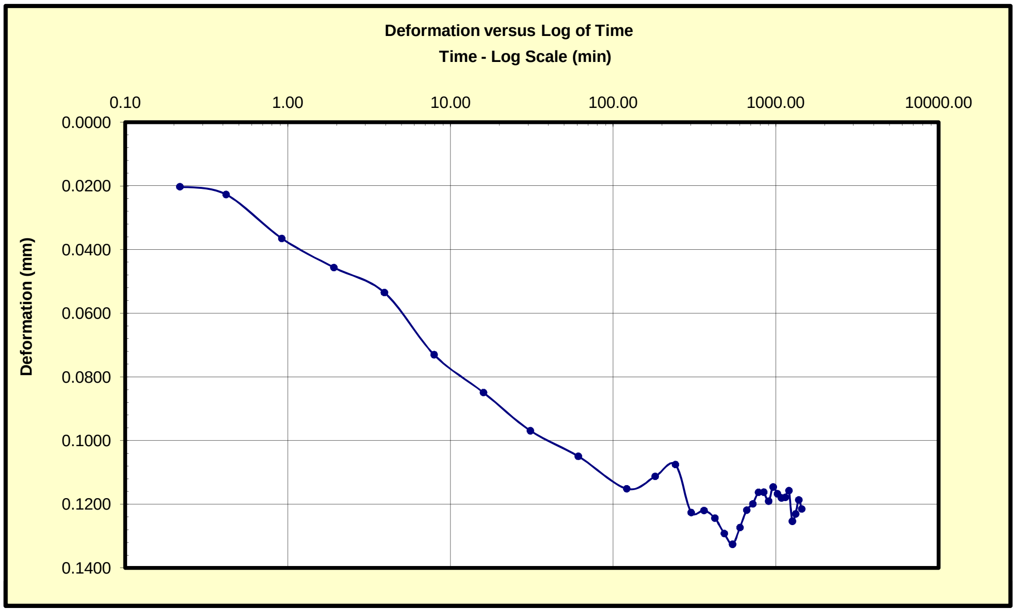
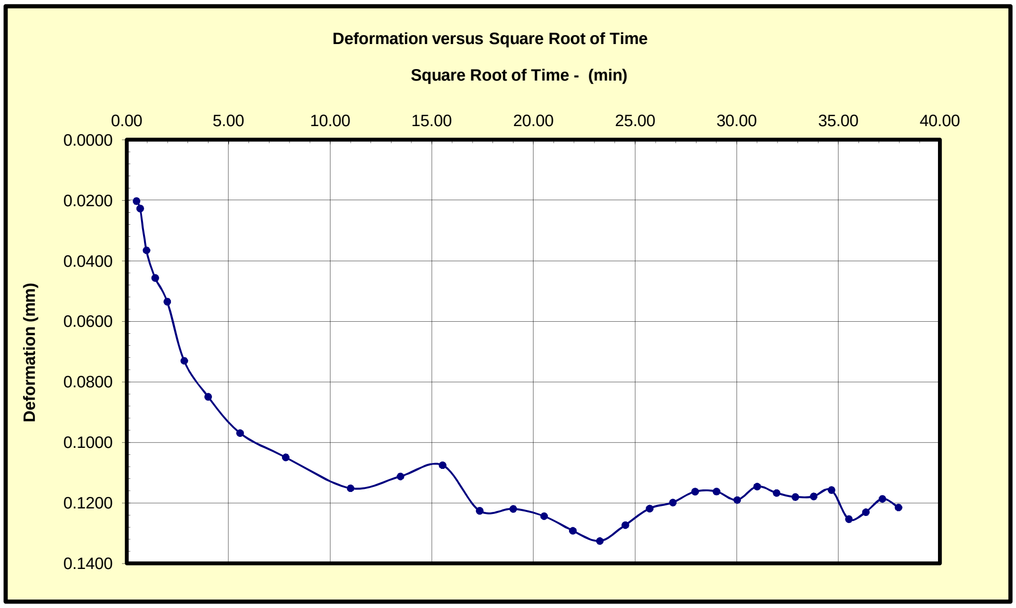
One-Dimensional Consolidation (ASTM D 2435)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 26, 2014
Borehole:	BH4	Station:	3
Sample No.:	S6	Depth (m):	8.15
Consolidation Step:	1	Vertical Stress (kPa):	5





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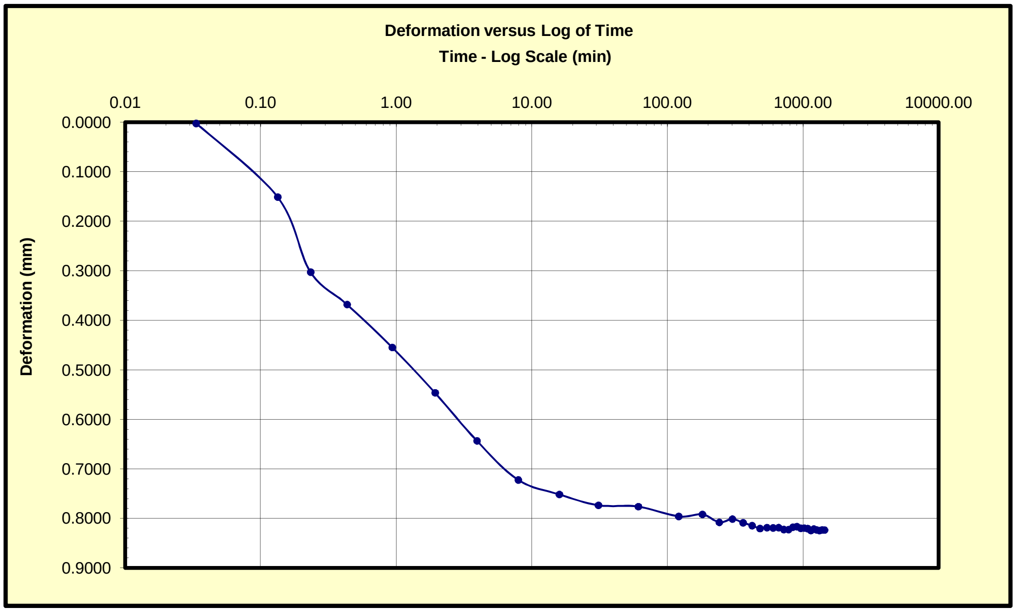
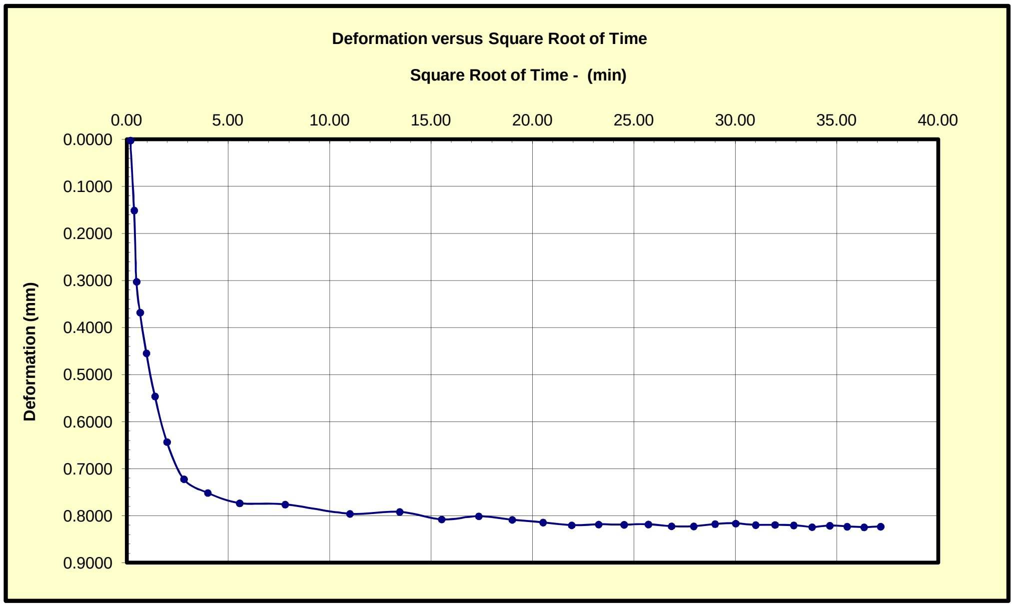
One-Dimensional Consolidation (ASTM D 2435)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 26, 2014
Borehole:	BH4	Station:	3
Sample No.:	S6	Depth (m):	8.15
Consolidation Step:	2	Vertical Stress (kPa):	48





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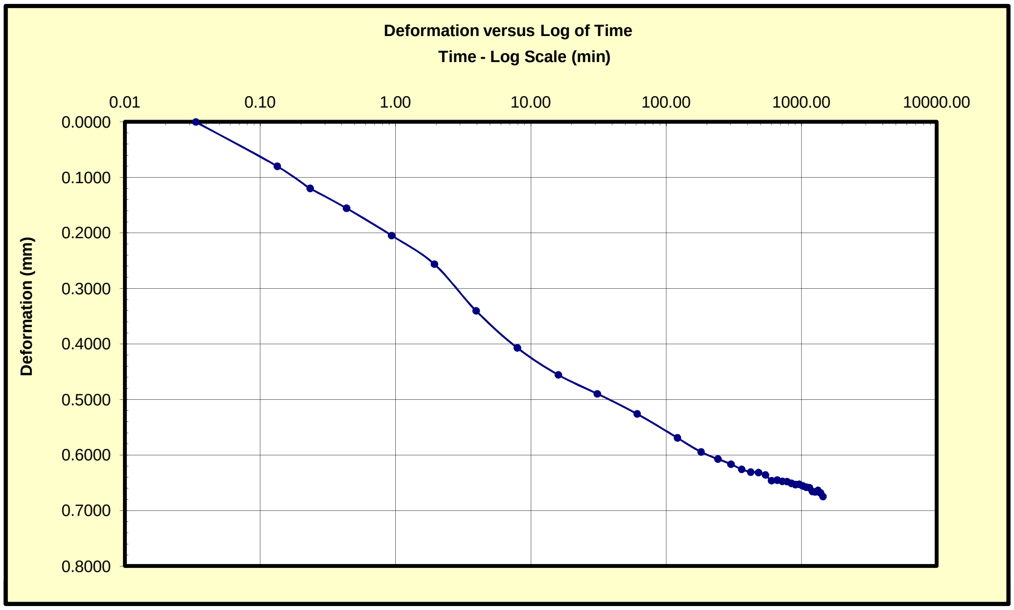
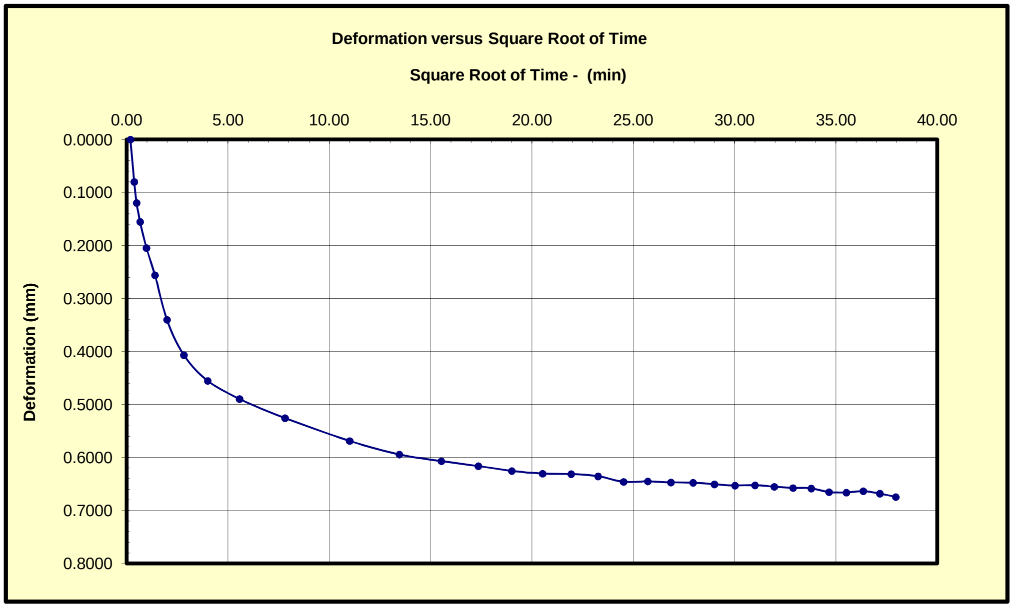
Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 26, 2014
Borehole:	BH4	Station:	3
Sample No.:	S6	Depth (m):	8.15
Consolidation Step:	3	Vertical Stress (kPa):	96





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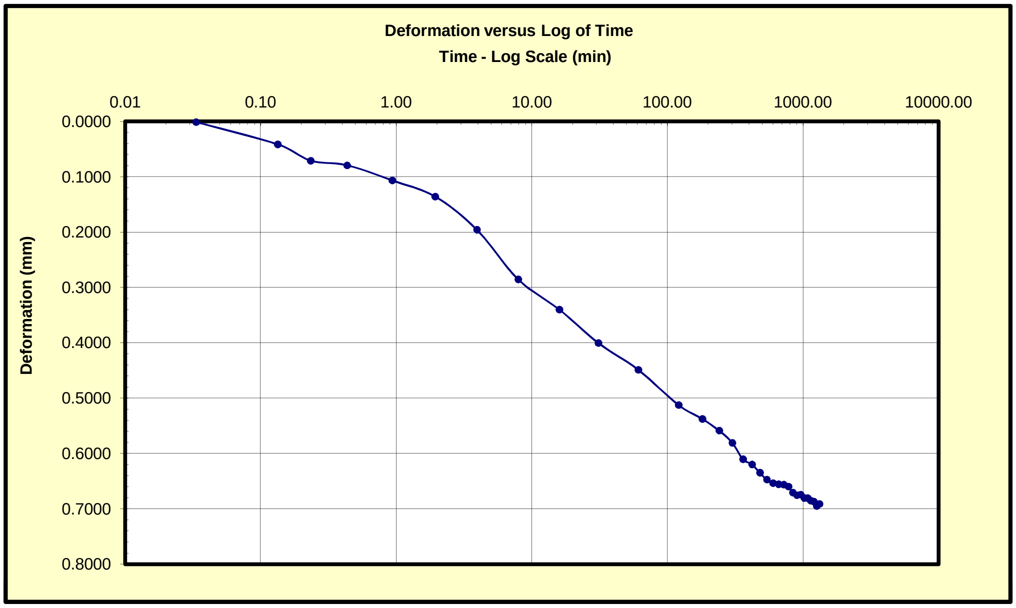
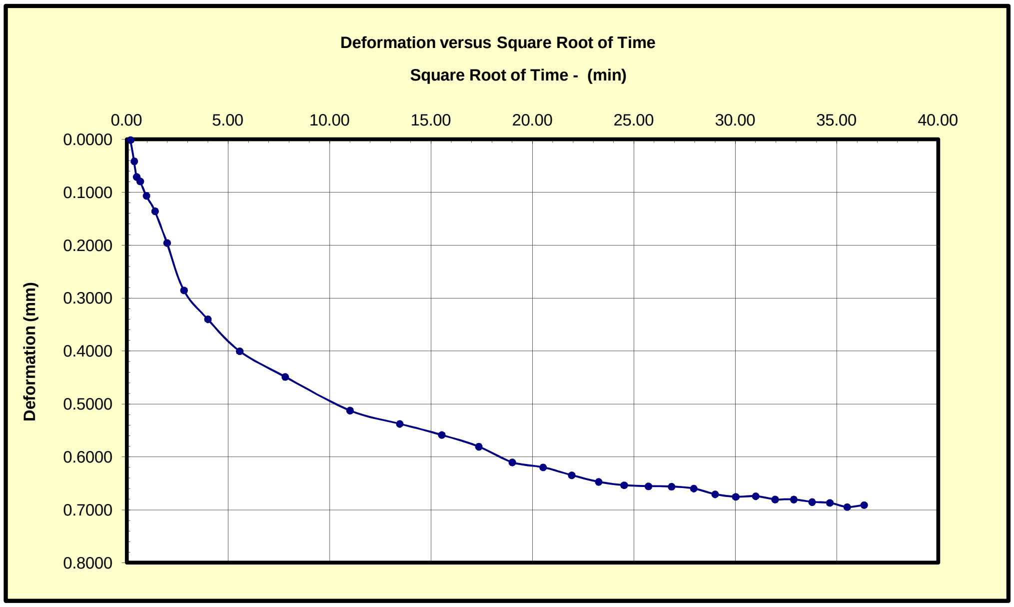
Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 26, 2014
Borehole:	BH4	Station:	3
Sample No.:	S6	Depth (m):	8.15
Consolidation Step:	4	Vertical Stress (kPa):	144





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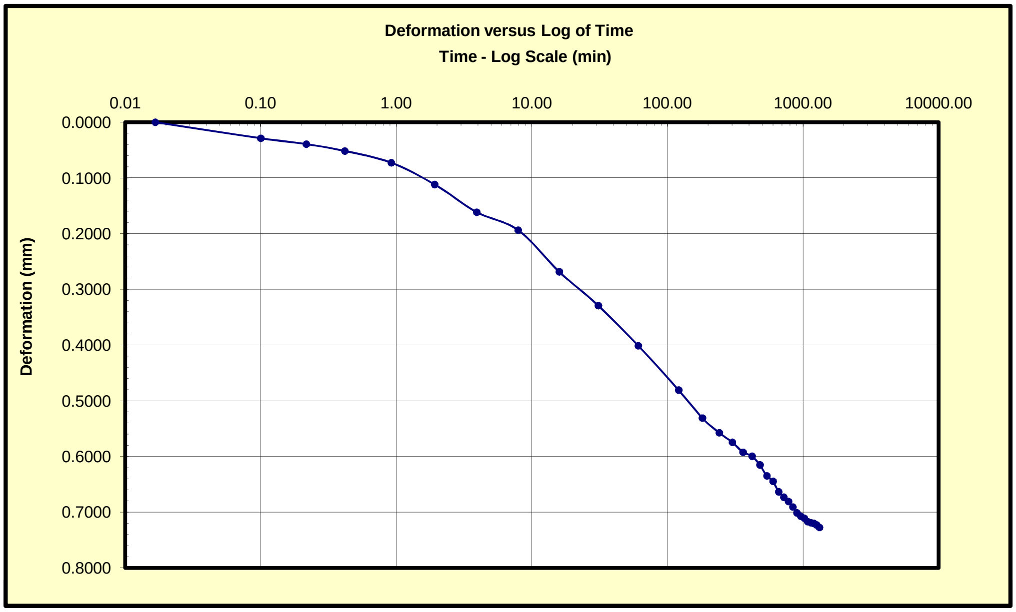
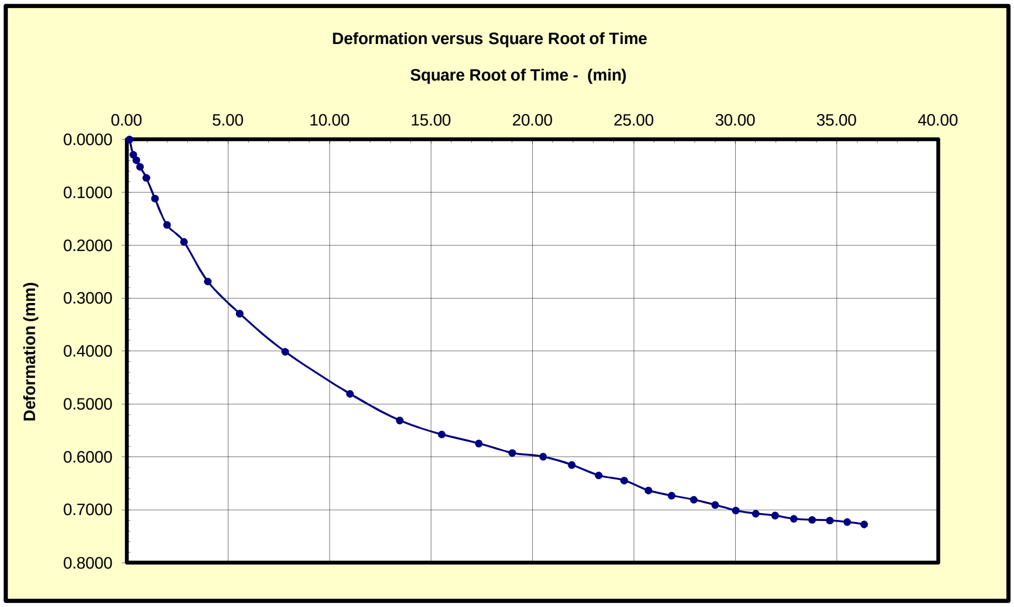
Location:	Oregon, USA	Date:	May 26, 2014
Borehole:	BH4	Station:	3
Sample No.:	S6	Depth (m):	8.15
Consolidation Step:	5	Vertical Stress (kPa):	192





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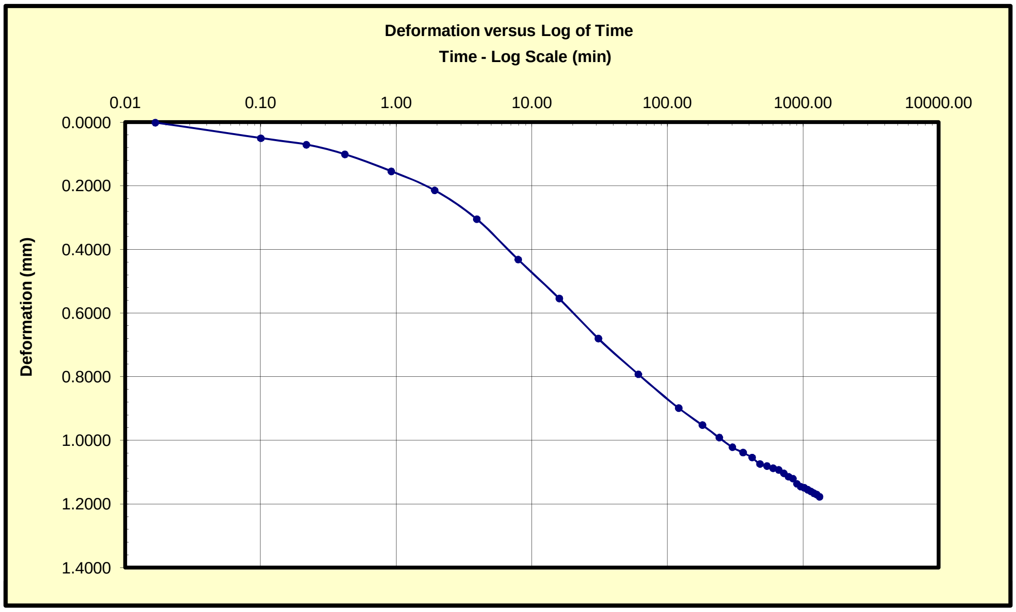
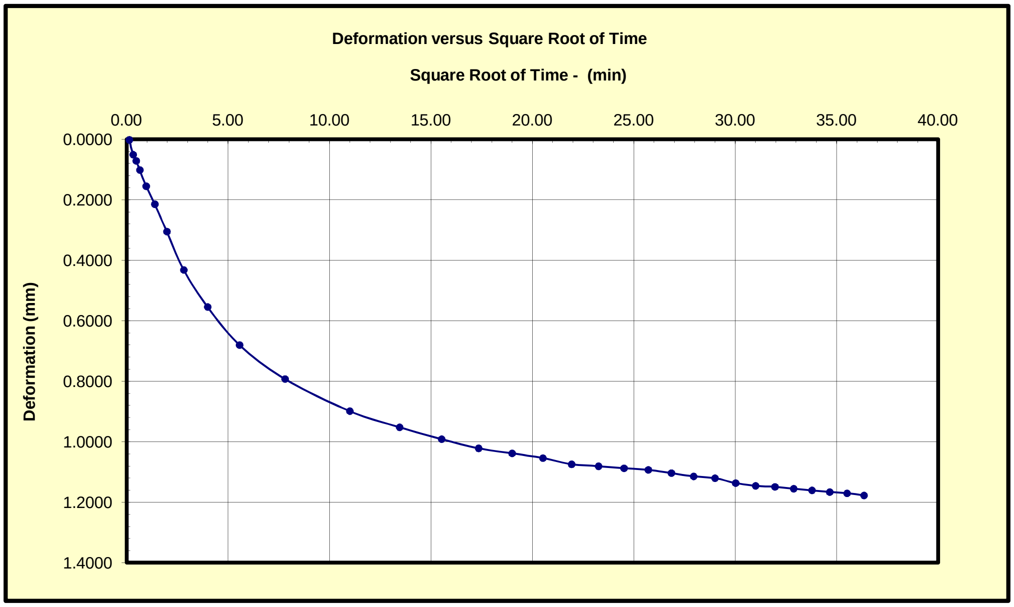
Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 26, 2014
Borehole:	BH4	Station:	3
Sample No.:	S6	Depth (m):	8.15
Consolidation Step:	6	Vertical Stress (kPa):	287





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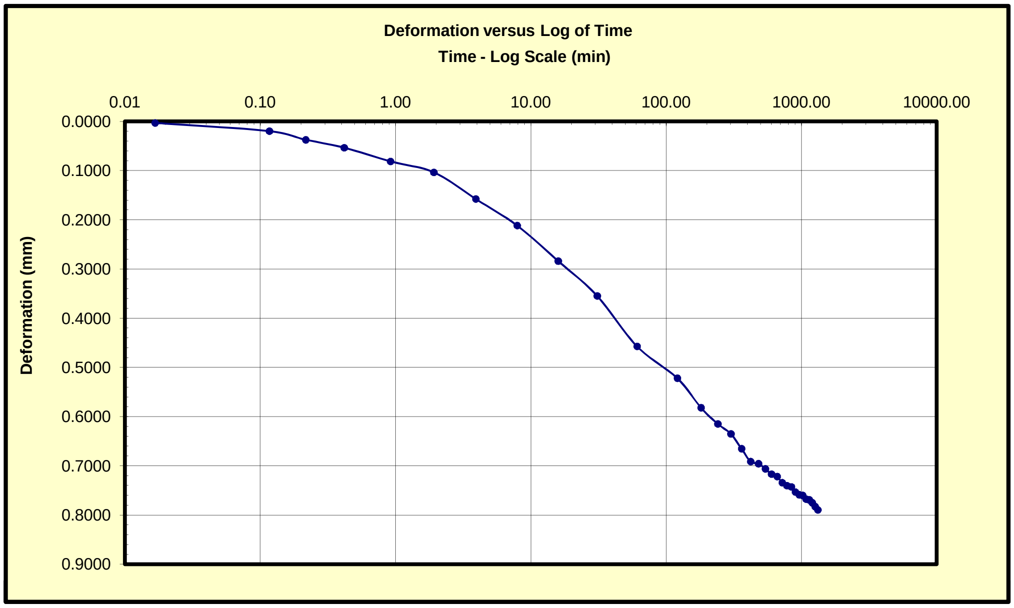
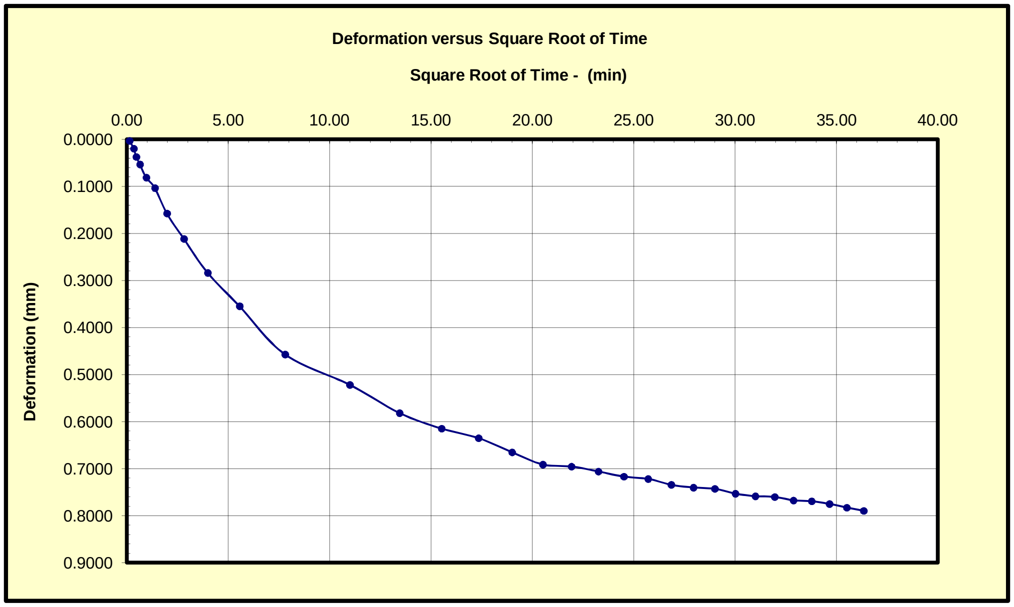
Location:	Oregon, USA	Date:	May 26, 2014
Borehole:	BH4	Station:	3
Sample No.:	S6	Depth (m):	8.15
Consolidation Step:	7	Vertical Stress (kPa):	383





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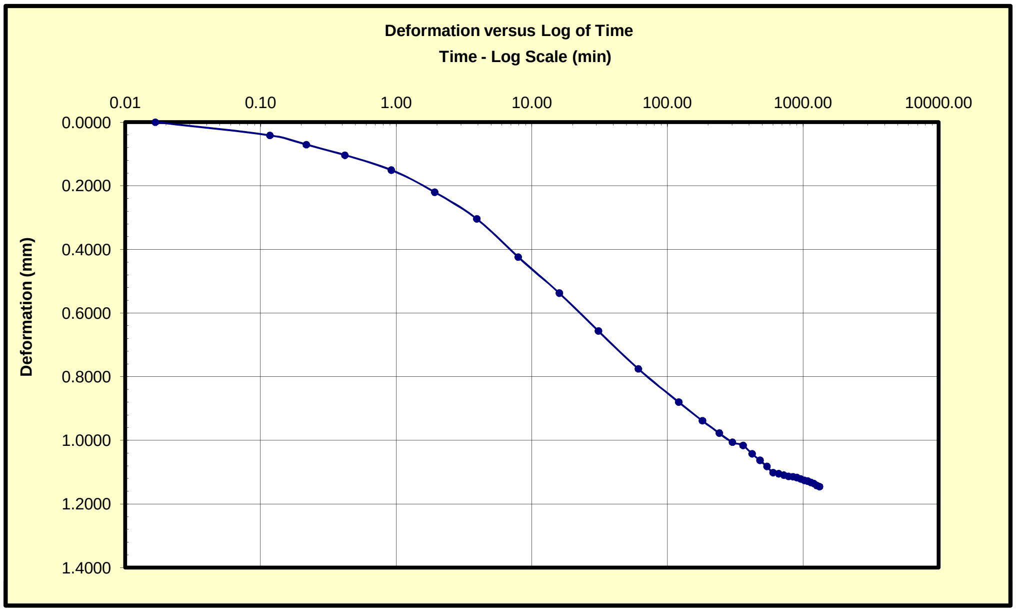
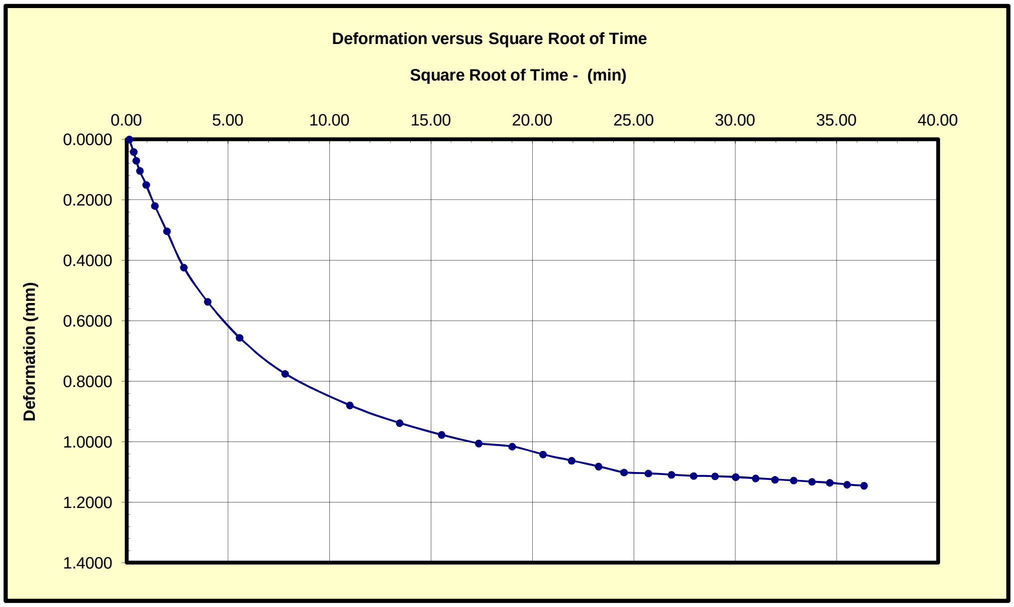
Location:	Oregon, USA	Date:	May 26, 2014
Borehole:	BH4	Station:	3
Sample No.:	S6	Depth (m):	8.15
Consolidation Step:	8	Vertical Stress (kPa):	575





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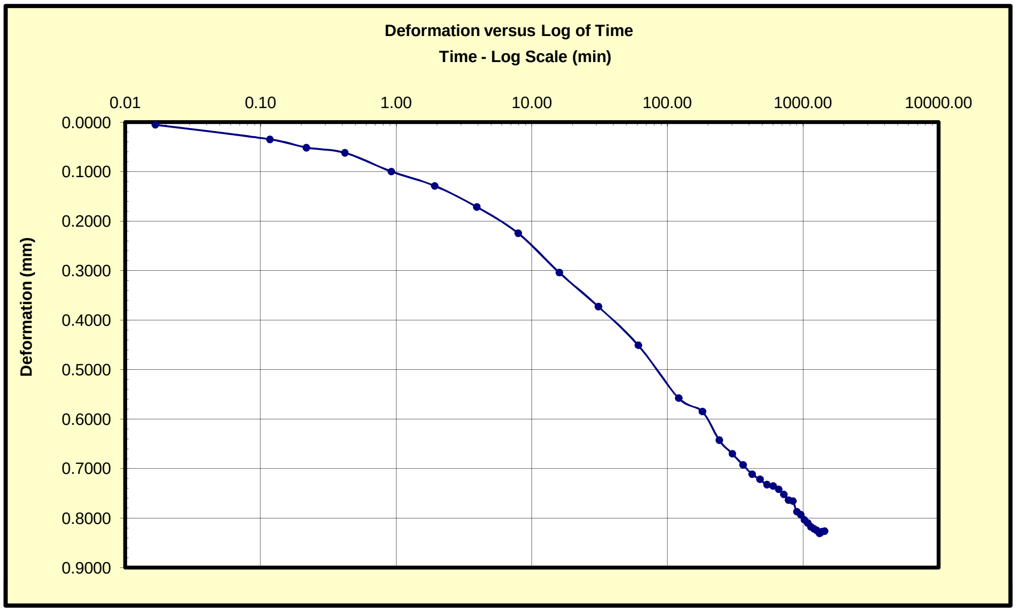
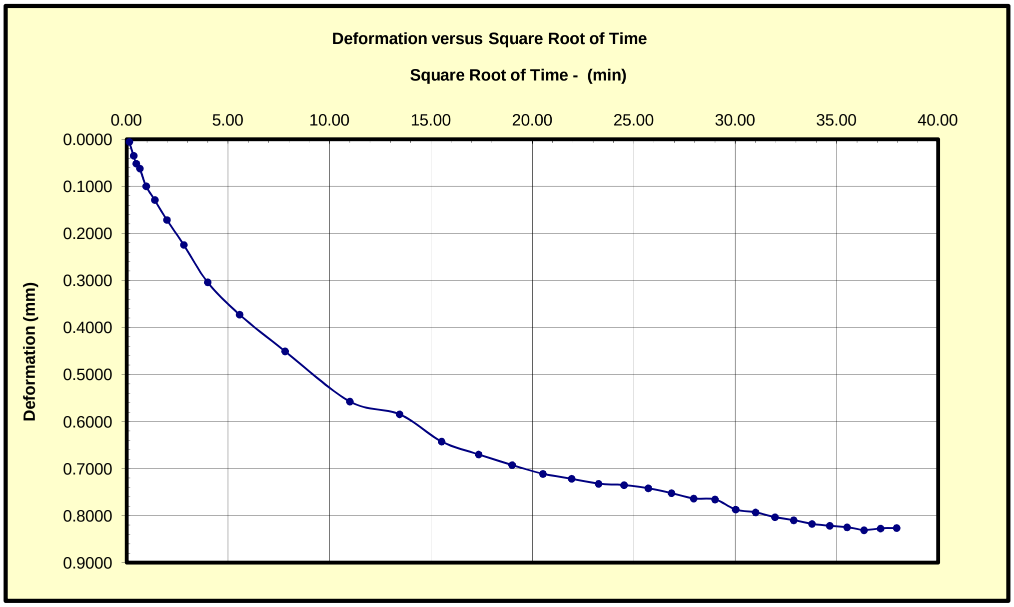
Location:	Oregon, USA	Date:	May 26, 2014
Borehole:	BH4	Station:	3
Sample No.:	S6	Depth (m):	8.15
Consolidation Step:	9	Vertical Stress (kPa):	766





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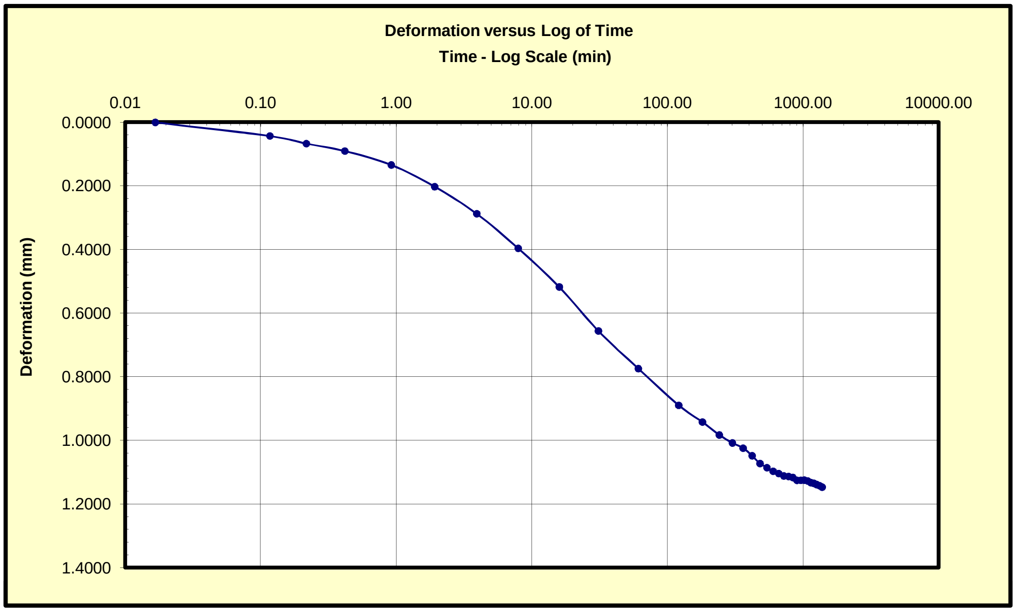
Date:	May 26, 2014	Date:	May 26, 2014	Date:	May 26, 2014

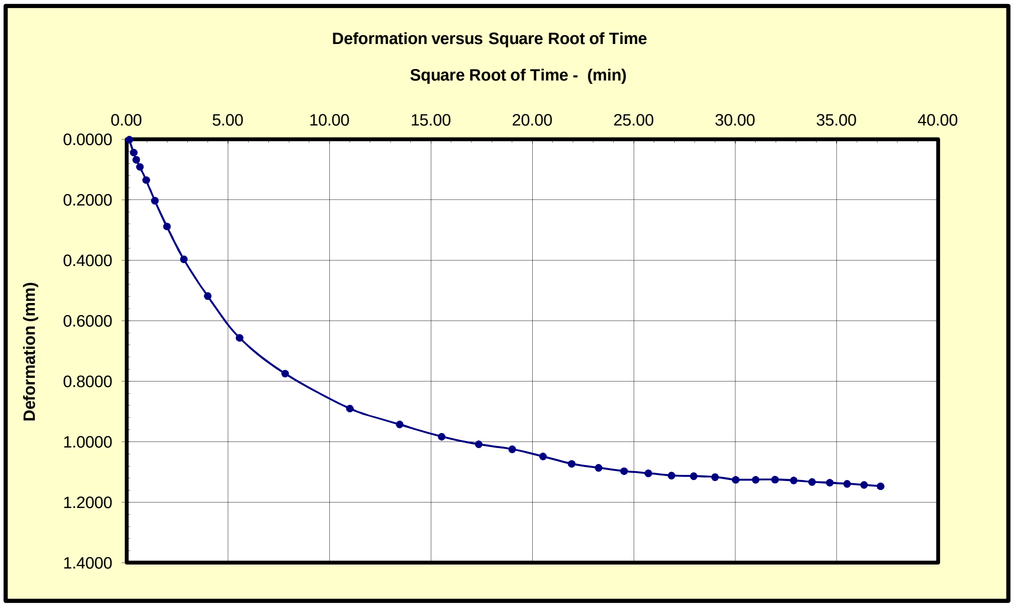
MEG TECHNICAL SERVICES

(A Division of MEG Consulting Limited)

One-Dimensional Consolidation (ASTM D 2435)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 26, 2014
Borehole:	BH4	Station:	3
Sample No.:	S6	Depth (m):	8.15
Consolidation Step:	10	Vertical Stress (kPa):	1149





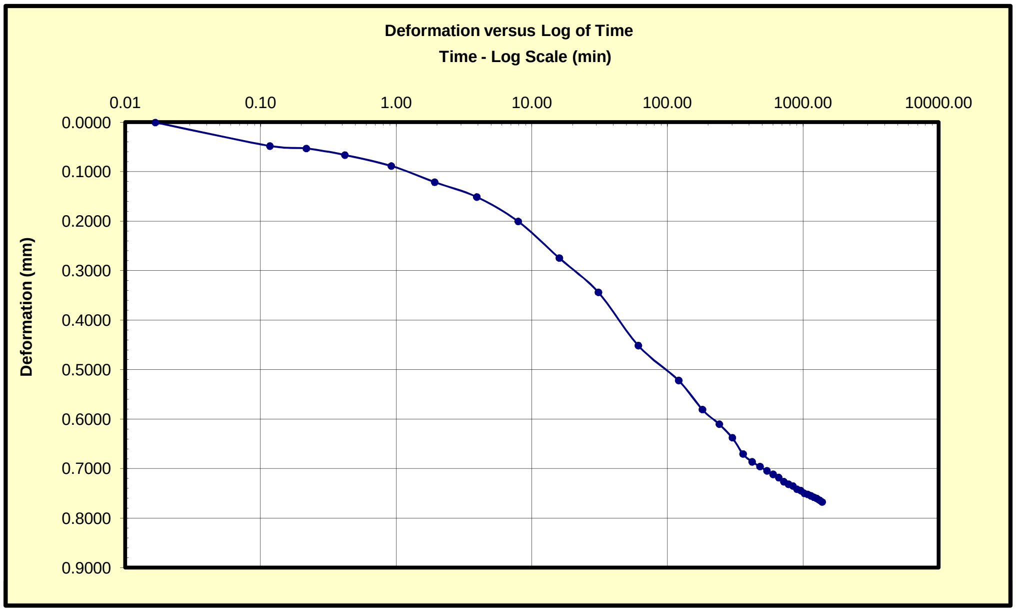
Prepared By:	PC	Checked By:	PS	Approved By:	EP
Date:	May 26, 2014	Date:	May 26, 2014	Date:	May 26, 2014

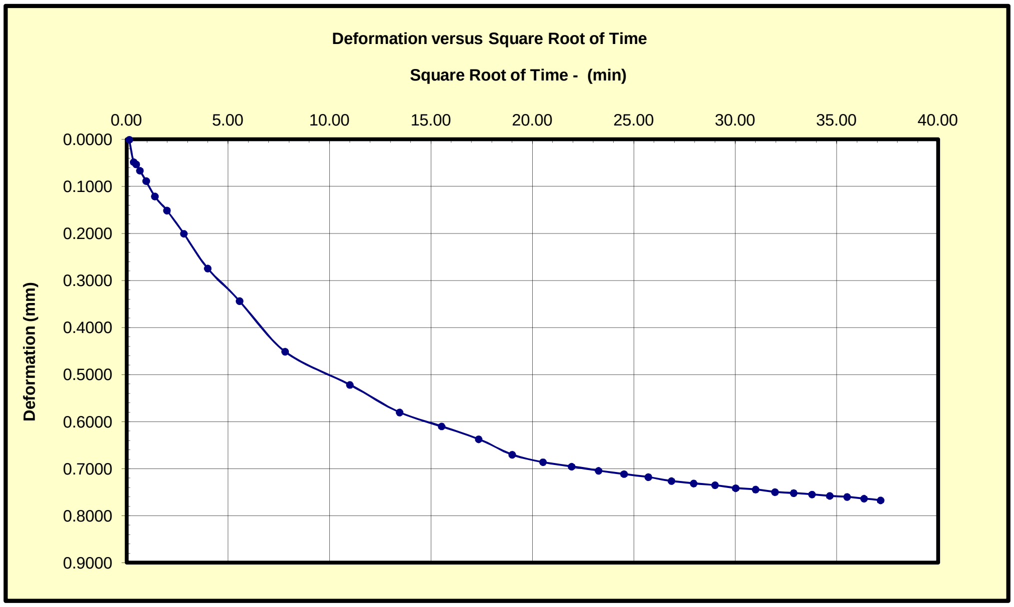
MEG TECHNICAL SERVICES

(A Division of MEG Consulting Limited)

One-Dimensional Consolidation (ASTM D 2435)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 26, 2014
Borehole:	BH4	Station:	3
Sample No.:	S6	Depth (m):	8.15
Consolidation Step:	11	Vertical Stress (kPa):	1532





Prepared By:	PC	Checked By:	PS	Approved By:	EP
Date:	May 26, 2014	Date:	May 26, 2014	Date:	May 26, 2014

MEG TECHNICAL SERVICES

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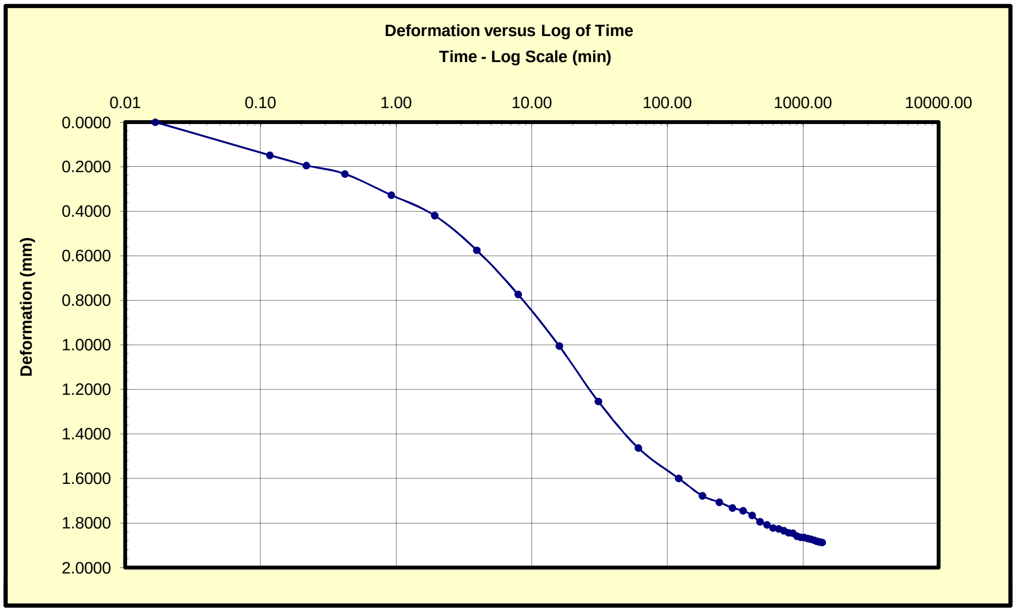
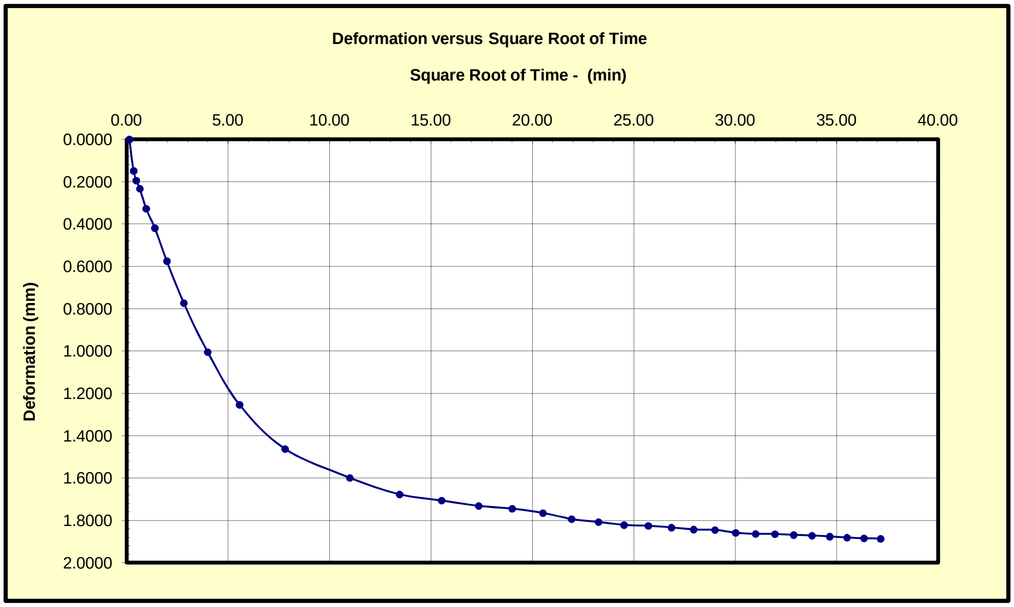
One-Dimensional Consolidation (ASTM D 2435)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 26, 2014
Borehole:	BH4	Station:	3
Sample No.:	S6	Depth (m):	8.15
Consolidation Step:	12	Vertical Stress (kPa):	3064





Prepared By:	PC	Checked By:	PS	Approved By:	EP
Date:	May 26, 2014	Date:	May 26, 2014	Date:	May 26, 2014

MEG TECHNICAL SERVICES

(A Division of MEG Consulting Limited)

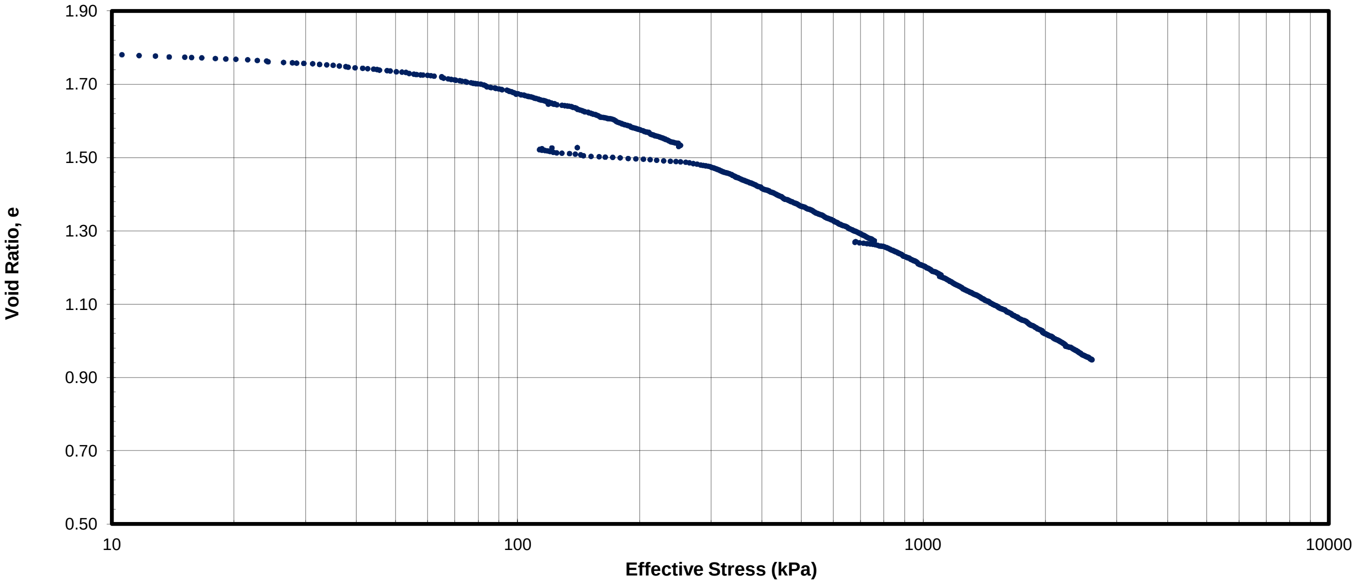
Constant Rate of Strain Consolidation (ASTM D4186)

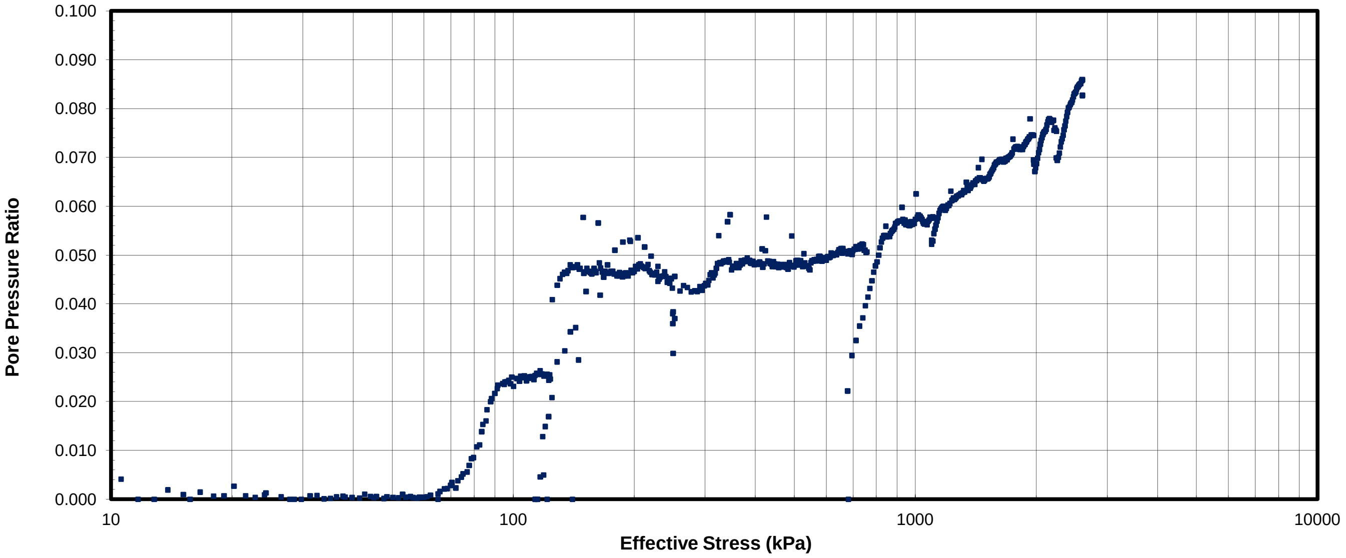
Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 15, 2014
Borehole:	BH4	Station:	2
Sample No.:	S9	Depth (m):	12.37

Weight of Ring (g):	211.96	**Ring + Wet Weight (g):**	339.65	**Initial Void Ratio, e:**	1.82
Initial Height (mm):	25.39	**Ring + Dry Weight (g):**	291.34	**Height of Soil, Hs (mm):**	9.01
Diameter of Ring (mm):	63.50	**Water Content (%):**	60.9	**Height of Void, Hv (mm):**	16.38
Unit Weight (kN/m^3)	15.57	**Specific Gravity, Gs:**	2.78		

Loading Strain Rate (%/hr):	2	**Max Stress (kPa):**	2601	**Backpressure (kPa):**	407
Unloading Strain Rate (%/hr):	N/A	**Max Strain (%):**	30.9		

Comments: test started at 2% per hour and rate was increased to 4% per hour at 6.17% strain (124 kPa)





Prepared By:	PC	Checked By:	PS	Approved By:	
Date:	May 16, 2014	Date:	May 16, 2014	Date:	

MEG TECHNICAL SERVICES

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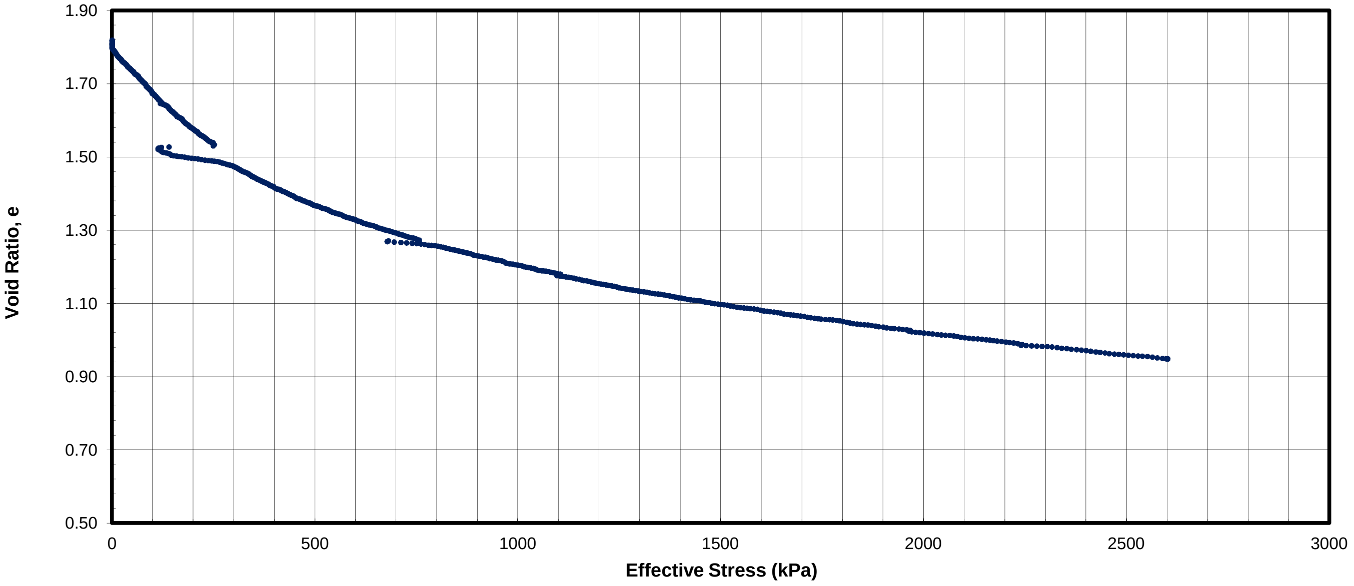
Constant Rate of Strain Consolidation (ASTM D4186)

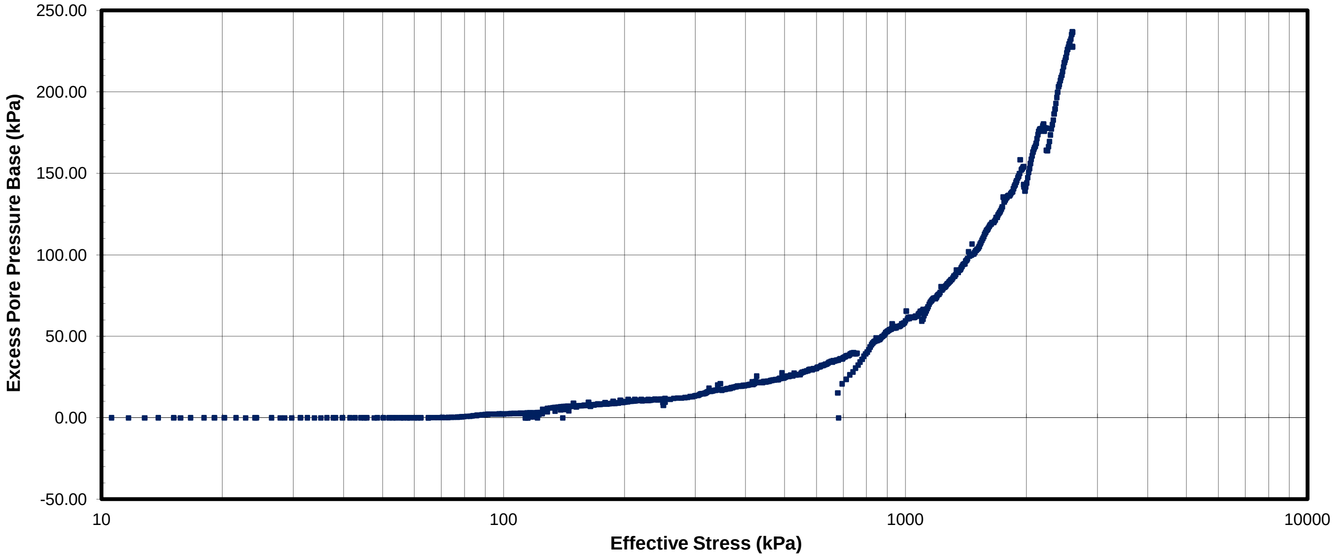
Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 15, 2014
Borehole:	BH4	Station:	2
Sample No.:	S9	Depth (m):	12.37

Weight of Ring (g):	211.96	**Ring + Wet Weight (g):**	339.65	**Initial Void Ratio, e:**	1.82
Initial Height (mm):	25.39	**Ring + Dry Weight (g):**	291.34	**Height of Soil, Hs (mm):**	9.01
Diameter of Ring (mm):	63.50	**Water Content (%):**	60.9	**Height of Void, Hv (mm):**	16.38
Unit Weight (kN/m^3)	15.57	**Specific Gravity, Gs:**	2.78		

Loading Strain Rate (%/hr):	2	**Max Stress (kPa):**	2601	**Backpressure (kPa):**	407
Unloading Strain Rate (%/hr):	N/A	**Max Strain (%):**	30.9		

Comments: test started at 2% per hour and rate was increased to 4% per hour at 6.17% strain (124 kPa)





Prepared By:	PC	Checked By:	PS	Approved By:	
Date:	May 16, 2014	Date:	May 16, 2014	Date:	

MEG TECHNICAL SERVICES

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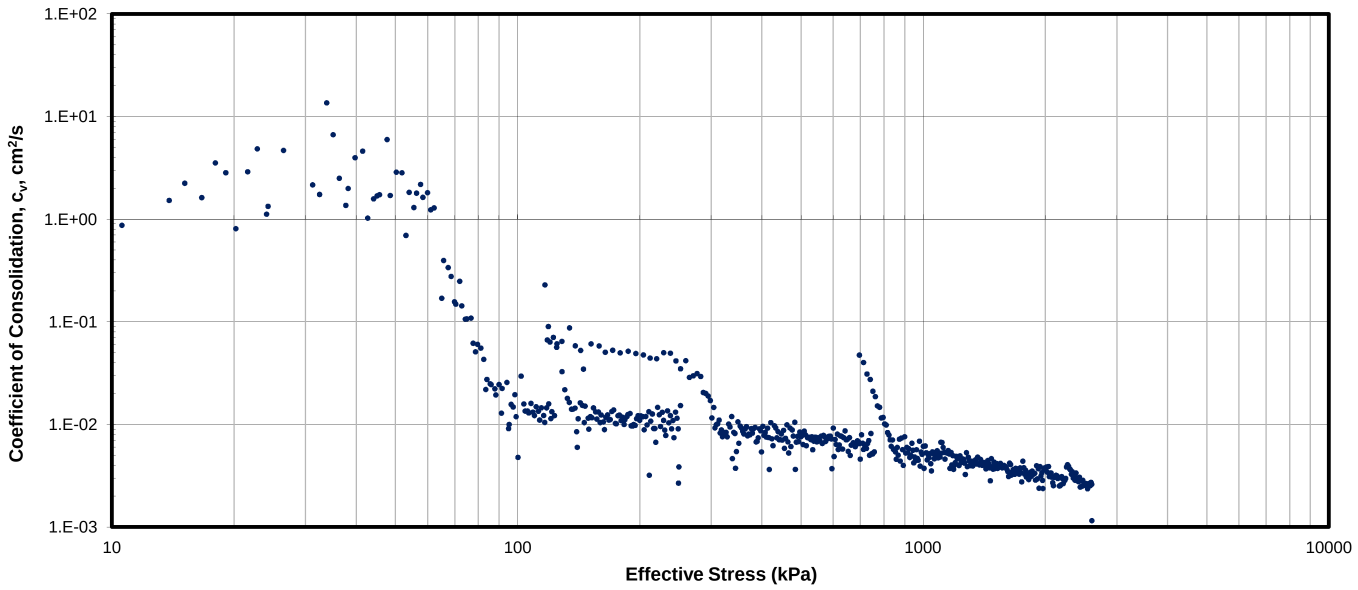
Constant Rate of Strain Consolidation (ASTM D4186)

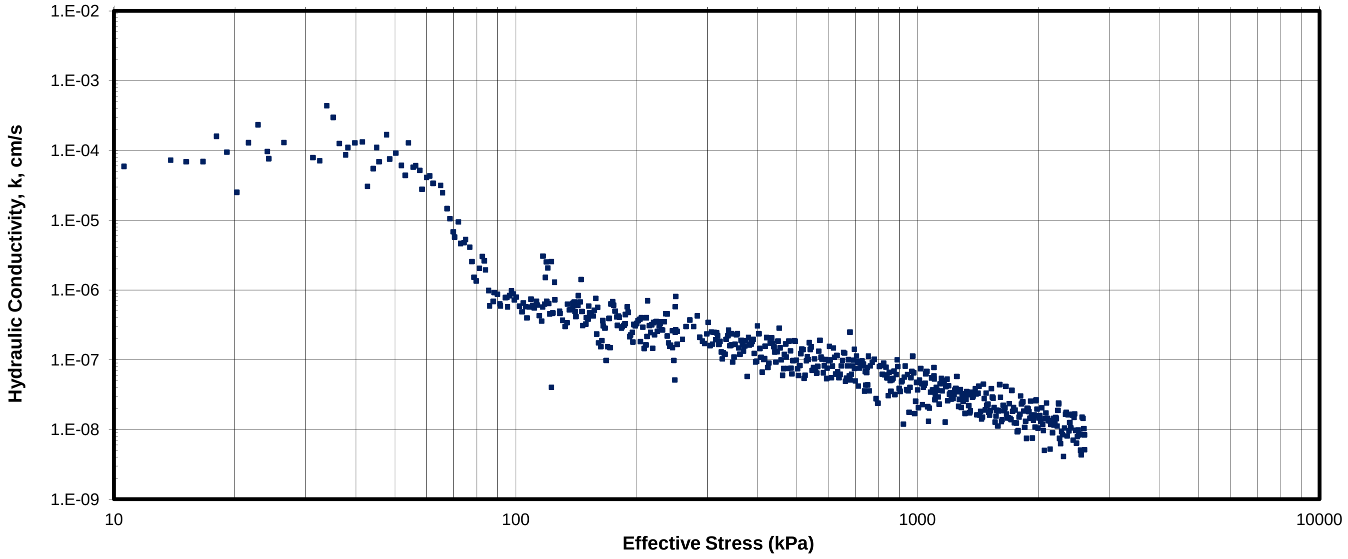
Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 15, 2014
Borehole:	BH4	Station:	2
Sample No.:	S9	Depth (m):	12.37

Weight of Ring (g):	211.96	**Ring + Wet Weight (g):**	339.65	**Initial Void Ratio, e:**	1.82
Initial Height (mm):	25.39	**Ring + Dry Weight (g):**	291.34	**Height of Soil, Hs (mm):**	9.01
Diameter of Ring (mm):	63.50	**Water Content (%):**	60.9	**Height of Void, Hv (mm):**	16.38
Unit Weight (kN/m^3)	15.57	**Specific Gravity, Gs:**	2.78		

Loading Strain Rate (%/hr):	2	**Max Stress (kPa):**	2601	**Backpressure (kPa):**	407
Unloading Strain Rate (%/hr):	N/A	**Max Strain (%):**	30.9		

Comments: test started at 2% per hour and rate was increased to 4% per hour at 6.17% strain (124 kPa)





Prepared By:	PC	Checked By:	PS	Approved By:	
Date:	May 16, 2014	Date:	May 16, 2014	Date:	

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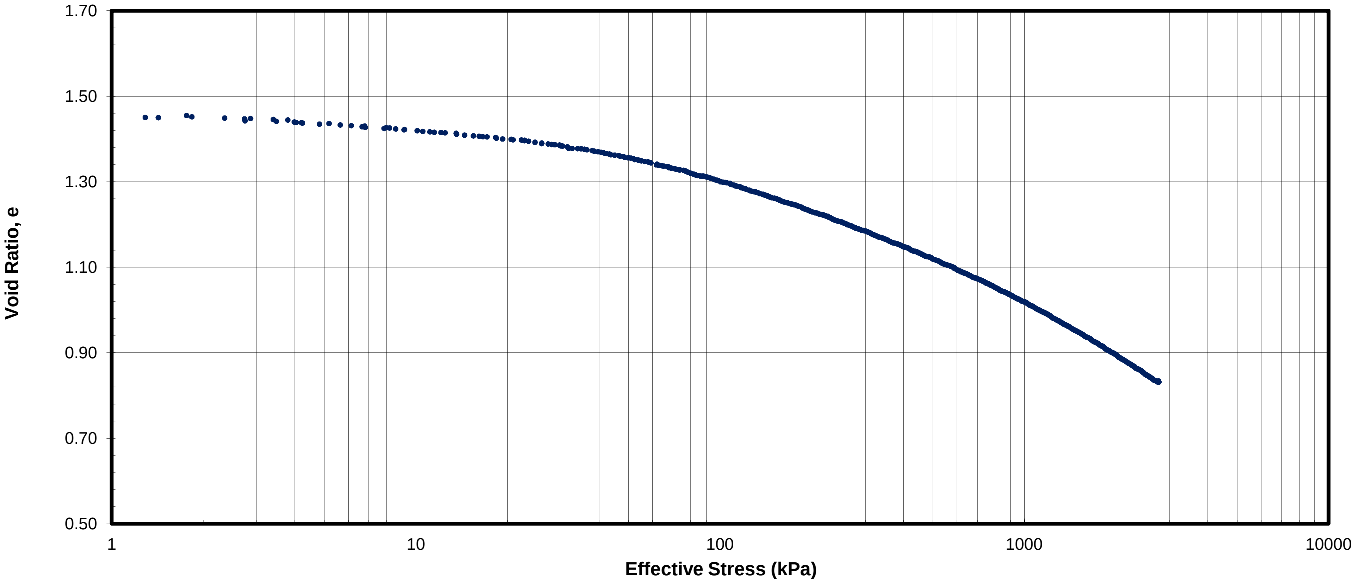
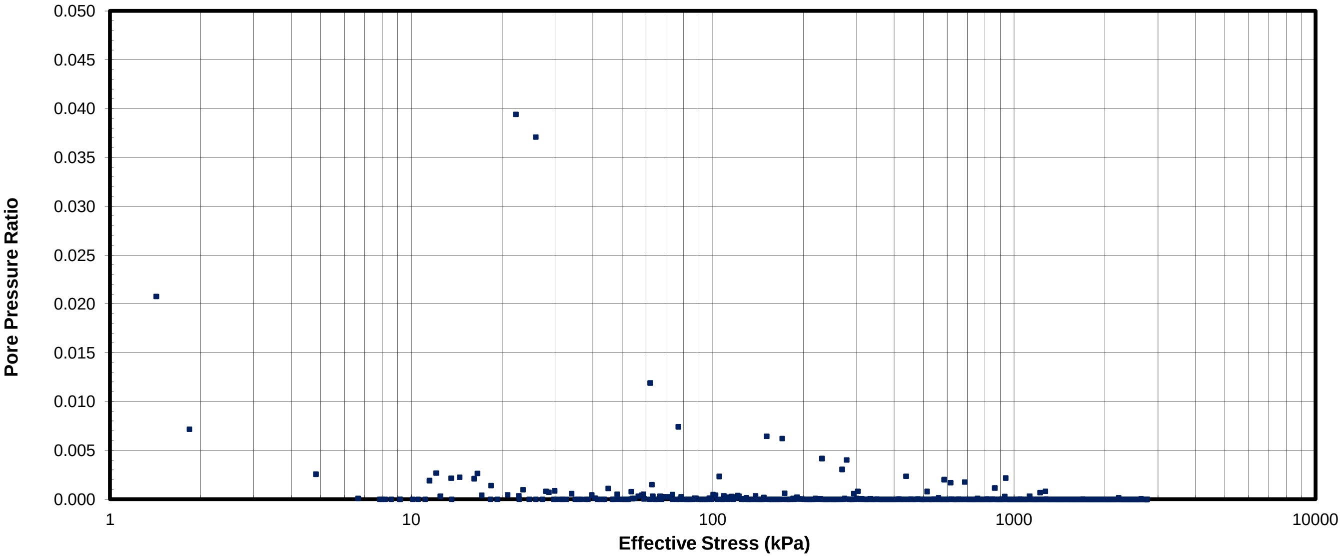
Constant Rate of Strain Consolidation (ASTM D4186)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 26, 2014
Borehole:	BH5	Station:	CRS 2
Sample No.:	S1	Depth (m):	3.53

Weight of Ring (g):	211.23	**Ring + Wet Weight (g):**	343.93	**Initial Void Ratio, e:**	1.49
Initial Height (mm):	25.39	**Ring + Dry Weight (g):**	297.14	**Height of Soil, Hs (mm):**	10.18
Diameter of Ring (mm):	63.50	**Water Content (%):**	54.5	**Height of Void, Hv (mm):**	15.21
Unit Weight (kN/m^3)	16.18	**Specific Gravity, Gs:**	2.66		

Loading Strain Rate (%/hr):	4	**Max Stress (kPa):**	2760	**Backpressure (kPa):**	407
Unloading Strain Rate (%/hr):	N/A	**Max Strain (%):**	26.6		





Prepared By:	PC	Checked By:	PS	Approved By:	EP
Date:	May 26, 2014	Date:	May 26, 2014	Date:	May 28, 2014

MEG TECHNICAL SERVICES

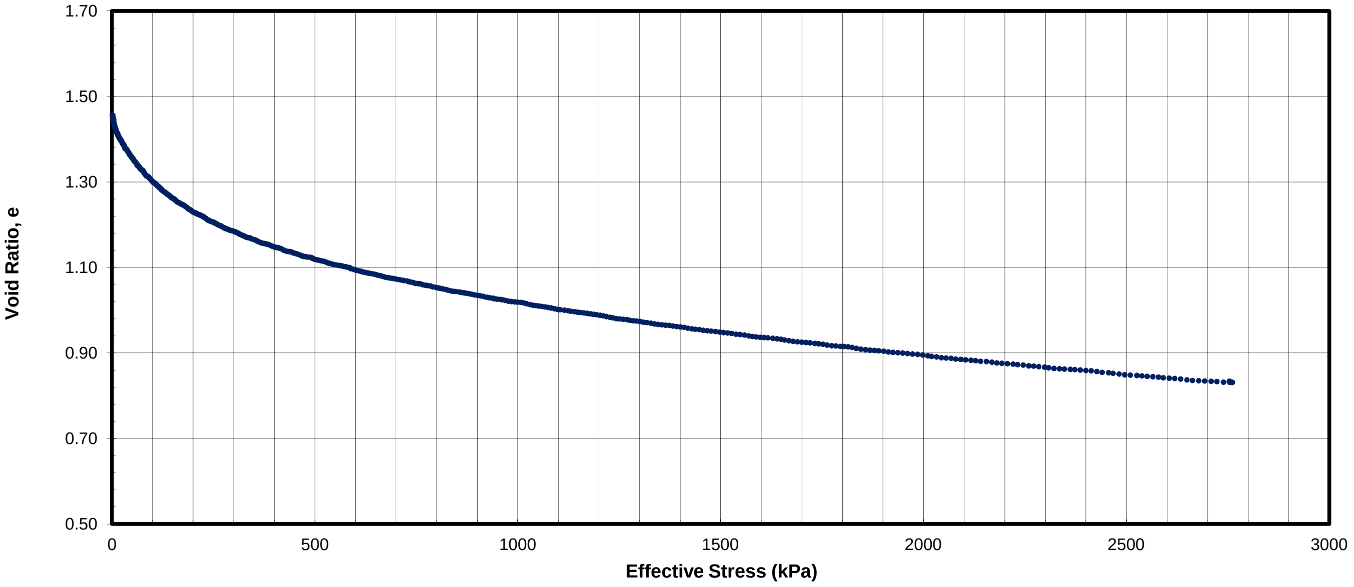
(A Division of MEG Consulting Limited)

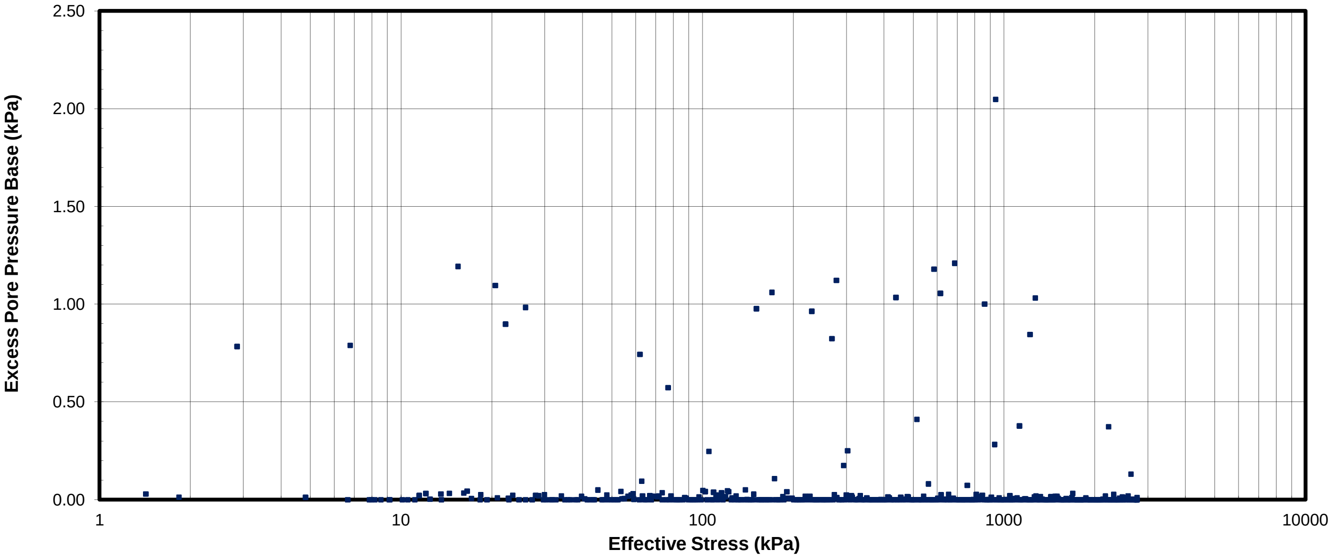
Constant Rate of Strain Consolidation (ASTM D4186)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 26, 2014
Borehole:	BH5	Station:	CRS 2
Sample No.:	S1	Depth (m):	3.53

Weight of Ring (g):	211.23	**Ring + Wet Weight (g):**	343.93	**Initial Void Ratio, e:**	1.49
Initial Height (mm):	25.39	**Ring + Dry Weight (g):**	297.14	**Height of Soil, Hs (mm):**	10.18
Diameter of Ring (mm):	63.50	**Water Content (%):**	54.5	**Height of Void, Hv (mm):**	15.21
Unit Weight (kN/m^3)	16.18	**Specific Gravity, Gs:**	2.66		

Loading Strain Rate (%/hr):	4	**Max Stress (kPa):**	2760	**Backpressure (kPa):**	407
Unloading Strain Rate (%/hr):	N/A	**Max Strain (%):**	26.6		





Prepared By:	PC	Checked By:	PS	Approved By:	EP
Date:	May 26, 2014	Date:	May 26, 2014	Date:	May 28, 2014

MEG TECHNICAL SERVICES

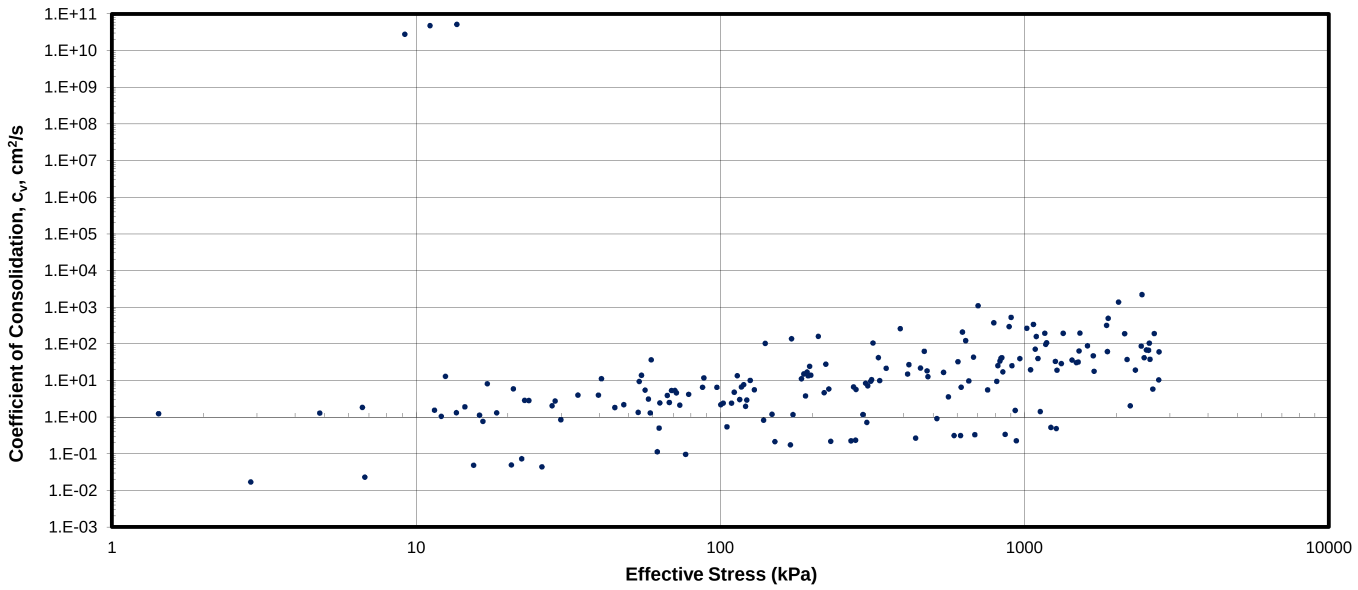
(A Division of MEG Consulting Limited)

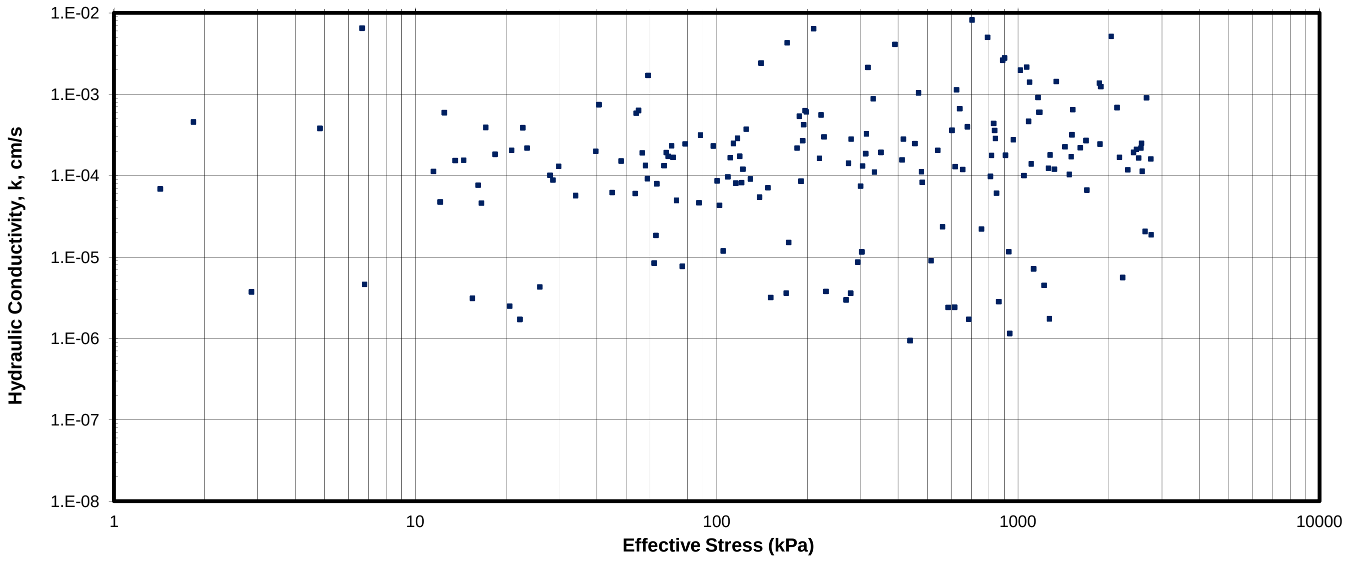
Constant Rate of Strain Consolidation (ASTM D4186)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 26, 2014
Borehole:	BH5	Station:	CRS 2
Sample No.:	S1	Depth (m):	3.53

Weight of Ring (g):	211.23	**Ring + Wet Weight (g):**	343.93	**Initial Void Ratio, e:**	1.49
Initial Height (mm):	25.39	**Ring + Dry Weight (g):**	297.14	**Height of Soil, Hs (mm):**	10.18
Diameter of Ring (mm):	63.50	**Water Content (%):**	54.5	**Height of Void, Hv (mm):**	15.21
Unit Weight (kN/m^3)	16.18	**Specific Gravity, Gs:**	2.66		

Loading Strain Rate (%/hr):	4	**Max Stress (kPa):**	2760	**Backpressure (kPa):**	407
Unloading Strain Rate (%/hr):	N/A	**Max Strain (%):**	26.6		





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Date:	May 26, 2014	Date:	May 26, 2014	Date:	May 28, 2014

MEG TECHNICAL SERVICES

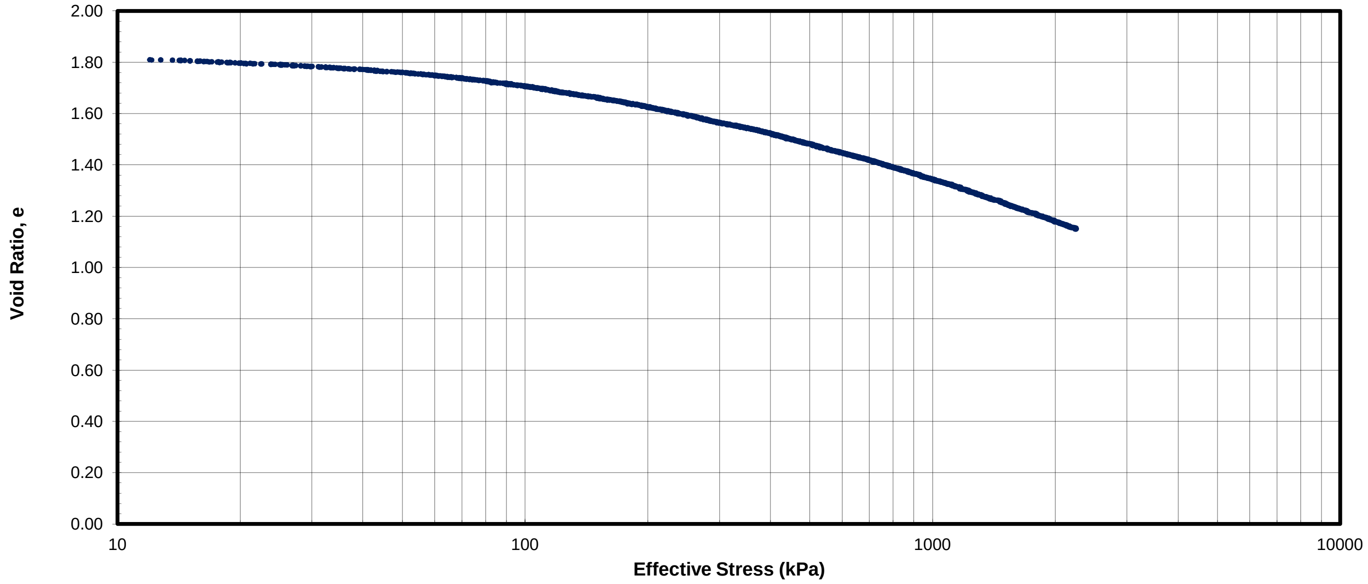
(A Division of MEG Consulting Limited)

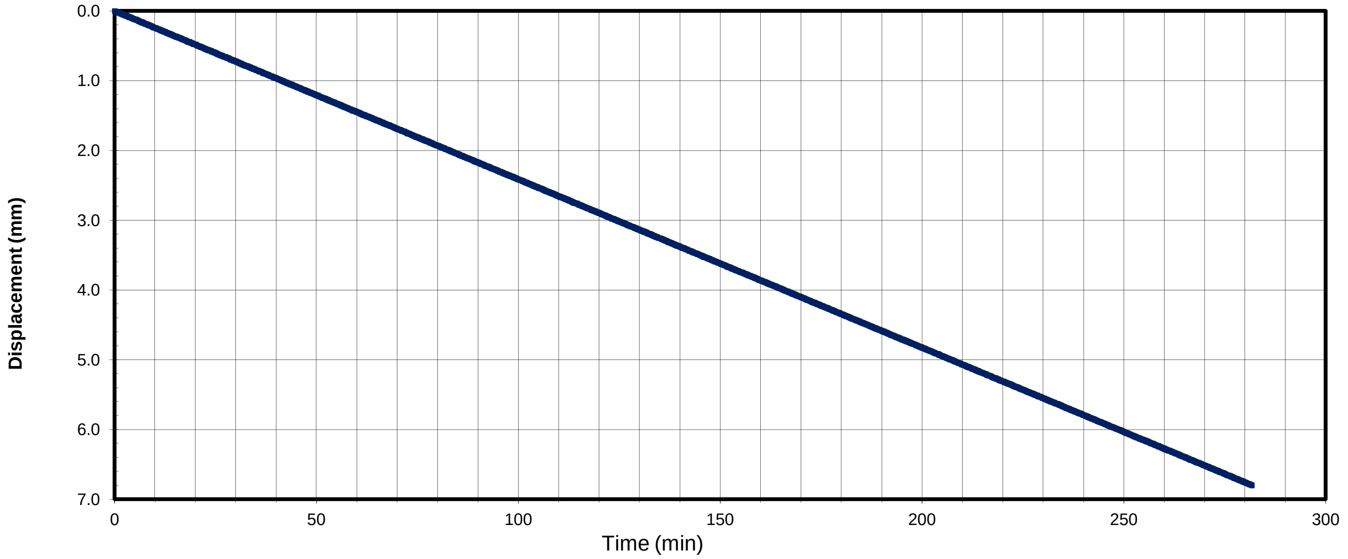
Constant Rate of Strain Consolidation

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BC1	Station:	DSS
Sample No.:	BH3-S1	Depth (m):	7.88

Weight of Ring (g):	2892.39	**Ring + Wet Weight (g):**	3082.27	**Initial Void Ratio, e:**	1.46
Initial Height (mm):	25.32	**Ring + Dry Weight (g):**	3012.32	**Height of Soil, Hs (mm):**	10.31
Diameter of Ring (mm):	73.33	**Water Content (%):**	58.3	**Height of Void, Hv (mm):**	15.02
Unit Weight (kN/m^3)	15.22	**Specific Gravity, Gs:**	2.76		

Loading Strain Rate (%/hr):	5	**Max Stress (kPa):**	2248	**Backpressure (kPa):**	N/A
Unloading Strain Rate (%/hr):	N/A	**Max Strain (%):**	23.5		





Prepared By:	PC	Checked By:	PS	Approved By:	EP
Date:	May 7, 2014	Date:	May 7, 2014	Date:	May 7, 2014

MEG TECHNICAL SERVICES

(A Division of MEG Consulting Limited)

Constant Rate of Strain Consolidation

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BC1	Station:	DSS
Sample No.:	BH3-S1	Depth (m):	7.88

Weight of Ring (g):	2892.39	**Ring + Wet Weight (g):**	3082.27	**Initial Void Ratio, e:**	1.80
Initial Height (mm):	28.89	**Ring + Dry Weight (g):**	3012.32	**Height of Soil, Hs (mm):**	10.31
Diameter of Ring (mm):	73.33	**Water Content (%):**	58.3	**Height of Void, Hv (mm):**	18.59
Unit Weight (kN/m^3)	15.22	**Specific Gravity, Gs:**	2.76		

Loading Strain Rate (%/hr):	5	**Max Stress (kPa):**	2248	**Backpressure (kPa):**	N/A
Unloading Strain Rate (%/hr):	N/A	**Max Strain (%):**	23.5		

Comments: Due to the nature of the sample, it was not possible to test the sample in the standard consolidation setup as the sample would have fallen apart during trimming. HDR requested that a sample be setup in the DSS testing equipment and the sample loaded similarly to a CRS consolidation The sample consisted of fine to medium gravel sized particles. After cutting the Shelby tube and a visual inspection of the sample, these particles looked like rounded gravel (see pictures attached). After setting up the sample and further examining these particles they were composed of fine silty sand or sandy silt (see pictures).

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Constant Rate of Strain Consolidation

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BC1	Station:	DSS
Sample No.:	BH3-S1	Depth (m):	7.88

SAMPLE PICTURES

Comments: pictures of gravel sized particles prior to testing

Prepared By:	PC	Checked By:	PS	Approved By:	EP
Date:	May 7, 2014	Date:	May 7, 2014	Date:	May 7, 2014

MEG TECHNICAL SERVICES

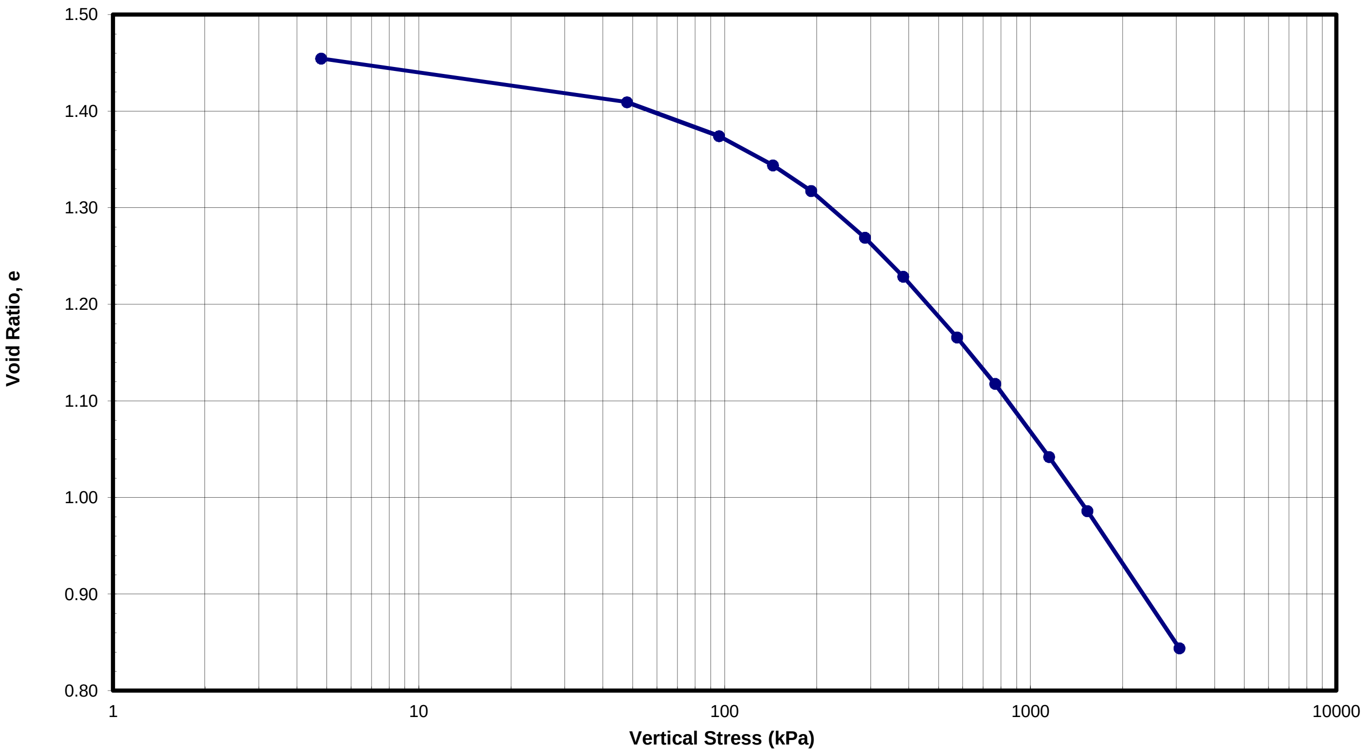
(A Division of MEG Consulting Limited)

One-Dimensional Consolidation (ASTM D 2435)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BH6	Station:	1
Sample No.:	S4	Depth (m):	5.80

Weight of Ring (g):	213.82	**Ring + Wet Weight (g):**	347.46	**Initial Void Ratio, e:**	1.46
Initial Height (mm):	25.40	**Ring + Dry Weight (g):**	301.53	**Height of Soil, Hs (mm):**	10.34
Diameter of Ring (mm):	63.50	**Water Content (%):**	52.4	**Height of Void, Hv (mm):**	15.06
Unit Weight (kN/m^3)	16.30	**Specific Gravity, Gs:**	2.68		

Step No.	**Vertical Stress (kPa)**	**Height of Sample (mm)**	**Vertical Strain (%)**	**Final Void Ratio e_f**	**Change in Void Ratio e**	**Coefficient of Compressibility a_v (m^2/MN)**	**Coefficient of Volume Compressibility m_v (m^2/MN)**
1	5	25.3771	0.0900	1.4546	0.00		
2	48	24.9098	1.9300	1.4094	0.05	1.0490	0.43
3	96	24.5466	3.3600	1.3743	0.04	0.7338	0.30
4	144	24.2341	4.5900	1.3440	0.03	0.6311	0.26
5	192	23.9598	5.6700	1.3175	0.03	0.5542	0.23
6	287	23.4594	7.6400	1.2691	0.05	0.5054	0.21
7	383	23.0429	9.2800	1.2288	0.04	0.4208	0.17
8	575	22.3926	11.8400	1.1659	0.06	0.3284	0.13
9	766	21.8948	13.8000	1.1178	0.05	0.2514	0.10
10	1149	21.1125	16.8800	1.0421	0.08	0.1976	0.08
11	1532	20.5334	19.1600	0.9861	0.06	0.1462	0.06
12	3064	19.0652	24.9400	0.8441	0.14	0.0927	0.04



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MEG TECHNICAL SERVICES

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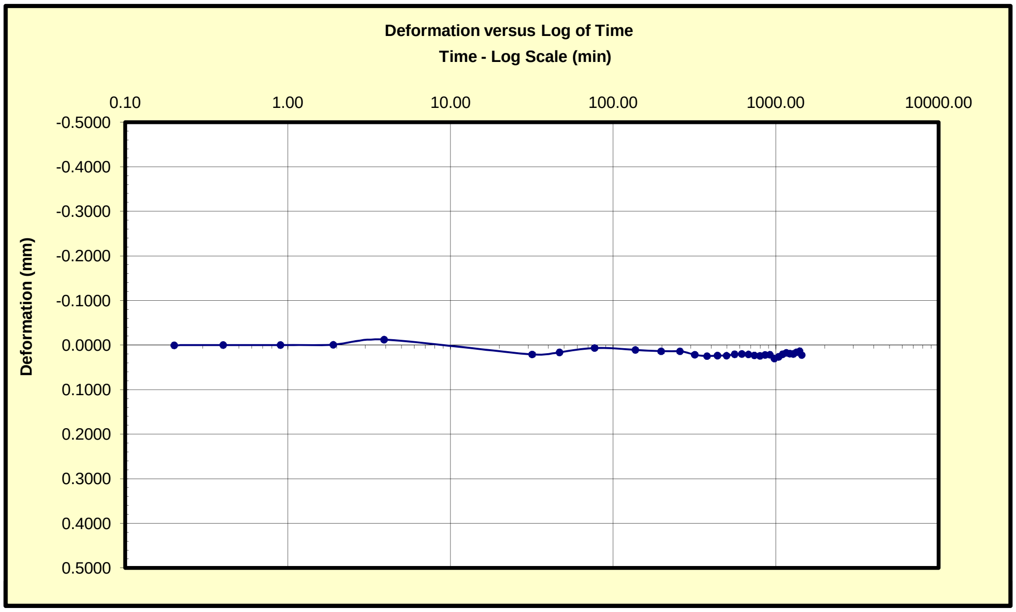
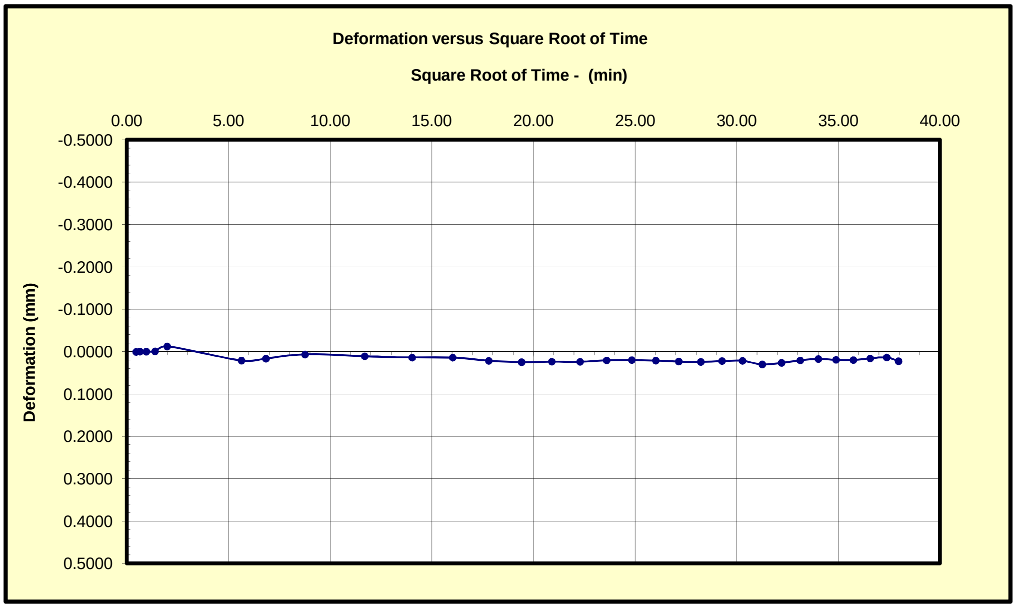
One-Dimensional Consolidation (ASTM D 2435)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BH6	Station:	1
Sample No.:	S4	Depth (m):	5.80
Consolidation Step:	1	Vertical Stress (kPa):	5





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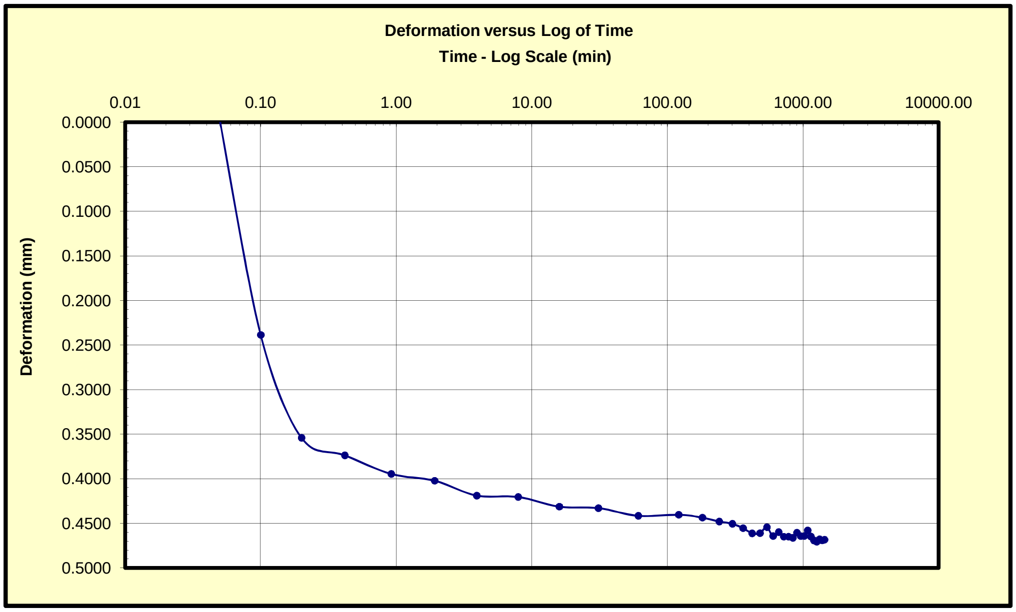
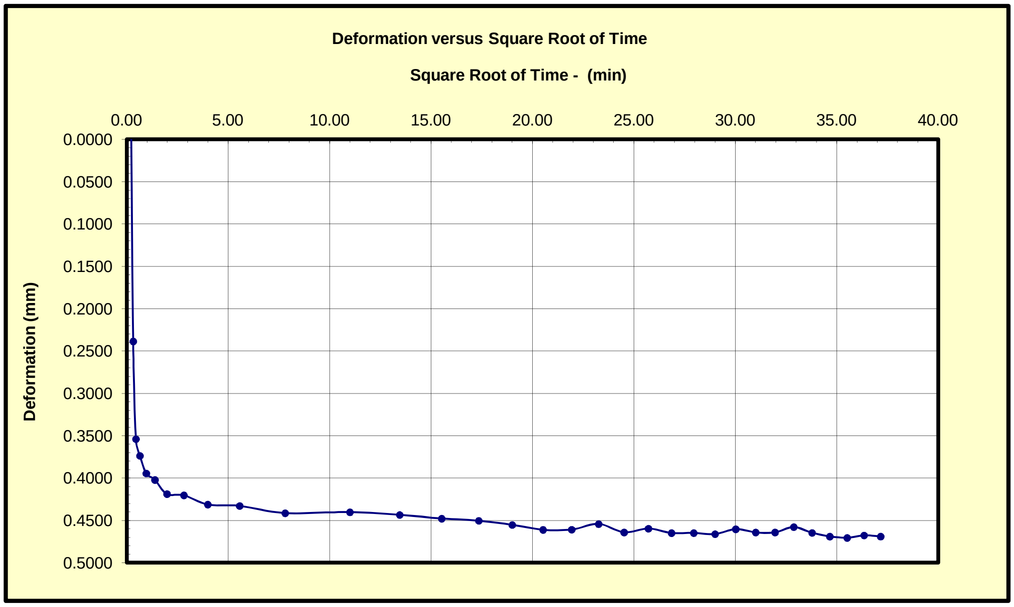
One-Dimensional Consolidation (ASTM D 2435)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BH6	Station:	1
Sample No.:	S4	Depth (m):	5.80
Consolidation Step:	2	Vertical Stress (kPa):	48





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Date:	May 7, 2014	Date:	May 7, 2014	Date:	May 7, 2014

MEG TECHNICAL SERVICES

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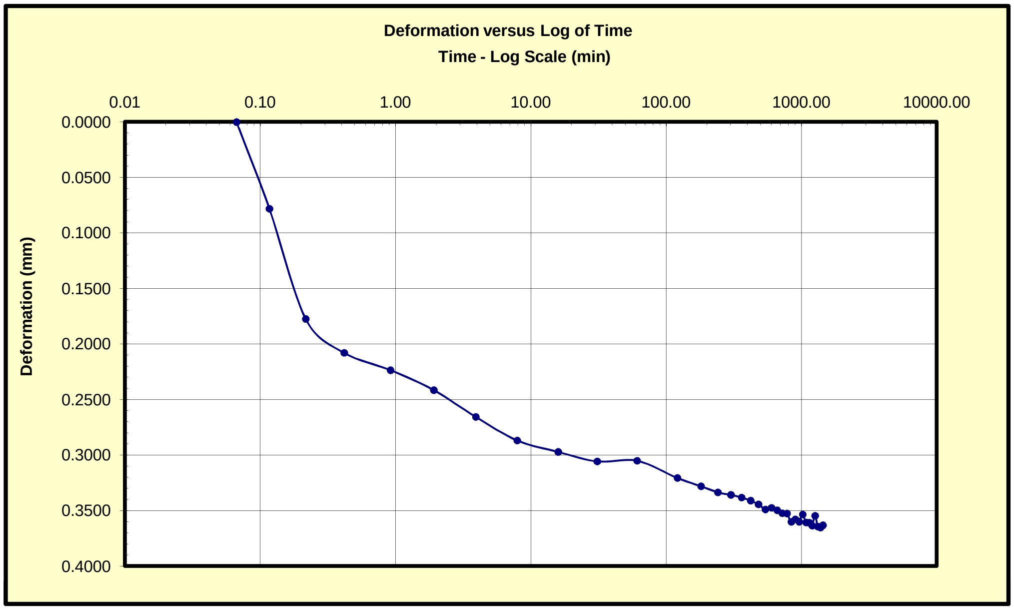
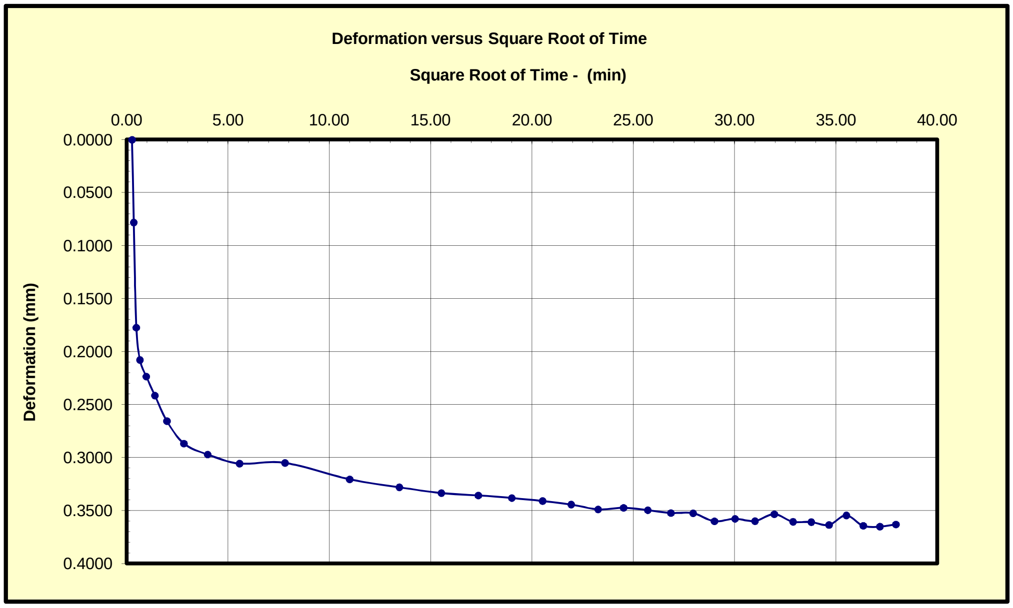
One-Dimensional Consolidation (ASTM D 2435)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BH6	Station:	1
Sample No.:	S4	Depth (m):	5.80
Consolidation Step:	3	Vertical Stress (kPa):	96





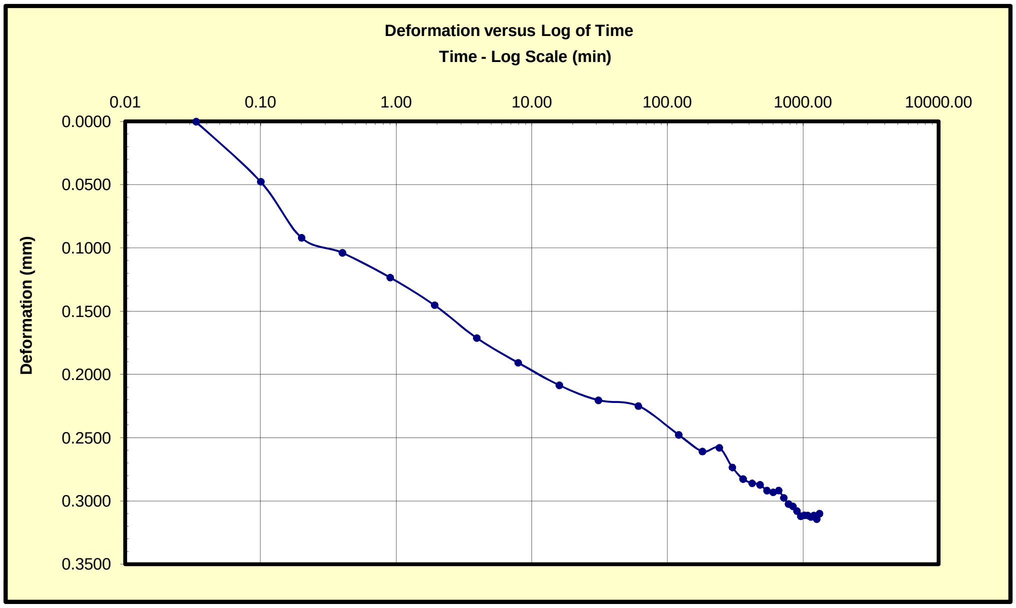
Prepared By:	PC	Checked By:	PS	Approved By:	EP
Date:	May 7, 2014	Date:	May 7, 2014	Date:	May 7, 2014

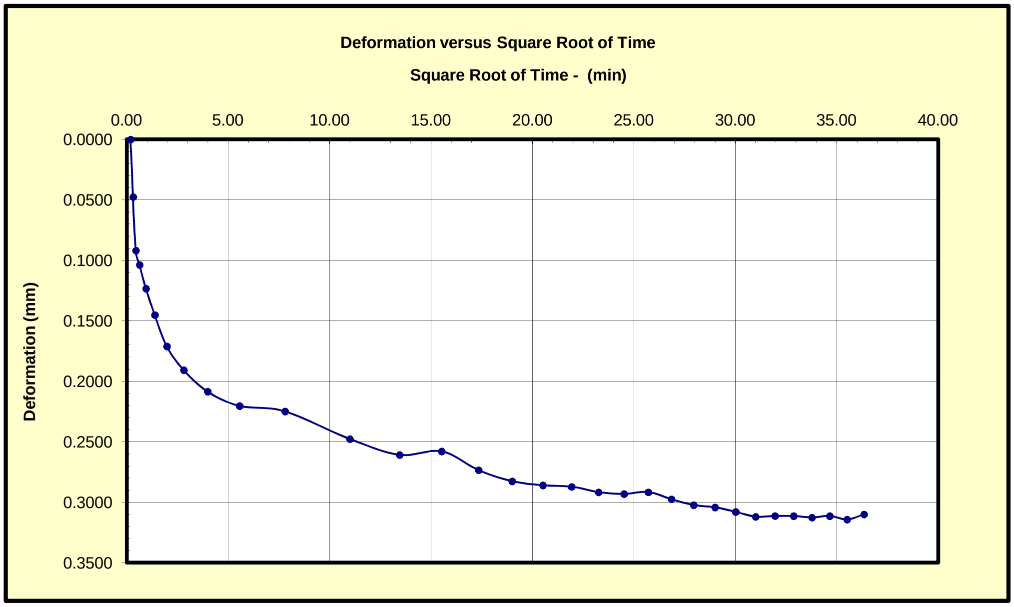
MEG TECHNICAL SERVICES

(A Division of MEG Consulting Limited)

One-Dimensional Consolidation (ASTM D 2435)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BH6	Station:	1
Sample No.:	S4	Depth (m):	5.80
Consolidation Step:	4	Vertical Stress (kPa):	144





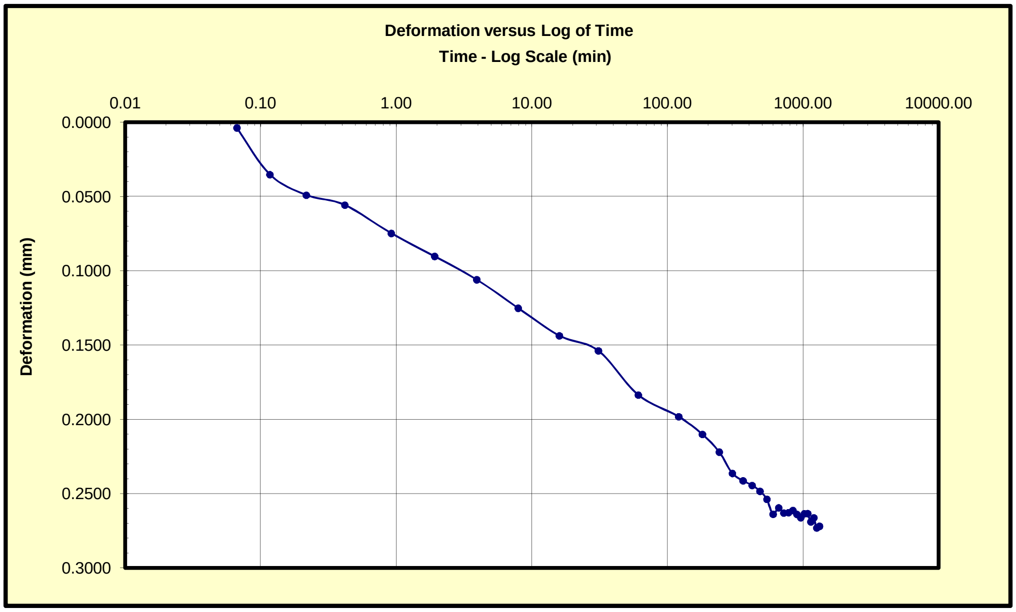
Prepared By:	PC	Checked By:	PS	Approved By:	EP
Date:	May 7, 2014	Date:	May 7, 2014	Date:	May 7, 2014

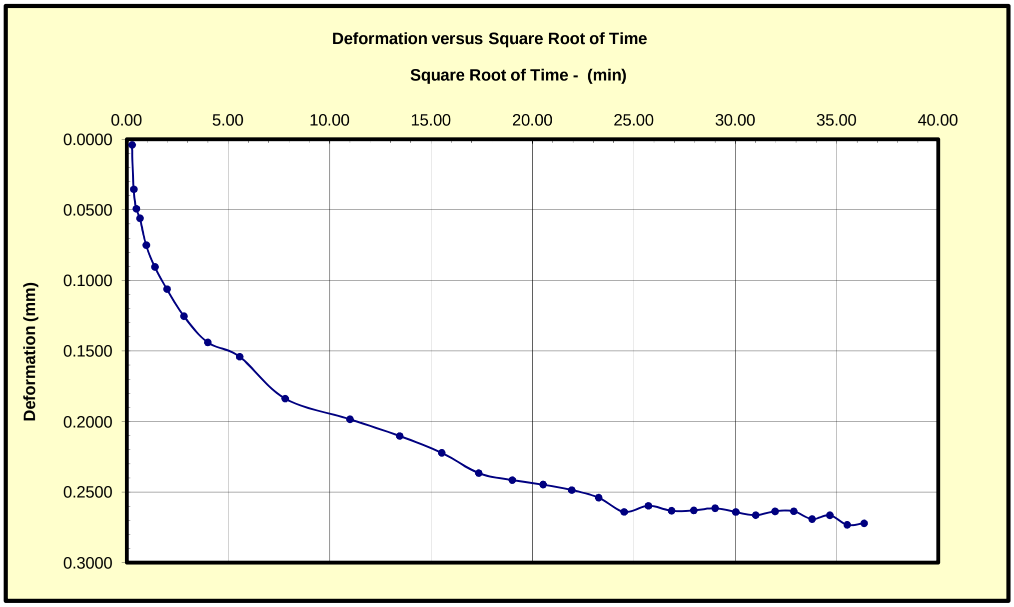
MEG TECHNICAL SERVICES

(A Division of MEG Consulting Limited)

One-Dimensional Consolidation (ASTM D 2435)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BH6	Station:	1
Sample No.:	S4	Depth (m):	5.80
Consolidation Step:	5	Vertical Stress (kPa):	192





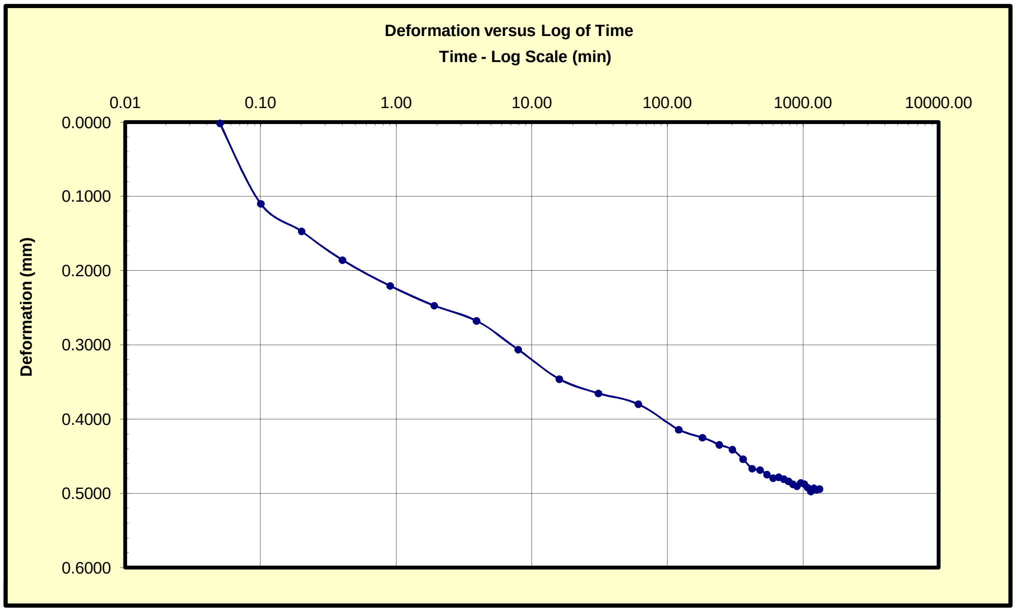
Prepared By:	PC	Checked By:	PS	Approved By:	EP
Date:	May 7, 2014	Date:	May 7, 2014	Date:	May 7, 2014

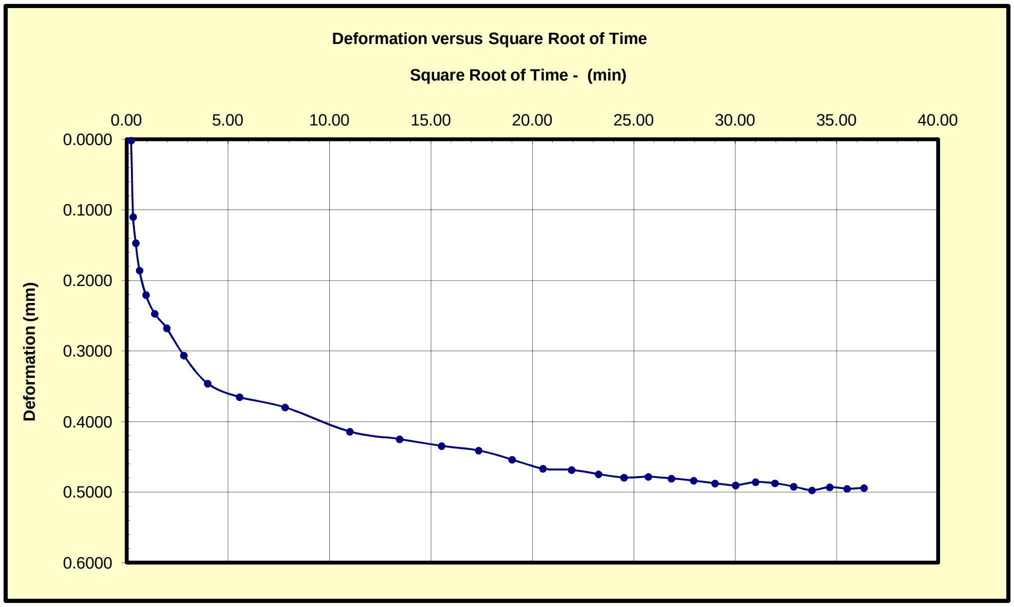
MEG TECHNICAL SERVICES

(A Division of MEG Consulting Limited)

One-Dimensional Consolidation (ASTM D 2435)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BH6	Station:	1
Sample No.:	S4	Depth (m):	5.80
Consolidation Step:	6	Vertical Stress (kPa):	287





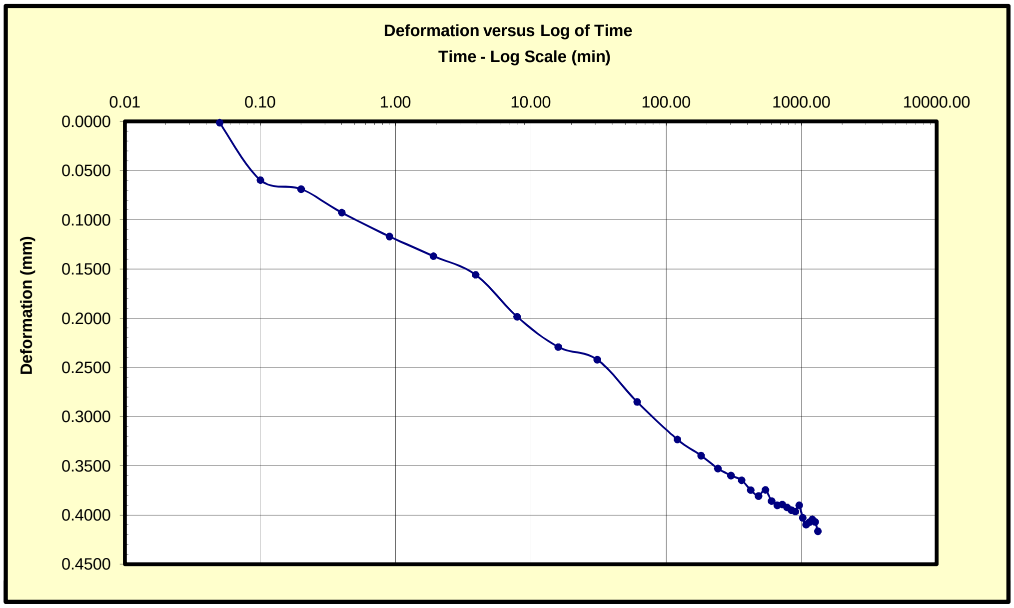
Prepared By:	PC	Checked By:	PS	Approved By:	EP
Date:	May 7, 2014	Date:	May 7, 2014	Date:	May 7, 2014

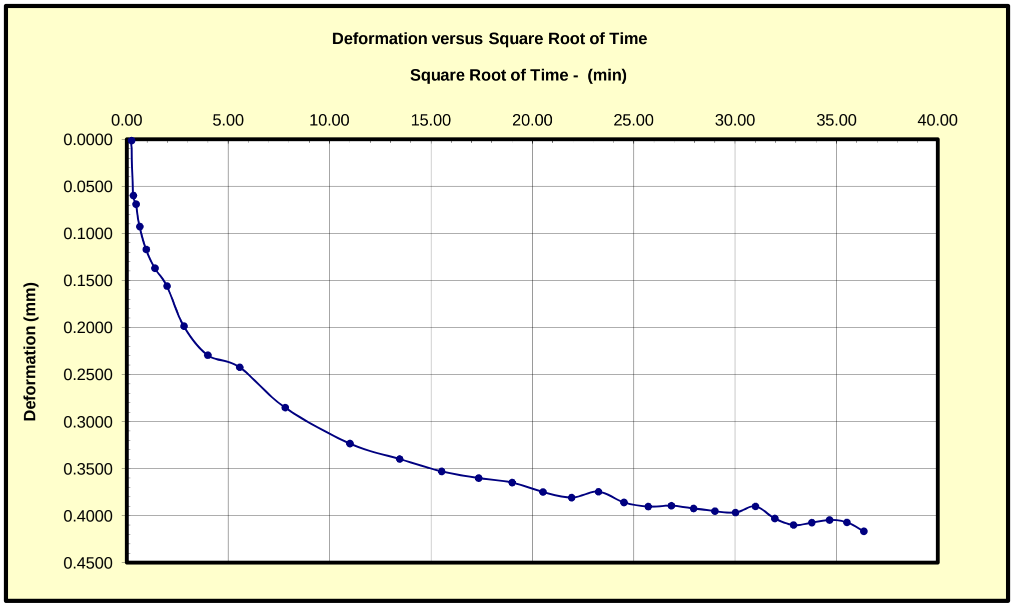
MEG TECHNICAL SERVICES

(A Division of MEG Consulting Limited)

One-Dimensional Consolidation (ASTM D 2435)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BH6	Station:	1
Sample No.:	S4	Depth (m):	5.80
Consolidation Step:	7	Vertical Stress (kPa):	383





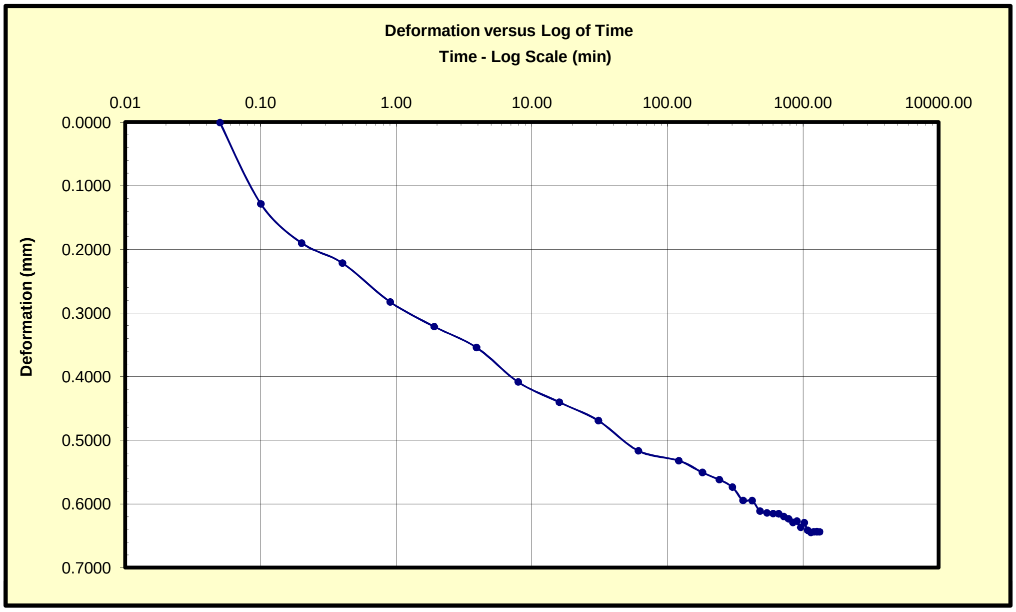
Prepared By:	PC	Checked By:	PS	Approved By:	EP
Date:	May 7, 2014	Date:	May 7, 2014	Date:	May 7, 2014

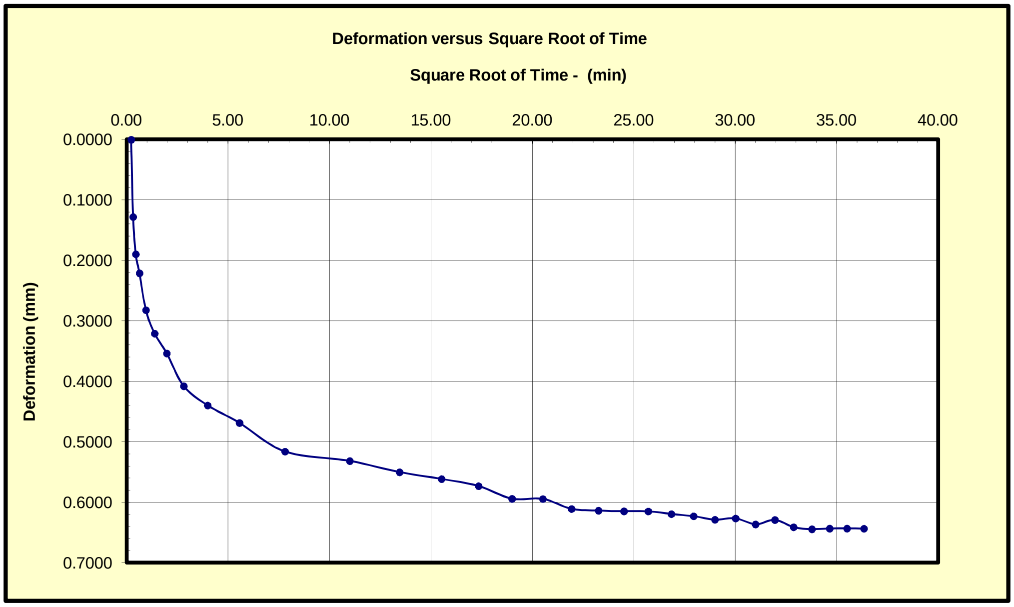
MEG TECHNICAL SERVICES

(A Division of MEG Consulting Limited)

One-Dimensional Consolidation (ASTM D 2435)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BH6	Station:	1
Sample No.:	S4	Depth (m):	5.80
Consolidation Step:	8	Vertical Stress (kPa):	575





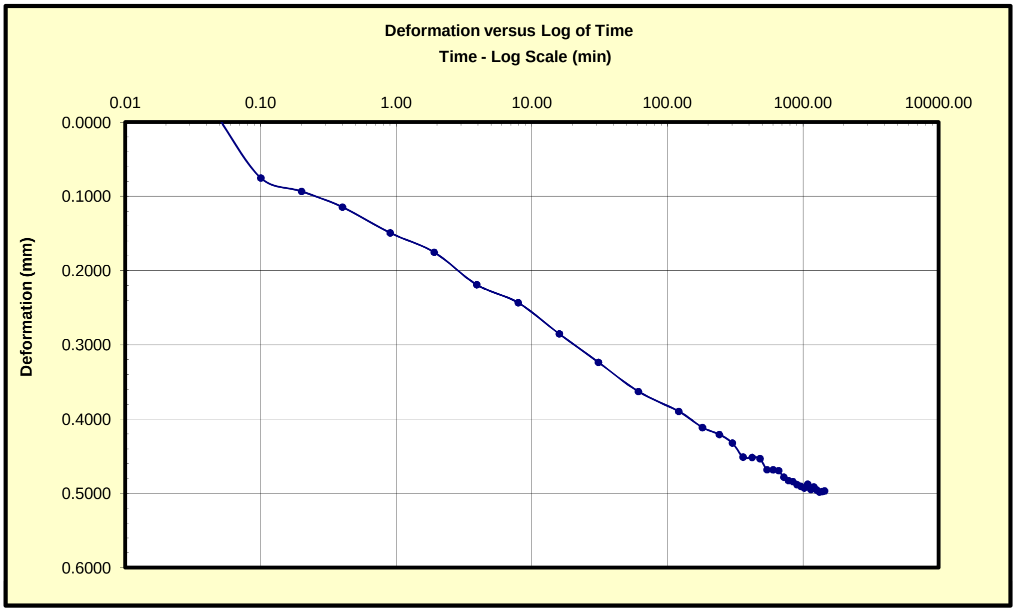
Prepared By:	PC	Checked By:	PS	Approved By:	EP
Date:	May 7, 2014	Date:	May 7, 2014	Date:	May 7, 2014

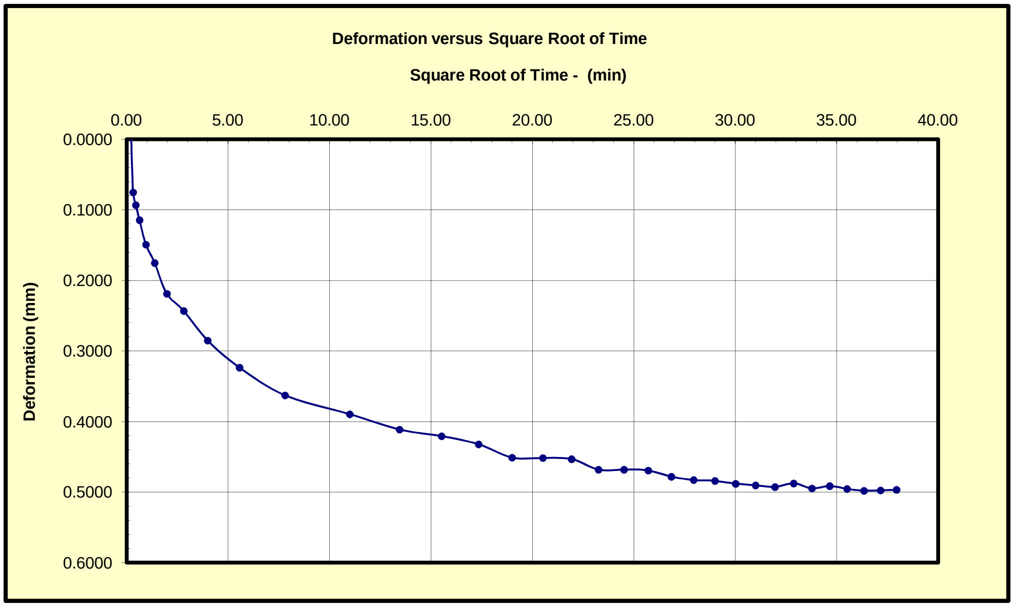
MEG TECHNICAL SERVICES

(A Division of MEG Consulting Limited)

One-Dimensional Consolidation (ASTM D 2435)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BH6	Station:	1
Sample No.:	S4	Depth (m):	5.80
Consolidation Step:	9	Vertical Stress (kPa):	766





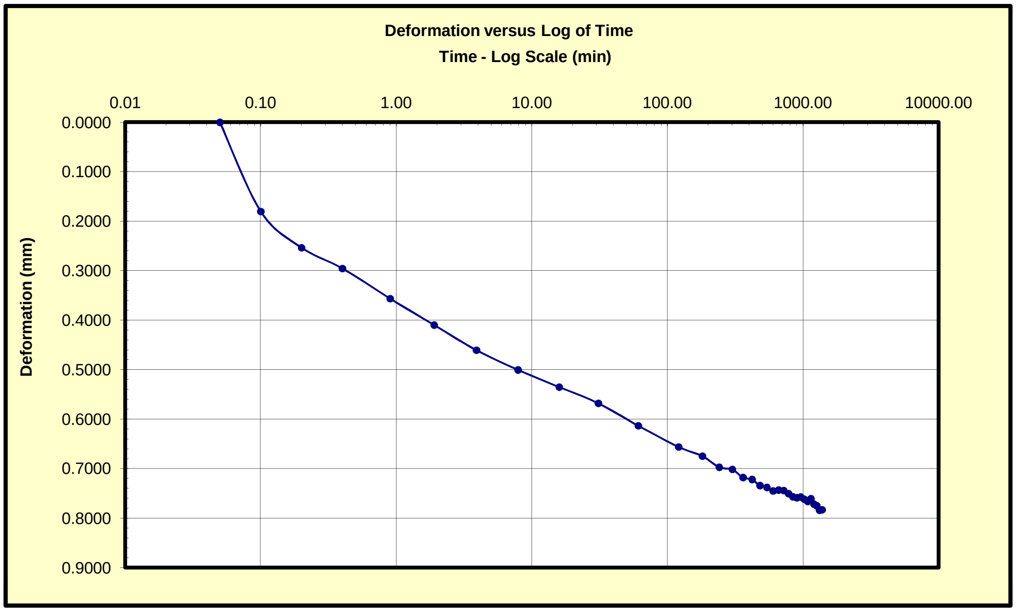
Prepared By:	PC	Checked By:	PS	Approved By:	EP
Date:	May 7, 2014	Date:	May 7, 2014	Date:	May 7, 2014

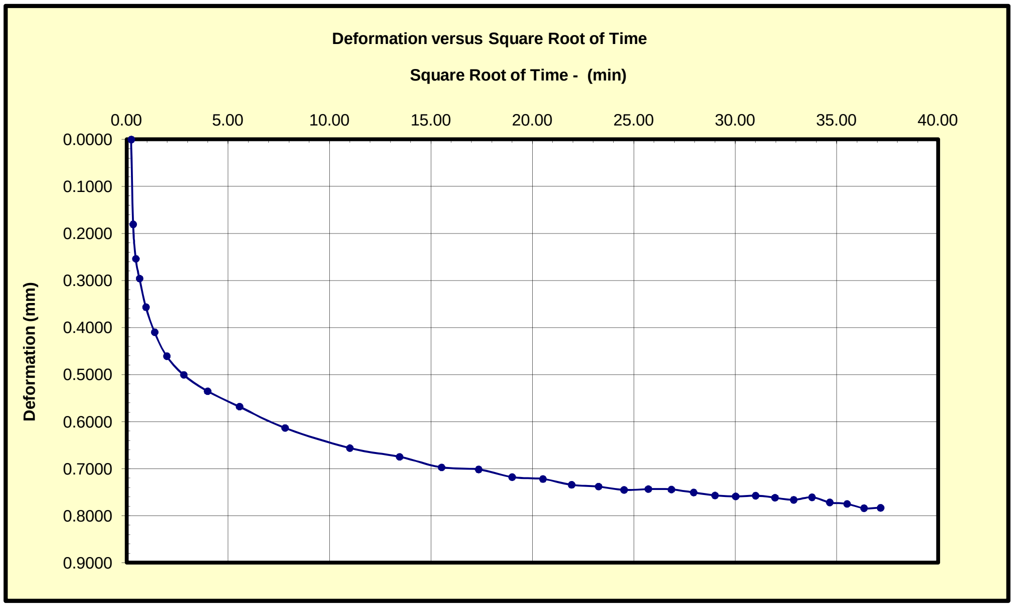
MEG TECHNICAL SERVICES

(A Division of MEG Consulting Limited)

One-Dimensional Consolidation (ASTM D 2435)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BH6	Station:	1
Sample No.:	S4	Depth (m):	5.80
Consolidation Step:	10	Vertical Stress (kPa):	1149





Prepared By:	PC	Checked By:	PS	Approved By:	EP
Date:	May 7, 2014	Date:	May 7, 2014	Date:	May 7, 2014

MEG TECHNICAL SERVICES

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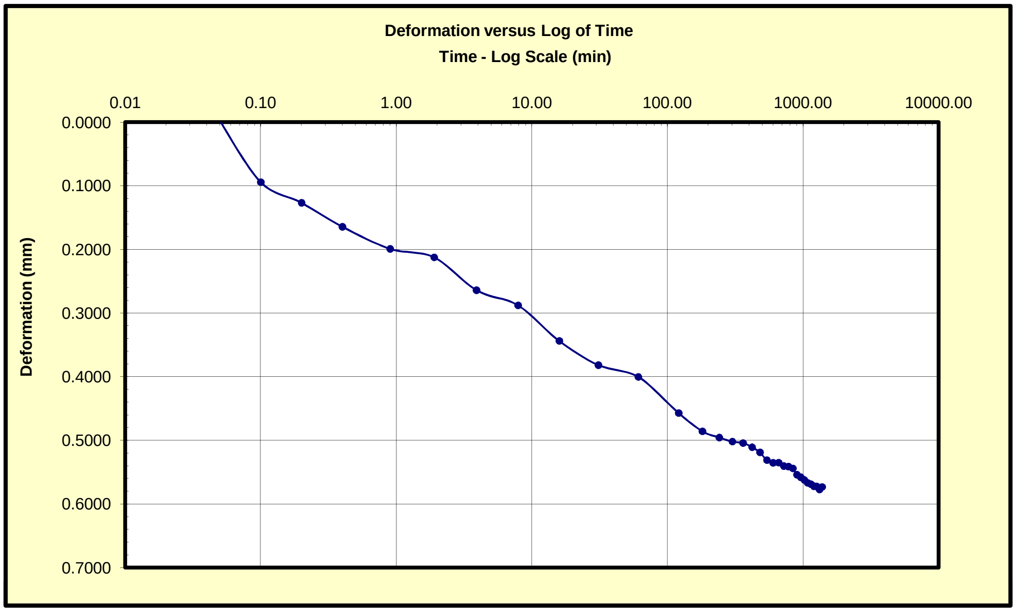
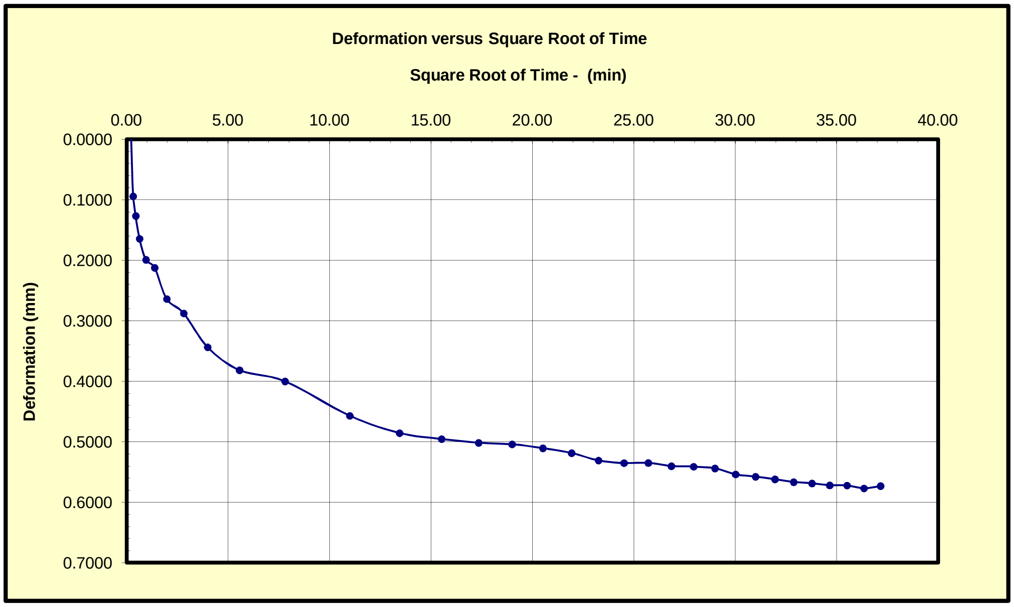
One-Dimensional Consolidation (ASTM D 2435)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BH6	Station:	1
Sample No.:	S4	Depth (m):	5.80
Consolidation Step:	11	Vertical Stress (kPa):	1532





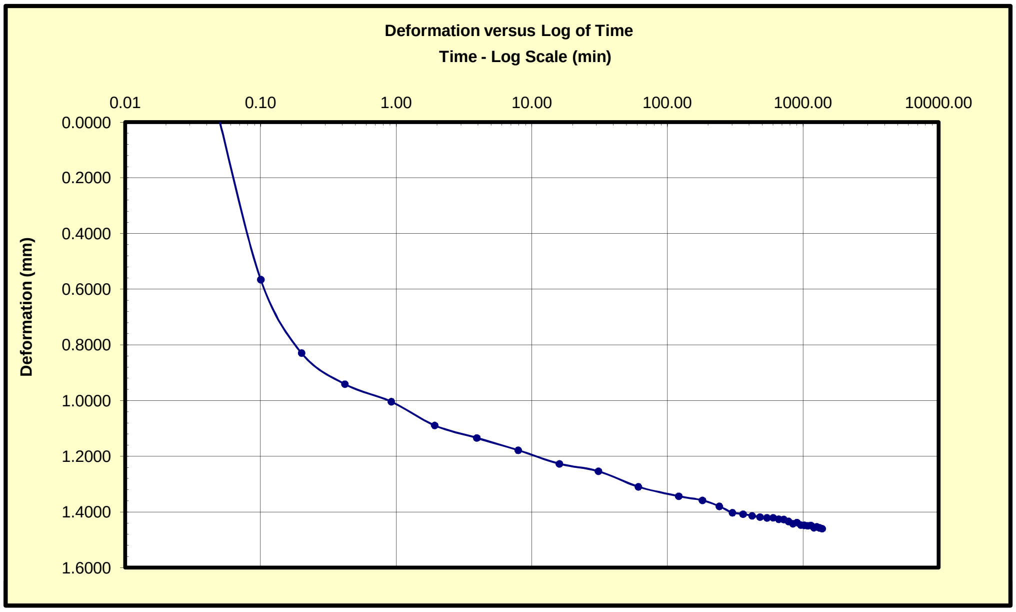
Prepared By:	PC	Checked By:	PS	Approved By:	EP
Date:	May 7, 2014	Date:	May 7, 2014	Date:	May 7, 2014

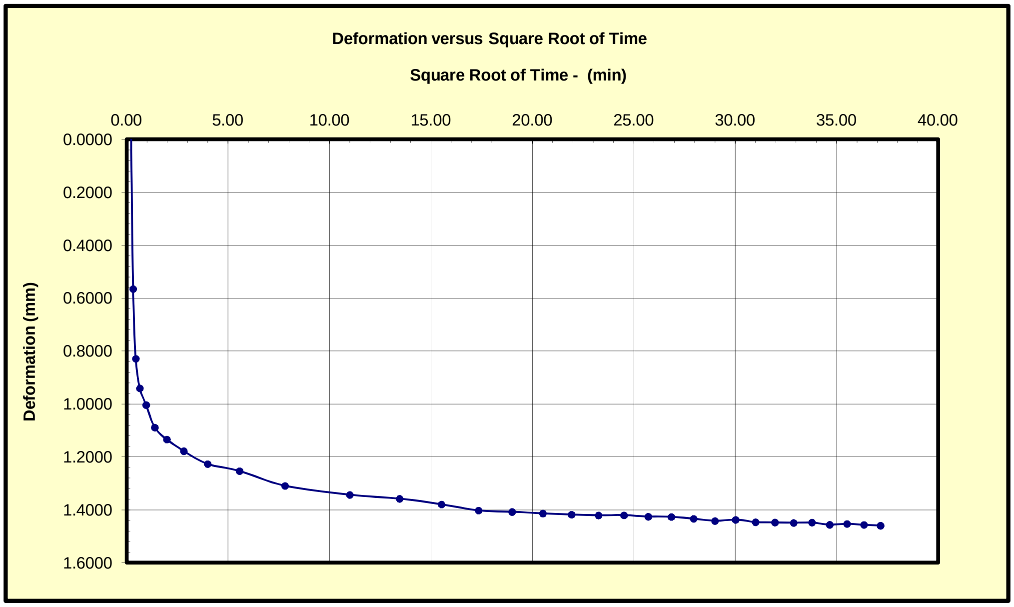
MEG TECHNICAL SERVICES

(A Division of MEG Consulting Limited)

One-Dimensional Consolidation (ASTM D 2435)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 7, 2014
Borehole:	BH6	Station:	1
Sample No.:	S4	Depth (m):	5.80
Consolidation Step:	12	Vertical Stress (kPa):	3064





Prepared By:	PC	Checked By:	PS	Approved By:	EP
Date:	May 7, 2014	Date:	May 7, 2014	Date:	May 7, 2014

MEG TECHNICAL SERVICES

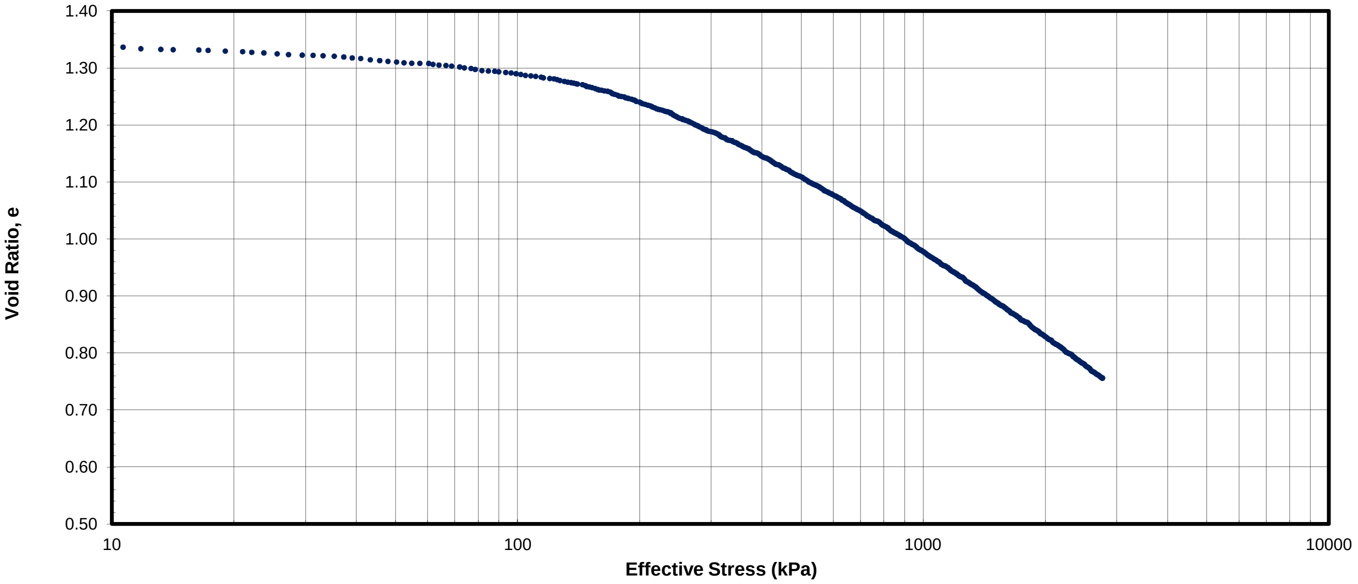
(A Division of MEG Consulting Limited)

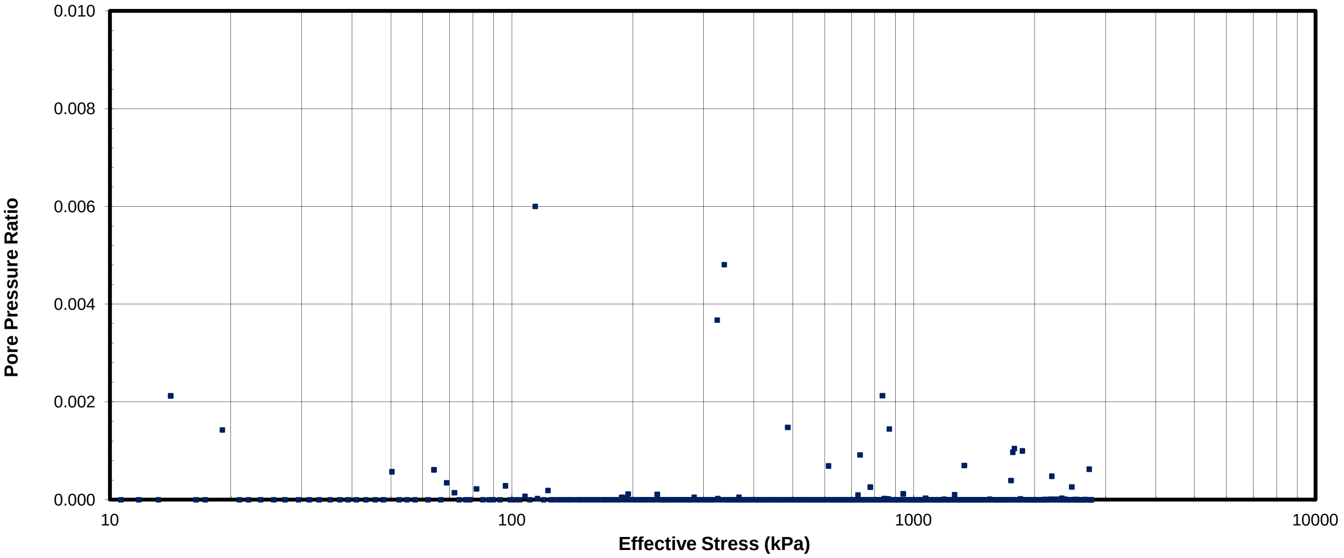
Constant Rate of Strain Consolidation (ASTM D4186)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 21, 2014
Borehole:	BC1	Station:	2
Sample No.:	BH3-S1	Depth (m):	8.10

Weight of Ring (g):	211.23	**Ring + Wet Weight (g):**	347.14	**Initial Void Ratio, e:**	1.35
Initial Height (mm):	25.41	**Ring + Dry Weight (g):**	304.80	**Height of Soil, Hs (mm):**	10.81
Diameter of Ring (mm):	63.50	**Water Content (%):**	45.2	**Height of Void, Hv (mm):**	14.60
Unit Weight (kN/m^3)	16.57	**Specific Gravity, Gs:**	2.73		

Loading Strain Rate (%/hr):	4	**Max Stress (kPa):**	2761	**Backpressure (kPa):**	408
Unloading Strain Rate (%/hr):	N/A	**Max Strain (%):**	25.3		





Prepared By:	PC	Checked By:	PS	Approved By:	EP
Date:	May 21, 2014	Date:	May 21, 2014	Date:	May 22, 2014

MEG TECHNICAL SERVICES

(A Division of MEG Consulting Limited)

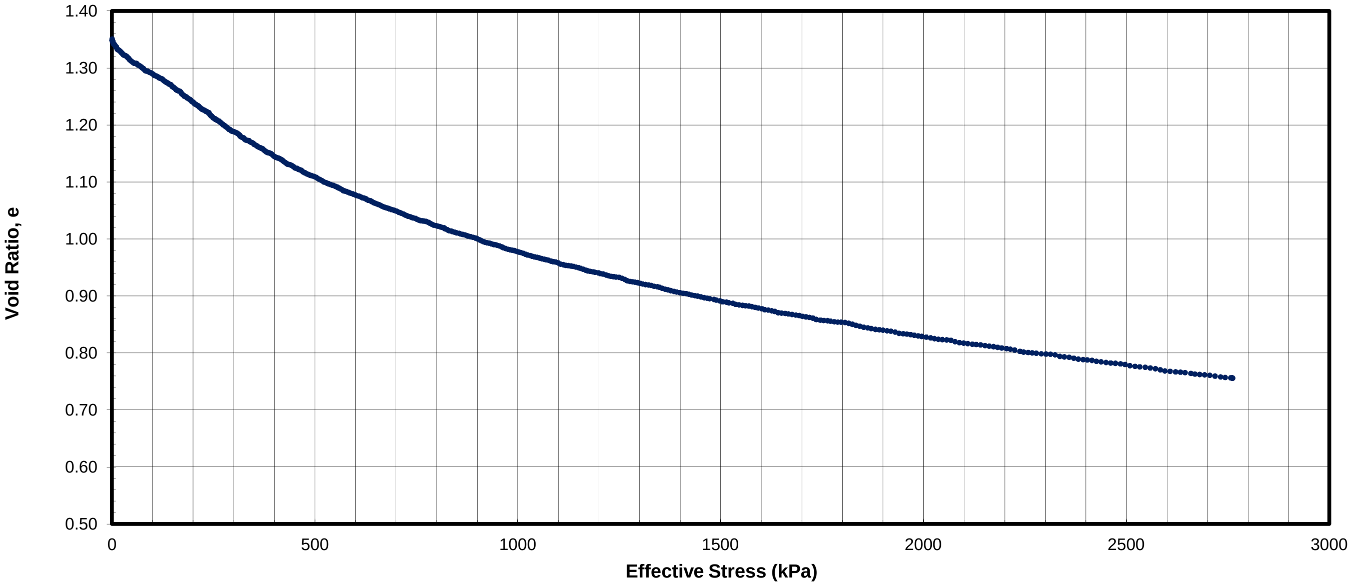
Marine + Earth MEG Geosciences

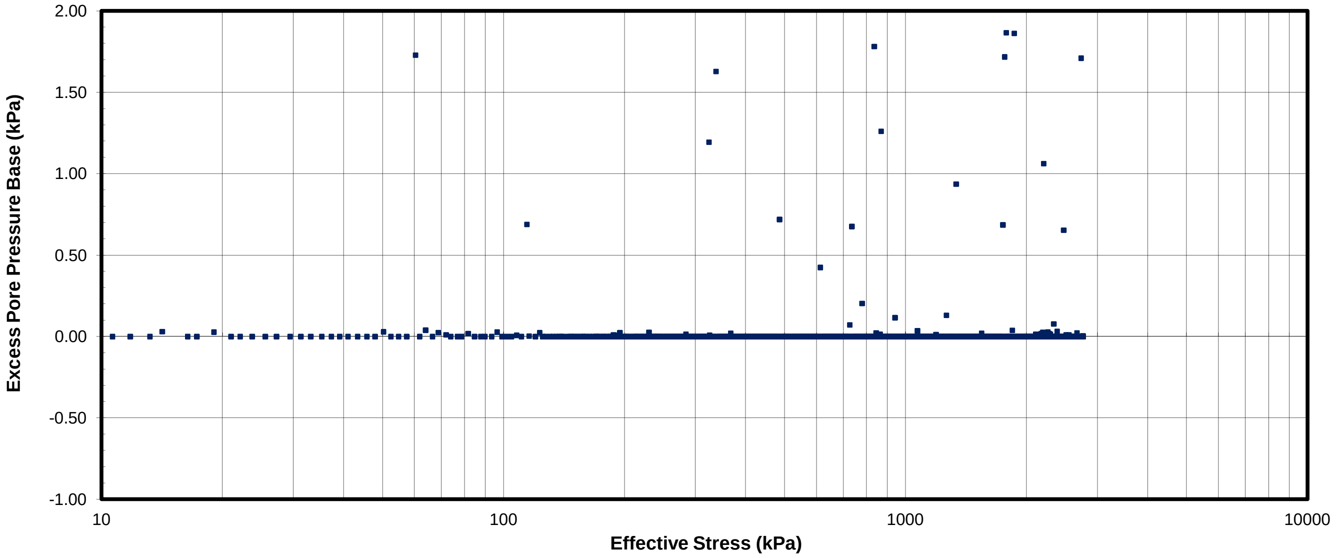
Constant Rate of Strain Consolidation (ASTM D4186)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 21, 2014
Borehole:	BC1	Station:	2
Sample No.:	BH3-S1	Depth (m):	8.10

Weight of Ring (g):	211.23	**Ring + Wet Weight (g):**	347.14	**Initial Void Ratio, e:**	1.35
Initial Height (mm):	25.41	**Ring + Dry Weight (g):**	304.80	**Height of Soil, Hs (mm):**	10.81
Diameter of Ring (mm):	63.50	**Water Content (%):**	45.2	**Height of Void, Hv (mm):**	14.60
Unit Weight (kN/m^3)	16.57	**Specific Gravity, Gs:**	2.73		

Loading Strain Rate (%/hr):	4	**Max Stress (kPa):**	2761	**Backpressure (kPa):**	408
Unloading Strain Rate (%/hr):	N/A	**Max Strain (%):**	25.3		





Prepared By:	PC	Checked By:	PS	Approved By:	EP
Date:	May 21, 2014	Date:	May 21, 2014	Date:	May 22, 2014

MEG TECHNICAL SERVICES

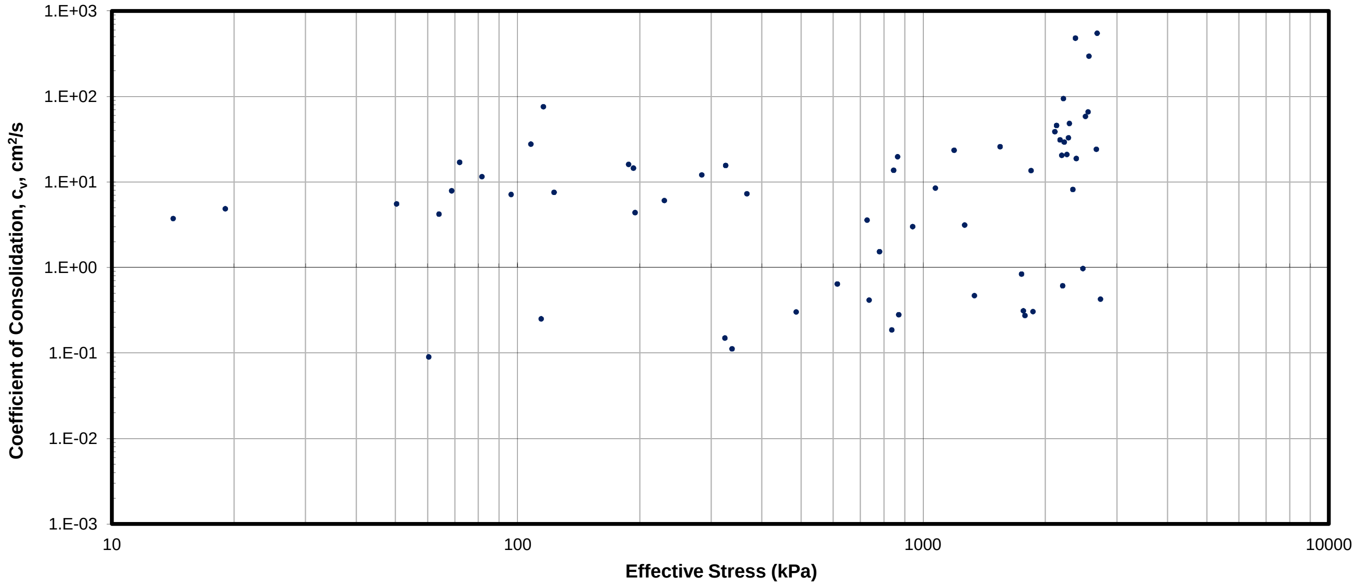
(A Division of MEG Consulting Limited)

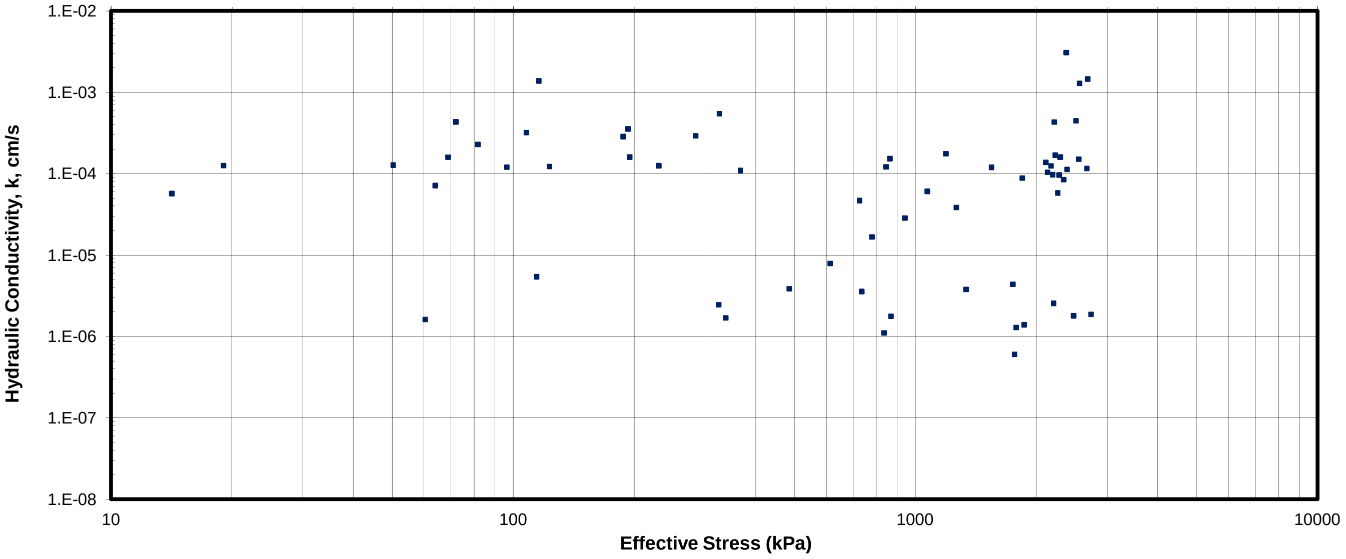
Constant Rate of Strain Consolidation (ASTM D4186)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	May 21, 2014
Borehole:	BC1	Station:	2
Sample No.:	BH3-S1	Depth (m):	8.10

Weight of Ring (g):	211.23	**Ring + Wet Weight (g):**	347.14	**Initial Void Ratio, e:**	1.35
Initial Height (mm):	25.41	**Ring + Dry Weight (g):**	304.80	**Height of Soil, Hs (mm):**	10.81
Diameter of Ring (mm):	63.50	**Water Content (%):**	45.2	**Height of Void, Hv (mm):**	14.60
Unit Weight (kN/m^3)	16.57	**Specific Gravity, Gs:**	2.73		

Loading Strain Rate (%/hr):	4	**Max Stress (kPa):**	2761	**Backpressure (kPa):**	408
Unloading Strain Rate (%/hr):	N/A	**Max Strain (%):**	25.3		





Prepared By:	PC	Checked By:	PS	Approved By:	EP
Date:	May 21, 2014	Date:	May 21, 2014	Date:	May 22, 2014

Static Strength Testing

MEG TECHNICAL SERVICES

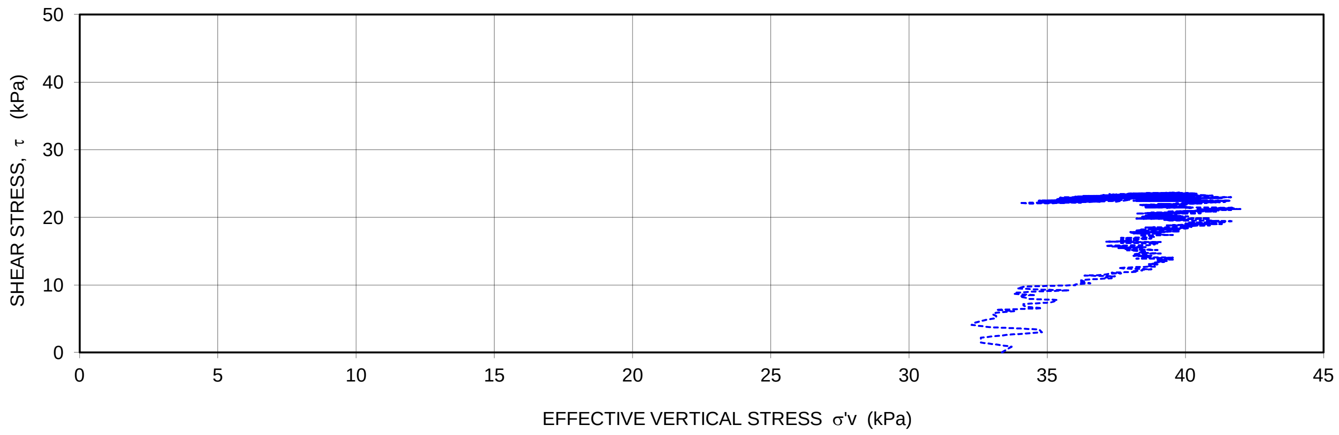
(A Division of MEG Consulting Limited)

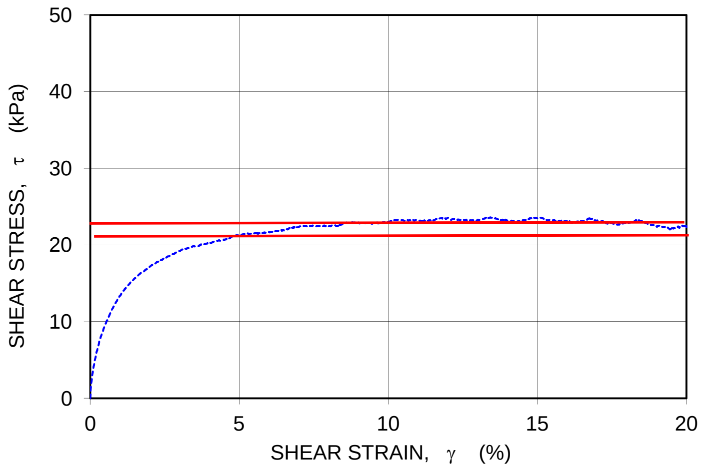
Marine + Earth
MEG
Geosciences

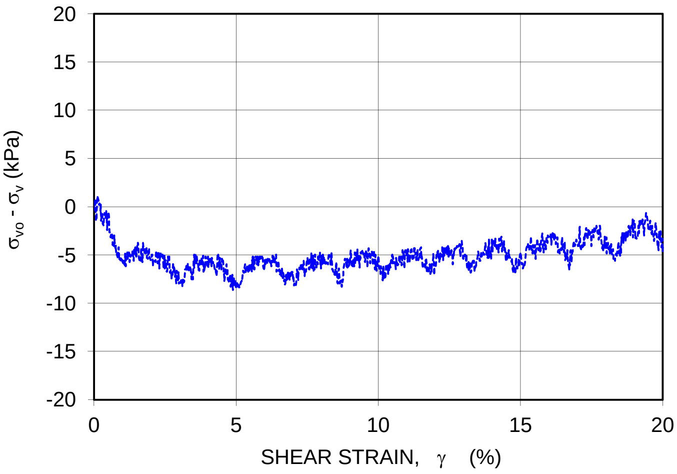
DIRECT SIMPLE SHEAR TEST (ASTM D 6528)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	June 5, 2014
Borehole:	BH-3	Depth (m):	4.69
Sample No.:	S-3	Station:	DSS1

Initial Height (mm):	23.6	**Weight of Specimen (g):**	139.92	**Initial Void Ratio, e_o:**	2.71
Diameter of Ring (mm):	73.2	**Total Unit Weight (kN/m^3):**	13.83	**Final Void Ratio, e_f:**	2.51
Specific Gravity, Gs:	2.57	**Dry Unit Weight (kN/m^3):**	6.78	**Natural Water Content (%):**	103.9
Final Water Content (%):	91.8	**Initial Degree of Saturation, Sr (%):**	98.3	**Final Degree of Saturation, Sr (%):**	93.8







Type of Test: Constant Volume				
Sample No.	Depth (m)	Total Unit Weight (kN/m^3)	Effective Vertical Stress, σ'v (kPa)	OCR
S-3	4.69	13.8	33.8	2.5

Comments: Sample sheared at 5% per hour

Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	June 5, 2014	Date:	June 5, 2014	Date:	June 6, 2014

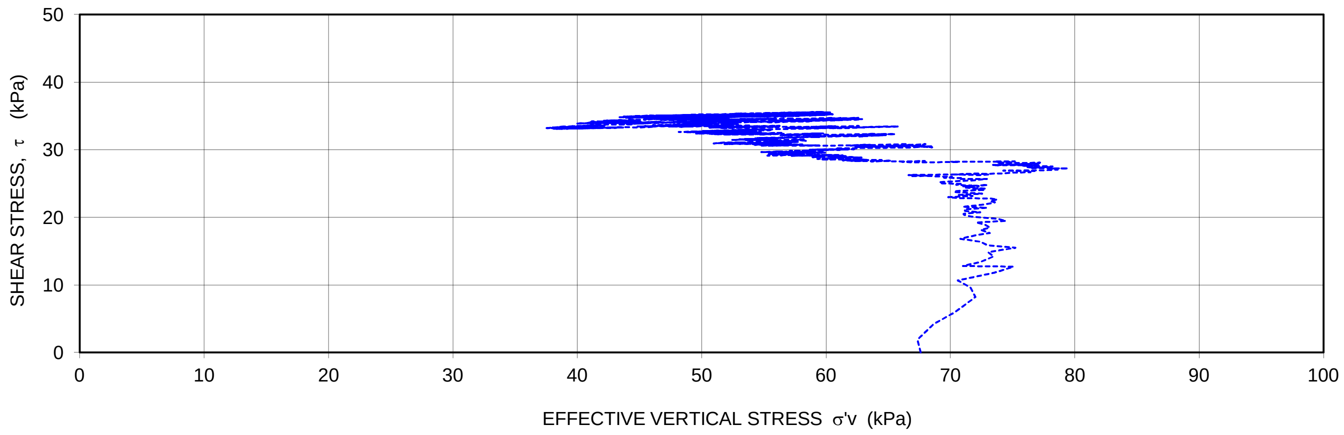
MEG TECHNICAL SERVICES

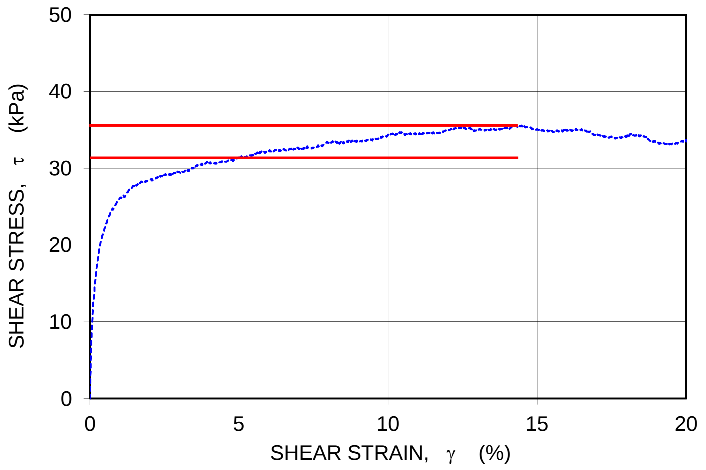
(A Division of MEG Consulting Limited)

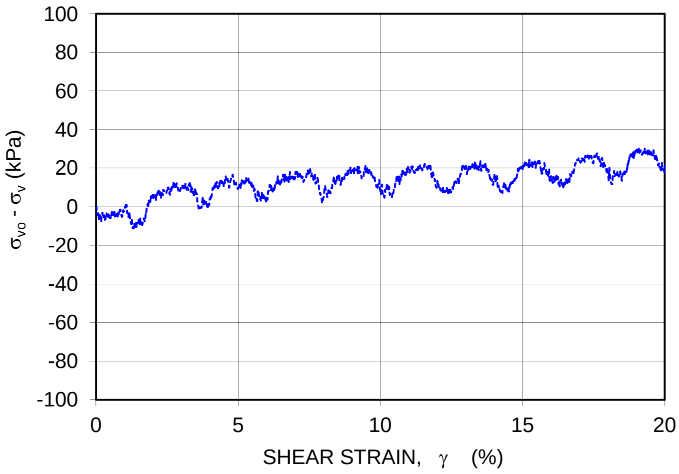
DIRECT SIMPLE SHEAR TEST (ASTM D 6528)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	June 5, 2014
Borehole:	BH-3	Depth (m):	8.91
Sample No.:	S-6	Station:	DSS2

Initial Height (mm):	23.3	**Weight of Specimen (g):**	159.12	**Initial Void Ratio, e_o:**	1.68
Diameter of Ring (mm):	73.2	**Total Unit Weight (kN/m^3):**	15.90	**Final Void Ratio, e_f:**	1.48
Specific Gravity, Gs:	2.73	**Dry Unit Weight (kN/m^3):**	10.00	**Natural Water Content (%):**	59.0
Final Water Content (%):	54.4	**Initial Degree of Saturation, Sr (%):**	96.1	**Final Degree of Saturation, Sr (%):**	99.9







Type of Test: Constant Volume

Sample No.	Depth (m)	Total Unit Weight (kN/m^3)	Effective Vertical Stress, σ'v (kPa)	OCR
S-6	8.91	15.9	67	1.9

Comments: Sample sheared at 5% per hour

Prepared By:	MF	Checked By:	PS	Approved By:	
Date:	June 5, 2014	Date:	June 5, 2014	Date:	

MEG TECHNICAL SERVICES

(A Division of MEG Consulting Limited)

Consolidated-Undrained Triaxial Compression Test
(ASTM D 4767)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	June 24, 2014
Borehole:	BH-3	Station:	10K
Sample No.:	S-8	Depth (m):	9.60

Specimen Data

Diameter (mm)	72.25
Height (mm)	151.60
Weight of container + sample (g)	1032.04
Weight of container (g)	0
Total Unit Weight (kN/m^3)	16.29
Dry Unit Weight (kN/m3)	10.30

Water Content

	Before Saturation	After Shear
Tin No.	16	H1
Weight of tin (g)	23.93	196.74
Tin + Wet weight (g)	107.19	753.17
Tin + Dry weight (g)	76.6	560.49
Water Content (%)	58.1	53.0

Consolidation

Specific Gravity, Gs:	2.65
Initial vertical effective stress, σ'_1 (kPa)	13.9
Final vertical effective stress, σ'_1 (kPa)	78.5
Initial effective isotropic stress, σ'_3 (kPa)	13.9
Final effective isotropic stress, σ'_3 (kPa)	77.4
Pore Pressure (kPa)	554.0
Ratio of horizontal to vertical stress, K	1.00
Volume change during consolidation, ΔV_c (cm^3)	15.33
Initial height of specimen (cm)	15.16
Initial area of specimen (cm^2)	41.00
Initial volume of specimen (cm^3)	621.63
Initial void ratio, e_i	1.52

Saturation

Vertical Seating Pressure (kPa)	13.8
Cell Pressure, σ_3 (kPa)	565.4
Back Pressure (kPa)	551.6
Effective Stress (kPa)	13.8
Pore pressure coefficient, B	1.00

Shear

Initial vertical effective stress, σ'_1 (kPa)	76.6
Initial Isotropic effective stress, σ'_3 (kPa)	75.4
Initial Pore Pressure (kPa)	554.1
Strain rate (%/hr)	0.4

Comments / Observations: Assumed Gs of 2.65 as no consolidation or Gs test performed

Performed By:	MF	Checked By:	PS	Approved By:	EP
Date:	June 24, 2014	Date:	June 25, 2014	Date:	June 26, 2014

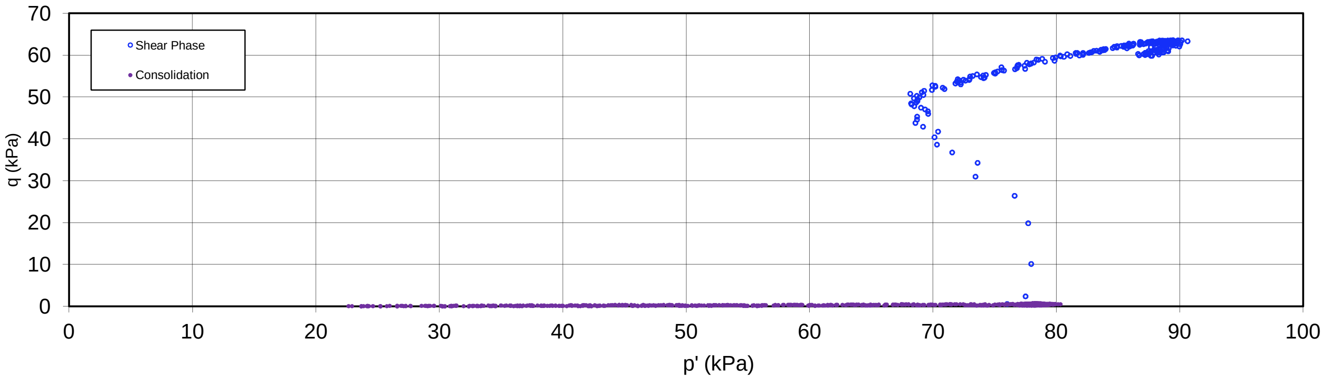
MEG TECHNICAL SERVICES

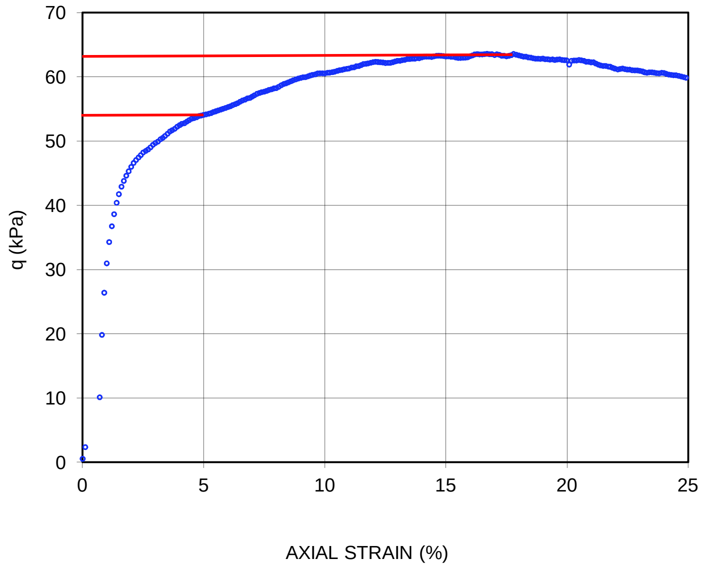
(A Division of MEG Consulting Limited)

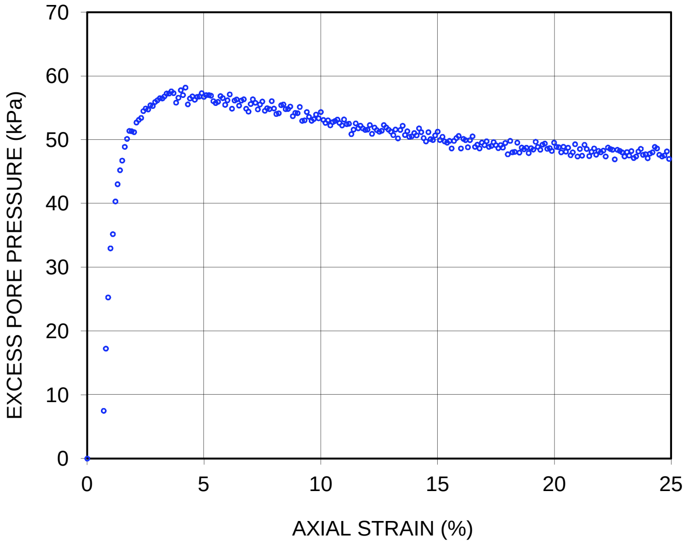
Consolidated-Undrained Triaxial Compression Test
(ASTM D 4767)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	June 24, 2014
Borehole:	BH-3	Depth (m):	9.60
Sample No.:	S-8		

Initial Height (mm):	151.6	**Weight of Specimen (g):**	1032.04	**Initial Void Ratio, e_i:**	1.52
Initial Diameter (mm):	72.3	**Total Unit Weight (kN/m^3):**	16.29	**Natural Water Content (%):**	58.1
Specific Gravity, Gs:	2.65	**Dry Unit Weight (kN/m^3):**	10.30		
Final Water Content (%):	53.0	**Pore pressure coefficient, B:**	1.00		







Comments: Assumed Gs of 2.65 as no consolidation or Gs test performed

Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	June 24, 2014	Date:	June 25, 2014	Date:	June 26, 2014

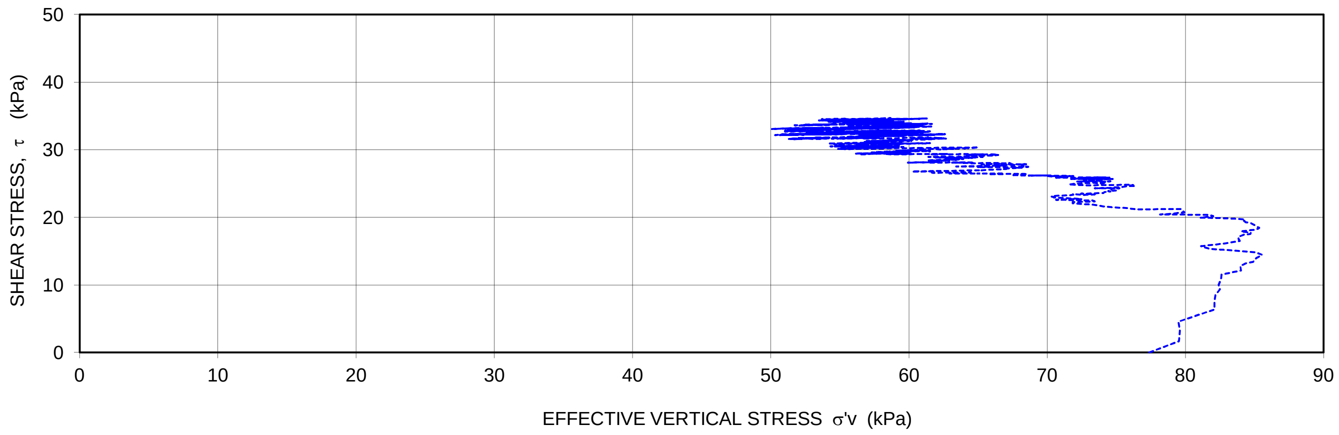
MEG TECHNICAL SERVICES
(A Division of MEG Consulting Limited)

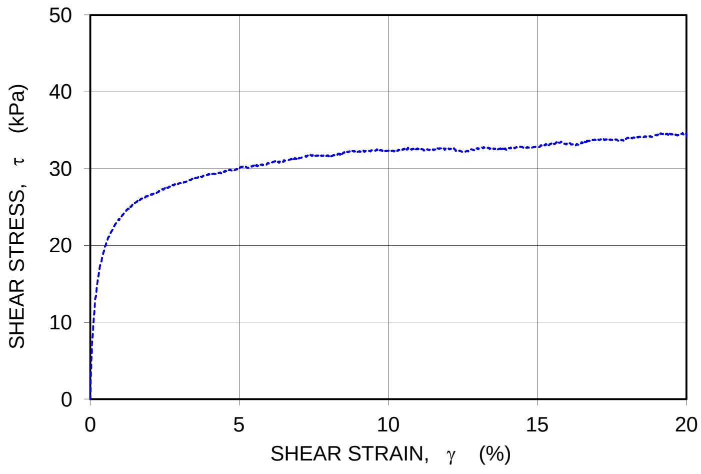
Marine + Earth
MEG
Geosciences

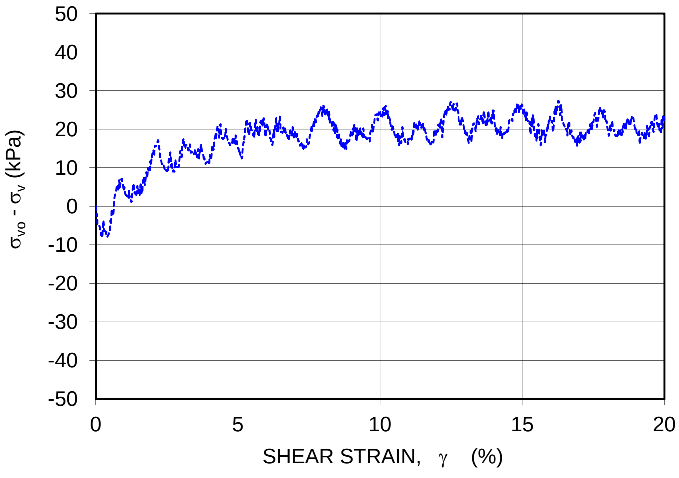
DIRECT SIMPLE SHEAR TEST (ASTM D 6528)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	July 7, 2014
Borehole:	BH-3	Depth (m):	10.60
Sample No.:	S-8	Station:	DSS2

Initial Height (mm):	23.4	**Weight of Specimen (g):**	150.29	**Initial Void Ratio, e_o:**	1.70
Diameter of Ring (mm):	72.9	**Total Unit Weight (kN/m^3):**	15.08	**Final Void Ratio, e_f:**	1.53
Specific Gravity, Gs:	2.70	**Dry Unit Weight (kN/m^3):**	9.82	**Natural Water Content (%):**	53.5
Final Water Content (%):	55.1	**Initial Degree of Saturation, Sr (%):**	85.1	**Final Degree of Saturation, Sr (%):**	97.0







Type of Test: Constant Volume				
Sample No.	Depth (m)	Total Unit Weight (kN/m^3)	Effective Vertical Stress, σ'v (kPa)	Test OCR
S-8	10.60	15.1	77.6	1.6

Comments: Sample sheared at 5% per hour
Assumed Gs of 2.70, no consolidation or Gs test performed

Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	July 7, 2014	Date:	July 8, 2014	Date:	July 9, 2014

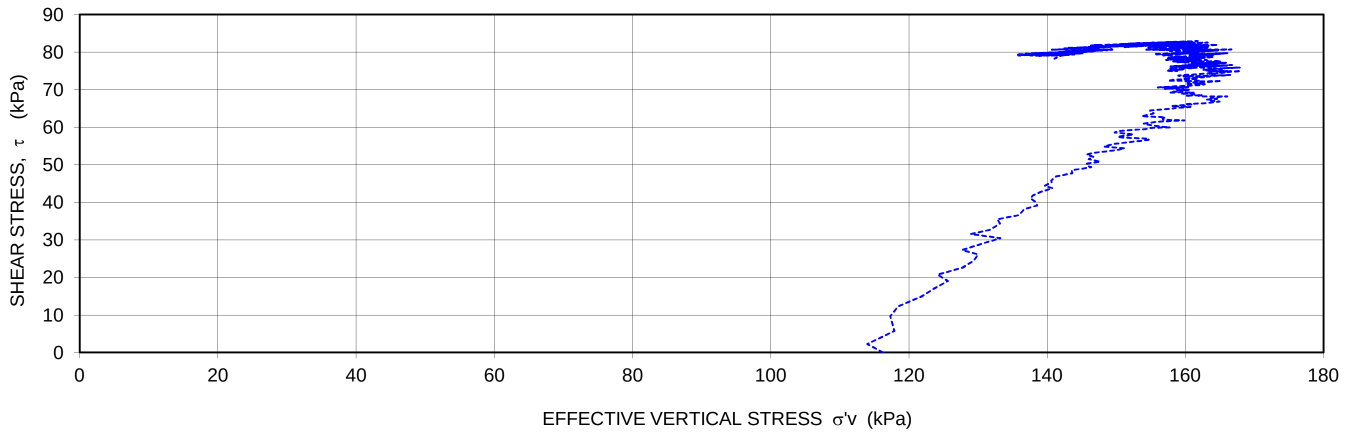
MEG TECHNICAL SERVICES

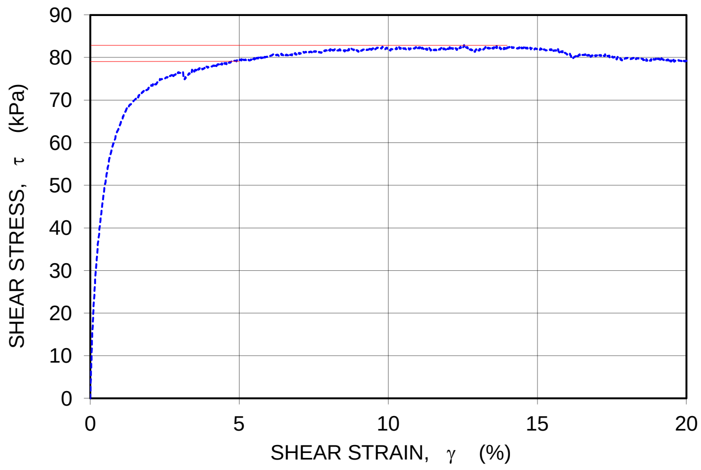
(A Division of MEG Consulting Limited)

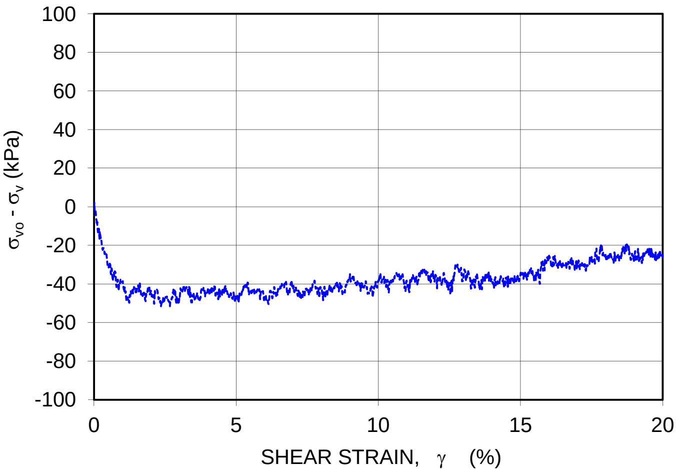
DIRECT SIMPLE SHEAR TEST (ASTM D 6528)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	July 14, 2014
Borehole:	BH-3	Depth (m):	10.00
Sample No.:	S-8	Station:	DSS2

Initial Height (mm):	23.4	**Weight of Specimen (g):**	144.44	**Initial Void Ratio, e_o:**	1.67
Diameter of Ring (mm):	72.9	**Total Unit Weight (kN/m^3):**	14.49	**Final Void Ratio, e_f:**	1.30
Specific Gravity, Gs:	2.70	**Dry Unit Weight (kN/m^3):**	9.93	**Natural Water Content (%):**	45.9
Final Water Content (%):	43.0	**Initial Degree of Saturation, Sr (%):**	74.4	**Final Degree of Saturation, Sr (%):**	89.3







Type of Test: Constant Volume

Sample No.	Depth (m)	Total Unit Weight (kN/m^3)	Effective Vertical Stress, σ'v (kPa)	Test OCR
S-8	10.00	14.5	116.33	4

Comments: Sample loaded to 3 times the estimated preconsolidation stress and unloaded to establish an OCR of 4. Sample sheared at 5% strain per hour. Assumed Gs of 2.70

Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	July 14, 2014	Date:	July 14, 2014	Date:	July 14, 2014

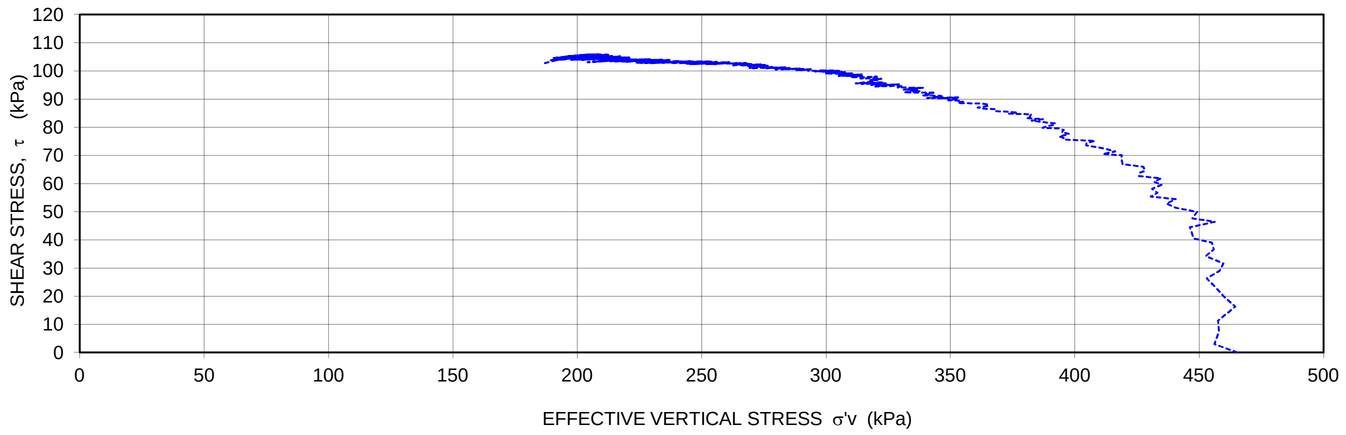
MEG TECHNICAL SERVICES

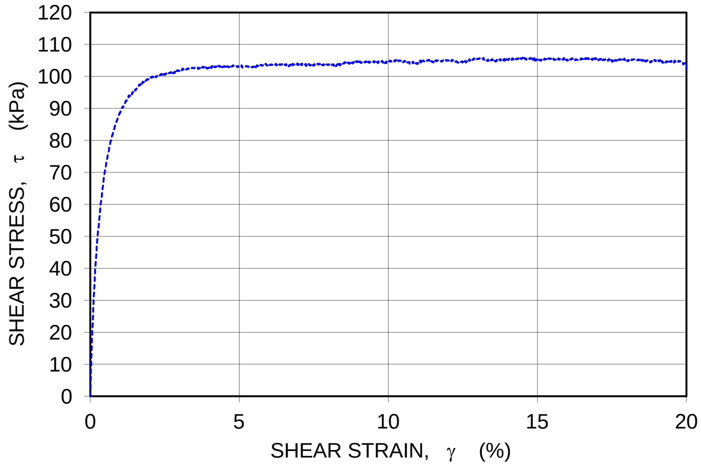
(A Division of MEG Consulting Limited)

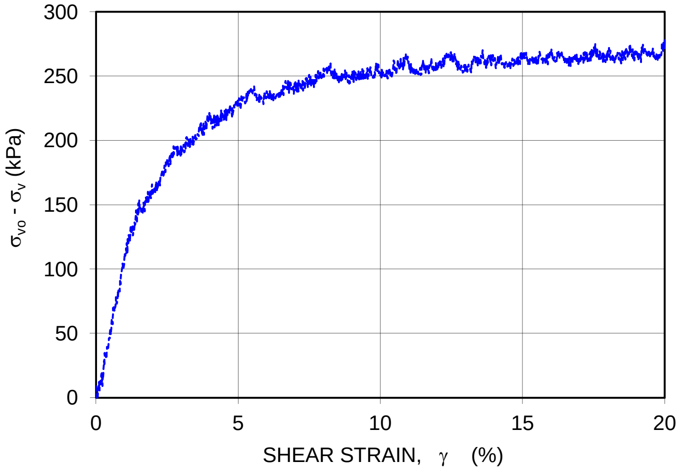
DIRECT SIMPLE SHEAR TEST (ASTM D 6528)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	July 7, 2014
Borehole:	BH-3	Depth (m):	10.04
Sample No.:	S-8	Station:	DSS2

Initial Height (mm):	23.4	**Weight of Specimen (g):**	152.59	**Initial Void Ratio, e_o:**	1.54
Diameter of Ring (mm):	72.9	**Total Unit Weight (kN/m^3):**	15.31	**Final Void Ratio, e_f:**	1.15
Specific Gravity, Gs:	2.70	**Dry Unit Weight (kN/m^3):**	10.42	**Natural Water Content (%):**	46.9
Final Water Content (%):	41.4	**Initial Degree of Saturation, Sr (%):**	82.1	**Final Degree of Saturation, Sr (%):**	97.0







Type of Test: Constant Volume				
Sample No.	Depth (m)	Total Unit Weight (kN/m^3)	Effective Vertical Stress, σ'v (kPa)	Test OCR
S-8	10.04	15.3	465.3	1.0

Comments: Sample loaded to 3 times the estimated pre-consolidation pressure and sheared at 5% per hour. Assumed Gs of 2.70, no consolidation or Gs test performed

Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	July 7, 2014	Date:	July 8, 2014	Date:	July 9, 2014

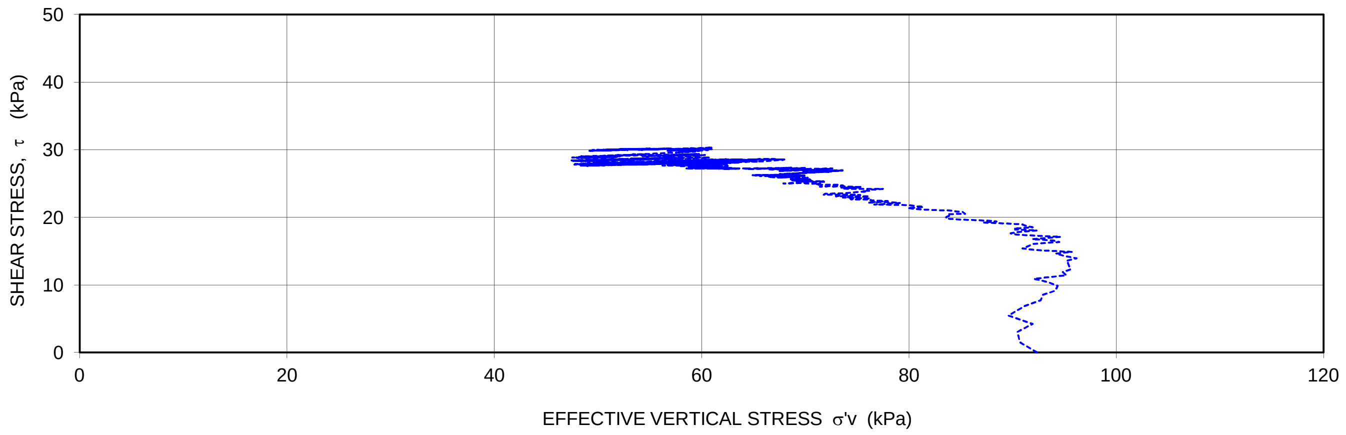
MEG TECHNICAL SERVICES

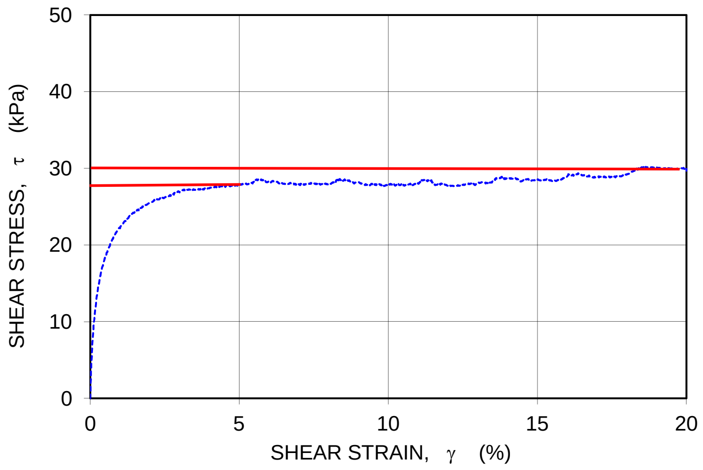
(A Division of MEG Consulting Limited)

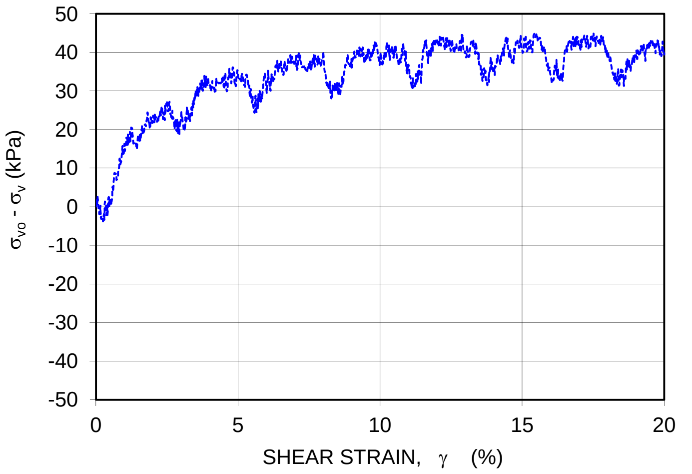
DIRECT SIMPLE SHEAR TEST (ASTM D 6528)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	June 23, 2014
Borehole:	BH-3	Depth (m):	11.48
Sample No.:	S-9	Station:	DSS2

Initial Height (mm):	23.6	**Weight of Specimen (g):**	146.62	**Initial Void Ratio, e_o:**	2.02
Diameter of Ring (mm):	73.1	**Total Unit Weight (kN/m^3):**	14.52	**Final Void Ratio, e_f:**	1.83
Specific Gravity, Gs:	2.70	**Dry Unit Weight (kN/m^3):**	8.78	**Natural Water Content (%):**	65.3
Final Water Content (%):	67.7	**Initial Degree of Saturation, Sr (%):**	87.4	**Final Degree of Saturation, Sr (%):**	99.7







Type of Test: Constant Volume				
Sample No.	Depth (m)	Total Unit Weight (kN/m^3)	Effective Vertical Stress, σ'v (kPa)	OCR
S-9	11.48	14.5	91.6	1.3

Comments: Sample sheared at 5% strain per hour
assumed Gs of 2.70, no consolidation or Gs test performed

Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	June 23, 2014	Date:	June 23, 2014	Date:	June 24, 2014

MEG TECHNICAL SERVICES

(A Division of MEG Consulting Limited)

Consolidated-Undrained Triaxial Compression Test
(ASTM D 4767)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	June 24, 2014
Borehole:	BH-4	Station:	1
Sample No.:	S-1	Depth (m):	3.34

Specimen Data

Diameter (mm)	72.42
Height (mm)	157.77
Weight of container + sample (g)	1117
Weight of container (g)	0
Total Unit Weight (kN/m^3)	16.86
Dry Unit Weight (kN/m3)	11.45

Water Content

	Before Saturation	After Shear
Tin No.	27	C31
Weight of tin (g)	23.55	196.55
Tin + Wet weight (g)	42.88	798.07
Tin + Dry weight (g)	36.67	602.25
Water Content (%)	47.3	48.3

Consolidation

Specific Gravity, Gs:	2.65
Initial vertical effective stress, σ'_1 (kPa)	13.5
Final vertical effective stress, σ'_1 (kPa)	33.6
Initial effective isotropic stress, σ'_3 (kPa)	13.5
Final effective isotropic stress, σ'_3 (kPa)	33.1
Pore Pressure (kPa)	555.5
Ratio of horizontal to vertical stress, K	1.00
Volume change during consolidation, ΔV_c (cm^3)	6.46
Initial height of specimen (cm)	15.78
Initial area of specimen (cm^2)	41.19
Initial volume of specimen (cm^3)	649.79
Initial void ratio, e_i	1.27

Saturation

Vertical Seating Pressure (kPa)	13.8
Cell Pressure, σ_3 (kPa)	565.4
Back Pressure (kPa)	551.6
Effective Stress (kPa)	13.8
Pore pressure coefficient, B	1.00

Shear

Initial vertical effective stress, σ'_1 (kPa)	35.0
Initial Isotropic effective stress, σ'_3 (kPa)	34.0
Initial Pore Pressure (kPa)	552.1
Strain rate (%/hr)	0.4

Comments / Observations: ______________________________

Performed By:	MF	Checked By:	PS	Approved By:	EP
Date:	June 24, 2014	Date:	June 25, 2014	Date:	June 26, 2014

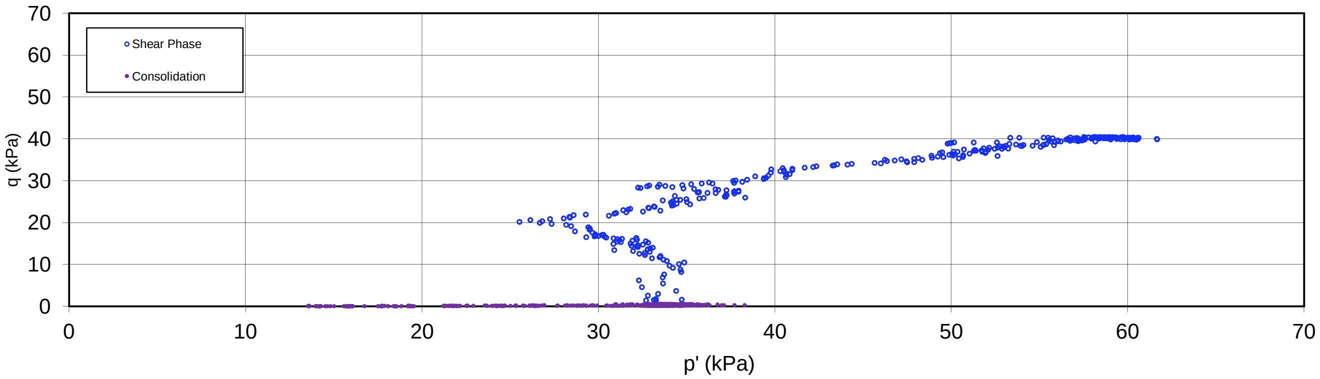
MEG TECHNICAL SERVICES

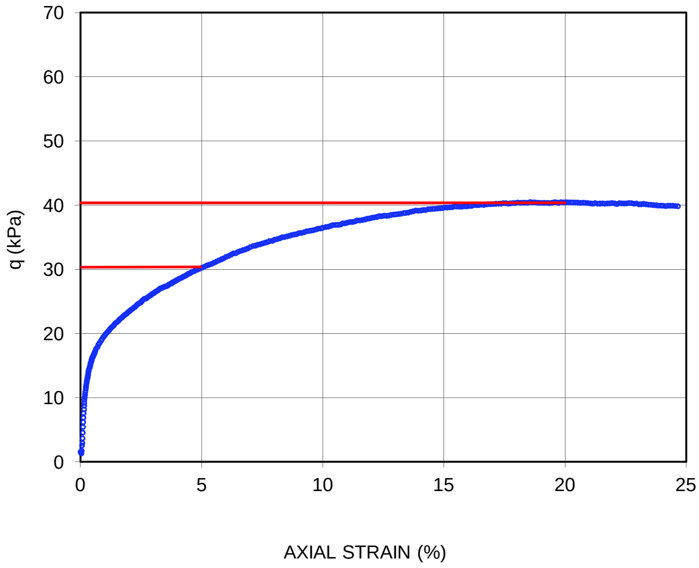
(A Division of MEG Consulting Limited)

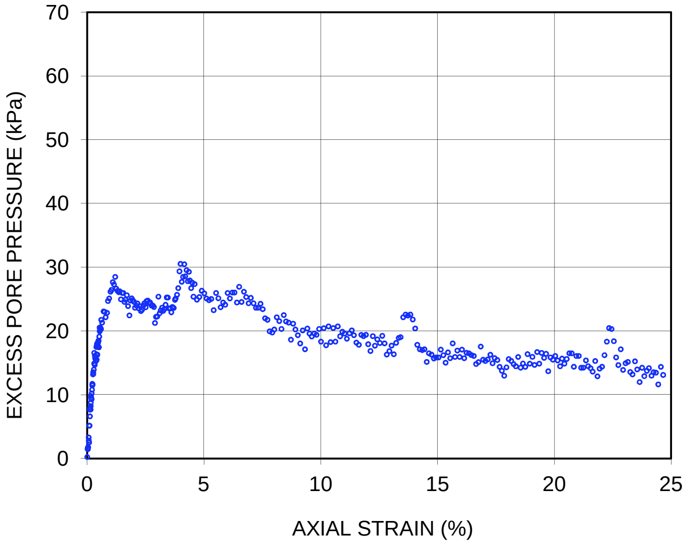
Consolidated-Undrained Triaxial Compression Test
(ASTM D 4767)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	June 24, 2014
Borehole:	BH-4	Depth (m):	3.34
Sample No.:	S-1		

Initial Height (mm):	157.8	**Weight of Specimen (g):**	1117.00	**Initial Void Ratio, e_i:**	1.27
Initial Diameter (mm):	72.4	**Total Unit Weight (kN/m^3):**	16.86	**Natural Water Content (%):**	47.3
Specific Gravity, Gs:	2.65	**Dry Unit Weight (kN/m^3):**	11.45		
Final Water Content (%):	48.3	**Pore pressure coefficient, B:**	1.00		







Comments: ______________________________

Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	June 24, 2014	Date:	June 25, 2014	Date:	June 26, 2014

MEG TECHNICAL SERVICES

(A Division of MEG Consulting Limited)

Consolidated-Undrained Triaxial Compression Test
(ASTM D 4767)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	June 24, 2014
Borehole:	BH-4	Station:	1
Sample No.:	S-1	Depth (m):	3.34

Specimen Data

Diameter (mm)	72.42
Height (mm)	157.77
Weight of container + sample (g)	1117
Weight of container (g)	0
Total Unit Weight (kN/m^3)	16.86
Dry Unit Weight (kN/m3)	11.45

Water Content

	Before Saturation	After Shear
Tin No.	27	C31
Weight of tin (g)	23.55	196.55
Tin + Wet weight (g)	42.88	798.07
Tin + Dry weight (g)	36.67	602.25
Water Content (%)	47.3	48.3

Consolidation

Specific Gravity, Gs:	2.65
Initial vertical effective stress, σ'_1 (kPa)	13.5
Final vertical effective stress, σ'_1 (kPa)	33.6
Initial effective isotropic stress, σ'_3 (kPa)	13.5
Final effective isotropic stress, σ'_3 (kPa)	33.1
Pore Pressure (kPa)	555.5
Ratio of horizontal to vertical stress, K	1.00
Volume change during consolidation, ΔV_c (cm^3)	6.46
Initial height of specimen (cm)	15.78
Initial area of specimen (cm^2)	41.19
Initial volume of specimen (cm^3)	649.79
Initial void ratio, e_i	1.27

Saturation

Vertical Seating Pressure (kPa)	13.8
Cell Pressure, σ_3 (kPa)	565.4
Back Pressure (kPa)	551.6
Effective Stress (kPa)	13.8
Pore pressure coefficient, B	1.00

Shear

Initial vertical effective stress, σ'_1 (kPa)	35.0
Initial Isotropic effective stress, σ'_3 (kPa)	34.0
Initial Pore Pressure (kPa)	552.1
Strain rate (%/hr)	0.4

Comments / Observations: ____________________

Performed By:	MF	Checked By:	PS	Approved By:	EP
Date:	June 24, 2014	Date:	June 25, 2014	Date:	June 26, 2014

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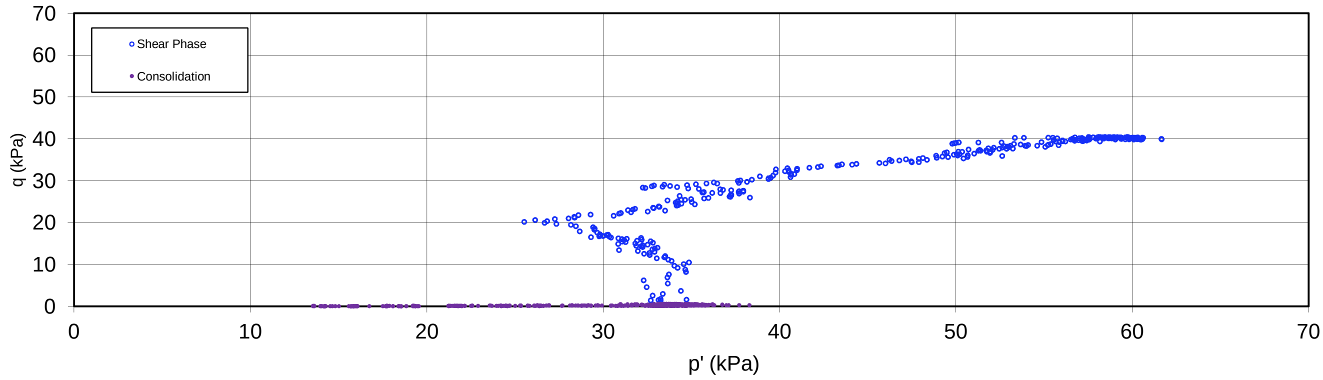
MEG TECHNICAL SERVICES

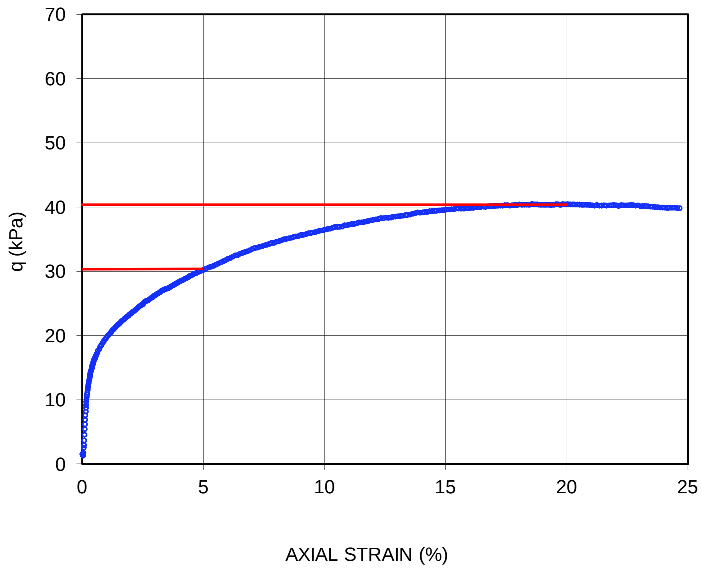
(A Division of MEG Consulting Limited)

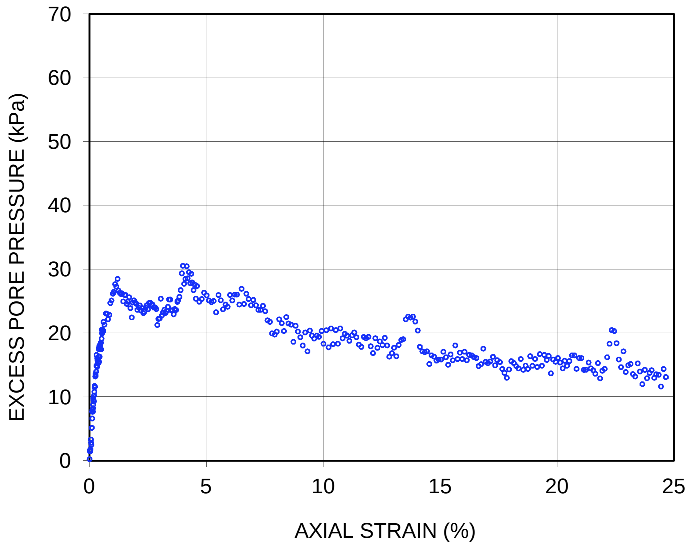
Consolidated-Undrained Triaxial Compression Test
(ASTM D 4767)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	June 24, 2014
Borehole:	BH-4	Depth (m):	3.34
Sample No.:	S-1		

Initial Height (mm):	157.8	**Weight of Specimen (g):**	1117.00	**Initial Void Ratio, e_i:**	1.27
Initial Diameter (mm):	72.4	**Total Unit Weight (kN/m^3):**	16.86	**Natural Water Content (%):**	47.3
Specific Gravity, Gs:	2.65	**Dry Unit Weight (kN/m^3):**	11.45		
Final Water Content (%):	48.3	**Pore pressure coefficient, B:**	1.00		







Comments: ______________________________

Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	June 24, 2014	Date:	June 25, 2014	Date:	June 26, 2014

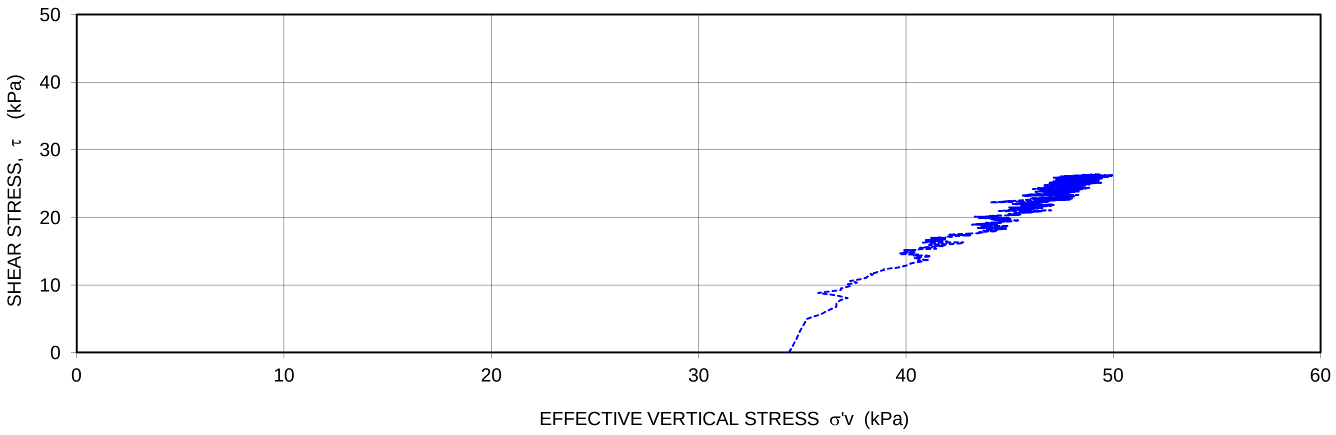
MEG TECHNICAL SERVICES

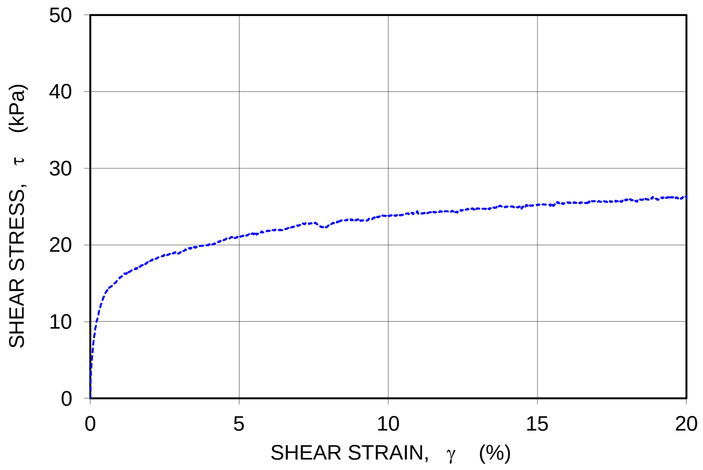
(A Division of MEG Consulting Limited)

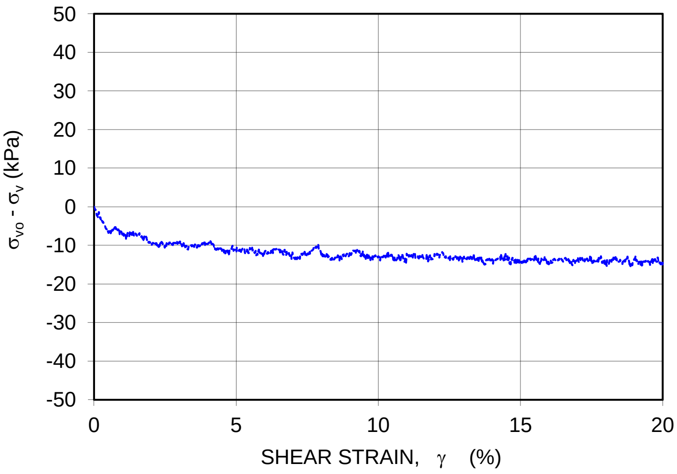
DIRECT SIMPLE SHEAR TEST (ASTM D 6528)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	July 7, 2014
Borehole:	BH-4	Depth (m):	3.32
Sample No.:	S-1	Station:	DSS1

Initial Height (mm):	23.5	**Weight of Specimen (g):**	166.30	**Initial Void Ratio, e_o:**	1.36
Diameter of Ring (mm):	73.3	**Total Unit Weight (kN/m^3):**	16.45	**Final Void Ratio, e_f:**	1.23
Specific Gravity, Gs:	2.65	**Dry Unit Weight (kN/m^3):**	10.98	**Natural Water Content (%):**	49.8
Final Water Content (%):	44.5	**Initial Degree of Saturation, Sr (%):**	96.6	**Final Degree of Saturation, Sr (%):**	95.6







Type of Test: Constant Volume				
Sample No.	Depth (m)	Total Unit Weight (kN/m^3)	Effective Vertical Stress, σ'v (kPa)	Test OCR
S-1	3.32	16.5	34.1	3.2

Comments: Sample sheared at 5% strain per hour

Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	July 7, 2014	Date:	July 8, 2014	Date:	July 9, 2014

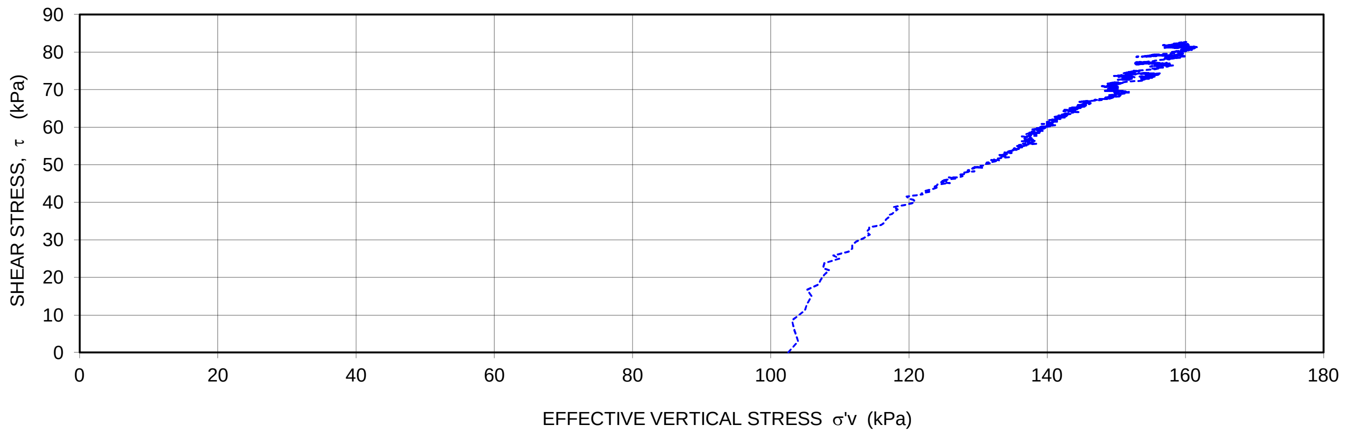
MEG TECHNICAL SERVICES

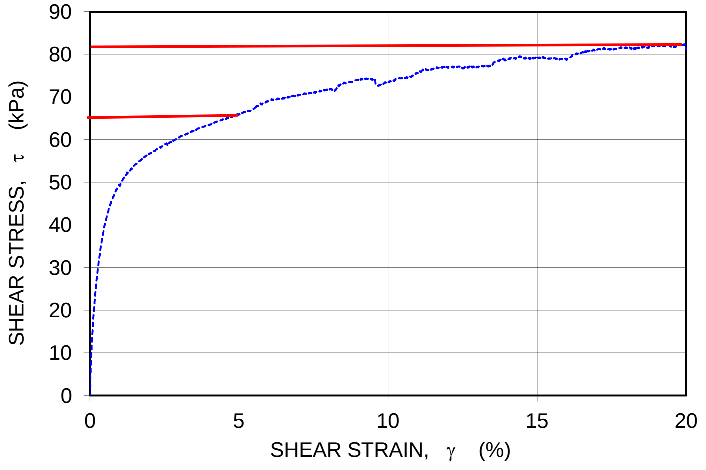
(A Division of MEG Consulting Limited)

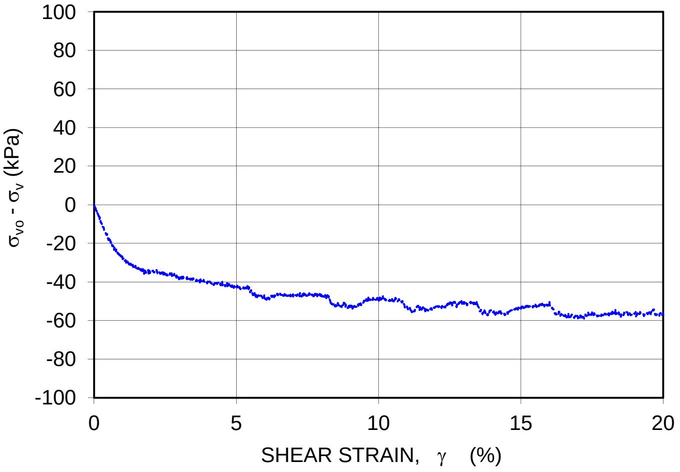
DIRECT SIMPLE SHEAR TEST (ASTM D 6528)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	July 14, 2014
Borehole:	BH-4	Depth (m):	3.26
Sample No.:	S-1	Station:	DSS1

Initial Height (mm):	23.5	**Weight of Specimen (g):**	167.14	**Initial Void Ratio, e_o:**	1.26
Diameter of Ring (mm):	73.3	**Total Unit Weight (kN/m^3):**	16.53	**Final Void Ratio, e_f:**	1.03
Specific Gravity, Gs:	2.65	**Dry Unit Weight (kN/m^3):**	11.50	**Natural Water Content (%):**	43.8
Final Water Content (%):	39.0	**Initial Degree of Saturation, Sr (%):**	92.1	**Final Degree of Saturation, Sr (%):**	100.0







Type of Test: Constant Volume				
Sample No.	Depth (m)	Total Unit Weight (kN/m^3)	Effective Vertical Stress, σ'v (kPa)	Test OCR
S-1	3.26	16.5	102.75	4

Comments: Sample loaded to 3 times the estimated preconsolidation stress and unloaded to establish an OCR of 4. Sample sheared at 5% strain per hour.

Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	July 14, 2014	Date:	July 14, 2014	Date:	July 14, 2014

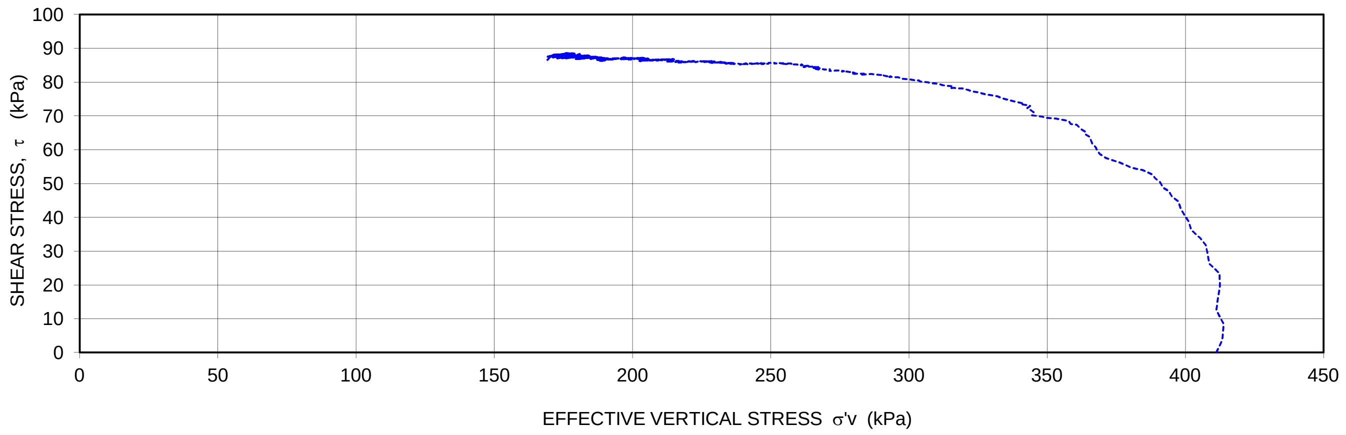
MEG TECHNICAL SERVICES

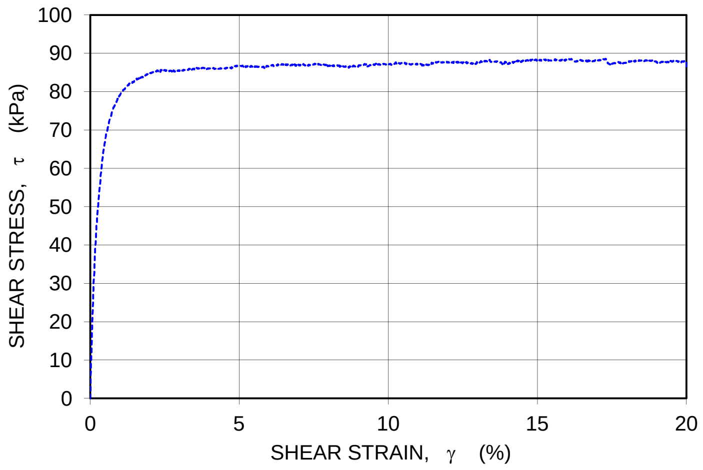
(A Division of MEG Consulting Limited)

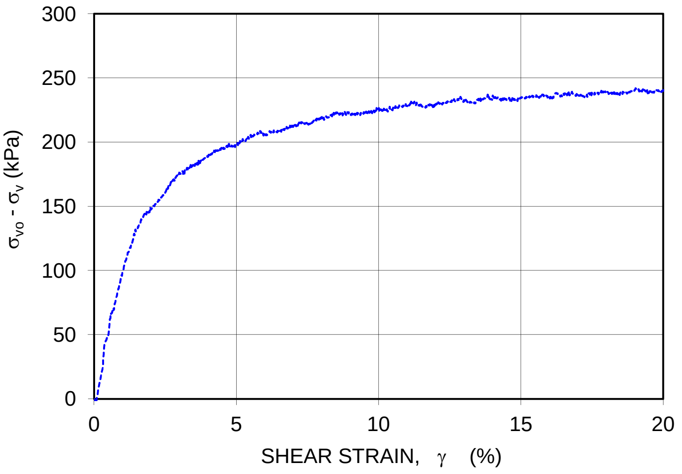
DIRECT SIMPLE SHEAR TEST (ASTM D 6528)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	July 8, 2014
Borehole:	BH-4	Depth (m):	3.30
Sample No.:	S-1	Station:	DSS1

Initial Height (mm):	23.5	**Weight of Specimen (g):**	158.24	**Initial Void Ratio, e_o:**	1.44
Diameter of Ring (mm):	73.3	**Total Unit Weight (kN/m^3):**	15.65	**Final Void Ratio, e_f:**	1.10
Specific Gravity, Gs:	2.65	**Dry Unit Weight (kN/m^3):**	10.65	**Natural Water Content (%):**	46.9
Final Water Content (%):	40.3	**Initial Degree of Saturation, Sr (%):**	86.4	**Final Degree of Saturation, Sr (%):**	97.0







Type of Test: Constant Volume				
Sample No.	Depth (m)	Total Unit Weight (kN/m^3)	Effective Vertical Stress, σ'v (kPa)	Test OCR
S-1	3.30	15.7	411	1.0

Comments: Sample loaded to 3 times the estimated pre-consolidation pressure and sheared at 5% per hour.

Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	July 8, 2014	Date:	July 9, 2014	Date:	July 9, 2014

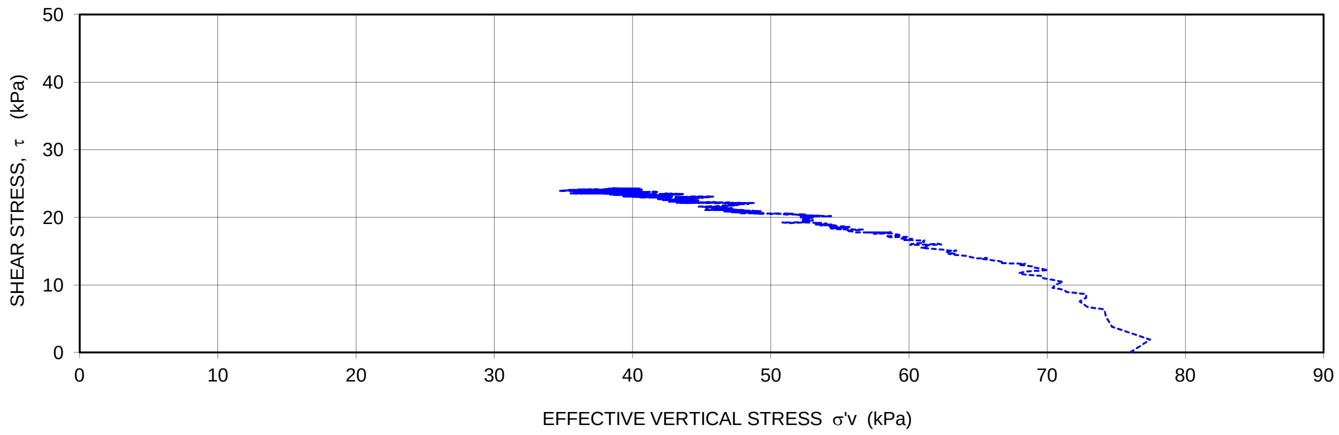
MEG TECHNICAL SERVICES

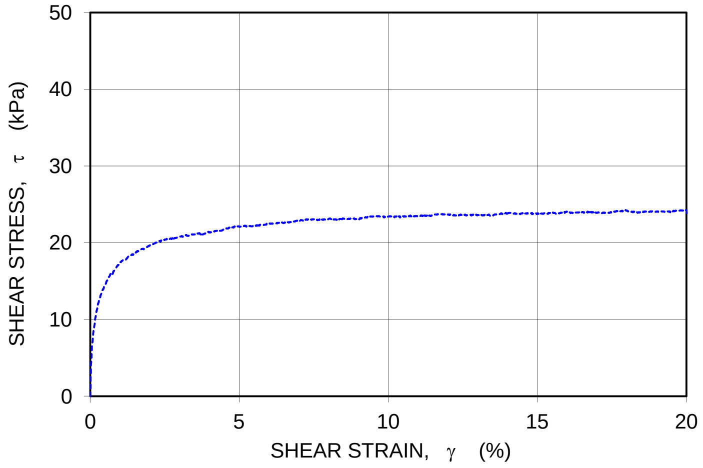
(A Division of MEG Consulting Limited)

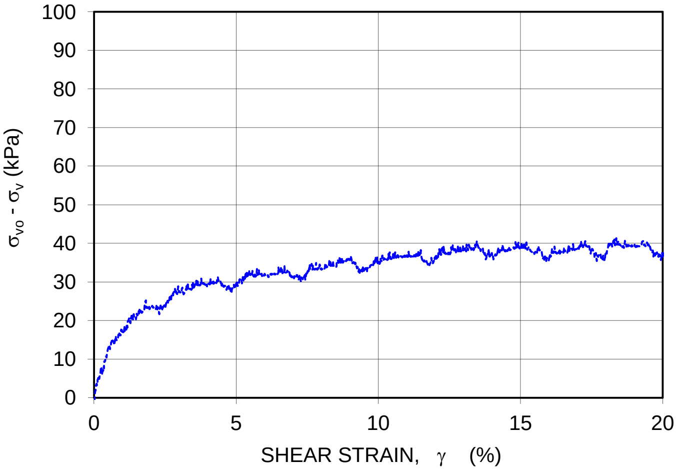
DIRECT SIMPLE SHEAR TEST (ASTM D 6528)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	June 9, 2014
Borehole:	BH-4	Depth (m):	7.11
Sample No.:	S-4	Station:	DSS1

Initial Height (mm):	23.4	**Weight of Specimen (g):**	153.52	**Initial Void Ratio, e_o:**	1.73
Diameter of Ring (mm):	73.1	**Total Unit Weight (kN/m^3):**	15.34	**Final Void Ratio, e_f:**	1.58
Specific Gravity, Gs:	2.70	**Dry Unit Weight (kN/m^3):**	9.69	**Natural Water Content (%):**	58.4
Final Water Content (%):	57.9	**Initial Degree of Saturation, Sr (%):**	90.9	**Final Degree of Saturation, Sr (%):**	98.9







Type of Test: Constant Volume				
Sample No.	Depth (m)	Total Unit Weight (kN/m^3)	Effective Vertical Stress, σ'v (kPa)	Effective Vertical Stress, σ'v (kPa)
S-4	7.11	15.3	75.2	1.0

Comments: Sample sheared at 5% per hour
Assumed Gs of 2.7 as requested by HDR

Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	June 9, 2014	Date:	June 9, 2014	Date:	June 10, 2014

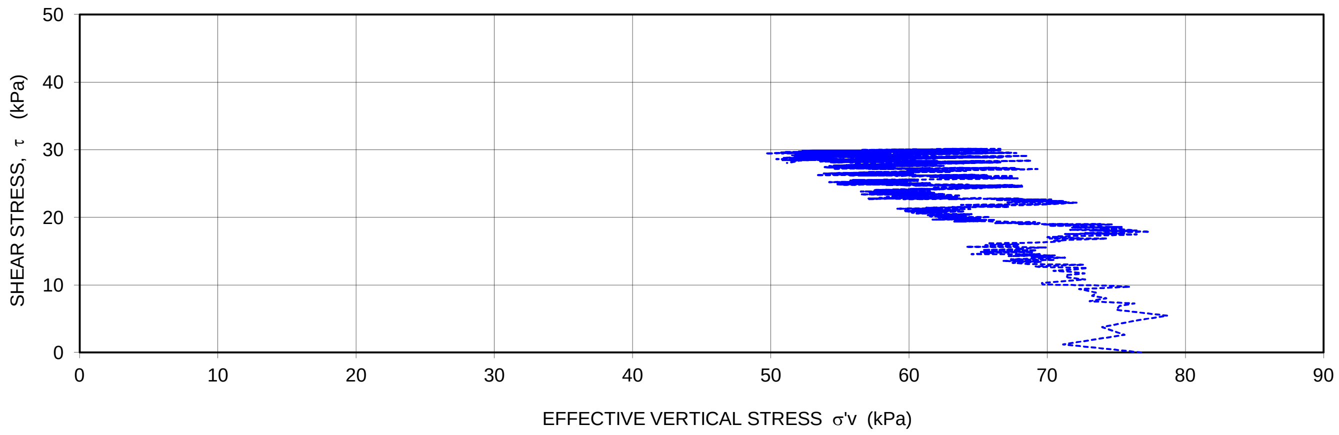
MEG TECHNICAL SERVICES

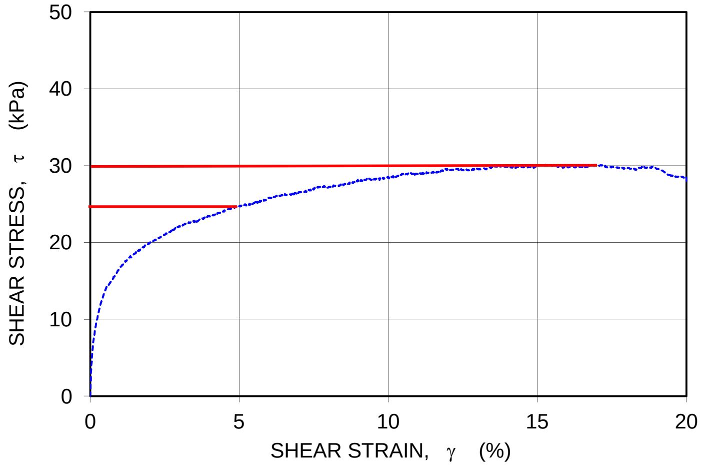
(A Division of MEG Consulting Limited)

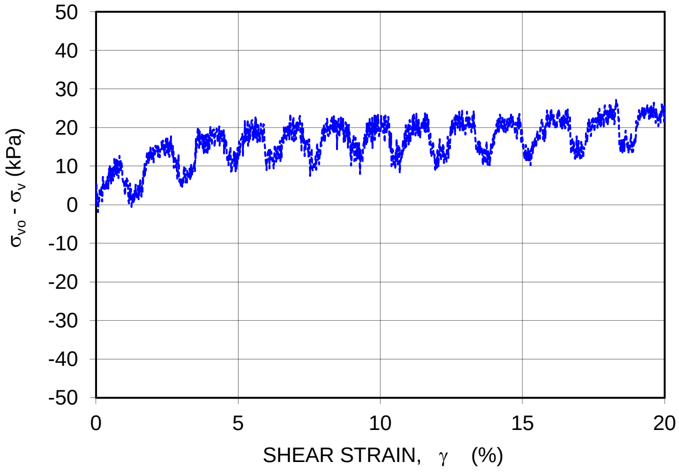
DIRECT SIMPLE SHEAR TEST (ASTM D 6528)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	June 20, 2014
Borehole:	BH-4	Depth (m):	7.98
Sample No.:	S-6	Station:	DSS2

Initial Height (mm):	23.6	**Weight of Specimen (g):**	143.55	**Initial Void Ratio, e_o:**	2.24
Diameter of Ring (mm):	73.1	**Total Unit Weight (kN/m^3):**	14.21	**Final Void Ratio, e_f:**	1.98
Specific Gravity, Gs:	2.69	**Dry Unit Weight (kN/m^3):**	8.16	**Natural Water Content (%):**	74.2
Final Water Content (%):	75.2	**Initial Degree of Saturation, Sr (%):**	89.3	**Final Degree of Saturation, Sr (%):**	102.5







Type of Test: Constant Volume				
Sample No.	Depth (m)	Total Unit Weight (kN/m^3)	Effective Vertical Stress, σ'v (kPa)	OCR
S-6	7.98	14.2	76.2	1.3

Comments: Sample sheared at 5% strain per hour

Some free water observed on the surface of the sample after test was completed

Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	June 23, 2014	Date:	June 23, 2014	Date:	June 24, 2014

MEG TECHNICAL SERVICES

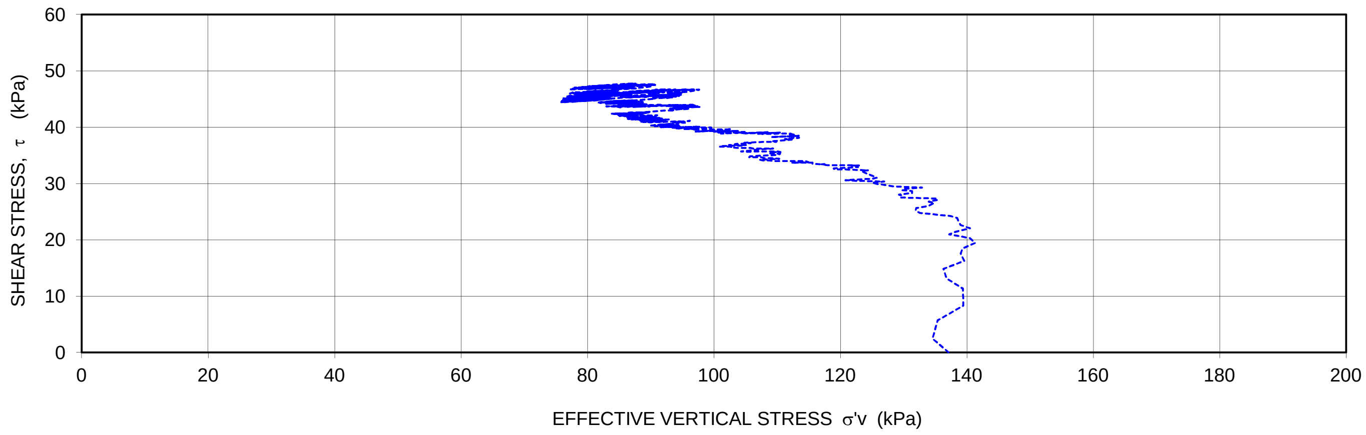
(A Division of MEG Consulting Limited)

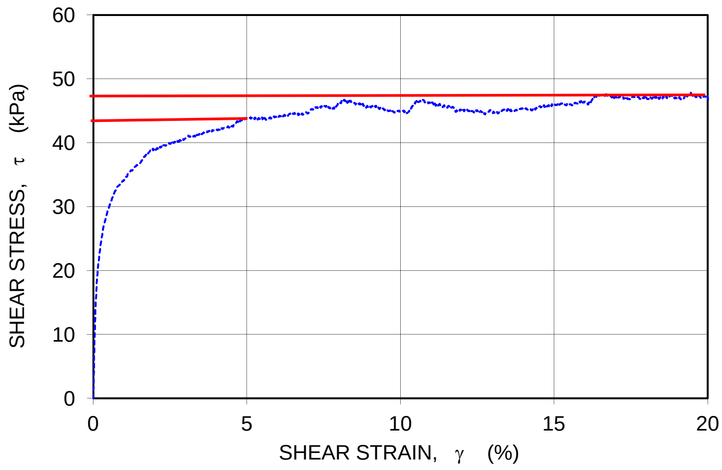
Marine + Earth
MEG
Geosciences

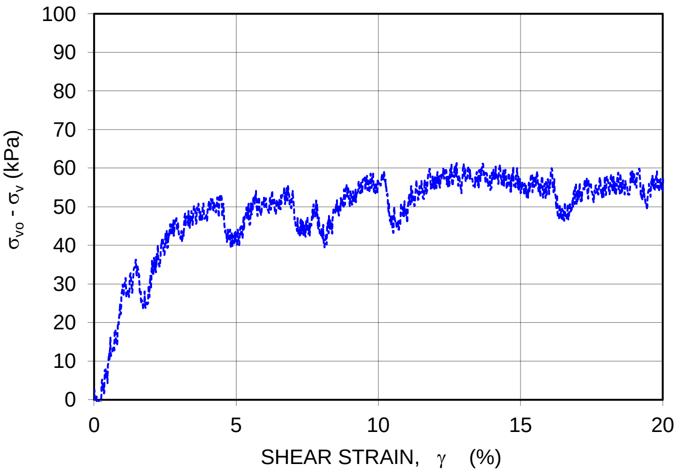
DIRECT SIMPLE SHEAR TEST (ASTM D 6528)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	June 9, 2014
Borehole:	BH-4	Depth (m):	12.74
Sample No.:	S-9	Station:	DSS2

Initial Height (mm):	23.6	**Weight of Specimen (g):**	169.80	**Initial Void Ratio, e_o:**	1.35
Diameter of Ring (mm):	73.1	**Total Unit Weight (kN/m^3):**	16.81	**Final Void Ratio, e_f:**	1.24
Specific Gravity, Gs:	2.78	**Dry Unit Weight (kN/m^3):**	11.60	**Natural Water Content (%):**	44.8
Final Water Content (%):	42.4	**Initial Degree of Saturation, Sr (%):**	92.2	**Final Degree of Saturation, Sr (%):**	95.4







Type of Test: Constant Volume				
Sample No.	Depth (m)	Total Unit Weight (kN/m^3)	Effective Vertical Stress, σ'v (kPa)	OCR
S-9	12.74	16.8	136.2	1.3

Comments: Sample sheared at 5% per hour

Prepared By:	PC	Checked By:	PS	Approved By:	EP
Date:	June 9, 2014	Date:	June 9, 2014	Date:	June 10, 2014

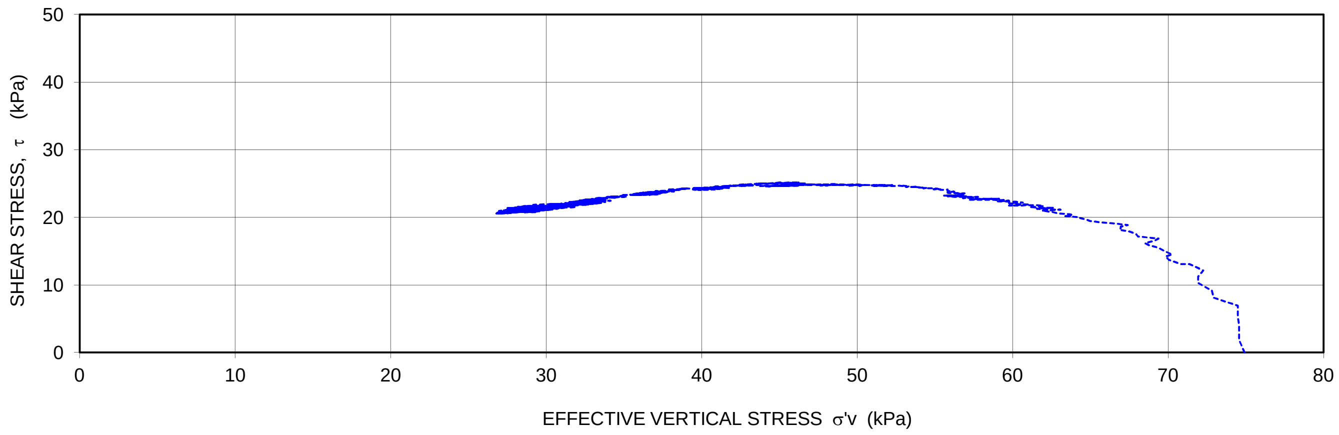
MEG TECHNICAL SERVICES

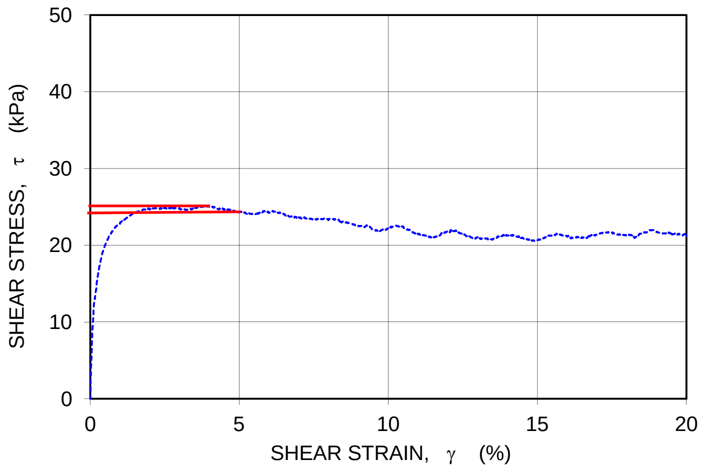
(A Division of MEG Consulting Limited)

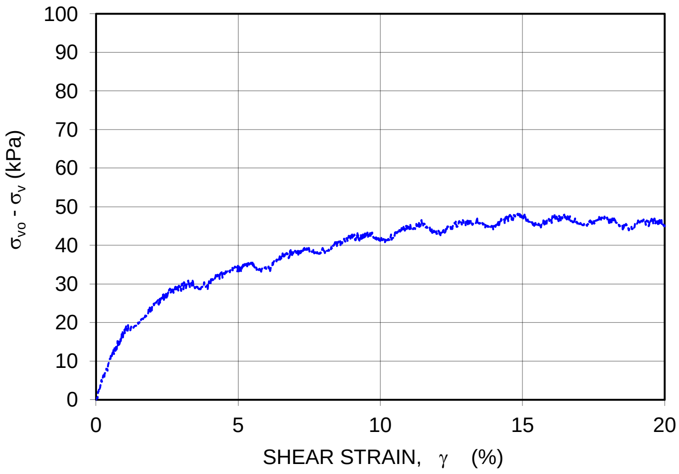
DIRECT SIMPLE SHEAR TEST (ASTM D 6528)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	June 16, 2014
Borehole:	BH-5	Depth (m):	7.62
Sample No.:	S-7	Station:	DSS1

Initial Height (mm):	23.5	**Weight of Specimen (g):**	161.60	**Initial Void Ratio, e_o:**	1.64
Diameter of Ring (mm):	73.1	**Total Unit Weight (kN/m^3):**	16.07	**Final Void Ratio, e_f:**	1.56
Specific Gravity, Gs:	2.76	**Dry Unit Weight (kN/m^3):**	10.25	**Natural Water Content (%):**	56.8
Final Water Content (%):	55.3	**Initial Degree of Saturation, Sr (%):**	95.7	**Final Degree of Saturation, Sr (%):**	97.6







Type of Test: Constant Volume				
Sample No.	Depth (m)	Total Unit Weight (kN/m^3)	Effective Vertical Stress, σ'v (kPa)	OCR
S-7	7.62	16.1	74	1.4

Comments: Sample sheared at 5% strain per hour

Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	June 16, 2014	Date:	June 16, 2014	Date:	June 17, 2017

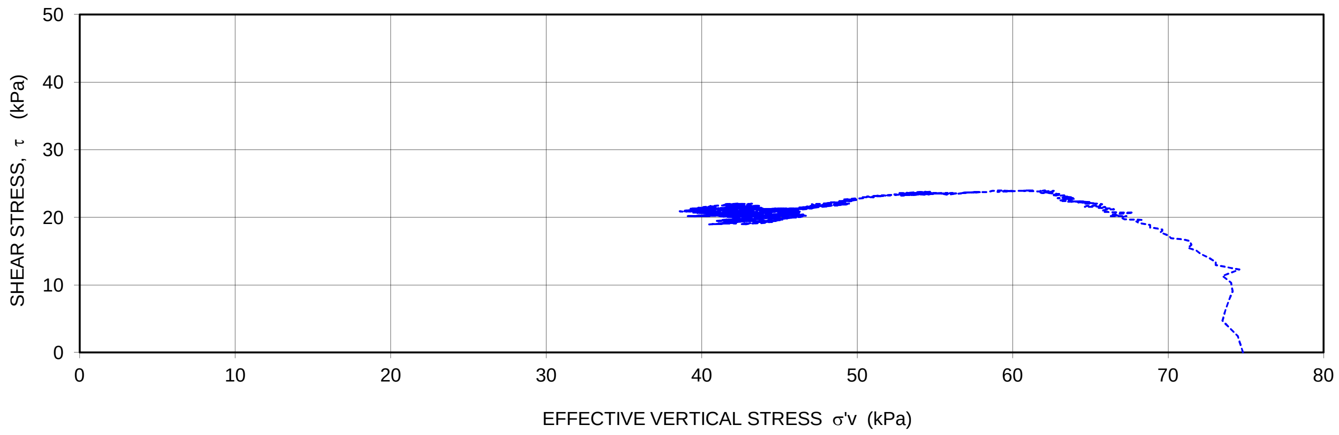
MEG TECHNICAL SERVICES

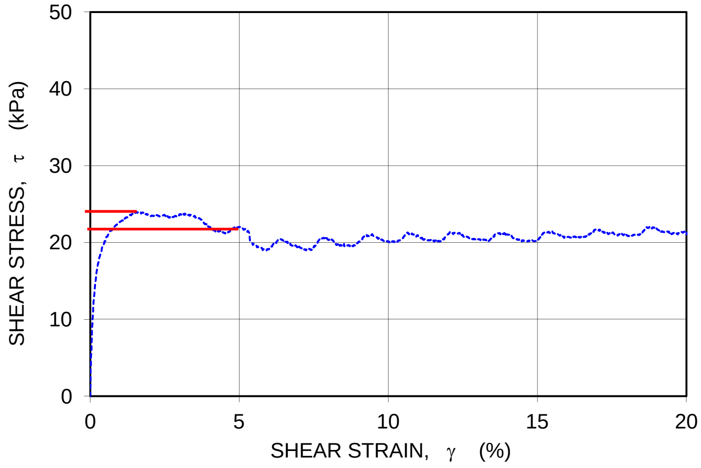
(A Division of MEG Consulting Limited)

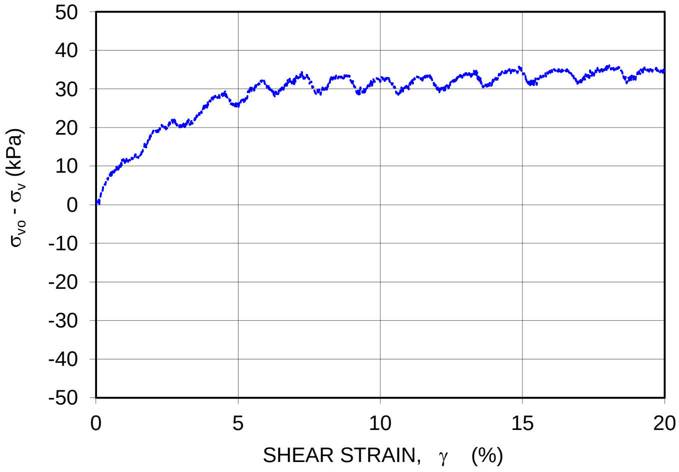
DIRECT SIMPLE SHEAR TEST (ASTM D 6528)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	June 23, 2014
Borehole:	BH-5	Depth (m):	8.75
Sample No.:	S-9	Station:	DSS1

Initial Height (mm):	23.5	**Weight of Specimen (g):**	157.86	**Initial Void Ratio, e_o:**	1.61
Diameter of Ring (mm):	73.1	**Total Unit Weight (kN/m^3):**	15.70	**Final Void Ratio, e_f:**	1.50
Specific Gravity, Gs:	2.70	**Dry Unit Weight (kN/m^3):**	10.13	**Natural Water Content (%):**	55.0
Final Water Content (%):	55.4	**Initial Degree of Saturation, Sr (%):**	92.0	**Final Degree of Saturation, Sr (%):**	99.6







Type of Test: Constant Volume				
Sample No.	Depth (m)	Total Unit Weight (kN/m^3)	Effective Vertical Stress, σ'v (kPa)	OCR
S-9	8.75	15.7	74.0	1.4

Comments: Sample sheared at 5% strain per hour

assumed Gs of 2.70, no consolidation or Gs test performed

Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	June 23, 2014	Date:	June 23, 2014	Date:	June 24, 2014

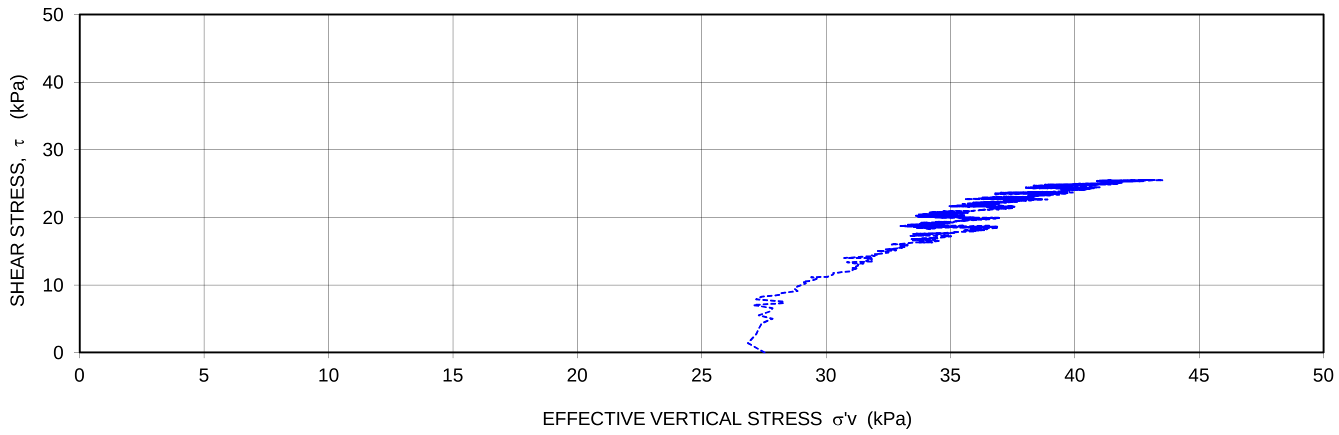
MEG TECHNICAL SERVICES

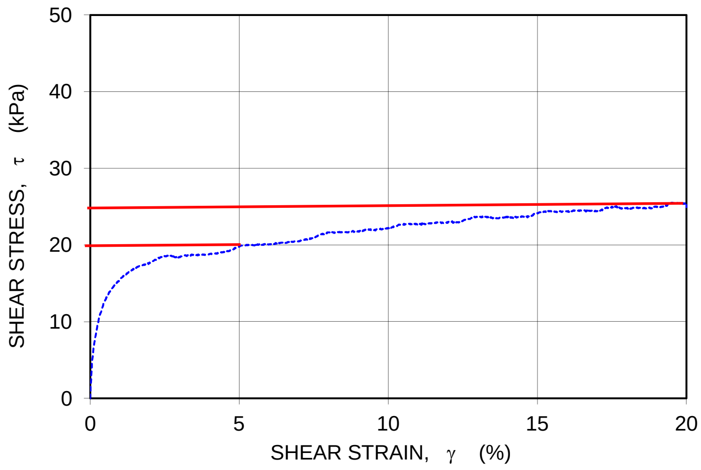
(A Division of MEG Consulting Limited)

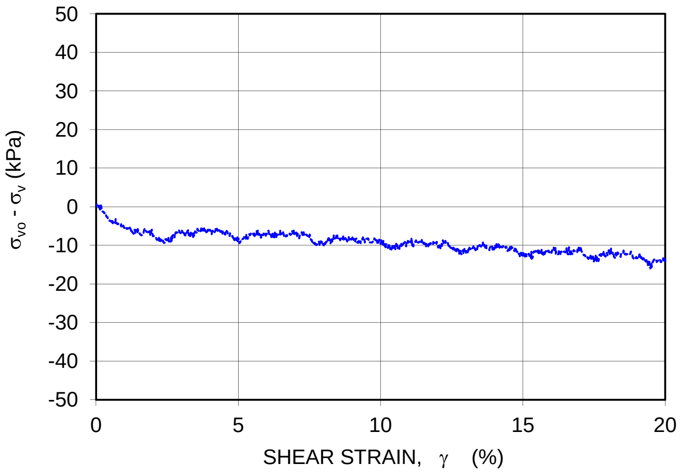
DIRECT SIMPLE SHEAR TEST (ASTM D 6528)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	June 23, 2014
Borehole:	BH-6	Depth (m):	3.57
Sample No.:	S-1	Station:	DSS1

Initial Height (mm):	23.5	**Weight of Specimen (g):**	139.23	**Initial Void Ratio, e_o:**	2.05
Diameter of Ring (mm):	73.1	**Total Unit Weight (kN/m^3):**	13.85	**Final Void Ratio, e_f:**	1.89
Specific Gravity, Gs:	2.79	**Dry Unit Weight (kN/m^3):**	8.97	**Natural Water Content (%):**	54.5
Final Water Content (%):	57.1	**Initial Degree of Saturation, Sr (%):**	74.1	**Final Degree of Saturation, Sr (%):**	84.2







Type of Test: Constant Volume				
Sample No.	Depth (m)	Total Unit Weight (kN/m^3)	Effective Vertical Stress, σ'v (kPa)	OCR
S-1	3.57	13.9	27.3	3.1

Comments: Sample sheared at 5% strain per hour

Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	June 23, 2014	Date:	June 23, 2014	Date:	June 24, 2014

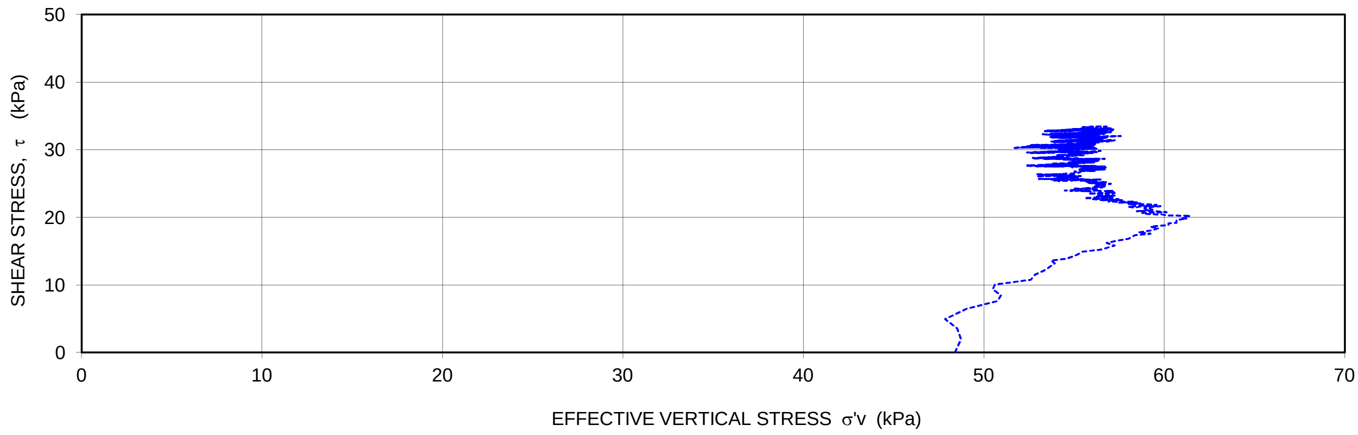
MEG TECHNICAL SERVICES

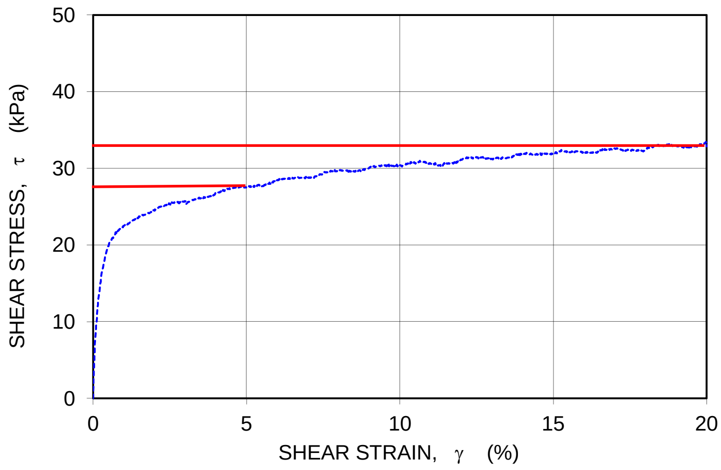
(A Division of MEG Consulting Limited)

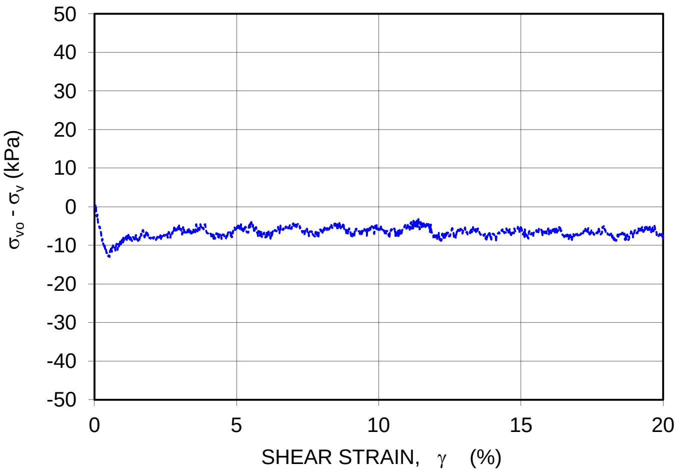
DIRECT SIMPLE SHEAR TEST (ASTM D 6528)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	June 19, 2014
Borehole:	BH-6	Depth (m):	5.49
Sample No.:	S-4	Station:	DSS1

Initial Height (mm):	23.5	**Weight of Specimen (g):**	165.25	**Initial Void Ratio, e_o:**	1.38
Diameter of Ring (mm):	73.1	**Total Unit Weight (kN/m^3):**	16.44	**Final Void Ratio, e_f:**	1.26
Specific Gravity, Gs:	2.68	**Dry Unit Weight (kN/m^3):**	11.05	**Natural Water Content (%):**	48.8
Final Water Content (%):	46.7	**Initial Degree of Saturation, Sr (%):**	94.8	**Final Degree of Saturation, Sr (%):**	99.2







Type of Test: Constant Volume

Sample No.	Depth (m)	Total Unit Weight (kN/m^3)	Effective Vertical Stress, σ'v (kPa)	OCR
S-4	5.49	16.4	47.7	2.8

Comments: Sample sheared at 5% strain per hour

Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	June 19, 2014	Date:	June 19, 2014	Date:	June 20, 2014

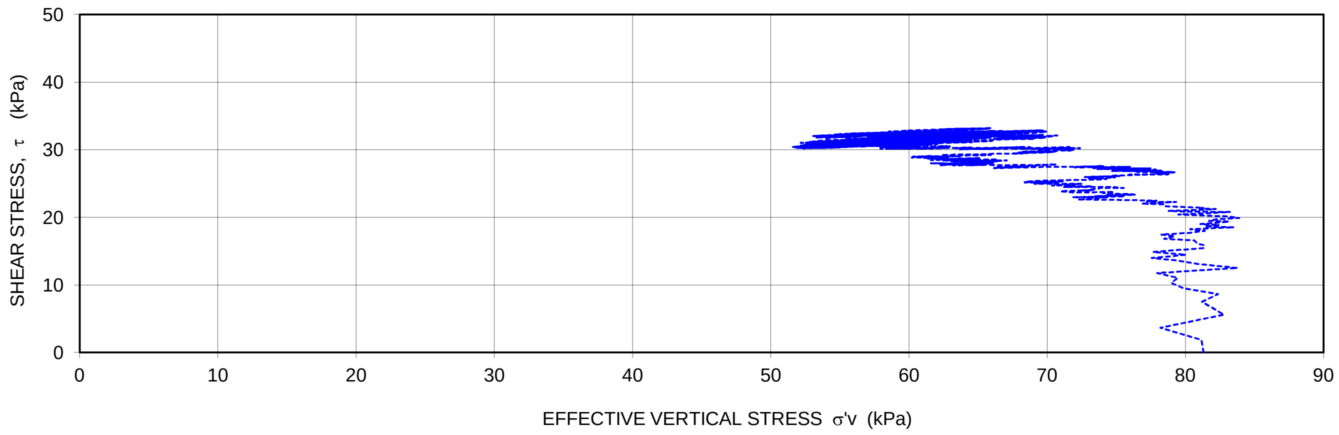
MEG TECHNICAL SERVICES

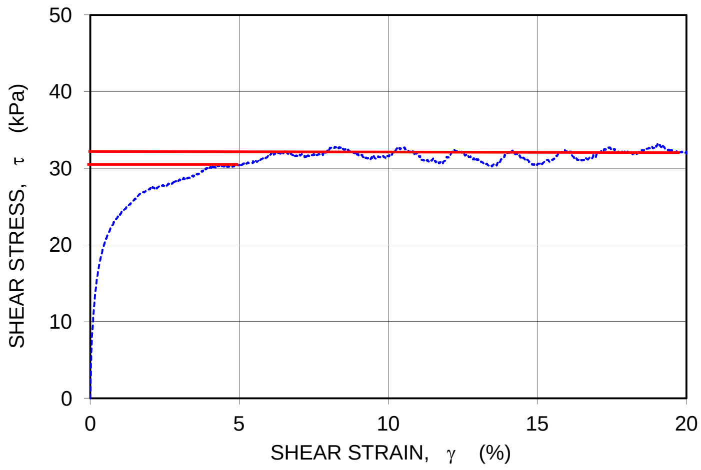
(A Division of MEG Consulting Limited)

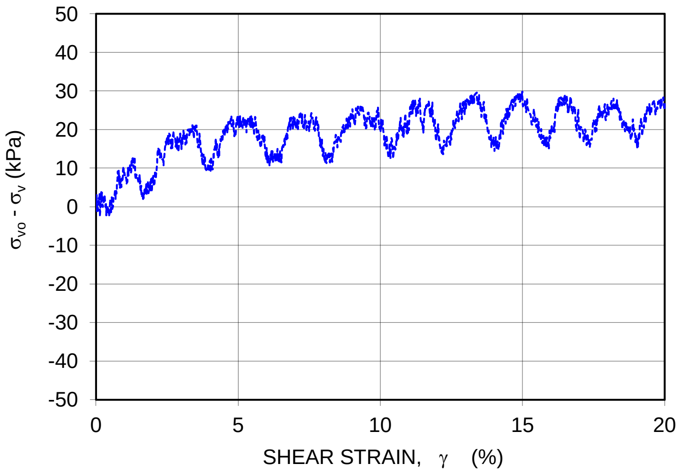
DIRECT SIMPLE SHEAR TEST (ASTM D 6528)

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2	Project No.:	14-MTS-001
Location:	Oregon, USA	Date:	June 16, 2014
Borehole:	BH-6	Depth (m):	7.92
Sample No.:	S-7	Station:	DSS2

Initial Height (mm):	23.6	**Weight of Specimen (g):**	164.91	**Initial Void Ratio, e_o:**	1.46
Diameter of Ring (mm):	73.1	**Total Unit Weight (kN/m^3):**	16.33	**Final Void Ratio, e_f:**	1.35
Specific Gravity, Gs:	2.73	**Dry Unit Weight (kN/m^3):**	10.90	**Natural Water Content (%):**	49.8
Final Water Content (%):	49.8	**Initial Degree of Saturation, Sr (%):**	93.3	**Final Degree of Saturation, Sr (%):**	100.0







Type of Test: Constant Volume

Sample No.	Depth (m)	Total Unit Weight (kN/m^3)	Effective Vertical Stress, σ'v (kPa)	OCR
S-7	7.92	16.3	80.7	1.6

Comments: Sample sheare at 5% strain per hour

Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	June 16, 2014	Date:	June 16, 2014	Date:	June 17, 2014

Cyclic Strength Testing

MEG TECHNICAL SERVICES

(A Division of MEG Consulting Limited)

STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2			Project No.:	14-MTS-001
Location:	Oregon, USA	Borehole:	BH-3	Depth:	10.55 m
Sample:	S-8	Station:	DSS1	Date:	July 25, 2014

0.15 stress ratio (τ_{cyc}/ σ'_{vc}) @ 1 Hz for 100 cycles, σ'_{vc}=77.6kPa
Test OCR=1.6 (Sample loaded to 124.1kPa and unloaded to 77.6kPa)

Initial sample Details		Final Sample Details	
Water Content (%):	48.4	Water Content (%):	48.1
Diameter (mm):	73.17	Diameter (mm):	73.17
Height (mm):	22.21	Change in Height, ΔH (mm):	1.07
Specific Gravity, Gs:	2.70	Final Height (mm):	21.14
Weight of Soil (g):	154.57	Weight of Soil (g):	154.28
Total Unit Weight (kN/m^3)	16.24	Total Unit Weight (kN/m^3)	17.03
Dry Unit Weight (kN/m^3)	10.95	Dry Unit Weight (kN/m^3)	11.50
Initial Void Ratio	1.42	Final Void Ratio	1.30

0.25 stress ratio (τcyc/ σ’vc) @ 1 Hz for 100 cycles, σ’vc=77.6kPa
Test OCR=1.6 (Sample loaded to 124.1kPa and unloaded to 77.6kPa)

Initial sample Details		Final Sample Details	
Water Content (%):	45.0	Water Content (%):	46.3
Diameter (mm):	73.17	Diameter (mm):	73.17
Height (mm):	22.21	Change in Height, ΔH (mm):	1.26
Specific Gravity, Gs:	2.70	Final Height (mm):	20.95
Weight of Soil (g):	151.88	Weight of Soil (g):	153.26
Total Unit Weight (kN/m^3)	15.95	Total Unit Weight (kN/m^3)	17.06
Dry Unit Weight (kN/m^3)	11.00	Dry Unit Weight (kN/m^3)	11.66
Initial Void Ratio	1.41	Final Void Ratio	1.27

Sample Description:

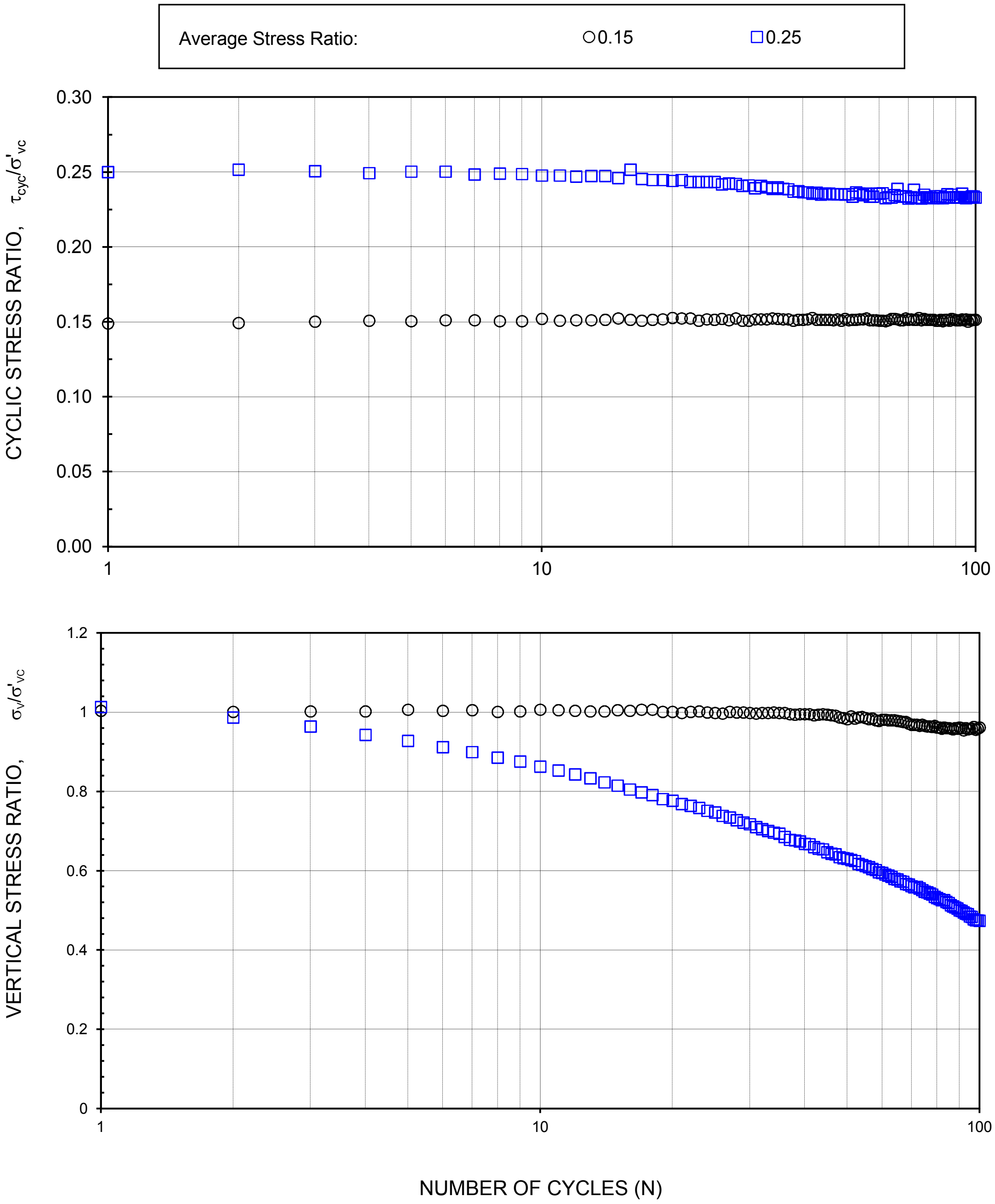
Prepared By:	MF	Checked By:	PS	Approved B	EP
Date:	July 25, 2014	Date:	July 25, 2014	Date:	July 30, 2014

MEG TECHNICAL SERVICES

(A Division of MEG Consulting Limited)

STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2			Project No.:	14-MTS-001
Location:	Oregon, USA	Borehole:	BH-3	Depth:	10.55 m
Sample:	S-8	Station:	DSS1	Date:	July 25, 2014



Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	July 25, 2014	Date:	July 25, 2014	Date:	July 30, 2014

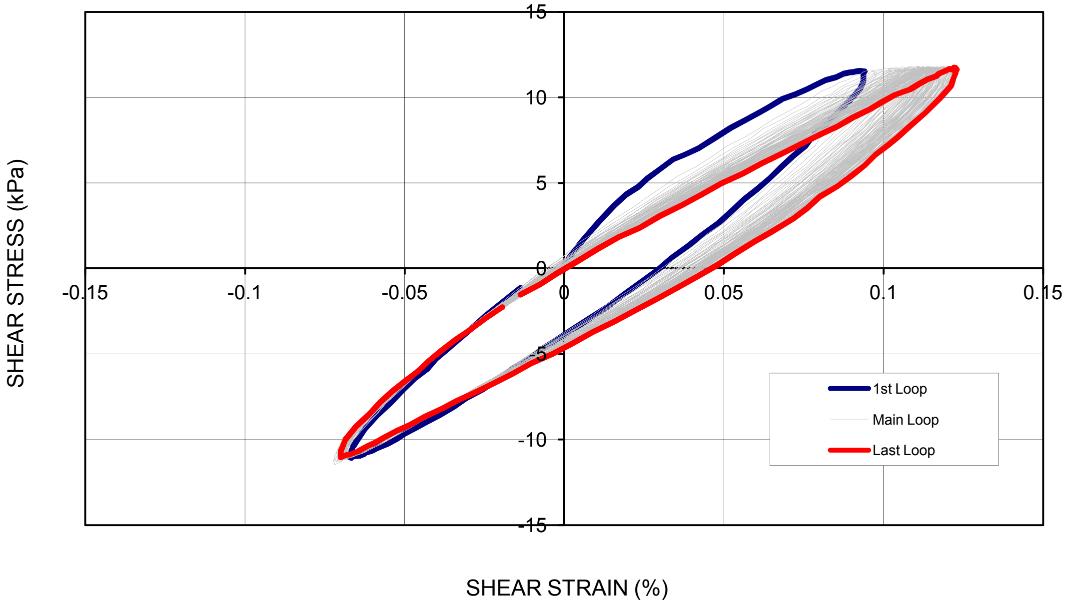
MEG TECHNICAL SERVICES

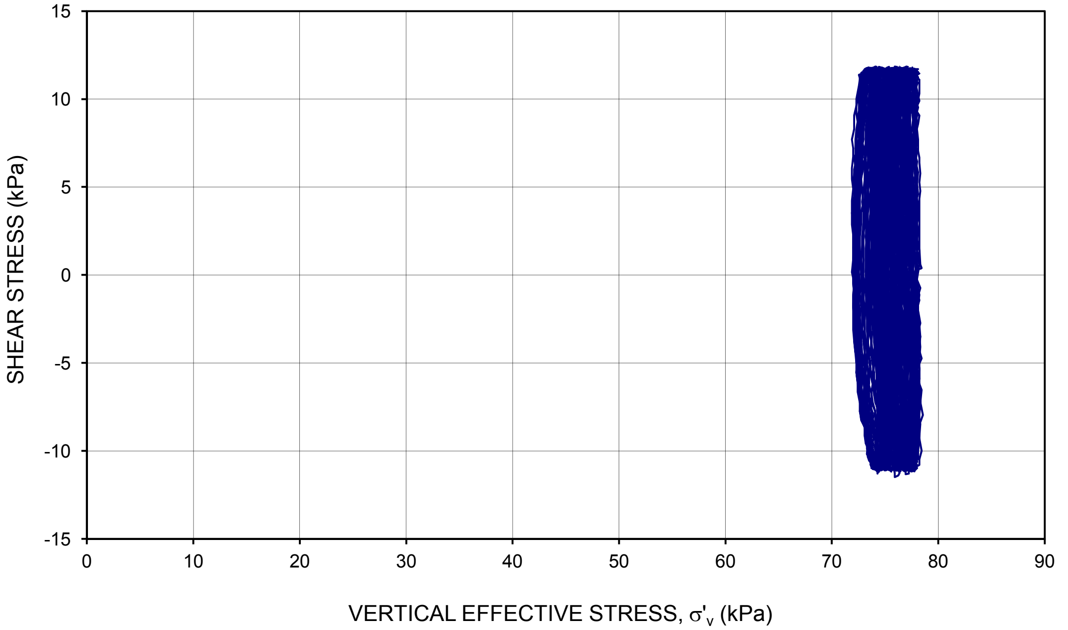
(A Division of MEG Consulting Limited)

STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2			Project No.:	14-MTS-001
Location:	Oregon, USA	Borehole:	BH-3	Depth:	10.55 m
Sample:	S-8	Station:	DSS1	Date:	July 25, 2014

0.15 stress ratio (τcyc/ σ'vc) @ 1 Hz for 100 cycles, σ'vc=77.6kPa
Test OCR=1.6 (Sample loaded to 124.1kPa and unloaded to 77.6kPa)





Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	July 25, 2014	Date:	July 25, 2014	Date:	July 25, 2014

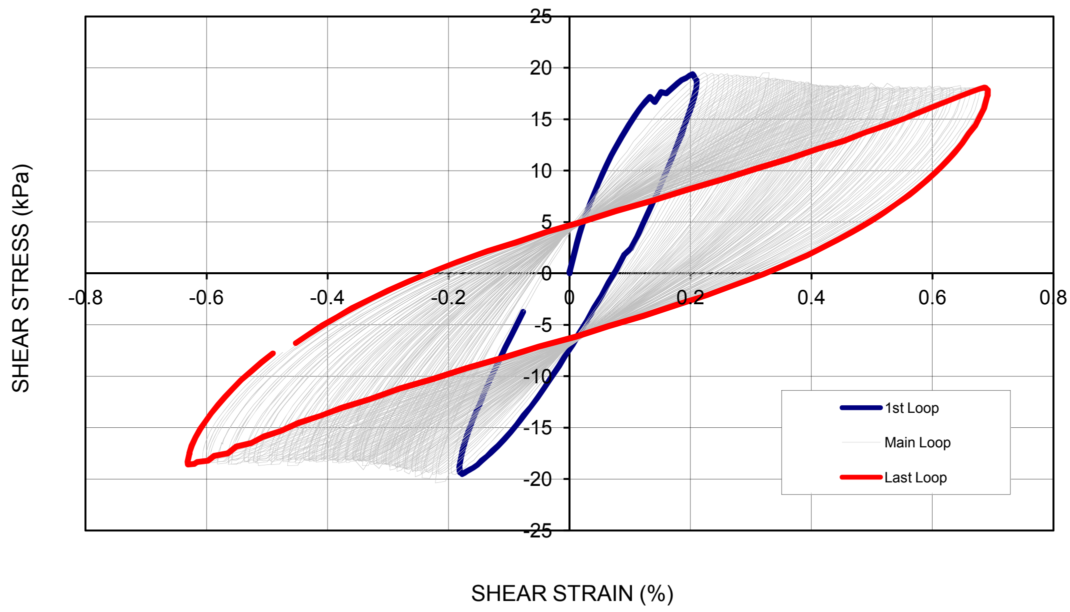
MEG TECHNICAL SERVICES

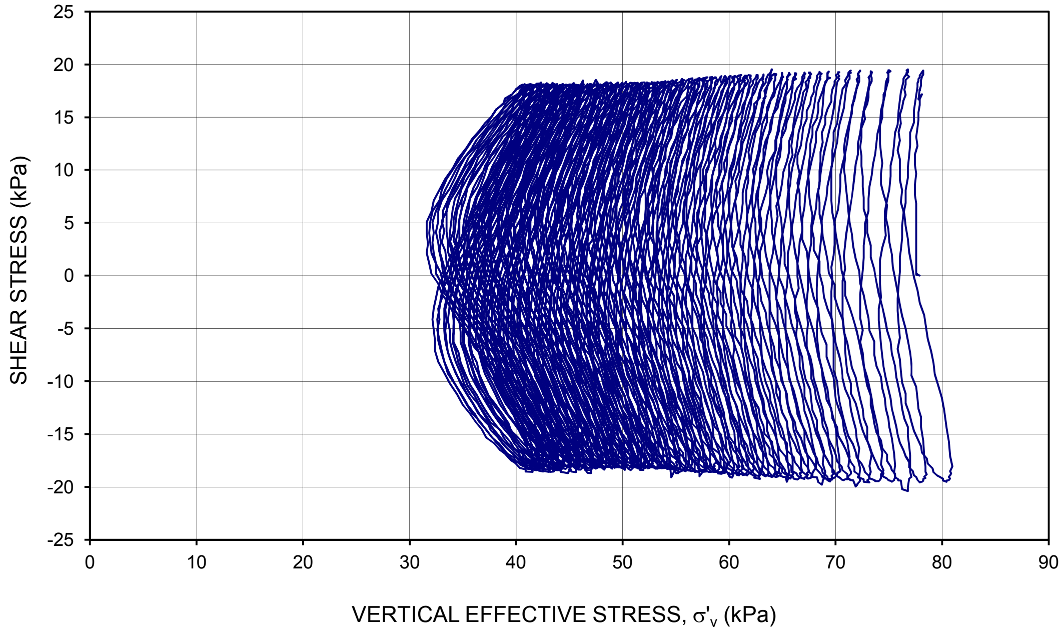
(A Division of MEG Consulting Limited)

STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2			Project No.:	14-MTS-001
Location:	Oregon, USA	Borehole:	BH-3	Depth:	10.55 m
Sample:	S-8	Station:	DSS1	Date:	July 25, 2014

0.25 stress ratio (τcyc/ σ’vc) @ 1 Hz for 100 cycles, σ’vc=77.6kPa
Test OCR=1.6 (Sample loaded to 124.1kPa and unloaded to 77.6kPa)





Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	July 29, 2014	Date:	July 29, 2014	Date:	July 30, 2014

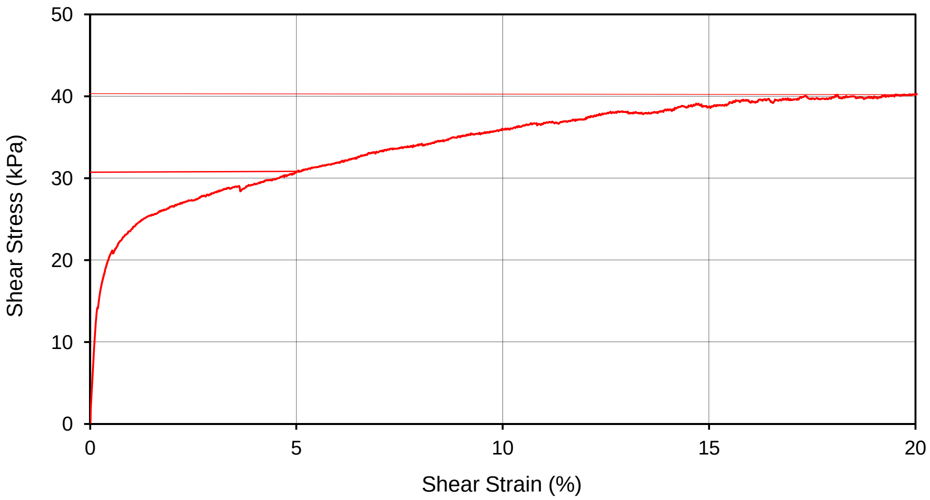
MEG TECHNICAL SERVICES

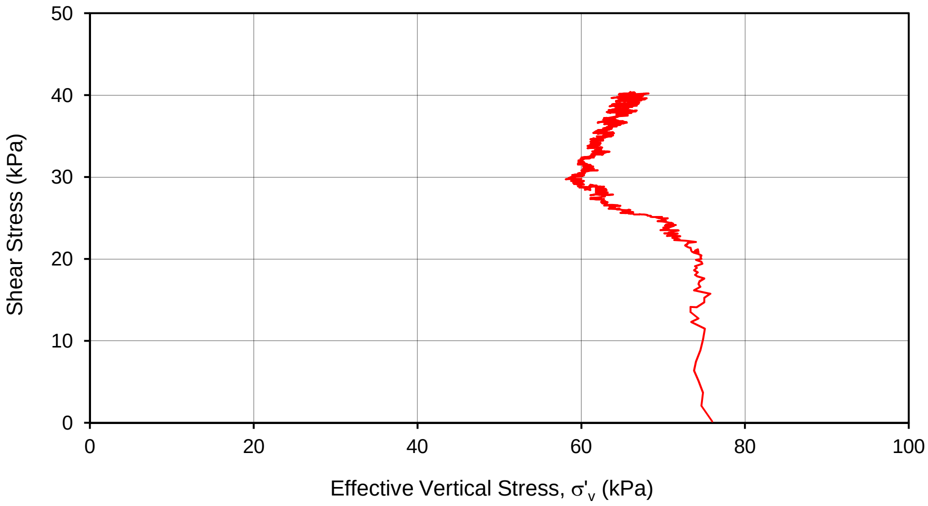
(A Division of MEG Consulting Limited)

POST-CYCLIC STATIC DIRECT SIMPLE SHEAR TEST

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2			Project No.:	14-MTS-001
Location:	Oregon, USA	Borehole:	BH-3	Depth:	10.55 m
Sample:	S-8	Station:	DSS1	Date:	July 25, 2014

POST-CYCLIC STATIC SHEAR TEST





Note: Test performed after stress-controlled DSS test at average cyclic stress ratio, CSR = 0.15 with 8% excess pore pressure. Post-cyclic tests performed under initial condition of zero shear stress

Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	July 25, 2014	Date:	July 25, 2014	Date:	July 25, 2014

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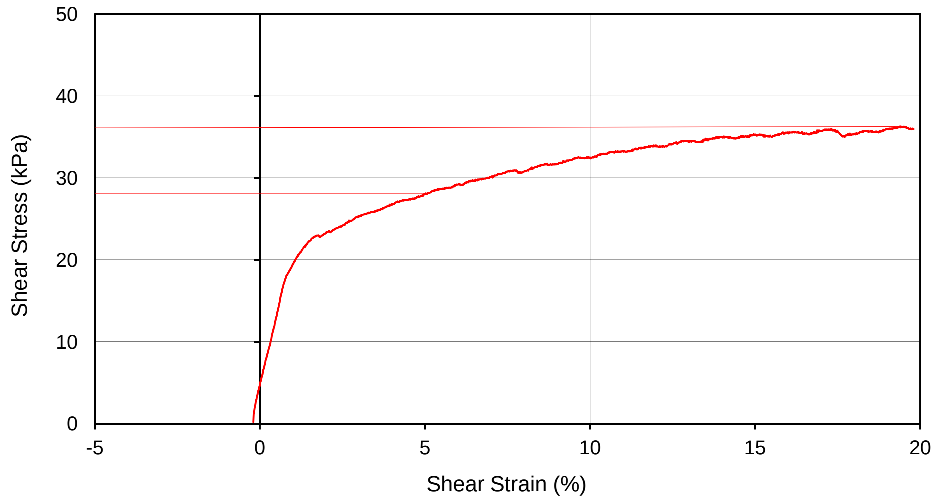
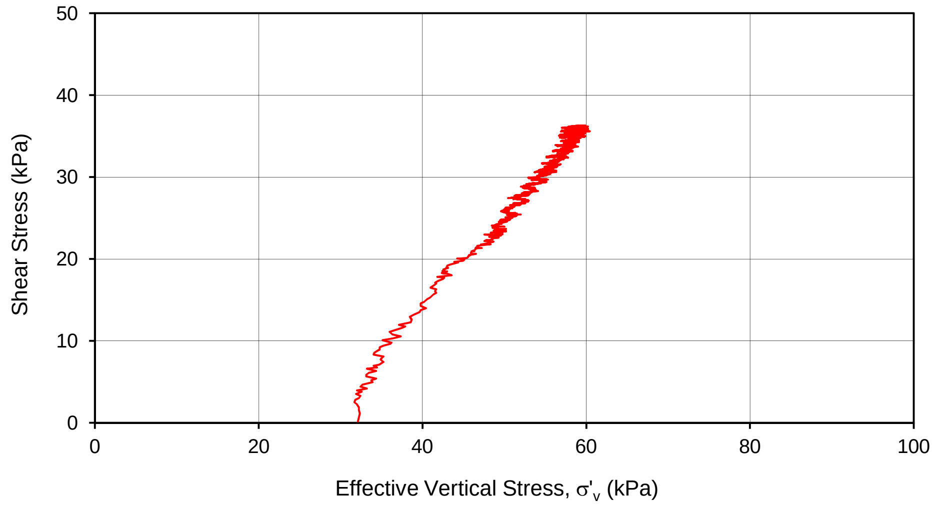
POST-CYCLIC STATIC DIRECT SIMPLE SHEAR TEST

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2			Project No.:	14-MTS-001
Location:	Oregon, USA	Borehole:	BH-3	Depth:	10.55 m
Sample:	S-8	Station:	DSS1	Date:	July 30, 2014

POST-CYCLIC STATIC SHEAR TEST





Note: Test performed after stress-controlled DSS test at average cyclic stress ratio, CSR = 0.25 with 59% excess pore pressure. Post-cyclic tests performed under initial condition of zero shear stress

Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	July 29, 2014	Date:	July 30, 2014	Date:	July 30, 2014

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STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2			Project No.:	14-MTS-001
Location:	Oregon, USA	Borehole:	BH-4	Depth:	3.55 m
Sample:	S-1	Station:	DSS1	Date:	August 5, 2014

0.15 stress ratio (τ_{cyc}/ σ'_{vc}) @ 1 Hz for 100 cycles, σ'_{vc}=34.1kPa
Test OCR=3.2 (sample consolidated to 109.6kPa and unloaded to 34.1kPa)

Initial sample Details		Final Sample Details	
Water Content (%):	51.0	Water Content (%):	46.9
Diameter (mm):	73.17	Diameter (mm):	73.17
Height (mm):	23.37	Change in Height, ΔH (mm):	0.94
Specific Gravity, Gs:	2.65	Final Height (mm):	22.43
Weight of Soil (g):	169.17	Weight of Soil (g):	164.49
Total Unit Weight (kN/m^3)	16.89	Total Unit Weight (kN/m^3)	17.11
Dry Unit Weight (kN/m^3)	11.18	Dry Unit Weight (kN/m^3)	11.65
Initial Void Ratio	1.32	Final Void Ratio	1.23

0.40 stress ratio (τ_{cyc}/ σ'_{vc}) @ 1 Hz for 100 cycles, σ'_{vc}=34.1kPa
Test OCR=3.2 (sample consolidated to 109.6kPa and unloaded to 34.1kPa)

Initial sample Details		Final Sample Details	
Water Content (%):	46.5	Water Content (%):	45.9
Diameter (mm):	73.22	Diameter (mm):	73.22
Height (mm):	23.30	Change in Height, ΔH (mm):	1.06
Specific Gravity, Gs:	2.65	Final Height (mm):	22.24
Weight of Soil (g):	167.35	Weight of Soil (g):	166.68
Total Unit Weight (kN/m^3)	16.73	Total Unit Weight (kN/m^3)	17.46
Dry Unit Weight (kN/m^3)	11.42	Dry Unit Weight (kN/m^3)	11.97
Initial Void Ratio	1.28	Final Void Ratio	1.17

0.60 stress ratio (τ_{cyc}/ σ'_{vc}) @ 1 Hz for 15 cycles, σ'_{vc}=34.1kPa
Test OCR=3.2 (sample consolidated to 109.6kPa and unloaded to 34.1kPa)

Final Sample Details	
Water Content (%):	45.9
Diameter (mm):	73.22
Change in Height, ΔH (mm):	1.08
Final Height (mm):	22.22
Weight of Soil (g):	166.72
Total Unit Weight (kN/m^3)	17.48
Dry Unit Weight (kN/m^3)	11.98
Final Void Ratio	1.17

Sample Description: Same specimen was used for test at CSR0.40 and 0.60. As requested by client, the CSR 0.40 sample was reconsolidated to 34.1 kPa and retested at CSR 0.60.

Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	August 20, 2014	Date:	August 20, 2014	Date:	August 21, 2014

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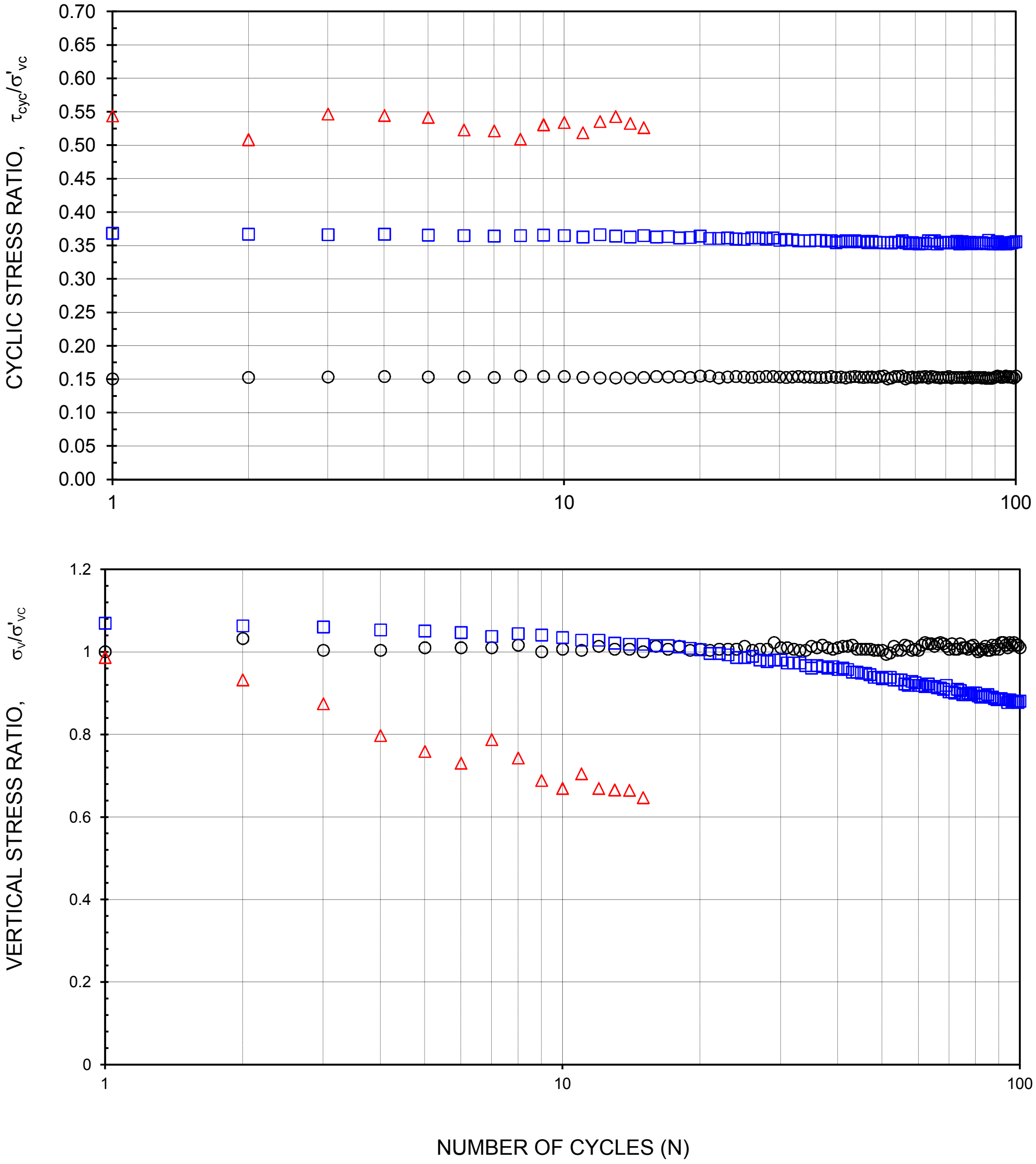
STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2			Project No.:	14-MTS-001
Location:	Oregon, USA	Borehole:	BH-4	Depth:	3.55 m
Sample:	S-1	Station:	DSS1	Date:	August 5, 2014

Average Stress Ratio: ○0.15 □0.40 △0.60



Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	August 20, 2014	Date:	August 20, 2014	Date:	August 21, 2014

MEG TECHNICAL SERVICES

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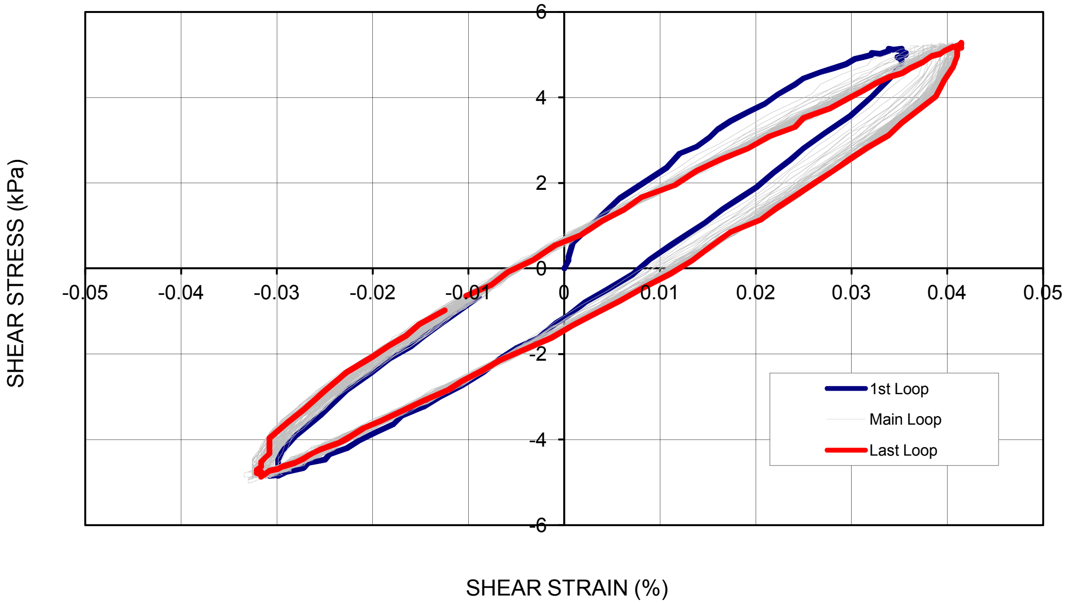
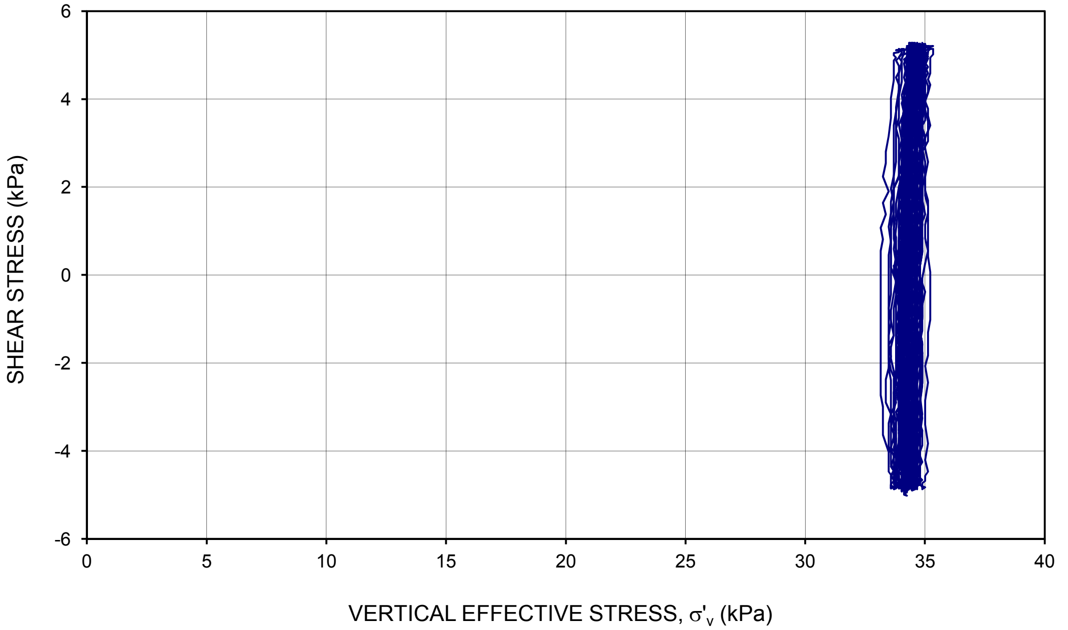
STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2			Project No.:	14-MTS-001
Location:	Oregon, USA	Borehole:	BH-4	Depth:	3.55 m
Sample:	S-1	Station:	DSS1	Date:	August 5, 2014

0.15 stress ratio (τ_{cyc}/ σ'_{vc}) @ 1 Hz for 100 cycles, σ'vc=34.1kPa
Test OCR=3.2 (sample consolidated to 109.6kpa and unloaded to 34.1kPa)





Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	August 5, 2014	Date:	August 5, 2014	Date:	August 5, 2014

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Marine + Earth
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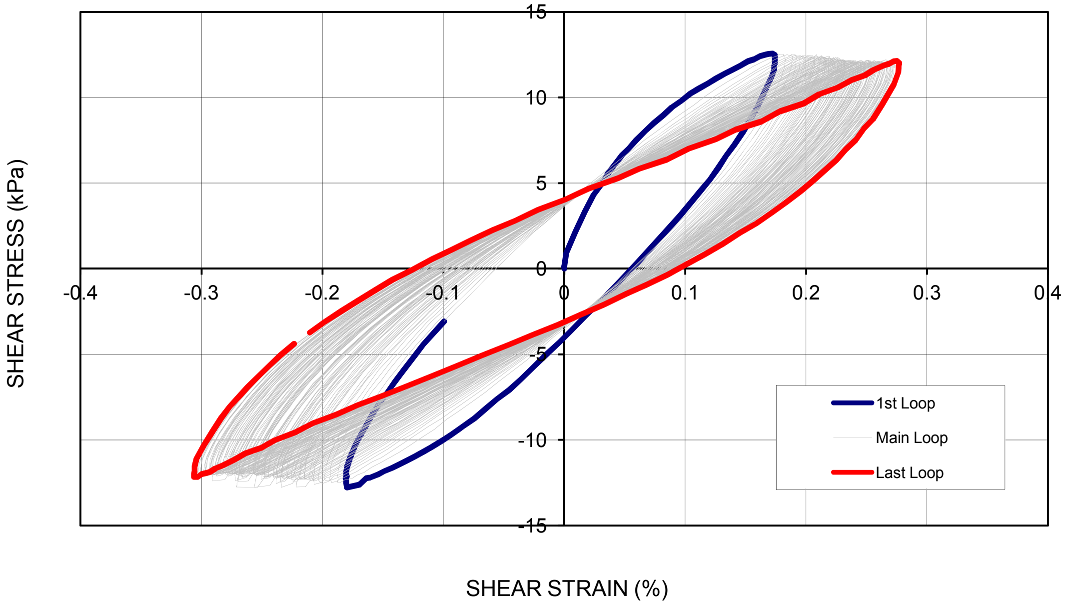
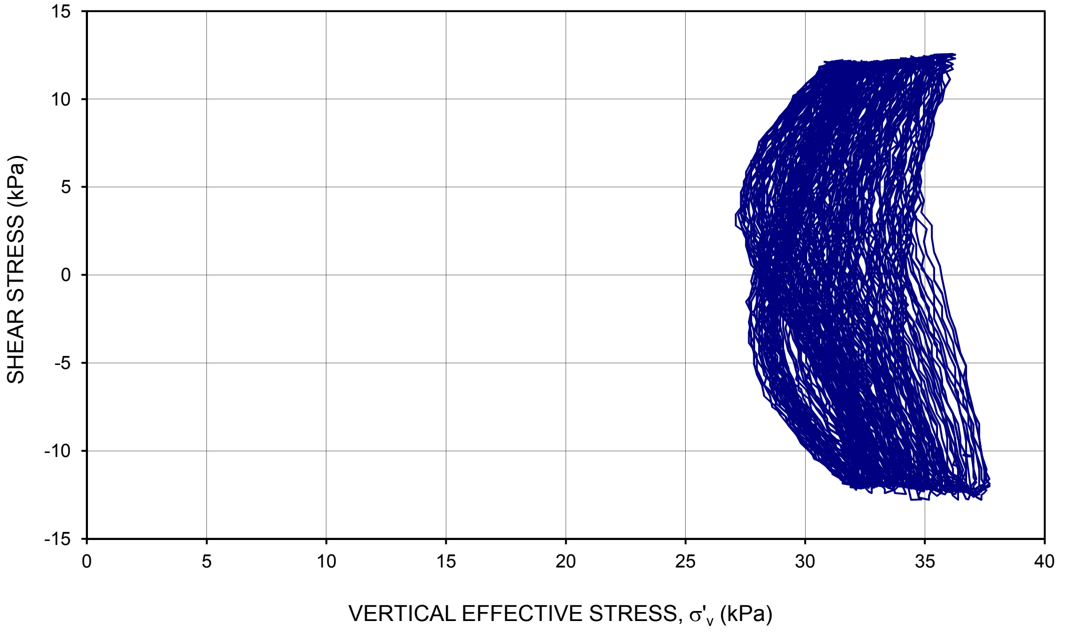
Geosciences

STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2			Project No.:	14-MTS-001
Location:	Oregon, USA	Borehole:	BH-4	Depth:	3.55 m
Sample:	S-1	Station:	DSS1	Date:	August 20, 2014

0.40 stress ratio (t_{cyc}/ s'_{vc}) @ 1 Hz for 100 cycles, σ'vc=34.1kPa
Test OCR=3.2 (sample consolidated to 109.6kpa and unloaded to 34.1kPa)





Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	August 20, 2014	Date:	August 20, 2014	Date:	August 21, 2014

MEG TECHNICAL SERVICES

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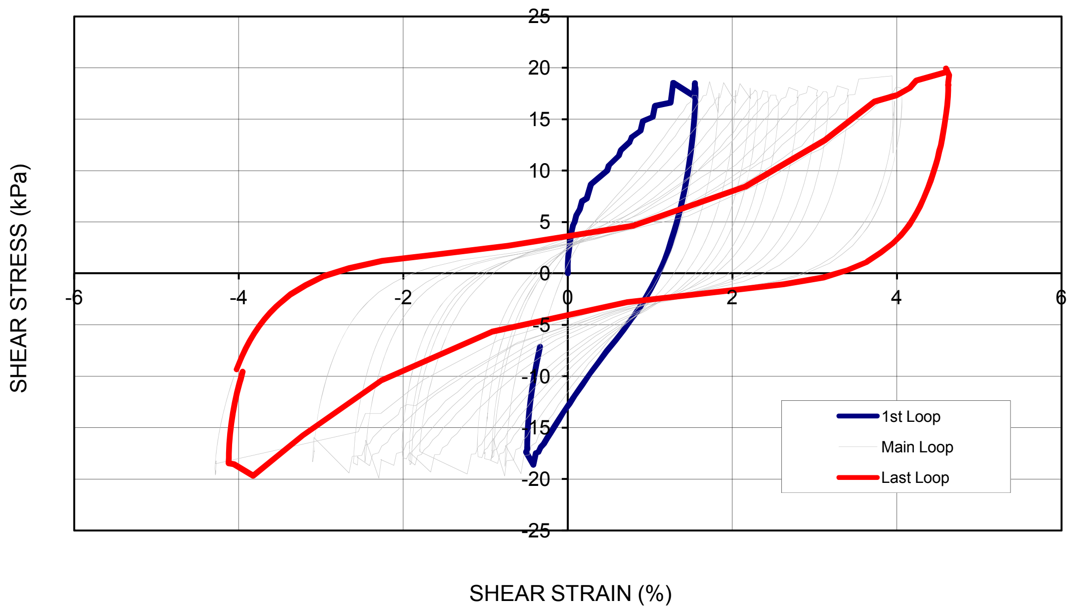
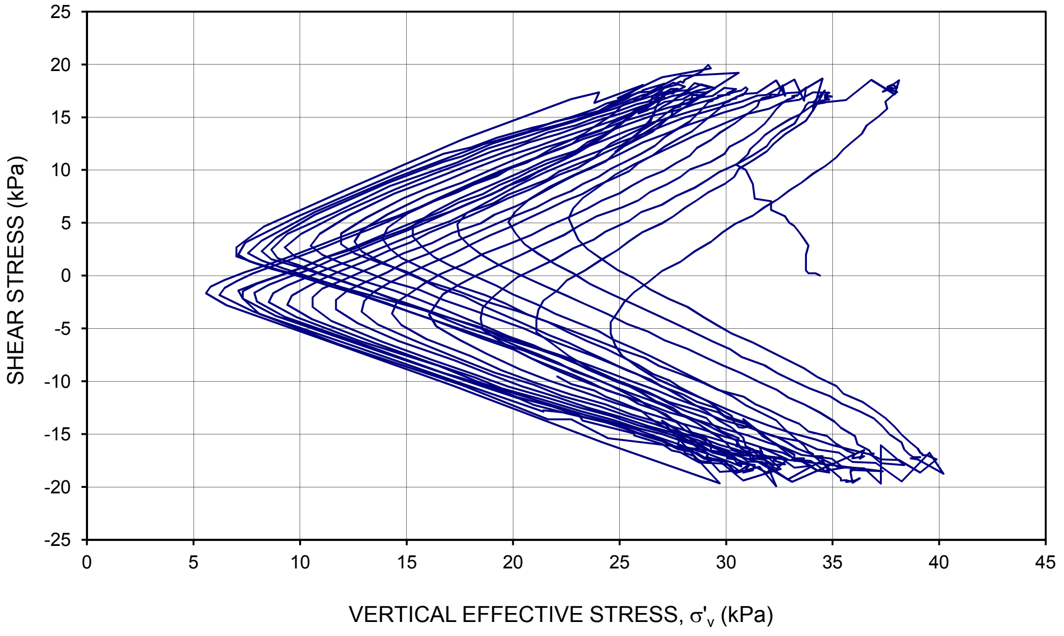
Marine + Earth Geosciences

STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2			Project No.:	14-MTS-001
Location:	Oregon, USA	Borehole:	BH-4	Depth:	3.55 m
Sample:	S-1	Station:	DSS1	Date:	August 20, 2014

0.60 stress ratio (τ_{cyc}/ σ'_{vc}) @ 1 Hz for 15 cycles, σ'_{vc}=34.1kPa
Test OCR=3.2 (sample consolidated to 109.6kpa and unloaded to 34.1kPa)





Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	August 20, 2014	Date:	August 20, 2014	Date:	August 21, 2014

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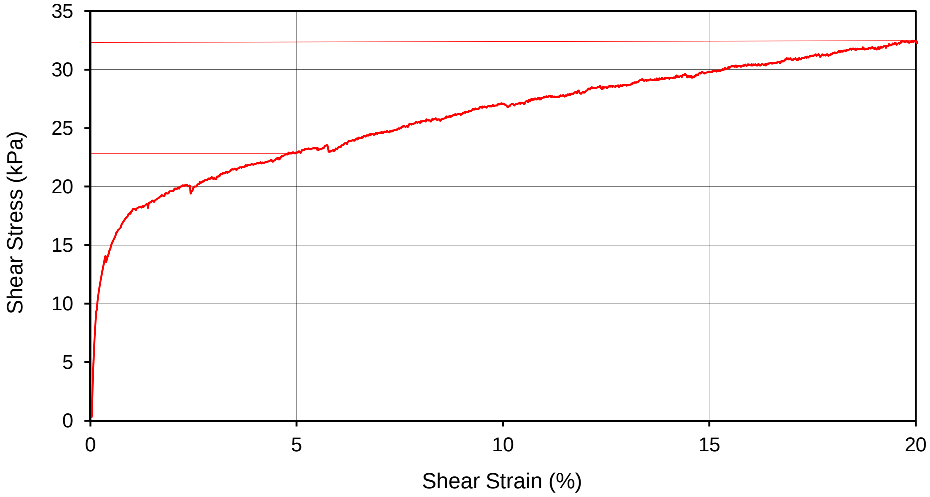
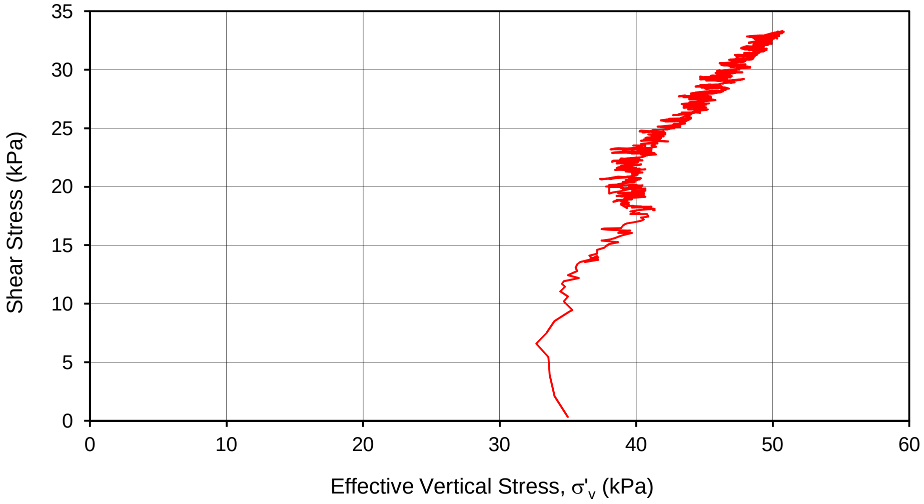
POST-CYCLIC STATIC DIRECT SIMPLE SHEAR TEST

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2			Project No.:	14-MTS-001
Location:	Oregon, USA	Borehole:	BH-4	Depth:	3.55 m
Sample:	S-1	Station:	DSS1	Date:	August 5, 2014

POST-CYCLIC STATIC SHEAR TEST





Note: Test performed after stress-controlled DSS test at average cyclic stress ratio, CSR = 0.15 with 2% excess pore pressure. Post-cyclic tests performed under initial condition of zero shear stress

Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	August 5, 2014	Date:	August 5, 2014	Date:	August 5, 2014

MEG TECHNICAL SERVICES

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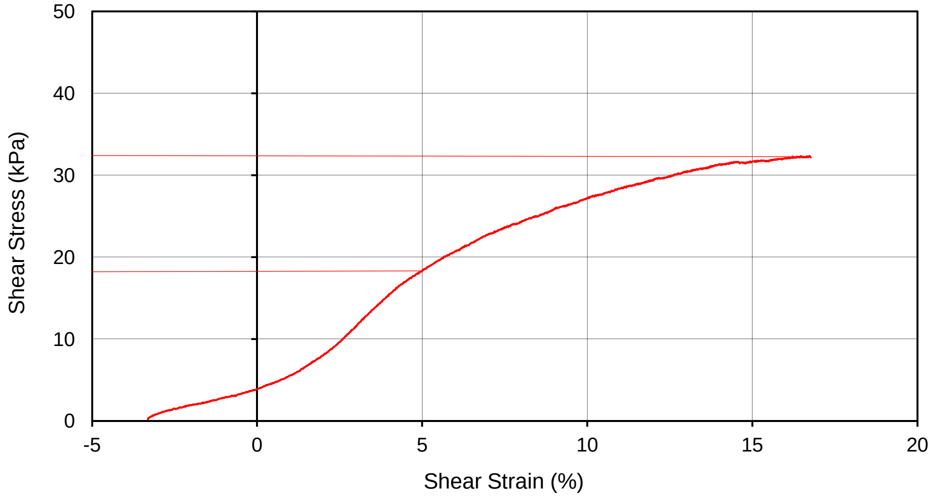
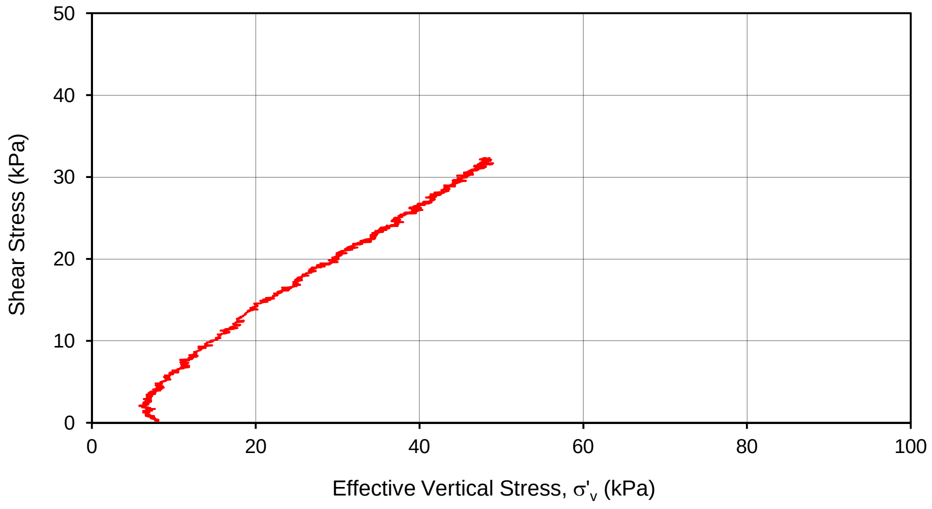
POST-CYCLIC STATIC DIRECT SIMPLE SHEAR TEST

Project: HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2
Project No.: 14-MTS-001
Location: Oregon, USA
Borehole: BH-4
Depth: 3.55 m
Sample: S-1
Station: DSS1
Date: August 19, 2014

POST-CYCLIC STATIC SHEAR TEST





Note: Test performed after stress-controlled DSS test at average cyclic stress ratio, CSR = 0.60 with 84% excess pore pressure. Post-cyclic tests performed under initial condition of zero shear stress

Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	August 20, 2014	Date:	August 20, 2014	Date:	August 21, 2014

MEG TECHNICAL SERVICES

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STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

Project: HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2

Project No.: 14-MTS-001

Location: Oregon, USA

Borehole: BH-4

Depth: 3.60 m

Sample: S-1

Station: DSS1

Date: August 15, 2014

0.10 stress ratio (τ_{cyc}/ σ'_{vc}) @ 1 Hz for 100 cycles, σ'_{vc}=411kPa

Initial sample Details		Final Sample Details	
Water Content (%):	52.5	Water Content (%):	42.0
Diameter (mm):	73.17	Diameter (mm):	73.17
Height (mm):	23.37	Change in Height, ΔH (mm):	2.77
Specific Gravity, Gs:	2.65	Final Height (mm):	20.60
Weight of Soil (g):	163.83	Weight of Soil (g):	152.59
Total Unit Weight (kN/m^3)	16.36	Total Unit Weight (kN/m^3)	17.29
Dry Unit Weight (kN/m^3)	10.73	Dry Unit Weight (kN/m^3)	12.17
Initial Void Ratio	1.42	Final Void Ratio	1.13

0.167 stress ratio (τ_{cyc}/ σ'_{vc}) @ 1 Hz for 72 cycles, σ'_{vc}=411kPa

Initial sample Details		Final Sample Details	
Water Content (%):	48.1	Water Content (%):	37.2
Diameter (mm):	73.22	Diameter (mm):	73.22
Height (mm):	23.30	Change in Height, ΔH (mm):	2.89
Specific Gravity, Gs:	2.65	Final Height (mm):	20.41
Weight of Soil (g):	165.47	Weight of Soil (g):	153.26
Total Unit Weight (kN/m^3)	16.54	Total Unit Weight (kN/m^3)	17.49
Dry Unit Weight (kN/m^3)	11.17	Dry Unit Weight (kN/m^3)	12.75
Initial Void Ratio	1.33	Final Void Ratio	1.04

Sample Description:

Prepared By:	MF	Checked By:	PS	Approved By	EP
Date:	August 15, 2014	Date:	August 15, 2014	Date:	August 18, 2014

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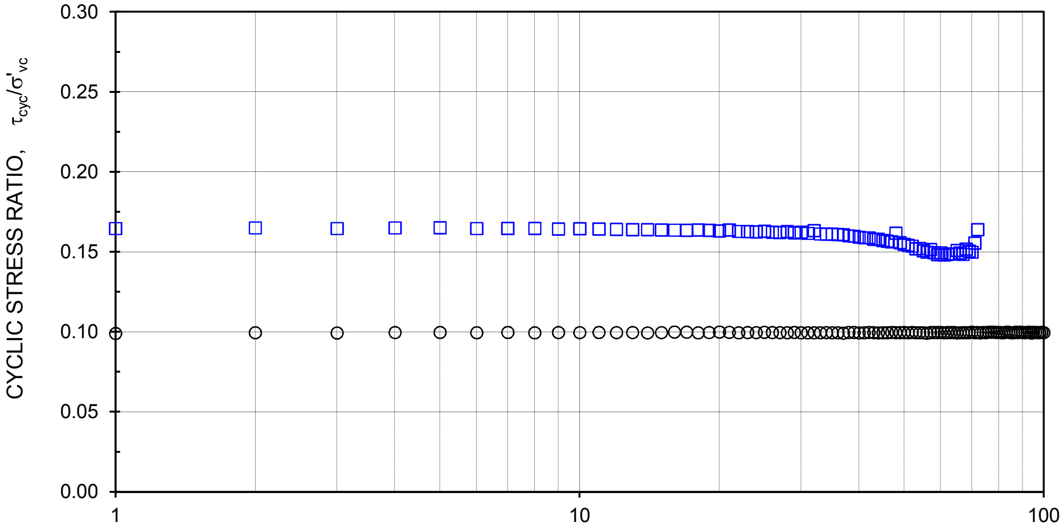
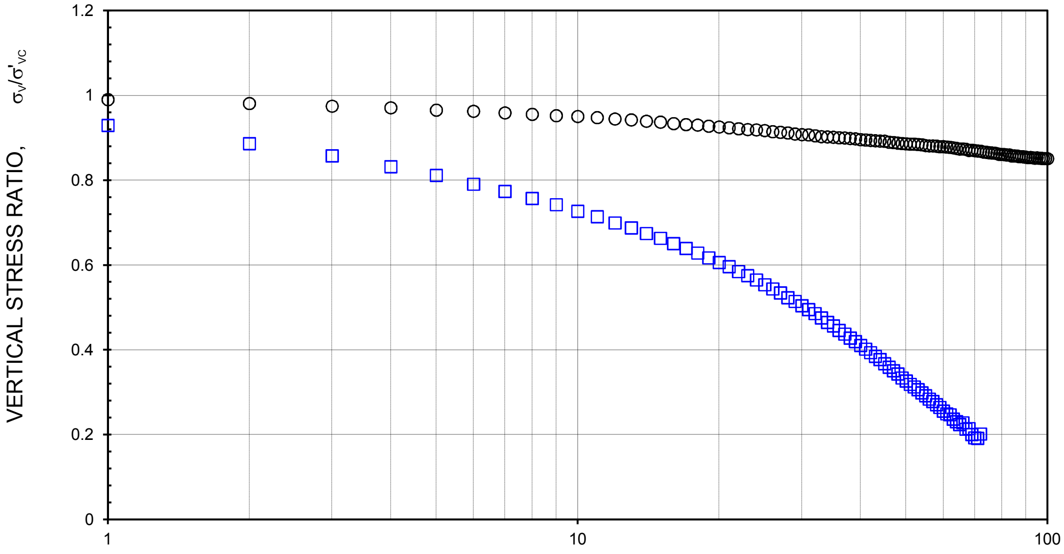
STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2			Project No.:	14-MTS-001
Location:	Oregon, USA	Borehole:	BH-4	Depth:	3.60 m
Sample:	S-1	Station:	DSS1	Date:	August 15, 2014

Average Stress Ratio: ○0.10 □0.167





NUMBER OF CYCLES (N)

Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	August 15, 2014	Date:	August 15, 2014	Date:	August 18, 2014

MEG TECHNICAL SERVICES

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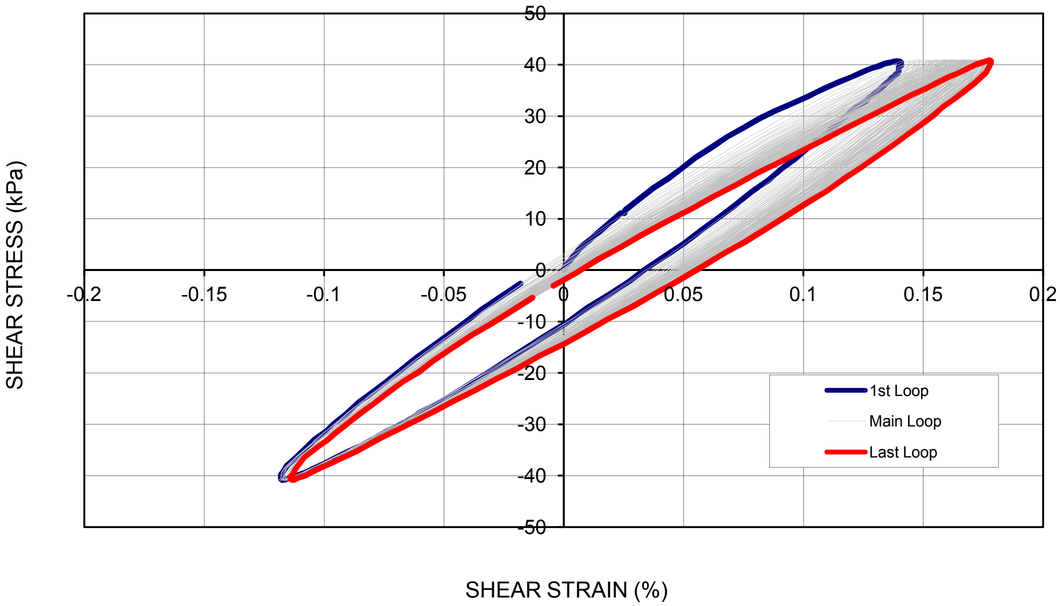
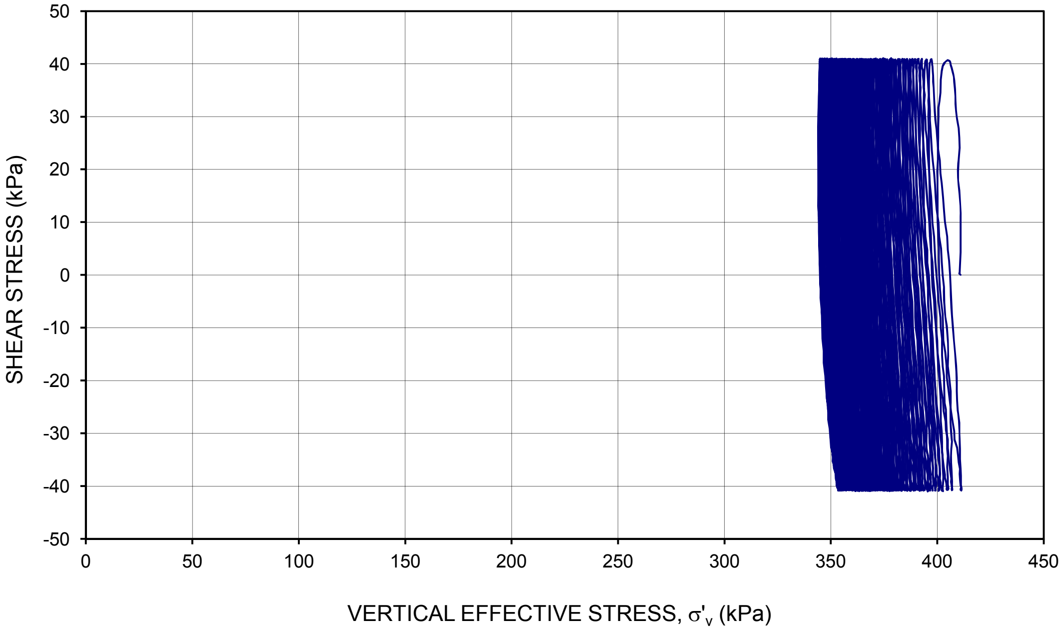
STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2			Project No.:	14-MTS-001
Location:	Oregon, USA	Borehole:	BH-4	Depth:	3.60 m
Sample:	S-1	Station:	DSS1	Date:	August 15, 2014

0.10 stress ratio (τ_{cyc}/ σ'_{vc}) @ 1 Hz for 100 cycles, σ'vc=411kPa





Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	August 15, 2014	Date:	August 15, 2014	Date:	August 18, 2014

MEG TECHNICAL SERVICES

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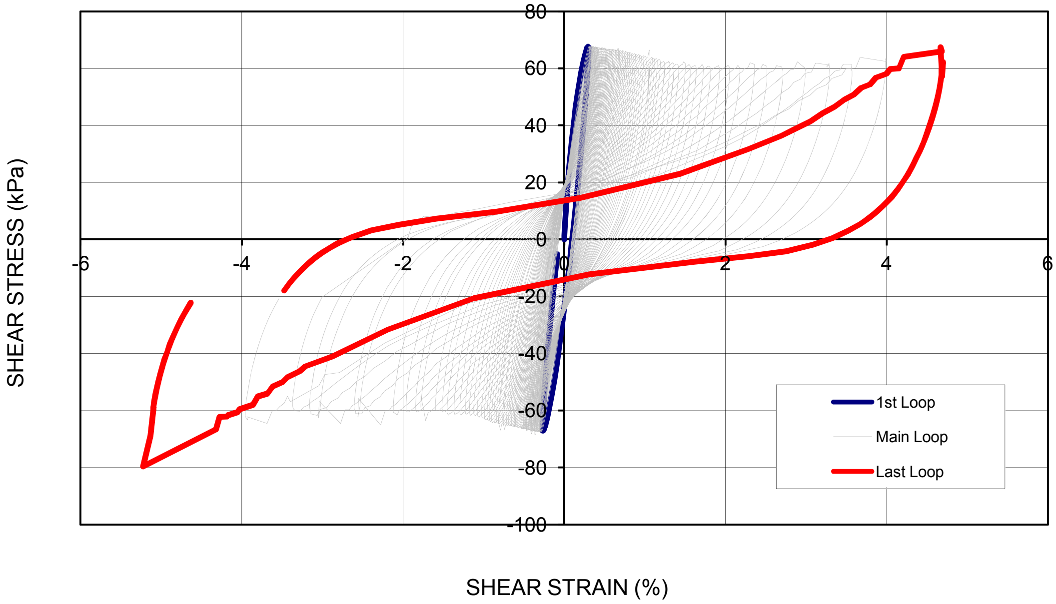
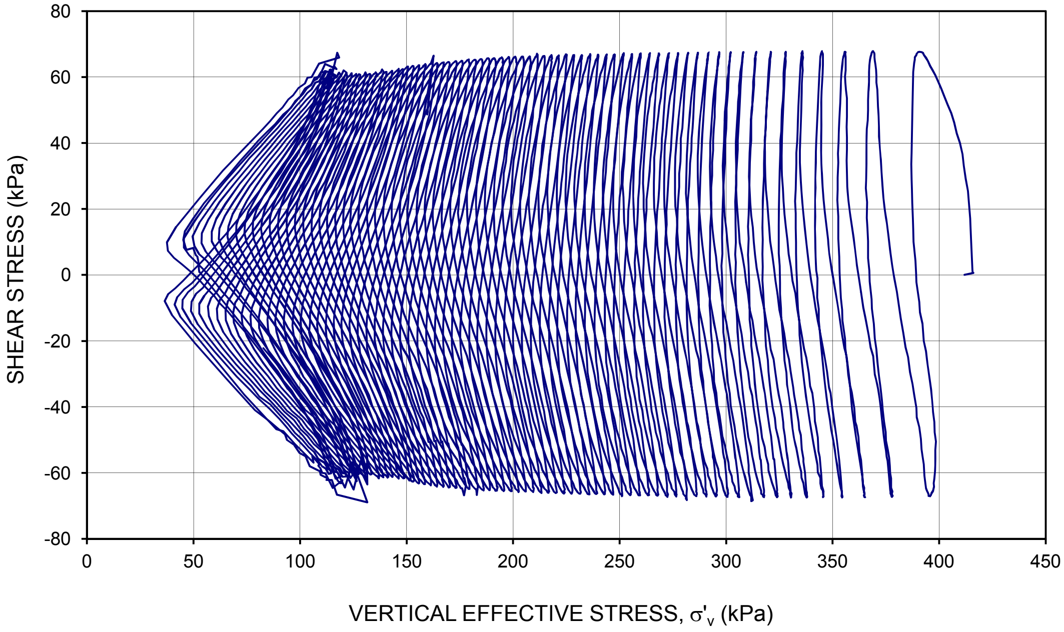
STRESS CONTROLLED CYCLIC DIRECT SIMPLE SHEAR TEST

Project: HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2
Project No.: 14-MTS-001
Location: Oregon, USA
Borehole: BH-4
Depth: 3.60 m
Sample: S-1
Station: DSS1
Date: August 15, 2014

0.167 stress ratio (τ_{cyc}/ σ'_{vc}) @ 1 Hz for 72 cycles, σ'vc=411kPa





Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	August 15, 2014	Date:	August 15, 2014	Date:	August 18, 2014

MEG TECHNICAL SERVICES

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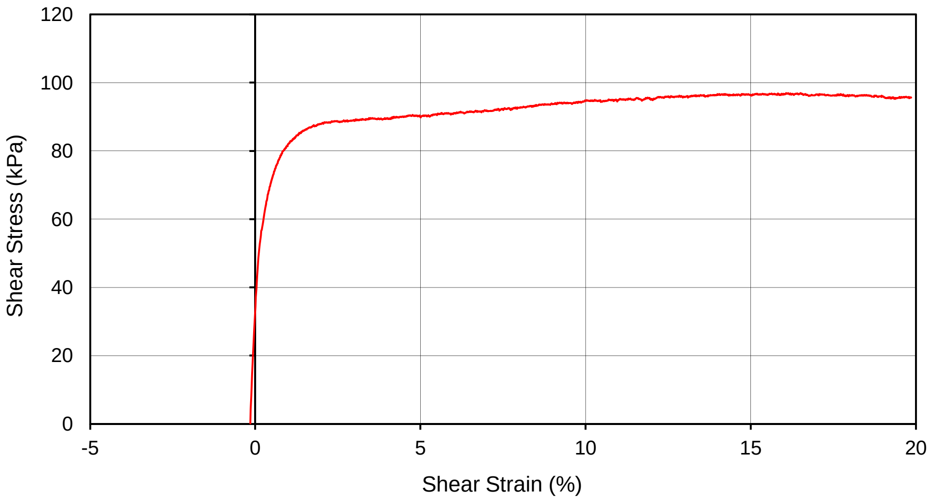
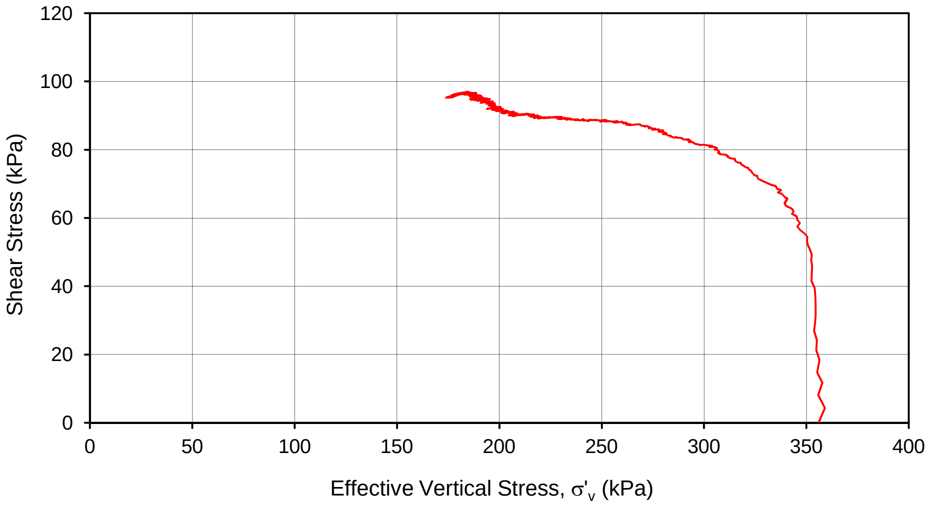
POST-CYCLIC STATIC DIRECT SIMPLE SHEAR TEST

Project:	HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2			Project No.:	14-MTS-001
Location:	Oregon, USA	Borehole:	BH-4	Depth:	3.60 m
Sample:	S-1	Station:	DSS1	Date:	July 30, 2014

POST-CYCLIC STATIC SHEAR TEST





Note: Test performed after stress-controlled DSS test at average cyclic stress ratio, CSR = 0.10 with 16% excess pore pressure. Post-cyclic tests performed under initial condition of zero shear stress

Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	July 30, 2014	Date:	July 30, 2014	Date:	July 31, 2014

MEG TECHNICAL SERVICES

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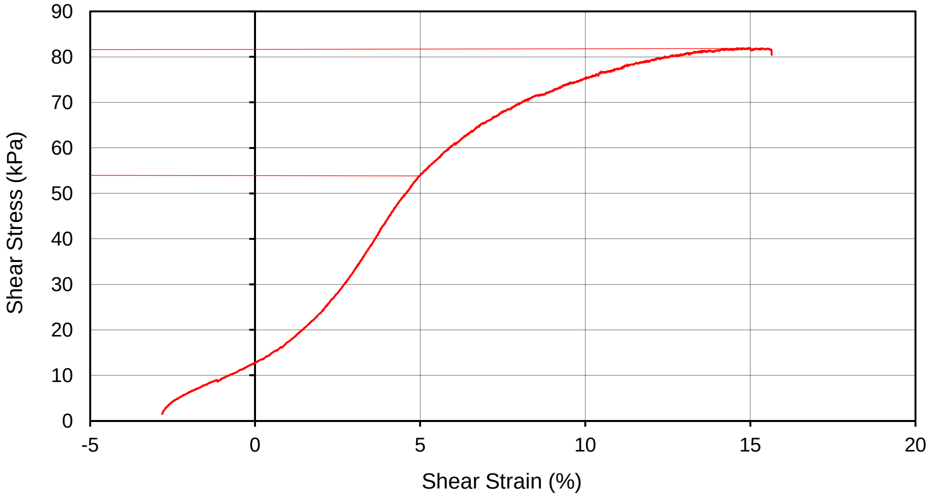
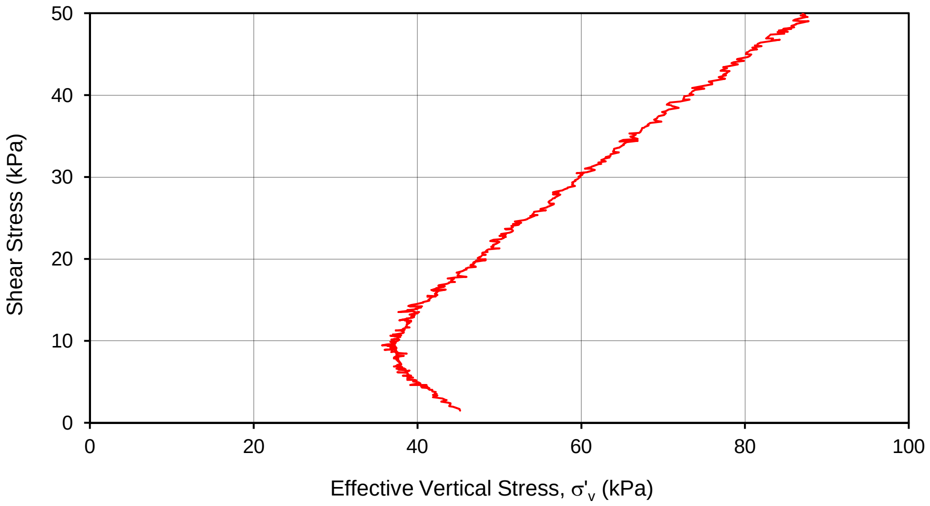
Marine + Earth MEG Geosciences

POST-CYCLIC STATIC DIRECT SIMPLE SHEAR TEST

Project: HDR Engineering - NewPort Oregon Big Creek Dam 1 & 2
Project No.: 14-MTS-001
Location: Oregon, USA
Borehole: BH-4
Depth: 3.60 m
Sample: S-1
Station: DSS1
Date: August 14, 2014

POST-CYCLIC STATIC SHEAR TEST





Note: Test performed after stress-controlled DSS test at average cyclic stress ratio, CSR = 0.22
with 87% excess pore pressure

Prepared By:	MF	Checked By:	PS	Approved By:	EP
Date:	August 15, 2014	Date:	August 15, 2014	Date:	August 18, 2014

Data Summaries and Plots

Consolidation Testing Results and Estimated Sample Quality, Table D-1.A.1

Boring	Sample	Depth		Estimated Preconsolidation Pressure				In-Situ Stress		OCR		Sample Quality Estimate $\Delta e/e_0$	Quality Rating
				Casagrande		Strain Energy				Casagrande	Strain-Energy		
		(m)	(ft)	(kPa)	(psf)	(kPa)	(psf)	(kPa)	(psf)				
BH-3	S-1	4.90	16.08	150.0	3132.8	105.0	2193.0	34.0	709.3	4.4	3.1	0.028	VG/E
BH-3	S-6	8.96	29.40	310.0	6474.5	162.0	3383.4	71.2	1486.4	4.4	2.3	0.043	F/G
BH-4	S-1	3.54	11.61	200.0	4177.1	137.0	2861.3	39.6	826.6	5.1	3.5	0.044	F/G
BH-4	S-6	8.15	26.74	210.0	4385.9	96.0	2005.0	91.2	1903.8	2.3	1.1	0.073	P to F/G
BH-4	S-9	12.37	40.58	220.0	4594.8	120.0	2506.3	138.2	2886.6	1.6	0.9	0.089	P
BH-5	S-1	3.53	11.58	180.0	3759.4	135.0	2819.5	34.5	720.3	5.2	3.9	0.075	P
BH-5	S-7	7.88	25.85	220.0	4594.8	130.0	2715.1	76.7	1601.6	2.9	1.7	0.044	F/G
BH-6	S-1	3.53	11.58	120.0	2506.3	120.0	2506.3	28.8	601.0	4.2	4.2	0.021	VG/E
BH-6	S-4	5.80	19.03	280.0	5847.9	166.0	3467.0	56.8	1187.3	4.9	2.9	0.039	F/G
BH-6	S-7	8.10	26.57	350.0	7309.9	162.0	3383.4	79.6	1662.8	4.4	2.0	0.038	VG/E or F/G

OCR	$\Delta e/e_0$ at in situ stresses for Quality Ratings 1 to 4			
	1 VG/E	2 F/G	3 P	4 VP
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

VG/E - Very Good to Excellent   Lunne et al. (2006)
F/G - Fair to Good
P - Poor
VP - Very Poor

Recalculated Casagrande Preconsolidation Pressures, Table D-1.A.2

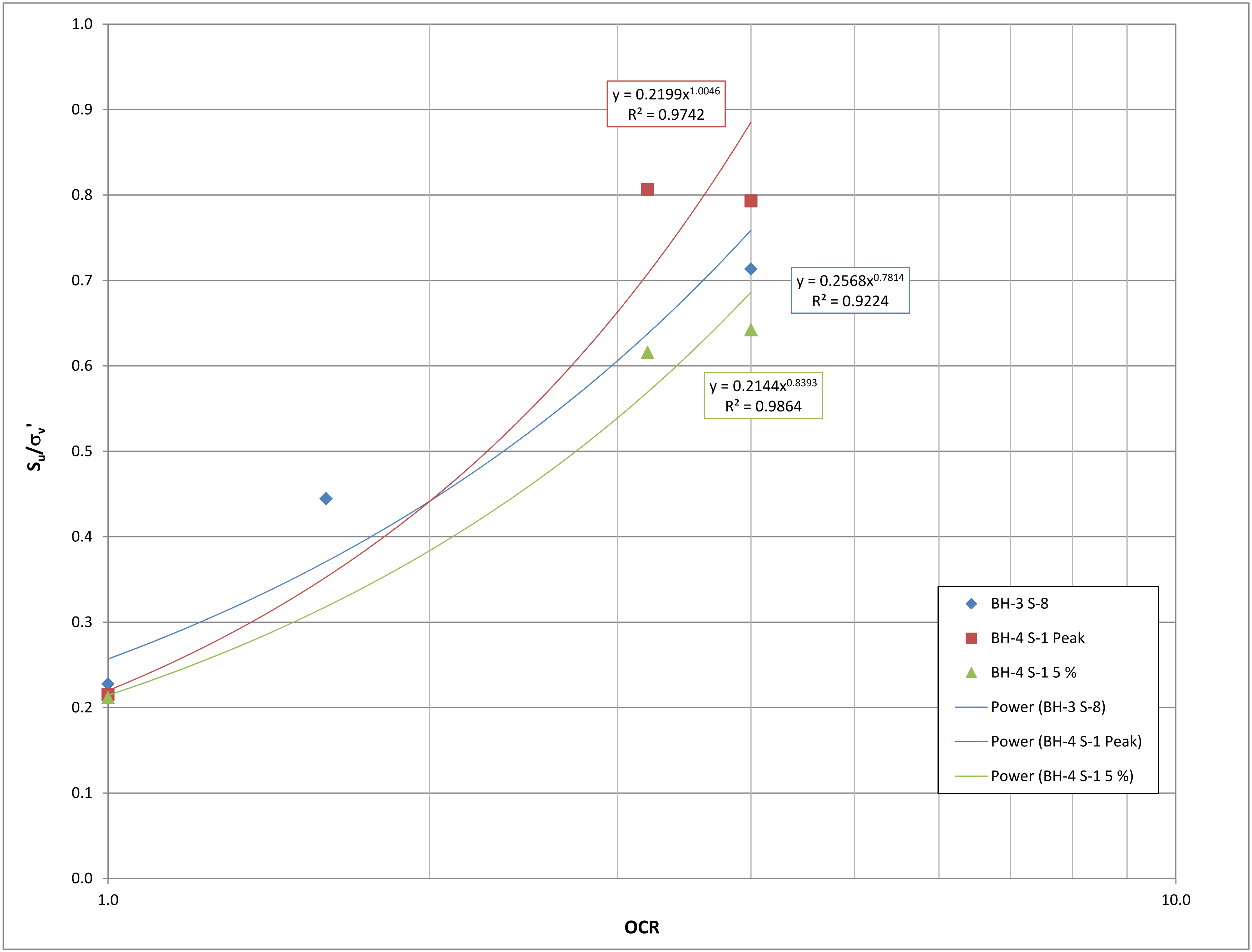
Boring	Sample	Depth		Estimated Preconsolidation Pressure								
				Revised Casagrande		Change in Preconsolidation Pressure	Strain Energy		In-Situ Stress		Revised OCR	
		(m)	(ft)	(kPa)	(psf)	(kPa)	(kPa)	(psf)	(kPa)	(psf)	Casagrande	Strain-Energy
BH-3	S-1	4.90	16.08	110	2297.4	-40.0	105.0	2193.0	34.0	709.3	3.2	3.1
BH-3	S-6	8.96	29.40	300	6265.6	-10.0	162.0	3383.4	71.2	1486.4	4.2	2.3
BH-4	S-1	3.54	11.61	180	3759.4	-20.0	137.0	2861.3	39.6	826.6	4.5	3.5
BH-4	S-6	8.15	26.74	110	2297.4	-100.0	96.0	2005.0	91.2	1903.8	1.2	1.1
BH-4	S-9	12.37	40.58	180	3759.4	-40.0	120.0	2506.3	138.2	2886.6	1.3	0.9
BH-5	S-1	3.53	11.58	200	4177.1	20.0	135.0	2819.5	34.5	720.3	5.8	3.9
BH-5	S-7	7.88	25.85	190	3968.2	-30.0	130.0	2715.1	76.7	1601.6	2.5	1.7
BH-6	S-1	3.53	11.58	120	2506.3		120.0	2506.3	28.8	601.0	4.2	4.2
BH-6	S-4	5.80	19.03	230	4803.6	-50.0	166.0	3467.0	56.8	1187.3	4.0	2.9
BH-6	S-7	8.10	26.57	260	5430.2	-90.0	162.0	3383.4	79.6	1662.8	3.3	2.0

Static Undrained Strength Testing - DSS and Triaxial, Table D-1.A.3

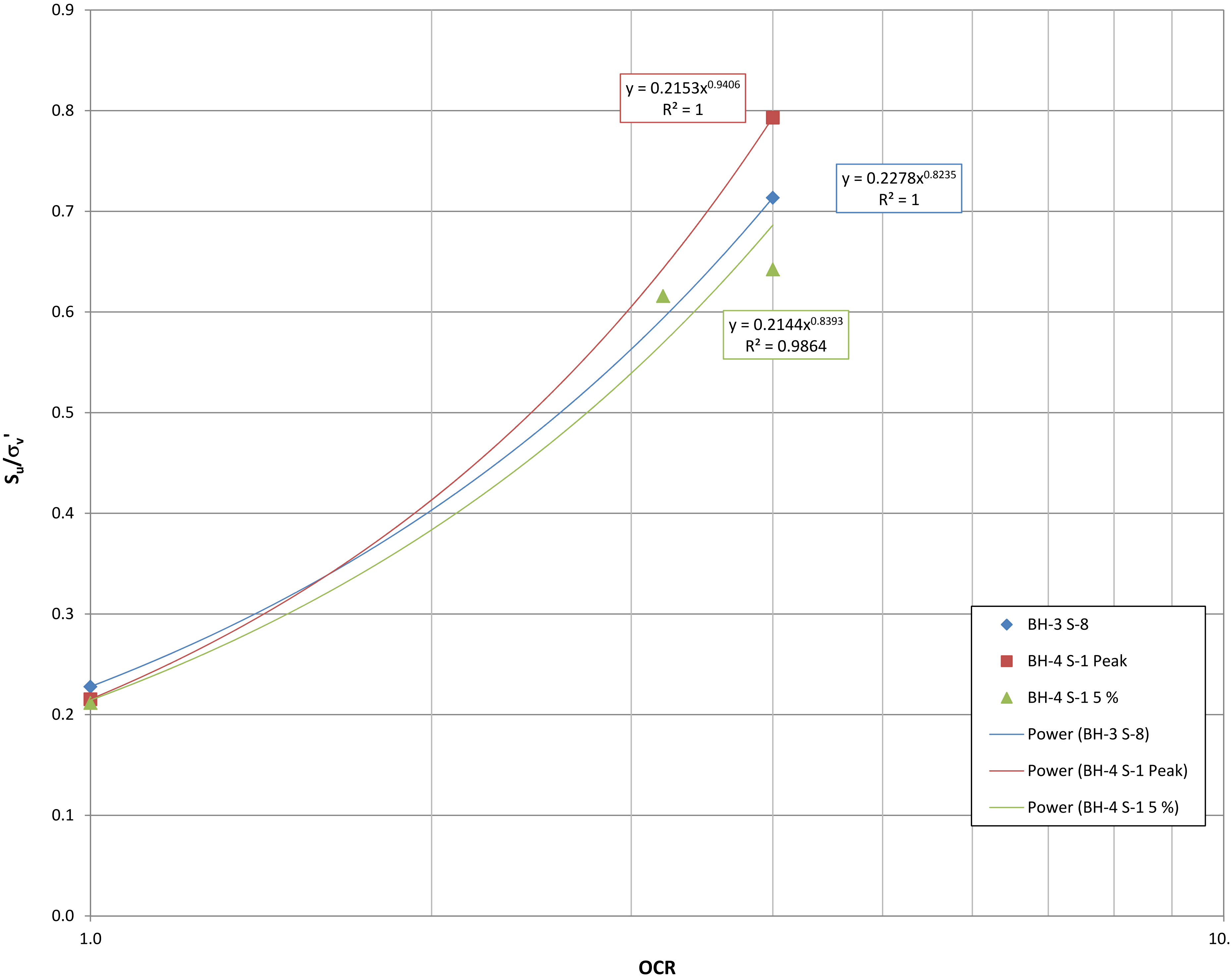
Dam	Boring	Sample	Depth (ft)	Depth (m)	Estimated In-Situ Effective Stress (kPa)	Estimated In-Situ Effective Stress (psf)	Estimated OCR	OCR in Testing	Type of Test	Estimated S_U from DSS, Peak (kPa)	Estimated S_U from DSS, Peak (psf)	Estimated S_U from DSS, @ 5 % Strain (kPa)	Estimated S_U from DSS, @ 5 % Strain (psf)	Testing Effective Vertical Stress (kPa)	S_U/p' Peak	S_U/p' 5 %	Initial Void Ratio, e_i	Final Void Ratio, e_f	Dry Unit Weight, γ_d (kN/m^3)	Dry Unit Weight, γ_d (pcf)	Natural Water Content, w (%)	Specific Gravity
BC-1	BH-3(U)	S-3	16	4.88	33.8	705.0	3.1	2.5	DSS	23.0	480.4	21.0	438.6				2.71	2.51	6.8	43.2	103.9	2.57
BC-1	BH-3(U)	S-6	28	8.53	67.0	1398.6	2.4	1.9	DSS	35.5	741.4	31.5	657.9	76.0	0.5	0.4	1.68	1.48	10.0	63.7	59.0	2.73
BC-1	BH-3(U)	S-8	31.5	9.60	77.6	1619.9	2	1.0	DSS	106.0	2213.9	103.0	2151.2	465.3	0.2	0.2	1.54	1.15	10.4	66.3	46.9	2.70^1
BC-1	BH-3(U)	S-8	31.5	9.60	77.6	1619.9	2	1.6	DSS	34.5	720.5	30.0	626.6	77.6	0.4	0.4	1.70	1.53	9.8	62.5	53.5	2.70^1
BC-1	BH-3(U)	S-8	31.5	9.60	77.6	1619.9	2	4.0	DSS	83.0	1733.5	79.0	1649.9	116.3	0.7	0.7	1.67	1.30	9.9	63.2	45.9	2.70^1
BC-1	BH-3(U)	S-8	31.5	9.60	77.6	1619.9	2		Triaxial	63.0	1315.8	54.0	1127.8				1.52		10.3	65.6	58.1	2.65^1
BC-1	BH-3(U)	S-9	37	11.28	91.6	1912.5	1.6	1.3	DSS	30.0	626.6	28.0	584.8				2.02	1.83	8.8	55.9	65.3	2.70^1
BC-1	BH-4(U)	S-1	10	3.05	34.1	712.0	4	3.2	DSS	40.2	839.6	30.2	630.7	34.0	1.2	0.9	1.27		11.5	72.9	47.3	2.65^1
BC-1	BH-4(U)	S-1	10	3.05	34.1	712.0	4	1.0	DSS	88.5	1848.4	87.0	1817.0	411.0	0.2	0.2	1.44	1.10	10.7	67.8	46.9	2.65^1
BC-1	BH-4(U)	S-1	10	3.05	34.1	712.0	4	3.2	DSS	27.5	574.3	21.0	438.6	34.1	0.8	0.6	1.36	1.23	11.0	69.9	49.8	2.65^1
BC-1	BH-4(U)	S-1	10	3.05	34.1	712.0	4	4.0	DSS	81.5	1702.2	66.0	1378.4	102.8	0.8	0.6	1.26	1.03	11.5	73.2	43.8	2.65^1
BC-1	BH-4(U)	S-4	22	6.71	75.2	1570.2	1.3	1.3	DSS	24.5	511.7	22.0	459.5	75.2	0.3	0.3	1.73	1.58	9.7	61.7	58.4	2.70^1
BC-1	BH-4(U)	S-6	26	7.92	88.8	1855.2	1.4	1.3	DSS	30.0	626.6	25.0	522.1	88.8	0.3	0.3	2.24	1.98	8.2	51.9	74.2	2.69
BC-1	BH-4(U)	S-9	40	12.19	136.2	2845.5	1.6	1.3	DSS	48.0	1002.5	44.5	929.4				1.35	1.24	11.6	73.8	44.8	2.78
BC-2	BH-5(U)	S-1	11	3.35	32.8	684.2	3.8	N/T		N/T	N/T	N/T	N/T				N/T	N/T	N/T	N/T	N/T	N/T
BC-2	BH-5(U)	S-7	25	7.62	74.0	1544.7	1.8	1.4	DSS	25.0	522.1	24.5	511.7				1.64	1.56	10.3	65.2	56.8	2.76
BC-2	BH-5(U)	S-9	28.5	8.69	85.2	1778.9	2	1.4	DSS	24.5	511.7	22.0	459.5				1.61	1.50	10.1	64.5	55.0	2.70^1
BC-2	BH-6(U)	S-1	11	3.35	27.3	570.9	3.9	3.1	DSS	25.0	522.1	20.0	417.7				2.05	1.89	9.0	57.1	54.5	2.79
BC-2	BH-6(U)	S-4	18	5.49	47.7	996.0	3.5	2.8	DSS	33.0	689.2	28.0	584.8				1.38	1.26	11.1	70.3	48.8	2.68
BC-2	BH-6(U)	S-7	26	7.92	80.7	1686.1	2	1.6	DSS	32.5	678.8	30.5	637.0				1.46	1.35	10.9	69.4	49.8	2.73

[1] Estimated Value

N/T = Not Testable



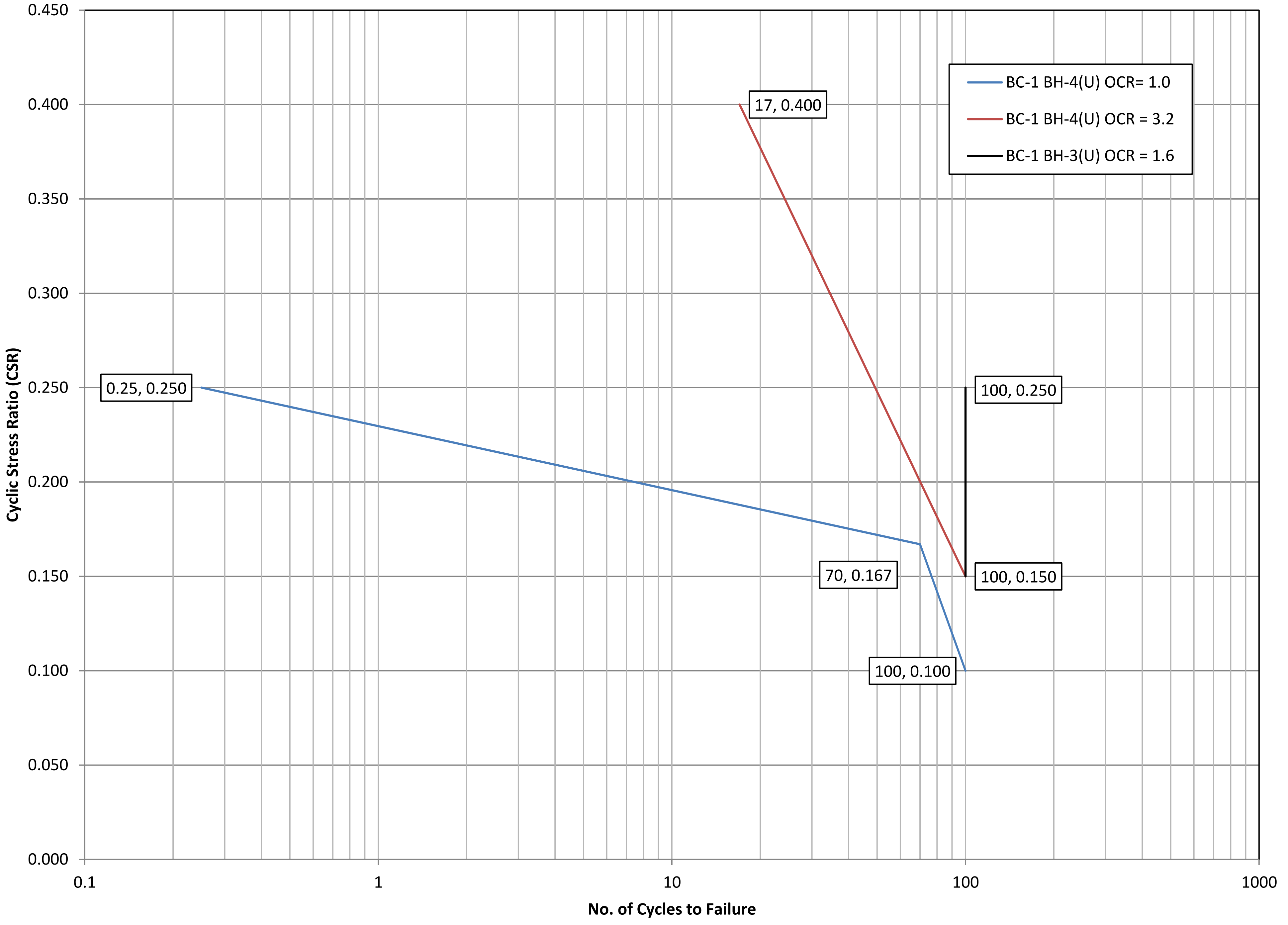
1.0
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
S_u/σ_v'
1.0
10.0
OCR
y = 0.2199x^1.0046
R² = 0.9742
y = 0.2568x^0.7814
R² = 0.9224
y = 0.2144x^0.8393
R² = 0.9864
BH-3 S-8
BH-4 S-1 Peak
BH-4 S-1 5 %
Power (BH-3 S-8)
Power (BH-4 S-1 Peak)
Power (BH-4 S-1 5 %)


y = 0.2153x^0.9406
R² = 1
y = 0.2278x^0.8235
R² = 1
y = 0.2144x^0.8393
R² = 0.9864
0.9
0.8
0.7
0.6
0.5
0.4
0.3
0.2
0.1
0.0
Su/σv'
1.0
10.0
OCR
BH-3 S-8
BH-4 S-1 Peak
BH-4 S-1 5 %
Power (BH-3 S-8)
Power (BH-4 S-1 Peak)
Power (BH-4 S-1 5 %)

Cyclic and Post Cyclic Monotonic Strength Testing - DSS - Stress Controlled, Table D-1.A.4

Dam	Boring	Sample	Depth (ft)	Depth (m)	Estimated In-Situ Effective Stress (kPa)	Estimated In-Situ Effective Stress (psf)	Estimated In-Situ OCR	OCR in Testing	Cyclic Stress Ratio	Number of Cycles to Failure	Post-Cyclic Monotonic Strength, $S_{u\text{-}post}$ 5% Strain (kPa)	Post-Cyclic Monotonic Strength, $S_{u\text{-}post}$ (kPa)
BC-1	BH-3(U)	S-8	31.5	9.60	77.6	1619.9	2	1.6	0.150	100	31	40.5
BC-1	BH-3(U)	S-8	31.5	9.60	77.6	1619.9	2	1.6	0.250	100	28.5	36
BC-1	BH-4(U)	S-1	10	3.05	34.1	712.0	4	1.0	0.100	100	90	96
BC-1	BH-4(U)	S-1	10	3.05	34.1	712.0	4	1.0	0.167	70	54	82
BC-1	BH-4(U)	S-1	10	3.05	34.1	712.0	4	1.0	0.250	0.25	N/A	N/A
BC-1	BH-4(U)	S-1	10	3.05	34.1	712.0	4	3.2	0.150	100	23	33
BC-1	BH-4(U)	S-1	10	3.05	34.1	712.0	4	3.2	0.400	100	N/A	N/A
BC-1	BH-4(U)	S-1	10	3.05	34.1	712.0	4	3.2	0.600	17	18	33

Cycles to Failure vs. Cyclic Stress Ratio (CSR)



Comparison of Laboratory Testing to CPT, Table D-1.A.5

Dam	Boring	Sample	Depth (ft)	Depth (m)	Estimated OCR	OCR from CPT	Ratio of OCR between Lab and CPT	Estimated S_U from DSS, Peak (kPa)	Estimated S_U from DSS, Peak (psf)	S_U from CPT (kPa)	S_U from CPT (psf)	Ratio of Lab to CPT for S_U
BC-1	BH-3(U)	S-3	16	4.88	3.1	17.9	0.17	23.0	480.4	29.3	611.9	0.78
BC-1	BH-3(U)	S-6	28	8.53	2.4	3.4	0.71	35.5	741.4	33.6	701.8	1.06
BC-1	BH-3(U)	S-8	31.5	9.60	2	3.6	0.56	63.0	1315.8	45.6	952.4	1.38
BC-1	BH-3(U)	S-9	37	11.28	1.6	5.8	0.28	30.0	626.6	101.8	2126.1	0.29
Average							0.43					0.88
Std. Dev.							0.25					0.46
COV							0.61					0.56
BC-1	BH-4(U)	S-1	10	3.05	4	12.3	0.33	40.2	839.6	26.2	547.2	1.53
BC-1	BH-4(U)	S-4	22	6.71	1.3	5.5	0.24	24.5	511.7	36.0	751.9	0.68
BC-1	BH-4(U)	S-6	26	7.92	1.4	4.3	0.33	30.0	626.6	38.1	795.7	0.79
BC-1	BH-4(U)	S-9	40	12.19	1.6	2.8	0.57	48.0	1002.5	52.0	1086.0	0.92
Average							0.36					0.98
Std. Dev.							0.14					0.38
COV							0.42					0.41
BC-2	BH-5(U)	S-1	11	3.35	3.8	115	0.03	N/T[2]	N/T[2]	27.8	580.6	
BC-2	BH-5(U)	S-7	25	7.62	1.8	161.3	0.01	25.0	522.1	14.9	311.2	1.68
BC-2	BH-5(U)	S-9	28.5	8.69	2	N/A[1]	N/A[1]	24.5	511.7	N/A[1]	N/A[1]	N/A[1]
Average							0.02					
Std. Dev.							0.02					
COV							0.79					
BC-2	BH-6(U)	S-1	11	3.35	3.9	26.9	0.14	25.0	522.1	35.3	737.3	0.71
BC-2	BH-6(U)	S-4	18	5.49	3.5	20.9	0.17	33.0	689.2	119.2	2489.5	0.28
BC-2	BH-6(U)	S-7	26	7.92	2	14.3	0.14	32.5	678.8	144.7	3022.1	0.22
Average							0.15					0.40
Std. Dev.							0.01					0.27
COV							0.11					0.71

N/A[1]	No CPT information at this depth
N/T[2]	No Testable Sample

Appendix D-2

Seismic Response Evaluation of RCC Dam Alternative A-2

1.0 Introduction

Previous site characterization and engineering analyses have confirmed that both Big Creek Dams No. 1 and 2 (BC 1 and BC 2, respectively), owned and operated by the City of Newport, have significant seismic response deficiencies that require corrective action. Both dams are under the jurisdiction of the Oregon Water Resources Department, Office of the State Engineer. In addition to the dam safety concerns, the City is also considering the need to increase long-term water supply through additional storage capacity within the system. A decision was recently reached to combine the consideration of the dam safety deficiencies at the Big Creek dams, and increased water supply needs through the evaluation of combined storage alternatives at the Big Creek dam sites.

Subsequently, three alternatives have been identified as possible solutions to a combined dam safety and increased water storage project for the City of Newport. One of these alternatives would involve the construction of a new Roller Compacted Concrete (RCC) dam across the stream channel immediately downstream of BC 2. This alternative has been assigned a designation of A-2. While several alternative storage capacities have been discussed, a reservoir with the maximum capacity has been selected for configuration level design and cost evaluation. This maximum capacity includes a combination of

1) The existing BC 2 capacity (970 acre-feet),
2) Recovery of storage lost in BC 2 due to sediment accumulation (100 acre-feet),
3) The storage capacity of BC 1 (to be abandoned; 200 acre-feet), and
4) A maximum required increased storage objective (1,000 acre-feet).

This corresponds to a total storage capacity to the crest of the principal spillway of 2,270 acre-feet.

An approximate area-capacity (A-C) curve was generated for the A-2 alternative dam site from existing LiDAR obtained topographic information. Based on this A-C curve, the principal spillway crest elevation was set at Elevation 112 feet. An allowance for routing the Probable Maximum Flood (PMF) through the reservoir was made by including a 30-foot-wide overflow spillway over the dam, and a crest elevation of 120 feet adjacent to the spillway overflow section.

An initial configuration design level layout of the dam in plan, profile, and along two cross sections was prepared based on recent experience with design of an RCC dam in a high seismic area. The initial cross section representative of the maximum height of the dam immediately adjacent to the central overflow spillway was used in the simplified seismic response evaluation. The simplified cross section is shown on Figure D-2.1 (All figures located at the end of this report). The following section presents the results of this seismic response evaluation along with appropriate conclusions and recommendations for the configuration design cross section of the dam.

1.1 RCC Dam Alternative A-2 Analysis Approach

A preliminary response spectra analysis of the proposed Alternative A-2 RCC dam cross section configuration was performed using the finite element program SAP2000. The results of the SAP2000 analyses were checked with a spreadsheet model based on Fenves and Chopra (1987) and a hand calculation check used for estimating maximum stresses during earthquake loading, overturning and sliding stability. Based on these initial response spectra analysis results, an additional model was established to perform a time history evaluation of the dam response including estimates of potential sliding deformation along a crack that could develop at the base of the dam in contact with the foundation bedrock.

The following sections outline the model geometry, and engineering properties are summarized in the subsections below. Loading conditions used in the analyses are summarized in Section 1.2. A summary of the analysis results is provided in Section 2.0. Figures and attachments detailing the analysis are appended at the end of this appendix.

1.1.1 Geometry

A generalized geometry of the preliminary RCC alternative section was developed for this initial assessment based on current state of the practice in RCC dam design and modified to account for the significant seismic loads anticipated for the site due to either a nearby crustal, or a Cascadia Subduction Zone (CSZ) design event. The representative section is shown on Figure D-2.1.

1.1.2 Material Properties

The material properties for the soil adjacent to the RCC dam are those described in the Appendix D1 Engineering Properties. Bedrock properties were estimated based on limited drilling into the underlying siltstone at the Dam No. 1 and Dam No. 2 sites. These rock properties including depth to the top of rock, rock strength (cohesion and friction angle) and rock modulus and will need to confirmed with a drilling and testing program in the proposed location of the RCC dam.

An estimated deformation modulus (E_D) of 2 x 10^6 psi and a Poisson's ratio of 0.3 was used for the bedrock in the model. Due to uncertainties and variability of the bedrock indicated by exploration results, a lower bound E_D of 1 x 10^6 psi was also considered. Typical properties based for the roller compacted concrete are based on U.S. Army Corps of Engineers (USACE) Engineer Manual EM 1110-2-2006 (2006), lift joints are assumed to be bonded and have an interface friction angle of 45 degree and cohesion of 125 psi. The RCC compressive strength and dynamic modulus of elasticity were assumed to be 2,500 pounds per square inch (psi) and 3.28×10^6 psi, respectively. The interface friction angle between the concrete and the underlying bedrock was assumed to be 45 degrees with an allowance for reduction of post seismic rock-concrete friction angle ranging from 30 to 45 degrees based on the significance of tension and sliding.

1.2 Loading Conditions

The following loading conditions were used in the analysis of the RCC dam section:

1.2.1 Uplift

A foundation drain efficiency of 37.5 percent was assumed based on an average of the USACE design recommendations (EM 1110-2-3506, 1984) and the U.S. Bureau of Reclamation (USBR) design criteria that allow consideration of a maximum suggested drain efficiency (USBR 1976). A section with zero drain efficiency following a large earthquake and significant sliding that would disrupt drain function also was evaluated (Section 2.0).

1.2.2 Hydrostatic load

A normal maximum reservoir pool elevation of 112 feet, and a tailwater elevation of 44 feet were used in the analyses.

1.2.3 Silt pressure and nominal dynamic earth pressure

Earth pressures corresponding to either silt or backfill to an elevation of 44 feet were included on the upstream face of the dam in the model.

1.2.4 Hydrodynamic force

Hydrodynamic forces from the reservoir pool were considered using the Westergaard added mass formulation.

1.2.5 Earthquake loading

Response spectrum corresponding to a CSZ and crustal events with a 4,975- and 2,475-recurrence interval were evaluated, see Figures D-2.3 and D-2.4

1.2.6 Extreme hydraulic loading:

PMF loading was not considered in the analysis at this time as the earthquake loadings described above were assumed to control the maximum stress conditions in the dam and any appropriate design configuration requirements.

2.0 Results Summary

The response spectra analysis yielded estimated Factors of Safety (FOS) under normal loading conditions for the no drain and with drain cases of 1.9 and 2.15, respectively. The analysis further indicated that no tension would exist under normal reservoir loading conditions and the full contact of the dam with the foundation rock would be in compression.

Due to the character of the spectral curves for the 4,975-year return period events, the crustal source (Figure D-2.3) yields larger stresses than Cascadia source (Figure D-2.4), response (about 30 percent larger).

For both cases, the earthquake horizontal force is larger than the weight of the section. It should be noted that in the response spectrum analysis only estimates of the maximum responses (stresses) are computed, unlike time history analysis, and does not allow evaluating the effect of earthquake duration or the number of pulses that result in stresses at or near the maximum computed stresses. Based on an assumed 45 degrees friction angle, the analyses indicated the potential for sliding in both upstream and downstream directions during an earthquake with an instantaneous minimum FOS of about 0.3. Consequently, the analyses suggest that significant base cracking at both upstream and downstream toes would occur. The FOS in sliding during earthquake and the tension zone at foundation suggests that an initial rock-concrete interface friction angle of greater than 45 degrees (before earthquake) would not change the conclusion of significance of sliding at rock-concrete interface. The results suggest that following sliding, it is possible that the drains would be ineffective and that a residual shear strength condition may exist along the foundation/rock interface and that it is assumed that the crack extends across the base of the dam. The high seismic tensile stresses computed at lift joints suggest that the lift joints would be expected to crack unless higher strength RCC is used. By increasing the bedding mortar strength, sliding would be limited to lower elevation joints or the foundation/rock contact that would still be sheared and opened up with or without gallery. The normal stresses associated with the earthquake load are high and as such the reduction in the uplift and placing the gallery would not meaningfully reduce the contact stresses during an earthquake and due to sliding the drains would most likely not be useful for post-seismic stability improvement.

Recommendations for uplift in the case of a fully propagated crack at the base for post seismic condition are not fully established. However, it is expected that uplift is a range between full reservoir pressure uplift (max) and linear distribution of reservoir head to tailwater head (min). For the full uplift condition the post-seismic stability FOS is always less than 1 for post-seismic friction angle of 43 degrees or lower.

In the case of linear uplift, a friction angle of 33 degrees or higher is required for an FOS of greater than 1.2.

Given the extent of tension and sliding, it can be concluded that with the preliminary analysis, sliding and extensive tension would occur at the base and refined analysis with the preliminary configuration would most likely not change the conclusion. If such damage is accepted and it can be assumed that the residual friction angle is greater than 33 degrees, a rigorous non-linear analysis would be required for final design. If the residual shear strength is about or less than 33 degrees, the section should be revised. A key factor for design can be provision of shear keys or similar mechanism to limit sliding during the earthquake and degradation of friction angle. High strength bedding mortar allows limiting sliding on lift joints and keeps friction angle at about 45 degrees, but at the rock concrete interface a mechanical mechanism is needed. Another option is to use anchors (this is not routine for new design) but would be extremely effective because the RCC weight and inertia force could be significantly reduced and at the same time compressive stress be added to the section. The anchor option to provide redundancy would be appealing especially for such high earthquake loading; although it would require provisions to avoid corrosion.

A study of a reduced foundation modulus, 2,475-year return period response spectra analysis and a single non-linear time-history analysis was performed as a follow on to the initial analyses.

The reduced foundation deformation modulus was used to estimate the response that is possible with a lower foundation deformation modulus. The deformation modulus of the underlying siltstone was not known and estimated values were used. The additional lower deformation modulus was used to estimate the variation of response with the variation of deformation moduli.

The reduced modulus lowers the peak stresses in both the upstream and downstream toes and also results in a smaller area of concentrated stresses. Figures D-2.6 and D-2.7 can be used as a general comparison of the changes from reducing the deformation modulus from 2×10^6 to 1×10^6 psi.

Both crustal and CSZ response spectra at the 2,475-year return period analysis were used to estimate the change in response between the previous analyses at 4,975-year return period.

As would be expected the higher frequency (lower return period) events reduced the stresses in both the upstream and downstream toe sections, as seen in Figure D-2.8. These stresses would also reduce the potential crack propagation lengths. Response spectrum analysis results in stresses that can be accepted for RCC, but causes tension at rock-concrete contact. Response spectra analysis also shows that sliding occurs in response to earthquake loading.

To estimate if drains used to assist in controlling uplift response continue to function and how much shear displacement occurs, a coarse mesh non-linear analysis using the Earthquake Analysis of Concrete Gravity Dams including Base Sliding program (EAGD-SLIDE; Chavez and Fenves 1994) was performed for one time history for each source, crustal and CSZ. Results show good comparison of stresses in a response spectrum analysis. The time-history analysis shows sliding of about 0.8 feet for the 4,975-year return period and 0.4 feet for the 2,475-year return period event. These analyses assumed an interface friction angle between the foundation and rock of 45 degrees, the actual friction angle my be higher or lower than this value and should be confirmed using direct shear testing with representative concrete and bedrock materials.

3.0 Conclusions and Recommendations

Based on the results of the preliminary analyses previously detailed, the following changes are recommended to the proposed RCC dam cross section to account for the issues identified in Section 2.0:

A heel section comprised of mass concrete that incorporates a shear key to assist in reducing the sliding response and also increasing the resistance to potential cracking in the most tension prone region would be needed. This mass concrete section would extend upward to a level approximately equal to the top of the drainage gallery.

The diameter of the drain holes would be sized to allow for the anticipated displacements while retaining a minimum level of efficiency to reduce the uplift potential.

Incorporation of these features into a non-linear time history analysis should provide the confirmation of the effectiveness of the design features for the final design of the RCC dam section.

4.0 References

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1999 EM 1110-2-6050, Response Spectra and Seismic Analysis for Concrete Hydraulic Structures

1995 EP 1110-2-12, Seismic Design Provisions for Roller Compacted Concrete Dams

Fenves, Gregory L., and Chopra, Anil K. 1987

Simplified Earthquake Analysis of Concrete Gravity Dams, Journal of Structural Engineering, Vol. 113, No. 8, August 1987, pp 1688-1708.

Figures

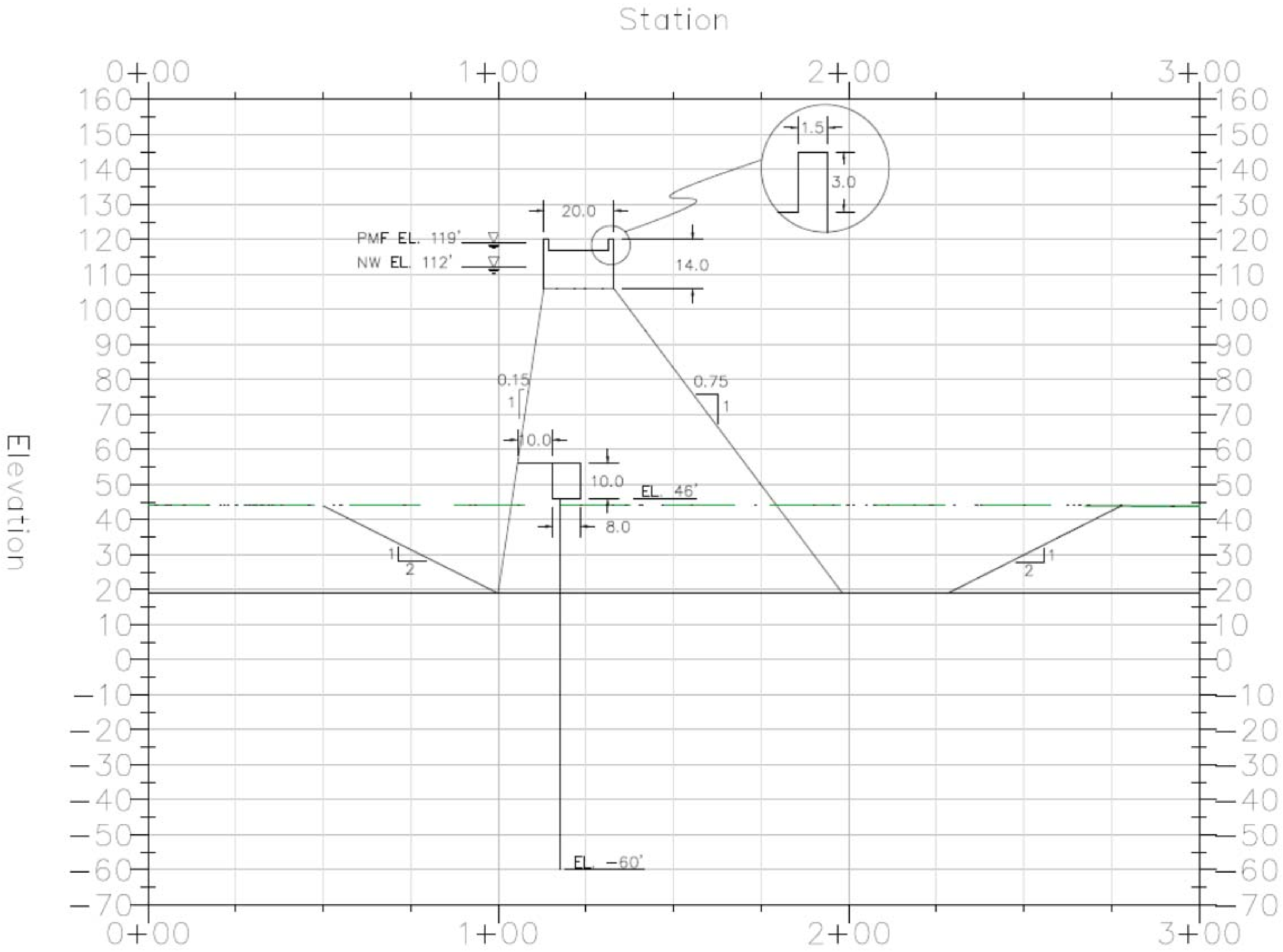
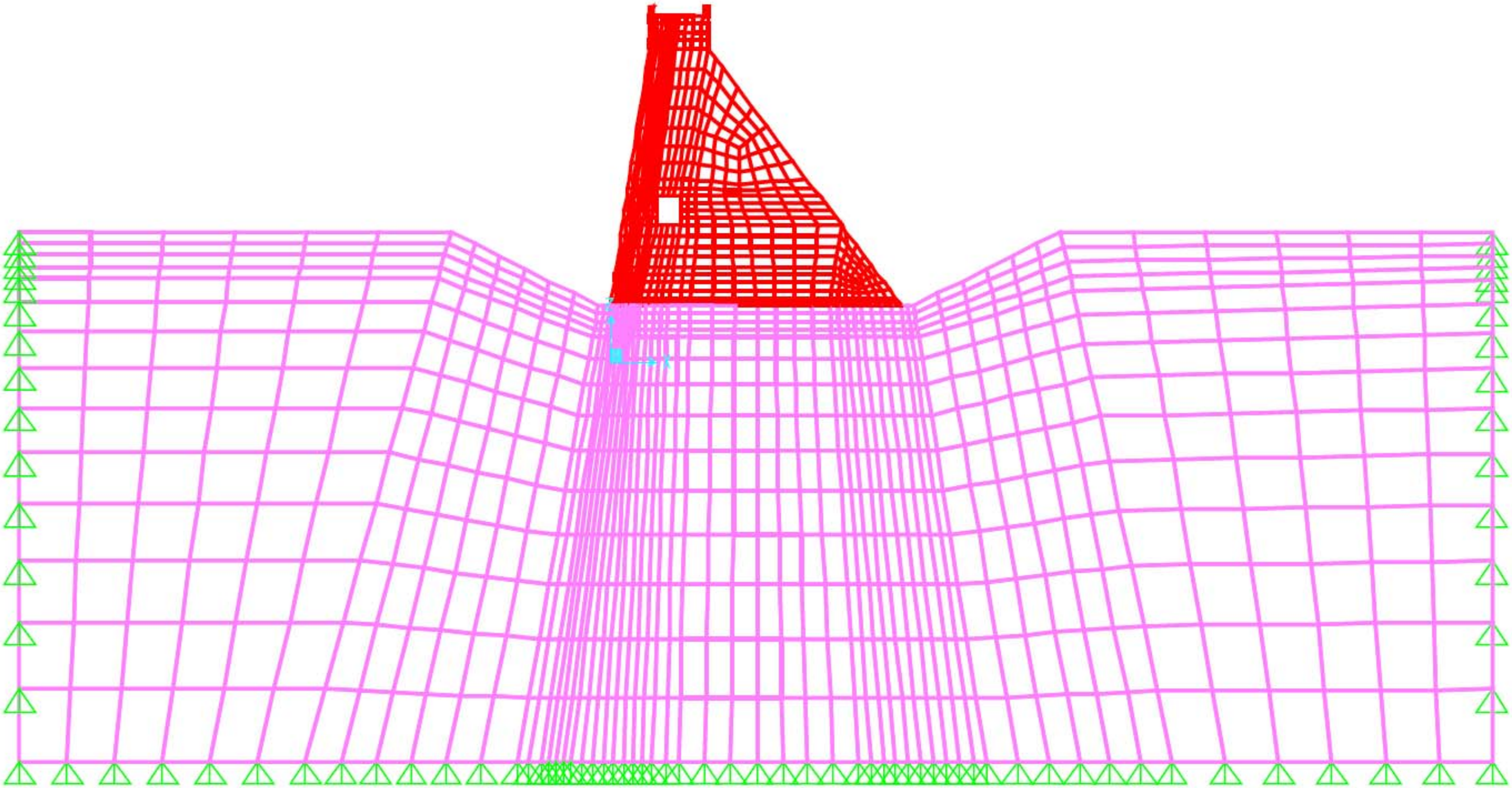


Figure D-2.1 – Initial RCC Dam Cross Section used in Seismic Response Evaluation



PROJECT NO. 221223	SAP2000 Finite Element Mesh		FIGURE
DRAWN: 05/12/15	BIG CREEK DAMS 1 AND 2 NEWPORT, OREGON		D-2.2
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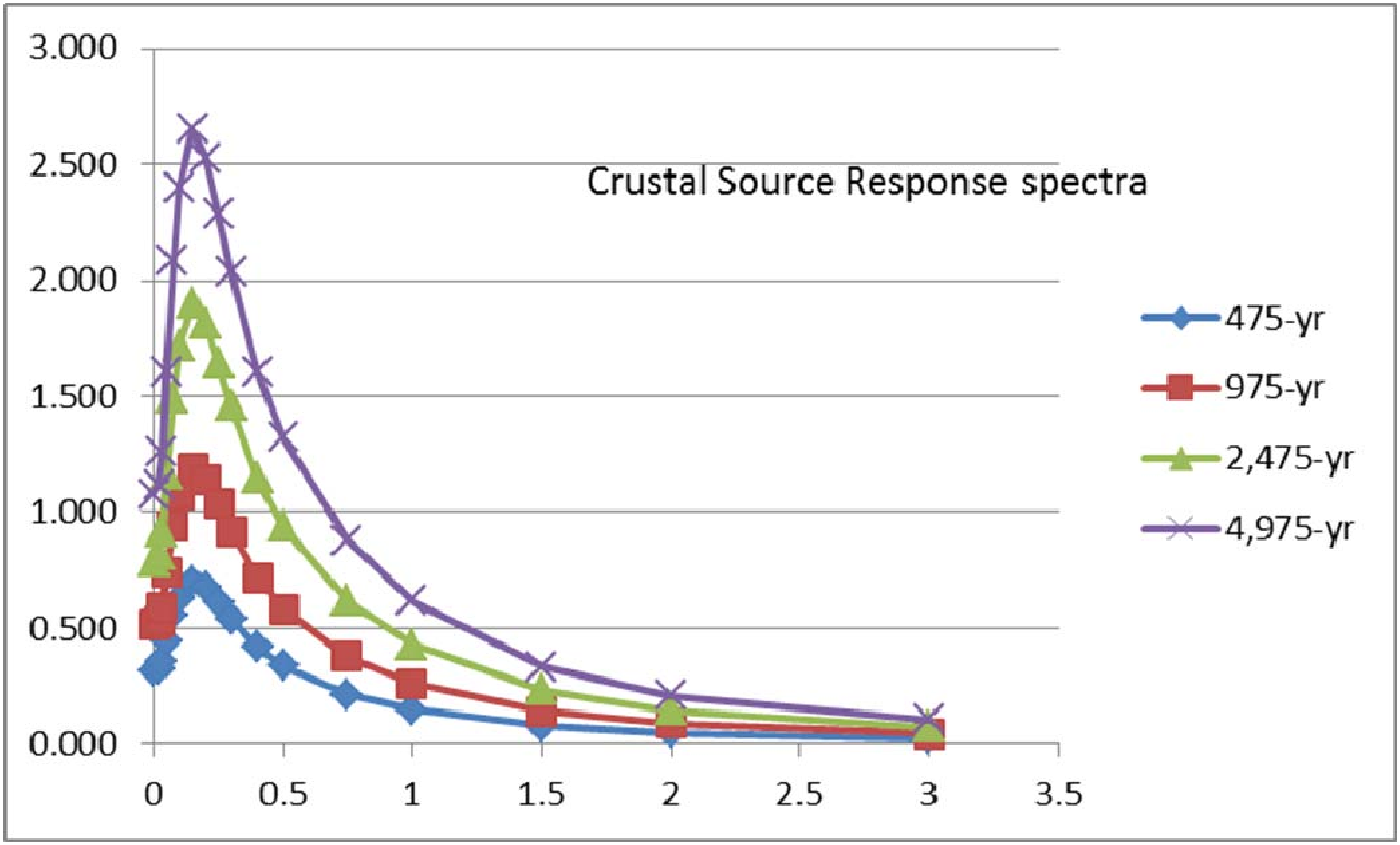

PROJECT NO.	221223	Response Spectra –Crustal Source		FIGURE
DRAWN:	05/12/15	BIG CREEK DAMS 1 AND 2 NEWPORT, OREGON		D-2.3
DRAWN BY:	STA			
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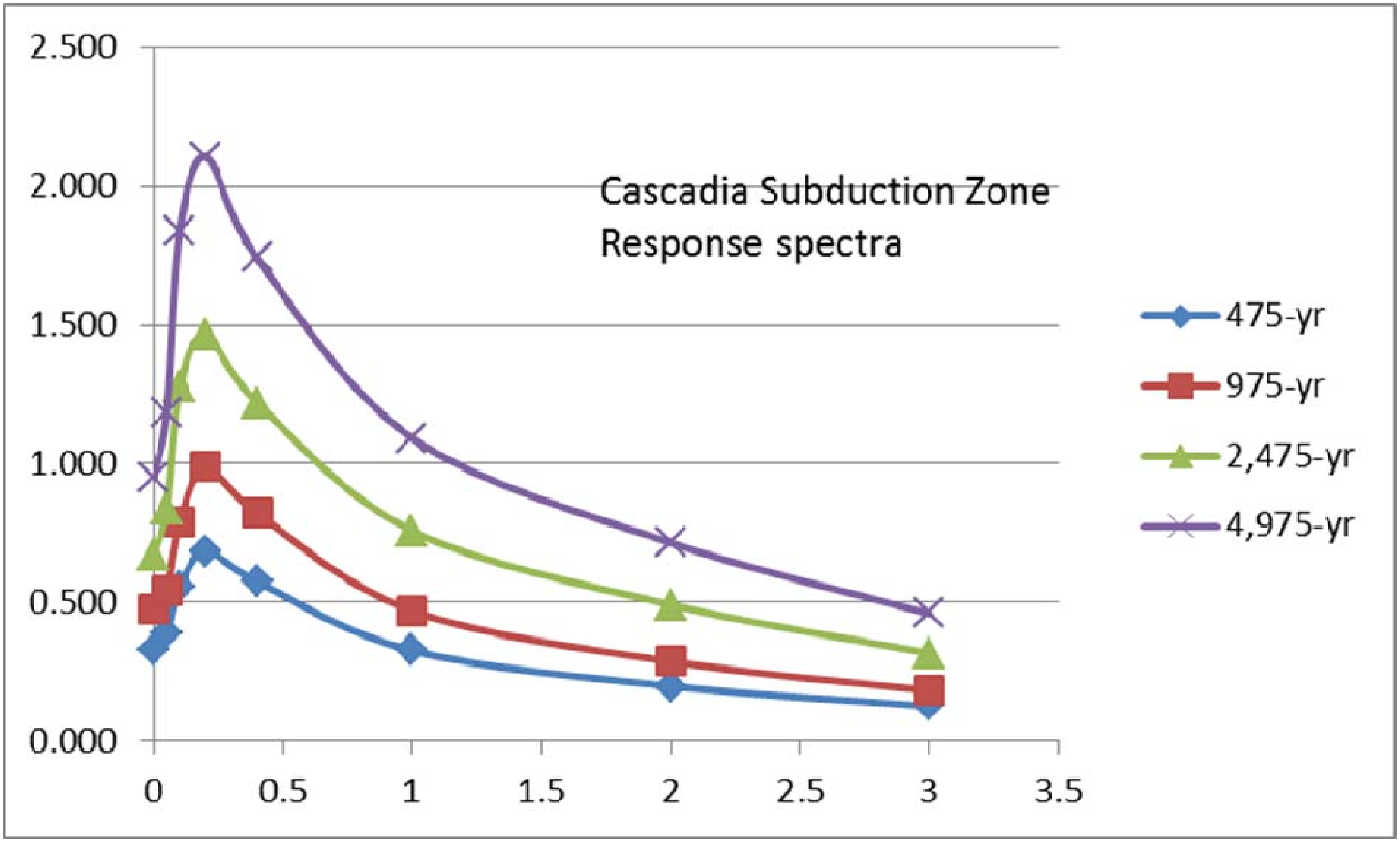

PROJECT NO. 221223	Response Spectra –CSZ Source			FIGURE
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DRAWN BY: STA				
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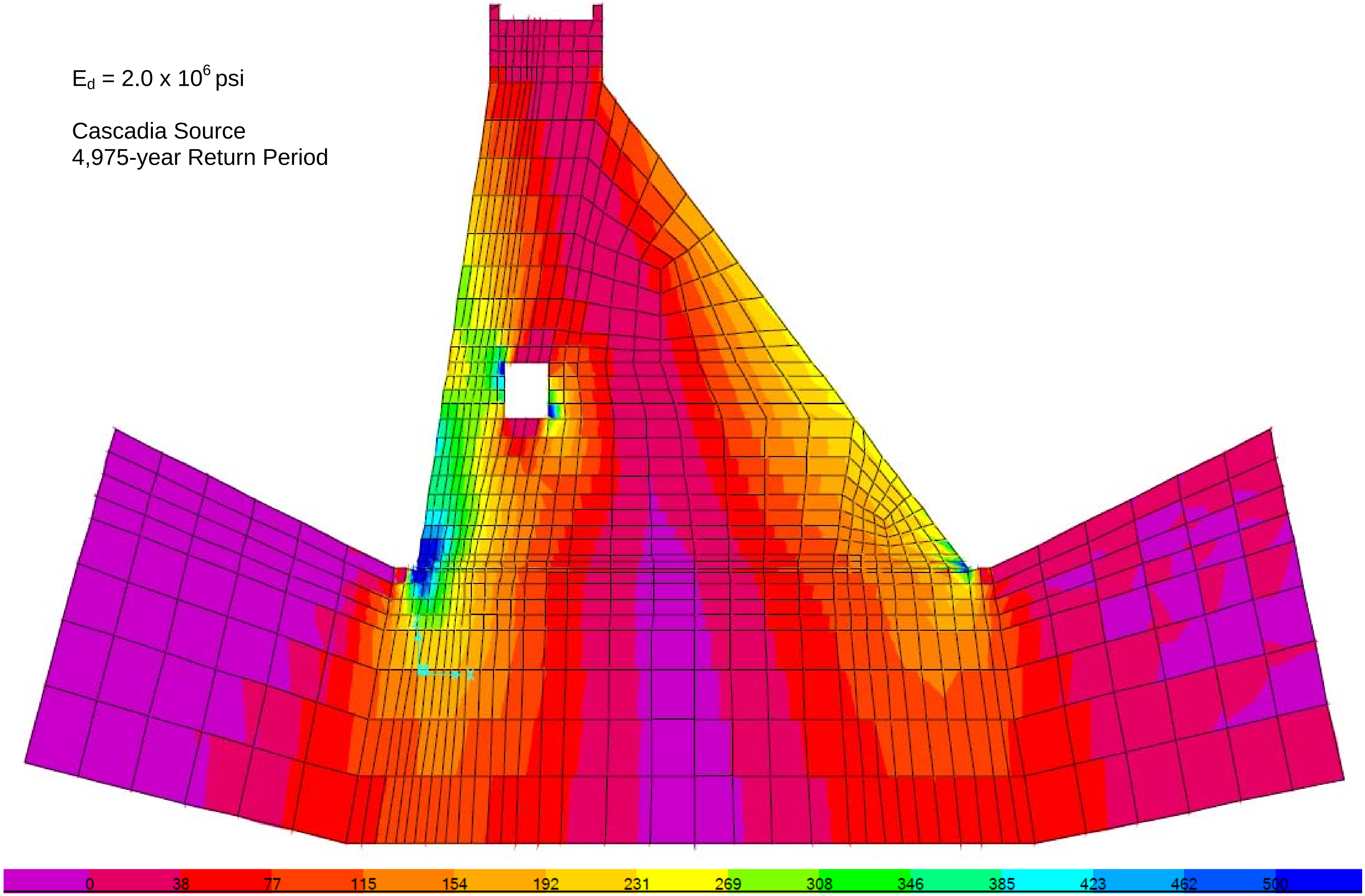
$E_d = 2.0 \times 10^6$ psi

Cascadia Source
4,975-year Return Period



Units—
pounds per
square inch
(psi)

PROJECT NO.	221223
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Stresses (S22) Response Spectra Analysis

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

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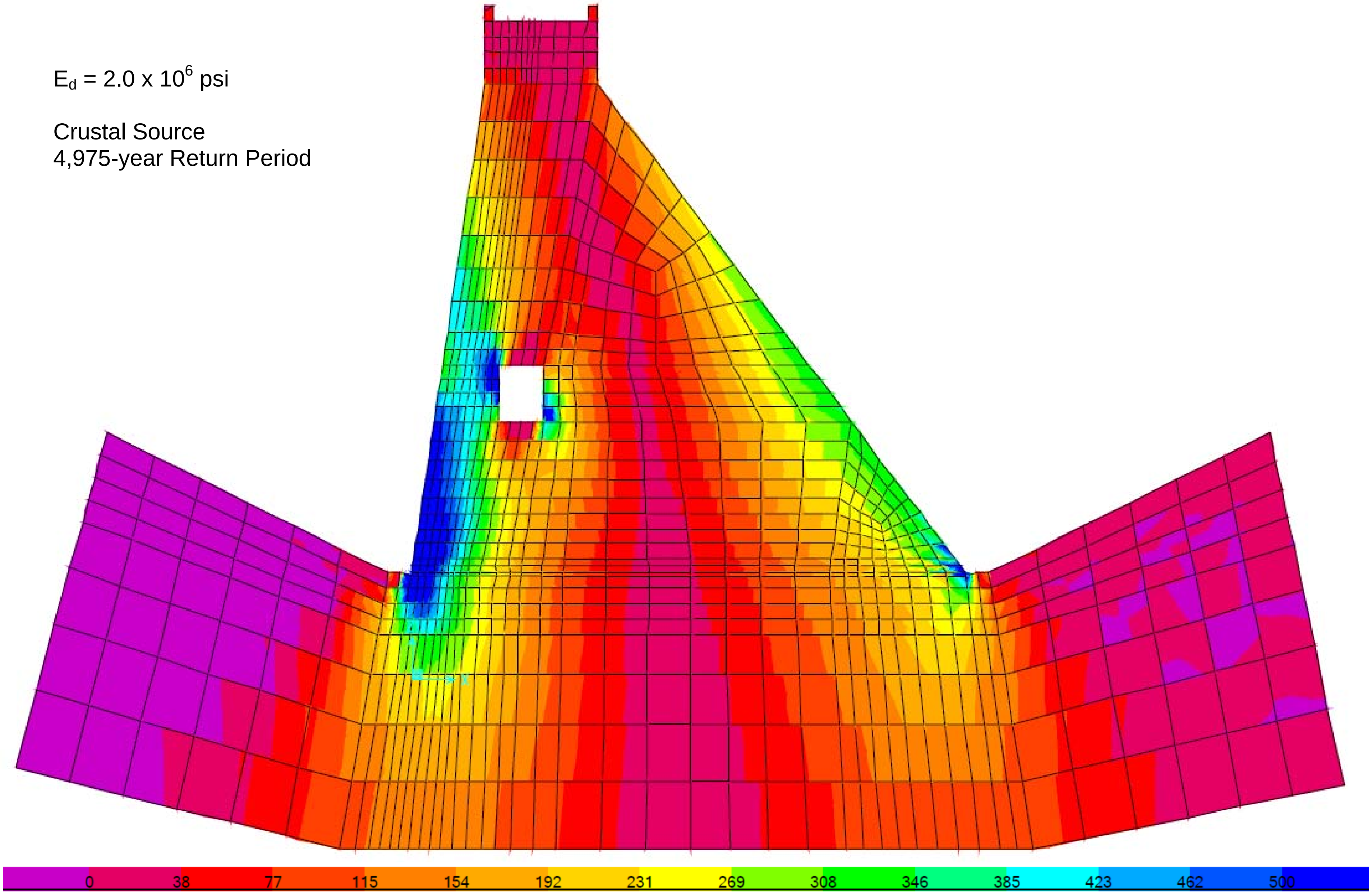
FIGURE
D-2.5



$E_d = 2.0 \times 10^6$ psi

Crustal Source
4,975-year Return Period

Units—
pounds per
square inch
(psi)

PROJECT NO. 221223
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Stresses (S22) Response Spectra Analysis

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

ORIGINATOR: <originated by>
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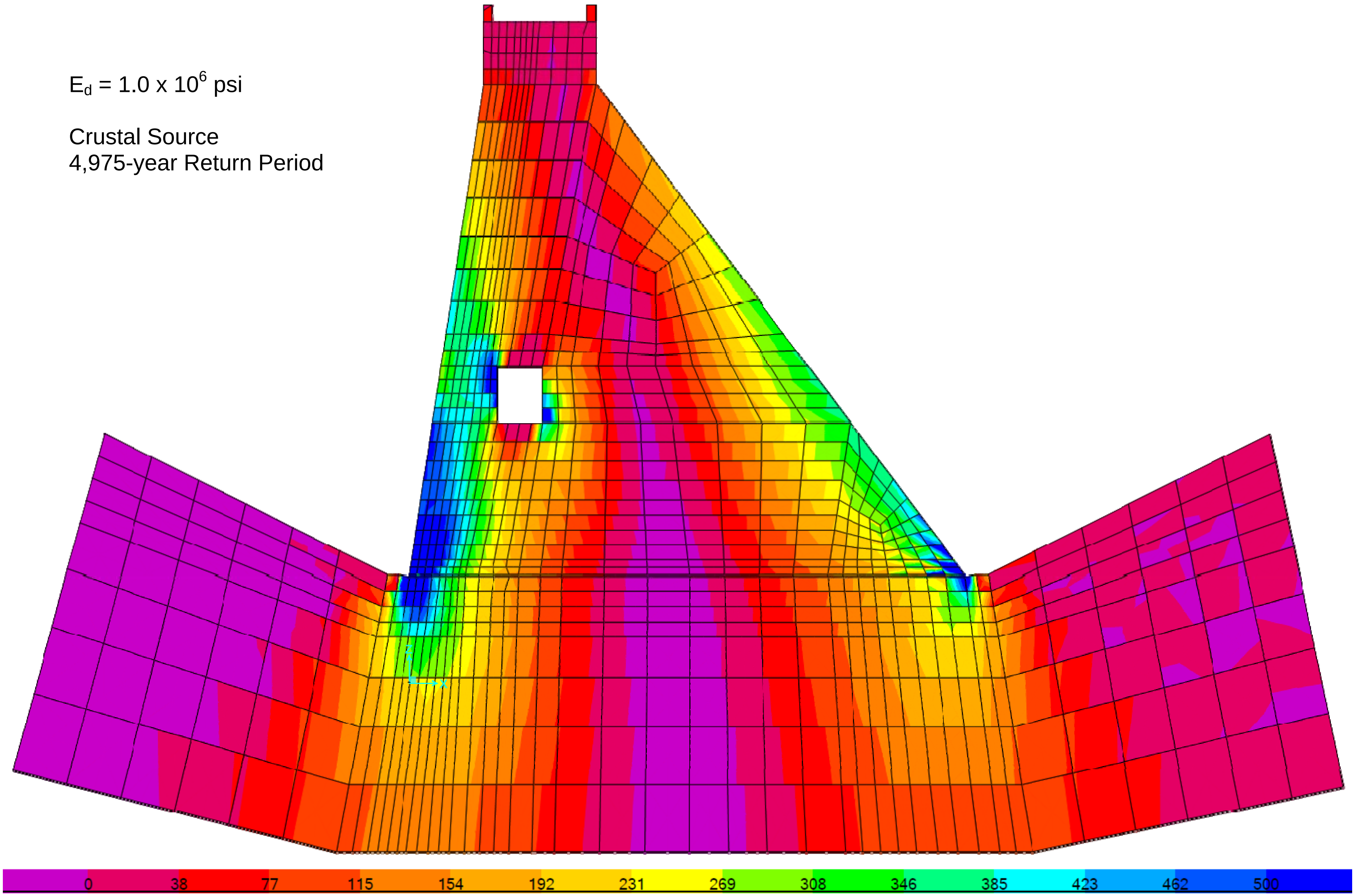
FIGURE
D-2.6



$E_d = 1.0 \times 10^6$ psi

Crustal Source
4,975-year Return Period

Units—
pounds per
square inch
(psi)

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Stresses (S22) Response Spectra Analysis

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

ORIGINATOR: <originated by>
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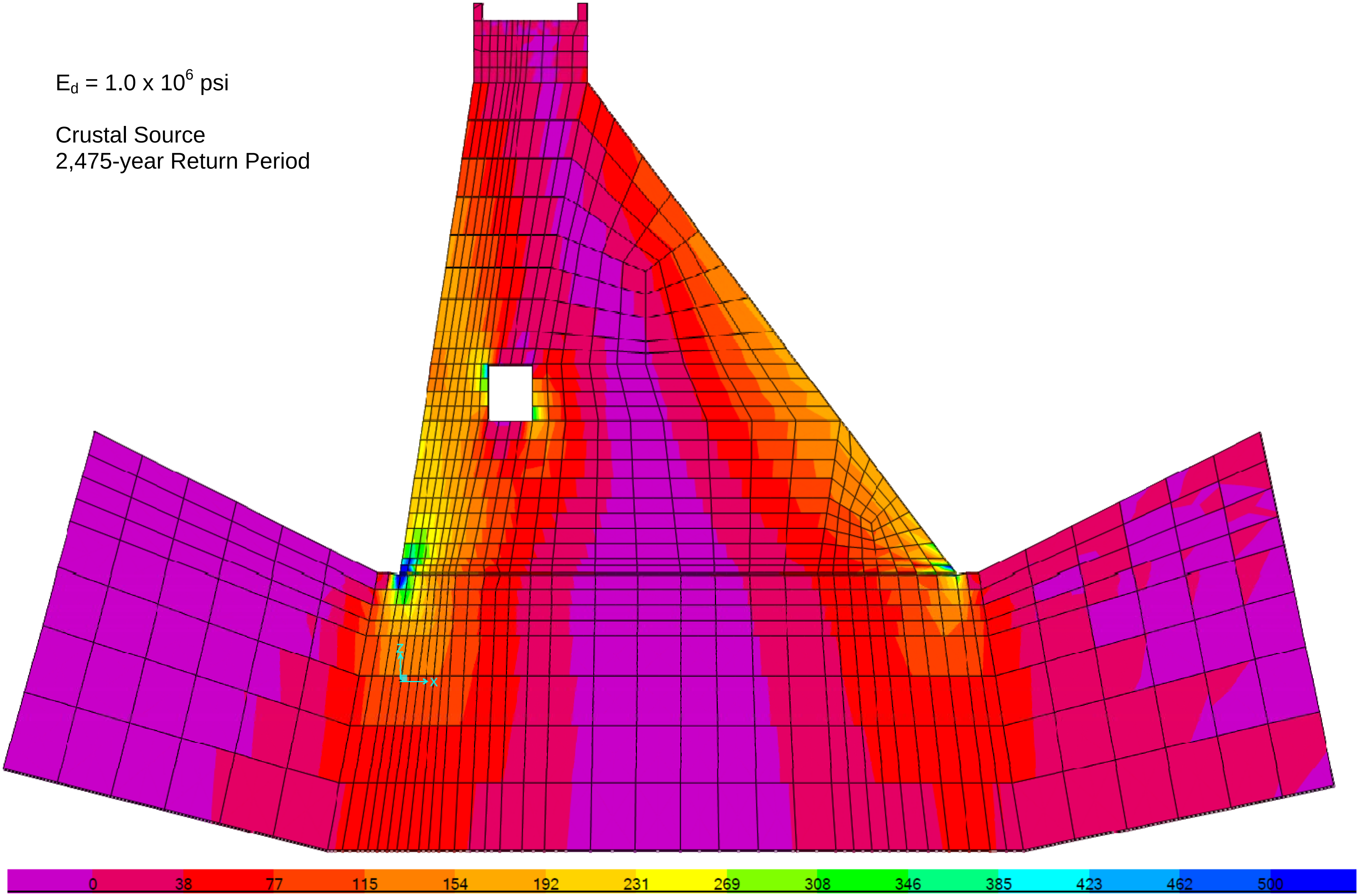
FIGURE
D-2.7



$E_d = 1.0 \times 10^6$ psi

Crustal Source
2,475-year Return Period

Units—
pounds per
square inch
(psi)

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Stresses (S22) Response Spectra Analysis

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

ORIGINATOR: <originated by>
APPROVED BY: <approved by>

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FIGURE
D-2.8

Appendix D-3

Evaluation of Embankment Dam Alternative A-3

1.0 Introduction

Previous site characterization and engineering analyses have confirmed that both Big Creek Dams No. 1 and 2 (BC 1 and BC 2, respectively), owned and operated by the City of Newport have significant seismic response deficiencies that require corrective action. Both of these dams are under the jurisdiction of the Oregon Water Resources Department, Office of the State Engineer. In addition to dam safety concerns, the City is also considering the need to increase long-term water supply through additional storage capacity within their system. A decision was recently reached to combine the consideration of the dam safety deficiencies at the Big Creek dams, and increased water supply needs through the evaluation of combined storage alternatives at the Big Creek dam sites.

Subsequently, three alternatives have been identified as possible solutions to a combined dam safety and increase water storage project for the City of Newport. One of these alternatives would involve the construction of a new embankment dam across the stream channel immediately downstream of BC 2. This alternative has been assigned a designation of A-3. While several alternative storage capacities have been discuss, a reservoir with the maximum capacity has been selected for configuration level design and cost evaluation. This maximum capacity includes a combination of:

1) The existing BC 2 capacity (970 acre-feet),
2) Recovery of storage lost in BC 2 due to sediment accumulation (100 acre-feet),
3) The storage capacity of BC 1 (to be abandoned)(200 acre-feet), and
4) A maximum required increased storage objective (1,000 acre-feet).

This corresponds to a total storage capacity to the crest of the principal spillway of 2,270 acre-feet.

An approximate area-capacity (A-C) curve was generated for the A-3 alternative dam site from existing LiDAR obtained topographic information. Based on this A-C curve, the principal spillway crest elevation was set at elevation 112 feet. An allowance for routing the Probable Maximum Flood (PMF) through the reservoir was made by including a 30-foot-wide overflow spillway over the dam, and a total crest elevation of 128 feet.

A configuration design level layout of the dam in plan, profile, and along the cross section is shown on Figure 4 of the main report.

1.1 Embankment Dam Alternative A-3 Analysis Approach

The proposed new embankment dam section (Alternative A-3) was analyzed for static and pseudo-static slope stability, along with estimated deformations based on Newmark-type analyses. The proposed dam section is a homogenous earthen dam with an internal filter system and 3 Horizontal to 1 Vertical (3H:1V) upstream and downstream slopes, the foundation is assumed to be founded directly on the underlying bedrock with the removal of existing alluvium and/or colluvial soils.

1.1.1 Material Properties and Loading Conditions

The material properties for the new embankment section are the described in detail in the Appendix D-1 Engineering Properties. The anticipated distributions of materials in and adjacent to the new embankment section are shown in Figure D-3.1 (All figures located at the end of this report), along with the anticipated geometry. An inclined filter and blanket drain is included in the analysis, but the final geometric configuration of the drainage system may differ from the initial conceptual analyses.

The full reservoir head is applied to the upstream face, to the maximum pool elevation of approximately 112 feet mean sea level of the dam for the analyses.

2.0 Results Summary

Static slope stability analyses were performed using the drained, undrained and post-earthquake parameters, as evaluated in Appendix D-1. Seepage analyses were used to evaluate the potential seepage patterns for upper and lower bound permeability parameters as discussed in the Appendix D-1. Static analysis cases considered maximum pool steady state seepage conditions drained strengths (long-term conditions), undrained strengths (short-term conditions) for both the up and downstream slopes. Both undrained (peak strengths) and post-earthquake (degraded or residual strengths) were evaluated for pseudo-static conditions for the upstream and downstream slope. A rapid drawdown analysis also was performed to evaluate the stability of the upstream slope during a rapid drawdown event.

2.1 Slope Stability Analysis Results

Slope stability analyses were performed for both the upstream and downstream slopes of the proposed embankment dam maximum section. Seepage parameter assumptions made some differences in the phreatic surfaces calculated, as seen in Figures D-3.2 and D-3.3. The changes in phreatic surface generally resulted in lower Factor of Safety (FOS) values for the upper bound seepage parameters.

In general, based on the strength parameters estimated from the laboratory testing program, the FOS values calculated and shown in Table D-3.1 indicate that the dam is stable under drained, peak undrained, and post-earthquake conditions at full reservoir loading.

Table D-3.1. Slope Stability Analysis Results for New Big Creek Embankment Dam

	Factor of Safety			
	Case 1(1)		Case 2(1)	
Section	**DS**	**US**	**DS**	**US**
Drained Strength Parameters	1.73	3.32	1.64	3.25
Peak Undrained Strength Parameters	2.52	4.02	2.59	3.93
Post Earthquake Undrained Strength Parameters	2.78	3.22	2.62	3.14

[1] *Case 1: Lower Bound Seepage Parameters*
[2] *Case 2: Upper Bound Seepage Parameters*

Graphical results from the slope stability analyses are shown in Figures D-3.4, D-3.5, D-3.6 and D-3.7. The drained strength results are not necessarily indicative of the actual FOS; as the drained strengths, which are generally higher than undrained strengths, were not evaluated as part of this or previous analyses. However, given the FOS values for a static undrained stability analysis, it can be assumed that the drained strength FOS values are at least those indicated in Table D-3.1.

A preliminary rapid drawdown analysis was performed using the Duncan-Wright triple stage procedure outlined in U.S. Army Corps of Engineers' (USACE) EM 1110-2-1902 Slope Stability (2003). The analysis was performed using the assumed drained strengths and Mohr-Coulomb parameters derived from the SHANSEP parameters. SHANSEP parameters cannot be used directly in the rapid drawdown method of Duncan and Wright, so SHANSEP strengths are converted to Mohr-Coulomb strengths for the analysis, the embankment materials used a constant undrained strength of ,S_u, 1,500 psf and drained strengths with an effective cohesion, c', of 200 psf and effective friction angle, ϕ', of 34 degrees. The alluvial soils assumed a constant undrained strength of ,S_u, 720 psf and drained strengths with a negligible effective cohesion and effective friction angle, ϕ', of 34 degrees. Since the elevation of the low level outlet for the outlet works has yet to be determined, it is assumed to be 60 feet, for a drawdown of approximately 52 feet. The FOS calculated for this case was 1.35, which is above both USACE and U.S. Bureau of Reclamation (USBR) criteria of 1.1 to 1.3.

2.2 Newmark Analysis Results

Newmark sliding block analyses were performed for the new embankment dam configuration. In addition to the rigid block analyses, both coupled and uncoupled sliding block analyses were performed.

Slope stability analysis using both the peak undrained and post-earthquake strengths were used to evaluate the yield acceleration of the dam cross section. The pseudo-static slope stability is performed and the seismic coefficients are varied until the FOS is approximately 1.0, indicating the point of anticipated failure. The vertical component of the seismic coefficient is taken as 50 percent of the horizontal component due to phase lag in the vertical wave with respect to the horizontal shear wave.

Both circular, wedge, and nonlinear shaped failure surfaces were considered with nonlinear surfaces based on both the circular and wedge types of analyses yielding the smallest yield coefficients. The surfaces based on the circular base shape were evaluated to have the lowest critical accelerations and tended to be in the alluvial materials, upstream and downstream of the new embankment dam, this would most likely lead to a widening of the base of the dam and potential cracking. Wedge shaped surfaces intercepting the crest yielded higher yield coefficients, although nearly identical for both the undrained and post-earthquake strengths.

Table D-3.2 lists the calculated yield coefficients, k_y (g), for both the upstream and downstream slopes for both the peak and post-earthquake undrained strengths.

Table D-3.2. Yield coefficients

Strength Envelope	Yield Acceleration	
	DS	US
Peak Undrained Strength Parameters - Circular	0.370	0.365
Peak Undrained Strength Parameters - Wedge	0.395	0.260
Post-Earthquake Undrained Strength Parameters - Circular	0.285	0.285
Post-Earthquake Undrained Strength Parameters - Wedge	0.389	0.250

One of the assumptions for the analysis is that the actual strength of the soil during shaking would shift from the peak undrained strength at the beginning of shaking to the post-earthquake strength at sometime during or immediately following shaking, depending on the rate of strength reduction, potential pore pressure generation and characteristics of the ground motion. Curves were generated using yield coefficients that vary between the post-earthquake to the peak undrained strengths to evaluate the range of possible deformations that could result depending on the rate of strength reduction.

Rigid-block analysis, first developed by Newmark (1965), treats a potential slope failure mass block as a rigid mass (no internal deformation) that slides in a perfectly plastic manner on an inclined plane. Thus, the mass experiences no permanent displacement until the base acceleration exceeds the critical (yield) acceleration of the block. When the base acceleration exceeds the critical acceleration, the block begins to move downslope. Displacements are estimated using a two-stage integration procedure: (1) the parts of the acceleration-time history that lie above the critical acceleration are integrated to yield a velocity-time history; and (2) the velocity-time history is then integrated to yield the cumulative displacement of the landslide block. Rigid-block analysis yields satisfactory results for relatively thin slope failures in stiff or brittle material having period ratios (T_s/T_m) less than about 0.1. The period ratio is the site period divided by the mean period of earthquake shaking as developed by Rathje et al. (2004). For thicker failure surfaces in softer materials, rigid-block analysis tends to be conservative to very conservative.

The decoupled sliding-block analysis is a modification of traditional Newmark analysis that does not require the potential failure mass to behave as a rigid block but rather models its dynamic response. The decoupled sliding-block analysis computes the dynamic response of the sliding mass without consideration of sliding and then uses the computed response in a rigid sliding-block analysis. The dynamic response of the sliding mass is computed using a one-dimensional, modal analysis in the time domain (Rathje and Bray, 1999). The sliding mass is defined by its height, shear-wave velocity, and damping ratio; the shear-wave velocity (V_s) below the sliding mass is also specified (this can be conservatively taken as rock). The modal analysis has a rigid base, but the effects of a visco-elastic base are modeled through additional damping that is assigned based on the V_s of the base and the V_s of the sliding mass (Lee, 2004). The dynamic response can be modeled as linear elastic or equivalent linear.

A coupled sliding-block analysis is an extension of the decoupled analysis. Coupled analysis models the interaction of sliding/limited shear stresses on the dynamic response of the sliding mass. Coupled analysis is considered the most rigorous and yields the most accurate estimates of displacement for deeper failures in softer material.

In our analyses the decoupled analyses generally yielded the larger deformations, followed by the coupled and then the rigid block analyses. The values for V_s for the alluvial material was estimated using an average of the shear wave velocities from the SCPT testing. The V_s values of the dam embankment and the underlying rock were estimated based on material type. The height of the failure for the analyses was taken as approximately 25 feet for the circular failure, essentially at the upstream and downstream toes of the dam and approximately 95 feet for the wedge type of failure, which is approximately the distance from the crest of the dam to the alluvium/rock interface, which is where the resulting failure surface obtained from the pseudo-static slope stability analysis is located.

Results of the Newmark analyses are presented in Table D-3.3. The maximum displacement of about 21 inches for a return period of 4,975-years is relatively small considering the percentage of overall height of the dam, approximately 2 percent. The results generally indicate that for the 4,975-year recurrence interval, the loss of freeboard is not significant and most likely would not result in a loss of containment.

Table D-3.3. Newmark Displacements

Yield Acceleration and Source	Failure Type	Estimated Mean Displacement (in.)
0.250g Crustal	Wedge	14.7
0.250g CSZ		21.0
0.395g Crustal		5.9
0.395g CSZ		6.0
0.285g Crustal	Circular	9.3
0.285g CSZ		10.7
0.370g Crustal		5.7
0.370g CSZ		5.5

3.0 Conclusions and Recommendations

Based on the static and pseudo-static slope stability analysis and Newmark deformation analysis, it appears that the proposed embankment dam alternative A-3 meets the general criteria from the USACE and USBR for these types of analyses.

4.0 References

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Application of likelihood ratio and logistic regression models to landslide susceptibility mapping using GIS.

Rathje, E.M., Faraj, Fadi, Russell, Stephanie, and Bray, J.D. 2004.

Empirical relationships for frequency content parameters of earthquake ground motions: Earthquake Spectra, v. 20, p. 119-144.

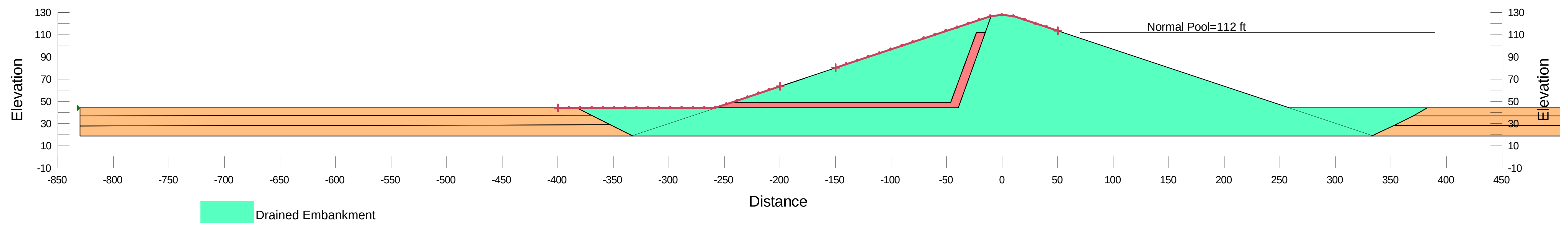
Rathje and Bray. 1999.

An examination of simplified earthquake-induced displacement procedures for earth structures: Canadian Geotechnical Journal, v. 36, p. 72-87.

U.S. Army Corps of Engineers. 2003.

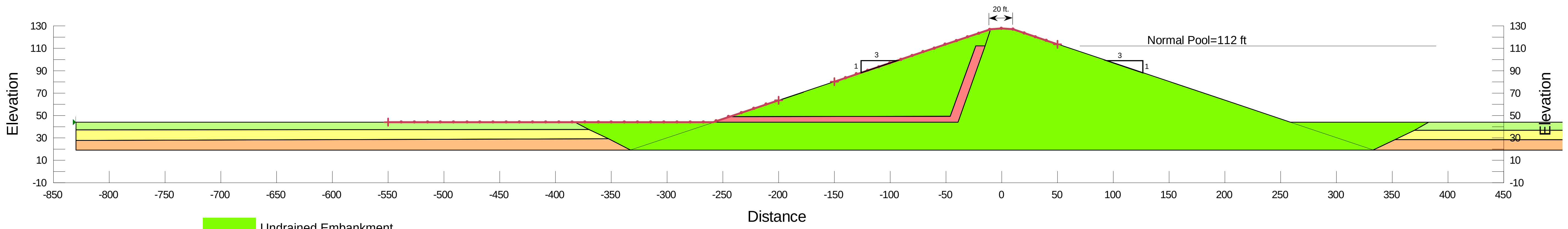
EM 1110-2-1902, Slope Stability.

Figures



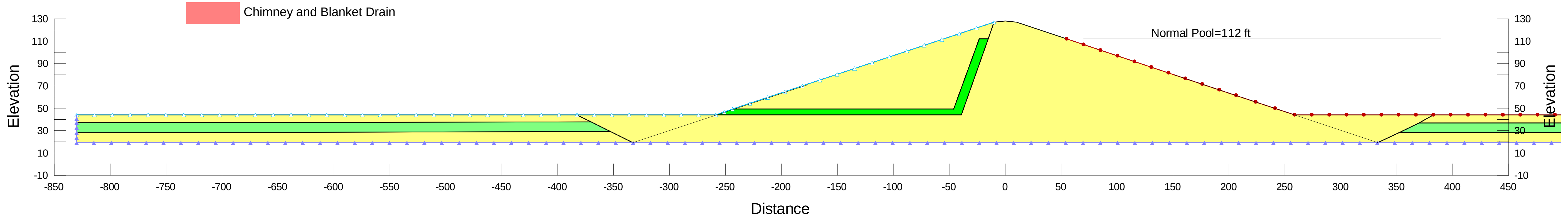
🟩 Drained Embankment
🟧 Drained Alluvium
🟥 Chimney and Blanket Drain

Drained Strength Stability Analysis



🟩 Undrained Embankment
🟧 Undrained Alluvium OCR = 2.0
🟨 Undrained Alluvium OCR = 3.0
🟩 Undrained Alluvium OCR = 4.0
🟥 Chimney and Blanket Drain

Undrained Strength Stability Analysis



🟨 MH Soils (Seepage)
🟩 SM/Drainage System Soils (Seepage)

Embankment Seepage Analysis

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New Embankment Dam Schematic

BIG CREEK DAMS 1 AND 2
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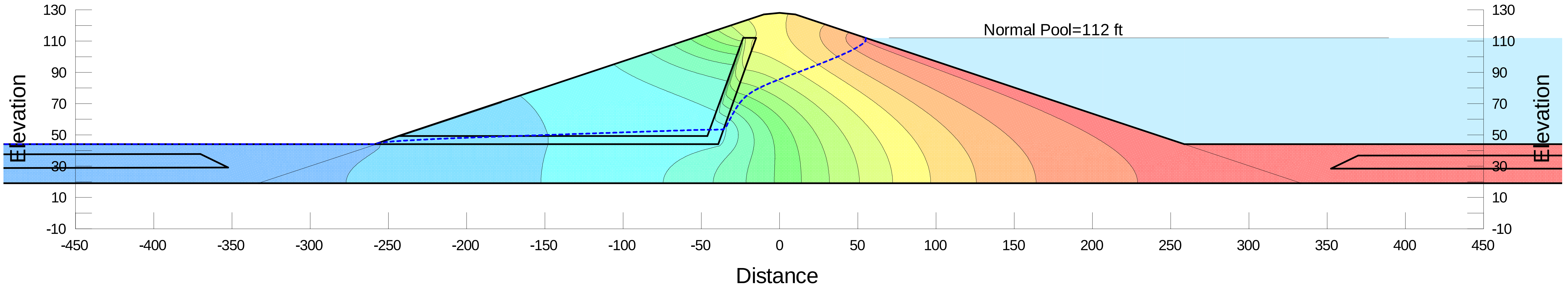
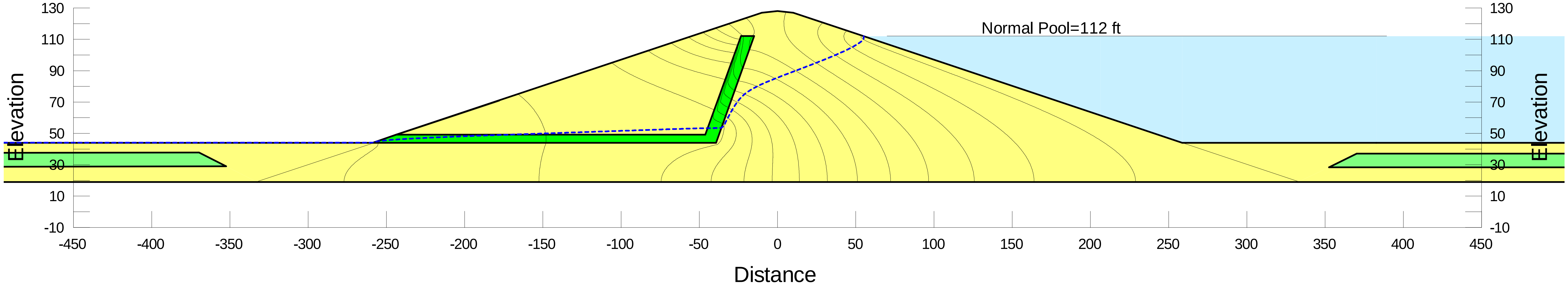
FIGURE
D-3.1





Head Contours

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Lower Bound Seepage Analyses

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

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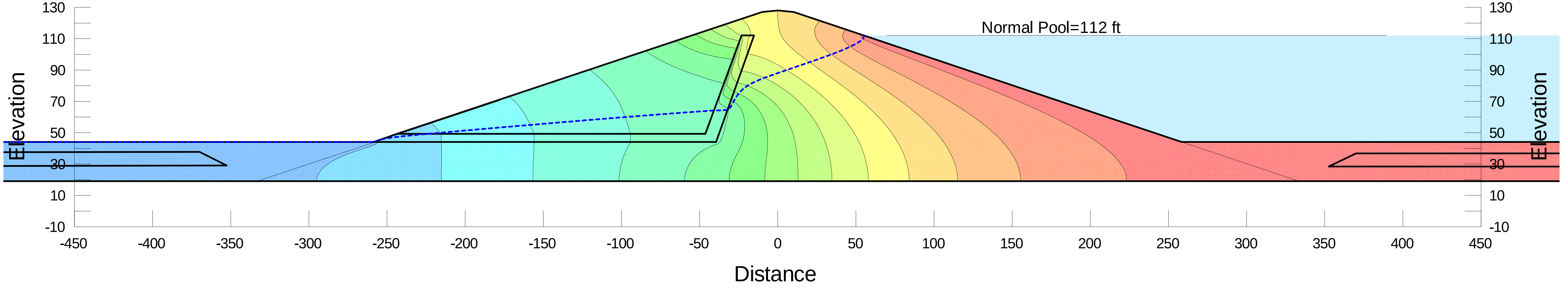
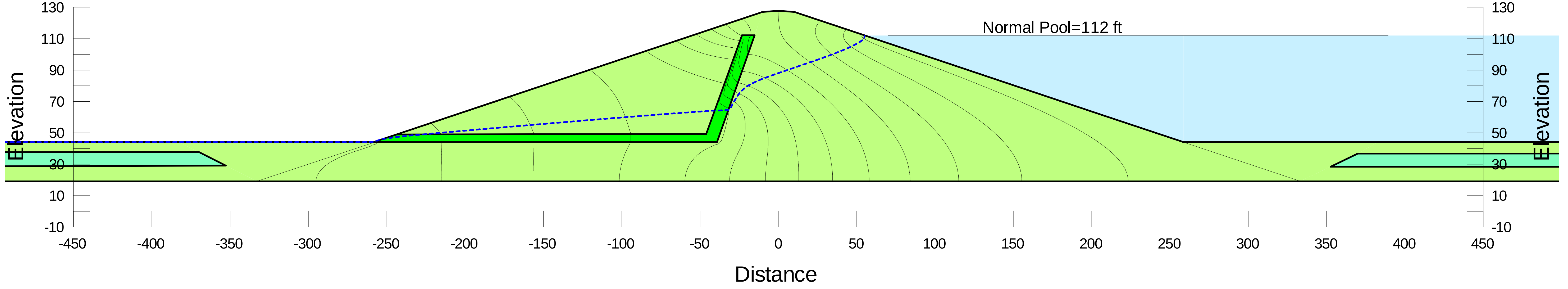
FIGURE
D-3.2





Head Contours

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PROJECT NO. 221223
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Upper Bound Seepage Analysis

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

ORIGINATOR: STA
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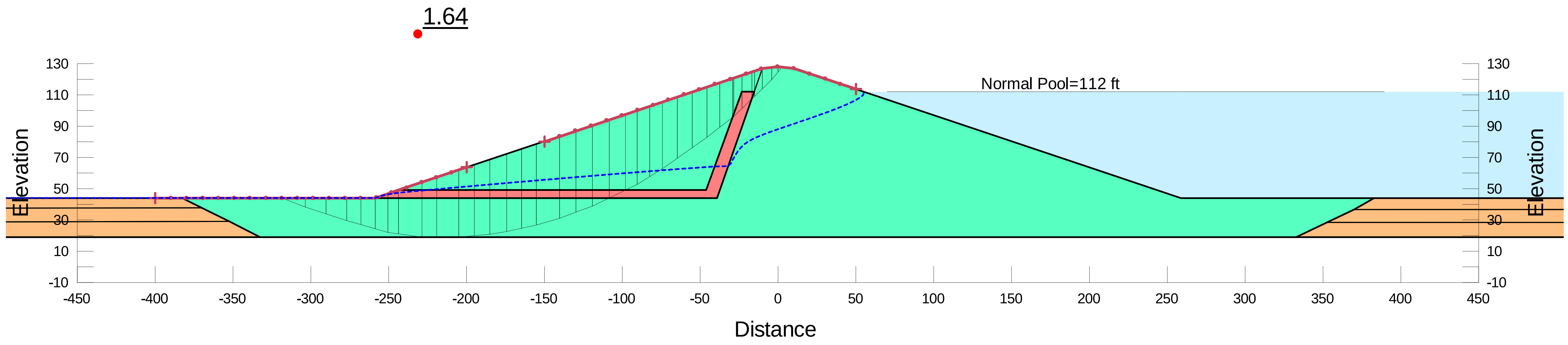
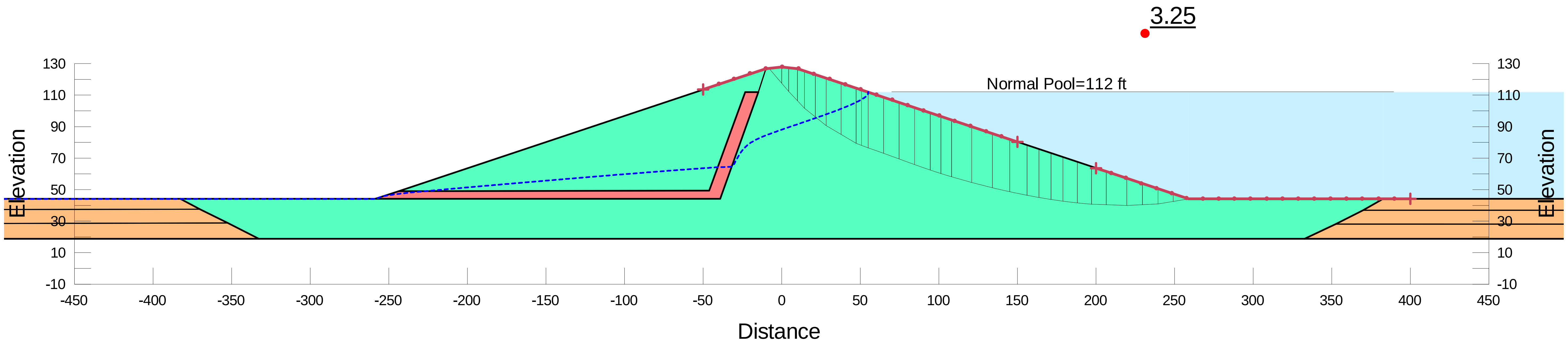
FIGURE
D-3.3





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Drained Slope Stability

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

ORIGINATOR: STA

APPROVED BY: <approved by>

DRAWING CATEGORY:
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FIGURE
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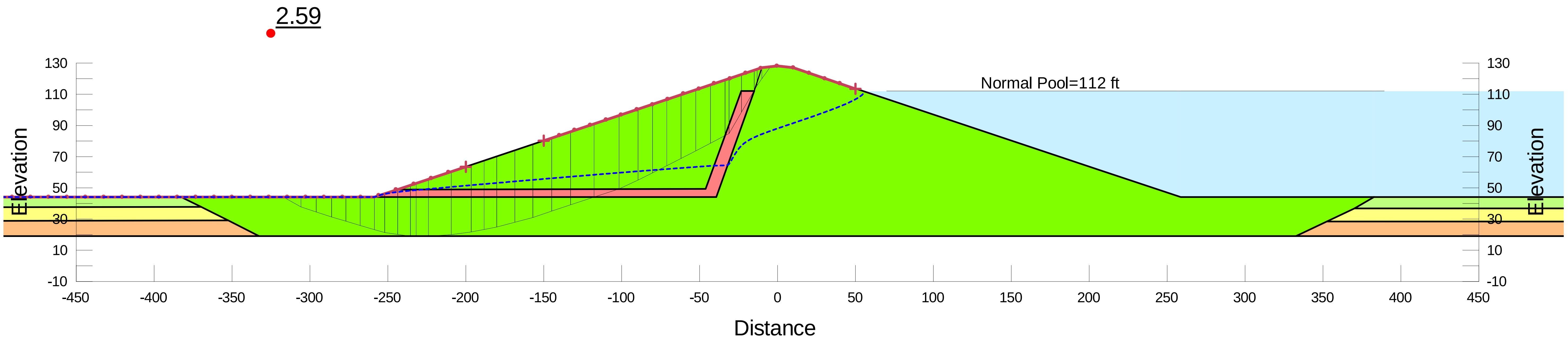

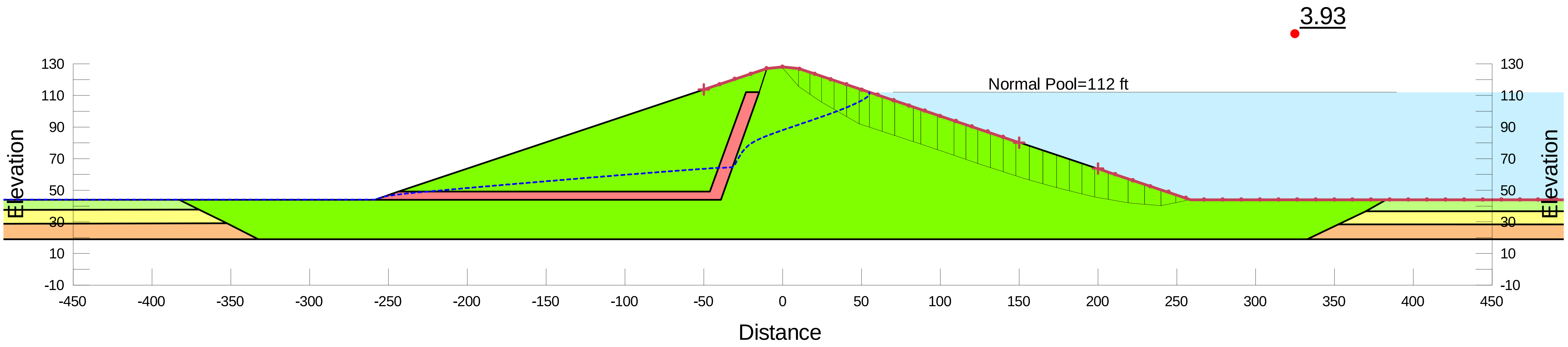

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Undrained Slope Stability

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

ORIGINATOR: STA
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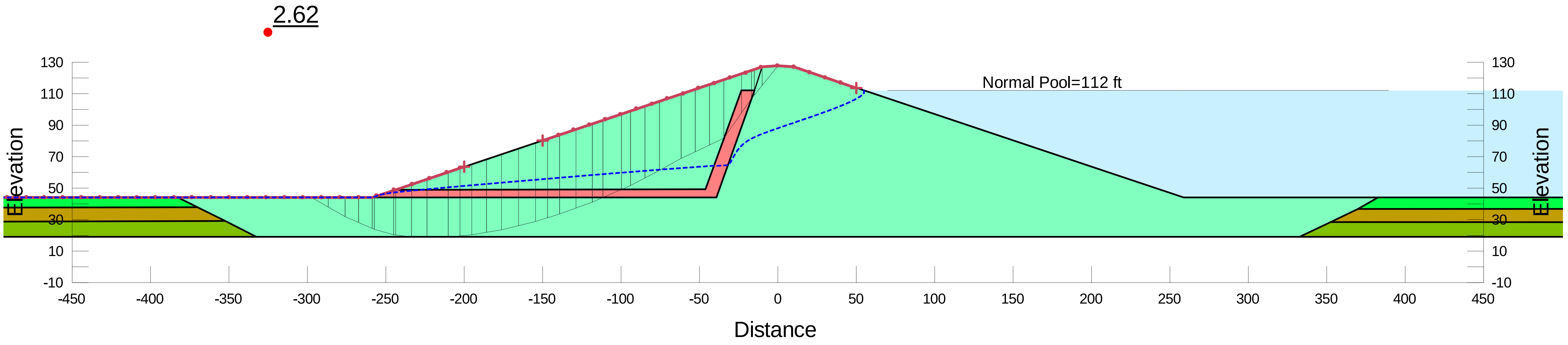
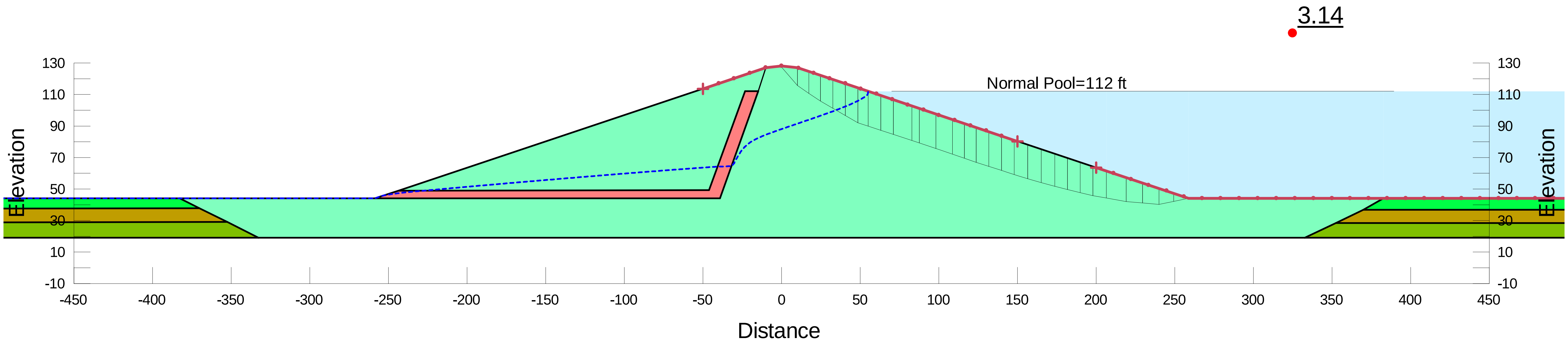
FIGURE
D-3.5





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Post-Earthquake Slope Stability

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

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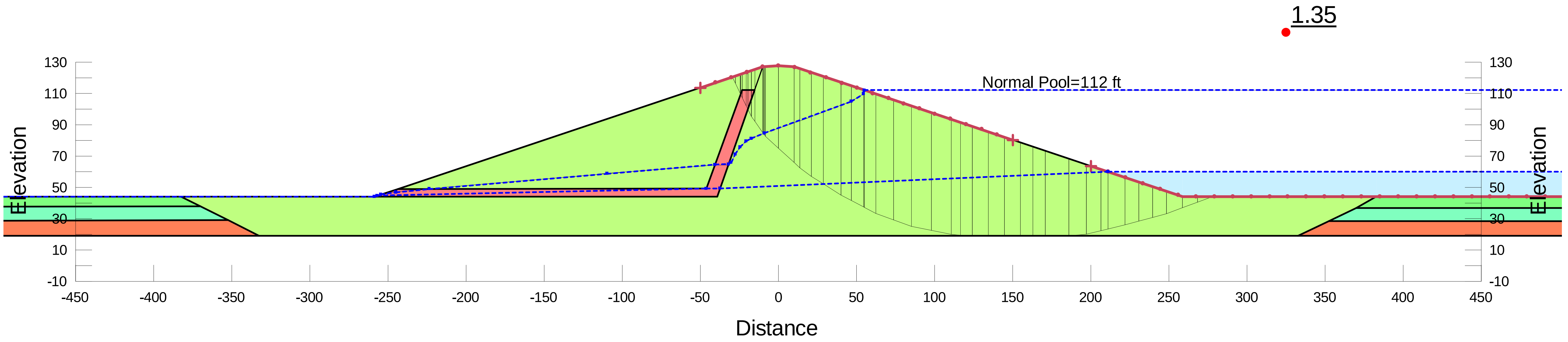
FIGURE
D-3.6



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Rapid Drawdown Slope Stability

BIG CREEK DAMS 1 AND 2
NEWPORT, OREGON

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APPROVED BY: <approved by>
DRAWING CATEGORY: <drawing category>

FIGURE
D-3.7