SHANNON & WILSON, INC.

GEOTECHNICAL REPORT PIONEER CROSSING, LEHI AND I-15 AMERICAN FORK INTERCHANGE, SEGMENT 1

Final Submittal
August 27, 2008

Prepared by:
Shannon & Wilson, Inc.,
Geotechnical Consultant for Kiewit/Clyde Joint Venture

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23-1-01178-010

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GEOTECHNICAL REPORT
PIONEER CROSSING, LEHI AND I-15 AMERICAN FORK INTERCHANGE
SEGMENT 1

1.0 INTRODUCTION

This report summarizes Shannon & Wilson's geotechnical explorations, testing, subsurface characterization, analyses, conclusions, and recommendations for the Pioneer Crossing, Lehi and I-15 American Fork Interchange. Segment 1 extends from Redwood Road to Station (Sta.) 725+00 as shown in the Vicinity Map, Figure 1, and the Exploration Plan, Figure 2. Shannon & Wilson was responsible for providing geologic interpretation and geotechnical recommendations for design of this segment of the project. Tasks included reviewing existing geotechnical information; performing geotechnical explorations, laboratory testing, and analyses; and providing design recommendations.

We prepared Geotechnical Design Memoranda for each design element. This report summarizes our geotechnical recommendations for the project and incorporates the Geotechnical Design Memoranda completed throughout the course of the project. Table 1 summarizes the memoranda and associated design elements. The final memorandum for each structure, feature, or design element is incorporated into Appendices A through F of this report.

2.0 PROJECT DESCRIPTION

The Pioneer Crossing, Lehi and I-15 American Fork Interchange project (project) consists of constructing an east to west urban arterial through Lehi connecting Redwood Road to I-15. The proposed arterial will parallel State Route 73 at approximately 900 South and will include new bridge structures to extend the roadway over the Union Pacific Railroad tracks near Mill Pond and over the Jordan River in Saratoga Springs. The project includes a new interchange at Main Street in American Fork. The geotechnical recommendations summarized in this report address the Jordan River Bridge foundations and fill embankments from Redwood Road at about Sta. 600+00 to Sta. 725+00 in Segment 1, as shown in the Vicinity Map, Figure 1, and the Site and Exploration Plan, Figure 2. Geotechnical studies and recommendations for project elements in Segment 2 (east of Sta. 725+00) were done by others.

The main design elements associated with constructing the roadway include:

- A new bridge crossing the Jordon River
- Bridge abutments
- Mechanically stabilized earth walls (Sta. 630+50 to 640+50)

- Post and panel walls (Sta. 650+50 to 660+00)
- Unreinforced soil slopes

The proposed roadway will be about 100 feet wide, comprising two lanes in each direction, a median and sidewalks. The new fill heights typically vary from about 2 to 18 feet depending on the elevation of the existing ground surface, with the majority of the fill embankments ranging between 4 to 8 feet. The higher embankments will form approaches to the east and west abutments of the Jordan River Bridge at 18 and 12 feet, respectively. Finished embankment slopes will typically be 3 horizontal to 1 vertical. In some locations, the embankment will be retained to reduce impacts to wetlands or reduce right-of-way encroachment. The retaining walls will vary in height up to about 10 feet to accommodate variation in the natural ground surface and proposed fill height. Details for each of these project elements are provided in the project plans. Major project element locations are presented in Figure 2.

3.0 SUBSURFACE EXPLORATIONS

Subsurface explorations were performed for baseline studies and provided in the Request for Proposal (RFP) (Utah Department of Transportation [UDOT], 2008). Terracon Consultants, Inc. (Terracon, 2008) performed nine borings and one cone penetration test (CPT) between Redwood Road and Sta. 725+00. Terracon's report is presented in Appendix H of this report. Shannon & Wilson performed 4 additional borings, 12 CPTs, and 17 test pits during the design phase. Subsurface explorations were performed in general accordance with the applicable ASTM International (ASTM) standards. The boring locations were surveyed following completion of exploratory work. Geotechnical Design Memorandum GD-5, Appendix E, presents the subsurface exploration logs and details regarding the subsurface methods, exploration dates, and sampling methods.

4.0 GEOTECHNICAL LABORATORY TESTING

Geotechnical laboratory tests were performed on selected samples retrieved from borings performed by Shannon & Wilson to classify the soil and evaluate index and engineering properties of the materials. Laboratory tests included visual classification, moisture content, Atterberg limits, grain size distribution, one-dimensional consolidation, and percent passing the number 200 sieve. Shannon & Wilson subcontracted with Terracon to perform laboratory tests on samples from selected borings that were drilled during the RFP. These tests included Atterberg Limits and grain size analyses. Laboratory tests were performed in general accordance with applicable ASTM standards. Laboratory test results are presented in Geotechnical Design Memoranda GD-5, Appendix E.

5.0 GEOLOGY AND SUBSURFACE CONDITIONS

The general geology of the project alignment is described in the Geotechnical Baseline Report prepared by Terracon, which is part of the RFP (UDOT, 2008) and included in Appendix H of this report. Subsurface conditions for each design element are described the applicable sections of Geotechnical Design Memorandum GD-5, Appendix E.

6.0 SEISMIC DESIGN CONSIDERATIONS

Terracon (2008), Appendix H, provided a preliminary description of the general seismicity of the project area and seismic hazards. Project ground motions and seismic design criteria are presented in Geotechnical Design Memorandum GD-3, Appendix C. The results of our liquefaction analyses and ground improvement recommendations are presented in Geotechnical Design Memorandum GD-4, Appendix D.

7.0 GEOTECHNICAL ENGINEERING ANALYSES AND RECOMMENDATIONS

Our geotechnical engineering analyses and recommendations for each project structure, feature, or design element are described in the individual Geotechnical Design Memoranda included in Appendices A and F. Our analyses were performed in general accordance with the UDOT Geotechnical Manual of Instruction (UDOT, 2006). Geotechnical analyses and design methodologies are summarized in Geotechnical Design Memorandum GD-2, Appendix B. Generally, each design memorandum provides a brief description of the structure or feature as it related to the geotechnical aspects we were addressing, followed by a discussion of subsurface conditions. Typically, we then described the assumptions made, the analyses completed, our results, and our recommendations. Finally, a discussion of construction considerations were generally included, as related to the geotechnical recommendations being made. Where appropriate, subsurface exploration and laboratory testing data were attached to the individual design memorandum.

We prepared Geotechnical Instrumentation Plan GIP-1 (Appendix G) to address settlement and slope monitoring. Project elements included in the plan include the Jordan River Bridge approach embankments and MSE wall R-575A.

8.0 LIMITATIONS

The limitations of this report and all design memoranda incorporated herein are presented in Geotechnical Design Memorandum GD-1, Appendix A.

9.0 REFERENCES

ASTM International (ASTM), 2007, Annual book of standards, construction, volume 04.08, soil and rock: West Conshohocken, Pa., ASTM.

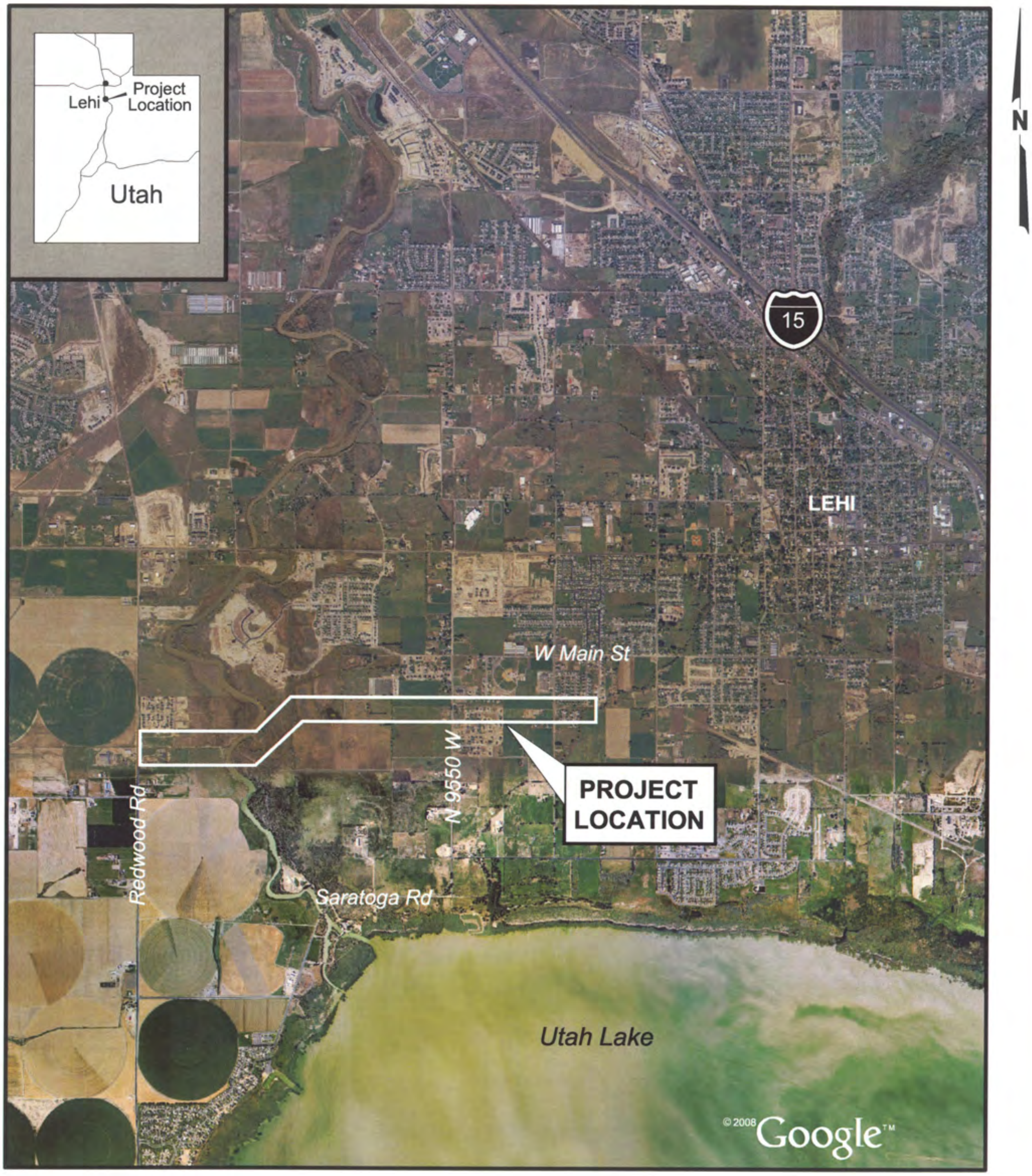
Terracon Consultants, Inc. (Terracon), 2008, Geotechnical baseline report East-west connector American Fork, Lehi, Saratoga Springs, Utah: UDOT project no. SR-399(38) Terracon project no. 61085026, report prepared by Terracon Consultants, Inc., Draper, Utah, for HDR, Inc., Salt Lake City, Utah, June 12.

Utah Department of Transportation (UDOT), 2006, Geotechnical manual of instruction: Salt Lake City, Utah, Utah Department of Transportation, 75 p., September.

Utah Department of Transportation (UDOT), 2008, Request for proposals: Pioneer Crossing, Lehi and I-15 American Fork Interchange, Utah, Utah Department of Transportation, June 20.

TABLE 1
SUMMARY OF GEOTECHNICAL DESIGN MEMORANDA AND LETTERS

Number	Title	100 Percent Submittal Date	Comment
GD-1	Limitations	December 10, 2008	
GD-2	Geotechnical Analyses and Design Methodologies	March 2, 2009	Criteria, assumptions, and design methodologies used in the geotechnical analyses including settlement, and stability for embankments and retaining walls, lateral earth pressures for embedded structures, axial and lateral bridge foundation capacity, and liquefaction analyses
GD-3	Ground Motions for Seismic Design	December 29, 2008	Ground motions (response spectra) for seismic design of the Jordan River Bridge and its approaches
GD-4	Liquefaction Analyses And Ground Improvement Recommendations	March 12, 2009	Evaluation of liquefaction potential and associated effects at the Jordan River Bridge and its approaches, mechanically stabilized earth (MSE) wall, and the post and panel wall
GD-5	Pioneer Crossing Approach Fill, Embankment Stability, and Jordan River Bridge	February 4, 2009	Geotechnical recommendations and considerations for design and construction of approach fills, embankment stability, and walls for Pioneer Crossing and foundations of the Jordan River Bridge
GD-5 Addendum 1	Pioneer Crossing Approach Fill, Embankment Stability, and Jordan River Bridge	April 7, 2009	Revised recommendations for nominal pile resistance, Wave Equation Analysis Pile analyses, and clarifies roadway and bridge approach settlement criteria
GIP-1	Segment 1 Geotechnical Instrumentation Plan	March 6, 2009	Plan for monitoring of the proposed MSE wall R-575A, and embankment settlement and stability during construction and surcharging



0 4000 8000

Approximate Scale in Feet

NOTE

Map adapted from aerial imagery provided by Google Earth Pro, Image © 2008 DigitalGlobe, and Image State of Utah reproduced by permission granted by Google Earth ™ Mapping Service.

Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

VICINITY MAP

August 2009 23-1-01178-010

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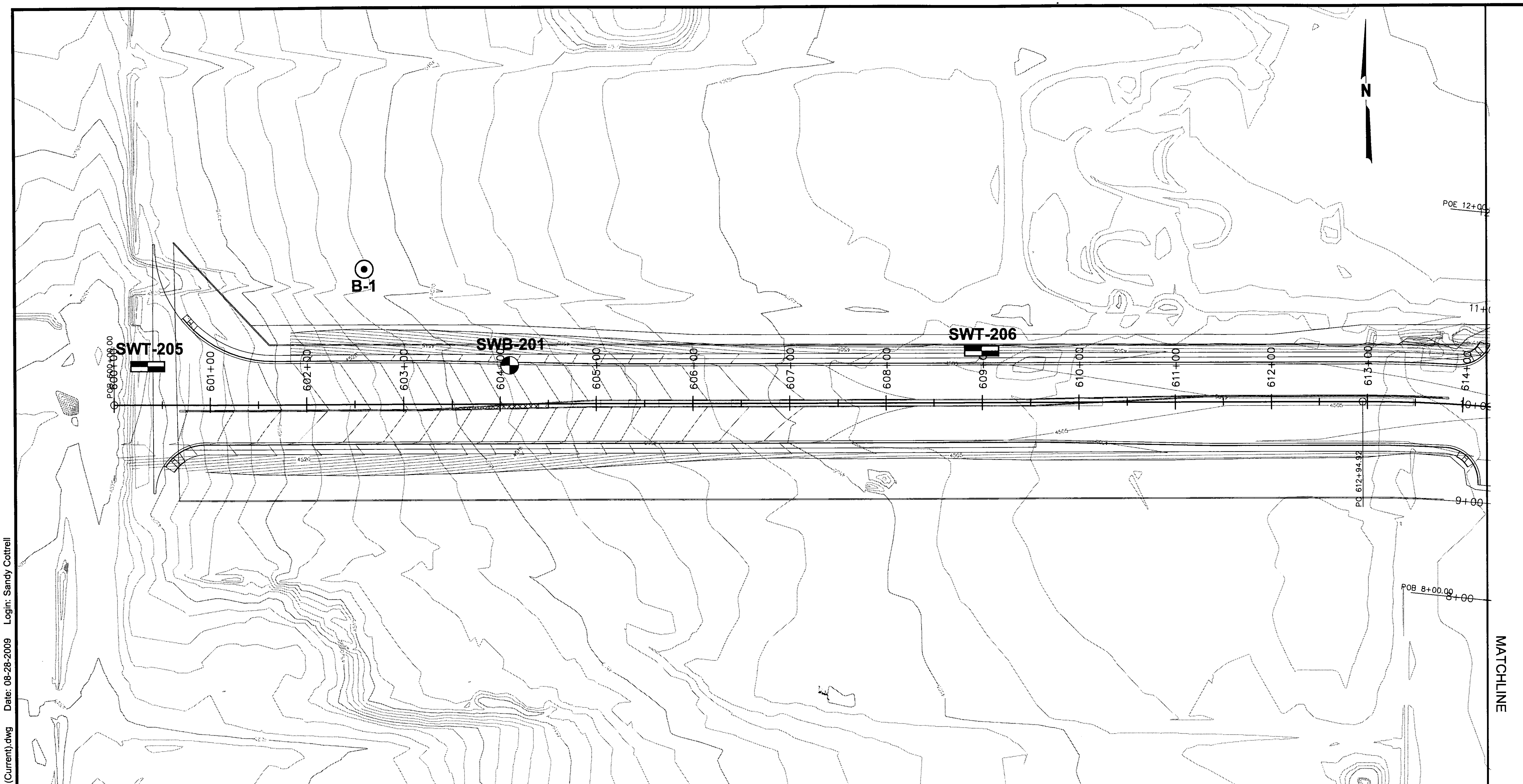
FIG. 1

Filename: J:\231\01178-010\23-1-01178-010 Fig 1.dwg Date: 08-27-2009 Login: Sandy Cottrell



LEGEND

B-1	Baseline Boring Designation and Approximate Location by Terracon
CPT-1	Baseline CPT Designation and Approximate Location by Terracon
SWB-1	Boring Designation and Approximate Location
SWC-1	CPT Designation and Approximate Location
SWT-1	Test Pit Designation and Approximate Location
A	Generalized Subsurface Profile Designation and Approximate Location

0 100 200

Scale in Feet

NOTE

Figure adapted from file *6697_S1_Design* provided by Utah Department of Transportation, received 12-18-2008.

Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

SITE AND EXPLORATION PLAN

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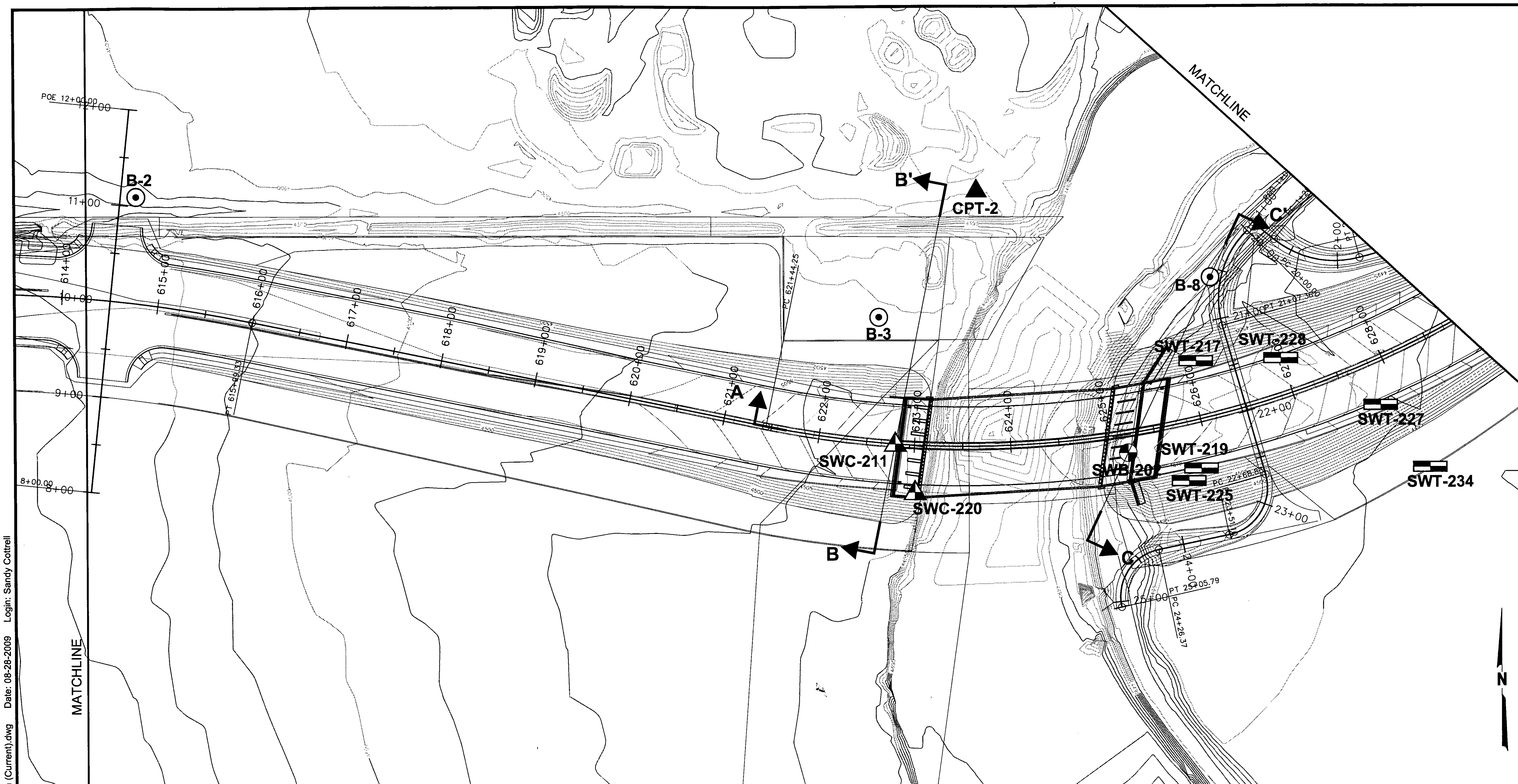
FIG. 2
Sheet 1 of 9

Filename: J:\231\01178-010\23-1-01178-010 Plan (Current).dwg Date: 08-28-2009 Login: Sandy Cottrell



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Scale in Feet

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Pioneer Crossing, Lehi & I-15
American Fork Interchange
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FIG. 2
Sheet 2 of 9

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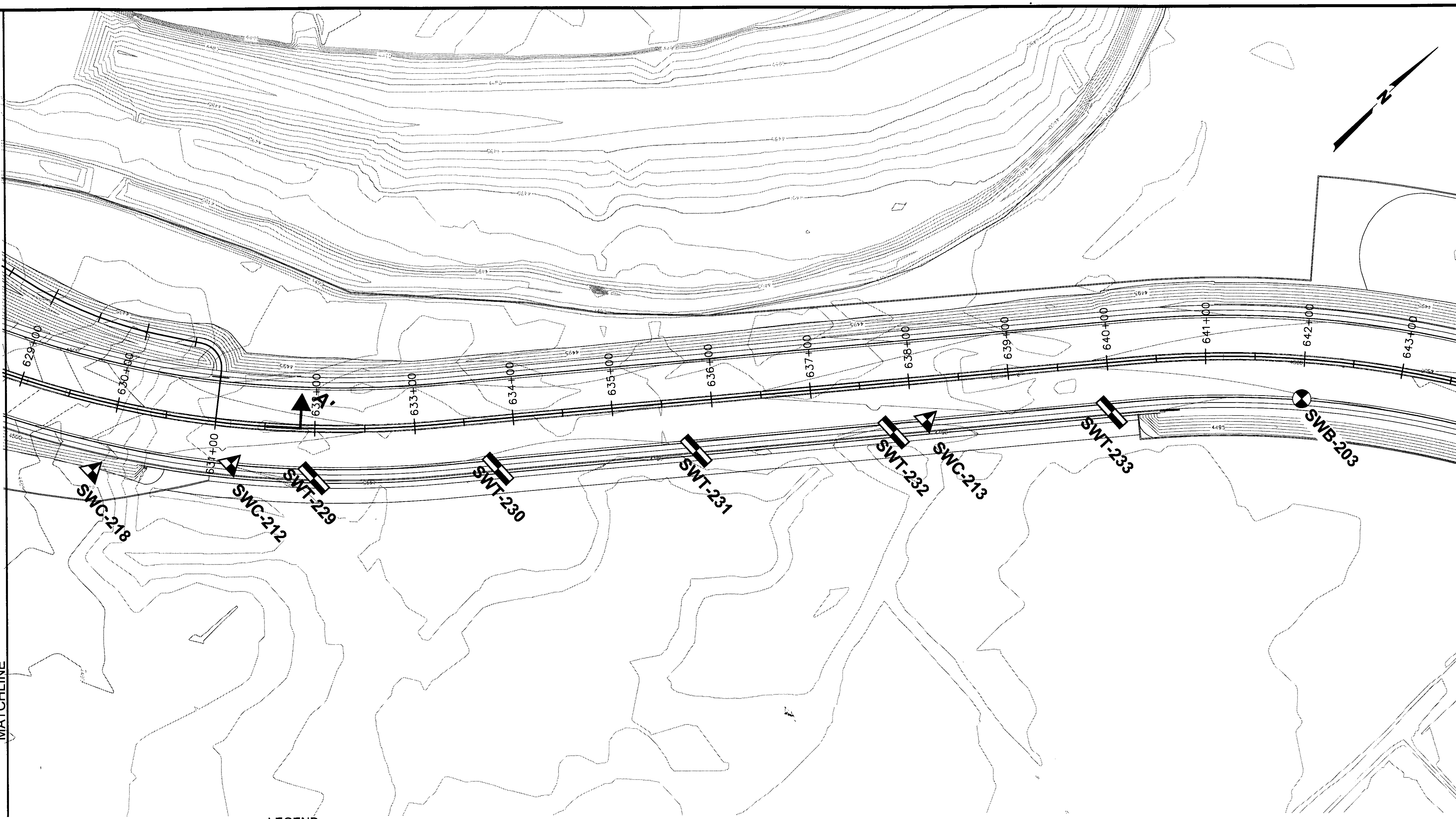

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American Fork Interchange
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SITE AND EXPLORATION PLAN

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FIG. 2
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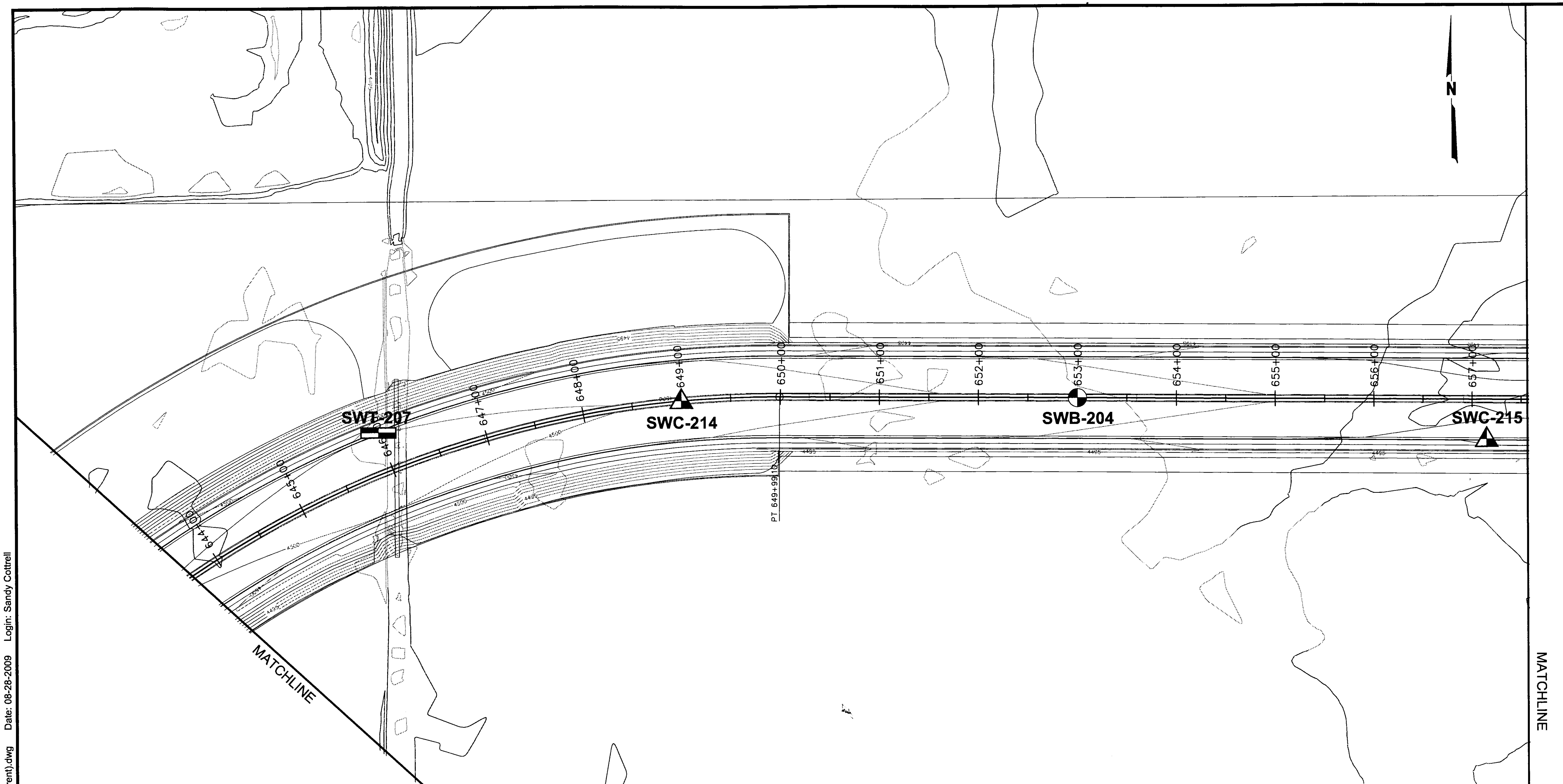

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FIG. 2
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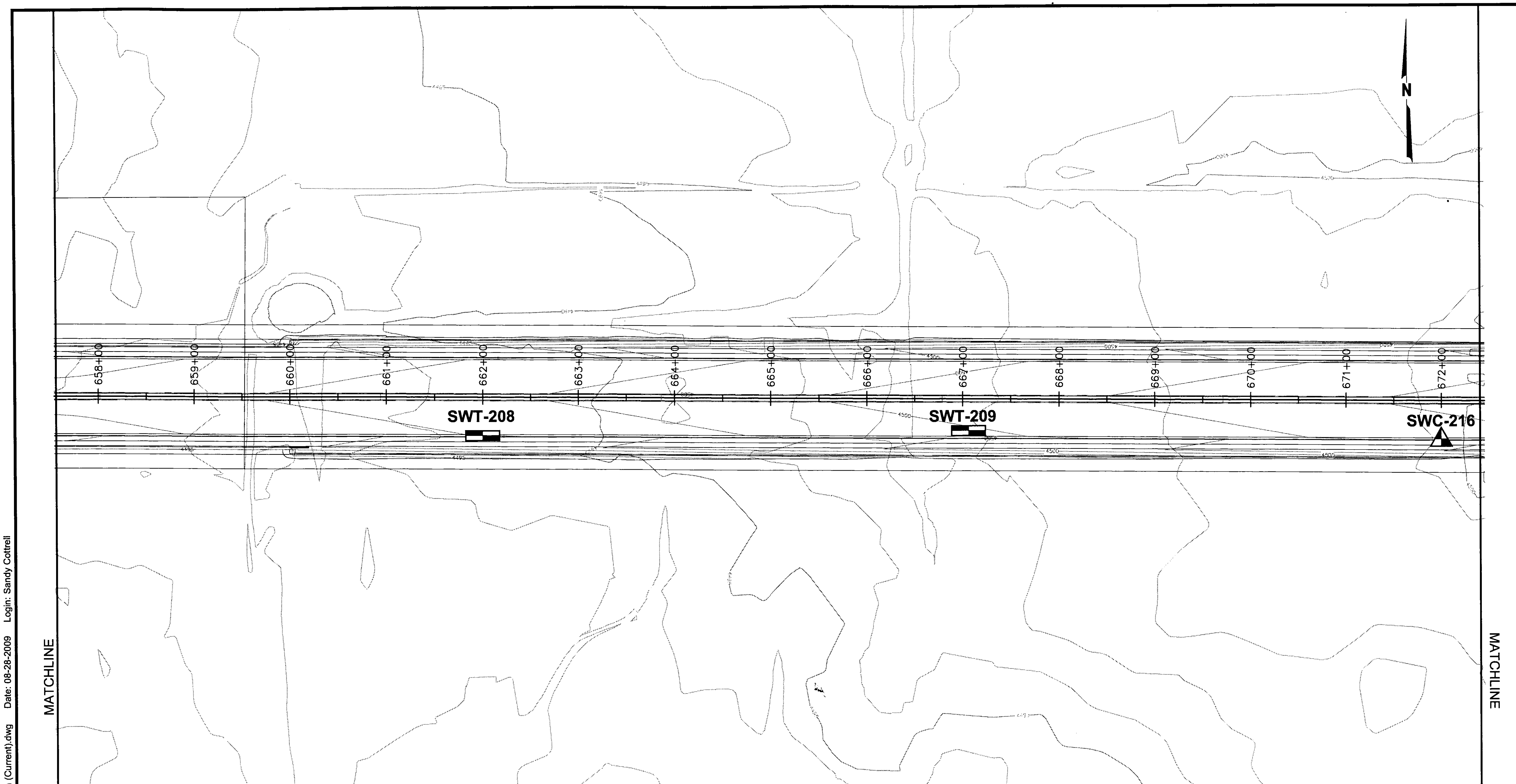

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FIG. 2
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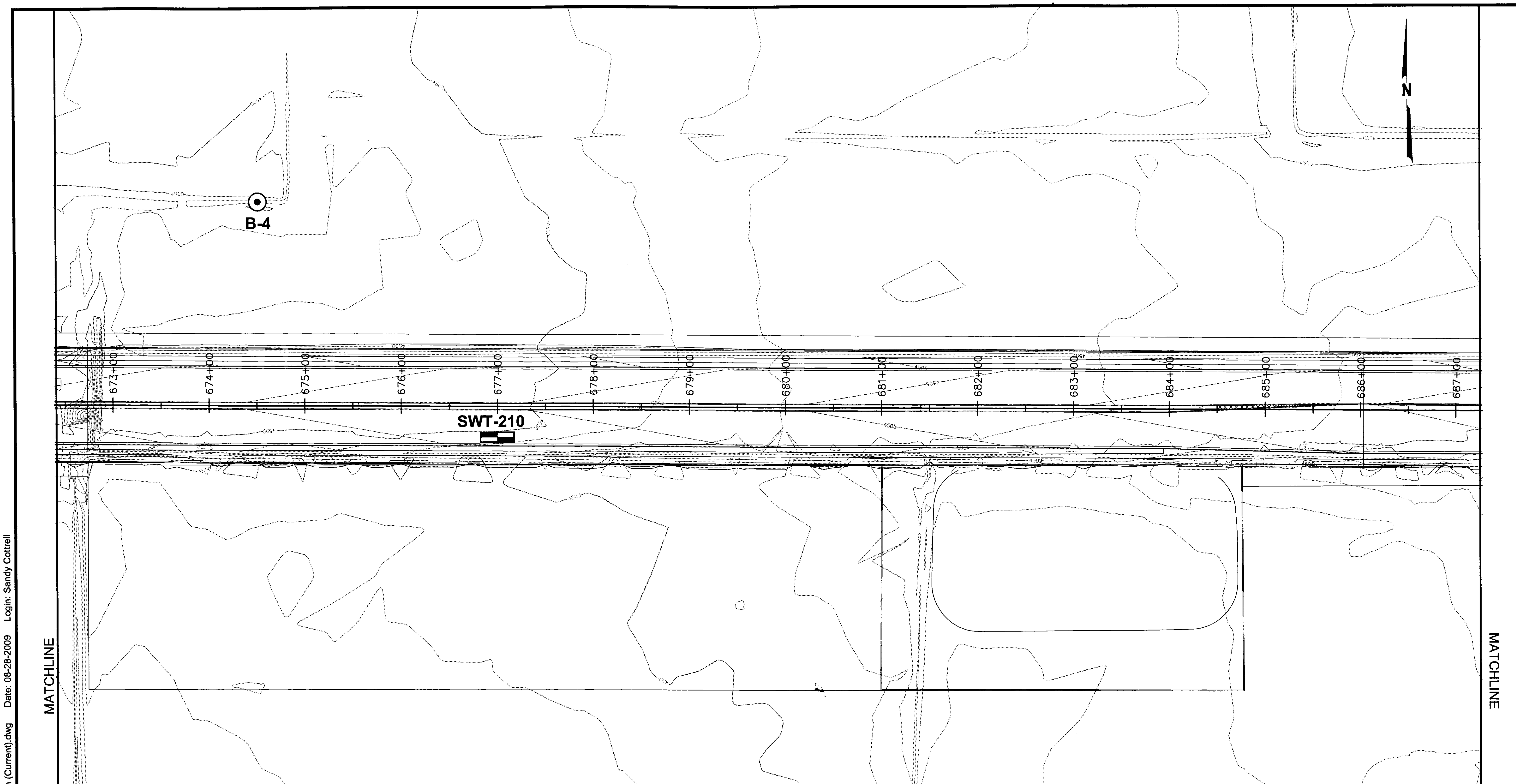

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FIG. 2
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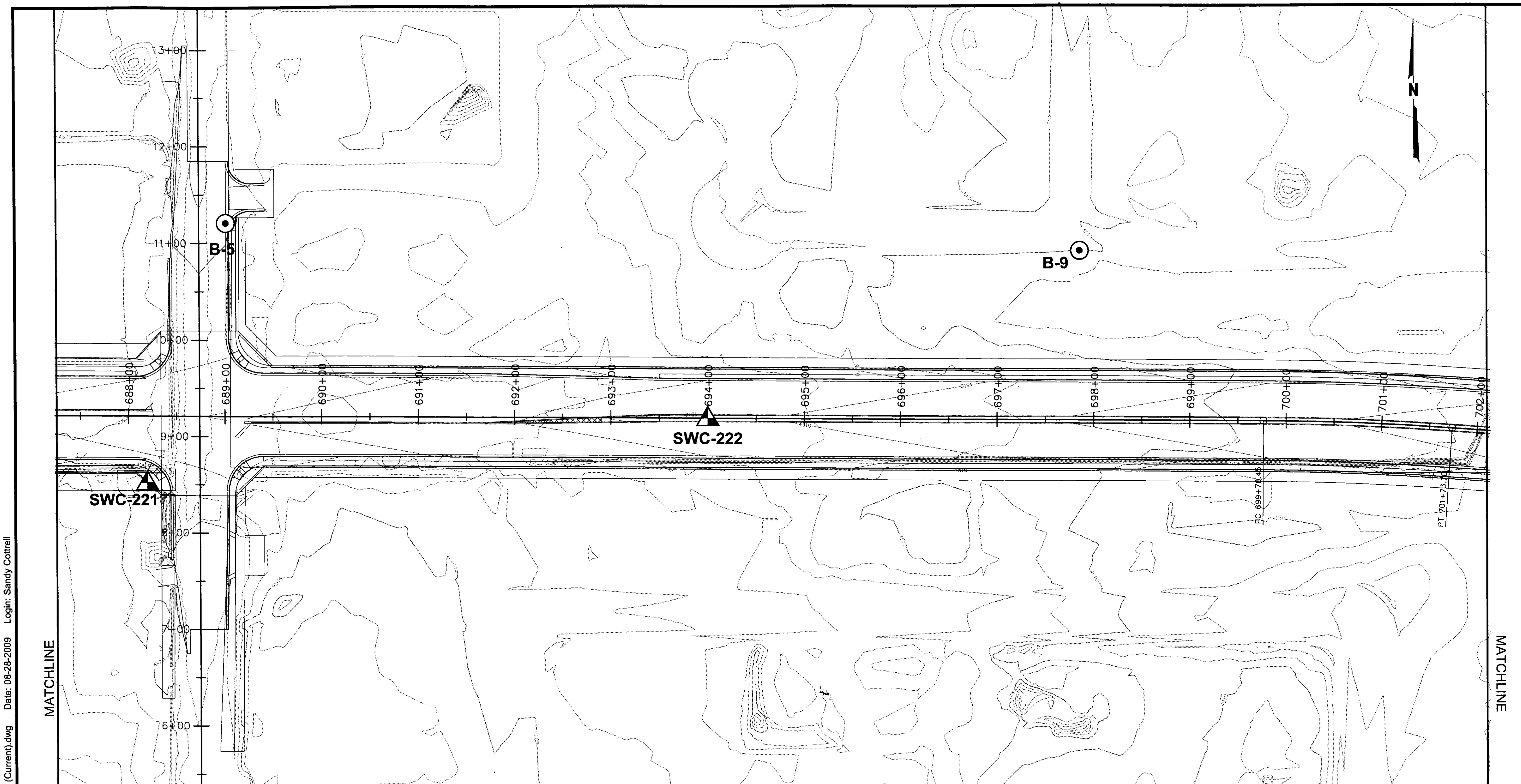

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FIG. 2
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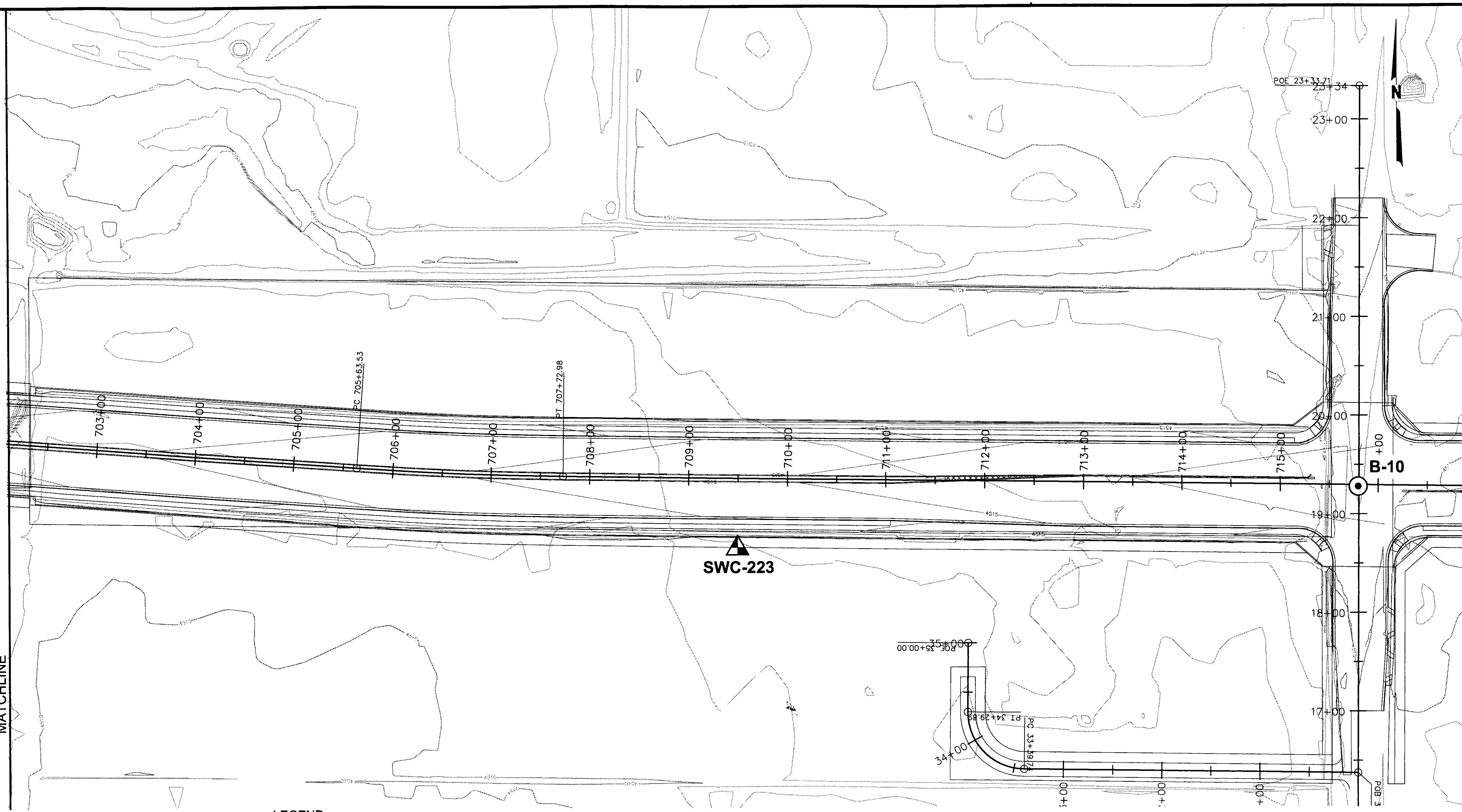

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FIG. 2
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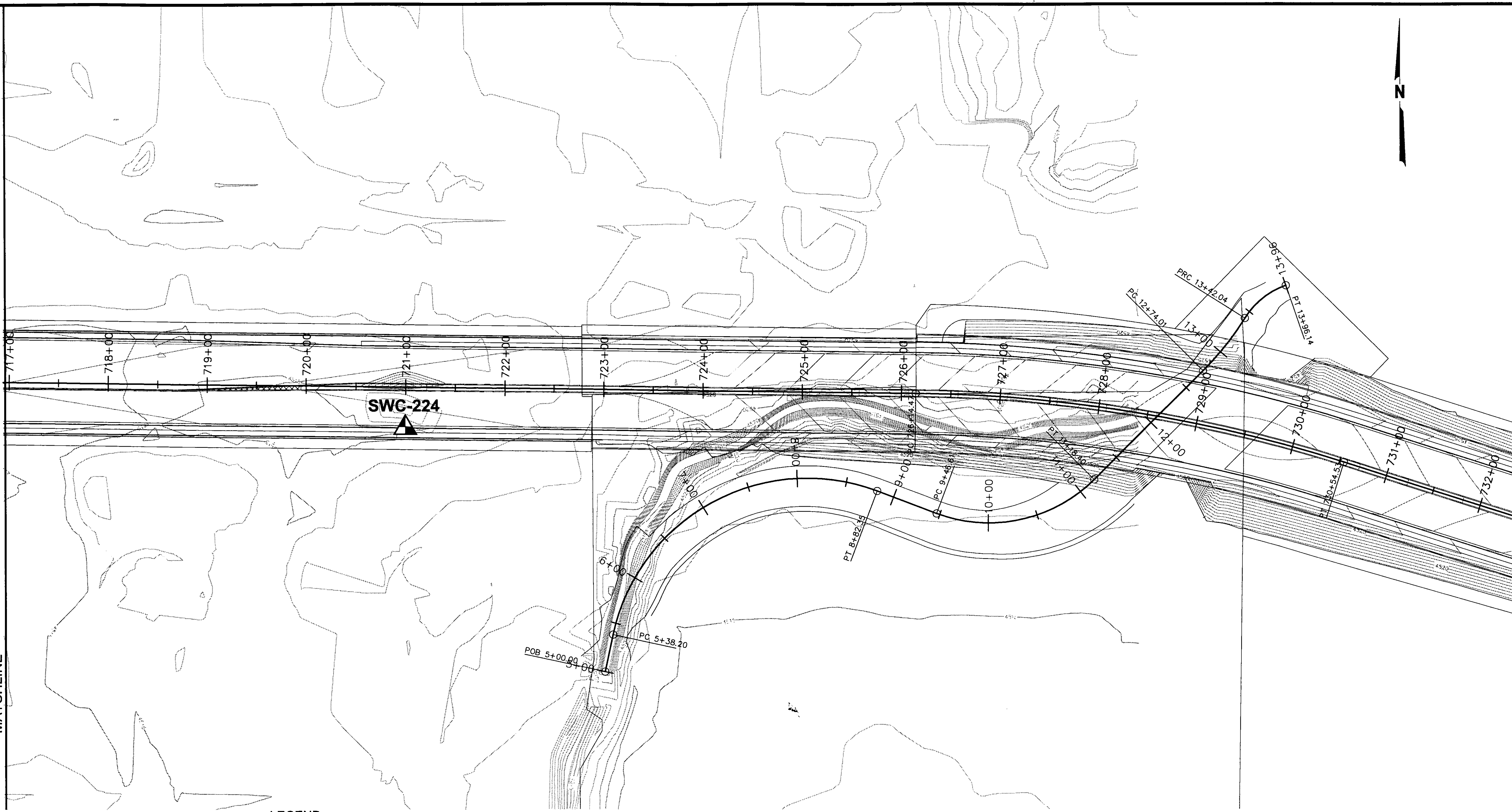

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FIG. 2
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APPENDIX A

GEOTECHNICAL DESIGN MEMORANDUM GD-1

GEOTECHNICAL DESIGN MEMORANDUM GD-1
PIONEER CROSSING, LEHI AND I-15 AMERICAN FORK INTERCHANGE

LIMITATIONS

December 10, 2008

Prepared by:
Shannon & Wilson, Inc.,
Geotechnical Consultant for Kiewit/Clyde Joint Venture

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Vice President

WJP:CAR/car

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ATTACHMENT

GEOTECHNICAL DESIGN MEMORANDUM GD-1
PIONEER CROSSING, LEHI AND I-15 AMERICAN FORM INTERCHANGE

1.0 DESIGN MEMORANDUM SCOPE

This geotechnical design memorandum presents general limitations that apply to the use of geotechnical memoranda, reports, designs, conclusions, and recommendations prepared by Shannon & Wilson, Inc. for Parsons Transportation Group Inc. and Kiewit/Clyde Joint Venture for the Pioneer Crossing, Lehi and I-15 American Fork Interchange (Project). A separate geotechnical design memorandum will be issued to address the site-specific details of our analyses and recommendations for each of the Project elements that require geotechnical engineering. Final versions of this and other geotechnical design memoranda issued for the Project will be compiled into the Pioneer Crossing, Lehi and I-15 American Fork Interchange Project Geotechnical Report (Report) upon completion of the geotechnical engineering design phase of the Project. Each geotechnical design memorandum that will be prepared will reference this memorandum (GD-1).

2.0 LIMITATIONS

Within the limitations of the scope, schedule, and budget, the analyses, conclusions, and recommendations presented in the geotechnical design memoranda and Report were prepared in accordance with generally accepted professional geotechnical engineering principles and practices at the time each geotechnical design memorandum and Report were produced. We make no other warranty, either express or implied.

The analyses, conclusions, and recommendations are based on our understanding of the Project, as described in the geotechnical design memoranda and Report and site conditions as they existed at the time our explorations were performed. For the purpose of presenting design recommendations, we assumed that the subsurface conditions in the Project area are not significantly different from those disclosed by the explorations, whether those explorations were performed by Shannon & Wilson, Inc. for this Project or by others for this or other projects on or near the site.

If, during construction, subsurface conditions different from those encountered in the explorations are observed or appear to be present, we should be advised at once so that we can review these conditions and reconsider our recommendations where necessary. If conditions have changed due to construction operations at or adjacent to the site, we recommend that the geotechnical design memoranda and Report be reviewed to determine the applicability of the conclusions and recommendations considering the changed conditions.

This Report was prepared for the exclusive use of Parsons Transportation Group Inc. and Kiewit/Clyde Joint Venture, other members of the Project design and construction team, and for the Utah State Department of Transportation. It should not be made available for use by others or for purposes other than those described in the individual geotechnical design memorandum and Report. Further, it only should be relied on for information based on factual data, such as those interpreted from the exploration logs and discussion of subsurface conditions included in the Report. Unanticipated soil conditions are commonly encountered and cannot be fully determined by merely taking soil samples from explorations. Such unexpected conditions frequently require that additional expenditures be made to attain properly constructed projects.

Certain construction activities may require the expertise of specialty contractors. The appropriate geotechnical design memoranda and Report should be provided to such contractors so that they can understand the engineering consequences associated with their particular construction means and methods. These activities may include, but are not limited to, ground modification and improvement, drilled shafts, driven piles, dewatering, tunneling, pipeline lining, mechanically stabilized earth retaining walls, and shoring.

The scope of our services did not include assessment or evaluation regarding the presence or absence of hazardous or toxic materials in the subsurface environment. If such contamination exists, it would not be possible to identify it within the geotechnical program completed for the Project.

Shannon & Wilson, Inc. has prepared the attachment, "Important Information About Your Geotechnical Report," to assist the design and construction team and others who might read the geotechnical design memoranda and Report in understanding the use and limitations of our work.

Date: December 10, 2008
To: Mr. Del Walker
Parsons

IMPORTANT INFORMATION ABOUT YOUR GEOTECHNICAL/ENVIRONMENTAL REPORT

CONSULTING SERVICES ARE PERFORMED FOR SPECIFIC PURPOSES AND FOR SPECIFIC CLIENTS.

Consultants prepare reports to meet the specific needs of specific individuals. A report prepared for a civil engineer may not be adequate for a construction contractor or even another civil engineer. Unless indicated otherwise, your consultant prepared your report expressly for you and expressly for the purposes you indicated. No one other than you should apply this report for its intended purpose without first conferring with the consultant. No party should apply this report for any purpose other than that originally contemplated without first conferring with the consultant.

THE CONSULTANT'S REPORT IS BASED ON PROJECT-SPECIFIC FACTORS.

A geotechnical/environmental report is based on a subsurface exploration plan designed to consider a unique set of project-specific factors. Depending on the project, these may include: the general nature of the structure and property involved; its size and configuration; its historical use and practice; the location of the structure on the site and its orientation; other improvements such as access roads, parking lots, and underground utilities; and the additional risk created by scope-of-service limitations imposed by the client. To help avoid costly problems, ask the consultant to evaluate how any factors that change subsequent to the date of the report may affect the recommendations. Unless your consultant indicates otherwise, your report should not be used: (1) when the nature of the proposed project is changed (for example, if an office building will be erected instead of a parking garage, or if a refrigerated warehouse will be built instead of an unrefrigerated one, or chemicals are discovered on or near the site); (2) when the size, elevation, or configuration of the proposed project is altered; (3) when the location or orientation of the proposed project is modified; (4) when there is a change of ownership; or (5) for application to an adjacent site. Consultants cannot accept responsibility for problems that may occur if they are not consulted after factors which were considered in the development of the report have changed.

SUBSURFACE CONDITIONS CAN CHANGE.

Subsurface conditions may be affected as a result of natural processes or human activity. Because a geotechnical/environmental report is based on conditions that existed at the time of subsurface exploration, construction decisions should not be based on a report whose adequacy may have been affected by time. Ask the consultant to advise if additional tests are desirable before construction starts; for example, groundwater conditions commonly vary seasonally.

Construction operations at or adjacent to the site and natural events such as floods, earthquakes, or groundwater fluctuations may also affect subsurface conditions and, thus, the continuing adequacy of a geotechnical/environmental report. The consultant should be kept apprised of any such events, and should be consulted to determine if additional tests are necessary.

MOST RECOMMENDATIONS ARE PROFESSIONAL JUDGMENTS.

Site exploration and testing identifies actual surface and subsurface conditions only at those points where samples are taken. The data were extrapolated by your consultant, who then applied judgment to render an opinion about overall subsurface conditions. The actual interface between materials may be far more gradual or abrupt than your report indicates. Actual conditions in areas not sampled may differ from those predicted in your report. While nothing can be done to prevent such situations, you and your consultant can work together to help reduce their impacts. Retaining your consultant to observe subsurface construction operations can be particularly beneficial in this respect.

A REPORT'S CONCLUSIONS ARE PRELIMINARY.

The conclusions contained in your consultant's report are preliminary because they must be based on the assumption that conditions revealed through selective exploratory sampling are indicative of actual conditions throughout a site. Actual subsurface conditions can be discerned only during earthwork; therefore, you should retain your consultant to observe actual conditions and to provide conclusions. Only the consultant who prepared the report is fully familiar with the background information needed to determine whether or not the report's recommendations based on those conclusions are valid and whether or not the contractor is abiding by applicable recommendations. The consultant who developed your report cannot assume responsibility or liability for the adequacy of the report's recommendations if another party is retained to observe construction.

THE CONSULTANT'S REPORT IS SUBJECT TO MISINTERPRETATION.

Costly problems can occur when other design professionals develop their plans based on misinterpretation of a geotechnical/environmental report. To help avoid these problems, the consultant should be retained to work with other project design professionals to explain relevant geotechnical, geological, hydrogeological, and environmental findings, and to review the adequacy of their plans and specifications relative to these issues.

BORING LOGS AND/OR MONITORING WELL DATA SHOULD NOT BE SEPARATED FROM THE REPORT.

Final boring logs developed by the consultant are based upon interpretation of field logs (assembled by site personnel), field test results, and laboratory and/or office evaluation of field samples and data. Only final boring logs and data are customarily included in geotechnical/environmental reports. These final logs should not, under any circumstances, be redrawn for inclusion in architectural or other design drawings, because drafters may commit errors or omissions in the transfer process.

To reduce the likelihood of boring log or monitoring well misinterpretation, contractors should be given ready access to the complete geotechnical engineering/environmental report prepared or authorized for their use. If access is provided only to the report prepared for you, you should advise contractors of the report's limitations, assuming that a contractor was not one of the specific persons for whom the report was prepared, and that developing construction cost estimates was not one of the specific purposes for which it was prepared. While a contractor may gain important knowledge from a report prepared for another party, the contractor should discuss the report with your consultant and perform the additional or alternative work believed necessary to obtain the data specifically appropriate for construction cost estimating purposes. Some clients hold the mistaken impression that simply disclaiming responsibility for the accuracy of subsurface information always insulates them from attendant liability. Providing the best available information to contractors helps prevent costly construction problems and the adversarial attitudes that aggravate them to a disproportionate scale.

READ RESPONSIBILITY CLAUSES CLOSELY.

Because geotechnical/environmental engineering is based extensively on judgment and opinion, it is far less exact than other design disciplines. This situation has resulted in wholly unwarranted claims being lodged against consultants. To help prevent this problem, consultants have developed a number of clauses for use in their contracts, reports and other documents. These responsibility clauses are not exculpatory clauses designed to transfer the consultant's liabilities to other parties; rather, they are definitive clauses that identify where the consultant's responsibilities begin and end. Their use helps all parties involved recognize their individual responsibilities and take appropriate action. Some of these definitive clauses are likely to appear in your report, and you are encouraged to read them closely. Your consultant will be pleased to give full and frank answers to your questions.

The preceding paragraphs are based on information provided by the
ASFE/Association of Engineering Firms Practicing in the Geosciences, Silver Spring, Maryland

APPENDIX B

GEOTECHNICAL DESIGN MEMORANDUM GD-2

GEOTECHNICAL DESIGN MEMORANDUM GD-2 PIONEER CROSSING, LEHI AND I-15 AMERICAN FORK INTERCHANGE

GEOTECHNICAL ANALYSES AND DESIGN METHODOLOGIES

June 11, 2009

Prepared by:
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GEOTECHNICAL DESIGN MEMORANDUM GD-2

PIONEER CROSSING, LEHI AND I-15 AMERICAN FORK INTERCHANGE DAVIS COUNTY, UTAH

GEOTECHNICAL ANALYSES AND DESIGN METHODOLOGIES

1.0 DESIGN MEMORANDUM SCOPE

This Geotechnical Design Memorandum presents the criteria, assumptions, and design methodologies used in our geotechnical analyses for the proposed project elements throughout the project alignment. The design analyses include settlement and stability for embankments and retaining walls, lateral earth pressures for embedded structures, axial and lateral bridge foundation capacity, and liquefaction analyses.

Separate geotechnical design memoranda were issued to address the site-specific details of our analyses and our geotechnical recommendations for each of the above elements within the project. Related analysis procedures and criteria are presented in the following geotechnical design memoranda:

- GD-3, Seismic Design Criteria (Shannon & Wilson, Inc., 2009b)
- GD-4, Liquefaction and Lateral Spreading (Shannon & Wilson, Inc., 2009c)
- GD-5, Pioneer Crossing Approach Fill, Embankment Stability, and Jordan River Bridge (Shannon & Wilson, Inc., 2009d)

Final versions of these and other geotechnical design memoranda issued for the project will be compiled into the Pioneer Crossing, Lehi and I-15 American Fork Interchange Project Geotechnical Report upon completion of the geotechnical engineering design phase of the project. General limitations on the use of this memorandum and recommendations presented herein are presented in Geotechnical Design Memorandum GD-1 (Shannon & Wilson, Inc., 2009a).

2.0 PROJECT ELEMENTS

The Pioneer Crossing, Lehi and I-15 American Fork Interchange project (project) will construct an east to west urban arterial through Lehi connecting Redwood Road to Interstate 15 (I-15). The proposed arterial will parallel State Route 73 at approximately 900 South and will include new bridge structures to extend the roadway over the Union Pacific Railroad tracks near Mill Pond and over the Jordan River in Saratoga Springs. The proposed roadway will be about 100 feet wide, comprising two lanes in each direction, a median, and sidewalks. The project includes a new Interchange at Main Street in American Fork.

This memorandum addresses design methodologies and analyses criteria that are performed for the following project components:

- Jordan River Bridge foundations, approach embankments, and abutment walls.
- General embankments for Pioneer Crossing from Redwood Road at about Station (Sta.) 600+00 to Sta. 725+00.
- General walls for Pioneer Crossing from Redwood Road at about Sta. 600+00 to Sta. 725+00 including mechanically stabilized earth (MSE) walls, and Post and Panel walls.
- Other miscellaneous project features.

The design methodologies and analyses criteria discussed herein are based on information provided in the project Request for Proposal (RFP) (Utah Department of Transportation [UDOT], 2008b), guidelines presented in the UDOT Geotechnical Manual of Instruction (UDOT, 2006) American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specifications 2008, and our experience on similar projects.

3.0 EMBANKMENTS AND RETAINING STRUCTURES

The analysis criteria discussed in this section are applicable to the roadway embankment slopes, retaining walls, and bridge abutment walls including MSE and concrete walls.

3.1 Level of Analyses

Our analyses for embankments and retaining structures consist of performing:

- Global stability analyses for static and seismic conditions for embankments, including ground improvement to meet global stability requirements.
- Global stability analyses for static and seismic conditions for MSE and concrete walls, including ground improvement, minimum retaining wall base dimensions (e.g., MSE wall minimum reinforcement length at the wall base), and retaining wall embedment to meet global stability requirements.
- Elastic, consolidation and secondary compression settlement analyses for general embankments, bridge approach embankments, and MSE walls, and providing estimates of total and differential settlements, including mitigation alternatives to meet warranty settlement requirements.
- Lateral earth pressure analyses for static and seismic conditions for proposed retaining walls.
- Bearing, overturning and sliding stability analyses of MSE walls, including evaluating sliding resistance and passive earth pressures.
- Analyses to evaluate engineering soil properties of native, embankment and retained soil.

Proprietary MSE walls will be designed by the supplier/manufacturer of the system. Our recommendations and geotechnical analyses for MSE walls will address global stability, settlement, and bearing capacity of the proposed walls. We will comment on other geotechnical issues that may affect the design, construction, or performance of the MSE wall and will provide geotechnical parameters for use in MSE design. Our analyses will not include an evaluation of sliding along the base or intermediate reinforcement, overturning of the wall system, design of reinforcement lengths, or design of wall facing elements. We assume that the MSE wall supplier/manufacturer will provide analyses and recommendations regarding these elements of the MSE wall using our recommended geotechnical parameters.

3.2 Global Stability Analyses Criteria

Embankments and retaining walls will be analyzed to meet the minimum allowable factors of safety per Table 5.1 of UDOT (2006) (enclosed).

For seismic conditions, retaining walls and embankments within 50 feet of a bridge foundation or which may affect the bridge performance or structural integrity are designed for the maximum considered earthquake ground motions that have a 2 percent probability of exceedance in 50 years. All other walls are designed for the expected earthquake that has a 10 percent probability of exceedance in 50 years. Embankments and retaining walls are analyzed will be analyzed in accordance with the RFP (UDOT, 2008b) and the UDOT (2006). Specific criteria used include:

- For global stability analyses of the design seismic event not considering liquefaction or seismic-event induced soil strength degradation, use a horizontal pseudo-static horizontal coefficient, k_h, of 0.5 peak ground acceleration and a vertical pseudo-static coefficient, k_v, equal to zero.
- For past seismic conditions (potentially liquefied conditions), global stability analyses consider the extent and reduced shear strength of potentially liquefiable soil. Because liquefaction is assumed to occur after strong ground motions ends, k_h and k_v will be set equal to zero.

3.2.1 Approach Embankments

In accordance with UDOT (2008a), bridge approach embankments within 50 feet of a bridge foundation will be analyzed for liquefaction hazards. UDOT will review the Design-Builder's findings, and in consultation with the Design-Builder's Geotechnical Engineer, will determine whether and where to proceed with lateral spread mitigation.

3.2.2 Retaining Walls

In addition to criteria presented in Section 3.2.1, retaining walls will be analyzed using the criteria and conditions specified in the UDOT (2008a), Section 11.6.5. Retaining walls and bridge abutment walls will be evaluated for static and seismic external stability. Static and seismic stability of MSE walls, except global stability, will be evaluated by the MSE wall supplier/manufacturer. The Mononobe-Okabe method as presented in AASHTO (2008) Appendix 11 and as updated by Zhang, et al. (1998) will be used to calculate the seismic earth pressure for retaining wall design.

MSE wall minimum reinforcement length and embedment depth are designed to meet AASHTO (2008), Sections 11.10.2.1 and 11.10.2.2, respectively.

MSE wall allowable differential settlement and maximum movement measured at the wall face are specified in UDOT (2008a), Section 9.2.1 (D):

- Single-stage walls: 1 percent (6 inches per 50 lineal feet of wall)
- Two-stage walls, first stage (wire facing): 5 percent (12 inches per 20 feet of wall)

- Two-stage walls, second stage (panels): 0.5 percent (3 inches per 50 feet of wall)
- Horizontal alignment: 0.8 percent (3 inches per 30 lineal feet of wall)
- Vertical alignment: 0.8 percent (1 inch per 10-foot vertical segment of wall)
- Overall plumbness: 0.6 percent (1.5 inches for 20-foot height of wall)

3.2.3 Soil Parameters

The soil parameters that are used in our analyses are evaluated on the site-specific subsurface conditions based on the available borings, cone penetration tests (CPTs), test pits, published data reported in the literature, and appropriate in situ test correlations.

In the geotechnical calculations for individual retaining wall or embankment, we will provide discussions and supporting information for the soil parameters used in our geotechnical analyses for that location.

3.2.4 Software

The following software programs are used for embankment and retaining wall stability analyses.

Program	Application
SLOPE/W V.7.13(Geo-Slope International, 2008)	Global stability analyses
PCSTABL / StedWin V 2.8 (Harald W. Van Aller, 2005)	Global stability analyses

Most slope stability analyses are performed using SLOPE/W 2007 (GeoSlope International, 2008). The factors of safety reported are calculated using the Morgenstern-Price method (1965), which satisfies both moment and force equilibrium, and considers both shear and normal interslice forces.

Depending on the specific geometry and configuration of the analyzed wall or embankment, the failure surface search routines used included exit and entry, grid and radius, block specified, or specified surface options. The SLOPE/W optimization technique is employed in our stability analyses to further refine the critical slip surface shape using a segmental technique. The optimization technique is used in all analyzed cases except in the seismic, where the factor of safety (FS) was estimated using the fully defined static critical slip surface.

Where soil improvement is included in the analyses, e.g., deep soil mixing (DSM) or stone columns, the proposed soil improvement is modeled assuming an equivalent homogeneous composite soil zone. The composite soil zone properties were calculated using an area- or volume-weighted average of the soil and the improved soil. Where wall supporting piles or shafts are included in the analyses to achieve the required FS, a shear force, and spacing for the piles or shafts are entered as an input in the model.

PCSTABLE / StedWin (Van Aller, 2005) will be used as needed for verification calculations.

3.3 Settlement Analyses Criteria

Embankments and retaining walls are analyzed to meet the pavement static settlement warranty requirements in Part 9.0 of the RFP, which specify the following maximum permissible settlement for pavement:

- Transverse direction: Maximum 0.25-inch per 12-foot lane width.
- Maximum tolerable longitudinal direction: 0.25-inch per 30 feet.
- Allowable total post-construction settlement: 2 inches, unless otherwise approved by the Department (UDOT, 2008a).

Due to the uncertainties and limitations of our analysis, our settlement estimates indicate likely estimates of settlement across the alignment. Variations in the assumed subsurface stratigraphy, estimated consolidation soil properties and/or in assumed construction sequencing may result in more or less actual settlement.

3.3.1 Settlement Analyses Approach

We evaluated settlement at representative sections selected based on subsurface conditions interpreted from the borings and CPTs and on the geometry of the existing and proposed embankment or wall cross sections.

In plastic silt and clay, we performed settlement analyses using one-dimensional vertical consolidation theory. For settlement due to horizontal consolidation using prefabricated vertical drains, we followed procedures outlined in the Federal Highway Administration (FHWA) manual for prefabricated vertical drains (Rixner et al., 1986). We performed elastic settlement analyses for non-plastic silt, sand and gravel. We assumed:

- Post-construction settlement within embankment materials would be negligible.
- Elastic settlement will occur relatively quickly, i.e., settlement should occur essentially as the load is applied.
- Secondary compression (creep) will start after 95 percent of primary consolidation has occurred.

Our settlement analyses considered multiple time stages to differentiate between the settlement that will occur during the construction phase and long-term settlement that will occur from after paving and until the end of the five-year warranty period. These time stages were chosen to check compliance with warranty settlement requirements. For settlement profiles at utility crossings, we considered total accumulated settlement occurring in the construction and warranty phases.

3.3.2 Soil Parameters

Elastic, vertical and horizontal consolidation, and secondary compression soil parameters that are used in our analyses are evaluated on site-specific subsurface conditions based on available borings, CPTs, laboratory consolidation test results, field dissipation test results, published data, and appropriate in situ test correlations. We used literature from the late-1990s I-15 construction, including Saye and Ladd (2000). In the geotechnical calculation for individual retaining wall or embankment, we will provide discussions and supporting information for the soil parameters used in our geotechnical analyses for that location.

Modulus of elasticity values were estimated based on field Standard Penetration Test (SPT) N-values and CPT tip resistance. Consolidation settlement parameters are based on laboratory one-dimensional consolidation testing data, correlations with Atterberg limits and in situ CPT testing, and published data. We evaluated overconsolidation ratios, strain compression indices, strain recompression indices, secondary strain compression indices, and coefficients of consolidation for individual stratum layers.

3.3.3 Software

We estimated embankment settlement using the commercial software Settle^{3D} developed by RocScience (2008). Specifically, Settle^{3D} was used to model Boussinesq stress distributions for complex embankment configurations and to estimate the elastic and time-rate primary consolidation settlement estimates for the construction sequencing. Settle^{3D} uses traditional one-dimensional consolidation theory to calculate settlement along user defined horizontal lines or discrete points at various depths. Long-term settlement was analyzed at the location of maximum primary consolidation settlement. Spreadsheet calculations were used in conjunction with Settle^{3D} output to calculate the secondary consolidation settlement. Total settlement profiles were developed using the Settle^{3D} output for primary consolidation settlement at specific construction sequencing stages and spreadsheet computation for secondary settlement. An Excel spreadsheet program was used to calculate DSM settlement using the equivalent raft method.

Shannon & Wilson spreadsheet programs were verified using hand calculations. The verification is included in the individual geotechnical calculation packages.

3.4 Lateral Earth Pressure and External Stability Criteria

We calculated lateral earth pressures for each wall based on the soil type and conditions of the soil retained by the wall. Active and at-rest pressures are calculated based on Rankine theory, passive earth pressure is calculated based on Coulomb theory that takes into account the friction between the soil and wall. Dynamic earth pressure increments are calculated using equations developed by Zhang et al. (1998), which update the relationships developed by Mononobe (1924) and Okabe (1924).

External stability for conventional cantilever walls and MSE walls are checked against sliding, bearing resistance, and overturning per AASHTO (2008), Sections 11.6.3 and 11.10.5, respectively. The coefficient of sliding friction is calculated based on the foundation soil-wall base interface friction angle, and on the foundation soil-reinforcement interface friction angle in case of MSE wall. In our analyses, we assumed foundation soil-reinforcement interface friction angle to be between one-half and two-thirds of the foundation soil friction angle. For a cohesive foundation soil, we assumed the nominal sliding resistance to be the lesser of undrained shear strength of the foundation soil or one-half of the effective overburden pressure per AASHTO (2008), Section 10.6.3.4.

The bearing resistance of the foundation soil underneath a retaining wall is calculated similar to that of a shallow foundation using soil mechanics theories. We followed the procedures outlined in AASHTO (2008), Section 10.6.3.1.2, using representative soil parameters. For an MSE wall, an equivalent footing is assumed that has a length equal to the wall length and width equal to the reinforcement length.

Overturning stability was evaluated for MSE walls so that the resultant of the reaction forces is within the middle one-half of the base (reinforcement length).

4.0 DEEP FOUNDATIONS

Sizes for driven piles for Jordan River bridge and for drilled-shaft for Post and Panel walls were selected based on applied loads, soil conditions, design and construction requirements, and structural analyses results.

4.1 Axial Capacity Analyses Procedures

We evaluated the axial capacity of driven piles and drilled shaft foundations in accordance with UDOT (2006) and AASHTO (2008), Articles 10.7 and 10.8, for driven piles and drilled shafts, respectively.

Drilled shaft axial capacity was evaluated at Service Limit, Strength Limit, and Extreme Limit states.

Static capacity of driven piles is in general for estimate of the piles length. Actual capacity is determined through pile load test, or dynamic testing and Pile Driving Analyzer (PDA)/Case Pile Wave Analysis Program (CAPWAP) analyses. Driven piles are usually constructed in groups, settlement is controlled by the pile group settlement rather than of a single pile. Mobilized tip resistances for driven piles are difficult to estimate, where piles may be driven to refusal, and hence all tip resistance is mobilized. Therefore, axial capacity for the driven piles is estimated at the Strength Limit and the Extreme Limit states and not for the Service Limit state. Axial capacity is evaluated as follows:

Service Limit State: Axial capacity was evaluated based on side friction resistance and tip resistance for service settlement limits of 0.5 and 1.0 inch (drilled shafts only).

- Strength Limit State: Axial capacity was evaluated based on ultimate side friction resistance and ultimate tip resistance.
- Extreme Limit State: Axial capacity was evaluated based on ultimate side friction resistance and ultimate tip resistance. Downdrag forces due to liquefaction-induced settlement are considered acting on the foundation when applicable.

According to the AASHTO (2008), we evaluated the following loading cases for bridge deep foundations.

LOADING CASES FOR SERVICE STRENGTH AND EXTREME LIMIT STATES

Limit State	Nominal Resistance	Load Combination*
Service (drilled shafts only)	Service ("Service Limit" plot)	DL+LL
Strength	Strength ("Strength Limit" plot)	DL+ LL
Extreme	Strength+ ("Strength Limit" plot)	DL+EQ
	Extreme ("Extreme Limit" plot)	DL+EQ
		DL+LL

Notes:
* Other loads like "wind loads" should be applied as appropriate.
\+ Strength nominal resistance using Extreme Limit Resistance Factors.
DL = dead load
EQ = earthquake load
LL = live load

We assumed the following AASHTO (2008) resistance factors for bridge deep foundation design:

DRILLED SHAFTS

Limit State	Side Friction	Tip Resistance	Uplift
Service Limit	1.00	1.00	1.00
Strength Limit (Cohesionless Soils)	0.55	0.50	0.45
Strength Limit (Cohesive Soils)	0.45	0.40	0.35
Extreme Limit	1.00	1.00	1.00

DRIVEN PILES

Limit State	Side Friction	End Bearing	Uplift
Strength Limit			
AASHTO LRFD[1]	0.25 - 0.5	0.25 - 0. 5	0.25- 0.4
AASHTO w/ PDA or CAPWAP	0.65	0.65	0.60
Extreme Limit	1.00	1.00	0.80

Notes:
[1] Depend on analytical method.
AASHTO = American Association of State Highway and Transportation Officials
CAPWAP = Case Pile Wave Analysis Program
LRFD = Load and Resistance Factor Design
PDA = pile-driving analyzer

4.2 Lateral Capacity Analysis

For drilled shafts and driven piles lateral analyses, we estimated soil properties and parameters for the structural designer to use when evaluating deep foundation lateral capacity using the program LPile ^{PLUS} (EnSoft, 2006). The soil parameters relevant to lateral resistance were selected based on correlations in the LPile ^{PLUS} program manual with consistency of cohesive soils, and relative density of cohesionless soils and their depth relative to the groundwater table.

Our recommendations include reduced soil properties for soil layers where liquefaction could occur. Reduced soil properties were calculated based on our liquefaction analyses described in Section 5.0. P-multiplier recommendations are according to AASHTO (2008), Section 10.7.2.4.

4.3 Soil Parameters

Soil parameters that are used in our analyses for estimating side friction and tip resistance of deep foundations are evaluated on site-specific subsurface conditions based on available borings, CPTs, published data reported in the literature, and appropriate in situ test correlations with field SPT (N-values) blow counts, and CPT tip resistance. Nominal unit skin friction and unit end-bearing values are estimated for each soil unit the deep foundation is driven or drilled into using empirical relationships and methods identified in the AASHTO (2008) and our experience in similar soil and project conditions. Our design methods are in general accordance with the Nordlund method and the α method (Hannigan et al., 2006) for driven piles in granular and cohesive soils, respectively. For drilled shafts, our design methods are in general accordance with Reese and O'Neill (1999) methods for granular and cohesive soil. The above methods and their corresponding resistance factors are referenced in the AASHTO (2008).

4.4 Software

The following software programs are used for deep foundations axial and lateral capacity analyses.

Program	Application
Shannon & Wilson Excel spreadsheet (2007)	Deep Foundations axial capacity analyses
LPile^{PLUS} V.7 (Ensoft, 2006)	Deep Foundations lateral capacity analyses

Axial capacity analyses are performed using Shannon & Wilson in-house Excel spreadsheet program using the methods outlines above. Hand verification calculations are provided with deep foundations axial capacity geotechnical calculation packages.

5.0 LIQUEFACTION ANALYSIS

We evaluated the soil liquefaction potential based on the Seed simplified semi-empirical procedure as revised by:

- Youd et al. (2001)
- Idriss and Boulanger (2004)
- Seed et al. (2003)

In this procedure, the SPT blow count is correlated to the liquefaction resistance of the soil (expressed as cyclic resistance ratio). The soil resistance is compared to the earthquake-induced loading (expressed as cyclic stress ratio) and a corresponding FS against liquefaction is calculated.

We estimate volumetric strain in the liquefiable soil based on relationships by Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992).

We used a detailed Shannon & Wilson Excel spreadsheet using the analysis methods described above to calculate liquefaction susceptibility, induced settlement, and liquefied soil strength. Hand calculations and verification of the spreadsheet is provided with liquefaction analysis geotechnical calculation packages.

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Table 5.1: Summary of Minimum Allowable Factors of Safety
Embankments and Retaining Wall Stability

		Condition	UDOT	AASHTO
Embankments Adjacent to Abutments*	Construction Global Stability	Static (Non-Impact)	1.1	
		Static (Adjacent Impact)	1.3	
	Long-Term Global Stability	Static	1.3	
		Dynamic (10% PE 250 yrs)	1.0	
		Post-Liquefaction Analysis	**	
	Overall Bearing Capacity	Static	2.0	
	Lateral Spreading	Static	2.0	
	Lateral Squeezing	Static	1.5	
Walls Adjacent to Abutments*	Construction Global Stability	Static (Non-Impact)	1.1	
		Static (Adjacent Impact)	1.3	
	Long-Term Global Stability	Static		1.5
		Dynamic (10% PE 250 yrs)	1.0	1.1
		Post-Liquefaction Analysis	**	---
	Sliding	Static		1.5
	Overturning	Static		2.0
	Bearing	Static		2.5
General Embankments	Construction Global Stability	Static (Non-Impact)	1.1	
		Static (Adjacent Impact)	1.3	
	Long-Term Global Stability	Static	1.2	
		Dynamic	N/R***	
	Overall Bearing Capacity	Static	1.5	
	Lateral Spreading	Static	1.5	
	Lateral Squeezing	Static	1.3	
General Walls	Construction Global Stability	Static (Non-Impact)	1.1	
		Static (Adjacent Impact)	1.3	
	Long-Term Global Stability	Static		1.3
		Dynamic (10% PE 50 yrs)	1.0	1.1
		Post-Liquefaction Analysis	**	—
	Sliding	Static		1.5
	Overturning	Static		2.0
	Bearing	Static		2.5

* *Within 50 feet of the foundation*
** *Post-Liquefaction Deformation Analysis required where FS < 1.1 for the designated seismic acceleration (See Section 5.4.1.1)*
*** *N/R- Not required*

APPENDIX C

GEOTECHNICAL DESIGN MEMORANDUM GD-3

SHANNON & WILSON, INC.

GEOTECHNICAL DESIGN MEMORANDUM GD-3 PIONEER CROSSING, LEHI AND I-15 AMERICAN FORK INTERCHANGE

GROUND MOTIONS FOR SEISMIC DESIGN

May 19, 2009

Prepared by:
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23-1-01178-010

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PIONEER CROSSING, LEHI AND I-15 AMERICAN FORK INTERCHANGE

GROUND MOTIONS FOR SEISMIC DESIGN

1.0 DESIGN MEMORANDUM SCOPE

This Geotechnical Design Memorandum provides ground motions (response spectra) for seismic design of the Jordan River Bridge and its approaches:

Related analyses procedures and criteria are presented in the following Geotechnical Design Memoranda:

- GD-2, Geotechnical Analysis and Design Methodologies (Shannon & Wilson, Inc., 2008b)
- GD-5, Pioneer Crossing Approach Fill, Embankment Stability, and Jordan River Bridge (Shannon & Wilson, Inc., 2008c)

Final versions of this and other geotechnical design memoranda issued for the project will be compiled into the Pioneer Crossing, Lehi and I-15 American Fork Interchange Project Geotechnical Report upon completion of the geotechnical engineering design phase of the project. General limitations on the use of this memorandum and recommendations presented herein are presented in Geotechnical Design Memorandum GD-1 (Shannon & Wilson, 2008a).

2.0 DESIGN RESPONSE SPECTRA

As required by the project Request for Proposal Part 4, Section 5 of the Utah Department of Transportation (UDOT) Geotechnical Manual of Instruction (UDOT, 2006), and Section 6 of the UDOT Structures Design Manual (UDOT, 2007), seismic design of the bridges will be in general accordance with the two level performance/ground motions in the Recommended LRFD Guidelines for the Seismic Design of Highway Bridges (2003) by the Applied Technology Council and Multidisciplinary Center for Earthquake Engineering Research (ATC-49).

The two ground motion levels in ATC-49 and modified in Section 6.2.2 of UDOT 2007 are defined as follows:

- Expected Earthquake (EE) – Ground motions with a 10 percent probability of exceedance in 50 years (about a 500-year return period).
- Maximum Considered Earthquake (MCE) – Ground motions with a 2 percent probability of exceedance in 50 years (about a 2,500-year return period).

Recommended EE and MCE design spectra for the bridge sites are shown in Figures GD3-1 and GD3-2, respectively. Spectral accelerations and corresponding periods are provided in Table GD3-1. These spectra were developed considering site soil conditions.

3.0 DESIGN RESPONSE SPECTRA DEVELOPMENT

3.1 General Procedure for Horizontal Motions

Computation of seismic forces and design spectra in ATC-49, and as modified in Section 6.3.3.1 of the UDOT Structures Design Manual (UDOT, 2007), is based on seismological input and site soil response factors.

The seismological inputs are the peak ground acceleration coefficient (PGA), short period spectral acceleration (S_S), and spectral acceleration at the 1 second period (S_1). The PGA, S_S, and S_1 values for this project are based on data from the U.S. Geological Survey (USGS) 2002 National Seismic Hazard Mapping Project (Frankel et al., 2002) and are listed in Figures GD3-1 and GD3-2 for EE and MCE ground motion levels, respectively.

The site soil response factors are based on determination of the Site Class. Based on the subsurface conditions at the Jordan River Bridge (Shannon & Wilson, 2008c), it is our opinion that the site corresponds to Site Class D. Site Class D is defined as a soil profile that has an average shear wave velocity within 100 feet of the ground surface (v_{s100}) between 600 and 1,200 feet per second (fps). Shear wave velocity measurements were made in CPT-2 and SWC-211 near the Jordan River Bridge. The v_{s100} from the cone penetrometer tests range between 697 and 737 fps, which corresponds to Site Class D. The F_{pga}, F_a, and F_v values corresponding to Site Class D and the PGA, S_S, and S_1 values are shown in Figure GD3-1 for the EE and Figure GD3-2 for the MCE.

3.2 Fault Directivity

Section 3.4 of ATC-49 requires that a site-specific procedure should be used to develop the design spectrum if the site is located within 10 kilometers (km) of a known active fault. Section 3.4.3 clarifies that for sites located within 10 km of an active fault, the site-specific studies should quantify near-fault effects on the design ground motions. The Jordan River Bridge site is located approximately 14 km west of the active Provo Segment of the Wasatch Fault. The USGS (Peterson et al., 2008) currently characterizes this fault segment as a north-south trending, west-dipping, normal fault with a dip of 50 degrees, a width of 20 km and a length of 77 km that is capable of producing an earthquake of a maximum magnitude of 7.4 with recurrence interval of approximately 2,400 years. No significant fault directivity effects are expected because of the distance and geometry between the fault and the bridge site.

4.0 REFERENCES

Applied Technology Council (ATC) and Multidisciplinary Center for Earthquake Engineering Research (MCEER), 2003, Recommended LRFD guidelines for the seismic design of highway bridges, part I: specifications and part II: commentary and appendices.

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TABLE GD3-1
DESIGN RESPONSE SPECTRA

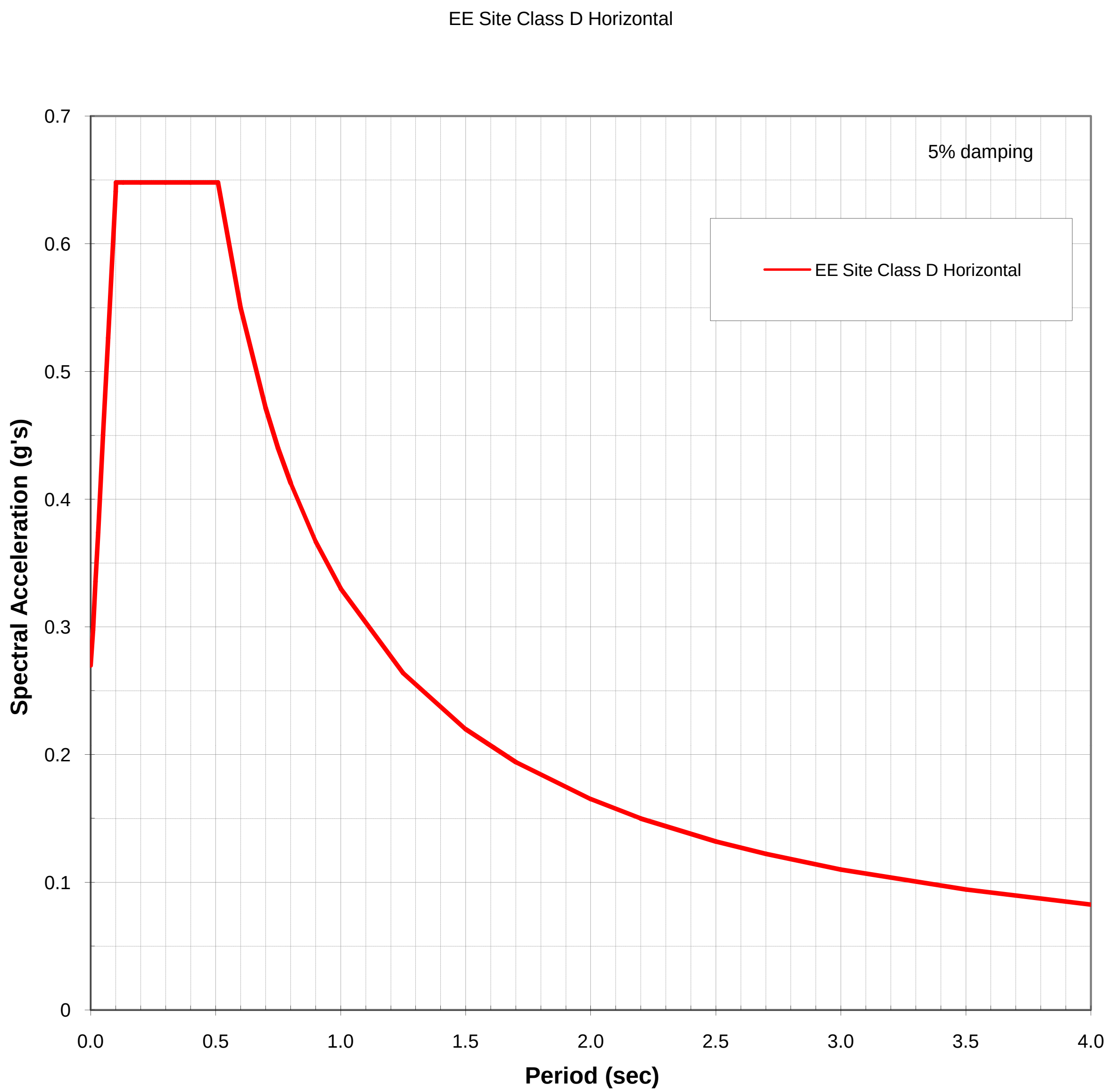
MCE Design Spectra

Period	Horizontal Spectral Acceleration (g's)
0.0 (PGA)	0.460
0.010	0.512
0.020	0.570
0.030	0.628
0.050	0.744
0.070	0.861
0.100	1.035
0.119	1.134
0.130	1.134
0.200	1.134
0.300	1.134
0.400	1.134
0.595	1.134
0.600	1.106
0.700	0.948
0.750	0.885
0.800	0.830
0.900	0.737
1.000	0.664
1.250	0.531
1.5	0.442
1.7	0.390
2.0	0.332
2.2	0.302
2.5	0.265
2.7	0.246
3.0	0.221
3.5	0.190
4.0	0.166

EE Design Spectra

Period	Horizontal Spectral Acceleration (g's)
0.0 (PGA)	0.270
0.010	0.297
0.020	0.336
0.030	0.374
0.050	0.450
0.070	0.526
0.100	0.641
0.102	0.648
0.130	0.648
0.200	0.648
0.300	0.648
0.400	0.648
0.509	0.648
0.509	0.648
0.600	0.550
0.700	0.471
0.750	0.440
0.800	0.413
0.900	0.367
1.000	0.330
1.250	0.264
1.5	0.220
1.7	0.194
2.0	0.165
2.2	0.150
2.5	0.132
2.7	0.122
3.0	0.110
3.5	0.094
4.0	0.083

Notes:
EE = Expected Earthquake
MCE = Maximum Considered Earthquake
PGA = peak ground acceleration



Response Spectrum Parameters[1]

Spectral Acceleration (g)

PGA = 0.19 g's	F_{pga} = 1.42
S_S = 0.45 g's	F_a = 1.44
S_1 = 0.15 g's	F_v = 2.20
S_{DS} = 0.65 g's	T_0 = 0.10 sec.
S_{D1} = 0.33 g's	T_S = 0.51 sec.
A_s = 0.27 g's	

NOTES

1. Per Section 6 of UDOT Structures Design Manual, "Expected Earthquake" (EE) ground motions correspond to a 10% probability of exceedance in 50 years.

2. The horizontal response spectra were constructed using a 3-point method required by Section 6 of the UDOT Structures Design Manual and as provided in 2008 AASHTO LRFD Guidelines.

Pioneer Crossing, Lehi
I-15 American Fork Interchange
Lehi, Utah

SITE CLASS D, EE
HORIZONTAL
RESPONSE SPECTRA

May 2009 | 23-1-01178-010

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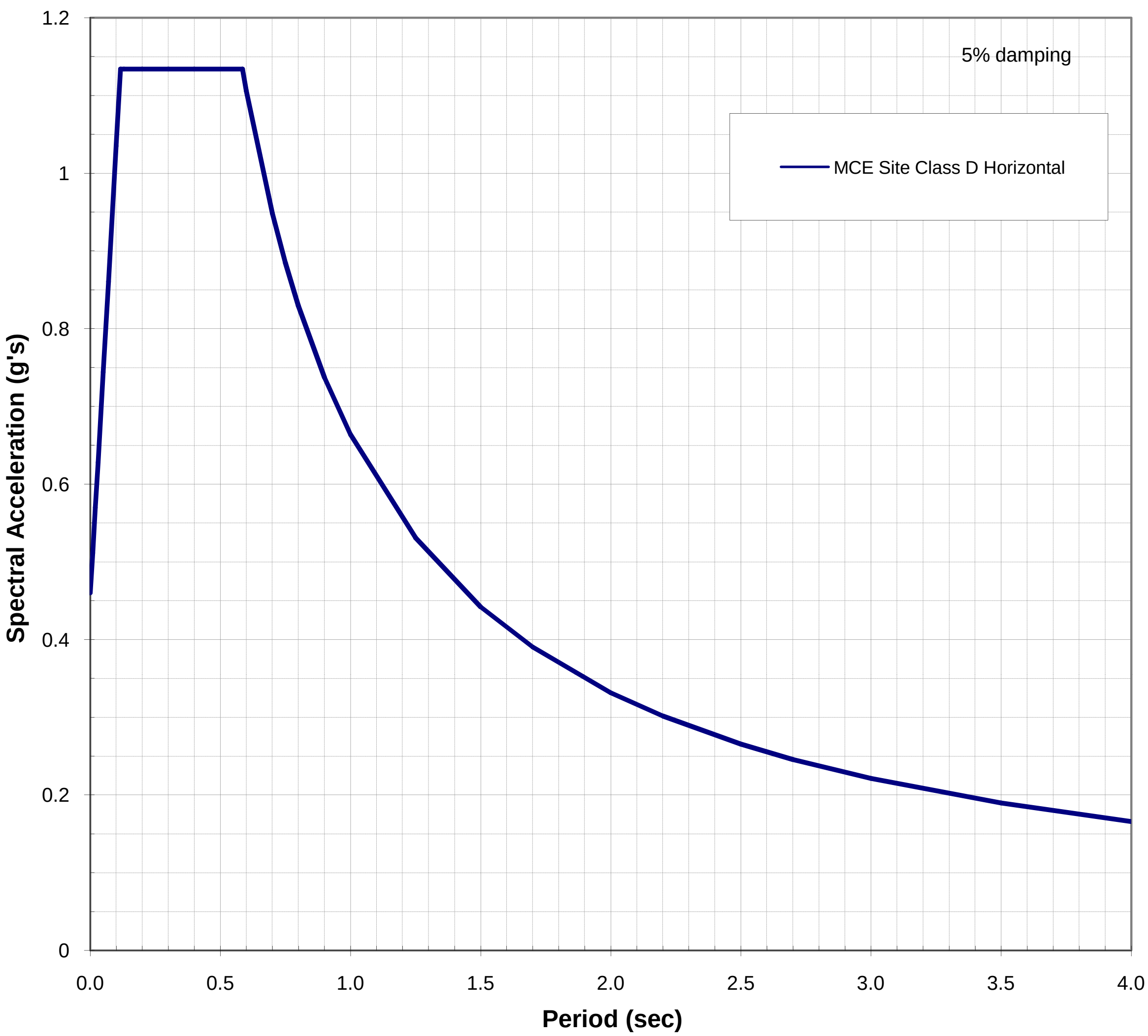
Geotechnical and Environmental Consultants

FIG. GD3-1

MCE Site Class D Horizontal



Response Spectrum Parameters[1]

Spectral Acceleration (g)

PGA = 0.43 g's	F_{pga} = 1.07
S_S = 1.05 g's	F_a = 1.08
S_1 = 0.42 g's	F_v = 1.58
S_{DS} = 1.13 g's	T_0 = 0.12 sec.
S_{D1} = 0.66 g's	T_S = 0.59 sec.
A_s = 0.46 g's	

NOTES

1. Response spectrum parameters are for the "Maximum Considered Earthquake" (MCE) as defined in ATC-49 and Section 6 of UDOT Structures Design Manual and corresponds to a 2% probability of exceedance in 50 years.

2. The horizontal response spectra were constructed using a 3-point method required by Section 6 of the UDOT Structures Design Manual and as provided in 2008 AASHTO LRFD Guidelines.

Pioneer Crossing, Lehi
I-15 American Fork Interchange
Lehi, Utah

SITE CLASS D, MCE HORIZONTAL RESPONSE SPECTRA

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FIG. GD3-2

APPENDIX D

GEOTECHNICAL DESIGN MEMORANDUM GD-4

GEOTECHNICAL DESIGN MEMORANDUM GD-4 PIONEER CROSSING, LEHI AND I-15 AMERICAN FORK INTERCHANGE

LIQUEFACTION ANALYSES AND GROUND IMPROVEMENT RECOMMENDATIONS

March 18, 2009

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PIONEER CROSSING, LEHI, AND I-15 AMERICAN FORK INTERCHANGE

LIQUEFACTION ANALYSES AND GROUND IMPROVEMENT RECOMMENDATIONS

1.0 DESIGN MEMORANDUM SCOPE

This Geotechnical Design Memorandum summarizes our evaluation of liquefaction potential and associated effects (e.g., reduced soil shear strength and lateral spreading) at the Jordan River Bridge and its approaches, the mechanically stabilized earth (MSE) wall section from Station 630+50 to 640+50, and the Post and Panel wall section from Station 650+50 to 660+00:

- Jordan River Bridge – Conveys proposed alignment over the Jordan River
- MSE Wall Section – Reduces impact on nearby wetlands
- Post and Panel Wall Section – Reduces impact on nearby wetlands

This memorandum provides our recommendations for ground improvement at the Jordan River Bridge east abutment to mitigate effects of liquefaction on abutment stability.

Related analyses procedures, criteria, and data are presented in the following Geotechnical Design Memoranda and report:

- GD-2, Geotechnical Analysis and Design Methodologies (Shannon & Wilson, 2009b)
- GD-3, Seismic Design Criteria (Shannon & Wilson, 2009c)
- GD-5, Pioneer Crossing Approach Fill, Embankment Stability, and Jordan River Bridge (Shannon & Wilson, 2009d)

Final versions of these and other geotechnical design memoranda issued for the project will be compiled into the Pioneer Crossing, Lehi, and I-15 American Fork Interchange Project Geotechnical Report upon completion of the geotechnical engineering design phase of the project. General limitations on the use of this memorandum and recommendations presented herein are presented in Geotechnical Design Memorandum GD-1 (Shannon & Wilson, 2009a).

2.0 LIQUEFACTION EVALUATION

2.1 Approach

We evaluated the liquefaction potential at the bridge and wall sites using the Seed simplified semi-empirical procedure as revised by:

- Youd et al. (2001)
- Idriss and Boulanger (2004)
- Seed et al. (2003)

In this procedure, the Standard Penetration Test (SPT) blow count or Cone Penetration Test (CPT) cone tip resistance is correlated to the liquefaction resistance of the soil (expressed as cyclic resistance ratio). The soil resistance is compared to the earthquake-induced loading (expressed as cyclic stress ratio), and a corresponding factor of safety (FS) against liquefaction is calculated.

In general, SPTs performed in mud-rotary borings are considered better indicators of liquefaction potential than those from hollow-stem auger (HSA) borings because of seepage pressure and soil disturbance (heave) that may occur in HSA borings (Seed et al., 1985). However, the borings provided in the Request for Proposals (RFPs) (Utah Department of Transportation [UDOT], 2008) used HSA drilling methods. Therefore, two HSA borings were used in the liquefaction analysis at the Jordan River Bridge.

SPTs were obtained using an auto-hammer. We assumed a hammer energy transfer rate of 75 percent in the liquefaction analyses, as recent calibration data for the specific hammers were not available. We based our assumption on work by Biringen and Davie (2008) who summarize the results of 220 tests run by them using auto-hammers together with a summary of UDOT and Florida Department of Transportation (FDOT) measurements. Biringen and Davie indicate that average energy transfer rates for their measurements, UDOT, and FDOT, were 81.5, 79.6, and 75.5 percent, respectively. Standard deviations for their data and FDOT data were 6.4 and 7.9 percent, respectively. For our analyses we used the lower (more conservative) UDOT average of 75 percent. For the site soil types, SPT blow counts (e.g., 1 to 4 in SWB-202) and ground motion levels, the results of the analyses are relatively insensitive to the range of hammer energy efficiencies used in the analyses.

CPT soundings were also used to calculate cyclic resistance ratios. Cyclic resistance ratios calculated from CPT data are sensitive to measured soil properties such as Atterberg Limit Plastic Index (PI). PI is measured directly from soil samples collected in SPTs and cannot be measured by a CPT. Therefore, we assumed that SPTs provide a quantitative assessment of liquefaction potential superior to CPTs. CPTs are assumed to provide a more qualitative assessment, showing relative trends of liquefaction FS but not necessarily absolute values.

The boring and CPT locations, boring and CPT logs, laboratory test data, and a description of field exploration procedures for the borings are provided in Geotechnical Design Memorandum GD-5 (Shannon & Wilson, 2009d).

We estimated reduced shear strengths for potentially liquefiable soil using relationships by Seed and Harder (1990), Olson and Stark (2002), Olson and Stark (2003), Idriss and Boulanger (2007), and Kramer (2008). We estimated volumetric strain using the relationships by Tokimatsu and Seed (1987) and Ishihara and Yoshimine (1992).

Section 4D.7.5.2 of the RFP indicates that the liquefaction assessments at bridge sites include, as a minimum:

- Supplemental exploratory borings (as required),
- Geologic reconnaissance and history research (including review of available aerial photographs), and
- Additional site-specific topographic surveys (as needed) to assess lateral spreading hazards and develop mitigation recommendations.

With regard to these requirements, our assessment of liquefaction potential is based on the RFP-phase and design phase borings and CPTs (as previously described). In our opinion, the data from these explorations are adequate to characterize the liquefaction potential. While we have reviewed current aerial photos of the area, the principle (and effective) geologic-reconnaissance-and-history-research activity was excavation and observation of geologic materials in the test pit excavations along the east bank of the Jordan River. Six test pits (SWT-217, SWT-219, SWT-225, SWT-227, SWT-228, and SWT-234) were excavated in the vicinity of east bridge abutment, as shown on Figure GD5-2 of GD-5. The geologic information from these test pits (test pit logs are provided in GD-5 Appendix A) are used to evaluate the lateral extent of and presence of groundwater in potentially-liquefiable near-surface soils. In our opinion, the topographic information developed in the current design phase is adequate to assess lateral spreading/embankment instability hazard and to develop mitigation recommendations, especially considering that the topography will be significantly modified by construction of the bridge approach embankment.

2.2 Ground Motions

Earthquake loading was determined for the two design ground motion levels specified in ATC-49 (2003) and modified in Section 6.2.2 of UDOT (2007). The two ground motion levels are defined as follows:

- Expected Earthquake (EE) – Ground motions with a 10 percent probability of exceedance in 50 years (about a 500-year return period).
- Maximum Considered Earthquake (MCE) – Ground motions with a 2 percent probability of exceedance in 50 years (about a 2,500-year return period).

Peak ground accelerations corresponding to EE and MCE ground motion levels are provided in Geotechnical Design Memorandum GD-3 (Shannon & Wilson, 2009c) and are 0.27g and 0.46g, respectively. We used a magnitude of M_W = 7.0 for both the EE and MCE. These magnitudes are modal values from the U.S. Geological Survey 2002 National Seismic Hazard Mapping Project (Frankel et al., 2002) ground motion hazard deaggregation.

2.3 Jordan River Bridge

For the Jordan River Bridge, cyclic resistance ratios were calculated for SPTs conducted in the two borings drilled for the proposal phase using HSA techniques (B-03 and B-08), one boring drilled for the design phase using mud-rotary drilling techniques (SWB-202), and two CPTs (SWC-211 and SWC-220) pushed for the design phase. Groundwater was assumed to be within about 4 feet of the existing ground surface.

2.3.1 Liquefaction Extent

Calculated FSs versus depth for the Jordan River Bridge borings and CPTs are shown in Figures GD4-1 through GD4-5 and GD4-6 through GD4-10 for the EE and MCE ground motion levels. Our calculations show that the FSs against liquefaction for layers or zones of sandy soil are less than 1.1 from the groundwater table to a depth of approximately 44 feet below the existing ground surface (bgs) for EE and MCE ground motion levels.

Beneath the west abutment, sandy layers between depths of approximately 13 and 17 feet and 25 to 32 feet have calculated FSs less than 1.1. These layers appear to be discontinuous. At a depth of about 39 to 44 feet bgs, there appears to be a relatively continuous layer of medium dense sand with FSs less than 1.1 for both MCE and EE ground motion levels, and it appears that this layer extends east across the river and beneath the east abutment.

Beneath the east abutment, calculated FSs against liquefaction are less than 1.1 from the groundwater table to a depth of 12 feet bgs. This zone of potentially liquefiable soil appears to be laterally continuous eastward in the direction perpendicular to the proposed roadway alignment. A discontinuous zone of potentially liquefiable soil was encountered at approximately 17 to 18 feet bgs at the east abutment. At a depth of about 34 to 36 feet bgs, a layer of medium dense sand with FSs less than 1.1 for both MCE and EE ground motion levels is present and appears to be laterally continuous with the sand layer observed between 39 and 44 feet bgs beneath the west abutment.

2.3.2 Liquefaction Effects

The effects of liquefaction may include reduced soil shear strength and associated lateral spreading or embankment instability, and settlement.

Liquefaction-induced reduced shear strength estimates are presented in Figures GD4-11 through GD4-13.

We performed stability analyses for the embankments that incorporate the reduced soil shear strengths (see GD-5, Appendix C, Figures GD-5-C-4, -8, -16, and -21). The discontinuous zones beneath the abutments that are not interbedded in soft cohesive soils were explicitly and conservatively modeled as continuous layers in the stability models. Discontinuous zones that are interbedded in soft cohesive soils were not explicitly modeled; the shear strength of the larger encompassing soft cohesive soil zone was reduced by 15 percent for post-seismic liquefied soil stability analyses. As described in GD-5, the post-seismic liquefied soil stability met UDOT requirements (UDOT, 2006).

To mitigate the liquefaction potential between the groundwater table and 12 feet bgs at the east abutment, a zone of improved ground was included in the stability analyses (see GD-5, Appendix C, Figure GD-5-C-1 through -4). In the east abutment stability analyses, the improved ground was modeled with the shear strength of compacted structural fill (phi = 36 degrees). Recommendations for ground improvement at the east abutment are provided in Section 3 of this memorandum.

In general, we estimate the volumetric strain in the liquefiable soil would be on the order of 2 to 4 percent which corresponds to about 1 to 2 inches of settlement for EE and MCE ground motions, respectively. Based on these strain levels and the thickness of the potentially liquefiable soils beneath each abutment, recommendations for liquefaction-induced down-drag on the abutment pile foundations are provided in GD-5. The depth of liquefaction-induced down-drag incorporated into the pile resistance recommendations in GD-5 are 44 and 35 feet bgs for the west and east abutments, respectively (see GD-5, Figures GD5-9 and -10).

2.4 Mechanically Stabilized Earth (MSE) Wall Section

For the MSE wall, cyclic resistance ratios were evaluated from SPTs conducted in one design phase mud-rotary boring (SWB-203) and two design phase CPTs (SWC-212 and SWC-213). No subsurface explorations were provided in the RFP (UDOT, 2008). The groundwater was assumed to be within about 2 feet of the existing ground surface.

2.4.1 Liquefaction Extent

Calculated FSs versus depth for the MSE Wall section boring and CPTs are shown in Figures GD4-14 through GD4-16, and GD4-17 through GD4-19 for the EE and MCE ground motion levels. Our calculations show FSs against liquefaction are less than 1.1 for a layer encountered 13 to 17 feet bgs for EE and MCE ground motion levels. For CPT SWC-212, our calculations show FSs that are less than 1.1 for zones 31 to 35 feet and 47 to 50 feet bgs.

2.4.2 Liquefaction Effects

The potential effects of liquefaction include a reduction in soil shear strength, settlement, and global wall instability.

Liquefaction-induced reduced shear strength estimates are presented in Figure GD4-20. Reduced shear strengths were incorporated into the post-seismic, liquefied-soil global stability analyses presented in GD-5, Appendix C, Figure GD5-C-25. As described in GD-5, the post-seismic liquefied soil stability met UDOT requirements.

We estimate volumetric strain in the liquefiable soil layer 13 to 17 feet bgs would be on the order of 2 to 4 percent, which corresponds to about 1 to 2 inches of settlement.

2.5 Post and Panel Wall Section

For the Post and Panel Wall section, cyclic resistance ratios were evaluated from SPTs conducted in one design phase mud-rotary boring (SWB-204) and two design phase CPT (SWC-214 and SWC-215). No subsurface explorations were provided in the RFP (UDOT, 2008). The groundwater was assumed to be within about 6 feet of the existing ground surface.

2.5.1 Liquefaction Extent

Calculated FSs versus depth for the Post and Panel Wall section boring and CPTs are shown in Figures GD4-21 through GD4-23, and GD4-24 through GD4-26 for the EE and MCE ground motion levels. Our calculations based on boring SWB-204 show FSs against liquefaction that are greater than 1.1 for length of the profile for EE and MCE ground motion levels. Our calculations based on CPT SWC-214 and SWC-215 show FSs against liquefaction less than 1.1 for scattered zones below the groundwater table to approximately 50 feet bgs.

Calculated FSs less than 1.1 are identified in lenses in CPT SWC-215 and are less numerous in CPT SWC-214. In our opinion, the potential for liquefaction at this location appear to be low. No liquefaction is indicated by the SPTs and the distribution of the low FSs for the CPT soundings indicates that if liquefaction were to occur, it would be in discontinuous scattered zones.

2.5.2 Liquefaction Effects

The potential effects of liquefaction include a reduction in soil shear strength, settlement, and global wall instability. Consistent with the low liquefaction potential of the soil at this location, the potential for lateral spreading, significant soil strength loss, and settlement are also low, in our opinion. For our post-seismic stability analyses, we assumed the majority of the shear strength of the soft cohesive soil would be reduced by 15 percent in post-seismic conditions (see GD-5, Appendix C, Figure GD5-C-29). As described in GD-5, the post-seismic stability met UDOT requirements.

3.0 JORDAN RIVER BRIDGE EAST ABUTMENT LIQUEFACTION MITIGATION

3.1 Mitigation Approach

As described in Section 2.3.2 of this memorandum, ground improvement is required to meet UDOT stability requirements for the post-seismic, liquefied-soil conditions. To meet stability requirements at the east abutment (see stability analyses in GD-5, Appendix C, Figure GD5-C-4), ground improvement was required for the potentially liquefiable soil between the groundwater table and 12 feet bgs. We modeled improved ground that extends from approximately 5 feet west of the east abutment to approximately 33 feet east of the east abutment. The improved ground was modeled with a shear strength of 36 degrees. This shear strength is typical of a well compacted, granular structural fill. At this modest improved-ground shear strength, the critical failure surface is forced below the improved ground and into the underlying cohesive soils. Deeper potential failure surfaces meet UDOT stability requirements.

Based on site constraints and specialty contractor availability, we understand that Kiewit/Clyde, in conjunction with Access Utah County, has selected to use deep soil mixing (DSM) to improve the ground.

Our DSM liquefaction mitigation recommendations are based on our review of published, peer-reviewed literature (e.g., Martin et al., 2004; Durgunoglu, 2006; O'Rourke and Goh, 1997) and recent studies by Shannon & Wilson (2008a) for liquefaction mitigation on specific projects (e.g., Alaskan Way Viaduct and Seawall Replacement Program, Dynamic Soil Structure Interaction Analyses Report, 2008). These studies show that replacement of potentially liquefiable soil by soil cement at an area replacement ratio of about 0.3 increases the overall stiffness of the soil mass, reduces shear strains, and can thereby reduce liquefaction potential at even relatively high levels of earthquake ground shaking, such as MCE ground motions at the Jordan River Bridge site.

3.2 Deep Soil Mixing (DSM) Ground Improvement Recommendations

Cement DSM is a process to treat soil in situ to increase shear strengths, improve compressive behavior, and decrease permeability. DSM typically involves a series of one to four hydraulically driven, 18- to 72-inch-diameter mixing augers. As penetration occurs, bentonite, cement, lime, or other stabilizing slurry is injected into the soil through the HSAs. The auger flights penetrate and break loose the soil and lift it to the mixing paddles, which blend the soil and slurry. As the augers continue to advance, the soil and slurry may be remixed by additional paddles. DSM has been used to treat soil as much as 90 feet deep, and has been used for structural cut-off walls, block treatment for foundations, and low strength piles and to mitigate liquefaction susceptible soil.

3.2.1 Deep Soil Mixing (DSM) Strength

The engineering properties of the soil cement (the product of DSM) vary with the soil, type of slurry, injection rate, auger advance rate, and several other factors. It is common to specify the soil cement properties and allow the contractor to select appropriate equipment and design the mix. Typically, soil cement strength increases with curing time and is inversely proportional to mixing water content.

Because of the alternating layers of sand, gravel, and clay in the improvement zone, we assumed a uniform minimum design unconfined compressive strength of 50 pounds per square inch (psi) will be achieved. This strength is consistent with the assumptions for improved ground strength used in our stability analyses. It is typical construction practice to specify the minimum soil strengths that must be obtained from the DSM. Therefore, we recommend:

- The average design unconfined compressive strength of the DSM soil cement be 100 psi at 28 days.
- The minimum allowable measured unconfined compressive strength of the DSM soil cement be 50 psi at 28 days.
- Perform testing on production DSM columns to measure strength as follows:
  - Core 2.5 percent of the DSM columns (approximately four columns) in a continuous core through the bottom of the DSM column. Cores should be retrieved no more than seven days following installation of the column. The holes generated from the coring operations should be immediately backfilled using cement slurry specifically designed to achieve a minimum of 100 psi unconfined compressive strength at 28 days.
  - Protect the retrieved core samples in plastic sheeting to maintain in situ moisture content until the strength tests are performed. Store cores in a cool, moist room typically specified for storage of concrete cores.
  - Perform unconfined compressive strength testing, in accordance with ASTM International (ASTM) D 2166, Standard Test Specification for the Unconfined

Compressive Strength of Cohesive Soil, on samples retrieved from cores of the DSM columns at intervals of 7, 14, and 28 days to observe unconfined compressive strength and the increase in strength with time.

– For each day of DSM installation, retrieve wet samples and cast into molds for strength testing immediately upon completion of a column. Screen off particles larger than 10 percent of the mold opening size. Perform unconfined compressive strength testing, in accordance with ASTM D 2166, Standard Test Specification for the Unconfined Compressive Strength of Cohesive Soil for pairs of samples at ages of 7, 14, and 28, days.
– Use clean, fresh water for mixing the soil cement. The water must be free from salt contamination; have a pH of no less than 6.0 or more than 8.0; have a total dissolved solids not greater than 500 parts per million (ppm); have oils, acids, organics, alkali, or other deleterious materials not greater than 50 ppm; and have a hardness not greater than 50 ppm.

Shannon & Wilson should observe the installation of the DSM columns during construction, observe the coring, select core intervals for unconfined compressive strength testing, and evaluate test results.

3.2.2 Deep Soil Mixing (DSM) Configuration

Our recommendations for ground improvement to mitigate potential liquefaction are based on our stability analyses that are presented in GD-5 and summarized in Section 2.3.2 of this memorandum. We recommend constructing a DSM ground improvement zone that is 38 feet wide measured along centerline extending from about 5 feet west of the pile-supported east abutment (Abutment 2) to about 1 foot east of the pedestrian tunnel east wall footing (Abutment 3). The DSM ground improvement should extend for the full length of the abutment. We recommend the DSM area replacement ratio be a minimum 33 percent. Based on understanding of readily available DSM installation equipment, we assumed that the column diameter that would be achieved in the field would be 4 feet. Plan sheets WS-108 and WS-109, included with this memorandum, present our recommended typical DSM column pattern and layout.

Our recommendation for a typical DSM column layout consists of secant DSM columns that form panels approximately 12 feet on center with a row of continuous tangent DSM columns immediately west of Abutment 2, and isolated DSM columns between the panels beneath the east edge of Abutment 3. Arranging the DSM in panels oriented perpendicular to the abutment creates a “shear” element in the direction of potential embankment movement toward the bridge abutments and Jordan River, thereby improving the embankment stability. The stability analyses include the overall composite strength of the DSM columns and surrounding soil in a liquefied condition. The analyses do not consider resistance that could be provided by the shear panel arrangement, its stiffening effect or associated potential to reduce pore pressure increases in the soils during an earthquake. Consequently, the results of the stability analyses should be considered conservative for the DSM panel arrangements shown.

The row of tangent DSM columns in front of the Abutment 2 piles is intended to improve the lateral resistance provided by the soil to the pile foundations. The isolated DSM columns between panels beneath the east edge of Abutment 3 are intended to provide a more uniform composite DSM/soil bearing pressure beneath the east side of the abutment spread footing foundation.

In general, columns that will be constructed in panels should have a minimum intersection of 6 inches to form a secant panel, increasing the shear resistance of the DSM column panels. We recommend similar DSM column layouts beneath the north and south wing walls to improve the global stability of these walls, improve post-seismic stability, and provide a DSM/soil subgrade condition similar to Abutment 3.

The DSM columns should penetrate through the loose liquefiable soil. We recommend they extend about 15 feet below the existing ground surface to an elevation no higher than 4,478 feet so that the columns penetrate through the potentially liquefiable soils and extend into the underlying non-liquefiable soils.

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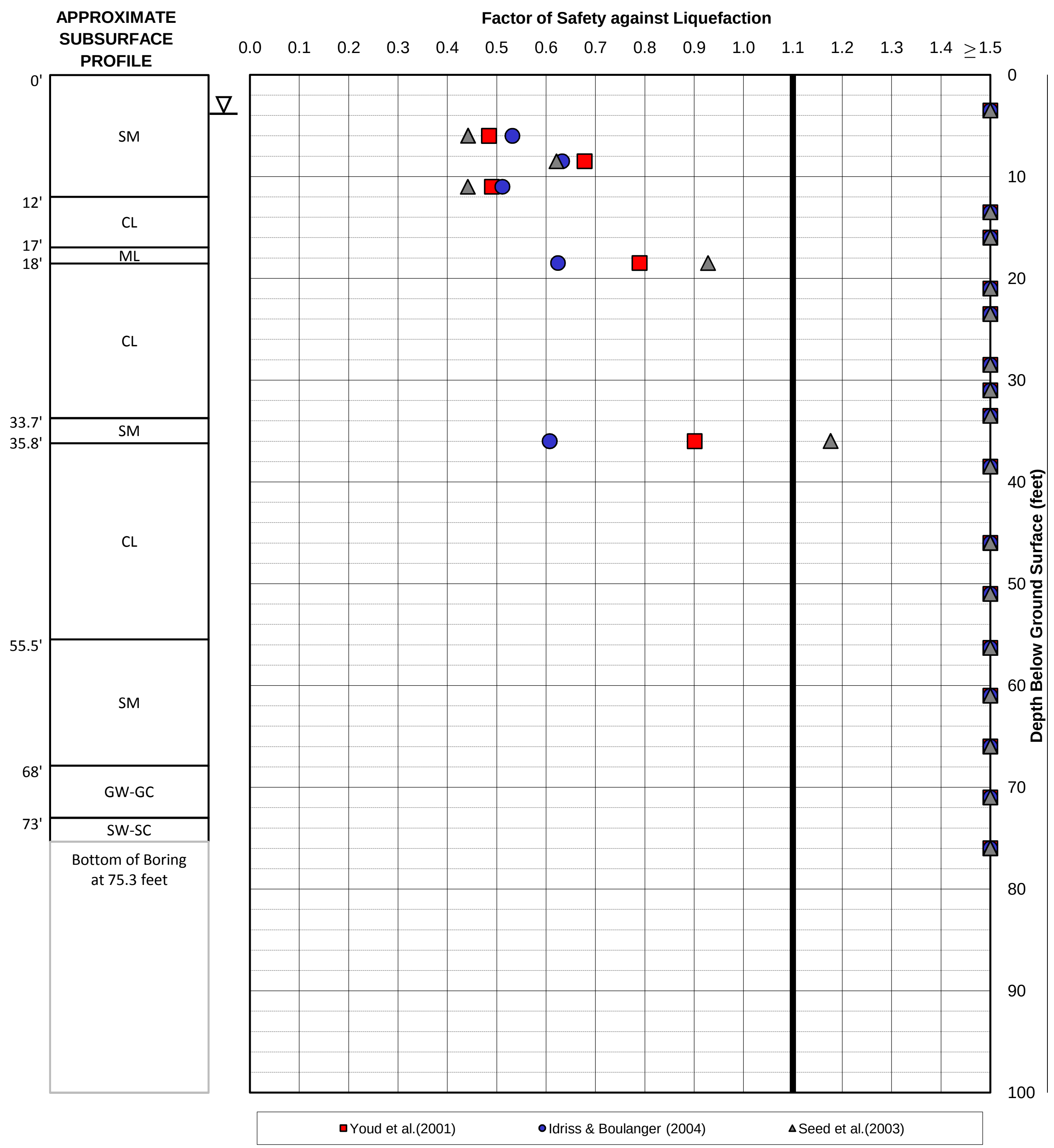

NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

Pioneer Crossing, Lehi
and I-15 American Fork Interchange
Lehi, Utah

LIQUEFACTION ANALYSES (EE)
BORING SWB-202
M = 7.0, PGA = 0.27

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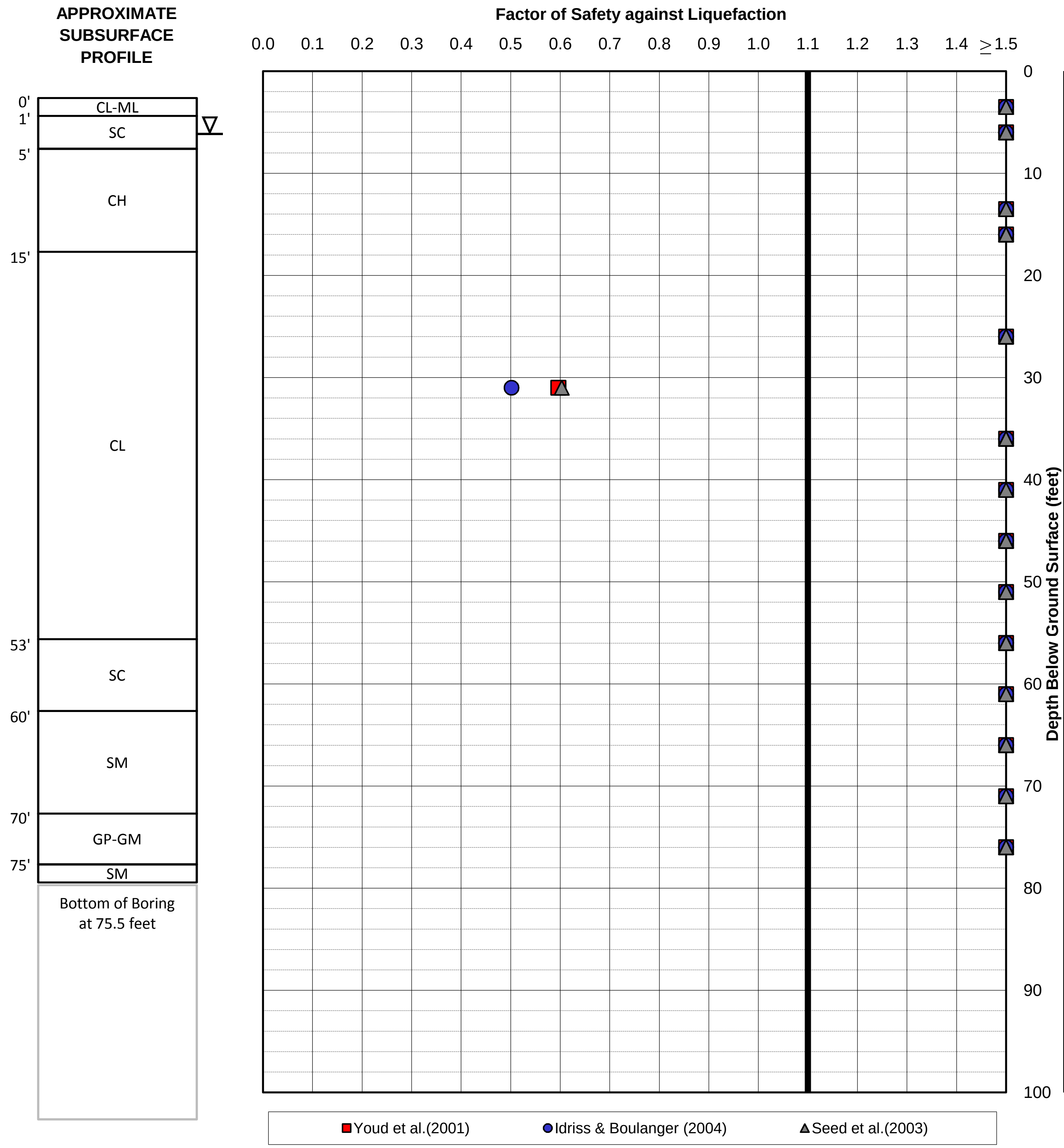
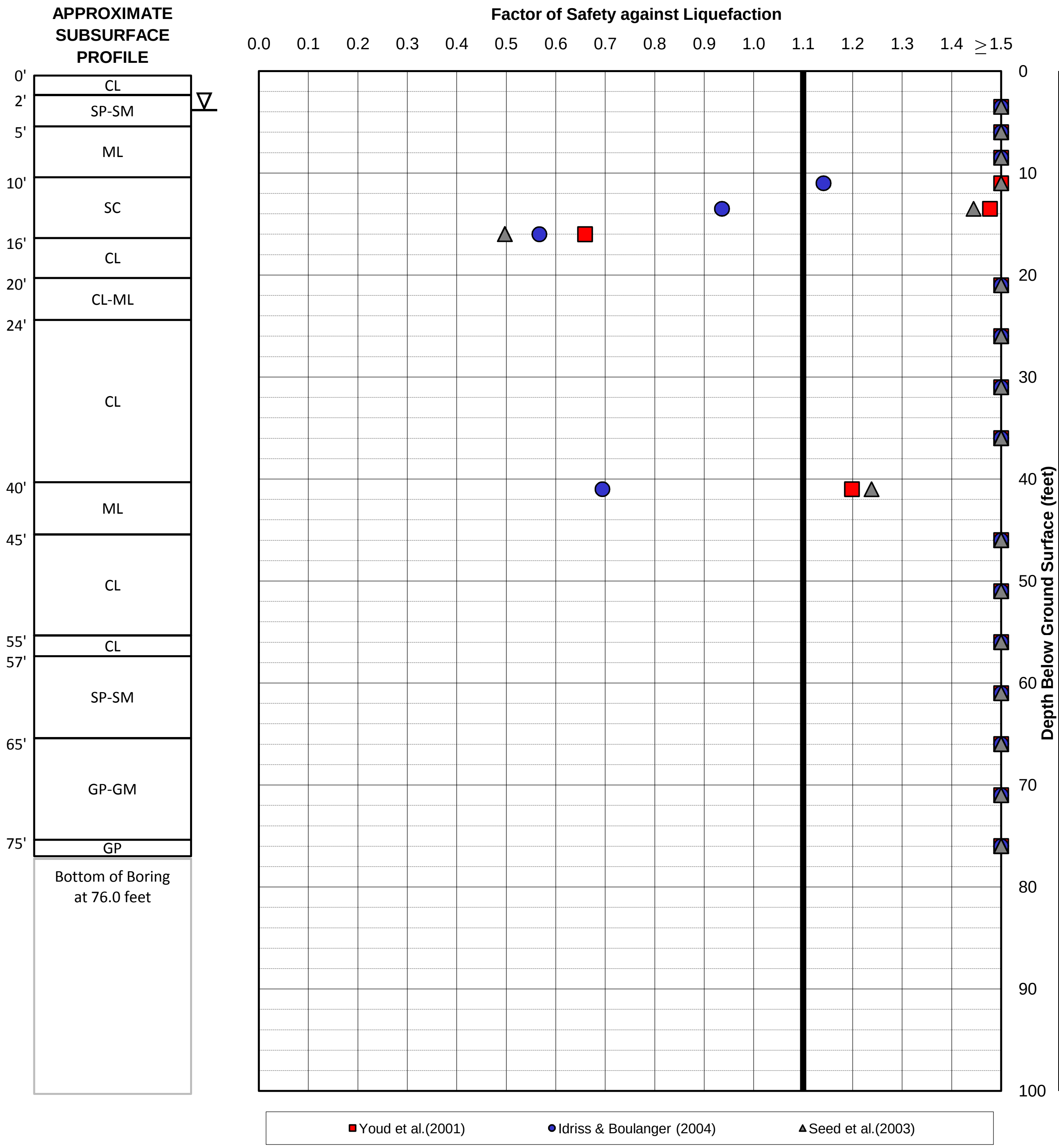
FIG. GD4-1



NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

Pioneer Crossing, Lehi and I-15 American Fork Interchange Lehi, Utah
LIQUEFACTION ANALYSES (EE) **BORING B-03** **M = 7.0, PGA = 0.27** March 2009 23-1-01178-010
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants — **FIG. GD4-2**



NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

Pioneer Crossing, Lehi
and I-15 American Fork Interchange
Lehi, Utah

LIQUEFACTION ANALYSES (EE)
BORING B-08
M = 7.0, PGA = 0.27

March 2009 23-1-01178-010

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FIG. GD4-3

RESULTS OF LIQUEFACTION ANALYSES (EE)

USING CPT DATA

Date: 3/18/2009
Time: 8:41 AM

Filename: LIQcpt_2007_SWC-211 EE.xlsm

CPT Hole Designation: **SWC-211** Design Earthquake Magnitude: 7 Project Name: **PIONEER CROSSING**
Groundwater Table Depth: 6.0 feet Peak Ground Acceleration: 0.27 g Job Number: **23-1-01178-010**

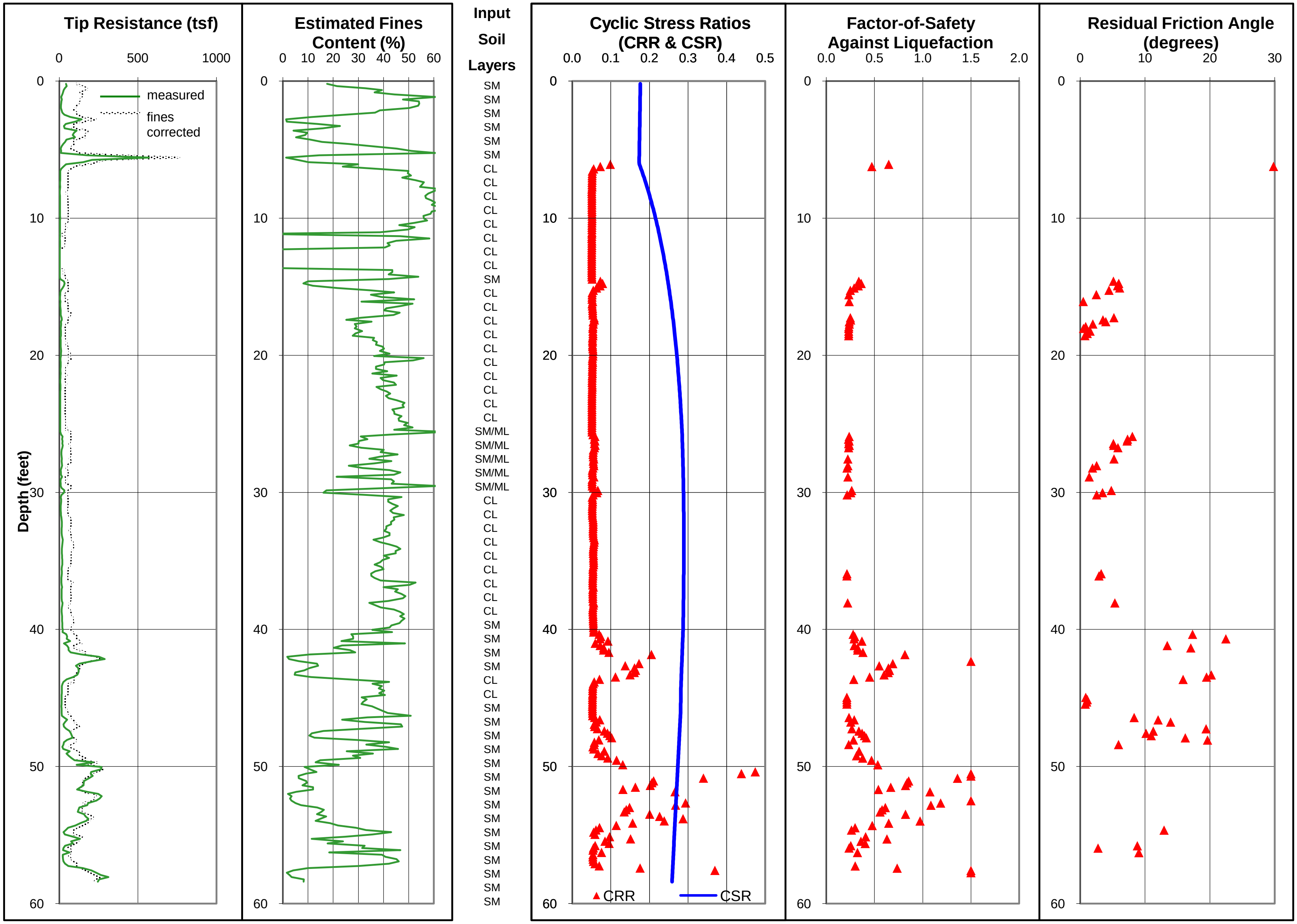


FIG. GD4-4

RESULTS OF LIQUEFACTION ANALYSES (EE)

USING CPT DATA

Date: 3/18/2009
Time: 8:45 AM

Filename: LIQcpt_2007_SWC-220 EE.xlsm

CPT Hole Designation: **SWC-220**
Groundwater Table Depth: 5.0 feet

Design Earthquake Magnitude: 7
Peak Ground Acceleration: 0.27 g

Project Name: **PIONEER CROSSING**
Job Number: **23-1-01178-010**

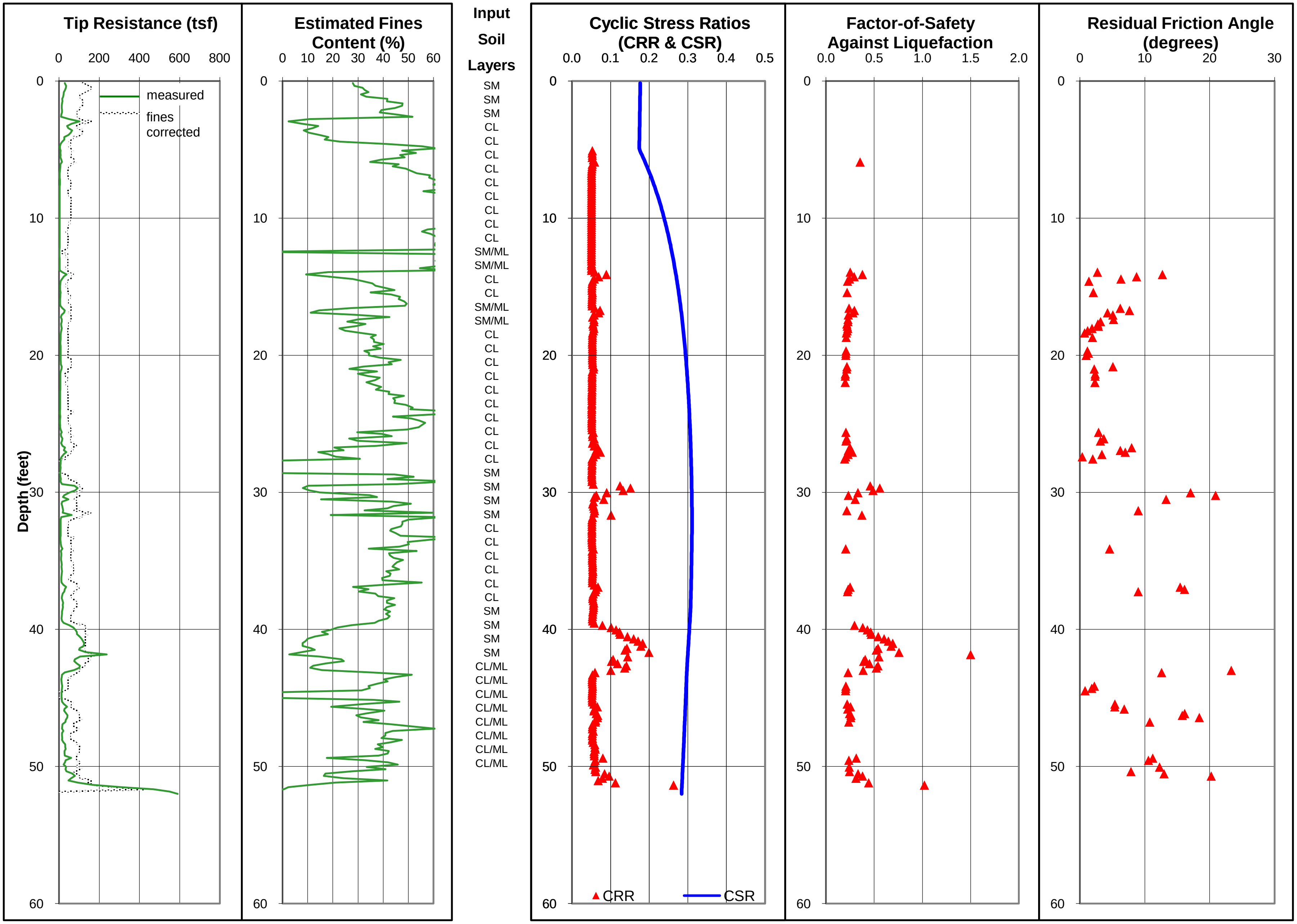
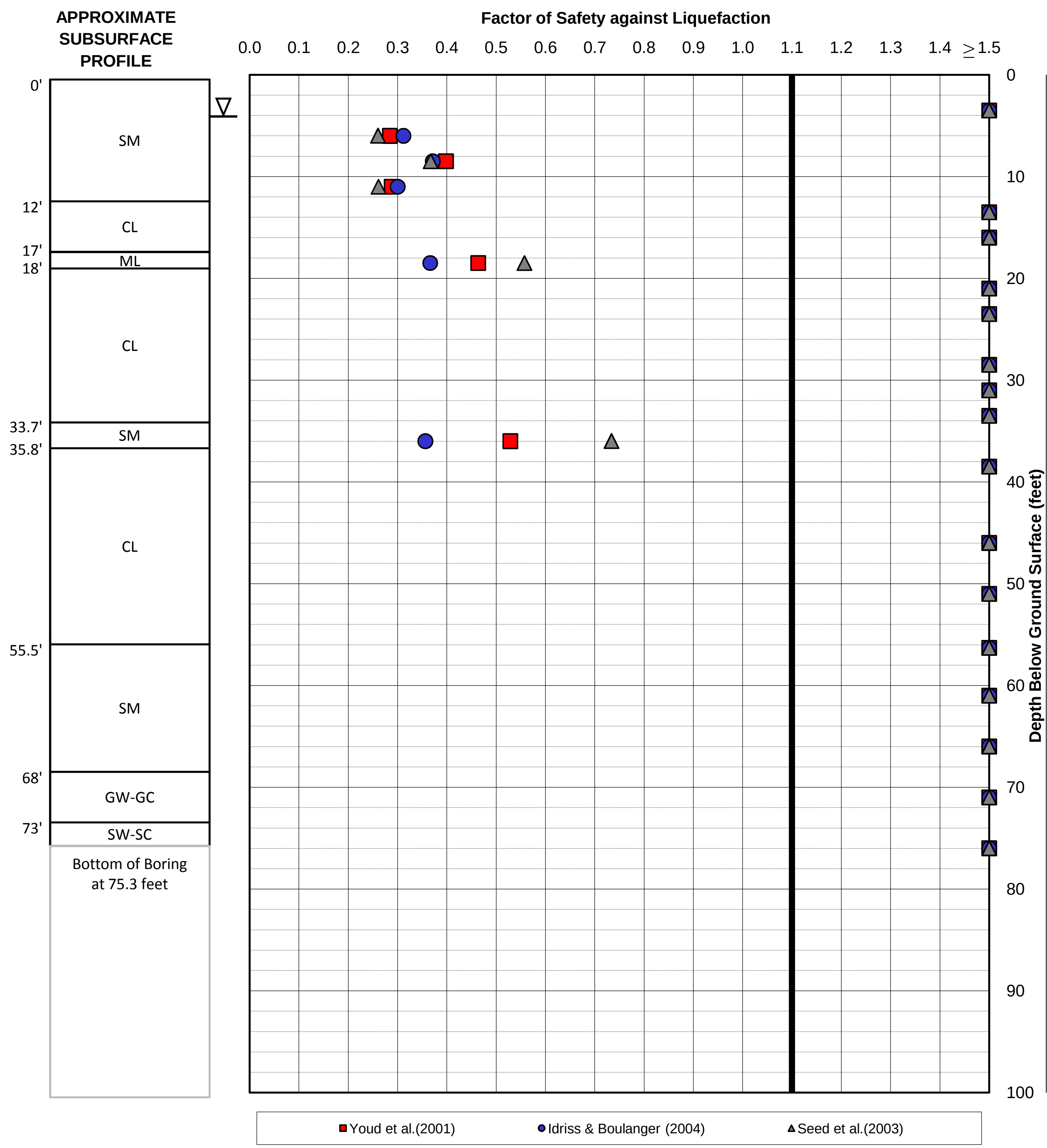


FIG. GD4-5



NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

Pioneer Crossing, Lehi
and I-15 American Fork Interchange
Lehi, Utah

LIQUEFACTION ANALYSES (MCE)
BORING SWB-202
M = 7.0, PGA = 0.46

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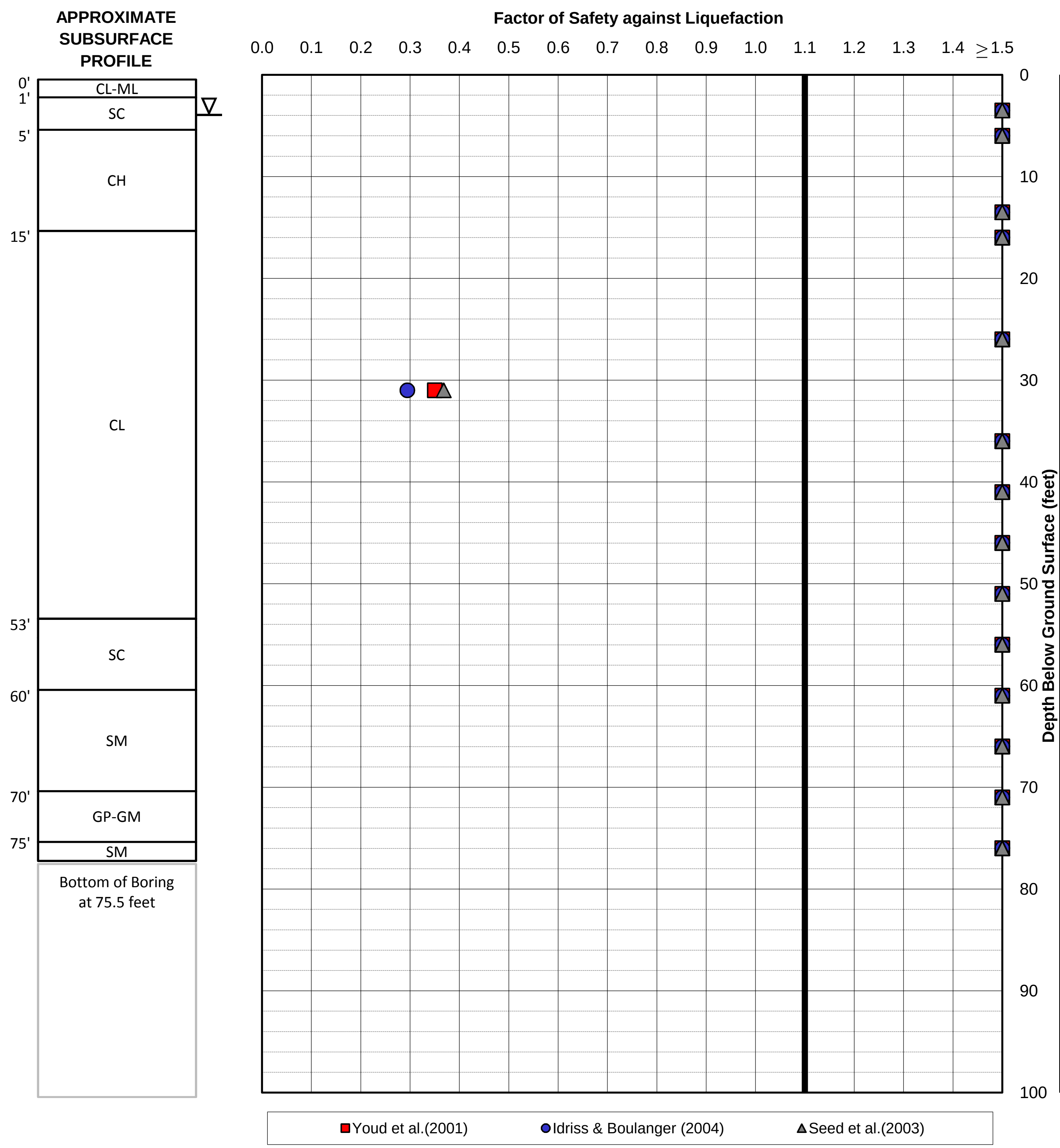
FIG. GD4-6



NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

Pioneer Crossing, Lehi
and I-15 American Fork Interchange
Lehi, Utah

LIQUEFACTION ANALYSES (MCE)
BORING B-03
M = 7.0, PGA = 0.46

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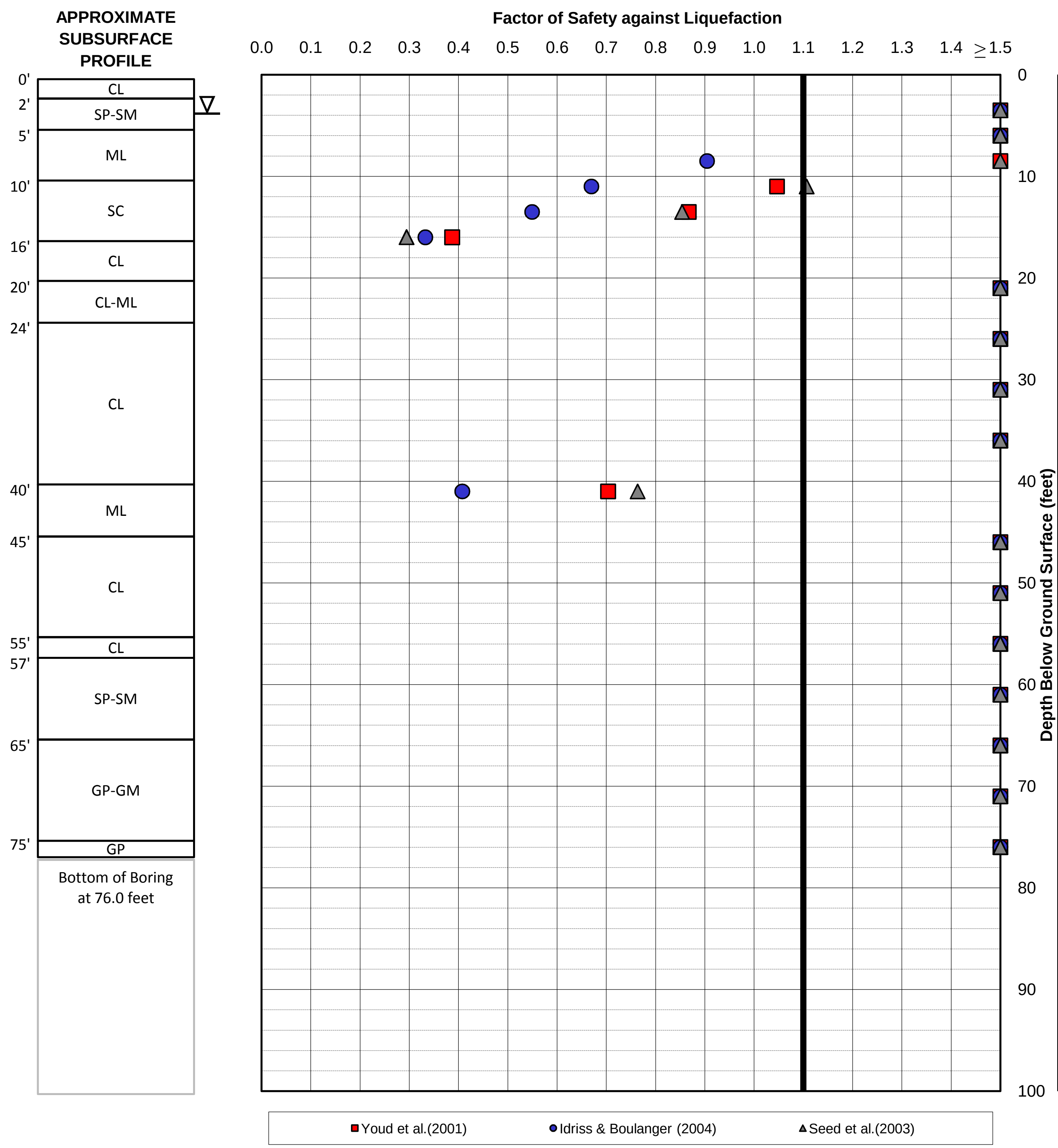
FIG. GD4-7



<u>NOTES</u>

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

Pioneer Crossing, Lehi
and I-15 American Fork Interchange
Lehi, Utah

LIQUEFACTION ANALYSES (MCE)
BORING B-08
M = 7.0, PGA = 0.46

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FIG. GD4-8

RESULTS OF LIQUEFACTION ANALYSES (MCE)

USING CPT DATA

Date: 3/18/2009
Time: 8:42 AM

Filename: LIQcpt_2007_SWC-211 MCE.xlsm

CPT Hole Designation: **SWC-211**
Groundwater Table Depth: 6.0 feet

Design Earthquake Magnitude: 7
Peak Ground Acceleration: 0.46 g

Project Name: **PIONEER CROSSING**
Job Number: **23-1-01178-010**

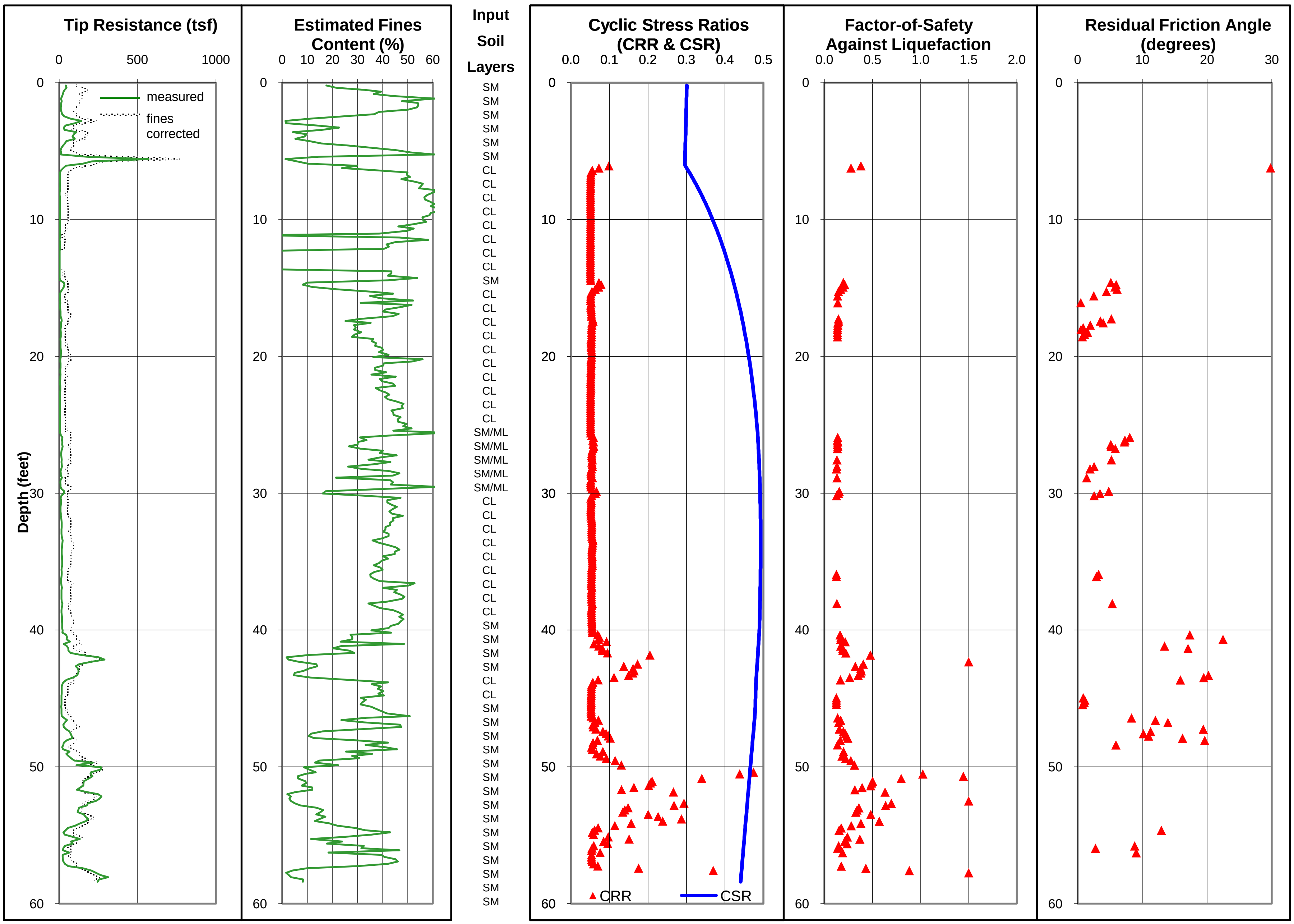


FIG. GD4-9

RESULTS OF LIQUEFACTION ANALYSES (MCE)

USING CPT DATA

Date: 3/18/2009
Time: 8:41 AM

Filename: LIQcpt_2007_SWC-220 MCE.xlsm

CPT Hole Designation: **SWC-220**
Groundwater Table Depth: 5.0 feet

Design Earthquake Magnitude: 7
Peak Ground Acceleration: 0.46 g

Project Name: **PIONEER CROSSING**
Job Number: **23-1-01178-010**

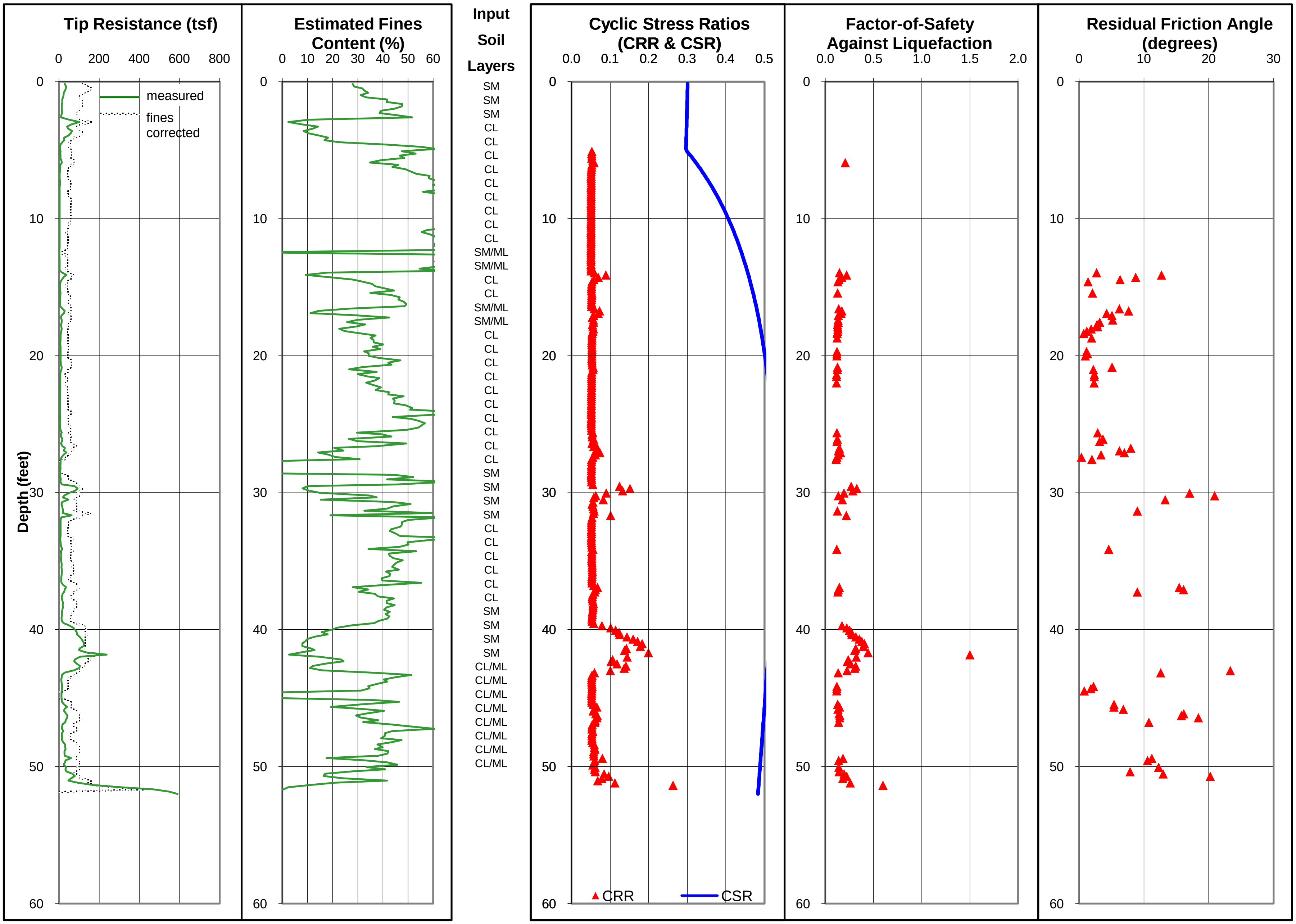
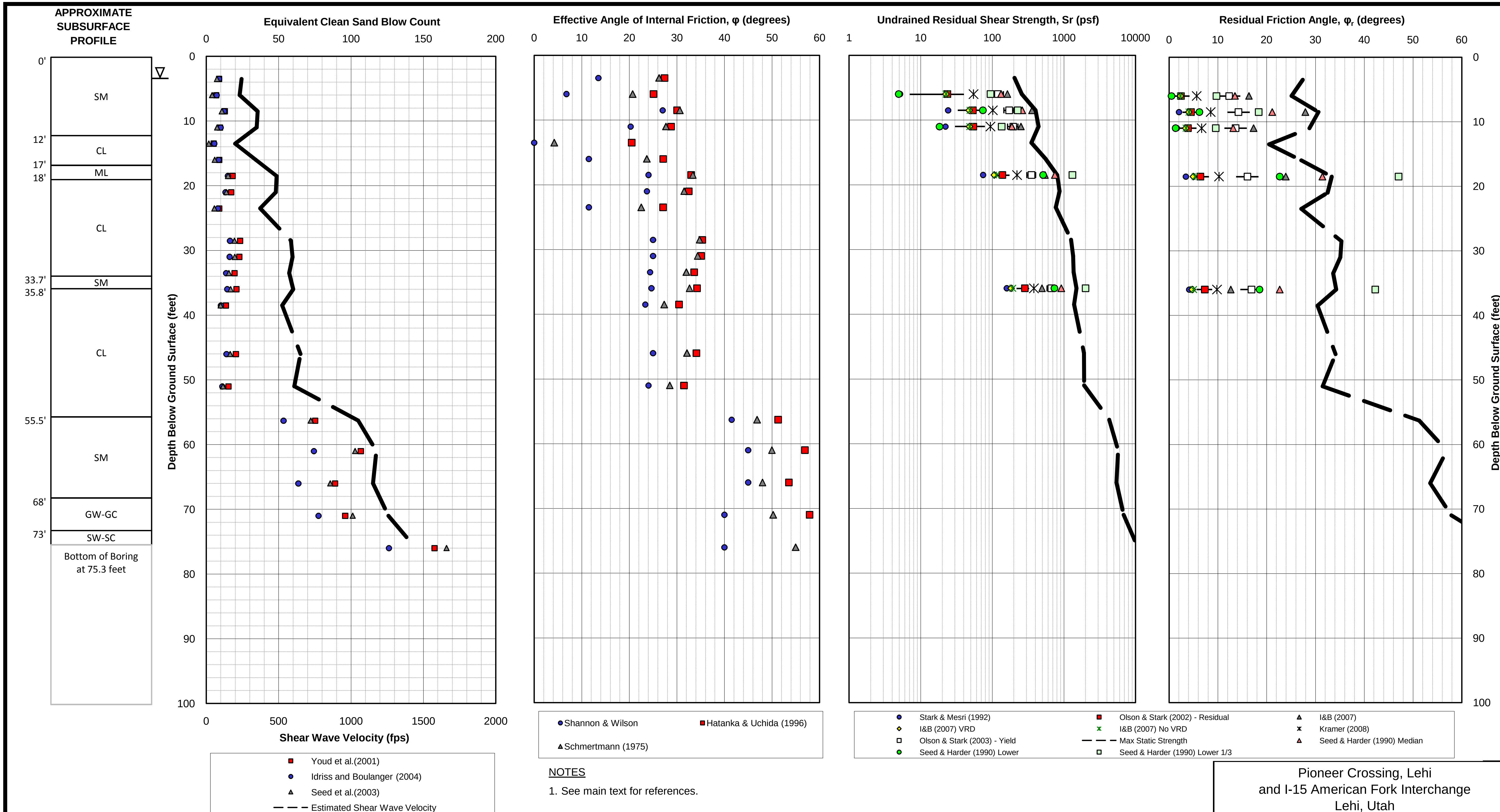


FIG. GD4-10

SPT Liquefaction v2.0.141_SWB202.xlsm Printed: 3/11/2009 5:13 PM

cij



NOTES

1. See main text for references.

Pioneer Crossing, Lehi
and I-15 American Fork Interchange
Lehi, Utah

LIQUEFIED SOIL PROPERTIES (MCE)
BORING SWB-202
M = 7.0, PGA = 0.46 G

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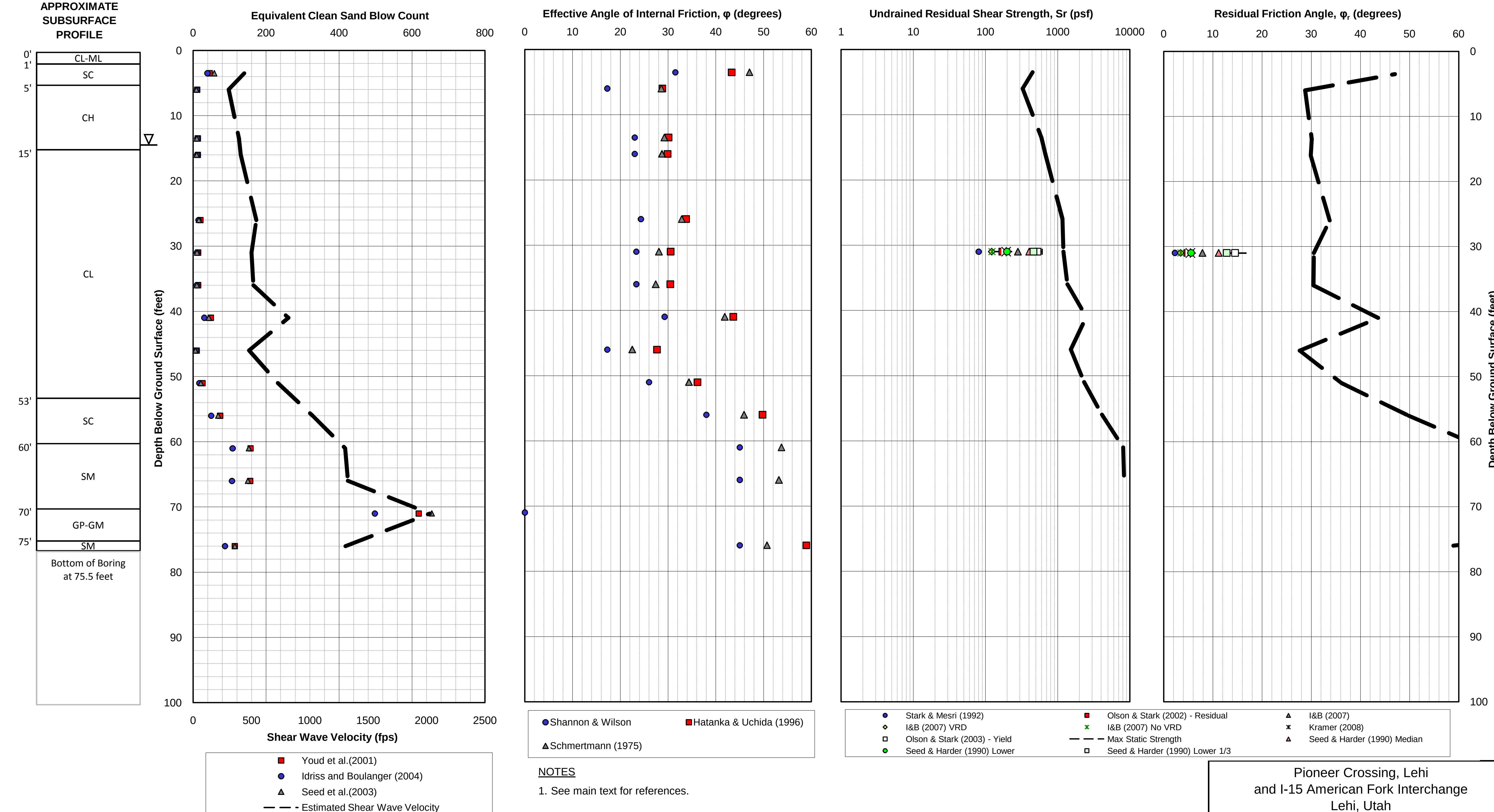
FIG. GD4-11

cij



NOTES

1. See main text for references.

Pioneer Crossing, Lehi
and I-15 American Fork Interchange
Lehi, Utah

LIQUEFIED SOIL PROPERTIES (MCE)
BORING B-03
M = 7.0, PGA = 0.46 G

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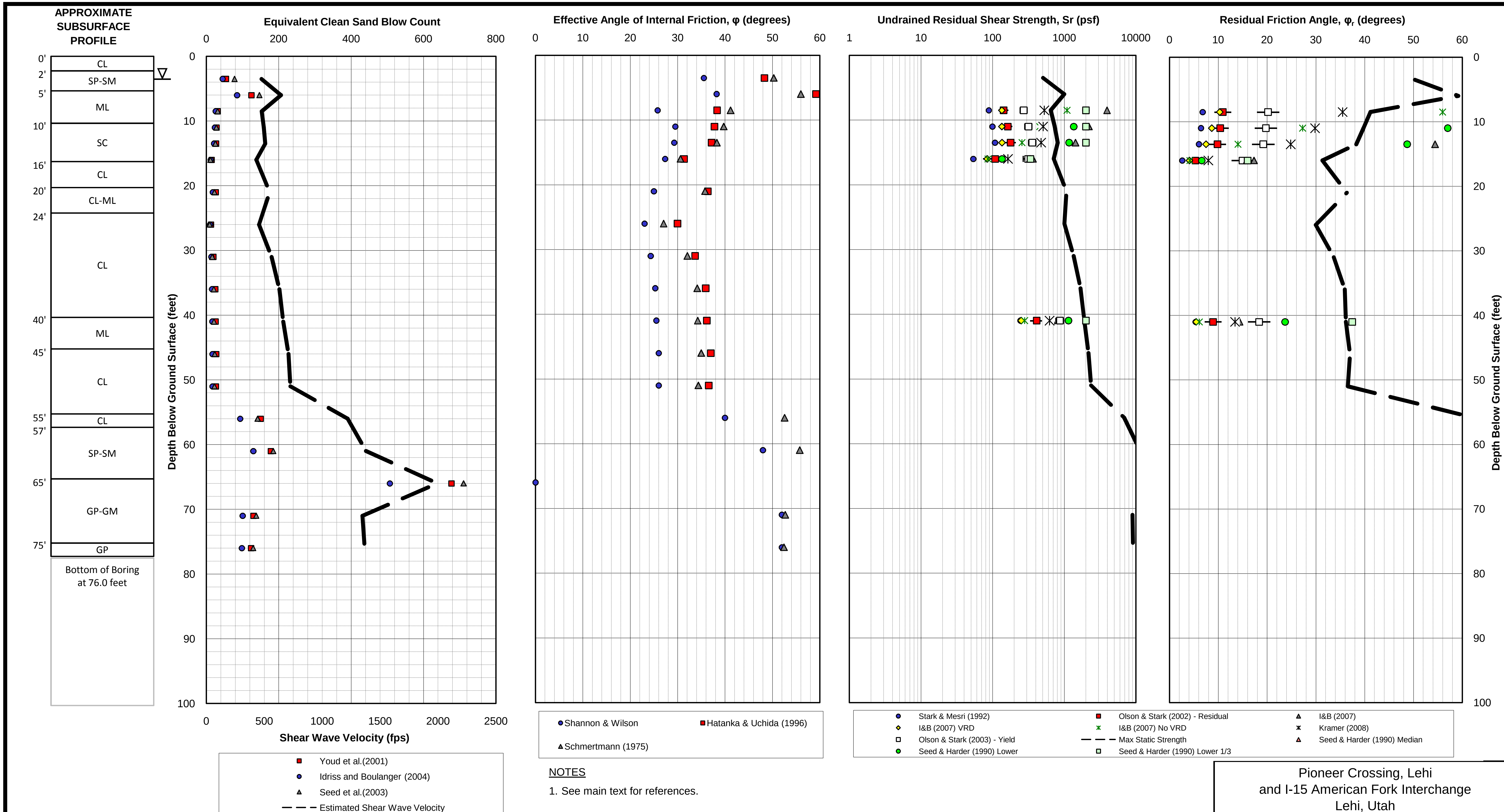
FIG. GD4-12



NOTES

1. See main text for references.

Pioneer Crossing, Lehi
and I-15 American Fork Interchange
Lehi, Utah

LIQUEFIED SOIL PROPERTIES (MCE)
BORING B-08
M = 7.0, PGA = 0.46 G

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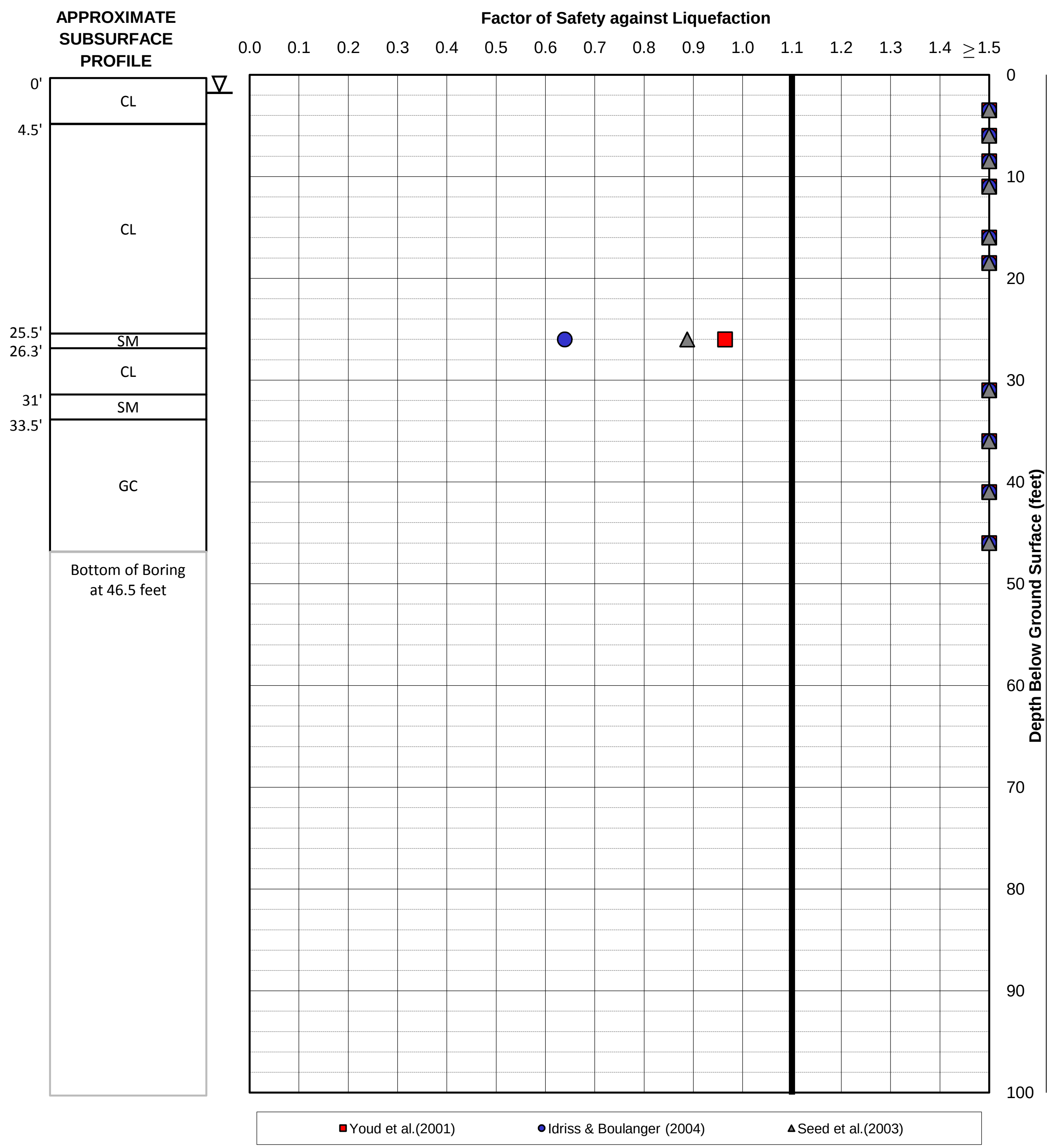
FIG. GD4-13



NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

Pioneer Crossing, Lehi
and I-15 American Fork Interchange
Lehi, Utah

LIQUEFACTION ANALYSES (EE)
BORING SWB-203
M = 7.0, PGA = 0.27

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FIG. GD4-14

RESULTS OF LIQUEFACTION ANALYSES (EE)

USING CPT DATA

Date: 3/18/2009
Time: 8:42 AM

Filename: LIQcpt_2007_SWC-212 EE.xlsm

CPT Hole Designation: **SWC-212**
Groundwater Table Depth: 2.0 feet

Design Earthquake Magnitude: 7
Peak Ground Acceleration: 0.27 g

Project Name: **Pioneer Crossing**
Job Number: **23-1-01178-010**

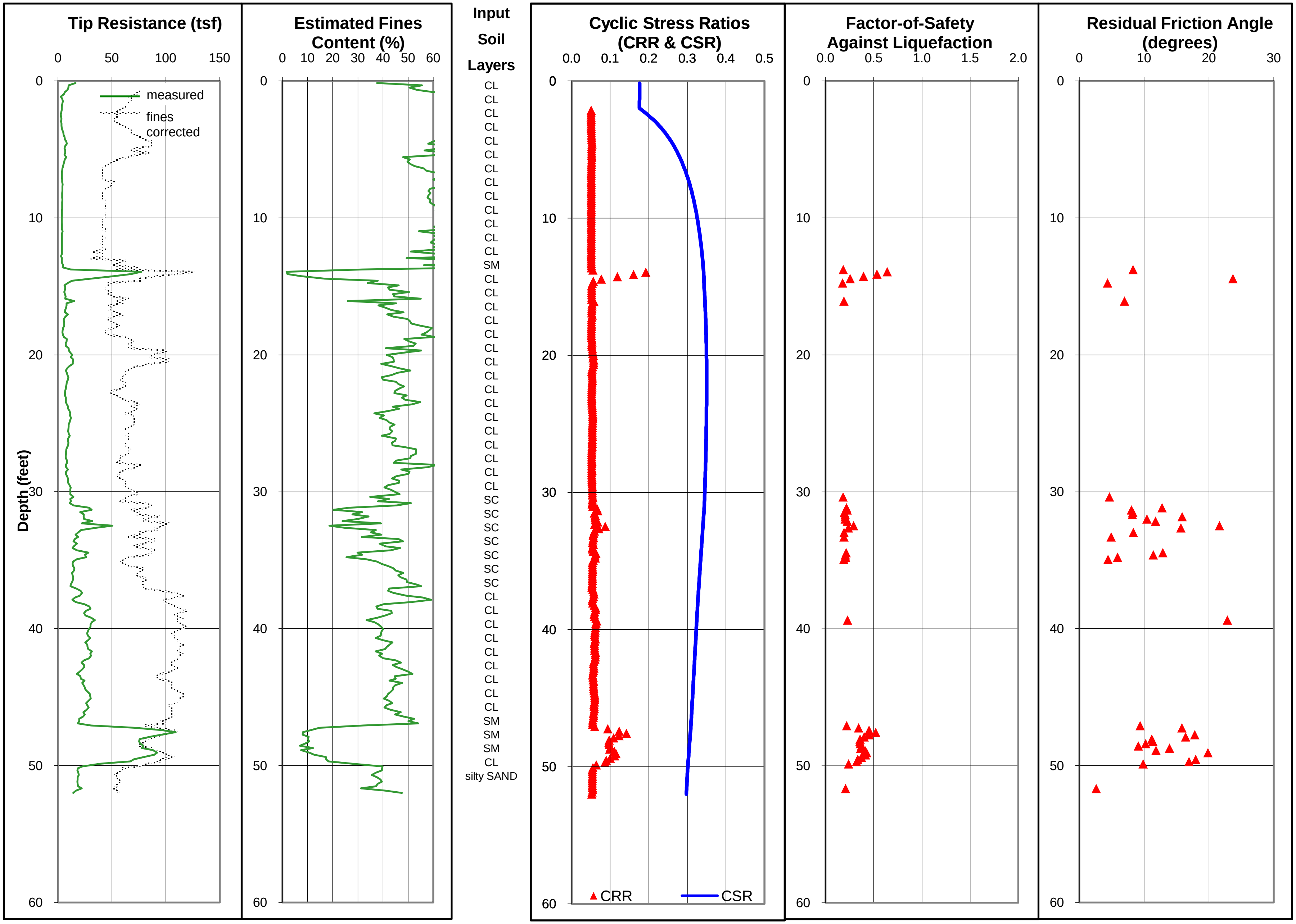


FIG. GD4-15

RESULTS OF LIQUEFACTION ANALYSES (EE)

USING CPT DATA

Date: 3/18/2009
Time: 8:43 AM

Filename: LIQcpt_2007_SWC-213 EE.xlsm

CPT Hole Designation: **SWC-213**
Groundwater Table Depth: 2.0 feet

Design Earthquake Magnitude: 7
Peak Ground Acceleration: 0.27 g

Project Name: **Pioneer Crossing**
Job Number: **23-1-01178-010**

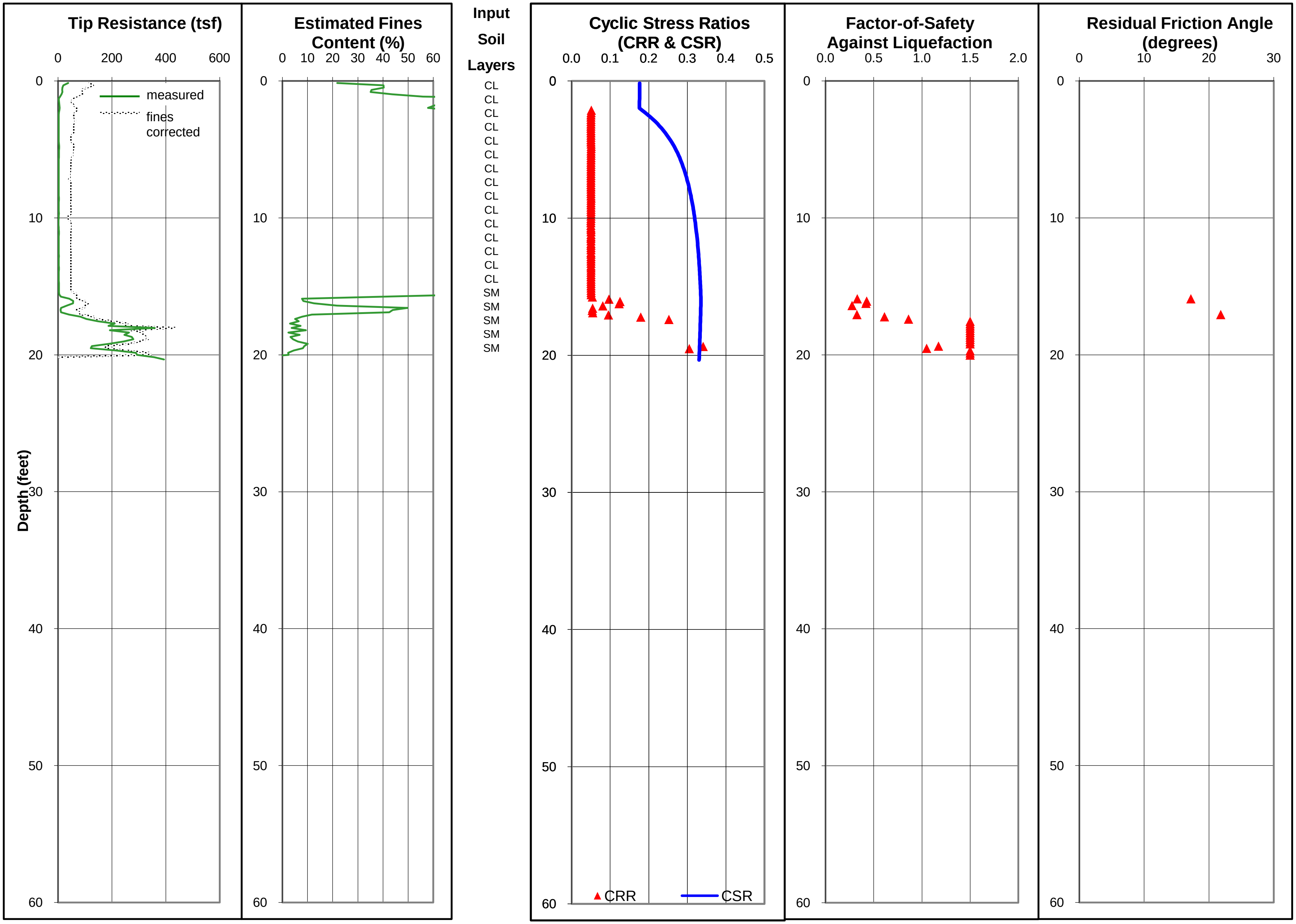
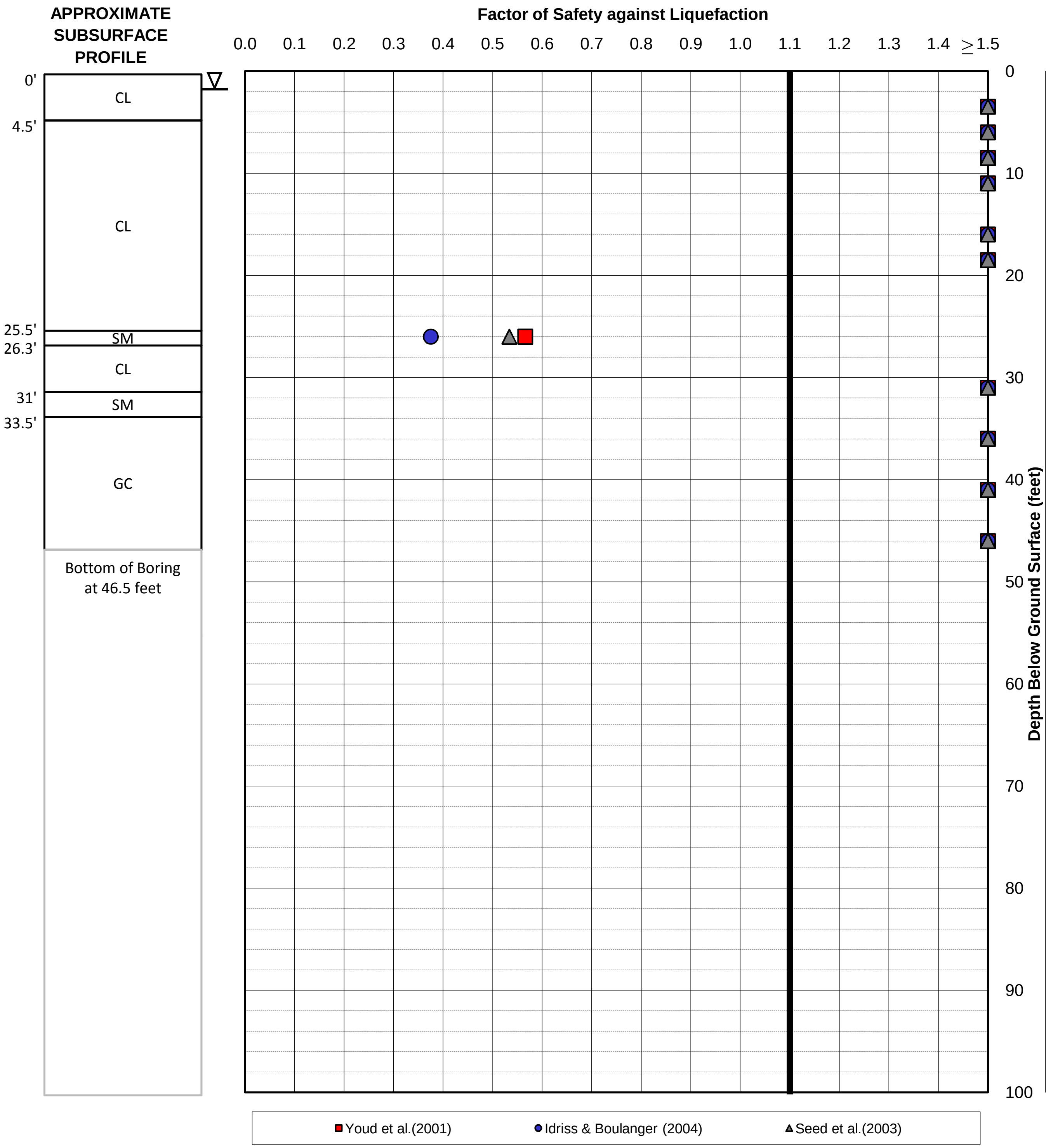


FIG. GD4-16



NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

Pioneer Crossing, Lehi and I-15 American Fork Interchange Lehi, Utah
LIQUEFACTION ANALYSES (MCE) **BORING SWB-203** **M = 7.0, PGA = 0.46**
March 2009 — 23-1-01178-010
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants — **FIG. GD4-17**

RESULTS OF LIQUEFACTION ANALYSES (MCE)

USING CPT DATA

Date: 3/18/2009
Time: 8:43 AM

Filename: LIQcpt_2007_SWC-212 MCE.xlsm

CPT Hole Designation: **SWC-212**
Groundwater Table Depth: 2.0 feet

Design Earthquake Magnitude: 7
Peak Ground Acceleration: 0.46 g

Project Name: **Pioneer Crossing**
Job Number: **23-1-01178-010**

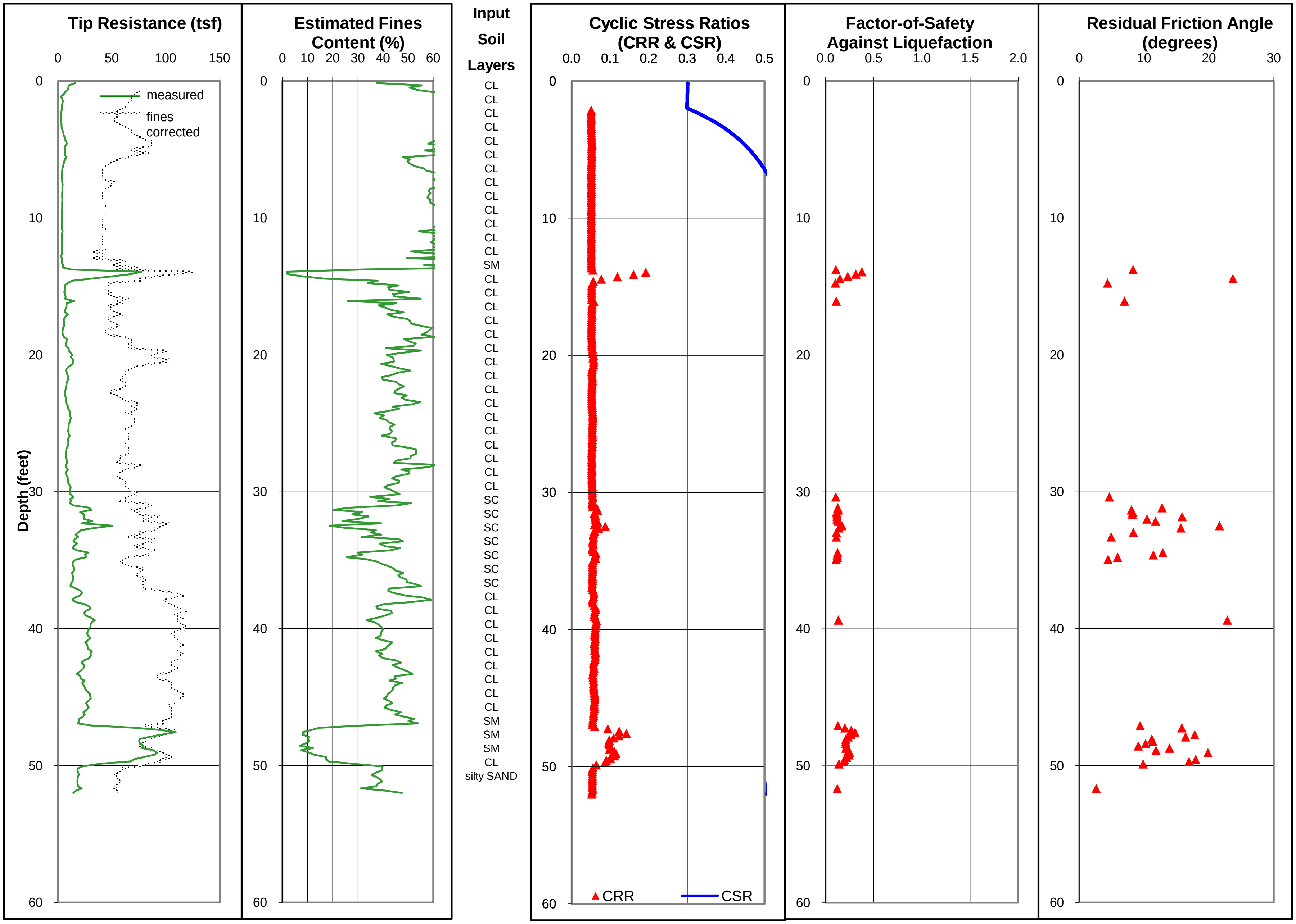


FIG. GD4-18

RESULTS OF LIQUEFACTION ANALYSES (MCE)

USING CPT DATA

Date: 3/18/2009
Time: 8:43 AM

Filename: LIQcpt_2007_SWC-213 MCE.xlsm

CPT Hole Designation: **SWC-213**
Groundwater Table Depth: 2.0 feet

Design Earthquake Magnitude: 7
Peak Ground Acceleration: 0.46 g

Project Name: **Pioneer Crossing**
Job Number: **23-1-01178-010**

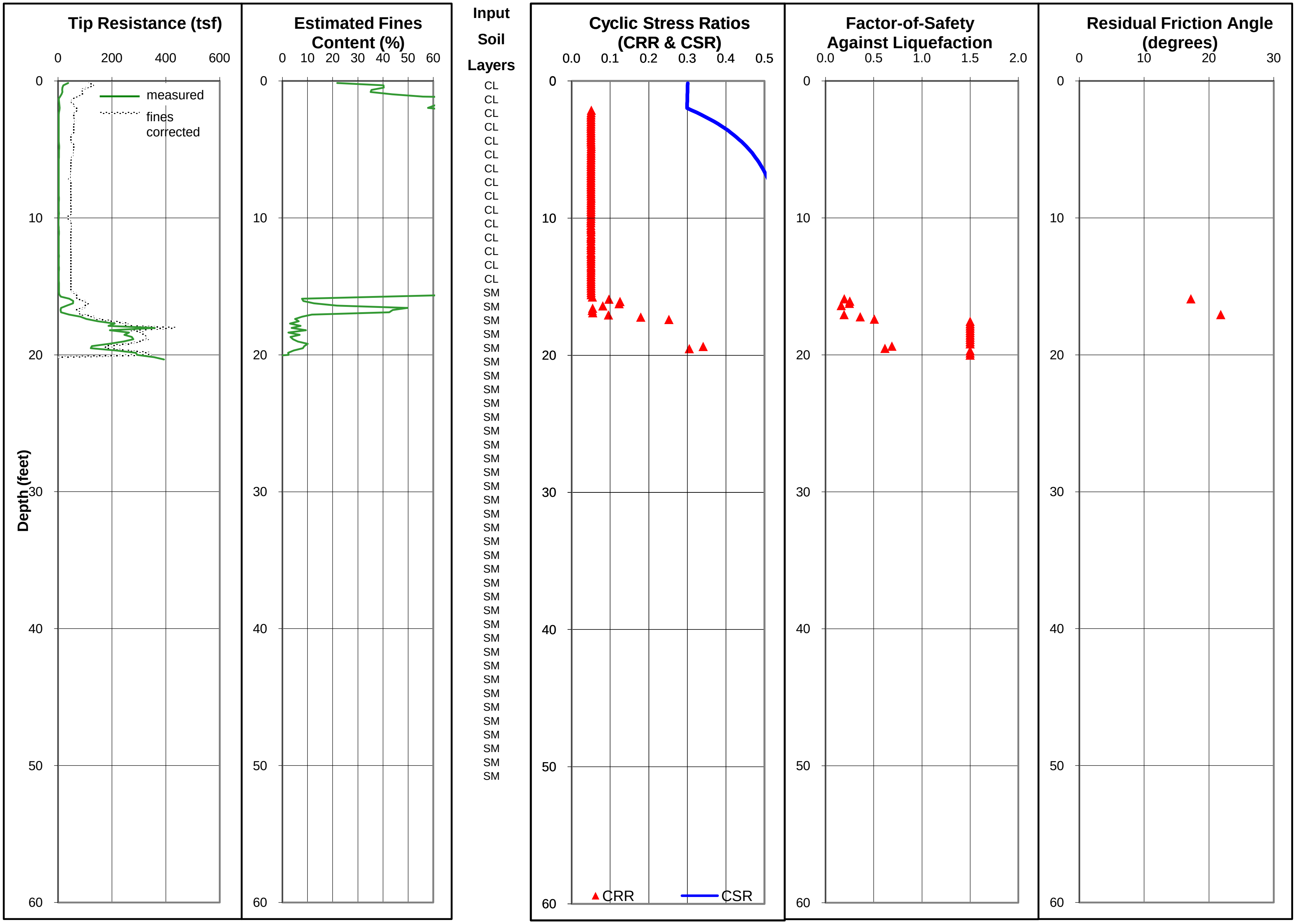
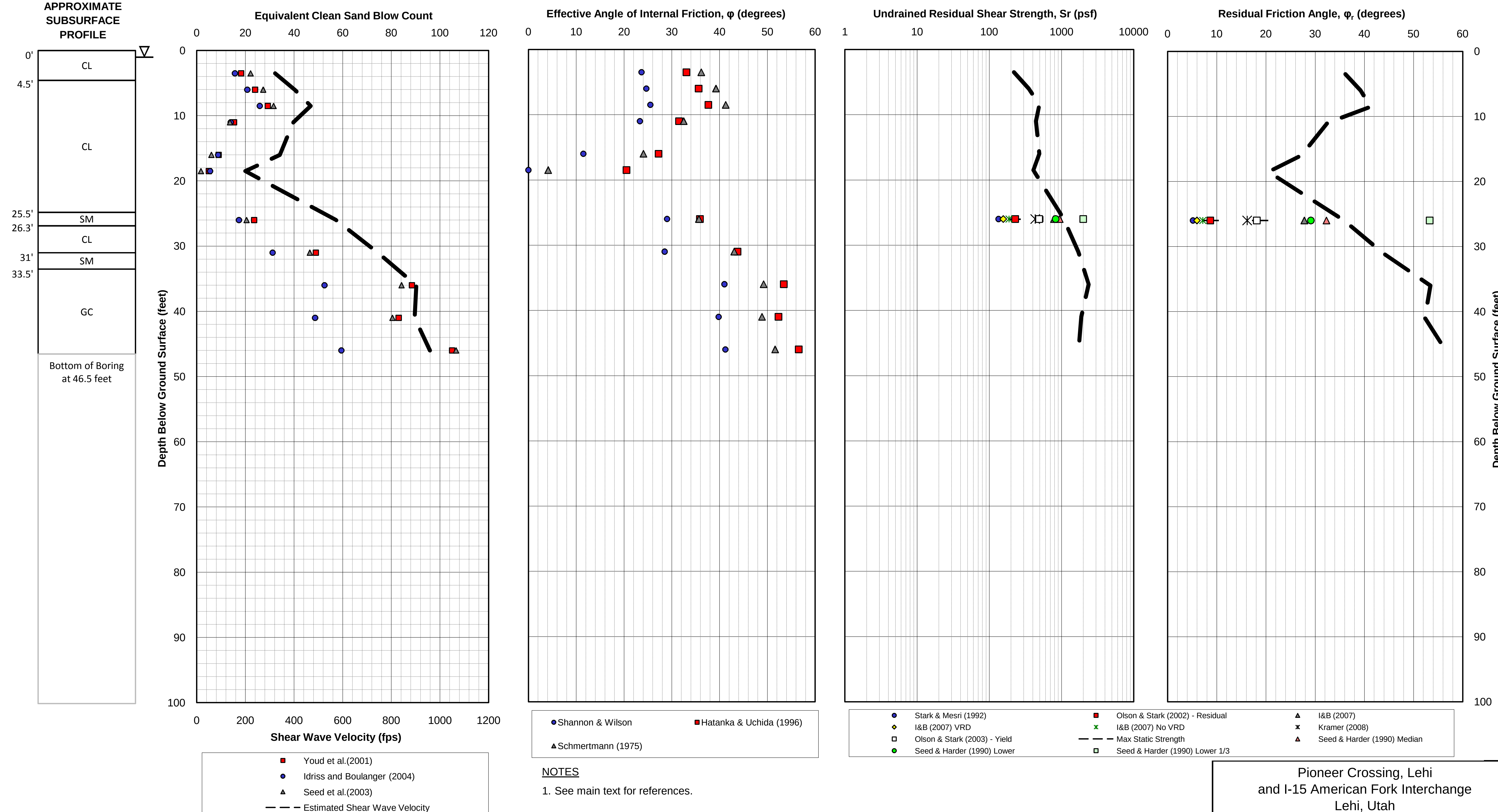


FIG. GD4-19



NOTES

1. See main text for references.

Pioneer Crossing, Lehi
and I-15 American Fork Interchange
Lehi, Utah

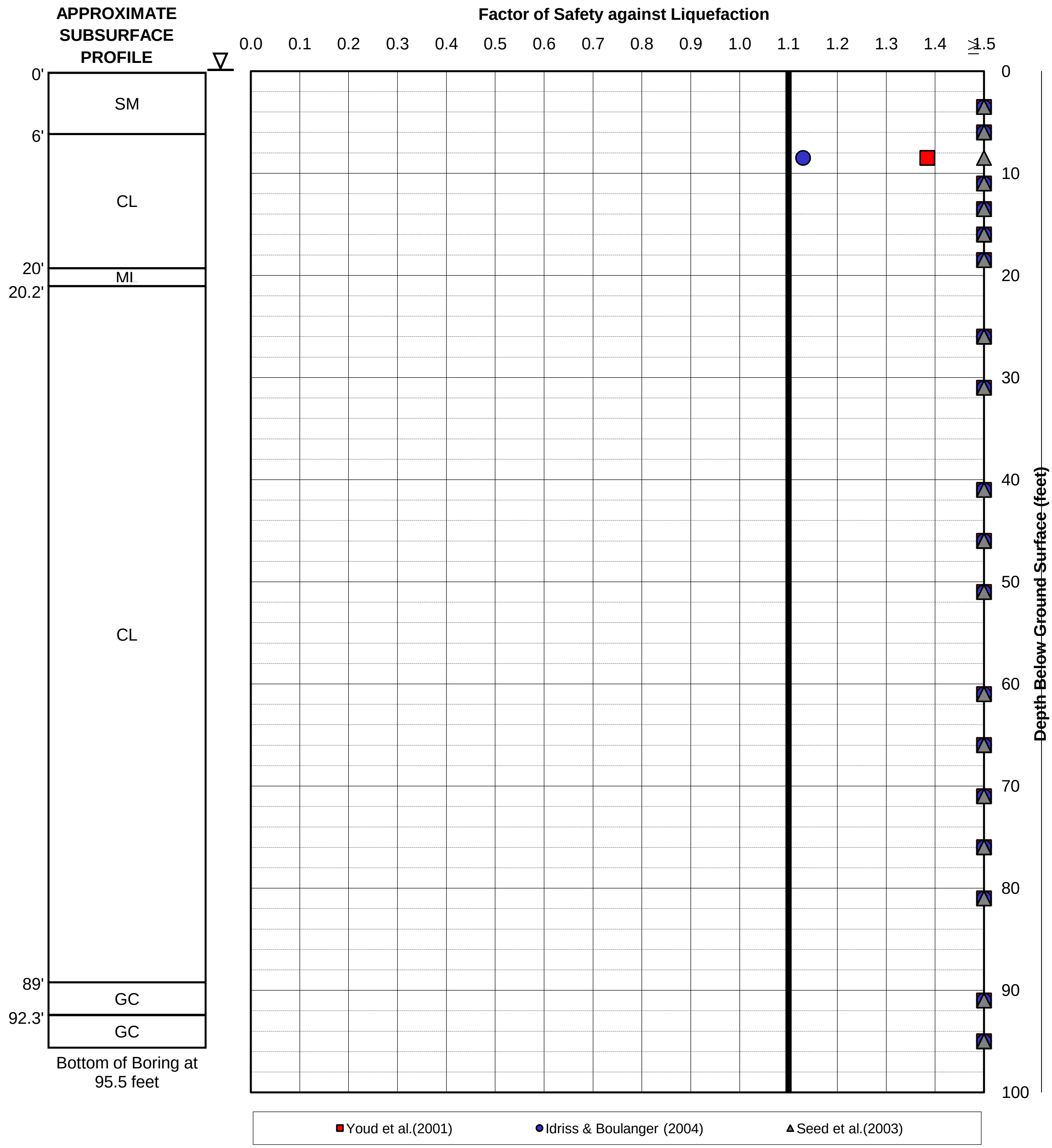
LIQUEFIED SOIL PROPERTIES (EE)
BORING SWB-203
M = 7.0, PGA = 0.27 G

March 2009 | 23-1-01178-010

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. GD4-20

SPT Liquefaction v2.0.141 SWB204.xlsm 3/11/2009



NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

Pioneer Crossing, Lehi
and I-15 American Fork Interchange
Lehi, Utah

LIQUEFACTION ANALYSES (EE)
BORING SWB-204
M = 7.0, PGA = 0.27

March 2009 | 23-1-01178-010

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. GD4-21

RESULTS OF LIQUEFACTION ANALYSES (EE)

USING CPT DATA

Date: 3/18/2009
Time: 8:44 AM

Filename: LIQcpt_2007_SWC-214 EE.xlsm

CPT Hole Designation: **SWC-214**
Groundwater Table Depth: 6.0 feet

Design Earthquake Magnitude: 7
Peak Ground Acceleration: 0.27 g

Project Name: **Pioneer Crossing**
Job Number: **23-1-01178-010**

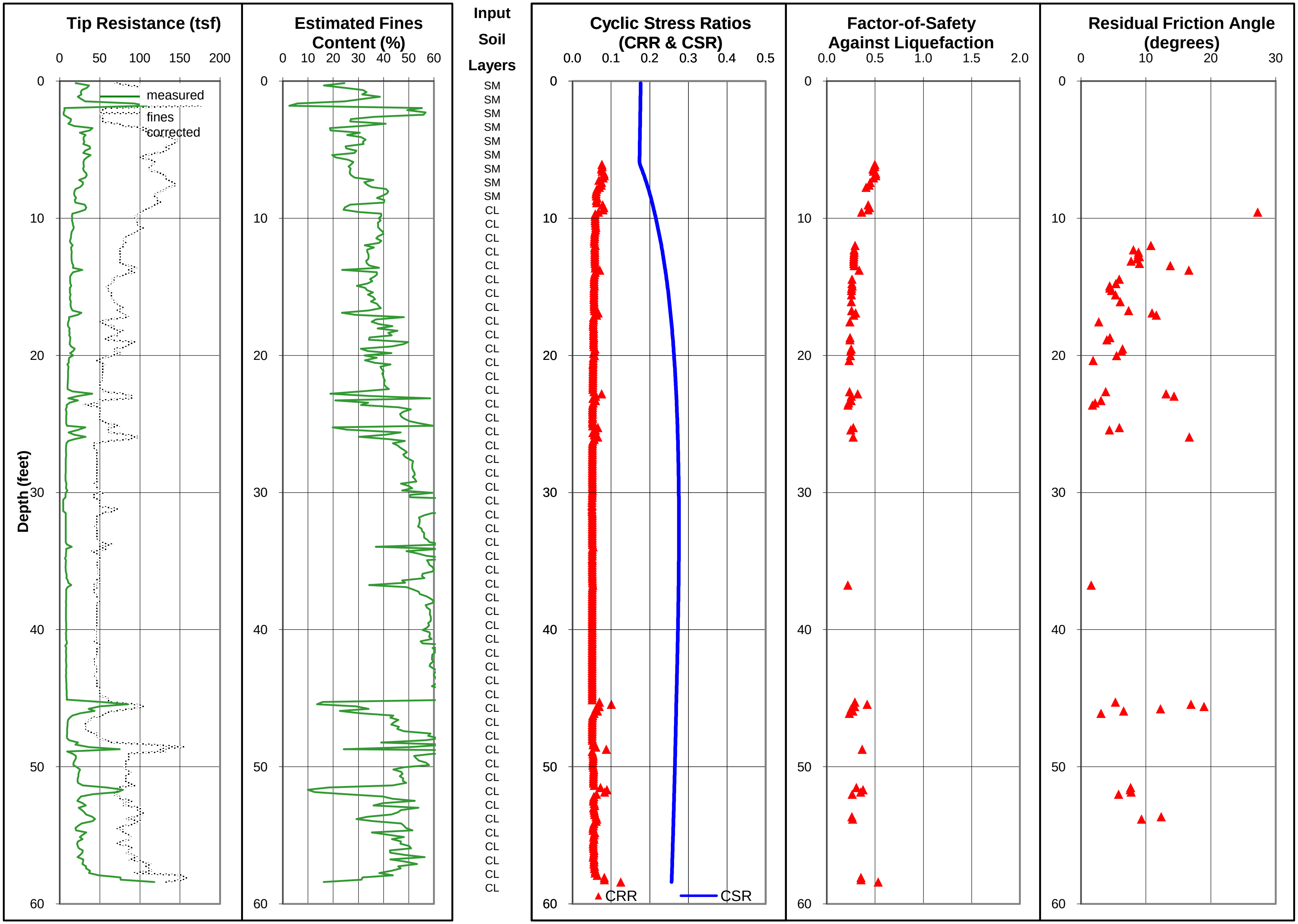


FIG. GD4-22

RESULTS OF LIQUEFACTION ANALYSES (EE)

USING CPT DATA

Date: 3/18/2009
Time: 8:44 AM

Filename: LIQcpt_2007_SWC-215 EE.xlsm

CPT Hole Designation: **SWC-215**
Groundwater Table Depth: 6.0 feet

Design Earthquake Magnitude: 7
Peak Ground Acceleration: 0.27 g

Project Name: **Pioneer Crossing**
Job Number: **23-1-01178-010**

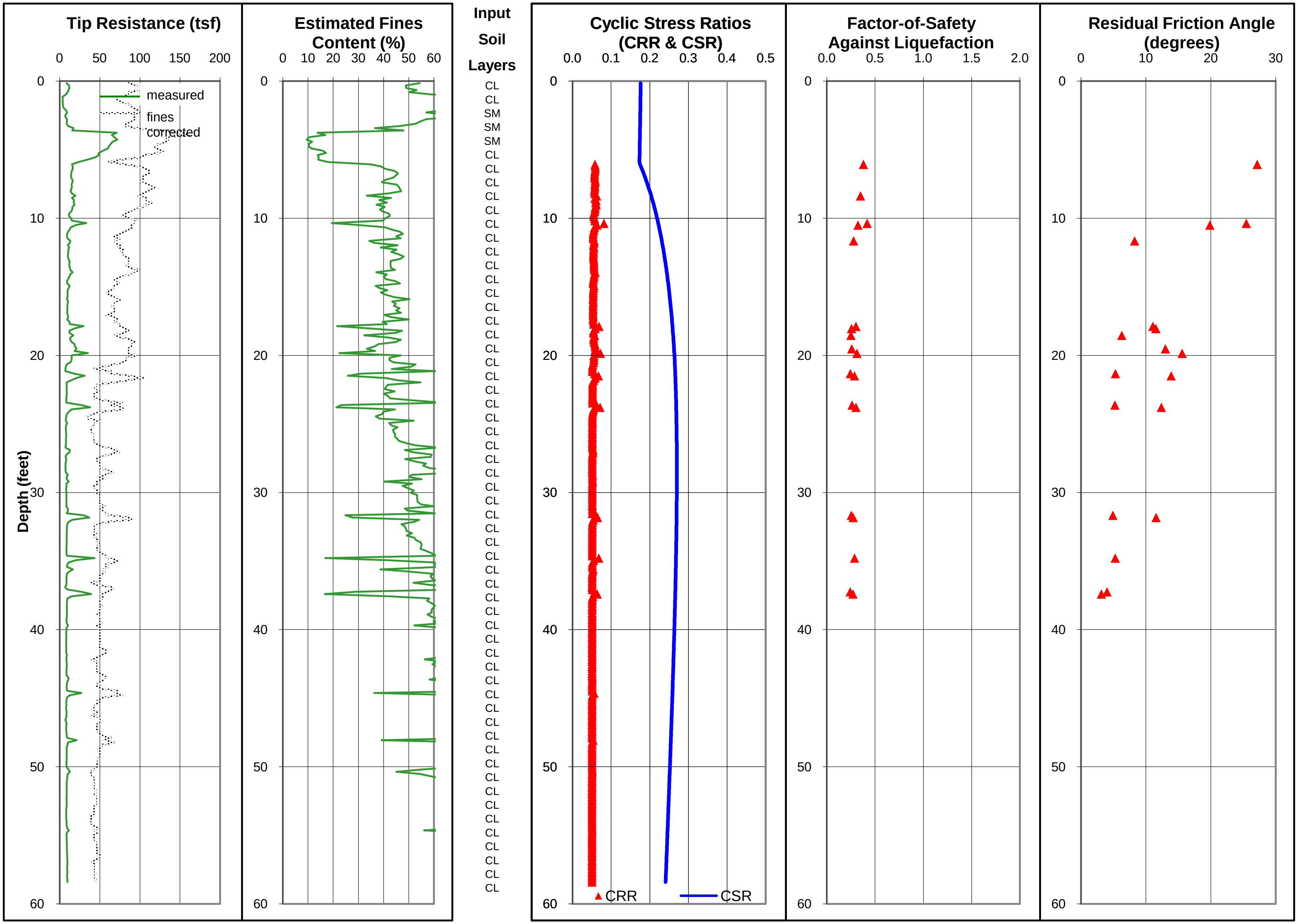
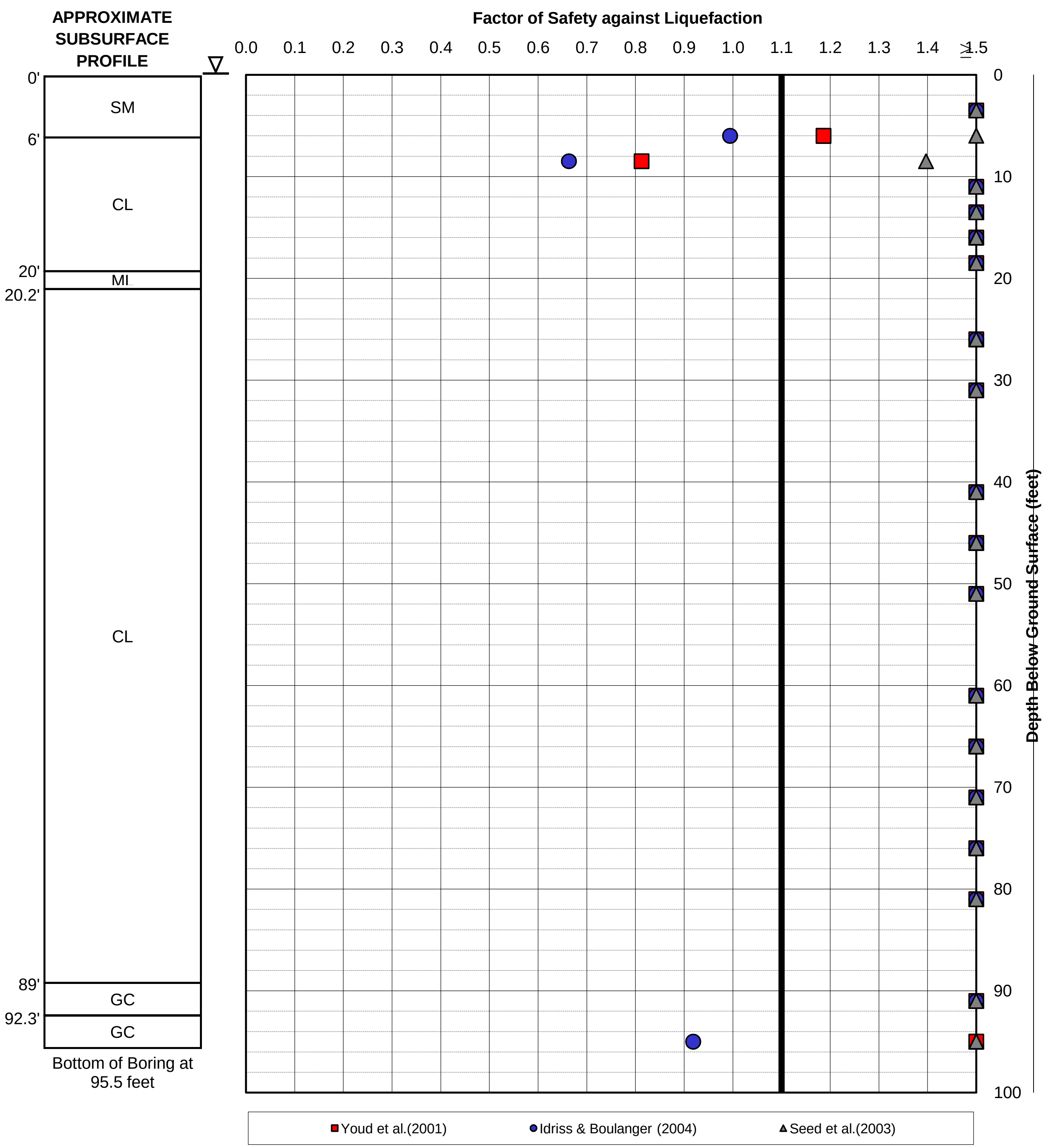


FIG. GD4-23

SPT Liquefaction v2.0.141 SWB204.xlsm 3/11/2009



NOTES

1. See main text for references.
2. The liquefaction resistance of a soil is based on its density and fines content. We used the results of the standard penetration testing to estimate the density, and the results of selected laboratory tests to estimate the fines content.

Pioneer Crossing, Lehi
and I-15 American Fork Interchange
Lehi, Utah

LIQUEFACTION ANALYSES (MCE)
BORING SWB-204
M = 7.0, PGA = 0.46

March 2009 | 23-1-01178-010

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. GD4-24

RESULTS OF LIQUEFACTION ANALYSES (MCE)

USING CPT DATA

Date: 3/18/2009
Time: 8:44 AM

Filename: LIQcpt_2007_SWC-214 MCE.xlsm

CPT Hole Designation: **SWC-214**
Groundwater Table Depth: 6.0 feet

Design Earthquake Magnitude: 7
Peak Ground Acceleration: 0.46 g

Project Name: **Pioneer Crossing**
Job Number: **23-1-01178-010**

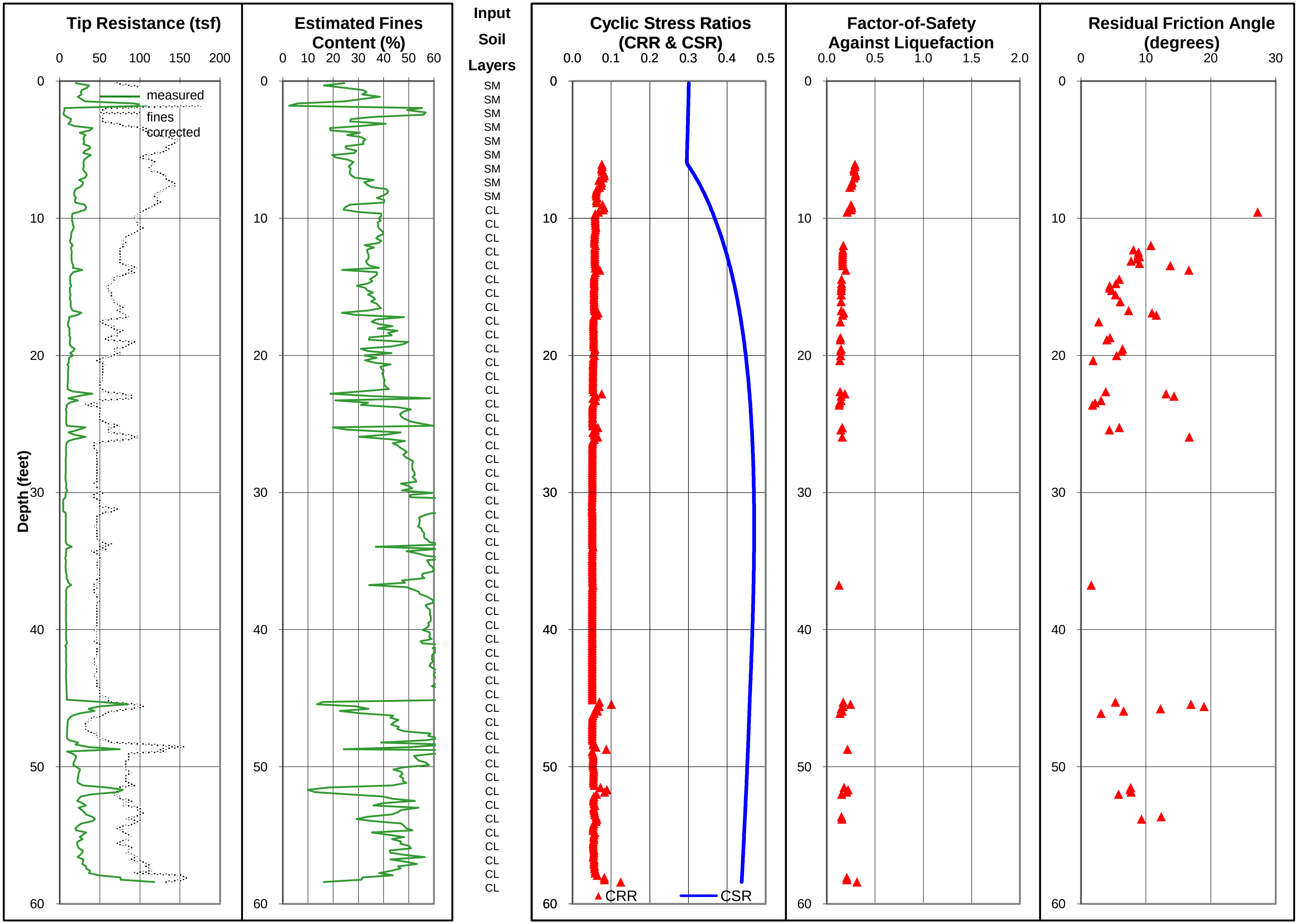


FIG. GD4-25

RESULTS OF LIQUEFACTION ANALYSES (MCE)

USING CPT DATA

Date: 3/18/2009
Time: 8:45 AM

Filename: LIQcpt_2007_SWC-215 MCE.xlsm

CPT Hole Designation: **SWC-215**
Groundwater Table Depth: 6.0 feet

Design Earthquake Magnitude: 7
Peak Ground Acceleration: 0.46 g

Project Name: **Pioneer Crossing**
Job Number: **23-1-01178-010**

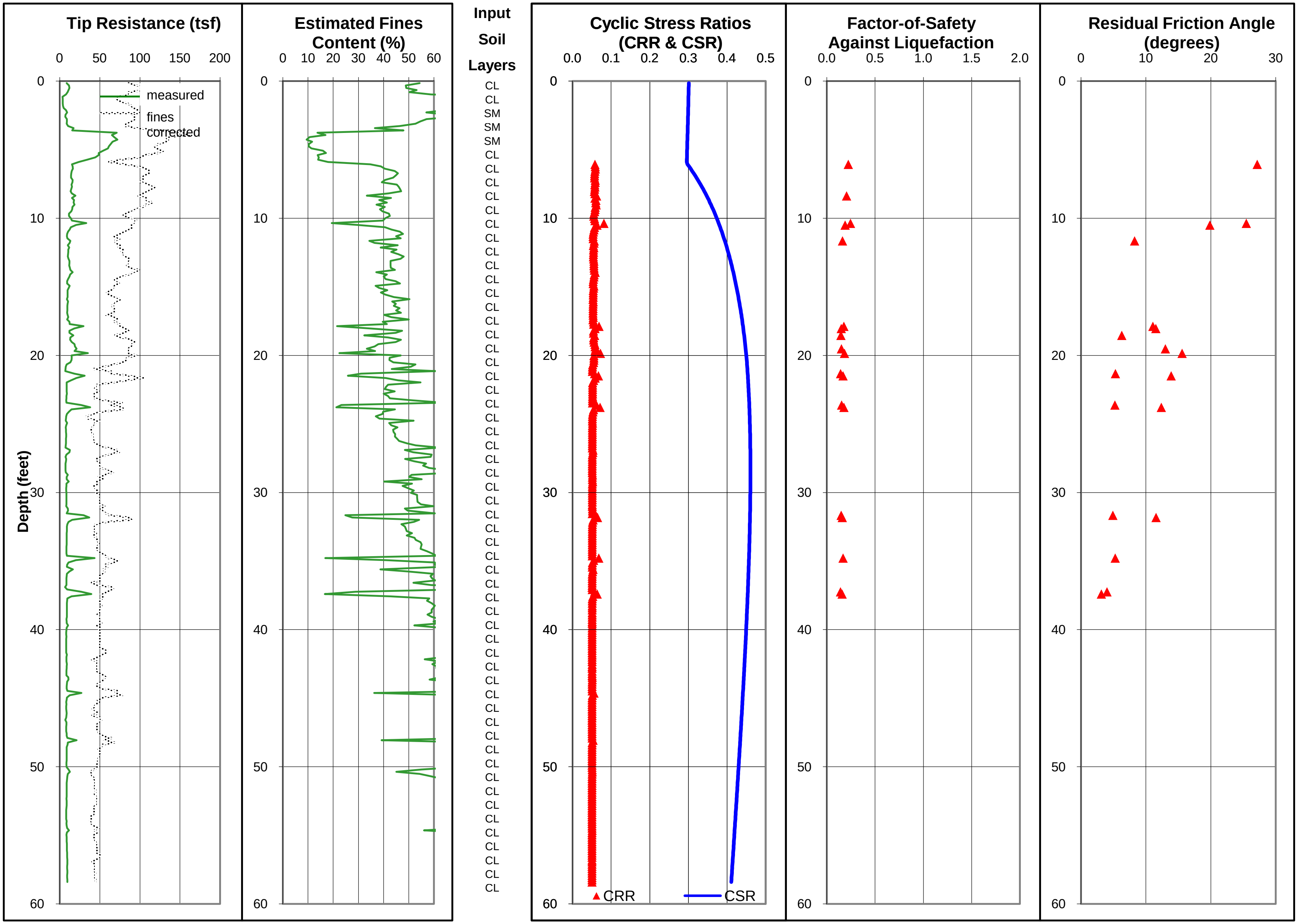
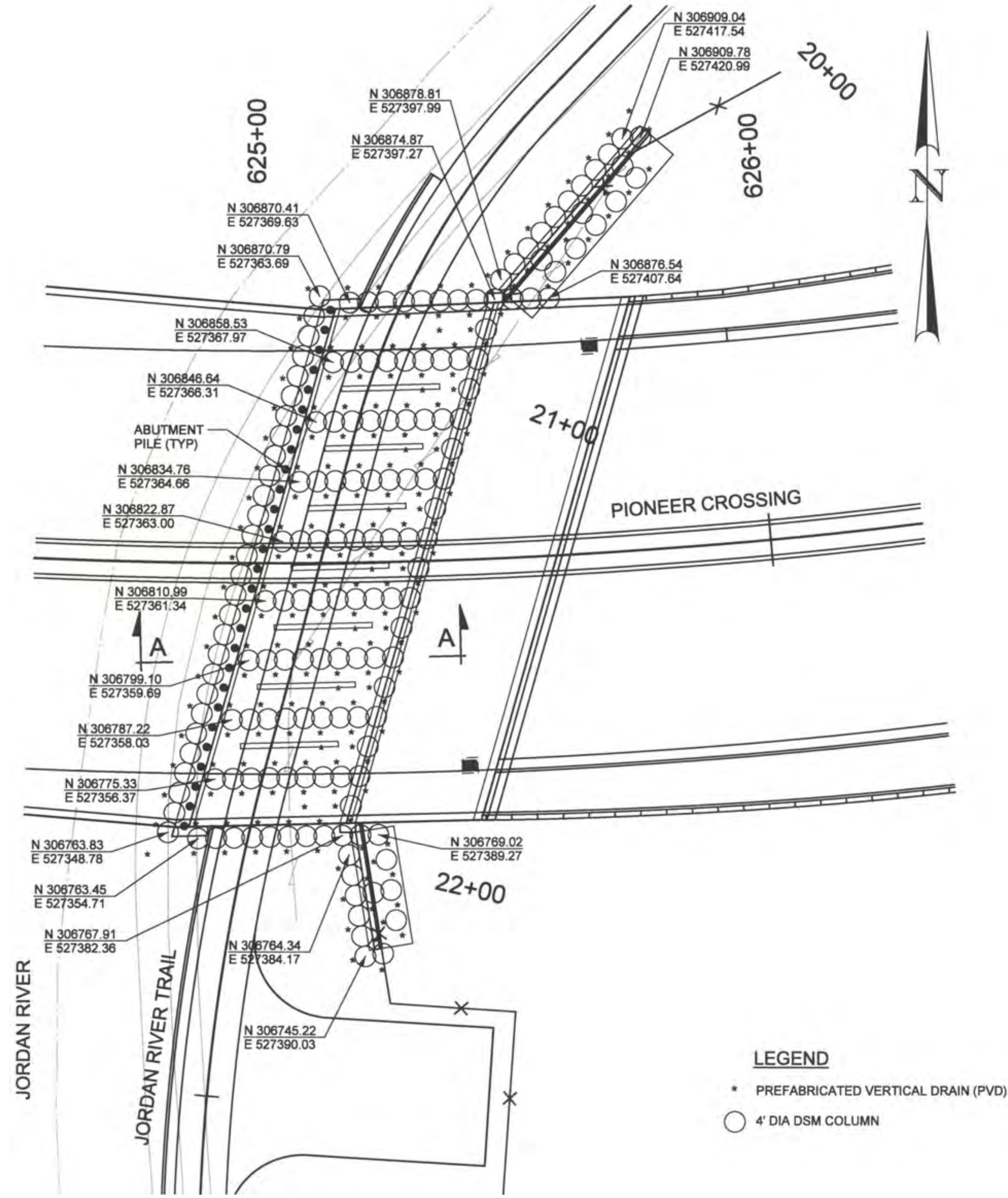
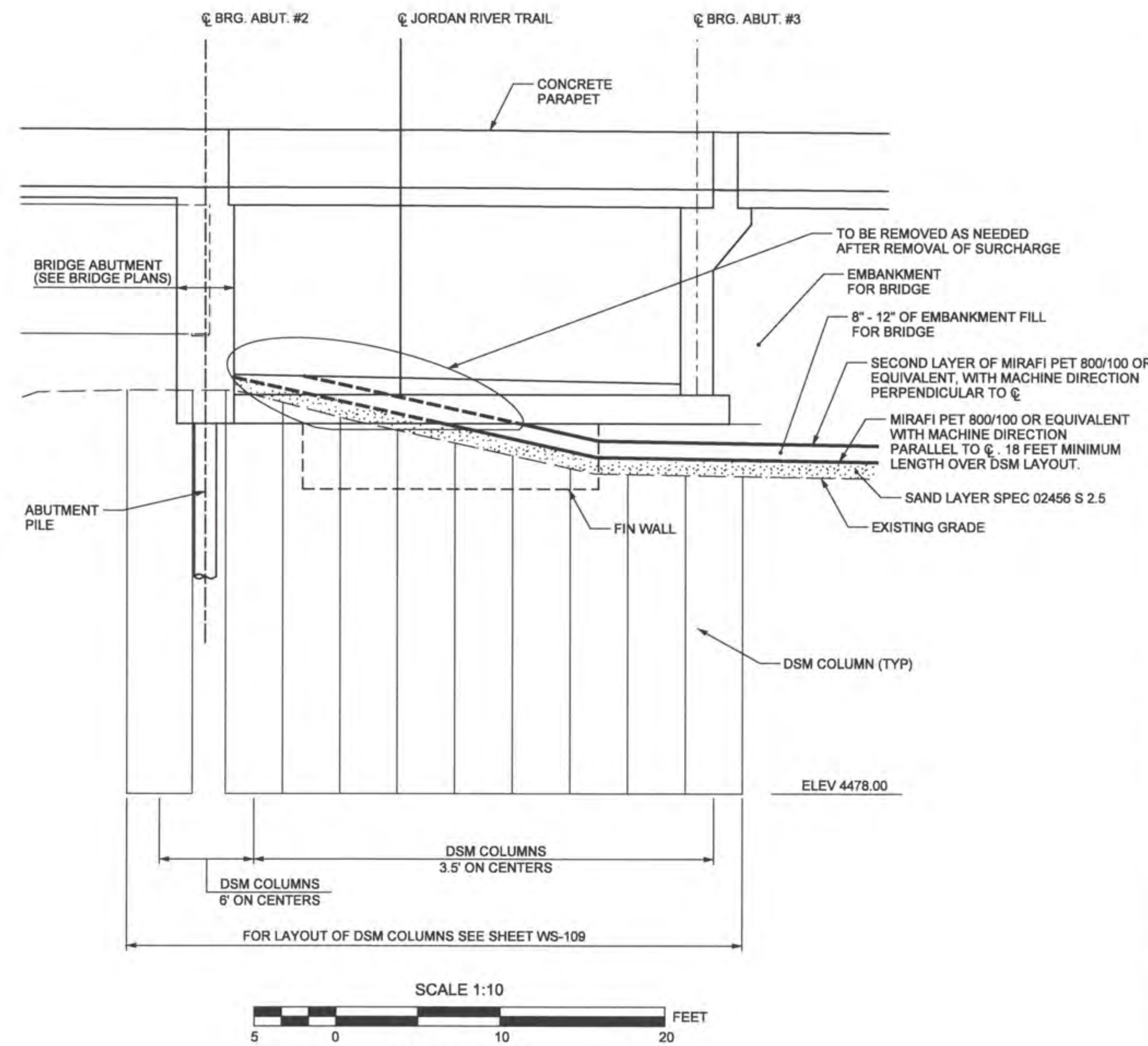


FIG. GD4-26



EAST PIONEER CROSSING ABUTMENT AND DSM COLUMN LAYOUT



SECTION A
THRU DSM GROUND IMPROVEMENT

CONSTRUCTION SEQUENCE

1. INSTALL TEMPORARY EROSION AND SEDIMENTATION CONTROL.
2. INSTALL DSM COLUMNS.
3. INSTALL PVD'S PER SHEET WS-004, WS-101 & WS-109. GEOTECHNICAL ENGINEER SHALL APPROVE FIELD ADJUSTMENTS TO PVD LOCATION. PVD'S OUTSIDE LIMITS OF DSM COLUMNS SHALL BE SPACED PER DETAIL ON SHEET WS-101.
4. INSTALL SAND LAYER, GEOTEXTILE AND EMBANKMENT FILL PER SHEETS WS-101 AND WS-104.

GEOTECHNICAL ASPECTS OF THE PVD, SURCHARGE AND EARTHWORK DESIGN WERE PREPARED UNDER THE DIRECT SUPERVISION OF CHRISTOPHER A. ROBERTSON, P.E.

SCALE 1:30
FEET
15 0 30 60

PROFESSIONAL ENGINEER
No. 7030264-2202
CHRISTOPHER ALAN ROBERTSON
3/16/2009
ELECTRONIC SEAL
STATE OF UTAH

REVISIONS

NO.	DATE	APPROVED BY	REMARKS
1	03/13/09	CAR	RFC - LIQUEFACTION

DRAWN BY: CSN
QC CHECKED BY: WJP

UTAH DEPARTMENT OF TRANSPORTATION
ROADWAY DESIGN

PROJECT: UTAH COUNTY ACCESS MANAGEMENT
PIONEER CROSSING - SEGMENT 1

PROJECT NUMBER: S-R399(42) & S-R399(59)

DSM COLUMN LAYOUT SHEET

WS-108

APPENDIX E

GEOTECHNICAL DESIGN MEMORANDUM GD-5

GEOTECHNICAL DESIGN MEMORANDUM GD-5
PIONEER CROSSING, LEHI AND I-15 AMERICAN FORK INTERCHANGE

PIONEER CROSSING APPROACH FILL, EMBANKMENT STABILITY, AND JORDAN RIVER BRIDGE

February 26, 2009

Prepared by:
Shannon & Wilson, Inc.,
Geotechnical Consultant for Kiewit/Clyde Joint Venture

PROFESSIONAL ENGINEER
No. 7030264-2202
CHRISTOPHER
ALAN
ROBERTSON
STATE OF UTAH

2/26/09

Christopher A. Robertson, P.E.
Vice President

CIJ:GSE:WJP:CAR/cij

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GEOTECHNICAL DESIGN MEMORANDUM GD-5
PIONEER CROSSING, LEHI AND I-15 AMERICAN FORK INTERCHANGE

1.0 DESIGN MEMORANDUM SCOPE

This design memorandum addresses geotechnical recommendations and considerations for design and construction of approach fills, embankment stability, and walls for Pioneer Crossing and foundations of the Jordan River Bridge. Related analysis procedures and criteria are presented in the following geotechnical design memoranda:

- GD-2, Geotechnical Analysis and Design Methodologies (Shannon & Wilson, Inc., 2009a)
- GD-3, Seismic Design Criteria (Shannon & Wilson, Inc., 2008b)
- GD-4, Liquefaction and Lateral Spreading (Shannon & Wilson, Inc., 2009b)

Final versions of this and other geotechnical design memoranda issued for the project will be compiled into the Pioneer Crossing, Lehi and I-15 American Fork Interchange Project Geotechnical Report upon completion of the geotechnical engineering design phase of the project. General limitations on the use of this memorandum and recommendations presented herein are presented in Geotechnical Design Memorandum GD-1 (Shannon & Wilson, Inc., 2008a).

2.0 PROJECT ELEMENT AND SITE DESCRIPTION

The Pioneer Crossing, Lehi and I-15 American Fork Interchange project (project) will construct an east to west urban arterial through Lehi connecting Redwood Road to I-15. The proposed arterial will parallel State Route (SR)-73 at approximately 900 South and will include new bridge structures to extend the roadway over the Union Pacific Railroad tracks near Mill Pond and over the Jordan River in Saratoga Springs. The project includes a new Interchange at Main Street in American Fork. The Vicinity Map, Figure GD5-1, shows the project area.

We understand the preliminary pavement design consists of approximately 11 inches of granular borrow overlain by 6 inches of untreated base coarse, 4 inches of lean concrete base, and

9 inches of plain jointed concrete pavement. Final pavement design is being done by Terracon Consultants, Inc. (Terracon).

The proposed arterial alignment begins at Redwood Road at approximately 600 North in Saratoga Springs and extends east. After crossing the Jordan River, it continues to the northeast to approximately 3200 West. The alignment then continues east along approximately 900 South in Lehi to approximately 1700 West where it turns slightly southeast and extends to approximately 790 West and 1250 South. It then turns east and continues to approximately 300 East and 1250 South where it bends to the northeast and extends to Mill Pond Road and 850 East. The alignment then continues east until it terminates at I-15.

This design memorandum addresses the proposed Jordan River Bridge foundations and fill embankments from Redwood Road at about Station (Sta.) 600+00 to Sta. 725+00.

The proposed roadway will be about 100 feet wide, comprising two lanes in each direction, a median and sidewalks. The new fill heights typically vary from about 2 to 18 feet depending on the elevation of the natural ground surface, with the majority of the fill embankments ranging between 4 to 8 feet. The higher embankments will form approaches to the east and west abutments of the Jordan River Bridge at 18 and 12 feet, respectively. Finished embankment slopes will typically be 3 horizontal to 1 vertical (3H:1V). In some locations, the embankment will be retained to reduce impacts to wetlands or reduce right-of-way encroachment. The retaining walls will vary in height up to about 10 feet to accommodate variation in the natural ground surface and proposed fill height.

Utility plans show two buried pipelines that could be affected by the proposed fill embankments near to the bridge approaches:

- Buried 10- and 14-inch forced sewer pipelines that cross the proposed Pioneer Crossing alignment in north-south direction at about Sta. 621+50.
- Buried 16-inch waterline that crosses the proposed Pioneer Crossing alignment in northwest-southeast direction at about Sta. 627+50.

This design memorandum does not address the overhead utilities.

The project site use includes agricultural fields, pasture, undeveloped land, some farm buildings and some commercial buildings at the east end of the corridor. The proposed alignment is mostly flat. Several inches to a few feet of standing water was visible between the Jordan River and approximately 2900 West in Saratoga Springs in November 2008. Vegetation includes grass, weeds, and occasional trees.

3.0 SUBSURFACE CONDITONS

3.1 Geology

The project is near the eastern edge of the Basin and Range physiographic province, which extends from the Sierra Nevada to the Wasatch Mountains. The Basin and Range province is characterized by north-trending mountain ranges and intervening sediment-filled valleys. The mountain ranges are typically bounded by high-angle normal faults that formed in response to regional crustal extension. The site is within about 8 miles of the Holocene Wasatch fault and the about 2 miles from the Utah Lake faults (VanHorn, 1982).

3.2 Existing Subsurface Data

Subsurface explorations were performed during the baseline studies data for the Request for Proposal (RFP) (Utah Department of Transportation [UDOT], 2008). Terracon (2008) performed 9 borings and 1 cone penetration test (CPT) between Redwood Road and Sta. 725+00. Shannon & Wilson performed an additional 4 borings, 12 CPTs, and 17 test pits during the design phase. See the Site and Exploration Plan (Figure GD5-2) for the subsurface exploration locations.

Appendix GD5-A presents a description of subsurface exploration methods, and boring, CPT and test pit logs.

3.3 Laboratory Tests

Shannon & Wilson performed geotechnical laboratory tests on selected samples. The testing includes visual classification, water content, grain size, percent passing the No. 200 sieve, Atterberg limits, and 1-D consolidation tests. Shannon & Wilson subcontracted with Terracon to perform laboratory tests on samples from selected borings that were drilled during the RFP. These tests included Atterberg Limits and grain size analyses. Laboratory test results are presented Appendix GD5-B and on the boring logs in Appendix GD5-A.

Appendix GD5-B presents selected laboratory test data that was provided in the RFP. Other RFP laboratory test results are incorporated in the boring logs that are presented in Appendix GD5-A.

3.4 Subsurface Conditions

Figures GD5-3, GD5-4, GD5-5, and GD5-6 present our interpretation of the geologic subsurface conditions along the proposed alignment from Sta. 600+00 to 725+00, near the proposed Jordan River Bridge, perpendicular to the west abutment, and perpendicular to the east abutment, respectively . The subsurface conditions near the Jordan River Bridge consist of variable fill underlain by about 20 to 30 feet of soft to medium stiff clay with scattered lenses and pockets of medium dense sand and clayey sand; 2 to 5 feet of medium dense, clayey sand; and 5 to 20 feet of stiff to hard clay with some dense, clayey sand interbeds. Very dense, silty sand and gravel were encountered in the borings and CPTs below about 50 to 55 feet depth.

Between Sta. 632+00 to 638+00, very soft to soft, silty clay to clayey silt containing trace to abundant organics was encountered in the upper 15 to 20 feet. Near Sta. 638+00, dense sand and gravel was encountered in a CPT at approximately 20 feet below the ground surface. Near Sta. 641+00 and 650+00, dense sand was encountered at 30 and 55 feet depth, respectively. The subsurface conditions from approximately Sta. 650+00 to 670+00 consist of 4 to 5 feet of medium dense, silty sand to sandy silt at the surface, underlain by nearly 85 feet soft to medium stiff clay. The moisture content is greater than the liquid limit for much of the layer. Very dense sand and gravel were encountered in borings and CPTs below about 90 feet depth.

From Sta. 670+00 to 735+00, more interfingering of sand, silt, and clay layers is present. Generally, the subsurface conditions consist of interfingered sand and silt in the upper 30 to 40 feet, approximately 20 feet of medium stiff clay, and interbeds of dense sand, gravel, and silt below 50 to 60 feet.

Groundwater was observed in wells installed in test pits and borings and interpreted from CPT dissipation tests at approximately between elevations of 4,486 and 4,507 feet. We assumed groundwater was at approximately elevation 4,490 feet near the Jordan River Bridge for our analyses.

3.5 Liquefaction and Lateral Spread

Several loose to medium dense, silty and clayey sand layers are present beneath the proposed Jordan River Bridge approach embankments (refer to Figures GD5-4, GD5-5, and GD5-6). A continuous, potentially liquefiable silty sand layer extends from near the ground surface to approximately 12 feet depth at the east abutment. Another approximately 4-foot-thick, loose to medium dense, clayey sand layer was encountered at approximately 35 feet below the ground surface (bgs). Potentially liquefiable layers were encountered from Sta. 670+00 to 725+00 at depths of 5, 10, 25 through 30, and 55 feet bgs and varied in thickness from 2 to 6 feet (refer to Figure GD5-3).

We evaluated liquefaction susceptibility of these soils using Standard Penetration Test (SPT) N-values, CPT measurements, and laboratory-measured soil fines contents using a well-verified computer program developed by Shannon & Wilson. Calculated factors of safety against liquefaction for a few of the SPT blow counts in the loose to medium dense land layers are less than 1 for the Maximum Considered Earthquake (MCE) ground motion suggesting liquefaction or elevated pore pressure could develop in these layers. Therefore, we evaluated the continuous layers of loose, silty sand below the proposed east and west embankments of the Jordan River Bridge for settlement, global instability, and lateral spreading. We evaluated the continuous layers of loose, silty sand below the proposed embankments from approximately Sta. 670+00 to 725+00 for settlement. Geotechnical Design Memorandum GD-4 (Shannon & Wilson, 2009b) presents the liquefaction analysis procedures and results. Stability analyses are presented in this Geotechnical Design Memorandum.

4.0 GEOTECHNICAL ANALYSES AND RECOMMENDATIONS

4.1 Analyses Criteria and Methods

The embankments and walls described herein were evaluated using the methods described in Geotechnical Design Memorandum GD-2 (Shannon & Wilson, 2009a).

4.2 Global Stability

We evaluated the global stability for the embankments at the east and west abutments of the Jordan River Bridge. The proposed approach embankments will be approximately 18 and

12 feet high at the east and west abutments, respectively. Our stability analyses included potential slope failure of the proposed east abutment towards the Jordan River (west) and the proposed west abutment towards the Jordan River (east). We also analyzed the stability of the east and west approach embankments, near Sta. 626+00 and 623+00, respectively, for failure perpendicular to the alignment (north). We analyzed each case for static stability during construction, static stability in the long term, seismic and post-seismic (liquefied) stability in accordance with the UDOT Manual of Geotechnical Instruction (UDOT, 2006).

4.2.1 Jordan River Bridge East Abutment

We understand the approach fill at the east abutment will typically have 3H:1V slopes to the north and south and a retained abutment. We performed analyses for the following cases:

1. Static slope stability during construction with combined 6 feet of surcharge and overbuild fill.

2. Static slope stability for the long term conditions assuming drained shear strength for the clay units.

3. Seismic slope stability assuming a seismic horizontal coefficient of 0.23g, which is one-half the peak ground acceleration of 0.46g (Shannon & Wilson, Inc., 2008b).

4. Post-seismic (liquefied) stability assuming liquefied strength for the potentially liquefiable sands/silts and reduced strength for soft clays.

Appendix GD5-C presents the results of our analyses for the above cases. Our analyses show factors of safety that meet or exceed UDOT minimum requirements for the static, seismic, and post-seismic conditions for failure to the west and north at/near the east abutment, provided the notes in the table below are followed. Factors of safety for the above cases are as follows:

FACTOR OF SAFETY AGAINST INSTABILITY – EAST ABUTMENT

Station (Direction of Failure)	Static		Seismic	Post-Seismic (Liquefied)
	End of Construction	Long-Term	k_h=0.23g	
East Abutment[6] (West)	1.3 [1,3]	1.8 [2,3]	1.0 [2,3,5]	1.4 [2,3]
626+00 (North)	1.6 [1,4]	2.6 [2,4]	1.1 [2,4]	1.5 [2,4]
626+50 (North)	1.3	2.3	Not Required	Not Required
626+00 (South)	Not Analyzed	Not Analyzed	1.0 [2,4]	Not Analyzed
UDOT (2006) Minimum Required Factor of Safety	1.1	1.5	1.0	>1.1 or deformation analysis

Notes:
1. Using a geotextile allowable construction strength of 20,000 lbs (machine direction).
2. Using a geotextile long-term strength of 15,000 lbs (machine direction).
3. Using a geotextile length of 80 feet behind abutment face.
4. Using a geotextile on the full width of the embankment.
5. Using a lateral pile resistance of 30,000 lbs, which can be achieved with less than 0.5 inch of pile deflection.
6. Overexcavate and replace potentially liquefiable silty sand layer. Lateral extent of overexcavation is described in Section 4.3.4.

UDOT = Utah Department of Transportation

The combined surcharge plus overbuild fill thickness recommended in Section 4.3 of this geotechnical design memorandum is 6.5 feet, i.e., 0.5 foot more than assumed for the stability analyses. By inspection we concluded the proposed embankment plus surcharge and overbuild fills should be stable during construction.

Our recent subsurface explorations encountered a zone of potentially liquefiable sand, silty sand, and silty gravel near the proposed east abutment of the Jordan River Bridge. This potentially liquefiable zone extends to about elevation 4,482 feet, which is a depth of about 12 feet. Groundwater is near elevation 4,490 feet. Our analyses show that some ground improvement will be needed for post-seismic stability. The ground improvement will need to extend through the potentially liquefiable zone. Ground improvement could consist of overexcavation and replacement, stone column (including Geopiers) or deep soil mixing. Deep compaction (deep dynamic compaction or rapid impact compaction) may be practical. However, we have concerns about the efficacy of deep compaction methods given the high groundwater and interbedded silt and clay layers.

For the purposes of this memorandum, we assumed the ground improvement would consist of overexcavation and replacement or other methods that would provide the same increase in shear strength in the potentially liquefiable layer. We can provide recommendations for other methods if desired. We recommend overexcavating for the entire embankment width through the potentially liquefiable layers. The extent of the overexcavation is described in Section 4.3.4. The backfill should consist of densely compacted granular fill material that has a minimum shear strength of 36 degrees.

Because the granular fill will be placed over very soft to soft clay, it may be necessary to place a construction geotextile or a geogrid to provide a working surface to compact the dense granular backfill.

Our analyses also show that a high strength geotextile or a geogrid will be required for global stability towards to west for the static construction, seismic and post-seismic (liquefied) cases. A high strength geotextile/geogrid will also be required for global stability towards the north and south within 50 feet of the abutment for the seismic case. One layer of geotextile/ geogrid with machine direction parallel to roadway centerline should extend 80 feet east of the abutment and to the toe of the fill to the west, north and south to provide additional shear strength for potential failure towards the west. A second layer of geotextile/geogrid with machine direction perpendicular to the roadway centerline should extend at least 50 feet east of the abutment and to the toes of the fills to the west, north and south to provide additional shear strength for potential failure towards the north and south. An 8- to 12-inch-thick lift of soil should be placed and compacted between the two layers of geotextile/geogrid. Our calculations show that Mirafi PET 800/100 high-strength woven polyester geotextile with 27,000 pounds per foot long-term design strength provides the required factors of safety. Other geotextile manufacturers could supply similar products. If deep soil mixing is used to mitigate liquefaction hazard, it may be practical to use the same but deeper soil improvement to improve stability instead of geotextile reinforcement.

4.2.2 Jordan River Bridge West Abutment

We understand the approach fill at the west abutment will typically have 3H:1V slopes to the north and south and a retained abutment. We performed analyses for the following cases:

1. Static slope stability during construction with a 4 feet of combined surcharge overbuild fill.

2. Static slope stability for the long-term condition assuming drained shear strength for the clay units.

3. Seismic slope stability assuming a seismic horizontal coefficient of 0.23g, which is one-half the peak ground acceleration of 0.46g (Shannon & Wilson, Inc., 2008b).

4. Post-seismic (liquefied) stability assuming liquefied strength for the potentially liquefiable sands/silts and reduced strength for soft clays.

Appendix GD5-C presents the results of our analyses for the above cases. Our analyses show factors of safety that meet or exceed UDOT minimum requirements for the static and seismic, and post-seismic conditions for failure to the east at the west abutment, provided the notes in the table below are followed. Factors of safety for the above cases are as follows:

FACTOR OF SAFETY AGAINST INTSTABILITY – WEST ABUTMENT

Station (Direction of Failure	Static		Seismic		
	End of Construction (vertical abutment)	End of Construction (sloping abutment)	Long-Term	k_h=0.23g	Post-Seismic (Liquefied)
West Abutment (east)	1.3 [1,3]	1.4 [1,3]	1.6 [2,3,4]	1.0 [2,3,4]	1.4 [2,3,4]
West Abutment (north)	1.4	1.7	2.3	1.1	1.5
UDOT (2006) Minimum Required Factor of Safety	1.1	1.1	1.5	1.0	>1.1 or deformation analysis

Notes:
1. Using a geotextile allowable construction strength of 25,000 lbs (machine direction).
2. Using a geotextile long term strength of 27,000 lbs (machine direction).
3. Using a geotextile length of 90 feet behind abutment face.
4. Using a lateral pile resistance of 10,000 lbs, which can be achieved with less than 0.5 inch of pile deflection.

UDOT = Utah Department of Transportation

The approach embankment will likely be constructed prior to the placement of the vertical abutment face. To account for variation in construction methods, we analyzed the stability of the west abutment for potential failure to the north and west for both a vertical face and a sloping abutment.

Our analyses show that a high strength geotextile or a geogrid will be required for global stability towards the east. One layer of geotextile/geogrid with machine direction parallel to roadway centerline should extend 90 feet west of the abutment to provide additional shear strength for potential failure towards the east. Our calculations show that Mirafi PET 800/100 high-strength woven polyester geotextile with 27,000 pounds per foot long-term design strength provides the required factors of safety. Other geotextile manufacturers could supply similar products.

4.2.3 Mechanically Stabilized Earth (MSE) Walls

We understand that a one-stage Terramesh MSE wall will retain the south side of the embankment bordering a wetland area approximately between Sta. 630+50 to 640+50. The proposed embankment/MSE wall heights vary from 5 to 7 feet, and additional combined surcharge and overbuild fill height will range between 3.5 and 4.0 feet. Subsurface explorations in this area included five test pits spaced approximately 200 feet apart, two CPTs near the west end of the MSE wall, and one CPT near the east end of the MSE wall. These explorations encountered very soft to soft, silty clay to clayey silt containing trace to abundant organics in the upper 15 to 20 feet (Figure GD5-3).

We analyzed the cross sections along the alignment with a 7-foot-high permanent embankment and minimum strap length (8-foot-long). We evaluated the global stability using limit equilibrium techniques for both short-term (construction), long-term static, pseudo-static seismic, and post liquefaction cases. During construction, we assumed 4 feet of combined surcharge plus overbuild fill would be present. Our global stability analyses for short-term construction and long-term permanent wall sections indicated that additional shear resistance would be required to achieve UDOT minimum factors of safety requirement. To achieve the required factors of safety, we recommend placing a single layer of geotextile fabric over the native subgrade soil that spans the entire embankment footprint with the machine direction perpendicular to the roadway centerline. Our calculations show that Mirafi PET 800/100 high-strength woven polyester geotextile with 27,000 pounds per foot long-term design strength provides the required factors of safety. Other geotextile manufacturers could supply similar products. Appendix GD5-C presents the results of our analyses for the above cases.

We understand that a 5-foot-diameter proposed water line may be installed either in the subgrade soil prior to embankment construction or in the embankment fill behind the proposed MSE wall. We do not have information regarding the invert depth and location; therefore, our global stability analyses do not account for this pipeline. If the pipeline were added in the embankment, we anticipate higher factors of safety because the pipeline should force larger potential critical failure surfaces. A summary of the calculated stability factor of safety for the MSE wall are presented in the following table along with the minimum UDOT requirements:

FACTOR OF SAFETY AGAINST INSTABILITY – MSE WALL

Stability Check	Calculated FS	UDOT (2006) Minimum FS
Static - End of Construction	1.1[1]	1.1
Static - Long-Term	1.9[1]	1.3
Seismic	1.6[1]	1.0
Post-Seismic (Liquefied)	1.3[1]	1.1

Notes:
1. Using a geotextile allowable construction strength of 27,000 lbs (machine direction) spanning the entire width of the embankment.
FS = factor of safety
MSE = mechanically stabilized earth
UDOT = Utah Department of Transportation

4.2.4 Post and Panel Walls

We understand that Post and Panel walls will retain the south and north sides of the embankment bordering a wetland area approximately between approximately Sta. 650+50 to 660+00. The proposed embankment and Post and Panel wall heights vary from approximately 4 to 7 feet, and additional combined surcharge and overbuild fill height will be about 2 feet. Subsurface explorations in this area included one CPT and one boring. These explorations encountered medium stiff to stiff clays underlying an about 5–foot-thick sand layer at the ground surface (Figure GD5-3).

We analyzed a cross section along the alignment with a 7-foot-high permanent Post and Panel wall retained embankment. We evaluated the global stability using limit equilibrium techniques for both short-term (construction), long-term static, pseudo-static seismic, and post liquefaction cases. During construction, we assumed 2 feet of combined surcharge plus overbuild fill would be present. Our global stability analyses show factors of safety that meet or exceed

UDOT (2006) minimum requirements for all the above analyzed cases. Appendix GD5-C presents the results of our analyses.

A summary of the calculated stability factor of safety for the Post and Panel wall are presented in the following table along with the minimum UDOT requirements:

FACTOR OF SAFETY AGAINST INSTABILITY - POST AND PANEL WALL

Stability Check	Calculated FS	UDOT (2006) Minimum FS
Static – End of Construction	1.2	1.1
Static – Long-Term	2.3	1.3
Seismic	1.2	1.0
Post-Seismic (Liquefied)	1.5	1.1

Notes:
FS = factor of safety
UDOT = Utah Department of Transportation

4.3 Settlement Analyses

This section presents recommendations for the early surcharge design for Segment 1 of the Pioneer Crossing project. It includes recommendations for surcharge and prefabricated vertical drains (PVD), also known as wick drains, to reduce post-construction settlement in accordance with the project warranty requirements presented below.

4.3.1 Settlement Analysis Approach

For our settlement analyses, we used Version 1.0 of the commercial software Settle^{3D} developed by RocScience. Settle^{3D} was used to model the Boussinesq pressure distribution with depth and to calculate the elastic and time-rate consolidation settlements for different construction staging. For granular soil (sand, gravel, and non-plastic silt layers) we used elastic settlement analysis procedures. We assumed elastic settlement will occur relatively quickly, i.e., settlement should occur essentially as the load is applied. For settlement in cohesive layers, we used the one-dimensional consolidation theory. We selected consolidation settlement parameters based on consolidations tests performed on selected samples from borings SWB-202, SWB-203, and SWB-204. We also considered published literature correlations with soil index properties, correlations with CPTs, and published soil properties for similar subsurface conditions in the Salt Lake area.

We developed settlement estimates along the roadway alignment based on the generalized soil profiles presented in Figures GD5-3 through GD5-6. We assigned different local soil profiles for different segments along the roadway alignment based on the continuity of the soil layers and their properties. A summary of the subsurface profiles, elastic, and consolidation settlement parameters used in our settlement analyses is presented in Tables GD5-1 through GD5-6. We evaluated the secondary compression rate that takes place in the clay layers after removal of the surcharge based on their degree of consolidation (over consolidation ratio) caused by the surcharge preloading using relationships presented in Saye and Ladd (2000).

In our analyses, we considered multiple stages to differentiate between the settlement that will occur during the construction phase and long-term settlement that will occur from after paving till the end of the five-year warranty period. In select areas, surcharge fills will be placed to slow secondary compression rate. After primary consolidation is complete, the surcharge fills will be removed and the roadway will be paved. In select areas, installation of PVDs will be combined with placing surcharge to expedite the time to primary consolidation.

4.3.2 Pavement Settlement Warranty Requirements

Section 9.0 of the RFP specifies the following maximum permissible settlement for pavement:

- Transverse direction: Maximum 0.25-inch per 12-foot lane width.
- Maximum tolerable longitudinal direction: 0.25-inch per 30 feet.
- Allowable total post-construction settlement: 2 inches, unless otherwise approved by the Department (UDOT, 2008).

4.3.3 Settlement Mitigation

Table GD5-7 summarizes our settlement estimates and our recommendations for settlement mitigation. Due to the uncertainties and limitations of our analysis, the settlement estimates presented in this table indicate likely estimates of settlement across the alignment. Variations in the assumed subsurface stratigraphy, estimated consolidation soil properties and/or in assumed construction sequencing may result in more or less actual settlement.

We recommend several methods for reducing settlement to meet the warranty requirements described above. Table GD5-7 shows the Stations (Sta.) where each of these settlement mitigation methods should be used, and details for each, e.g., surcharge height and duration. In the following discussion, finished grade means top of pavement and pavement subgrade means the top of the Granular Borrow or base of the untreated base course (UTBC) layer.

No Settlement Mitigation. No settlement mitigation is recommended in areas where anticipated settlement is less than the warranty requirements after construction. Paving can take place immediately after fill placement and compaction is complete.

Embankment Preload. Where settlement greater than 2 inches is expected following fill placement, we recommend constructing the embankment to the pavement subgrade elevation and then allowing fills to settle for the times shown in the Table GD5-7. After the preloading period, the embankment should be regraded to the pavement subgrade elevation and the pavement layers can then be constructed. If Kiewit/Clyde prefers to reduce placing new fills, they could overbuild above the pavement subgrade elevation by the amount of settlement that is expected to occur.

These recommendations do not include provisions for protecting the subgrade from deterioration that could occur during wet and/or freezing weather. Before paving, the soil loosened by weather and other factors should be reworked and recompacted to meet the embankment fill material specifications, or should be removed.

Surcharge. Where moderately thick fills will support the traveled way, we recommend placing a surcharge fill to consolidate underlying compressible soils to reduce post-construction settlement. The surcharge fill should remain in place for several months to allow the underlying clay to consolidate. In addition to the surcharge, some of the overbuilding for the embankment is needed to compensate for settlement of the underlying native soil. Figure GD5-7 shows our recommended surcharge fills configuration.

Table GD5-7 shows our recommendations for surcharge thickness plus overbuild thickness, and for consolidation duration. The overbuild thickness should consist of embankment material that will remain in place after the surcharge fill is removed. Therefore, we recommend that the overbuild thickness be equal to or greater than the estimated settlement. Following the consolidation period, the surcharge fill should be removed down to the pavement subgrade elevation. The surcharge plus overbuild height shown in the Table GD5-7 is referenced to the finished grade. Please note this is different from the recommendations above for no settlement mitigation and embankment preload, which recommend building the embankments to the pavement subgrade elevation.

Transitions from thicker to thinner surcharge plus overbuild fills should be at a 20H:1V slope. The full surcharge plus overbuild height should be constructed between the stations shown in Table GD5-7. The 20H:1V transition should occur over the adjacent stations that have thinner surcharge plus overbuild fills. Where no adjacent surcharge fills are recommended, the surcharge plus overbuild fill thickness should taper down at 20H:1V to the pavement subgrade elevation. At the east and west abutments of the Jordan River Bridge, the surcharge fill plus overbuild should taper as shown in Figure GD5-7.

Surcharge with PVDs. Where thick fills are required to support the traveled way, we recommend placing surcharge fills and using PVDs to overconsolidate underlying compressible soil to reduce post-construction settlement. Surcharge recommendations are the same as presented above. PVDs reduce primary consolidation time, thereby increasing the post-construction degree of consolidation and overconsolidation. Our analyses show that PVDs will be needed to reduce post-construction settlement beneath portions of the proposed Jordan River Bridge abutments. Table GD5-7 presents our recommendations for areas using PVDs.

We recommend a maximum PVDs spacing of 5.5 feet measured center-to-center in a triangular pattern, with an equivalent drain diameter of 3 inches. The PVDs should be installed to a maximum elevation of 4,450 feet or to refusal. If refusal is encountered at a shallower elevation, pre-drilling could be required. PVDs installed through the existing “levee” may require either predrilling or removing part of the “levee.”

The PVD installation may need to consider the liquefaction mitigation measures. If the liquefaction measures consist of overexcavation and replacement with dense sand or shot rock, then PVD installation likely will need to be done before placing the dense fill. For this case, we recommend performing the overexcavation, placing a working layer of sand, installing the PVDs, covering the PVDs with sand, and then placing the dense compacted backfill. The sand layers should be densely compacted. Alternatively, the PVDs could be installed from the current ground surface before making the overexcavation. Once the overexcavation extends to its design depth, we recommend placing a sand layer to drain water discharging from the PVDs.

4.3.4 Jordan River Bridge Approaches

This section presents recommendations for meeting the RFP longitudinal settlement warranty requirements at Jordan River Bridge approaches. It provides longitudinal settlement estimates for varying slab lengths and a discussion of mitigation alternatives. The following assumptions were made in our longitudinal settlement analyses:

- Total settlement mitigation measures currently planned for both bridge approaches include installing PVDs, placing temporary surcharge fills during the time-rate settlement period, and conducting a settlement monitoring program to determine an appropriate time to remove surcharge fills.
- For the east abutment, our seismic and post-liquefaction stability analysis show that ground improvement mitigation is required. We understand that overexcavation of potentially liquefiable material may be selected. Overexcavation will extend the full depth of the liquefiable layer from at least 5 feet west of the abutment and wing walls to at least 33 feet east of the abutment. At the east end the overexcavation will extend north and south to the toes of the fills. The overexcavation will taper up to the ground surface at slopes required for stability during construction.
- Bridge abutments will be founded on closed-end pipe piles at each abutment such that little post-construction abutment settlement should occur. Therefore, longitudinal differential settlement between the abutment and end of approach slab will be essentially the total post-construction settlement for a single span approach slab. If two approach slabs are considered, the differential settlement for each span will need to be considered (as discussed in further detail in the east abutment approach slab).

4.3.4.1 West Abutment Longitudinal Settlement

For the west abutment, we assumed that permanent embankment fill height will extend up to 12 feet in height (including pavement thickness) at the abutment. To reduce post-construction settlement, we recommended an overbuild plus surcharge height of 4 feet from abutment to Sta. 622+00, and using PVDs to accelerate primary consolidation from the 30 feet east of the bridge abutment to Sta. 622+00 (see Table GD5-7). Our longitudinal settlement estimates for the west abutment is summarized in Table GD5-8.

We estimate the total post-construction to end of warranty period settlement will be on the order of 1 inch (rows 1 plus 2 in the Table GD5-8). If a 30-foot-long approach slab were built, the allowable differential settlement at the end of warranty is 0.25 inch across the slab length (row 3 in Table GD5-8). To meet the warranty requirement, the end of the approach slab would have to be constructed 0.75 inch above design grade (see row 4 in Table GD5-8) to accommodate the amount of settlement expected in the warranty period. At the beginning of the warranty period, we predict that the end of the approach slab would be about 0.35 inch above design grade (see row 5 in Table GD5-8). If UDOT applied the longitudinal differential settlement criterion as an allowable differential grade criterion across the slab, the end of the approach slab would be 0.1 inch too high. Please note that our settlement estimates could vary as described above.

The last column in Table GD5-8 presents a similar analysis for a 60-foot-long approach slab. If actual settlement does not vary from our predictions, our calculations indicate the longitudinal settlement warranty requirements could be met by building the end of the slab 0.5 inch above design grade.

4.3.4.2 East Abutment Longitudinal Settlement

For the east abutment, we assumed that permanent embankment fill height will extend up to 18 feet in height (including pavement thickness) at the abutment. To reduce post-construction settlement, we recommend the settlement mitigation presented in Section 4.3.3 and Table GD5-8. To mitigate liquefaction hazard, we recommended a zone of overexcavated material under the pedestrian underpass (see Section 3.5). Our longitudinal settlement estimates for the east abutment is summarized in Table GD5-9.

We understand that the trail could cross under the east approach using a two-approach slab configuration, as shown in Figure GD5-8, or in a concrete box structure that would be under and independent from a single 60-foot approach slab.

For the two-approach slab alternative, the overexcavation of the subgrade material under the pedestrian underpass (Sta. 625+30) will result in a slower, long-term settlement rate than the unimproved subgrade 60 feet east of the abutment (Sta. 625+60). At Sta. 625+30, we estimate the total post-construction to end of warranty period settlement will be on the order of 0.8 inch (rows 1 plus 2 in Table GD5-9), and at Sta. 625+60 we estimate total post-construction settlement of 1.7 inches. If two 30-foot approach slabs are built, the allowable differential grade across the first slab, assuming the abutment does not settle, is 0.25 inch. The allowable differential grade for the second slab is the differential grade of the first slab (0.25 inch) plus 0.25 inch over the 30-foot slab; effectively 0.5 inch over 60 feet. To meet the warranty requirement, the end of the approach slabs would have to be constructed 0.55 and 1.2 inches, respectively, above design grade (see row 4 in Table GD5-9). At the beginning of the warranty period, we predict that the differential grade from the abutment to Sta. 625+30 would be 0.35 inch above plan grade and 0.25 inch above grade between Sta. 625+30 and 625+60 (see row 5 in Table GD5-9). If UDOT applied the longitudinal differential settlement criterion across the slab, the first slab would be 0.1 inch high at Sta. 625+30, and the net differential settlement across the second slab would be at allowable longitudinal settlement requirement. Please note that our settlement estimates could vary as described previously.

For the single 60-foot approach slab, we presented a similar analysis, as described the west abutment in Table GD5-9. The overbuild requirement at Sta. 625+30 is the same as the two slab approach (1.2 inches), but will result in a grade differential at the start of warranty of 0.6 inch across the slab or 0.1 inch above the possible UDOT 0.25 inch/30 feet criteria over the 60-foot slab.

4.3.4.3 Mitigation Alternatives

Methods that could be used to meet the longitudinal warranty criteria:

(a) Build the finished pavement surface higher than the design grades to accommodate the expected settlement. Refer to Figure GD5-8 for a conceptual drawing showing this approach. Tables GD5-8 and GD5-9 present estimated overbuild heights for 30- and 60-foot slab approach lengths. As discussed above, uncertainties and assumptions made in our long-term settlement calculations could result in more or less actual settlement.

(b) Accept the risk that actual settlement will exceed the warranty criteria and repair pavement after construction if needed. Pavement repair alternatives could include jacking and pumping concrete under the slab (where the slab meets the embankment) to raise the grade, white topping the pavement surface to raise the grade, or grinding the slab surface to lower grade. If grinding of the pavement surface is considered, than a thicker slab should be constructed to accommodate the potential loss of pavement surface.

(c) Delay approach slab and final pavement near the bridge approaches construction until near the beginning of the warranty period to reduce row 1 settlement shown in Tables GD5-8 and GD5-9.

(d) Extending the approach slab beyond 30 or 60 feet.

4.3.5 Mechanically Stabilized Earth (MSE) Wall

The UDOT (2006) differential settlement warranty criterion for single stage MSE wall is 6 inches over 50 feet in a two-year period. We estimate total settlement along the MSE wall face will be less than 6 inches in the two-year warranty period. In our opinion, the proposed wall will meet the UDOT (2006) differential settlement criterion.

4.3.6 Seismic Settlement

Settlement that may potentially occur as a result of an earthquake is described in Geotechnical Design Memorandum GD-4, Liquefaction and Lateral Spread (Shannon & Wilson, 2009b). In general, we anticipate post-seismic settlement will be 1 to 5 inches. Assuming the liquefaction mitigation measures described in this memorandum are adopted, we estimate that post-seismic settlement near the east and west abutments of the Jordan River should be about 1 to 2 inches.

4.4 Jordan River Bridge Abutment Foundations

The Jordan River crossing bridge runs generally in the east-west direction between Sta. 623+10 and 625+10. We understand that the bridge abutments will be supported on single row of concrete-filled, driven, closed-end steel pipe piles. The east and west abutment walls will retain about 18 feet and 12 feet of fill, respectively, measured from the existing grades.

4.4.1 Recommended Deep Foundation Axial Capacity

We understand that 16-inch-diameter, closed-ended steel pipe piles that are concrete-filled (CIP) will support the Jordan River Bridge west and east abutments. Driven piles will support axial loads through a combination of skin friction and end bearing. Figures GD5-9 and GD5-10 present our recommendations for geotechnical axial capacity versus depth at the west and east abutment piles, respectively.

We evaluated the axial capacity of proposed driven pile bridge foundations in accordance with the American Association of State Highway and Transportation Officials (AASHTO) Load and Resistance Factor Design (LRFD) Bridge Design Specification (AASHTO, 2008) Article 10.7 for driven piles. Our settlement estimates indicate that downdrag loads likely cannot be eliminated at the bridge abutment foundations. We estimate that post-construction settlement of the soil adjacent to the pile foundation under fill loads will be on the order of 1 to 2 inches. According to AASHTO (2008) Section 3.11.8, downdrag loads can be induced on pile foundations when relative settlement between soil and foundation on the order of 0.4 inch or more occurs. In our opinion, post-construction settlement will induce downdrag loads on the pile foundation. In our analyses, we limited unit tip resistance value to 400 kips per square foot (ksf) and side friction to 4 ksf. Uplift resistance was assumed to be equal to the side resistance.

Axial capacity was evaluated at Strength Limit and Extreme Limit states as follows:

1. Strength Limit State: Axial capacity was evaluated based on ultimate side friction resistance and ultimate tip resistance. Downdrag loads due to consolidation settlement acting on the foundation should be considered with other loads in this state.

2. Extreme Limit State: Axial capacity was evaluated based on ultimate side friction resistance and ultimate tip resistance. Downdrag loads due to liquefaction-induced settlement acting on the foundation should be considered with other loads in this state.

According to AASHTO (2008), the following loading cases should be considered when designing the bridge foundations.

AASHTO (2008) RECOMMENDED LOADING CASES

Limit State	Nominal Resistance	Load Combination*
Strength	Strength (“Strength Limit” plot)	DL+ LL, or DL+ DD_{Static} (whichever is larger)
Extreme	Strength+ (“Strength Limit” plot)	DL+EQ
	Extreme (“Extreme Limit” plot)	DL+EQ or DL+$DD_{seismic}$ (whichever is larger)
		DL+LL

Notes:
* Other loads like “wind loads” should be applied as appropriate.
+ Strength nominal resistance using Extreme Limit Resistance Factors.
AASHTO = American Association of State Highway and Transportation Officials
DL = dead load
LL = live load
EQ = earthquake load
DD_{static} = static downdrag
$DD_{seismic}$ = seismic downdrag

All loads should be multiplied by the appropriate load factors according to the AASHTO LRDF. Soil nominal resistance should be multiplied by the appropriate Resistance Factors as indicated in Figures GD5-9 and GD5-10.

We anticipate that hard driving conditions will be experienced when the CIP piles reach the very dense sand layer. Considering that the steel pipe piles would likely be driven to refusal in this layer, the ultimate bearing capacity for these piles could reach the maximum allowable stress acting over the drivable steel cross-sectional area. Per AASHTO LRFD (AASHTO, 2008), this maximum allowable stress is limited to one-third of the yield strength of the steel. Assuming steel yield strength of 50 kips per square inch (ksi), an ultimate unit end bearing of about 143 tons per square foot (tsf) could be reached. We estimate an ultimate end bearing of 117 tsf based on the assumed sand layer strength parameters for the geotechnical axial capacity calculations shown in Figures GD5-9 and GD5-10.

4.4.2 Recommend Foundation Lateral Capacities

Table GD5-10 presents recommended soil properties and parameters to evaluate deep foundation lateral capacities using the program LPILE^{PLUS}, including reduced soil properties corresponding to potential liquefaction expected for post-seismic conditions. Figure GD5-11 presents the AASHTO LRFD recommended p-multiplier for group effect on laterally loaded piles or drilled shafts.

For earthquake loading conditions, the lateral capacity and stiffness provided by the abutments depends on whether or not the expansion joint between the bridge superstructure and the abutment can accommodate the relative displacements between them. We understand that the current proposed design for the bridge does not include an expansion joint between the abutment walls and the superstructure. Therefore, the lateral capacity will be controlled by passive soil pressures. The lateral stiffness of the abutments can be estimated by assuming that the passive earth pressure is fully mobilized when the horizontal displacement of the abutment equals 2 percent of the abutment height. Passive earth pressures are provided in the abutment wall design recommendations portion of this report.

4.5 Retaining Wall Design Recommendations

We understand that the proposed Jordan River Bridge abutment and wing walls will be concrete walls. The wing walls on the west abutment will be cantilevered on the west abutment pile cap. On the east abutment, the wing walls of the bridge and of the underpass trail will be conventional concrete cantilever walls supported on shallow foundations.

As shown in Figure GD5-2, two proposed types of retaining walls; Post and Panel walls, and MSE walls will be constructed to reduce embankment footprint in wetland areas. The post and panel walls are supported by soldier piles that consist of a moment resisting beam in a drilled shaft. The following sections present lateral earth pressure and foundation recommendations for the bridge abutment and wing walls, and design recommendations for MSE, and Post and Panel walls.

4.5.1 Lateral Earth Pressures

For the bridge abutment walls and wing walls, we recommend estimating unfactored active and at-rest lateral resistance of soil acting on the abutment walls using the values presented in the table below that are expressed as equivalent fluid weights in pound per cubic foot (pcf). A lateral earth pressure coefficient (K) of 0.26 (active) and 0.41 (at-rest) and a soil unit weight of 130 pcf may be used to estimate lateral earth pressure acting on walls with level backfill due to surcharge. The pressures assume that the embankment fill is placed and compacted behind the wall in accordance with UDOT Standard Specifications and with the recommendations in this report, and that the water level is below the supported height of the wall. Active earth pressure is applicable to walls that are allowed to yield at the top 0.001 H or greater, where H is the height of the wall. Rigid buried walls or those that cannot yield a sufficient amount to cause active conditions should be designed using at-rest earth pressure. The following table also presents uniform dynamic increments for expected earthquake and maximum considered earthquake ground motions in pounds per square foot (psf), which should be added to the static earth pressures.

LATERAL EARTH PRESSURES

Lateral Earth Pressure Condition	Backslope	
	Horizontal	3H:1V
Active Lateral Earth Pressure (pcf)	33	39
At-Rest Lateral Earth Pressure (pcf)	54	60
Dynamic Earth Pressure Increment EE (psf)	6H	10H
Dynamic Earth Pressures Increment MCE (psf)	11H	20H

Notes:
EE =expected earthquake
H:V = horizontal to vertical
MCE = Maximum Considered Earthquake
pcf = pounds per cubic foot
psf = pounds per square foot

Unfactored passive earth pressures, assuming horizontal ground surface in front of the wall equal to the buried depth of the wall backfilled with granular structural fill, can be estimated using a recommended equivalent fluid weight of 1,000 pcf. We recommend that the resistance acting on buried portions of the wall within 2 feet of the ground surface be ignored. AASHTO LRFD recommends a passive resistance factor of 0.5 and that appropriate load factors should be applied.

For the proposed post and panel walls, we recommend assuming equivalent fluid weights of 38 and 63 pcf to calculate active and at rest earth pressures. These values differ from our recommendations above, because of expected variability in the embankment backfill material.

4.5.2 Foundations

The abutment walls will be supported on driven piles as previously described in this memorandum. We understand the proposed wing walls that extend away from the east abutment will be supported on spread footing foundations. Bearing pressure is a function of footing width. For initial design, we recommend assuming an unfactored soil bearing pressure of 8,000 psf for a 10-foot-wide footing. This recommendation assumes the liquefaction mitigation measures recommended in this memorandum are adopted and that wall footings would be supported on the improved ground. We can provide bearing capacity recommended for other footing sizes as needed. A coefficient of friction of 0.45 may be assumed along the base of the footing to resist sliding. AASHTO LRFD design recommends the use of Resistance Factor of 0.5, and 0.8 for bearing resistance and sliding, respectively.

4.5.3 Mechanically Stabilized Earth (MSE) Walls

For static long-term conditions, we analyzed the MSE wall for bearing capacity, sliding, and overturning stability. Our evaluation of the factors of safety against sliding and overturning instability is for guidance only. The MSE wall manufacturer/provider should evaluate and submit these analyses.

The long-term bearing capacity factor of safety is below the UDOT-required factor of safety. In our opinion, the factor of safety we calculate is sufficient for MSE walls. In our experience, walls built on soft subgrade commonly cannot achieve a 2.5 factor of safety for bearing capacity, yet are stable as shown by global stability analyses and long-term performance. Therefore, we recommend requesting a variance for UDOT. We can provide an MSE wall design expert if requested. If not, ground improvement would be required. Our bearing capacity calculations do not consider eccentric loading because MSE wall have flexible foundations.

We analyzed lateral squeeze in accordance with MSE wall design procedures developed by the Federal Highway Administration (Elias et al., 2001). The lateral squeeze factor of safety is the ratio of three times underdrained shear strength (for soft bearing soils) to the height of the embankment times the MSE wall backfill unit weight. The resulting factor of safety is 1.1 for a 7-foot permanent embankment height and 0.7 for the 7-foot embankment plus surcharge height. Over time, the clay subgrade will consolidate and the undrained shear strength will increase. During construction, we recommend a monitoring the wall performance with inclinometers during the surcharge and primary consolidation period to determine if excessive lateral movement is beginning. A summary of the calculated stability factors of safety for the MSE wall are presented in the following table along with the minimum UDOT requirements:

CALCULATED FACTORS OF SAFETY FOR THE CRITICAL MSE WALL SECTION

Stability Check[1]	Calculated FS	UDOT (2006) Minimum FS
Bearing Capacity	2.2	2.5
Sliding	3.3	1.5
Overturning	14.5	2.0

Notes:
FS = factor of safety
MSE = mechanically stabilized earth
UDOT = Utah Department of Transportation

4.5.4 Post and Panel Walls

We understand that the post and panel walls will be supported by soldier piles that consist of a moment resisting beam in a 2.5-foot-diameter drilled shafts. The drilled shafts will support axial loads through a combination of skin friction and end bearing. Figure GD5-12 presents our recommendations for geotechnical axial capacity versus depth at the location of the Post and Panel walls between Sta. 650+50 to Sta. 660+00.

We evaluated the axial capacity of the proposed drilled shaft foundation in accordance with the AASHTO LRFD (AASHTO, 2008) Article 10.8 for drilled shafts. Axial capacity was evaluated at Service Limit, Strength Limit and Extreme Limit states as follows:

1. Service Limit State: Shaft axial capacity was evaluated based on side friction resistance and tip resistance for service settlement limits of 0.5 and 1.0 inch.
2. Strength Limit State: Axial capacity was evaluated based on ultimate side friction resistance and ultimate tip resistance.
3. Extreme Limit State: Axial capacity was evaluated based on ultimate side friction resistance and ultimate tip resistance.

Our settlement and liquefaction analyses indicated that no static or liquefaction-induced downdrag loads should be considered for the design of the drilled shafts at the location of the Post and Panel walls. According to AASHTO (2008), the following loading cases should be considered when designing the bridge foundations.

AASHTO (2008) RECOMMENDED LOADING CASES

Limit State	Nominal Resistance	Load Combination*
Service	Service ("Strength Limit" plot)	DL+ LL
Strength	Strength ("Strength Limit" plot)	DL+ LL
Extreme	Strength ("Strength Limit" plot)	DL+EQ
	Extreme ("Extreme Limit" plot)	DL+EQ
		DL+LL

Notes:
* Other loads like "wind loads" should be applied as appropriate.
\+ Strength nominal resistance using Extreme Limit Resistance Factors.
AASHTO = American Association of State Highway and Transportation Officials
DL = dead load
LL = live load
EQ = earthquake load

All loads should be multiplied by the appropriate load factors according to the AASHTO LRDF. Soil nominal resistance should be multiplied by the appropriate Resistance Factors as indicated in Figure GD5-12. Table GD5-11 presents recommended soil properties and parameters to evaluate the drilled shafts lateral capacities using the program LPILE^{PLUS}.

5.0 CONSTRUCTION CONSIDERATIONS

5.1 Driven Pile Installation

5.1.1 Wave Equation Analysis for Pile (WEAP) Driving

To establish driving criteria for pile installation, we performed a preliminary Wave Equation Analyses for Pile driving (WEAP) analysis assuming 16-inch closed end pipe pile will be used at the west and east abutments. Kiewit/Clyde provided three different hammers that may be used to drive the piles. The three hammers are IHC S-70, IHC S-90, and Delmag D-36 (APE 36-32). WEAP analyses allow evaluation of driving stresses so that an appropriate pile-driving hammer size can be selected to obtain the desired pile capacity without damaging the pile. The analyses also provide estimates of the ultimate pile capacity for a given driving resistance (blows per foot). All piles should be driven to ultimate loads as calculated by WEAP. We performed WEAP analyses using the computer program GRLWEAP (PDI, 2005). GRLWEAP is a one-dimensional Wave Equation Analysis program that simulates the pile response to pile-driving equipment. The subsurface conditions at each bridge abutment were input in the analysis. We assumed a pile length at each location that will achieve at least 5 feet of penetration into the very dense, silty sand layer. No pile cushion was used in our analyses.

Results of the WEAP for the west and east abutments are presented in Figures GD5-13 and GD5-14, respectively. The results are presented as plots of driving resistance (blows/foot) versus ultimate pile capacity, maximum compression and tensile stress in the steel, energy at the pile tip, and hammer stroke. Efficient pile driving can be defined as driving the pile to the desired ultimate capacity at a reasonable blow count of 100 blows per foot or less, and as close but not exceeding 90 percent of the yield strength of the pile material.

The WEAP results indicate that the Delmag D 36 hammer could drive the 16-inch closed end pipe piles with reasonable driving resistance of about 40 blows per foot, without exceeding the maximum allowable compression stresses of the steel piles (90 percent of 50 ksi). Based on these results, we recommend that the piles be considered to have reached refusal when the driving resistance reaches at least 3 blows per inch (bpi) for the last 6 inches of driving the piles. The hammer stroke of the Delmag D 36 should be at least 8 feet.

The WEAP results indicate that the other two hammers considered, IHC S-70 and IHC S-90 should be able to drive the piles to the required ultimate capacity but would exceed the allowable compressive stress of the pile material.

The following sections provide recommendations for monitoring and testing. For the LRFD resistance factors that are being used for pile design, Pile-Driving Analyzer (PDA) and Case Pile Wave Analysis Program (CAPWAP) are required.

5.1.2 Test Pile Program

The recommended pile capacities and corresponding penetrations are based on theoretical and empirical data, subsurface conditions encountered at the site in limited number of borings, and our engineering judgment and experience. To substantiate our recommendations, and be able to use a resistance factor of 0.65 per the AASHTO LRFD, we recommend that a test pile program be undertaken. The test pile program could consist of driving indicator piles and performing dynamic pile tests using a PDA. We recommend a minimum of four piles per abutment be driven as indicator piles. This is the minimum number of piles to be tested according to AASHTO LRFD Table 10.5.5.2.3-3 for 26 to 51 piles per site of low variability of soil conditions. During indicator pile driving, we recommend that dynamic measurements using a PDA be taken and a Case Pile Wave Analysis Program be performed on each test pile. Based on our experience, dynamic pile tests are a cost-effective method for determining the total ultimate pile capacities and load distributions. Test piles may be used as production piles if they meet the specified installation procedures and requirements.

5.1.3 Monitoring Pile Driving

Shannon & Wilson should observe and evaluate all pile driving by making a continuous driving record of each pile. For this purpose, the Contractor should mark the pile in 1-foot increments. During restrike, additional 1-inch increments between the 1-foot marks would be required.

The pile-driving record would include hammer stroke (diesel hammers), blows per foot, time, date, reasons for delays, and other pertinent information. In addition, the record would include tip elevation, specified criteria, and the initials of inspectors making final acceptance of the pile. The pile-driving records should be reviewed on a daily basis by Shannon & Wilson.

5.1.4 Pile Driving Vibrations and Movement Monitoring

We recommend developing and implementing vibration criteria for the existing forced sewer pipeline and waterline west and east of the bridge. The criteria should consider the type and frequency of the vibrations and the existing condition of the pipelines. Prior to construction, we recommend that threshold vibrations levels be determined and, if appropriate, vibration monitors be placed on the pipelines to monitor vibration during construction. Developing vibration threshold criteria and monitoring vibrations during construction will aid in assessing the need for mitigating measures, as well as resolving potential disputes.

5.2 Fill and Earthwork Recommendations

In general, the proposed embankment project will require between about 2 and 18 feet of permanent structural fill and up to approximately 6.5 feet of surcharge plus overbuild fills. Prior to the placing fills, the subgrade will need to be properly prepared in order to receive the new embankment material. The following sections present our recommendations for site preparation and grading, structural fill placement and compaction, and construction operations during wet weather.

5.2.1 Site Preparation and Grading

Subgrade soil that will support fills and roadway structures should be prepared in accordance with UDOT Standard Specifications and the special provision Section 02231 with some modifications. We recommend removing any snow, debris, brush and other material that would impede fill placement. In areas where very soft clay underlies a thin desiccated crust, existing grass may be left in place and upper desiccated crust should not be disturbed to provide initial working surface. Any surface water or groundwater should be drained or pumped from areas requiring grading.

Upon completion of excavation to obtain desired grades, the exposed surface should be prepared for placement of fill in accordance with UDOT Standard Specifications Section 02056. Should proof-rolling reveal the presence of soft zones, they should either be removed and replaced with structural fill or, as described in Part 3.2 of UDOT Standard Specification Section 02056, a working platform may be constructed across soft, wet ground. The use of a geogrid (e.g., Tensar BX1100 or similar) across the soft, wet ground could facilitate construction of the working platform. Alternatively, a 2-foot lift of granular fill with a maximum 6-inch particle size could be placed, and then tracked and wheel rolled in place. Subsequent lifts of structural fill should be placed and compacted in accordance with UDOT Standard Specifications.

5.2.2 Backfill for Structures and Pavement Subgrade

All fill beneath structures within 150 feet of bridge abutments, within 2 feet of pavement subgrades, and other areas where settlements are to be minimized or backfill will be relied upon for passive resistance should be densely compacted structural fill. Structural fill soil should consist of a well-graded sand and gravel of on-site or imported granular soil, free of organic debris, rubbish, and contaminants.

We recommend using granular fill that contains not more than 15 percent fines (material passing the No. 200-mesh sieve, based on the minus -¾ -inch fraction). Our recommendations for structural fill generally follow that for granular borrow in Part 2.3, Section 02056, of the UDOT Standard Specifications, and the moisture content of the soil should be within ±2 percent of its optimum to allow proper compaction. All structural fill material should have a maximum particle size smaller than 3 inches. Existing embankment fill material may be reused provided they met these criteria. Fill material should not contain frozen soil or be placed on frozen ground. Fill soil should have a temperature of at least 35 degrees when it is placed during freezing conditions to reduce the possibility of freezing before it is compacted.

If fill placement occurs during wet weather, wet conditions or below the water level in any excavation, structural fill material should consist of aggregate that conforms to UDOT granular borrow specifications and for Contractor convenience has not more than 5 percent fines by weight passing the No. 200 mesh sieve based on a wet sieve analysis of the fraction passing the ¾-inch sieve. The fines should be nonplastic and the fill material should have a maximum particle size smaller than 3 inches.

Structural fill should be constructed in accordance with Part 3.3 and 3.4, Section 02056, of the UDOT Standard Specifications, as appropriate.

If subgrade or fill soils become loosened or disturbed, additional excavation to expose competent, undisturbed soils and replacement with properly compacted structural fill will be required.

5.2.3 Common Embankment and Surcharge Fill

Our recommendations for material to be used for common embankment and surcharge fill construction generally follow UDOT Standard Specification Section 02056, Part 3.2 (UDOT, 2008). However, during wet or freezing conditions we do not recommend the use of materials classified as A-4 in AASHTO M 145, which can be difficult to condition and compact during wet or freezing conditions.

Common embankment fill should constructed be in accordance with Part 3.3, Section 02056, of the UDOT Standard Specifications.

If subgrade or fill soils become loosened or disturbed, additional excavation to expose competent, undisturbed soils and replacement with properly compacted structural fill will be required.

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TABLE GD5-1
ESTIMATED SOIL PROPERTIES
STATION 600+41 TO 606+00

Soil Type	Top Depth (ft)	Bottom Depth (ft)	Thickness (ft)	Unit Weight (pcf)	Elastic or Consolidation Settlement	Elastic Modulus (ksf)		OCR	$C_{c\varepsilon}$		C_v (ft^2/month)		$C_{\alpha\varepsilon}$	Continiously Drained Layers
						Loading	Unloading/ Reloading		Loading	Unloading/ Reloading	Loading	Unloading/ Reloading	Loading	
Stiff silty CLAY	0	10	10	110	Consolidation	-	-	4	0.16	0.016	9	16	0.006	-
Loose to m. dense clayey SAND	10	16	6	110	Elastic	200	200	-	-	-	-	-	-	X
Stiff silty CLAY	16	24	8	105	Consolidation	-	-	2	0.18	0.018	9	16	0.007	-
M. stiff silty CLAY to clayey SILT	24	35	11	105	Consolidation	-	-	2	0.16	0.016	35	61	0.003	-
Dense sandy GRAVEL	35	43	8	125	Elastic	1000	1000	-	-	-	-	-	-	X

Notes:

1) Soil profile is based on SWB-201, SWT-205, and B-01.

2) Groundwater depth is assumed at 16 feet.

TABLE GD5-2
ESTIMATED SOIL PROPERTIES
STATION 621+00 TO WEST ABUTMENT

Soil Type	Top Depth (ft)	Bottom Depth (ft)	Thickness (ft)	Unit Weight (pcf)	Elastic or Consolidation Settlement	Su (psf)	ϕ (deg.)	Elastic Modulus (ksf)		OCR	$C_{c\varepsilon}$		C_v (ft^2/month)		C_h (ft^2/month)	$C_{\alpha\varepsilon}$	Continiously Drained Layers
								Loading	Unloading/ Reloading		Loading	Unloading/ Reloading	Loading	Unloading/ Reloading		Loading	
M. dense clayey SAND	0	6	6	120	Elastic	1000	32	350	450	-	-	-	-	-	-	-	X
Soft CLAY	6	14	8	105	Consolidation	450	-	-	-	2	0.18	0.018	10	17.5	12	0.007	
Loose to m. dense SAND	14	16	2	100	Elastic	-	30	250	350	-	-	-	-	-	-	-	X
M. stiff to stiff CLAY	16	26	10	100	Consolidation	800	-	-	-	2	0.16	0.016	14	24.5	16	0.006	
M. dense SAND	26	27.5	1.5	120	Elastic	-	33	400	500	-	-	-	-	-	-	-	X
M. stiff to stiff CLAY	27.5	40	12.5	110	Consolidation	1500	-	-	-	2	0.18	0.018	6	10.5	8	0.007	
M. dense SAND	40	44	4	120	Elastic	-	34	400	500	-	-	-	-	-	-	-	X
M. stiff CLAY	44	51	7	110	Consolidation	2100	-	-	-	2	0.16	0.016	32	56	32	0.003	
V. dense clayey SAND	51	70	19	130	Elastic	-	36	800	950	-	-	-	-	-	-	-	X
V. dense sandy GRAVEL and gravelly SAND	70	80	10	140	Elastic	-	40	1000	1000	-	-	-	-	-	-	-	X

Notes:
1) Soil profile is based on SWC-211, SWC-220, and B-3.
2) Groundwater Depth is assumed at 8 feet (Elev. 4490 feet).

TABLE GD5-3
ESTIMATED SOIL PROPERTIES
EAST ABUTMENT TO STATION 630+00

Soil Type	Top Depth (ft)	Bottom Depth (ft)	Thickness (ft)	Unit Weight (pcf)	Elastic or Consolidation Settlement	Su (psf)	ϕ (deg.)	Elastic Modulus (ksf)		OCR	$C_{c\varepsilon}$		C_v (ft^2/month)		C_h (ft^2/month)	$C_{\alpha\varepsilon}$	Continiously Drained Layers
								Loading	Unloading/ Reloading		Loading	Unloading/ Reloading	Loading	Unloading/ Reloading		Loading	
V. stiff to hard CLAY	0	6	6	115	Consolidation	1000		-	-	1.5	0.16	0.016	14	25	16	0.006	-
Clayey, silty SAND	6	8	2	120	Elastic	-	30	200	300	-	-	-	-	-	-	-	X
Soft CLAY	8	25	17	100	Consolidation	500	-	-	-	2	0.16	0.016	14	25	16	0.006	-
M. stiff to stiff CLAY	25	34	9	110	Consolidation	1500	-	-	-	2	0.18	0.018	6	11	8	0.007	-
M. dense SAND	34	36	2	115	Elastic	-	34	250	375	-	-	-	-	-	-	-	X
M. stiff CLAY	36	42	6	110	Consolidation	2100	-	-	-	2	0.16	0.016	32	56	32	0.003	-
M. stiff CLAY	42	56	14	110	Consolidation	2100	-	-	-	2	0.16	0.016	32	56	32	0.003	-
V. dense clayey SAND	56	68	12	130	Elastic	-	36	800	950	-	-	-	-	-	-	-	X
V. dense sandy GRAVEL and gravelly SAND	68	75	7	140	Elastic	-	40	1000	1000	-	-	-	-	-	-	-	X

Notes:

1) Soil profile is based on SWB-202 and SWT-219.

2) Groundwater Depth is assumed at 4 feet (Elevation 4490 feet).

TABLE GD5-4
ESTIMATED SOIL PROPERTIES
STATION 630+00 TO 640+50

Soil Type	Top Depth (ft)	Bottom Depth (ft)	Thickness (ft)	Unit Weight (pcf)	Elastic or Consolidation Settlement	Elastic Modulus (ksf)		OCR	$C_{c\varepsilon}$		C_v (ft^2/month)		$C_{\alpha\varepsilon}$	Continiously Drained Layers
						Loading	Unloading/ Reloading		Loading	Unloading/ Reloading	Loading	Unloading/ Reloading	Loading	
Loose silty SAND	0	2	2	130	Elastic	200	200	-	-	-	-	-	-	X
Soft silty CLAY	2	15	13	105	Consolidation	-	-	1.5	0.2	0.02	6	11	0.007	-
D. silty SAND	15	18	3	125	Elastic	600	600	-	-	-	-	-	-	X
M. stiff silty CLAY	18	31	13	105	Consolidation	-	-	2	0.2	0.02	6	11	0.007	-
Sandy SILT to silty SAND	31	38	7	110	Elastic	300	300	-	-	-	-	-	-	X
M. stiff sandy silty CLAY	38	47	9	115	Consolidation	-	-	2	0.19	0.019	16	28	0.007	-
M. dense silty SAND	47	51	4	110	Elastic	400	400	-	-	-	-	-	-	X
M. stiff silty CLAY	51	55	4	105	Consolidation	-	-	2	0.195	0.0195	7	12	0.007	-
M. dense silty SAND	55	56	1	120	Elastic	400	400	-	-	-	-	-	-	X
Stiff silty CLAY	56	64	8	110	Consolidation	-	-	2.5	0.195	0.0195	7	12	0.004	-
D. sandy GRAVEL	64	70	6	125	Elastic	1000	1000	-	-	-	-	-	-	X

Notes:

1) Soil profile is based on SWC-213, SWC-212, and SWC-218.

2) Groundwater depth is assumed at 4 feet.

TABLE GD5-5
ESTIMATED SOIL PROPERTIES
STATION 640+50 TO 649+00

Soil Type	Top Depth (ft)	Bottom Depth (ft)	Thickness (ft)	Unit Weight (pcf)	Elastic or Consolidation Settlement	Elastic Modulus (ksf)		OCR	$C_{c\varepsilon}$		C_v (ft^2/month)		$C_{\alpha\varepsilon}$	Continiously Drained Layers
						Loading	Unloading/ Reloading		Loading	Unloading/ Reloading	Loading	Unloading/ Reloading	Loading	
V. loose silty SAND	0	2	2	120	Elastic	200	200	-	-	-	-	-	-	X
Stiff silty CLAY	2	5	3	110	Consolidation	-	-	3	0.185	0.0185	15	26	0.007	-
M. dense silty SAND	5	8	3	120	Elastic	400	400	-	-	-	-	-	-	X
M. stiff silty CLAY	8	22	14	110	Consolidation	-	-	3	0.185	0.0185	15	26	0.007	-
M. dense SILT	22	24	2	100	Elastic	200	200	-	-	-	-	-	-	X
V. soft silty CLAY	24	30	6	100	Consolidation	-	-	2.5	0.195	0.0195	8	14	0.007	-
Soft silty CLAY	30	45	15	100	Consolidation	-	-	2	0.195	0.0195	8	14	0.007	-
Silty SAND	45	47	2	100	Elastic	200	200	-	-	-	-	-	-	X
Clayey SILT	47	58	11	100	Consolidation	-	-	2	0.14	0.014	45	79	0.003	-
D. sandy GRAVEL	58	65	7	125	Elastic	1000	1000	-	-	-	-	-	-	X

Notes:
1) Soil profile is based on SWC-214, SWT-207, and SWC-214.
2) Groundwater depth is assumed at 4 feet.

TABLE GD5-6
ESTIMATED SOIL PROPERTIES
STATION 649+00 TO 723+50 AND STATION 606+00 TO 621+00

Soil Type	Top Depth (ft)	Bottom Depth (ft)	Thickness (ft)	Unit Weight (pcf)	Elastic or Consolidation Settlement	Elastic Modulus (ksf)		OCR	$C_{c\varepsilon}$		C_v (ft^2/month)		$C_{\alpha\varepsilon}$	Continiously Drained Layers
						Loading	Unloading/ Reloading		Loading	Unloading/ Reloading	Loading	Unloading/ Reloading	Loading	
M. dense silty SAND	0	3	3	125	Elastic	200	200	-	-	-	-	-	-	X
Stiff CLAY	3	5	2	120	Consolidation	-	-	2.5	0.16	0.016	11	19	0.006	-
M. dense silty SAND	5	7	2	120	Elastic	400	400	-	-	-	-	-	-	X
M. Stiff CLAY	7	24	17	110	Consolidation	-	-	2.5	0.16	0.016	11	19	0.006	-
Sandy SILT	24	25	1	110	Elastic	200	200	-	-	-	-	-	-	X
M. Stiff CLAY	25	31	6	110	Consolidation	-	-	2.5	0.16	0.016	11	19	0.006	-
V. soft CLAY	31	51	20	100	Consolidation	-	-	2.3	0.18	0.018	9	16	0.007	-
V. soft CLAY	51	71	20	100	Consolidation	-	-	1.6	0.19	0.019	7	12	0.007	-
M. stiff CLAY	71	92	21	95	Consolidation	-	-	1.6	0.19	0.019	7	12	0.007	-
M. dense to dense sandy GRAVEL	92	95	3	115	Elastic	1000	1000	-	-	-	-	-	-	X

Notes:

1) Soil profile is based on SWB-204, SWC-215, SWT-208, SWT-209, and SWC-216.

2) Groundwater depth is assumed at 4 feet.

TABLE GD5-7
SETTLEMENT ESTIMATES AND RECOMMENDED SETTLEMENT MITIGATION

Stations	Proposed Embankment Height[1] (feet)	Estimated Total Settlement (inches)	PVDs	Surcharge + Overbuild[2] Thickness (ft) / Stress (ksf)		Duration (month)
				Start	End	
600+41 to 606+00	4 – 8	4 - 9		-	-	3
606+00 to 620+00	2 – 4	< 2		-	-	0
620+00 to 622+00	4 – 9	2 - 6		-	-	3
622+00 to West Bridge Abutment[3]	9 -12	8 -10	Yes	4.0 / 0.52	4.0 / 0.52	3
West Bridge Abutment to 30 feet[3]	Slope fill down at 1.5H: 1 V from abutment	varies	Yes	4.0 / 0.52	0	3
West edge of trail/ levee to east bridge abutment[4]	Slope fill up at 1.5H: 1V from west edge of trail/ levee	varies	Yes	0	varies	3
East bridge abutment[4] to 626+50	17 - 18	31 - 32	Yes	6.5 / 0.85	6.5 / 0.85	3
626+50 to 627+75	12 - 17	20 - 31	Yes	5 /0.65	5 /0.65	3
627+75 to 628+75	8 - 12	12 - 20	Yes	4.5 / 0.6	4.5 / 0.6	3
628+75 to 629+00	8	12		4.5 / 0.6	4.5 / 0.6	3
629+00 to 636+50	5 - 8	10 - 12		4 / 0.52	4 / 0.52	3
636+50 to 640+00	5 - 7	10-13		3.5 / 0.46	3.5 / 0.46	3
640+00 to 649+00	7 - 9	5 - 6		-	-	3
649+00 to 653+50	6 - 7	3 - 5		2 / 0.26	2 / 0.26	3
653+50 to 663+50	4 - 6	2 - 4		-	-	3
663+50 to 723+50	2 -4	< 2		-	-	0
723+50 to 725+00	8 - 10	-[4]		-[5]	-[5]	-[5]

Notes:
1. To finished grade.
2. Measured from finished grade.
3. Start of the full surcharge plus overbuild height (4.0 feet) will vary based on where the 1.5H:1V slope from the ordinary high water intersects with the full surcharge + overbuild elevation.
4. Start of the full surcharge plus overbuild height (6.5 feet) will vary based on where the 1.5H:1V slope from the trail/levee intersects with the full surcharge + overbuild elevation. At some locations the intersection will occur east of the abutment.
5. Surcharge for Dry Creek similar to recommendations provided by Terracon. Surcharge will not be over the full width of the embankment.
- Means no settlement mitigation or embankment preload as described above.
ft = feet
ksf = kips per square foot
PVDs = prefabricated vertical drains

TABLE GD5-8
WEST ABUTMENT APPROACH SLAB LONGITUDINAL SETTLEMENT ESTIMATES

Construction Stage		Approach Slab Length	
		30 Feet (Slab to 622+86)	**60 Feet (Slab to 622+56)**
1	Post-Construction Total Settlement (inches), (Slab Construction to Start of Warranty)[1]	~0.4	~0.4
2	Warranty Period Total Settlement (inches), (5 years)	0.6	0.6
3	Warranty Allowable Longitudinal Settlement[2]	0.25 inch / 30 feet	0.5 inch / 60 feet
4	Required Overbuild Thickness (inches)	1.0 - 0.25 = 0.75	1.0 - 0.5 = 0.5
5	Differential Grade at Start of Warranty (inches)	0.75 - 0.4 = 0.35	0.5 - 0.4 = 0.1
	Considerations	0.35 inch / 30 feet exceeds warranty limits by 0.1 inch across the slab length.	

Notes:
1. Total post-construction settlement is primarily composed of secondary settlement and, therefore, is a function of time. To reduce post-construction settlement, we assumed the approach slabs would be constructed eight months before the warranty period begins.
2. Based on the warranty requirements, as specified in the Request for Proposal. Magnitude of allowable longitudinal settlement assumed for differential grade at start of warranty (row 5).

TABLE GD5-9
EAST ABUTMENT APPROACH SLAB LONGITUDINAL SETTLEMENT ESTIMATES

	Construction Stage	Approach Slab Length	
		30 Feet (Slab to 625+30)	**30 and 60 Feet (Slab to 625+60)**
1	Post-Construction Total Settlement (inches), (Slab Construction to Start of Warranty)[1]	~0.2	~0.6
2	Warranty Period Total Settlement (inches), (5 years)	0.6	1.1
3	Warranty Allowable Longitudinal Settlement[2]	0.25 inch / 30 feet	0.5 inch / 60 feet
4	Required Overbuild Thickness (inches)	0.2 + 0.6 - 0.25 = 0.55	0.6 + 1.1 – 0.5 = 1.2
5	Differential Grade at Start of Warranty (inches)	0.55 – 0.2 = 0.35	**60 foot Slab:** 1.2 – 0.6 = 0.6 **30 foot Slab:** 1.2 – 0.6 - 0.35 = 0.25
	Considerations	0.35 inch / 30 feet exceeds warranty limits by 0.1 inch across slab length.	**60 foot Slab:** 0.6 inch / 60 feet exceeds warranty limits by 0.1 inch across slab length. **30 foot Slab:** 0.25 inch / 30 feet is at warranty limits.

Notes:
1. Total post-construction settlement is primarily composed of secondary settlement and therefore is a function of time. To reduce post-construction settlement, we assumed the approach slabs would be constructed eight months before the warranty period begins.
2. Based on the warranty requirements, as specified in the Request for Proposal. Magnitude of allowable longitudinal settlement assumed for differential grade at start of warranty (row 5).

TABLE GD5-10
JORDAN RIVER BRIDGE
RECOMMENDED PARAMETERS FOR LATERAL RESISTANCE ANALYSIS USING LPILE[PLUS]

Abutment	Representative Explorations[1]	Ground Surface Elevation (feet)/ Depth to Groundwater (feet)	LPILE Soil Type	Depth of Soil Layer (feet)	Total Unit Weight[2], γ (pcf)	ε_{50}	Static: Assumed Friction Angle, ϕ (degrees)	Static: Cohesion, c (psf)	Static: Initial Modulus of Subgrade Reaction, k (pci)	Dynamic – Liquefied or Strength Reduced: Assumed Friction Angle, ϕ (degrees)	Dynamic – Liquefied or Strength Reduced: Cohesion, c (psf)	Dynamic – Liquefied or Strength Reduced: Initial Modulus of Subgrade Reaction, k (pci)
West Abutment	B-3 SWC-211 SWC-220	4497/ 7	Sand (Reese)	0 - 7	120	-	34	-	85	34	-	85
			Soft Clay	7 - 14	105	0.02	-	520	60	-	440	50
			Sand (Reese)	14- 16	110	-	30	-	10	-	200	2
			Stiff Clay w/ free water	16- 26	105	0.01	-	800	100	-	800	100
			Stiff Clay w/ free water	26 - 40	110	0.013	-	1,000	240	-	1,000	240
			Sand (Reese)	40 - 44	120	-	34	-	70	-	850	30
			Stiff Clay w/ free water	44 - 51	110	0.008	-	2,000	280	-	2,000	280
			Sand (Reese)	51 - 70	130	-	39	-	125	36	-	125
			Sand (Reese)	70 - 80	140	-	42	-	200	40	-	200
East Abutment	SWB-202 B-8	4494 / 4	Sand (Reese)*	0 - 4	130	-	36	-	140	36	-	140
			Sand (Reese)*	4 - 8	130	-	36	-	85	36	-	85
			Soft Clay	8 - 25	105	0.02	-	700	100	-	600	85
			Stiff Clay w/ free water	25 - 34	110	0.008	-	1,200	280	-	1,200	280
			Sand (Reese)	34 - 36	115	-	34	-	25	-	400	5
			Stiff Clay w/ free water	36 - 56	110	0.009	-	2,000	160	-	2,100	160
			Sand (Reese)	56 - 68	130	-	39	-	125	36	-	125
			Sand (Reese)	68 - 75	140	-	42	-	200	40	-	200

Notes:
1. Subsurface information and recommended geotechnical parameters are based on the results of above indicated field explorations.
2. LPile requires effective unit weight of the soil layer. Effective unit weight = total unit weight – unit weight of water. $\gamma' = \gamma t - \gamma w$, where, $\gamma w = 62.4$ pcf
The values on the table above assume that the foundation consists of a single pile or drilled shaft, no lateral efficiency factor was considered due to group effect.
If applicable, modifications to the p-y curves for sloping ground conditions should be determined in accordance with LPILE [PLUS] (2004) manual.
* Improved sand layer.
pcf = pounds per cubic foot ; pci = pounds per cubic inch; psf= pounds per square foot

TABLE GD5-11
POST AND PANEL WALLS - RECOMMENDED PARAMETERS FOR LATERAL RESISTANCE ANALYSIS USING LPILE^PLUS

Exploration[1]	Depth to Groundwater (feet)	Wall Stations		LPILE Soil Type	Depth of Soil Layer (feet)	Total Unit Weight[2], γ (pcf)	ε_{50}	Static			Dynamic – Liquefied or Strength Reduced		
		From	To					Cohesion, c (psf)	Assumed Friction Angle, ϕ (degrees)	Initial Modulus of Subgrade Reaction, k (pci)	Cohesion, c (psf)	Assumed Friction Angle, ϕ (degrees)	Initial Modulus of Subgrade Reaction, k (pci)
SWB-204	6*	650+00	655+00	Sand (Reese)	0.0 – 6.0	115	-	-	32	50	-	32	50
				Stiff Clay w free water	6.0 – 20.0**	115	0.009	1,000	-	275	1,000	-	275
SWC-215	6	655+00	660+00	Stiff Clay w/o free water	0.0 – 3.0	105	0.017	500	-	60	500	-	60
				Sand (Reese)	3.0 – 6.0	120	-	-	30	40	-	30	40
				Stiff Clay w/ free water	6.0 – 12.0	115	0.008	1,200	-	350	1,200	-	350
				Stiff Clay w/ free water	12.0 – 20.0**	115	0.009	1,000	-	275	1,000	-	275

Notes:

1. Subsurface information and recommended geotechnical parameters are based on results of above indicated field explorations.
2. LPile requires effective unit weight of the soil layer. Effective unit weight = total unit weight – unit weight of water ($\gamma' = \gamma t - \gamma w$, where, $\gamma w = 62.4$ pcf).
3. The values on the table above assume that the foundation consists of a single pile or drilled shaft; no lateral efficiency factor was considered due to group effect.
4. If applicable, modifications to the p-y curves for sloping ground conditions should be determined in accordance with LPILE^PLUS (2004) manual.

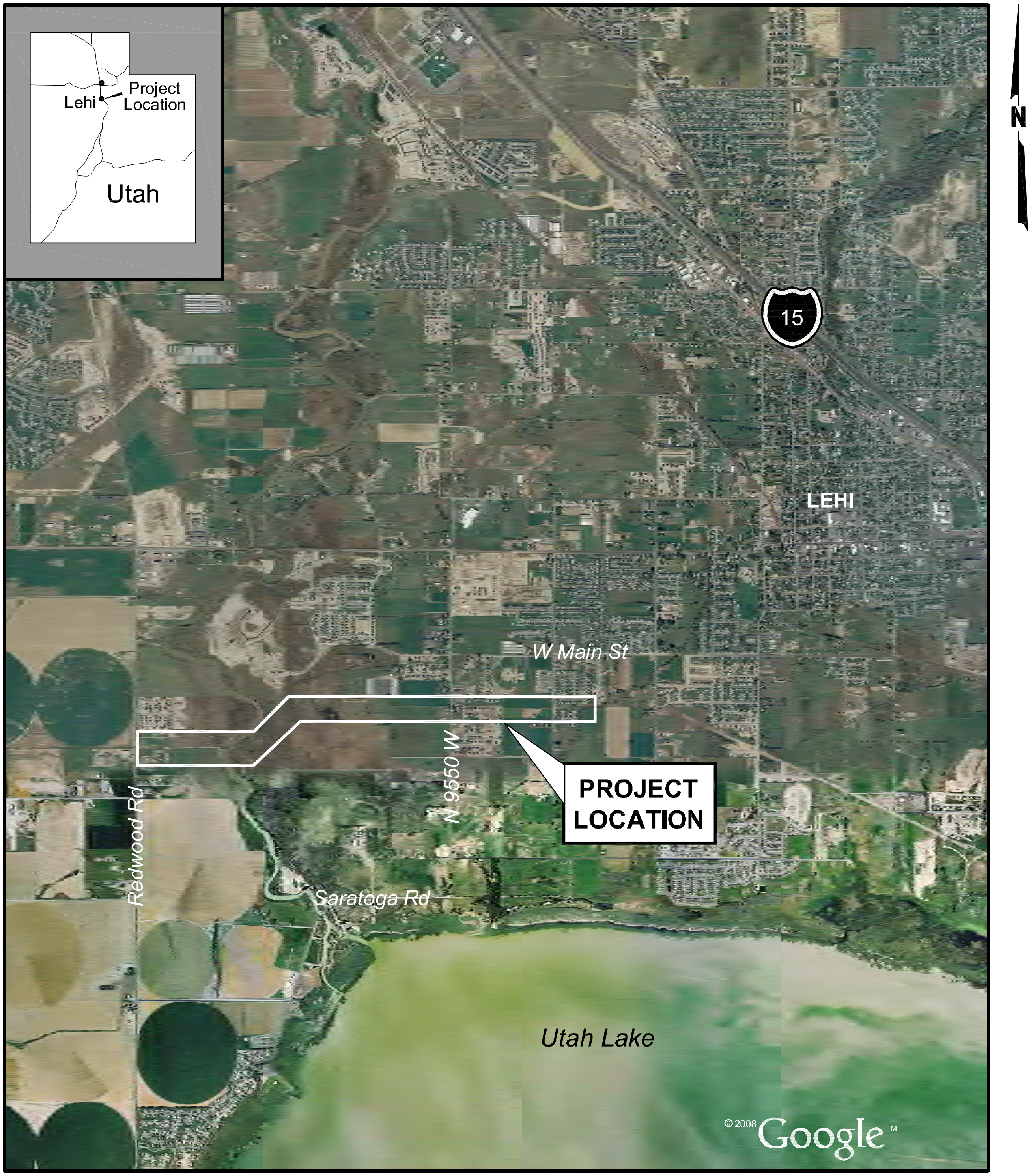
* Groundwater was not encountered during drilling. Depth was estimated from nearby cone penetrometer tests

** Sensitive clay below 20 feet. Please consult with Shannon & Wilson if foundation extends below 20 feet

pcf = pounds per cubic foot

pci = pounds per cubic inch

psf = pounds per square foot



0 4000 8000

Approximate Scale in Feet

NOTE

Map adapted from aerial imagery provided by Google Earth Pro, Image © 2008 DigitalGlobe, and Image State of Utah reproduced by permission granted by Google Earth ™ Mapping Service.

Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

VICINITY MAP

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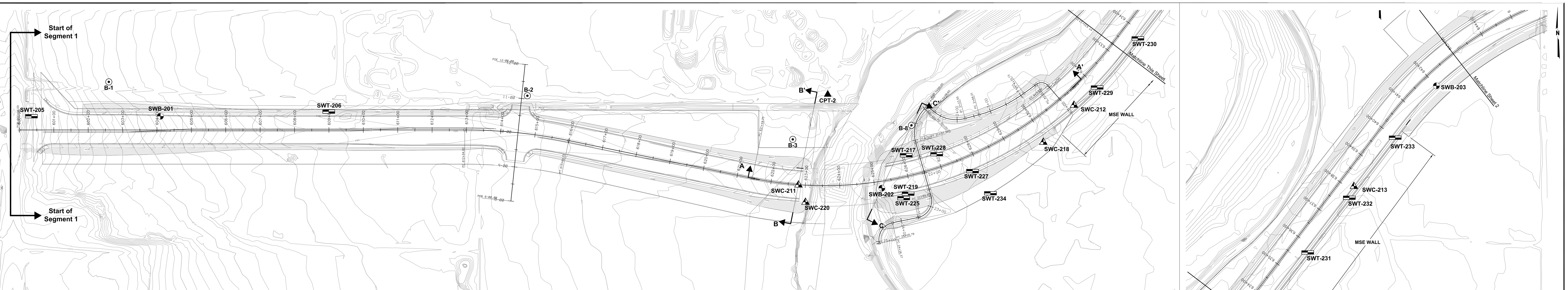
FIG. GD5-1

Filename: J:\231\01178-010\23-1-01178-010 Fig 1.dwg Date: 02-25-2009 Login: cnt



LEGEND

B-1	⊙	Baseline Boring Designation and Approximate Location by Terracon
CPT-1	▲	Baseline CPT Designation and Approximate Locationby Terracon
SWB-1		Boring Designation and Approximate Location
SWC-1		CPT Designation and Approximate Location
SWT-1		Test Pit Designation and Approximate Location

0 100 200
Scale in Feet

NOTE

Figure adapted from file *6697_S1_Design* provided by Utah Department of Transportation, received 12/18/2008.

Filename: J:\231\01178-010\23-1-01178-010 Plan.dwg Date: 02-25-2009 Login: cnt

Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

SITE AND EXPLORATION PLAN

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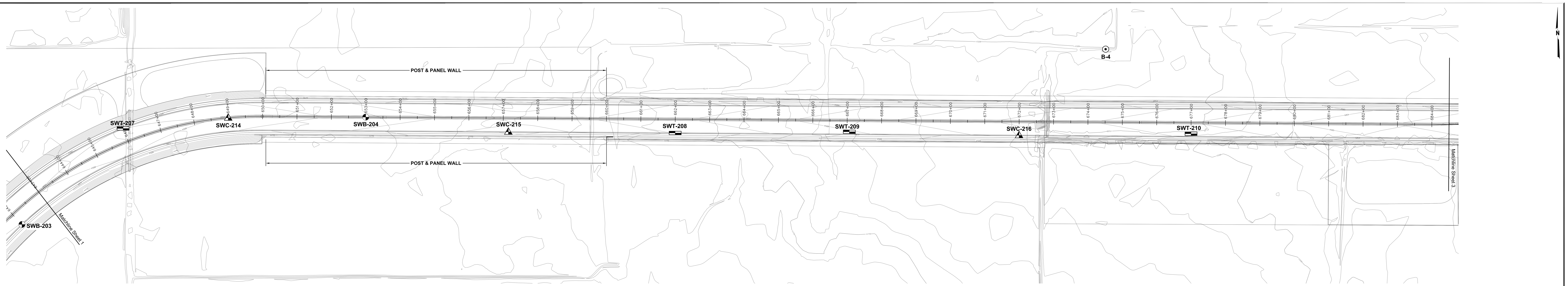
FIG. GD5-2
Sheet 1 of 3



LEGEND

B-1	⊙	Baseline Boring Designation and Approximate Location by Terracon
CPT-1	▲	Baseline CPT Designation and Approximate Locationby Terracon
SWB-1		Boring Designation and Approximate Location
SWC-1		CPT Designation and Approximate Location
SWT-1		Test Pit Designation and Approximate Location

0 100 200
Scale in Feet

NOTE

Figure adapted from file *6697_S1_Design* provided by Utah Department of Transportation, received 12/18/2008.

Filename: J:\231\01178-010\23-1-01178-010 Plan.dwg Date: 02-25-2009 Login: cnt

Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

SITE AND EXPLORATION PLAN

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FIG. GD5-2
Sheet 2 of 3

LEGEND

B-1	⊙	Baseline Boring Designation and Approximate Location by Terracon
CPT-1	▲	Baseline CPT Designation and Approximate Locationby Terracon
SWB-1		Boring Designation and Approximate Location
SWC-1		CPT Designation and Approximate Location
SWT-1		Test Pit Designation and Approximate Location

0 100 200
Scale in Feet

NOTE

Figure adapted from file *6697_S1_Design* provided by Utah Department of Transportation, received 12/18/2008.

Filename: J:\231\01178-010\23-1-01178-010 Plan.dwg Date: 02-25-2009 Login: cnt

Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

SITE AND EXPLORATION PLAN

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FIG. GD5-2
Sheet 3 of 3


SWT-205
B-01
PROPOSED GRADE
Existing Ground Surface
SWB-201
SWT-206
B-02
SWC-211
B-03
SWC-220
SWB-202
SWT-219
SWT-225
SWT-217
SWT-228
SWT-227
SWT-234
SWC-218
SWC-212
SWT-229
SWT-230
SWT-231
SWC-213
SWC-232
SWT-233
SWB-203
MATCHLINE
LEGEND
Horizontal Scale in Feet
Vertical Scale in Feet
Vertical Exaggeration = 10X
Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah
GENERALIZED SUBSURFACE
PROFILE ALONG ALIGNMENT
February 2009
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SHANNON & WILSON, INC.
FIG. GD5-3


SWT-207
SWC-214
PROPOSED GRADE
Existing Ground Surface
SWB-204
SWC-215
SWT-208
SWT-209
SWC-216
B-04
SWT-210
MATCHLINE
LEGEND
Horizontal Scale in Feet
Vertical Scale in Feet
Vertical Exaggeration = 10X
Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah
GENERALIZED SUBSURFACE
PROFILE ALONG ALIGNMENT
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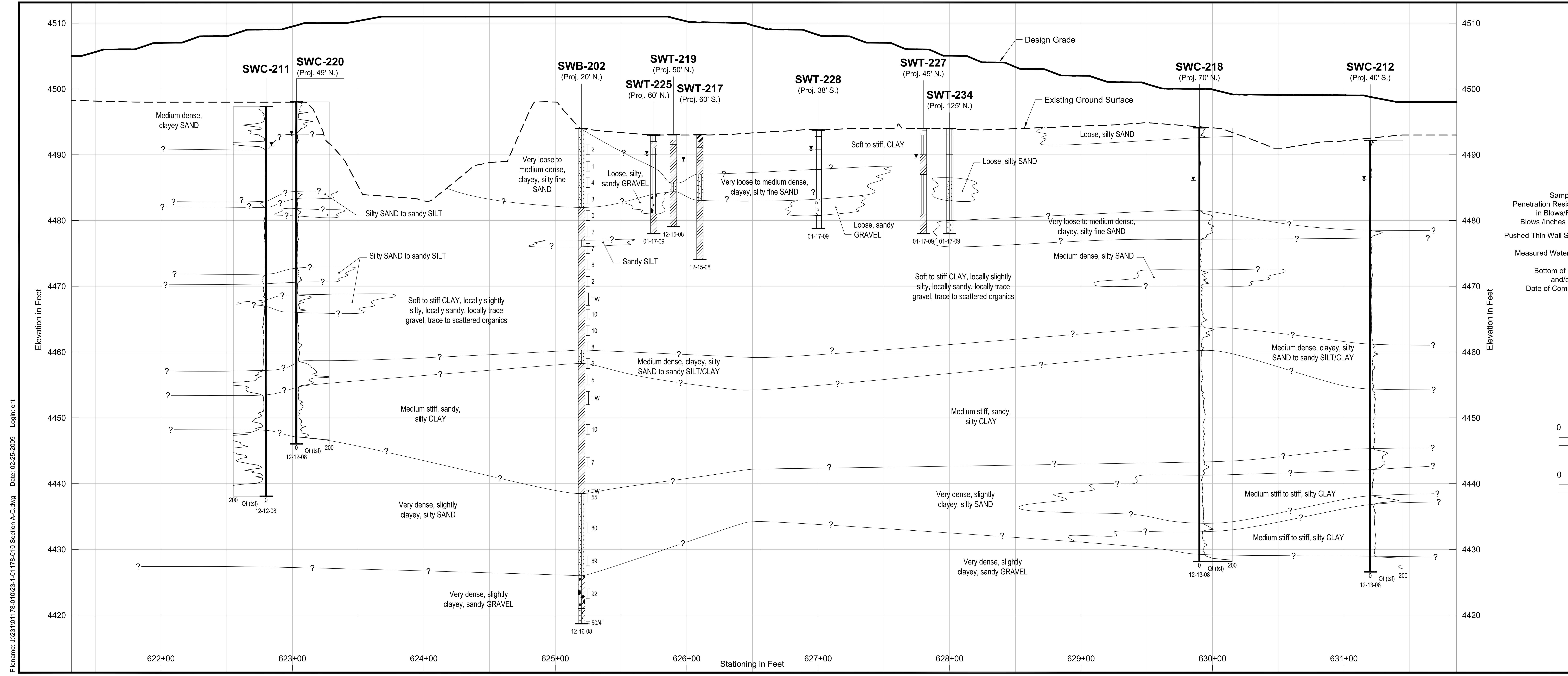
SHANNON & WILSON, INC.
FIG. GD5-3


SWC-221
B-05
SWC-222
PROPOSED GRADE
Existing Ground Surface
SWC-223
B-10
SWC-224
MATCHLINE
LEGEND
Horizontal Scale in Feet
Vertical Scale in Feet
Vertical Exaggeration = 10X
Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah
GENERALIZED SUBSURFACE
PROFILE ALONG ALIGNMENT
February 2009
23-1-01178-010
SHANNON & WILSON, INC.
FIG. GD5-3



Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

GENERALIZED SUBSURFACE PROFILE A-A'

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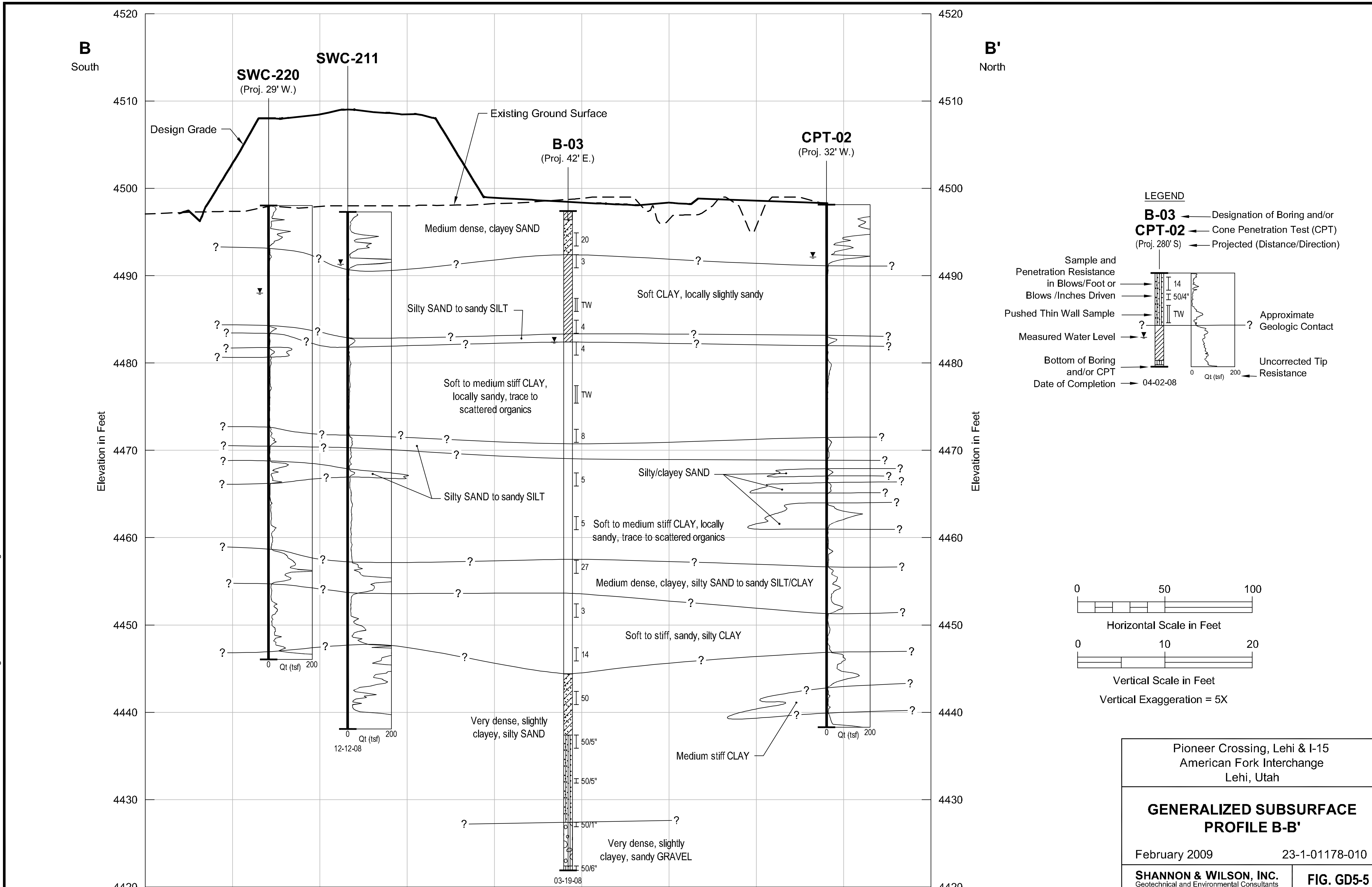
FIG. GD5-4



Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

GENERALIZED SUBSURFACE PROFILE B-B'

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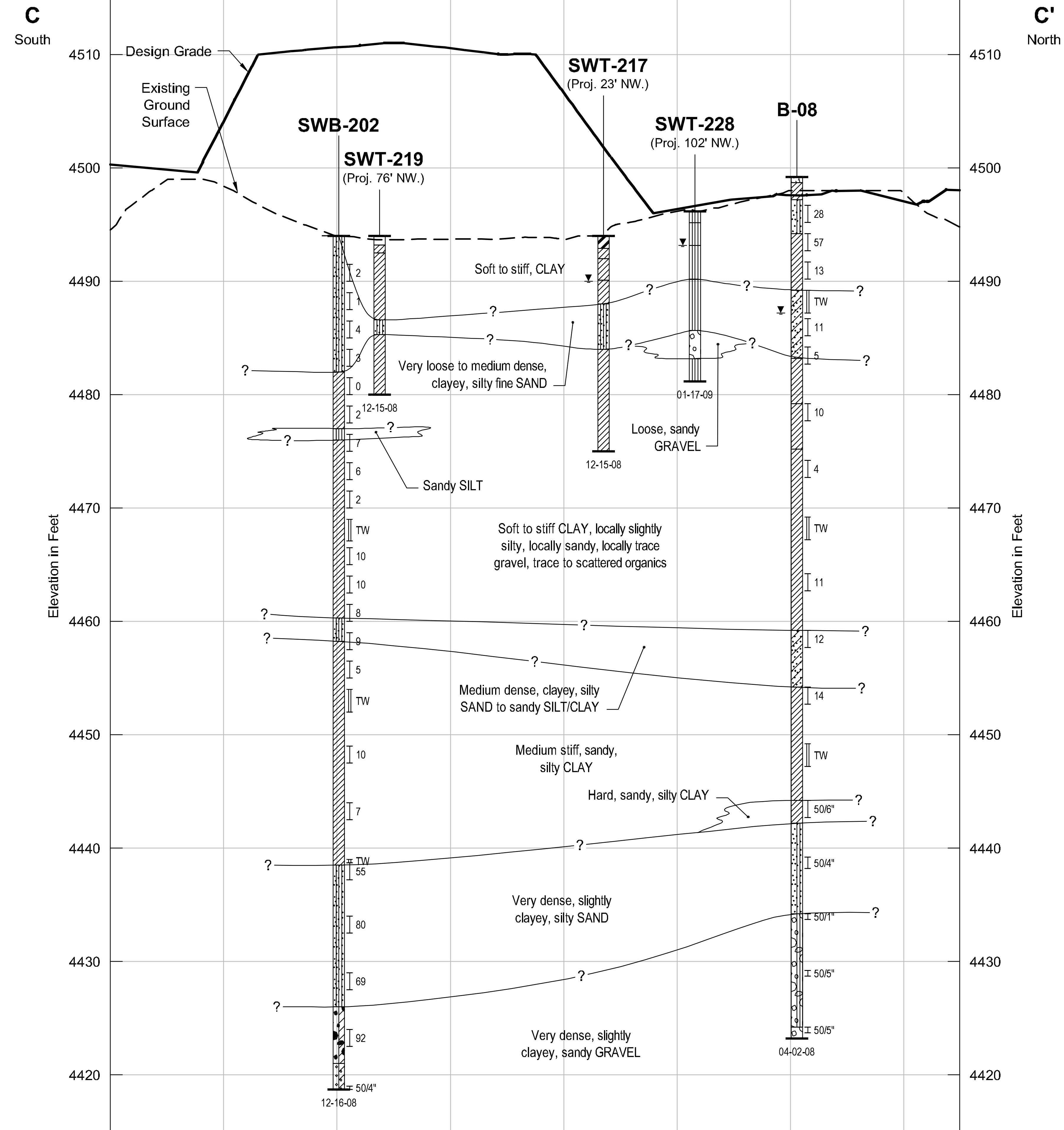
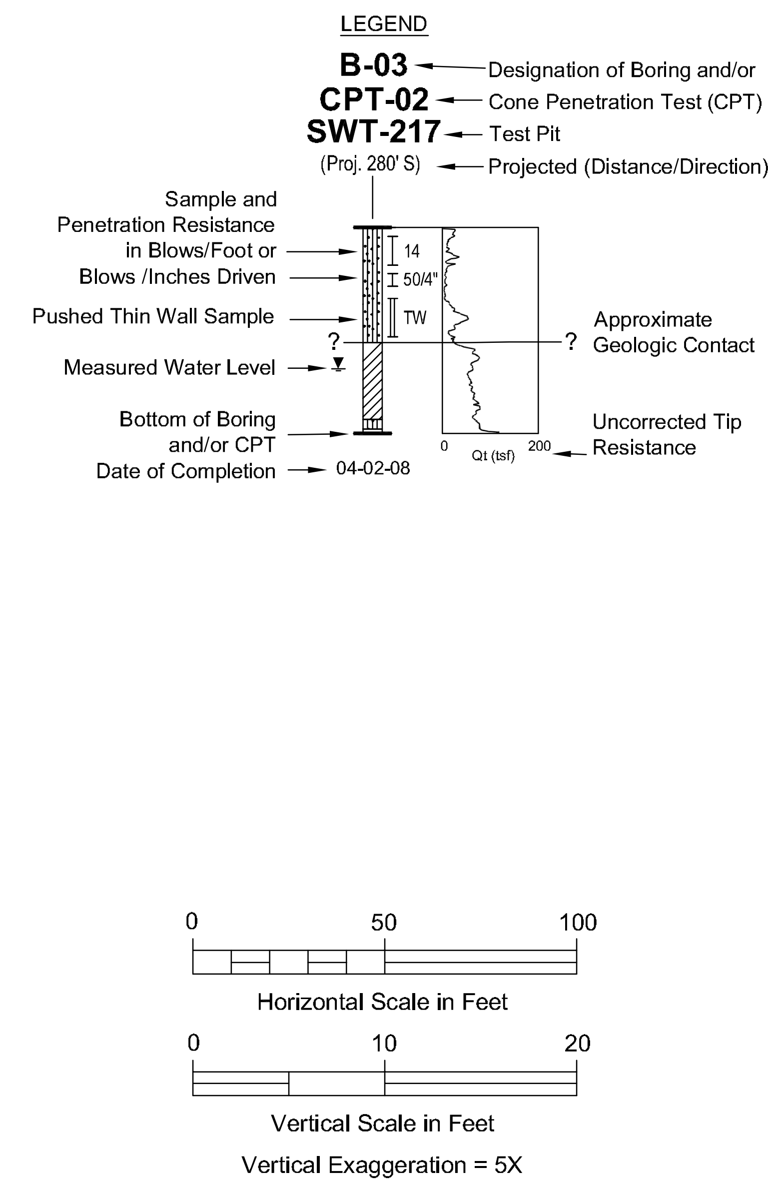
FIG. GD5-5





Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

GENERALIZED SUBSURFACE PROFILE C-C'

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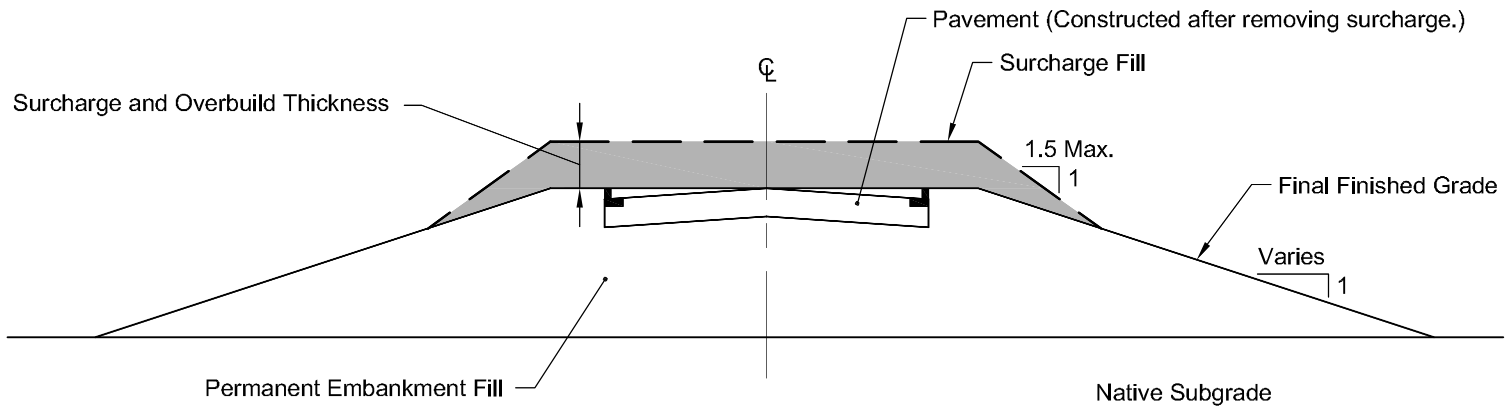
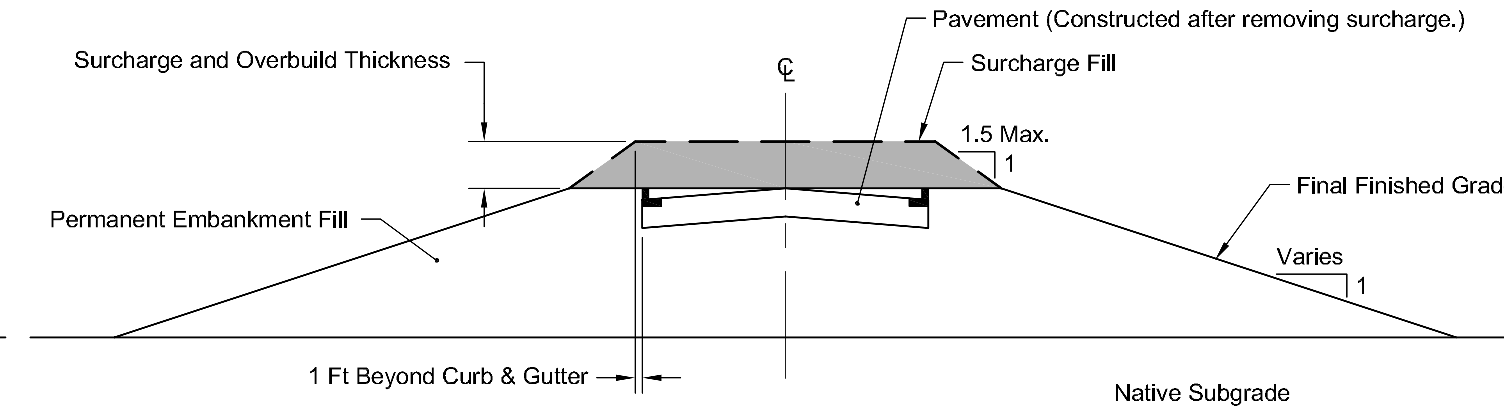
FIG. GD5-6

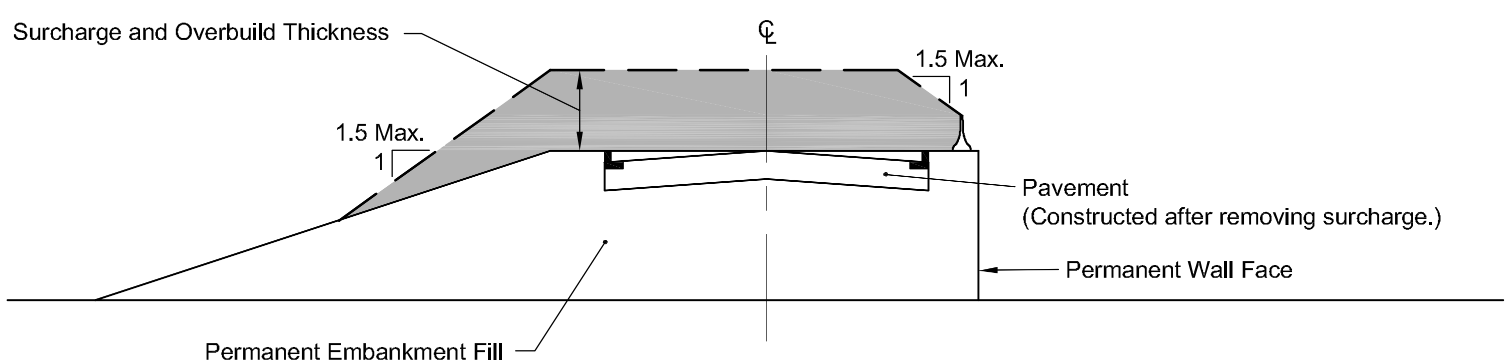
Filename: J:\231\01178-010\23-1-01178-010 Section A-C.dwg Date: 02-25-2009 Login: cnt



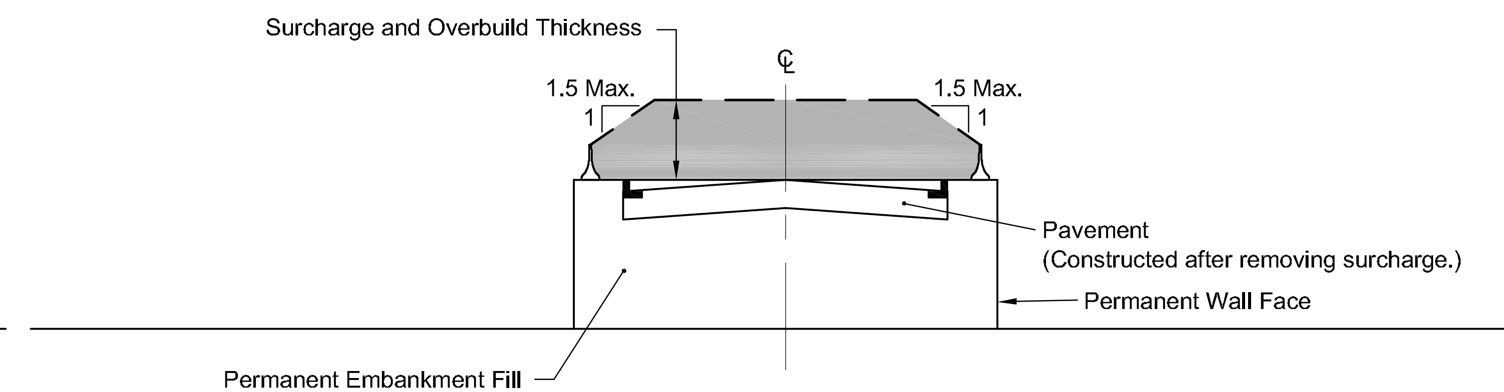
Surcharge Fill Configuration A



Surcharge Fill Configuration C



Surcharge Fill Configuration B



Surcharge Fill Configuration D

Surcharge Fill Schedule

Station Range	Surcharge Fill Configuration
621+70 to W. Abutment	C
E. Abutment to 627+75	C
627+75 to 629+50	A
629+50 to 641+20	B
648+10 to 650+00	D
650+00 to 654+40	C

NOTE

Drawing is not to scale.

LEGEND

Embankment Fill

Recommended Surcharge Fill Configuration

Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

RECOMMENDED SURCHARGE FILLS CONFIGURATION

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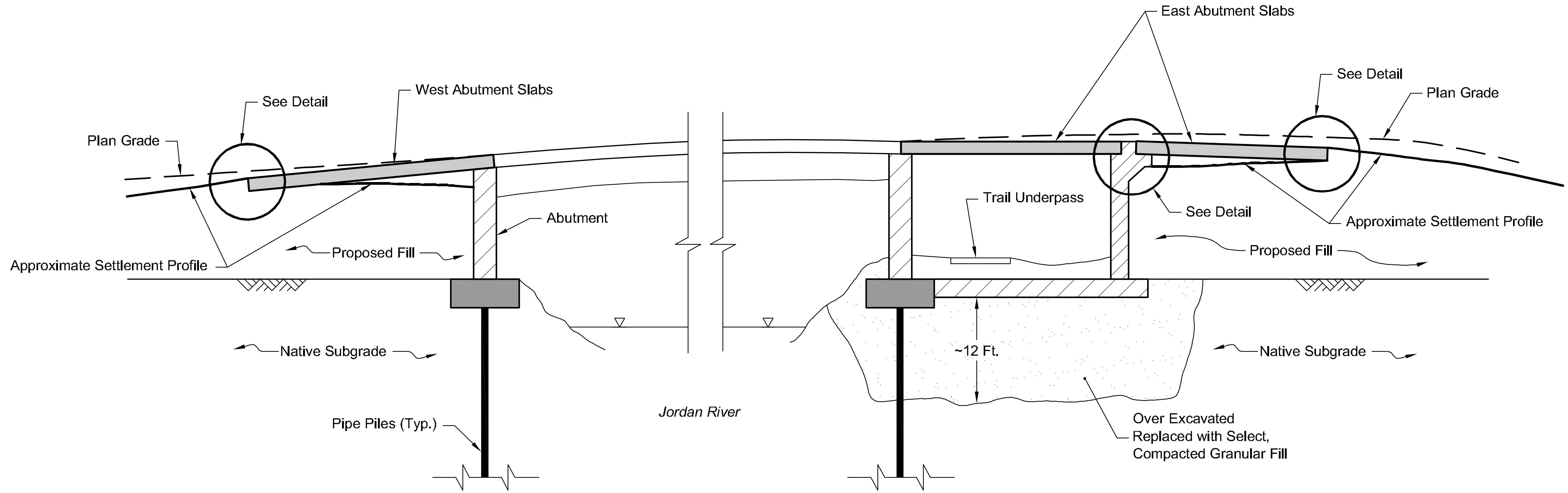
FIG. GD5-7

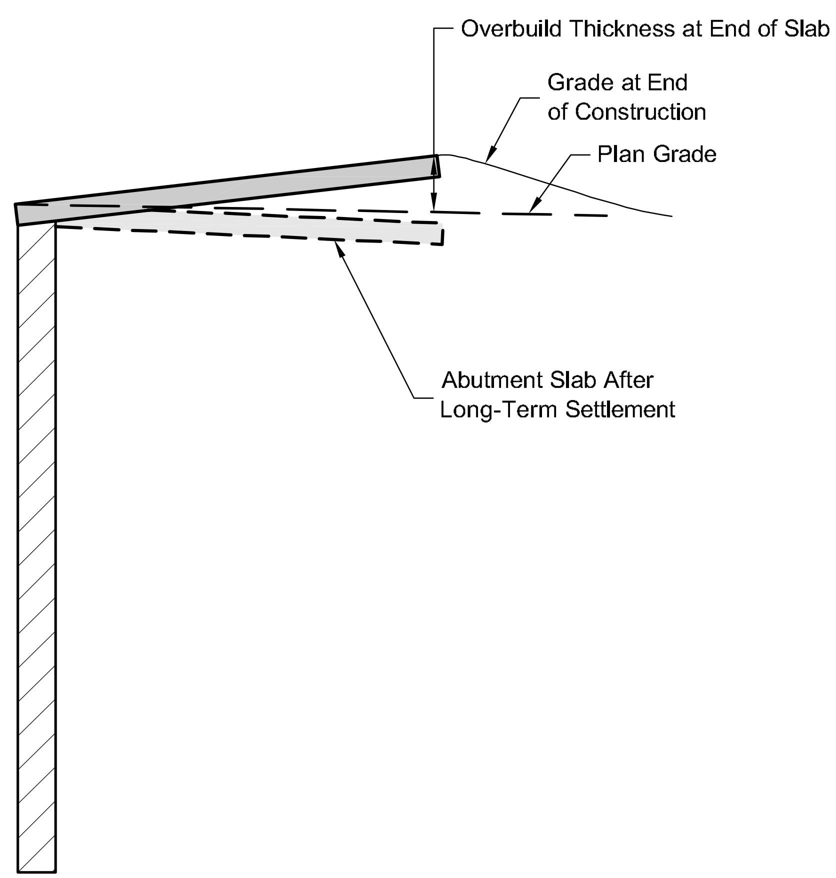
Filename: J:\231\01178-010\23-1-01178-010 Fig GD5-7.dwg Date: 02-25-2009 Login: cnt



Expected Long-Term Settlement Profile

NOTE

1. Drawing is not to scale.
2. Trail underpass could use a concrete box structure under a single independent approach slab.



Overbuild Settlement Mitigation Conceptual Detail

Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

APPROACH FILL SETTLEMENT SCHEMATIC

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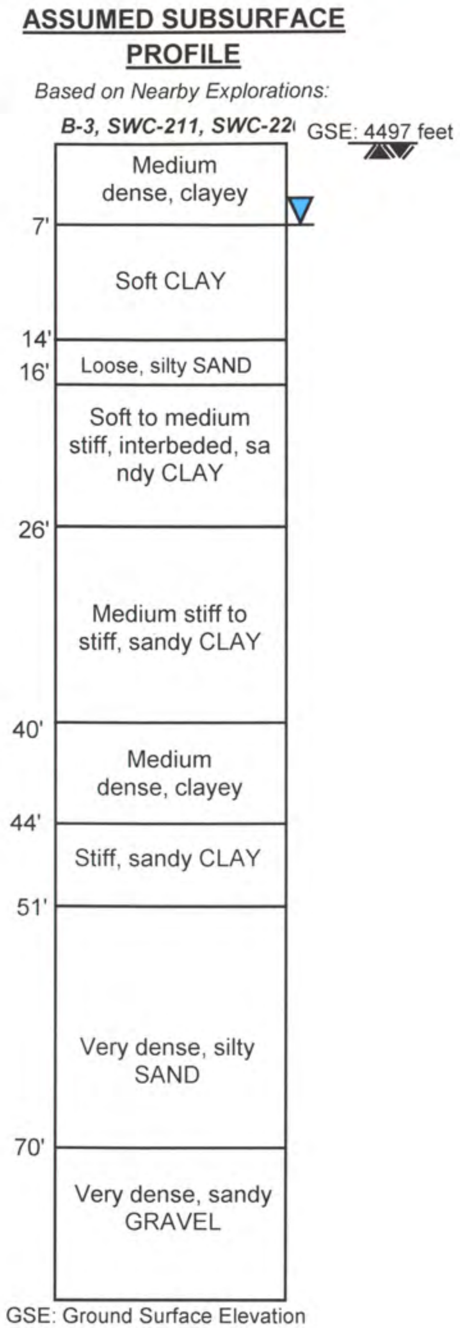
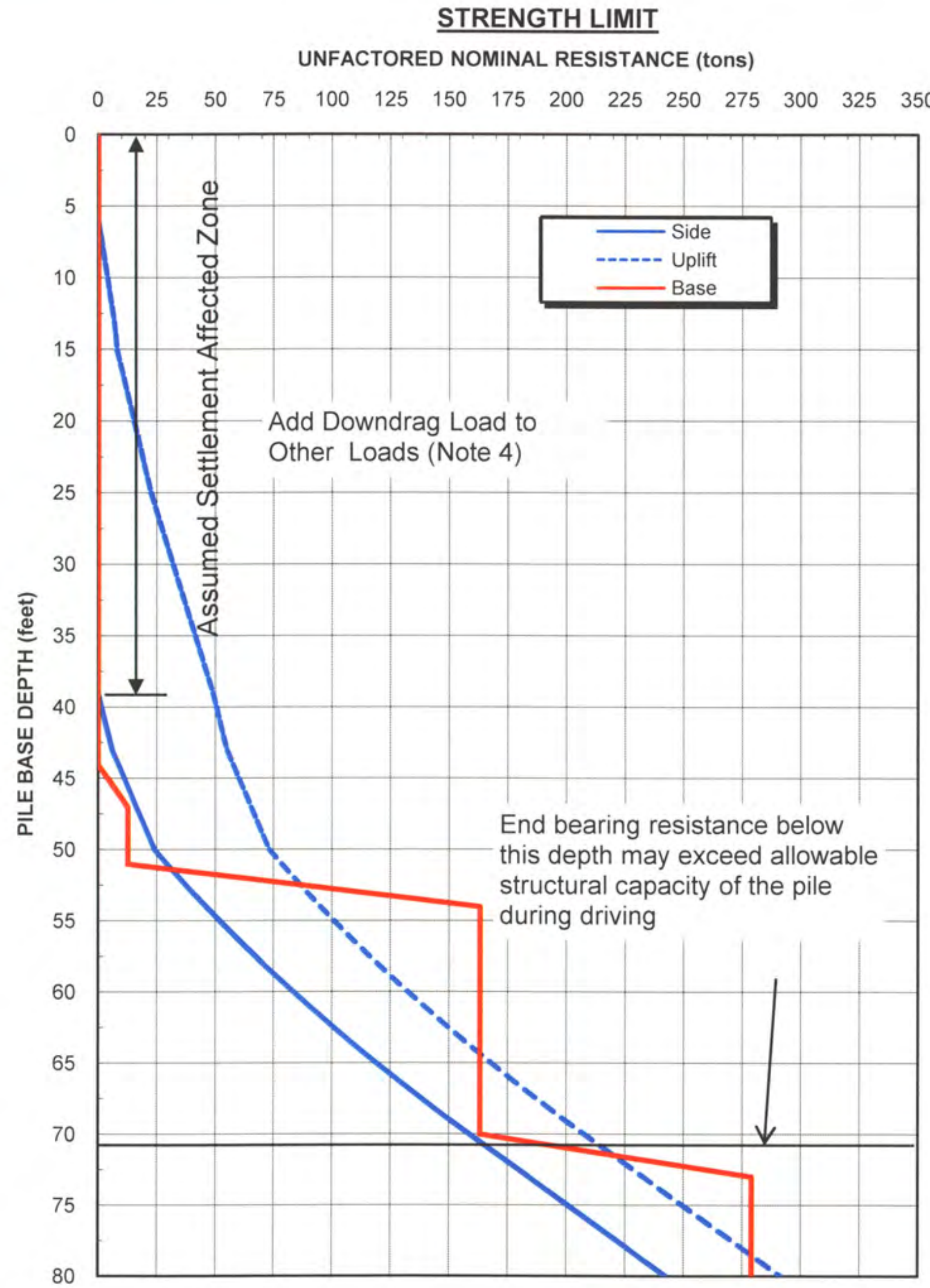
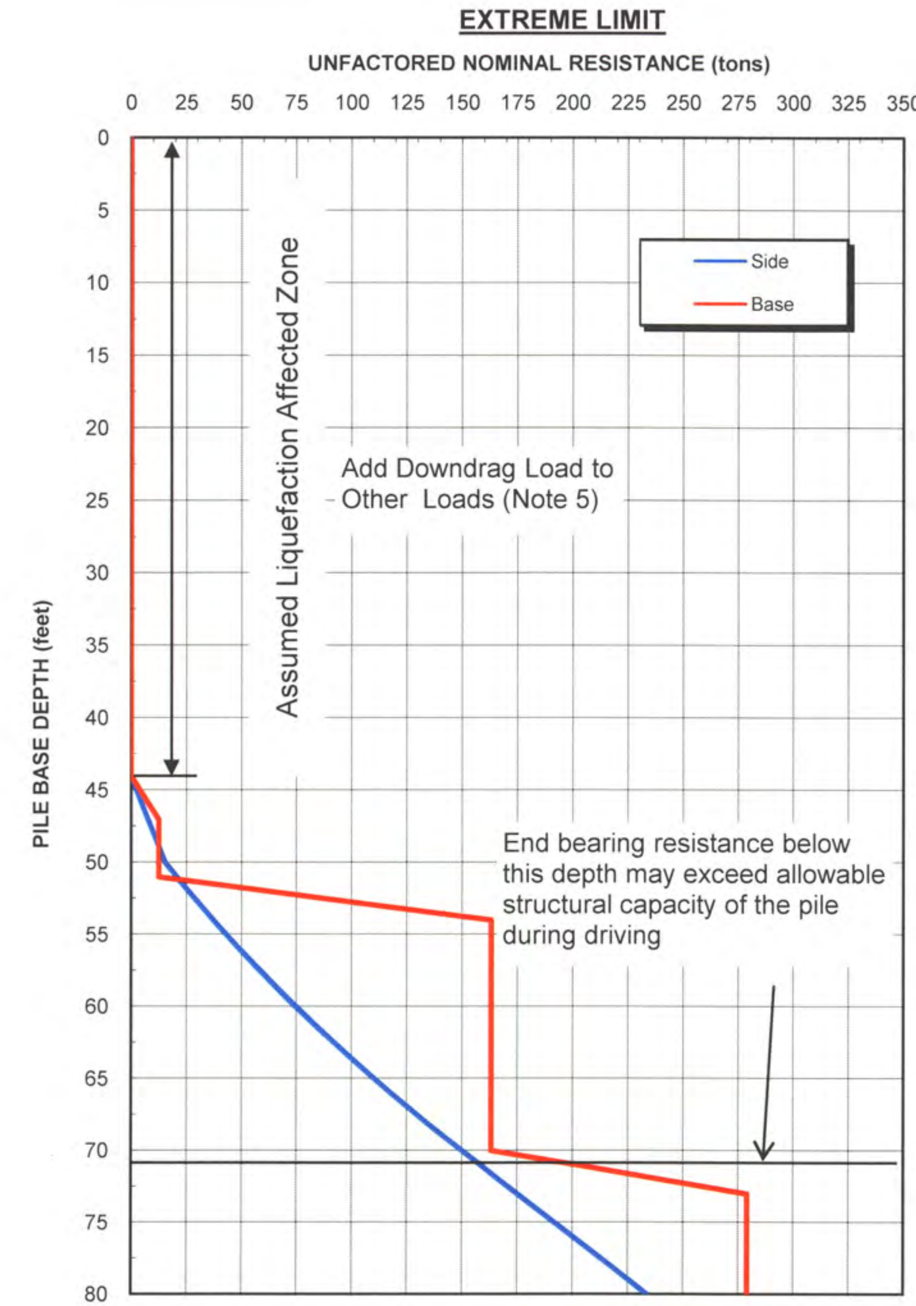
FIG. GD5-8







Notes:

1. The analyses are based on a single pile and do not consider group action of closely spaced piles (closer than 2.5 diameters, center to center).
2. The unfactored nominal resistances on the above plots should be multiplied by the appropriate resistance factors as noted in the table. Total pile compressive capacity considers side and base resistances.
3. Pile uplift capacity can be estimated by using the unfactored side resistance shown in the plot and a recommended resistance factor.
4. Unfactored downdrag force is estimated to be **49** tons due to embankment loading-induced secondary compression settlement.
5. Unfactored downdrag force is estimated to be **45 tons** due to liquefaction-induced settlement. Downdrag force is recommended to be applied with post-earthquake loading

RESISTANCE FACTORS

Limit State	Side Friction	End Bearing	Uplift*
Strength			
AASHTO LRFD**	0.25 - 0.50	0.25 - 0.50	0.25- 0.40
AASHTO w/ PDA and CAPWAP**	0.65	0.65	0.60
Extreme	1.00	1.00	0.80

* See Note 3

** Per AASHTO LRFD Bridge Design Specifications Table 10.5.5.2.3-1 based on design or test method, see Note 4.

Pioneer Crossing, Lehi and I-15 American Fork Interchange Lehi, Utah

JORDAN RIVER BRIDGE-WEST ABUTMENT ESTIMATED AXIAL CAPACITY FOR 16-INCH CIP PILE

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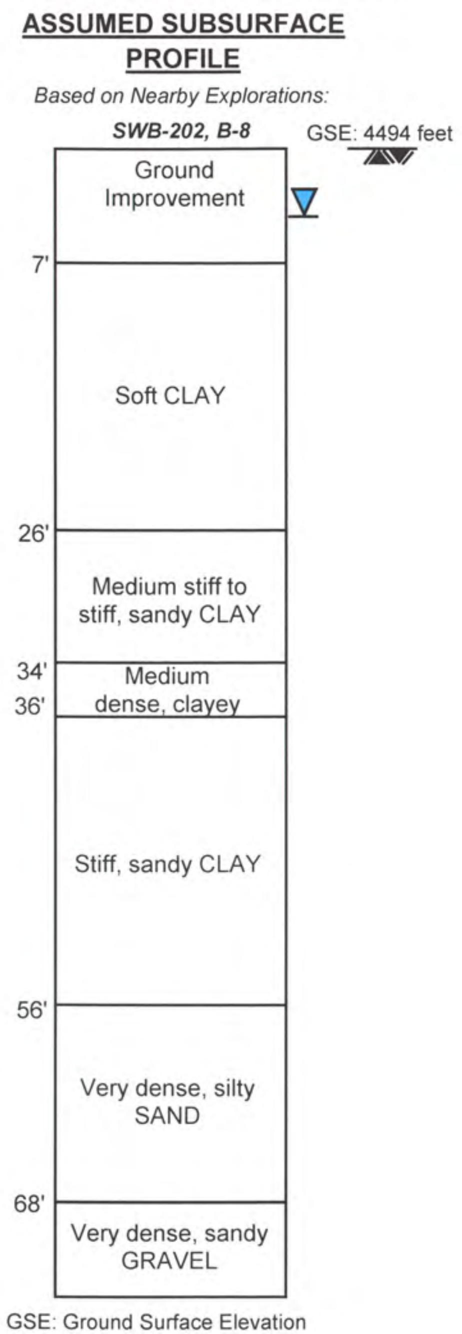
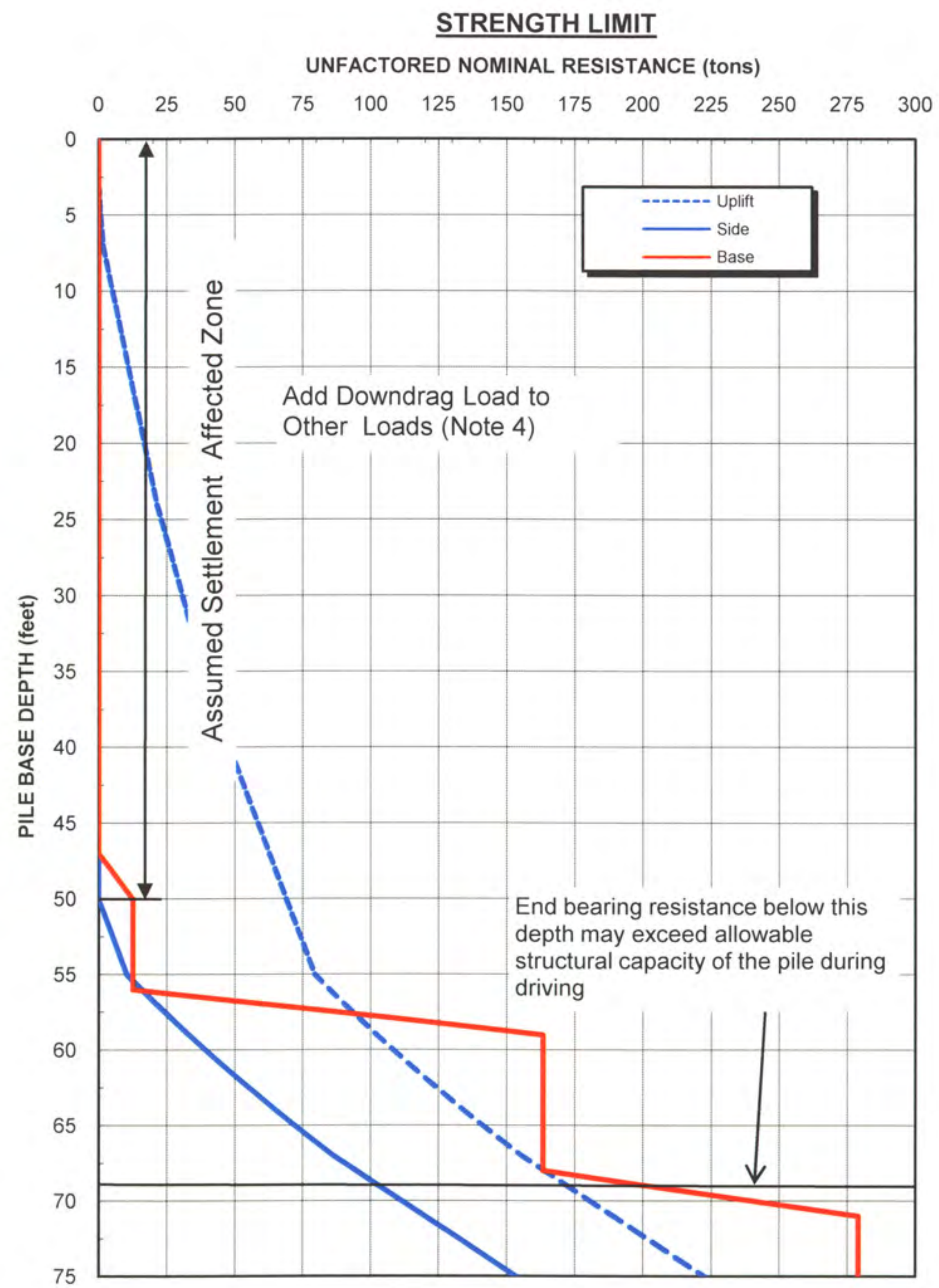
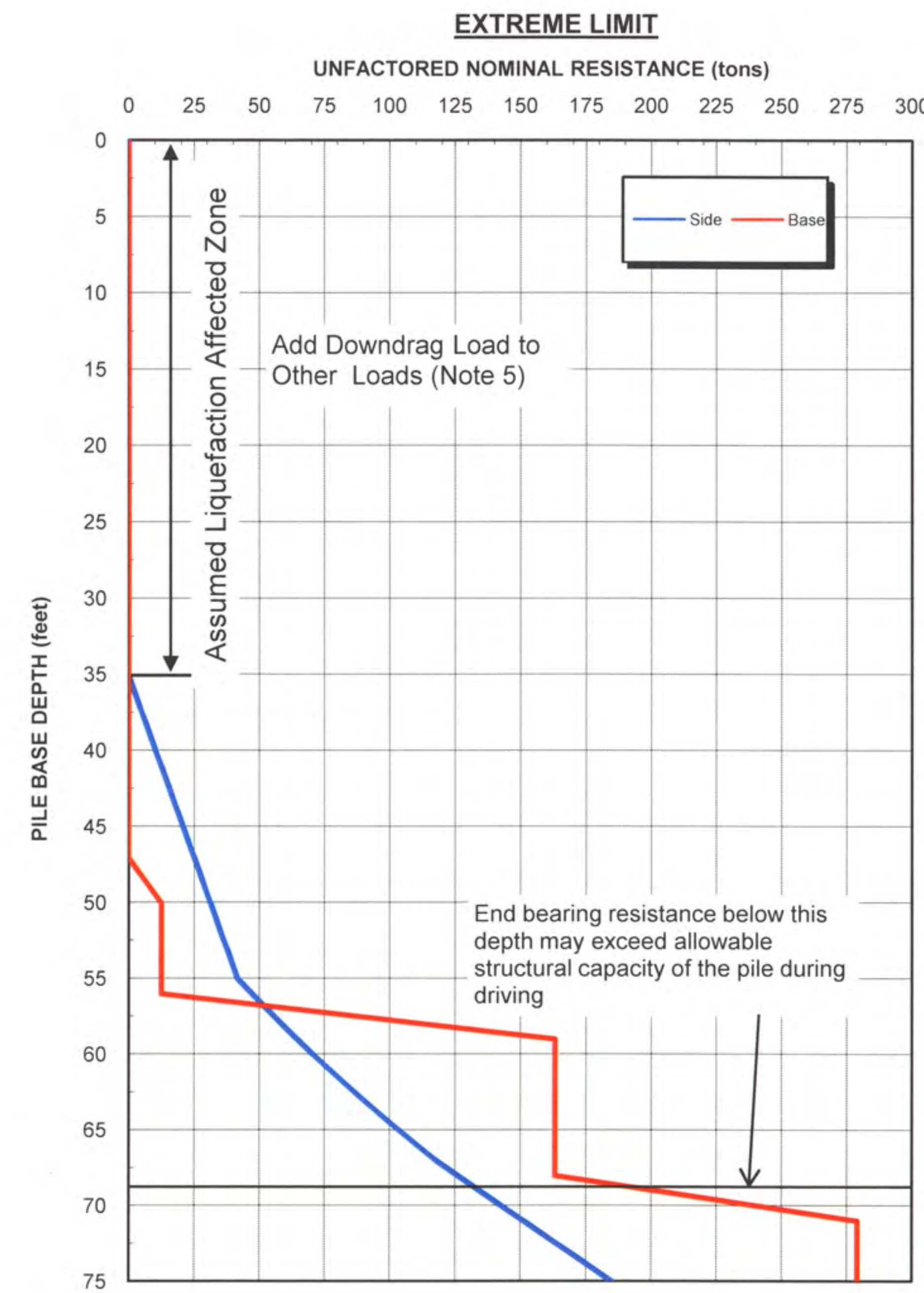
FIG GD5-9

2/26/2009-Figure GD5-10_Jordan River_16 CIP_East abutment.xls







Notes:

1. The analyses are based on a single pile and do not consider group action of closely spaced piles (closer than 2.5 diameters, center to center).
2. The unfactored nominal resistances on the above plots should be multiplied by the appropriate resistance factors as noted in the table. Total pile compressive capacity considers side and base resistances.
3. Pile uplift capacity can be estimated by using the unfactored side resistance shown in the plot and a recommended resistance factor.
4. Unfactored downdrag force is estimated to be **69** tons due to embankment loading-induced secondary compression settlement.
5. Unfactored downdrag force is estimated to be **26.5 tons** due to liquefaction-induced settlement. Downdrag force is recommended to be applied with post-earthquake loading

RESISTANCE FACTORS

Limit State	Side Friction	End Bearing	Uplift*
Strength			
AASHTO LRFD**	0.25 - 0.5	0.25 - 0.5	0.25- 0.4
AASHTO w/ PDA and CAPWAP**	0.65	0.65	0.60
Extreme	1.00	1.00	0.80

* See Note 3

** Per AASHTO LRFD Bridge Design Specifications Table 10.5.5.2.3-1 based on design or test method, see Note 4.

Pioneer Crossing, Lehi and I-15 American Fork Interchange Lehi, Utah

JORDAN RIVER BRIDGE-EAST ABUTMENT ESTIMATED AXIAL CAPACITY FOR 16-INCH CIP PILE

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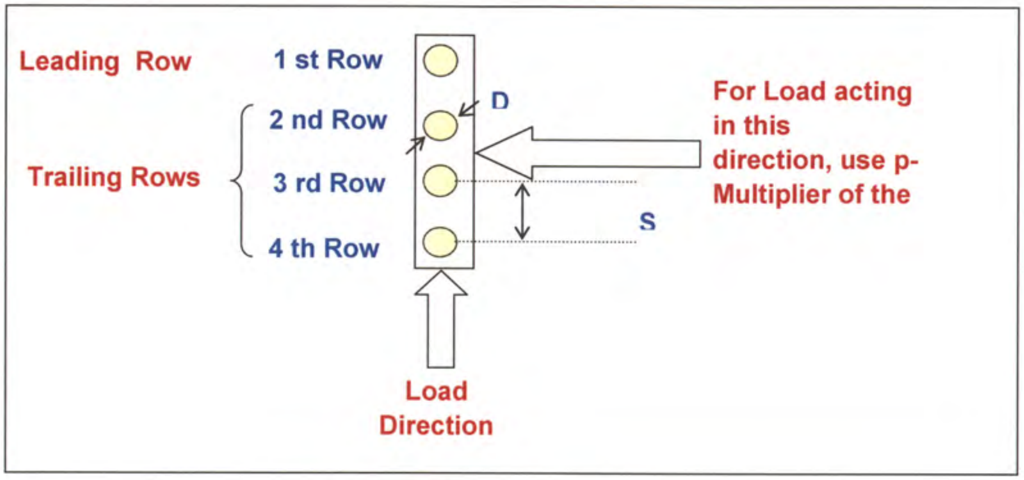
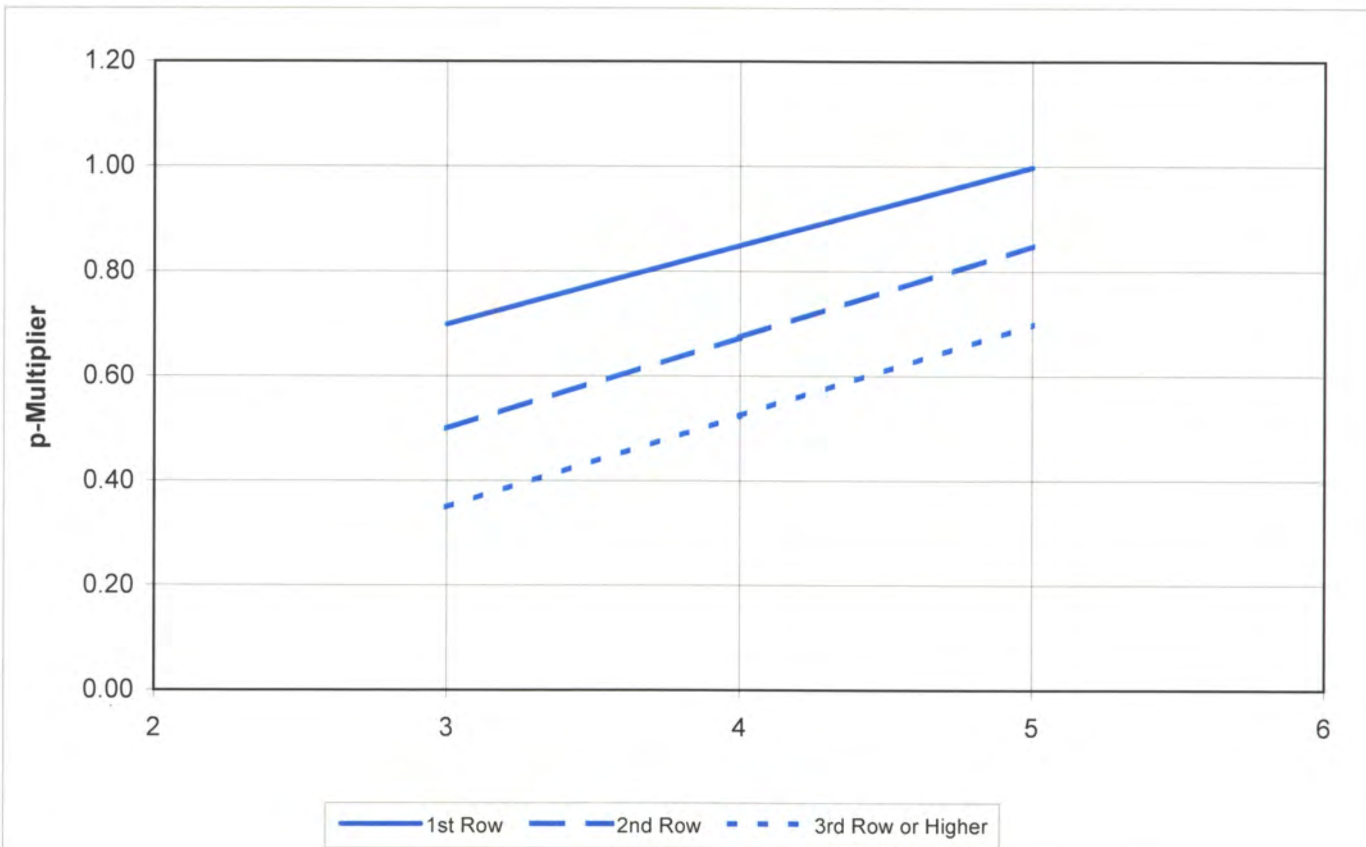
FIG GD5-10

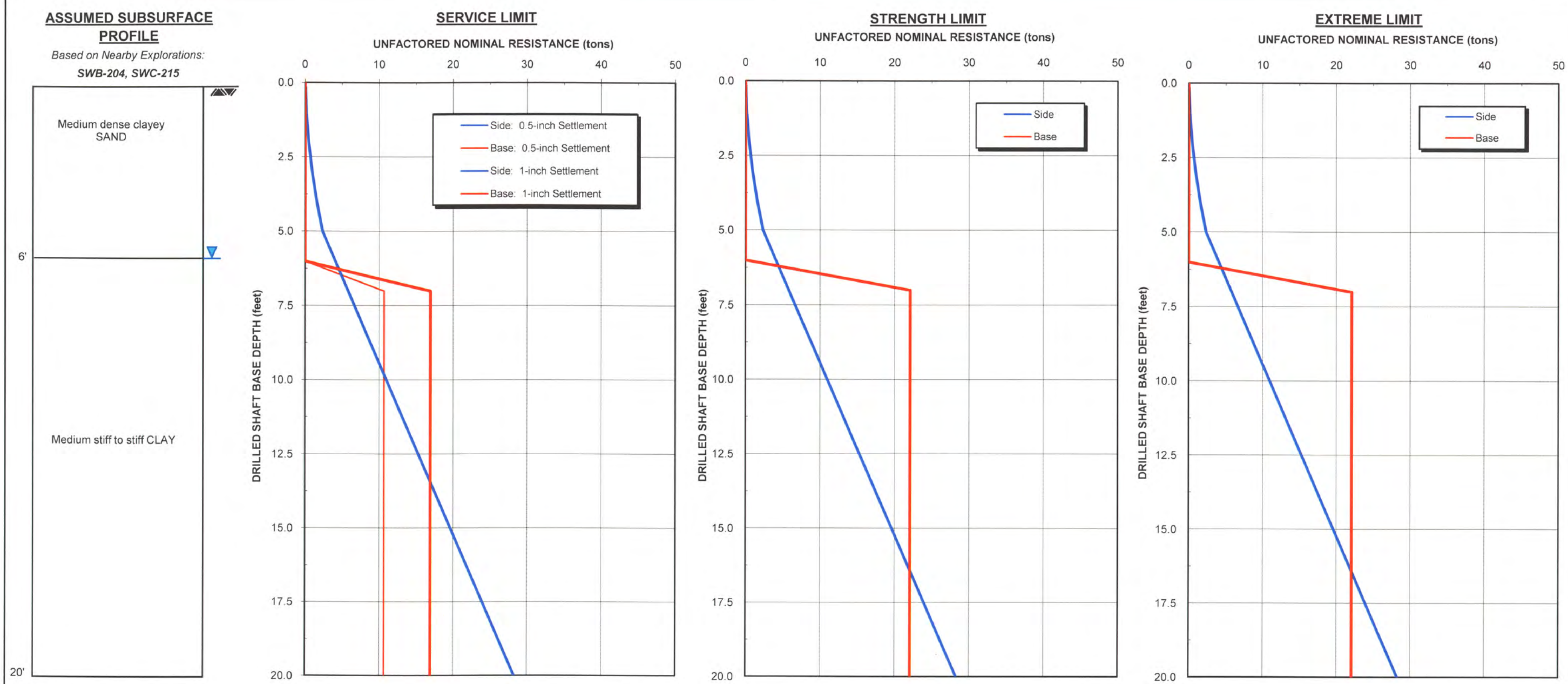




Reference: AASHTO LRFD Bridge Design Specifications- Table 10.7.2.4-1 (Hannigan et al., 2005)

Pioneer Crossing, Lehi and 1-15 American Fork Interchange Lehi, Utah	
P-MULTIPLIER GROUP EFFECTS ON LATERALLY LOADED PILES AND DRILLED SHAFTS	
February 2009	23-1-01178-010
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	FIG. GD5-11

Figure GD5-12-Post and Panel Wall 2.5 ft DS.xls



GENERAL NOTES

1. We performed the shaft axial capacity analyses using experience and recommendations in the AASHTO LRFD Bridge Design Specifications. The analyses are based on a single shaft and do not consider group action of shafts spaced closer than 4 diameters, center to center.
2. The unfactored nominal resistances on the above plots should be multiplied by the appropriate resistance factors as noted in the table. Total shaft compressive capacity is the sum of side and base resistances.
3. Estimated capacities assume that if casing is used, it will be removed after the shaft installation. If, the casing is left in place, grouting should be used to fill all potential voids around the casing and the estimated capacities given above should be re-evaluated.
4. Downdrag loads due to potential fill embankment settlement have not been included.
5. Shaft uplift capacity can be estimated by using the unfactored side resistance shown in the plot and a recommended resistance factor.

RESISTANCE FACTORS**

Limit State	Side Friction	End Bearing	Uplift*
Service	1.00	1.00	1.00
Strength	0.45	0.40	0.35
Extreme	1.00	1.00	0.80

* See Note 5
** Per AASHTO LRFD Bridge Design Specifications Table 10.5.5.2.4-1

Pioneer Crossing, Lehi and
I-15 American Fork Interchange
Lehi, Utah

POST AND PANEL WALLS ESTIMATED AXIAL CAPACITY FOR 2.5-foot DRILLED SHAFT

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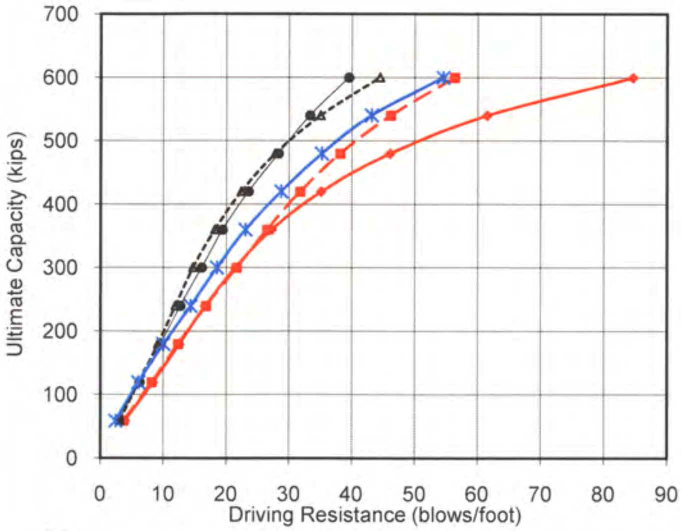
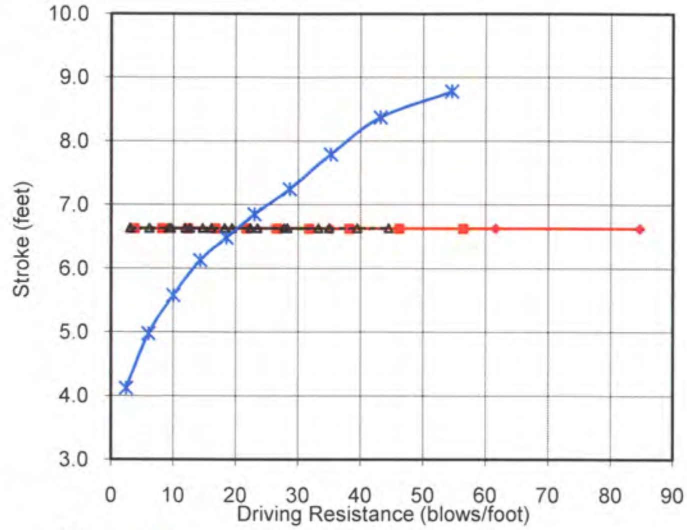
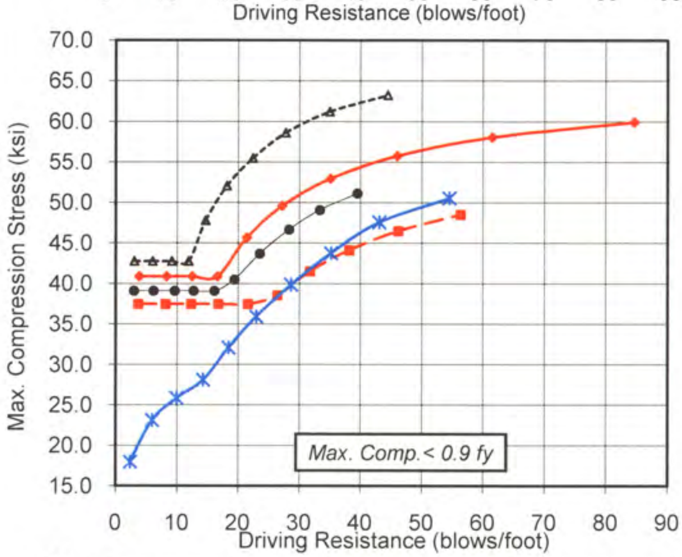
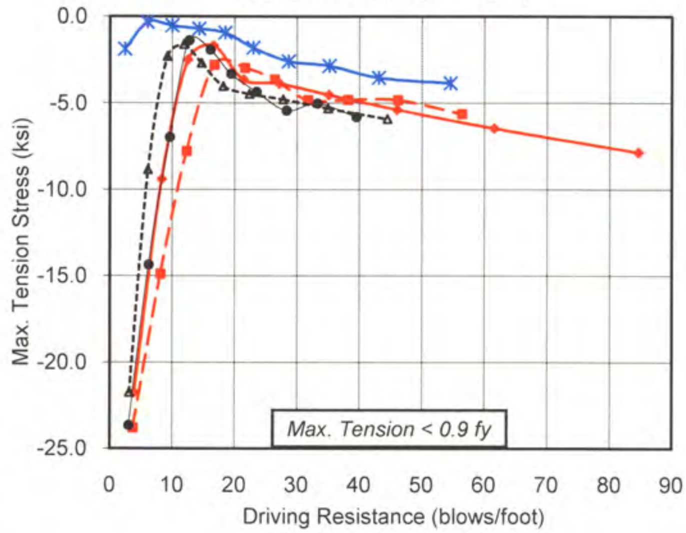
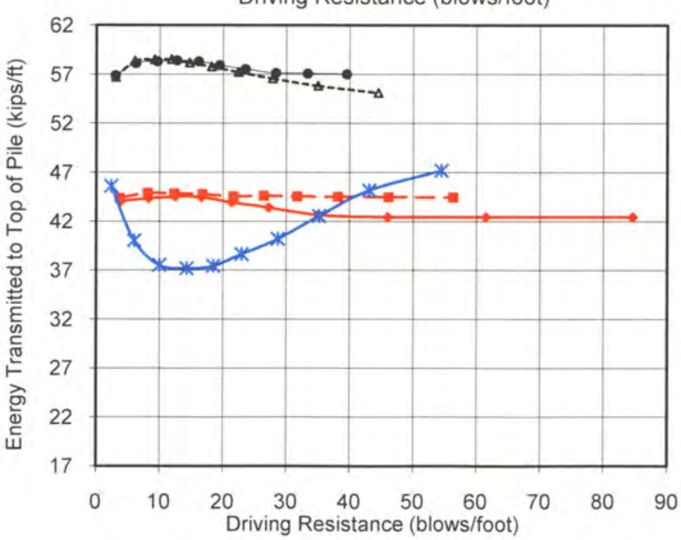
FIG. GD5-12











GRLWEAP INPUT PARAMETERS

	Pile Data
Size	16 in
Total Length	varies
Penetration	80 ft
X-Sect. Area	201 in^2
Circumference	4.2 ft
Material	Steel
Elast. Modulus	30000 ksi
Spec. Weight	492 pcf
Yield Strength (fy)	45 ksi

	Pile Cushion
Thickness	-
Area	-
Elast. Modulus	-
C.O.R.	-

	IHC S-70/90 Hammer	DELMAG D-36 Hammer
Helmet Weight	2.7 kips	2.7 kips
Efficiency	0.95	0.8
	Hammer Cushion	
Thickness	2 in	2 in
Area	415 in^2	415 in^2
Elast. Modulus	530 ksi	530 ksi
C.O.R.	0.8	0.8
	Soil Parameters	
	Quake	Damping
Shaft	0.1	0.08
Toe	0.133	0.15
Shaft Resist.	40%	*Simple Dist.*

LEGEND:

- IHC S-70 Wall Thickness = 0.5 inch
- IHC S-70 Wall Thickness = 0.75 inch
- IHC S-90 Wall Thickness = 0.5 inch
- IHC S-90 Wall Thickness = 0.75 inch
- DELMAG D-36 Wall Thickness = 0.5 inch

NOTES:

1. The computer program GRLWEAP (Version 2005) was used with the above assumed input parameters to perform the analyses.
2. Standard GRLWEAP recommended values used for cushions and helmets.

Pioneer Crossing, Lehi and I-15 American Fork Interchange Lehi, Utah

WEAP ANALYSIS, 16-INCH CONCRETE-FILLED PIPE PILE WEST ABUTMENT

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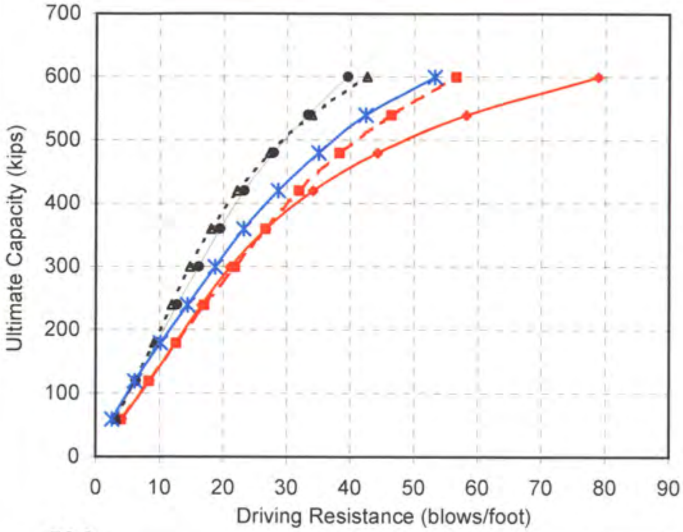
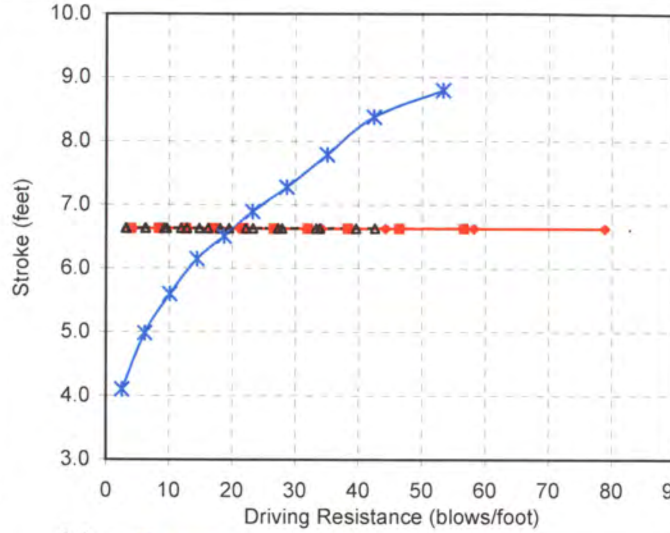
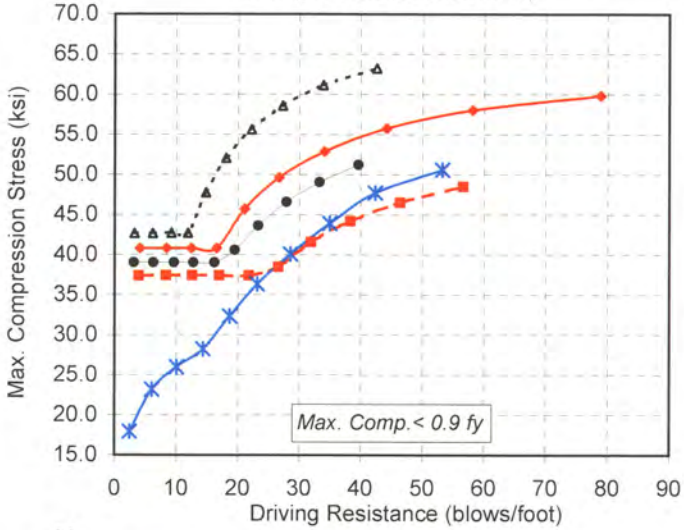
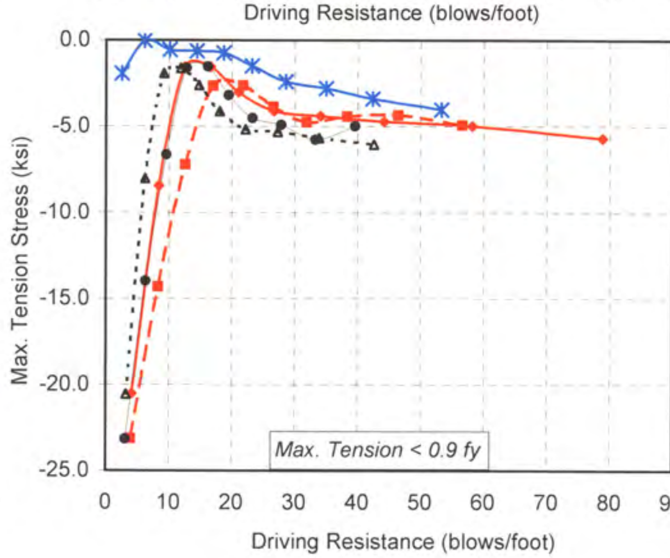
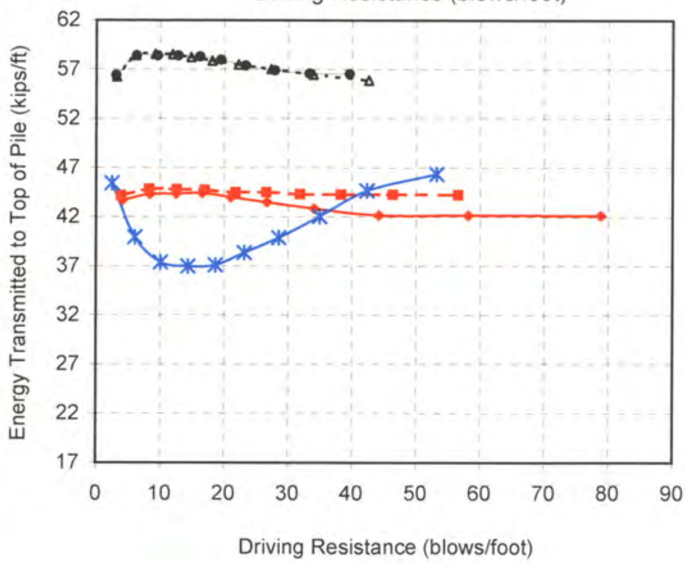
FIG. GD5-13











LEGEND:

- IHC S-70 Wall Thickness = 0.5 inch
- IHC S-70 Wall Thickness = 0.75 inch
- IHC S-90 Wall Thickness = 0.5 inch
- IHC S-90 Wall Thickness = 0.75 inch
- DELMAG D-36 Wall Thickness = 0.5 inch

NOTES:

1. The computer program GRLWEAP (Version 2005) was used with the above assumed input parameters to perform the analyses.
2. Standard GRLWEAP recommended values used for cushions and helmets.

GRLWEAP INPUT PARAMETERS

	Pile Data
Size	16 in
Total Length	varies
Penetration	80 ft
X-Sect. Area	201 in^2
Circumference	4.2 ft
Material	Steel
Elast. Modulus	30000 ksi
Spec. Weight	492 pcf
Yield Strength (fy)	45 ksi

	Pile Cushion
Thickness	-
Area	-
Elast. Modulus	-
C.O.R.	-

	IHC S-70/90 Hammer	DELMAG D-36 Hammer
Helmet Weight	2.7 kips	2.7 kips
Efficiency	0.95	0.8
Hammer Cushion		
Thickness	2 in	2 in
Area	415 in^2	415 in^2
Elast. Modulus	530 ksi	530 ksi
C.O.R.	0.8	0.8
Soil Parameters	Quake	Damping
Shaft	0.1	0.08
Toe	0.133	0.15
Shaft Resist.	40%	*Simple Dist.*

Pioneer Crossing, Lehi and I-15 American Fork Interchange Lehi, Utah

WEAP ANALYSIS, 16-INCH CONCRETE-FILLED PIPE PILE EAST ABUTMENT

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FIG. GD5-14

APPENDIX GD5-A

SUBSURFACE EXPLORATIONS

APPENDIX GD5-A

SUBSURFACE EXPLORATIONS

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Figure No.

LIST OF FIGURES (cont.)

Figure No.

LIST OF FIGURES (cont.)

Figure No.

APPENDIX GD5-A

SUBSURFACE EXPLORATIONS

GD5-A.1 INTRODUCTION

The subsurface exploration program for the Pioneer Crossing project consisted of drilling borings, excavating test pits, and pushing cone penetration test (CPT) probes. The subsurface explorations occurred in two phases: the Request for Proposal (RFP) and final design phase. Shannon & Wilson used 12 borings, 17 test pits, and 13 CPTs for designing this project. The borings, test pits, and CPTs were performed at the approximate locations shown in the Site and Exploration Plan, presented as Figure GD5-2, after the main report text. The following table lists the boring/test pit/CPT designation, company name, date drilled, and depth of the exploration:

DESCRIPTION OF EXPLORATIONS

Designation	Company Name	Total Depth (feet)	Date Drilled
SWB-201	Shannon & Wilson	41.5	December 11, 2008
SWB-202	Shannon & Wilson	75.3	December 16, 2008
SWB-203	Shannon & Wilson	46.5	December 12, 2008
SWB-204	Shannon & Wilson	95.5	December 14, 2008
B-01	Terracon	21.5	March 18, 2008
B-02	Terracon	21.5	March 18, 2008
B-03	Terracon	75.5	March 19, 2008
B-04	Terracon	21.5	April 2, 2008
B-05	Terracon	26.5	March 18, 2008
B-08	Terracon	76.0	April 1, 2008
B-09	Terracon	21.5	April 2, 2008
B-10	Terracon	26.5	April 3, 2008
SWC-211	Shannon & Wilson	59.2	December 12, 2008
SWC-212	Shannon & Wilson	65.6	December 13, 2008
SWC-213	Shannon & Wilson	20.3	December 13, 2008
SWC-214	Shannon & Wilson	64.0	December 14, 2008
SWC-215	Shannon & Wilson	94.6	December 14, 2008
SWC-216	Shannon & Wilson	93.8	December 15, 2008
SWC-218	Shannon & Wilson	65.9	December 13, 2008
SWC-220	Shannon & Wilson	52.0	December 12, 2008
SWC-221	Shannon & Wilson	56.1	December 15, 2008
SWC-222	Shannon & Wilson	58.9	December 15, 2008
SWC-223	Shannon & Wilson	84.1	December 16, 2008

Designation	Company Name	Total Depth (feet)	Date Drilled
SWC-224	Shannon & Wilson	60.7	December 17, 2008
CPT-2	Terracon	60.0	March 19, 2008
SWT-205	Shannon & Wilson	14	December 17, 2008
SWT-206	Shannon & Wilson	12	December 17, 2008
SWT-207	Shannon & Wilson	16	December 16, 2008
SWT-208	Shannon & Wilson	14	December 13, 2008
SWT-209	Shannon & Wilson	15	December 15, 2008
SWT-210	Shannon & Wilson	16	December 15, 2008
SWT-217	Shannon & Wilson	19	December 15, 2008
SWT-219	Shannon & Wilson	14	December 15, 2008
SWT-225	Shannon & Wilson	15	January 17, 2009
SWT-227	Shannon & Wilson	16	January 17, 2009
SWT-228	Shannon & Wilson	15	January 17, 2009
SWT-229	Shannon & Wilson	15	January 17, 2009
SWT-230	Shannon & Wilson	14	January 17, 2009
SWT-231	Shannon & Wilson	14	January 17, 2009
SWT-232	Shannon & Wilson	6	January 17, 2009
SWT-233	Shannon & Wilson	15	January 18, 2009
SWT-234	Shannon & Wilson	16	January 17, 2009

The approximate locations of the Shannon & Wilson subsurface explorations were surveyed by Kiewit/Clyde before they were performed. The actual locations may vary. Terracon provided coordinates for the RFP explorations for their explorations. Boring, test pits, and CPT locations and elevations should be considered accurate to the degree implied by the method used.

GD5-A.2 BORINGS

The subsurface conditions along the Pioneer Crossing alignment were explored by Shannon & Wilson with four soil borings. These four borings, designated SWB-201 through SWB-204 were advanced to depths ranging from 41.5 to 95.5 feet.

GD5-A.2.1 Drilling Procedures

Direct Push Services, LLC, drilled the soil borings under subcontract to Shannon & Wilson using a truck-mounted CME-75 drill rig. The borings were drilled using mud-rotary techniques. The drilling mud was a mixture of bentonite powder and water. Cuttings are transported from the bottom of the borehole to the surface by drilling mud flowing between the drilling rods and the sides of the borehole. The cuttings are deposited in a settling tank at the ground surface and the mud is recirculated. Soil samples are taken from the bottom of the mud-filled open hole.

Artesian groundwater conditions were encountered at approximately 65 feet below the ground surface (bgs) in boring SWB-202. After drilling and sampling were completed, a well was placed in borings SWB-202 and SWB-203 and then boreholes were sealed with bentonite grout and chips.

GD5-A.2.2 Soil Sampling

To obtain disturbed soil samples from borings, Standard Penetration Tests (SPTs) were performed in general accordance with the ASTM International (ASTM) Designation: D 1586, Test Method for Penetration Test and Split-Barrel Sampling of Soils. In borings SWB-201, SWB-203, and SWB-204, SPTs were performed every 2.5 feet to a depth of 20 feet and then at 5-foot intervals to the bottom of the borings. In boring SWB-202, SPTs were performed every 2.5 feet to a depth of 40 feet and then at 5-foot intervals to the bottom of boring. In the SPT, a 2-inch outside-diameter, 1⅜-inch inside-diameter, split-spoon sampler is driven a distance of 18 inches into the bottom of the borehole with a 140-pound hammer falling 30 inches. At the time of this report, the hammer efficiency was not available, so we assumed a hammer efficiency of 75 percent.

Our field representative recorded the number of blows required to achieve each of three 6-inch increments of sampler penetration. The number of blows required to cause the last 12 inches of penetration is termed the Standard Penetration Resistance (N-value). This value is an empirical parameter that provides a means for evaluating the relative density, or compactness, of granular soils and the consistency, or stiffness, of cohesive soils. The terminology used to describe the relative density or consistency of the soil is presented in Figure GD5-A-1. Generally, whenever 50 or more blows were required to cause 6 inches or less of penetration, the test was terminated, and the number of blows and the corresponding penetration were recorded. The N-values are plotted at the appropriate depths on the boring logs presented as Figures GD5-A-2 through GD5-A-5.

The split-spoon sampler used during the SPT penetration testing recovers a disturbed sample of the soil, which is useful for identification and classification purposes. The samples were classified and recorded in field logs by our representatives. The samples were then sealed in jars and returned to our laboratory for testing.

At select locations, relatively undisturbed samples were obtained in general accordance with ASTM Designation: D 1587-00, Standard Practice for Thin-Walled Tube Geotechnical Sampling of Soils. The relatively undisturbed samples were obtained using a Shelby sampler, which is a 3-inch outer diameter, thin wall steel sampling tube with a sharp cutting edge that is connected to a sampling head attached to the drill rods. The sampling tube is pushed for a distance of 2 feet and then retracted to obtain the sample. The tube is then capped and sealed at both ends to preserve the field moisture conditions. The sample tubes were then stored upright and driven to our laboratory for testing.

GD5-A.2.3 Soil Classification

A representative from Shannon & Wilson, Inc. was present throughout the field drilling period to observe the drilling and sampling operation, retrieve representative soil samples for subsequent laboratory testing, and to prepare descriptive field logs of the boring explorations. Boring sample classifications were based on the ASTM Designation: D 2487-98, Standard Test Method for Classification of Soil for Engineering Purposes, and ASTM Designation: D 2488-93, Standard Recommended Practice for Description of Soils (Visual-Manual Procedure). The Unified Soil Classification System (USCS), as described in Figure GD5-A-1 of this appendix, was used to classify the material encountered. The boring logs presented in Figures GD5-A-2 through GD5-A-5 represent our interpretation of the samples and the results of geotechnical laboratory testing.

GD5-A.2.4 Boring Logs

The design phase boring logs for Pioneer Crossing are presented as Figures GD5-A-2 through GS5-A-5. A boring log is a written record of the subsurface conditions encountered and graphically illustrate the geologic units (layers) encountered in each boring and the USCS symbol of each geologic layer. It includes the blow count, corrected blow count $(N1)_{60}$ and natural water content (where tested). Other information shown in the boring logs includes the groundwater level observations made during drilling, ground water observations from well measurements (when available), ground surface elevations, station and offset, types and depths of sampling, fines content (where tested), and Atterberg Limits (where tested).

GD5-A.3 CONE PENETRATION TESTING

Twelve CPT probes, designated SWC-211 through SWC-216, SWC-218, and SWC-220 through SWC-224 were performed at the site in December 2008 to supplement the subsurface information obtained in the borings and test pits and to perform additional in situ testing. The CPT is an electric piezocone test that develops a nearly continuous subsurface profile of soil conditions at a particular location. The testing was performed by In Situ Engineering under subcontract to Shannon & Wilson, in general accordance with procedures outlined in ASTM Designation: D 3441-94, Test Method for Deep, Quasi-static, Cone and Friction-Cone Penetration Tests of Soil. Soil samples are not obtained in this test method. The 12 CPT probes ranged in depth from about 20 to 94 feet bgs.

GD5-A.3.1 Field Equipment

The piezocone apparatus used by In Situ Engineering is a Hogentogler system. In this test, steel rods with a cone tip on the end are pushed hydraulically into the soil at a relatively constant rate of approximately 2 centimeters per second (cm/sec). Readings are recorded every 5 cm. The cone tip is connected to a stationary friction sleeve and has a cross-sectional area of 10 cm^2, a surface area of 15 cm^2, and an angle of 30 degrees from the axis. The area ratio for the tip was 0.8. The stationary friction sleeve had the same diameter as the cone tip but a surface area of 150 cm^2. The cone tip and friction sleeve assembly is about 50 cm long and is pushed into the ground by steel rods. Each rod is about 1 meter long. An electronic cable is prestrung through the rods. This cable provides power to the instruments and communication between the instrument and a computer. The entire system is powered by a 12-volt deep cycle battery, which is periodically recharged.

The CPT instrument is capable of recording tip resistance, sleeve friction, pore pressure, and inclination as it penetrates into the ground. The cone had a tip capacity of 10 tons or approximately 1,000 tons per square foot. The cone was a subtraction type cone, which senses the tip resistance on one set of strain gauges and senses tip resistance plus side friction on another set of strain gauges. The frictional reading was determined by electronically subtracting the tip reading from the combined reading. The pore pressure sensor had a capacity of 500 pounds per square inch. The pore pressure filter element, located behind the cone tip, was a high-air-entry polypropylene disk that was discarded and replaced after every test hole. Disks were pre-saturated by subjecting to a vacuum of 25 to 28 inches of mercury for 30 minutes while submerged in a 50 percent solution of glycerin and water. This filter element transmits pore pressures to the pressure transducer located within the cone tip.

GD5-A.3.2 Testing Procedures

As the cone penetrated through the soil, measurements of tip resistance, sleeve friction, pore pressure, and inclination were electrically transmitted through the electronic cable to the ground surface and then displayed and recorded on a portable computer. The cone was pushed into ground at a rate of 2 cm/sec and readings were recorded at 5 cm intervals. Termination of the testing resulted when either the penetration resistance exceeded the capacity of the hydraulic system, or the rebounding (bending), of the cone penetrometer push rods became excessive. The tip, filter element, and friction sleeve assemblies were disassembled and cleaned between holes. A new pore pressure filter element was placed in the assembly prior to each hole, and pore pressure cavities were filled with a 50 percent glycerin and water solution. A syringe was used in filling void spaces to assist in removing air bubbles and increase saturation.

GD5-A.3.3 Interpretation

The CPT data consists of cone tip resistance, sleeve friction, friction ratio (ratio of sleeve friction to cone tip resistance), and pore pressure versus depth. This data was processed and interpreted by Shannon & Wilson and In Situ Engineering. Soil parameters were estimated based on published correlations as shown in the following table:

SOIL PARAMETERS ESTIMATED FROM CONE PENETRATION TESTS

Soil Parameter	Published Reference and Year
Soil Behavior Type	Robertson & Wride, 1998
Angle of Internal Friction	Wolff, 1989
	Kulhawy & Mayne, 1990
	Hatanaka & Uchida, 1996
	Senneset et al. 1989
Equivalent SPT N_{60} Value (uncorrected)	Jeffries & Davies, 1993
	Robertson, 1990
In situ Unit Weight	Mayne, 2001

GD5-A.3.4 Cone Penetration Test (CPT) Logs

The CPT logs are presented as Figures GD5-A-14 through GD5-A-25. A CPT log is a written record of the subsurface conditions encountered. It graphically illustrates the cone tip resistance, the friction ratio, the pore pressure, the interpreted soil behavior type, and the correlated SPT N-value with depth.

GD5-A.4 TEST PIT EXCAVATIONS

The test pit excavations performed at the sites consisted of digging and sampling 17 test pits. The excavations were made using a CAT, track-mounted backhoe between December 13, 2008, and January 18, 2009. The 17 test pits, designated SWT-205 through SWT-210, SWT-217, SWT-219, SWT-225, and SWT-227 through SWT-234 were excavated at selected locations along the alignment to depths ranging from 6 to 19 feet.

Disturbed soil samples were collected in test pits SWT-205 through SWT-210, SWT-217, SWT-225, and SWT-227 through SWT-234. The samples were classified and recorded in field logs by our representatives. The samples were then sealed in jars and returned to our laboratory for testing. The results of the laboratory test performed on one test pit sample are presented in Appendix GD5-B.

Piezometer wells were installed in select test pits to allow for potentially stabilized water level measurements at a later date. A description of the test pit wells is provided below:

TEST PIT PIEZOMETER WELLS

Test Pit Designation	Well Depth (ft)	Screen Interval (ft)
SWT-206	12	2-12
SWT-208	14	4-14
SWT-209	15	5-15
SWT-210	15	5-15

The test pit logs are presented as Figures GD5-A-27 through GD5-A-43.

A test pit log is a written record of the subsurface conditions encountered. It graphically illustrates the geologic units (layers) encountered in the test pit and the USCS symbol of each geologic layer. Other information shown in the test pit logs includes the estimated density, a plot of soil sample depth and a description of the groundwater-level observations made during excavation.

GD5-A.5 PREVIOUS SUBSURFACE EXPLORATIONS

The subsurface explorations that were performed by Terracon during RFP are presented in Figures GD5-A-6 through GD5-A-13 (borings) and Figure GD5-A-26 (CPT).

GD5-A.6 SEISMIC CONE LOGS

Shear wave velocities along the Pioneer Crossing alignment were measured by performing two seismic cone tests. One seismic cone test was performed by Terracon in CPT-02 during the RFP phase and one was performed by Shannon & Wilson in SWC-211 during the design phase. The seismic cone logs are presented in Figures GD5-A-44 and GD5-A-45.

GD5-A.7 DISSIPATION TESTS

Dissipation tests were performed along the Pioneer Crossing alignment by Shannon & Wilson during the design phase. The dissipation tests were performed in clayey soils to aid in determining permeability parameters and equalization tests were performed in sandy soils to aid in determining the elevation of the ground water table. See table below for CPT designation and depth:

DISSIPATION TESTS

Cone Penetration Test (CPT)	Depth (ft)
SWC-211	20
SWC-211	25
SWC-212	5
SWC-212	13
SWC-212	19
SWC-212	28
SWC-212	48
SWC-213	10
SWC-214	19
SWC-214	30
SWC-215	28
SWC-215	41
SWC-215	94
SWC-216	18
SWC-216	33

Cone Penetration Test (CPT)	Depth (ft)
SWC-216	93
SWC-218	46
SWC-218	55
SWC-220	8
SWC-220	24
SWC-221	32
SWC-222	9
SWC-222	26
SWC-222	38
SWC-223	31
SWC-223	44
SWC-223	62
SWC-223	78
SWC-224	4
SWC-224	43

GD5-A.8 REFERENCES

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Wolff, T.E., 1989, Pile capacity prediction using parameter functions: Predicted and Observed Axial Behavior of Piles, Results of a Pile Prediction Symposium, Geotechnical Special Publication No. 23.

Shannon & Wilson, Inc. (S&W), uses a soil classification system modified from the Unified Soil Classification System (USCS). Elements of the USCS and other definitions are provided on this and the following page. Soil descriptions are based on visual-manual procedures (ASTM D 2488-93) unless otherwise noted.

S&W CLASSIFICATION OF SOIL CONSTITUENTS

- MAJOR constituents compose more than 50 percent, by weight, of the soil. Major consituents are capitalized (i.e., SAND).
- Minor constituents compose 12 to 50 percent of the soil and precede the major constituents (i.e., silty SAND). Minor constituents preceded by "slightly" compose 5 to 12 percent of the soil (i.e., slightly silty SAND).
- Trace constituents compose 0 to 5 percent of the soil (i.e., slightly silty SAND, trace of gravel).

MOISTURE CONTENT DEFINITIONS

Dry	Absence of moisture, dusty, dry to the touch
Moist	Damp but no visible water
Wet	Visible free water, from below water table

ABBREVIATIONS

ATD	At Time of Drilling
Elev.	Elevation
ft	feet
FeO	Iron Oxide
MgO	Magnesium Oxide
HSA	Hollow Stem Auger
ID	Inside Diameter
in	inches
lbs	pounds
Mon.	Monument cover
N	Blows for last two 6-inch increments
NA	Not applicable or not available
NP	Non plastic
OD	Outside diameter
OVA	Organic vapor analyzer
PID	Photo-ionization detector
ppm	parts per million
PVC	Polyvinyl Chloride
SS	Split spoon sampler
SPT	Standard penetration test
USC	Unified soil classification
WOH	Weight of hammer
WOR	Weight of drill rods
WLI	Water level indicator

GRAIN SIZE DEFINITION

DESCRIPTION	SIEVE NUMBER AND/OR SIZE
FINES	< #200 (0.08 mm)
SAND*	
- Fine	#200 to #40 (0.08 to 0.4 mm)
- Medium	#40 to #10 (0.4 to 2 mm)
- Coarse	#10 to #4 (2 to 5 mm)
GRAVEL*	
- Fine	#4 to 3/4 inch (5 to 19 mm)
- Coarse	3/4 to 3 inches (19 to 76 mm)
COBBLES	3 to 12 inches (76 to 305 mm)
BOULDERS	> 12 inches (305 mm)

* Unless otherwise noted, sand and gravel, when present, range from fine to coarse in grain size.

RELATIVE DENSITY / CONSISTENCY

COARSE-GRAINED SOILS		FINE-GRAINED SOILS	
N, SPT, BLOWS/FT.	RELATIVE DENSITY	N, SPT, BLOWS/FT.	RELATIVE CONSISTENCY
0 - 4	Very loose	Under 2	Very soft
4 - 10	Loose	2 - 4	Soft
10 - 30	Medium dense	4 - 8	Medium stiff
30 - 50	Dense	8 - 15	Stiff
Over 50	Very dense	15 - 30	Very stiff
		Over 30	Hard

WELL AND OTHER SYMBOLS

Bent. Cement Grout	Surface Cement Seal
Bentonite Grout	Asphalt or Cap
Bentonite Chips	Slough
Silica Sand	Bedrock
PVC Screen	
Vibrating Wire	

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SOIL CLASSIFICATION AND LOG KEY

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FIG. GD5-A-1
Sheet 1 of 2

BORING_CLASS1 23-01178.GPJ SWNEW.GDT 2/25/09

UNIFIED SOIL CLASSIFICATION SYSTEM (USCS)
(From ASTM D 2487-98 & 2488-93)

MAJOR DIVISIONS			GROUP/GRAPHIC SYMBOL	TYPICAL DESCRIPTION
COARSE-GRAINED SOILS *(more than 50% retained on No. 200 sieve)*	Gravels *(more than 50% of coarse fraction retained on No. 4 sieve)*	Clean Gravels *(less than 5% fines)*	GW	Well-graded gravels, gravels, gravel/sand mixtures, little or no fines.
			GP	Poorly graded gravels, gravel-sand mixtures, little or no fines
		Gravels with Fines *(more than 12% fines)*	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	Sands *(50% or more of coarse fraction passes the No. 4 sieve)*	Clean Sands *(less than 5% fines)*	SW	Well-graded sands, gravelly sands, little or no fines
			SP	Poorly graded sand, gravelly sands, little or no fines
		Sands with Fines *(more than 12% fines)*	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE-GRAINED SOILS *(50% or more passes the No. 200 sieve)*	Silts and Clays *(liquid limit less than 50)*	Inorganic	ML	Inorganic silts of low to medium plasticity, rock flour, sandy silts, gravelly silts, or clayey silts with slight plasticity
			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
		Organic	OL	Organic silts and organic silty clays of low plasticity
	Silts and Clays *(liquid limit 50 or more)*	Inorganic	MH	Inorganic silts, micaceous or diatomaceous fine sands or silty soils, elastic silt
			CH	Inorganic clays or medium to high plasticity, sandy fat clay, or gravelly fat clay
		Organic	OH	Organic clays of medium to high plasticity, organic silts
HIGHLY-ORGANIC SOILS	Primarily organic matter, dark in color, and organic odor		PT	Peat, humus, swamp soils with high organic content (see ASTM D 4427)

NOTE: No. 4 size = 5 mm; No. 200 size = 0.075 mm

NOTES

1. Dual symbols *(symbols separated by a hyphen, i.e., SP-SM, slightly silty fine SAND)* are used for soils with between 5% and 12% fines or when the liquid limit and plasticity index values plot in the CL-ML area of the plasticity chart.

2. Borderline symbols *(symbols separated by a slash, i.e., CL/ML, silty CLAY/clayey SILT; GW/SW, sandy GRAVEL/gravelly SAND)* indicate that the soil may fall into one of two possible basic groups.

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and I-15 American Fork Interchange
Lehi, Utah

SOIL CLASSIFICATION AND LOG KEY

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FIG. GD5-A-1
Sheet 2 of 2

BORING_CLASS2 23-01178.GPJ SWNEW.GDT 2/25/09

Total Depth:	41.5 ft.	Northing:	~	Drilling Method:	Mud Rotary	Hole Diam.:	3.78 in.
Top Elevation:	~ 4508 ft.	Easting:	~	Drilling Company:	Direct Push Services, Inc	Rod Diam.:	
Vert. Datum:		Station:	~ 604+10 ft.	Drill Rig Equipment:	CME 75	Hammer Type:	Automatic
Horiz. Datum:		Offset:	~ 40'L	Other Comments:			

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

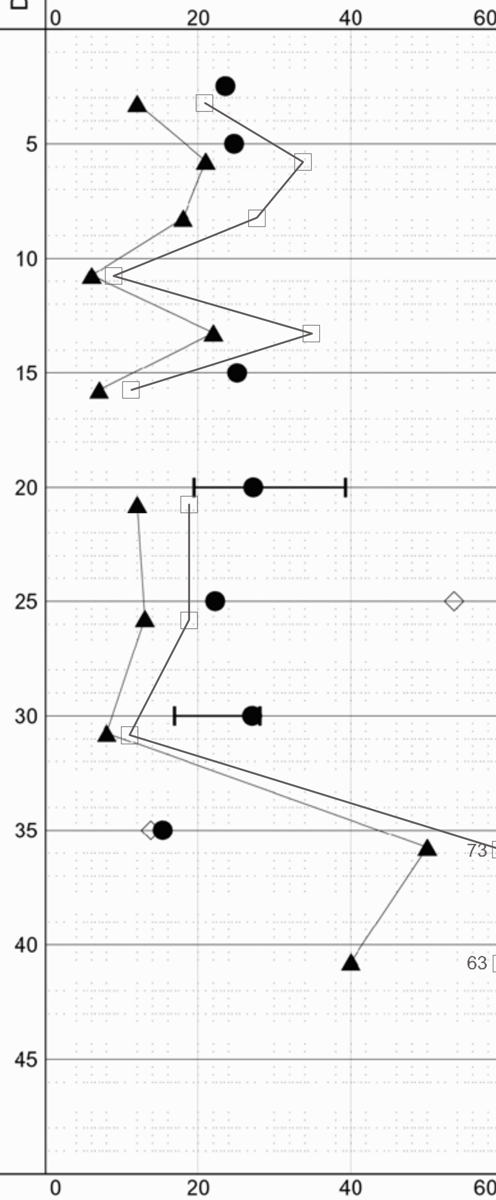
Soil Description	Depth, ft.	Samples
Stiff to very stiff, light brown to brown CLAY; damp grading to moist; CL.		1, 2
Stiff to very stiff, brown, sandy CLAY; moist to wet; CL.	7.0	3
Loose to medium dense, brown, trace to slightly gravelly, clayey SAND; wet; occasional orange/red oxidation; SC.	8.7	4, 5
Medium stiff to stiff, light brown CLAY; wet; trace of sand and gravel below 19 feet; CL. Pp = 500 psf	15.2	6, 7, 8
Stiff, tan to light brown, fine sandy, gravelly, silty CLAY; wet; occasional orange/red oxidations; CL.	23.0	9, 10
Dense to very dense, light brown, slightly silty to silty, fine to coarse sandy GRAVEL; wet; GM/GP-GM.	34.0	11, 12
BOTTOM OF BORING COMPLETED 12/11/2008	41.5	

Ground Water: None Observed During Drilling

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: 140 lbs / 30 inches

□ Corrected Blow Count, $(N_1)_{60}$



LEGEND

* Sample Not Recovered

Standard Penetration Test

Thin Wall Sample

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit

Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. USCS designation is based on visual-manual classification and selected lab testing.
6. The hole location was measured from existing site features and should be considered approximate.

Pioneer Crossing, Lehi
and I-15 American Fork Interchange
Lehi, Utah

LOG OF BORING SWB-201

February 2009 23-1-01178-010

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

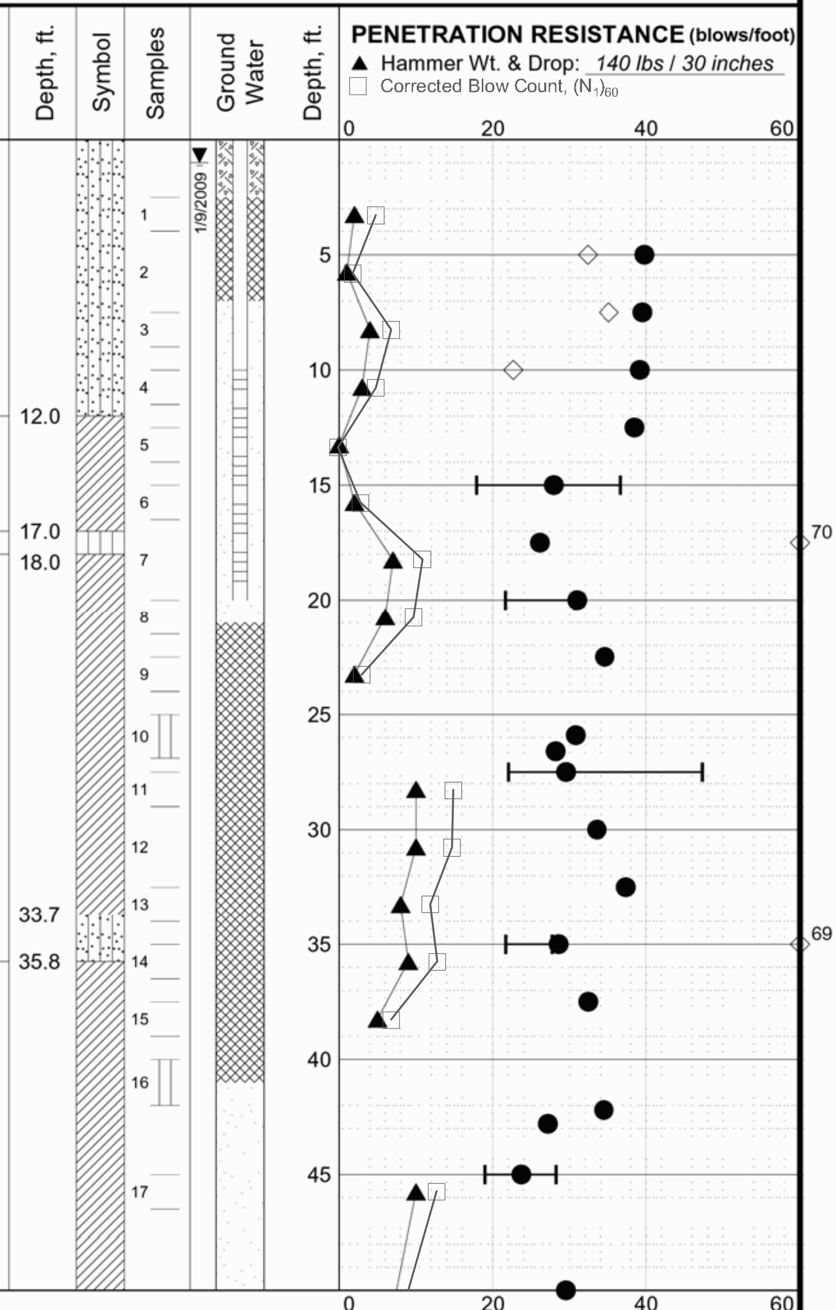
FIG. GD5-A-2

Total Depth:	75.3 ft.	Northing:	~ 306,807 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	3.78 in.
Top Elevation:	~ 4494.1 ft.	Easting:	~ 527,382 ft.	Drilling Company:	Direct Push Services, Inc	Rod Diam.:	
Vert. Datum:		Station:	~ 625+21.7 ft.	Drill Rig Equipment:	CME 75	Hammer Type:	Automatic
Horiz. Datum:		Offset:	~ 14'R	Other Comments:			

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Soil Description	Depth, ft.
Very loose, brown-gray, clayey, silty, fine SAND; wet; trace to scattered shells and organics; SM.	
Soft to very soft, gray-brown, silty CLAY, trace of fine to medium sand and fine gravel; wet; scattered hard tan pockets, trace of organics; CL.	12.0
Loose, gray, slightly clayey, sandy SILT; wet; ML/SM.	17.0
Soft to stiff, brown to gray, locally slightly sandy, locally silty CLAY; moist; local pockets of cemented silt/clay pieces; CL/CH.	18.0
Loose to medium dense, brown-red, clayey, silty, fine SAND; wet; SM.	33.7
Medium stiff to stiff, light brown, sandy grading to slightly sandy, silty CLAY; wet; CL/ML.	35.8



CONTINUED NEXT SHEET

LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- Thin Wall Sample
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout
- ▼ Ground Water Level in Well

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit

Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. USCS designation is based on visual-manual classification and selected lab testing.
6. The hole location was measured from existing site features and should be considered approximate.

Pioneer Crossing, Lehi
and I-15 American Fork Interchange
Lehi, Utah

LOG OF BORING SWB-202

February 2009 | 23-1-01178-010

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. GD5-A-3
Sheet 1 of 2

Total Depth:	75.3 ft.	Northing:	~ 306,807 ft.	Drilling Method:	Mud Rotary
Top Elevation:	~ 4494.1 ft.	Easting:	~ 527,382 ft.	Drilling Company:	Direct Push Services, Inc
Vert. Datum:		Station:	~ 625+21.7 ft.	Drill Rig Equipment:	CME 75
Horiz. Datum:		Offset:	~ 14'R	Other Comments:	

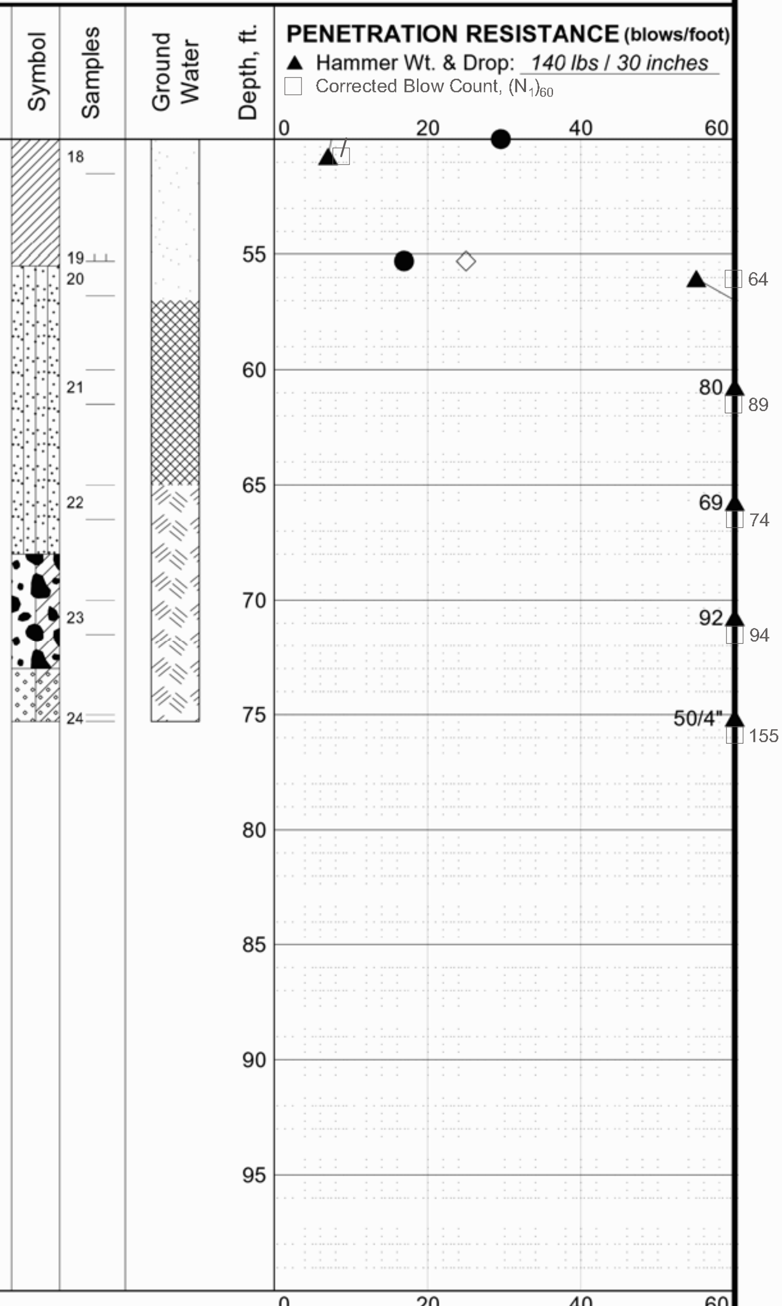
Hole Diam.:	3.78 in.
Rod Diam.:	
Hammer Type:	Automatic

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Soil Description	Depth, ft.
	55.5
Very dense, orange-brown, slightly gravelly to gravelly, clayey, silty, medium to coarse SAND; wet; SM/SC. - Artesian pressure below +/- 65 feet.	68.0
Very dense, brown, slightly clayey, sandy, fine to coarse GRAVEL; wet; GW-GC. - Clay grading out with depth.	73.0
Very dense, orange-brown, slightly clayey, gravelly, coarse SAND; wet; SW-SC.	75.3

BOTTOM OF BORING
COMPLETED 12/16/2008



LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- Thin Wall Sample
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout
- ▼ Ground Water Level in Well

◇ % Fines (<0.075mm)
● % Water Content
Plastic Limit ├─●─┤ Liquid Limit
Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. USCS designation is based on visual-manual classification and selected lab testing.
6. The hole location was measured from existing site features and should be considered approximate.

Pioneer Crossing, Lehi
and I-15 American Fork Interchange
Lehi, Utah

LOG OF BORING SWB-202

February 2009 23-1-01178-010

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. GD5-A-3
Sheet 2 of 2

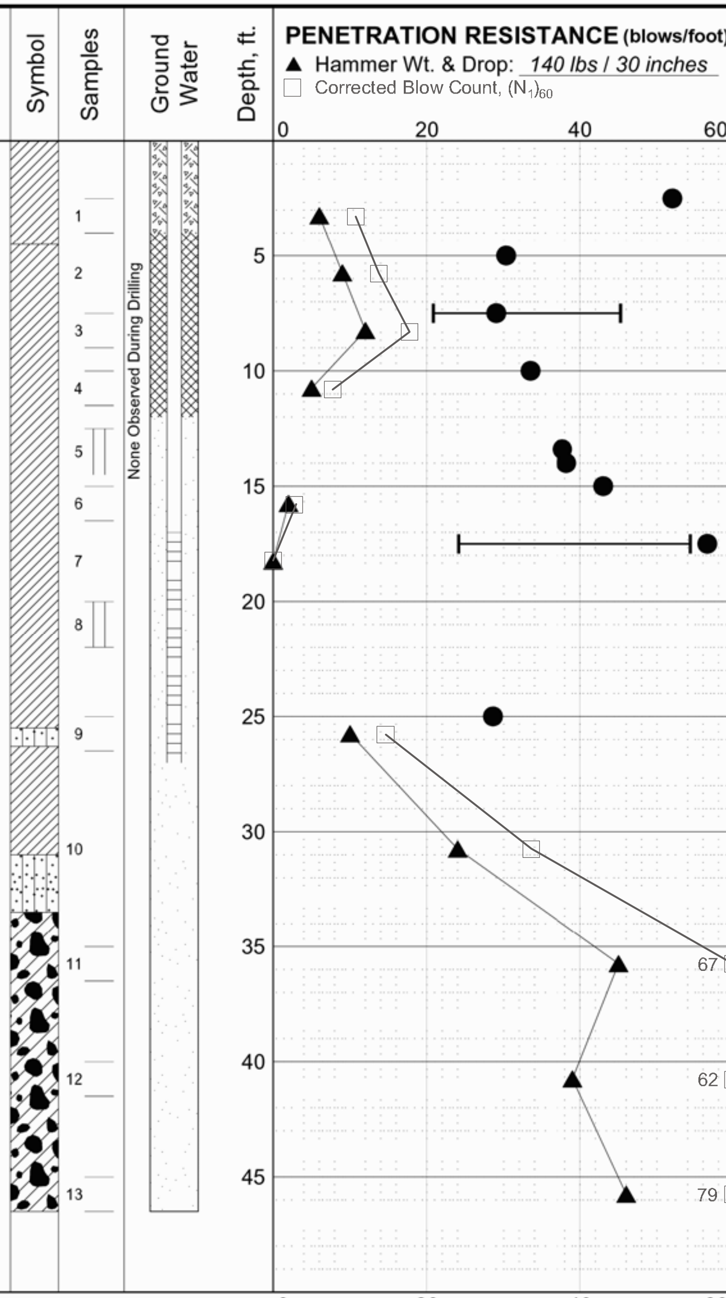
Total Depth:	46.5 ft.	Northing:	~ 307,882 ft.	Drilling Method:	Mud Rotary	Hole Diam.:	3.78 in.
Top Elevation:	~ 4492.6 ft.	Easting:	~ 528,623 ft.	Drilling Company:	Direct Push Services, Inc	Rod Diam.:	
Vert. Datum:		Station:	~ 642+02.0 ft.	Drill Rig Equipment:	CME 75	Hammer Type:	Automatic
Horiz. Datum:		Offset:	~ 40.5'R	Other Comments:			

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Soil Description	Depth, ft.
Medium stiff, dark brown to black CLAY, trace organics; moist; occasional tan pockets; CL/CH.	4.5
Soft to stiff, gray, silty CLAY; moist; locally slightly fine sandy, locally trace organics; CL/CH. Pp = 500 psf - Very soft below 14.5 feet.	25.5
Loose, dark brown-yellow, silty, fine SAND; wet; SM.	26.3
Medium stiff to stiff, tan, sandy, silty CLAY; moist; CL.	31.0
Medium dense, light brown, slightly silty, clayey, gravelly, medium to coarse SAND; wet; SM/SC.	33.5
Dense, brown and gray, clayey, sandy GRAVEL; wet; GC.	46.5

BOTTOM OF BORING
COMPLETED 12/12/2008



LEGEND

- * Sample Not Recovered
- Standard Penetration Test
- Thin Wall Sample
- Piezometer Screen and Sand Filter
- Bentonite-Cement Grout
- Bentonite Chips/Pellets
- Bentonite Grout

◇ % Fines (<0.075mm)
● % Water Content
Plastic Limit ├─●─┤ Liquid Limit
Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. USCS designation is based on visual-manual classification and selected lab testing.
6. The hole location was measured from existing site features and should be considered approximate.

Pioneer Crossing, Lehi
and I-15 American Fork Interchange
Lehi, Utah

LOG OF BORING SWB-203

February 2009 23-1-01178-010

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. GD5-A-4

Total Depth:	*95.5 ft.*	Northing:	*~*	Drilling Method:	*Mud Rotary*
Top Elevation:	*~ 4492 ft.*	Easting:	*~*	Drilling Company:	*Direct Push Services, Inc*
Vert. Datum:		Station:	*~ 653+00 ft.*	Drill Rig Equipment:	*CME 75*
Horiz. Datum:		Offset:	*~ Centerline*	Other Comments:	

Hole Diam.:	*3.78 in.*
Rod Diam.:	
Hammer Type:	*Automatic*

SOIL DESCRIPTION

Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

Soil Description	Depth, ft.	Samples
Medium dense, light brown, silty, fine SAND; damp; trace of small shells, trace of fine organics; oxidized orange to red; SM.		1, 2
Medium stiff to stiff, gray, silty CLAY; moist; trace fine sand pockets, occasional orange oxidation; CL/CH.	6.0	3, 4, 5, 6, 7
Loose to medium dense, gray SILT; wet; ML.	20.0	
Very soft, gray CLAY; wet; locally trace fine organics; CL/CH. Pp = 1,500 psf Tv = 600 psf	20.2	8, 9, 10
- Gray, silty, fine SAND lense. Pp = 250 psf Tv = 200 psf		11, 12
- Gray, silty, fine sand to sandy silt pocket.		13

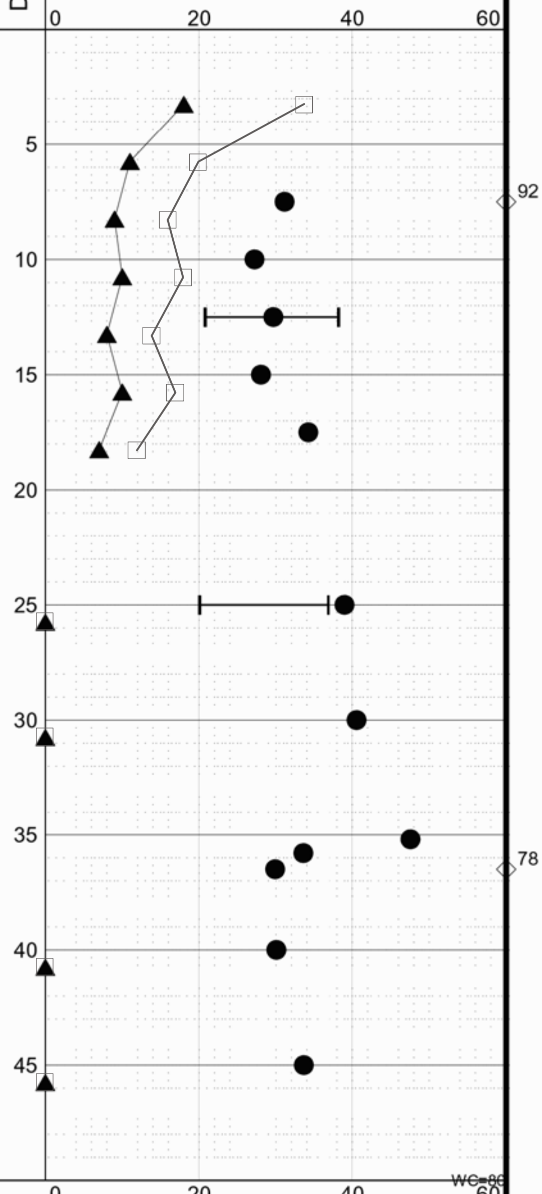
Ground Water: None Observed During Drilling

CONTINUED NEXT SHEET

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*

□ Corrected Blow Count, $(N_1)_{60}$



◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit

Natural Water Content

LEGEND

* Sample Not Recovered

Standard Penetration Test

Thin Wall Sample

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. USCS designation is based on visual-manual classification and selected lab testing.
6. The hole location was measured from existing site features and should be considered approximate.

Pioneer Crossing, Lehi
and I-15 American Fork Interchange
Lehi, Utah

LOG OF BORING SWB-204

February 2009 23-1-01178-010

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. GD5-A-5

Sheet 1 of 2

Total Depth:	*95.5 ft.*	Northing:	~	Drilling Method:	*Mud Rotary*	Hole Diam.:	*3.78 in.*
Top Elevation:	*~ 4492 ft.*	Easting:	~	Drilling Company:	*Direct Push Services, Inc.*	Rod Diam.:	
Vert. Datum:		Station:	*~ 653+00 ft.*	Drill Rig Equipment:	*CME 75*	Hammer Type:	*Automatic*
Horiz. Datum:		Offset:	*~ Centerline*	Other Comments:			

SOIL DESCRIPTION

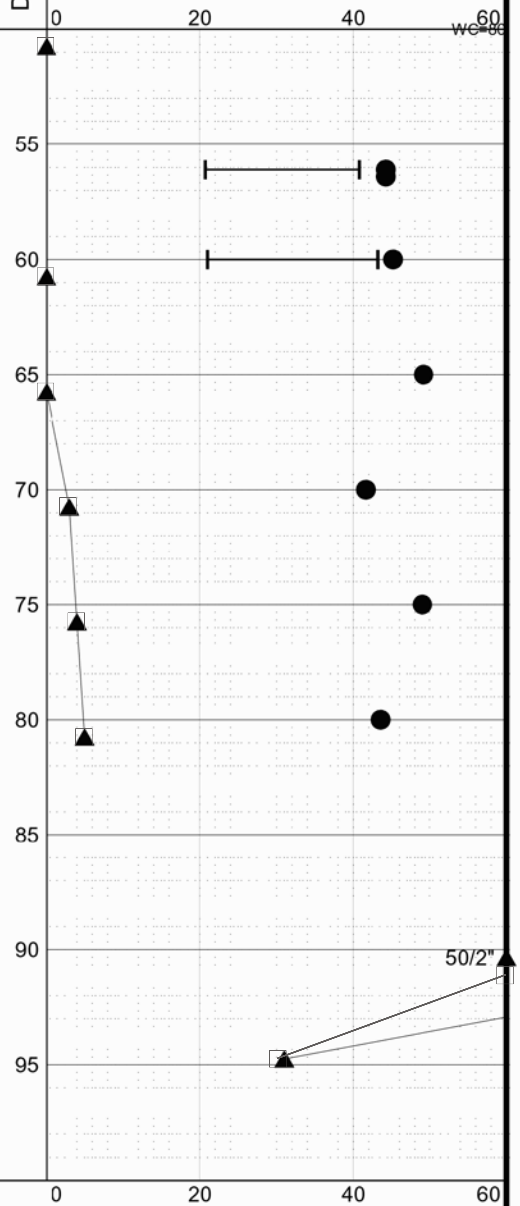
Refer to the report text for a proper understanding of the subsurface materials and drilling methods. The stratification lines indicated below represent the approximate boundaries between material types, and the transition may be gradual.

SOIL DESCRIPTION	Depth, ft.	Samples
		14
		15
		16
- Grading to dark gray to black below approximately 63 feet. - Grades to soft.		17
		18
		19
- Grades to medium stiff.		20
Pp = 800 psf		21
Very dense, dark gray, slightly clayey, slightly sandy, fine to coarse GRAVEL; moist; GC.	89.0	22
Interbedded, dense, gray, slightly clayey, sandy GRAVEL and stiff, light gray, slightly sandy CLAY; moist to wet; GC, CH.	92.3	23
BOTTOM OF BORING COMPLETED 12/14/2008	95.5	

PENETRATION RESISTANCE (blows/foot)

▲ Hammer Wt. & Drop: *140 lbs / 30 inches*

□ Corrected Blow Count, $(N_1)_{60}$



LEGEND

* Sample Not Recovered

Standard Penetration Test

Thin Wall Sample

◇ % Fines (<0.075mm)

● % Water Content

Plastic Limit ├─●─┤ Liquid Limit

Natural Water Content

NOTES

1. Refer to KEY for explanation of symbols, codes, abbreviations and definitions.
2. The stratification lines represent the approximate boundaries between soil types, and the transition may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the nature of the subsurface materials.
4. Groundwater level, if indicated above, is for the date specified and may vary.
5. USCS designation is based on visual-manual classification and selected lab testing.
6. The hole location was measured from existing site features and should be considered approximate.

Pioneer Crossing, Lehi
and I-15 American Fork Interchange
Lehi, Utah

LOG OF BORING SWB-204

February 2009 23-1-01178-010

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. GD5-A-5
Sheet 2 of 2

LOG OF BORING NO. B-01

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: **West of Jordan River near Redwood Road**

GRAPHIC LOG	Description	DEPTH, ft.	USCS SYMBOL	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES RECOVERY, in.	SAMPLES PENETRATION RESISTANCE BLOWS / ft.	TESTS WATER CONTENT, %	TESTS DRY UNIT WEIGHT, PCF	TESTS LIQUID LIMIT	TESTS PLASTICITY INDEX	TESTS % PASSING NO. 200 SIEVE	PID
0.25	**TOPSOIL:** clay, approximately 3 inches thick												
	SILTY CLAY (CL-ML): very stiff to hard, brown with white and red mottling, with trace gravel,	3		1	SS	10	41						
		6		2	SS	16	56						
		8		3	SS	18	27						
12.5		11		4	SS	18	20						
	CLAY (CL): with sand, very stiff, red to brown	13		5	SS	18	18						
20		16		6	SS	18	17						
21.5	**SANDY CLAY (CL):** medium stiff, red to brown	21		7	SS	18	4						
	BOTTOM OF BORING AT APPROXIMATELY 21.5 FEET												

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	▽ 20	WD	▼
WL	▼		▼
WL			

Terracon

BORING STARTED			3-18-08
BORING COMPLETED			3-18-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

FIG. GD5-A-6
(SHEET 1 OF 2)

LOG OF BORING NO. B-01A

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: **West of Jordan River near Redwood Road**

Approx. Surface Elev.: 4518.3 ft

GRAPHIC LOG / DESCRIPTION	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, in.	SAMPLES: PENETRATION RESISTANCE BLOWS / ft.	TESTS: WATER CONTENT, %	TESTS: DRY UNIT WEIGHT, PCF	TESTS: LIQUID LIMIT	TESTS: PLASTICITY INDEX	TESTS: % PASSING NO. 200 SIEVE	PID
1 **SILTY SAND (SM):** red-brown, with organics with trace gravel	1											
SILTY CLAY (CL-ML): soft to hard, brown, with trace gravel	2											
	3		1	SS	6	33	9					
	4											
	5											
	6		2	SS	5	14						pH, Res.
	7											
	8		3	SS	5	6						
	9											
	10											
	11		4	SS	3	5	11					
	12											
	13		5	SS	1	4						
	14											
	15											
16.5	16		6	SS	3	2						

BOTTOM OF BORING AT
APPROXIMATELY 16.5 FEET

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	∇ NE	WD	▼
WL	▼		∇
WL			

Terracon

BORING STARTED			3-18-08
BORING COMPLETED			3-18-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

FIG. GD5-A-6
(SHEET 2 OF 2)

LOG OF BORING NO. B-02

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: **Between Jordan River and Redwood Road**

Approx. Surface Elev.: 4502.6 ft

Depth (graphic log)	Description	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	TYPE	RECOVERY, in.	PENETRATION RESISTANCE BLOWS / ft.	TESTS: WATER CONTENT, %	DRY UNIT WEIGHT, PCF	LIQUID LIMIT	PLASTICITY INDEX	% PASSING NO. 200 SIEVE	PID
0.5	**TOPSOIL:** approximately 6 inches thick												
	SILT (MH): medium stiff, brown to gray	3		B	BS	48	---	41		52	18	71	
		4		1	SS	0	4						
		6		2	SS	8	4						
7.5	**LEAN CLAY (CL):** soft to medium stiff, gray to brown, trace organics	8		3	SS	14	4						ph, Res.
		11		4	SS	1	5						uu triax
		14		5	ST	24	---			28	10		
		16		6	SS	18	2						
20	**SANDY CLAY (CL):** soft, brown	21		7	SS	15	2						
21.5	BOTTOM OF BORING AT APPROXIMATELY 21.5 FEET												

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	∇ 8	WD	▼
WL	▼		▼
WL			

Terracon

BORING STARTED	3-18-08		
BORING COMPLETED	3-18-08		
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

FIG. GD5-A-7

LOG OF BORING NO. B-03

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: **Jordan River Bridge near West Abutment**

Approx. Surface Elev.: **4497.4 ft**

Strata depth (ft)	Description	Sample Depth, ft.	USCS Symbol	Sample Number	Sample Type	Recovery, in.	Penetration Resistance Blows / ft.	Water Content, %	Dry Unit Weight, PCF	Liquid Limit	Plasticity Index	% Passing No. 200 Sieve	PID
0–1	**SILTY CLAY (CL-ML):** brown, with organics												
1–5	**CLAYEY SAND (SC):** medium dense, brown, trace organics	3–4.5		1	SS	8	20						
5–15	**LEAN CLAY (CL):** soft to medium stiff, brown to gray, with organics, trace sand	5.5–7		2	SS	18	3						
		10–12		3	ST	24	--			38	11		uu triax pH, Res.
		12.5–14		4	SS	18	4						
15 ▽	**SANDY LEAN CLAY:** medium to stiff, brown to gray, some organics	15–16.5		5	SS	18	4						
		20–22		6	ST	24	--			29	10		uu triax
		25–26.5		7	SS	18	8						
		30–31.5		8	SS	18	5	33				67	
		35–36.5		9	SS	18	5						

Continued Next Page

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	▽ 15	WD	▼
WL	▼		▼
WL			

Terracon

BORING STARTED			3-19-08
BORING COMPLETED			3-19-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LSI ENV SLC 61085026.GPJ TERRACON.GDT 6/12/08

FIG. GD5-A-8
(SHEET 1 OF 2)

LOG OF BORING NO. B-03

CLIENT: **HDR Inc**

SITE: **I-15 to Redwood Road** **American Fork, Lehi, Saratoga Springs, UT**

PROJECT: **East to West Connector; I-15 to Redwood Road**

Graphic log depth	Description	DEPTH, ft.	USCS SYMBOL	NUMBER	TYPE	RECOVERY, in.	PENETRATION RESISTANCE BLOWS / ft.	WATER CONTENT, %	DRY UNIT WEIGHT, PCF	LIQUID LIMIT	PLASTICITY INDEX	% PASSING NO. 200 SIEVE	PID
	SANDY LEAN CLAY: medium to stiff, brown to gray, some organics	41		10	SS	12	27	32				51	
		46		11	SS	15	3						
		51		12	SS	16	14						
53	**CLAYEY SAND (SC):** very dense, red-brown	56		13	SS	16	50	26				50	
60	**SILTY SAND(SM):** very dense, brown, with gravel	61		14	SS	12	50/5						
		65		15	SS	3	50/5	15				40	
70	**GRAVEL (GP-GM):** very dense, gray	70		16	SS	2	50/1						
75		75		17	SS	7	50/6	13				25	
75.5	**SILTY SAND (SM):** very dense, gray to brown, with gravel												
	AUGER REFUSAL AT APPROXIMATELY 75.5 FEET												

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL	∇ 15	WD ▼
WL	∇	∇
WL		

Terracon

BORING STARTED			3-19-08
BORING COMPLETED			3-19-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LSI ENV SLC 61085026.GPJ TERRACON.GDT 6/12/08

FIG. GD5-A-8

(SHEET 2 OF 2)

LOG OF BORING NO. B-04

CLIENT: **HDR Inc**

SITE: **I-15 to Redwood Road, American Fork, Lehi, Saratoga Springs, UT**

PROJECT: **East to West Connector; I-15 to Redwood Road**

Boring Location: Approximately 2600 West

Approx. Surface Elev.: 4499.4 ft

Depth (strata)	Description	DEPTH, ft.	USCS SYMBOL	NUMBER	TYPE	RECOVERY, in.	PENETRATION RESISTANCE BLOWS / ft.	WATER CONTENT, %	DRY UNIT WEIGHT, PCF	LIQUID LIMIT	PLASTICITY INDEX	% PASSING NO. 200 SIEVE	PID
1	**TOPSOIL:** approximately 1 foot thick: sandy clay, dark brown, with organics												
5	**SANDY CLAY:** with sand lenses, soft, brown, trace organics	3		1	SS	15	3						
7.5	**SILTY SAND:** loose, brown	6		2	SS	10	6	19				16	
10	**SANDY CLAY:** soft, gray-brown	8		3	SS	10	2						
12.5	**CLAYEY SAND:** medium dense, gray-brown with orange mottling, trace organics	11		4	SS	18	18						
21.5	**CLAY:** with sand lenses, stiff to hard, gray to brown gray, with orange mottling	13		5	SS	10	26	23		34	18		
		16		6	SS	18	28						
		21		7	SS	15	7						
	BOTTOM OF BORING AT APPROXIMATELY 21.5 FEET												

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	∇ 2	WD	▼
WL	▼		▼
WL			

Terracon

BORING STARTED			4-2-08
BORING COMPLETED			4-2-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LSI ENV SLC 61085026.GPJ TERRACON.GDT 6/12/08

FIG. GD5-A-9

LOG OF BORING NO. B-05

CLIENT **HDR Inc**

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT **East to West Connector; I-15 to Redwood Road**

Boring Location: 2300 West Intersection

Approx. Surface Elev.: 4504.7 ft

Depth (strata)	Description	DEPTH, ft.	USCS SYMBOL	SAMPLES NUMBER	TYPE	RECOVERY, in.	PENETRATION RESISTANCE BLOWS / ft.	WATER CONTENT, %	DRY UNIT WEIGHT, PCF	LIQUID LIMIT	PLASTICITY INDEX	% PASSING NO. 200 SIEVE	PID
1	**TOPSOIL:** sandy clay, dark brown, with organics												
4	**SANDY CLAY (CL):** medium stiff, dark brown, trace organics	3		1	SS	2	4						CBR Proctor
6	**SAND (SP-SM):** loose, tan to gray	6		2	SS	16	6						
7.5	**CLAY:** medium stiff, dark brown												
20.5	**SILTY SAND (SM):** loose to dense, gray to tan, with trace gravel and occasional clay layers ∇	8		3	ST	20	---			NP	NP		uu triax pH, Res.
		11		4	SS	6	6						
		13		5	SS	14	12						
24	**CLAY (CL):** hard, brown	20		6	SS	14	35						
26.5	**SANDY CLAY (CL):** very stiff, brown to gray	25		7	SS	10	21						

BOTTOM OF BORING AT APPROXIMATELY 26.5 FEET

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	∇ 10	WD	▼
WL	▼		▼
WL			

Terracon

BORING STARTED			3-18-08
BORING COMPLETED			3-18-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LSI ENV SLC 61085026.GPJ TERRACON.GDT 6/12/08

FIG. GD5-A-10

LOG OF BORING NO. B-08

CLIENT
HDR Inc

SITE I-15 to Redwood Road
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: Jordan River Bridge; East Abutment

Approx. Surface Elev.: 4499.2 ft

Depth to (ft)	Description	Sample No.	Type	Recovery, in.	Penetration Resistance Blows / ft.	Water Content, %	Dry Unit Weight, PCF	Liquid Limit	Plasticity Index	% Passing No. 200 Sieve	PID
0.5	**TOPSOIL:** silty clay, brown, with organics, approximately 6 inches thick										
2	**SANDY CLAY(CL):** brown, trace organics	1 (3 ft)	SS	8	28						
5	**SAND (SP-SM):** medium dense, brown to gray, with gravel	2 (6 ft)	SS	8	57						
10	**SANDY LEAN CLAY (CL):** very stiff to hard, brownto gray, trace organics, with gravel	3 (8 ft)	SS	13	13						
16	**CLAYEY SAND (SC):** loose to medium dense, brown to gray, with organics	4 (11 ft)	ST	18	---	37	84				uu triax
		5 (13 ft)	SS	18	11						
		6 (16 ft)	SS	14	5						
20	**CLAY (CL):** medium stiff, gray, with organics										
24	**SANDY CLAY (CL):** stiff, brown, with organics, trace gravel	7 (21 ft)	SS	16	10						
40	**CLAY (CL):** medium stiff to stiff, red to brown	8 (26 ft)	SS	16	4						
		9 (31 ft)	ST	22	---	27	99	37	23		uu triax
		10 (36 ft)	SS	18	11						

Continued Next Page

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL	▽ 12	WD ▼
WL	▼	▼
WL		

Terracon

BORING STARTED	4-1-08
BORING COMPLETED	4-2-08
RIG B-80	FOREMAN DAF
LOGGED DAF	JOB # 61085026

FIG. GD5-A-11
(SHEET 1 OF 2)

LOG OF BORING NO. B-08

CLIENT
HDR Inc

SITE I-15 to Redwood Road
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

DESCRIPTION	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	TYPE	RECOVERY, in.	PENETRATION RESISTANCE BLOWS / ft.	TESTS: WATER CONTENT, %	DRY UNIT WEIGHT, PCF	LIQUID LIMIT	PLASTICITY INDEX	% PASSING NO. 200 SIEVE	PID
CLAYEY SAND (SC): medium dense, red to brown	41		11	SS	9	12						
45												
SANDY CLAY (CL): very stiff, red to brown, trace organics	46		12	SS	18	14						
	51		13	ST	20	---	26	99	26	9		uu triax
55												
CLAY (CL): hard, red to brown, trace sand	56		14	SS	18	50/6	27				73	
57												
SAND (SP-SM): very dense, brown to gray, with gravel	60		15	SS	14	50/4						
65												
GRAVEL (GP-GM): very dense, gray	65		16	SS	3	50/1						
	70		17	SS	3	50/5						
75												
76 **SANDY GRAVEL (GP):** very dense, brown to gray, trace clay	75		18	SS	3	50/5						
AUGER REFUSAL AT APPROXIMATELY 76 FEET	76											

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL	∇ 12	WD ▼
WL	▼	▼
WL		

Terracon

BORING STARTED			4-1-08
BORING COMPLETED			4-2-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LSI ENV SLC 61085026.GPJ TERRACON.GDT 6/12/08

FIG. GD5-A-11
(SHEET 2 OF 2)

LOG OF BORING NO. B-09

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: Approximately 2000 West

Approx. Surface Elev.: 4511.1 ft

GRAPHIC LOG (depth, ft)	DESCRIPTION	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, in.	SAMPLES: PENETRATION RESISTANCE BLOWS / ft.	TESTS: WATER CONTENT, %	TESTS: DRY UNIT WEIGHT, PCF	TESTS: LIQUID LIMIT	TESTS: PLASTICITY INDEX	TESTS: % PASSING NO. 200 SIEVE	PID
1	**TOPSOIL:** sandy clay, dark brown, with organics, approximately 1 foot thick												
5	**SILTY SAND (SM):** medium dense, brown, trace gravel ▽	3		1	SS	4	10						
7.5	**SANDY CLAY (CL):** brown	6		2	ST	18	---	12				31	uu triax
12.5	**CLAYEY SAND (SC):** very loose to loose, brown to gray	8		3	SS	12	7						
		11		4	SS	1	2						
19	**CLAY (CL):** medium stiff, brown, trace sand	13		5	SS	18	4						
		16		6	ST	24	---						uu triax
21.5	**SAND (SP-SM):** dense, orange-brown	21		7	SS	10	35	21				15	

BOTTOM OF BORING AT APPROXIMATELY 21.5 FEET

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	▽ 5	WD	▼
WL	▼		▼
WL			

Terracon

BORING STARTED			4-2-08
BORING COMPLETED			4-2-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LOG OF BORING NO. B-10

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: 1700 West Intersection

Approx. Surface Elev.: 4514.1 ft

GRAPHIC LOG (depth, ft)	Description	DEPTH, ft.	USCS SYMBOL	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES RECOVERY, in.	SAMPLES PENETRATION RESISTANCE BLOWS / ft.	TESTS WATER CONTENT, %	TESTS DRY UNIT WEIGHT, PCF	TESTS LIQUID LIMIT	TESTS PLASTICITY INDEX	TESTS % PASSING NO. 200 SIEVE	PID
1	**TOPSOIL:** silty clay, dark brown, with organics												
7.5	**SILTY SAND (SM):** loose to medium dense, brown, trace clay and organics	3		1	SS	10	9						
		6		2	SS	0	10						
10	**CLAYEY SAND (SC):** very loose, brown	8		3	SS	9	2						
12.5	**SANDY CLAY (CL):** soft, brown	11		4	SS	6	3						
16	**CLAY (CL):** medium stiff, brown, trace sand	13		5	SS	18	6						
24	**CLAYEY SAND (SC):** medium dense, brown	16		6	ST	24	---	26	87	28	10		
		21		7	SS	7	10						
26.5	**CLAY (CL):** very stiff, gray with orange mottling	26		8	SS	16	26						

BOTTOM OF BORING AT APPROXIMATELY 26.5 FEET

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	∇ 8	WD	▼
WL	▼		▼
WL			

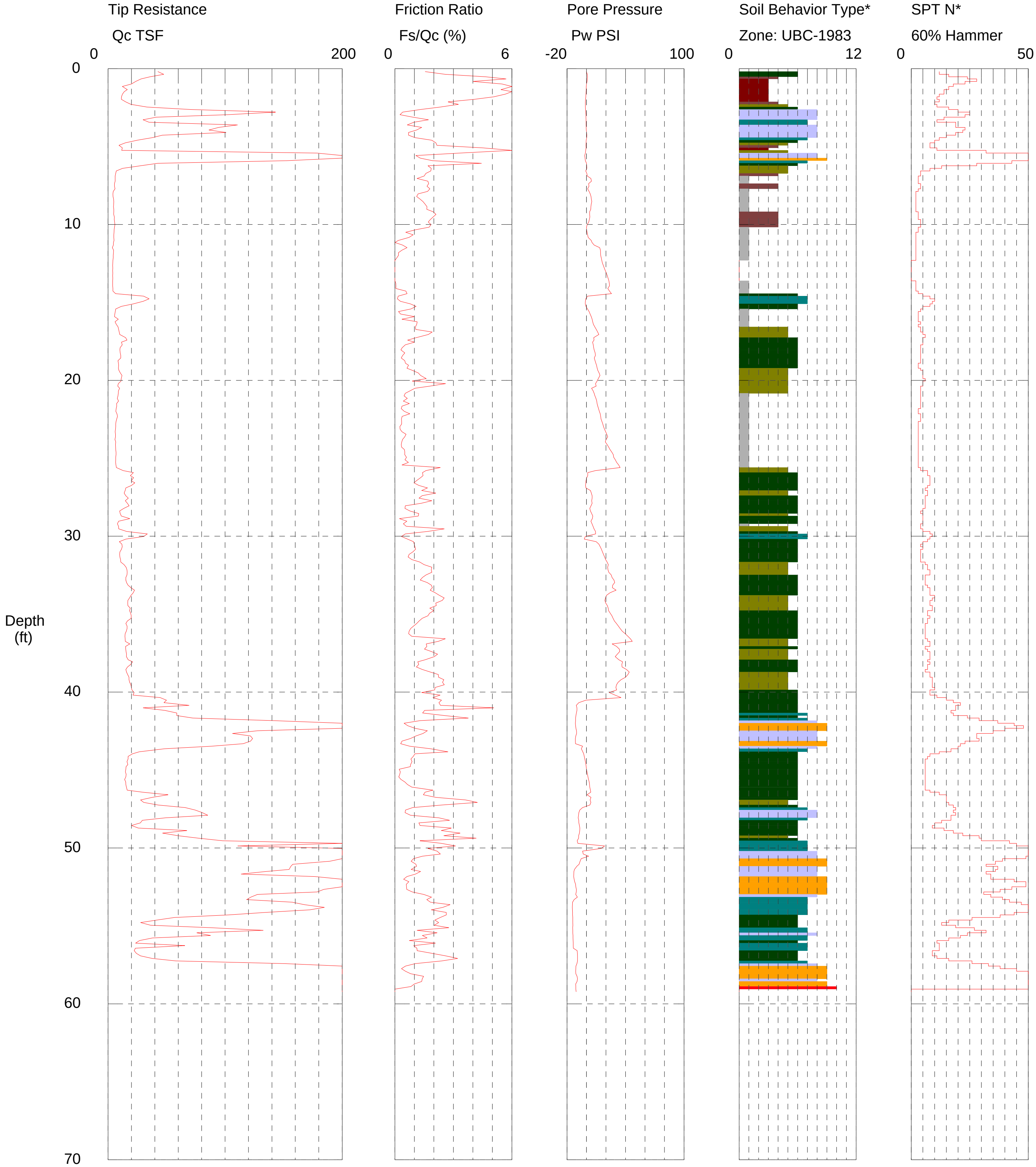
Terracon

BORING STARTED			4-3-08
BORING COMPLETED			4-3-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

Shannon & Wilson

Operator: Brown
Sounding: SWC 211
Cone Used: DSG1065

CPT Date/Time: 12/12/2008 10:48:09 AM
Location: Pioneer Crossing Sta 622+80, Centerline, Elev. ~4498'
Job Number: 21-1-01178-010



Maximum Depth = 59.22 feet

Depth Increment = 0.164 feet

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

Location of test was 5 ft west of staked position.

Seismic Testing performed on this test
In Situ Engineering

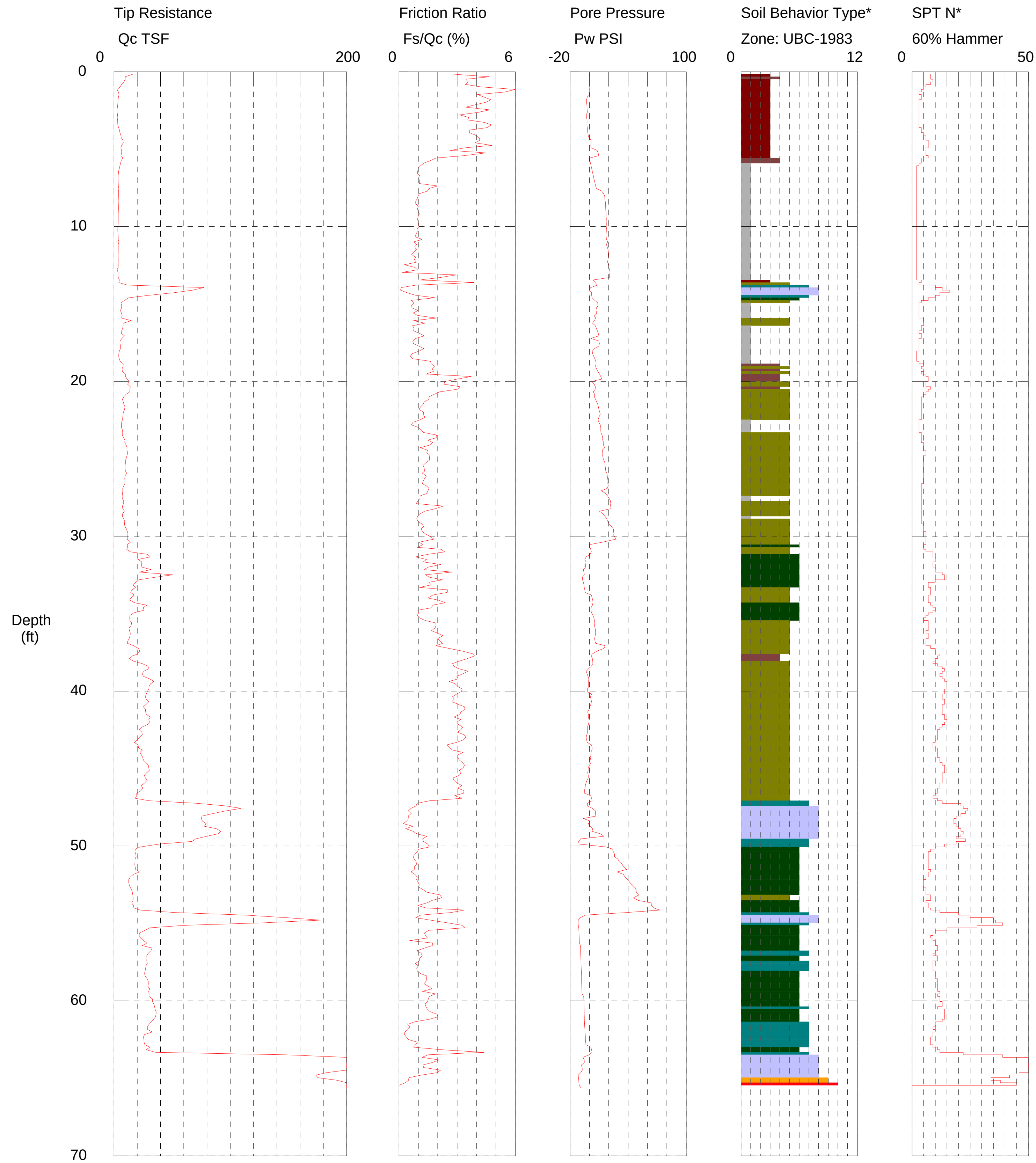
FIG. GD5-A-14

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

Shannon & Wilson

Operator: Brown
Sounding: SWC 212
Cone Used: DSG1065

CPT Date/Time: 12/13/2008 11:32:18 AM
Location: Pioneer Crossing STA 631+20, +/- 40' R, Elev. ~4492'
Job Number: 21-1-01178-010



Maximum Depth = 65.62 feet
Depth Increment = 0.164 feet

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

FIG. GD5-A-15

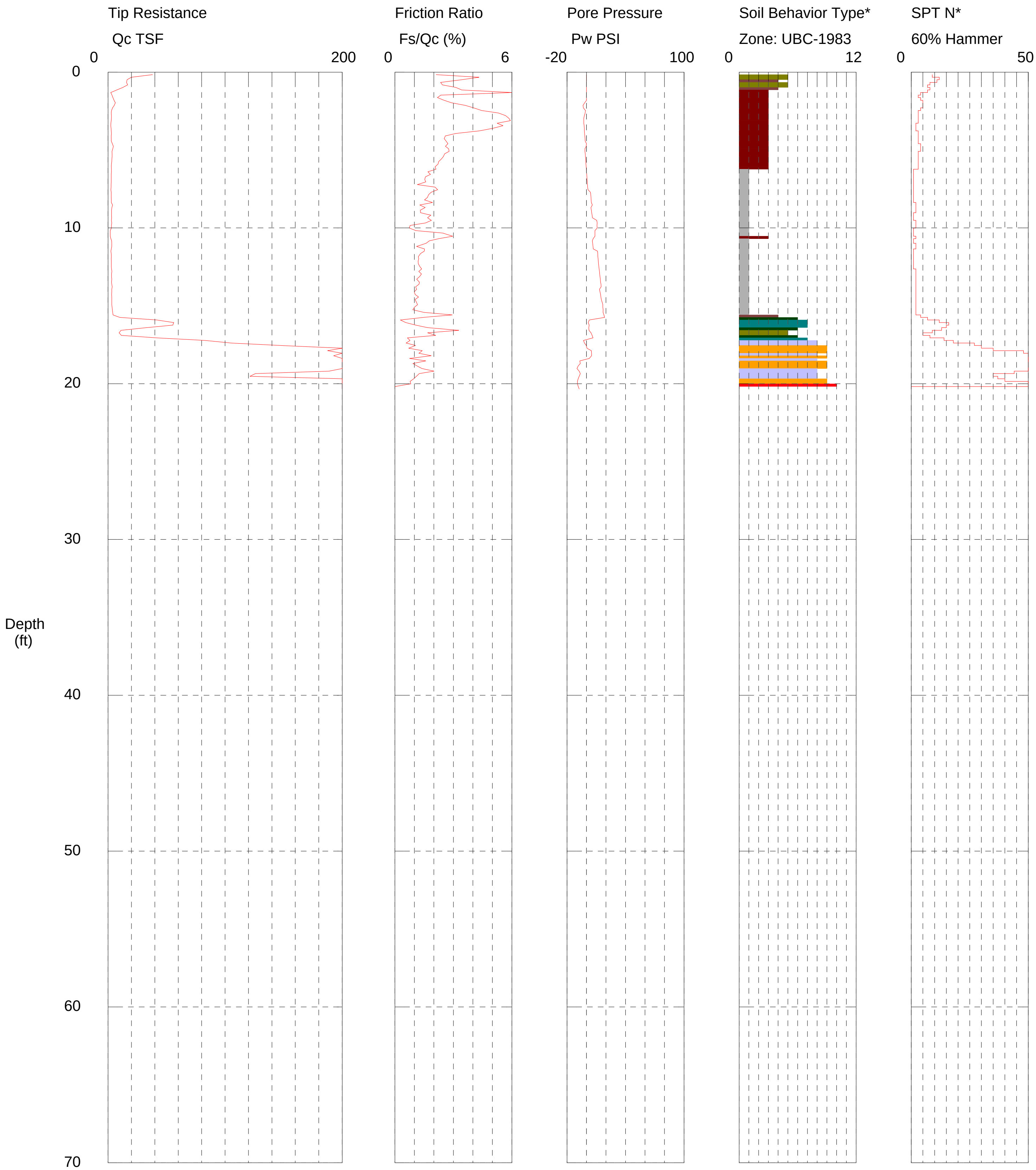
In Situ Engineering

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

Shannon & Wilson

Operator: Brown
Sounding: SWC 213
Cone Used: DSG1065

CPT Date/Time: 12/13/2008 9:35:42 AM
Location: Pioneer Crossing STA 638+15, +/- 40' R, Elev. ~4491'
Job Number: 21-1-01178-010



Maximum Depth = 20.34 feet

Depth Increment = 0.164 feet

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

FIG. GD5-A-16

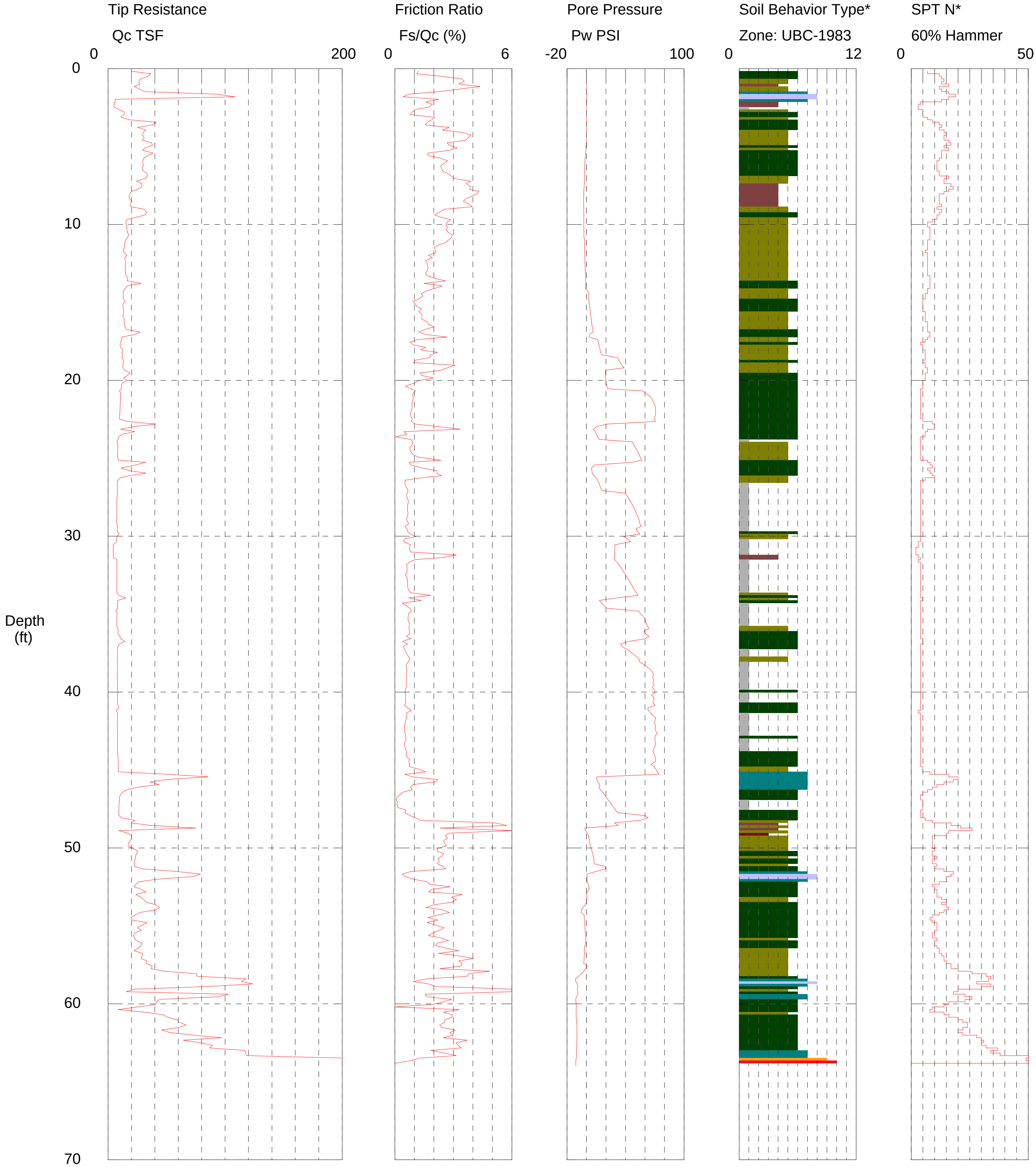
In Situ Engineering

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

Shannon & Wilson

Operator: Brown
Sounding: SWC 214
Cone Used: DSG1065

CPT Date/Time: 12/14/2008 9:29:37 AM
Location: Pioneer Crossing STA 649+00, Centerline, Elev. ~4492'
Job Number: 21-1-01178-010



Maximum Depth = 63.98 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 (*)

In Situ Engineering

FIG. GD5-A-17

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

Shannon & Wilson

Operator: Brown
Sounding: SWC 215
Cone Used: DSG1065

CPT Date/Time: 12/14/2008 12:43:17 PM
Location: Pioneer Crossing STA 657+15, +/- 40' R, Elev. ~4492'
Job Number: 21-1-01178-010

Maximum Depth = 94.65 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

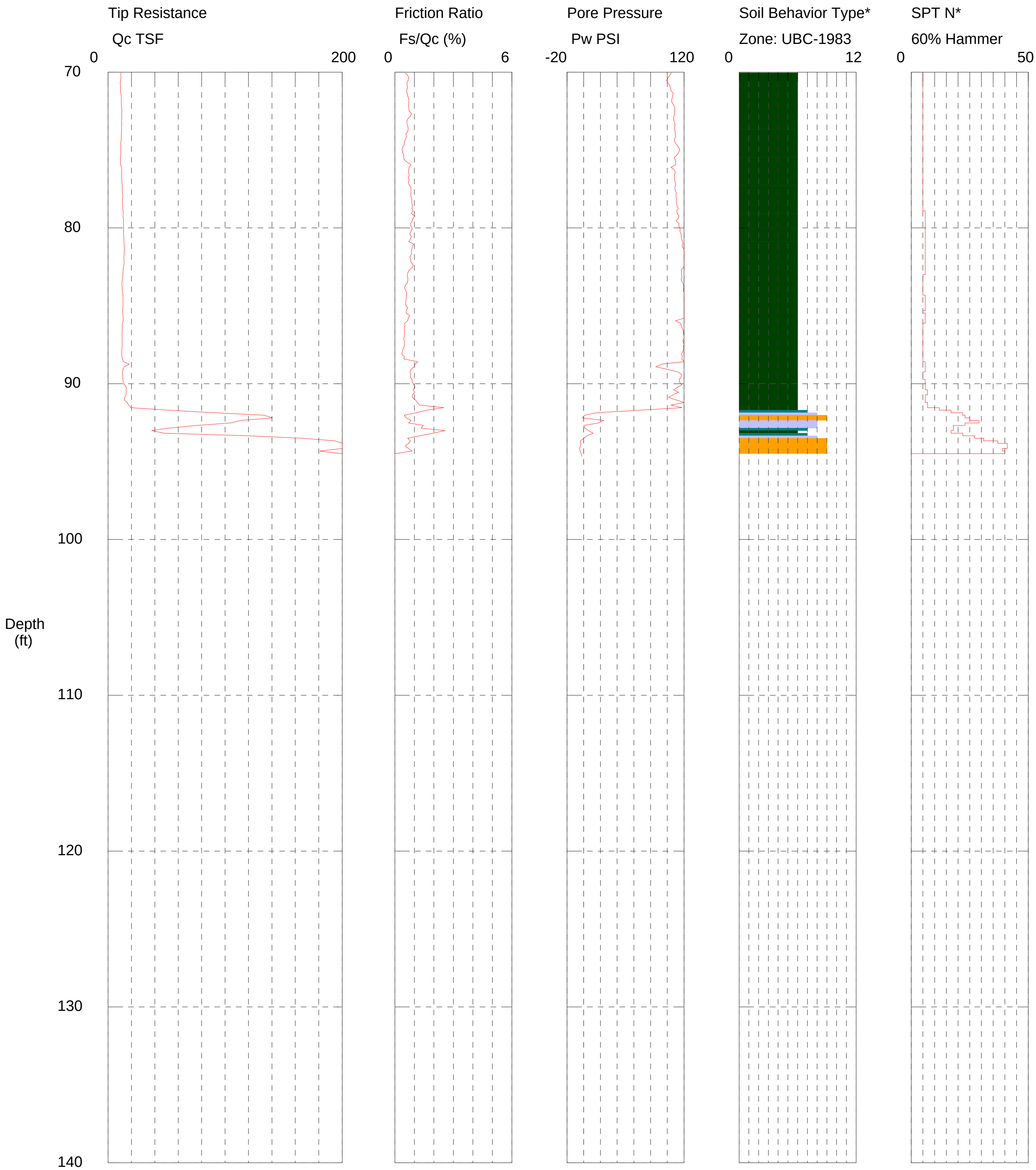
In Situ Engineering

FIG. GD5-A-18 (SHEET 1 OF 2)

Shannon & Wilson

Operator: Brown
Sounding: SWC 215
Cone Used: DSG1065

CPT Date/Time: 12/14/2008 12:43:17 PM
Location: Pioneer Crossing STA 657+15, +/- 40' R, Elev. ~4492'
Job Number: 21-1-01178-010



Maximum Depth = 94.65 feet

Depth Increment = 0.164 feet

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

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

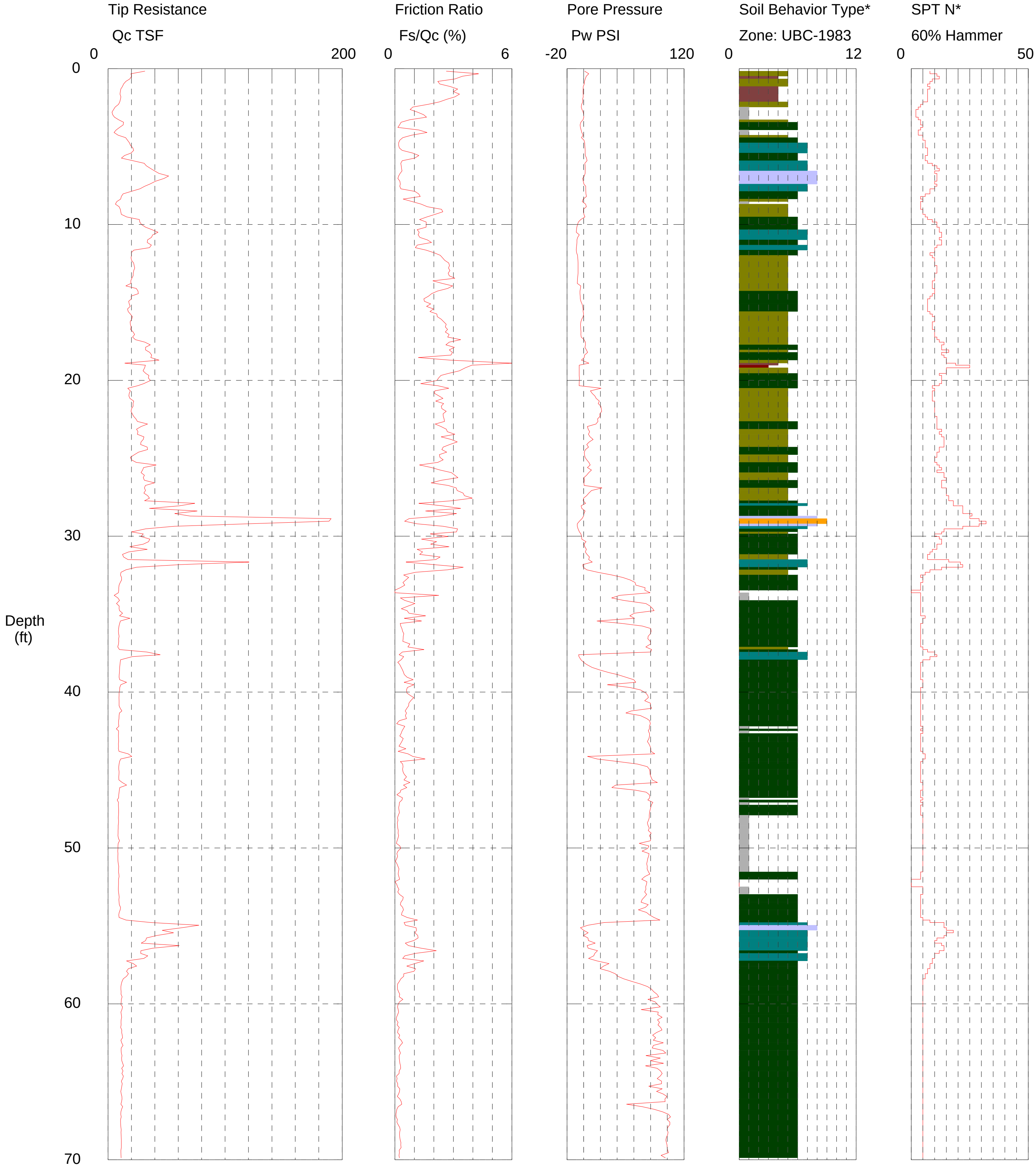
In Situ Engineering

FIG. GD5-A-18 (SHEET 2 OF 2)

Shannon & Wilson

Operator: Brown
Sounding: SWC 216
Cone Used: DSG1065

CPT Date/Time: 12/15/2008 9:40:30 AM
Location: Pioneer Crossing STA 672+00, +/- 40' R, Elev. ~4498'
Job Number: 21-1-01178-010



Maximum Depth = 93.83 feet

Depth Increment = 0.164 feet

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

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

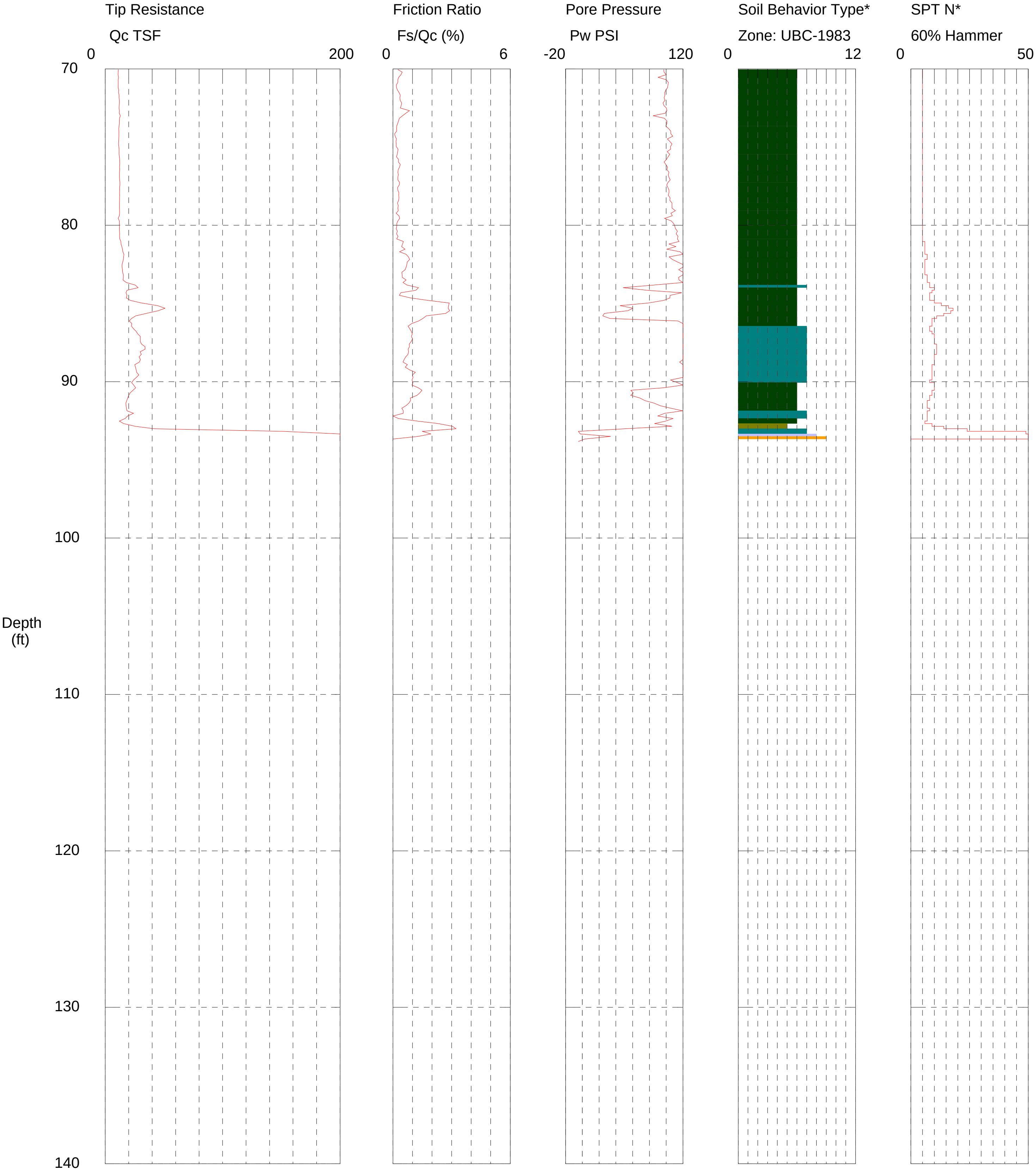
In Situ Engineering

FIG. GD5-A-19 (SHEET 1 OF 2)

Shannon & Wilson

Operator: Brown
Sounding: SWC 216
Cone Used: DSG1065

CPT Date/Time: 12/15/2008 9:40:30 AM
Location: Pioneer Crossing STA 672+00, +/- 40' R, Elev. ~4498'
Job Number: 21-1-01178-010



Maximum Depth = 93.83 feet

Depth Increment = 0.164 feet

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

FIG. GD5-A-19 (SHEET 2 OF 2)

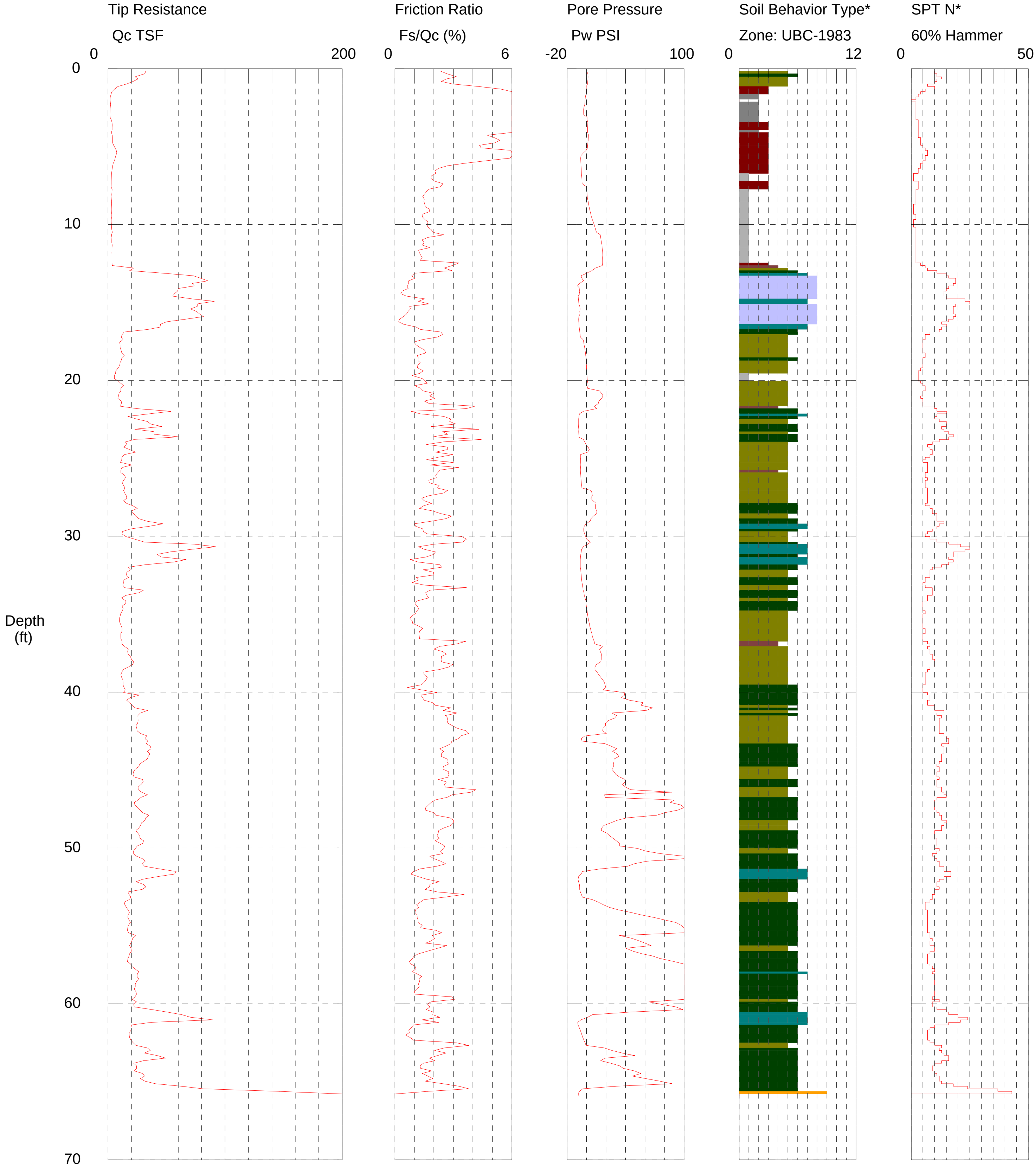
In Situ Engineering

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

Shannon & Wilson

Operator: Brown
Sounding: SWC 218
Cone Used: DSG1065

CPT Date/Time: 12/13/2008 2:46:00 PM
Location: Pioneer Crossing STA 630+10, +/- 70' R, Elev. ~4494'
Job Number: 21-1-01178-010



Maximum Depth = 65.94 feet

Depth Increment = 0.164 feet

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

Test location is approximately 20 feet East of staked location

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

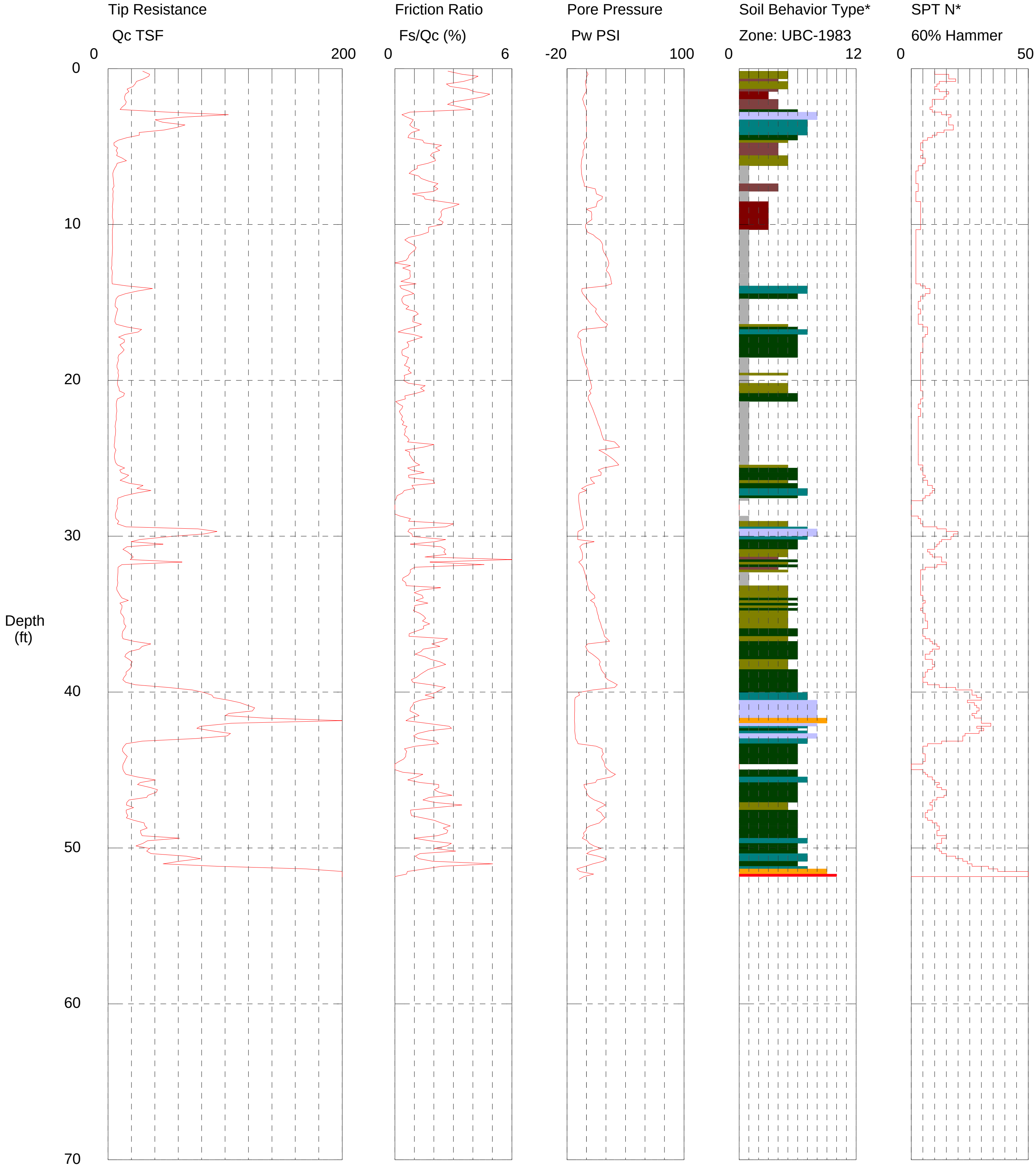
In Situ Engineering

FIG. GD5-A-20

Shannon & Wilson

Operator: Brown
Sounding: SWC 220
Cone Used: DSG1065

CPT Date/Time: 12/12/2008 1:32:52 PM
Location: Pioneer Crossing 50 ft S, 20 ft E SWC 211, Elev. ~4498'
Job Number: 21-1-01178-010



Maximum Depth = 52.00 feet

Depth Increment = 0.197 feet

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

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

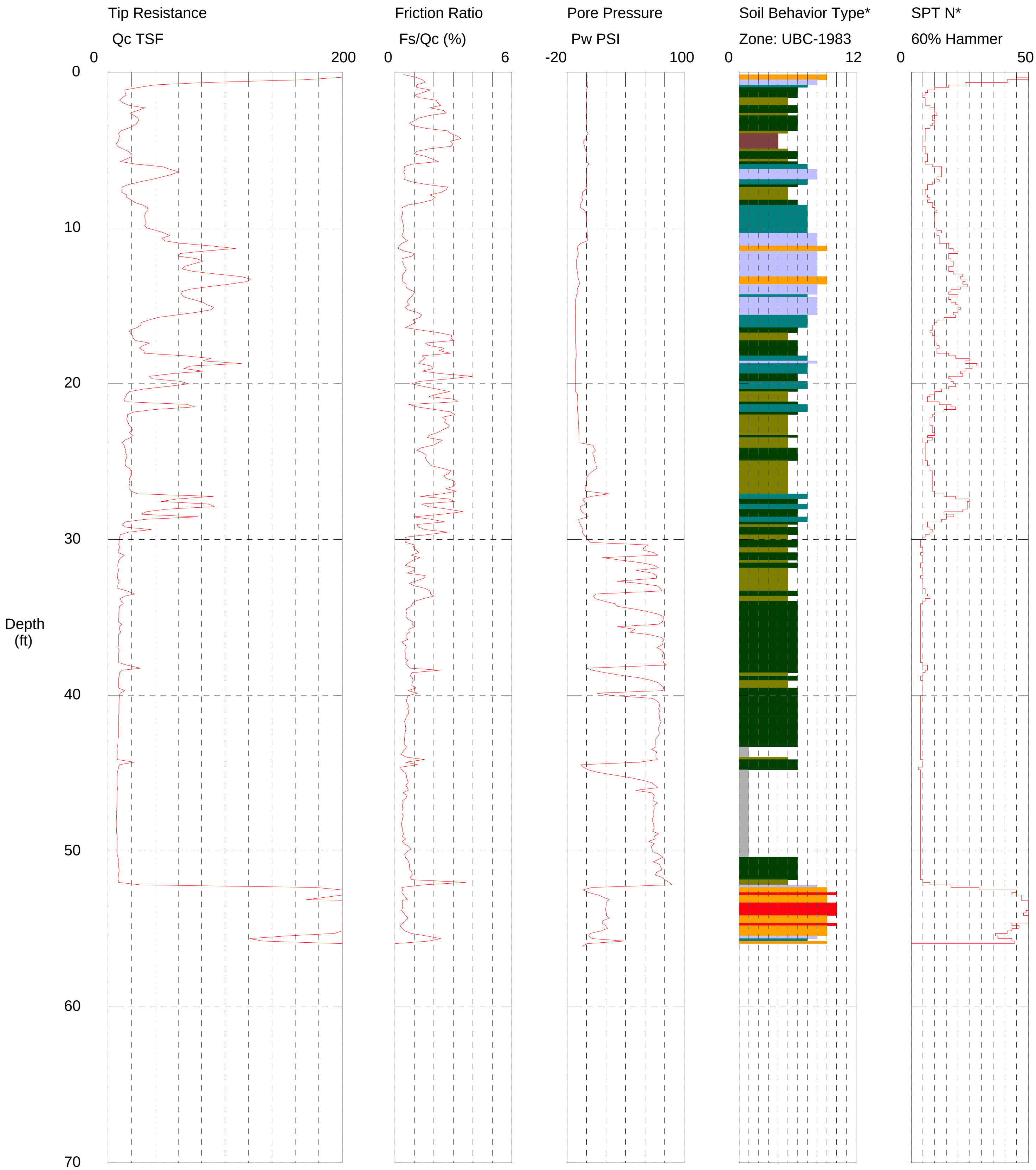
In Situ Engineering

FIG. GD5-A-21

Shannon & Wilson

Operator: Brown
Sounding: SWC 221 (TCC101)
Cone Used: DSG1065

CPT Date/Time: 12/15/2008 1:02:38 PM
Location: Pioneer Crossing STA 688+20, 69' R, Elev. ~4505'
Job Number: 21-1-01178-010



Maximum Depth = 56.10 feet

Depth Increment = 0.164 feet

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

Test location is 9 feet south of staked location.

In Situ Engineering

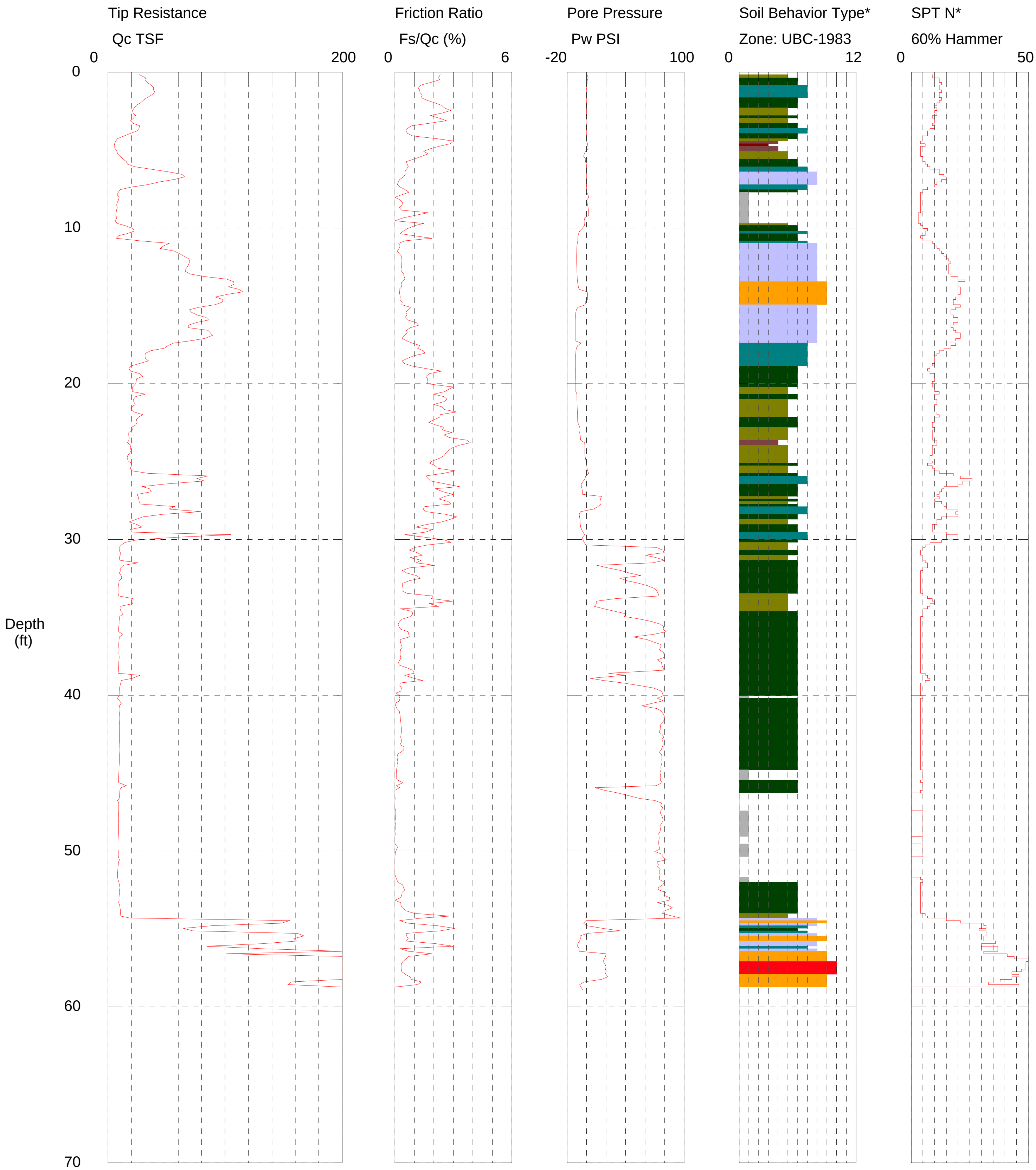
FIG. GD5-A-22

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

Shannon & Wilson

Operator: Brown
Sounding: SWC 222 (TCC102)
Cone Used: DSG1065

CPT Date/Time: 12/15/2008 3:34:19 PM
Location: Pioneer Crossing STA 694+00, Centerline, Elev. ~4507'
Job Number: 21-1-01178-010



Maximum Depth = 58.89 feet

Depth Increment = 0.197 feet

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

In Situ Engineering

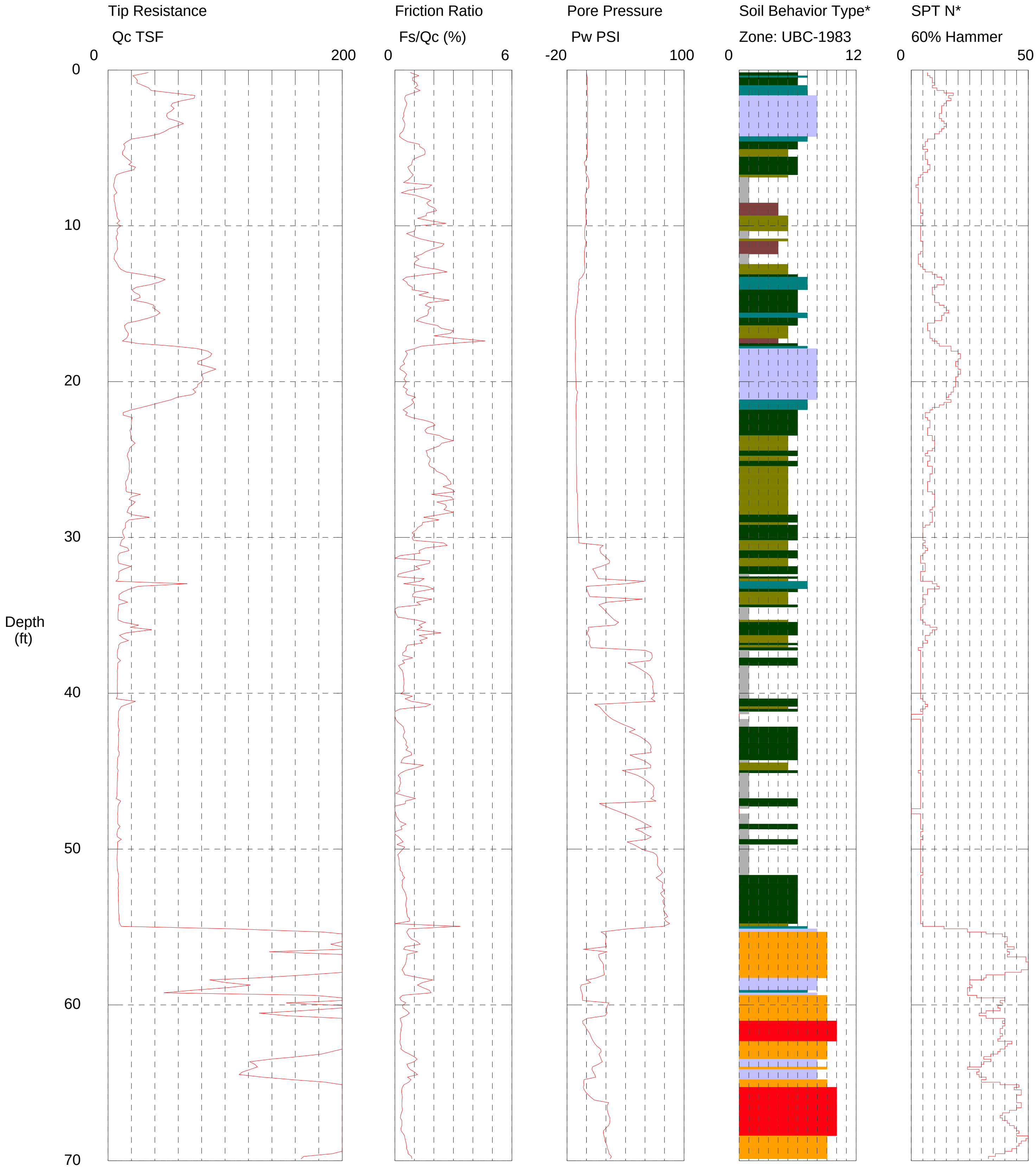
FIG. GD5-A-23

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

Shannon & Wilson

Operator: Brown
Sounding: SWC 223 (TCC103)
Cone Used: DSG1065

CPT Date/Time: 12/16/2008 9:48:38 AM
Location: Pioneer Crossing STA 709+50, +/- 70' R, Elev. ~4511'
Job Number: 21-1-01178-010



Maximum Depth = 84.15 feet

Depth Increment = 0.197 feet

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

Stake was not in the field. Location estimated by pacing.

In Situ Engineering

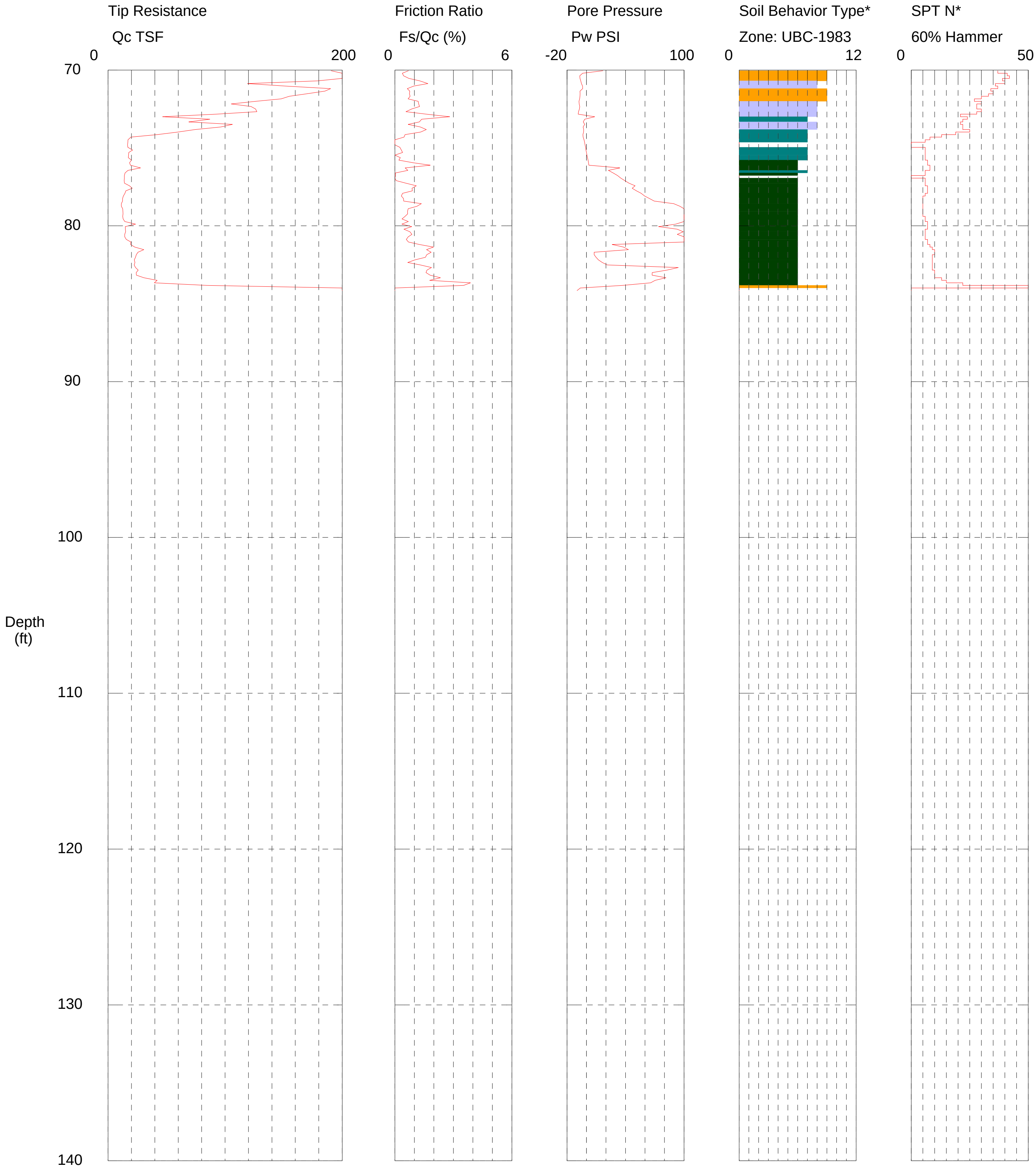
FIG. GD5-A-24 (SHEET 1 OF 2)

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

Shannon & Wilson

Operator: Brown
Sounding: SWC 223 (TCC103)
Cone Used: DSG1065

CPT Date/Time: 12/16/2008 9:48:38 AM
Location: Pioneer Crossing STA 709+50, +/- 70' R, Elev. ~4511'
Job Number: 21-1-01178-010



Maximum Depth = 84.15 feet

Depth Increment = 0.197 feet

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

Stake was not in the field. Location estimated by pacing.

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

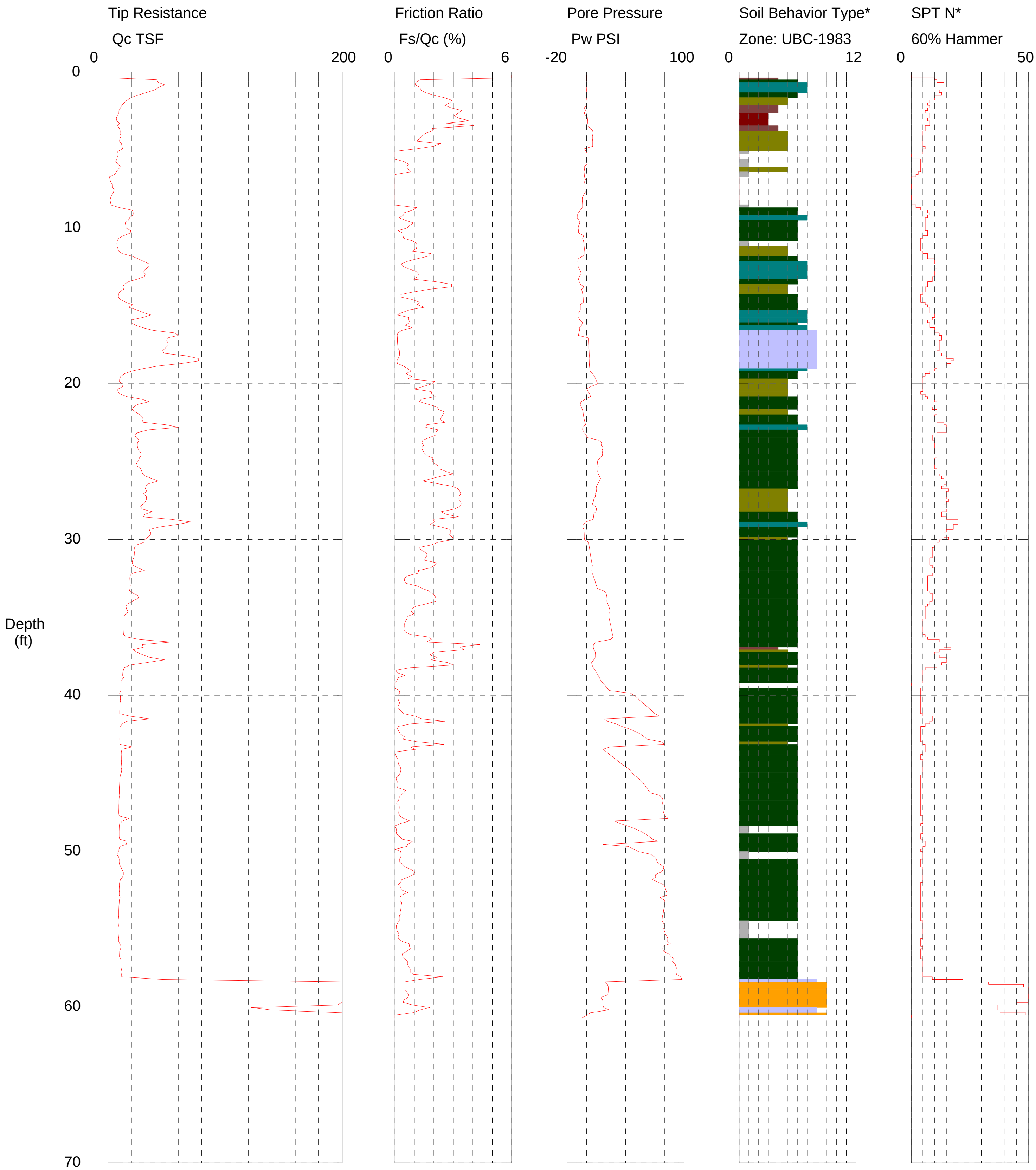
In Situ Engineering

FIG. GD5-A-24 (SHEET 2 OF 2)

Shannon & Wilson

Operator: Brown
Sounding: SWC 224 (TCC104)
Cone Used: DSG1065

CPT Date/Time: 12/17/2008 10:10:48 AM
Location: Pioneer Crossing STA 721+00, +/-40' R, Elev. ~ 4515'
Job Number: 21-1-01178-010



Maximum Depth = 60.70 feet

Depth Increment = 0.164 feet

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

FIG. GD5-A-25

In Situ Engineering

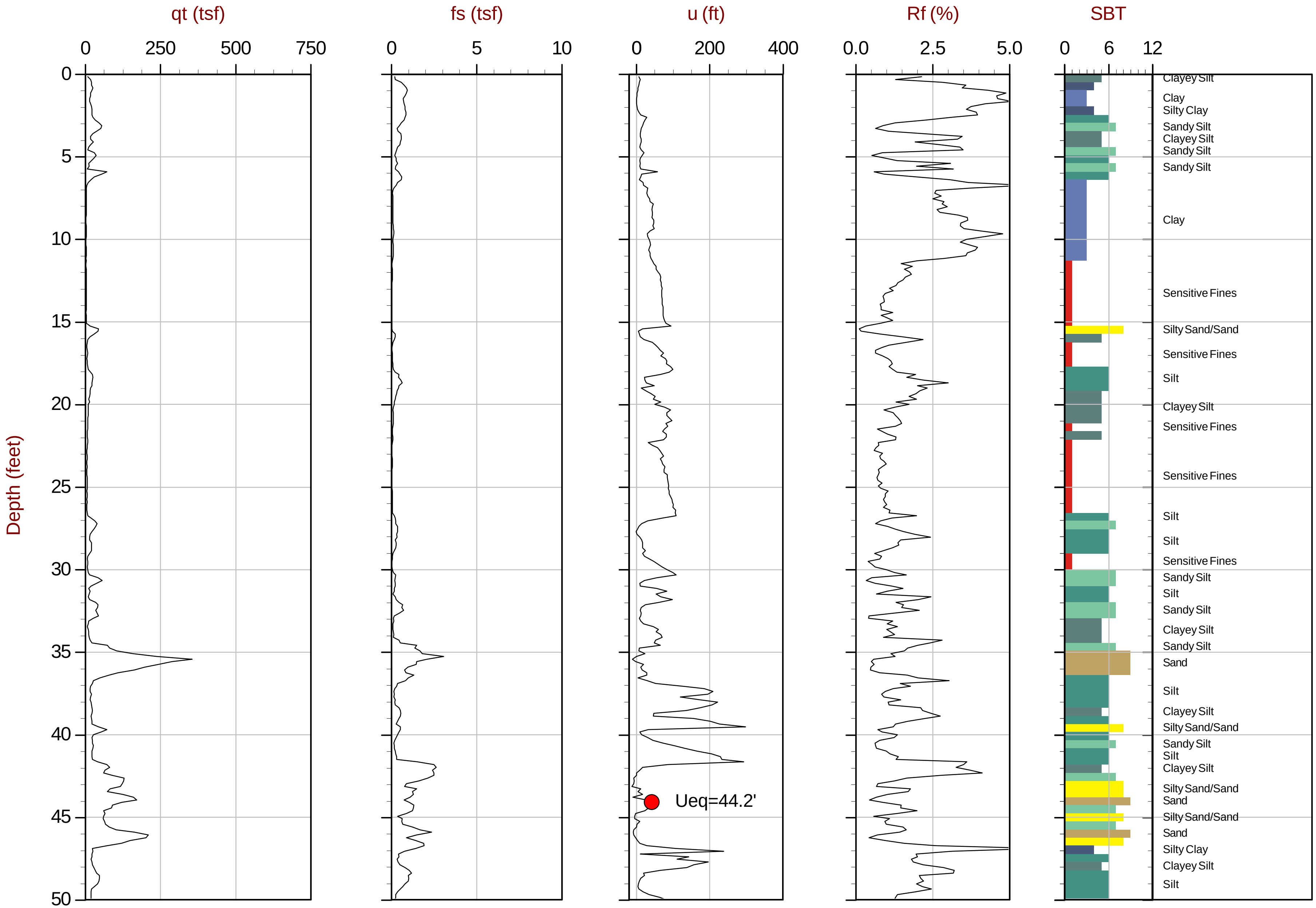
*Soil behavior type and SPT based on data from UBC-1983

CONETEC

Terracon

Job No: 08-340
Date: 03:19:08 11:54
Site: EAST WEST CONNECTOR

Sounding: CPT-02
Cone: STD 20T AD197



Max Depth: 18.250 m / 59.87 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.150 m

File: 340CP02.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 2
● Equilibrium Pore Pressure from Dissipation

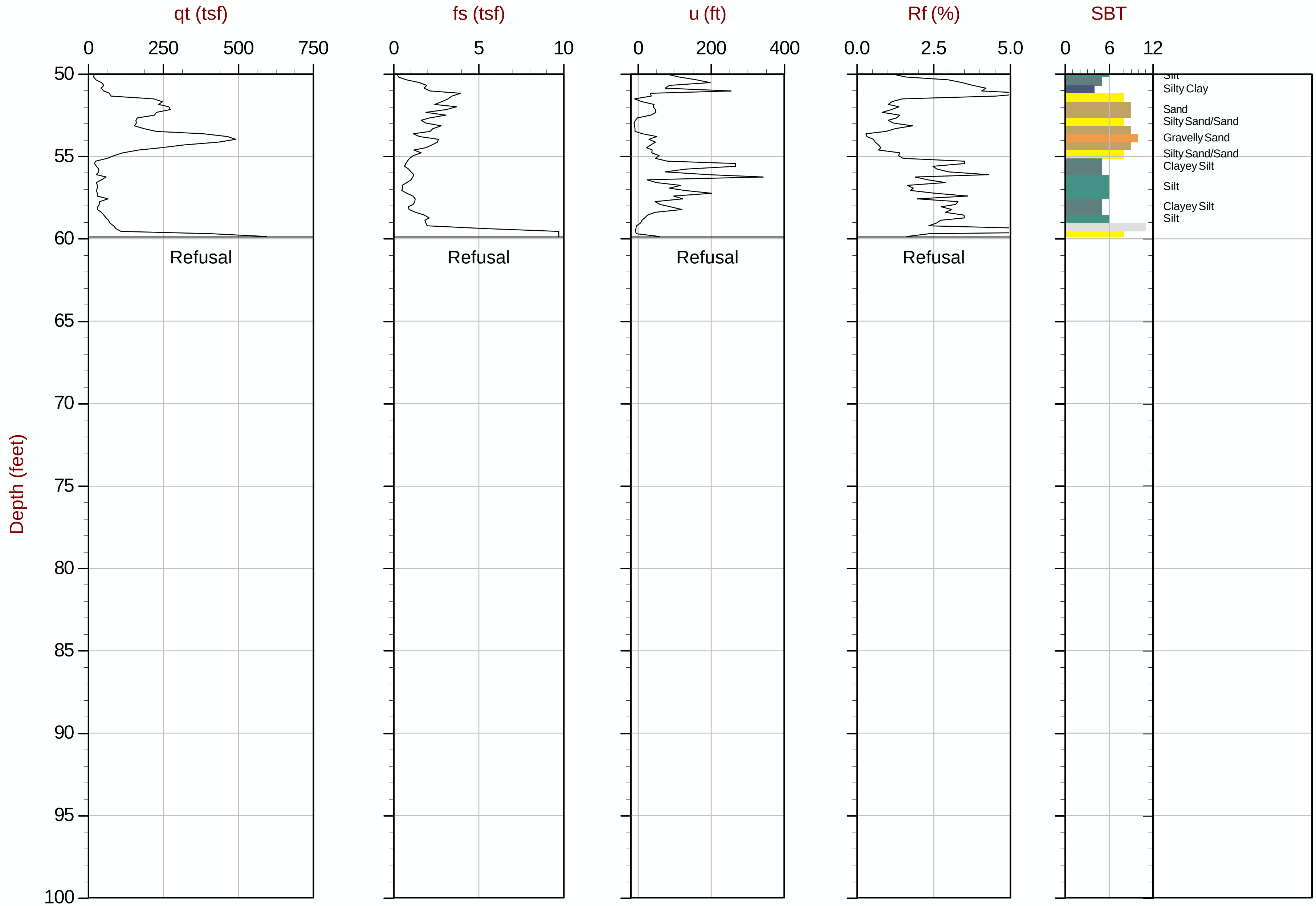
FIG. GD5-A-26 (SHEET 1 OF 2)

CONETEC

Terracon

Job No: 08-340
Date: 03:19:08 11:54
Site: EAST WEST CONNECTOR

Sounding: CPT-02
Cone: STD 20T AD197



Max Depth: 18.250 m / 59.87 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.150 m

File: 340CP02.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
Page No: 2 of 2
Equilibrium Pore Pressure from Dissipation

FIG. GD5-A-26 (SHEET 2 OF 2)

Filename: J:\231\01178-010\23-1-01178-010 TPs.dwg   Layout: SWT-205   Date: 02-25-2009   Login: cnt

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT SWT-205

JOB NO: 23-1-01178-010   DATE: 12-17-08   LOCATION: Sta 600+35, ±40' Left

PROJECT: Pioneer Crossing

SOIL DESCRIPTION

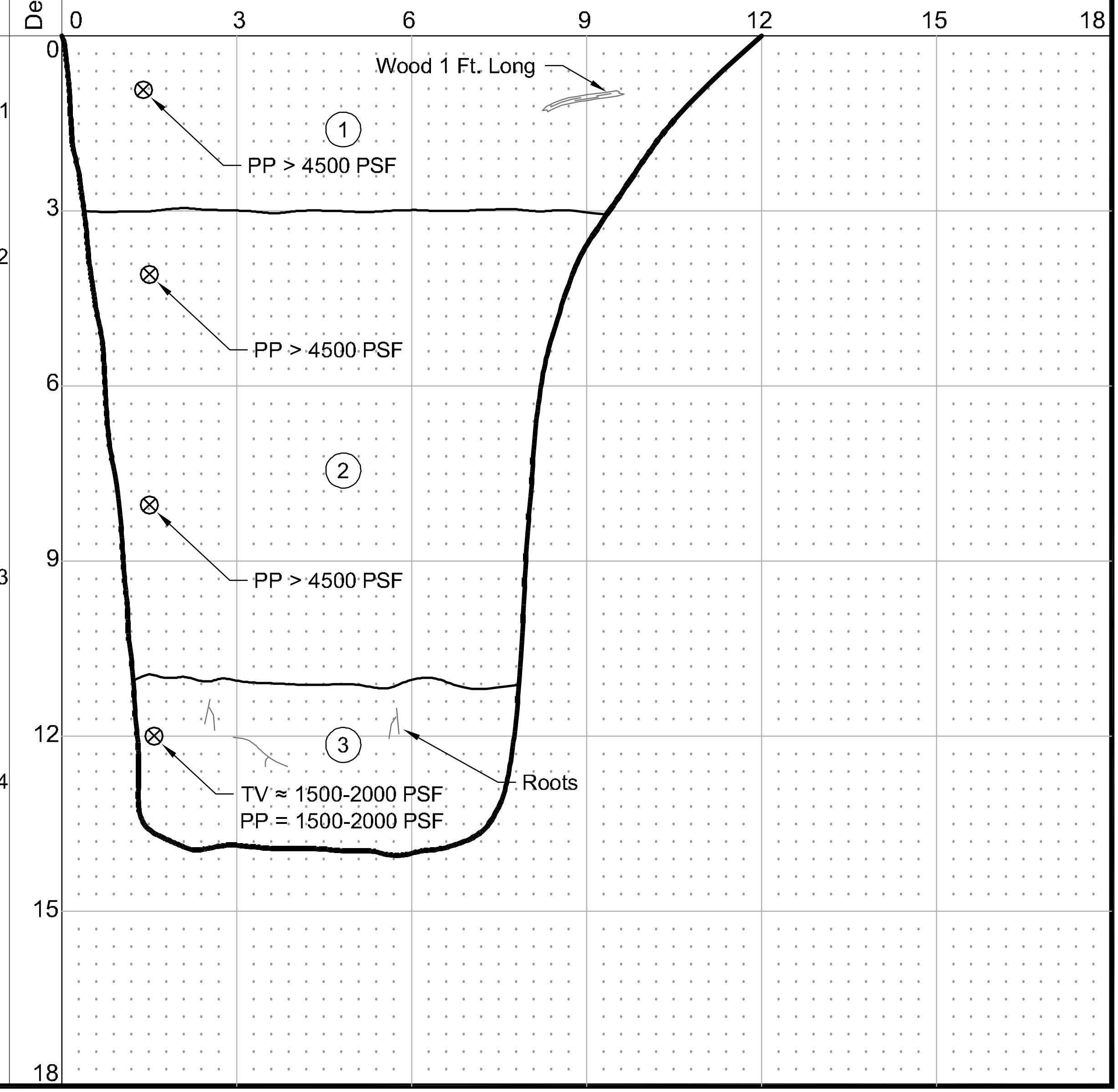
1. Medium dense, tan to brown, clayey, silty, gravelly SAND; moist; scattered fine roots; (Fill) SM.
2. Very stiff to hard, tan to brown, clayey SILT; moist; trace of fine roots, numerous iron-oxide stains; ML.
3. Stiff, reddish-brown to light gray, silty CLAY; moist; occasional fine roots; CL.

Ground Water: None Observed

% Water Content

Samples: Bulk Sample; S-1; S-2; S-3; S-4

Sketch of South Side of Pit   Surface Elevation: Approx. 4521 Ft.



LEGEND

TV = Torvane Readings
PP = Pocket Penetrometer Readings
PSF = Pounds per Square Foot

FIG. GD5-A-27

Filename: J:\231\01178-010\23-1-01178-010 TPs.dwg Layout: SWT-206 Date: 02-25-2009 Login: cnt

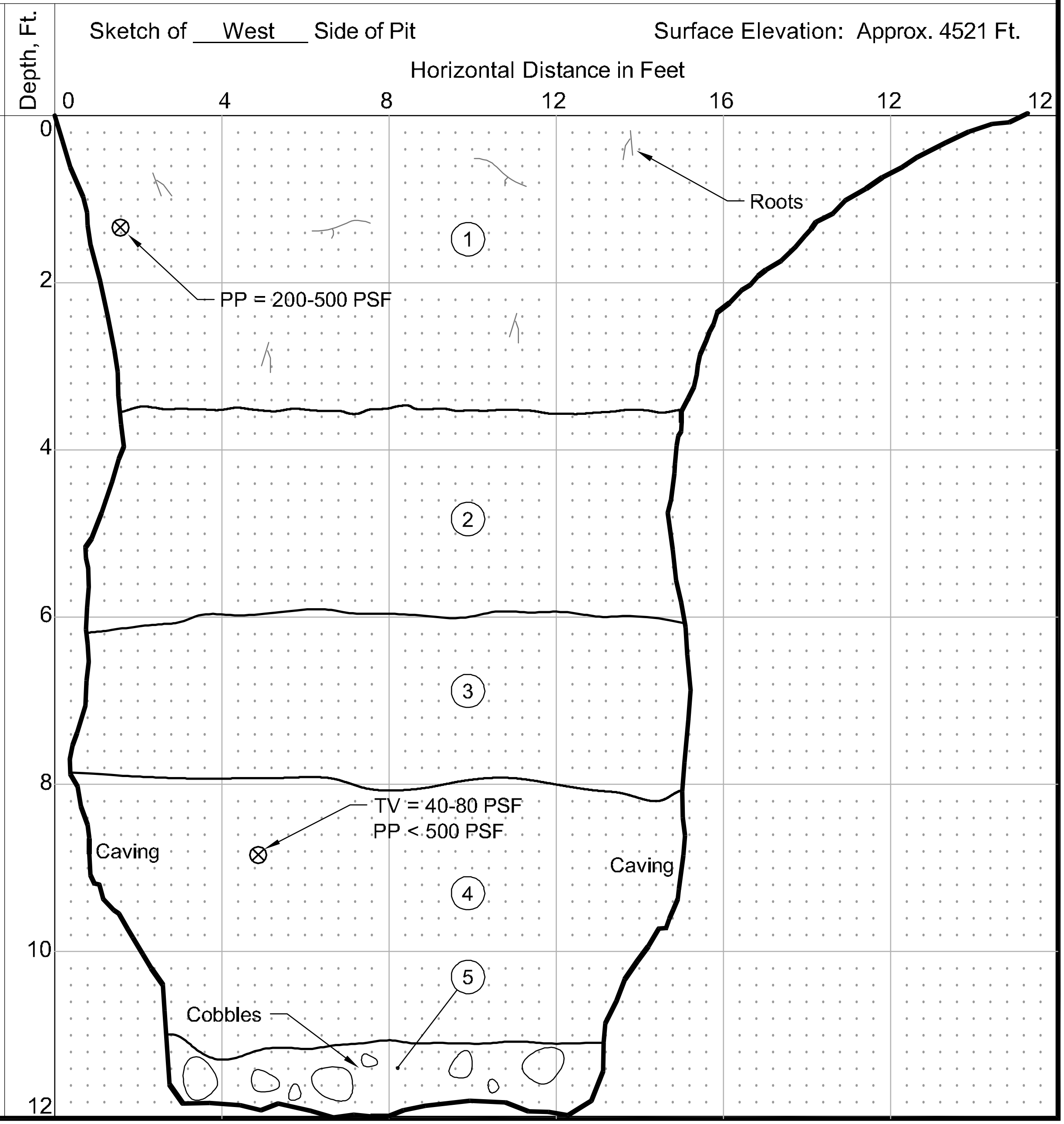
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT SWT-206

JOB NO: 23-1-01178-010 DATE: 12-17-08 LOCATION: Sta 609+00, ±40' Left

PROJECT: Pioneer Crossing

SOIL DESCRIPTION	Ground Water	% Water Content	Samples
(1) Soft, dark brown to brown, clayey SILT; moist; scattered fine roots; ML.			
(2) Loose, light gray, clayey, angular gravel; wet; GC.			S-1
(3) Loose, light brown, gravelly SAND; wet; scattered cobbles; SM.			
(4) Very soft, gray, fine sandy, clayey SILT; wet; ML.			
(5) Dense, cobbles and boulders; GM.	▽		S-2
	▼ ▼		S-3
			S-4



LEGEND

TV = Torvane Readings
PP = Pocket Penetrometer Readings
PSF = Pounds per Square Foot
▽ = Well Measurement at Time of Drilling
▼ = Well Measurement 12/22/08
▼ = Well Measurement 1/9/09

FIG. GD5-A-28

Filename: J:\231\01178-010\23-1-01178-010 TPs.dwg Layout: SWT-207 Date: 02-25-2009 Login: cnt

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT SWT-207

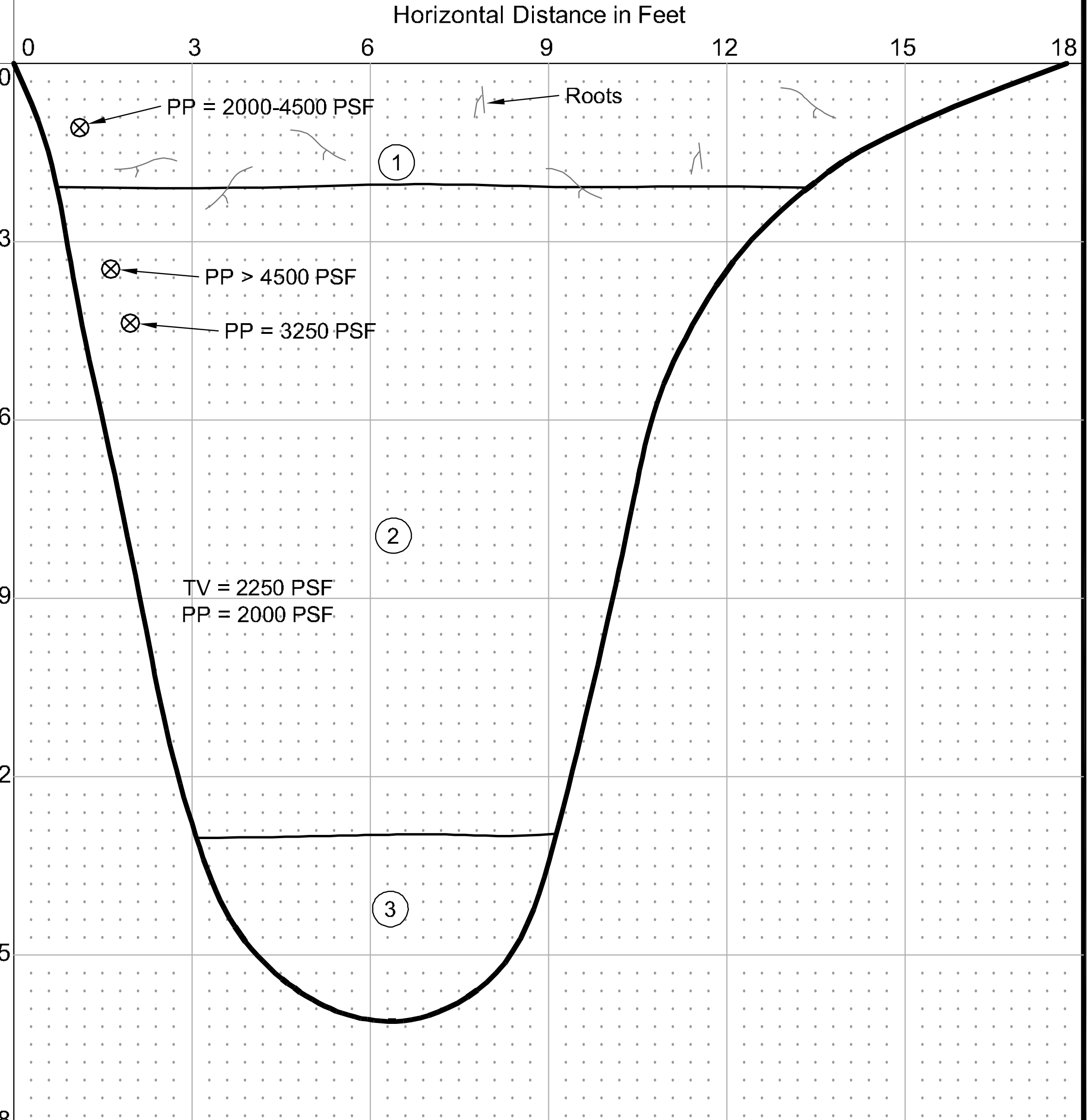
JOB NO: 23-1-01178-010 DATE: 12-16-08 LOCATION: Sta 646+00, ±54' Left

PROJECT: Pioneer Crossing

Sketch of East Side of Pit

Surface Elevation: Approx. 4502 Ft.

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) Stiff, black, organic CLAY; damp; friable; CH.	None Observed		S-1	0
(2) Hard grading to stiff, tan and gray, CLAY; moist; pockets of gray, silty sand and tan, slightly cemented silty sand oxidized orange, very slow seepage from sand seams/pockets, small shells at bottom of silty sand seepage layer; CL.			S-2	3
(3) Very soft, gray to dark gray, CLAY; wet; local pockets of gray sand; CL.				6
				9
				12
			S-3	15
				18



LEGEND

TV = Torvane Readings
PP = Pocket Penetrometer Readings
PSF = Pounds per Square Foot

FIG. GD5-A-29

Filename: J:\231\01178-010\23-1-01178-010 TPs.dwg Layout: SWT-208 Date: 02-25-2009 Login: cnt

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT SWT-208

JOB NO: 23-1-01178-010 DATE: 12-13-08 LOCATION: Sta 662+00, ±40' Right

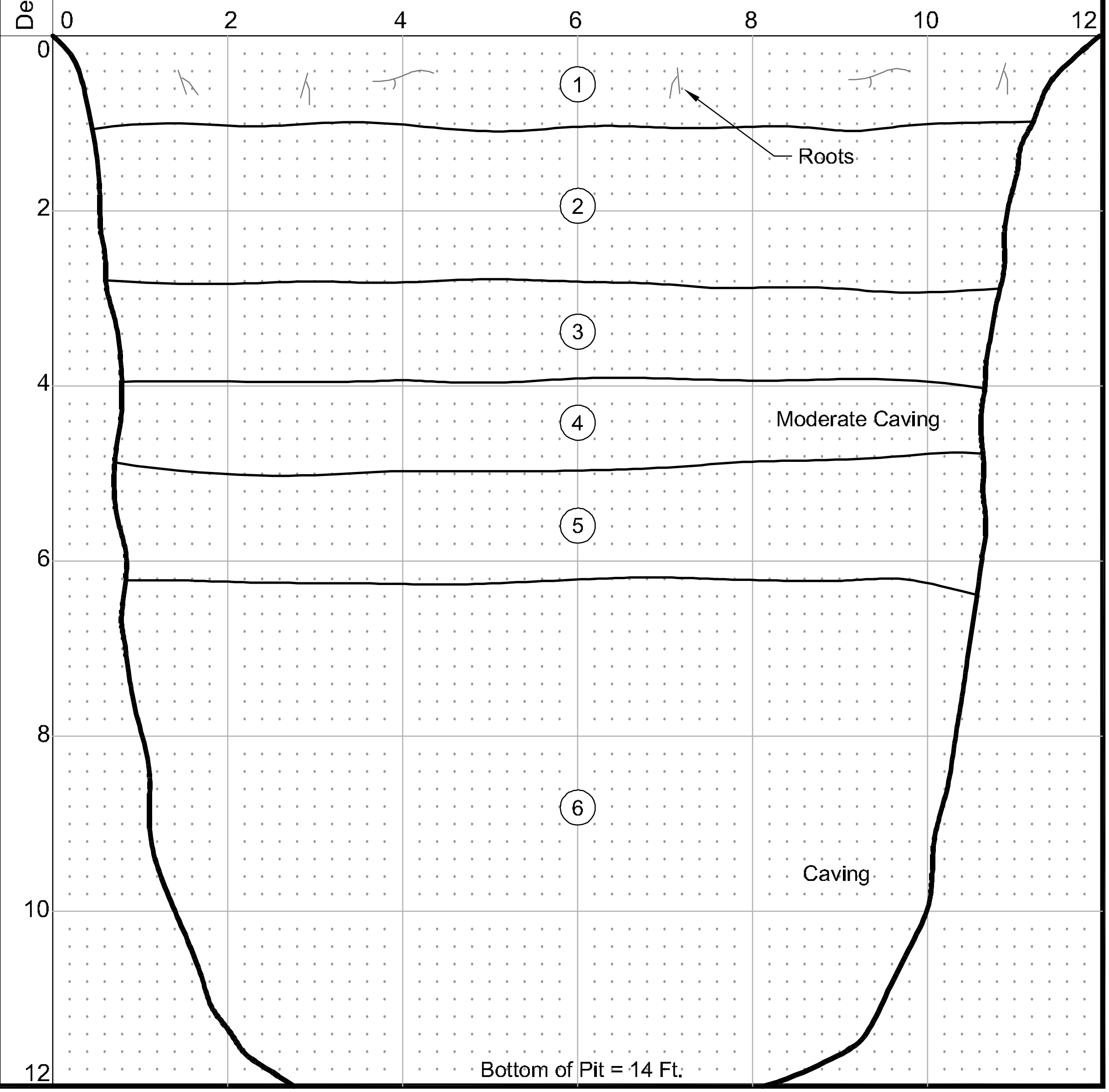
PROJECT: Pioneer Crossing

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) Stiff, brown-gray, CLAY; damp; CL/CH.			S-1	0
(2) Medium dense, brown, silty, fine SAND to sandy SILT: moist; SM/ML.			S-2	2
(3) Medium dense, black, clayey SAND; wet; SC.			S-3	
(4) Medium dense, brown-gray, medium SAND; wet; SP.	▽			4
(5) Black, slightly sandy, CLAY; wet; CH.	▼ ▽			
(6) Stiff, tan, CLAY; wet; oxidized orange, fractured, grading to gray oxidized orange, small shell layers at 6 and 7 feet; CL/CH.				6

Sketch of North Side of Pit

Surface Elevation: Approx. 4494 Ft.

Horizontal Distance in Feet



LEGEND

TV = Torvane Readings
PP = Pocket Penetrometer Readings
PSF = Pounds per Square Foot
▽ = Well Measurement at Time of Drilling
▼ = Well Measurement 12/22/08
▽ = Well Measurement 1/9/09

FIG. GD5-A-30

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

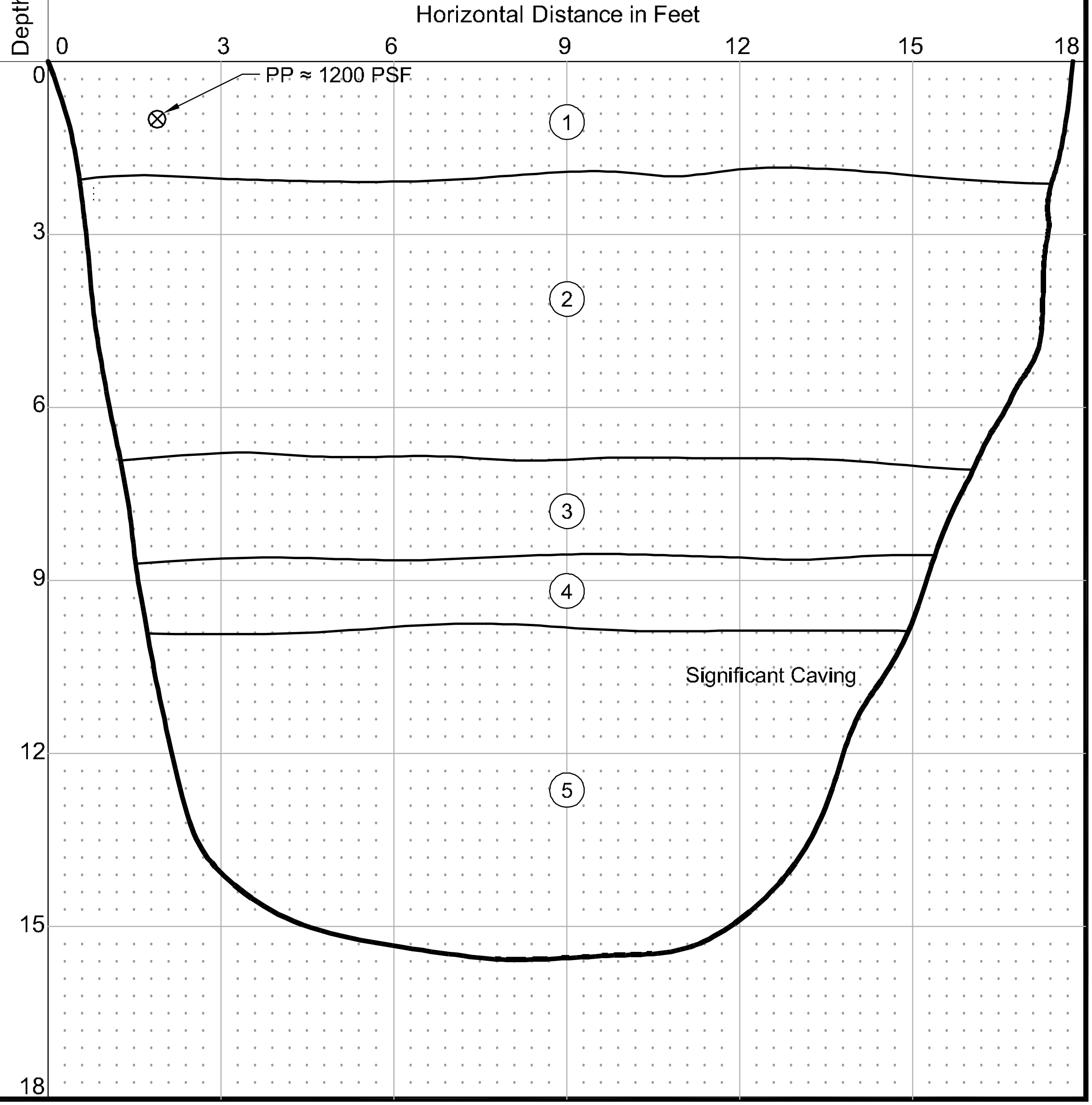
LOG OF TEST PIT SWT-209

JOB NO: 23-1-01178-010 DATE: 12-15-08 LOCATION: Sta 667+06, 33' Right

PROJECT: Pioneer Crossing

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) Medium dense, black, clayey, medium to coarse SAND, trace of fine gravel; moist to wet; SC.			S-1, S-2, S-3	0
(2) Loose to medium dense, light brown, slightly clayey grading to trace, slightly gravelly, medium to coarse SAND; SP-SC.	▽ ▼			3
(3) Stiff, black, CLAY; moist; trace of fine organics; CH.			S-4	6
(4) Gray to tan, CLAY; moist; stands vertical; CL/CH.				9
(5) Stiff, gray, CLAY; moist; fractured, oxidized orange, significant caving along fractures; CH.				12

Sketch of South Side of Pit Surface Elevation: Approx. 4997 Ft.



LEGEND

TV = Torvane Readings
PP = Pocket Penetrometer Readings
PSF = Pounds per Square Foot
▽ = Well Measurement at Time of Drilling
▼ = Well Measurement 12/22/08

FIG. GD5-A-31

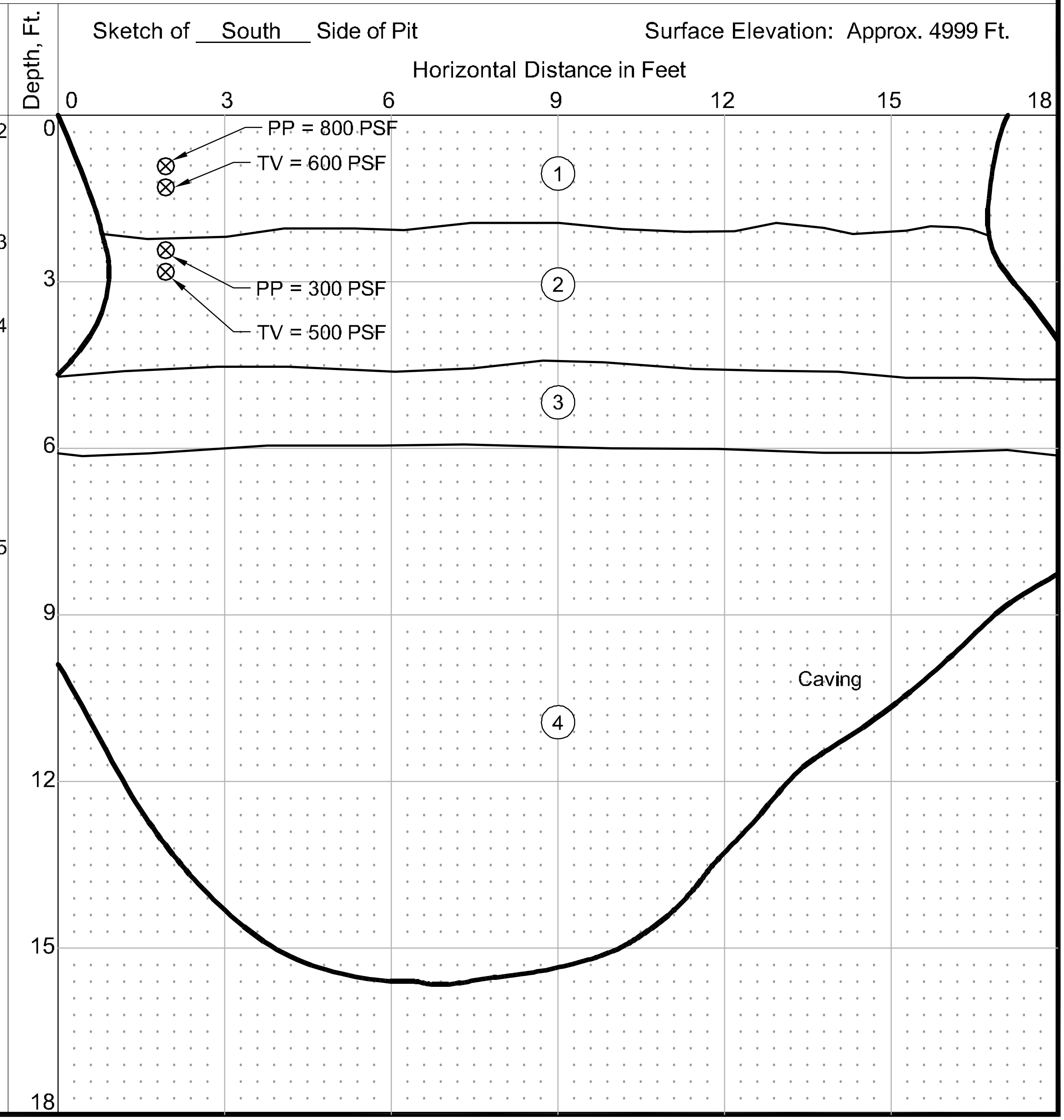
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT SWT-210

JOB NO: 23-1-01178-010 DATE: 12-15-08 LOCATION: Sta 677+00, 32' Right

PROJECT: Pioneer Crossing

SOIL DESCRIPTION	Ground Water	% Water Content	Samples
1 Stiff, dark gray to black, sandy CLAY; moist; scattered organics; CH.			S-2, S-1
2 Medium stiff to stiff, light gray-brown, sandy, silty CLAY; wet; scattered organics, pockets of dark brown to black, clayey, medium sand; CL.	▽ ▼		S-3
3 Loose to medium dense, gray with orange and white, slightly gravelly, medium to coarse SAND; wet; SP.	▽		S-4
4 Medium stiff to stiff, gray, CLAY; wet grading to moist; occasional fine to medium sand seams, fractured; CL/CH.			S-5



LEGEND

TV = Torvane Readings
PP = Pocket Penetrometer Readings
PSF = Pounds per Square Foot
▽ = Well Measurement at Time of Drilling
▼ = Well Measurement 12/22/08
▽ = Well Measurement 1/9/09

FIG. GD5-A-32

Filename: J:\231\01178-010\23-1-01178-010 TPs.dwg Layout: SWT-217 Date: 02-25-2009 Login: cnt

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

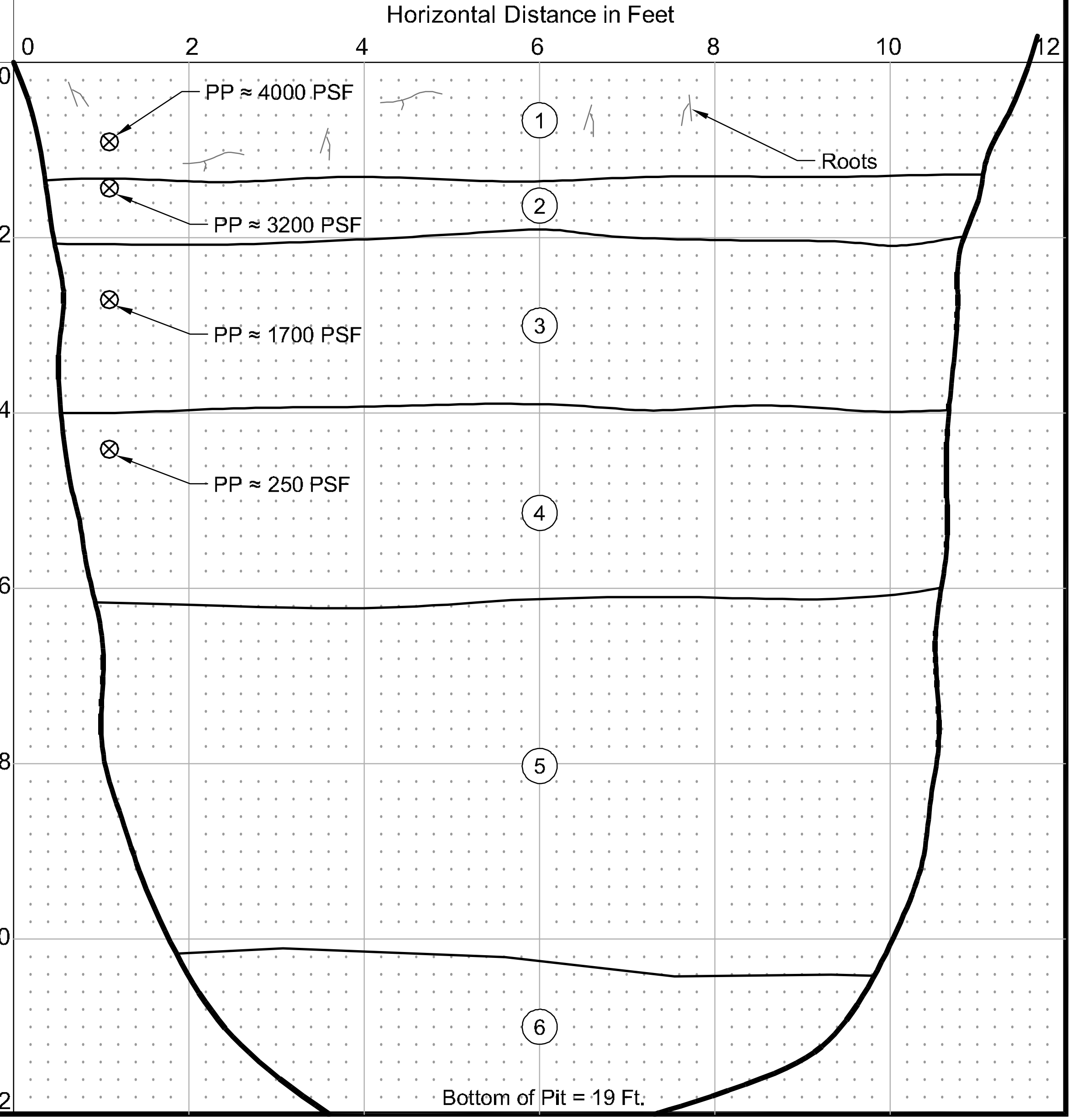
LOG OF TEST PIT SWT-217

JOB NO: 23-1-01178-010 DATE: 12-15-08 LOCATION: Sta 626+10, ±60' Left

PROJECT: Pioneer Crossing

Sketch of Southwest Side of Pit — Surface Elevation: 4493 Ft.

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) Very stiff to hard, brown-gray, CLAY; damp; CH.				0
(2) Stiff to very stiff, black, CLAY; damp; friable; CL/CH.				2
(3) Very stiff grading to stiff, light gray-brown, CLAY; moist grading to wet; trace organics, slightly oxidized dark yellow-red; CL.			S-1	4
(4) Very soft, light gray-brown, fine sandy, CLAY; wet; numerous organics; CL.	▽			
(5) Medium dense, gray, silty, fine to medium SAND; wet; SM.				6
(6) Very soft to soft, gray, slightly sandy, CLAY; wet; CL.				8
				10
				12



LEGEND

TV = Torvane Readings
PP = Pocket Penetrometer Readings
PSF = Pounds per Square Foot

FIG. GD5-A-33

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT SWT-219

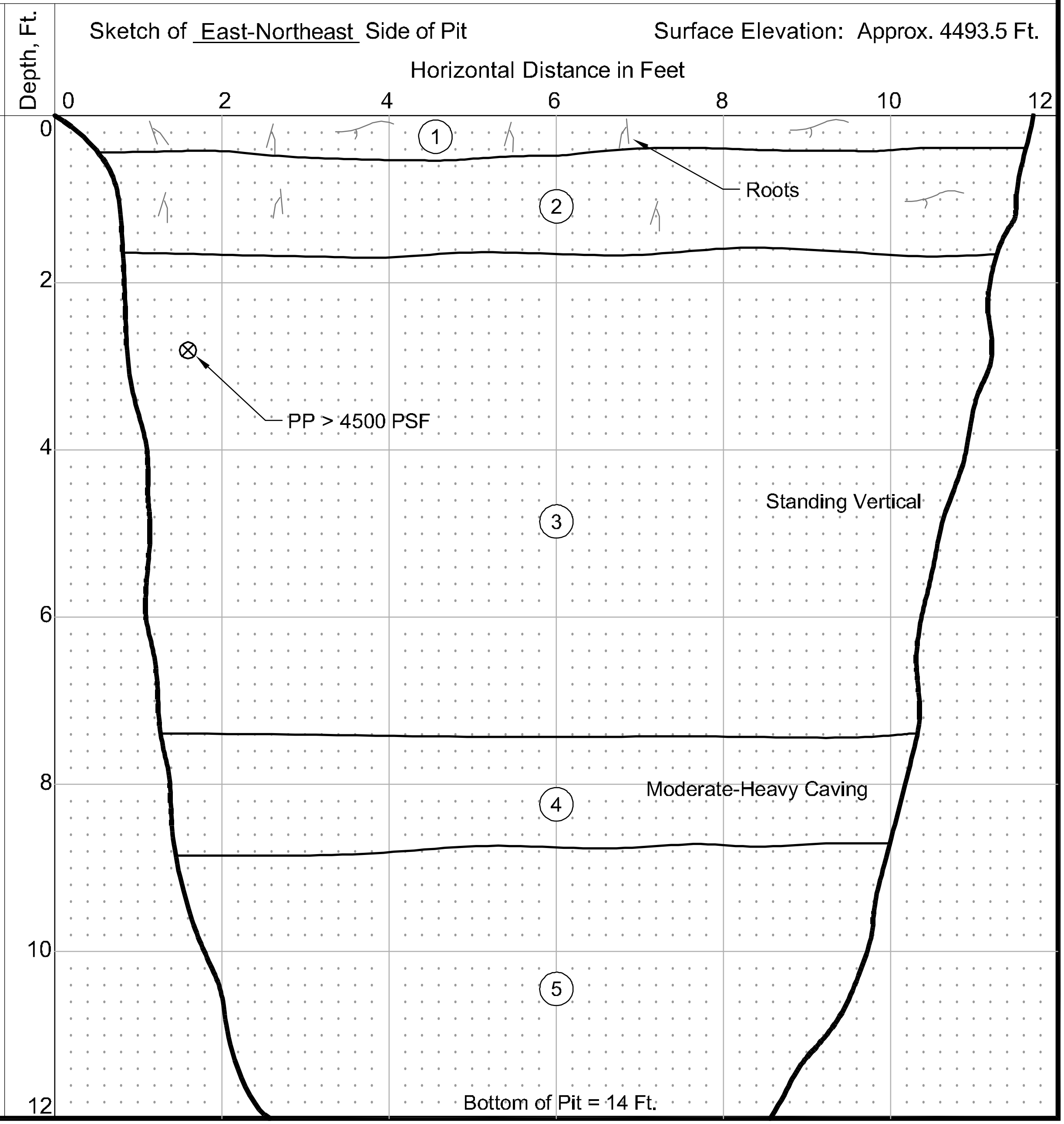
JOB NO: 23-1-01178-010 DATE: 12-15-08 LOCATION: Sta 626+48, ±69.7' Right

PROJECT: Pioneer Crossing

SOIL DESCRIPTION	Ground Water	% Water Content	Samples
(1) DUFF: Loose soil, dense roots. (2) Stiff, dark brown, slightly sandy, CLAY; damp; friable, numerous roots; CL. (3) Very stiff to hard, light brown, CLAY; damp grading to moist; trace organics, oxidized yellowish-red; CL/CH. (4) Medium dense, gray, slightly gravelly, silty, medium to coarse SAND; wet; SM. (5) Soft, gray to brown-gray, CLAY; wet; CL/CH.	None Observed		

Sketch of East-Northeast Side of Pit

Surface Elevation: Approx. 4493.5 Ft.



LEGEND

TV = Torvane Readings
PP = Pocket Penetrometer Readings
PSF = Pounds per Square Foot

FIG. GD5-A-34

Filename: J:\231\01178-010\23-1-01178-010 TPs.dwg Layout: SWT-225 Date: 02-25-2009 Login: cnt

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

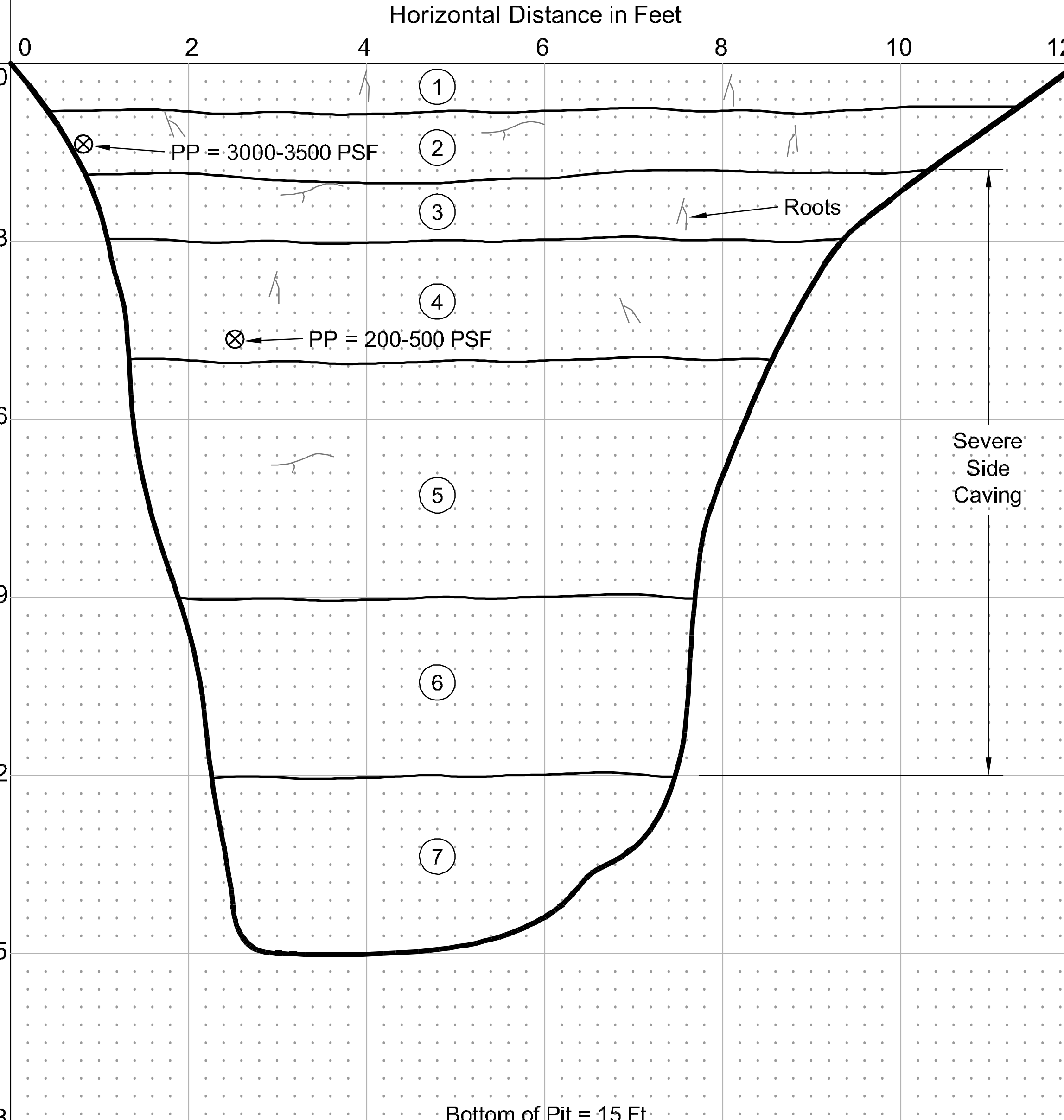
LOG OF TEST PIT SWT-225

JOB NO: 23-1-01178-010 DATE: 1-17-09 LOCATION: Sta 625+75, ±60' Right

PROJECT: Pioneer Crossing

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) Soft, dark brown, clayey SILT; moist; numerous organics and roots; (Topsoil) ML.			S-1	0
(2) Stiff, dark brown, silty CLAY; moist; scattered roots; CL.	∇		S-2	
(3) Soft, light brown, sandy SILT; wet; scattered roots; ML/SM.			S-3	3
(4) Soft, light gray, clayey SILT; wet; scattered roots; ML.			S-4	
(5) Loose, gray, interbedded slightly clayey, sandy SILT to silty SAND; wet; occasional roots; ML/SM.				6
(6) Loose, light gray, silty, sandy GRAVEL; wet; numerous shell fragments; (Waterbearing) GM.			S-5	9
(7) Stiff, brown-gray, silty CLAY; wet; numerous shell fragments; CL.			S-6	12

Sketch of West Side of Pit Surface Elevation: Approx. 4493.2 Ft.



LEGEND

TV = Torvane Readings
PP = Pocket Penetrometer Readings
PSF = Pounds per Square Foot

FIG. GD5-A-35

Filename: J:\231\01178-010\23-1-01178-010 TPs.dwg Layout: SWT-227 Date: 02-25-2009 Login: cnt

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT SWT-227

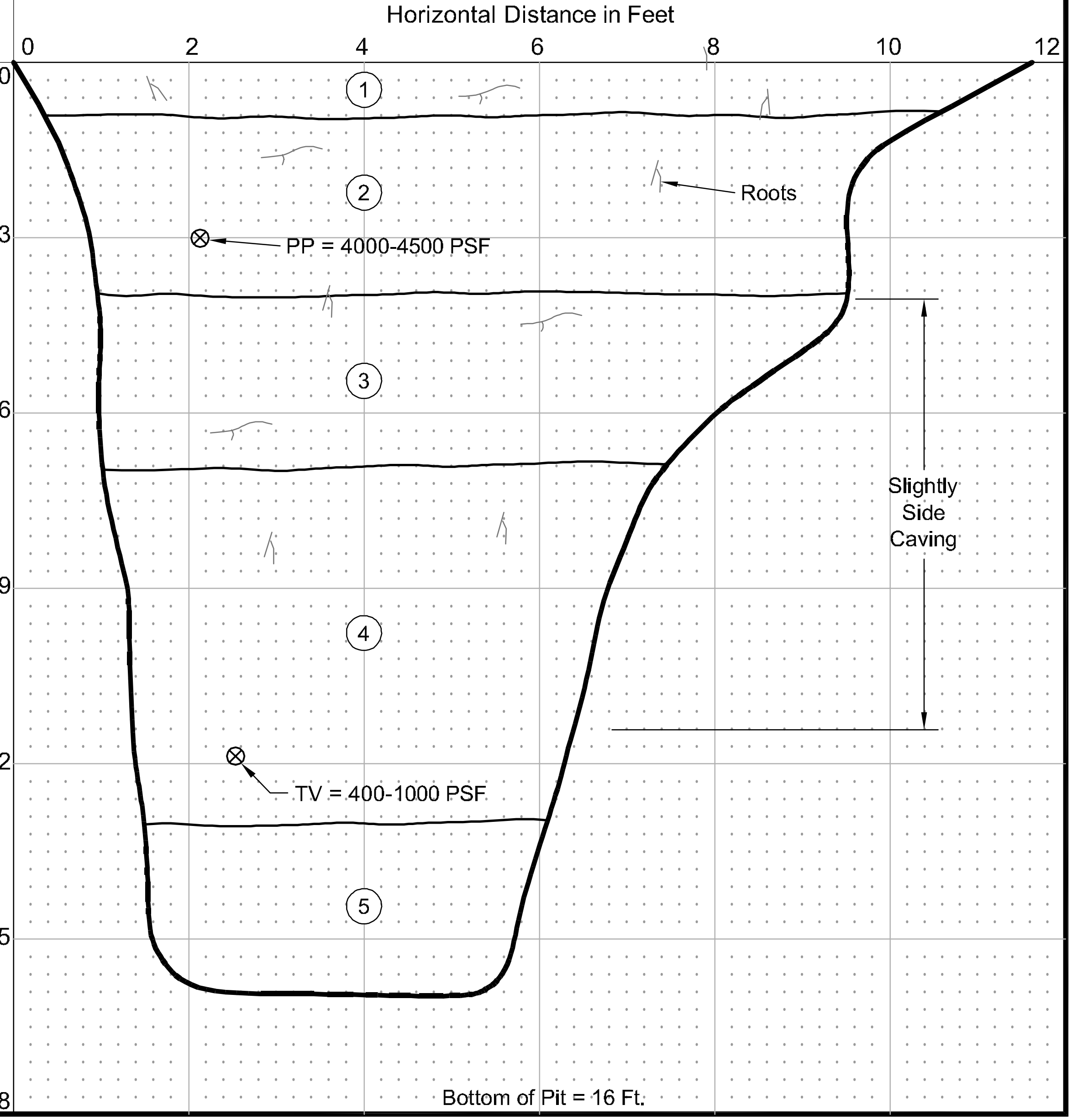
JOB NO: 23-1-01178-010 DATE: 1-17-09 LOCATION: Sta 627+80, ±45' Right

PROJECT: Pioneer Crossing

Sketch of West Side of Pit

Surface Elevation: Approx. 4493.6 Ft.

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) Medium stiff, dark brown, organic clayey SILT; numerous roots; (Topsoil) OL.				0
(2) Very stiff, light brown-olive, clayey SILT to silty CLAY; moist; scattered roots; ML/CL.			S-1	3
(3) Medium stiff to soft, light brown, silty CLAY; wet; CL.	∇		S-2	
(4) Soft, brown-gray, clayey SILT; wet; numerous layers of shell fragments and fibrous roots; ML.				6
(5) Stiff, greenish-gray, silty CLAY; wet; scattered tuff clasts; CL.			S-3	9
				12
			S-4	15
				18



LEGEND

TV = Torvane Readings
PP = Pocket Penetrometer Readings
PSF = Pounds per Square Foot

FIG. GD5-A-36

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

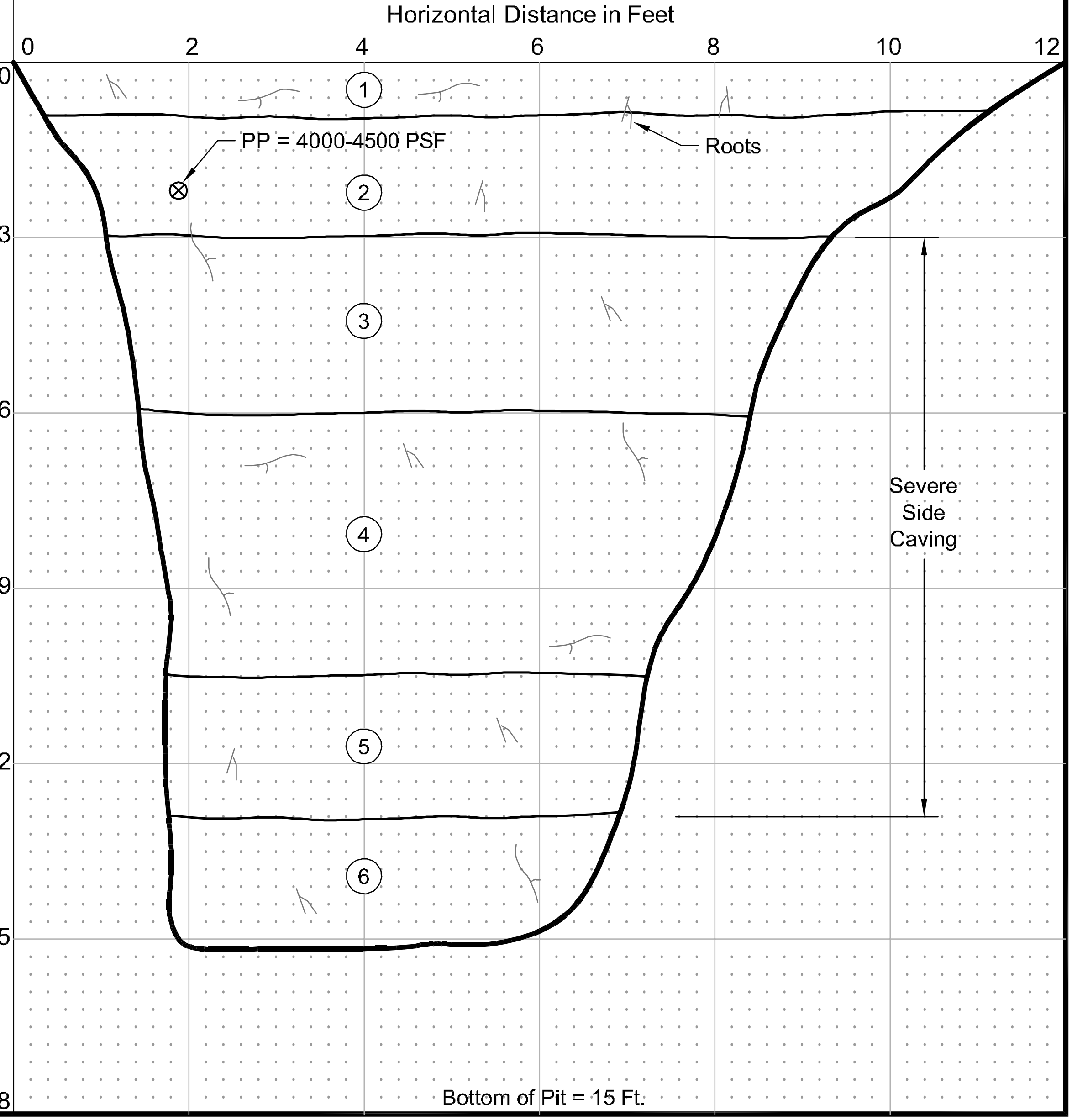
LOG OF TEST PIT SWT-228

JOB NO: 23-1-01178-010   DATE: 1-17-09   LOCATION: Sta 627+00, ±38' Left

PROJECT: Pioneer Crossing

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) Loose, dark brown, clayey SILT; numerous roots and organics; (Topsoil) ML.				0
(2) Very stiff, brown, clayey SILT; moist; scattered roots; ML.	∇		S-1	
(3) Very soft, light gray, clayey SILT; wet; scattered roots; ML.			S-2	3
(4) Loose, gray, alternating layers of sandy SILT and silty SAND; wet; scattered shell fragments and roots; ML/SM.			S-3	6
(5) Loose, gray, sandy, fine to medium GRAVEL; wet; numerous shell fragments; GP.				9
(6) Soft to medium stiff, brown-gray, clayey SILT; wet; scattered roots; ML.			S-4	12
			S-5	15
				18

Sketch of East Side of Pit   Surface Elevation: Approx. 4493.7 Ft.



LEGEND

TV = Torvane Readings
PP = Pocket Penetrometer Readings
PSF = Pounds per Square Foot

FIG. GD5-A-37

Filename: J:\231\01178-010\23-1-01178-010 TPs.dwg Layout: SWT-229 Date: 02-25-2009 Login: cnt

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT SWT-229

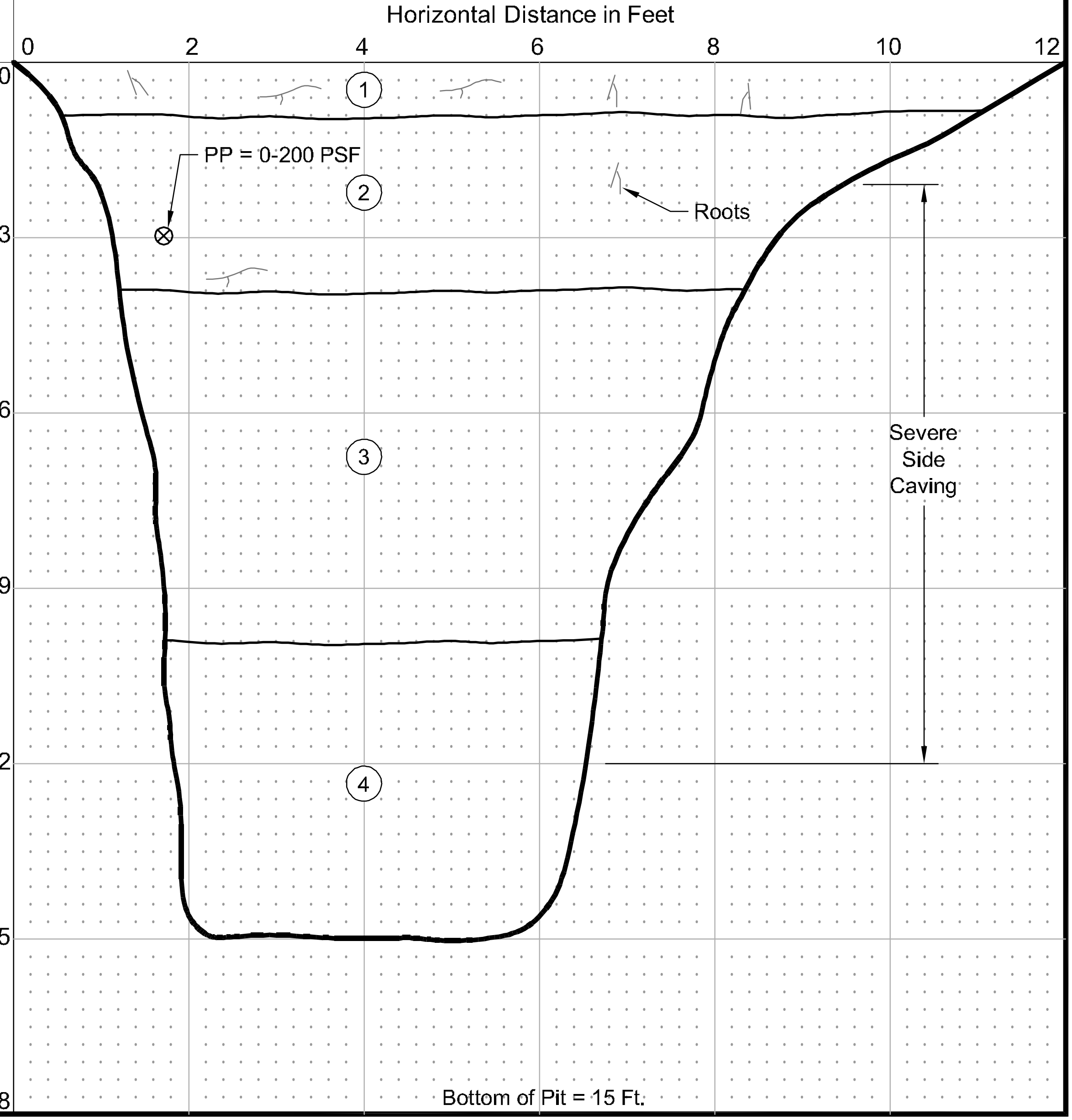
JOB NO: 23-1-01178-010 DATE: 1-17-09 LOCATION: Sta 632+00, ±50' Right

PROJECT: Pioneer Crossing

Sketch of North Side of Pit

Surface Elevation: Approx. 4492.5 Ft.

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) Soft, dark brown, organic SILT; moist; numerous roots; (Topsoil) OL.				0
(2) Soft, light brown, silty CLAY; moist; scattered roots; CL.	▽		S-1	3
(3) Very soft, gray, clayey SILT; wet; numerous shell rich layers, trace of sand, abundant organics; ML.			S-2	6
(4) Very soft, dark gray-gray, clayey SILT; wet; ML.			S-3	9, 12, 15, 18



LEGEND

TV = Torvane Readings
PP = Pocket Penetrometer Readings
PSF = Pounds per Square Foot

FIG. GD5-A-38

Filename: J:\231\01178-010\23-1-01178-010 TPs.dwg Layout: SWT-230 Date: 02-25-2009 Login: cnt

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT SWT-230

JOB NO: 23-1-01178-010 DATE: 1-17-09 LOCATION: Sta 633+80, ±50'Right

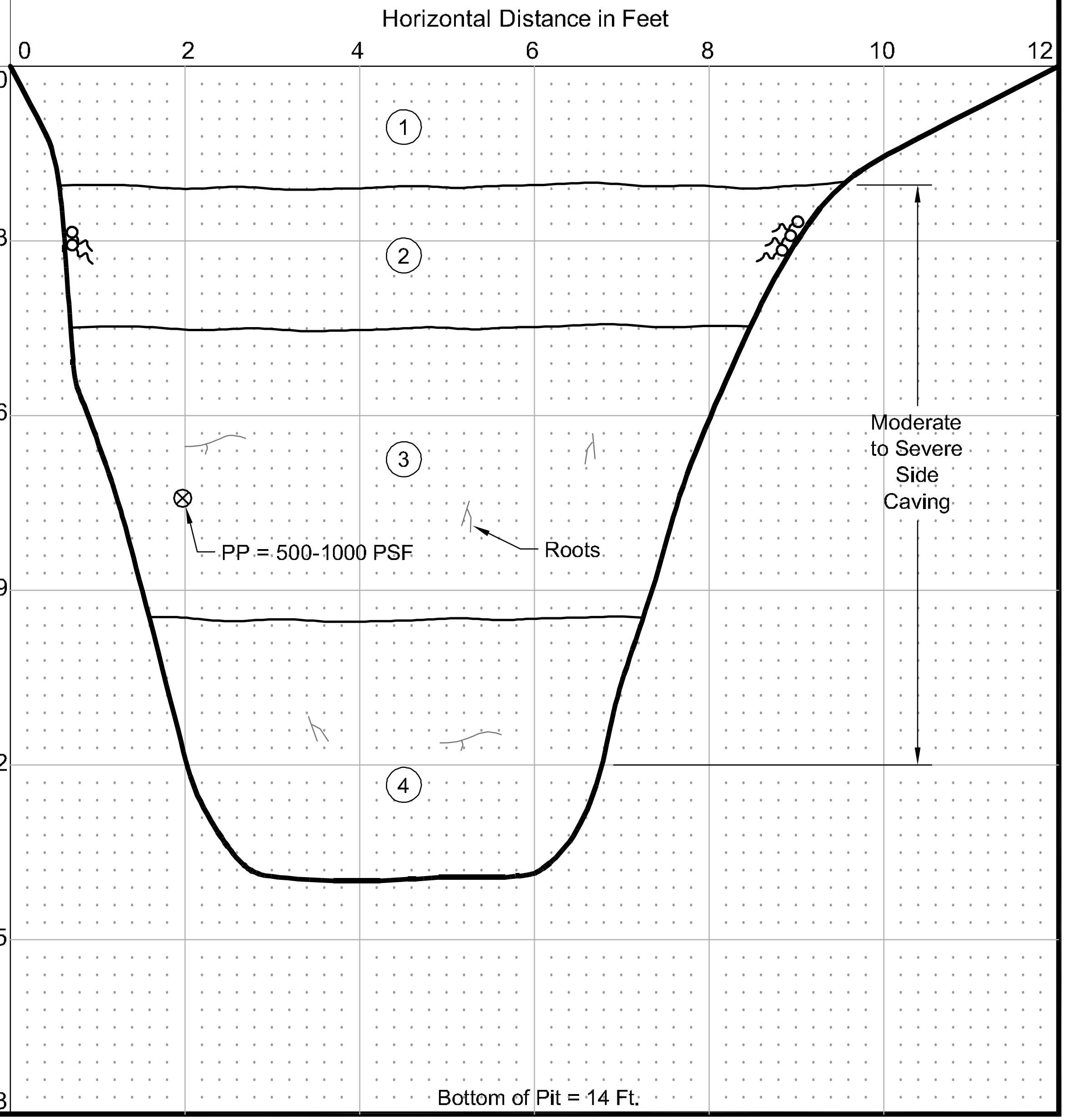
PROJECT: Pioneer Crossing

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) Soft, dark brown, organic clayey SILT; moist to wet; (Topsoil); OL.	∇			0
(2) Very soft, dark gray to gray, peaty, clayey SILT; wet; OL.			S-1	3
(3) Medium stiff, gray, clayey SILT; wet; scattered to numerous roots; ML.			S-2	6
(4) Very soft, gray, clayey SILT; wet; scattered roots; ML.			S-3	9, 12, 15, 18

NOTE

Gushing water observed at approx. 3'.

Sketch of West Side of Pit Surface Elevation: Approx. 4492.9 Ft.



LEGEND

TV = Torvane Readings
PP = Pocket Penetrometer Readings
PSF = Pounds per Square Foot

FIG. GD5-A-39

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT SWT-231

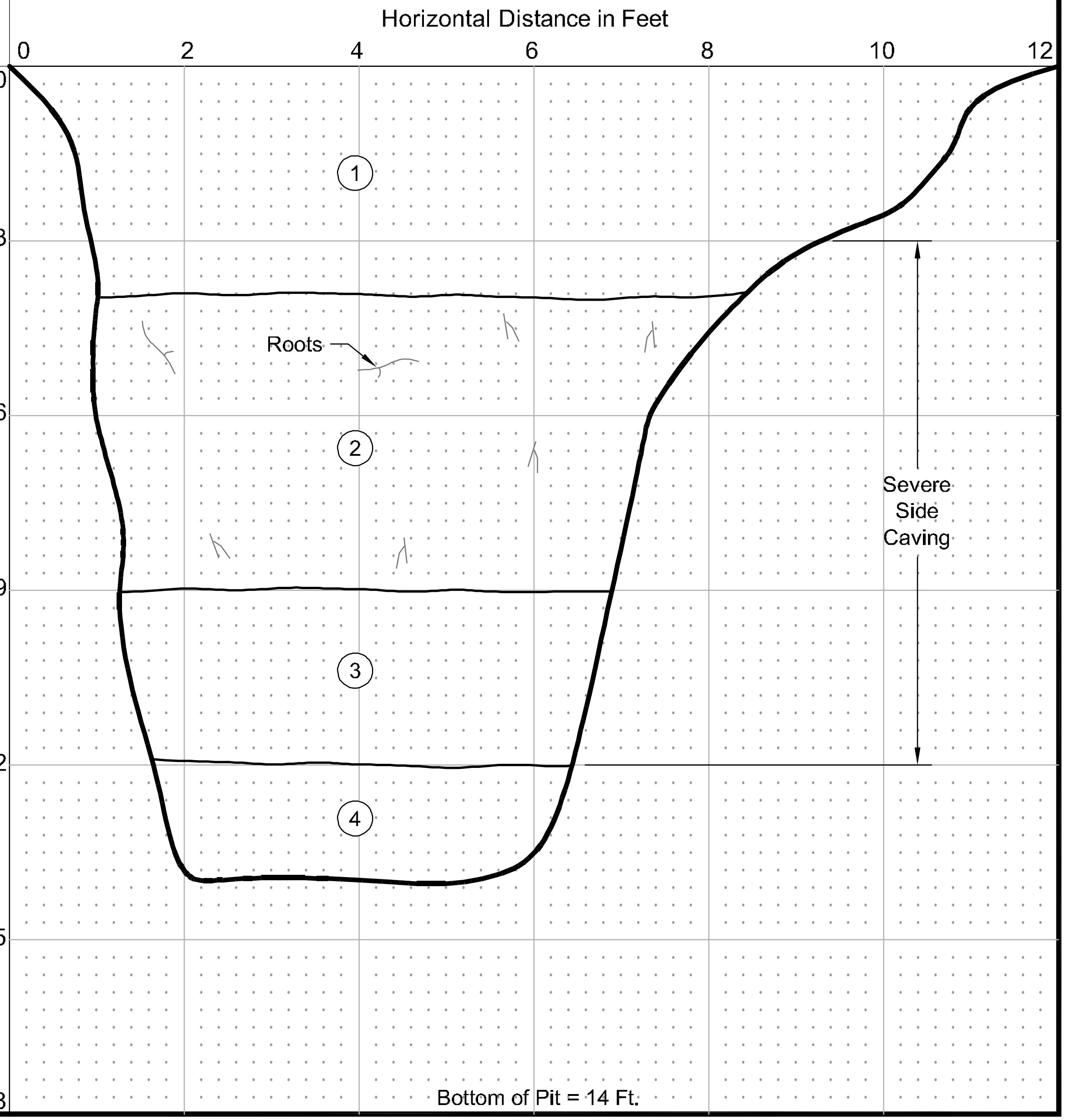
JOB NO: 23-1-01178-010 DATE: 1-17-09 LOCATION: Sta 635+80, ±50' Right

PROJECT: Pioneer Crossing

Sketch of North Side of Pit

Surface Elevation: Approx. 4493.1 Ft.

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
1 Soft, dark brown, peaty clayey SILT; moist to wet; (Swamp Soil) OL.				0
2 Very soft, gray, silty CLAY; wet; numerous roots; CL.	∇		S-1	3
3 Loose, gray-brown, silty SAND; wet; SM.				
4 Soft, light gray, sandy, gravelly, clayey SILT; wet; ML.			S-2	6
				9
			S-3	
				12
			S-4	
				15
				18



LEGEND

TV = Torvane Readings
PP = Pocket Penetrometer Readings
PSF = Pounds per Square Foot

FIG. GD5-A-40

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT SWT-232

JOB NO: 23-1-01178-010 DATE: 1-17-09 LOCATION: Sta 637+80, ±50' Right

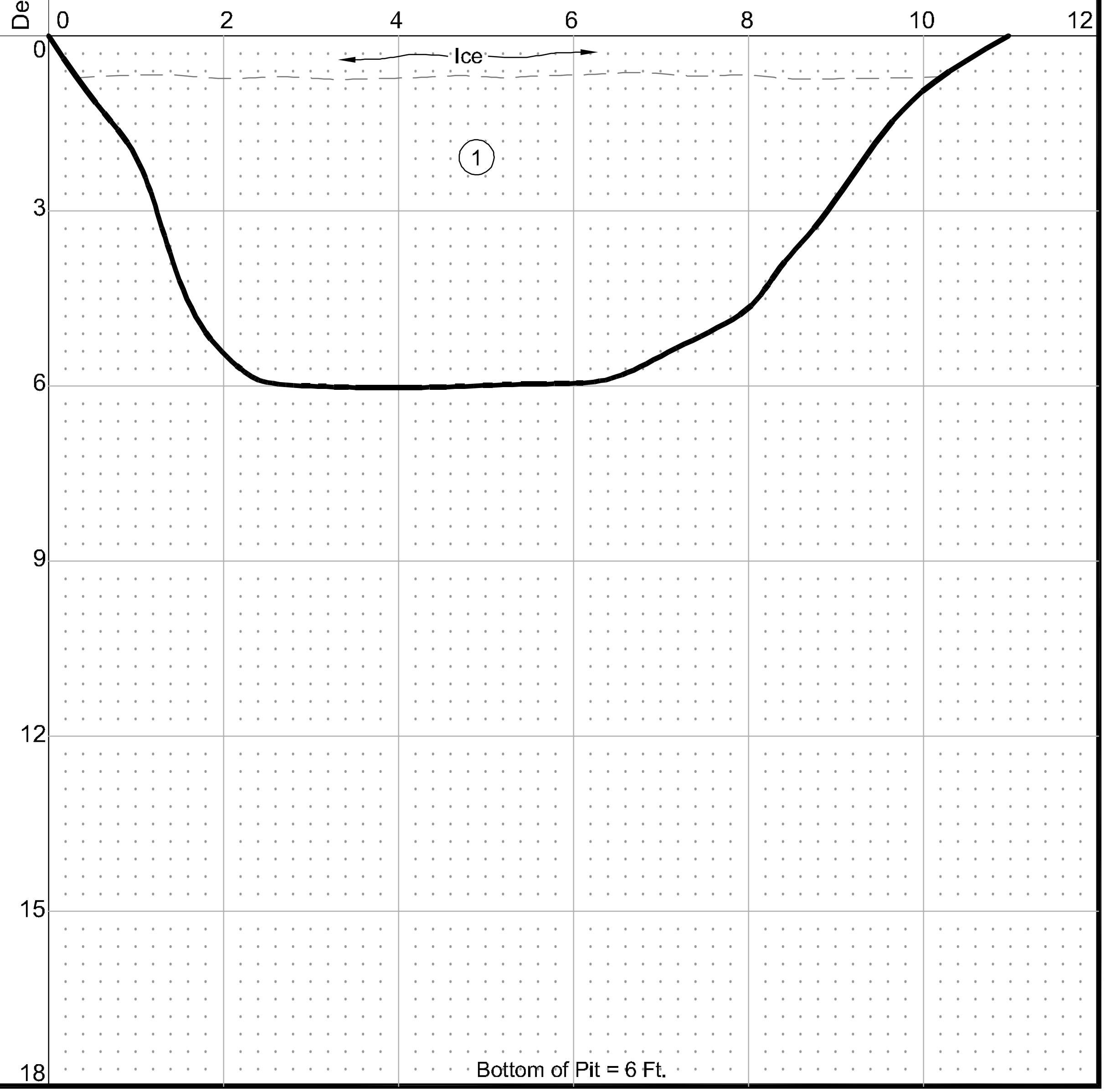
PROJECT: Pioneer Crossing

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) Very soft, dark brown, peaty, clayey SILT; wet; (Swamp Soil) OL.	(At Surface)			0
			S-1	3
			S-2	6
				9
			S-3	
				12
			S-4	
				15
				18

Sketch of North Side of Pit

Surface Elevation: Approx. 4491.4 Ft.

Horizontal Distance in Feet



Bottom of Pit = 6 Ft.

NOTES

1. Test pit is located in a pond.
2. Test pit was stopped at 6' due to concerns of damage to the wetland.

LEGEND

TV = Torvane Readings
PP = Pocket Penetrometer Readings
PSF = Pounds per Square Foot

FIG. GD5-A-41

Filename: J:\231\01178-010\23-1-01178-010 TPs.dwg   Layout: SWT-233   Date: 02-25-2009   Login: cnt

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

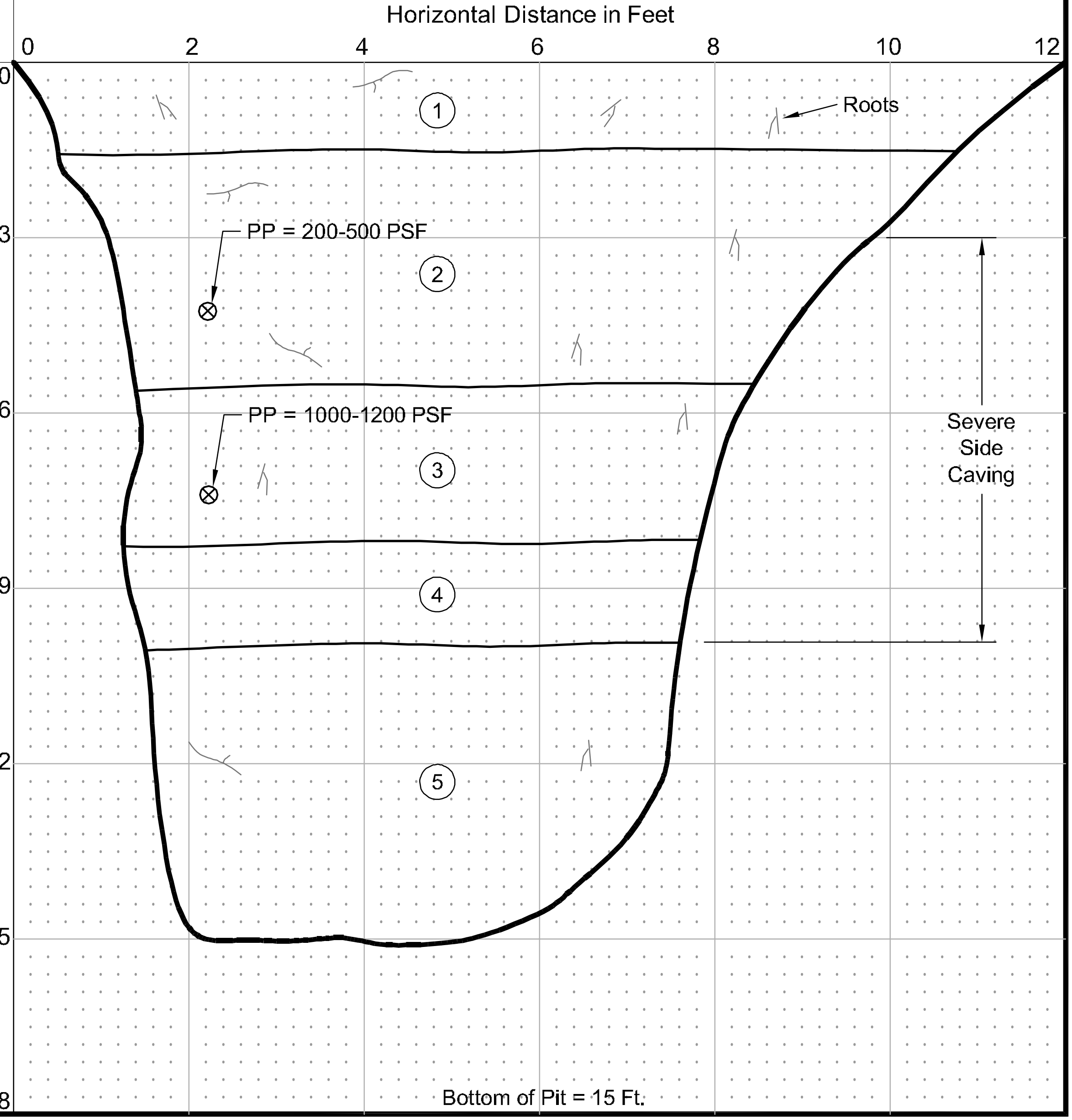
LOG OF TEST PIT SWT-233

JOB NO: 23-1-01178-010   DATE: 1-18-09   LOCATION: Sta 640+00, ±50' Right

PROJECT: Pioneer Crossing

SOIL DESCRIPTION	Ground Water	% Water Content	Samples	Depth, Ft.
(1) Soft, dark brown, organic clayey SILT; moist; OL.				0
(2) Medium stiff, dark gray to dark brown, silty CLAY; wet; scattered roots; CL.			S-1	3
(3) Stiff, brown-gray, silty CLAY; moist; scattered roots, occasional gravel; CL.	▽			
(4) Very soft, brown, gray, clayey SILT; wet; trace of sand, scattered shell fragments; ML.			S-2	6
(5) Soft, gray, silty CLAY; wet; scattered roots; CL.			S-3	9
			S-4	12
				15
				18

Sketch of South Side of Pit   Surface Elevation: Approx. 4492.2 Ft.



LEGEND

TV = Torvane Readings
PP = Pocket Penetrometer Readings
PSF = Pounds per Square Foot

FIG. GD5-A-42

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

LOG OF TEST PIT SWT-234

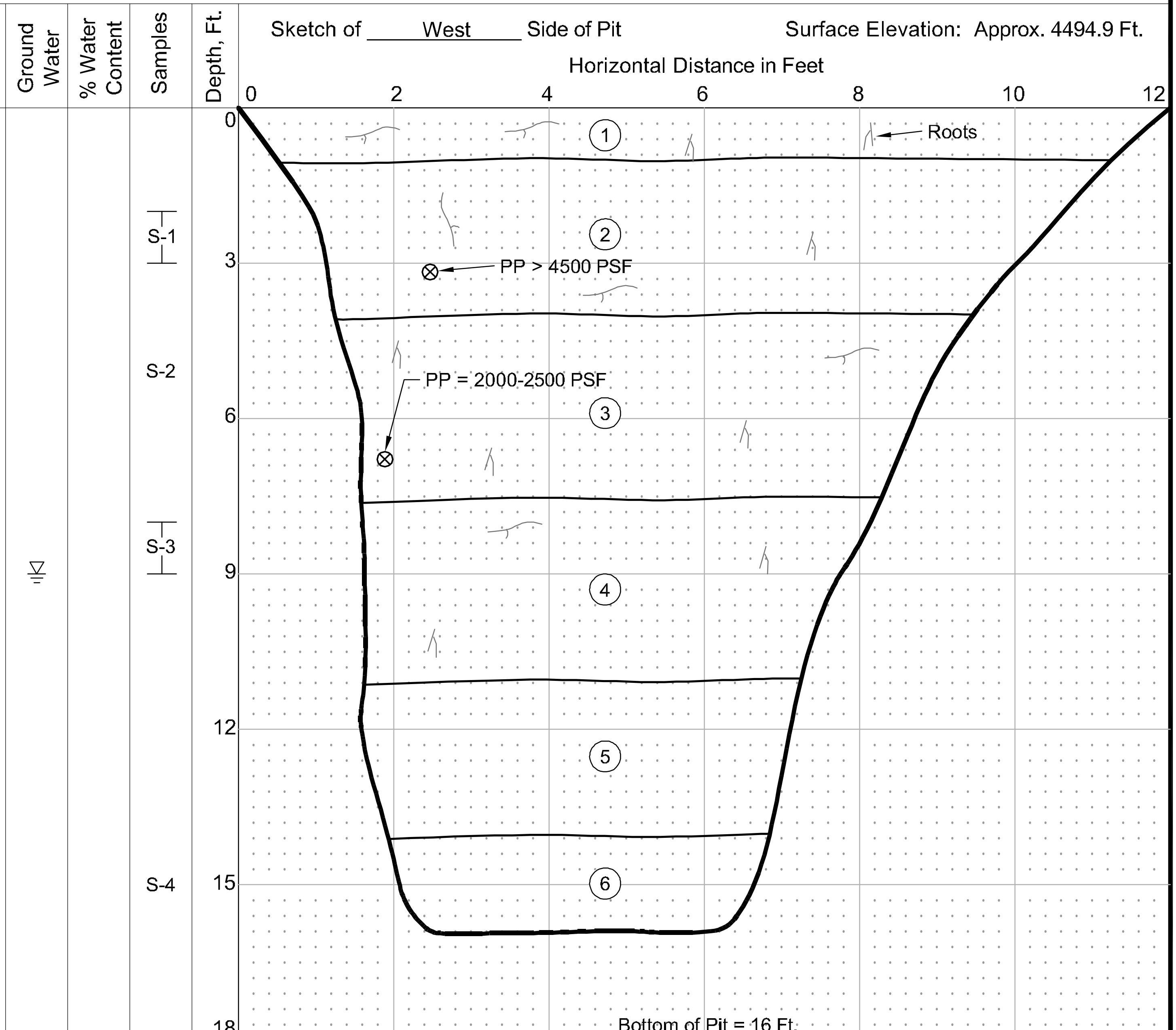
JOB NO: 23-1-01178-010 DATE: 1-17-09 LOCATION: Sta 628+00, ±125' Right

PROJECT: Pioneer Crossing

Sketch of West Side of Pit — Surface Elevation: Approx. 4494.9 Ft.

SOIL DESCRIPTION

1. Loose, dark brown, clayey SILT; moist; numerous roots and organics; (Topsoil) ML.
2. Very stiff, light brown, clayey SILT; moist; scattered roots; ML.
3. Medium stiff to stiff, light brown, trace to slightly sandy, clayey SILT; moist; scattered roots; ML.
4. Loose, gray, silty SAND; moist to wet; micaceous; numerous shell layers; scattered roots; SM.
5. Soft, dark brown-gray, clayey SILT to silty CLAY, trace of sand; numerous organic layers and shell layers; ML/CL.
6. Loose, gray, clean fine to coarse SAND; wet; trace of gravel and shells; SP.

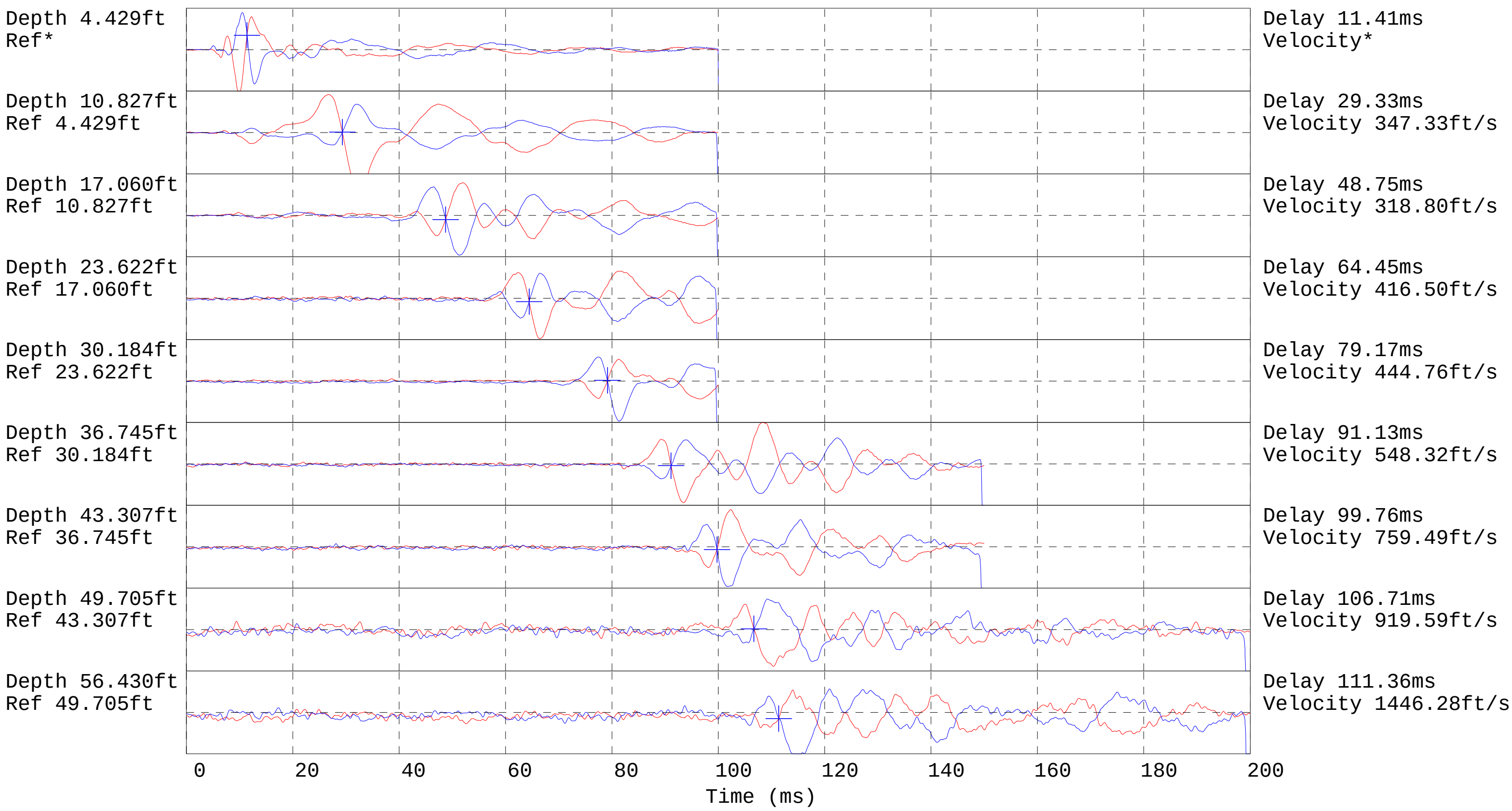


LEGEND

TV = Torvane Readings
PP = Pocket Penetrometer Readings
PSF = Pounds per Square Foot

FIG. GD5-A-43

Pioneer Crossing Shear Wave Velocity Plots
SWC 211



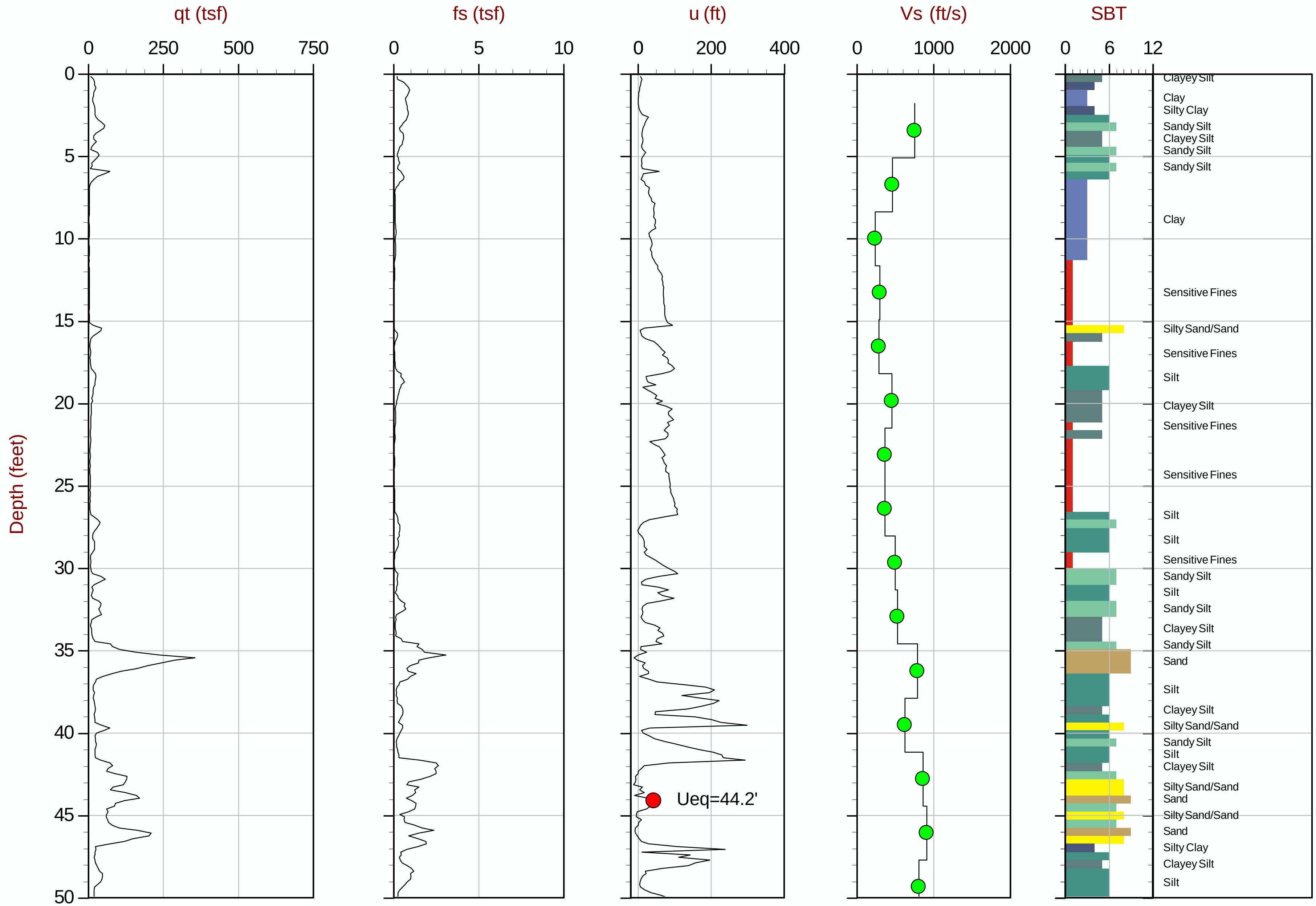
Hammer to Rod String Distance 0.5 (m)
* = Not Determined

FIG. GD5-A-44

Terracon

Job No: 08-340
Date: 03:19:08 11:54
Site: EAST WEST CONNECTOR

Sounding: CPT-02
Cone: STD 20T AD197



Max Depth: 18.250 m / 59.87 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.150 m

File: 340CP02.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 2
Equilibrium Pore Pressure from Dissipation

FIG. GDG-A-45 (SHEET 1 OF 2)

CONETEC

Terracon

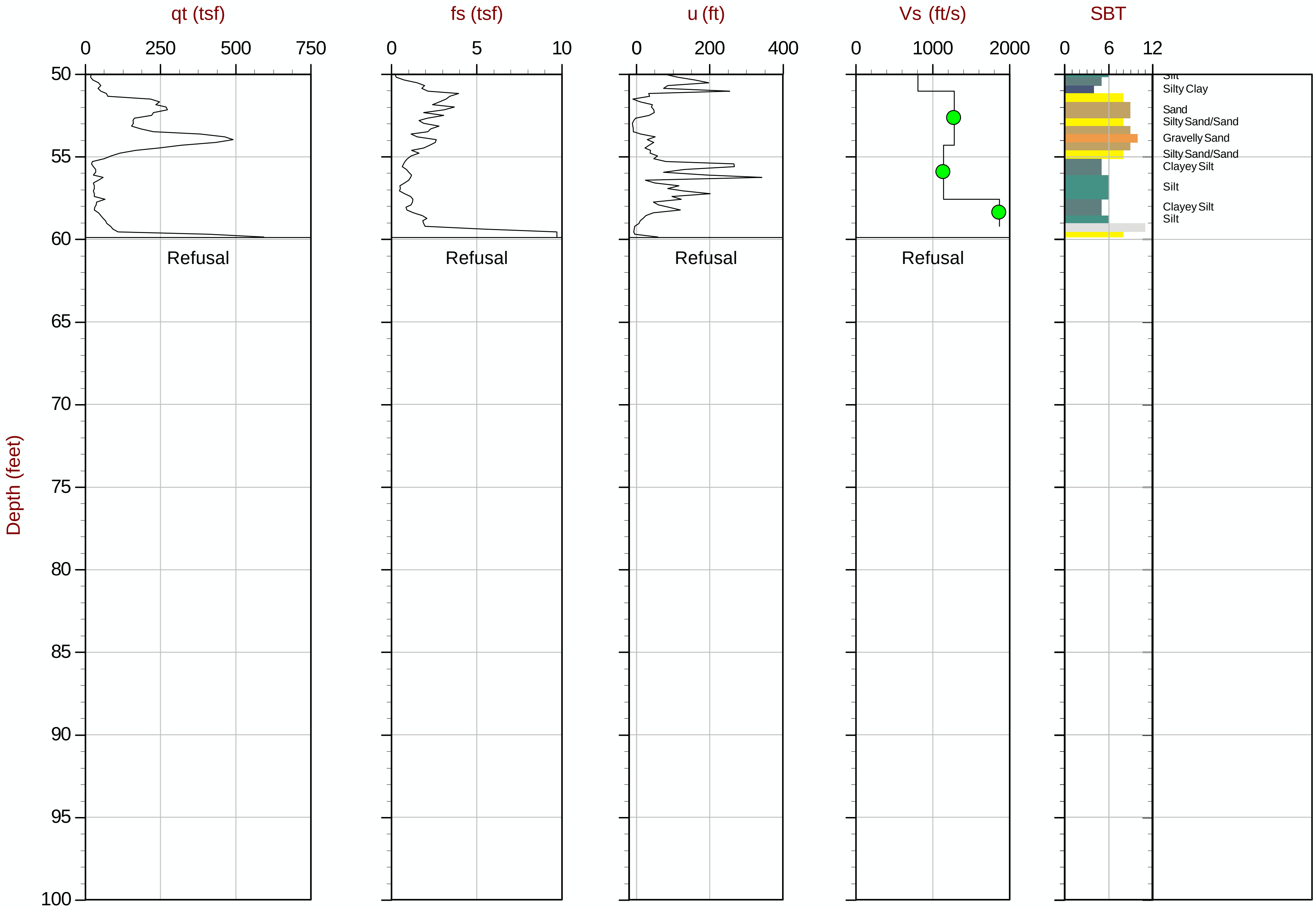
Job No: 08-340

Date: 03:19:08 11:54

Site: EAST WEST CONNECTOR

Sounding: CPT-02

Cone: STD 20T AD197



Max Depth: 18.250 m / 59.87 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.150 m

File: 340CP02.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
Page No: 2 of 2

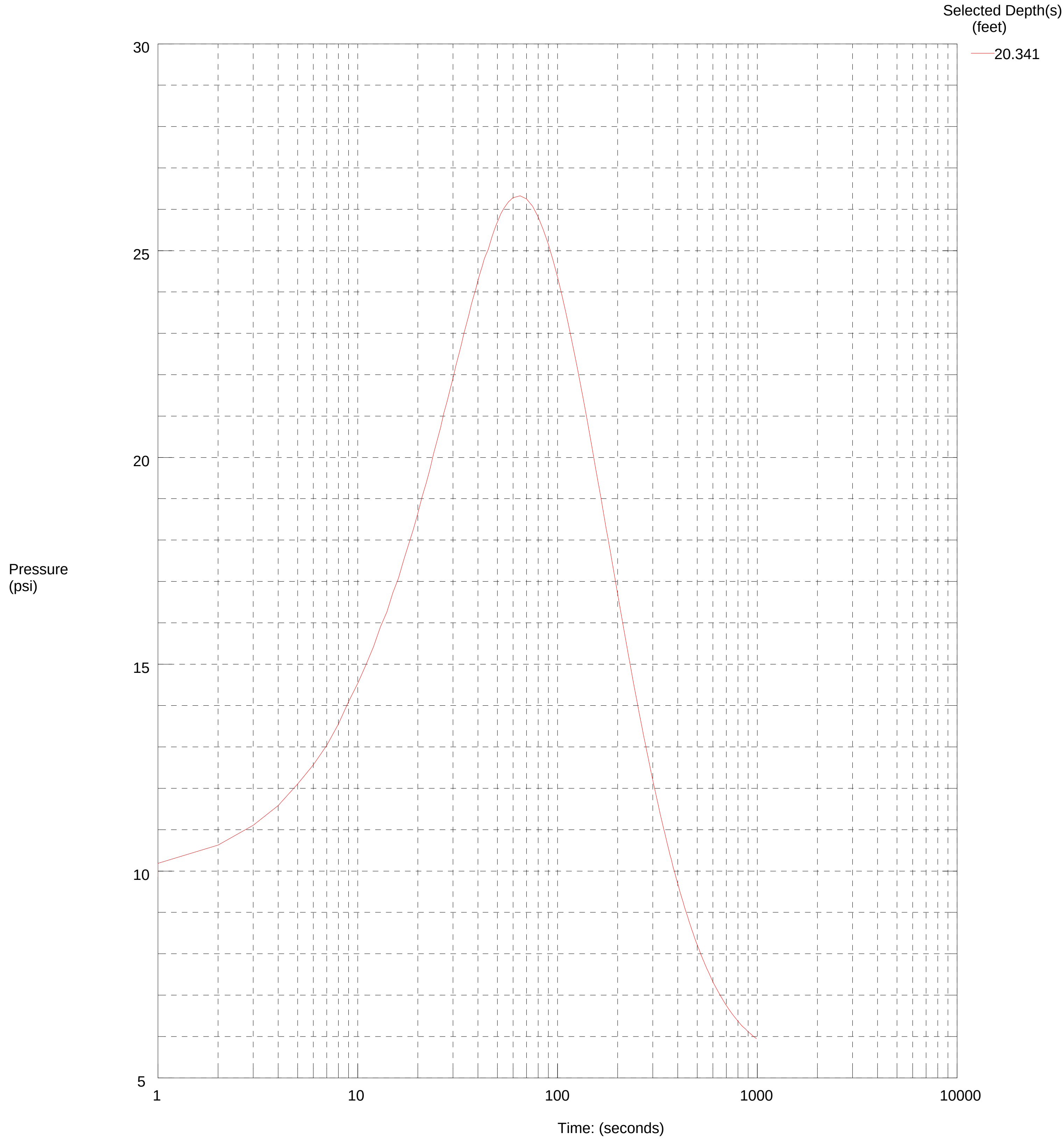
Equilibrium Pore Pressure from Dissipation

FIG. GD5-A-45 (SHEET 2 OF 2)

Shannon & Wilson

Operator Brown
Sounding: SWC 211
Cone Used: DSG1065

CPT Date/Time: 12/12/2008 10:48:09 AM
Location: Pioneer Crossing Sta 622+80
Job Number: 23-1-01178-010



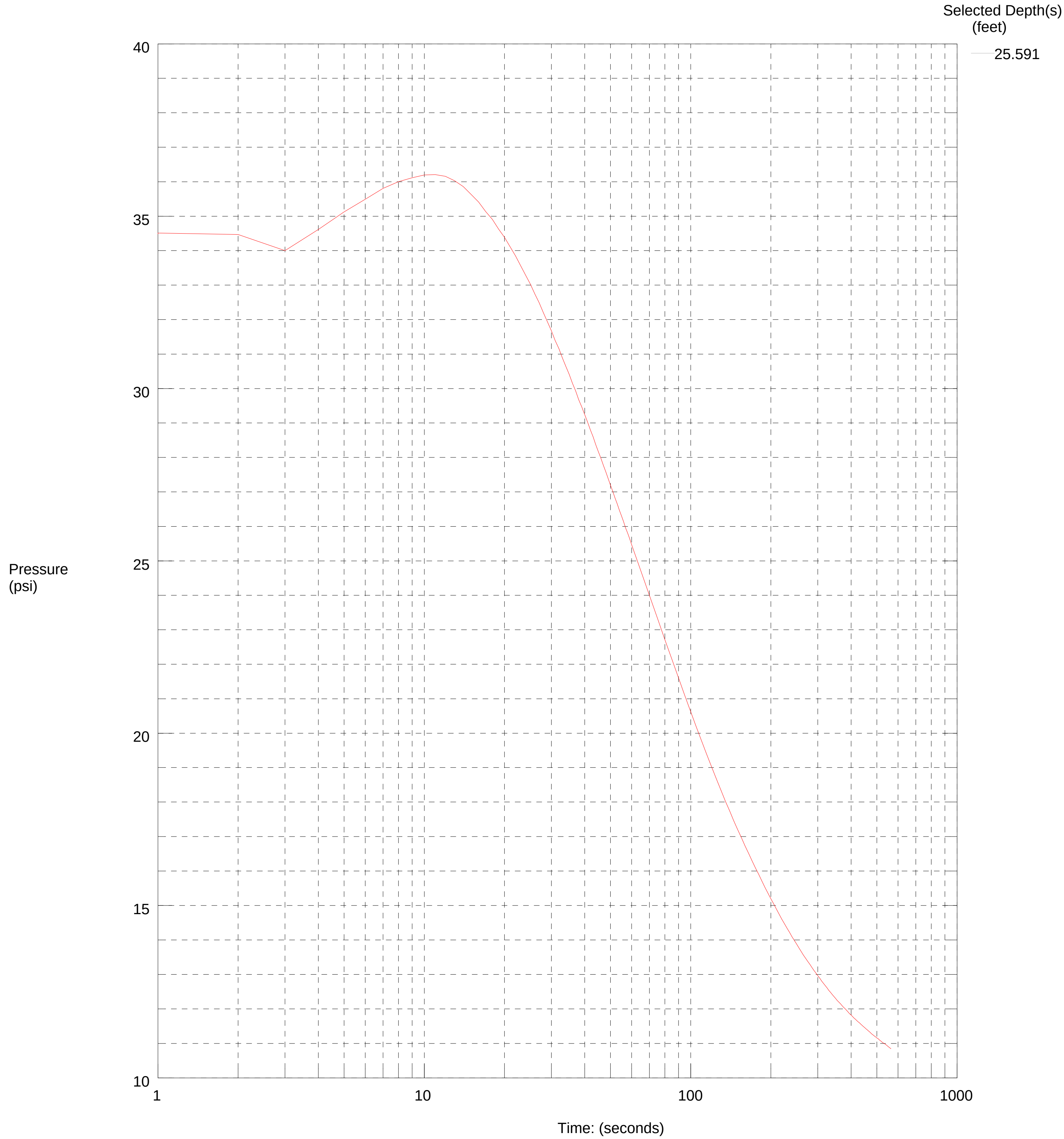
Maximum Pressure = 26.328 psi

FIG. GD5-A-46

Shannon & Wilson

Operator Brown
Sounding: SWC 211
Cone Used: DSG1065

CPT Date/Time: 12/12/2008 10:48:09 AM
Location: Pioneer Crossing Sta 622+80
Job Number: 23-1-01178-010



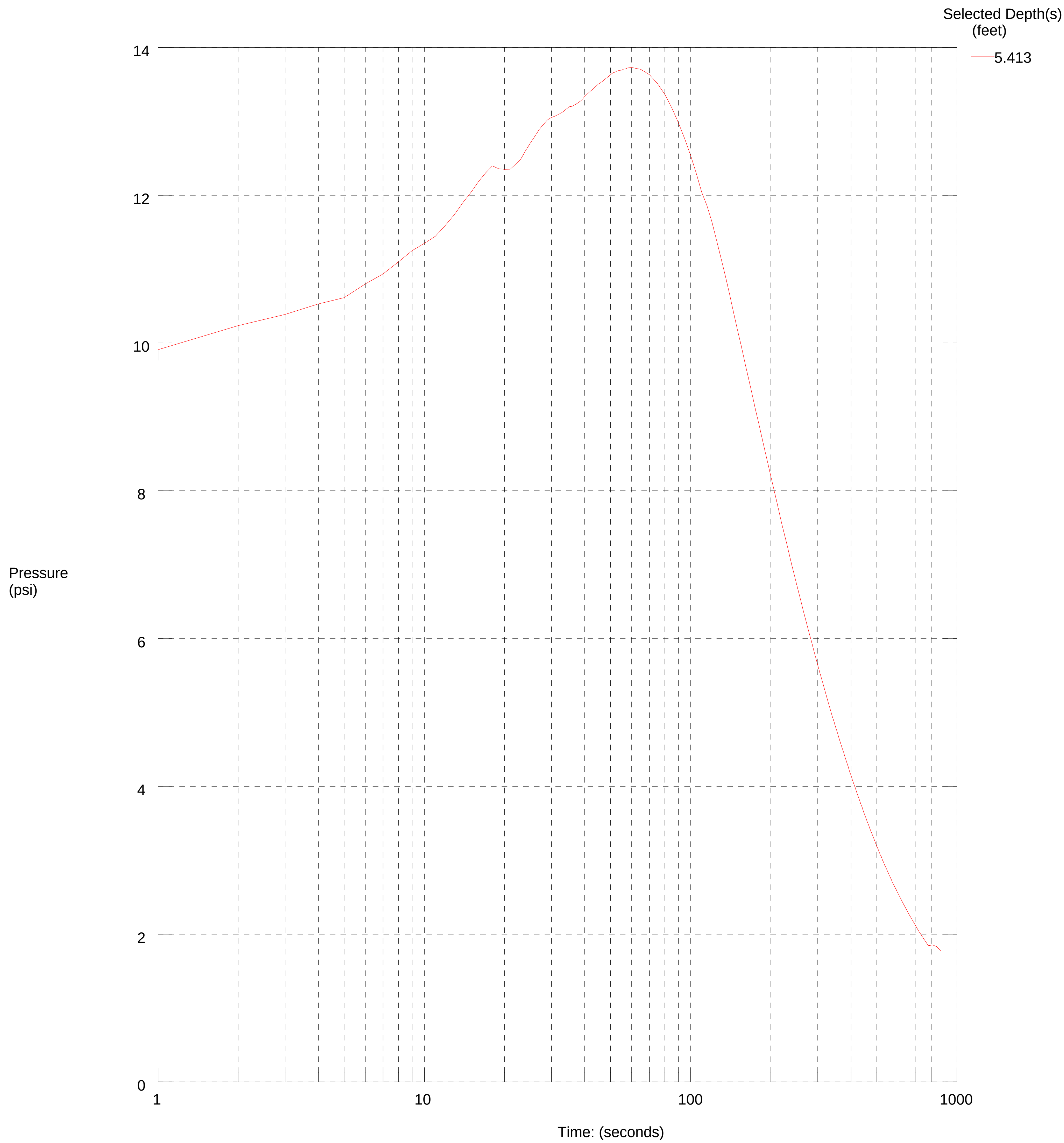
Maximum Pressure = 36.207 psi

FIG. GD5-A-47

Shannon & Wilson

Operator Brown
Sounding: SWC-212
Cone Used: DSG1065

CPT Date/Time: 12/13/2008 11:32:18 AM
Location: Pioneer Crossing STA 631+20
Job Number: 23-1-01178-010



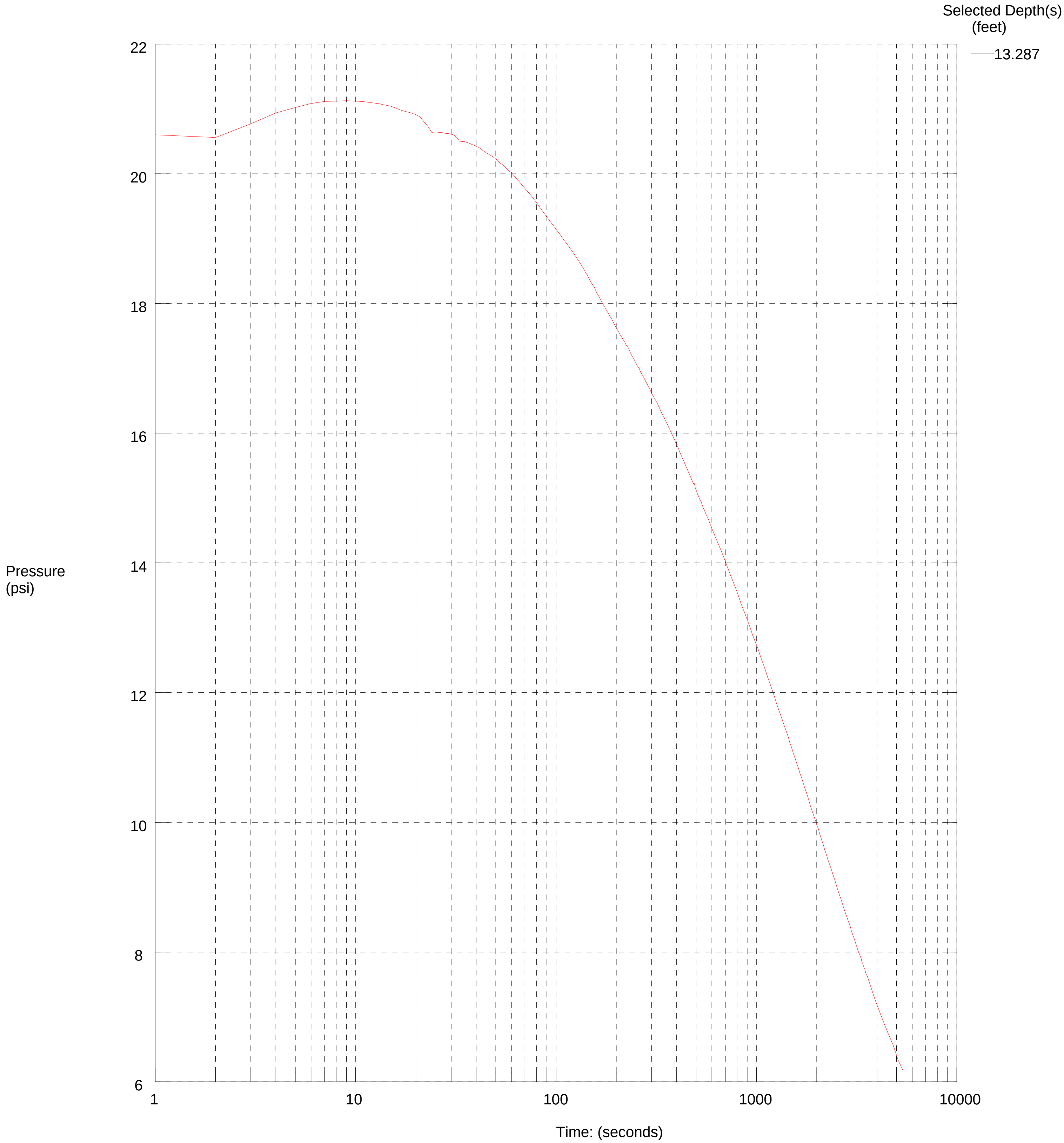
Maximum Pressure = 13.728 psi
.

FIG. GD5-A-48

Shannon & Wilson

Operator Brown
Sounding: SWC-212
Cone Used: DSG1065

CPT Date/Time: 12/13/2008 11:32:18 AM
Location: Pioneer Crossing STA 631+20
Job Number: 23-1-01178-010



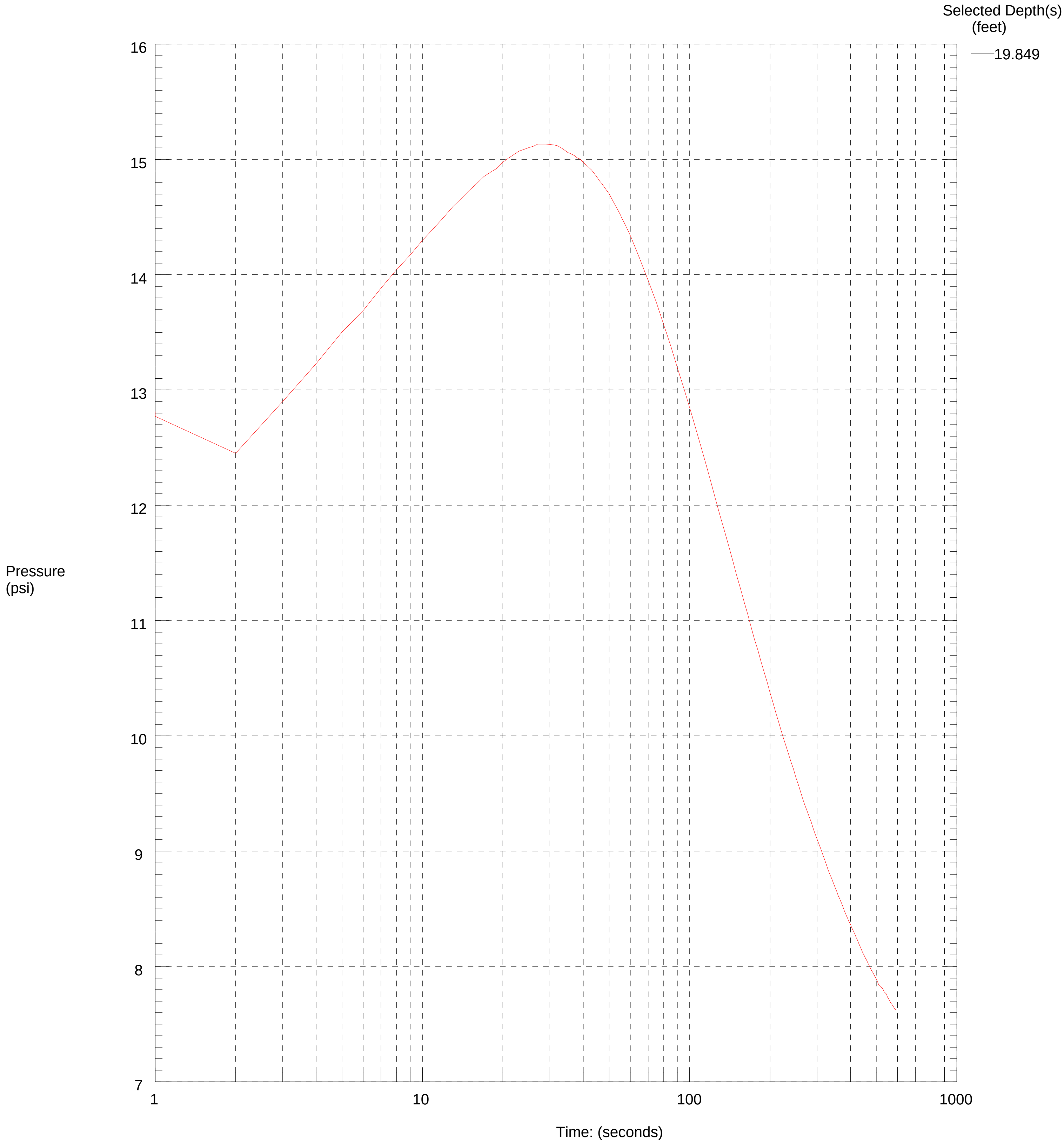
Maximum Pressure = 21.126 psi

FIG. GD5-A-49

Shannon & Wilson

Operator Brown
Sounding: SWC-212
Cone Used: DSG1065

CPT Date/Time: 12/13/2008 11:32:18 AM
Location: Pioneer Crossing STA 631+20
Job Number: 23-1-01178-010



Maximum Pressure = 15.132 psi

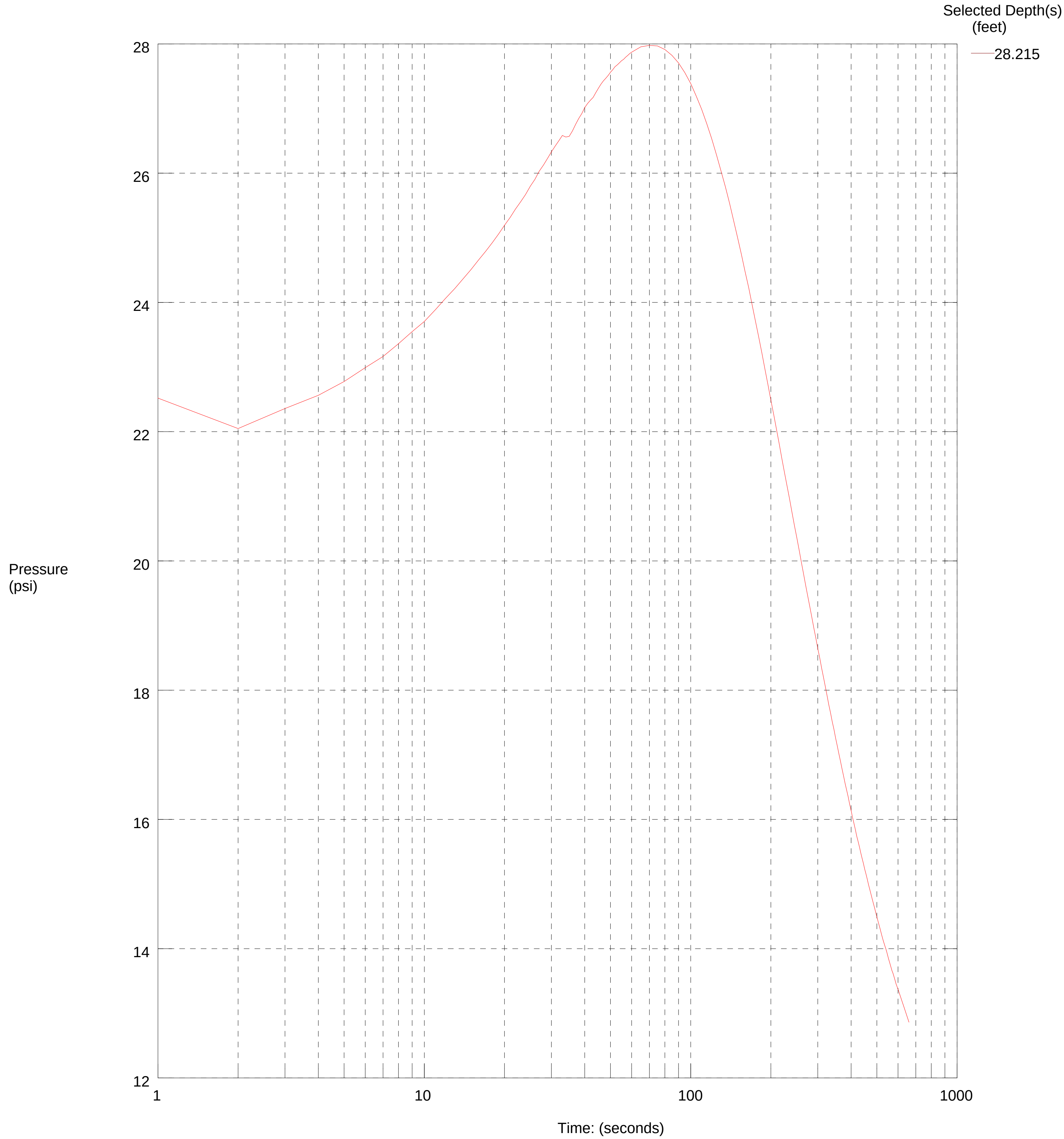
.

FIG. GD5-A-50

Shannon & Wilson

Operator Brown
Sounding: SWC-212
Cone Used: DSG1065

CPT Date/Time: 12/13/2008 11:32:18 AM
Location: Pioneer Crossing STA 631+20
Job Number: 23-1-01178-010



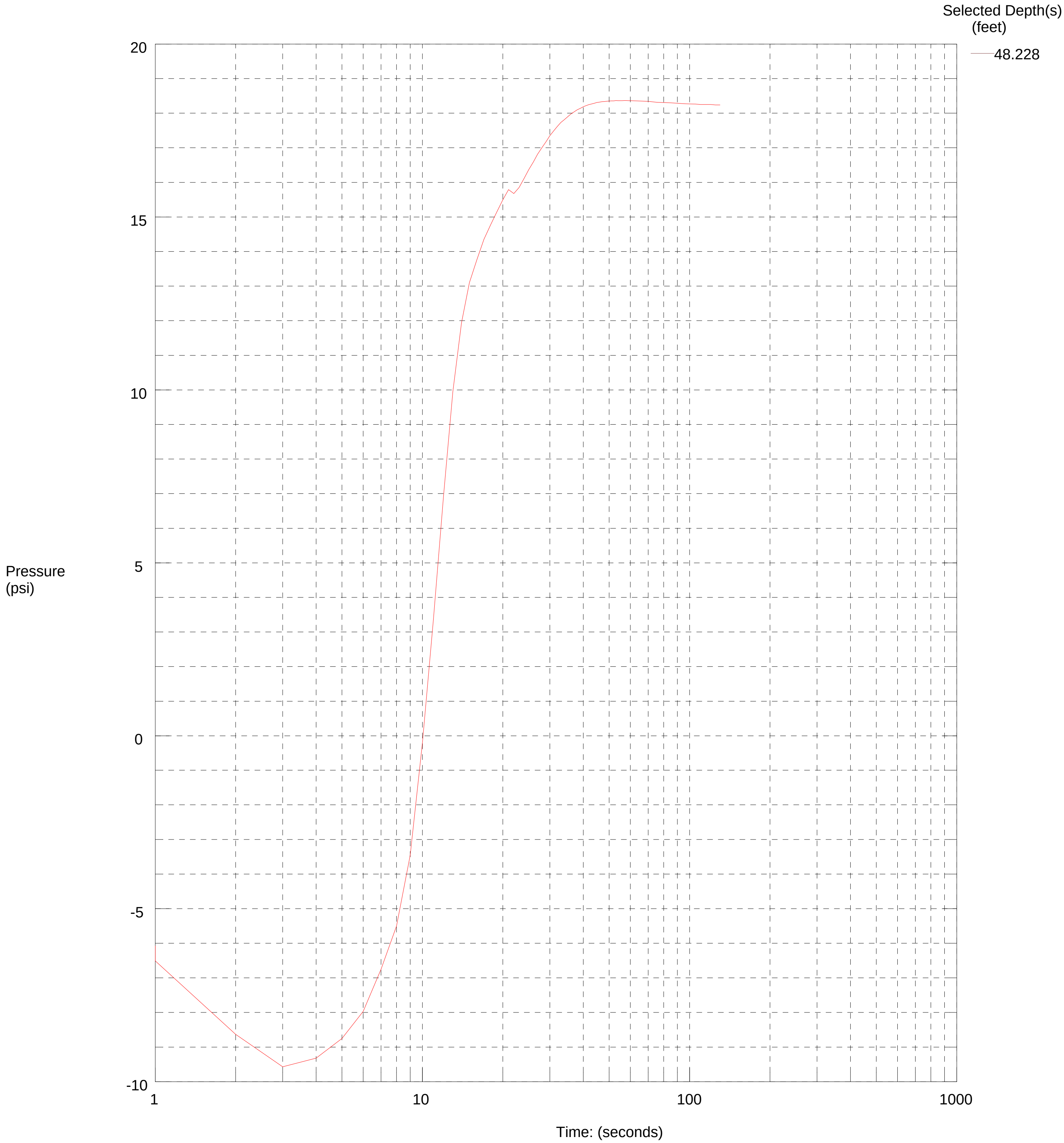
Maximum Pressure = 27.976 psi

FIG. GD5-A-51

Shannon & Wilson

Operator Brown
Sounding: SWC-212
Cone Used: DSG1065

CPT Date/Time: 12/13/2008 11:32:18 AM
Location: Pioneer Crossing STA 631+20
Job Number: 23-1-01178-010



Maximum Pressure = 18.367 psi

FIG. GD5-A-52

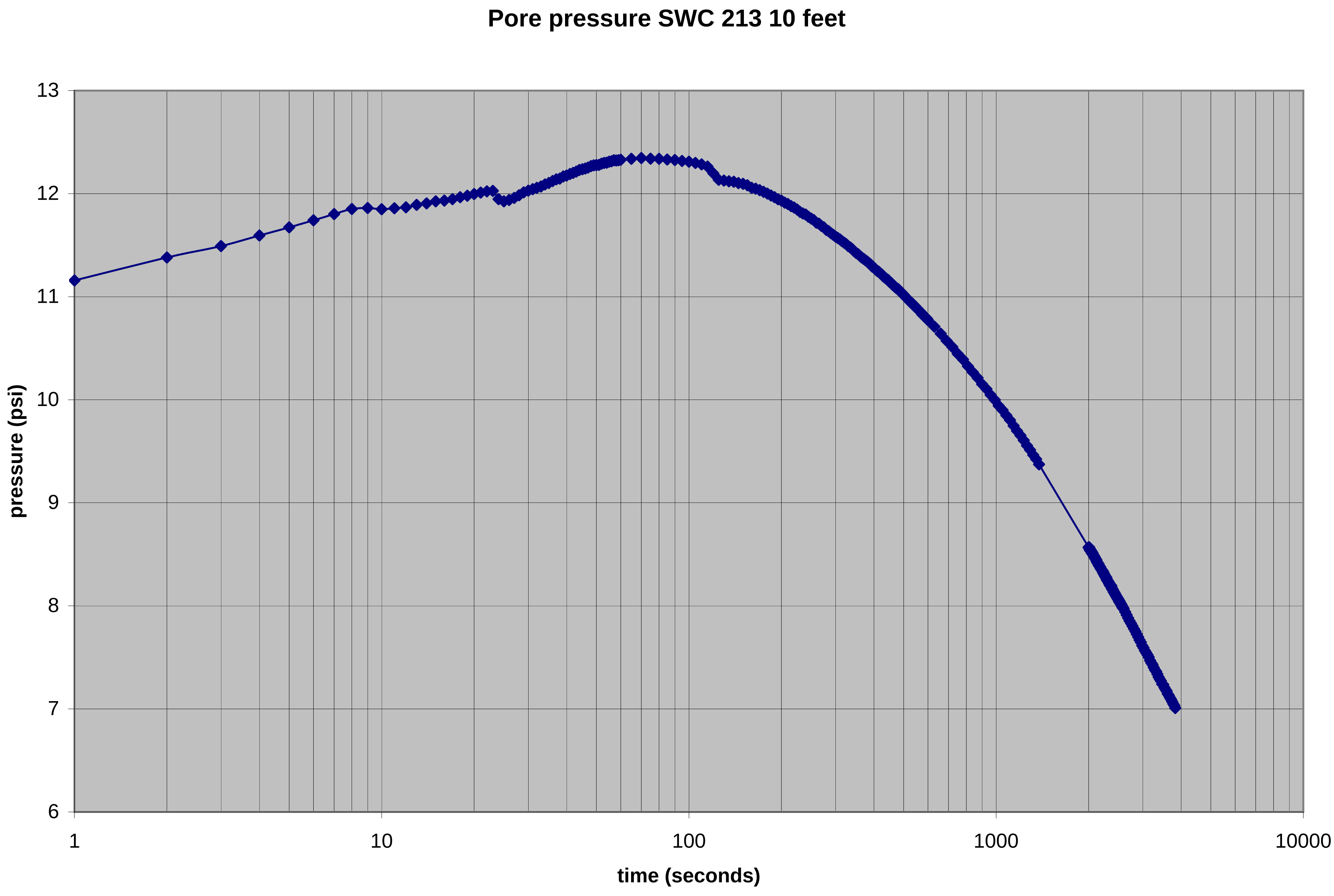


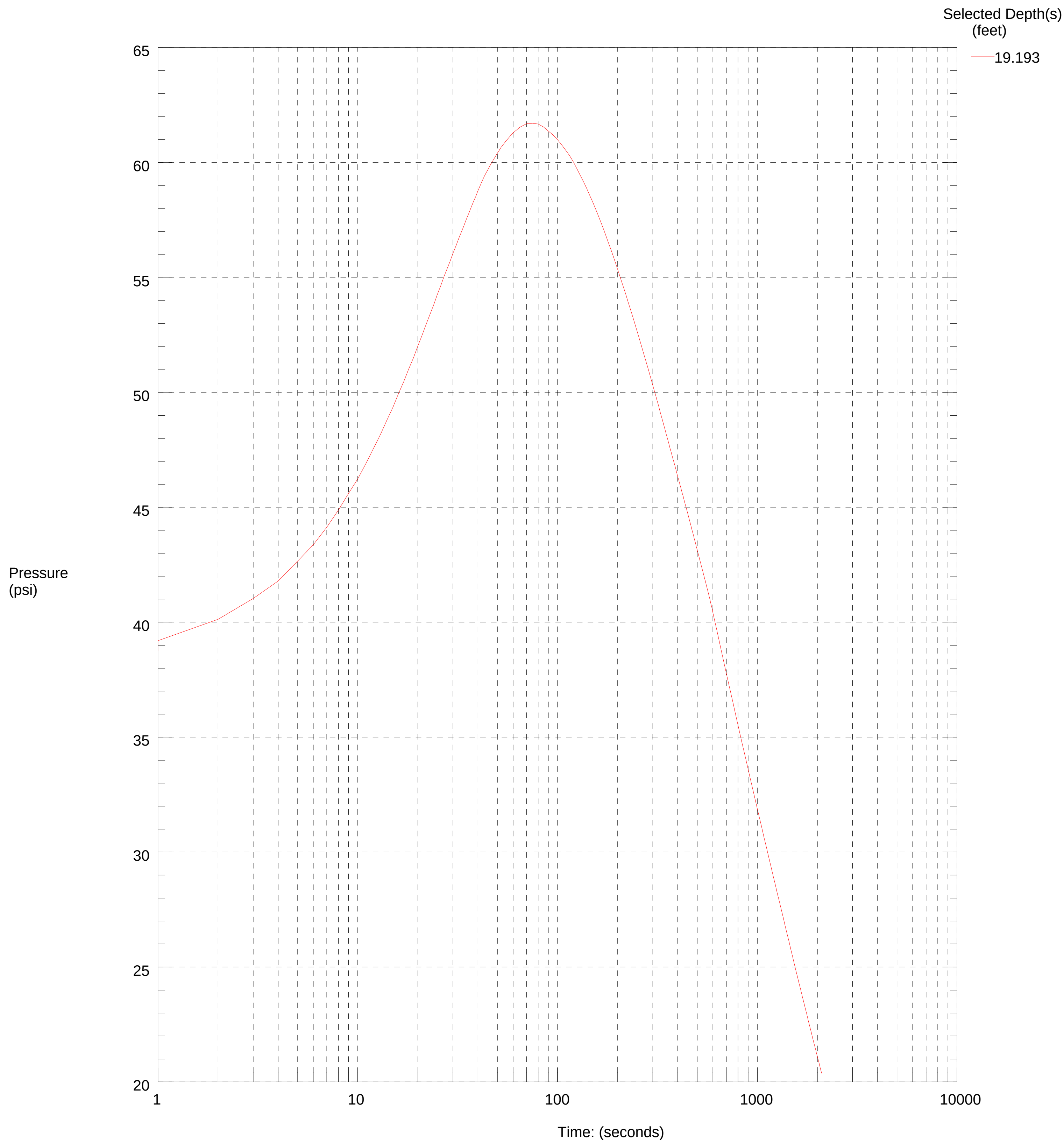
FIG. GD5-A-53

FIG. GD5-A-53

Shannon & Wilson

Operator Brown
Sounding: SWC 214
Cone Used: DSG1065

CPT Date/Time: 12/14/2008 9:29:37 AM
Location: Pioneer Crossing STA 649+00 Elev 4492.5
Job Number: 23-1-01178-010



Maximum Pressure = 61.702 psi

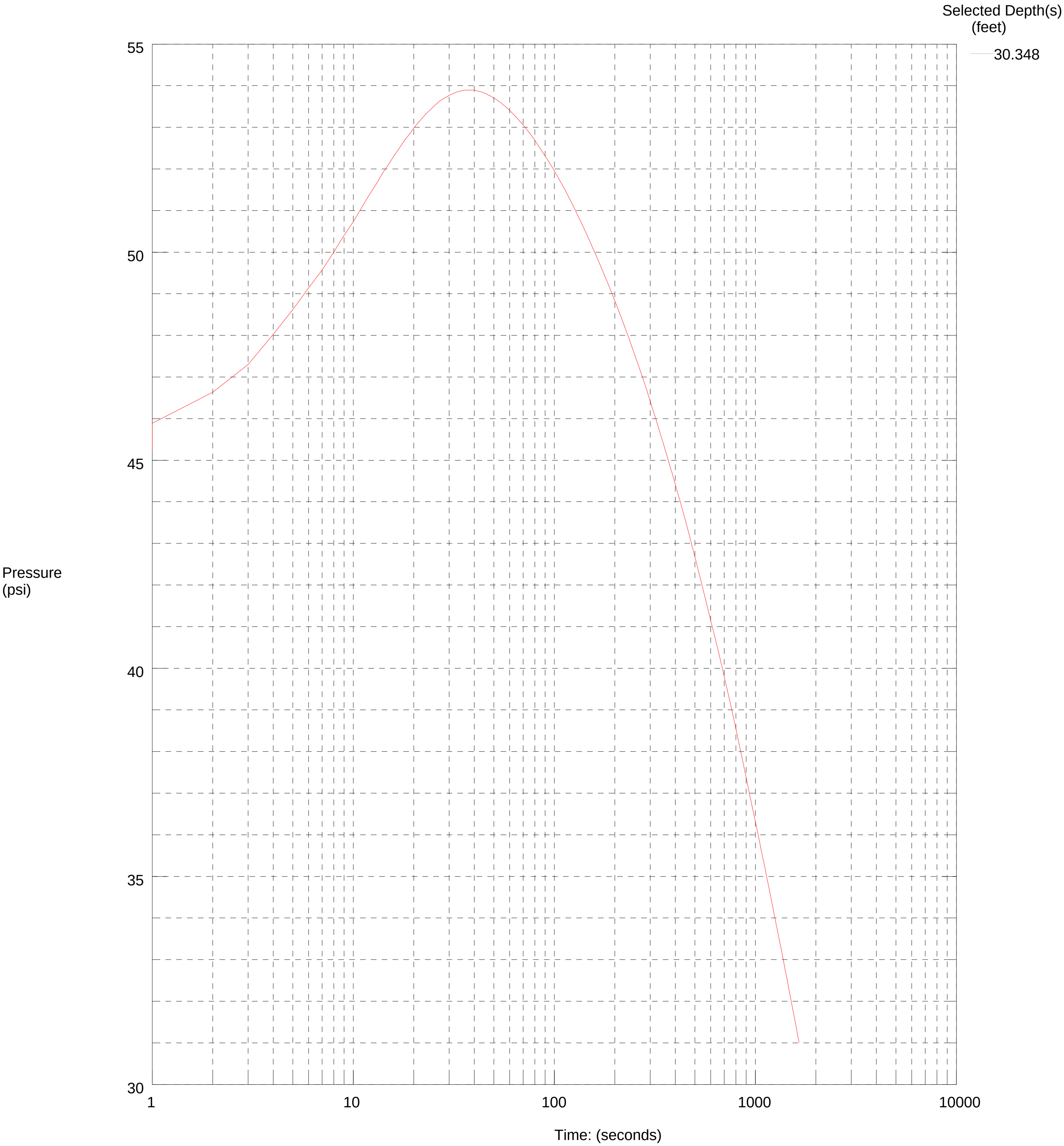
.

FIG. GD5-A-54

Shannon & Wilson

Operator Brown
Sounding: SWC 214
Cone Used: DSG1065

CPT Date/Time: 12/14/2008 9:29:37 AM
Location: Pioneer Crossing STA 649+00 Elev 4492.5
Job Number: 23-1-01178-010



Maximum Pressure = 53.897 psi

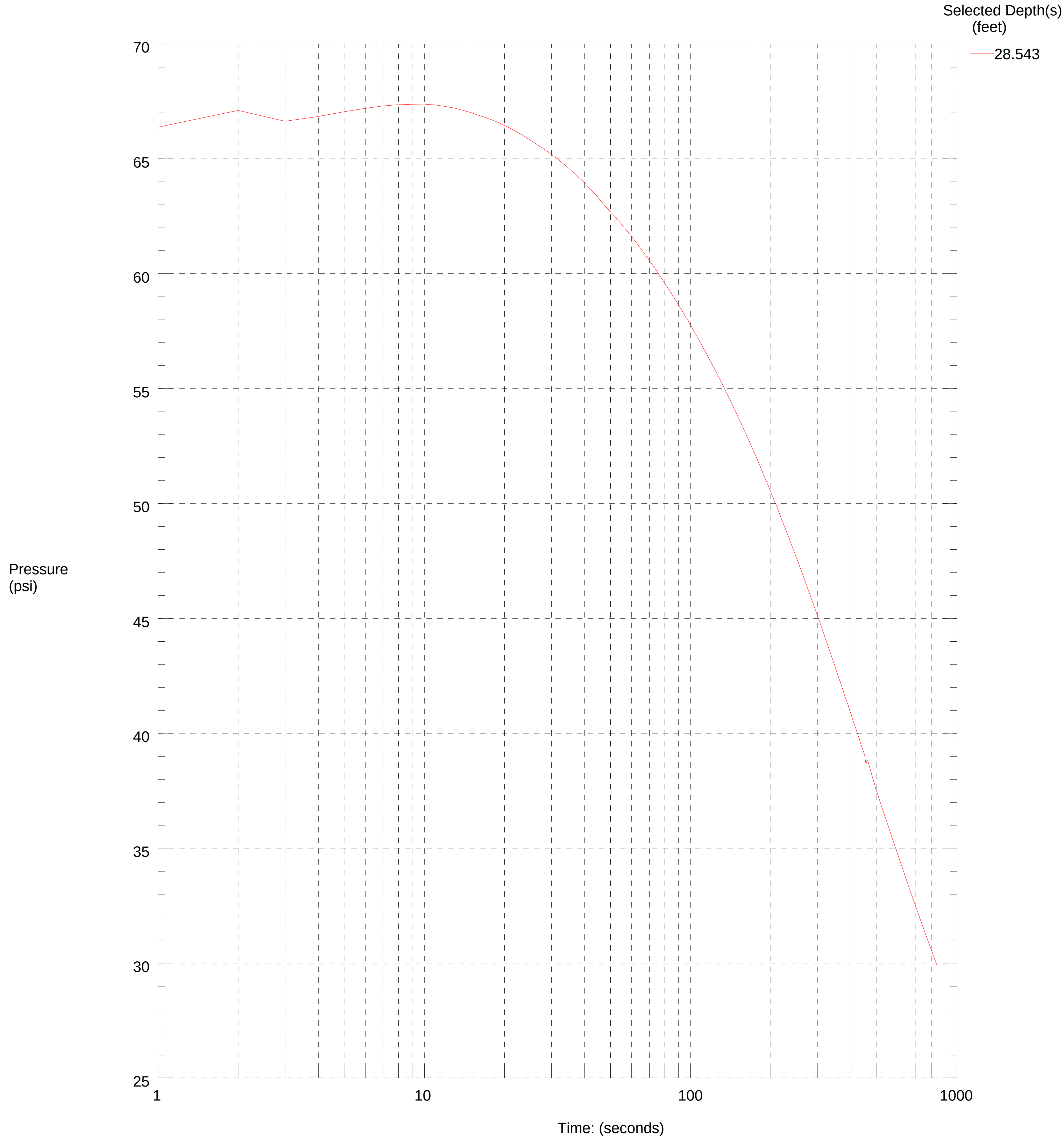
.

FIG. GD5-A-55

Shannon & Wilson

Operator Brown
Sounding: SWC 215
Cone Used: DSG1065

CPT Date/Time: 12/14/2008 12:43:17 PM
Location: Pioneer Crossing STA 657+15 Elev 4492.2
Job Number: 23-1-01178-010



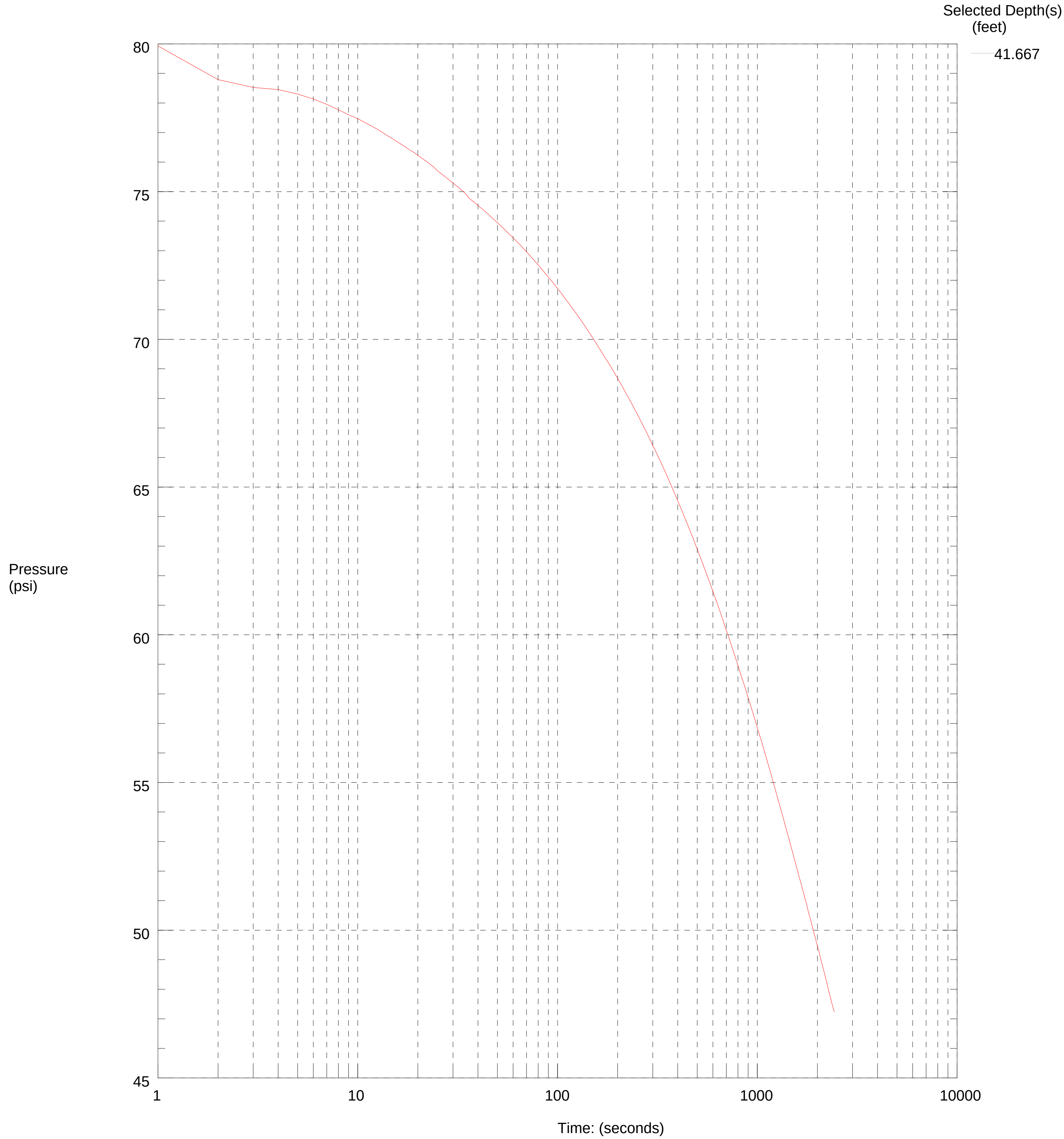
Maximum Pressure = 67.377 psi
.

FIG. GD5-A-56

Shannon & Wilson

Operator Brown
Sounding: SWC 215
Cone Used: DSG1065

CPT Date/Time: 12/14/2008 12:43:17 PM
Location: Pioneer Crossing STA 657+15 Elev 4492.2
Job Number: 23-1-01178-010



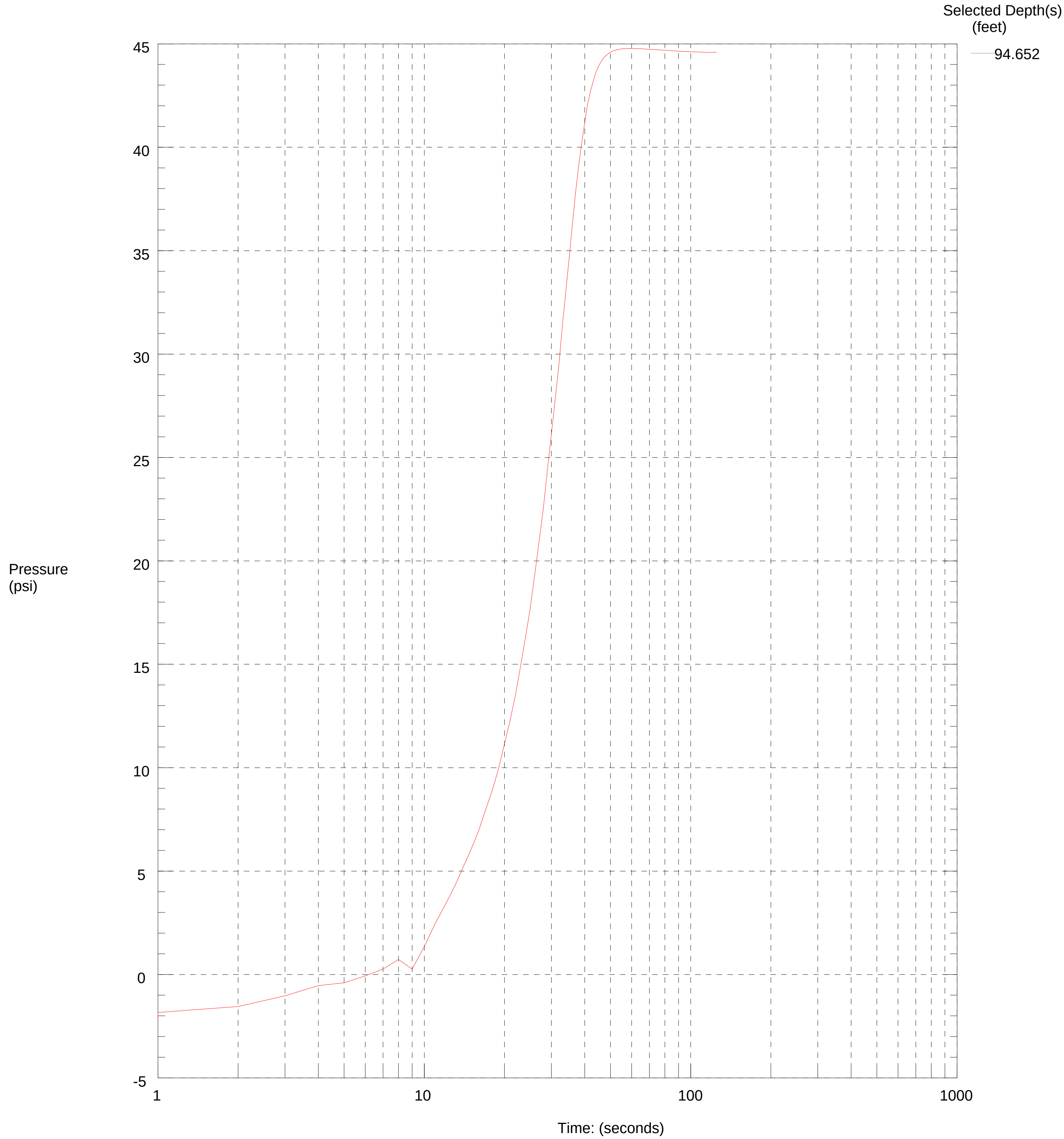
Maximum Pressure = 79.97 psi

FIG. GD5-A-57

Shannon & Wilson

Operator Brown
Sounding: SWC 215
Cone Used: DSG1065

CPT Date/Time: 12/14/2008 12:43:17 PM
Location: Pioneer Crossing STA 657+15 Elev 4492.2
Job Number: 21-1-01178-010



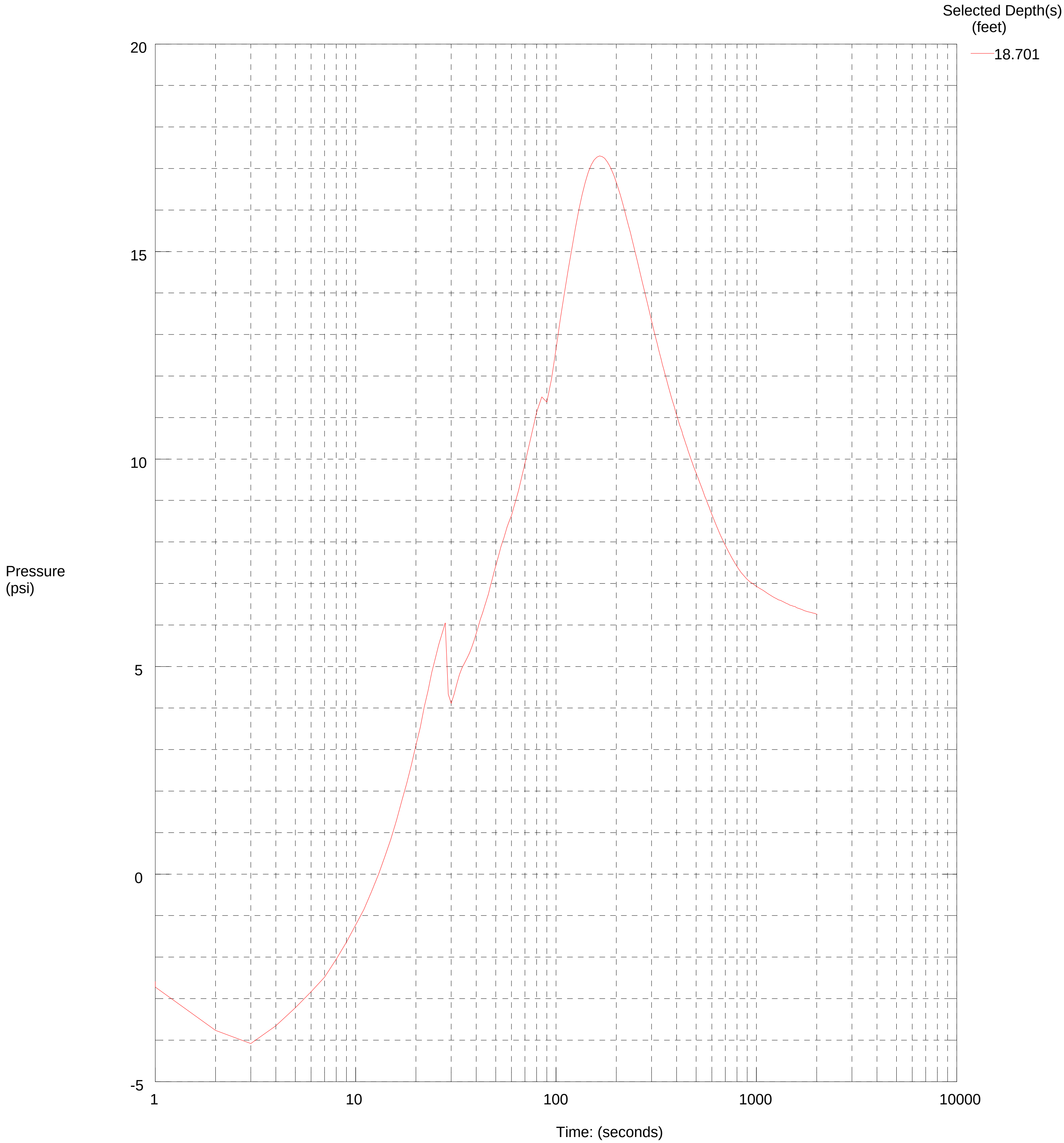
Maximum Pressure = 44.779 psi

FIG. GD5-A-58

Shannon & Wilson

Operator Brown
Sounding: SWC 216
Cone Used: DSG1065

CPT Date/Time: 12/15/2008 9:40:30 AM
Location: Pioneer Crossing STA 672+00 Elev 4498.8
Job Number: 23-1-01178-010



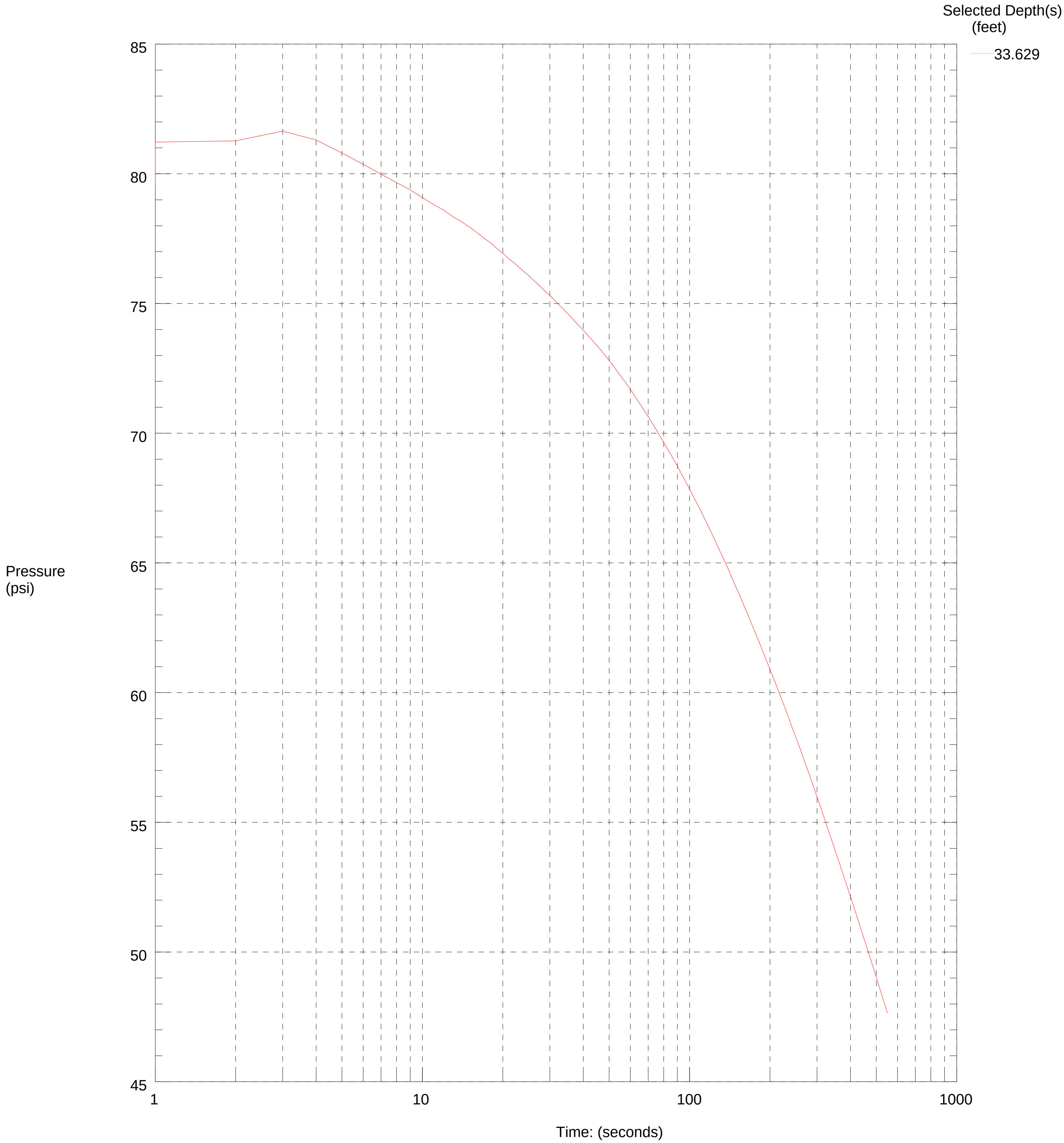
Maximum Pressure = 17.304 psi

FIG. GD5-A-59

Shannon & Wilson

Operator Brown
Sounding: SWC 216
Cone Used: DSG1065

CPT Date/Time: 12/15/2008 9:40:30 AM
Location: Pioneer Crossing STA 672+00 Elev 4498.8
Job Number: 23-1-01178-010



Maximum Pressure = 81.643 psi

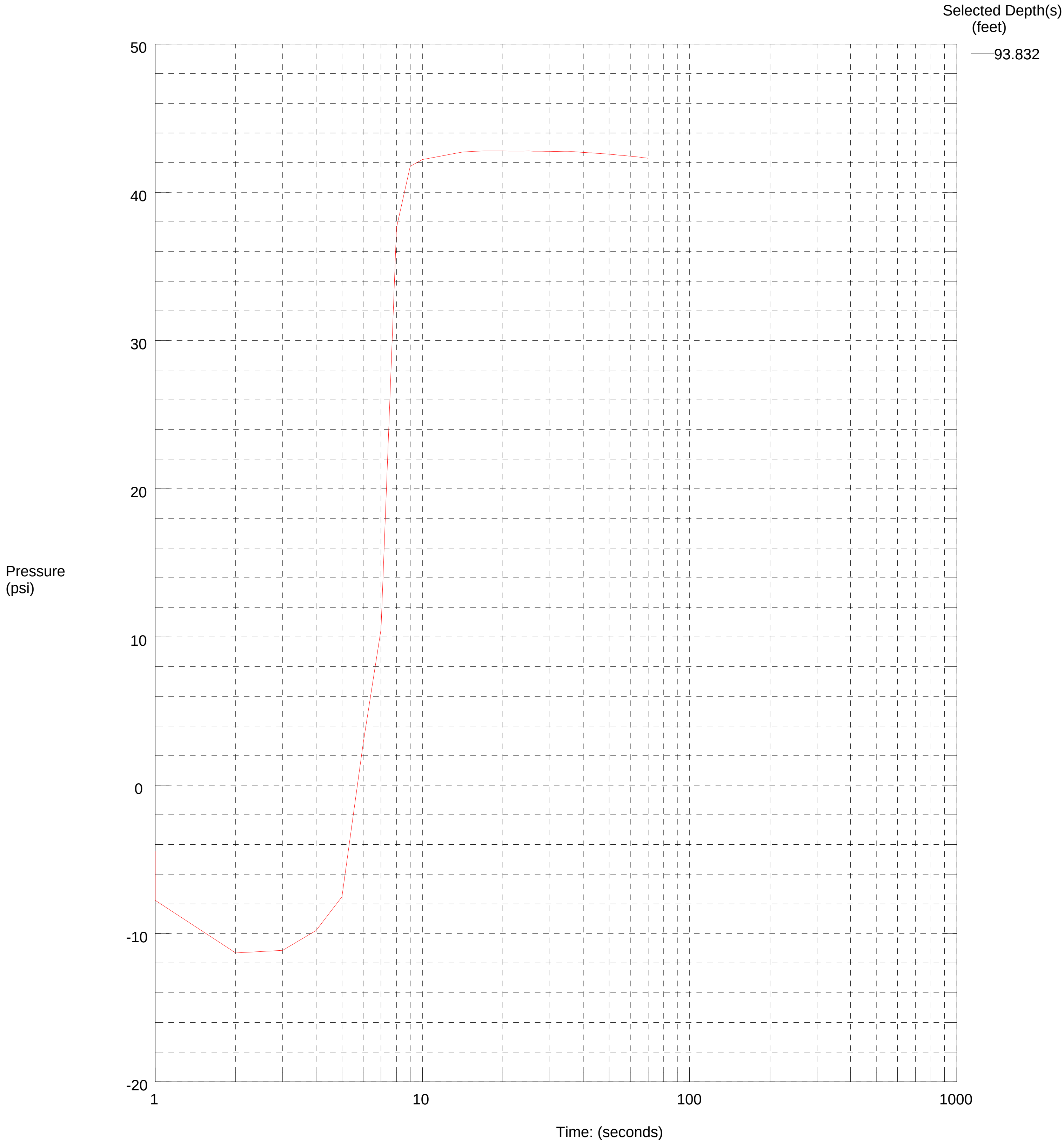
.

FIG. GD5-A-60

Shannon & Wilson

Operator Brown
Sounding: SWC 216
Cone Used: DSG1065

CPT Date/Time: 12/15/2008 9:40:30 AM
Location: Pioneer Crossing STA 672+00 Elev 4498.8
Job Number: 23-1-01178-010



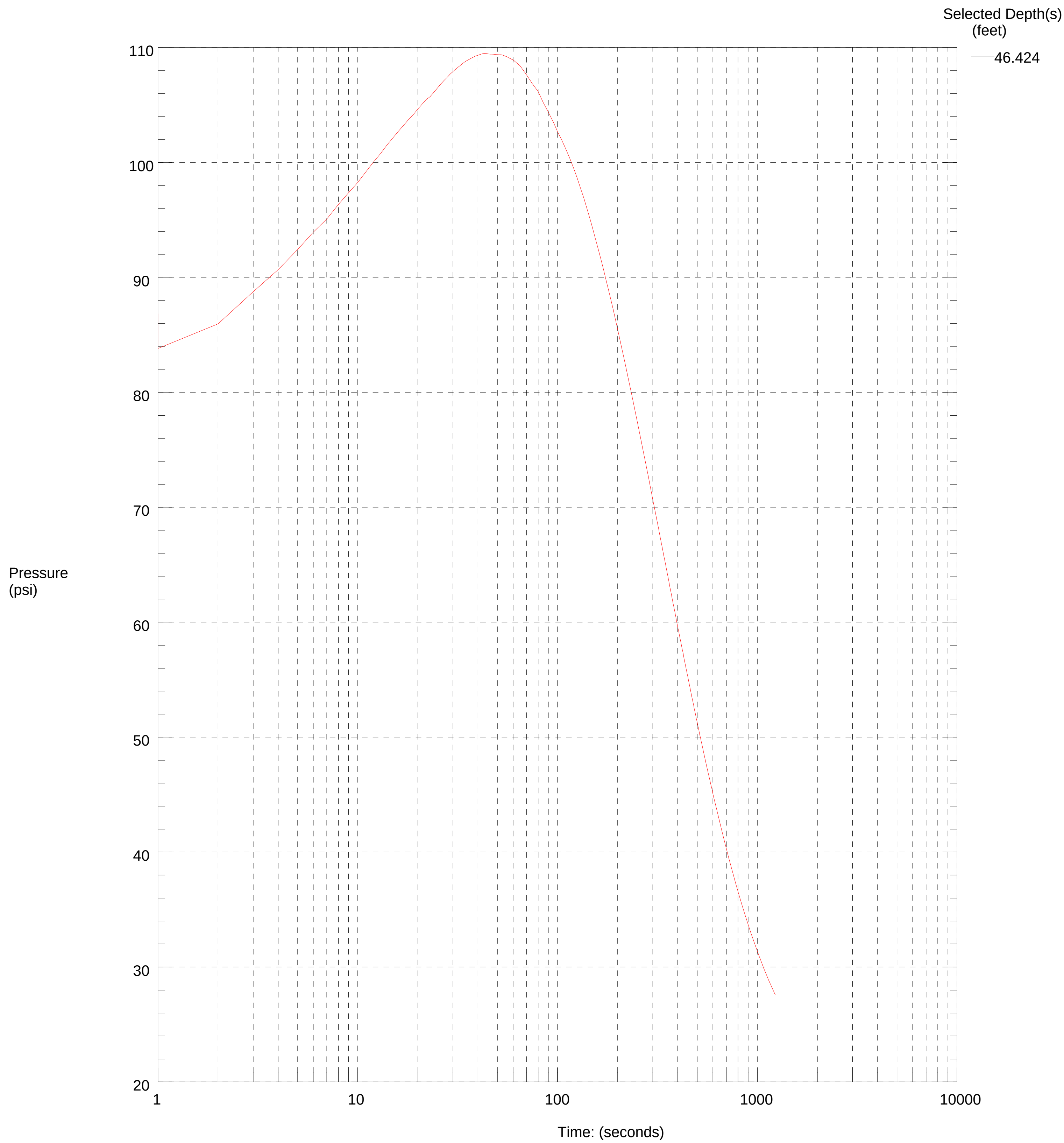
Maximum Pressure = 42.79 psi

FIG. GD5-A-61

Shannon & Wilson

Operator Brown
Sounding: SWC-218
Cone Used: DSG1065

CPT Date/Time: 12/13/2008 2:46:00 PM
Location: Pioneer Crossing STA 630+10
Job Number: 23-1-01178-010



Maximum Pressure = 109.474 psi

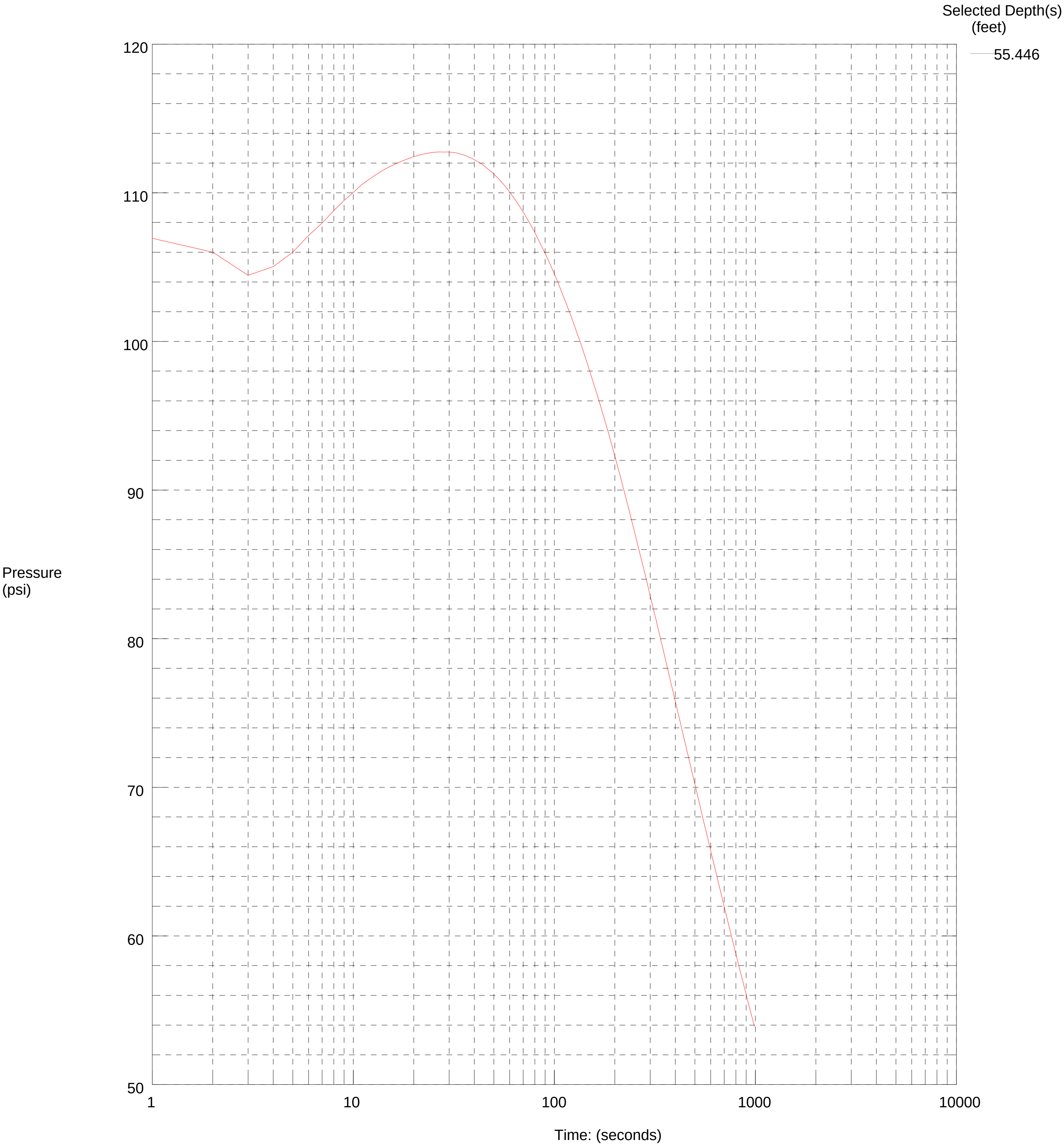
.

FIG. GD5-A-62

Shannon & Wilson

Operator Brown
Sounding: SWC-218
Cone Used: DSG1065

CPT Date/Time: 12/13/2008 2:46:00 PM
Location: Pioneer Crossing STA 630+10
Job Number: 23-1-01178-010



Maximum Pressure = 112.75 psi

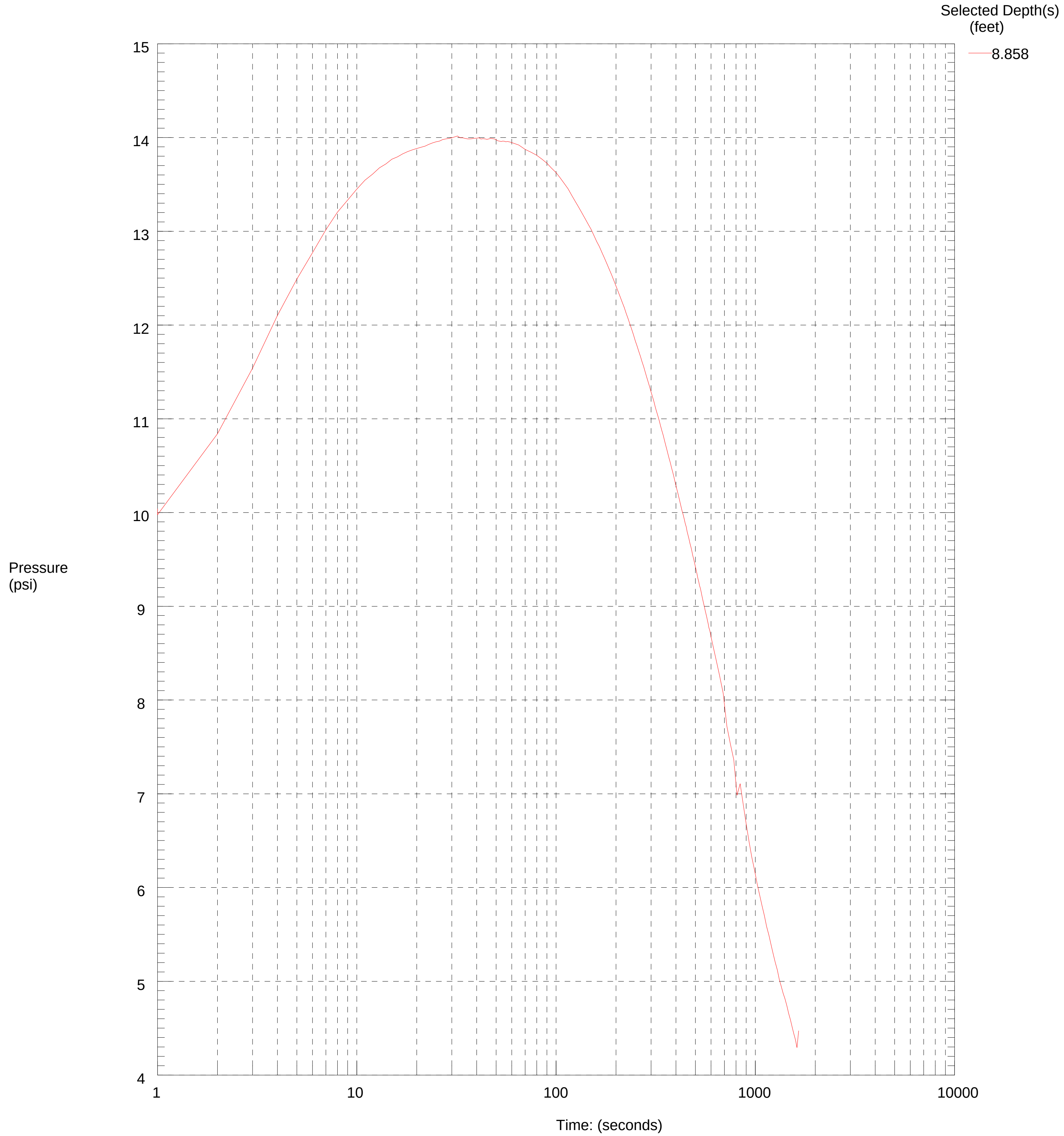
.

FIG. GD5-A-63

Shannon & Wilson

Operator Brown
Sounding: SWC 220
Cone Used: DSG1065

CPT Date/Time: 12/12/2008 1:32:52 PM
Location: Pioneer Crossing 50 ft S, 20 ft E SWC 211
Job Number: 23-1-01178-010



Maximum Pressure = 14.016 psi

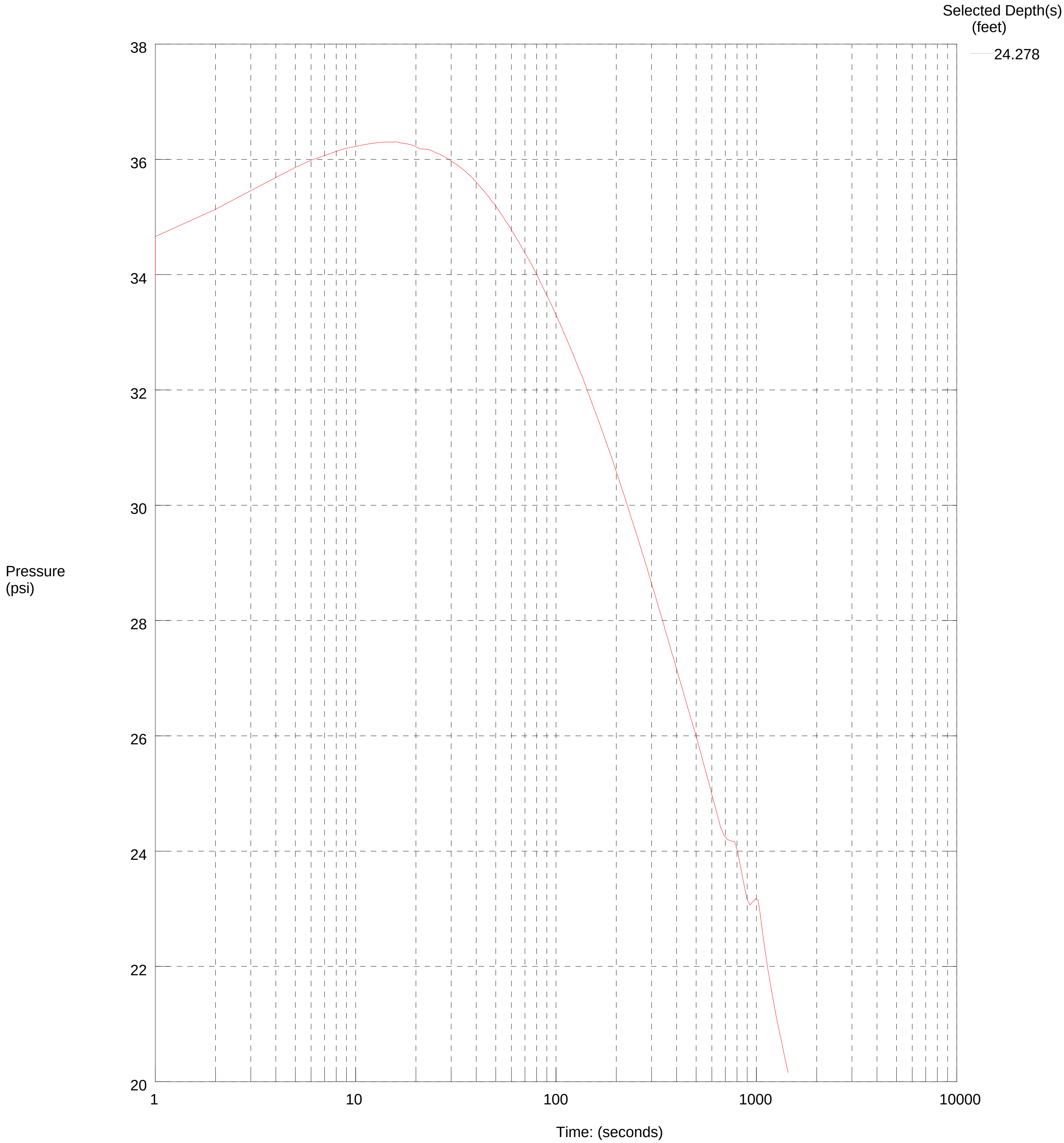
.

FIG. GD5-A-64

Shannon & Wilson

Operator Brown
Sounding: SWC 220
Cone Used: DSG1065

CPT Date/Time: 12/12/2008 1:32:52 PM
Location: Pioneer Crossing 50 ft S, 20 ft E SWC 211
Job Number: 23-1-01178-010



Maximum Pressure = 36.302 psi

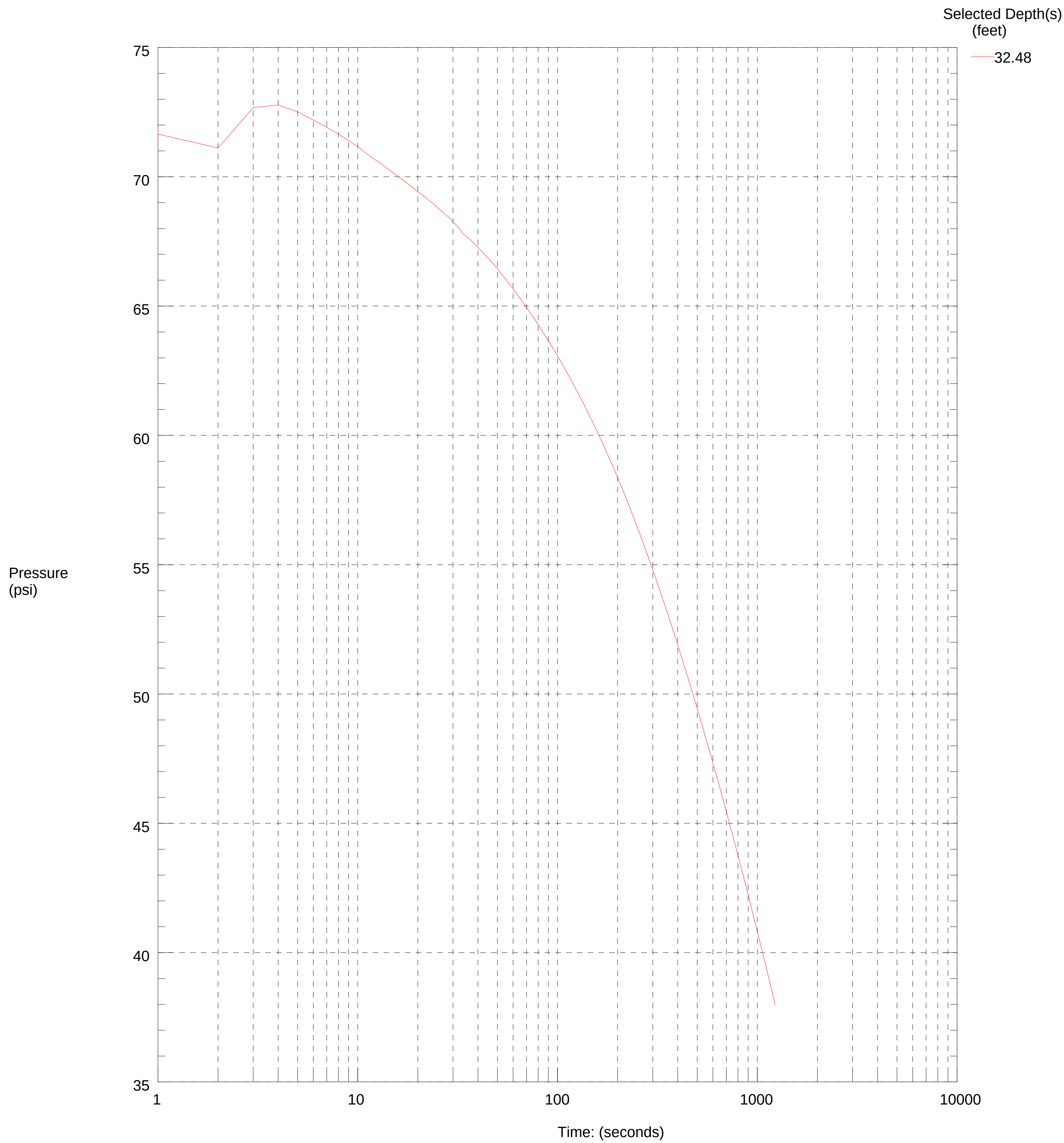
.

FIG. GD5-A-65

Shannon & Wilson

Operator Brown
Sounding: SWC 221 (TCC101)
Cone Used: DSG1065

CPT Date/Time: 12/15/2008 1:02:38 PM
Location: Pioneer Crossing STA 688+20 69R Elev 4505.90
Job Number: 23-1-01178-010



Maximum Pressure = 72.78 psi

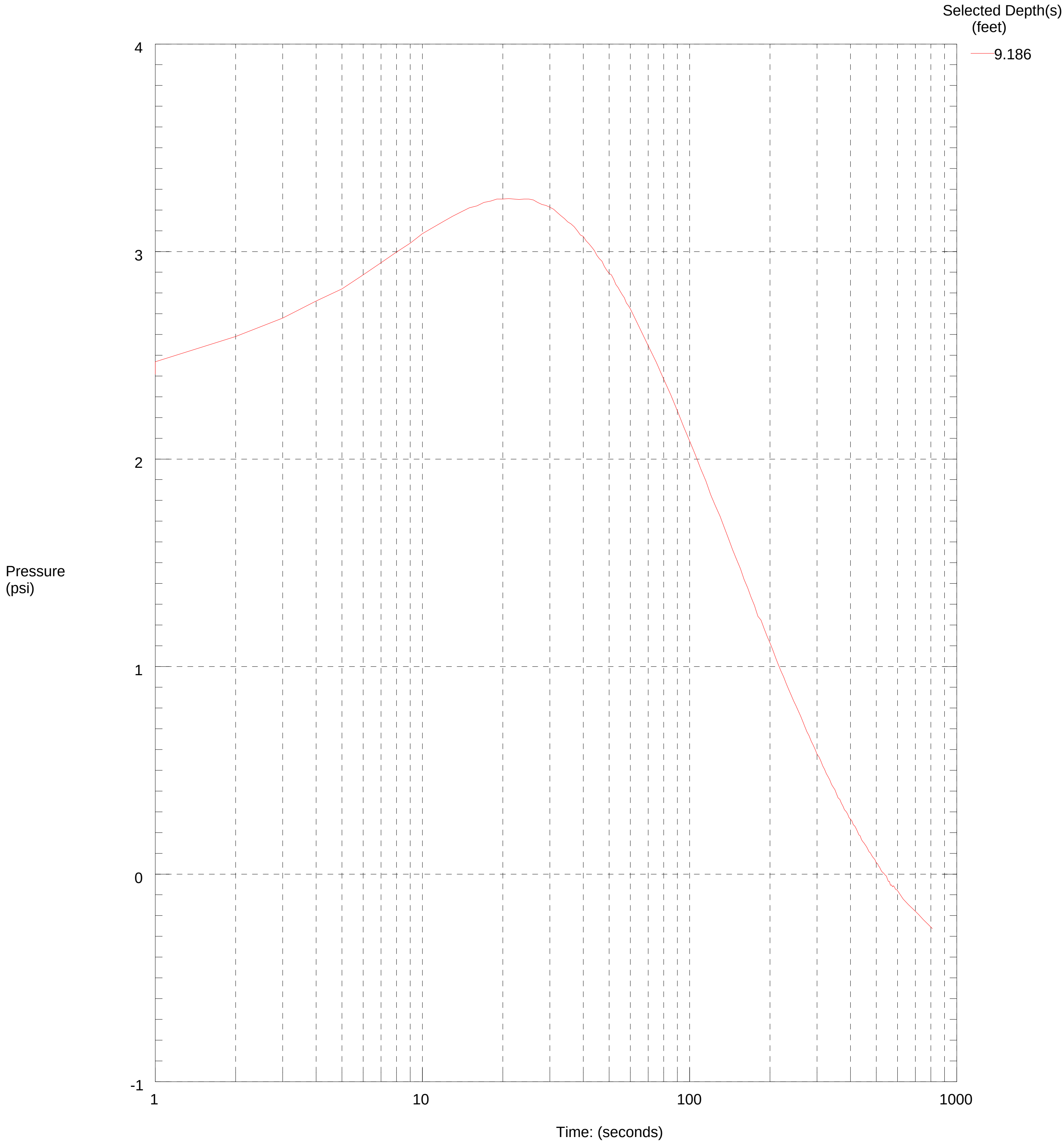
.

FIG. GD5-A-66

Shannon & Wilson

Operator Brown
Sounding: SWC 222 (TCC102)
Cone Used: DSG1065

CPT Date/Time: 12/15/2008 3:34:19 PM
Location: Pioneer Crossing STA 694+00 Elev 4507.3
Job Number: 23-1-01178-010



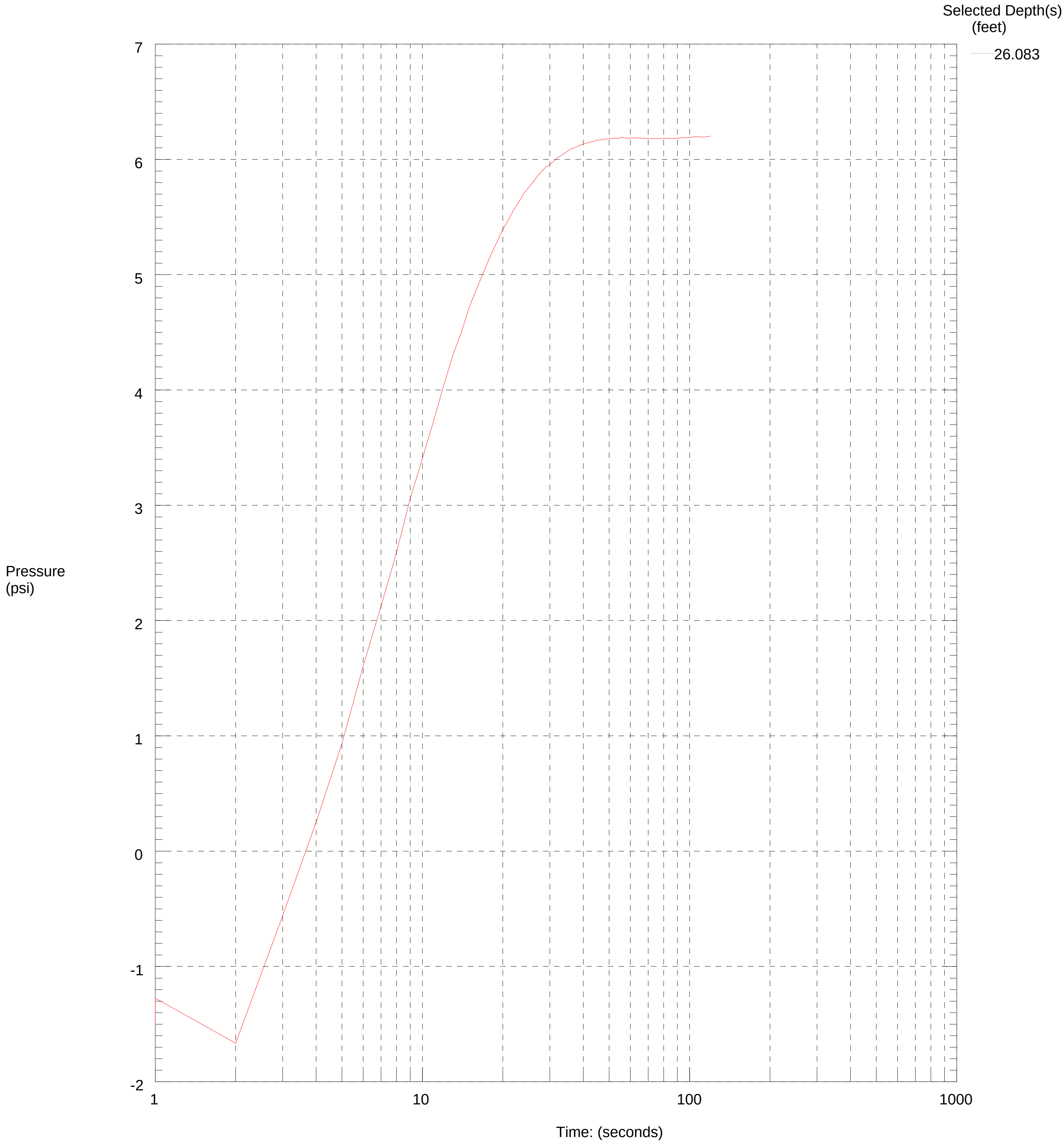
Maximum Pressure = 3.255 psi

FIG. GD5-A-67

Shannon & Wilson

Operator Brown
Sounding: SWC 222 (TCC102)
Cone Used: DSG1065

CPT Date/Time: 12/15/2008 3:34:19 PM
Location: Pioneer Crossing STA 694+00 Elev 4507.3
Job Number: 23-1-01178-010



Maximum Pressure = 6.202 psi

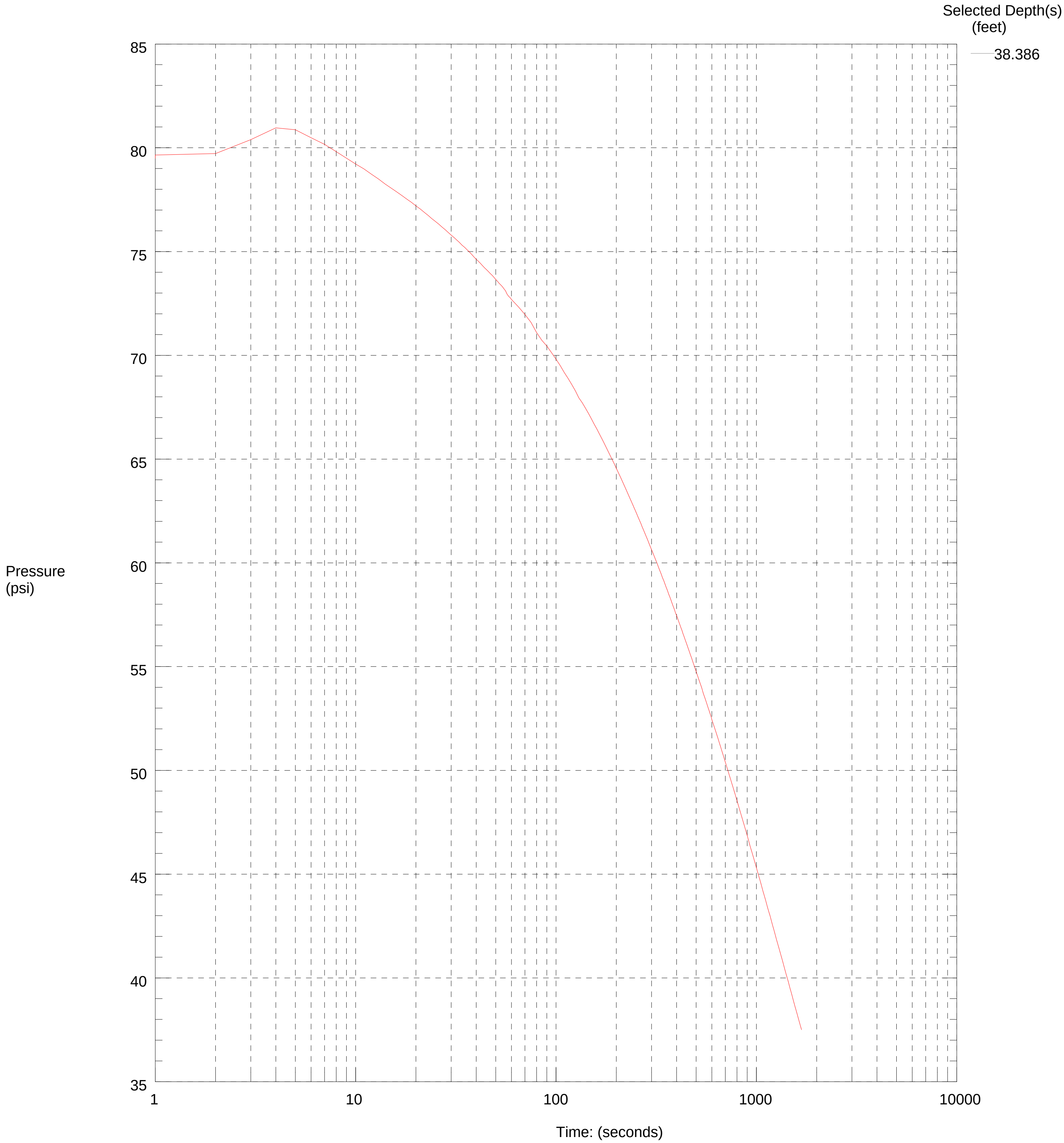
.

FIG. GD5-A-68

Shannon & Wilson

Operator Brown
Sounding: SWC 222 (TCC102)
Cone Used: DSG1065

CPT Date/Time: 12/15/2008 3:34:19 PM
Location: Pioneer Crossing STA 694+00 Elev 4507.3
Job Number: 23-1-01178-010



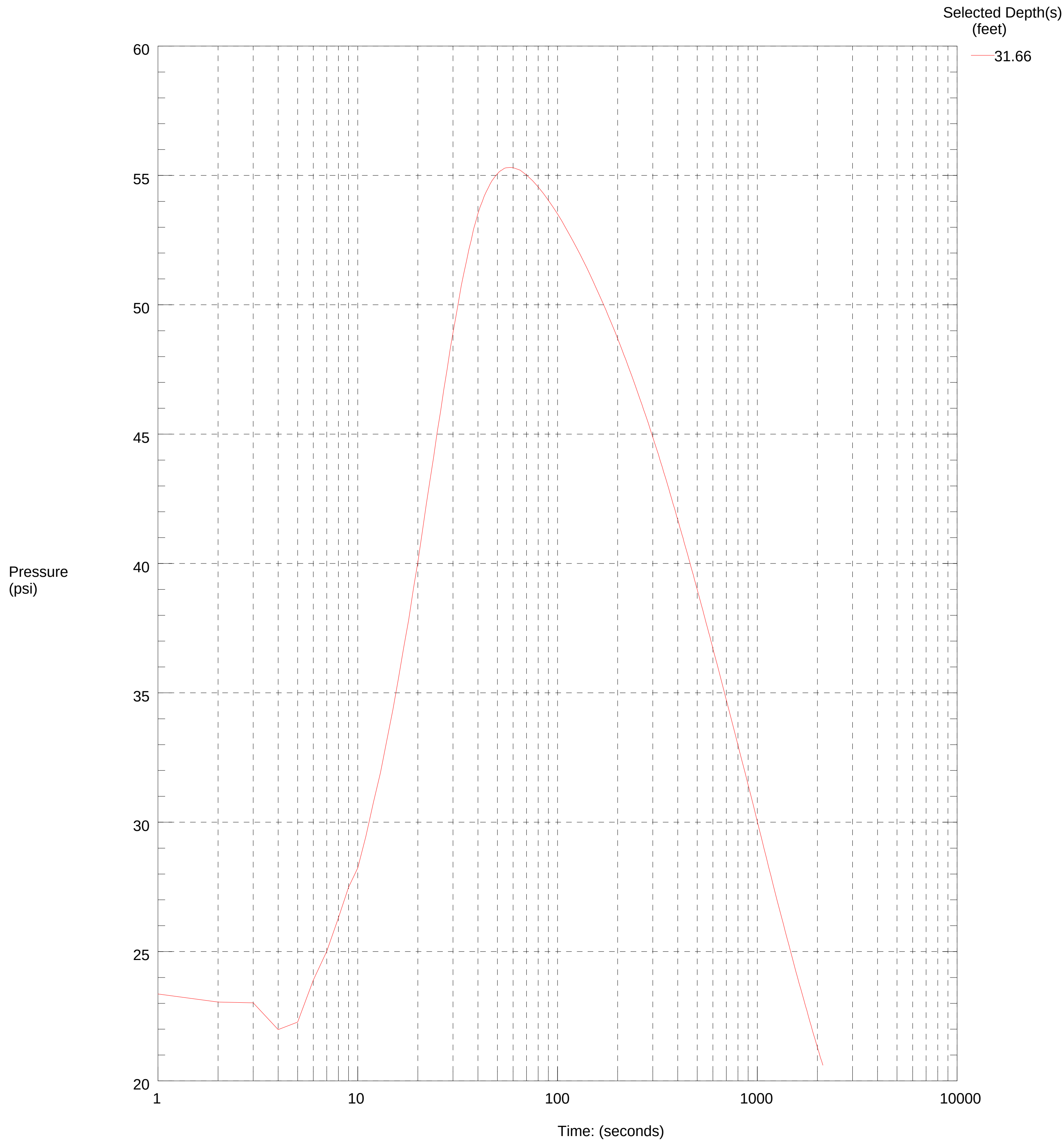
Maximum Pressure = 80.96 psi
.

FIG. GD5-A-69

Shannon & Wilson

Operator Brown
Sounding: SWC 223 (TCC103)
Cone Used: DSG1065

CPT Date/Time: 12/16/2008 9:48:38 AM
Location: Pioneer Crossing STA 709+50 estimated Elev unknown
Job Number: 23-1-01178-010



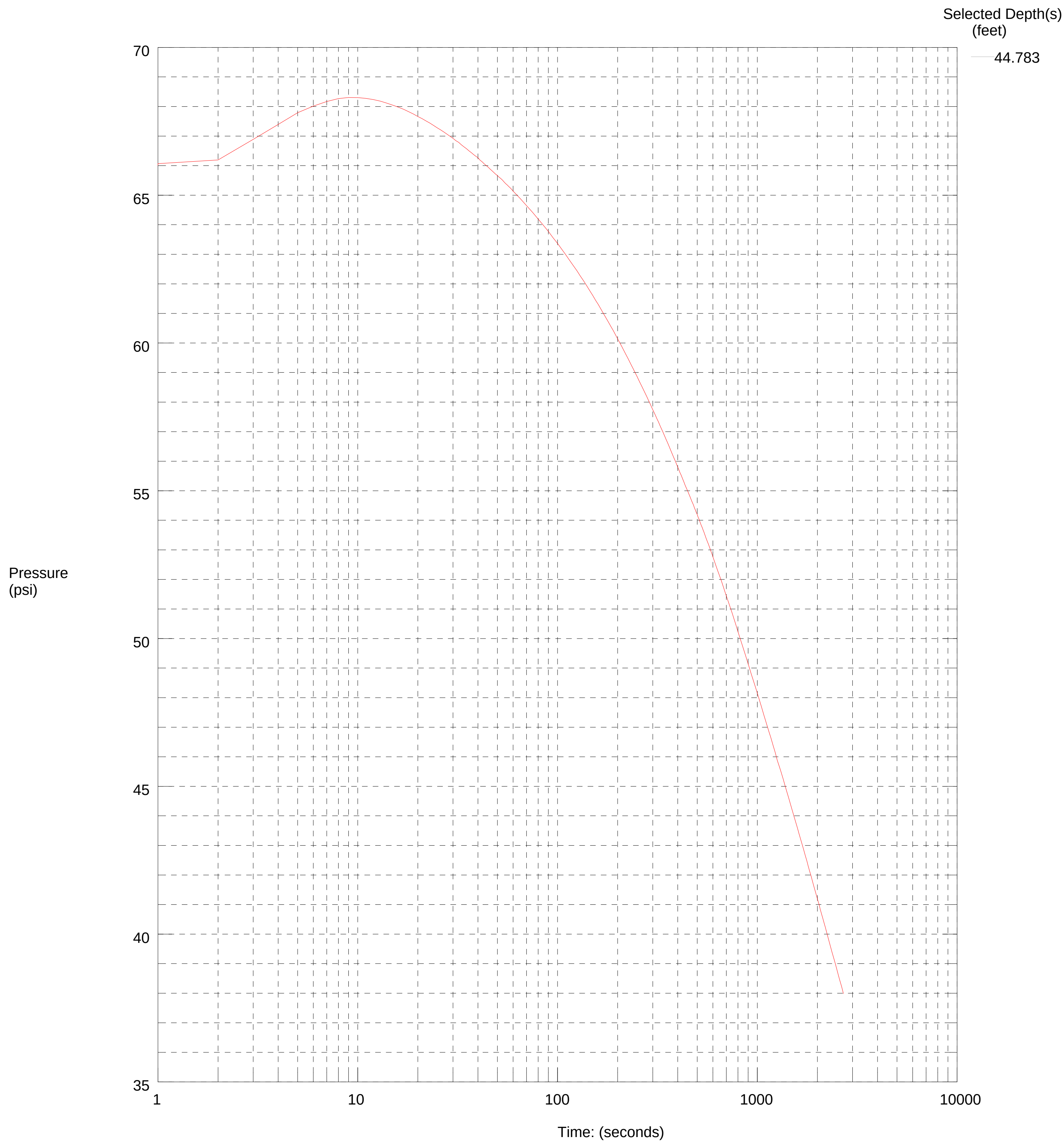
Maximum Pressure = 55.306 psi

FIG. GD5-A-70

Shannon & Wilson

Operator Brown
Sounding: SWC 223 (TCC103)
Cone Used: DSG1065

CPT Date/Time: 12/16/2008 9:48:38 AM
Location: Pioneer Crossing STA 709+50 estimated Elev unknown
Job Number: 23-1-01178-010



Maximum Pressure = 68.306 psi

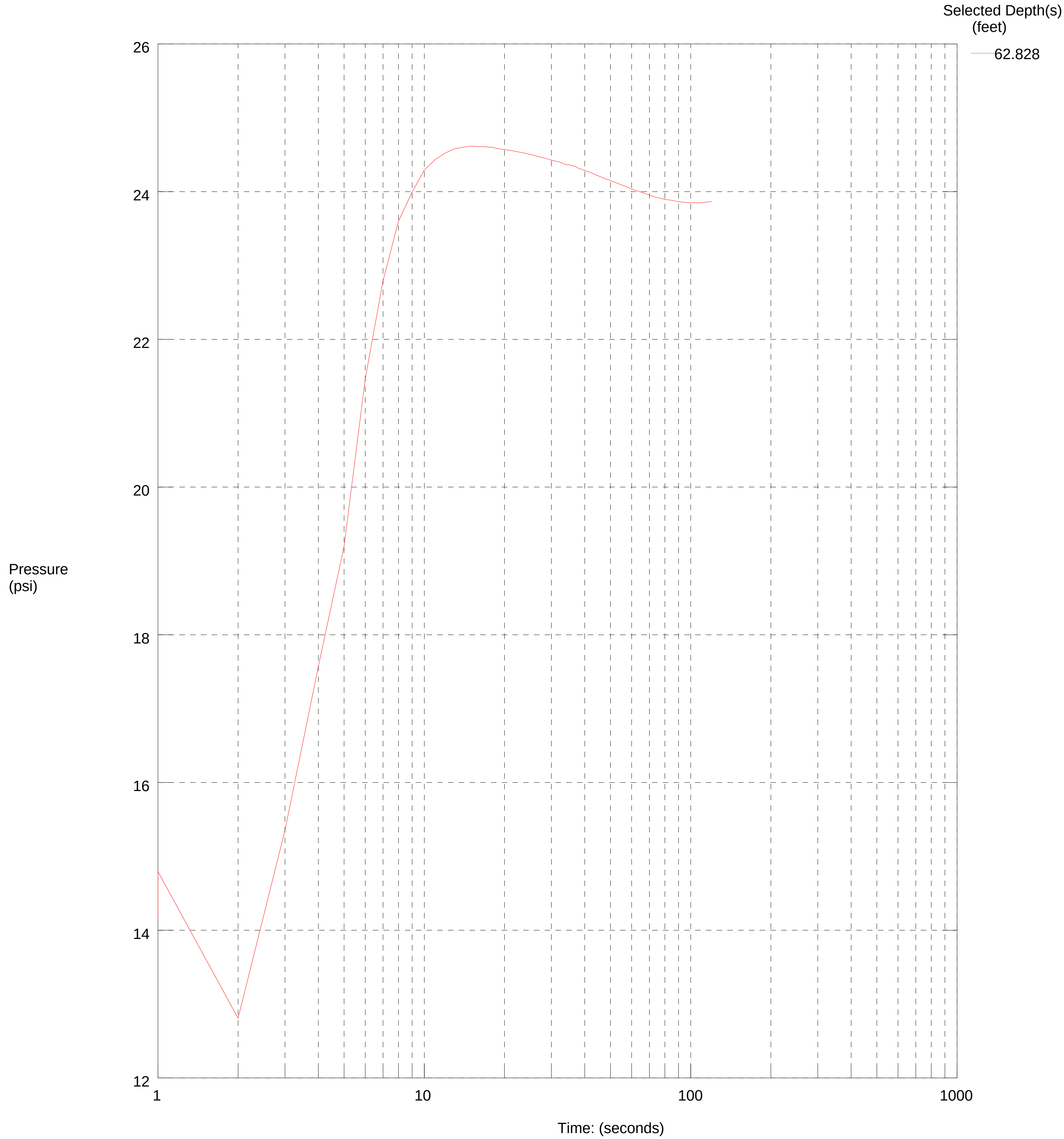
.

FIG. GD5-A-71

Shannon & Wilson

Operator Brown
Sounding: SWC 223 (TCC103)
Cone Used: DSG1065

CPT Date/Time: 12/16/2008 9:48:38 AM
Location: Pioneer Crossing STA 709+50 estimated Elev unknown
Job Number: 23-1-01178-010



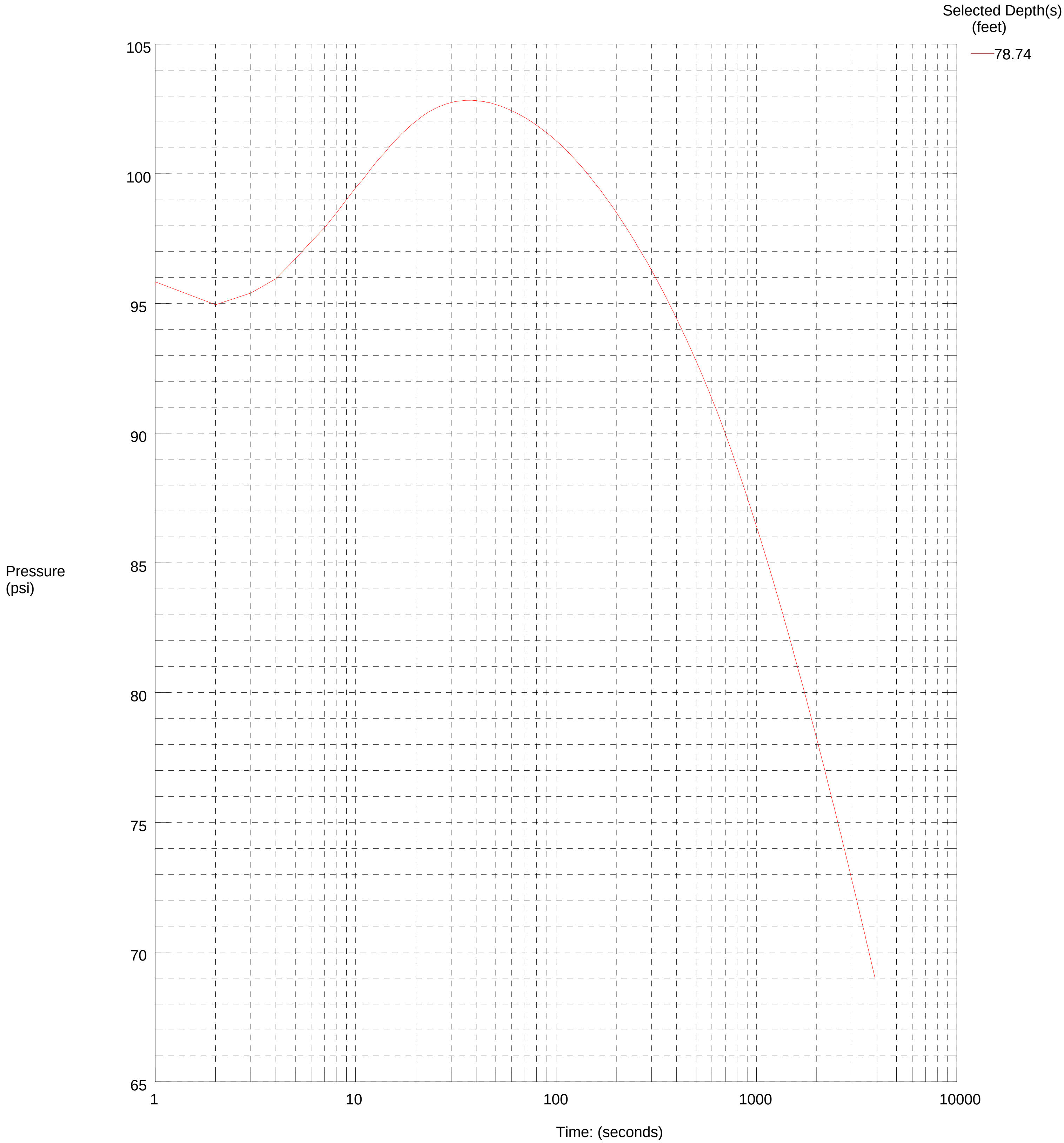
Maximum Pressure = 24.615 psi
.

FIG. GD5-A-72

Shannon & Wilson

Operator Brown
Sounding: SWC 223 (TCC103)
Cone Used: DSG1065

CPT Date/Time: 12/16/2008 9:48:38 AM
Location: Pioneer Crossing STA 709+50 estimated Elev unknown
Job Number: 23-1-01178-010



Maximum Pressure = 102.832 psi

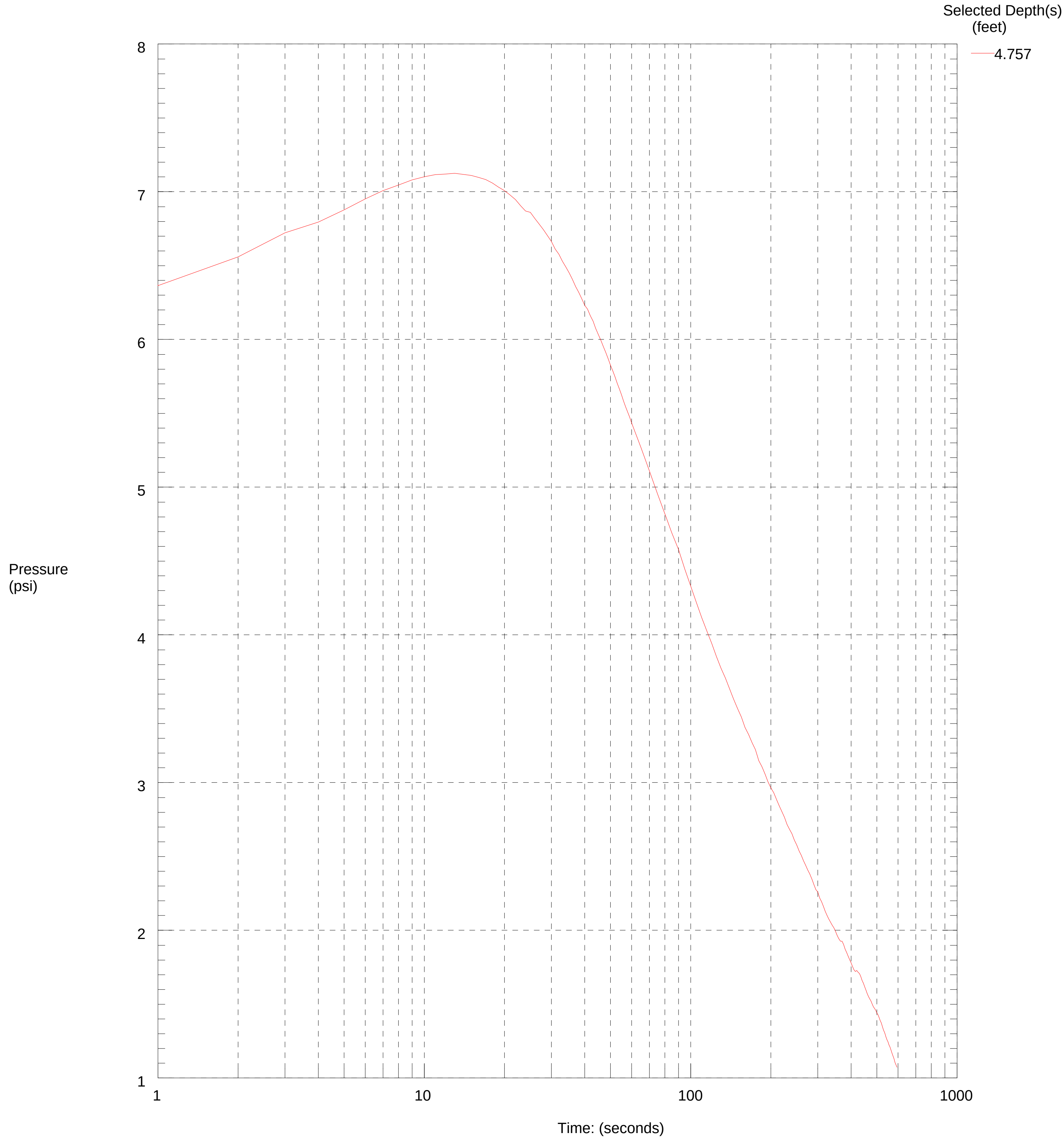
.

FIG. GD5-A-73

Shannon & Wilson

Operator Brown
Sounding: SWC 224 (TCC104)
Cone Used: DSG1065

CPT Date/Time: 12/17/2008 10:10:48 AM
Location: Pioneer Crossing STA 721+00 Elev 4515.1
Job Number: 23-1-01178-010



Maximum Pressure = 7.124 psi

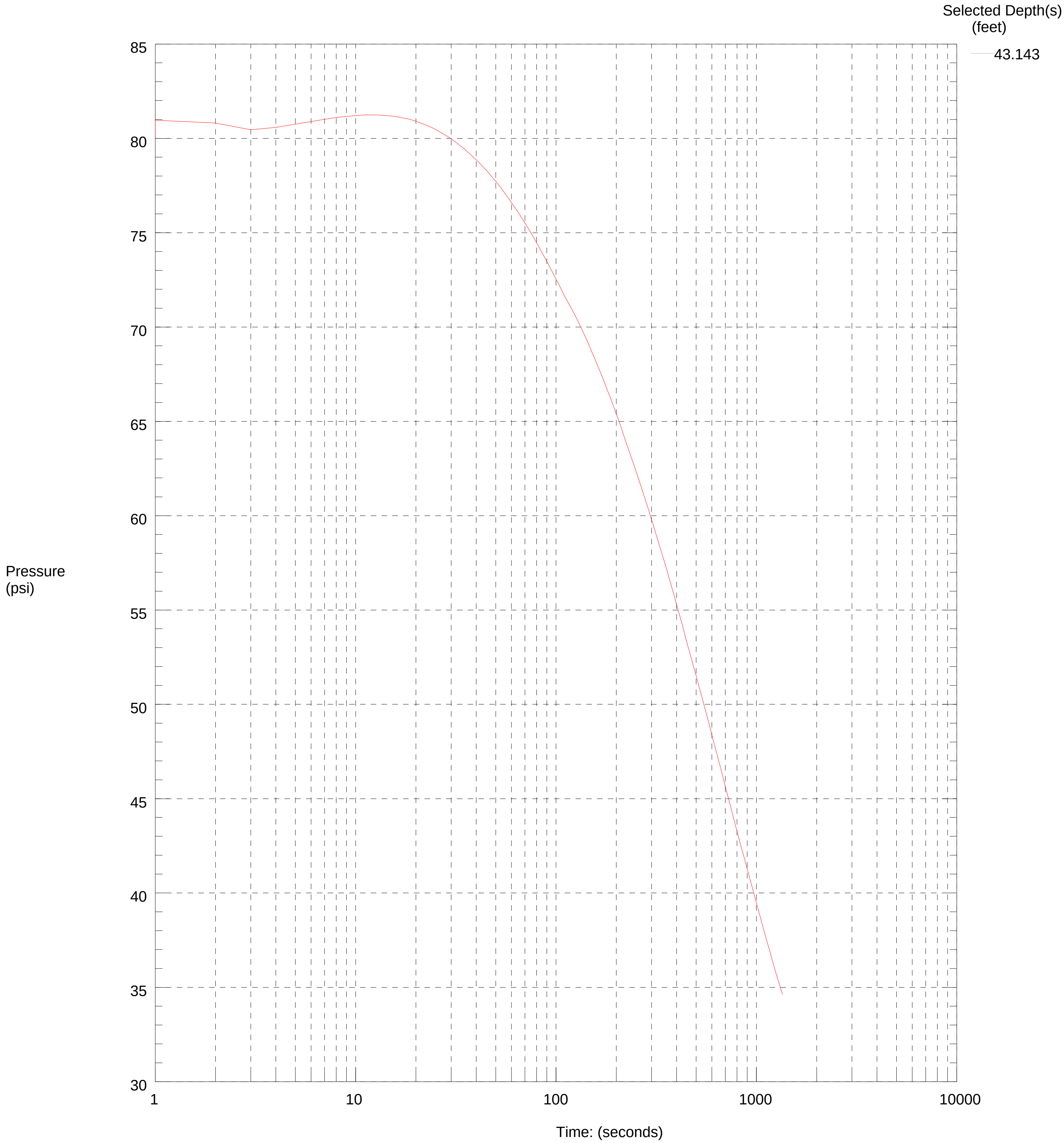
.

FIG. GD5-A-74

Shannon & Wilson

Operator Brown
Sounding: SWC 224 (TCC104)
Cone Used: DSG1065

CPT Date/Time: 12/17/2008 10:10:48 AM
Location: Pioneer Crossing STA 721+00 Elev 4515.1
Job Number: 23-1-01178-010



Maximum Pressure = 81.243 psi

.

FIG. GD5-A-75

APPENDIX GD5-B

GEOTECHNICAL LABORATORY TESTING PROCEDURES AND RESULTS

APPENDIX GD5-B

GEOTECHNICAL LABORATORY TESTING PROCEDURES AND RESULTS

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APPENDIX GD5-B

GEOTECHNICAL LABORATORY TESTING PROCEDURES AND RESULTS

GD5-B.1 INTRODUCTION

Disturbed samples collected between December 11, 2008, and January 18, 2009, from borings SWB-202, SWB-203, and SWB-204 and test pits SWT-206, SWT-207, SWT-208, SWT-209, SWT-210, SWT-217, SWT-219, SWT-225, SWT-227, SWT-228, SWT-229, SWT-230, SWT-231, SWT-232, SWT-233, and SWT-234 were sealed in jars and returned to our laboratory in Seattle, Washington, for testing. Relatively undisturbed samples obtained from SWB-201, SWB-202, SWB-203, and SWB-204 were also returned to our Seattle lab for testing. The samples were tested in December 2008 and January 2009 to measure the basic index properties and the engineering characteristics of the subsurface soils at the site. Tests were conducted in general accordance with applicable ASTM International (ASTM) standards. The following sections describe laboratory tests performed by Shannon & Wilson.

The figures presenting laboratory test analyses are grouped by firm as shown in the Appendix GD5-B Table of Contents. We obtained additional geotechnical data from the Baseline Geotechnical Field Exploration Report provided by Terracon Consultants, Inc. (Terracon, 2008) during the Request for Proposal (RFP) phase. Shannon & Wilson, Inc. cannot be held responsible for laboratory test analyses performed by others for the RFP baseline reports.

GD5-B.2 VISUAL CLASSIFICATION

Each of the soil samples recovered from the borings was visually reclassified in our laboratory using a system based on the ASTM Designation: D 2487, Standard Test Method for Classification of Soil for Engineering Purposes, or ASTM Designation: D 2488, Standard Recommended Practice for Description of Soils (Visual-Manual Procedure). These ASTM standards use the Unified Soil Classification System (USCS) described in Figure GD5-A-1. The individual samples classifications have been incorporated into our boring logs presented in Figures GD5-A-2 through GD5-A-5.

GD5-B.3 INDEX TESTS

GD5-B.3.1 Water Content Determination

The natural water content of all soil samples recovered from the field explorations was determined in general accordance with ASTM Designation: D 2216, Standard Method of Laboratory Determination of Water (Moisture) Content of Soil, Rock, and Soil-Aggregate Mixtures. Comparison of water content of a soil with its index properties can be useful in characterizing soil unit weight, consistency, compressibility, and strength. Water content is plotted in the boring logs presented in Appendix GD5-A.

GD5-B.3.2 Grain Size Distribution Analyses

Grain size distribution analyses were performed on 11 samples in general accordance with ASTM Designation: D 422, Standard Method for Particle-Size Analysis of Soils and D 1140, Standard Test Methods for Amount of Material in Soil is Finer than No. 200 Sieve. The general procedures to measure the grain size distribution of a soil sample include sieve analysis, hydrometer analysis, and combined analysis.

Grain size distributions are used to assist in classifying soils and to provide correlation with soil properties, including permeability, liquefaction potential, behavior when excavated, capillary action, and sensitivity to moisture. Results of the grain size analyses are plotted on grain size distribution curves presented in Figures GD5-B-1 and GD5-B-2. Along with each grain size distribution is a tabulated summary containing the group symbol according to the USCS, the sample description, percentage of fines passing the No. 200 sieve, and the natural water content.

GD5-B.3.3 Atterberg Limit Test

Atterberg Limit tests were performed on 13 samples of fine-grained soil in general accordance with ASTM Designation: D 4318, Standard Test Method for Liquid Limit, Plastic Limit, and Plasticity Index of Soils. The Atterberg Limits include Liquid Limit (LL), Plastic Limit (PL), and Plasticity Index (PI=LL-PL). They are generally used to assist in classification of soil, to indicate soil consistency (when compared with natural water content), and to provide correlation to soil properties including compressibility and strength.

The results of the Atterberg Limits determinations are shown in the appropriate boring logs in Appendix GD5-A, and in the plasticity charts presented in Figures GD5-B-3 and GD5-B-4.

GD5-B.4 CONSOLIDATION TESTS

Consolidation tests were performed on relatively undisturbed samples of silt or clay soil to provide data for estimating settlement. Consolidation tests were performed in general accordance with ASTM D 2435, Test Method for One-Dimensional Consolidation Properties of Soils. The results of these tests are included as Figures GD5-B-5 through GD5-B-9.

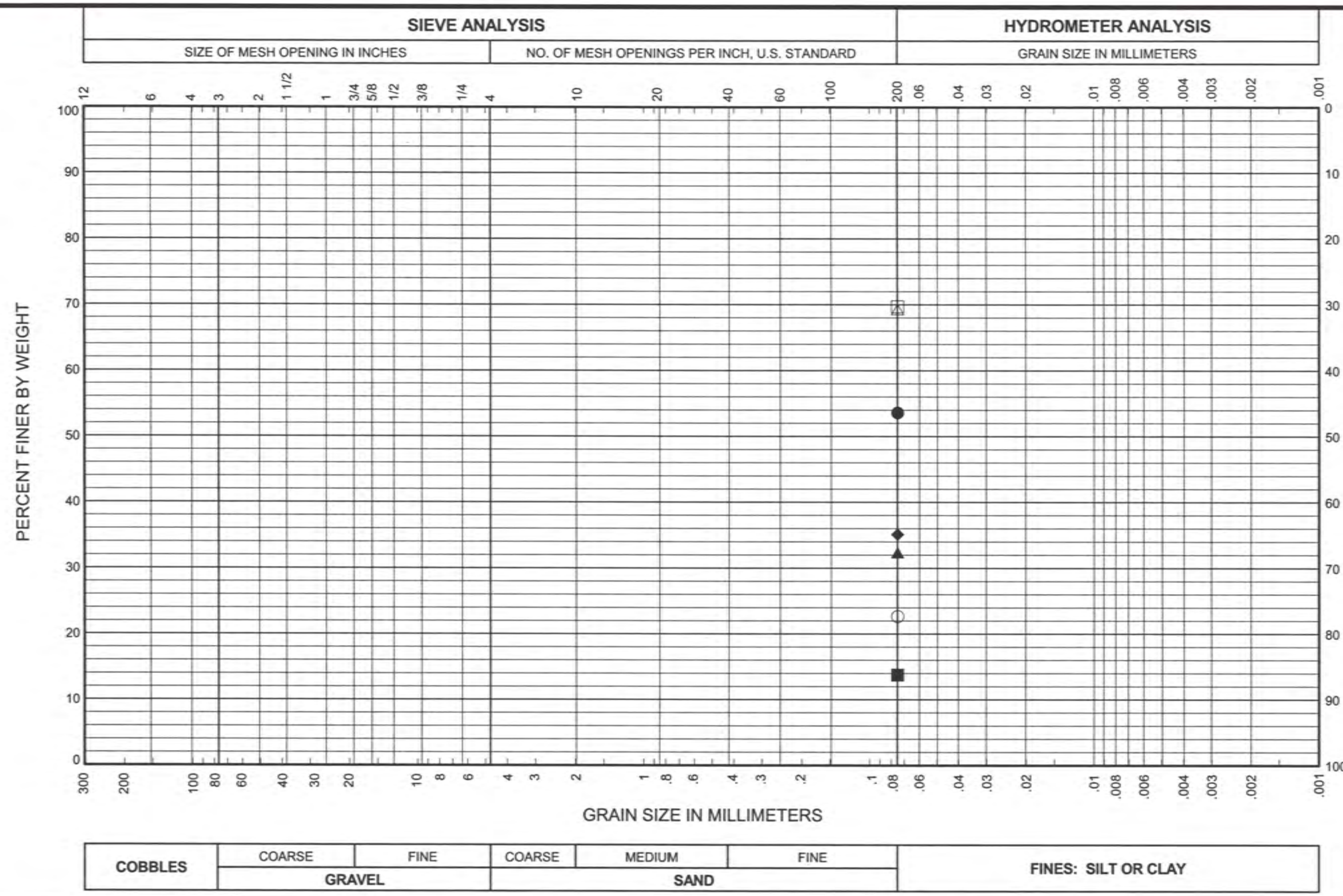
GD5-B.5 TRIAXIAL COMPRESSION TESTS

Consolidated-undrained triaxial compression tests were performed by Terracon on samples selected from boring B-02, B-03, B-05, B-08, and B-09 in general accordance with ASTM D 4767, Test Method for Consolidated-undrained Triaxial Compression Test for Cohesive Soils. Test results are presented in Figures GD5-B-10 through GD5-B-14.

GD5-B.6 REFERENCES

ASTM International (ASTM), 2007, 2007 Annual book of standards, Construction, v. 04.08, Soil and rock (I): D 420 – D 5611: West Conshohocken, Pa.

Terracon Consultants, Inc. (Terracon), 2008, Baseline geotechnical field exploration report, revision A, I-15 widening, 500 North to I-215, UDOT project no. BRF-15-7(213)320, north Salt Lake City & Salt Lake City, Utah: Report prepared by Terracon Consultants, Inc., Draper, Utah, project no. 61075070A, for Michael Baker Jr., Inc., Midvale, Utah, March.



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	LL %	PL %	PI %	NAT. W.C. %	PASS. #200,%	TEST BY	CKD BY	ASTM STND
● SWB-201, S-9	25.0	ML	Light brown, sandy, gravelly SILT				22.2	53.6	JJM	JFL	D1140
■ SWB-201, S-11	35.0	GM	Brown, silty, sandy GRAVEL				15.3	13.8	JJM	JFL	D1140
▲ SWB-202, S-2	5.0	SM	Gray, silty SAND; trace of organics				39.8	32.4	AKV	JFL	D1140
◆ SWB-202, S-3	7.5	SM	Gray, silty SAND				39.5	35.1	JJM	JFL	D1140
○ SWB-202, S-4	10.0	SM	Gray, silty fine SAND; trace of shell fragments				39.2	22.7	AKV	JFL	D1140
□ SWB-202, S-7	17.5	ML	Brown, slightly clayey, sandy SILT				26.1	69.7	JJM	JFL	D1140
△ SWB-202, S-14	35.0	CL-ML	Brown, sandy, clayey SILT	28	22	6	28.5	69.3	JJM	JFL	D1140

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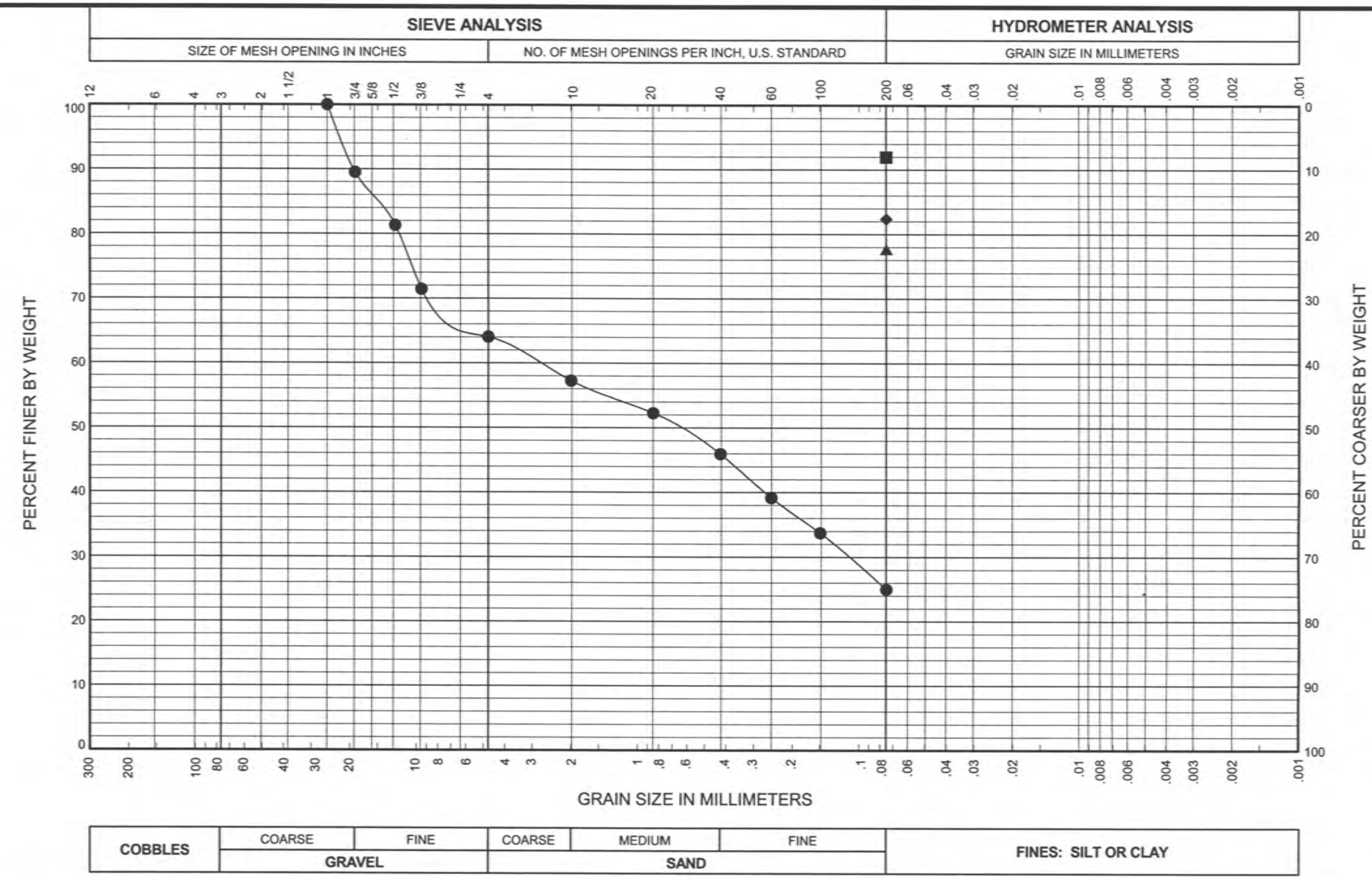
FIG. GD5-B-1



BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	LL %	PL %	PI %	NAT. W.C. %	PASS. #200,%	TEST BY	CKD BY	ASTM STND
● SWB-202, S-20	55.3	SM	Light brown, silty, gravelly SAND				16.9	25.0	JJM	JFL	D422
■ SWB-204, S-3	7.5	CL	Brown, slightly sandy, silty CLAY				31.1	92.0	AKV	JFL	D1140
▲ SWB-204, S-11	36.5	ML	Gray-brown, fine sandy SILT				29.9	77.7	AKV	JFL	D1140
◆ SWT-217	3.7	CH	Light brown, sandy, silty CLAY; trace of shell fragments; scattered fine organics				41.2	82.3	AKV	JFL	D1140

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FIG. GD5-B-2

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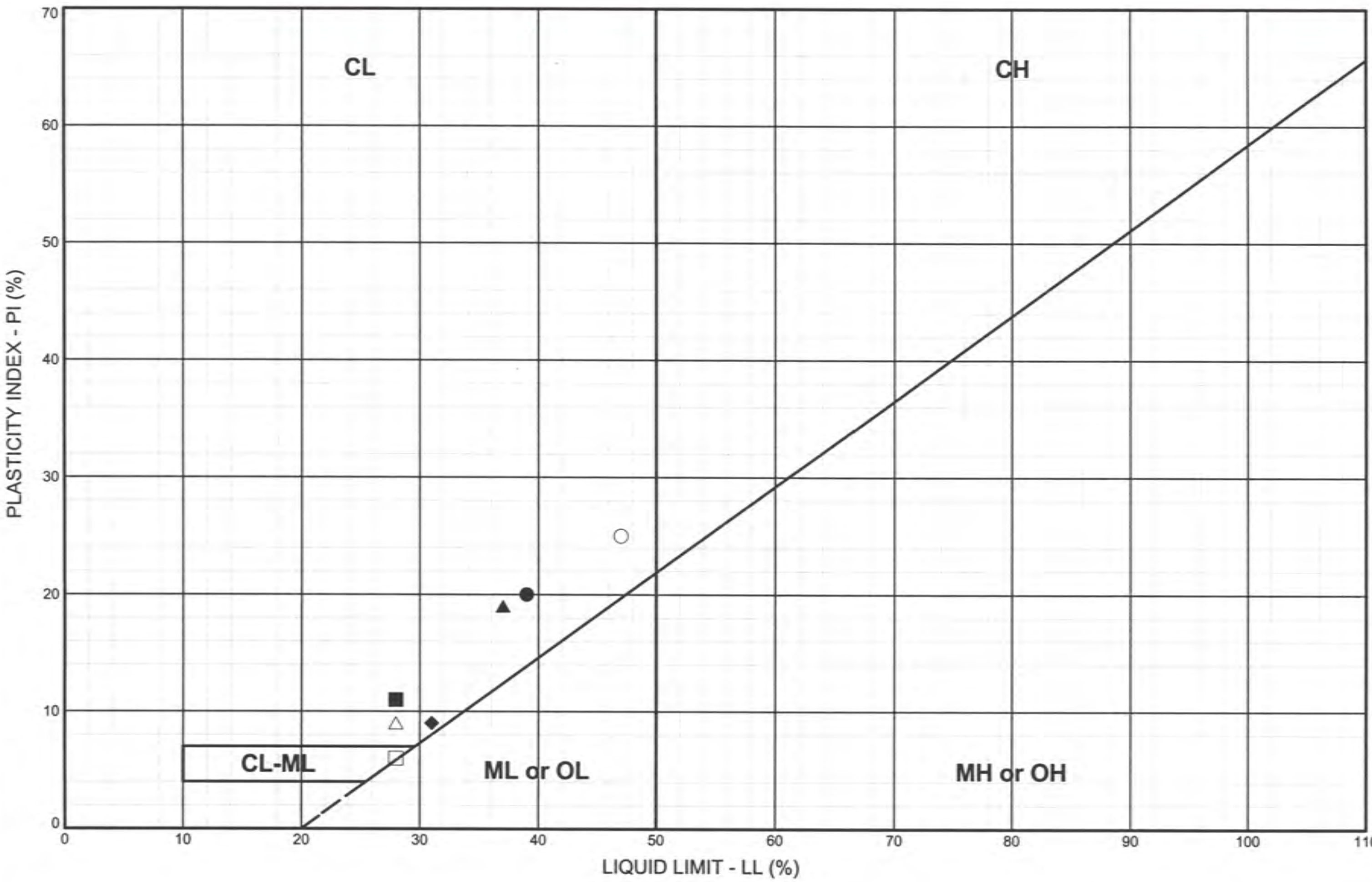
FIG. GD5-B-2



LEGEND

CL: Low plasticity inorganic clays; sandy and silty clays

CH: High plasticity inorganic clays

ML or OL: Inorganic and organic silts and clayey silts of low plasticity

MH or OH: Inorganic and organic silts and clayey silts of high plasticity

CL-ML: Silty clays and clayey silts

NOTES

AD Sample air dried before testing
ND Sample not air dried

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BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	LL %	PL %	PI %	NAT. W.C. %	PASS. #200,%	TEST BY	CKD BY	SMPL PREP.
● SWB-201, S-8	20.0	CL	Light brown, silty CLAY	39	19	20	27.2		TJS	JFL	ND
■ SWB-201, S-10	30.0	CL	Light brown, sandy, silty CLAY	28	17	11	27.0		TJS	JFL	ND
▲ SWB-202, S-6	15.0	CL	Gray, slightly sandy, silty CLAY	37	18	19	27.9		TJS	JFL	ND
◆ SWB-202, S-8	20.0	CL	Light brown, silty CLAY, trace of sand	31	22	9	31.0		AKV	JFL	ND
○ SWB-202, S-11	27.5	CL	Light brown, silty CLAY	47	22	25	29.5		AKV	JFL	ND
□ SWB-202, S-14	35.0	CL-ML	Brown, sandy, clayey SILT	28	22	6	28.5	69.3	AKV	JFL	ND
△ SWB-202, S-17	45.0	CL	Light brown, slightly sandy, silty CLAY	28	19	9	23.7		AKV	JFL	ND

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PLASTICITY CHART

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FIG. GD5-B-3

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FIG. GD5-B-3

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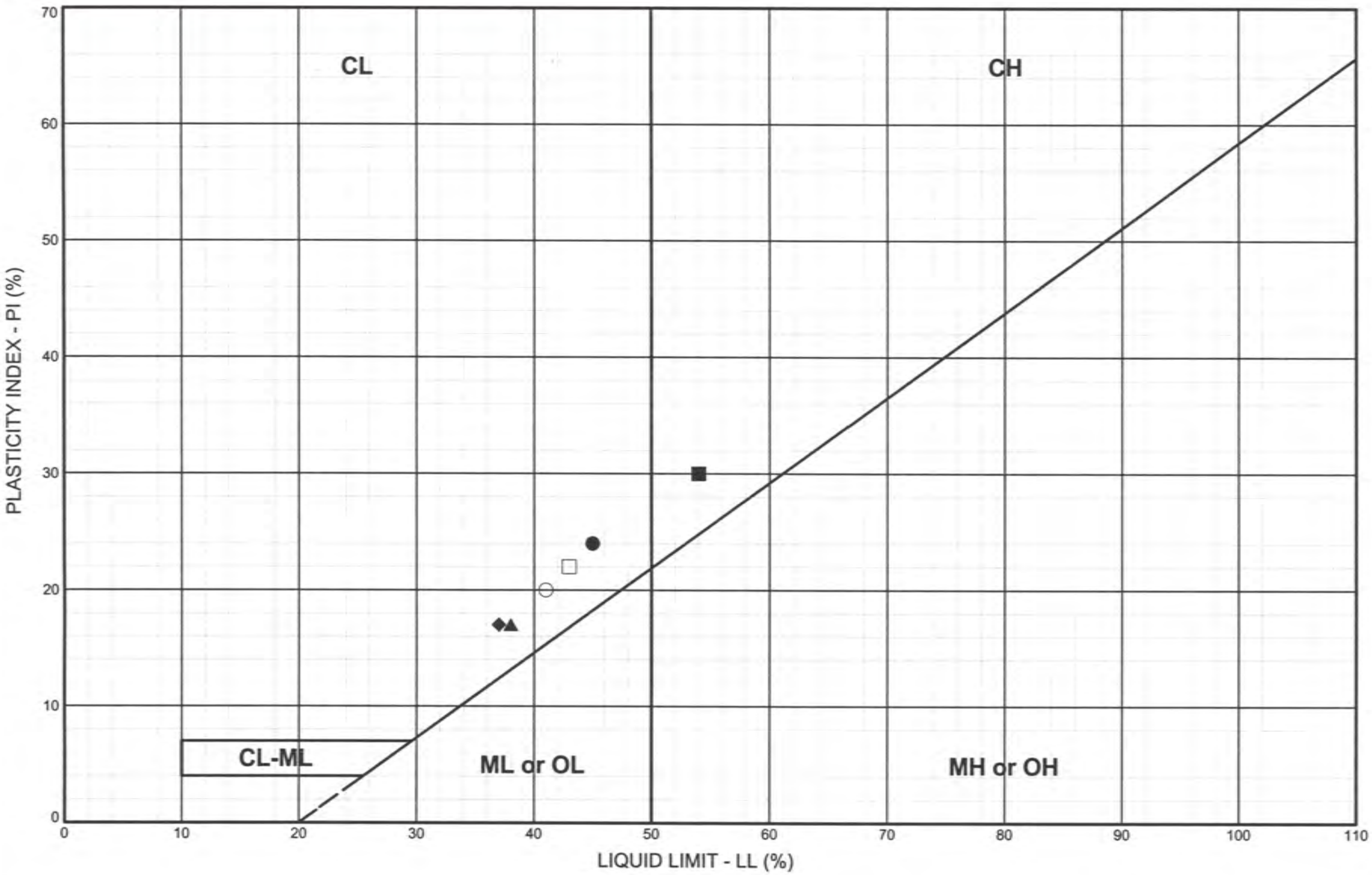

LEGEND

CL: Low plasticity inorganic clays; sandy and silty clays

CH: High plasticity inorganic clays

ML or OL: Inorganic and organic silts and clayey silts of low plasticity

MH or OH: Inorganic and organic silts and clayey silts of high plasticity

CL-ML: Silty clays and clayey silts

NOTES

AD Sample air dried before testing
ND Sample not air dried

BORING AND SAMPLE NO.	DEPTH (feet)	U.S.C.S. SYMBOL	SOIL CLASSIFICATION	LL %	PL %	PI %	NAT. W.C. %	PASS. #200,%	TEST BY	CKD BY	SMPL PREP.
● SWB-203, S-3	7.5	CL	Gray, silty CLAY	45	21	24	29.0		AKV	JFL	ND
■ SWB-203, S-7	17.5	CH	Gray, silty CLAY	54	24	30	56.5		AKV	JFL	ND
▲ SWB-204, S-5	12.5	CL	Gray, silty CLAY	38	21	17	29.6		AKV	JFL	ND
◆ SWB-204, S-9	25.0	CL	Gray, silty CLAY	37	20	17	39.0		TJS	JFL	ND
○ SWB-204, S-15	56.1	CL	Gray, silty CLAY; trace of fine organics	41	21	20	44.2		AKV	JFL	ND
□ SWB-204, S-16	60.0	CL	Gray, silty CLAY	43	21	22	45.2		TJS	JFL	ND
											ND

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PLASTICITY CHART

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FIG. GD5-B-4

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FIG. GD5-B-4

ONE DIMENSIONAL CONSOLIDATION TEST NO. 1
SUMMARY OF TEST DATA

Boring	SWB-202	Tested By / Date	GK 12/19/08
Sample	S-10	Calc. By / Date	CIJ 1/5/09
Depth, ft	26.75	Check By / Date	CIJ 1/19/09

CLASSIFICATION:
Brown, silty CLAY; CH

SAMPLE DATA:

Spec. Grav. (est.) :	2.70
Liquid Limit :	50
Plastic Limit :	29
Plasticity Index :	21
Specimen :	UNDISTURBED

SPECIMEN DATA:

	Before Test	After Test
Height, inches :	1.000	.794
Diameter, inches :	2.500	2.500
Wet Density, pcf :	119.1	139.6
Dry Density, pcf :	90.1	113.5
Water Content, % :	32.2	23.0
Void Ratio :	.865	.484
Saturation, % :	100	128

Spec Load kg	d 100 0.01mm	Defl Corr 0.01mm	Consol Pressure tsf	Settlement %	Void Ratio	t 50 min.	d 50 0.01mm	Coeff of Consol cm2/sec	Coeff of Perm cm/sec
3.9	5.9	.0	.12	.2	.865	.4	4.6	1.32E-02	
7.7	18.2	.0	.25	.7	.856	.3	15.5	1.74E-02	6.76E-07
30.9	66.7	.0	1.00	2.6	.821	.3	56.4	1.69E-02	4.30E-07
61.8	115.5	.0	2.00	4.5	.785	.6	104.4	8.12E-03	1.56E-07
123.7	179.5	.0	4.00	7.1	.738	.6	163.3	7.73E-03	9.74E-08
247.4	279.4	.0	8.00	11.0	.664	1.8	248.9	2.39E-03	2.35E-08
61.8	293.2	.0	2.00	11.5	.654	.2	299.2	2.06E-02	
15.5	272.9	.0	.50	10.7	.669	.1	279.9	4.19E-02	
61.8	263.8	.0	2.00	10.4	.676	.6	261.4	7.10E-03	1.71E-08
123.7	280.6	.0	4.00	11.0	.663	.4	275.1	1.05E-02	3.49E-08
247.4	309.2	.0	8.00	12.2	.642	.5	300.7	8.23E-03	2.31E-08
494.7	407.4	.0	16.00	16.0	.570	2.6	367.0	1.49E-03	7.20E-09
989.4	523.9	.0	32.00	20.6	.484	2.7	471.2	1.30E-03	3.73E-09

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CONSOLIDATION TEST
BORING SWB-202, S-10

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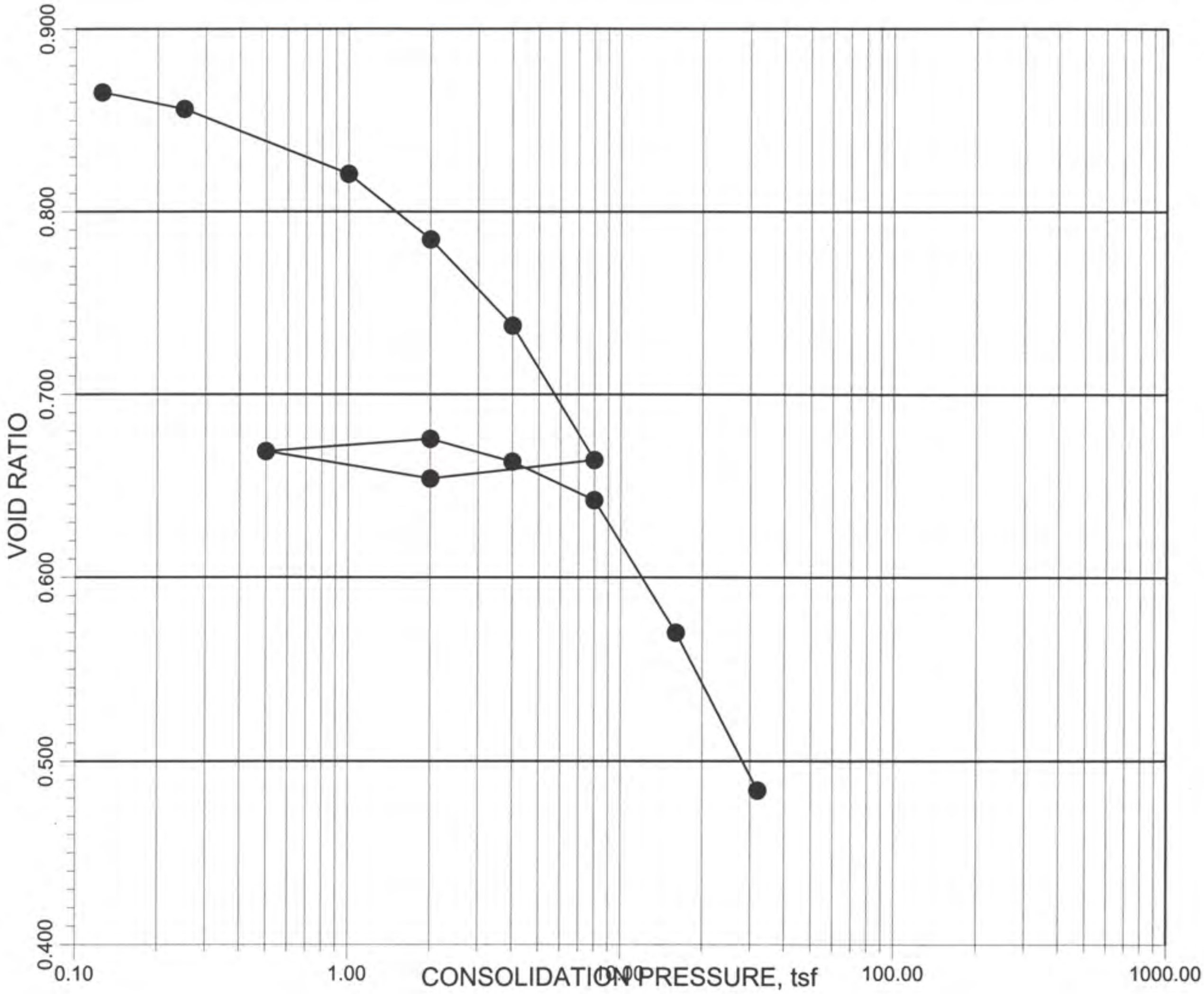
FIG. GD5-B-5a

ONE DIMENSIONAL CONSOLIDATION TEST NO. 1
VOID RATIO VS LOG10(CONSOLIDATION PRESSURE)

Boring	SWB-202	Tested	By / Date	GK	12/19/08
Sample	S-10	Calc.	By / Date	CIJ	1/5/09
Depth, ft	26.75	Check	By / Date	CIJ	1/19/09



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CONSOLIDATION TEST
BORING SWB-202, S-10

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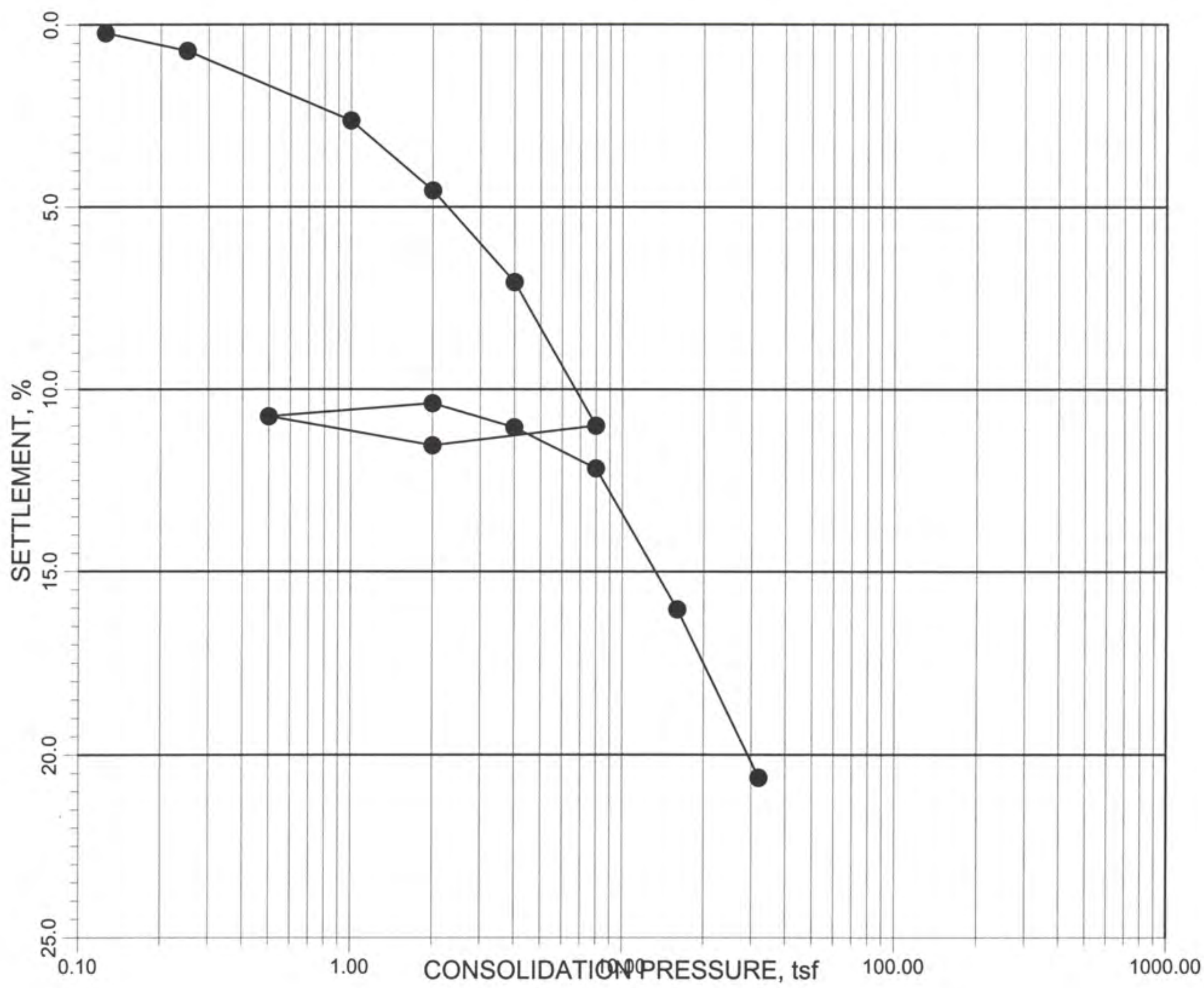
FIG. GD5-B-5b

ONE DIMENSIONAL CONSOLIDATION TEST NO. 1
SETTLEMENT VS LOG10(CONSOLIDATION PRESSURE)

Boring	SWB-202	Tested	By / Date GK	12/19/08
Sample	S-10	Calc.	By / Date CIJ	1/5/09
Depth, ft	26.75	Check	By / Date CIJ	1/19/09



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CONSOLIDATION TEST
BORING SWB-202, S-10

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FIG. GD5-B-5c

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ONE DIMENSIONAL CONSOLIDATION TEST NO. 2
SUMMARY OF TEST DATA

Boring SWB-202
Sample S-16
Depth, ft 41.5

Tested By / Date GK 12/19/08
Calc. By / Date CIJ 1/5/09
Check By / Date CIJ 1/19/09

CLASSIFICATION:
Light brown, silty CLAY, trace of medium to coarse sand; fissured; calcareous nodules; CL

SAMPLE DATA:
Spec. Grav. (est.) : 2.70
Liquid Limit : 28
Plastic Limit : 18
Plasticity Index : 10
Specimen : UNDISTURBED

SPECIMEN DATA:

	Before Test	After Test
Height, inches :	1.000	.836
Diameter, inches :	2.500	2.500
Wet Density, pcf :	119.6	134.4
Dry Density, pcf :	92.1	110.2
Water Content, % :	29.8	21.9
Void Ratio :	.828	.529
Saturation, % :	97	112

Spec Load kg	d 100 0.01mm	Defl Corr 0.01mm	Consol Pressure tsf	Settlement %	Void Ratio	t 50 min.	d 50 0.01mm	Coeff of Consol cm2/sec	Coeff of Perm cm/sec
3.9	2.0	.0	.12	.1	.828	.3	1.5	1.76E-02	
7.7	8.6	.0	.25	.3	.823	.2	7.7	2.63E-02	5.47E-07
15.5	21.3	.0	.50	.8	.814	.1	19.8	5.21E-02	1.04E-06
30.9	42.9	.0	1.00	1.7	.798	.2	40.4	2.56E-02	4.36E-07
61.8	77.0	.0	2.00	3.0	.774	.1	72.9	5.00E-02	6.69E-07
123.7	131.8	.0	4.00	5.2	.734	.1	123.7	4.79E-02	5.18E-07
247.4	214.9	.0	8.00	8.5	.675	.2	202.7	2.24E-02	1.83E-07
61.8	219.5	.0	2.00	8.6	.671	.1	219.7	4.42E-02	
15.5	208.0	.0	.50	8.2	.679	.1	209.3	4.46E-02	
30.9	208.8	.0	1.00	8.2	.679	.0	208.5	1.12E-01	6.68E-08
61.8	213.9	.0	2.00	8.4	.675	.1	213.4	4.44E-02	8.89E-08
123.7	221.2	.0	4.00	8.7	.670	.1	220.5	4.42E-02	6.40E-08
247.4	234.7	.0	8.00	9.2	.660	.1	232.7	4.37E-02	5.79E-08
494.7	311.9	.0	16.00	12.3	.605	.1	297.2	4.13E-02	1.57E-07
989.4	417.1	.0	32.00	16.4	.529	.1	402.1	3.75E-02	9.71E-08

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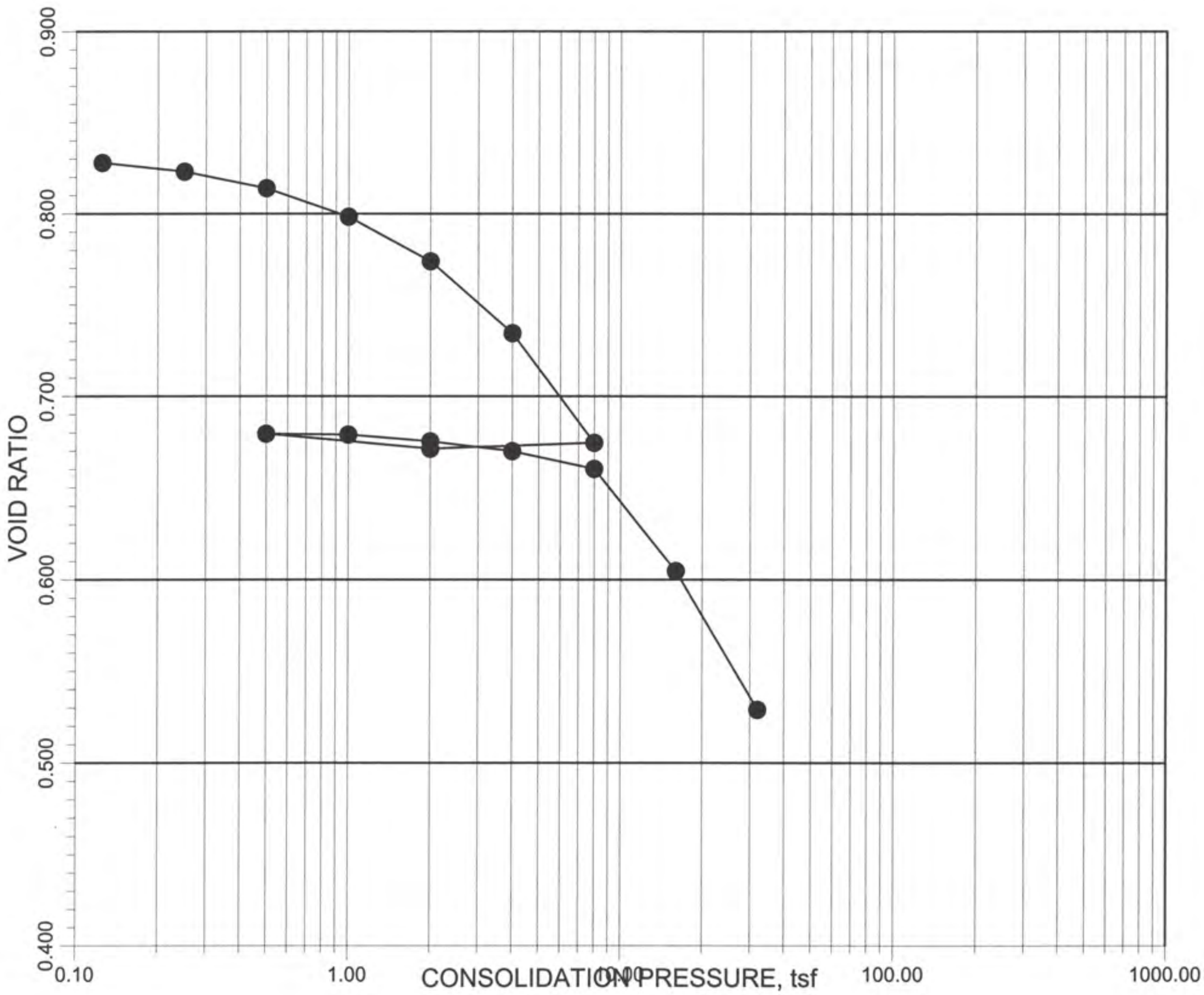
FIG. GD5-B-6a

ONE DIMENSIONAL CONSOLIDATION TEST NO. 2
VOID RATIO VS LOG10(CONSOLIDATION PRESSURE)

Boring	SWB-202	Tested	By / Date	GK	12/19/08
Sample	S-16	Calc.	By / Date	CIJ	1/5/09
Depth, ft	41.5	Check	By / Date	CIJ	1/19/09



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CONSOLIDATION TEST
BORING SWB-202, S-16

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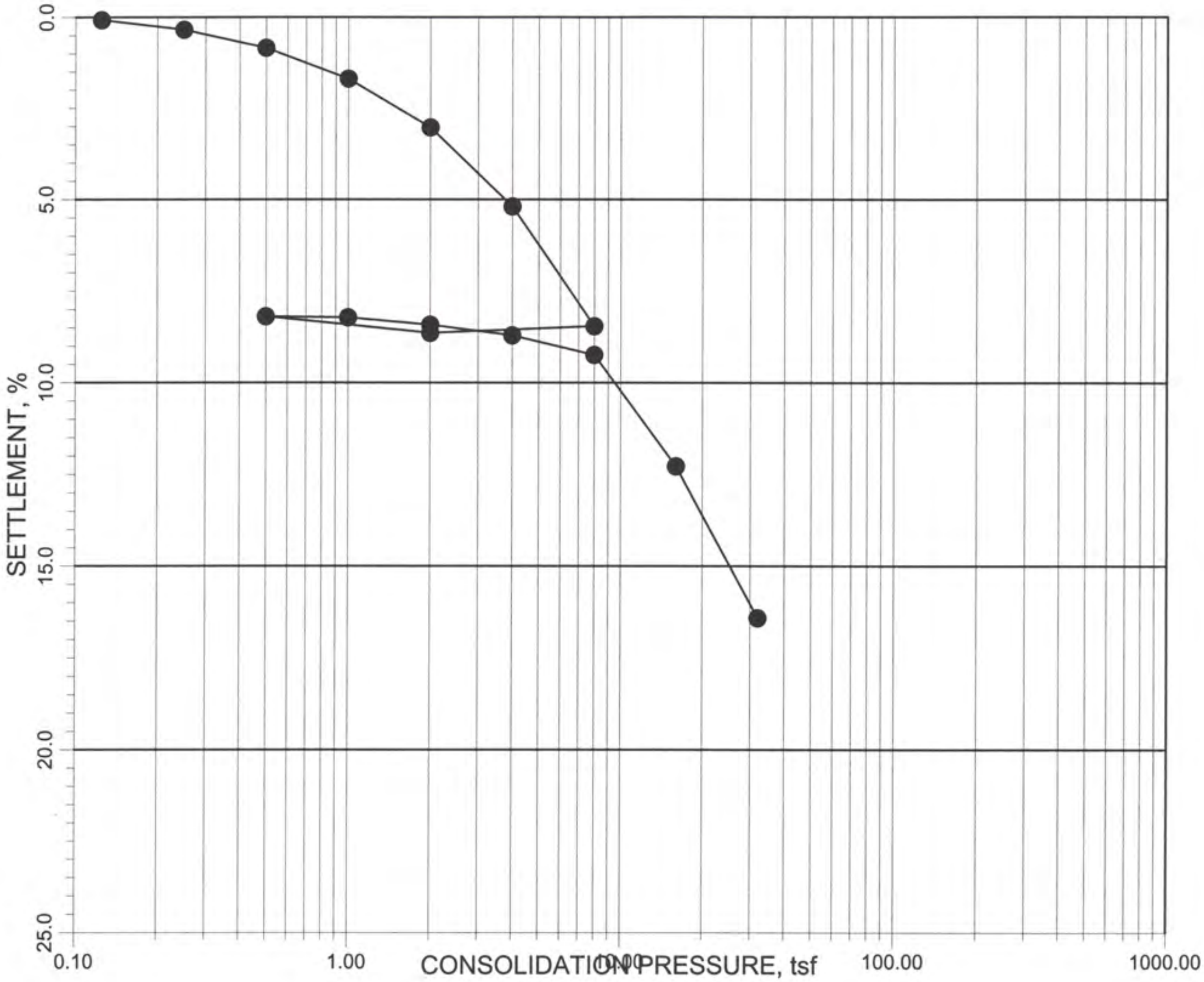
FIG. GD5-B-6b

ONE DIMENSIONAL CONSOLIDATION TEST NO. 2
SETTLEMENT VS LOG10(CONSOLIDATION PRESSURE)

Boring	SWB-202	Tested By / Date	GK 12/19/08
Sample	S-16	Calc. By / Date	CIJ 1/5/09
Depth, ft	41.5	Check By / Date	CIJ 1/19/09



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CONSOLIDATION TEST
BORING SWB-202, S-16

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FIG. GD5-B-6c

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ONE DIMENSIONAL CONSOLIDATION TEST NO. 3
SUMMARY OF TEST DATA

Boring SWB-203
Sample S-5
Depth, ft 13.5

Tested By / Date GK 12/29/08
Calc. By / Date CIJ 1/5/09
Check By / Date CIJ 1/19/09

CLASSIFICATION:
Gray, silty CLAY; trace of fibrous organics; occasional orange oxidation; CH.

SAMPLE DATA:
Spec. Grav. (est.) : 2.70
Liquid Limit : 50
Plastic Limit : 23
Plasticity Index : 27
Specimen : UNDISTURBED

SPECIMEN DATA:

	Before Test	After Test
Height, inches :	1.000	.798
Diameter, inches :	2.500	2.500
Wet Density, pcf :	116.1	132.8
Dry Density, pcf :	85.0	106.5
Water Content, % :	36.6	24.6
Void Ratio :	.981	.581
Saturation, % :	101	114

Spec Load kg	d 100 0.01mm	Defl Corr 0.01mm	Consol Pressure tsf	Settlement %	Void Ratio	t 50 min.	d 50 0.01mm	Coeff of Consol cm2/sec	Coeff of Perm cm/sec
3.9	2.0	.0	.12	.1	.981	1.0	1.3	5.29E-03	
7.7	8.9	.0	.25	.3	.976	.5	6.9	1.05E-02	2.27E-07
15.5	21.8	.0	.50	.9	.965	.5	18.0	1.04E-02	2.13E-07
30.9	54.6	.0	1.00	2.1	.940	2.6	41.9	1.97E-03	5.08E-08
61.8	87.1	.0	2.00	3.4	.915	1.6	73.2	3.12E-03	4.00E-08
123.7	141.2	.0	4.00	5.6	.872	1.7	120.9	2.83E-03	3.01E-08
247.4	246.4	.0	8.00	9.7	.790	3.2	204.2	1.40E-03	1.45E-08
61.8	247.1	.0	2.00	9.7	.790	.7	255.5	6.12E-03	
15.5	203.7	.0	.50	8.0	.824	4.0	223.5	1.10E-03	
30.9	206.5	.0	1.00	8.1	.821	1.2	205.5	3.73E-03	8.19E-09
61.8	219.0	.0	2.00	8.6	.812	1.5	213.6	2.96E-03	1.45E-08
123.7	241.6	.0	4.00	9.5	.794	1.2	231.4	3.65E-03	1.62E-08
247.4	274.3	.0	8.00	10.8	.768	1.1	259.3	3.88E-03	1.25E-08
494.7	378.5	.0	16.00	14.9	.687	2.7	328.2	1.49E-03	7.62E-09
989.4	514.1	.0	32.00	20.2	.581	2.6	447.8	1.38E-03	4.61E-09

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CONSOLIDATION TEST
BORING SWB-203, S-5

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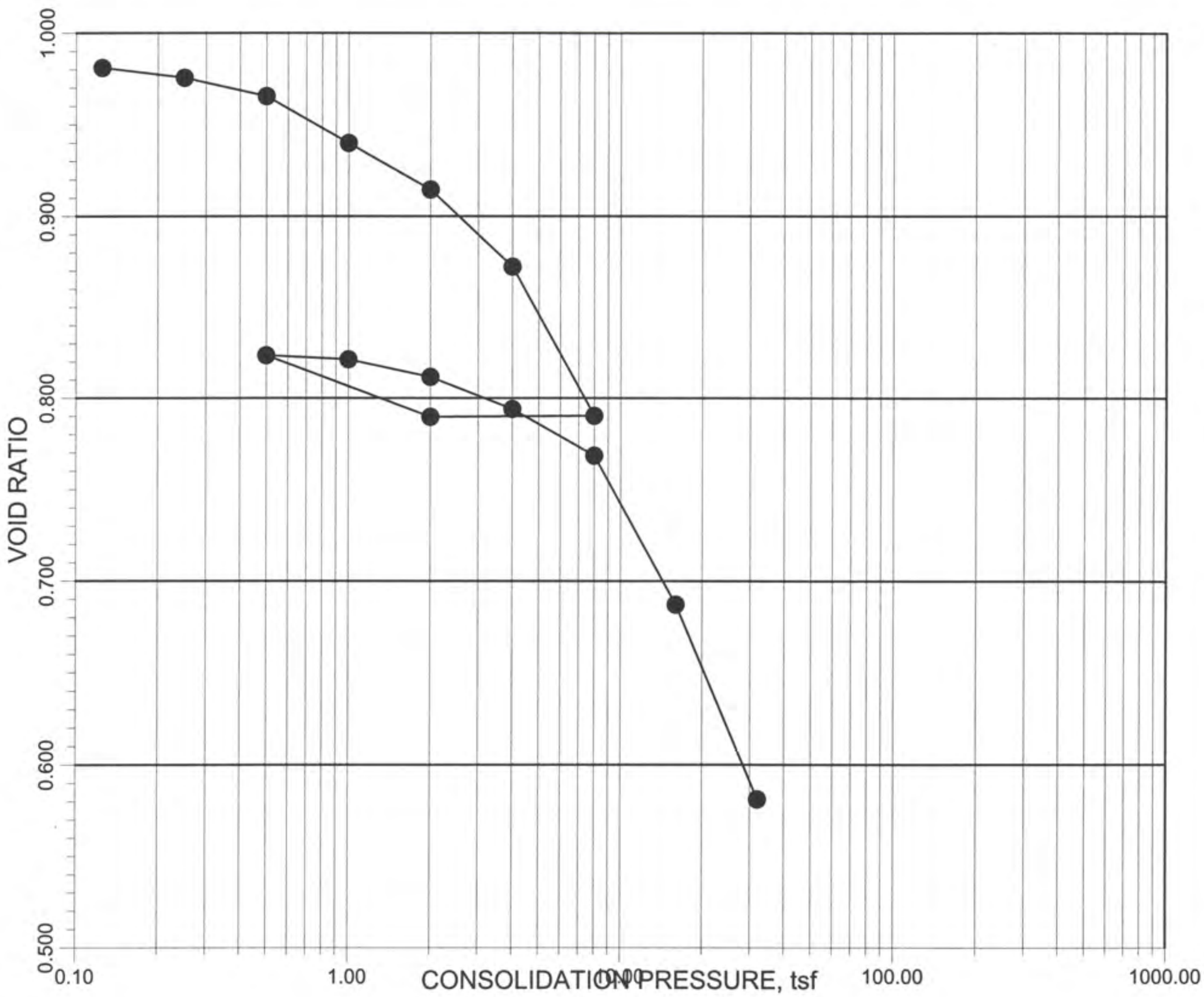
FIG. GD5-B-7a

ONE DIMENSIONAL CONSOLIDATION TEST NO. 3
VOID RATIO VS LOG10(CONSOLIDATION PRESSURE)

Boring	SWB-203	Tested	By / Date	GK	12/29/08
Sample	S-5	Calc.	By / Date	CIJ	1/5/09
Depth, ft	13.5	Check	By / Date	CIJ	1/19/09



Pioneer Crossing, Lehi
& I-15 American Fork Interchange
Lehi, Utah

CONSOLIDATION TEST
BORING SWB-203, S-5

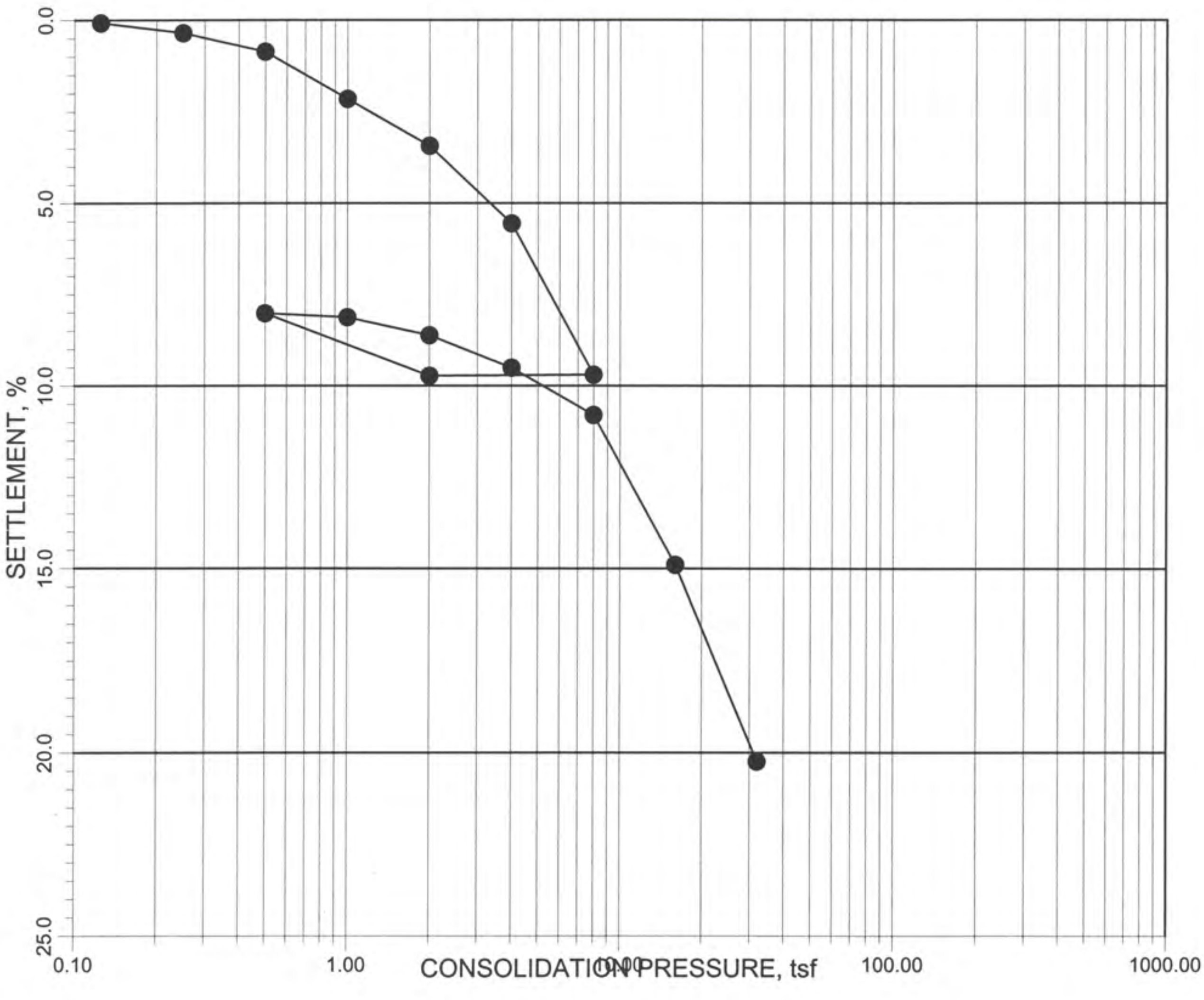
February 2009 23-1-01178-010

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. GD5-B-7b

ONE DIMENSIONAL CONSOLIDATION TEST NO. 3
SETTLEMENT VS LOG10(CONSOLIDATION PRESSURE)

Boring	SWB-203	Tested	By / Date	GK	12/29/08
Sample	S-5	Calc.	By / Date	CIJ	1/5/09
Depth, ft	13.5	Check	By / Date	CIJ	1/19/09



Pioneer Crossing, Lehi
& I-15 American Fork Interchange
Lehi, Utah

CONSOLIDATION TEST
BORING SWB-203, S-5

February 2009 23-1-01178-010

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. GD5-B-7c

ONE DIMENSIONAL CONSOLIDATION TEST NO. 4
SUMMARY OF TEST DATA

Boring SWB-204
Sample S-11
Depth, ft 35.3

Tested By / Date GK 12/29/08
Calc. By / Date CIJ 1/5/09
Check By / Date CIJ 1/19/09

CLASSIFICATION:
Gray, silty CLAY; CL.

SAMPLE DATA:
Spec. Grav. (est.) : 2.70
Liquid Limit : 36
Plastic Limit : 22
Plasticity Index : 14
Specimen : UNDISTURBED

SPECIMEN DATA:

	Before Test	After Test
Height, inches :	1.000	.793
Diameter, inches :	2.500	2.500
Wet Density, pcf :	117.3	133.5
Dry Density, pcf :	85.7	108.0
Water Content, % :	36.9	23.6
Void Ratio :	.965	.560
Saturation, % :	103	114

Spec Load kg	d 100 0.01mm	Defl Corr 0.01mm	Consol Pressure tsf	Settlement %	Void Ratio	t 50 min.	d 50 0.01mm	Coeff of Consol cm2/sec	Coeff of Perm cm/sec
3.9	3.0	.0	.12	.1	.965	.3	2.1	2.11E-02	
7.7	11.7	.0	.25	.5	.958	.6	9.1	8.76E-03	2.40E-07
15.5	23.7	.0	.50	.9	.949	.4	20.1	1.30E-02	2.47E-07
30.9	41.4	.0	1.00	1.6	.935	.5	35.3	1.03E-02	1.43E-07
61.8	68.2	.0	2.00	2.7	.914	.6	57.5	8.43E-03	8.91E-08
123.7	161.8	.0	4.00	6.4	.842	2.4	120.0	2.00E-03	3.69E-08
247.4	304.3	.0	8.00	12.0	.731	2.3	248.3	1.87E-03	2.63E-08
61.8	313.5	.0	2.00	12.3	.724	.3	319.7	1.35E-02	
15.5	283.7	.0	.50	11.2	.747	1.4	297.1	2.95E-03	
30.9	282.8	.0	1.00	11.1	.748	.5	282.1	8.37E-03	6.13E-09
61.8	293.9	.0	2.00	11.6	.739	.6	288.9	6.93E-03	3.02E-08
123.7	310.5	.0	4.00	12.2	.726	.5	302.2	8.22E-03	2.70E-08
247.4	337.9	.0	8.00	13.3	.705	.5	325.9	8.05E-03	2.17E-08
494.7	427.3	.0	16.00	16.8	.636	1.1	383.6	3.47E-03	1.53E-08
989.4	524.9	.0	32.00	20.7	.560	.7	477.4	4.99E-03	1.20E-08

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Pioneer Crossing, Lehi
& I-15 American Fork Interchange
Lehi, Utah

CONSOLIDATION TEST
BORING SWB-204, S-11

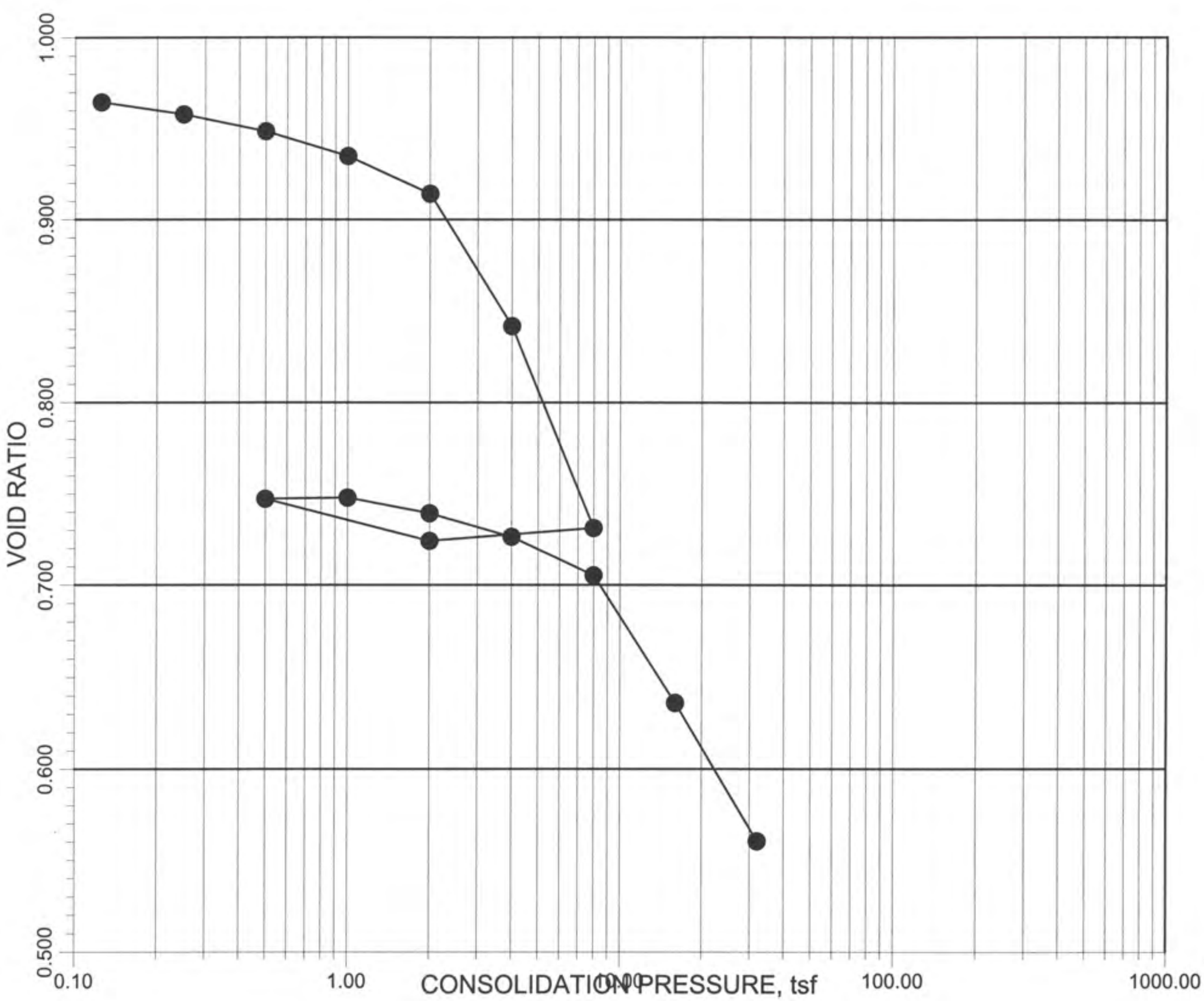
February 2009 23-1-01178-010

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. GD5-B-8a

ONE DIMENSIONAL CONSOLIDATION TEST NO. 4
VOID RATIO VS LOG10(CONSOLIDATION PRESSURE)

Boring	SWB-204	Tested	By / Date	GK	12/29/08
Sample	S-11	Calc.	By / Date	CIJ	1/5/09
Depth, ft	35.3	Check	By / Date	CIJ	1/19/09



Pioneer Crossing, Lehi
& I-15 American Fork Interchange
Lehi, Utah

CONSOLIDATION TEST
BORING SWB-204, S-11

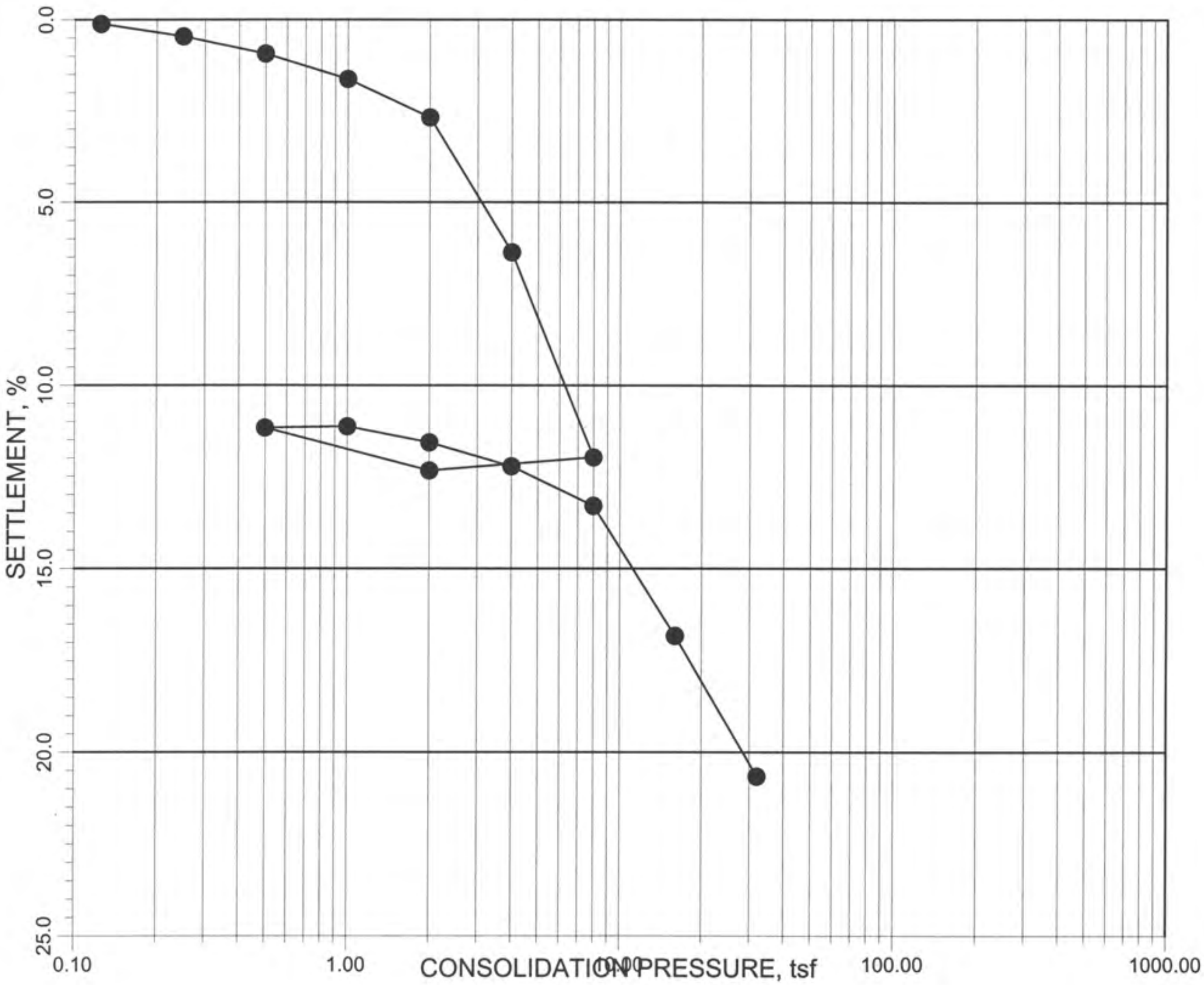
February 2009 | 23-1-01178-010

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. GD5-B-8b

ONE DIMENSIONAL CONSOLIDATION TEST NO. 4
SETTLEMENT VS LOG10(CONSOLIDATION PRESSURE)

Boring	SWB-204	Tested By / Date	GK 12/29/08
Sample	S-11	Calc. By / Date	CIJ 1/5/09
Depth, ft	35.3	Check By / Date	CIJ 1/19/09



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Pioneer Crossing, Lehi
& I-15 American Fork Interchange
Lehi, Utah

CONSOLIDATION TEST
BORING SWB-204, S-11

February 2009 23-1-01178-010

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. GD5-B-8c

ONE DIMENSIONAL CONSOLIDATION TEST NO. 5
SUMMARY OF TEST DATA

Boring	SWB-204	Tested By / Date	JFL 1/6/09
Sample	S-15	Calc. By / Date	JFL 1/19/09
Depth, ft	56.1	Check By / Date	GJW 2-25-09

CLASSIFICATION:
Gray, silty CLAY, trace of sand and gravel; trace of fine organics; CL.

SAMPLE DATA:

Spec. Grav. (est.) : 2.70
Liquid Limit : 41
Plastic Limit : 21
Plasticity Index : 20
Specimen : UNDISTURBED

SPECIMEN DATA:

	Before Test	After Test
Height, inches :	.785	.631
Diameter, inches :	2.508	2.508
Wet Density, pcf :	109.0	121.1
Dry Density, pcf :	73.8	91.6
Water Content, % :	47.7	32.2
Void Ratio :	1.282	.836
Saturation, % :	100	104

Spec Load kg	d 100 0.01mm	Defl Corr 0.01mm	Consol Pressure tsf	Settlement %	Void Ratio	t 50 min.	d 50 0.01mm	Coeff of Consol cm2/sec	Coeff of Perm cm/sec
.2	1.5	.7	.06	.0	1.282	.9	.9	3.63E-03	
.4	6.8	2.7	.13	.2	1.278	.7	5.0	4.65E-03	1.20E-07
.8	14.1	6.6	.26	.4	1.274	.5	11.8	6.49E-03	8.74E-08
1.6	26.5	10.8	.51	.8	1.265	.6	22.2	5.38E-03	8.54E-08
3.2	44.6	15.0	1.03	1.5	1.249	.6	38.5	5.31E-03	7.23E-08
6.4	69.5	19.8	2.06	2.5	1.226	.5	61.2	6.26E-03	6.12E-08
12.8	162.4	25.4	4.11	6.9	1.126	2.1	125.5	1.40E-03	2.99E-08
25.6	333.7	31.6	8.23	15.2	.937	2.2	290.4	1.12E-03	2.26E-08
51.2	456.7	38.5	16.45	21.0	.804	1.2	423.3	1.77E-03	1.25E-08
102.4	569.2	47.5	32.90	26.2	.686	1.0	517.4	1.91E-03	6.01E-09
25.6	561.5	39.1	8.23	26.2	.685	.3	566.1	5.89E-03	
6.4	524.0	30.2	2.06	24.8	.718	1.1	538.4	1.65E-03	
1.6	468.5	25.6	.51	22.2	.776	5.4	493.3	3.54E-04	
.4	413.1	22.5	.13	19.6	.836	16.7	441.4	1.22E-04	

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Pioneer Crossing, Lehi
& I-15 American Fork Interchange
Lehi, Utah

CONSOLIDATION TEST
BORING SWB-204, S-15

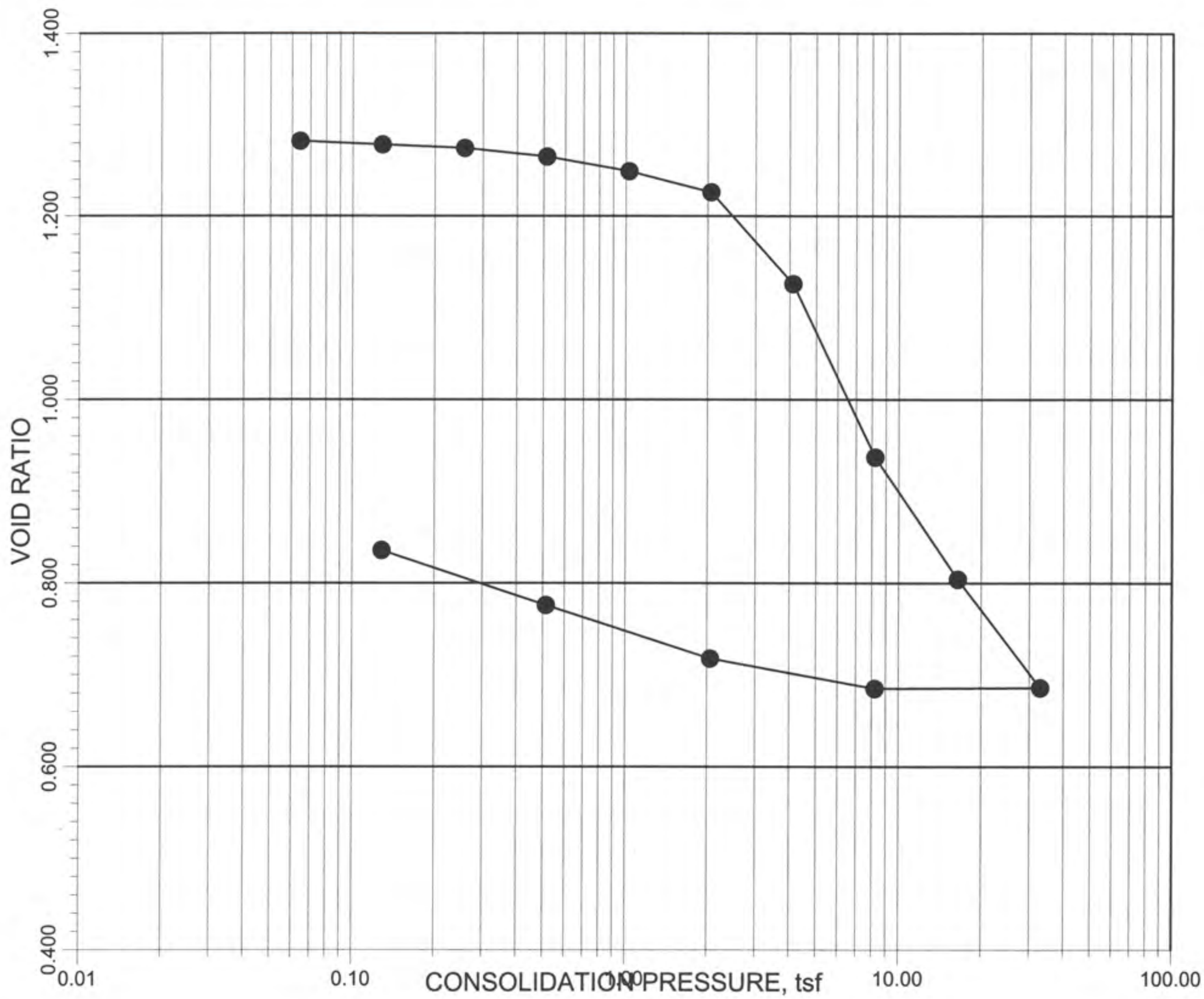
February 2009 23-1-01178-010

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. GD5-B-9a

ONE DIMENSIONAL CONSOLIDATION TEST NO. 5
VOID RATIO VS LOG10(CONSOLIDATION PRESSURE)

Boring	SWB-204	Tested By / Date	JFL 1/6/09
Sample	S-15	Calc. By / Date	JFL 1/19/09
Depth, ft	56.1	Check By / Date	ajd 2-25-09



Pioneer Crossing, Lehi
& I-15 American Fork Interchange
Lehi, Utah

CONSOLIDATION TEST
BORING SWB-204, S-15

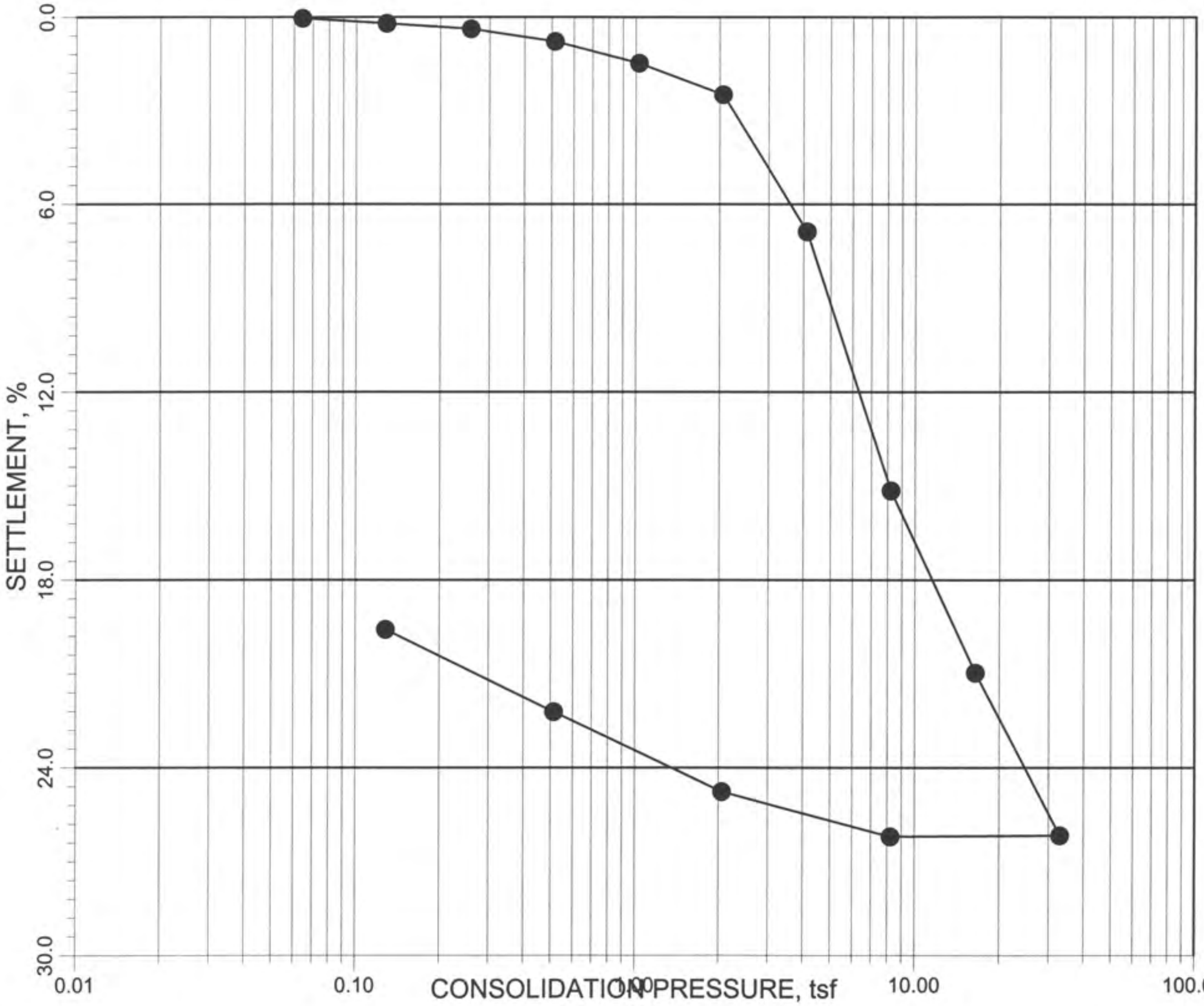
February 2009 | 23-1-01178-010

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. GD5-B-9b

ONE DIMENSIONAL CONSOLIDATION TEST NO. 5
SETTLEMENT VS LOG10(CONSOLIDATION PRESSURE)

Boring	SWB-204	Tested By / Date	JFL 1/6/09
Sample	S-15	Calc. By / Date	JFL 1/19/09
Depth, ft	56.1	Check By / Date	2-25-09



Pioneer Crossing, Lehi
& I-15 American Fork Interchange
Lehi, Utah

CONSOLIDATION TEST
BORING SWB-204, S-15

February 2009 23-1-01178-010

SHANNON & WILSON, INC.
GEOTECHNICAL AND ENVIRONMENTAL CONSULTANTS

FIG. GD5-B-9c

MINUS NO. 200 SIEVE ANALYSIS			Job:	1-15 East West Connector			Tested by:	N.M		
			Job #:	61081077			Date:	11/25/08		
Boring No.	B-8									
Depth, Ft.	35.0'									
Tare wt., g	74.18									
Wet wt. + tare before wash, g	305.20									
Dry wt. + tare before wash, g	254.80									
Wet wt. - tare before wash, g	231.02									
Dry wt. - tare before wash, g	180.62									
Weight of water, g	50.40									
Moisture Content	**27.9%**									
Tare wt., g	74.18									
Dry wt. + tare before wash, g	254.80									
Dry wt. + tare after wash, g	78.30									
Dry wt. - tare before wash, g	180.62									
Dry wt. - tare after wash, g	4.12									
Wt. washed thru No. 200, g	176.50									
Wt. on pan after rotapping, g	0.30									
Total wt. Passing, No. 200	176.80									
PERCENT MINUS NO. 200	**97.9%**									
Boring No.										
Depth, Ft.										
Tare wt., g										
Wet wt. + tare before wash, g										
Dry wt. + tare before wash, g										
Wet wt. - tare before wash, g										
Dry wt. - tare before wash, g										
Weight of water, g										
Moisture Content										
Tare wt., g										
Dry wt. + tare before wash, g										
Dry wt. + tare after wash, g										
Dry wt. - tare before wash, g										
Dry wt. - tare after wash, g										
Wt. washed thru No. 200, g										
Wt. on pan after rotapping, g										
Total wt. Passing, No. 200										
PERCENT MINUS NO. 200										

FIG. GD5-B-10

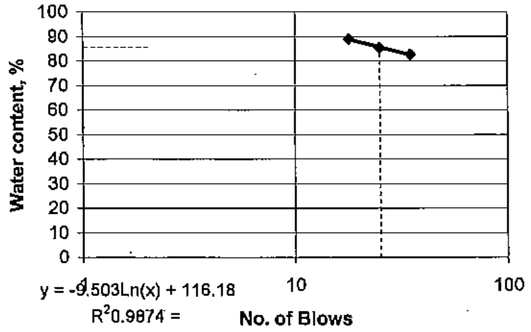
ATTERBERG LIMITS

Job No. 61081077 Checked by: ______

Sample No. B-3 @ 5.0'

LIQUID LIMIT DETERMINATION				PLASTIC LIMIT	
Can No.	1	2	3	1	2
Wet Weight + Tare	22.47	20.69	22.55	24.43	24.07
Dry Weight + Tare	16.49	15.46	16.37	23.82	23.46
Tare	9.25	9.32	9.42	22.03	21.69
Weight of Dry Sample	7.24	6.14	6.95	1.79	1.77
Weight of Water	5.98	5.23	6.18	0.61	0.61
% Moisture (w)	82.6	85.18	88.92	34.08	34.46
No. of Blows (N)	35	25	16		

Liquid Limit **85.6**
Plastic Limit **34**
Plasticity Index **51.6** **Fines class: CH**



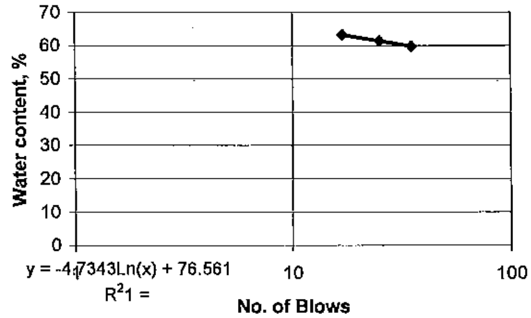
ATTERBERG LIMITS

Job No. 61081077 Checked by: ______

Sample No. B-3 @ 12.5'

LIQUID LIMIT DETERMINATION				PLASTIC LIMIT	
Can No.	1	2	3	1	2
Wet Weight + Tare	19.81	20.86	23.02	24.27	24.01
Dry Weight + Tare	15.88	16.42	17.64	23.8	23.53
Tare	9.3	9.18	9.12	22.21	21.9
Weight of Dry Sample	6.58	7.24	8.52	1.59	1.63
Weight of Water	3.93	4.44	5.38	0.47	0.48
% Moisture (w)	59.73	61.33	63.15	29.56	29.45
No. of Blows (N)	35	25	17		

Liquid Limit **61**
Plastic Limit **30**
Plasticity Index **31** **Fines class: CH**



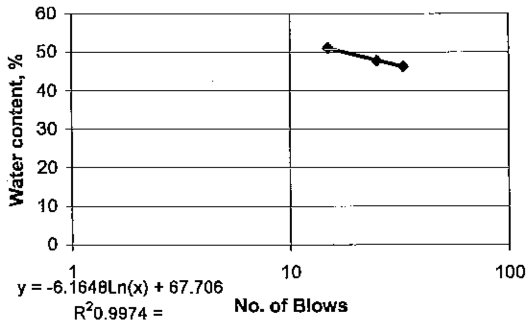
ATTERBERG LIMITS

Job No. 61081077 Checked by: ______

Sample No. B-3 @ 15.0'

LIQUID LIMIT DETERMINATION				PLASTIC LIMIT	
Can No.	1	2	3	1	2
Wet Weight + Tare	20.63	21.76	25.66	26.22	26.02
Dry Weight + Tare	17.06	17.68	20.13	25.45	25.3
Tare	9.34	9.13	9.3	21.99	22.1
Weight of Dry Sample	7.72	8.55	10.83	3.46	3.2
Weight of Water	3.57	4.08	5.53	0.77	0.72
% Moisture (w)	46.24	47.72	51.06	22.25	22.5
No. of Blows (N)	33	25	15		

Liquid Limit **48**
Plastic Limit **22**
Plasticity Index **26** **Fines class: CL**



ATTERBERG LIMITS

Job No. 61081077 Checked by: ______

Sample No. B-3 @ 25.0'

LIQUID LIMIT DETERMINATION				PLASTIC LIMIT	
Can No.	1	2	3	1	2
Wet Weight + Tare	24.09	31.4	24.08	24.72	24.49
Dry Weight + Tare	19.86	28.69	19.71	24.22	23.96
Tare	9.09	21.61	9.33	22.03	21.61
Weight of Dry Sample	10.77	7.08	10.38	2.19	2.35
Weight of Water	4.23	2.71	4.37	0.5	0.53
% Moisture (w)	39.28	38.28	42.1	22.83	22.55
No. of Blows (N)	35	25	16		

Liquid Limit **40**
Plastic Limit **23**
Plasticity Index **17** **Fines class: CL**

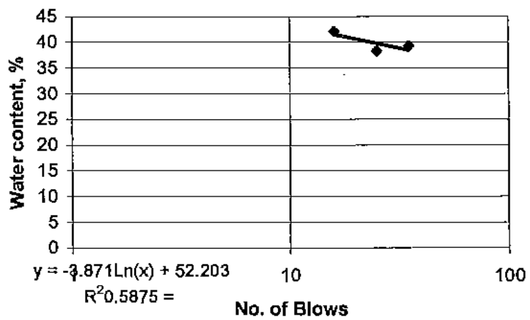
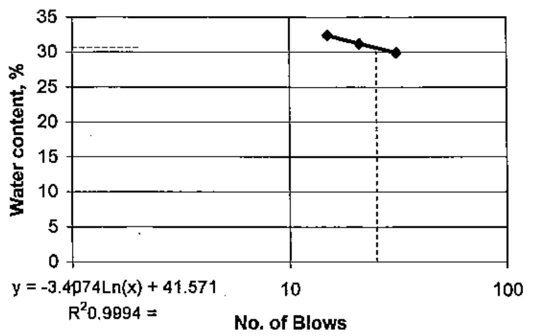


FIG. GD5-B-11
(SHEET 1 OF 3)

ATTERBERG LIMITS Job No. 61081077 Checked by: ______
Sample No. B-3 @ 35.0'

LIQUID LIMIT DETERMINATION				PLASTIC LIMIT	
Can No.	1	2	3	1	2
Wet Weight + Tare	20.59	23.98	21.59	25.52	25.35
Dry Weight + Tare	17.96	20.44	18.59	24.97	24.84
Tare	9.16	9.08	9.32	21.72	21.66
Weight of Dry Sample	8.8	11.36	9.27	3.25	3.18
Weight of Water	2.63	3.54	3	0.55	0.51
% Moisture (w)	29.89	31.16	32.36	16.92	16.04
No. of Blows (N)	31	21	15		

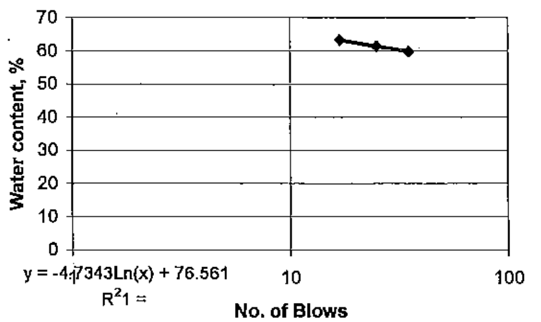
Liquid Limit 30.6
Plastic Limit 16
Plasticity Index 14.6 **Fines class: CL**



ATTERBERG LIMITS Job No. 61081077 Checked by: ______
Sample No. B-3 @ 12.5'

LIQUID LIMIT DETERMINATION				PLASTIC LIMIT	
Can No.	1	2	3	1	2
Wet Weight + Tare	19.81	20.86	23.02	24.27	24.01
Dry Weight + Tare	15.88	16.42	17.64	23.8	23.53
Tare	9.3	9.18	9.12	22.21	21.9
Weight of Dry Sample	6.58	7.24	8.52	1.59	1.63
Weight of Water	3.93	4.44	5.38	0.47	0.48
% Moisture (w)	59.73	61.33	63.15	29.56	29.45
No. of Blows (N)	35	25	17		

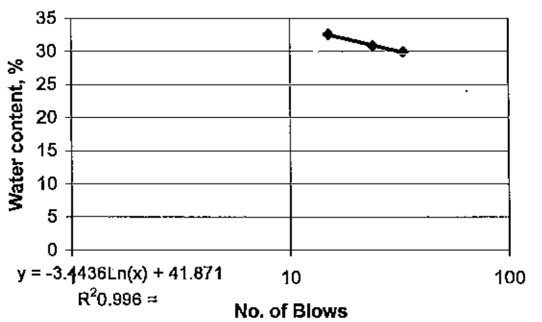
Liquid Limit 61
Plastic Limit 30
Plasticity Index 31 **Fines class: CH**



ATTERBERG LIMITS Job No. 61081077 Checked by: ______
Sample No. B-3 @ 45.0'

LIQUID LIMIT DETERMINATION				PLASTIC LIMIT	
Can No.	1	2	3	1	2
Wet Weight + Tare	16.96	18.48	19.92	24.87	25.2
Dry Weight + Tare	14.53	15.65	16.58	24.43	24.71
Tare	6.4	6.47	6.33	21.65	21.72
Weight of Dry Sample	8.13	9.18	10.25	2.78	2.99
Weight of Water	2.43	2.83	3.34	0.44	0.49
% Moisture (w)	29.89	30.83	32.59	15.83	16.39
No. of Blows (N)	33	24	15		

Liquid Limit 31
Plastic Limit 16
Plasticity Index 15 **Fines class: CL**



ATTERBERG LIMITS Job No. 61081077 Checked by: ______
Sample No. B-3 @ 7.5'

LIQUID LIMIT DETERMINATION				PLASTIC LIMIT	
Can No.	1	2	3	1	2
Wet Weight + Tare	18.53	21.75	19.87	21.51	18.26
Dry Weight + Tare	15.1	18.11	16.59	20.56	17.24
Tare	6.36	9.14	9.07	17.11	13.56
Weight of Dry Sample	8.74	8.97	7.52	3.45	3.68
Weight of Water	3.43	3.64	3.28	0.95	1.02
% Moisture (w)	39.24	40.58	43.62	27.54	27.72
No. of Blows (N)	33	25	16		

Liquid Limit 41
Plastic Limit 28
Plasticity Index 13 **Fines class: ML**

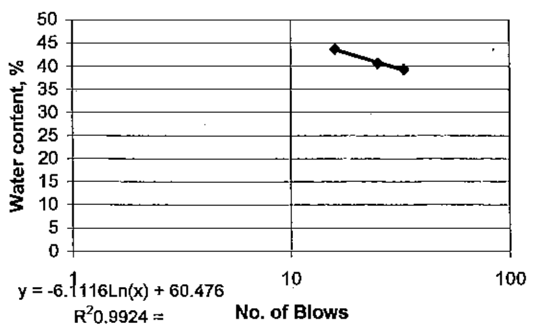
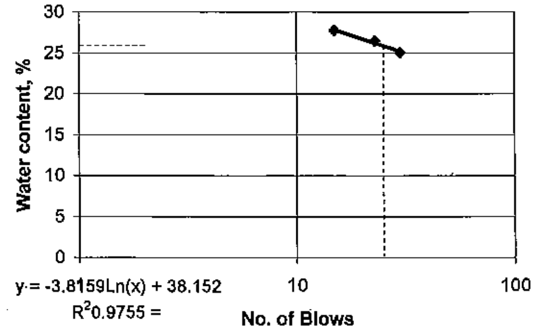


FIG. GD5-B-11
(SHEET 2 OF 3)

ATTERBERG LIMITS Job No. 61081077 Checked by: ____
Sample No. B-8 @ 20.0'

LIQUID LIMIT DETERMINATION	1	2	3	PLASTIC LIMIT 1	2
Can No.	1	2	3	1	2
Wet Weight + Tare	22.3	21.83	20.24	27.12	26.62
Dry Weight + Tare	19.66	19.24	17.9	26.25	25.77
Tare	9.11	9.44	9.46	22.04	21.63
Weight of Dry Sample	10.55	9.8	8.44	4.21	4.14
Weight of Water	2.64	2.59	2.34	0.87	0.85
% Moisture (w)	25.02	26.43	27.73	20.67	20.53
No. of Blows (N)	30	23	15		

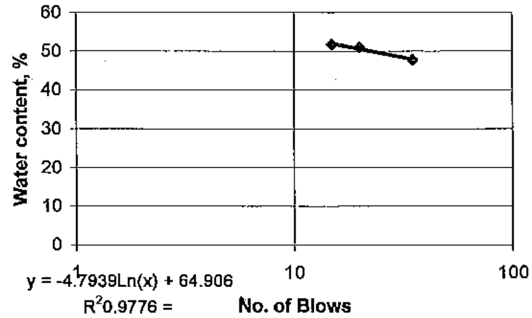
Liquid Limit 25.9
Plastic Limit 21
Plasticity Index 4.87 **Fines class: CL-ML**



ATTERBERG LIMITS Job No. 61081077 Checked by: ____
Sample No. B-8 @ 35.0'

LIQUID LIMIT DETERMINATION	1	2	3	PLASTIC LIMIT 1	2
Can No.	1	2	3	1	2
Wet Weight + Tare	21.46	20.37	22.83	26.08	25.7
Dry Weight + Tare	17.55	16.69	18.24	25.26	25.05
Tare	9.36	9.46	9.36	21.61	22.13
Weight of Dry Sample	8.19	7.23	8.88	3.65	2.92
Weight of Water	3.91	3.68	4.59	0.82	0.65
% Moisture (w)	47.74	50.9	51.69	22.47	22.26
No. of Blows (N)	35	20	15		

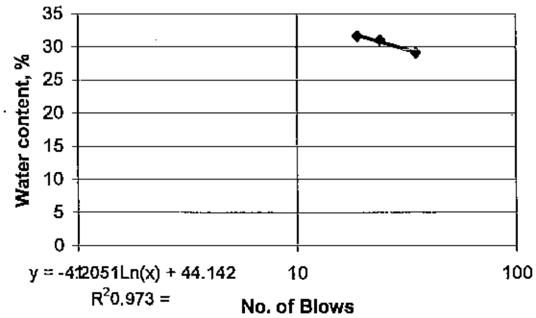
Liquid Limit 49
Plastic Limit 22
Plasticity Index 27 **Fines class: CL**



ATTERBERG LIMITS Job No. 61081077 Checked by: ____
Sample No. B-8 @ 40.0'

LIQUID LIMIT DETERMINATION	1	2	3	PLASTIC LIMIT 1	2
Can No.	1	2	3	1	2
Wet Weight + Tare	22.38	22.88	23.8	24.56	24.66
Dry Weight + Tare	19.45	19.61	20.26	24.06	24.11
Tare	9.38	9.07	9.06	21.84	21.73
Weight of Dry Sample	10.07	10.54	11.2	2.22	2.38
Weight of Water	2.93	3.27	3.54	0.5	0.55
% Moisture (w)	29.1	31.02	31.61	22.52	23.11
No. of Blows (N)	35	24	19		

Liquid Limit 31
Plastic Limit 23
Plasticity Index 8 **Fines class: ML**



ATTERBERG LIMITS Job No. 61081077 Checked by: ____
Sample No.

LIQUID LIMIT DETERMINATION				PLASTIC LIMIT	
Can No.					
Wet Weight + Tare					
Dry Weight + Tare					
Tare					
Weight of Dry Sample					
Weight of Water					
% Moisture (w)					
No. of Blows (N)					

Liquid Limit
Plastic Limit NP
Plasticity Index NP **Fines class: ML**

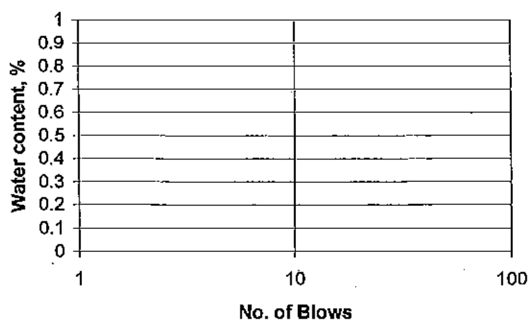


FIG. GD5-B-11
(SHEET 3 OF 3)

Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

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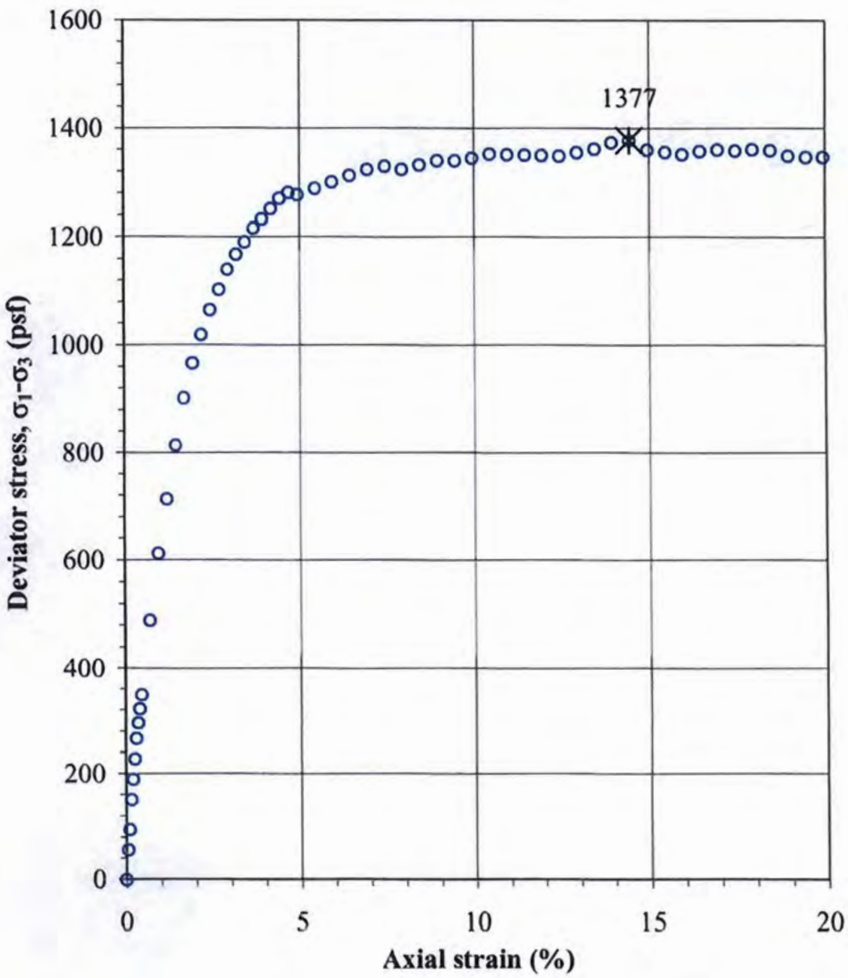
Project: Terracon (61085026)
No: M00385-078
Location: East/ West Connector Lehi
Date: 4/16/2008
By: BRR

Boring No.: B-2
Sample:
Depth: 10
Sample Description: gray clay
Sample type: Undisturbed

Sample height, H (in.)	5.980
Sample diameter, D (in.)	2.840
Sample volume, V (ft^3)	0.0219
Wt. rings + wet soil (g)	1213.05
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1213.05
Moist unit wt., γ_m (pcf)	122.0
Dry unit wt., γ_d (pcf)	**92.8**

Wet soil + tare (g)	559.08
Dry soil + tare (g)	458.65
Tare (g)	139.73
Moisture content, w (%)	**31.5**
Confining stress, σ3 (psf)	706
Shear rate (in/min)	0.0179
Strain at failure, ε_f (%)	14.45
Deviator stress at failure, $(\sigma1\text{-}\sigma3)_f$ (psf)	1377
Shear stress at failure, $q_f = (\sigma1_f\text{-}\sigma3_f)/2$ (psf)	689

Axial Strain (%)	σd σ1-σ3 (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	56.6	28.3
0.10	95.2	47.6
0.15	151.7	75.8
0.20	190.2	95.1
0.25	228.7	114.3
0.30	267.2	133.6
0.35	296.6	148.3
0.40	323.2	161.6
0.45	349.6	174.8
0.70	490.5	245.2
0.95	612.9	306.4
1.20	714.0	357.0
1.45	814.6	407.3
1.70	902.9	451.4
1.95	967.4	483.7
2.20	1019.9	509.9
2.45	1066.2	533.1
2.70	1103.7	551.8
2.95	1140.8	570.4
3.20	1169.1	584.5
3.45	1191.5	595.7
3.70	1216.6	608.3
3.95	1233.0	616.5
4.20	1252.2	626.1
4.45	1271.2	635.6
4.70	1281.5	640.7
4.95	1277.6	638.8
5.45	1289.6	644.8
5.95	1301.3	650.6
6.45	1312.9	656.4
6.95	1324.2	662.1
7.45	1329.8	664.9
7.95	1324.3	662.1
8.45	1332.4	666.2
8.95	1340.3	670.1
9.45	1339.9	669.9
9.95	1344.9	672.4
10.45	1352.3	676.1
10.95	1351.6	675.8
11.45	1350.8	675.4
11.95	1349.9	674.9
12.45	1348.9	674.4
12.95	1355.6	677.8
13.45	1362.2	681.1
13.95	1373.7	686.8
14.45	1377.3	688.6
14.95	1360.4	680.2
15.45	1356.3	678.1
15.95	1352.1	676.0
16.45	1357.8	678.9
16.95	1360.9	680.4
17.45	1358.8	679.4
17.95	1361.6	680.8
18.45	1359.4	679.7
18.95	1349.8	674.9
19.45	1347.4	673.7
19.95	1347.4	673.7
19.97	1344.7	672.3



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Reviewed: DS

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FIG. GD5-B-12

Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

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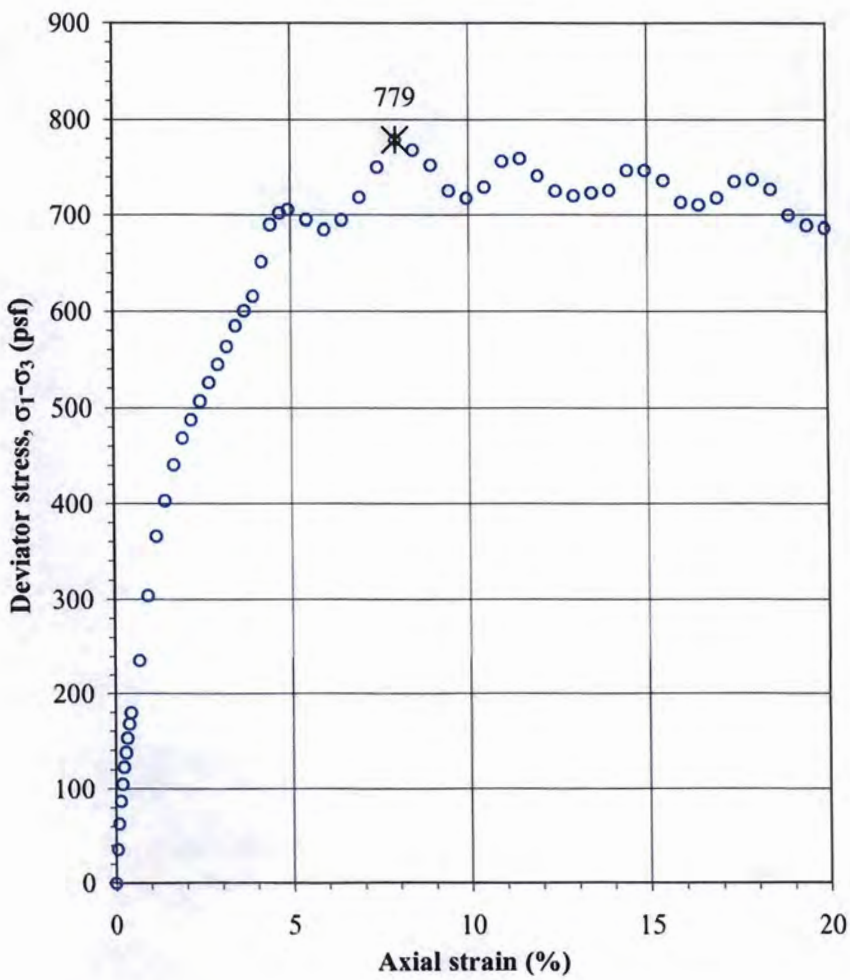
Project: Terracon (61085026)
No: M00385-078
Location: East/ West Connector Lehi
Date: 4/16/2008
By: BRR

Boring No.: B-3
Sample:
Depth: 10
Sample Description: gray clay
Sample type: Undisturbed

Sample height, H (in.)	5.990
Sample diameter, D (in.)	2.820
Sample volume, V (ft^3)	0.0217
Wt. rings + wet soil (g)	1031.70
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1031.70
Moist unit wt., γ_m (pcf)	105.1
Dry unit wt., γ_d (pcf)	**70.4**

Wet soil + tare (g)	350.19
Dry soil + tare (g)	281.32
Tare (g)	141.45
Moisture content, w (%)	**49.2**
Confining stress, $\sigma3$ (psf)	706
Shear rate (in/min)	0.0180
Strain at failure, ε_f (%)	7.95
Deviator stress at failure, $(\sigma1-\sigma3)_f$ (psf)	779
Shear stress at failure, $q_f = (\sigma1_f-\sigma3_f)/2$ (psf)	390

Axial Strain (%)	σd $\sigma1-\sigma3$ (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	36.2	18.1
0.10	63.3	31.7
0.15	87.4	43.7
0.20	105.4	52.7
0.25	123.4	61.7
0.30	138.4	69.2
0.35	153.3	76.6
0.40	168.2	84.1
0.45	180.1	90.0
0.70	236.4	118.2
0.95	304.3	152.2
1.20	366.0	183.0
1.45	403.4	201.7
1.70	440.7	220.4
1.95	468.9	234.4
2.20	488.0	244.0
2.45	507.0	253.5
2.70	526.0	263.0
2.95	544.8	272.4
3.20	563.5	281.8
3.45	585.0	292.5
3.70	600.6	300.3
3.95	616.1	308.1
4.20	651.8	325.9
4.45	690.2	345.1
4.70	702.4	351.2
4.95	705.9	353.0
5.45	695.7	347.9
5.95	685.5	342.8
6.45	695.2	347.6
6.95	718.9	359.4
7.45	750.6	375.3
7.95	779.2	389.6
8.45	768.6	384.3
8.95	752.5	376.3
9.45	725.6	362.8
9.95	718.1	359.0
10.45	729.5	364.7
10.95	756.9	378.5
11.45	759.9	379.9
11.95	741.4	370.7
12.45	725.7	362.9
12.95	720.7	360.4
13.45	723.6	361.8
13.95	726.3	363.2
14.45	747.1	373.6
14.95	747.0	373.5
15.45	736.6	368.3
15.95	713.6	356.8
16.45	710.9	355.5
16.95	718.4	359.2
17.45	735.6	367.8
17.95	737.7	368.8
18.45	727.3	363.7
18.95	699.9	349.9
19.45	689.7	344.9
19.95	686.9	343.5
20.00	686.4	343.2



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Reviewed:

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FIG. GD5-B-13
(SHEET 1 OF 2)

Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

IGES

Project: Terracon (61085026)
No: M00385-078
Location: East/ West Connector Lehi
Date: 4/16/2008
By: BRR

Boring No.: B-3
Sample:
Depth: 20
Sample Description: gray silty clay
Sample type: Undisturbed

Sample height, H (in.)	5.810
Sample diameter, D (in.)	2.820
Sample volume, V (ft^3)	0.0210
Wt. rings + wet soil (g)	1131.12
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1131.12
Moist unit wt., γ_m (pcf)	118.7
Dry unit wt., γ_d (pcf)	**87.8**

Wet soil + tare (g)	407.99
Dry soil + tare (g)	341.93
Tare (g)	154.30
Moisture content, w (%)	**35.2**
Confining stress, σ3 (psf)	1397
Shear rate (in/min)	0.0174
Strain at failure, ε_f (%)	10.95
Deviator stress at failure, $(\sigma1-\sigma3)_f$ (psf)	1346
Shear stress at failure, $q_f = (\sigma1_f-\sigma3_f)/2$ (psf)	673

Axial Strain (%)	σd σ1-σ3 (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	48.3	24.2
0.10	84.5	42.2
0.15	126.7	63.3
0.20	162.8	81.4
0.25	195.8	97.9
0.30	231.9	115.9
0.35	258.8	129.4
0.40	294.8	147.4
0.45	330.7	165.3
0.71	467.6	233.8
0.95	574.0	287.0
1.20	655.9	327.9
1.45	731.4	365.7
1.70	791.6	395.8
1.95	839.6	419.8
2.20	887.4	443.7
2.45	923.1	461.5
2.70	961.5	480.7
2.95	990.9	495.4
3.20	1026.0	513.0
3.45	1060.9	530.4
3.70	1095.7	547.8
3.95	1130.2	565.1
4.20	1167.4	583.7
4.45	1192.9	596.4
4.70	1209.5	604.7
4.95	1217.4	608.7
5.45	1238.7	619.3
5.95	1265.5	632.7
6.45	1272.0	636.0
6.95	1292.5	646.2
7.45	1315.5	657.7
7.95	1321.5	660.7
8.45	1321.7	660.8
8.95	1324.6	662.3
9.45	1324.6	662.3
9.95	1321.9	660.9
10.45	1329.9	664.9
10.95	1345.8	672.9
11.45	1342.7	671.3
11.95	1334.2	667.1
12.45	1331.0	665.5
12.95	1327.8	663.9
13.45	1327.0	663.5
13.95	1321.0	660.5
14.45	1320.2	660.1
14.95	1311.5	655.7
15.45	1297.7	648.8
15.95	1294.1	647.0
16.45	1285.5	642.7
16.95	1271.8	635.9
17.45	1245.6	622.8
17.95	1229.7	614.8
18.45	1221.2	610.6
18.95	1207.8	603.9
19.45	1201.8	600.9
19.95	1193.3	596.6
19.98	1195.2	597.6

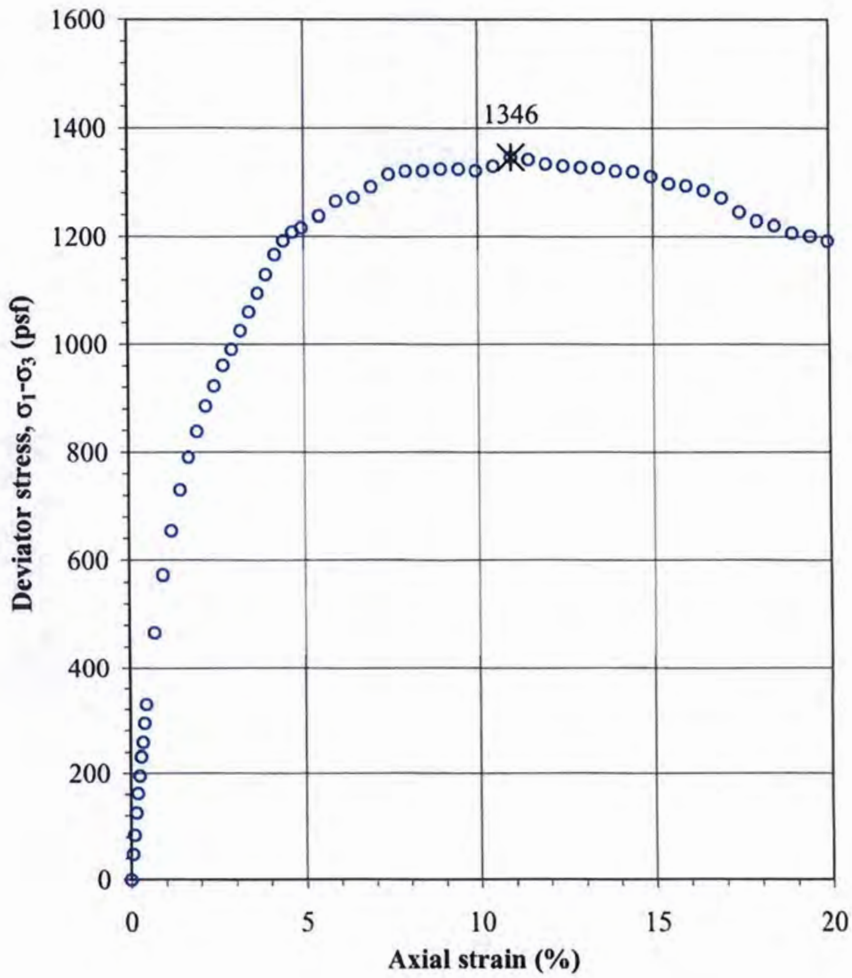

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FIG. GD5-B-13
(SHEET 2 OF 2)

Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

IGES

(ASTM D2850)

Project: Terracon (61085026)
No: M00385-078
Location: East/ West Connector Lehi
Date: 4/16/2008
By: BRR

Boring No.: B-5
Sample:
Depth: 7.5
Sample Description: gray clay w/ gravelly sand
Sample type: Undisturbed

Sample height, H (in.)	5.970
Sample diameter, D (in.)	2.870
Sample volume, V (ft^3)	0.0224
Wt. rings + wet soil (g)	1397.41
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1397.41
Moist unit wt., γ_m (pcf)	137.8
Dry unit wt., γ_d (pcf)	**119.2**

Wet soil + tare (g)	824.57
Dry soil + tare (g)	732.05
Tare (g)	140.89
Moisture content, w (%)	**15.7**
Confining stress, σ3 (psf)	605
Shear rate (in/min)	0.0179
Strain at failure, ε_f (%)	19.93
Deviator stress at failure, $(\sigma1-\sigma3)_f$ (psf)	2406
Shear stress at failure, $q_f = (\sigma1_f-\sigma3_f)/2$ (psf)	1203

Axial Strain (%)	σd σ1-σ3 (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	32.1	16.0
0.10	61.2	30.6
0.15	93.2	46.6
0.20	122.3	61.1
0.25	66.8	33.4
0.30	107.5	53.7
0.35	66.6	33.3
0.40	98.5	49.3
0.45	51.9	26.0
0.70	237.2	118.6
0.95	326.1	163.0
1.20	397.2	198.6
1.45	467.9	233.9
1.70	526.8	263.4
1.95	579.6	289.8
2.20	632.2	316.1
2.45	693.0	346.5
2.70	742.1	371.1
2.95	796.7	398.3
3.20	862.2	431.1
3.45	935.9	468.0
3.70	1012.0	506.0
3.95	1070.8	535.4
4.20	1132.2	566.1
4.45	1137.3	568.7
4.70	1209.3	604.7
4.95	1255.8	627.9
5.45	1342.6	671.3
5.95	1420.2	710.1
6.45	1480.4	740.2
6.95	1548.1	774.1
7.45	1625.7	812.9
7.95	1697.1	848.6
8.45	1756.9	878.5
8.95	1810.5	905.3
9.45	1821.2	910.6
9.95	1847.5	923.8
10.45	1875.8	937.9
10.95	1916.9	958.5
11.45	1962.5	981.3
11.95	2002.2	1001.1
12.45	2031.2	1015.6
12.95	2049.6	1024.8
13.45	2054.9	1027.5
13.95	2080.1	1040.1
14.45	2120.0	1060.0
14.95	2149.2	1074.6
15.45	2182.9	1091.5
15.95	2213.6	1106.8
16.45	2221.8	1110.9
16.95	2237.0	1118.5
17.45	2254.3	1127.2
17.95	2292.8	1146.4
18.45	2335.3	1167.7
18.95	2367.8	1183.9
19.45	2387.9	1194.0
19.93	2405.7	1202.9

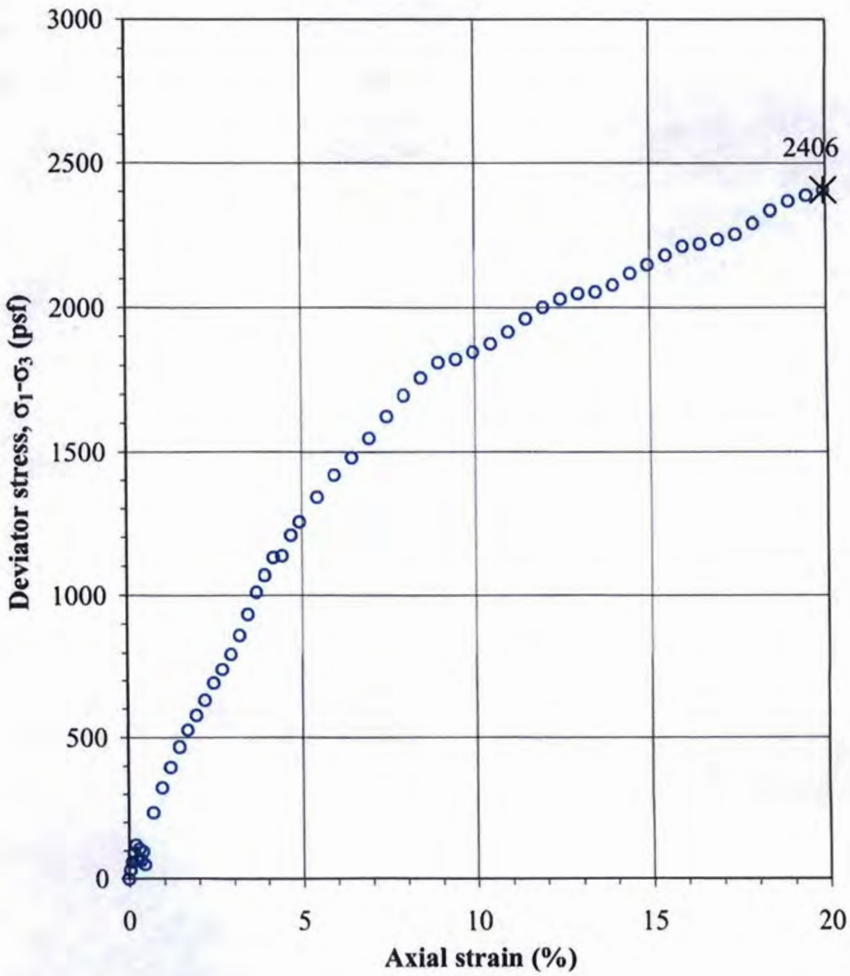

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FIG. GD5-B-14

Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

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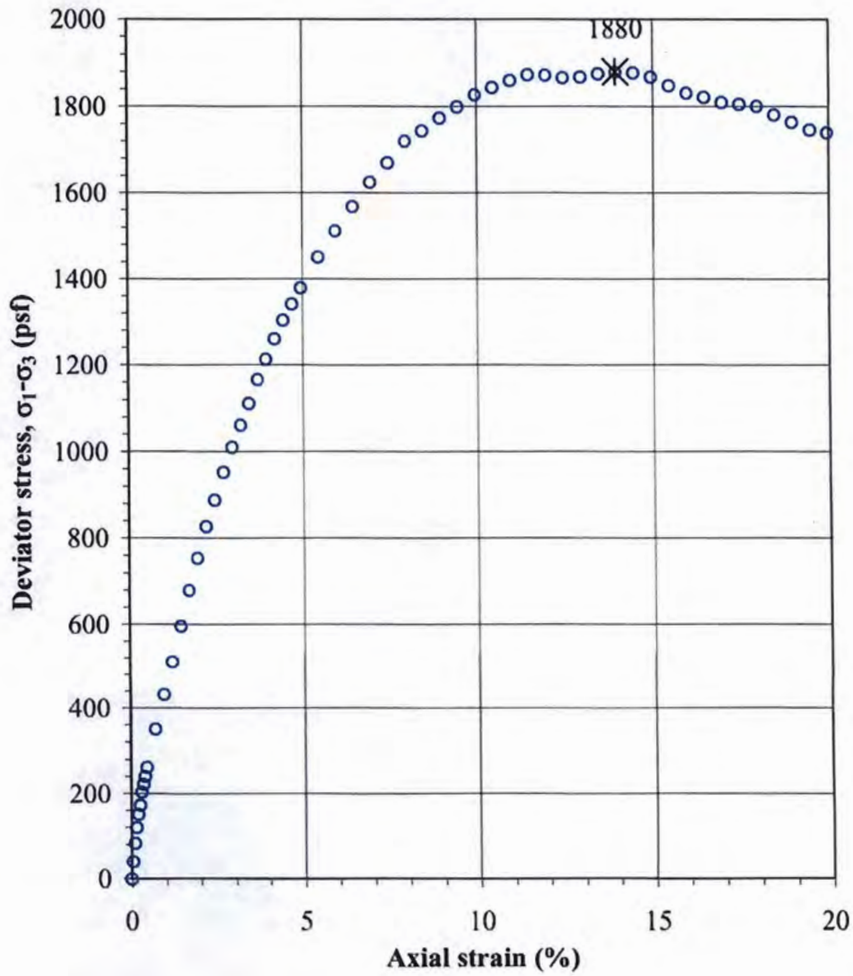
Project: Terracon (61075055)
No: M00385-079
Location: EW Connector
Date: 4/16/2008
By: BRR

Boring No.: B-8
Sample:
Depth: 10
Sample Description: gray silty clay
Sample type: Undisturbed

Sample height, H (in.)	6.000
Sample diameter, D (in.)	2.800
Sample volume, V (ft^3)	0.0214
Wt. rings + wet soil (g)	1109.26
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1109.26
Moist unit wt., γ_m (pcf)	114.4
Dry unit wt., γ_d (pcf)	**83.6**

Wet soil + tare (g)	423.55
Dry soil + tare (g)	358.01
Tare (g)	179.70
Moisture content, w (%)	**36.8**
Confining stress, σ3 (psf)	706
Shear rate (in/min)	0.0180
Strain at failure, ε_f (%)	13.95
Deviator stress at failure, $(\sigma1-\sigma3)_f$ (psf)	1880
Shear stress at failure, $q_f = (\sigma1_f-\sigma3_f)/2$ (psf)	940

Axial Strain (%)	σd σ1-σ3 (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	40.4	20.2
0.10	83.9	41.9
0.15	121.1	60.5
0.20	152.0	76.0
0.25	173.6	86.8
0.30	204.5	102.3
0.35	223.0	111.5
0.40	241.4	120.7
0.45	262.9	131.4
0.70	351.5	175.7
0.95	433.5	216.7
1.20	512.0	256.0
1.45	596.2	298.1
1.70	680.0	340.0
1.95	754.1	377.1
2.20	827.9	414.0
2.45	889.1	444.6
2.70	953.1	476.6
2.95	1010.6	505.3
3.20	1061.8	530.9
3.45	1112.8	556.4
3.70	1166.4	583.2
3.95	1213.8	606.9
4.20	1260.9	630.5
4.45	1304.7	652.4
4.70	1342.4	671.2
4.95	1379.8	689.9
5.45	1451.1	725.6
5.95	1512.7	756.4
6.45	1567.8	783.9
6.95	1625.1	812.6
7.45	1670.1	835.1
7.95	1720.3	860.2
8.45	1744.1	872.1
8.95	1773.2	886.6
9.45	1799.1	899.6
9.95	1827.4	913.7
10.45	1844.1	922.1
10.95	1860.5	930.3
11.45	1873.8	936.9
11.95	1873.0	936.5
12.45	1866.8	933.4
12.95	1868.5	934.3
13.45	1875.6	937.8
13.95	1879.7	939.9
14.45	1878.3	939.2
14.95	1868.8	934.4
15.45	1848.7	924.4
15.95	1831.4	915.7
16.45	1822.0	911.0
16.95	1809.9	905.0
17.45	1805.5	902.8
17.95	1801.1	900.6
18.45	1781.2	890.6
18.95	1764.1	882.1
19.45	1747.0	873.5
19.95	1739.9	870.0
20.02	1738.2	869.1



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Reviewed:

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FIG. GD5-B-15

(SHEET 1 OF 3)

Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

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Project: Terracon (61075055)
No: M00385-079
Location: EW Connector
Date: 4/16/2008
By: BRR

Boring No.: B-8
Sample:
Depth: 30
Sample Description: brown clay
Sample type: Undisturbed

Sample height, H (in.)	5.980
Sample diameter, D (in.)	2.840
Sample volume, V (ft^3)	0.0219
Wt. rings + wet soil (g)	1256.48
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1256.48
Moist unit wt., γ_m (pcf)	126.4
Dry unit wt., γ_d (pcf)	**99.5**

Wet soil + tare (g)	445.16
Dry soil + tare (g)	395.31
Tare (g)	210.28
Moisture content, w (%)	**26.9**
Confining stress, σ3 (psf)	1699
Shear rate (in/min)	0.0179
Strain at failure, ε_f (%)	9.95
Deviator stress at failure, $(\sigma1\text{-}\sigma3)_f$ (psf)	1173
Shear stress at failure, $q_f = (\sigma1_f\text{-}\sigma3_f)/2$ (psf)	587

Axial Strain (%)	σd σ1-σ3 (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	63.5	31.7
0.10	120.9	60.4
0.15	169.1	84.5
0.20	205.2	102.6
0.25	232.2	116.1
0.30	253.1	126.5
0.35	274.0	137.0
0.40	291.8	145.9
0.45	303.7	151.8
0.70	371.5	185.8
0.95	430.1	215.0
1.20	485.3	242.7
1.45	534.3	267.1
1.70	574.1	287.0
1.95	613.6	306.8
2.20	650.0	325.0
2.45	683.3	341.6
2.70	713.4	356.7
2.95	746.3	373.1
3.20	779.0	389.5
3.45	805.6	402.8
3.70	835.1	417.6
3.95	864.3	432.2
4.20	890.5	445.3
4.45	910.8	455.4
4.70	933.8	466.9
4.95	956.7	478.4
5.45	990.6	495.3
5.95	1021.3	510.7
6.45	1054.3	527.2
6.95	1084.2	542.1
7.45	1105.2	552.6
7.95	1128.7	564.4
8.45	1149.1	574.6
8.95	1160.9	580.5
9.45	1172.6	586.3
9.95	1173.1	586.6
10.45	1170.8	585.4
10.95	1160.3	580.2
11.45	1152.5	576.3
11.95	1131.4	565.7
12.45	1107.8	553.9
12.95	1076.5	538.3
13.45	1029.7	514.9
13.95	959.9	480.0
14.45	919.3	459.7
14.95	881.8	440.9
15.45	854.8	427.4
15.95	840.8	420.4
16.45	837.0	418.5
16.95	830.7	415.4
17.45	819.3	409.7
17.95	803.1	401.6
18.45	787.0	393.5
18.95	773.4	386.7
19.45	769.7	384.9
19.95	773.3	386.6
20.17	770.5	385.2

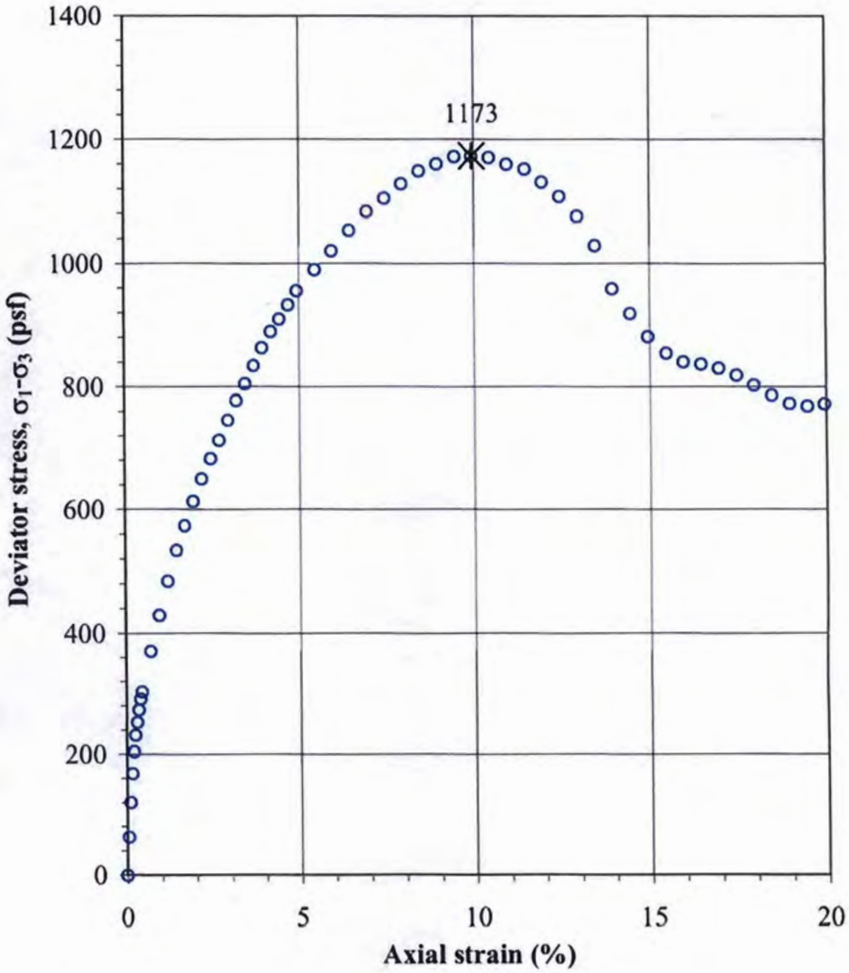

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FIG. GD5-B-15
(SHEET 2 OF 3)

Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

IGES

Project: Terracon (61075055)
No: M00385-079
Location: EW Connector
Date: 4/16/2008
By: BRR

Boring No.: B-8
Sample:
Depth: 50
Sample Description: tan clay
Sample type: Undisturbed

Sample height, H (in.)	5.990
Sample diameter, D (in.)	2.870
Sample volume, V (ft^3)	0.0224
Wt. rings + wet soil (g)	1260.63
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1260.63
Moist unit wt., γ_m (pcf)	123.9
Dry unit wt., γ_d (pcf)	**98.6**

Wet soil + tare (g)	424.23
Dry soil + tare (g)	380.79
Tare (g)	211.67
Moisture content, w (%)	**25.7**
Confining stress, σ3 (psf)	2779
Shear rate (in/min)	0.0180
Strain at failure, ε_f (%)	18.95
Deviator stress at failure, $(\sigma1\text{-}\sigma3)_f$ (psf)	2658
Shear stress at failure, $q_f = (\sigma1_f\text{-}\sigma3_f)/2$ (psf)	1329

Axial Strain (%)	σd σ1-σ3 (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	23.7	11.8
0.10	50.3	25.1
0.15	73.9	36.9
0.20	100.4	50.2
0.25	124.0	62.0
0.30	153.5	76.7
0.35	179.9	90.0
0.40	212.3	106.1
0.45	238.7	119.4
0.70	367.4	183.7
0.95	492.6	246.3
1.20	611.2	305.6
1.45	723.4	361.7
1.70	835.0	417.5
1.95	934.4	467.2
2.20	1027.5	513.7
2.45	1117.1	558.5
2.70	1206.4	603.2
2.95	1286.5	643.2
3.20	1363.3	681.6
3.45	1431.0	715.5
3.70	1498.4	749.2
3.95	1559.8	779.9
4.20	1620.8	810.4
4.45	1678.6	839.3
4.70	1733.2	866.6
4.95	1787.6	893.8
5.45	1889.8	944.9
5.95	1985.2	992.6
6.45	2065.6	1032.8
6.95	2125.7	1062.8
7.45	2182.2	1091.1
7.95	2232.6	1116.3
8.45	2285.0	1142.5
8.95	2336.7	1168.3
9.45	2382.3	1191.1
9.95	2419.3	1209.6
10.45	2442.3	1221.1
10.95	2459.7	1229.8
11.45	2484.6	1242.3
11.95	2511.7	1255.8
12.45	2543.5	1271.7
12.95	2569.7	1284.8
13.45	2585.0	1292.5
13.95	2587.2	1293.6
14.45	2604.5	1302.2
14.95	2618.8	1309.4
15.45	2630.3	1315.1
15.95	2646.5	1323.2
16.45	2652.3	1326.1
16.95	2653.0	1326.5
17.45	2651.0	1325.5
17.95	2651.2	1325.6
18.45	2653.7	1326.8
18.95	2658.3	1329.1
19.45	2657.9	1328.9
19.95	2652.5	1326.2
20.11	2651.7	1325.8

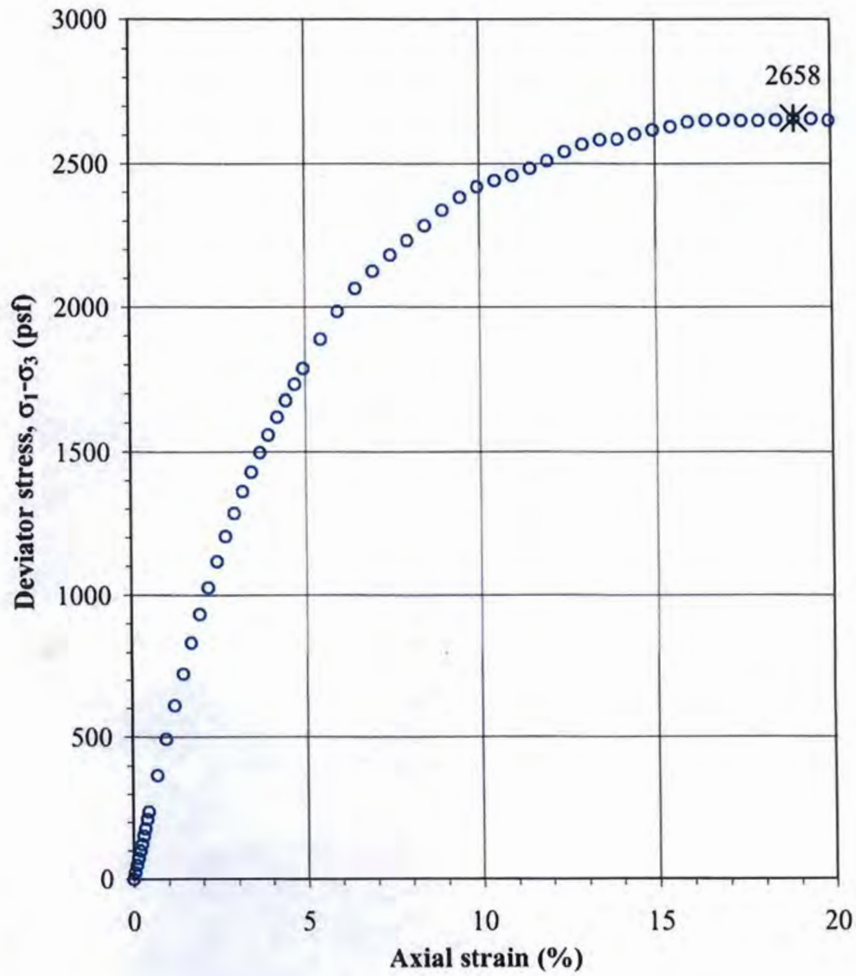

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FIG. GD5-B-15
(SHEET 3 OF 3)

Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

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Project: Terracon
No: M00385-080 (61085026)
Location: E.W. Connector
Date: 5/5/2008
By: BRR

Boring No.: B-9
Sample:
Depth: 5
Sample Description: dark brown clay w/ sand & gravel
Sample type: Undisturbed

Sample height, H (in.)	5.980
Sample diameter, D (in.)	2.880
Sample volume, V (ft^3)	0.0225
Wt. rings + wet soil (g)	1313.88
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1313.88
Moist unit wt., γ_m (pcf)	128.5
Dry unit wt., γ_d (pcf)	**107.1**

Wet soil + tare (g)	856.83
Dry soil + tare (g)	739.35
Tare (g)	150.62
Moisture content, w (%)	**20.0**
Confining stress, σ3 (psf)	605
Shear rate (in/min)	0.0179
Strain at failure, ε_f (%)	11.95
Deviator stress at failure, $(\sigma1-\sigma3)_f$ (psf)	1706
Shear stress at failure, $q_f = (\sigma1_f - \sigma3_f)/2$ (psf)	853

Axial Strain (%)	σd σ1-σ3 (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	118.8	59.4
0.10	191.2	95.6
0.15	243.2	121.6
0.20	292.2	146.1
0.25	338.3	169.2
0.30	369.9	185.0
0.35	413.0	206.5
0.40	444.6	222.3
0.45	470.3	235.1
0.70	584.0	292.0
0.95	688.6	344.3
1.20	786.8	393.4
1.45	890.3	445.2
1.70	981.8	490.9
1.95	1055.8	527.9
2.20	1123.7	561.9
2.45	1163.0	581.5
2.70	1190.7	595.4
2.95	1221.1	610.6
3.20	1251.4	625.7
3.45	1281.4	640.7
3.70	1319.7	659.9
3.95	1363.3	681.7
4.20	1398.3	699.2
4.45	1435.9	718.0
4.70	1487.1	743.6
4.95	1518.7	759.4
5.45	1534.8	767.4
5.95	1545.1	772.6
6.45	1544.3	772.2
6.95	1562.3	781.2
7.45	1601.6	800.8
7.95	1645.6	822.8
8.45	1689.1	844.6
8.95	1684.5	842.3
9.45	1677.1	838.6
9.95	1664.5	832.3
10.45	1680.5	840.3
10.95	1696.2	848.1
11.45	1698.8	849.4
11.95	1706.4	853.2
12.45	1701.1	850.6
12.95	1683.0	841.5
13.45	1670.1	835.1
13.95	1654.7	827.4
14.45	1661.7	830.9
14.95	1693.2	846.6
15.45	1702.1	851.1
15.95	1691.2	845.6
16.45	1673.1	836.6
16.95	1655.1	827.6
17.45	1656.3	828.2
17.95	1676.5	838.3
18.45	1686.8	843.4
18.95	1685.0	842.5
19.45	1669.1	834.6
19.95	1655.6	827.8
20.00	1652.3	826.2

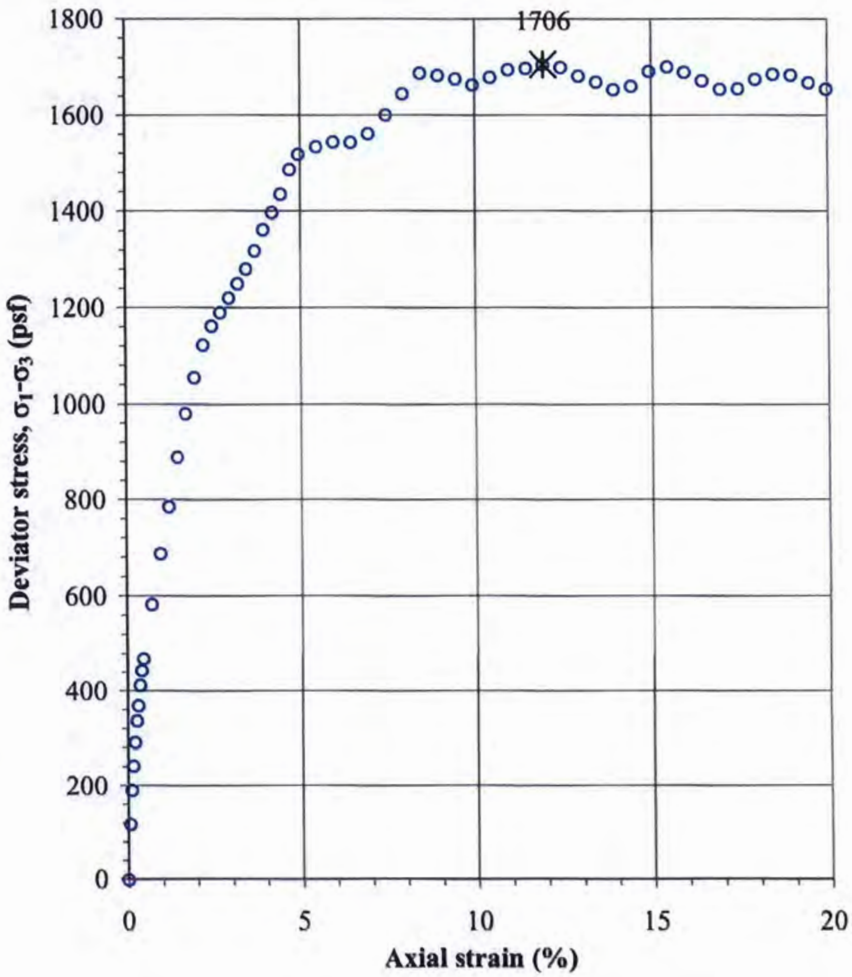

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FIG. GD5-B-16
(SHEET 1 OF 2)

Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

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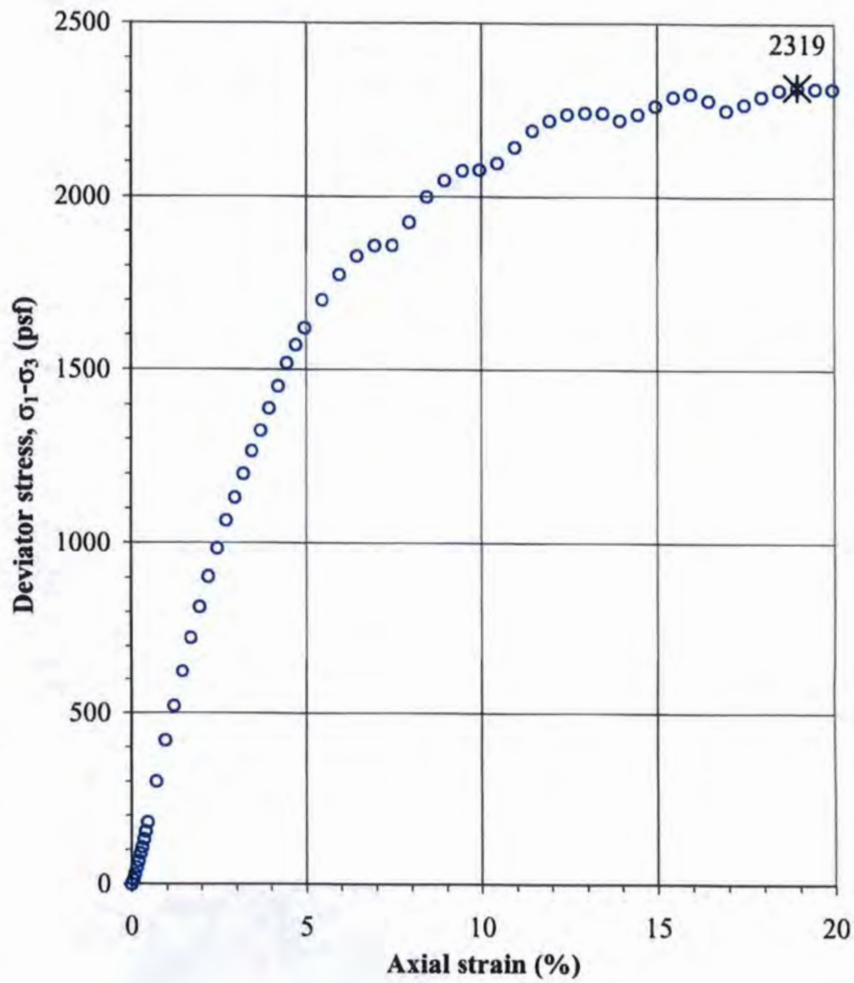
Project: Terracon
No: M00385-080 (61085026)
Location: E.W. Connector
Date: 5/5/2008
By: BRR

Boring No.: B-9
Sample:
Depth: 15
Sample Description:
Sample type: Undisturbed

Sample height, H (in.)	5.990
Sample diameter, D (in.)	2.850
Sample volume, V (ft^3)	0.0221
Wt. rings + wet soil (g)	1236.21
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1236.21
Moist unit wt., γ_m (pcf)	123.2
Dry unit wt., γ_d (pcf)	**95.5**

Wet soil + tare (g)	769.19
Dry soil + tare (g)	627.90
Tare (g)	140.59
Moisture content, w (%)	**29.0**
Confining stress, σ3 (psf)	1699
Shear rate (in/min)	0.0180
Strain at failure, ε_f (%)	18.95
Deviator stress at failure, $(\sigma 1-\sigma 3)_f$ (psf)	2319
Shear stress at failure, $q_f = (\sigma 1_f - \sigma 3_f)/2$ (psf)	1160

Axial Strain (%)	σd σ1-σ3 (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	11.8	5.9
0.10	29.5	14.8
0.15	50.2	25.1
0.20	67.9	33.9
0.25	91.5	45.7
0.30	109.1	54.5
0.35	132.6	66.3
0.40	156.1	78.1
0.45	182.6	91.3
0.70	302.6	151.3
0.95	422.0	211.0
1.20	523.2	261.6
1.45	626.8	313.4
1.70	727.0	363.5
1.95	818.0	409.0
2.20	905.5	452.8
2.45	986.8	493.4
2.70	1067.7	533.9
2.95	1133.8	566.9
3.20	1202.5	601.3
3.45	1267.8	633.9
3.70	1327.1	663.6
3.95	1391.8	695.9
4.20	1453.3	726.7
4.45	1520.1	760.1
4.70	1572.4	786.2
4.95	1621.6	810.8
5.45	1702.4	851.2
5.95	1776.7	888.4
6.45	1830.6	915.3
6.95	1861.8	930.9
7.45	1862.4	931.2
7.95	1928.3	964.2
8.45	2001.6	1000.8
8.95	2049.7	1024.9
9.45	2078.2	1039.1
9.95	2079.7	1039.9
10.45	2099.6	1049.8
10.95	2145.6	1072.8
11.45	2193.5	1096.8
11.95	2222.5	1111.3
12.45	2240.6	1120.3
12.95	2245.5	1122.8
13.45	2245.0	1122.5
13.95	2224.0	1112.0
14.45	2241.1	1120.6
14.95	2265.4	1132.7
15.45	2291.8	1145.9
15.95	2300.2	1150.1
16.45	2281.2	1140.6
16.95	2252.4	1126.2
17.45	2270.2	1135.1
17.95	2292.5	1146.3
18.45	2311.9	1156.0
18.95	2319.0	1159.5
19.45	2316.2	1158.1
19.95	2315.6	1157.8
19.97	2310.4	1155.2



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Reviewed: LOS

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FIG. GD5-B-16
(SHEET 2 OF 2)

APPENDIX GD5-C

STABILITY ANALYSES

APPENDIX GD5-C

STABILITY ANALYSES

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Figure No.

LIST OF FIGURES (cont.)

Figure No.

APPENDIX GD5-C

STABILITY ANALYSES

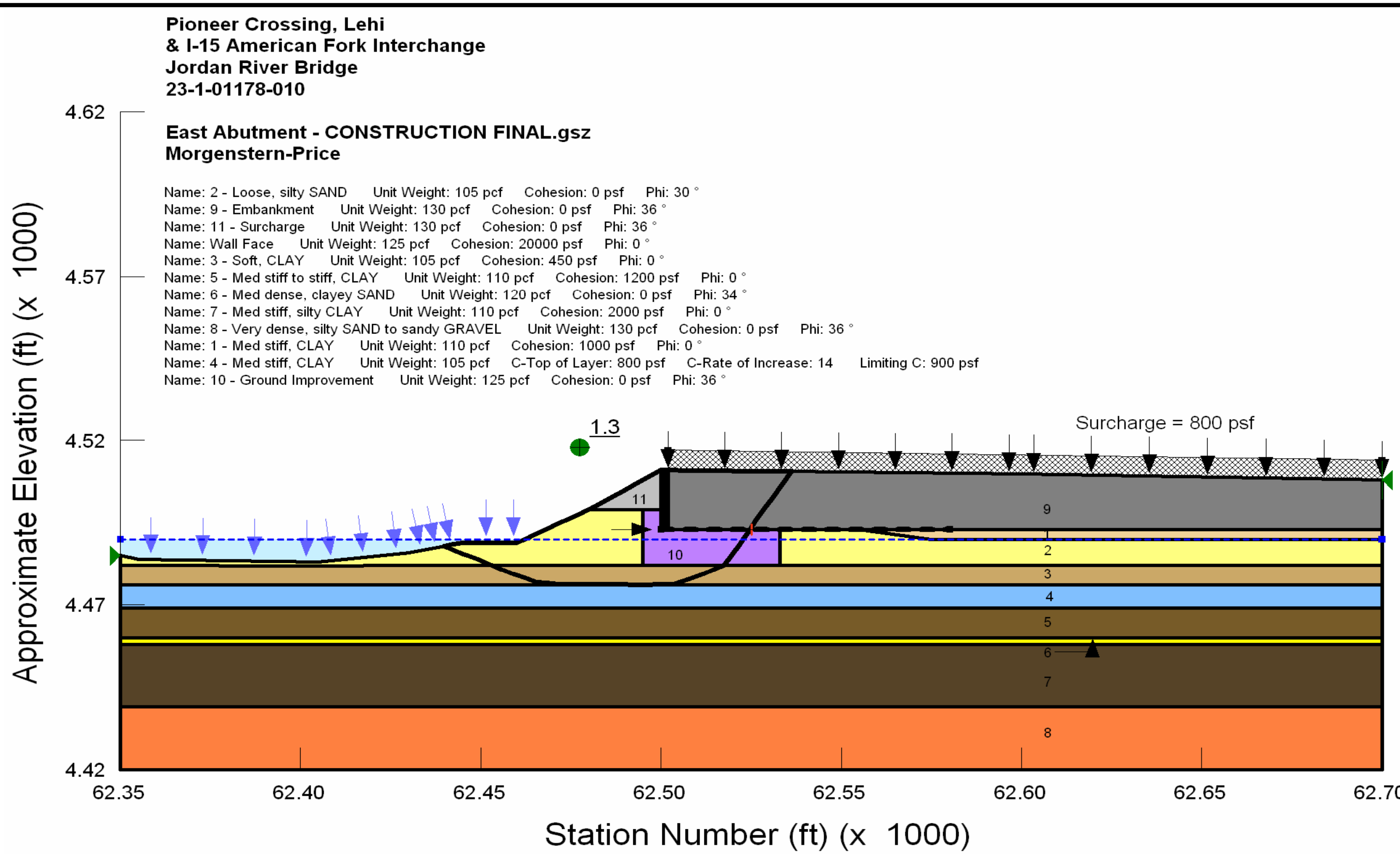
GD5-C.1 INTRODUCTION

This appendix presents the results from the global slope stability analyses (Shannon & Wilson, 2009). Please see the calculation package for analysis details.

At the east and west abutment, Layer 8 is modeled as a very dense, sand/gravel unit and is a combination of two separate layers in the soil profile. For the stability analyses, this layer was modeled with a conservative shear strength value that differs from the strength value used in other portions of report. Using a conservative value for this unit does not affect the results of the analyses since none of the failure surfaces intersect this layer.

GD5-C.2 REFERENCE

Shannon & Wilson, Inc., 2009, Geotechnical calculation GC-02, Pioneer Crossing, Lehi and I-15 American Fork Interchange, Utah, Global Stability Analyses: Calculation prepared by Shannon & Wilson, Inc., Seattle, Wash., 23-1-01178-010, for Kiewit/Clyde, in progress.



Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

GLOBAL STABILITY PROFILE
EAST ABUTMENT
STATIC - END OF CONSTRUCTION

February 2009 23-1-01178-010

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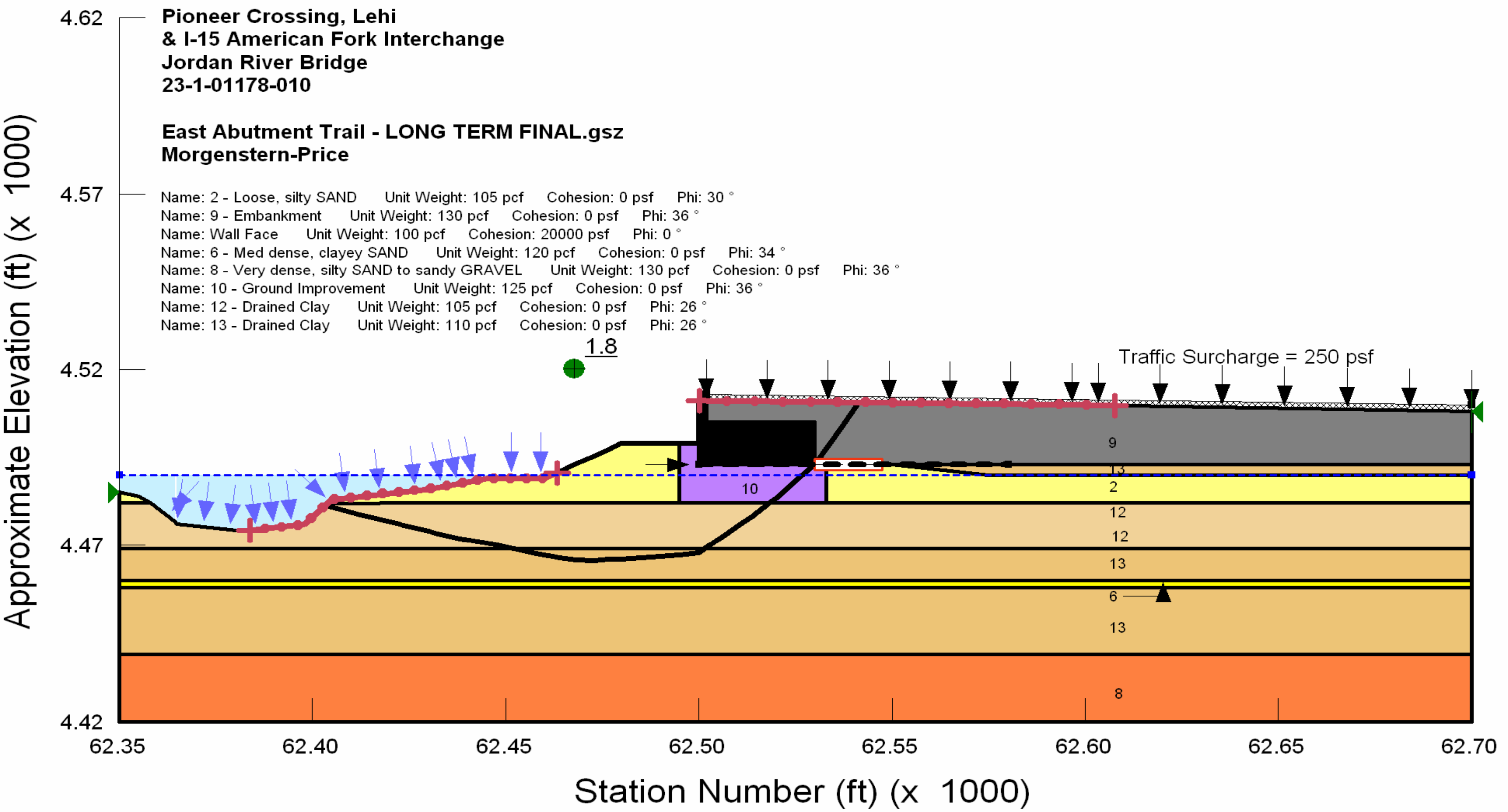
FIG. GD5-C-1

FIG. GD5-C-1

Stability Figures.xls - 2/25/2009



Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

GLOBAL STABILITY PROFILE
EAST ABUTMENT
STATIC -LONG TERM

February 2009 23-1-01178-010

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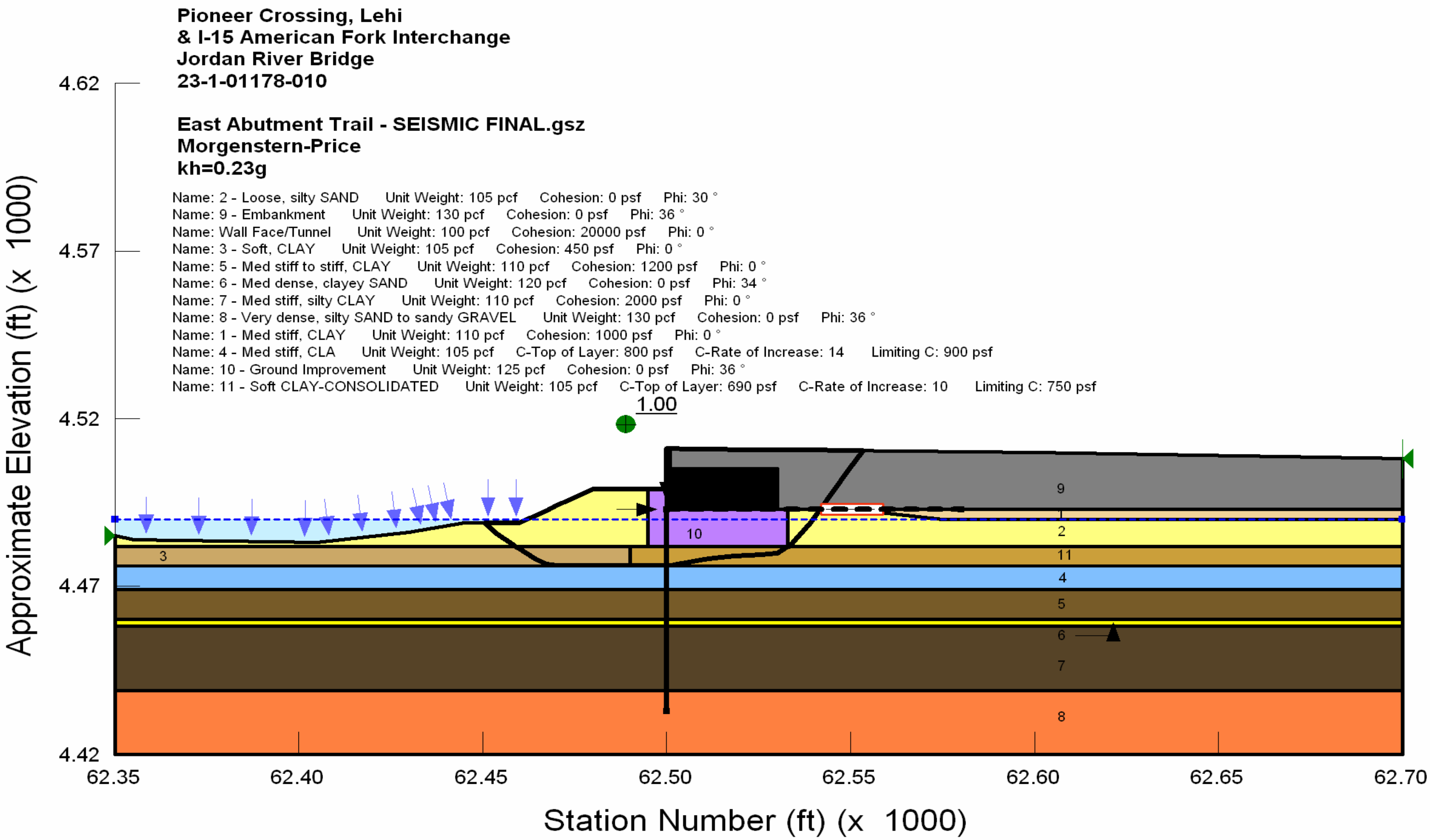
FIG. GD5-C-2

FIG. GD5-C-2



Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

GLOBAL STABILITY PROFILE
EAST ABUTMENT
SEISMIC

February 2009 | 23-1-01178-010

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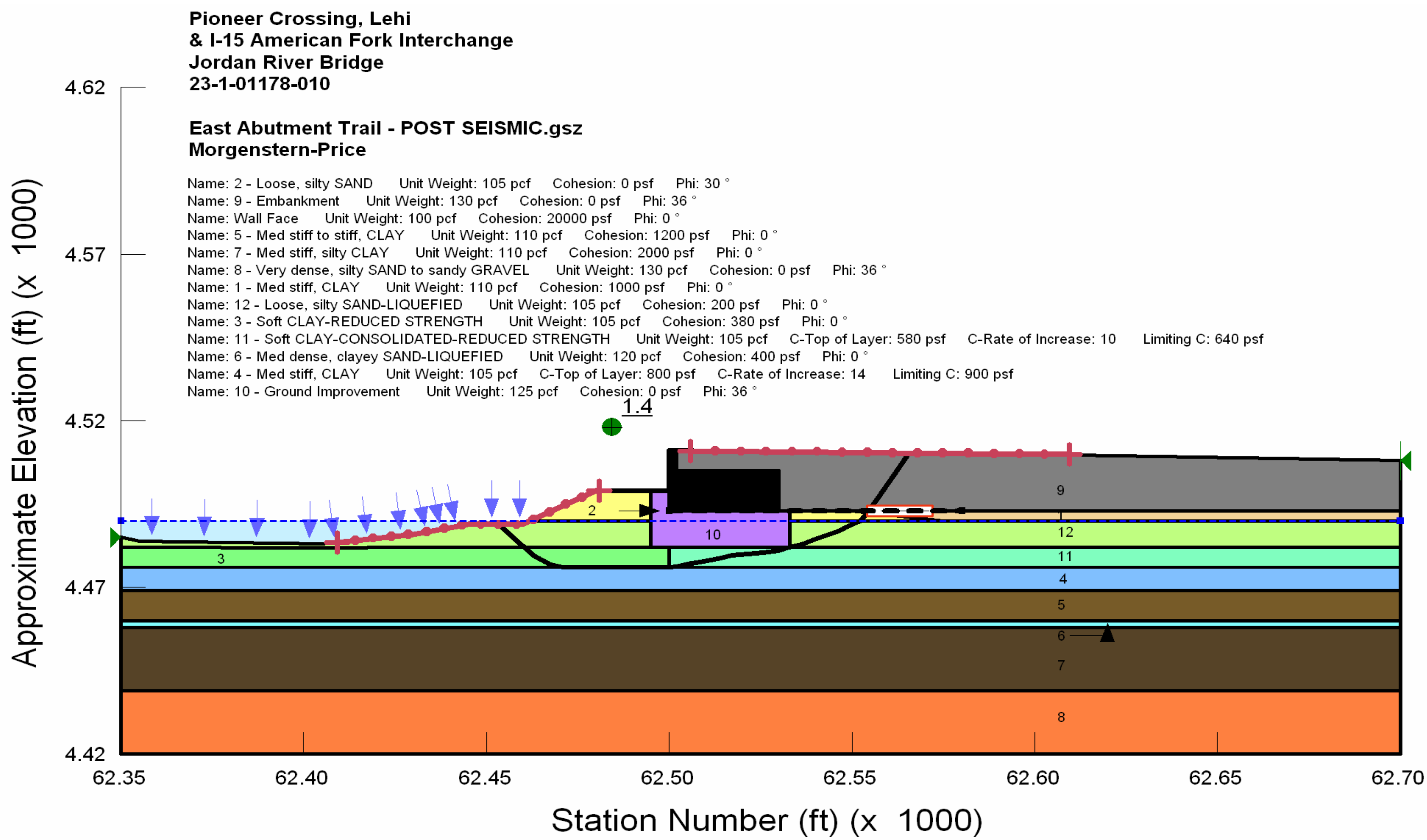
FIG. GD5-C-3

FIG. GD5-C-3

Stability Figures.xls - 2/25/2009



Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

GLOBAL STABILITY PROFILE
EAST ABUTMENT
POST SEISMIC (LIQUEFIED)

February 2009 23-1-01178-010

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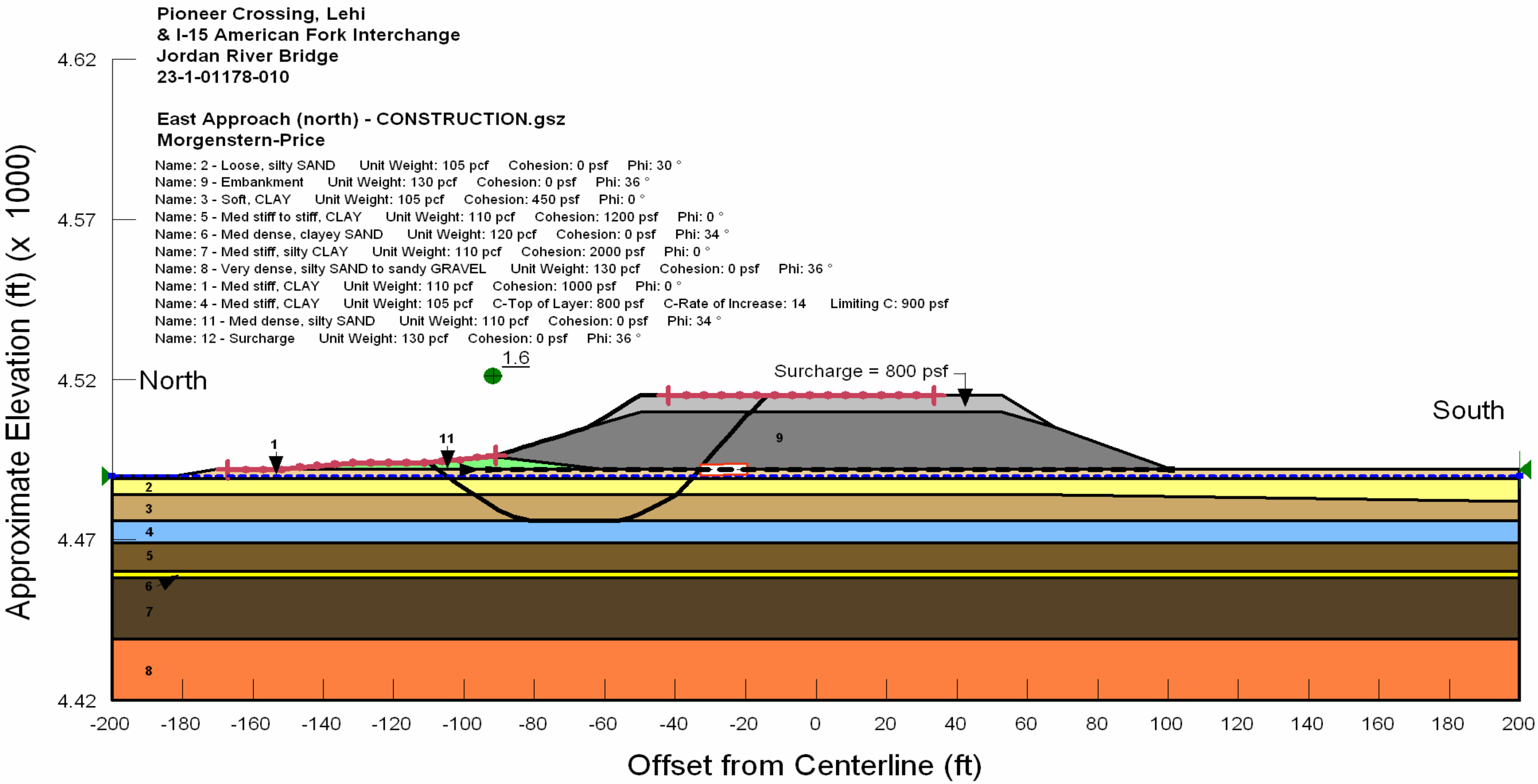
FIG. GD5-C-4

FIG. GD5-C-4



Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

GLOBAL STABILITY PROFILE
EAST APPROACH (NORTH)
STATIC - END OF CONSTRUCTION

February 2009 23-1-01178-010

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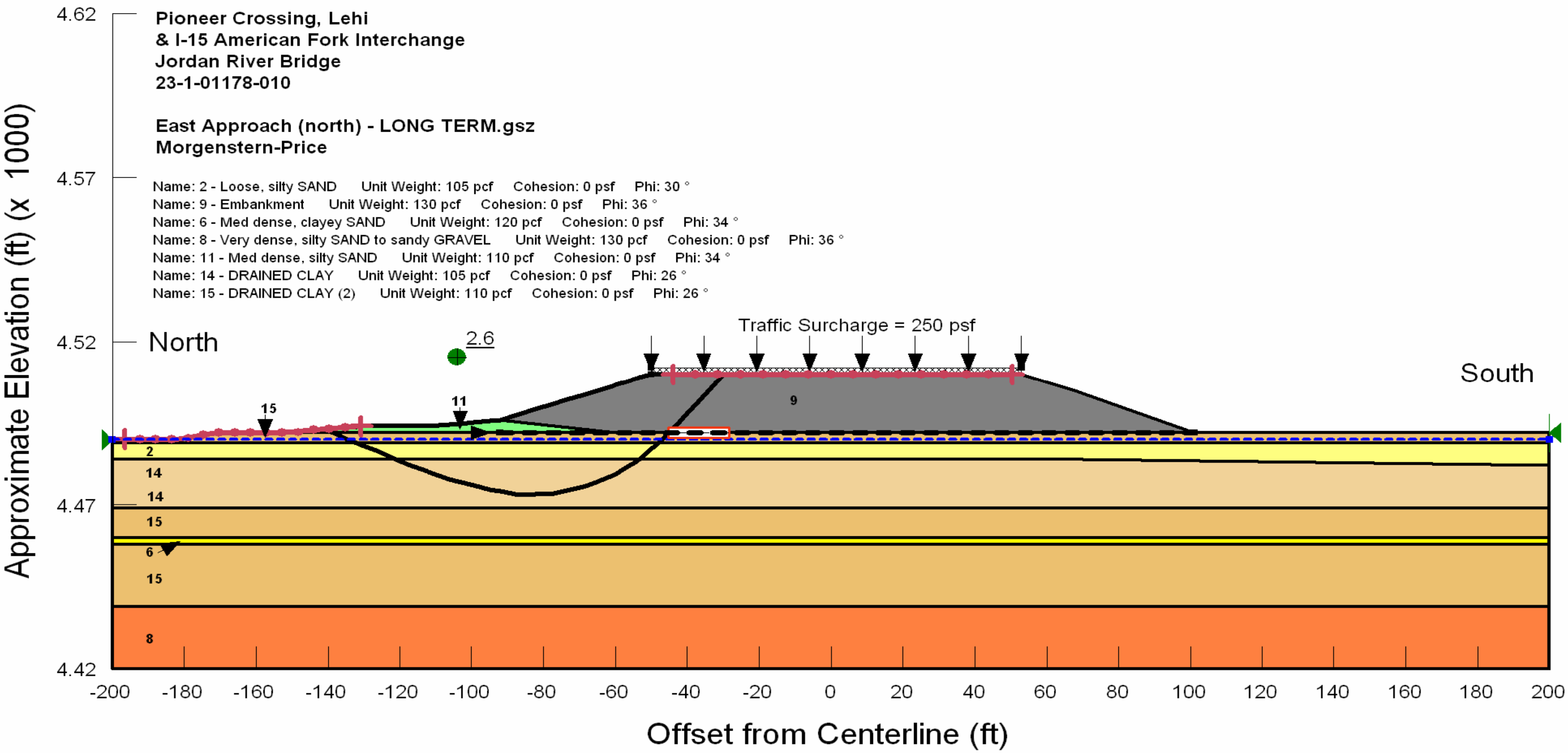
FIG. GD5-C-5

FIG. GD5-C-5



Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

GLOBAL STABILITY PROFILE
EAST APPROACH (NORTH)
STATIC -LONG TERM

February 2009 23-1-01178-010

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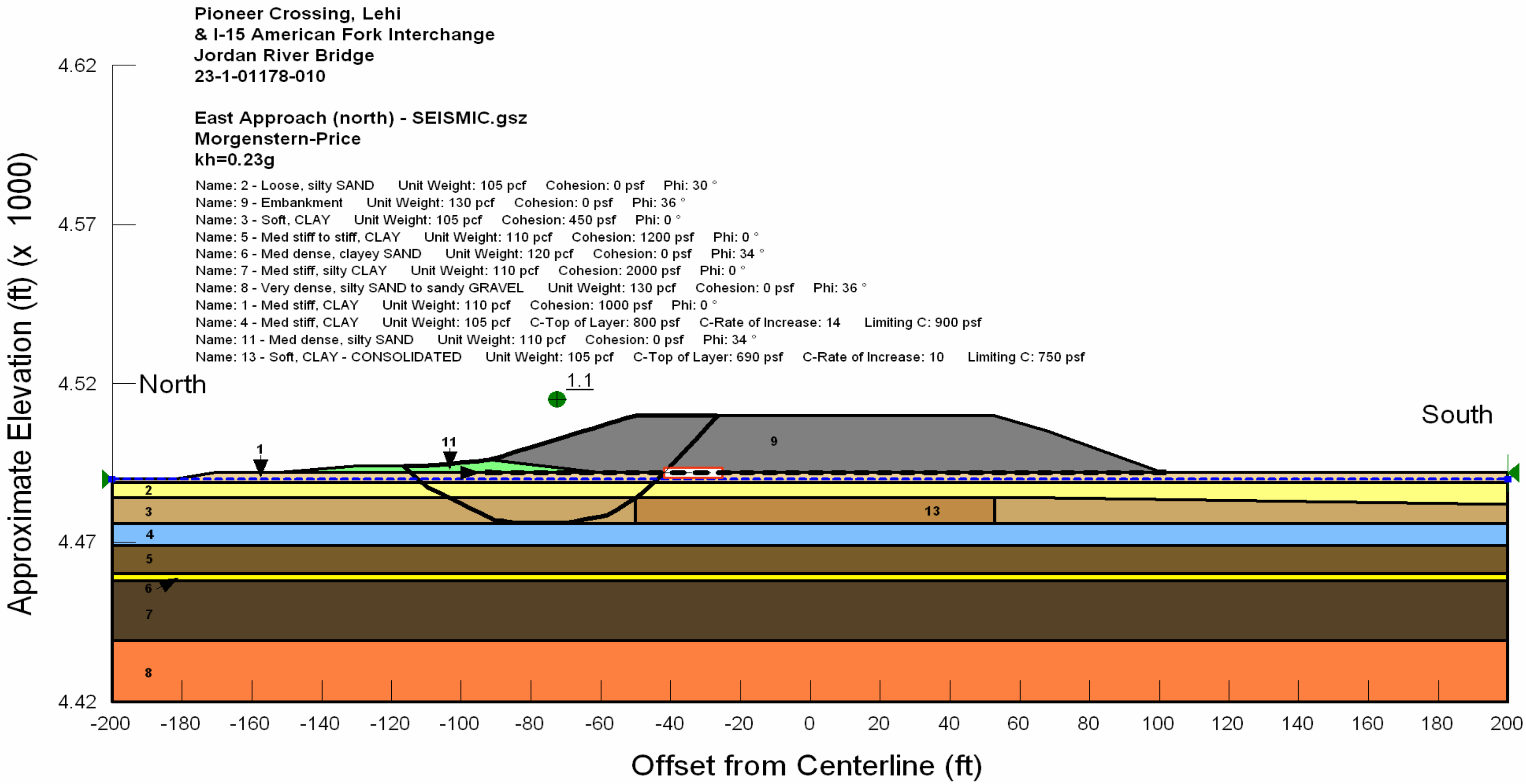
FIG. GD5-C-6

FIG. GD5-C-6



Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

GLOBAL STABILITY PROFILE
EAST APPROACH (NORTH)
SEISMIC

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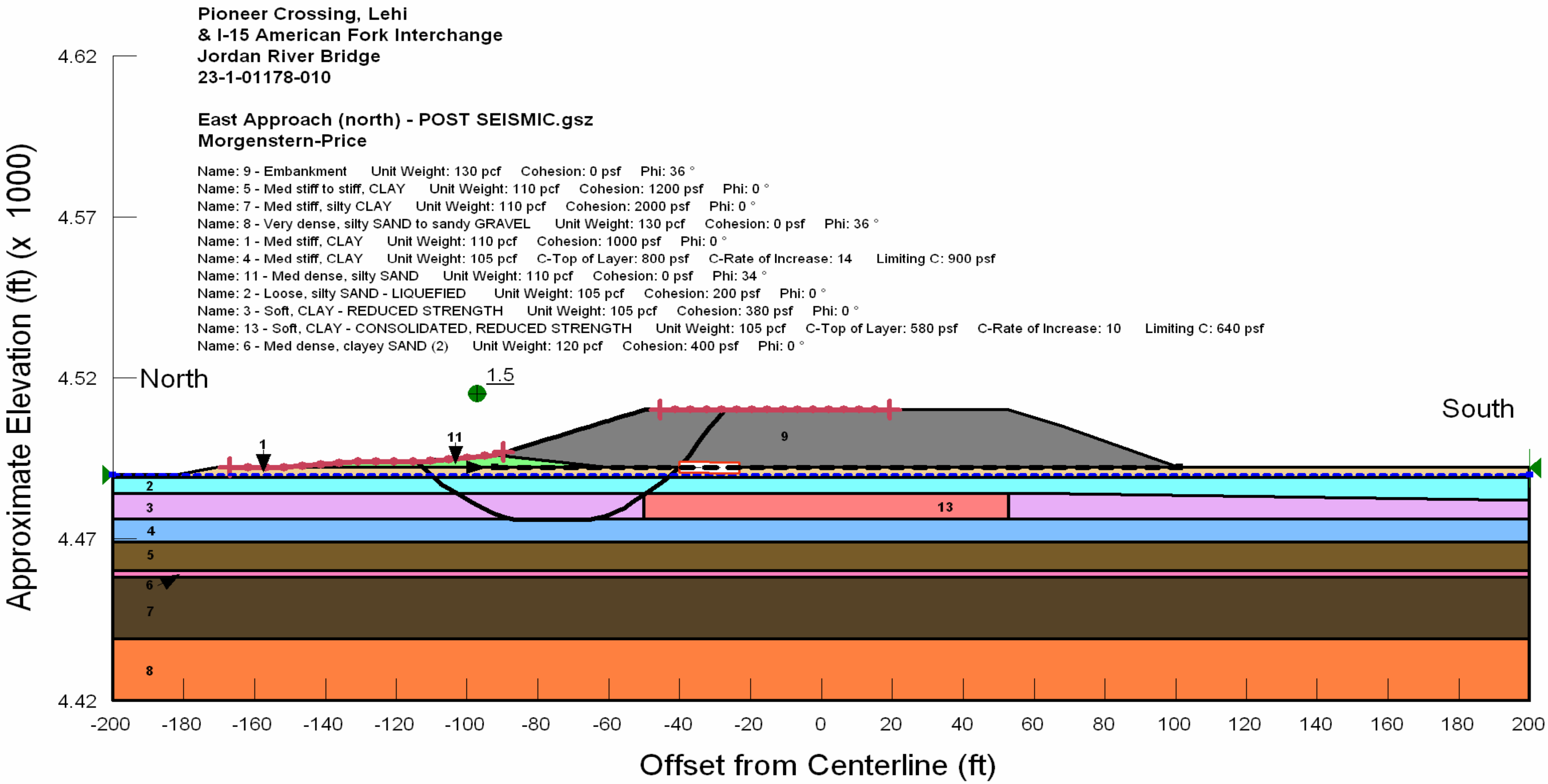
FIG. GD5-C-7

FIG. GD5-C-7

Stability Figures.xls - 2/25/2009



Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

GLOBAL STABILITY PROFILE
EAST APPROACH (NORTH)
POST SEISMIC (LIQUEFIED)

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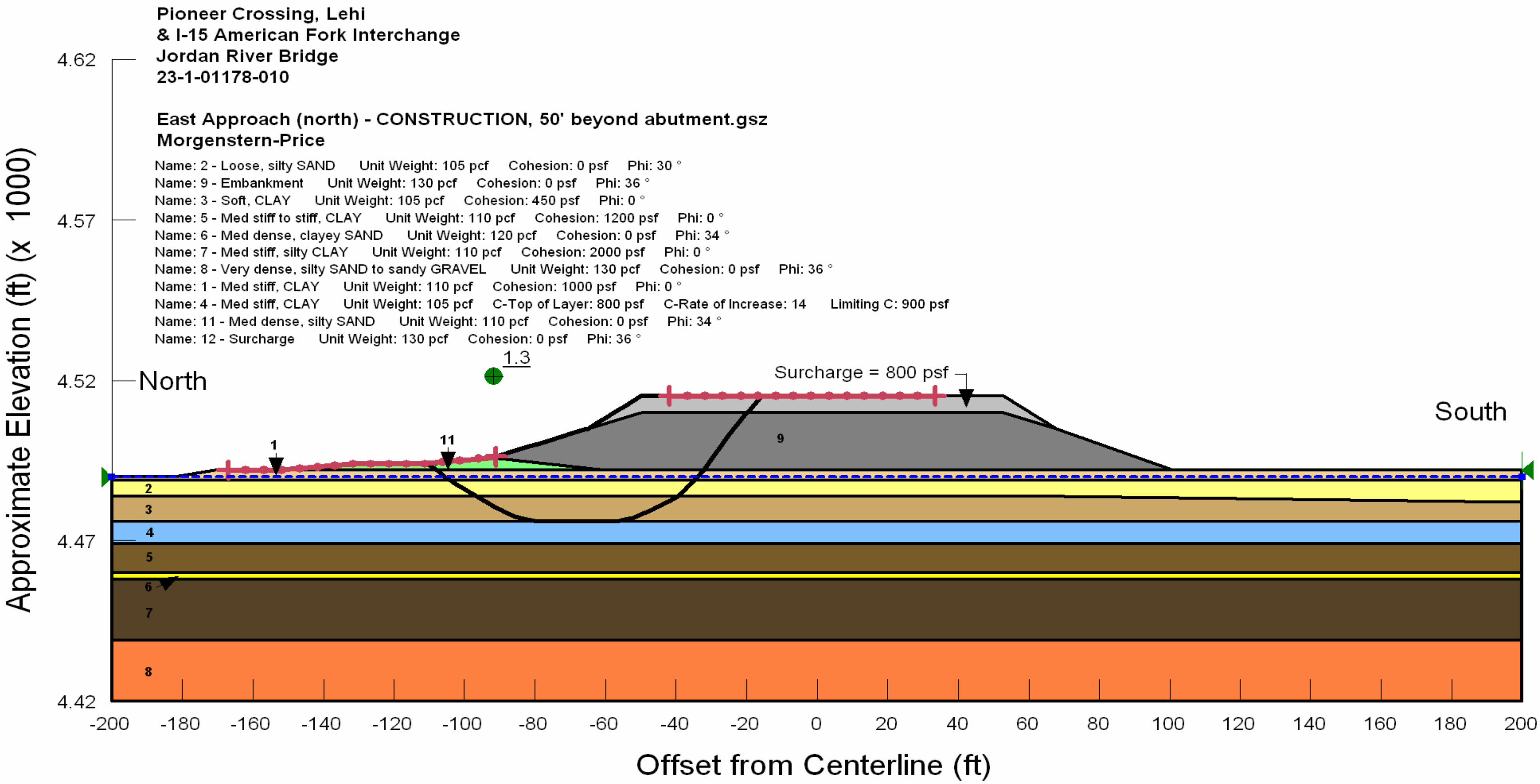
FIG. GD5-C-8

FIG. GD5-C-8



Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

GLOBAL STABILITY PROFILE
EAST APPROACH, 50' EAST OF ABUTMENT (NORTH)
STATIC - END OF CONSTRUCTION

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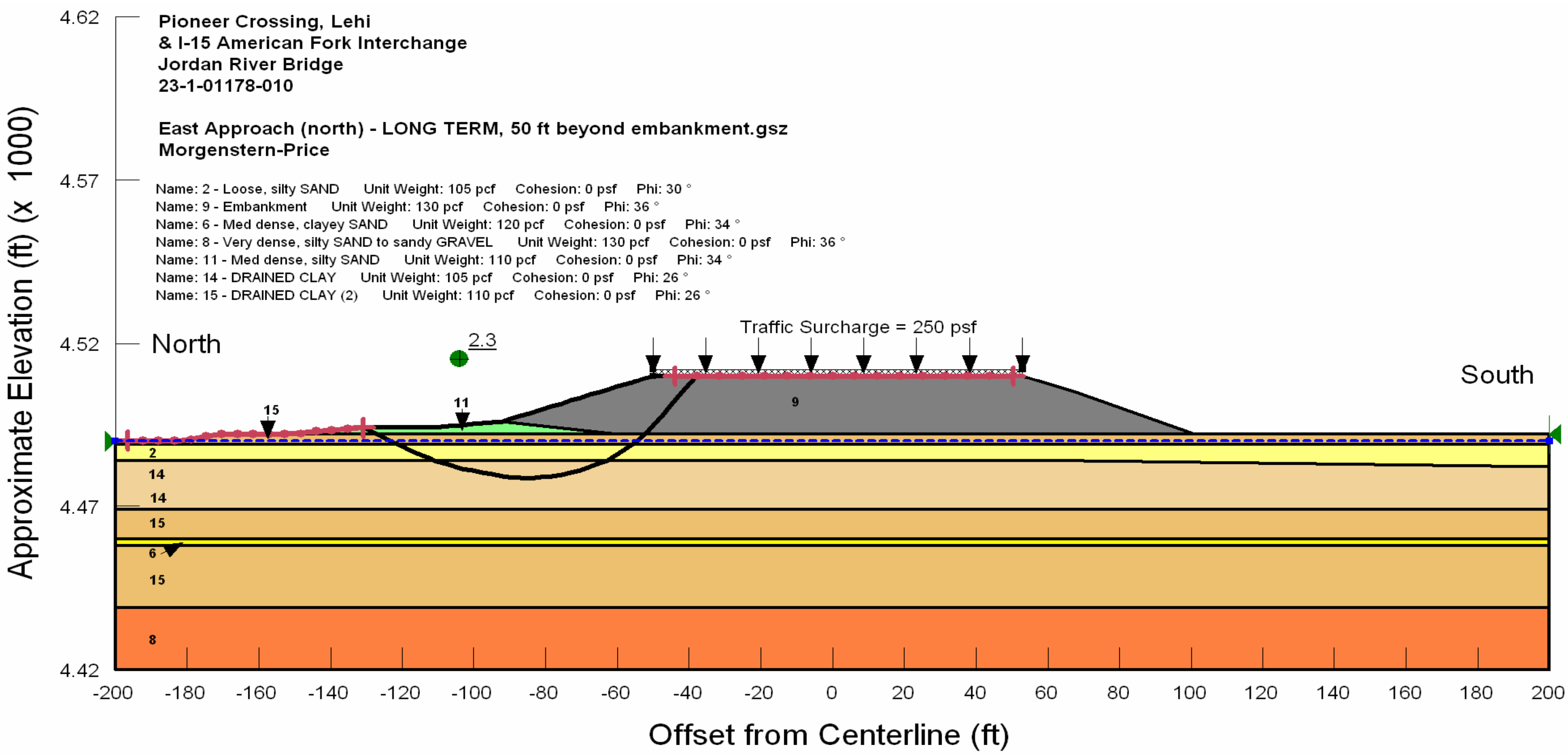
FIG. GD5-C-9

FIG. GD5-C-9



Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

GLOBAL STABILITY PROFILE
EAST APPROACH, 50' EAST OF ABUTMENT (NORTH)
STATIC -LONG TERM

February 2009 23-1-01178-010

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FIG. GD5-C-10

FIG. GD5-C-10

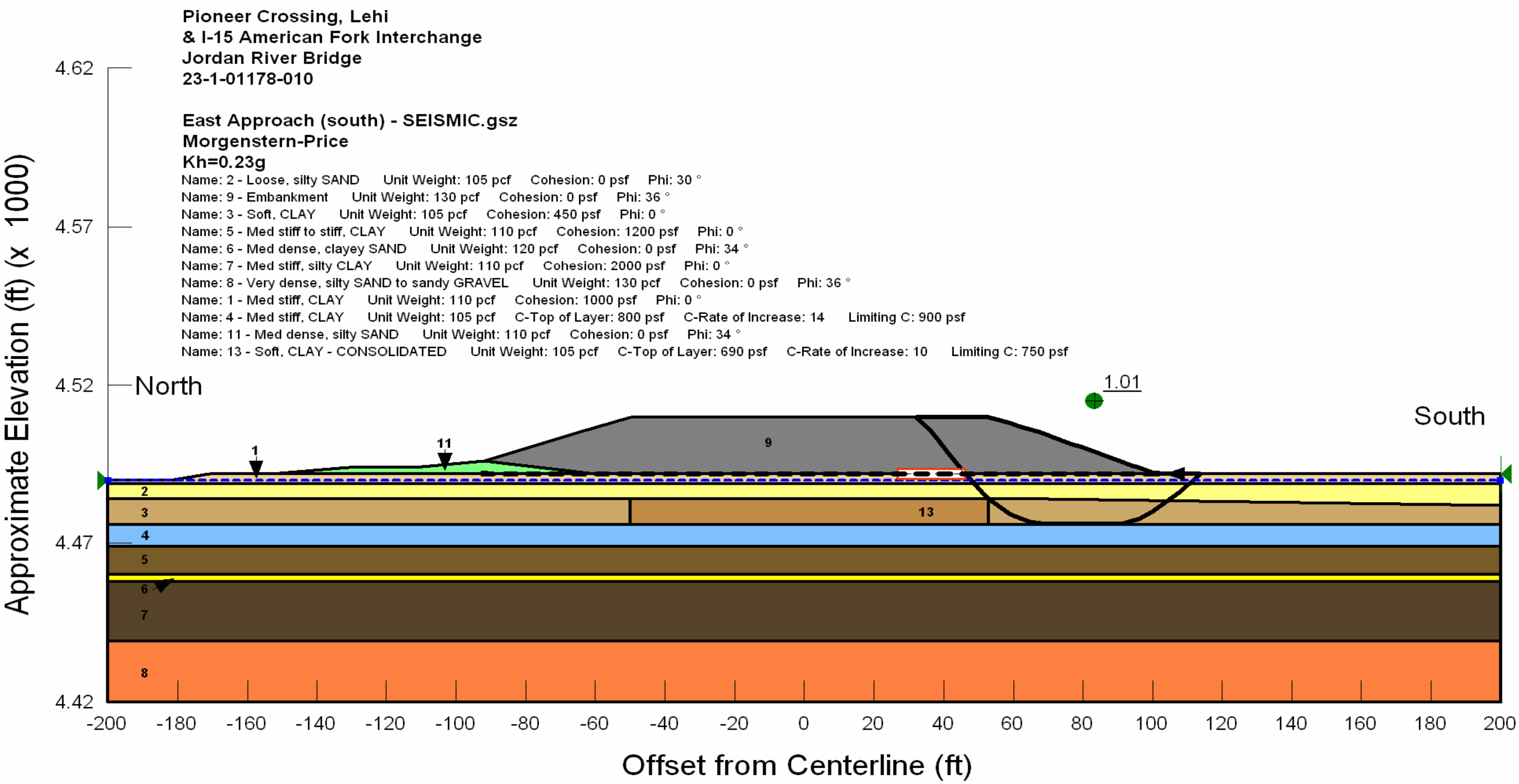

FIG. GD5-C-11

Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

GLOBAL STABILITY PROFILE
EAST APPROACH (SOUTH)
SEISMIC

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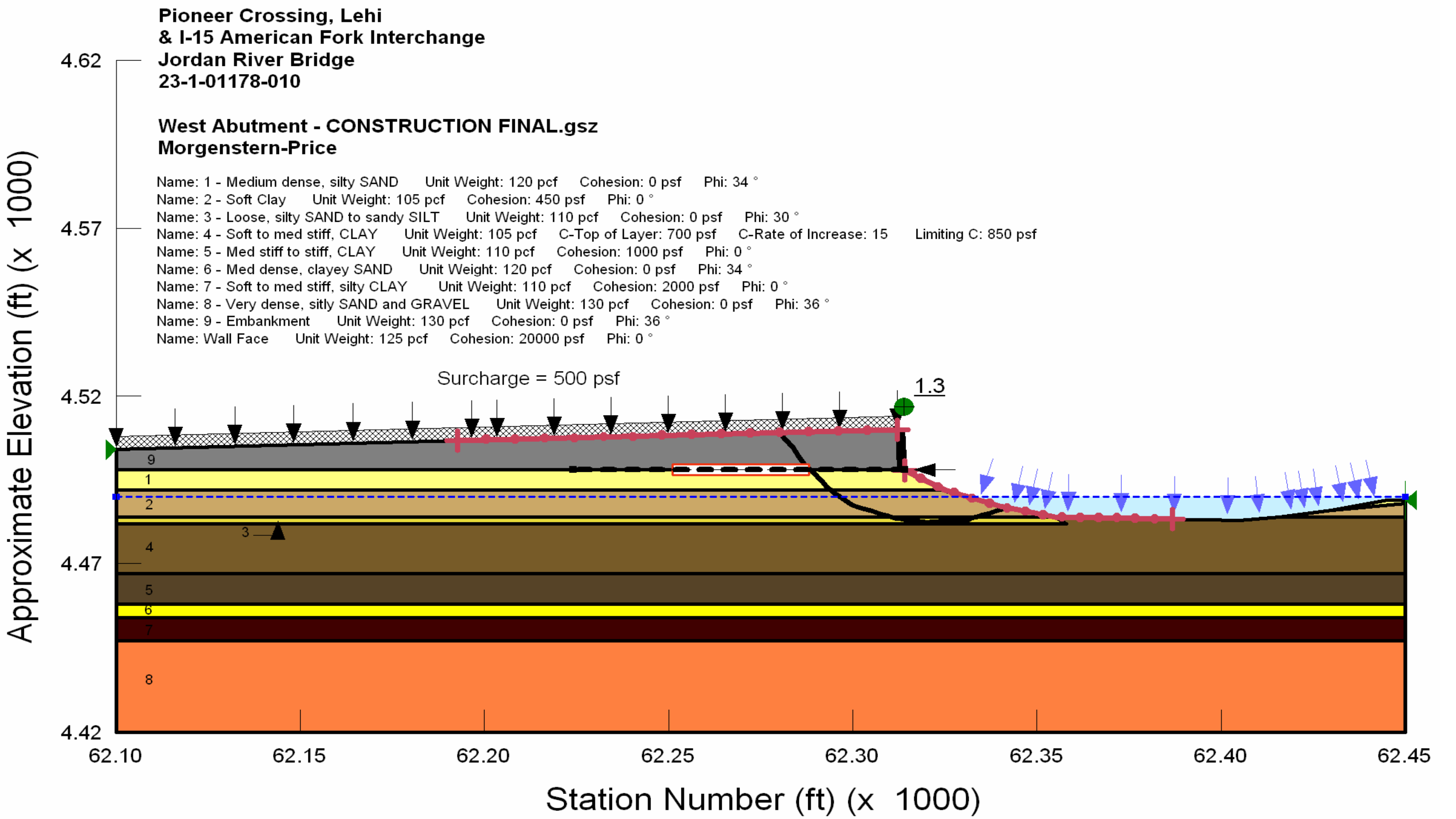
FIG. GD5-C-11



Pioneer Crossing, Lehi & I-15 American Fork Interchange Lehi, Utah	
GLOBAL STABILITY PROFILE WEST (VERTICAL) ABUTMENT STATIC - END OF CONSTRUCTION	
February 2009	23-1-01178-010
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. GD5-C-12**

FIG. GD5-C-12

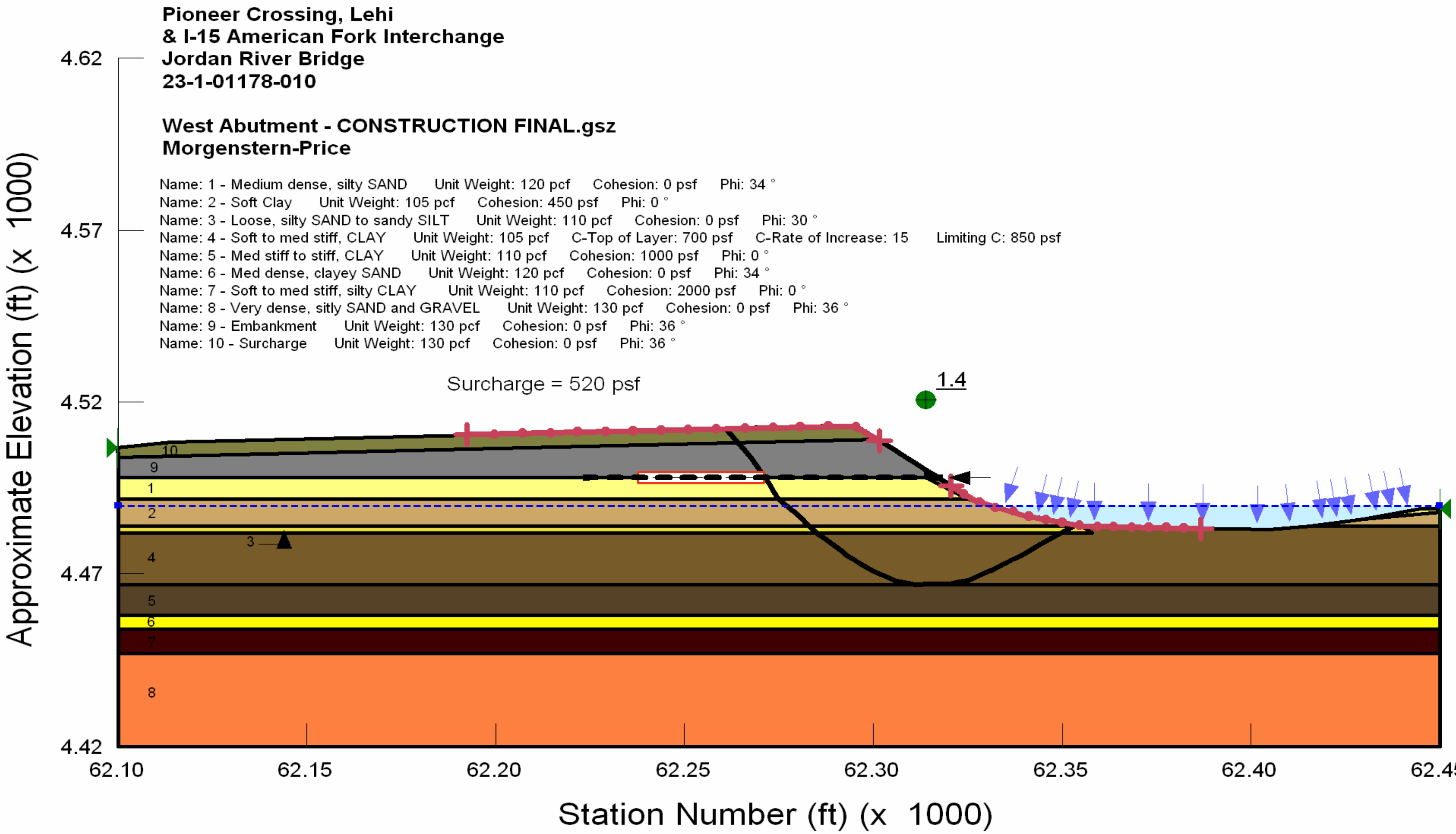
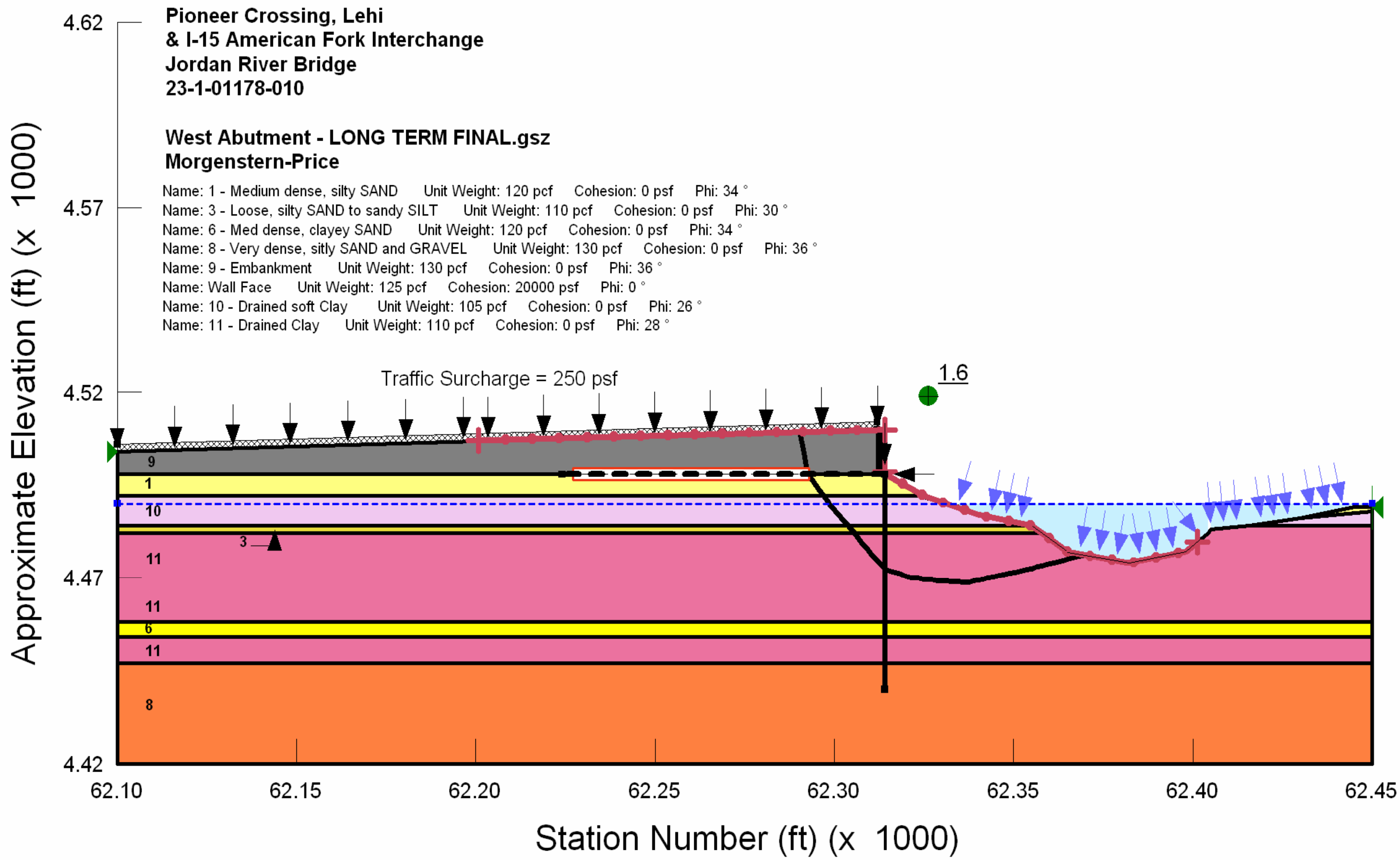


FIG. GD5-C-13

Pioneer Crossing, Lehi & I-15 American Fork Interchange Lehi, Utah	
GLOBAL STABILITY PROFILE WEST (SLOPING) ABUTMENT STATIC - END OF CONSTRUCTION	
February 2009	23-1-01178-010
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants	**FIG. GD5-C-13**



Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

GLOBAL STABILITY PROFILE
WEST ABUTMENT
STATIC -LONG TERM

February 2009 23-1-01178-010

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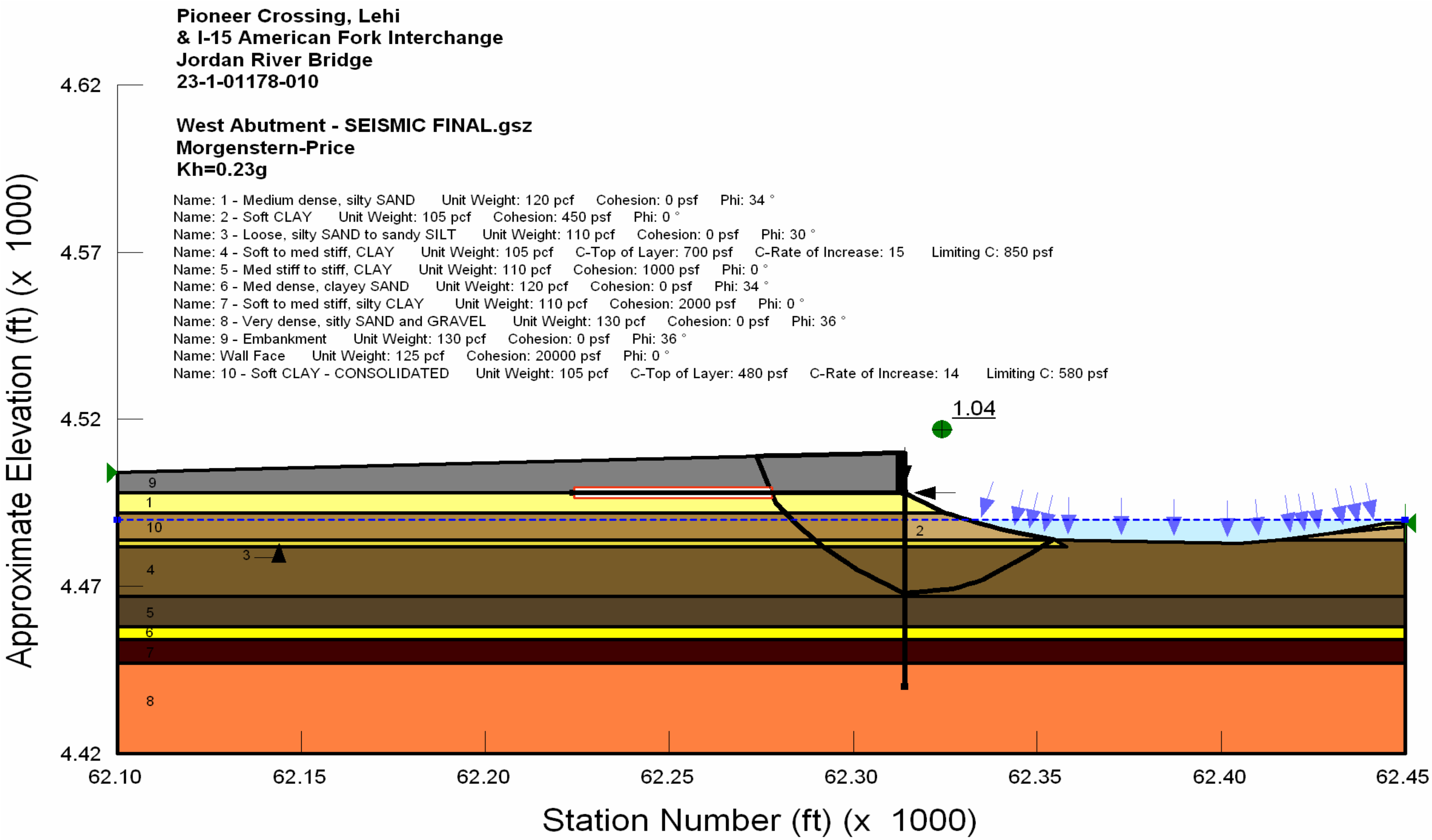
FIG. GD5-C-14

FIG. GD5-C-14



Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

GLOBAL STABILITY PROFILE
WEST ABUTMENT
SEISMIC

February 2009 23-1-01178-010

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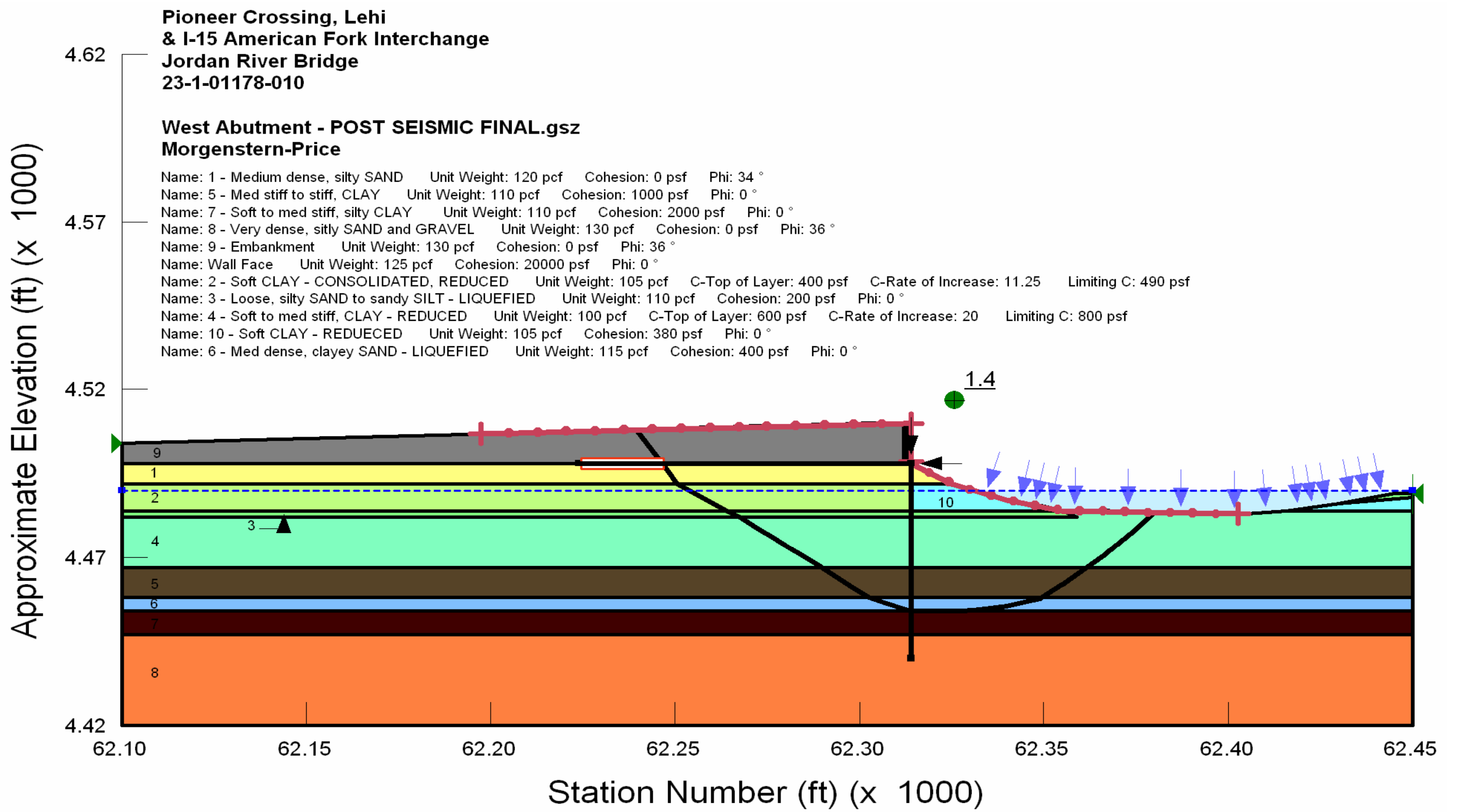
FIG. GD5-C-15

FIG. GD5-C-15



Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

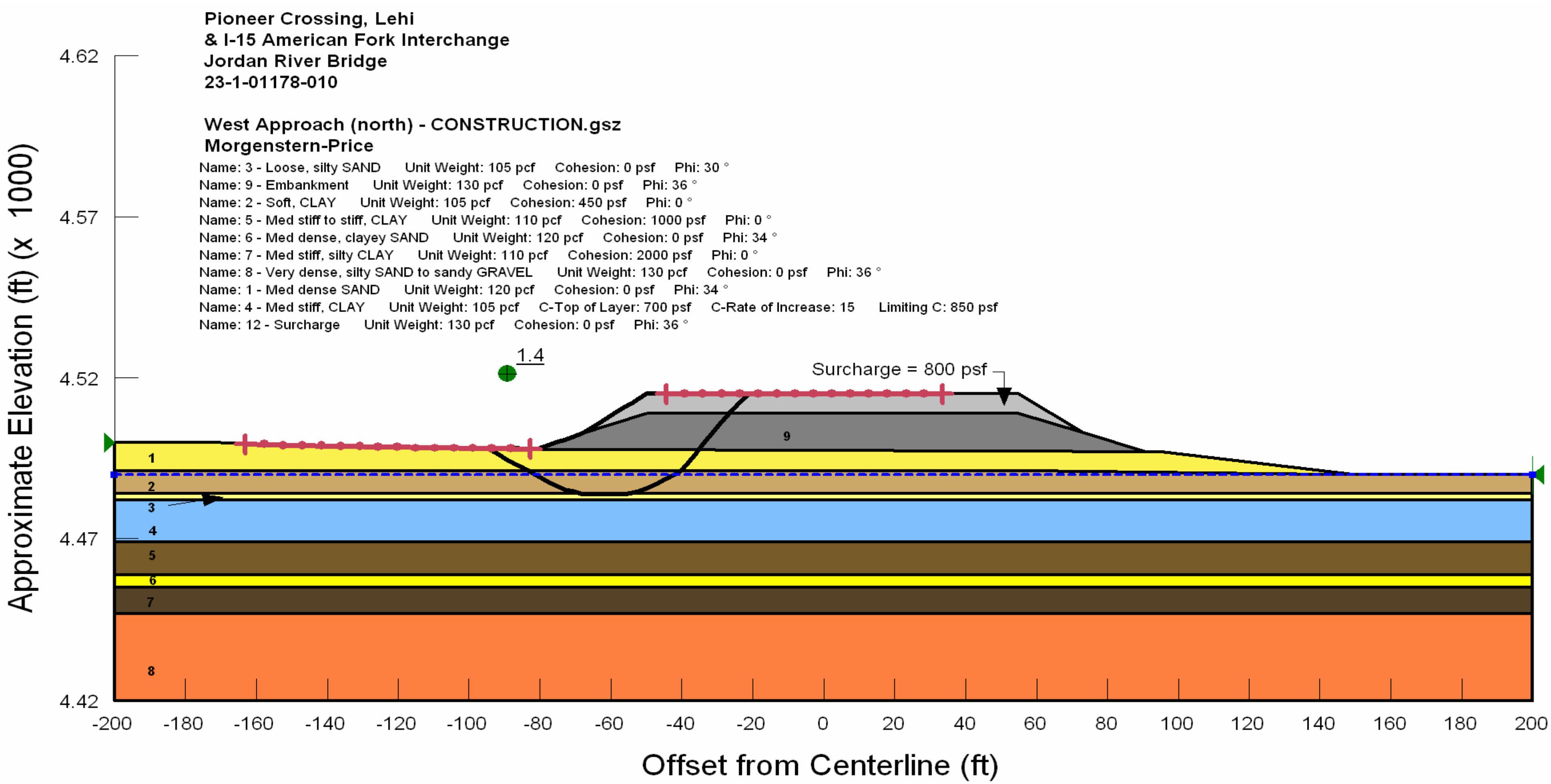
GLOBAL STABILITY PROFILE
WEST ABUTMENT
POST SEISMIC (LIQUEFIED)

February 2009 23-1-01178-010

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

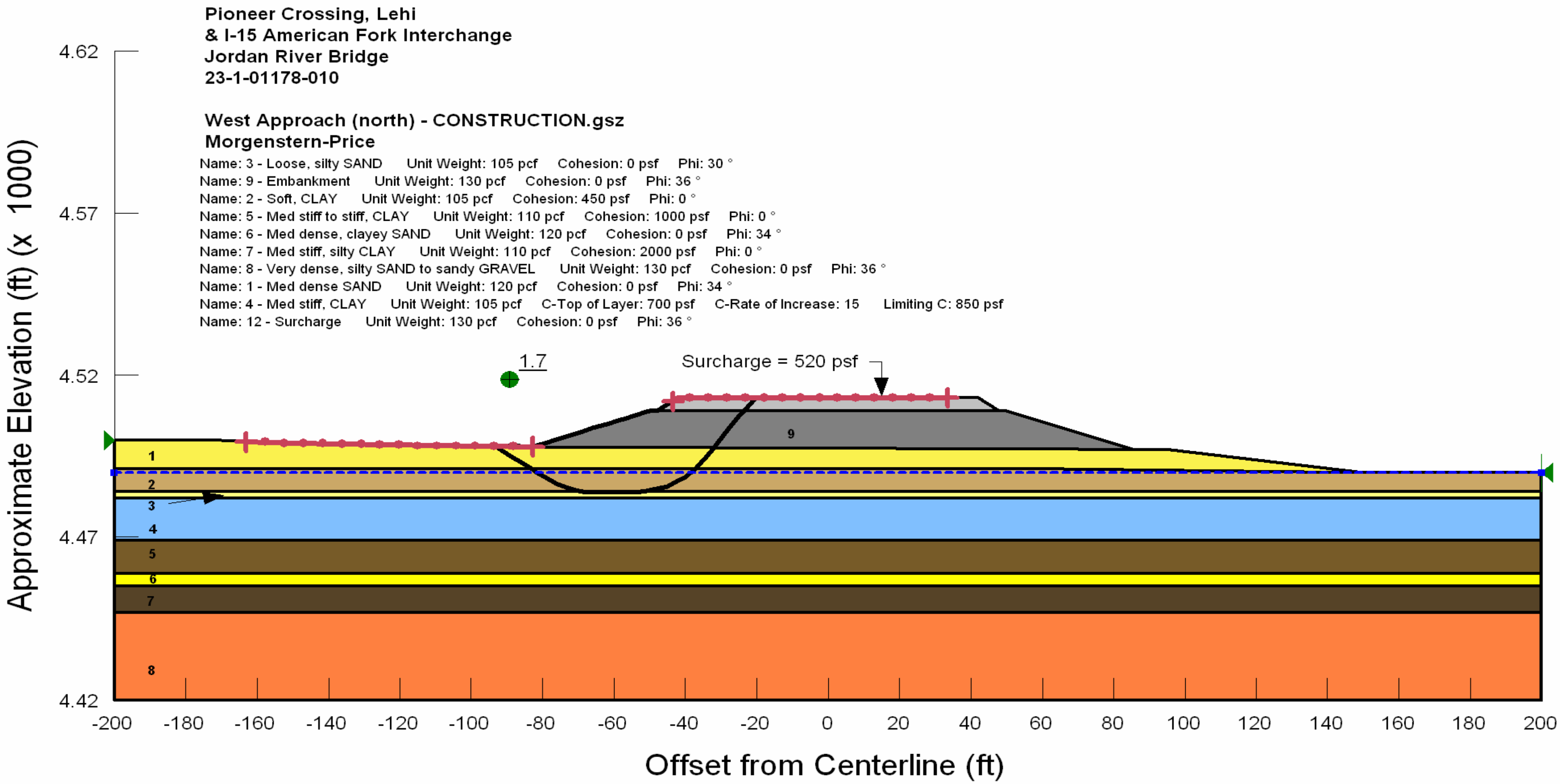
FIG. GD5-C-16

FIG. GD5-C-16



Pioneer Crossing, Lehi & I-15 American Fork Interchange Lehi, Utah
GLOBAL STABILITY PROFILE WEST APPROACH (NORTH) STATIC - END OF CONSTRUCTION (1)
February 2009 23-1-01178-010
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants — **FIG. GD5-C-17**

FIG. GD5-C-17



Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

GLOBAL STABILITY PROFILE
WEST APPROACH (NORTH)
STATIC - END OF CONSTRUCTION (2)

February 2009 23-1-01178-010

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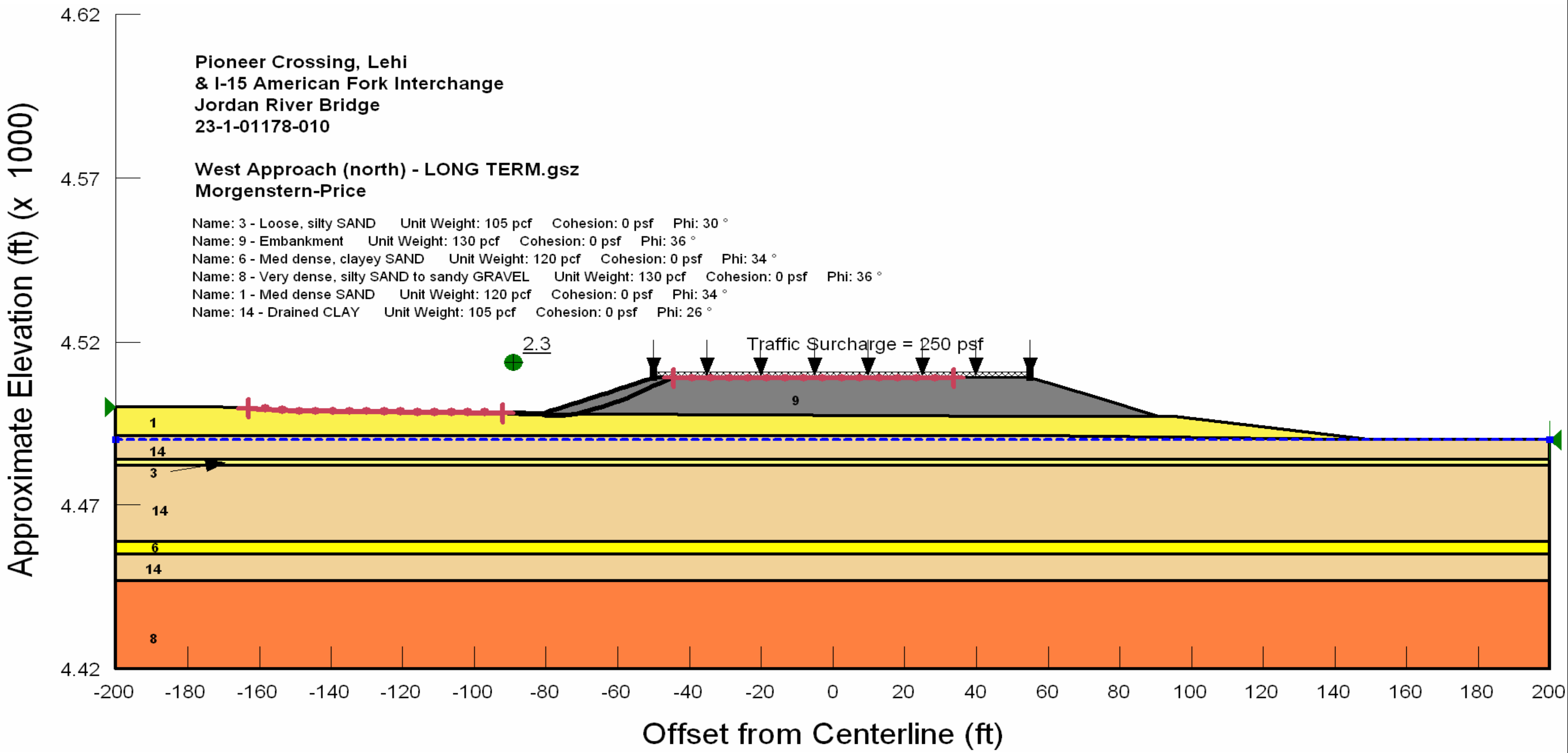
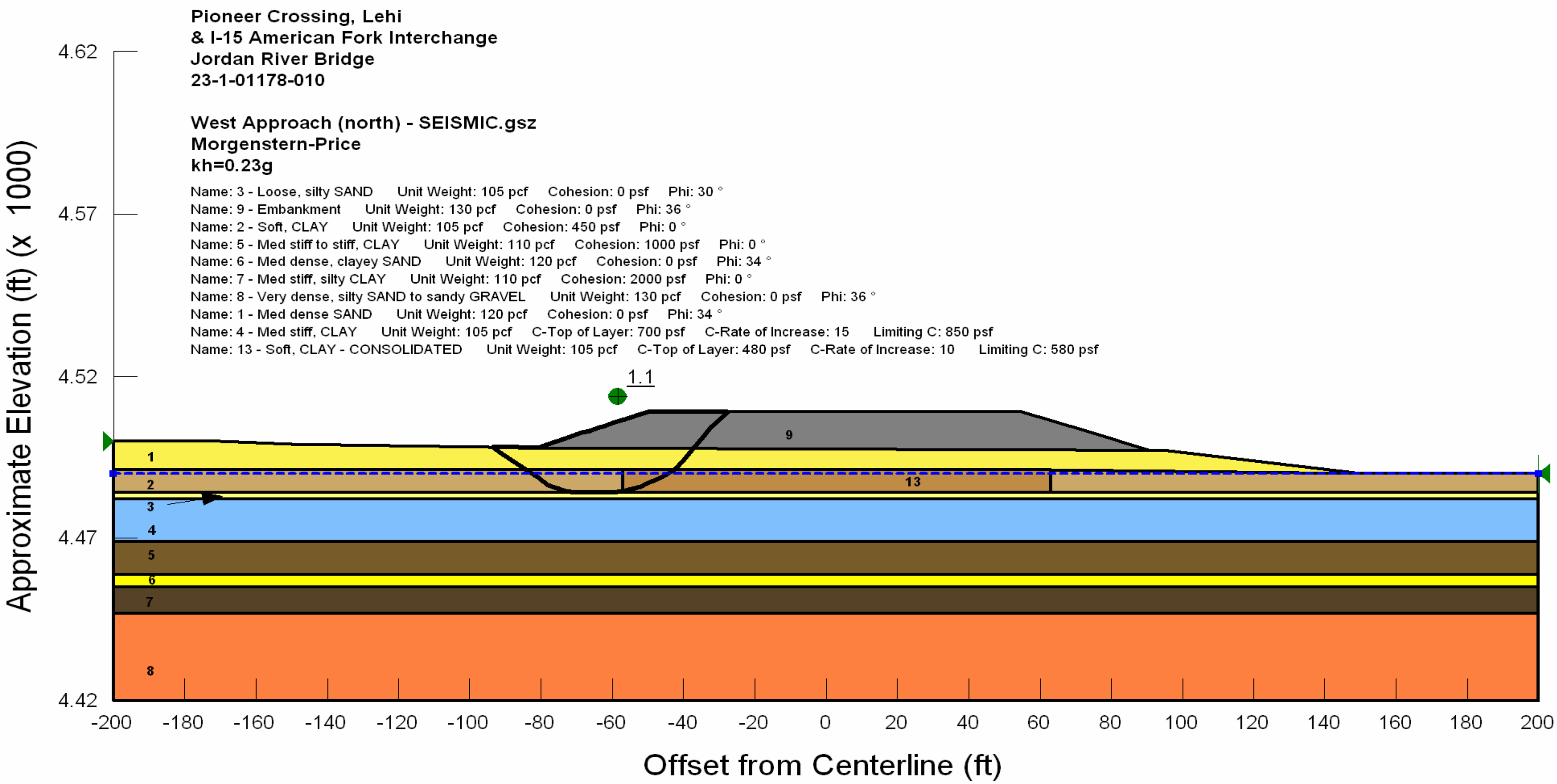
FIG. GD5-C-18

FIG. GD5-C-18



Pioneer Crossing, Lehi & I-15 American Fork Interchange Lehi, Utah
GLOBAL STABILITY PROFILE **WEST APPROACH (NORTH)** **STATIC - LONG TERM**
February 2009 — 23-1-01178-010
SHANNON & WILSON, INC. Geotechnical and Environmental Consultants — **FIG. GD5-C-19**

FIG. GD5-C-19



Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

GLOBAL STABILITY PROFILE
WEST APPROACH (NORTH)
SEISMIC

February 2009 23-1-01178-010

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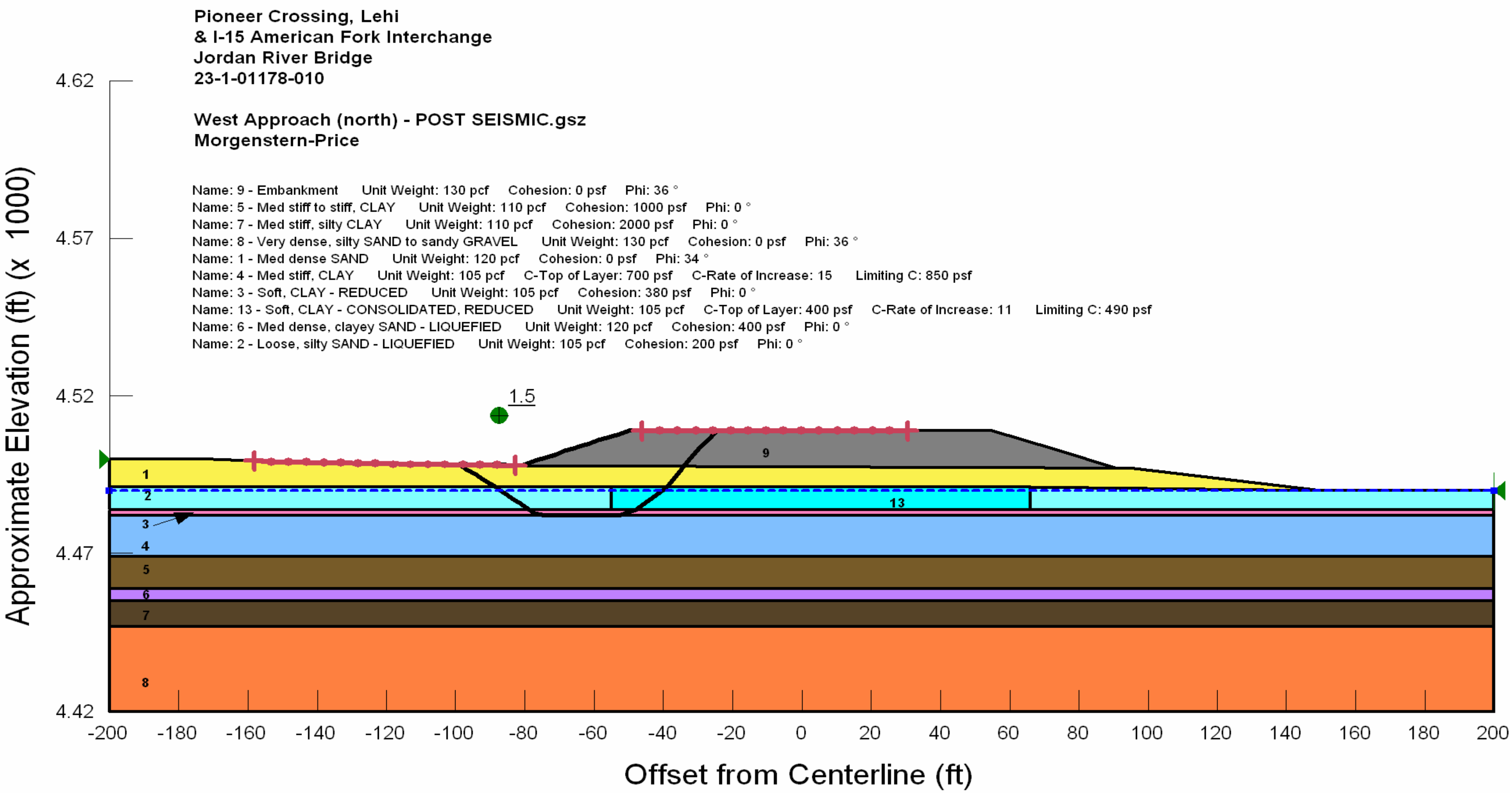
FIG. GD5-C-20

FIG. GD5-C-20



Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

GLOBAL STABILITY PROFILE
WEST APPROACH (NORTH)
POST SEISMIC (LIQUEFIED)

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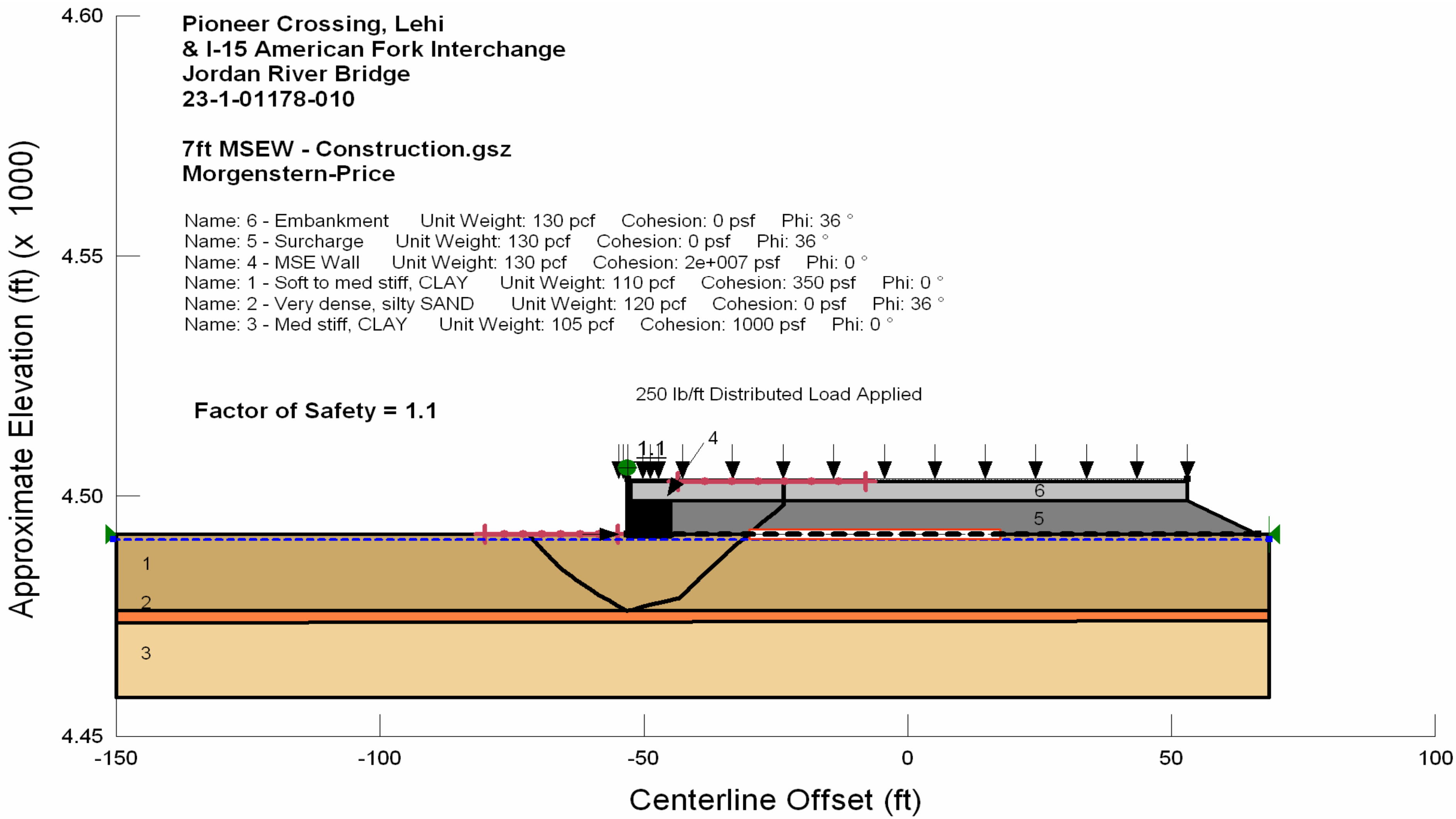
FIG. GD5-C-21

FIG. GD5-C-21



Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

GLOBAL STABILITY PROFILE
MSE WALL
STATIC - END OF CONSTRUCTION

February 2009 23-1-01178-010

SHANNON & WILSON, INC.
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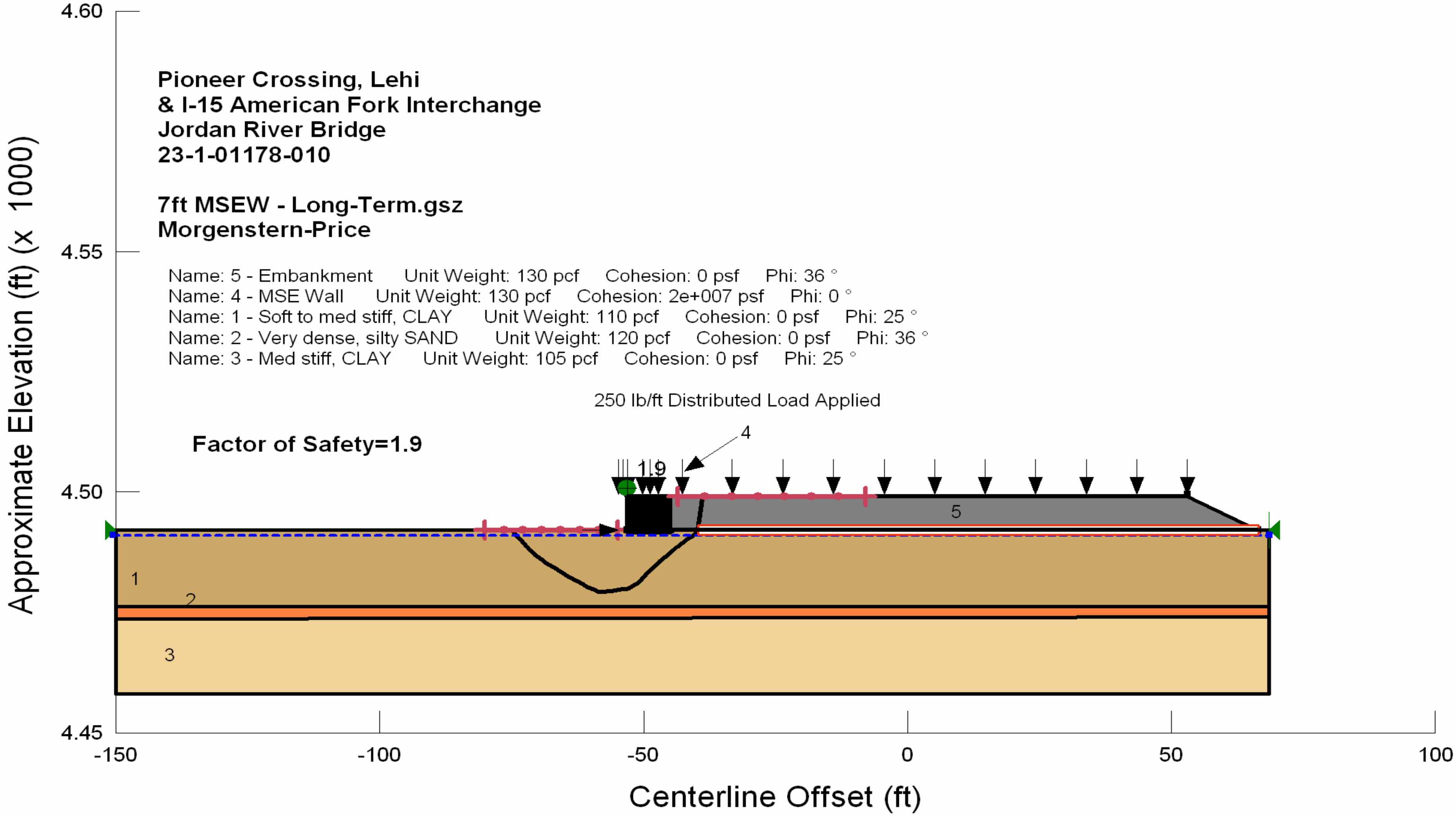
FIG. GD5-C-22

FIG. GD5-C-22



Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

GLOBAL STABILITY PROFILE
MSE WALL
STATIC -LONG TERM

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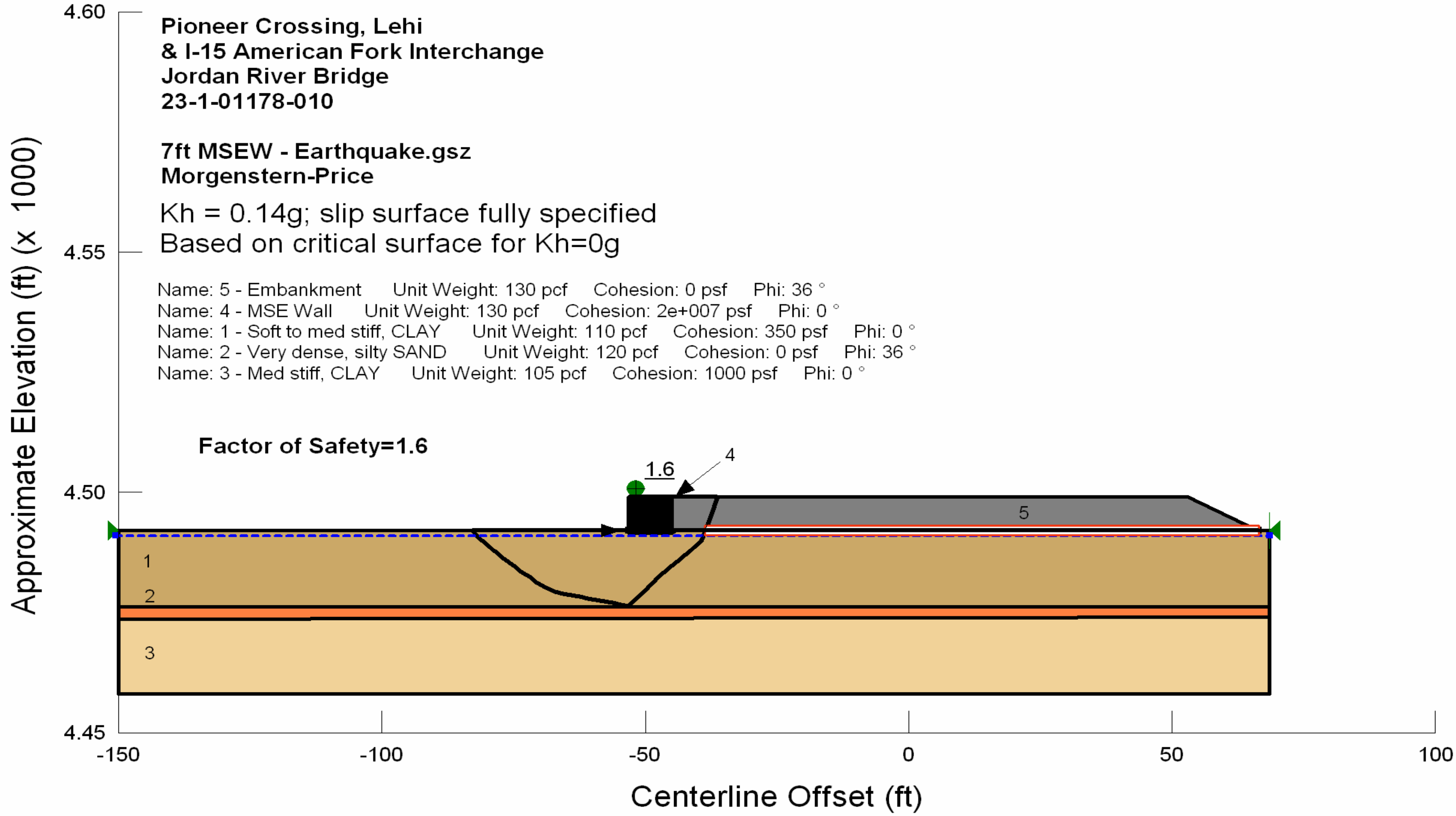
FIG. GD5-C-23

FIG. GD5-C-23



Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

GLOBAL STABILITY PROFILE
MSE WALL
SEISMIC

February 2009 23-1-01178-010

SHANNON & WILSON, INC.
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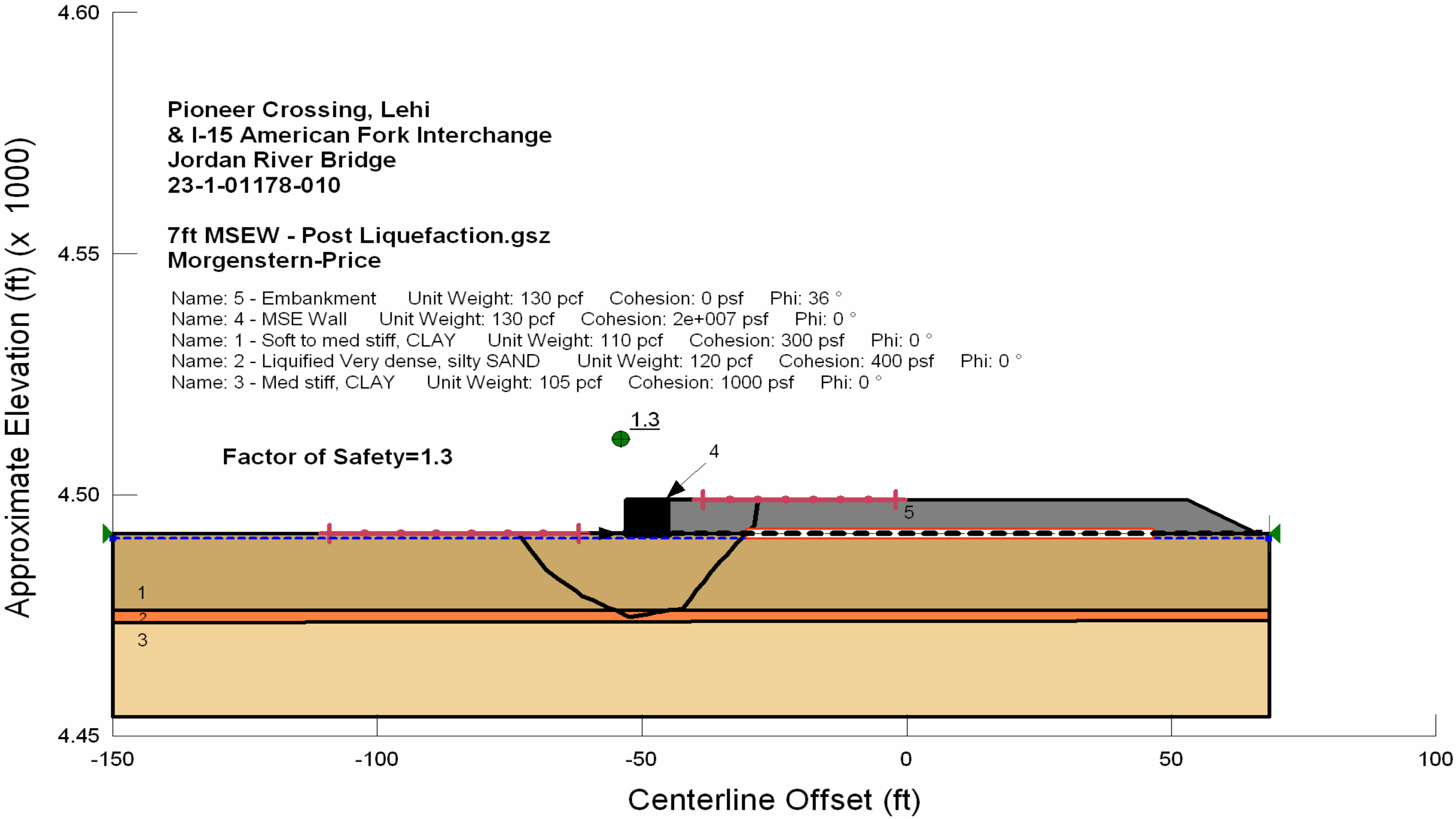
FIG. GD5-C-24

FIG. GD5-C-24



Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

GLOBAL STABILITY PROFILE
MSE WALL
POST SEISMIC (LIQUEFIED)

February 2009 23-1-01178-010

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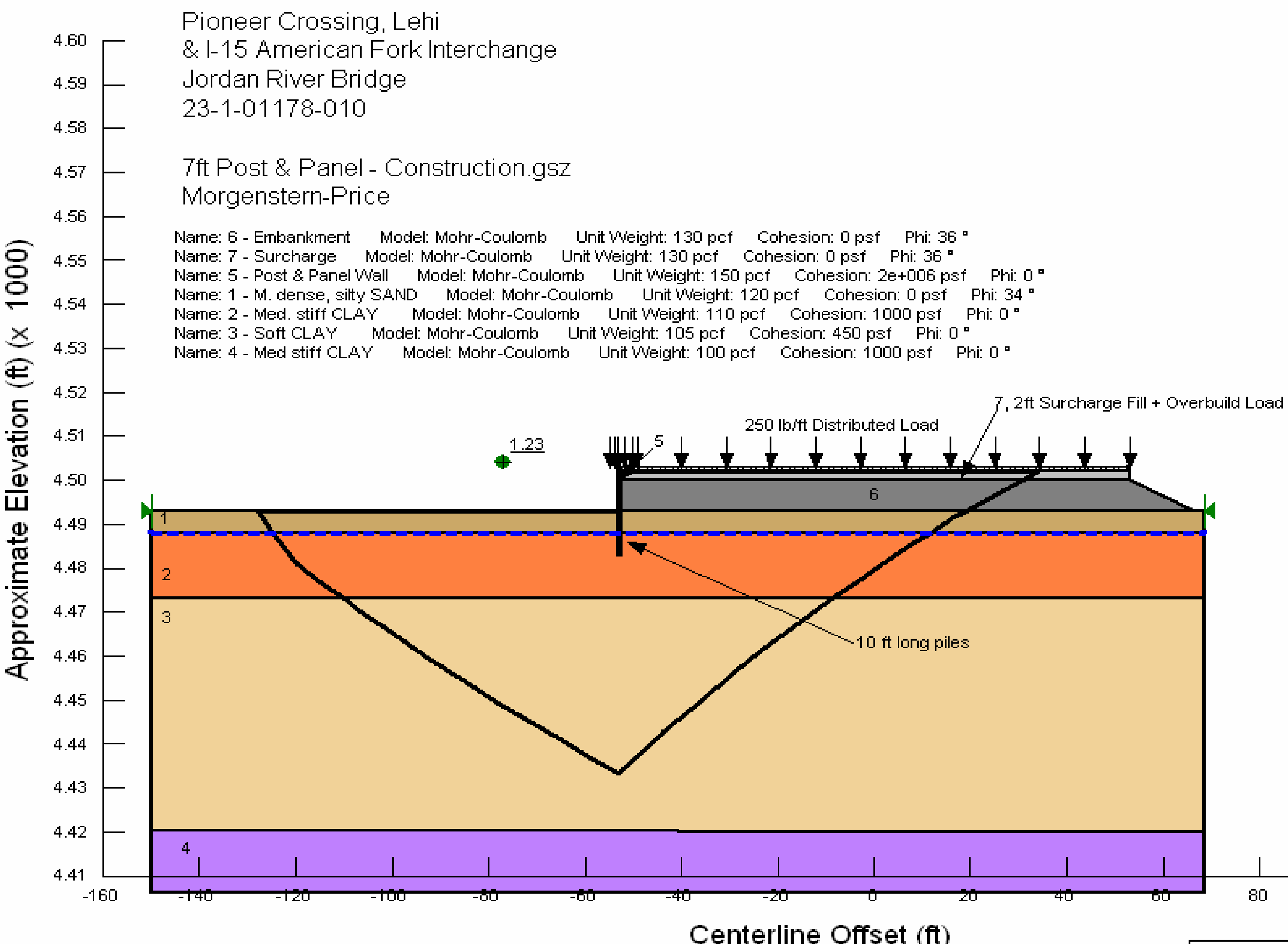
FIG. GD5-C-25

FIG. GD5-C-25



Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

GLOBAL STABILITY PROFILE
POST & PANEL WALL
STATIC - END OF CONSTRUCTION

February 2009 23-1-01178-010

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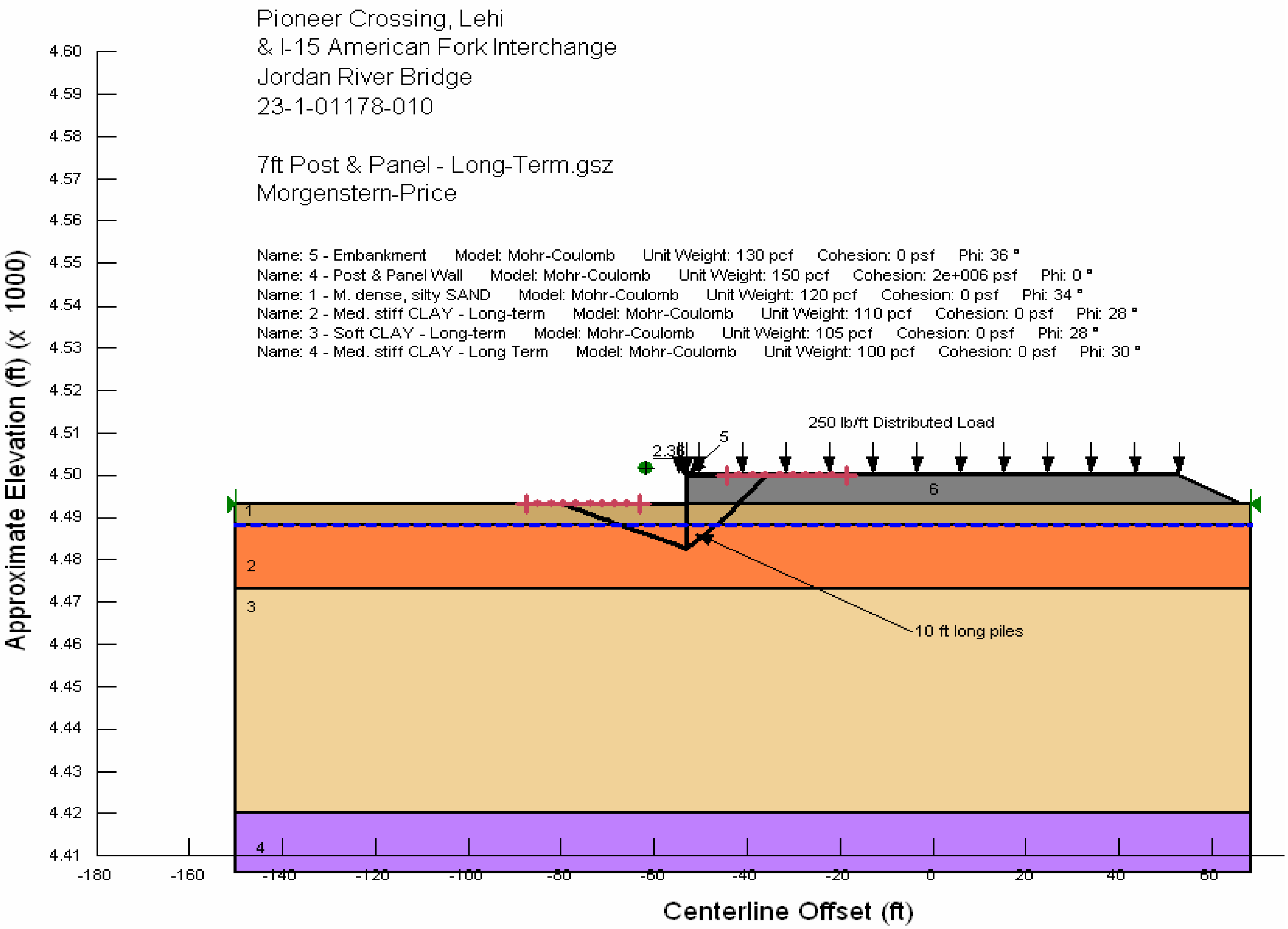
Geotechnical and Environmental Consultants

FIG. GD5-C-26

FIG. GD5-C-26



Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

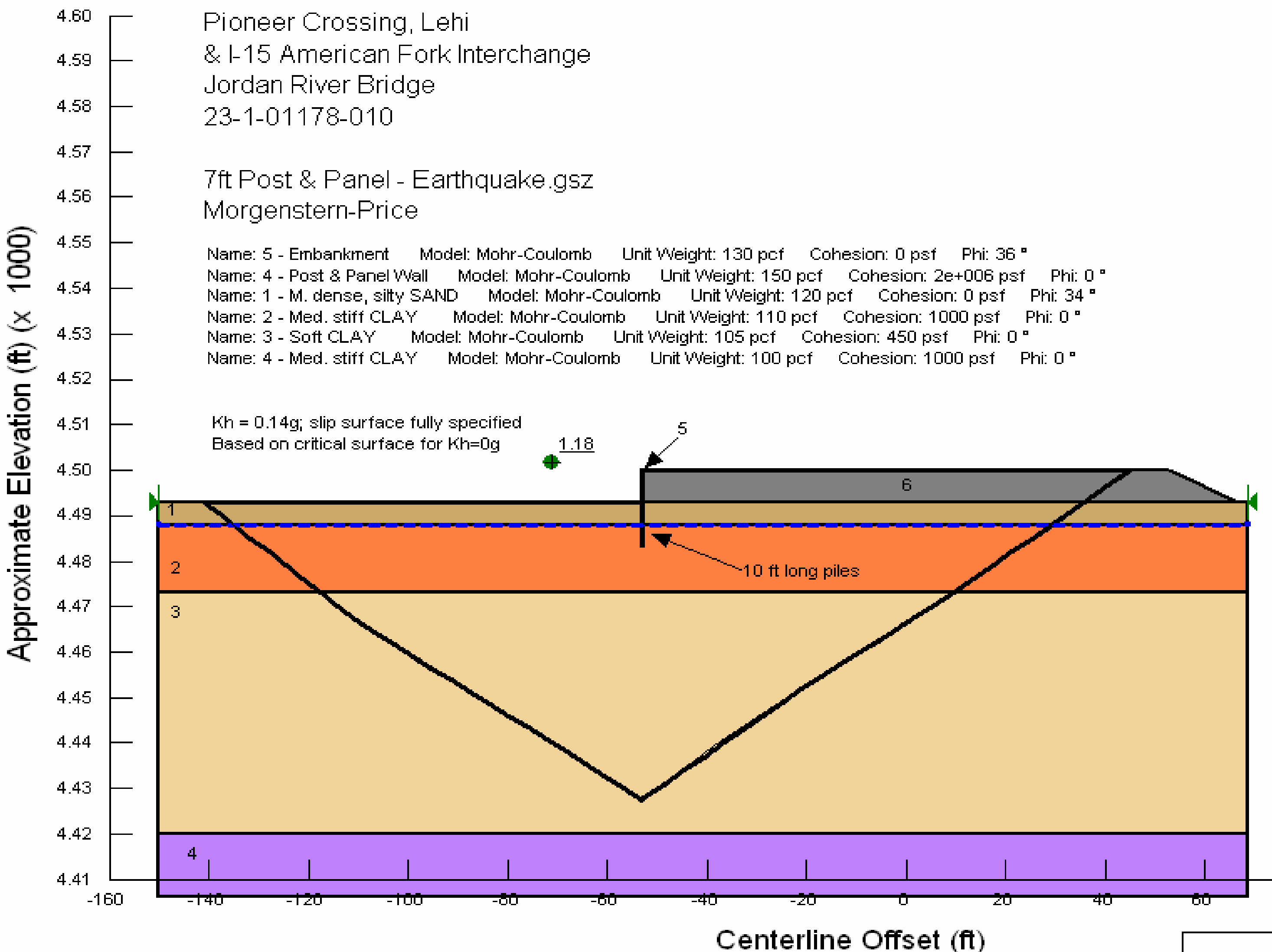
GLOBAL STABILITY PROFILE
POST & PANEL WALL
STATIC -LONG TERM

February 2009 23-1-01178-010

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. GD5-C-27

FIG. GD5-C-27



Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

GLOBAL STABILITY PROFILE
POST & PANEL WALL
SEISMIC

February 2009 23-1-01178-010

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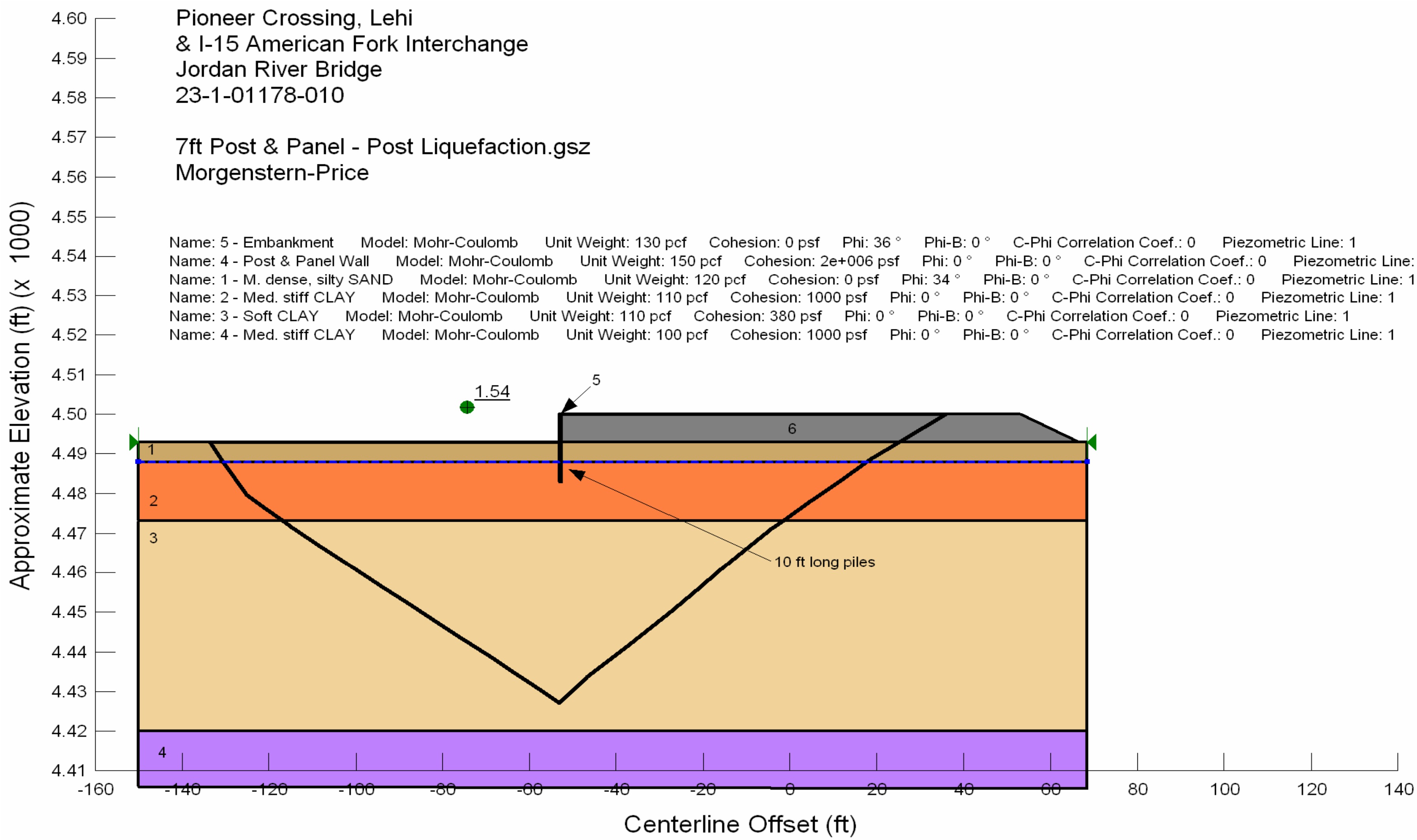
FIG. GD5-C-28

FIG. GD5-C-28



Pioneer Crossing, Lehi & I-15
American Fork Interchange
Lehi, Utah

GLOBAL STABILITY PROFILE
POST & PANEL WALL
POST SEISMIC (LIQUEFIED)

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FIG. GD5-C-29

FIG. GD5-C-29

APPENDIX F

GEOTECHNICAL DESIGN MEMORANDUM GD-5 ADDENDUM 1

GEOTECHNICAL DESIGN MEMORANDUM GD-5
ADDENDUM 1
PIONEER CROSSING, LEHI AND I-15 AMERICAN FORK INTERCHANGE

PIONEER CROSSING APPROACH FILL, EMBANKMENT STABILITY, AND JORDAN RIVER BRIDGE

April 7, 2009

Prepared by:
Shannon & Wilson, Inc.,
Geotechnical Consultant for Kiewit/Clyde Joint Venture

PROFESSIONAL ENGINEER
No. 7030264-2202
CHRISTOPHER ALAN ROBERTSON
STATE OF UTAH

4/7/09

Christopher A. Robertson, P.E.
Vice President

GSE:CAR/car

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GEOTECHNICAL DESIGN MEMORANDUM GD-5, ADDENDUM 1
PIONEER CROSSING, LEHI AND I-15 AMERICAN FORK INTERCHANGE

1.0 DESIGN MEMORANDUM SCOPE

This addendum to Geotechnical Design Memorandum GD-5 (GD-5), dated February 26, 2009, provides revised recommendations for nominal pile resistance, Wave Equation Analyses for Pile Driving (WEAP) analyses, and clarifies roadway and bridge approach settlement criteria. Please see GD-5 for details regarding the project and site descriptions, subsurface conditions, and geotechnical design recommendations.

This addendum is a revision to and supersedes Addendum 1 dated March 26, 2009. We made revisions in response to UDOT comments provided on April 6, 2009:

Comment No.	Comment (paraphrased)	Response
369	(kip-ft instead of kip/ft) Correct typo on Figures GD5-13 and GD5-14	Corrected typo on Figures GD5-13 and GD5-14.
370	Perform WEAP analyses for anticipated pile lengths.	See revised WEAP analyses in Figures GD5-13 and GD5-14.
371	Correct pile cross-section area on Figures GD5-13 and GD5-14.	Corrected on Figure sGD5-13 and GD5-14.
376	Provide pile settlement estimates.	See Section 2.2.

Final versions of this addendum will be incorporated with other geotechnical design memoranda issued for the project and will be compiled into the Pioneer Crossing, Lehi and I-15 American Fork Interchange Project Geotechnical Report upon completion of the geotechnical engineering design phase of the project. General limitations on the use of this memorandum and recommendations presented herein are presented in Geotechnical Design Memorandum GD-1 (Shannon & Wilson, Inc., 2008a).

2.0 GEOTECHNICAL ANALYSES AND RECOMMENDATIONS

The embankments and walls described herein were evaluated using the methods described in Geotechnical Design Memorandum GD-2 (Shannon & Wilson, 2009).

2.0 GEOTECHNICAL ANALYSES AND RECOMMENDATIONS

The embankments and walls described herein were evaluated using the methods described in Geotechnical Design Memorandum GD-2 (Shannon & Wilson, 2009).

2.1 Settlement Analyses

Section 9.0 of the Request for Proposals (RFP) (Utah Department of Transportation [UDOT], 2008) specifies the following maximum permissible settlement for pavement:

- Transverse direction: Maximum 0.25-inch per 12-foot lane width.
- Maximum tolerable longitudinal direction: 0.25-inch per 30 feet.
- Allowable total post-construction settlement: 2 inches, unless otherwise approved by UDOT (UDOT, 2008).

Section 4.3 of GD-5 presented recommendations for settlement mitigation alternatives to meet these settlement criteria. In response to Request for Information (RFI) 019, dated March 5, 2009, UDOT clarified that the longitudinal settlement criterion for bridge approaches differs from the RFP pavement longitudinal settlement criterion. A copy of RFI-019 is presented in Appendix D. In their response, UDOT concurred with the RFI recommendation to:

> … limit the post-construction settlement at the end of the 25-foot approach slab to 1.5 inches, and to not require the smoothness requirement for pavement of 0.25 inches in 30 feet …

Our discussion in Section 4.3 of GD-5 presents post-construction settlement estimates at the proposed bridge abutments in Tables GD5-8 and GD5-9. These settlement estimates are not changed by the UDOT concurrence to RFI-019. The mitigation recommendations presented in Section 4.3.4.3 are valid; however, likely will not be required to meet the settlement criteria per the UDOT concurrence with RFI-019.

2.2 Jordan River Bridge Abutment Foundations

We understand that 16-inch-diameter, closed-ended steel pipe piles that are concrete-filled will support the Jordan River Bridge west and east abutments. Driven piles will support axial loads through a combination of skin friction and end bearing. This addendum presents revised

We anticipate total pile settlement should be about 1 inch or less under the design foundation loads. In our opinion, differential settlement between piles in an abutment should be about ½ inch or less, and differential settlement between the east and west abutments should be ¾ inch or less.

3.0 CONSTRUCTION CONSIDERATIONS

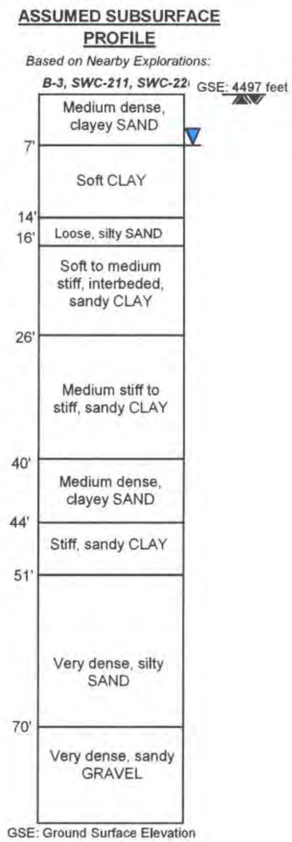
To establish driving criteria for pile installation, we performed preliminary WEAP analysis assuming 16-inch closed end pipe piles will be used at the west and east abutments. Kiewit/Clyde provided two different hammers that may be used to drive the piles. Two hammers are IHC S-70 and IHC S-90. This addendum presents revised WEAP analyses that incorporate different load transfer assumptions. In GD-5, we assumed constant shaft resistance during driving. Our revised WEAP analyses assume a proportional shaft resistance during driving. The proportional shaft resistance assumption results in lower maximum compressive stress during driving.

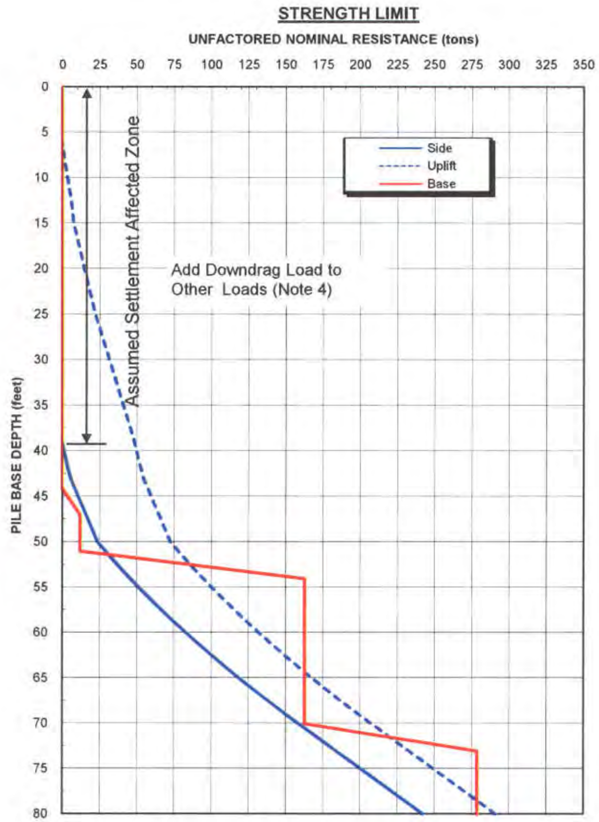
Results of the revised WEAP for the west and east abutments are presented in Figures GD5-13 and GD5-14, respectively. The results are presented as plots of driving resistance (blows/foot) versus ultimate pile capacity, maximum compression and tensile stress in the steel, energy at the pile tip, and hammer stroke. The analyses show that the maximum driving stresses in the pile should be below the allowable level of 90 percent of the yield 50 kips per square inch strength.

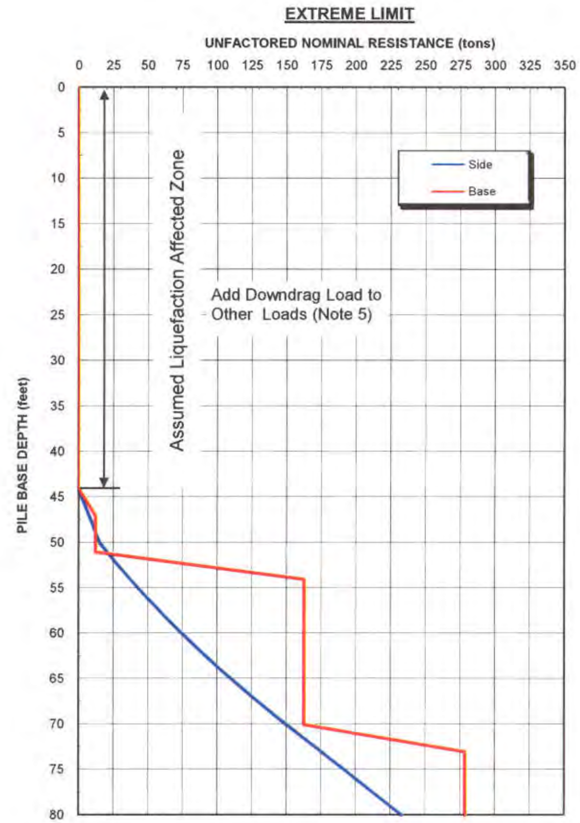
4.0 REFERENCES

Shannon & Wilson, Inc., 2009, Geotechnical design memorandum GD-1, Pioneer Crossing, Lehi and I-15 American Fork Interchange, Utah, Limitations: Memorandum prepared by Shannon & Wilson, Inc., Seattle, Wash., 23-1-01178-010, for Kiewit/Clyde, in progress.

Utah Department of Transportation, 2008, Request for proposals: Pioneer Crossing, Lehi and I-15 American Fork Interchange, Utah, Utah Department of Transportation, June 20.







Notes:

1. The analyses are based on a single pile and do not consider group action of closely spaced piles (closer than 2.5 diameters, center to center).
2. The unfactored nominal resistances on the above plots should be multiplied by the appropriate resistance factors as noted in the table. Total pile compressive capacity considers side and base resistances.
3. Pile uplift capacity can be estimated by using the unfactored side resistance shown in the plot and a recommended resistance factor.
4. Unfactored downdrag force is estimated to be **49** tons due to embankment loading-induced secondary compression settlement.
5. Unfactored downdrag force is estimated to be **45 tons** due to liquefaction-induced settlement. Downdrag force is recommended to be applied with post-earthquake loading

RESISTANCE FACTORS

Limit State	Side Friction	End Bearing	Uplift*
Strength			
AASHTO LRFD**	0.25 - 0.50	0.25 - 0.50	0.25- 0.40
AASHTO w/ PDA and CAPWAP**	0.65	0.65	0.60
Extreme	1.00	1.00	0.80

* See Note 3

** Per AASHTO LRFD Bridge Design Specifications Table 10.5.5.2.3-1 based on design or test method, see Note 4.

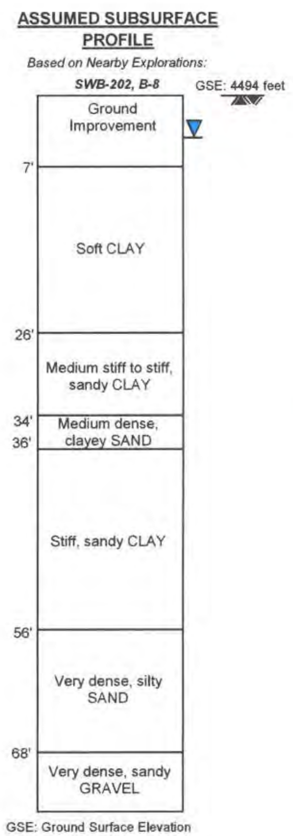
Pioneer Crossing, Lehi and
I-15 American Fork Interchange
Lehi, Utah

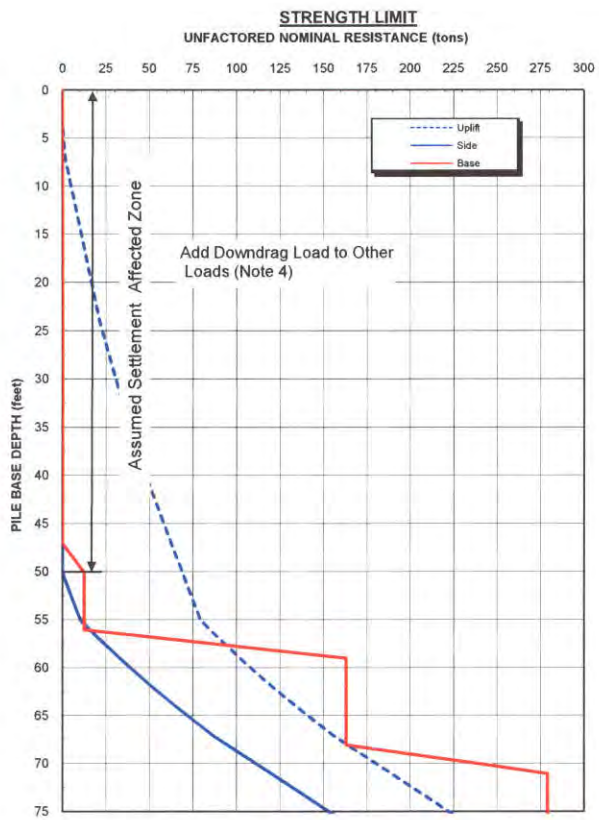
JORDAN RIVER BRIDGE-WEST ABUTMENT
ESTIMATED AXIAL CAPACITY FOR
16-INCH CIP PILE

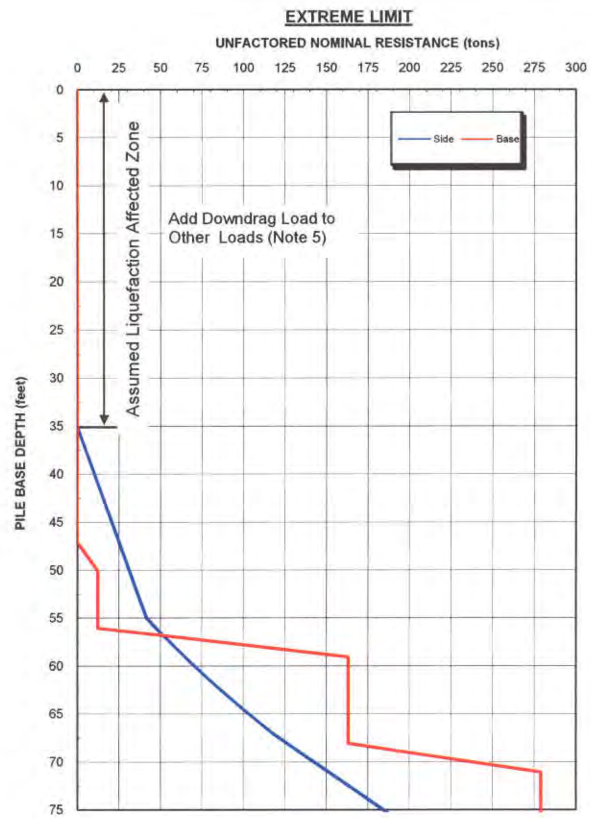
March 2009 | 23-1-01178-010

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG GD5-9







Notes:

1. The analyses are based on a single pile and do not consider group action of closely spaced piles (closer than 2.5 diameters, center to center).
2. The unfactored nominal resistances on the above plots should be multiplied by the appropriate resistance factors as noted in the table. Total pile compressive capacity considers side and base resistances.
3. Pile uplift capacity can be estimated by using the unfactored side resistance shown in the plot and a recommended resistance factor.
4. Unfactored downdrag force is estimated to be **69** tons due to embankment loading-induced secondary compression settlement.
5. Unfactored downdrag force is estimated to be **26.5 tons** due to liquefaction-induced settlement. Downdrag force is recommended to be applied with post-earthquake loading

RESISTANCE FACTORS

Limit State	Side Friction	End Bearing	Uplift*
Strength			
AASHTO LRFD**	0.25 - 0.5	0.25 - 0.5	0.25- 0.4
AASHTO w/ PDA and CAPWAP**	0.65	0.65	0.60
Extreme	1.00	1.00	0.80

* See Note 3

** Per AASHTO LRFD Bridge Design Specifications Table 10.5.5.2.3-1 based on design or test method, see Note 4.

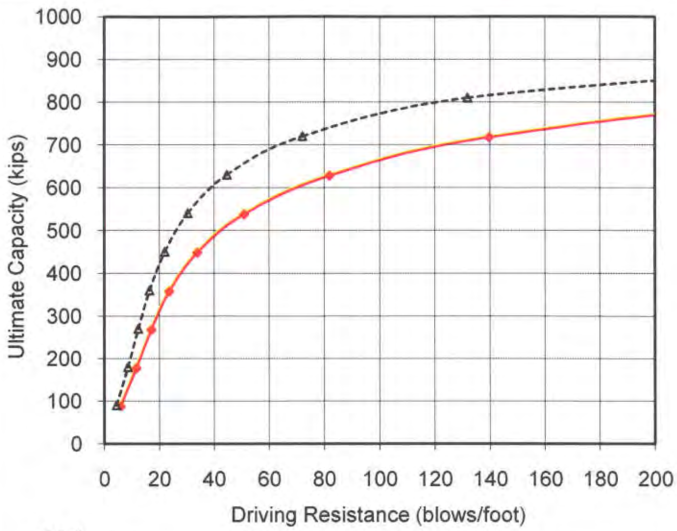
Pioneer Crossing, Lehi and
I-15 American Fork Interchange
Lehi, Utah

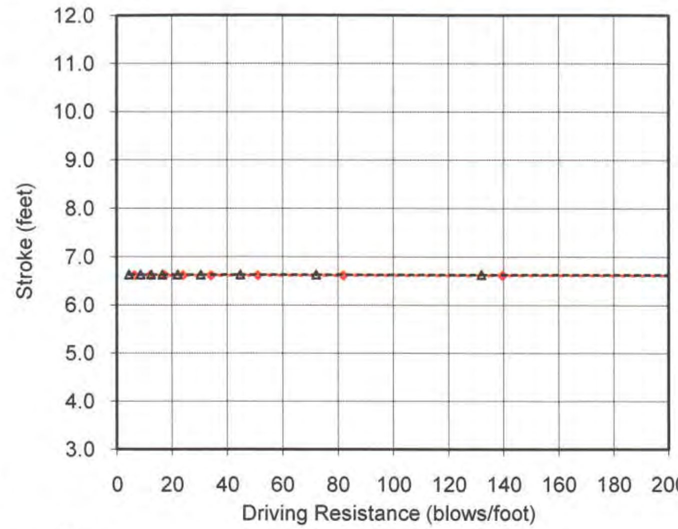
JORDAN RIVER BRIDGE-EAST ABUTMENT
ESTIMATED AXIAL CAPACITY FOR
16-INCH CIP PILE

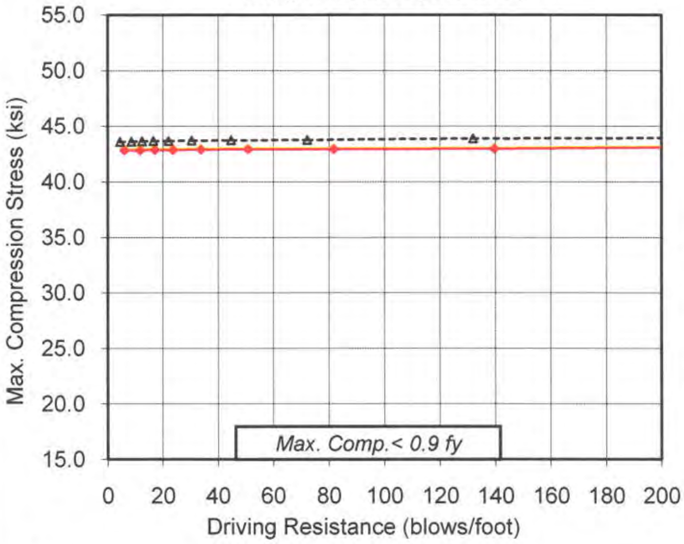
March 2009 23-1-01178-010

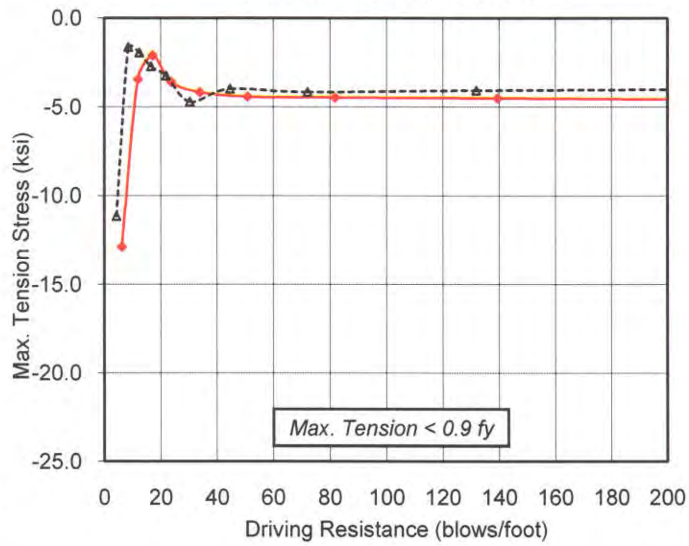
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

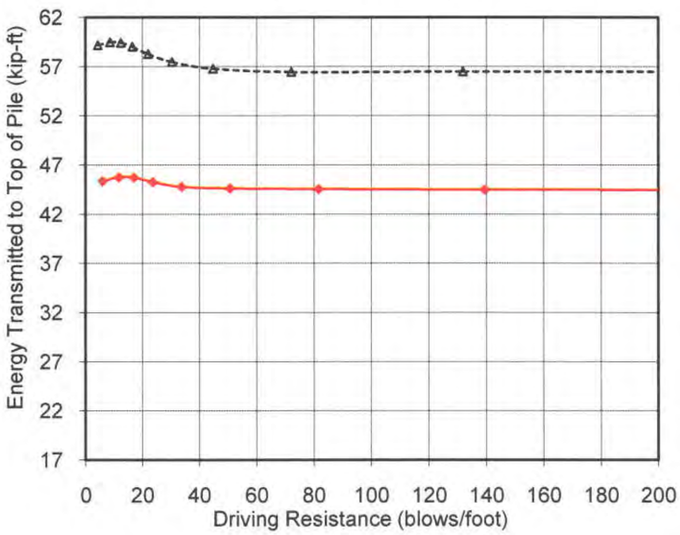
FIG GD5-10











GRLWEAP INPUT PARAMETERS

Pile Data		IHC S-70/90 Hammer		
Size	16 in			
Total Length	80 ft	Helmet Weight	1.72 kips	
Penetration	70 ft	Efficiency	0.95	
X-Sect. Area	24.35 in^2			
Circumference	4.2 ft	Hammer Cushion		
Material	Steel	Thickness	-	
Elast. Modulus	30000 ksi	Area	-	
Spec. Weight	492 pcf	Elast. Modulus	-	
Yield Strength (fy)	50 ksi	C.O.R.	-	
		Soil Parameters	Quake	Damping
Pile Cushion				
Thickness	-	Shaft	0.1	0.107
Area	-	Toe	0.133	0.15
Elast. Modulus	-			
C.O.R.	-	Shaft Resist.	50%	*Simple Dist.*

LEGEND:

IHC S-70 Wall Thickness = 0.5 inch

IHC S-90 Wall Thickness = 0.5 inch

NOTES:

1. The computer program GRLWEAP (Version 2005) was used with the above assumed input parameters to perform the analyses.
2. Standard GRLWEAP recommended values used for cushions and helmets.

Pioneer Crossing, Lehi and I-15 American Fork Interchange Lehi, Utah

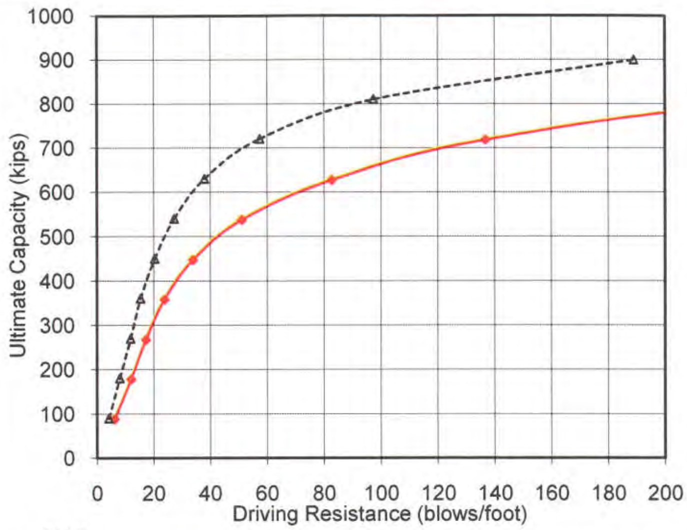
WEAP ANALYSIS, 16-INCH CONCRETE-FILLED PIPE PILE EAST ABUTMENT

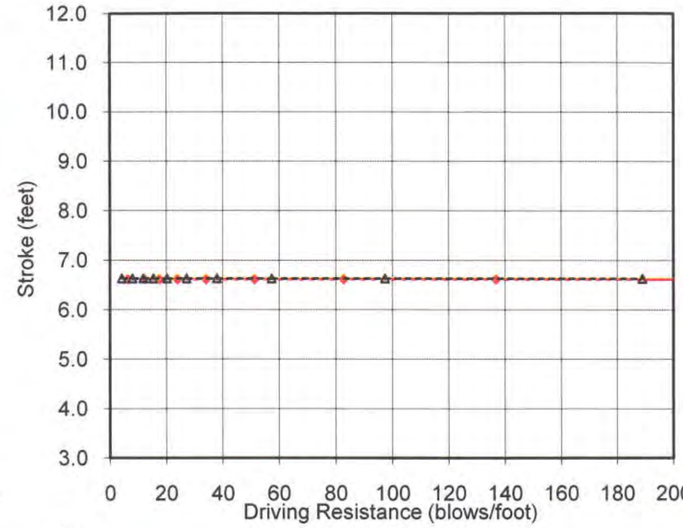
April 2009

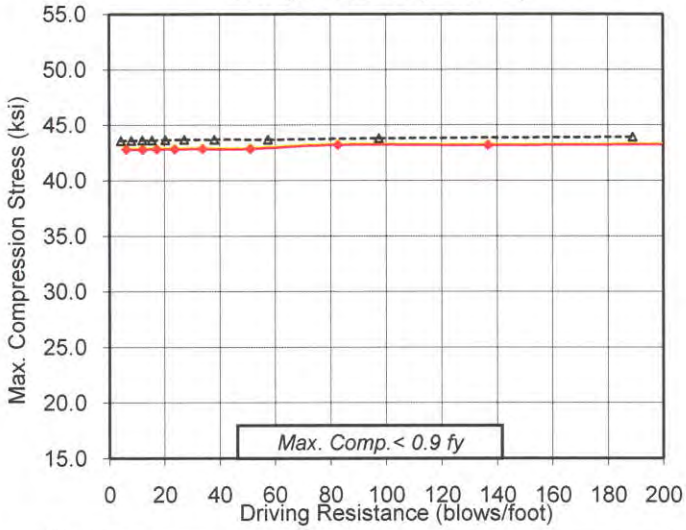
23-1-01178-010

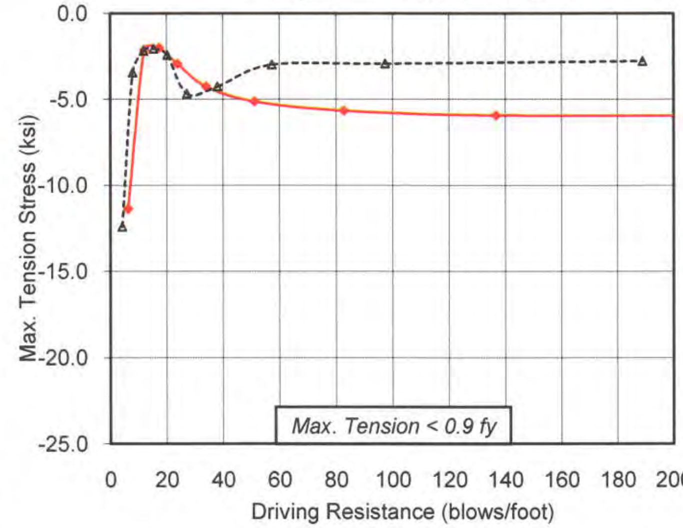
SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

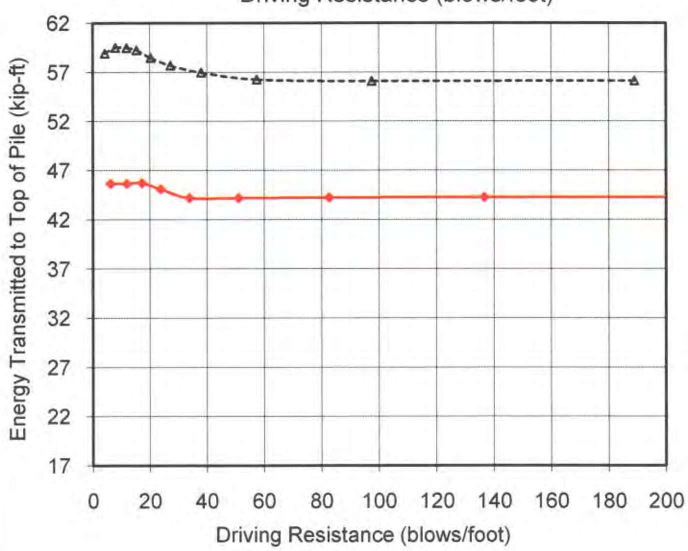
FIG. GD5-13











GRLWEAP INPUT PARAMETERS

	Pile Data		IHC S-70/90 Hammer	
Size	16 in			
Total Length	65 ft	Helmet Weight	1.72 kips	
Penetration	65 ft	Efficiency	0.95	
X-Sect. Area	24.35 in^2			
Circumference	4.2 ft		Hammer Cushion	
Material	Steel	Thickness	-	
Elast. Modulus	30000 ksi	Area	-	
Spec. Weight	492 pcf	Elast. Modulus	-	
Yield Strength (fy)	50 ksi	C.O.R.	-	
			Soil Parameters	
	Pile Cushion		Quake	Damping
Thickness	-	Shaft	0.1	0.103
Area	-	Toe	0.133	0.15
Elast. Modulus	-			
C.O.R.	-	Shaft Resist.	50%	*Simple Dist.*

LEGEND:

IHC S-70 Wall Thickness = 0.5 inch

IHC S-90 Wall Thickness = 0.5 inch

NOTES:

1. The computer program GRLWEAP (Version 2005) was used with the above assumed input parameters to perform the analyses.
2. Standard GRLWEAP recommended values used for cushions and helmets.

Pioneer Crossing, Lehi and I-15 American Fork Interchange Lehi, Utah

WEAP ANALYSIS, 16-INCH CONCRETE-FILLED PIPE PILE WEST ABUTMENT

April 2009 | 23-1-01178-010

SHANNON & WILSON, INC.
Geotechnical and Environmental Consultants

FIG. GD5-14

APPENDIX GD5-D

REQUEST FOR INFORMATION 019

PIONEER CROSSING
I-15 to Redwood Road

Pioneer Crossing Design-Build

KIEWIT / WW CLYDE, a Joint Venture

REQUEST FOR INFORMATION (OWNER)

OWNER PROJECT No.:	S-R399(42)/S-R399(59)	RFI No.:	019	DATE:	03/05/09
CONTRACTOR JOB No.:	478-12656	CONTRACTOR:	Kiewit/ Clyde		
CONTRACT FOR:	Pioneer Crossing, Lehi/I-15, American Fork Interchange				

To Be Completed By Contractor

Originator / Company: Tom Melton/Wilson & Co.

Reference: Drawing #s / Specification Sections: RFP Part 9, 9.2 Warranty Requirements, 9.2.1 A Pavements

Subject: Approach Slab Settlement

Request: ☑ Clarification ☐ Change

It is our understanding that the referenced Section of the RFP Contract Documents refers to pavement settlement and does not address the structure approach slabs at the end of a bridge. Furthermore, it is our understanding that profilometer measurements stop short of the approach slabs near a bridge. This limit of pofilometer measurement is logical in that it is unrealistic to expect 0.25" in 30 feet smoothness at the end of a bridge no matter how good the soil is underneath the approach slab.

Recommended Solution: ☐ CHECK IF CHANGE ORDER MAY BE REQUIRED **Priority Designation: 2 Day**

Please clarify the intent of the Warranty Requirements for approach slabs at the end of a bridge. We propose to limit the post-construction settlement at the end of the 25 foot approach slab to 1.5 inches, and to not require the smoothness requirement for pavement of 0.25 inches in 30 feet which is listed in 9.2.1.A.2. The logic behind the 1.5 inch settlement is based on allowing a maximum 1.0% grade break at the approach slab joint. Thus, using 0.5% either side of approach slab joint (for a total of 1%) allows 25 feet times 0.5% on the approach slab for a total of 1.5 inches.

Recommended By: Tom Melton

To Be Completed By Purchaser / A/E

Response:

The following instruction is issued to:

☐ CLARIFY DRAWINGS, SPECIFICATIONS, AND/OR PROCEDURES

☐ APPROVE USE OF MATERIALS OR EQUIPMENT

☑ NOTICE TO PROCEED SUBJECT TO CONDITIONS STIPULATED HEREIN

☐ DISPOSITION TO BE DETERMINED BY PURCHASER / A/E BY: ______ Date

Conditions:

AUC concurs with the recommended solution provided above.

☐ Reference Documents Attached

Recommended By: Darin Sjoblom, Boyd Wheeler

AUTHORIZATION TO PROCEED:

Jose Luis Rodriguez
UDOT

03/11/09
Date

RECEIVED 03/05/09

APPENDIX G

GEOTECHNICAL DESIGN MEMORANDUM GIP-1

SEGMENT 1 GEOTECHNICAL INSTRUMENTATION PLAN GIP-1

PIONEER CROSSING, LEHI AND I-15 AMERICAN FORK INTERCHANGE, UTAH

March 6, 2009

Prepared by:
Shannon & Wilson, Inc.,
Geotechnical Consultant for Kiewit/Clyde Joint Venture

PROFESSIONAL ENGINEER
No. 7030264-2202
CHRISTOPHER
ALAN
ROBERTSON
STATE OF UTAH

8/28/09

Christopher A. Robertson, P.E.
Vice President

WJP:CAR/wjp

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PLAN SHEETS

SEGMENT 1 GEOTECHNICAL INSTRUMENTATION PLAN GIP-1

PIONEER CROSSING, LEHI AND AMERICAN FORK INTERCHANGE, UTAH

1.0 INSTRUMENTATION PLAN SCOPE

This geotechnical instrumentation plan addresses monitoring of the proposed mechanically stabilized earth (MSE) wall R-575A and embankment settlement and stability during construction and surcharging. A description of the embankments and associated MSE wall, wick drains, and temporary surcharge fills are provided in Geotechnical Design Memorandum GD-5, "Pioneer Crossing Approach Fill, Embankment Stability, and Jordan River Bridge" (Shannon & Wilson, 2009).

2.0 INSTRUMENTATION PLAN DESCRIPTION

Instrumentation should be installed to monitor consolidation and elastic settlement of the embankments and the response of the planned MSE wall to embankment construction. Data collected from the monitoring program will be used to evaluate the behavior of the embankment fills and new MSE wall. To obtain this data, the instrumentation plan consists of the following. Instrumentation locations are shown in the enclosed Wick Drain & Surcharge Plan – Segment 1 sheets WS-001, WS-004, WS-005, WS-006, WS-007, and WS-107.

- Optical Surveys

  Consolidation Settlement Platforms (SPs) – Beneath permanent and surcharge fills. Nineteen points total. SPs will be used to monitor embankment settlement and to evaluate the percent of primary consolidation and when to remove surcharge fills.

- Inclinometers

  Vertical Inclinometers (INs) –Eight inclinometers total, which will be located near abutment faces, sides of abutment fill, and in front of the MSE wall. The data from the INs will be used to measure lateral soil movement that could be indicative of embankment instability.

3.0 INSTRUMENTATION, MONITORING FREQUENCY, REPORTING, AND ACTIONS

The following sections describe the proposed instrumentation, installation, reading frequency, reporting requirements, and action plans.

3.1 Instrumentation

SPs and top of INs casing will be monitored using optical survey methods accurate to 0.01 foot or better. Surveying will include horizontal and vertical measurements.

Each instrument or monitoring point will be surveyed within one to two working days after installation has been completed.

3.1.1 Consolidation Settlement Platforms (SPs)

Install SPs at locations shown in Wick Drain & Surcharge Plan – Segment 1 sheets WS-001, WS-004, WS-005, WS-006, and WS-007 and as indicated on WS-107. Install SPs prior to fill placement and in accordance with the Request for Proposal Technical Special Provision 02312S.

SPs shall be a square steel plate onto which is welded a steel coupler to receive a steel riser pipe. Pipe sections shall have standard threads and shall be joined using standard steel couplings. If directed by the Geotechnical Engineer, the pipe shall be protected and isolated from the surrounding ground by a 3-inch polyvinyl chloride (PVC) tube. The pipe shall be centered in the PVC tube.

As the construction of the embankment proceeds, extend steel pipe and PVC pipe in 5-foot increments with couplers, maintaining top of pipes at least 2 feet above fill surface at all time.

3.1.2 Vertical Inclinometers (INs)

Install INs at locations shown in Wick Drain & Surcharge Plan – Segment 1 sheets WS-004, WS-005, and WS-006, and as indicated on WS-107 prior to construction of adjacent embankment fills.

IN casing shall be Slope Indicator Co. Models 51101100 or 51150210, or equal of 2.75-inch outside diameter, 2.32-inch inside diameter with internal grooves at 90-degree intervals. Use at least two telescoping casings (Slope Indicator Co. Models 51150220 or 51107400 or equal) in each IN. Use couplings and end caps as provided by casing manufacturer.

Install inclinometer casings so that they extend at least 5 feet into the dense sand present near elevation 4,448 feet beneath the west abutment and elevation 4,440 feet beneath the east abutment or as recommended by the Geotechnical Engineer. For the MSE wall, install inclinometers approximately 35 feet below the existing ground surface or as recommended by the Geotechnical Engineer.

Orient inclinometer casing so that the orthogonal grooves are positioned parallel and perpendicular to the expected direction of movement, typically perpendicular to the long axis of the face of the embankment. Temporarily close top of casing to prevent entry of foreign material.

Fill casing with water and anchor casing as required to prevent it from floating out of the borehole during installation. Fill the annular void between the drill hole and the inclinometer casing with bentonite-cement grout consisting of 94 lbs. of Portland Type I or II cement, 30 gallons of water, and 25 lbs. of bentonite. Pump grout into annular void in vertical stages through a tremie tube.

Install minimum 6-inch-diameter cast-iron or steel, lockable enclosure grouted in place for protective cap concentric with inclinometer casing to a depth of at least 3 feet below ground surface. Center inclinometer casing inside steel casing and fill annulus with grout to 6 inches below top of inclinometer casing. Backfill around outside of 6-inch-diameter casing to ground surface with concrete to ensure that casing will remain in position.

3.2 Monitoring Frequency

Table 1 describes the monitoring frequency, including:

- Initial measurements
- Timing relative to construction activities
- Comments

The frequency of reading an instrument may be increased by at the discretion of the Geotechnical Engineer due to increased rates of embankment construction or other conditions.

3.3 Data Reduction and Reporting

Data will be collected, reduced, and presented in useful, legible, and well-labeled tables and plots. Reduced and interpreted data will be made available to the Geotechnical Engineer the same day that measurements are made, or if impracticable, by 10 a.m. the following day. In general, plots and accompanying documentation will include construction information such as describing areas of subgrade preparation, overexcavation, fill placement, and ground modification.

3.4 Monitoring Action Plan

3.4.1 Settlement and Surcharge Removal

The Geotechnical Engineer shall review settlement data to evaluate the progress of settlement. The Asaoka (1978)1 method will be used to evaluate the surface settlement data. East of Station 607+00, surcharge removal is planned when surface settlements are approximately 95 percent of the primary consolidation settlement estimated using the Asaoka (1978) method. This degree of consolidation was assumed in our design so layers contributing to large settlement should be slightly overconsolidated when the surcharge is removed. A degree of overconsolidation was assumed in our secondary settlement calculations. Based on the review of settlement data, the Geotechnical Engineer will provide written recommendations for when surcharge may be removed. Surcharge will not be removed until written recommendations are provided.

3.4.2 Embankment Stability During Construction

The Geotechnical Engineer will review the instrumentation and monitoring data and evaluate the stability of the embankments during fill construction. Where surcharge is shown in the plans, the fill placement for individual work areas shall stop when the permanent grade levels are established. The Geotechnical Engineer shall review all instrumentation and monitoring data associated with the active fill area and provide written recommendations to the Construction Manager that the surcharge fill may be placed. The surcharge fill shall not be placed until this written recommendation is provided.

If monitoring during construction indicates embankment instability, the following actions shall be taken:

- The Geotechnical Engineer will notify the Construction Manager, who will halt embankment construction.
- A corrective action plan will be developed by the Geotechnical Engineer and the Construction Manager before work resumes.
- Implement approved corrective actions.
- Verify success of corrective actions.
- If corrective actions are not successful, cease all related operations contributing to the displacements and repeat the process listed above.

---

1 Asaoka, A., 1978, Observational procedures or settlement prediction: Soils and Foundation, v. 18, no. 4, p. 87-100.

- If necessary, corrective actions may require modification of construction procedures or methods.

For embankment stability, a displacement ratio (horizontal displacement/vertical settlement) of 0.4 is the threshold level above which the action plan is implemented. This ratio is a tangent and not a secant value. This ratio is based on reported ratios and embankment performance for previous I-15 widening projects in Salt Lake City.

4.0 INSTRUMENTATION REPORTING PROCEDURES AND PERSONNEL

The "geotechnical engineer" referenced these documents is Mr. Robertson (Shannon & Wilson [S&W]) or his designee Mr. Perkins (S&W).

Optical survey measurements will be performed by Kiewit/Clyde (KC).

Initial vertical inclinometer readings will be performed by Xiao Hui Liu (S&W). Mr. Liu will train KC personnel to make subsequent inclinometer readings. After training, KC personnel will perform inclinometer readings with periodic assistance from Mr. Liu as needed.

Prior to placement of embankment fill, Mr. Liu will confirm with KC that instrumentation has been installed and that the initial readings have been made.

Instrumentation data will be collected at the frequency shown on Table 1 and assembled by KC surveyors. This information will include embankment/surcharge cross-sections at the time the readings are made. KC will forward (e-mail) data to Mr. Lui (S&W), William Chou (S&W), Bill Perkins (S&W), and Chris Robertson (S&W) within 24 hours from the time when the measurements are made. Mr. Lui will have primary responsibility to make sure the data are received, plotted, and interpreted and that reports are prepared and provided. Mr. Chou will be available as the secondary contact in case Mr. Liu cannot respond (illness, vacation, etc.). Messrs. Perkins and Robertson will receive the data for further backup purposes in case of unplanned absences. E-mail addresses are:

Xiao Hiu Liu	xhl@shanwil.com
William Chou	yhc@shanwil.com
Bill Perkins	wjp@shanwil.com
Chris Robertson	car@shanwil.com

S&W will plot and review the instrumentation data daily as it provided and prepare weekly instrumentation summary reports under the supervision of Mr. Christopher A. Robertson, P.E..

The reports will be submitted weekly to KC, Access Utah County (AUC)/Utah Department of Transportation (UDOT) resident engineer, and UDOT geotechnical engineer Darin Sjoblom (dsjoblom@utah.gov).

5.0 HOLD POINTS

At the end of embankment construction and prior surcharge placement, instrumentation data and embankment stability will be reviewed by the Geotechnical Engineer prior to proceeding with surcharge placement. Written notification to proceed with surcharge placement will be provided to KC with copies to the AUC/UDOT resident engineer and Mr. Sjoblom (UDOT) from Mr. Robertson (S&W) or his designee, Mr. Perkins (S&W).

For settlement surcharge removal east of Station 607+00, the Geotechnical Engineer will confer with Steve Saye (KC) and Mr. Sjoblom (UDOT) prior to providing written notification for surcharge removal. Surcharge west of Station 607+00 is related to reducing settlements of a proposed 60-inch-diameter water line to be placed beneath the roadway and has been designed by Terracon Consultants, Inc. Therefore, Terracon is responsible for determining the degree of consolidation and when the surcharge west of Station 607+00 can be removed for water line settlement purposes.

TABLE 1
INSTRUMENTATION READING FREQUENCY

Instrumentation	Initial Reading[2]	Reading Frequency[1]: Embankment Construction to Surcharge Removal	Reading Frequency[1]: After Surcharge Removal	Comments
Consolidation Settlement Platform (SP)	Twice prior to fill placement	Daily[3]		Include time, temperature/weather, fill height and geometry peculiarities at time of reading.
Vertical Inclinometer (IN)	Twice prior to fill placement	Weekly	Monthly	Includes optical survey of top of casing.

Notes:
[1] Take readings at the same time of day for a given instrument.
[2] Initial readings are required to establish baseline. If the baseline is not established with the indicated number of initial readings, additional readings should be made until a reliable baseline is established.
[3] Daily measurements are not needed during periods when embankment or surcharge are not actively being placed prior to completion of surcharge placement.

APPENDIX H

REQUEST FOR PROPOSAL BASELINE GEOTECHNICAL REPORT

APPENDIX H

REQUEST FOR PROPOSAL BASELINE GEOTECHNICAL REPORT

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REPORT

GEOTECHNICAL BASELINE REPORT

EAST-WEST CONNECTOR
AMERICAN FORK, LEHI, SARATOGA SPRINGS, UTAH
UDOT PROJECT NO. SR-399(38)

Terracon Project No. 61085026
June 12, 2008

Prepared for:

HDR, Inc.
3995 South 700 East, Suite 100
Salt Lake City, Utah 84107

Prepared by:

TERRACON CONSULTANTS, INC.
12217 South Lone Peak Pkwy. Suite 100
Draper, Utah 84020

Terracon

Terracon
Consulting Engineers & Scientists
Terracon Consultants, Inc.
12217 South Lone Peak Parkway, Suite 100
Draper, Utah 84020
1550 North 320 West, Suite J
Layton, Utah 84041
Phone 801.545.8500
Fax 801.545.8600
www.terracon.com

June 12, 2008

HDR, Inc.
3995 South 700 East, Suite 100
Salt Lake City, Utah, 84107

Attn: Mr. John Buttenob

Re: Geotechnical Baseline Report
East – West Connector
American Fork, Lehi, Saratoga Springs, Utah
UDOT Project No. SR-399(38)
Terracon Project No. 61085026

Mr. Buttenob:

At your request, Terracon Consultants, Inc. (Terracon) has prepared this geotechnical baseline report based on our field exploration activities and literature review. Preparation of this report was authorized by you on April 4, 2008 and has been completed in general conformance with our Scope and Fee Estimate for Preliminary Geotechnical Engineering Services, dated January 31, 2008. The report describes the exploration and presents preliminary geotechnical data for reference by design-build teams preparing design-build proposals for the project.

We appreciate the opportunity to be of service to you on this project. If you have any questions concerning this report, or if we may be of further service, please contact us.

Sincerely,
TERRACON CONSULTANTS

Rick L. Chesnut, P.E., P.G.
Principal / Office Manager

Curtis J. Tanner, P.E.
Geotech Department Manager

RLC/CJT/sm
Copies To: Addressee (2, Electronic)
N:\Projects\2008\61085026\61085026 rpt.doc

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GEOTECHNICAL BASELINE REPORT

EAST – WEST CONNECTOR
AMERICAN FORK, LEHI, SARATOGA SPRINGS, UTAH
UDOT PROJECT NO. SR-399(38)

Terracon Project No. 61085026
June 12, 2008

1.0 GENERAL

This report has been prepared to aid Design-Build contractors in evaluating subsurface conditions along the project, to assist the owner and designers in reviewing the Contractors submittal and operations and to establish a geotechnical baseline which will serve as a basis for identification of differing site conditions.

The information and recommendations presented in this report should be considered preliminary in nature. We understand that this report will be used by design-build contractors to prepare design-build bid submittals for the project. Additional field exploration, laboratory testing and analysis may be performed by the design-build contractors. Final design recommendations will be developed by the selected design-build contractor and geotechnical engineer.

2.0 PROJECT DESCRIPTION

Based on our understanding of the subject project and an alignment map provided by Horrocks Engineers we understand that this project consists of constructing a major east to west corridor through Lehi connecting Redwood Road to I-15. The corridor will parallel SR- 73 to the south at approximately 900 South. Current land use in the corridor is mostly agricultural. As part of the project we understand that the existing I-15 interchange located at Main Street in American Fork will be replaced with a new Single Point Urban Interchange (SPUI). New bridge structures will be constructed to extend the roadway over the UPRR tracks near Mill Pond and over the Jordan River in Saratoga Springs. In addition to the proposed structures we understand that approximately 6 miles of new pavement will be constructed along the project corridor.

2.1.1 Proposed Alignment

The proposed alignment begins on Redwood Road at approximately 600 North in Saratoga Springs and will extend east to the Jordan River. The alignment crosses the Jordan River and continues to the northeast to approximately 3200 West (county address) and then extends directly east along approximately 900 South in Lehi to approximately 1700 West.

The alignment then turns slightly southeast and extends to 790 West and 1250 South and then turns east continuing to approximately 300 East and 1250 South. East of this point the alignment bends northeast and extends to Mill Pond Road and 850 East. The alignment then continues east to I-15.

The majority of the alignment consists of agricultural fields, preserved corridor and undeveloped land. The portion of the alignment between the Jordan River and approximately 2900 West (county address) in Saratoga Springs was covered with several inches to two feet of standing water.

Existing commercial facilities are located on the proposed alignment between Mill Pond Raod and I-15. These facilities will be removed to facilitate construction of the proposed roadway.

2.1.2 Proposed Improvements

Proposed improvements include construction of new pavement sections along the corridor. The pavement section may either consist of a flexible asphaltic concrete section or a Portland Cement Concrete Pavement (PCCP) section. New on and off ramps to I-15 from the proposed roadway will be constructed to facilitate the proposed SPUI at I-15. A new lane will be constructed along I-15 to facilitate merging of traffic onto I-15 from the proposed ramps.

A bridge structure will be constructed at the Jordan River. We understand that this structure will be a single-span bridge constructed at-grade. It is anticipated that approach fills to the bridge will be minimal.

A bridge structure will be constructed to extend the proposed roadway over Mill Pond Road and the existing UPRR tracks. We understand that this structure will be a multi-span, elevated bridge. Approach fills to the bridge abutments may be 25 to 38 feet tall.

A new bridge and SPUI will be constructed at I-15. The existing bridge alignment will be moved to the south and the alignment skewed to the southwest. We understand that the SPUI bridge will be a multi-span, elevated structure. Approach fills to the bridge may be 15 to 28 feet tall.

3.0 PREVIOUS REPORTS AND INVESTIGATIONS

UDOT completed previous geotechnical explorations at the existing I-15 American Fork Main Street interchange. Subsurface conditions encountered during those explorations are summarized on the bridge (C-344) soil data sheets. These plan sheets were provided to Terracon by UDOT and are included in Appendix D.

4.0 EXISTING FACILITIES

4.1 Bridges

The existing interchange at I-15 consists of a diamond-shape configuration. The existing bridge is an elevated structure over I-15 with multiple spans. One center bent supports the bridge between the north and southbound I-15 travel lanes. No other bridge structures were observed along the alignment.

4.2 Pavements

The majority of the alignment crosses agricultural fields and preserved corridor. Existing roadway crossings along the alignment include Mill Pond Road, 300 East, Lehi Center Street, 500 West, 1100 West, 1700 West and Saratoga Road (9550 West) streets. Pavements on these roads consist of asphaltic concrete sections. Asphaltic concrete pavements were also observed on the existing I-15 ramps.

4.3 Buildings

There are existing commercial buildings on the proposed alignment between Mill Pond Road and I-15. These facilities consist of an automobile salvage yard and a concrete batch plant. We understand that the two facilities will be demolished and removed from the site prior to construction.

5.0 FINDINGS

5.1 Site Conditions

The majority of the site consists of undeveloped land and preserved corridor. With the exception of the east end of the corridor where commercial buildings exist as described above agricultural fields and pasture make up the majority of the existing corridor. Standing water was encountered on the west portion of the corridor near the east side of the Jordan River. Site conditions adjacent to the alignment are generally flat and moderately vegetated with grass and weeds and some trees.

Surface soils along portions of the alignment were observed to be soft and susceptible to pumping and rutting. These conditions may become more prevalent following grubbing of topsoil and near-surface fills and during times of precipitation.

5.2 Geology

A published geologic map[1] indicates geologic units mapped along the corridor consist of Holocene to Upper Pleistocene alluvial and lacustrine soil deposits. Individual deposits identified on the referenced map are described as follows.

- Holocene to Upper Pleistocene younger undifferentiated alluvial fan deposits (Qafy) - estimated to be up to tens of feet in thickness.
- Holocene to Upper Pleistocene Lacustrine and alluvial deposits (Qlay) - well-sorted fine-grained sand, silt and clay deposited in Mill Pond area. Estimated to be less than 15 feet in thickness.
- Upper Pleistocene Lacustrine silt and clay (Qlmp) – calcarious silt (marl) with minor clay and fine-grained.

Bedrock units were not identified on the referenced map or observed at the site during our preliminary field exploration.

The site is located within 2 miles of the Holocene Wasatch fault and Utah Lake faults. These faults are described in more detail in the Seismic Considerations Section of the report.

No landslides or lateral spread areas were identified on reviewed maps or observed at the site during our field exploration.

5.3 Soil Materials

Subsurface conditions encountered at the site are indicated on the boring logs and CPT logs in Appendix A. The stratification lines shown on the logs represent the approximate boundary between the soil types encountered; the actual transition may be gradual.

Fill consisting of sand and gravel was encountered to depths ranging between 2 and 5 feet below existing grade in borings B-7, B-19 and B-20 through B-28. Soil conditions encountered below the fill generally consist of sandy clay, lean clay, silty clay, clayey sand, silty sand, sand, sandy gravel, clayey gravel and gravel to the maximum depth explored of 103 feet.

The clays were generally very soft to hard with N-values ranging from 2 blows per foot of penetration and refusal (greater than 50 blows per foot of penetration). The sands and gravels were generally very loose to very dense with N-values ranging from 2 blows per foot of penetration and refusal. Laboratory test results are summarized on the attached boring logs and laboratory test summary sheets.

5.4 Geohydrologic Conditions

[1] Biek, Robert F., 2005, *Geological Map of the Lehi Quadrangle and Part of the Timpanogos Cave Quadrangle, Salt Lake and Utah Counties, Utah*, Utah Geologic Survey Map 210.

The presence of and approximate level of groundwater during drilling was observed. At the time of our field exploration, groundwater was encountered at depths ranging between about 2 and 20 feet in the borings while drilling. Groundwater was not encountered in borings B-21, B-22 and B-25 through B-28. It should be recognized that fluctuations of the groundwater table may occur due to seasonal and long term variations in the amount of rainfall, runoff and other factors not evident at the time the borings were performed. Evaluating these factors is beyond the scope of this exploration.

5.5 Potentially Hazardous Materials

Evaluating potentially hazardous materials is beyond the scope of the geotechnical baseline report. Limited soil sampling and analytical testing was completed in borings B-19 and B-20 as requested by the client. Results of those samples and tests will be summarized in a separate report.

6.0 SEISMIC CONSIDERATIONS

6.1 Fault Rupture

Published geologic maps were reviewed to aid in assessing the location and distance of late Quaternary faults near the site (Hecker, 2003). Maximum Considered Earthquake (MCE) data presented in available literature was gathered for each fault. Where MCE information was not readily available, a magnitude was estimated based on a correlation by Wells and Coppersmith (1994) for fault rupture length.

QUATERNARY FAULTS

Fault	Magnitude	Distance (mi)
Wasatch Fault – Provo Segment	7.3[1]	1.2 to 8.6
Utah Lake faults	6.5[1]	1.5 to 2
Wasatch Fault – Salt Lake Segment	7.2[1]	8 to 8.6
West Valley Fault Zone	6.5[1]	18 to 19

[1]Calculated from mapped rupture length from U.S. Geological Survey (2007)

The Provo segment of the Wasatch fault, located approximately 1.2 to 8.6 miles east of the site appears to produce the highest potential ground motion at the site. A review of geologic literature indicates that the Provo segment of the Wasatch fault is capable of producing an earthquake of magnitude 7.3 based on calculations from the mapped rupture length. The Wasatch fault occurs as a west facing scarp along the western base of the Wasatch Range. The most recent well-documented event on the segment occurred about 500 to 600 years ago.

6.2 Liquefaction and Dynamic Settlement

The proposed bridge sites are located in areas mapped on county planning amps as having moderate to high potential for liquefaction. A preliminary analysis of liquefaction was performed at each bridge site. Based on this analysis, liquefiable soils were encountered at each site. The following table summarizes approximate liquefiable soil layer depths and preliminary estimated settlement from those layers.

LIQUEFIABLE SOIL LAYERS – I-15 SPUI

Layer Depth Interval (ft)	Liquefaction Factor of Safety	Estimated Settlement (in)
16 to 22	0.4	2.0
35 to 36	0.5	0.5
44 to 45	0.8	0.5
	Total Settlement (in):	3.0

LIQUEFIABLE SOIL LAYERS – MILL POND ROAD BRIDGE

Layer Depth Interval (ft)	Liquefaction Factor of Safety	Estimated Settlement (in)
10 to 12	0.8	0.5
*21 to 24	0.4	0.4
*33 to 40	0.7	0.8
*47 to 50	0.7	0.4
	Total Settlement (in):	2.1

* Thin sand layers interbedded with clay within depth interval.

LIQUEFIABLE SOIL LAYERS – JORDAN RIVER BRIDGE

Layer Depth Interval (ft)	Liquefaction Factor of Safety	Estimated Settlement (in)
15 to 16	0.7	0.5
*27 to 45	0.6	3.2
	Total Settlement (in):	3.7

* Thin sand layers interbedded with clay within depth interval.

6.3 Lateral Spread

A preliminary estimate of lateral spread potential at each of the three bridge sites was performed in general conformance with procedures by Youd (2002)[2]. Lateral spread was estimated at all three of the bridge sites using a sloping ground condition.

[2] *Youd, T. Leslie, Corbett M. Hansen, and Steven F. Bartlett, "Revised Multilinear Regression Equations for Prediction of Lateral Spread Displacement", Journal of Geotechnical and Geoenvironmental Engineering, Vol. 128, No. 12, December 1, 2002*

A free face condition was apparent at the Jordan River however the depths of the liquefiable layers were significantly below the bottom level of the free face. Preliminary estimates of lateral spread were 13 to 18 inches at the proposed Jordan River and Mill Pond Road bridges and 16 to 20 inches at the proposed I-15 SPUI.

6.4 Seismic Design Criteria

Earthquake databases (2002 NEHRP) were reviewed to determine spectral accelerations for MCEER Soil Profile Type B, Firm Rock Sites at the bridge locations. These accelerations are summarized in the following table for events representative of 2% and 10% probabilities of exceedance (PE) in a 50 year period. These events correlate to return intervals of about 2,475 years and 475 years, respectively.

RESPONSE SPECTRAL ACCELERATION VALUES – SITE CLASS B

Structure Location	Spectral Acceleration (%g)			
	2% in 50 Year		10% in 50 Year	
	0.2 sec S_S	1.0 sec S_1	0.2 sec, S_S	1.0 sec S_1
I-15 Bridge	1.18	0.50	0.47	0.16
Mill Pond Road Bridge	1.17	0.50	0.47	0.16
Jordan River Bridge	1.04	0.42	0.45	0.15

Based on the results of our exploration the subsurface soil profile in the upper 100-feet at the sites is best represented by Site Class D according to MCEER guidelines. MCEER Site Coefficients F_a and F_v for Site Class D are 1.0 and 1.5, respectively.

PGA VALUES

Structure Location	PGA (%g)	
	2% in 50 Year	10% in 50 Year
I-15 Bridge	0.51	0.20
Mill Pond Road Bridge	0.50	0.20
Jordan River Bridge	0.43	0.19

Proposed bridge structure locations at the Jordan River, Mill Pond Road/Union Pacific Railroad (UPRR) crossings and at the proposed I-15 SPUI are located in areas mapped as having moderate to high potential for liquefaction based on maps published by the Utah Geological Survey[3]. This report provides preliminary estimates of liquefaction-induced settlement at the proposed bridge locations. In addition, 13 to 20 inches of lateral spread has been estimated at the proposed bridge sites. The Design-Builder is responsible for final liquefaction and lateral spread evaluation along the project and design of structures to accommodate the occurrence of these hazards. If it is determined by the Design-Builder that structural design of the bridge structures cannot effectively accommodate the projected settlements or ground displacement, a ground improvement program consisting of properly designed stone columns or other approved system should be prepared to mitigate lateral spread and/or liquefaction at the bridge sites. At a minimum, the Design-Builder should design the structures or mitigate the site for the following estimated liquefaction-induced settlement and lateral spread values.

MINIMUM SEISMIC-INDUCED GROUND DEFORMATION VALUES

Bridge	Liquefaction Settlement (in)	Lateral Spread (in)
Jordan River Bridge	3.7	18
Mill Pond Road/UPRR Tracks Bridge	2.1	18
I-15 SPUI	3.0	20

7.0 FIELD AND LABORATORY TEST DATA

7.1 Field Exploration

The subsurface exploration included drilling 28 borings to depths of approximately 11.5 to 103 feet below existing grade and completing 3 Cone Penetrometer Test (CPT) soundings to refusal at depths of approximately 55 to 76 feet. The CPTs were terminated upon refusal in dense sand layers. The approximate boring and CPT locations in relation to the proposed construction are shown on the Boring Location Plan, included in Appendix A. The borings and CPTs were located by reference to existing on-site features. The locations are approximate and should be considered accurate only to the degree implied by the means and methods used to determine them.

The borings were drilled with Mobile B-80 equipment using hollow-stem augers. Disturbed soil samples were collected at various depths utilizing a 2-inch outside-diameter split spoon sampler driven in general accordance with the standard penetration test (SPT). This test consists of driving the sampler into the ground with a 140-pound hammer free-falling through a distance of 30 inches.

[3] *"Liquefaction-Potential Map for a Part of Utah County, Utah"*, Utah Geological Survey, Public Information Series 28, 1994.

The number of blows required to advance the sampler the last 12 inches, or the interval indicated, of a typical 18-inch penetration is recorded as the standard penetration resistance value (N-value). These values are indicated on the boring logs at the respective sample depths.

The standard penetration test provides a rough estimate of the in-place density of sandy type materials, but only provides an indication of the relative stiffness of cohesive materials since the blow count in these soils may be affected by the water content. In addition, considerable care should be exercised in interpreting the N-values in gravelly soils, particularly where the size of the gravel particle exceeds the inside diameter of the sampler.

Relatively undisturbed samples were collected in cohesive soils utilizing 3-inch diameter thin-walled tubes pushed into the soil with the hydraulic head of the drill rig.

Terracon personnel prepared a log of each boring during drilling. The soil samples were packaged and transported to our Draper laboratory for further observation and testing. CPT soundings were completed and logged by ConeTec, Inc.

7.2 Laboratory Testing

Samples obtained during the field exploration were visually classified in the laboratory in general accordance with the Unified Soil Classification System (USCS). Where laboratory testing was completed, AASHTO soil classifications are included on the soil boring logs. The USCS and AASHTO soil classification system are described in Appendix B.

Selected soil samples were tested to determine physical and engineering properties and to aid in classification. Following are the laboratory tests performed and a brief description of each test:

Natural Water Content: The percentage of water in the soil at the sample location.

Percent Passing the No. 200 Sieve: Amount of combined clay and silt-sized particles in the soil sample.

Atterberg Limits: Consistency and range of moisture content within which the material exhibits a plastic behavior.

One-Dimensional Consolidation: Measurement of soil compressibility upon loading and soil behavior upon wetting at given loads.

Dry Unit Weight: Dry unit weight of soil at the sample location.

Triaxial Shear Test: Stress-strain strength properties of the soil under unconsolidated and undrained conditions.

Sulfate Content: Concentration of water-soluble sulfate constituents in the sample.

pH and Resistivity: Measurement of acidity in the sample and the ability of the sample to resist the flow of electricity.

California Bearing Ration (CBR): Potential strength of pavement subgrade material over varying moisture contents or compactive efforts.

Moisture-density Relationship (Proctor): Relationship of dry density versus moisture content under "modified" energy as described in ASTM D-1557.

Results of the laboratory tests are summarized on the boring logs in Appendix A and on the laboratory summary sheets in Appendix B.

8.0 GEOTECHNICAL CONSIDERATIONS

The Design-Builder is responsible for final geotechnical exploration and design for this project. Based on this preliminary exploration, it is our opinion that the alignment is suitable for the proposed construction, but the amount of consolidation and secondary settlement expected under the bridge approach fills and the potential for liquefaction and lateral spread at the bridge locations are significant design issues that must be considered.

Relatively large primary consolidation settlement is anticipated below the approach fills and should be evaluated by the design-builder. The wall manufacturer should be made aware of projected settlements so the differential settlements may be considered in the wall-type selection and design. It is anticipated that two-stage MSE wall systems may be required where settlements exceed about 8 to 10 inches.

The near-surface native soils encountered in the borings may be susceptible to disturbance or rutting under the weight of construction equipment. The contractor should consider this when planning construction activities and staging to avoid disturbance of the subgrade. Crushed, angular stone in combination with geotextiles may be required to provide a working surface.

9.0 EARTHWORK

Construction and earthwork operations should be performed in conformance with current UDOT bridge construction specifications.

10.0 CORROSION

Testing Engineers International tested nine soil samples for corrosion properties. Results of those tests are summarized in the following table.

Sample Location and Depth (feet)	PH	Resistivity (ohm-cm)	Soluble Sulfates (ppm)	Soluble Chlorides (ppm)
B-1A @ 5 – 7.5	7.9	502	--	--
B-2 @ 7.5	7.7	474	--	--
B-3 @ 10	8.8	1,040	--	--
B-5 @ 7.5	8.3	3.990	--	--
B-4 @ 10	7.7	1,110	17	41
B-9 @ 7.5	8.0	4,250	<10	94
B-12 @ 20	7.3	1,310	<10	58
B-15 @ 2.5	8.0	2,130	243	35
B-12 @ 10	8.0	4,630	<10	23

These values can be used to estimate the corrosion potential of the on-site soils in contact with buried metal and concrete.

11.0 CLOSURE

This report has been prepared for the exclusive use of our client for specific application to the project discussed and has been prepared in accordance with generally accepted geotechnical engineering practices. No warranties, either express or implied, are intended or made. Site safety, excavation support, and dewatering requirements are the responsibility of others. In the event that changes in the nature, design, or location of the project as outlined in this report are planned, the conclusions and recommendations contained in this report shall not be considered valid unless Terracon reviews the changes and either verifies or modifies the conclusions of this report in writing.

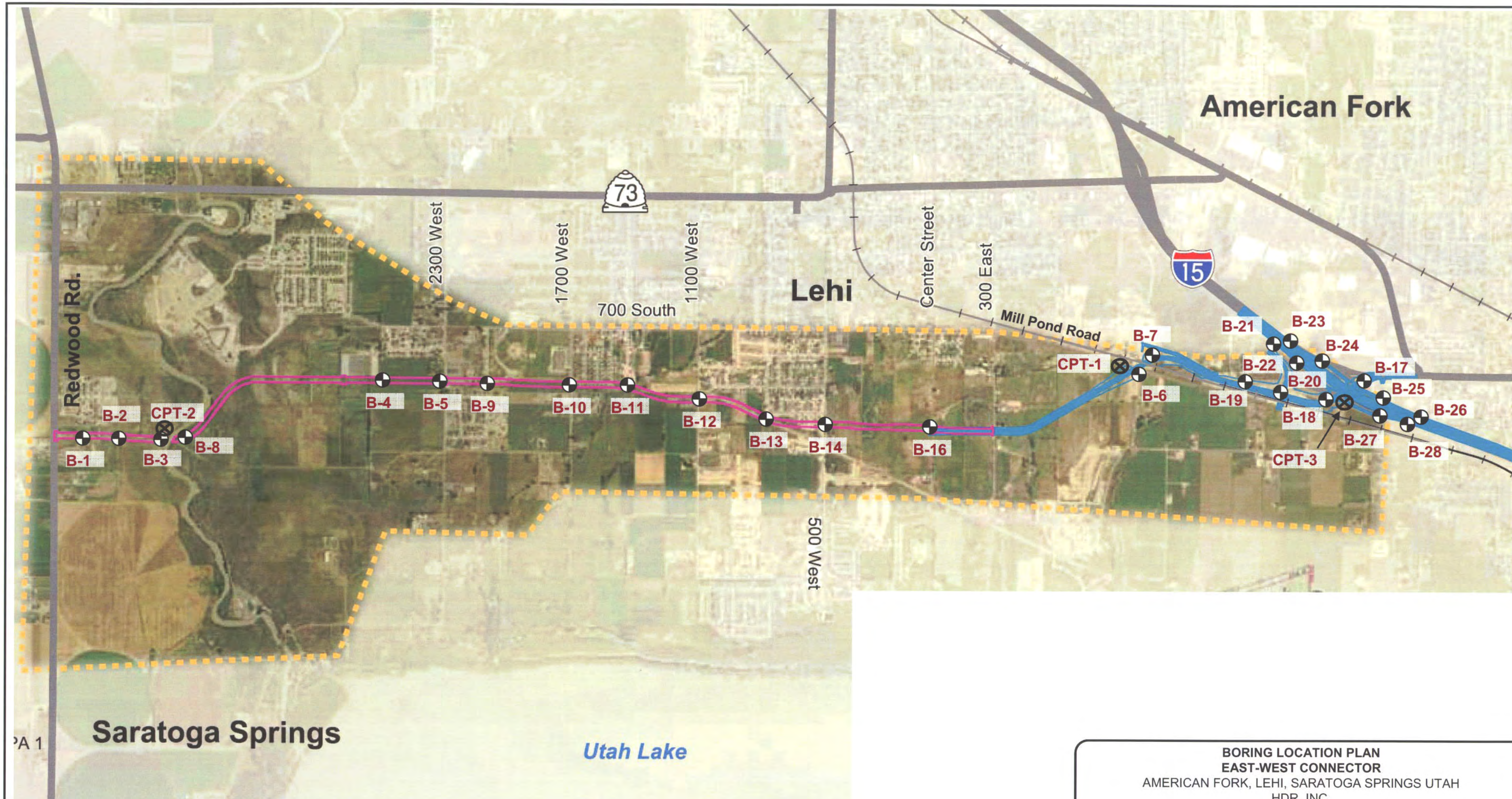
12.0 LIMITATIONS

The analysis and recommendations presented in this report are based upon the data obtained from the borings performed at the indicated locations and from other information discussed in this report. This report does not reflect variations that may occur between borings, across the site, or due to the modifying effects of weather. The nature and extent of such variations may not become evident until during or after construction. If variations appear, we should be immediately notified so that further evaluation and supplemental recommendations can be provided.

The scope of services for this project does not include either specifically or by implication any environmental assessment of the site or identification or prevention of pollutants, hazardous materials or conditions. If the owner is concerned about the potential for such contamination or pollution, other studies should be undertaken.

APPENDIX A

Boring Location Plan
Boring Logs



BORING LOCATION PLAN
EAST-WEST CONNECTOR
AMERICAN FORK, LEHI, SARATOGA SPRINGS UTAH
HDR, INC.

Project Mngr:	RLC	Terracon	**Project No.**	61085026
Designed By:	OTHER		**Scale:**	Not to Scale
Checked By:	RLC	12217 Lone Peak Pkwy., Ste. 100 Draper, Utah 84020 P: 801.545.8500 F: 801.545.8600	**Date:**	4/29/2008
Approved By:	RLC		**Drawn By:**	AC (61)
File Name:	N:\Projects\2008\61085026\Figure 2 Location Plan.ppt		**Figure No.**	2

REFERENCE: http://www.udot.utah.gov/ewconnector

DIAGRAM IS FOR GENERAL LOCATION ONLY, AND IS NOT INTENDED FOR CONSTRUCTION PURPOSES.

LOG OF BORING NO. B-01

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: **West of Jordan River near Redwood Road**

Depth (strata)	Description	DEPTH, ft.	USCS SYMBOL	NUMBER	TYPE	RECOVERY, in.	PENETRATION RESISTANCE BLOWS / ft.	WATER CONTENT, %	DRY UNIT WEIGHT, PCF	LIQUID LIMIT	PLASTICITY INDEX	% PASSING NO. 200 SIEVE	PID
0.25	**TOPSOIL:** clay, approximately 3 inches thick												
	SILTY CLAY (CL-ML): very stiff to hard, brown with white and red mottling, with trace gravel,	3–4		1	SS	10	41						
		5–6		2	SS	16	56						
		8–9		3	SS	18	27						
12.5		10–11		4	SS	18	20						
	CLAY (CL): with sand, very stiff, red to brown	13–14		5	SS	18	18						
20	▽	15–16		6	SS	18	17						
21.5	**SANDY CLAY (CL):** medium stiff, red to brown	20–21		7	SS	18	4						
	BOTTOM OF BORING AT APPROXIMATELY 21.5 FEET												

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL	▽ 20	WD ▼
WL	▼	▼
WL		

Terracon

BORING STARTED	3-18-08		
BORING COMPLETED	3-18-08		
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LOG OF BORING NO. B-01A

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: **West of Jordan River near Redwood Road**

Approx. Surface Elev.: 4518.3 ft

GRAPHIC LOG	Description	DEPTH, ft.	USCS SYMBOL	SAMPLES NUMBER	SAMPLES TYPE	SAMPLES RECOVERY, in.	SAMPLES PENETRATION RESISTANCE BLOWS / ft.	TESTS WATER CONTENT, %	TESTS DRY UNIT WEIGHT, PCF	TESTS LIQUID LIMIT	TESTS PLASTICITY INDEX	TESTS % PASSING NO. 200 SIEVE	PID
	1 **SILTY SAND (SM):** red-brown, with organics with trace gravel	1											
	SILTY CLAY (CL-ML): soft to hard, brown, with trace gravel	2											
		3		1	SS	6	33	9					
		6		2	SS	5	14						pH, Res.
		8		3	SS	5	6						
		11		4	SS	3	5	11					
		13		5	SS	1	4						
	16.5	16		6	SS	3	2						
	BOTTOM OF BORING AT APPROXIMATELY 16.5 FEET												

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	NE	WD	
WL			
WL			

Terracon

BORING STARTED			3-18-08
BORING COMPLETED			3-18-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LOG OF BORING NO. B-02

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: **Between Jordan River and Redwood Road**

Approx. Surface Elev.: 4502.6 ft

Depth (strata), ft	Description	Depth, ft.	USCS Symbol	Number	Type	Recovery, in.	Penetration Resistance Blows / ft.	Water Content, %	Dry Unit Weight, PCF	Liquid Limit	Plasticity Index	% Passing No. 200 Sieve	PID
0.5	**TOPSOIL:** approximately 6 inches thick												
	SILT (MH): medium stiff, brown to gray	3		B	BS	48	---	41		52	18	71	
		4		1	SS	0	4						
		6		2	SS	8	4						
7.5	**LEAN CLAY (CL):** soft to medium stiff, gray to brown, trace organics	8		3	SS	14	4						ph, Res.
		11		4	SS	1	5						uu triax
		14		5	ST	24	---			28	10		
		16		6	SS	18	2						
20	**SANDY CLAY (CL):** soft, brown	21		7	SS	15	2						
21.5	BOTTOM OF BORING AT APPROXIMATELY 21.5 FEET												

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL	∇ 8	WD ▼
WL	▼	▼
WL		

Terracon

BORING STARTED			3-18-08
BORING COMPLETED			3-18-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LSI ENV SLC 61085026.GPJ TERRACON.GDT 6/12/08

LOG OF BORING NO. B-03

CLIENT: HDR Inc	
SITE: I-15 to Redwood Road, American Fork, Lehi, Saratoga Springs, UT	PROJECT: East to West Connector; I-15 to Redwood Road

Boring Location: Jordan River Bridge near West Abutment

Approx. Surface Elev.: 4497.4 ft

Depth (ft)	Description
0–1	**SILTY CLAY (CL-ML):** brown, with organics
1–5	**CLAYEY SAND (SC):** medium dense, brown, trace organics
5–15	**LEAN CLAY (CL):** soft to medium stiff, brown to gray, with organics, trace sand
15–40	**SANDY LEAN CLAY:** medium to stiff, brown to gray, some organics

DEPTH, ft.	USCS SYMBOL	NUMBER	TYPE	RECOVERY, in.	PENETRATION RESISTANCE BLOWS / ft.	WATER CONTENT, %	DRY UNIT WEIGHT, PCF	LIQUID LIMIT	PLASTICITY INDEX	% PASSING NO. 200 SIEVE	PID
3.5		1	SS	8	20						
6		2	SS	18	3						
11		3	ST	24	--			38	11		uu triax pH, Res.
13.5		4	SS	18	4						
16		5	SS	18	4						
21		6	ST	24	--			29	10		uu triax
26		7	SS	18	8						
31		8	SS	18	5	33				67	
36		9	SS	18	5						

Continued Next Page

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL	▽ 15	WD ▼
WL	▼	▼
WL		

Terracon

BORING STARTED			3-19-08
BORING COMPLETED			3-19-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LOG OF BORING NO. B-03

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Depth to base, ft.	Description	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, in.	SAMPLES: PENETRATION RESISTANCE BLOWS / ft.	TESTS: WATER CONTENT, %	TESTS: DRY UNIT WEIGHT, PCF	TESTS: LIQUID LIMIT	TESTS: PLASTICITY INDEX	TESTS: % PASSING NO. 200 SIEVE	PID
53	**SANDY LEAN CLAY:** medium to stiff, brown to gray, some organics	41		10	SS	12	27	32				51	
		46		11	SS	15	3						
		51		12	SS	16	14						
60	**CLAYEY SAND (SC):** very dense, red-brown	56		13	SS	16	50	26				50	
70	**SILTY SAND(SM):** very dense, brown, with gravel	61		14	SS	12	50/5						
		65		15	SS	3	50/5	15				40	
75	**GRAVEL (GP-GM):** very dense, gray	70		16	SS	2	50/1						
75.5	**SILTY SAND (SM):** very dense, gray to brown, with gravel	75		17	SS	7	50/6	13				25	

AUGER REFUSAL AT APPROXIMATELY 75.5 FEET

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL	∇ 15	WD ▼
WL	▼	∇
WL		

Terracon

BORING STARTED			3-19-08
BORING COMPLETED			3-19-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LOG OF BORING NO. B-04

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: Approximately 2600 West

Approx. Surface Elev.: 4499.4 ft

GRAPHIC LOG (depth, ft)	Description	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	TYPE	RECOVERY, in.	PENETRATION RESISTANCE BLOWS / ft.	TESTS: WATER CONTENT, %	DRY UNIT WEIGHT, PCF	LIQUID LIMIT	PLASTICITY INDEX	% PASSING NO. 200 SIEVE	PID
1	**TOPSOIL:** approximately 1 foot thick: sandy clay, dark brown, with organics	1–2											
5	**SANDY CLAY:** with sand lenses, soft, brown, trace organics	3		1	SS	15	3						
7.5	**SILTY SAND:** loose, brown	6		2	SS	10	6	19				16	
10	**SANDY CLAY:** soft, gray-brown	8		3	SS	10	2						
12.5	**CLAYEY SAND:** medium dense, gray-brown with orange mottling, trace organics	11		4	SS	18	18						
21.5	**CLAY:** with sand lenses, stiff to hard, gray to brown gray, with orange mottling	13		5	SS	10	26	23		34	18		
		16		6	SS	18	28						
		21		7	SS	15	7						

BOTTOM OF BORING AT APPROXIMATELY 21.5 FEET

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	▽ 2	WD	▼
WL	▼		▼
WL			

Terracon

BORING STARTED			4-2-08
BORING COMPLETED			4-2-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LOG OF BORING NO. B-05

CLIENT **HDR Inc**

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT **East to West Connector; I-15 to Redwood Road**

Boring Location: 2300 West Intersection

Approx. Surface Elev.: 4504.7 ft

Depth (ft)	Description	Sample Depth, ft.	USCS Symbol	Number	Type	Recovery, in.	Penetration Resistance Blows / ft.	Water Content, %	Dry Unit Weight, PCF	Liquid Limit	Plasticity Index	% Passing No. 200 Sieve	PID
1	**TOPSOIL:** sandy clay, dark brown, with organics												
4	**SANDY CLAY (CL):** medium stiff, dark brown, trace organics	3–4		1	SS	2	4						CBR Proctor
6	**SAND (SP-SM):** loose, tan to gray	5.5–6.5		2	SS	16	6						
7.5	**CLAY:** medium stiff, dark brown												
20.5	**SILTY SAND (SM):** loose to dense, gray to tan, with trace gravel and occasional clay layers ▽	8–10		3	ST	20	---			NP	NP		uu triax pH, Res.
		10.5–11.5		4	SS	6	6						
		13–14		5	SS	14	12						
24	**CLAY (CL):** hard, brown	20.5–21.5		6	SS	14	35						
26.5	**SANDY CLAY (CL):** very stiff, brown to gray	25.5–26.5		7	SS	10	21						

BOTTOM OF BORING AT APPROXIMATELY 26.5 FEET

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	▽ 10	WD	▼
WL	▼		▽
WL			

Terracon

BORING STARTED			3-18-08
BORING COMPLETED			3-18-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LOG OF BORING NO. B-06

CLIENT **HDR Inc**

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT **East to West Connector; I-15 to Redwood Road**

Boring Location: **Mill Pond Road and Railroad Bridge; Southwest Abutment East Side**

Approx. Surface Elev.: 4525.9 ft

Depth of layer bottom	Description	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	TYPE	RECOVERY, in.	PENETRATION RESISTANCE BLOWS / ft.	TESTS: WATER CONTENT, %	DRY UNIT WEIGHT, PCF	LIQUID LIMIT	PLASTICITY INDEX	% PASSING NO. 200 SIEVE	PID
0.25	**GRAVEL (GP-GM):** loose, gray												
5	**SILTY SAND (SM):** loose, light brown, trace gravel and organics												
10	**SAND (SP-SM):** medium dense, brown, with clay	6	SW	1	SS	12	11						
		7	SM										
15.8	**CLAYEY SAND (SC):** medium dense, brown to gray	11		2	SS	15	16						
24	**SANDY CLAY (CL):** very stiff, gray to brown, with sand lenses	16		3	ST	22	--						
		21		4	SS	16	14						
37	**LEAN CLAY (CL):** medium stiff, gray	26		5	SS	14	6						
		31		6	ST	22	--	24	97	27	9		Consol uu triax
		36		7	SS	3	30	22				69	
40	**SAND (SP-SC):** dense, gray, with clay												

Continued Next Page

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		BORING STARTED	3-26-08
WL ∇ 15	WD ▼	BORING COMPLETED	3-27-08
WL ▼	▼	RIG B-80	FOREMAN DAF
WL		LOGGED DAF	JOB # 61085026

Terracon

LOG OF BORING NO. B-06

CLIENT HDR Inc

SITE I-15 to Redwood Road, American Fork, Lehi, Saratoga Springs, UT

PROJECT East to West Connector; I-15 to Redwood Road

Depth to base, ft	Description	Depth, ft.	USCS Symbol	Sample Number	Type	Recovery, in.	Penetration Resistance Blows / ft.	Water Content, %	Dry Unit Weight, PCF	Liquid Limit	Plasticity Index	% Passing No. 200 Sieve	PID
44	**SILTY CLAY (CL-ML):** hard, gray, with sand lenses	41		8	SS	14	30						
46	**SAND (SP-SM):** medium dense, gray	46		9	SS	18	16						
50	**CLAY (CL):** very stiff, gray												
64	**SAND (SP-SM):** very dense, gray, with gravel	51		10	SS	18	50/6	15				6	
		56		11	SS	18	50/5						
		61		12	SS	18	50/6						
75	**LEAN CLAY (CL):** very stiff, light brown	66		13	SS	18	17	27		38	21		
		71		14	SS	16	22						
79	**SAND (SP-SM):** dense, brown	76		15	SS	0	37						

Continued Next Page

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL	▽ 15	WD ▼
WL	▼	▼
WL		

BORING STARTED			3-26-08
BORING COMPLETED			3-27-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LOG OF BORING NO. B-06

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

GRAPHIC LOG	DESCRIPTION	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, in.	SAMPLES: PENETRATION RESISTANCE BLOWS / ft.	TESTS: WATER CONTENT, %	TESTS: DRY UNIT WEIGHT, PCF	TESTS: LIQUID LIMIT	TESTS: PLASTICITY INDEX	TESTS: % PASSING NO. 200 SIEVE	PID
85	**SILTY CLAY (CL-ML):** hard, brown, with sand	81		16	SS	16	35						
90	**SAND (SP-SM):** medium dense, brown to gray, trace gravel	86		17	SS	12	15	13				10	
95	**SANDY CLAY (CL):** hard, brown	91		18	SS	16	36						
99	**SAND (SP-SM):** dense, brown	96		19	SS	0	38						
101.5	**LEAN CLAY (CL):** hard, gray-brown, with sand lenses	101		20	SS	15	50/6	15		35	19		

BOTTOM OF BORING AT APPROXIMATELY 101.5 FEET

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	▽ 15	WD	▼
WL	▼		▼
WL			

Terracon

BORING STARTED			3-26-08
BORING COMPLETED			3-27-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LOG OF BORING NO. B-07

CLIENT: **HDR Inc**

SITE: **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT: **East to West Connector; I-15 to Redwood Road**

Boring Location: **Mill Pond Road and Raildroad Bridge; Northeast Abutment**

Approx. Surface Elev.: 4526.2 ft

GRAPHIC LOG / DESCRIPTION	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	TYPE	RECOVERY, in.	PENETRATION RESISTANCE BLOWS / ft.	TESTS: WATER CONTENT, %	DRY UNIT WEIGHT, PCF	LIQUID LIMIT	PLASTICITY INDEX	% PASSING NO. 200 SIEVE	PID
FILL: silty sand and gravel, brown, scattered debris and vegetation at surface (to 5)	0–5											
SILTY SAND (SM): loose, brown with orange mottling, very fine ▽ to fine grained (to 10)	6		1	SS	6	8						
SANDY LEAN CLAY (CL): brown with black organics (to 15)	11		2	ST	18				25	8		UU TRIAX
CLAYEY SAND (SC): medium dense, brown to gray with orange mottling and clay seams (to 20)	16		3	SS	13	26						
SANDY LEAN CLAY (CL): stiff to very stiff, brown with black and orange mottling, with sand lenses	21		4	ST	18				44	21		
	26		5	SS	15	15						
	35		6	ST	18							uu triax
	36		7	SS	14	30						

Continued Next Page

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL	▽ 7	WD ▼
WL	▼	▼
WL		

Terracon

BORING STARTED			3-20-08
BORING COMPLETED			
RIG	B-80	FOREMAN	SCD
LOGGED	SCD	JOB #	61085026

LSI ENV SLC 61085026.GPJ TERRACON.GDT 6/12/08

LOG OF BORING NO. B-07

CLIENT: **HDR Inc**

SITE: **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT: **East to West Connector; I-15 to Redwood Road**

Strata depth, ft	Description	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	TYPE	RECOVERY, in.	PENETRATION RESISTANCE BLOWS / ft.	TESTS: WATER CONTENT, %	DRY UNIT WEIGHT, PCF	LIQUID LIMIT	PLASTICITY INDEX	% PASSING NO. 200 SIEVE	PID
42		41		8	SS	18	45	14				88	
45	**SILTY SAND (SM):** dense, gray												
50	**SANDY CLAY (CL):** stiff, dark gray	46		9	SS	14	8						
60	**CLAYEY GRAVEL (GC):** very dense, brown, with sand	50		10	SS	5	50/5"						
		56		11	SS	8	71						
63	**SILTY SAND (SM):** medium dense, brown	61		12	SS	12	20	22				10	
75	**SANDY CLAY (CL):** stiff to very stiff, brown, with gravels	66		13	SS	6	22						
		71		14	SS	12	12	26		39	21	84	
78	**CLAYEY GRAVEL (GC):** very dense, brown, fine to coarse gravel, with sand	75		15	SS	1	50/5"						
	SANDY CLAY (CL): very stiff, brown, with gravel												

Continued Next Page

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	▽ 7	WD	▼
WL	▼		▼
WL			

Terracon

BORING STARTED			3-20-08
BORING COMPLETED			
RIG	B-80	FOREMAN	SCD
LOGGED	SCD	JOB #	61085026

LOG OF BORING NO. B-07

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

GRAPHIC LOG	DESCRIPTION	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, in.	SAMPLES: PENETRATION RESISTANCE BLOWS / ft.	TESTS: WATER CONTENT, %	TESTS: DRY UNIT WEIGHT, PCF	TESTS: LIQUID LIMIT	TESTS: PLASTICITY INDEX	TESTS: % PASSING NO. 200 SIEVE	PID
	SANDY CLAY (CL): very stiff, brown, with gravel	81		16	SS	8	21						
85	**SILTY SAND (SM):** very dense, brown, with sand	86		17	SS	8	50/5"	15				14	
90	**CLAYEY GRAVEL (GC):** very dense, brown, with sand	90		18	SS	1	50/5"						
92	BOTTOM OF BORING (AUGER REFUSAL) AT APPROXIMATELY 92 FEET	92											

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	∇ 7	WD	▼
WL	▼		▼
WL			

Terracon

BORING STARTED			3-20-08
BORING COMPLETED			
RIG	B-80	FOREMAN	SCD
LOGGED	SCD	JOB #	61085026

LOG OF BORING NO. B-08

CLIENT
HDR Inc

SITE I-15 to Redwood Road
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: Jordan River Bridge; East Abutment

Approx. Surface Elev.: 4499.2 ft

Graphic log depth	Description	Depth, ft.	USCS Symbol	Samples: Number	Type	Recovery, in.	Penetration Resistance Blows / ft.	Tests: Water Content, %	Dry Unit Weight, PCF	Liquid Limit	Plasticity Index	% Passing No. 200 Sieve	PID
0.5	**TOPSOIL:** silty clay, brown, with organics, approximately 6 inches thick												
2	**SANDY CLAY(CL):** brown, trace organics	3		1	SS	8	28						
5	**SAND (SP-SM):** medium dense, brown to gray, with gravel	6		2	SS	8	57						
	SANDY LEAN CLAY (CL): very stiff to hard, brownto gray, trace organics, with gravel	8.5		3	SS	13	13						
10	**CLAYEY SAND (SC):** loose to medium dense, brown to gray, with organics ▽	11		4	ST	18	---	37	84				uu triax
		13.5		5	SS	18	11						
		16		6	SS	14	5						
16	**CLAY (CL):** medium stiff, gray, with organics												
20	**SANDY CLAY (CL):** stiff, brown, with organics, trace gravel	21		7	SS	16	10						
24	**CLAY (CL):** medium stiff to stiff, red to brown	26		8	SS	16	4						
		31		9	ST	22	---	27	99	37	23		uu triax
		36		10	SS	18	11						
40		40											

Continued Next Page

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	▽ 12	WD	▼
WL	▼		▼
WL			

Terracon

BORING STARTED			4-1-08
BORING COMPLETED			4-2-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LOG OF BORING NO. B-08

CLIENT
HDR Inc

SITE I-15 to Redwood Road
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Graphic Log Depth	Description	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, in.	SAMPLES: PENETRATION RESISTANCE BLOWS / ft.	TESTS: WATER CONTENT, %	TESTS: DRY UNIT WEIGHT, PCF	TESTS: LIQUID LIMIT	TESTS: PLASTICITY INDEX	TESTS: % PASSING NO. 200 SIEVE	PID
	CLAYEY SAND (SC): medium dense, red to brown	41		11	SS	9	12						
45	**SANDY CLAY (CL):** very stiff, red to brown, trace organics	46		12	SS	18	14						
		51		13	ST	20	---	26	99	26	9		uu triax
55	**CLAY (CL):** hard, red to brown, trace sand	56		14	SS	18	50/6	27				73	
57	**SAND (SP-SM):** very dense, brown to gray, with gravel	61		15	SS	14	50/4						
65	**GRAVEL (GP-GM):** very dense, gray	66		16	SS	3	50/1						
		70		17	SS	3	50/5						
75	**SANDY GRAVEL (GP):** very dense, brown to gray, trace clay	75		18	SS	3	50/5						
76	AUGER REFUSAL AT APPROXIMATELY 76 FEET	76											

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL	∇ 12	WD ▼
WL	▼	∇
WL		

Terracon

BORING STARTED			4-1-08
BORING COMPLETED			4-2-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LOG OF BORING NO. B-09

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: Approximately 2000 West

Approx. Surface Elev.: 4511.1 ft

Depth (strata)	Description	Sample Depth, ft.	USCS Symbol	Number	Type	Recovery, in.	Penetration Resistance Blows / ft.	Water Content, %	Dry Unit Weight, PCF	Liquid Limit	Plasticity Index	% Passing No. 200 Sieve	PID
1	**TOPSOIL:** sandy clay, dark brown, with organics, approximately 1 foot thick												
5	**SILTY SAND (SM):** medium dense, brown, trace gravel ▽	3		1	SS	4	10						
7.5	**SANDY CLAY (CL):** brown	6		2	ST	18	---	12				31	uu triax
12.5	**CLAYEY SAND (SC):** very loose to loose, brown to gray	8		3	SS	12	7						
		11		4	SS	1	2						
19	**CLAY (CL):** medium stiff, brown, trace sand	13		5	SS	18	4						
		16		6	ST	24	---						uu triax
21.5	**SAND (SP-SM):** dense, orange-brown	21		7	SS	10	35	21				15	

BOTTOM OF BORING AT APPROXIMATELY 21.5 FEET

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	▽ 5	WD	▼
WL	▼		▼
WL			

Terracon

BORING STARTED			4-2-08
BORING COMPLETED			4-2-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LOG OF BORING NO. B-10

CLIENT
HDR Inc

SITE I-15 to Redwood Road
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: 1700 West Intersection

Approx. Surface Elev.: 4514.1 ft

Depth to bottom, ft	Description	Sample depth, ft	USCS SYMBOL	NUMBER	TYPE	RECOVERY, in.	PENETRATION RESISTANCE BLOWS / ft.	WATER CONTENT, %	DRY UNIT WEIGHT, PCF	LIQUID LIMIT	PLASTICITY INDEX	% PASSING NO. 200 SIEVE	PID
1	**TOPSOIL:** silty clay, dark brown, with organics												
7.5	**SILTY SAND (SM):** loose to medium dense, brown, trace clay and organics	3		1	SS	10	9						
		6		2	SS	0	10						
10	**CLAYEY SAND (SC):** very loose, brown	8.5		3	SS	9	2						
12.5	**SANDY CLAY (CL):** soft, brown	11		4	SS	6	3						
16	**CLAY (CL):** medium stiff, brown, trace sand	13.5		5	SS	18	6						
24	**CLAYEY SAND (SC):** medium dense, brown	16		6	ST	24	---	26	87	28	10		
		21		7	SS	7	10						
26.5	**CLAY (CL):** very stiff, gray with orange mottling	26		8	SS	16	26						

BOTTOM OF BORING AT APPROXIMATELY 26.5 FEET

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	▽ 8	WD	▼
WL	▼		▼
WL			

Terracon

BORING STARTED			4-3-08
BORING COMPLETED			4-3-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LOG OF BORING NO. B-11

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: 1400 West Intersection

Approx. Surface Elev.: 4517.4 ft

Depth to base (ft)	Description
0.5	**TOPSOIL:** clayey sand, dark brown, trace organics
12.5	**SAND (SP-SM):** loose to medium dense, brown, trace clay and gravel
20	**LEAN CLAY (CL):** very stiff, orange-brown, trace gravel
21.5	**CLAYEY SAND (SC):** loose, orange-brown

BOTTOM OF BORING AT APPROXIMATELY 21.5 FEET

DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	TYPE	RECOVERY, in.	PENETRATION RESISTANCE BLOWS / ft.	TESTS: WATER CONTENT, %	DRY UNIT WEIGHT, PCF	LIQUID LIMIT	PLASTICITY INDEX	% PASSING NO. 200 SIEVE	PID
3–4		1	SS	15	10						
6		2	SS	8	16						
8.5		3	SS	11	4						
11		4	SS	4	14						
13.5		5	SS	15	17						
16		6	ST	16	---	25	85	25	10		uu triax
21		7	SS	15	5						

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL	∇ 9	WD ▼
WL	▼	▼
WL		

BORING STARTED			4-3-08
BORING COMPLETED			4-3-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LOG OF BORING NO. B-12

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: 1100 West Intersection

Approx. Surface Elev.: 4518.2 ft

GRAPHIC LOG (depth)	Description	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	TYPE	RECOVERY, in.	PENETRATION RESISTANCE BLOWS / ft.	TESTS: WATER CONTENT, %	DRY UNIT WEIGHT, PCF	LIQUID LIMIT	PLASTICITY INDEX	% PASSING NO. 200 SIEVE	PID
0.5	**TOPSOIL:** silt, brown, with organics												
7.5	**SAND (SP-SM):** very loose to loose, brown	3		1	SS	3	6						
		6		2	SS	0	2						
10	**CLAYEY SAND (SC):** loose, brown, trace gravel ▽	9		3	ST	4	---	16					
12.5	**SANDY CLAY (CL):** stiff, brown	11		4	SS	18	8						
26.5	**LEAN CLAY (CL):** stiff to hard, brown to red-brown, with sand	13.5		5	SS	18	7						
		16		6	SS	18	10	28		33	13		
		21		7	SS	14	30						
		26		8	SS	14	28						

BOTTOM OF BORING AT
APPROXIMATELY 26.5 FEET

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	▽ 10	WD	▼
WL	▼		▼
WL			

Terracon

BORING STARTED			4-9-08
BORING COMPLETED			4-9-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LOG OF BORING NO. B-13

CLIENT
HDR Inc

SITE I-15 to Redwood Road
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: Approximately 800 West

Approx. Surface Elev.: 4516.3 ft

GRAPHIC LOG depth	Description	DEPTH, ft.	USCS SYMBOL	SAMPLES NUMBER	TYPE	RECOVERY, in.	PENETRATION RESISTANCE BLOWS / ft.	WATER CONTENT, %	DRY UNIT WEIGHT, PCF	LIQUID LIMIT	PLASTICITY INDEX	% PASSING NO. 200 SIEVE	PID
0.5	**TOPSOIL:** silt, trace clay, dark brown, with organics												
	SILTY CLAY (CL-ML): soft to medium stiff, brown, trace organics	3		1	SS	3	2	24					
	▽	6		2	SS	18	4						
7.5	**SANDY CLAY (CL):** soft, brown, trace organics	8		3	SS	12	3						
11	**SAND (SP-SM):** loose to medium dense, brown to gray	11		4	ST	20	---						
		13		5	SS	18	3	26				5	
		16		6	SS	16	27						
18	**LEAN CLAY (CL):** very stiff, gray, with clay												
21.5		21		7	SS	15	19	25		30	11	95	

BOTTOM OF BORING AT APPROXIMATELY 21.5 FEET

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL	▽ 5	WD ▼
WL	▼	▼
WL		

Terracon

BORING STARTED			4-9-08
BORING COMPLETED			4-9-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LOG OF BORING NO. B-14

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: 500 West Intersection

Approx. Surface Elev.: 4517.6 ft

GRAPHIC LOG (depth, ft)	Description	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	TYPE	RECOVERY, in.	PENETRATION RESISTANCE BLOWS / ft.	TESTS: WATER CONTENT, %	DRY UNIT WEIGHT, PCF	LIQUID LIMIT	PLASTICITY INDEX	% PASSING NO. 200 SIEVE	PID
5	**CLAYEY SAND (SC):** loose, brown, trace gravel	3		1	SS	0	8						
7.5	**SANDY CLAY (CL):** soft, brown	6		2	SS	4	3						
10	**SAND (SP-SM):** medium dense, brown-gray ▽	8		3	SS	4	22	23				13	
12.5	**CLAYEY SAND (SC):** medium dense, brown to gray	11		4	SS	14	15						
15	**SANDY LEAN CLAY (CL):** stiff, gray to brown with orange mottling	14		5	ST	24	---	28	89	42	23		uu triax
24	**CLAY (CL):** very stiff, brown with gray and red mottling, with sand, trace gravel	16		6	SS	15	17						
		21		7	SS	12	20						
26	**SANDY CLAY (CL):** dense, gray												
26.5	**SAND (SP-SM):** dense, gray	26		8	SS	16	35	21				60	

BOTTOM OF BORING AT APPROXIMATELY 26.5 FEET

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL	▽ 8	WD ▼
WL	▼	▼
WL		

Terracon

BORING STARTED			4-9-08
BORING COMPLETED			4-9-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LOG OF BORING NO. B-15

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: Center Streeet Intersection

Approx. Surface Elev.: 4513.8 ft

Depth (strata)	Description	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	TYPE	RECOVERY, in.	PENETRATION RESISTANCE BLOWS / ft.	TESTS: WATER CONTENT, %	DRY UNIT WEIGHT, PCF	LIQUID LIMIT	PLASTICITY INDEX	% PASSING NO. 200 SIEVE	PID
1	**TOPSOIL:** silty clay, dark brown, with organics												
7.5	**LEAN CLAY (CL-ML):** stiff to very stiff, gray to brown, trace organics, trace gravel	3		1	SS	16	24						
		6		2	SS	18	11	22		26	7		
12.5	**SANDY CLAY (CL):** medium stiff, gray to brown with orange mottling	8		3	SS	8	4						
	∇	11		4	SS	15	5						
24	**LEAN CLAY (CL):** medium stiff to stiff, dark gray	14		5	ST	24	---	48	67	25	8		uu triax
		16		6	SS	10	4						
		21		7	SS	15	8						
26.5	**CLAYEY SAND (SC):** medium dense, dark gray	26		8	SS	12	16	19				46	

BOTTOM OF BORING AT APPROXIMATELY 26.5 FEET

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL	∇ 12	WD
WL		
WL		

Terracon

BORING STARTED			4-9-08
BORING COMPLETED			4-9-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LOG OF BORING NO. B-17

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: **I-15 SPUI Interchange, West Abutment, North Side**

Approx. Surface Elev.: 4546.9 ft

Depth of change (ft)	Description	Sample Depth, ft.	USCS Symbol	Number	Type	Recovery, in.	Penetration Resistance Blows / ft.	Water Content, %	Dry Unit Weight, PCF	Liquid Limit	Plasticity Index	% Passing No. 200 Sieve	PID
0.5	**TOPSOIL:** silty clay, brown												
	SANDY LEAN CLAY (CL): soft, gray to brown, trace gravel	3		1	SS	6	9						
		6		2	SS	8	3	27				56	
		8.5		3	SS	10	3						
		11		4	ST	24	--	32	92	38	18		Consol uu triax
		13.5		5	SS	7	3						
		16		6	SS	12	17	27				80	
19	**SAND (SP-SM):** medium dense, gray, with clay	21		7	SS	9	13						
25	**LEAN CLAY (CL):** soft, gray to brown, trace sand	26		8	SS	16	3						
		31		9	ST	24	--	52	71	32	13		Consol uu triax
32.5	**SAND (SP-SM):** medium dense to very dense, gray to brown, trace clay	36		10	SS	14	18						

Continued Next Page

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL	∇ 3	WD ▼
WL	▼	▼
WL		

Terracon

BORING STARTED			3-27-08
BORING COMPLETED			3-28-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LOG OF BORING NO. B-17

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

DESCRIPTION	DEPTH, ft.	USCS SYMBOL	NUMBER	TYPE	RECOVERY, in.	PENETRATION RESISTANCE BLOWS / ft.	WATER CONTENT, %	DRY UNIT WEIGHT, PCF	LIQUID LIMIT	PLASTICITY INDEX	% PASSING NO. 200 SIEVE	PID
SAND (SP-SM): medium dense to very dense, gray to brown, trace clay	41		11	SS	12	50	24				17	
	46		12	SS	12	28						
48 **CLAY (CL):** very stiff, dark gray	51		13	SS	4	18						
54.5 **SANDY LEAN CLAY (CL):** very stiff, brown	56		14	ST	16	--			27	10		Consol
60 **SILTY SAND (SM):** very dense, brown-gray	61		15	SS	18	50/5	16				20	
65 **CLAYEY SAND (SC):** medium dense to dense, brown, trace gravel	66		16	SS	4	32						
	71		17	SS	0	24						
75 **GRAVEL (GP-GM):** very dense, gray, trace sand and clay	75		18	SS	3	50/3						

Continued Next Page

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL	∇ 3	WD ▼
WL	▼	∇
WL		

Terracon

BORING STARTED			3-27-08
BORING COMPLETED			3-28-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LOG OF BORING NO. B-17

CLIENT
HDR Inc

SITE I-15 to Redwood Road
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

DESCRIPTION	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, in.	SAMPLES: PENETRATION RESISTANCE BLOWS / ft.	TESTS: WATER CONTENT, %	TESTS: DRY UNIT WEIGHT, PCF	TESTS: LIQUID LIMIT	TESTS: PLASTICITY INDEX	TESTS: % PASSING NO. 200 SIEVE	PID
GRAVEL (GP-GM): very dense, gray, trace sand and clay	81		19	SS	0	50/5						
84												
LEAN CLAY (CL): soft to stiff, gray to light brown, trace gravel	86		20	SS	12	12	23		37	21		
	91		21	SS	1	2						
95												
SANDY CLAY (CL): hard, brown	96		22	SS	0	45						
98												
LEAN CLAY (CL): hard, orange-brown	101		23	SS	12	50/6	24		26	8		
103												

AUGER REFUSAL AT APPROXIMATELY 103 FEET

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL	∇ 3	WD ▼
WL	▼	▼
WL		

Terracon

BORING STARTED			3-27-08
BORING COMPLETED			3-28-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LOG OF BORING NO. B-18

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: I-15 SPUI Interchange East Abutment

Approx. Surface Elev.: 4543.5 ft

Depth to base (ft)	Description	Samples: Number	Type	Recovery, in.	Penetration Resistance Blows / ft.	Water Content, %	Dry Unit Weight, PCF	Liquid Limit	Plasticity Index	% Passing No. 200 Sieve	PID
7.5	**CLAY (CL):** soft to medium stiff, dark brown										
	(3 ft)	1	SS	0	3						
	(5.5 ft)	2	SS	0	4						
10	**SANDY GRAVEL (GP-GM):** dark gray										
	(8.5 ft)	3	ST	3	---			23	5		
16	**SANDY CLAY (CL):** soft to medium stiff, dark brown, with gravel										
	(10.5 ft)	4	SS	8	2						
	(13 ft)	5	SS	10	5	29		36	12		
	(15.5 ft)	6	SS	10	6						
19	**CLAYEY SAND (SC):** loose, dark brown, with gravel										
21	**SANDY SILT (ML):** very stiff, dark brown										
	(20.5 ft)	7	SS	8	15	21				24	
23	**SILTY SAND (SM):** medium dense, brown										
31	**CLAY (CL):** very stiff, gray										
	(25.5 ft)	8	SS	10	17						
	(30.5 ft)	9	ST	24	---						
32	**SILTY SAND (SM):** medium dense, gray										
36	**CLAY (CL):** medium stiff, gray										
	(35.5 ft)	10	SS	18	21	24		26	7		
38	**SILTY SAND (SM):** medium dense, gray										
	SAND (SP-SM): medium dense to very dense, gray										

Continued Next Page

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	∇ 5	WD	▼
WL	▼		▼
WL			

Terracon

BORING STARTED			4-17-08
BORING COMPLETED			4-17-08
RIG	B-80	FOREMAN	JWG
LOGGED	JWG	JOB #	61085026

LOG OF BORING NO. B-18

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

DESCRIPTION	Bottom depth, ft.
SAND (SP-SM): medium dense to very dense, gray	48
CLAY (CL): medium stiff, gray to black	53
SANDY CLAY (CL): stiff, brown	57
SANDY GRAVEL (GP-GM): very dense, gray to brown, trace clay	64
SILTY SAND (SM): very dense, brown, trace gravel	69
CLAYEY GRAVEL (GP-GM): very dense, brown, with sand	74
CLAYEY SAND (SC): very dense, brown, with gravel	

DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	TYPE	RECOVERY, in.	PENETRATION RESISTANCE BLOWS / ft.	TESTS: WATER CONTENT, %	DRY UNIT WEIGHT, PCF	LIQUID LIMIT	PLASTICITY INDEX	% PASSING NO. 200 SIEVE	PID
41		11	SS	12	11						
46		12	SS	18	74						
51		13	SS	18	6						
56		14	ST	24	---	27	99				uu triax
61		15	SS	14	78						
66		16	SS	6	51	12				21	
70		17	SS	3	50/4						
76		18	SS	8	50						

Continued Next Page

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL	∇ 5	WD ▼
WL	▼	∇
WL		

Terracon

BORING STARTED			4-17-08
BORING COMPLETED			4-17-08
RIG	B-80	FOREMAN	JWG
LOGGED	JWG	JOB #	61085026

LOG OF BORING NO. B-18

CLIENT
HDR Inc

SITE I-15 to Redwood Road
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

GRAPHIC LOG	DESCRIPTION	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	TYPE	RECOVERY, in.	PENETRATION RESISTANCE BLOWS / ft.	TESTS: WATER CONTENT, %	DRY UNIT WEIGHT, PCF	LIQUID LIMIT	PLASTICITY INDEX	% PASSING NO. 200 SIEVE	PID
81	**SANDY CLAY (CL):** hard, brown with red mottling, trace gravel	81		19	SS	12	34	19		25	7		
84		82–85											
86.5	**SAND (SP):** very dense, brown, with gravel and trace clay	86		20	SS	8	75	9				17	
	BOTTOM OF BORING AT APPROXIMATELY 86.5 FEET												

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL	∇ 5	WD ▼
WL	▼	▼
WL		

Terracon

BORING STARTED			4-17-08
BORING COMPLETED			4-17-08
RIG	B-80	FOREMAN	JWG
LOGGED	JWG	JOB #	61085026

LOG OF BORING NO. B-19

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: Between I-15 and Railroad Bridge

Approx. Surface Elev.: 4528.2 ft

Depth of layer (ft)	Description	Number	Type	Recovery, in.	Penetration Resistance Blows / ft.
1.5	**CONCRETE:**				
3	**FILL:**				
4.5	**SANDY SILT (MC):** hard, dark gray, with clay	1	SS	8	73
6	**CLAY (CL):** hard, brown	2	SS	10	40
7	**SILTY SAND (SM):** medium dense, brown	3	SS	10	28
21.5	**CLAY (CL):** soft to very stiff, brown to gray, trace silt	4	SS	6	2
		5	ST	0	---
		6	SS	8	13
		7	SS	8	18

Column headings: GRAPHIC LOG; DEPTH, ft.; USCS SYMBOL; SAMPLES: NUMBER, TYPE, RECOVERY, in., PENETRATION RESISTANCE BLOWS / ft.; TESTS: WATER CONTENT, %, DRY UNIT WEIGHT, PCF, LIQUID LIMIT, PLASTICITY INDEX, % PASSING NO. 200 SIEVE; PID

Sample depths (ft): 1 at 4; 2 at 5.5; 3 at 7; 4 at 8.5; 5 at 13–15; 6 at 16; 7 at 21.

BOTTOM OF BORING AT
APPROXIMATELY 21.5 FEET

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL	▽ 7.5	WD ▼
WL	▽	▼
WL		

Terracon

BORING STARTED			4-15-08
BORING COMPLETED			4-15-08
RIG	B-80	FOREMAN	JWG
LOGGED	JWG	JOB #	61085026

LOG OF BORING NO. B-20

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: Between I-15 and Railroad Bridge

Approx. Surface Elev.: 4526.6 ft

Depth (ft)	Description	DEPTH, ft.	USCS SYMBOL	NUMBER	TYPE	RECOVERY, in.	PENETRATION RESISTANCE BLOWS / ft.	WATER CONTENT, %	DRY UNIT WEIGHT, PCF	LIQUID LIMIT	PLASTICITY INDEX	% PASSING NO. 200 SIEVE	PID
0.5	**CONCRETE:**												
2	**FILL:** silty sand with gravel, dark gray	2		1	SS	4	44						
5	**SANDY CLAY (CL):** stiff to hard, brown to gray ▽	3		2	SS	8	7						
		5		3	SS	10	30						
12.5	**SILTY SAND (SM):** medium dense to dense, red-brown to gray	6		4	SS	6	37						
		7		5	SS	6	20						
22	**SANDY CLAY (CL):** stiff, brown	13		6	SS	8	8						
		16		7	SS	8	9						
		21		8	ST	8	---	29	94				uu triax

BOTTOM OF BORING AT APPROXIMATELY 22 FEET

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL	▽ 5	WD ▼
WL	▼	▼
WL		

Terracon

BORING STARTED			4-15-08
BORING COMPLETED			4-15-08
RIG	B-80	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LOG OF BORING NO. B-21

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: **Southbound I-15, Right Shoulder at Exit 278**

Depth to base of layer	Description	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	TYPE	RECOVERY, in.	PENETRATION RESISTANCE BLOWS / ft.	TESTS: WATER CONTENT, %	DRY UNIT WEIGHT, PCF	LIQUID LIMIT	PLASTICITY INDEX	% PASSING NO. 200 SIEVE	PID
0.7	**ASPHALT:** approximately 8.5 inches thick												
3	**FILL:** sandy gravel, tan	3		1	SS	15	45						
7	**SILTY SAND (SM):** dense, light brown, trace gravel	6		2	SS	15	33	9				26	
11.5	**SANDY CLAY (CL):** medium stiff to stiff, gray to black, trace organics	8		3	SS	14	11	25				70	
		11		4	SS	15	4	49		44	19		

BOTTOM OF BORING AT APPROXIMATELY 11.5 FEET

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL	▽ NE	WD ▼
WL	▼	▽
WL		

Terracon

BORING STARTED			5-30-08
BORING COMPLETED			5-30-08
RIG	CME-75	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LOG OF BORING NO. B-22

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: **Southbound I-15, Right Lane, Approximately 200 Feet North of Overpass**

Depth to base of layer, ft	Description	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, in.	SAMPLES: PENETRATION RESISTANCE BLOWS / ft.	TESTS: WATER CONTENT, %	TESTS: DRY UNIT WEIGHT, PCF	TESTS: LIQUID LIMIT	TESTS: PLASTICITY INDEX	TESTS: % PASSING NO. 200 SIEVE	PID
0.75	**ASPHALT:** approximately 9 inches thick	1											
2.5	**FILL:** sandy gravel, tan	2											
		3		1	SS	16	44	6				27	
7	**SILTY SAND (SM):** dense to very dense, light brown, trace gravel	6		2	SS	16	56						
11.5	**SANDY CLAY (CL):** medium stiff to soft, dark gray to light brown, trace gravel and organics	8		3	SS	12	6	28		43	15	59	
		11		4	SS	18	0	28					

BOTTOM OF BORING AT APPROXIMATELY 11.5 FEET

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	∇ NE	WD	▼
WL	∇		∇
WL			

Terracon

BORING STARTED			5-30-08
BORING COMPLETED			5-30-08
RIG	CME-75	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LOG OF BORING NO. B-23

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: **Northbound I-15, Right Shoulder between Right Lane and On Ramp**

Depth to base	Description	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, in.	SAMPLES: PENETRATION RESISTANCE BLOWS / ft.	TESTS: WATER CONTENT, %	TESTS: DRY UNIT WEIGHT, PCF	TESTS: LIQUID LIMIT	TESTS: PLASTICITY INDEX	TESTS: % PASSING NO. 200 SIEVE	PID
0.83	**ASPHALT:** approximately 10 inches thick	1											
2.5	**FILL:** sandy gravel, tan	2–3		1	SS	12	32	5				19	
10	**SILTY SAND (SM):** loose to dense, light brown, trace gravel	6		2	SS	16	40	5				13	
		8		3	SS	3	7						
11.5	**SILTY CLAY (CL-ML):** soft, brown to gray, trace gravel	11		4	SS	17	2						

BOTTOM OF BORING AT APPROXIMATELY 11.5 FEET

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL	▽ 10	WD ▼
WL	▼	▼
WL		

Terracon

BORING STARTED			5-30-08
BORING COMPLETED			5-30-08
RIG	CME-75	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LOG OF BORING NO. B-24

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: **Northbound I-15, Right Lane, Approximately 200 Feet North of Overpass**

Depth of strata (ft)	Description	DEPTH, ft.	USCS SYMBOL	NUMBER	TYPE	RECOVERY, in.	PENETRATION RESISTANCE BLOWS / ft.	WATER CONTENT, %	DRY UNIT WEIGHT, PCF	LIQUID LIMIT	PLASTICITY INDEX	% PASSING NO. 200 SIEVE	PID
0.96	**ASPHALT:** approximately 11.5 inches thick	1											
2.5	**FILL:** sandy gravel, tan	2											
		3		1	SS	12	66						
	SAND (SP-SM): dense to very dense, light brown, trace gravel	4											
		5											
7		6		2	SS	16	34	4				12	CBR Proctor
	SANDY CLAY (CL): medium stiff, brown and gray, trace organics and gravel	7		5	BS	--	--						
		8		3	SS	12	4	25		29	12	72	
	▽	9											
		10											
11.5		11		4	SS	8	4						

BOTTOM OF BORING AT APPROXIMATELY 11.5 FEET

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	▽ 10	WD	▼
WL	▼		▼
WL			

Terracon

BORING STARTED			5-30-08
BORING COMPLETED			5-30-08
RIG	CME-75	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LOG OF BORING NO. B-25

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: **Northbound I-15, Right Lane, Just South of Exit 278**

Depth (graphic log)	Description	DEPTH, ft.	USCS SYMBOL	NUMBER	TYPE	RECOVERY, in.	PENETRATION RESISTANCE BLOWS / ft.	WATER CONTENT, %	DRY UNIT WEIGHT, PCF	LIQUID LIMIT	PLASTICITY INDEX	% PASSING NO. 200 SIEVE	PID
0.94	**ASPHALT:** approximately 11.25 inches thick	1											
2.5	**FILL:** sandy gravel, tan	2											
		3		1	SS	12	28	10				38	
	SILTY SAND (SM): loose to dense, light brown, trace silt and gravel	6		2	SS	15	48						
		8		3	SS	14	31	12				17	
10.75		10											
11.5	**SILTY CLAY (CL-ML):** medium stiff, brown and gray	11		4	SS	15	8						

BOTTOM OF BORING AT APPROXIMATELY 11.5 FEET

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL	∇ NE	WD ▼
WL	▼	▼
WL		

Terracon

BORING STARTED			5-30-08
BORING COMPLETED			5-30-08
RIG	CME-75	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LOG OF BORING NO. B-26

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: **Northbound I-15, Right Shoulder, Approximately 500 Feet South of Exit 278**

GRAPHIC LOG (depth)	DESCRIPTION	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, in.	SAMPLES: PENETRATION RESISTANCE BLOWS / ft.	TESTS: WATER CONTENT, %	TESTS: DRY UNIT WEIGHT, PCF	TESTS: LIQUID LIMIT	TESTS: PLASTICITY INDEX	TESTS: % PASSING NO. 200 SIEVE	PID
0.75	**ASPHALT:** approximately 9 inches thick	1											
2.5	**FILL:** sandy gravel, tan	2											
	SILTY SANDS (SM): loose to dense, brown, trace gravel	3		1	SS	12	36	4				15	
		6		2	SS	15	24						
7	**SANDY CLAY (CL):** medium stiff, brown	8		3	SS	16	4	20		23	8	71	
9	**SILTY SAND (SM):** loose, brown, trace gravel and clay	11		4	SS	15	8	15					
11.5	BOTTOM OF BORING AT APPROXIMATELY 11.5 FEET												

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL	▽ NE	WD ▼
WL	▼	▼
WL		

Terracon

BORING STARTED			5-31-08
BORING COMPLETED			5-31-08
RIG	CME-75	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

LSI ENV SLC 61085026.GPJ TERRACON.GDT 6/12/08

LOG OF BORING NO. B-27

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: **Southbound I-15, Right Lane, South of Overpass**

Graphic Log Depth	Description	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	SAMPLES: TYPE	SAMPLES: RECOVERY, in.	SAMPLES: PENETRATION RESISTANCE BLOWS / ft.	TESTS: WATER CONTENT, %	TESTS: DRY UNIT WEIGHT, PCF	TESTS: LIQUID LIMIT	TESTS: PLASTICITY INDEX	TESTS: % PASSING NO. 200 SIEVE	PID
0.7	**ASPHALT:** approximately 8.5 inches thick	1											
2.5	**FILL:** sandy gravel, tan	2											
	SILTY SAND (SM): medium dense to dense, light brown	3		1	SS	15	36	9				40	
		6		2	SS	16	32	7				13	CBR Proctor
		8		3	SS	14	28	7				19	
11.5		11		4	SS	15	29						

BOTTOM OF BORING AT APPROXIMATELY 11.5 FEET

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft		
WL	NE	WD
WL		
WL		

Terracon

BORING STARTED	5-31-08
BORING COMPLETED	5-31-08
RIG CME-75	FOREMAN DAF
LOGGED DAF	JOB # 61085026

LOG OF BORING NO. B-28

CLIENT
HDR Inc

SITE **I-15 to Redwood Road**
American Fork, Lehi, Saratoga Springs, UT

PROJECT
East to West Connector; I-15 to Redwood Road

Boring Location: **Southbound I-15, Right Shoulder, Approximately 500 Feet South of On Ramp**

Depth to base, ft	Description	DEPTH, ft.	USCS SYMBOL	SAMPLES: NUMBER	TYPE	RECOVERY, in.	PENETRATION RESISTANCE BLOWS / ft.	TESTS: WATER CONTENT, %	DRY UNIT WEIGHT, PCF	LIQUID LIMIT	PLASTICITY INDEX	% PASSING NO. 200 SIEVE	PID
0.83	**ASPHALT:** approximately 10 inches thick	1											
2.5	**FILL:** sandy gravel, tan	2											
		3		1	SS	16	28	7				27	
7	**SILTY SAND (SM):** medium dense, light brown, trace gravel	6		2	SS	15	28						
9.5	**SANDY CLAY (CL):** medium stiff, brown to dark brown, trace organics	8		3	SS	12	5	21		29	12	72	
11.5	**SILTY SAND (SM):** very loose, brown, with clay	11		4	SS	12	3	16					

BOTTOM OF BORING AT APPROXIMATELY 11.5 FEET

The stratification lines represent the approximate boundary lines between soil and rock types: in-situ, the transition may be gradual.

WATER LEVEL OBSERVATIONS, ft			
WL	▽ NE	WD	▼
WL	▼		▽
WL			

Terracon

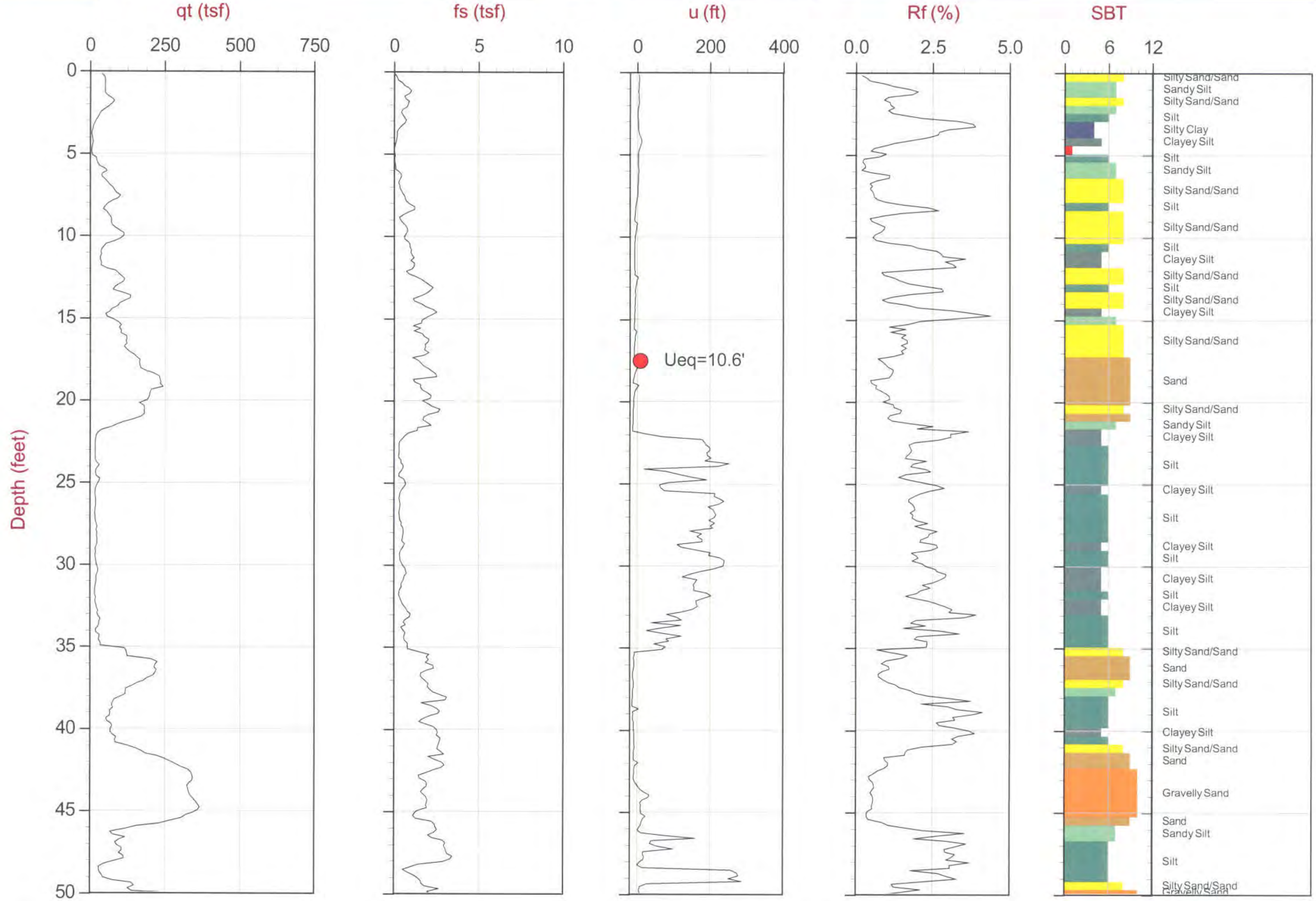
BORING STARTED			5-31-08
BORING COMPLETED			5-31-08
RIG	CME-75	FOREMAN	DAF
LOGGED	DAF	JOB #	61085026

CONETEC

Terracon

Job No: 08-340
Date: 03:19:08 09:26
Site: EAST WEST CONNECTOR

Sounding: CPT-01
Cone: STD 20T AD197



Max Depth: 16.650 m / 54.63 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.150 m

File: 340CP01.COR
Unit Wt: SBT Chart Soil Zones

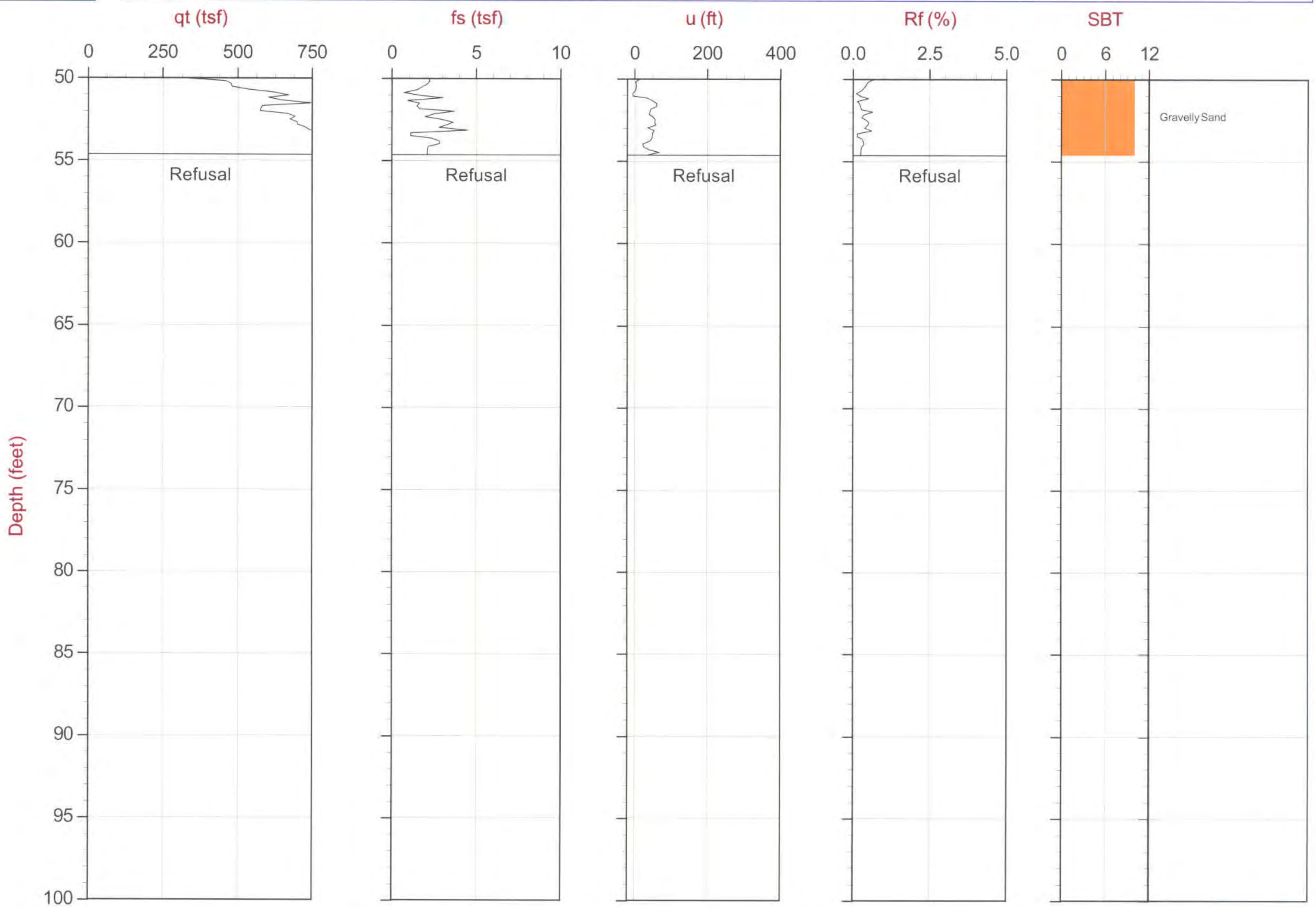
SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 2

● Equilibrium Pore Pressure from Dissipation

CONETEC **Terracon**

Job No: 08-340
Date: 03:19:08 09:26
Site: EAST WEST CONNECTOR

Sounding: CPT-01
Cone: STD 20T AD197



Max Depth: 16.650 m / 54.63 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.150 m

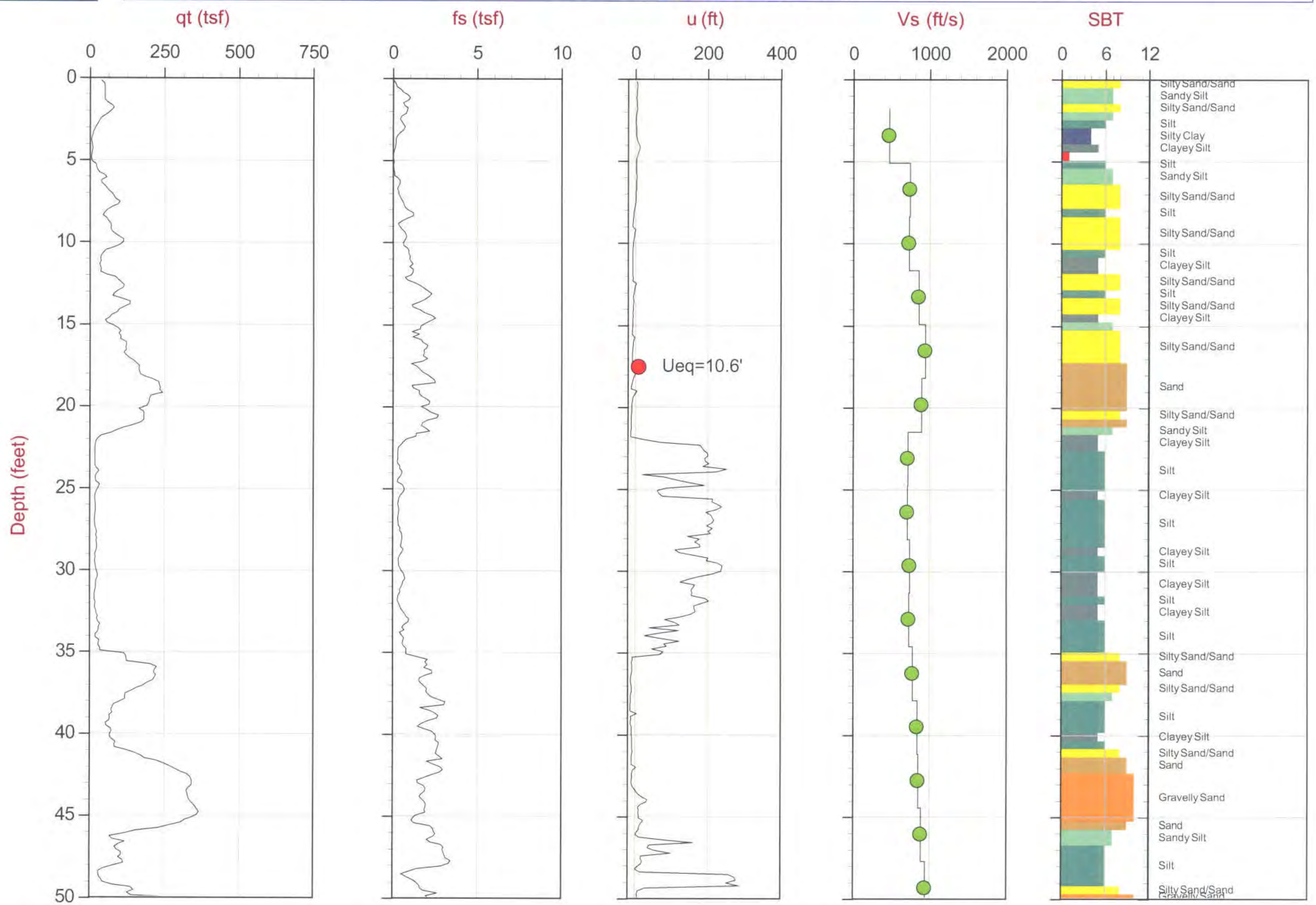
File: 340CP01.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
Page No: 2 of 2
● Equilibrium Pore Pressure from Dissipation

CONETEC | Terracon

Job No: 08-340
Date: 03:19:08 09:26
Site: EAST WEST CONNECTOR

Sounding: CPT-01
Cone: STD 20T AD197



Max Depth: 16.650 m / 54.63 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.150 m

File: 340CP01.COR
Unit Wt: SBT Chart Soil Zones

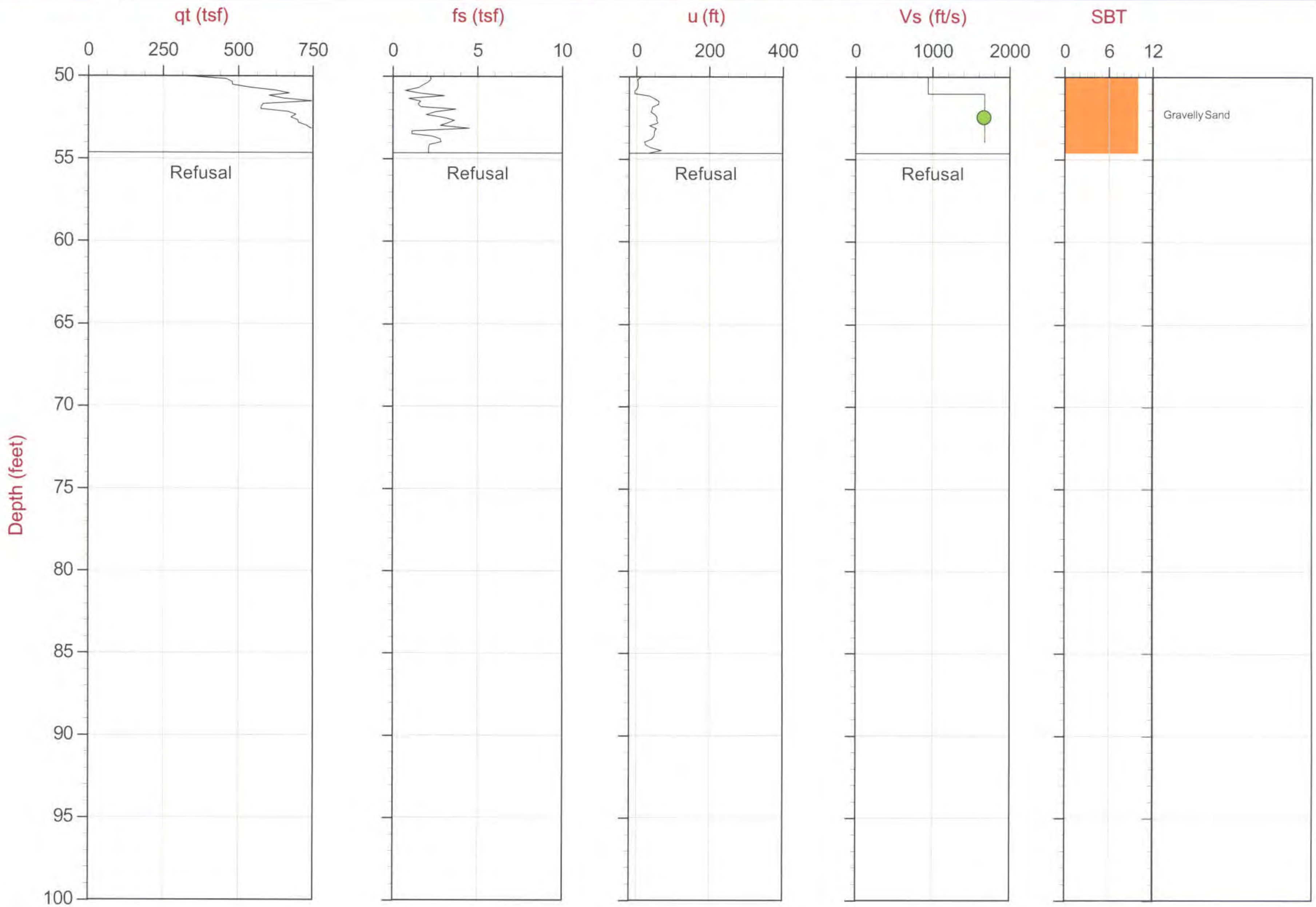
SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 2
● Equilibrium Pore Pressure from Dissipation

CONETEC

Terracon

Job No: 08-340
Date: 03:19:08 09:26
Site: EAST WEST CONNECTOR

Sounding: CPT-01
Cone: STD 20T AD197



Max Depth: 16.650 m / 54.63 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.150 m

File: 340CP01.COR
Unit Wt: SBT Chart Soil Zones

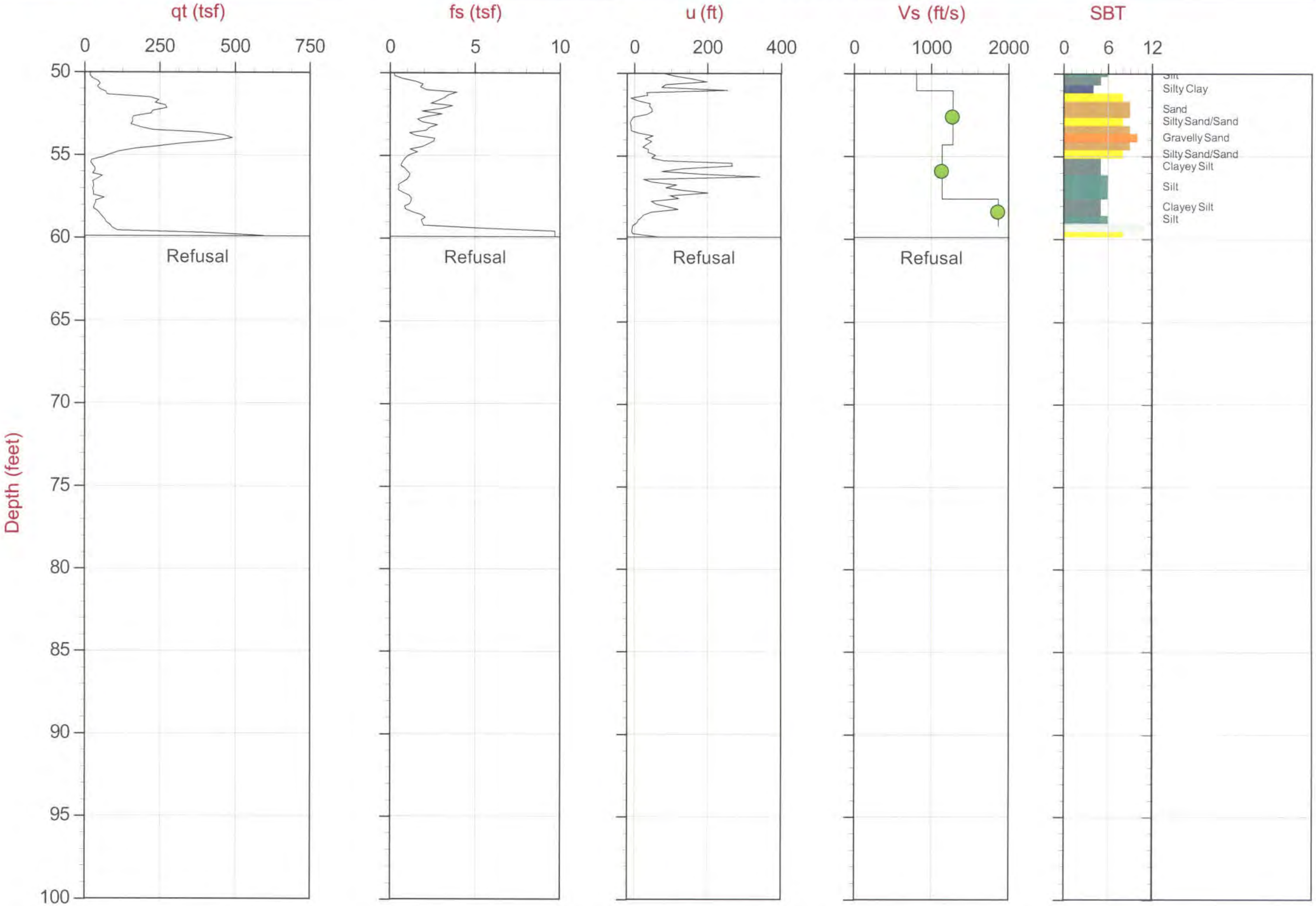
SBT: Lunne, Robertson and Powell, 1997
Page No: 2 of 2
Equilibrium Pore Pressure from Dissipation

ConeTec

Terracon

Job No: 08-340
Date: 03:19:08 11:54
Site: EAST WEST CONNECTOR

Sounding: CPT-02
Cone: STD 20T AD197



Max Depth: 18.250 m / 59.87 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.150 m

File: 340CP02.COR
Unit Wt: SBT Chart Soil Zones

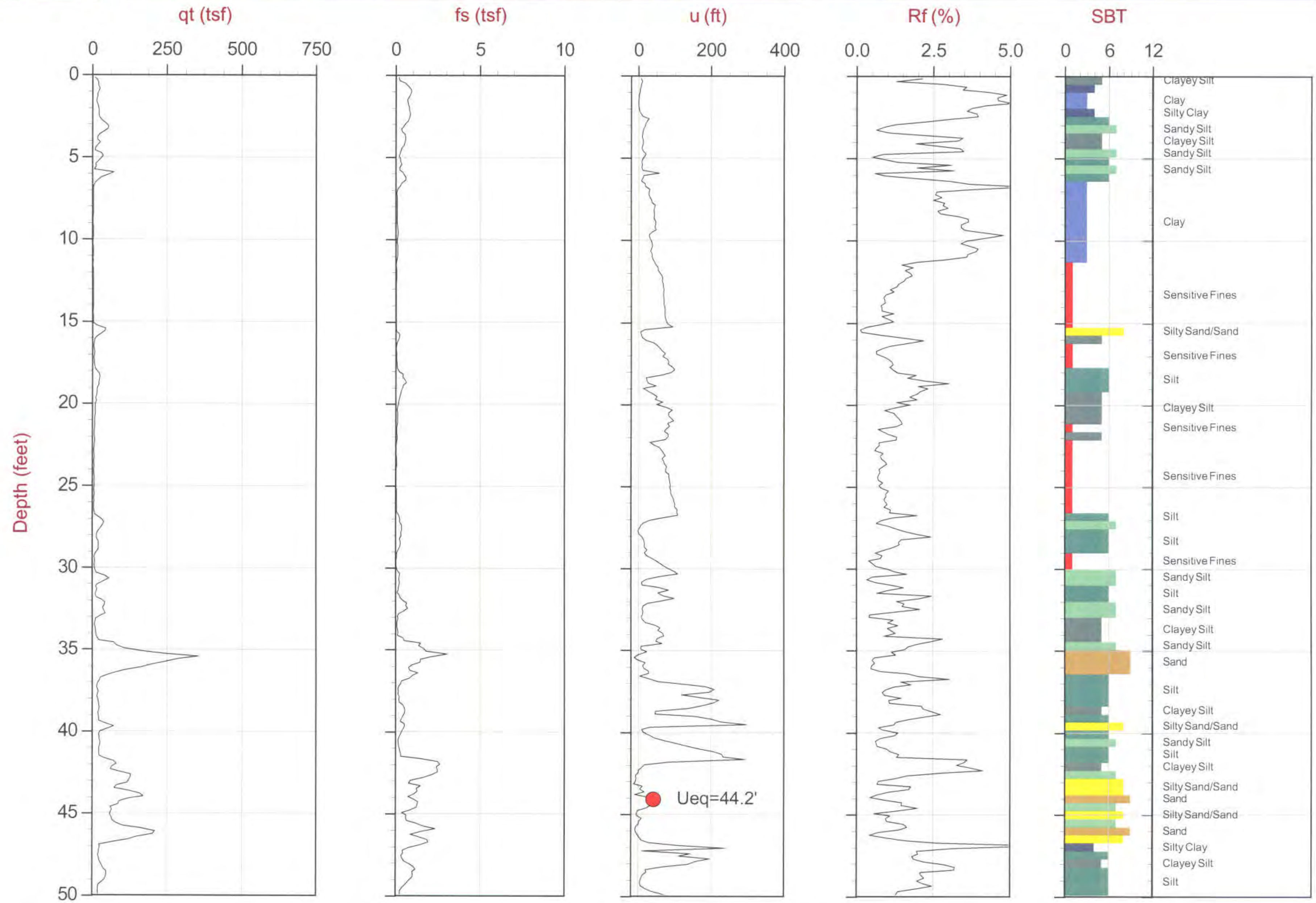
SBT: Lunne, Robertson and Powell, 1997
Page No: 2 of 2
● Equilibrium Pore Pressure from Dissipation

CONETEC

Terracon

Job No: 08-340
Date: 03:19:08 11:54
Site: EAST WEST CONNECTOR

Sounding: CPT-02
Cone: STD 20T AD197



Max Depth: 18.250 m / 59.87 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.150 m

File: 340CP02.COR
Unit Wt: SBT Chart Soil Zones

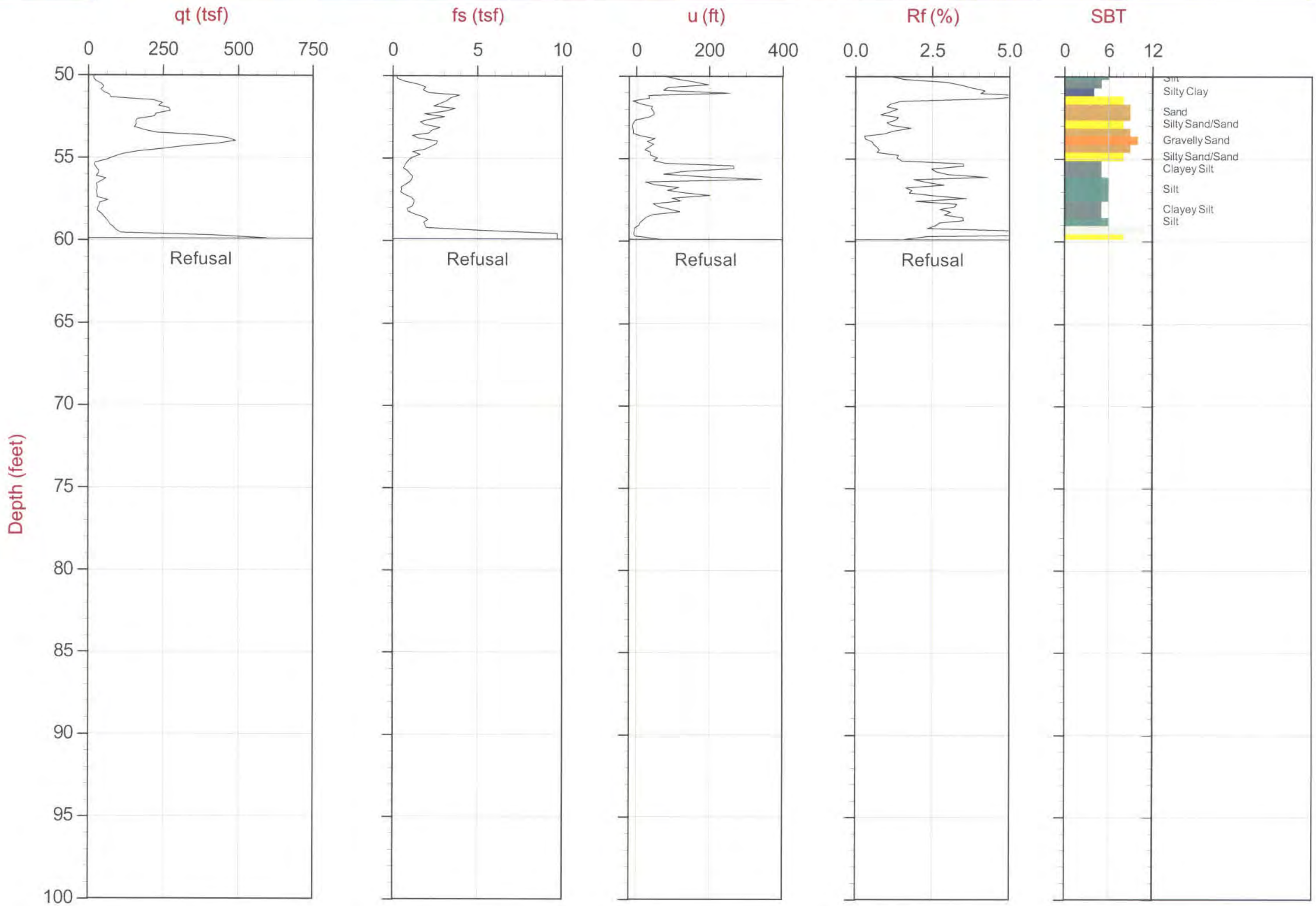
SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 2
● Equilibrium Pore Pressure from Dissipation

ConeTec

Terracon

Job No: 08-340
Date: 03:19:08 11:54
Site: EAST WEST CONNECTOR

Sounding: CPT-02
Cone: STD 20T AD197



Max Depth: 18.250 m / 59.87 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.150 m

File: 340CP02.COR
Unit Wt: SBT Chart Soil Zones

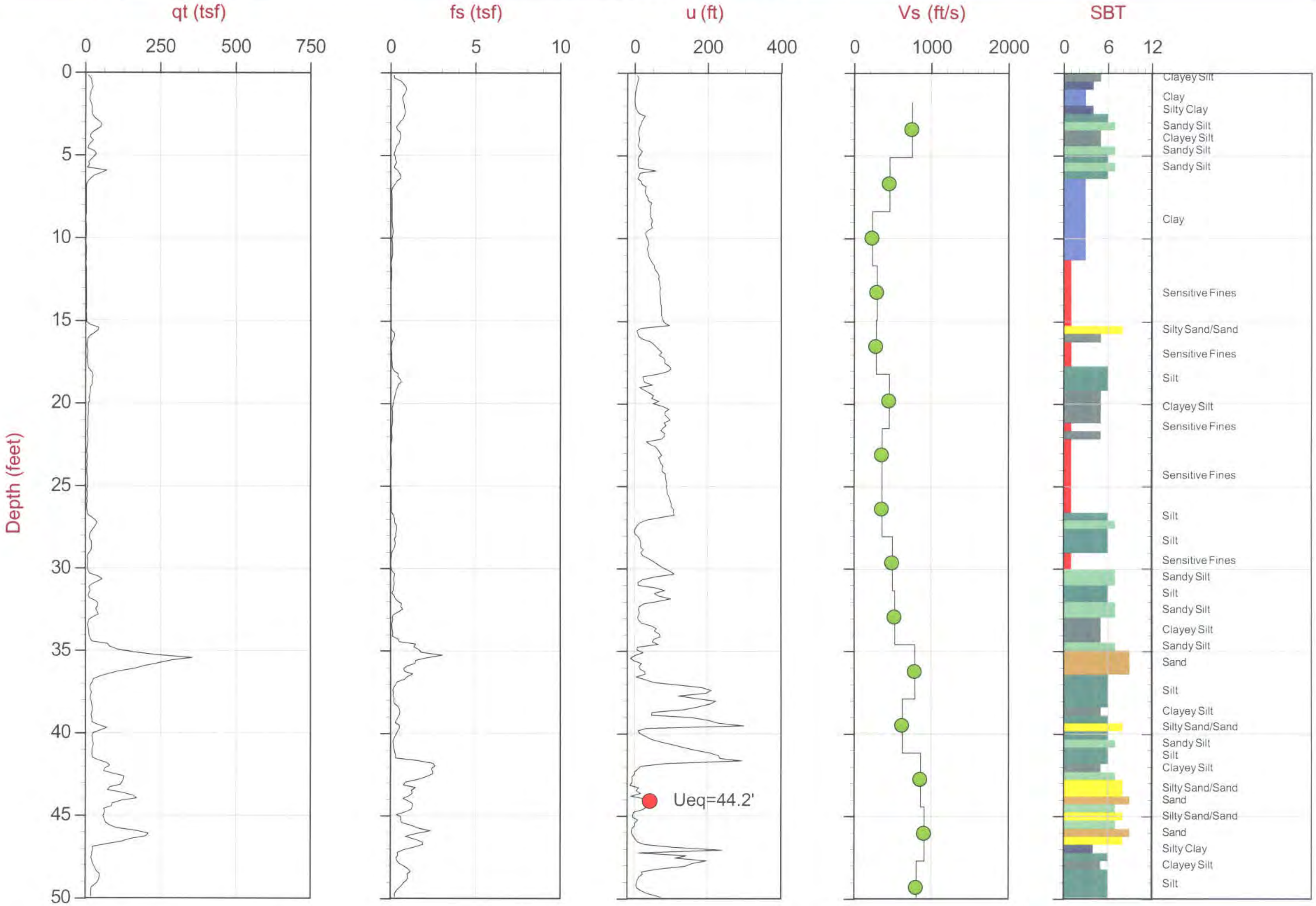
SBT: Lunne, Robertson and Powell, 1997
Page No: 2 of 2

Equilibrium Pore Pressure from Dissipation

ConeTec **Terracon**

Job No: 08-340
Date: 03:19:08 11:54
Site: EAST WEST CONNECTOR

Sounding: CPT-02
Cone: STD 20T AD197



Max Depth: 18.250 m / 59.87 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.150 m

File: 340CP02.COR
Unit Wt: SBT Chart Soil Zones

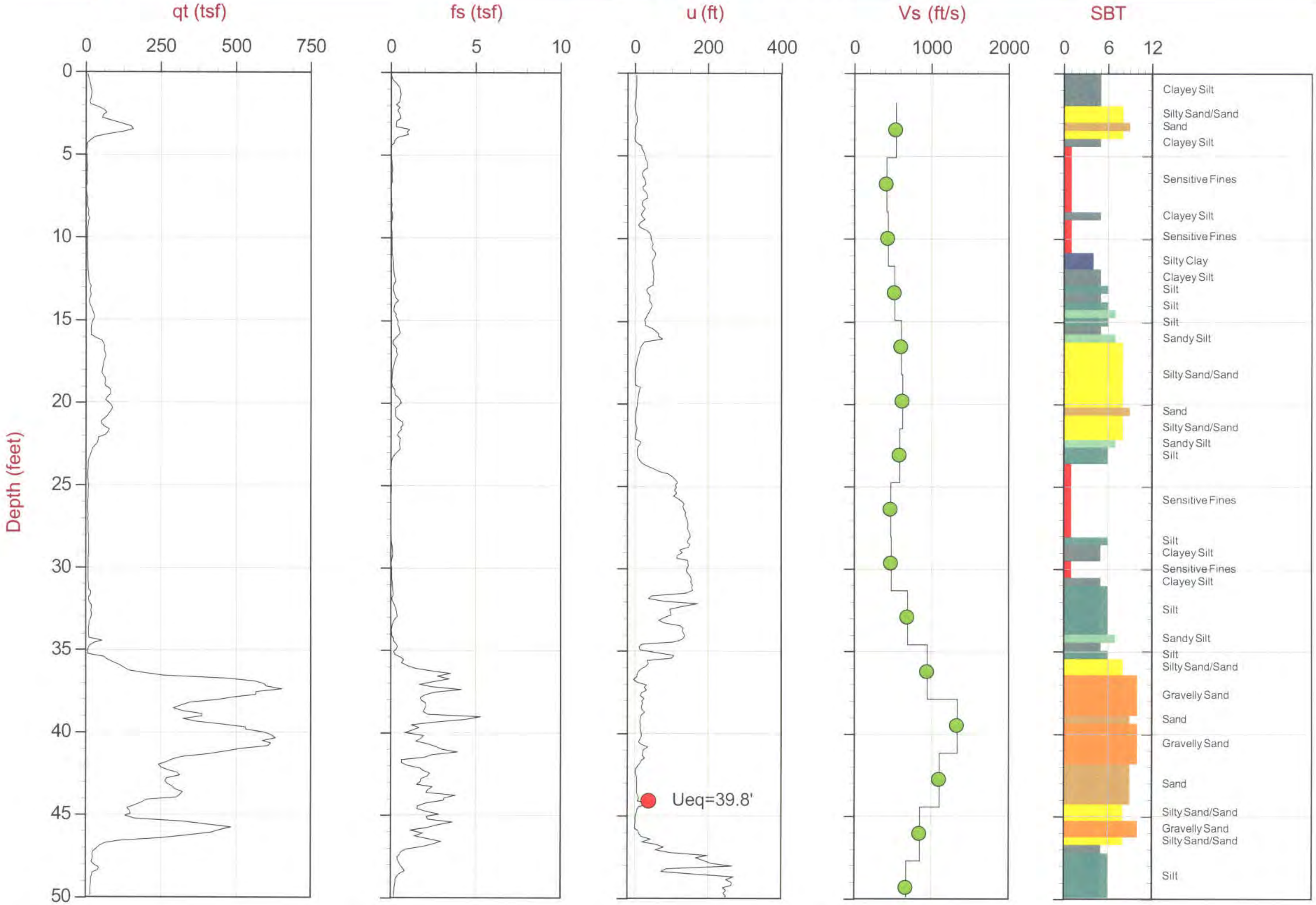
SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 2
● Equilibrium Pore Pressure from Dissipation

CONETEC

Terracon

Job No: 08-340
Date: 03:19:08 13:27
Site: EAST WEST CONNECTOR

Sounding: CPT-03
Cone: STD 20T AD197



Max Depth: 23.100 m / 75.79 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.150 m

File: 340CP03.COR
Unit Wt: SBT Chart Soil Zones

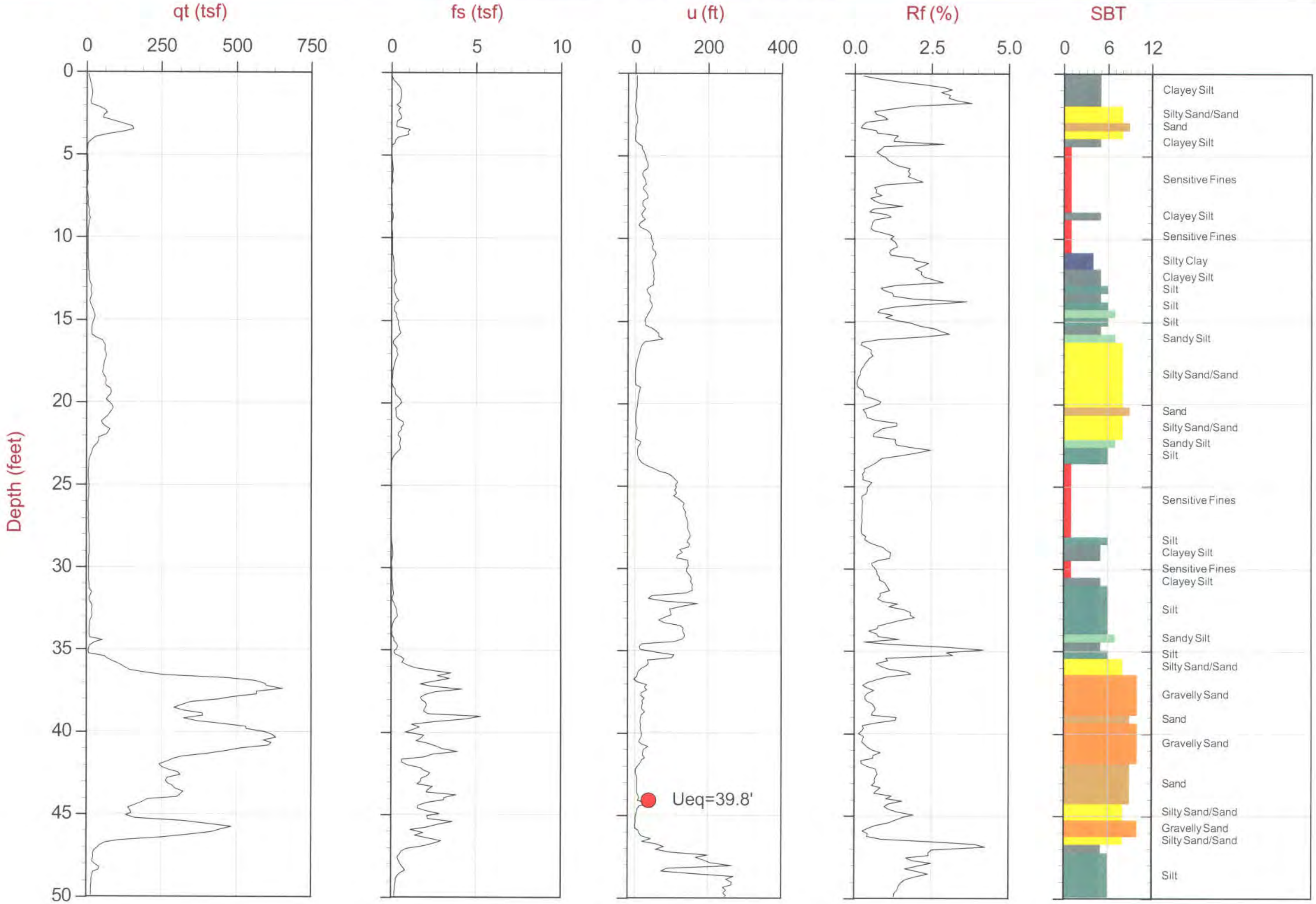
SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 2
● Equilibrium Pore Pressure from Dissipation

CONETEC

Terracon

Job No: 08-340
Date: 03:19:08 13:27
Site: EAST WEST CONNECTOR

Sounding: CPT-03
Cone: STD 20T AD197



Max Depth: 23.100 m / 75.79 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.150 m

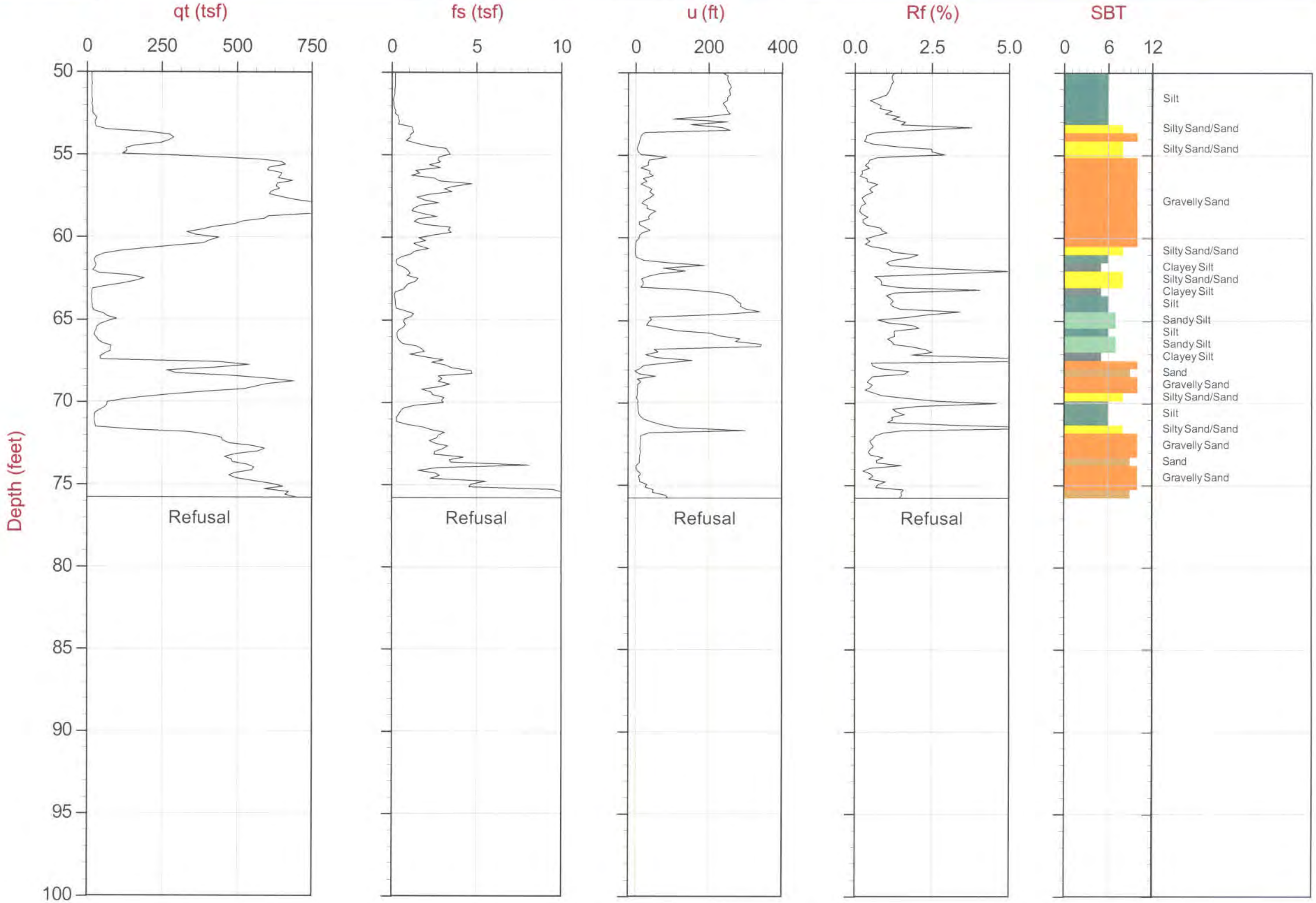
File: 340CP03.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
Page No: 1 of 2
● Equilibrium Pore Pressure from Dissipation

Terracon

Job No: 08-340
Date: 03:19:08 13:27
Site: EAST WEST CONNECTOR

Sounding: CPT-03
Cone: STD 20T AD197



Max Depth: 23.100 m / 75.79 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.150 m

File: 340CP03.COR
Unit Wt: SBT Chart Soil Zones

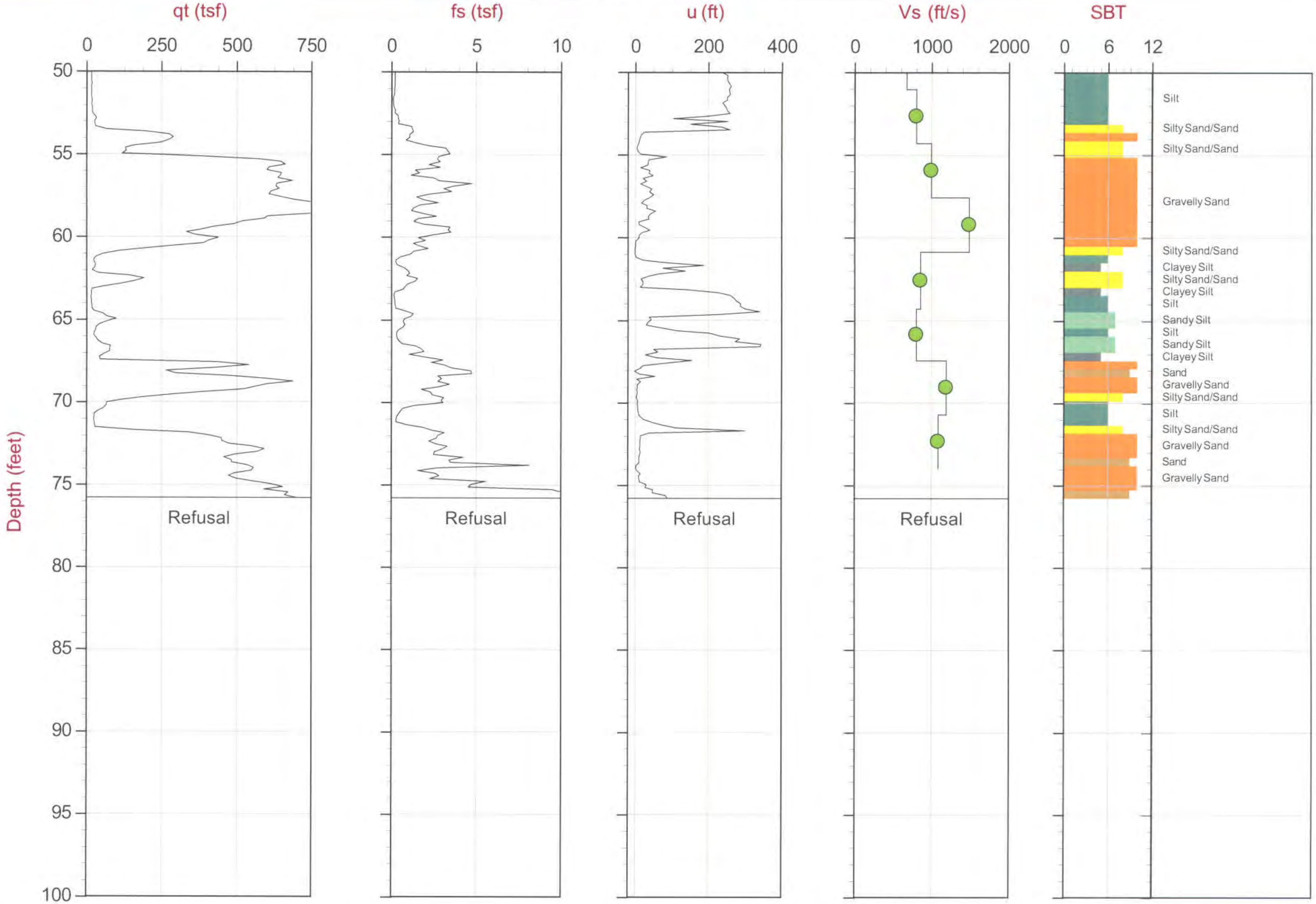
SBT: Lunne, Robertson and Powell, 1997
Page No: 2 of 2

Equilibrium Pore Pressure from Dissipation

ConeTec | Terracon

Job No: 08-340
Date: 03:19:08 13:27
Site: EAST WEST CONNECTOR

Sounding: CPT-03
Cone: STD 20T AD197



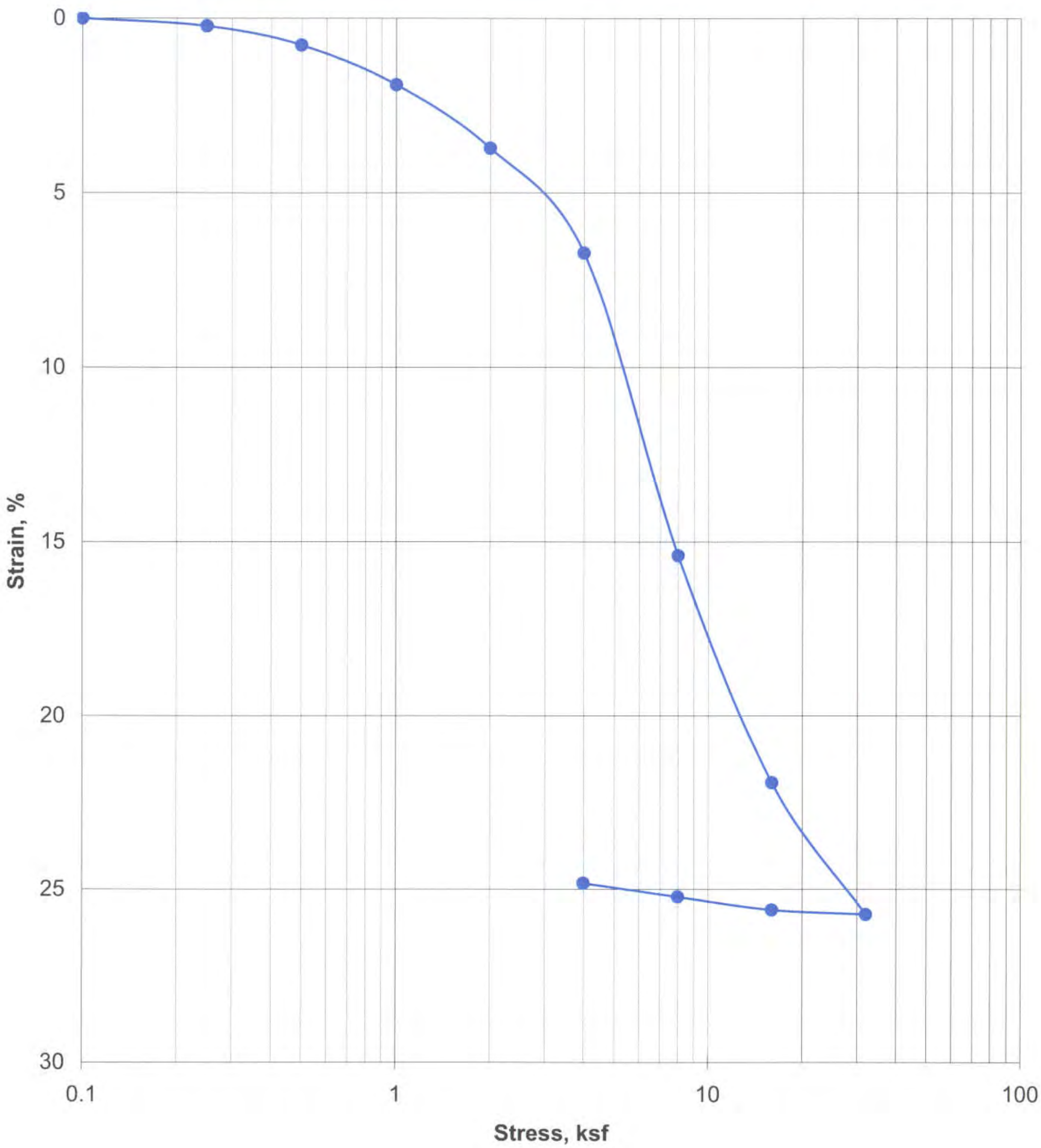
Max Depth: 23.100 m / 75.79 ft
Depth Inc: 0.050 m / 0.164 ft
Avg Int: 0.150 m

File: 340CP03.COR
Unit Wt: SBT Chart Soil Zones

SBT: Lunne, Robertson and Powell, 1997
Page No: 2 of 2
Equilibrium Pore Pressure from Dissipation

APPENDIX B

Laboratory Test Results

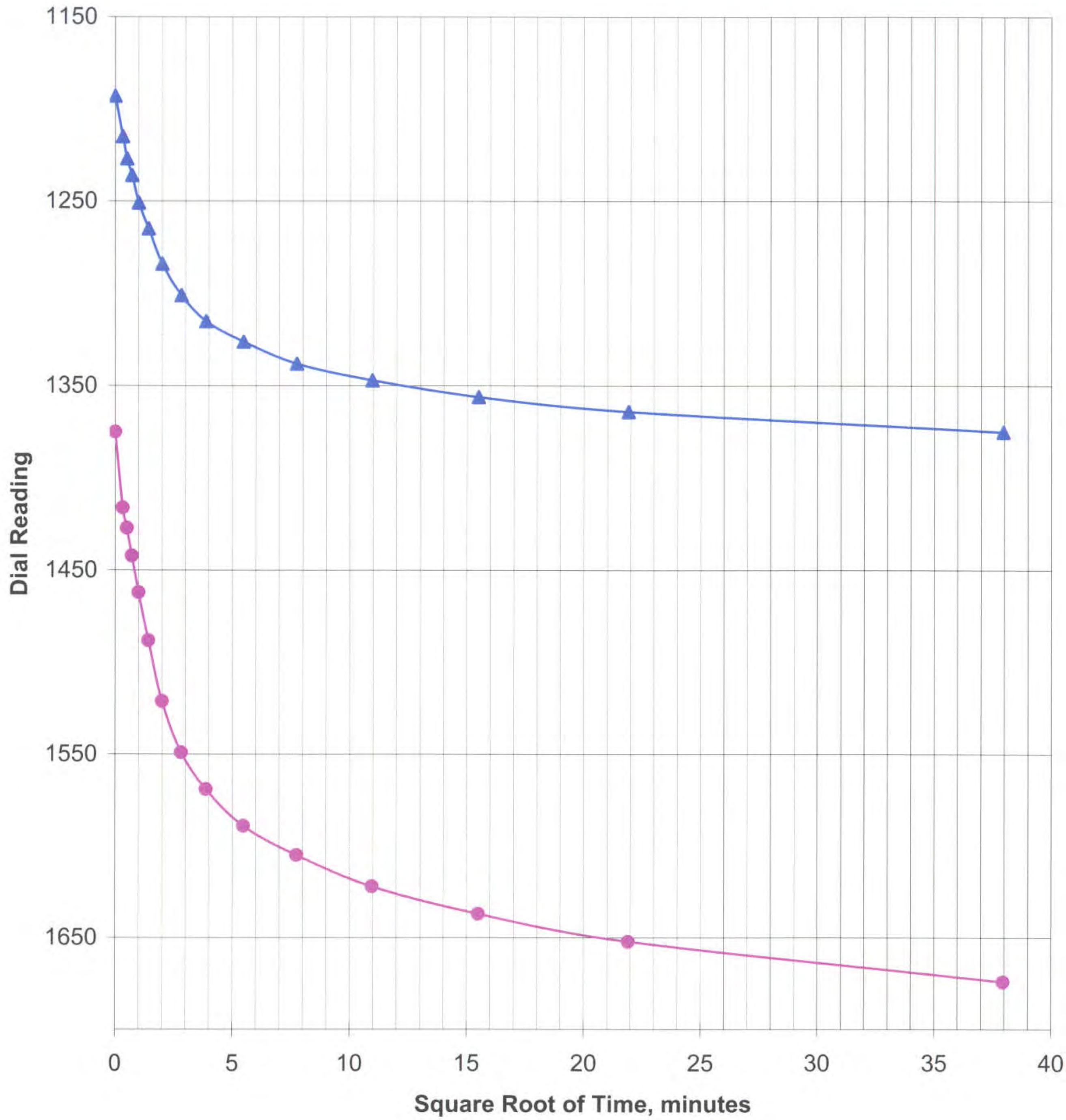


Specimen Identification		Classification	DD	MC%
●	B-6 @ 30 feet	Lean Clay (CL)	97	24
●				

Terracon

CONSOLIDATION TEST RESULTS

Project Name: East-West Connector
Location: Lehi, Utah
Project No.: 61085026
Date: 4/29/08

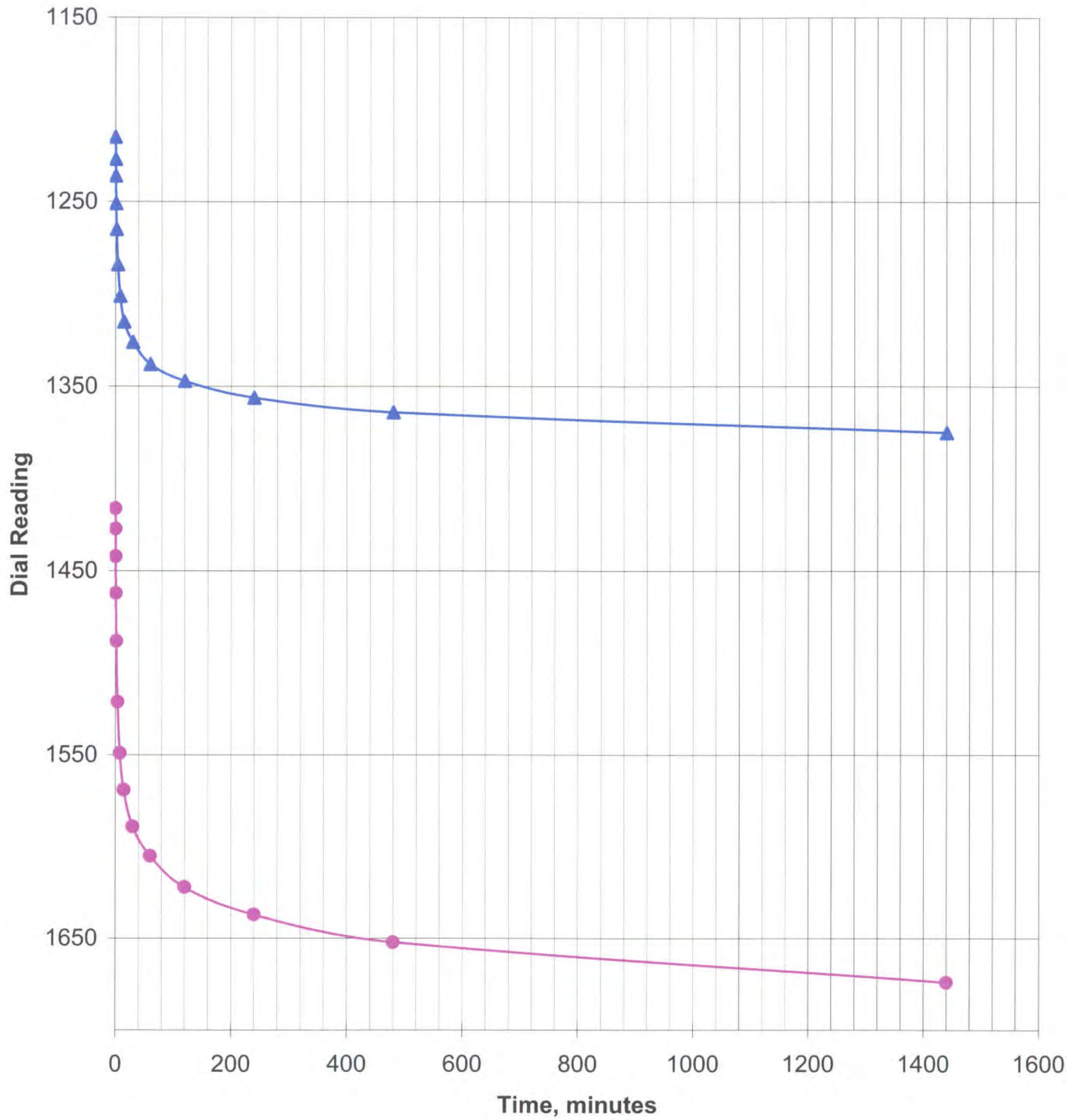


Specimen Identification		Classification
▲	B-6 @ 30 feet, Timed at Load of 2 ksf	Lean Clay (CL)
●	B-6 @ 30 feet, Timed at Load of 4 ksf	Lean Clay (CL)

Terracon

TIMED CONSOLIDATION TEST RESULTS

Project Name:	**East-West Connector**
Location:	**Lehi, Utah**
Project No.:	**61085026**
Date:	**4/29/08**

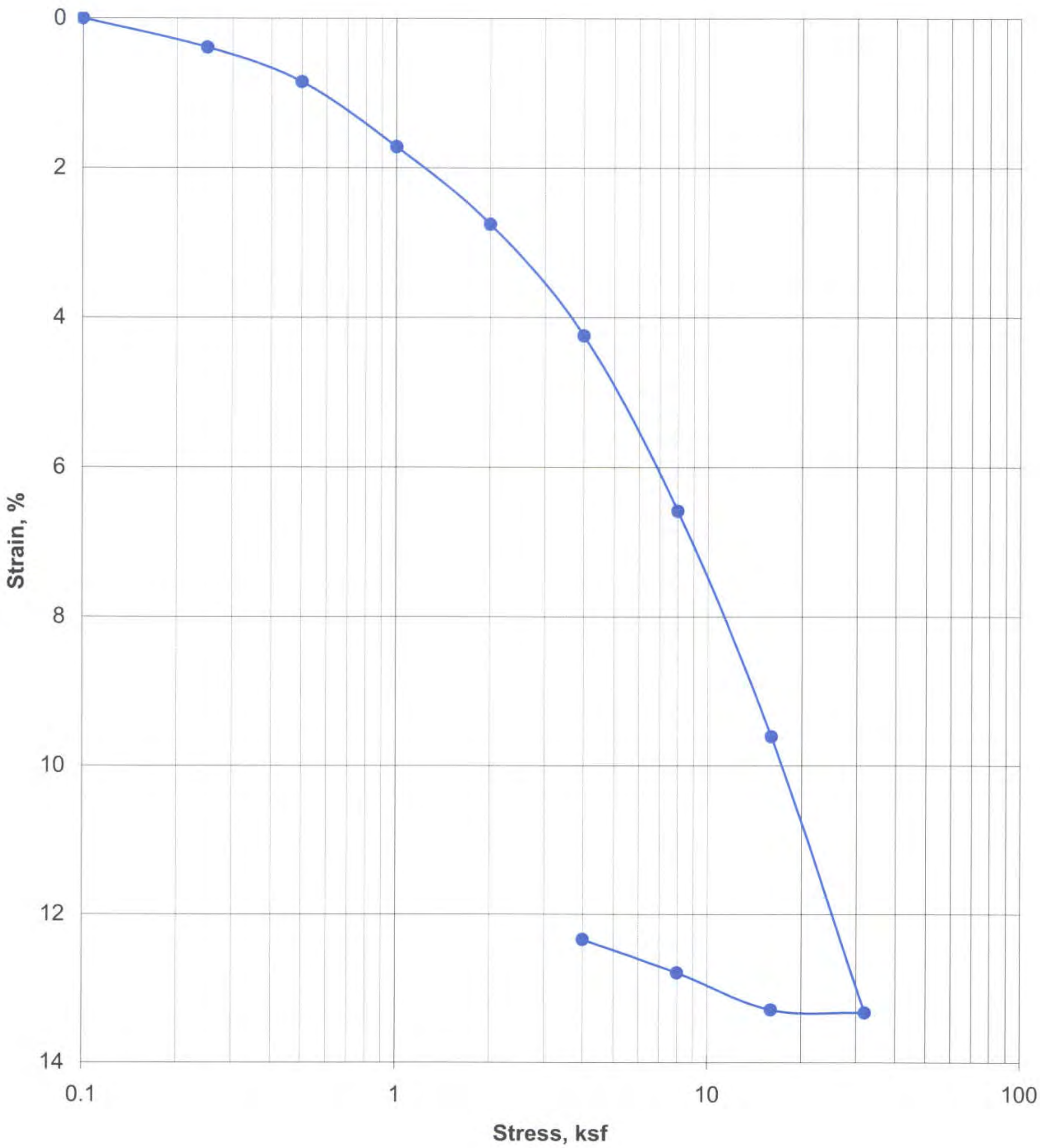


Specimen Identification		Classification
▲	B-6 @ 30 feet, Timed at Load of 2 ksf	Lean Clay (CL)
●	B-6 @ 30 feet, Timed at Load of 4 ksf	Lean Clay (CL)

Terracon

TIMED CONSOLIDATION TEST RESULTS

Project Name: **East-West Connector**
Location: **Lehi, Utah**
Project No.: **61085026**
Date: **4/29/08**

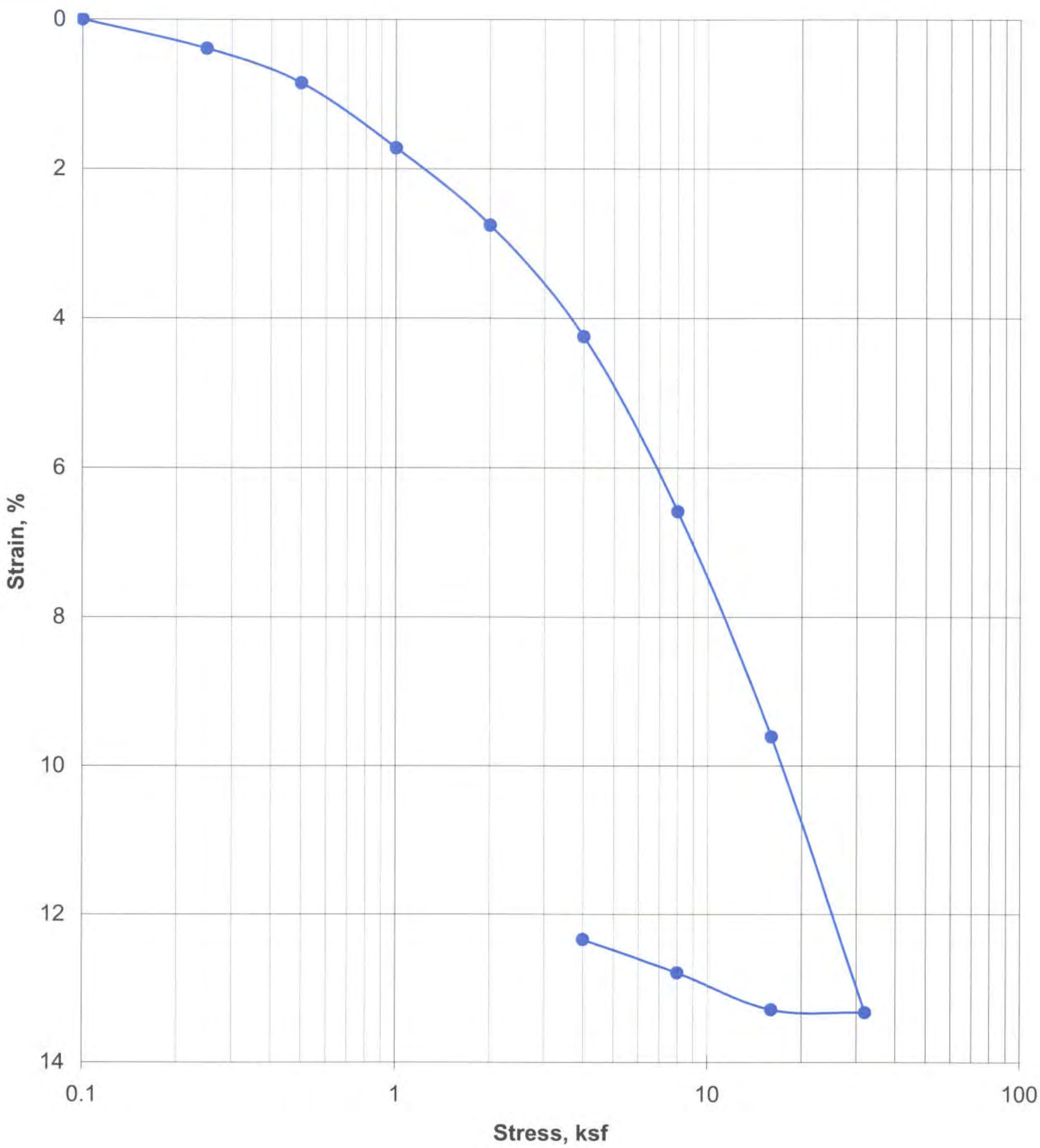


Specimen Identification		Classification	DD	MC%
●	B-17 @ 30 feet	Lean Clay (CL)	71	52
●				

Terracon

CONSOLIDATION TEST RESULTS

Project Name: East-West Connector
Location: Lehi, Utah
Project No.: 61085026
Date: 4/29/08

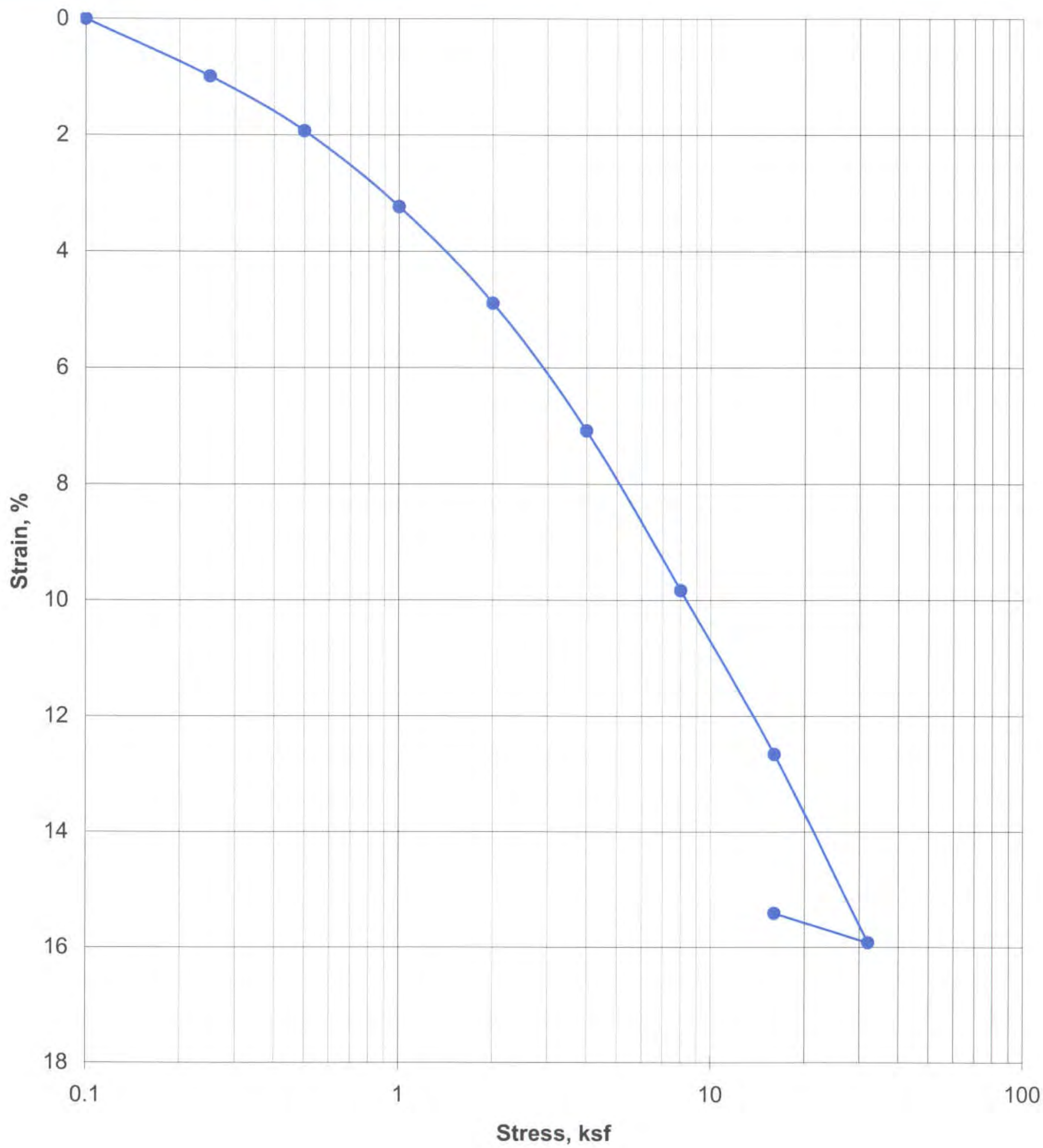


Specimen Identification		Classification	DD	MC%
●	B-17 @ 30 feet	Lean Clay (CL)		
●				

Terracon

CONSOLIDATION TEST RESULTS

Project Name: East-West Connector
Location: Lehi, Utah
Project No.: 61085026
Date: 4/29/08

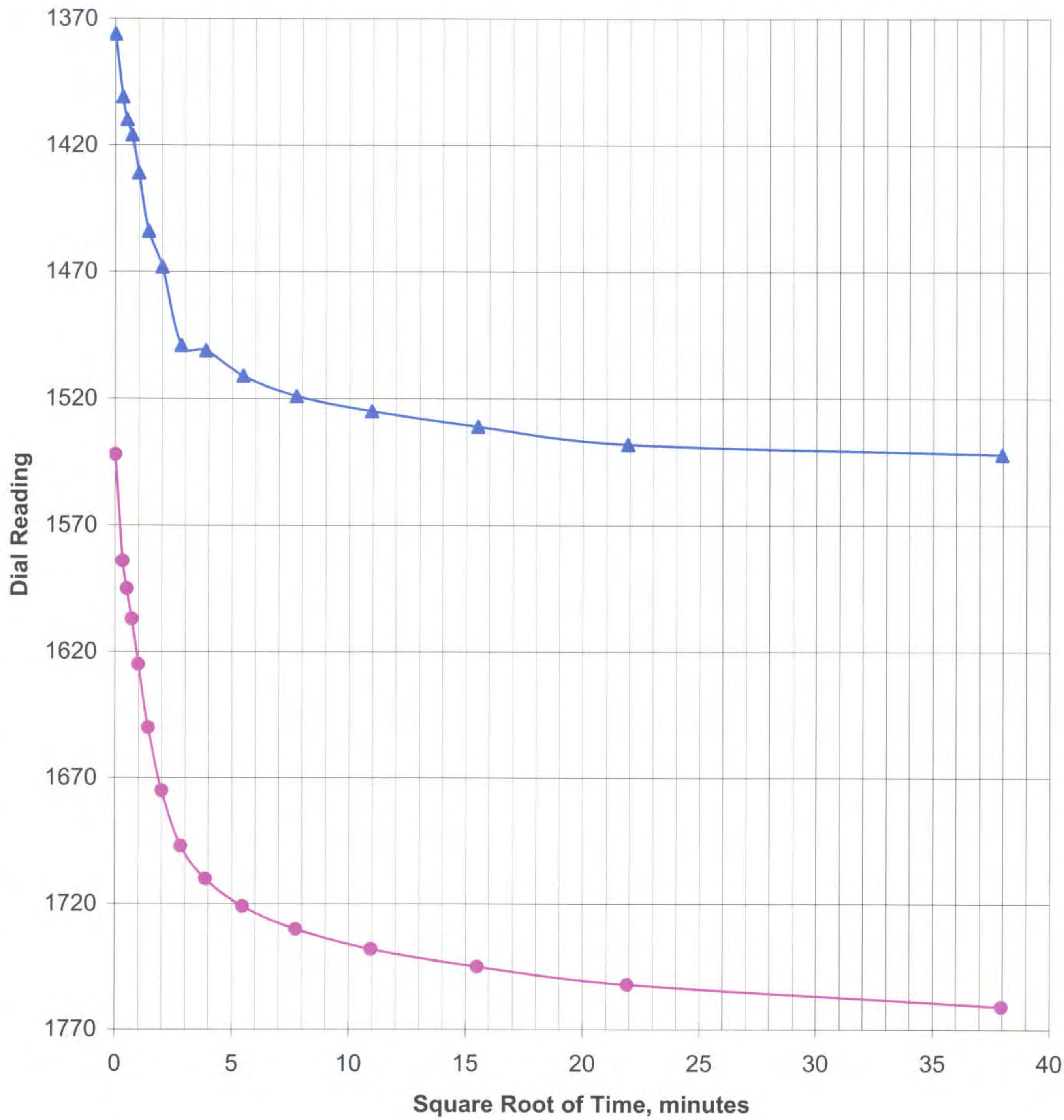


Specimen Identification		Classification	DD	MC%
●	B-17 @ 10 feet	Lean Clay (CL)	92	32
●				

Terracon

CONSOLIDATION TEST RESULTS

Project Name: East-West Connector
Location: Lehi, Utah
Project No.: 61085026
Date: 4/29/08

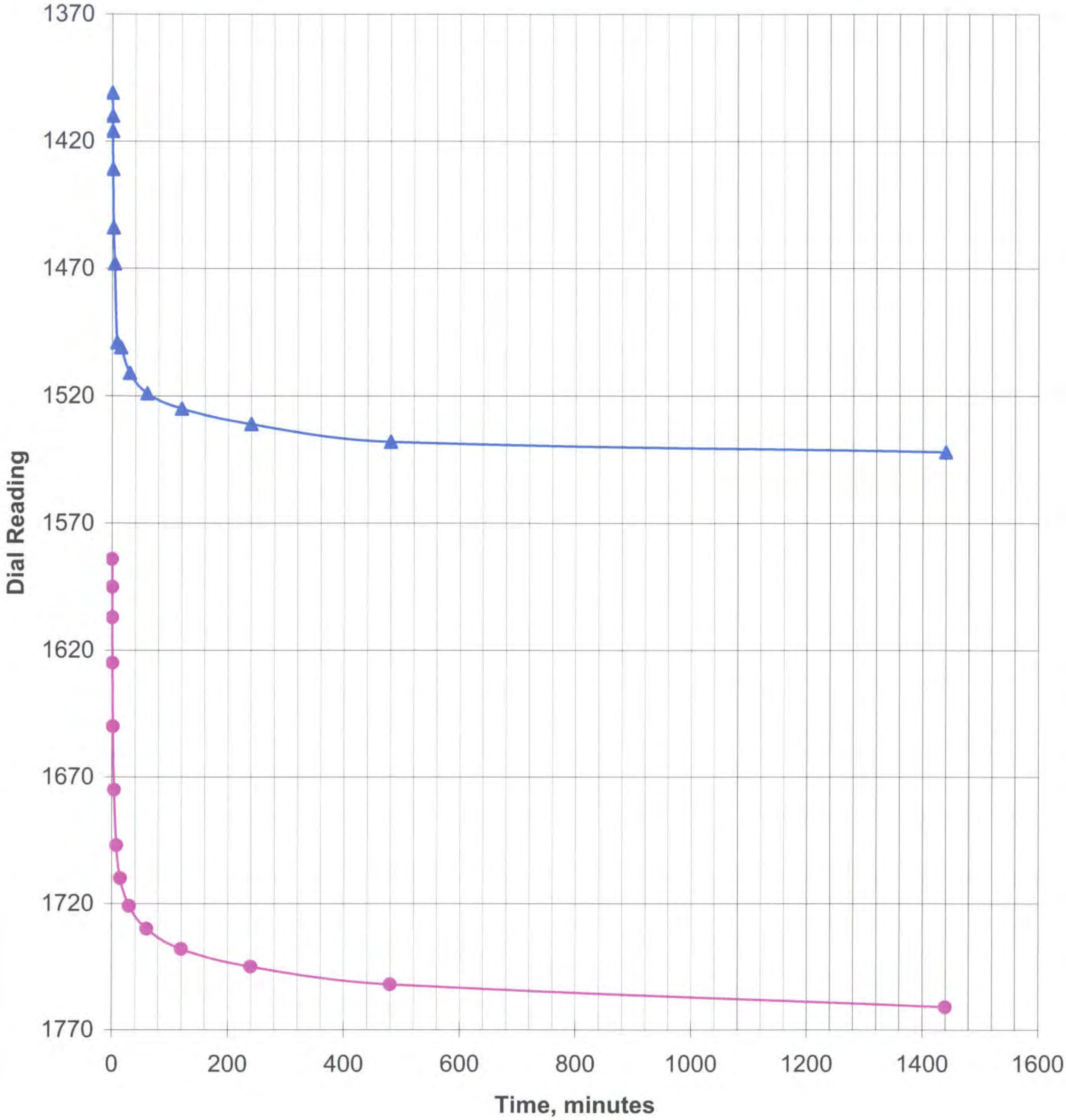


Specimen Identification		Classification
▲	B-17 @ 10 feet, Timed at Load of 2 ksf	Lean Clay (CL)
●	B-17 @ 10 feet, Timed at Load of 4 ksf	Lean Clay (CL)

Terracon

TIMED CONSOLIDATION TEST RESULTS

Project Name: **East-West Connector**
Location: **Lehi, Utah**
Project No.: **61085026**
Date: **4/29/08**

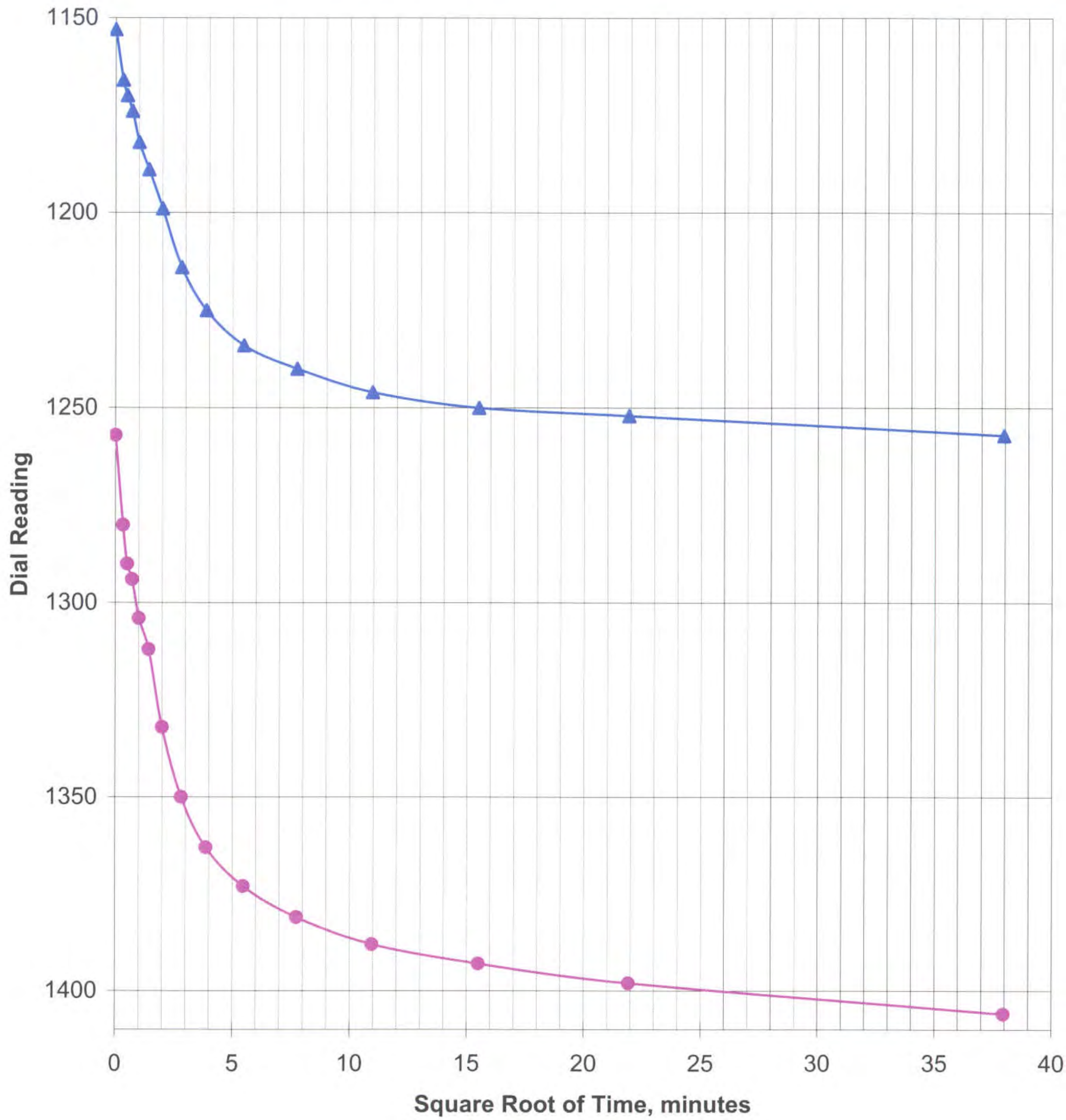


Specimen Identification		Classification
▲	B-17 @ 10 feet, Timed at Load of 2 ksf	Lean Clay (CL)
●	B-17 @ 10 feet, Timed at Load of 4 ksf	Lean Clay (CL)

Terracon

TIMED CONSOLIDATION TEST RESULTS

Project Name: East-West Connector
Location: Lehi, Utah
Project No.: 61085026
Date: 4/29/08

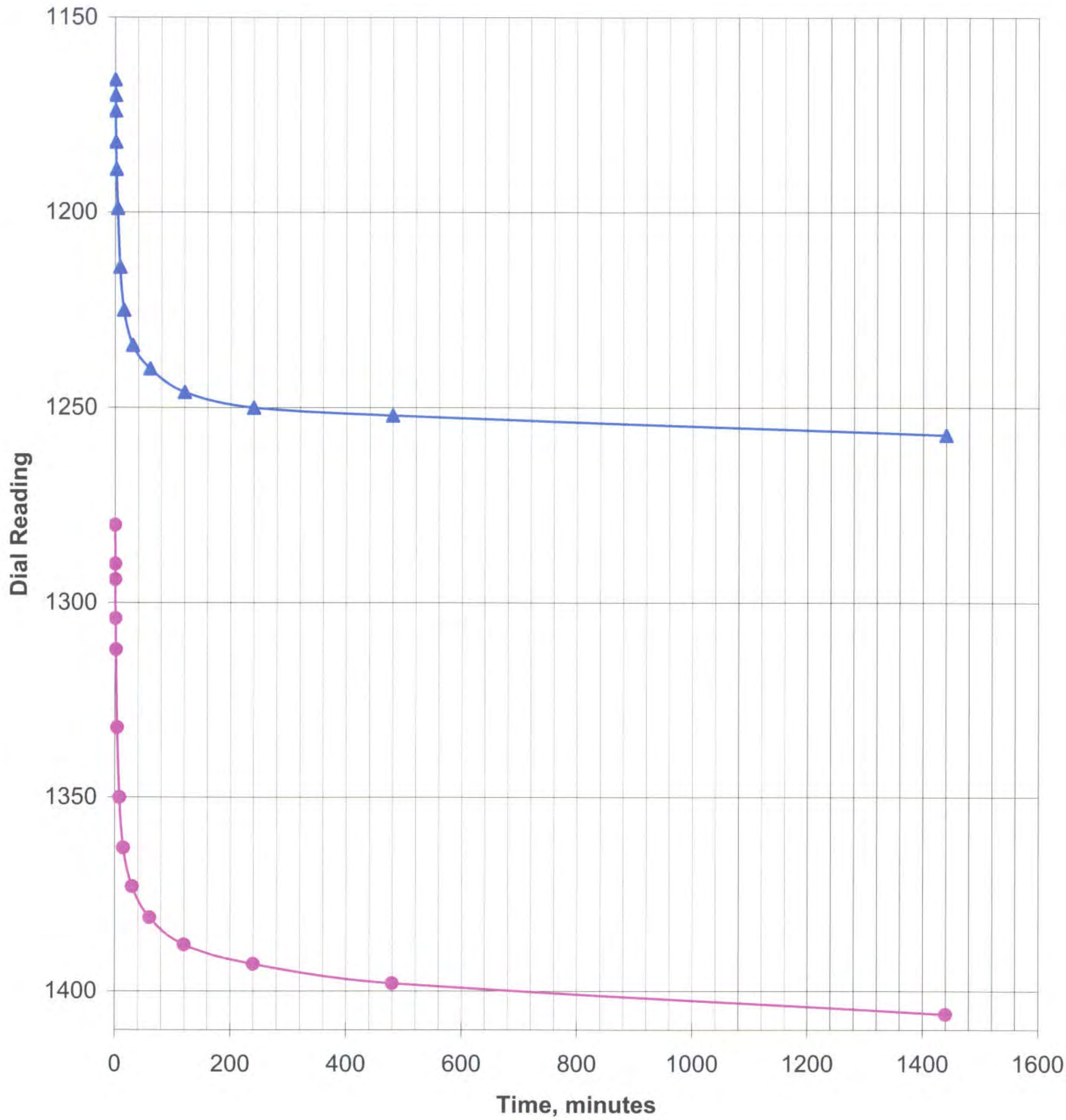


Specimen Identification	Classification
▲ **B-17 @ 30 feet, Timed at Load of 2 ksf**	**Lean Clay (CL)**
● **B-17 @ 30 feet, Timed at Load of 4 ksf**	**Lean Clay (CL)**

Terracon

TIMED CONSOLIDATION TEST RESULTS

Project Name: **East-West Connector**
Location: **Lehi, Utah**
Project No.: **61085026**
Date: **4/29/08**

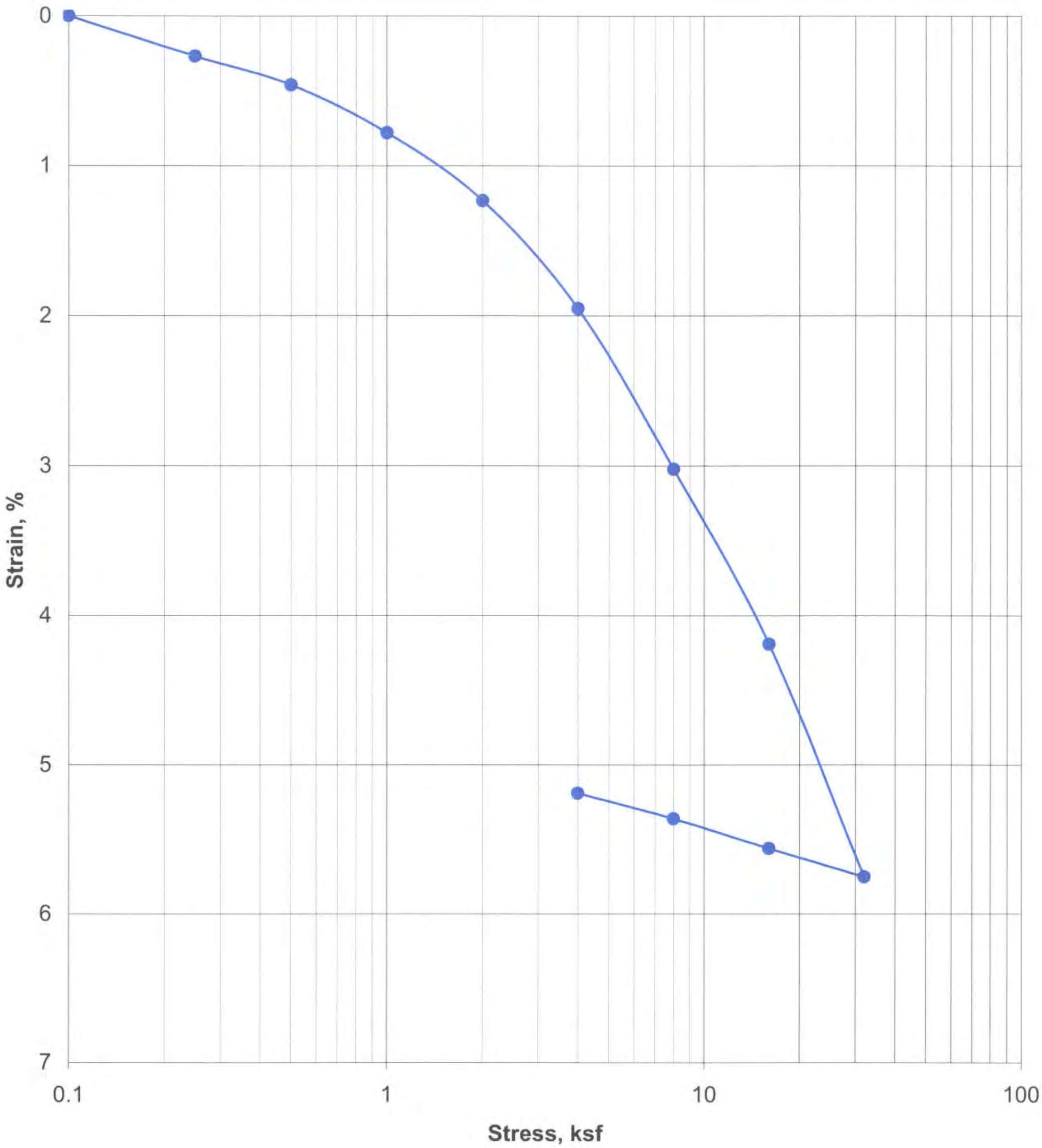


	Specimen Identification	Classification
▲	B-17 @ 30 feet, Timed at Load of 2 ksf	Lean Clay (CL)
●	B-17 @ 30 feet, Timed at Load of 4 ksf	Lean Clay (CL)

Terracon

TIMED CONSOLIDATION TEST RESULTS

Project Name: **East-West Connector**
Location: **Lehi, Utah**
Project No.: **61085026**
Date: **4/29/08**

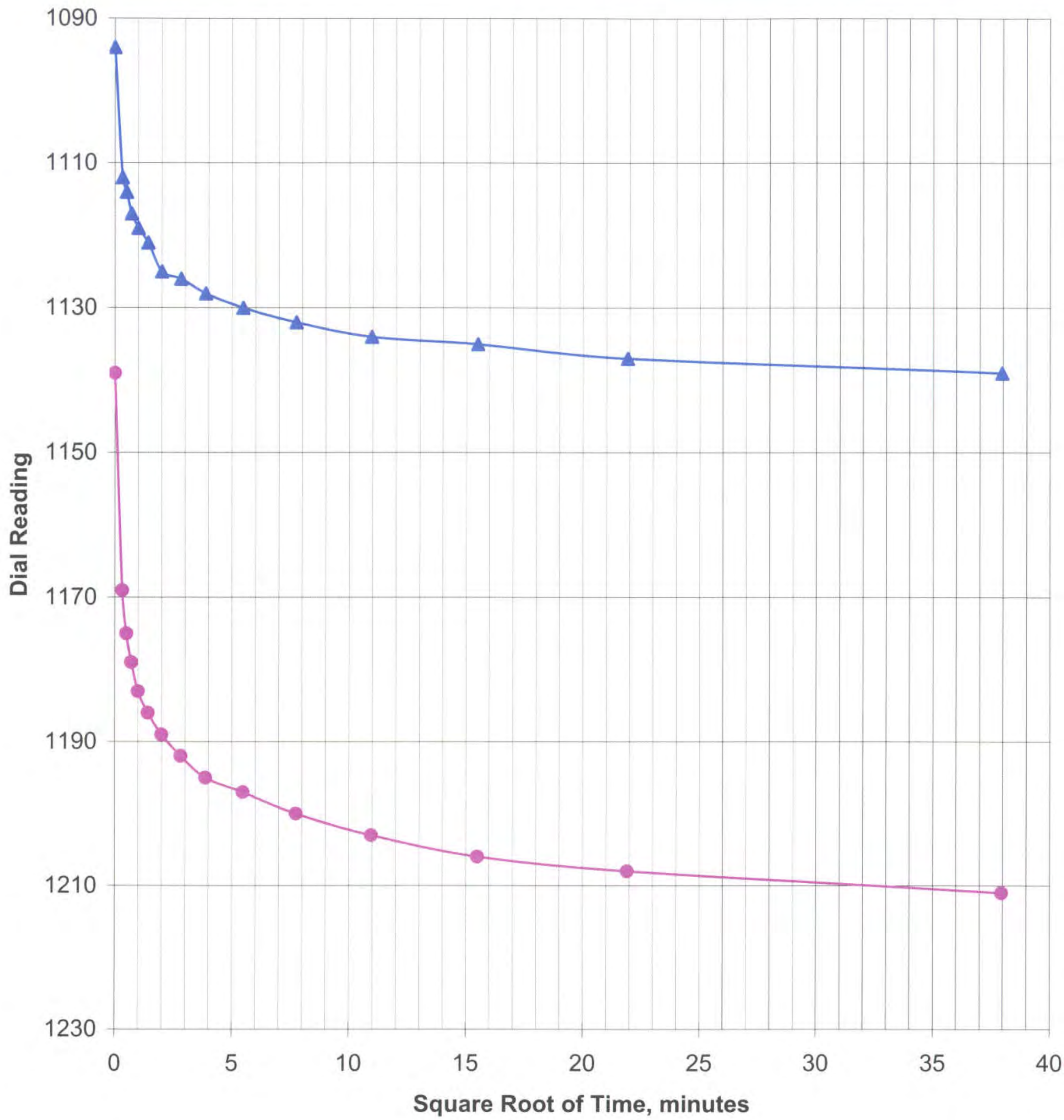


Specimen Identification		Classification	DD	MC%
●	B-17 @ 55 feet	Lean Clay (CL)		
●				

Terracon

CONSOLIDATION TEST RESULTS

Project Name: East-West Connector
Location: Lehi, Utah
Project No.: 61085026
Date: 4/29/08

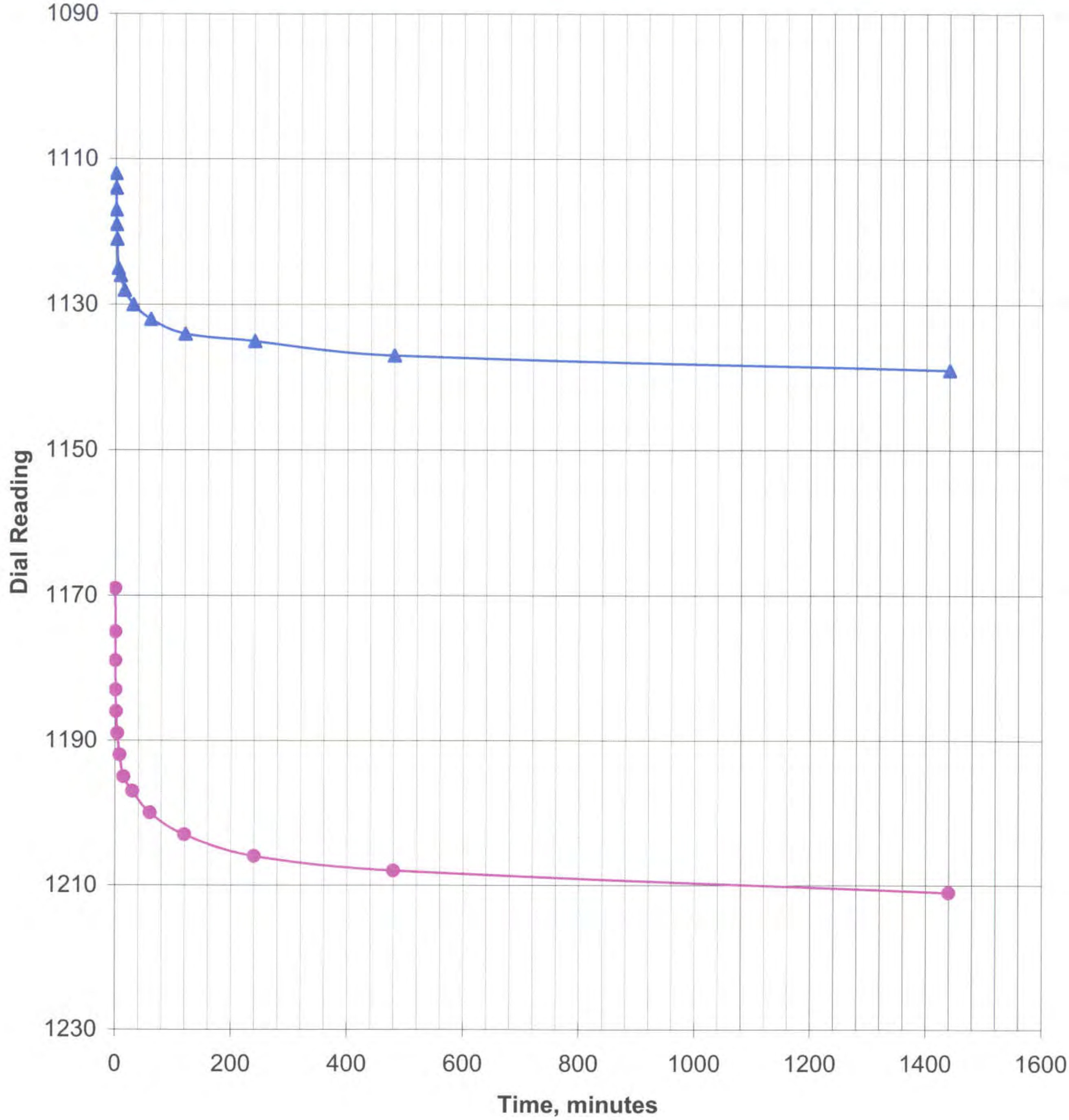


Specimen Identification		Classification
▲	B-17 @ 55 feet, Timed at Load of 2 ksf	Lean Clay (CL)
●	B-17 @ 55 feet, Timed at Load of 4 ksf	Lean Clay (CL)

Terracon

TIMED CONSOLIDATION TEST RESULTS

Project Name: **East-West Connector**
Location: **Lehi, Utah**
Project No.: **61085026**
Date: **4/29/08**



Specimen Identification		Classification
▲	**B-17 @ 55 feet, Timed at Load of 2 ksf**	**Lean Clay (CL)**
●	**B-17 @ 55 feet, Timed at Load of 4 ksf**	**Lean Clay (CL)**

Terracon

TIMED CONSOLIDATION TEST RESULTS

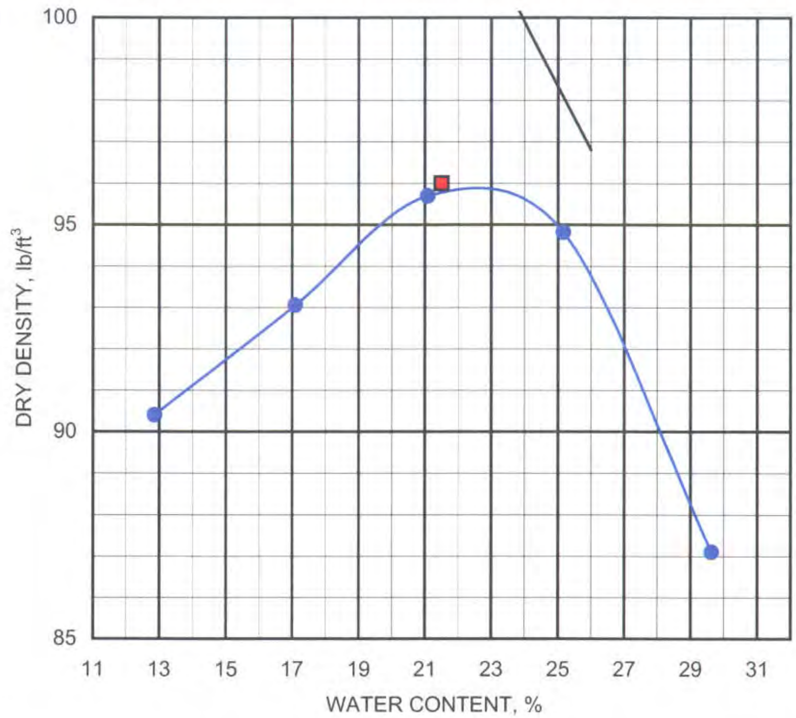
Project Name:	**East-West Connector**
Location:	**Lehi, Utah**
Project No.:	**61085026**
Date:	**4/29/08**

GRADATION RESULTS

SIEVE SIZE	PERCENT PASSING
No. 8	99
No. 20	97
No. 40	94
No. 80	89
No. 100	88
No. 200	81

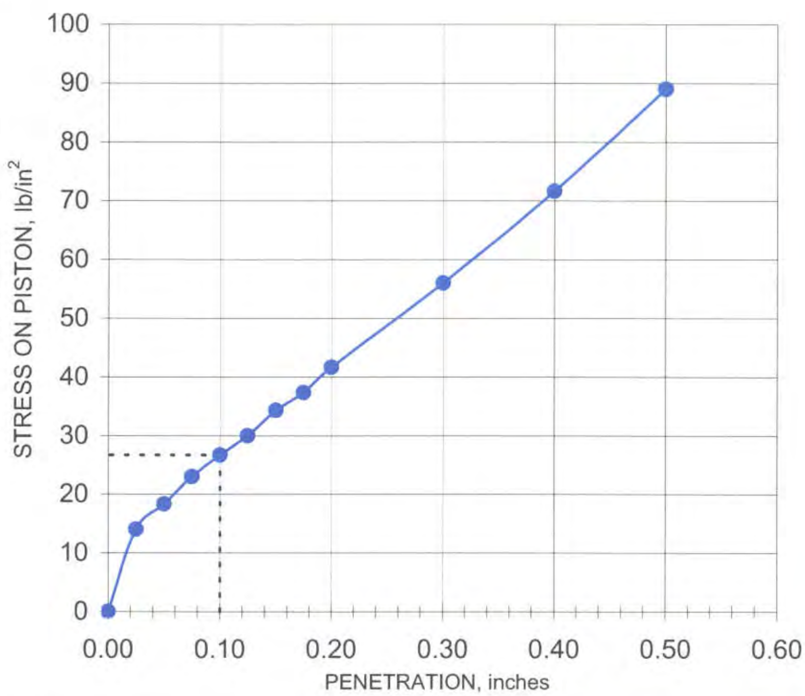
Liquid Limit:
Plasticity Index:

Maximum Dry Density, lb/ft^3: **96** Test Method: **ASTM D 698**
Optimum Moisture, %: **21.5** (ZERO AIR VOIDS FOR SPECIFIC GRAVITY OF 2.6)



CALIFORNIA BEARING RATIO

Dry Density,	
Before Soaking, lb/ft^3:	95.2
After Soaking, lb/ft^3:	
Relative Compaction, %:	99
Moisture Content,	
Before Compaction, %:	20.1
Top 1-inch After Soaking, %:	34.4
Average After Soaking, %:	29.5
Surcharge Weight, lb:	15
Soaking Period, hr:	96
Swell, %:	
CBR Value, %:	2.7



SAMPLE - IDENTIFICATION
B-5 @ 1.0 - 3.0 ft

CLASSIFICATION
Sandy Clay (CL)

Terracon

GRADATION, MOISTURE-DENSITY AND CALIFORNIA BEARING RATIO TEST RESULTS

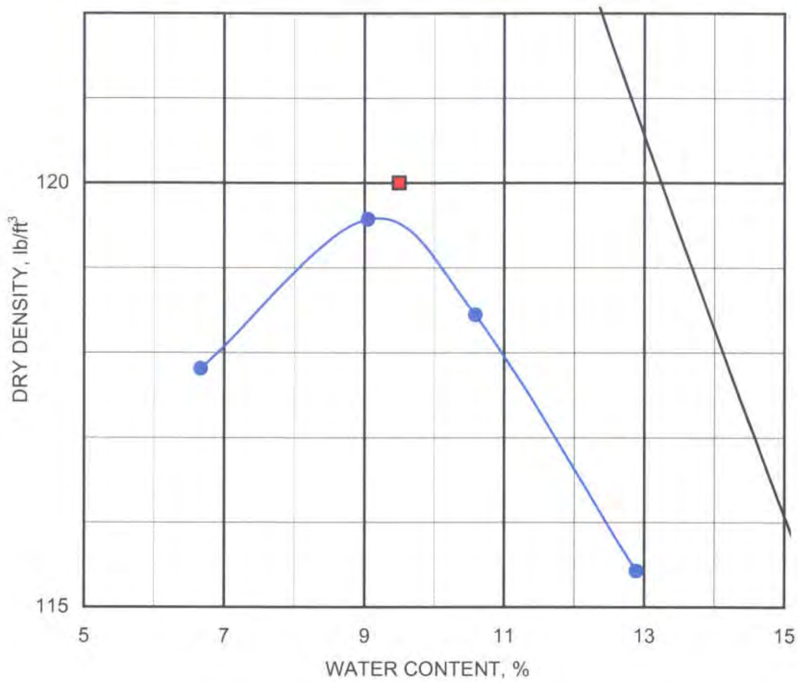
Project Name: **East West Connector**
Location: **Lehi, Utah**
Project No.: **61085026**
Date: **6/12/08**

GRADATION RESULTS

SIEVE SIZE	PERCENT PASSING
1/2"	**100**
No. 8	**95**
No. 20	**89**
No. 40	**84**
No. 80	**70**
No. 100	**65**
No. 200	**42**

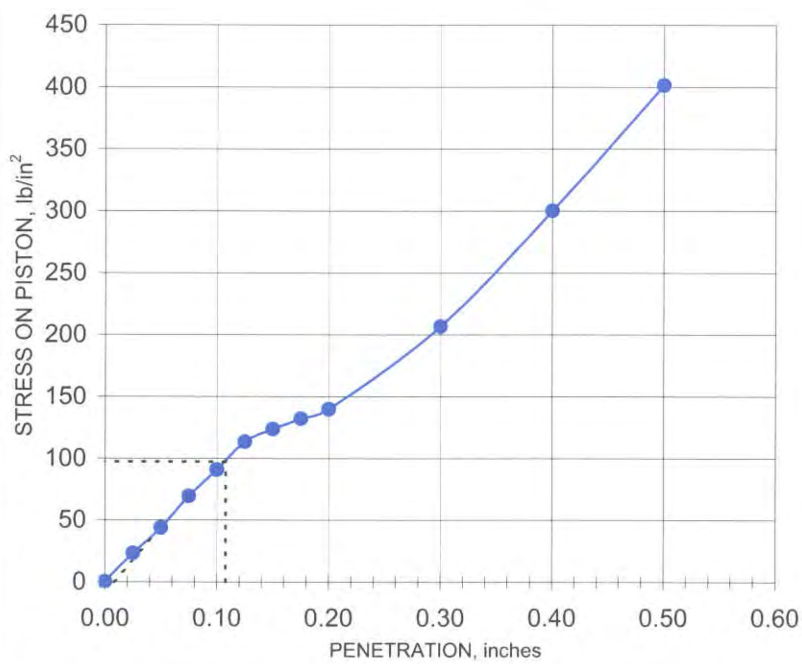
Liquid Limit: --
Plasticity Index: --

Maximum Dry Density, lb/ft^3: **120** Test Method: **ASTM D 698**
Optimum Moisture, %: **9.5** (ZERO AIR VOIDS FOR SPECIFIC GRAVITY OF 2.58)



CALIFORNIA BEARING RATIO

Dry Density,	
Before Soaking, lb/ft^3:	**118.8**
After Soaking, lb/ft^3:	**119.7**
Relative Compaction, %:	**99**
Moisture Content,	
Before Compaction, %:	**6.8**
Top 1-inch After Soaking, %:	**11.2**
Average After Soaking, %:	**11.7**
Surcharge Weight, lb:	**15**
Soaking Period, hr:	**96**
Swell, %:	**0.0**
CBR Value, %:	**9.7**



SAMPLE - IDENTIFICATION
B24 @ 6'-8

CLASSIFICATION

Terracon

GRADATION, MOISTURE-DENSITY AND CALIFORNIA BEARING RATIO TEST RESULTS

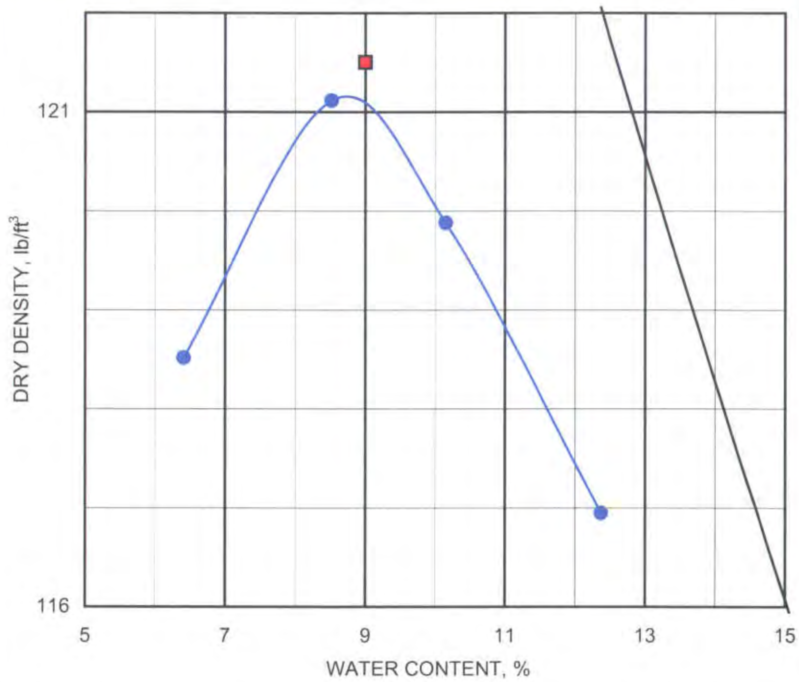
Project Name: **E-W Connector**
Location: **B-24 @ 6'-8'**
Project No.: **61085026**
Date: **6/12/2008**

GRADATION RESULTS

SIEVE SIZE	PERCENT PASSING
1-1/2"	100
1"	100
3/4"	99
3/8"	96
No. 4	90
No. 8	83
No. 10	81
No. 16	72
No. 20	66
No. 30	58
No. 40	50
No. 50	42
No. 80	29
No. 100	25
No. 200	13

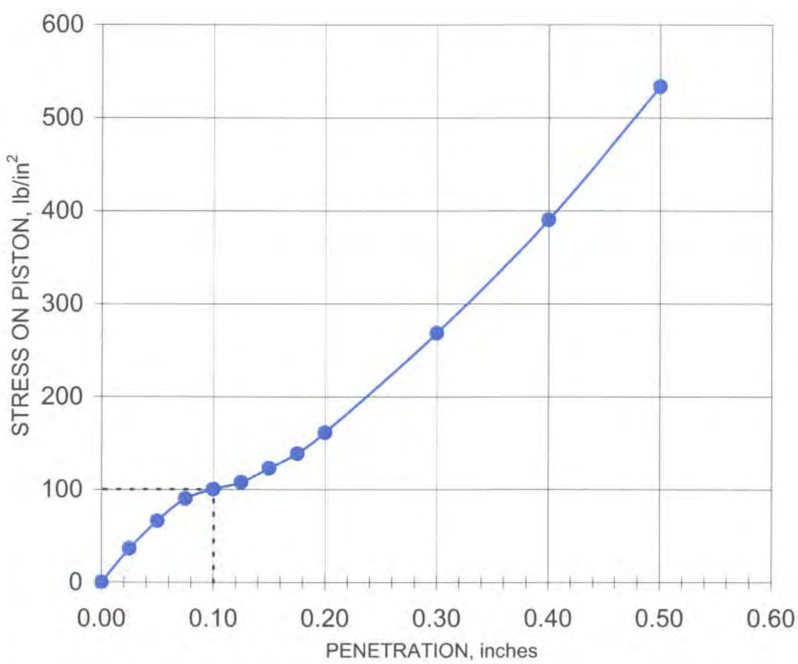
Liquid Limit: --
Plasticity Index: --

Maximum Dry Density, lb/ft^3: **121.5** Test Method: **ASTM D 698**
Optimum Moisture, %: **9** (ZERO AIR VOIDS FOR SPECIFIC GRAVITY OF 2.58)



CALIFORNIA BEARING RATIO

Dry Density,	
Before Soaking, lb/ft^3:	120.5
After Soaking, lb/ft^3:	120.9
Relative Compaction, %:	99
Moisture Content,	
Before Compaction, %:	8.4
Top 1-inch After Soaking, %:	11.3
Average After Soaking, %:	11.6
Surcharge Weight, lb:	15
Soaking Period, hr:	96
Swell, %:	0.2
CBR Value, %:	10.0



SAMPLE - IDENTIFICATION
B-27.3'-6

CLASSIFICATION

Terracon

GRADATION, MOISTURE-DENSITY AND CALIFORNIA BEARING RATIO TEST RESULTS

Project Name: **E-W Connector**
Location: **B-27 @ 3'-6'**
Project No.: **61085026**
Date: **6/12/2008**

Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

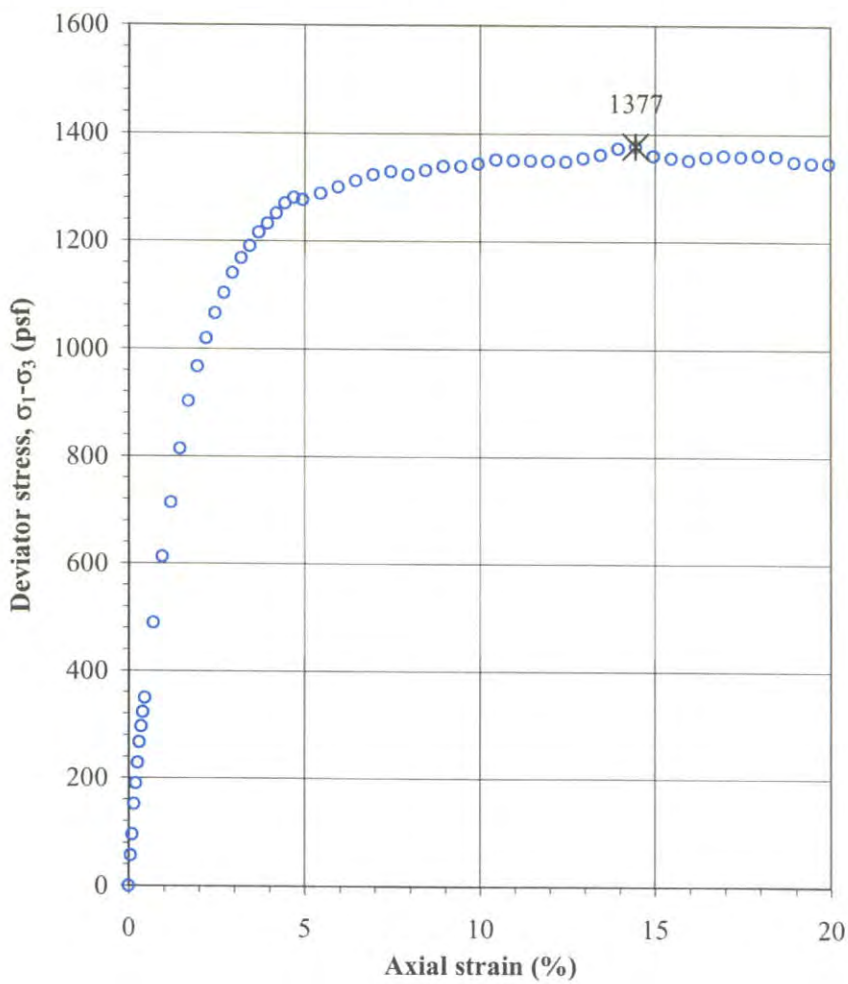
Project: Terracon (61085026)
No: M00385-078
Location: East/ West Connector Lehi
Date: 4/16/2008
By: BRR

Boring No.: B-2
Sample:
Depth: 10
Sample Description: gray clay
Sample type: Undisturbed

Sample height, H (in.)	5.980
Sample diameter, D (in.)	2.840
Sample volume, V (ft^3)	0.0219
Wt. rings + wet soil (g)	1213.05
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1213.05
Moist unit wt., γ_m (pcf)	122.0
Dry unit wt., γ_d (pcf)	**92.8**

Wet soil + tare (g)	559.08
Dry soil + tare (g)	458.65
Tare (g)	139.73
Moisture content, w (%)	**31.5**
Confining stress, σ3 (psf)	706
Shear rate (in/min)	0.0179
Strain at failure, ε_f (%)	14.45
Deviator stress at failure, $(\sigma 1-\sigma 3)_f$ (psf)	1377
Shear stress at failure, $q_f = (\sigma 1_f - \sigma 3_f)/2$ (psf)	689

Axial Strain (%)	σd σ1-σ3 (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	56.6	28.3
0.10	95.2	47.6
0.15	151.7	75.8
0.20	190.2	95.1
0.25	228.7	114.3
0.30	267.2	133.6
0.35	296.6	148.3
0.40	323.2	161.6
0.45	349.6	174.8
0.70	490.5	245.2
0.95	612.9	306.4
1.20	714.0	357.0
1.45	814.6	407.3
1.70	902.9	451.4
1.95	967.4	483.7
2.20	1019.9	509.9
2.45	1066.2	533.1
2.70	1103.7	551.8
2.95	1140.8	570.4
3.20	1169.1	584.5
3.45	1191.5	595.7
3.70	1216.6	608.3
3.95	1233.0	616.5
4.20	1252.2	626.1
4.45	1271.2	635.6
4.70	1281.5	640.7
4.95	1277.6	638.8
5.45	1289.6	644.8
5.95	1301.3	650.6
6.45	1312.9	656.4
6.95	1324.2	662.1
7.45	1329.8	664.9
7.95	1324.3	662.1
8.45	1332.4	666.2
8.95	1340.3	670.1
9.45	1339.9	669.9
9.95	1344.9	672.4
10.45	1352.3	676.1
10.95	1351.6	675.8
11.45	1350.8	675.4
11.95	1349.9	674.9
12.45	1348.9	674.4
12.95	1355.6	677.8
13.45	1362.2	681.1
13.95	1373.7	686.8
14.45	1377.3	688.6
14.95	1360.4	680.2
15.45	1356.3	678.1
15.95	1352.1	676.0
16.45	1357.8	678.9
16.95	1360.9	680.4
17.45	1358.8	679.4
17.95	1361.6	680.8
18.45	1359.4	679.7
18.95	1349.8	674.9
19.45	1347.4	673.7
19.95	1347.4	673.7
19.97	1344.7	672.3



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Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

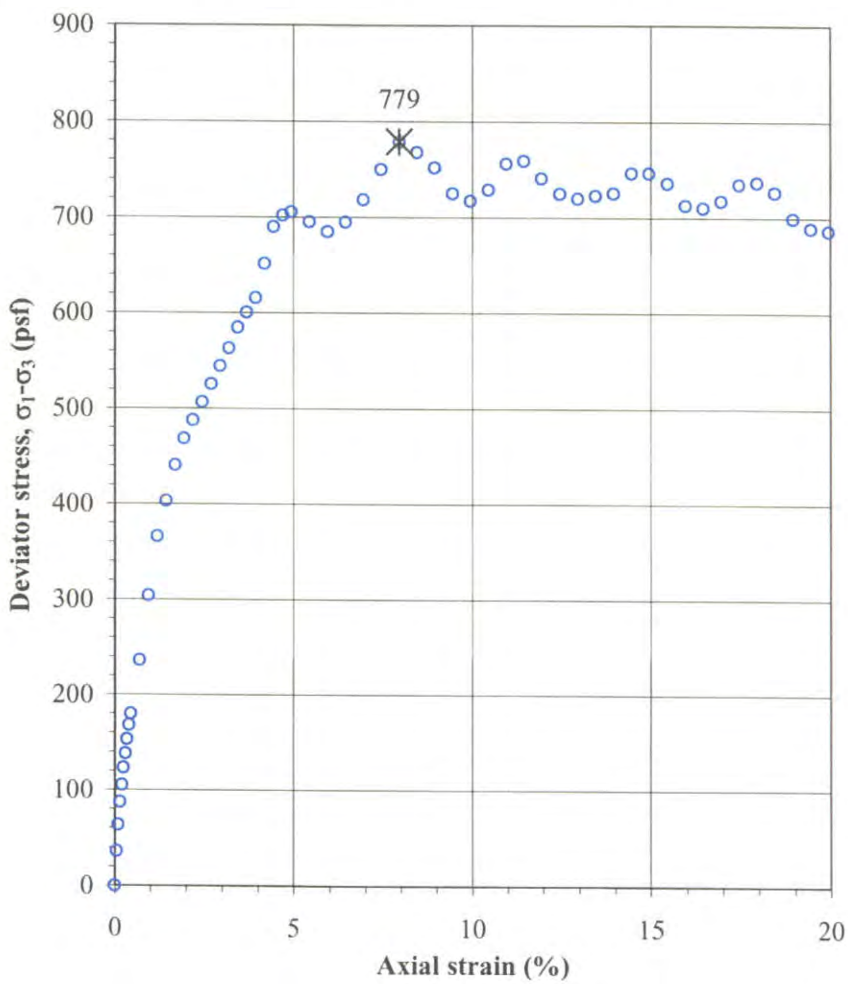
Project: Terracon (61085026)
No: M00385-078
Location: East/ West Connector Lehi
Date: 4/16/2008
By: BRR

Boring No.: B-3
Sample:
Depth: 10
Sample Description: gray clay
Sample type: Undisturbed

Sample height, H (in.)	5.990
Sample diameter, D (in.)	2.820
Sample volume, V (ft^3)	0.0217
Wt. rings + wet soil (g)	1031.70
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1031.70
Moist unit wt., γ_m (pcf)	105.1
Dry unit wt., γ_d (pcf)	**70.4**

Wet soil + tare (g)	350.19
Dry soil + tare (g)	281.32
Tare (g)	141.45
Moisture content, w (%)	**49.2**
Confining stress, σ3 (psf)	706
Shear rate (in/min)	0.0180
Strain at failure, ε_f (%)	7.95
Deviator stress at failure, $(\sigma1-\sigma3)_f$ (psf)	779
Shear stress at failure, $q_f = (\sigma1_f-\sigma3_f)/2$ (psf)	390

Axial Strain (%)	σd σ1-σ3 (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	36.2	18.1
0.10	63.3	31.7
0.15	87.4	43.7
0.20	105.4	52.7
0.25	123.4	61.7
0.30	138.4	69.2
0.35	153.3	76.6
0.40	168.2	84.1
0.45	180.1	90.0
0.70	236.4	118.2
0.95	304.3	152.2
1.20	366.0	183.0
1.45	403.4	201.7
1.70	440.7	220.4
1.95	468.9	234.4
2.20	488.0	244.0
2.45	507.0	253.5
2.70	526.0	263.0
2.95	544.8	272.4
3.20	563.5	281.8
3.45	585.0	292.5
3.70	600.6	300.3
3.95	616.1	308.1
4.20	651.8	325.9
4.45	690.2	345.1
4.70	702.4	351.2
4.95	705.9	353.0
5.45	695.7	347.9
5.95	685.5	342.8
6.45	695.2	347.6
6.95	718.9	359.4
7.45	750.6	375.3
7.95	779.2	389.6
8.45	768.6	384.3
8.95	752.5	376.3
9.45	725.6	362.8
9.95	718.1	359.0
10.45	729.5	364.7
10.95	756.9	378.5
11.45	759.9	379.9
11.95	741.4	370.7
12.45	725.7	362.9
12.95	720.7	360.4
13.45	723.6	361.8
13.95	726.3	363.2
14.45	747.1	373.6
14.95	747.0	373.5
15.45	736.6	368.3
15.95	713.6	356.8
16.45	710.9	355.5
16.95	718.4	359.2
17.45	735.6	367.8
17.95	737.7	368.8
18.45	727.3	363.7
18.95	699.9	349.9
19.45	689.7	344.9
19.95	686.9	343.5
20.00	686.4	343.2



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Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

IGES

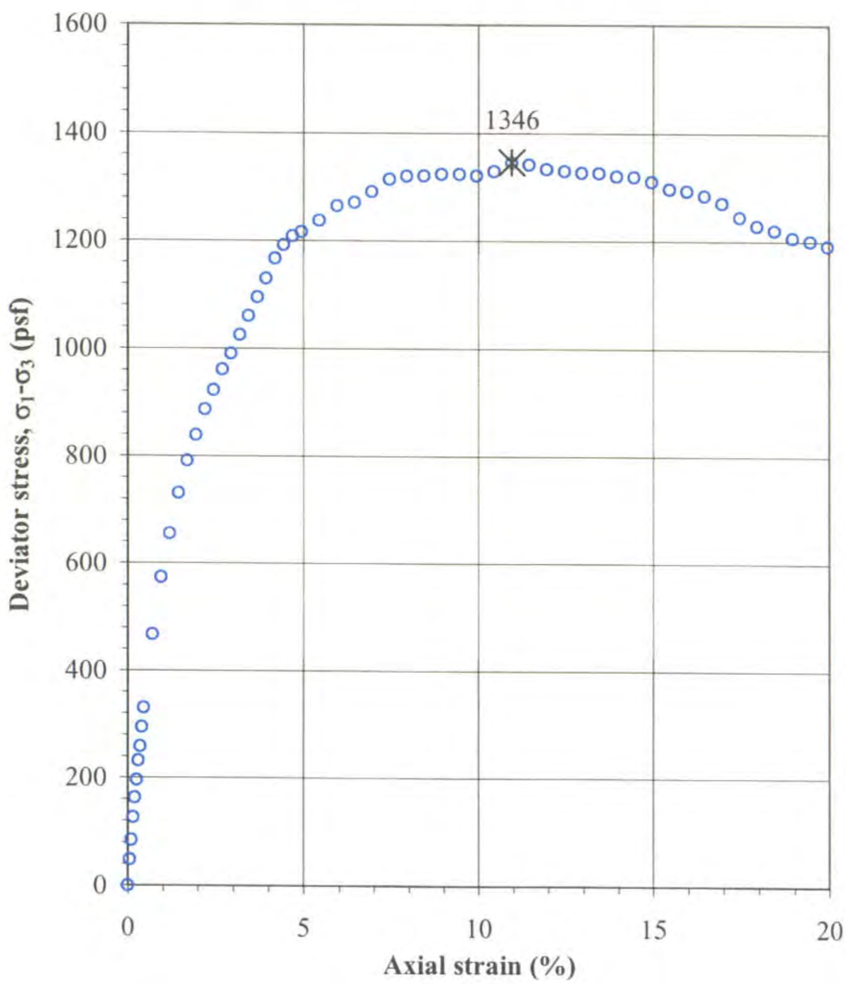
Project: Terracon (61085026)
No: M00385-078
Location: East/ West Connector Lehi
Date: 4/16/2008
By: BRR

Boring No.: B-3
Sample:
Depth: 20
Sample Description: gray silty clay
Sample type: Undisturbed

Sample height, H (in.)	5.810
Sample diameter, D (in.)	2.820
Sample volume, V (ft^3)	0.0210
Wt. rings + wet soil (g)	1131.12
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1131.12
Moist unit wt., γ_m (pcf)	118.7
Dry unit wt., γ_d (pcf)	**87.8**

Wet soil + tare (g)	407.99
Dry soil + tare (g)	341.93
Tare (g)	154.30
Moisture content, w (%)	**35.2**
Confining stress, σ3 (psf)	1397
Shear rate (in/min)	0.0174
Strain at failure, ε_f (%)	10.95
Deviator stress at failure, $(\sigma1-\sigma3)_f$ (psf)	1346
Shear stress at failure, $q_f = (\sigma1_f-\sigma3_f)/2$ (psf)	673

Axial Strain (%)	σd σ1-σ3 (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	48.3	24.2
0.10	84.5	42.2
0.15	126.7	63.3
0.20	162.8	81.4
0.25	195.8	97.9
0.30	231.9	115.9
0.35	258.8	129.4
0.40	294.8	147.4
0.45	330.7	165.3
0.71	467.6	233.8
0.95	574.0	287.0
1.20	655.9	327.9
1.45	731.4	365.7
1.70	791.6	395.8
1.95	839.6	419.8
2.20	887.4	443.7
2.45	923.1	461.5
2.70	961.5	480.7
2.95	990.9	495.4
3.20	1026.0	513.0
3.45	1060.9	530.4
3.70	1095.7	547.8
3.95	1130.2	565.1
4.20	1167.4	583.7
4.45	1192.9	596.4
4.70	1209.5	604.7
4.95	1217.4	608.7
5.45	1238.7	619.3
5.95	1265.5	632.7
6.45	1272.0	636.0
6.95	1292.5	646.2
7.45	1315.5	657.7
7.95	1321.5	660.7
8.45	1321.7	660.8
8.95	1324.6	662.3
9.45	1324.6	662.3
9.95	1321.9	660.9
10.45	1329.9	664.9
10.95	1345.8	672.9
11.45	1342.7	671.3
11.95	1334.2	667.1
12.45	1331.0	665.5
12.95	1327.8	663.9
13.45	1327.0	663.5
13.95	1321.0	660.5
14.45	1320.2	660.1
14.95	1311.5	655.7
15.45	1297.7	648.8
15.95	1294.1	647.0
16.45	1285.5	642.7
16.95	1271.8	635.9
17.45	1245.6	622.8
17.95	1229.7	614.8
18.45	1221.2	610.6
18.95	1207.8	603.9
19.45	1201.8	600.9
19.95	1193.3	596.6
19.98	1195.2	597.6



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Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

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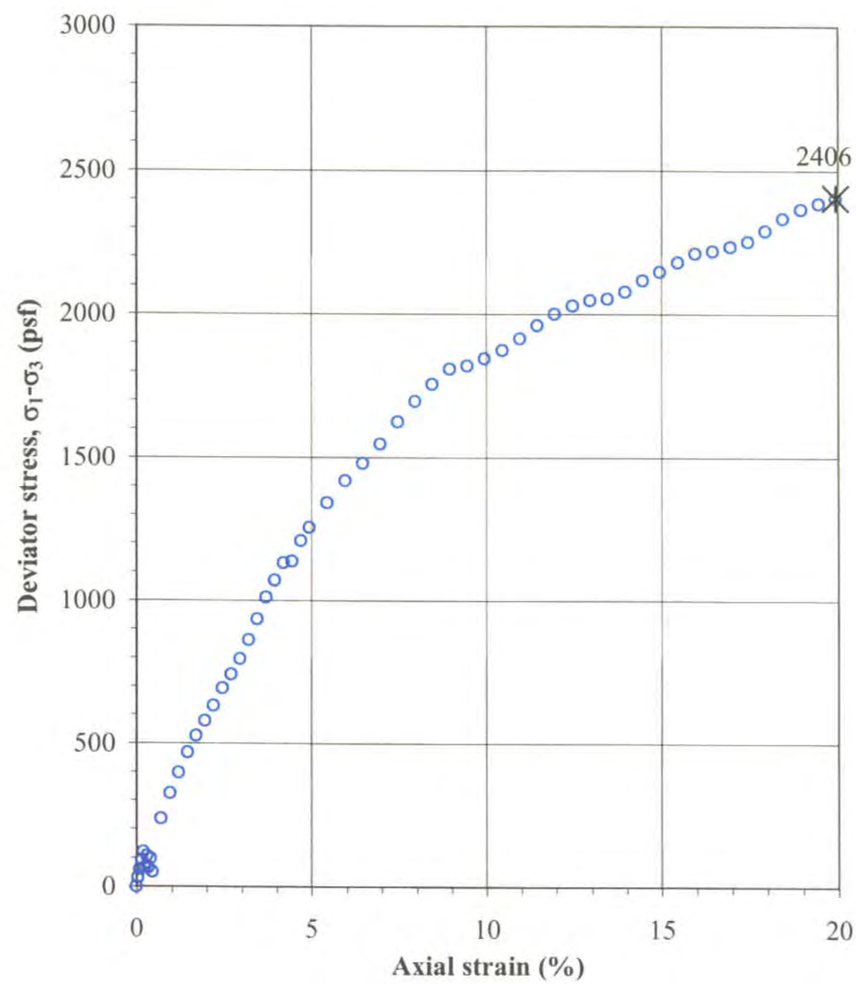
Project: Terracon (61085026)
No: M00385-078
Location: East/ West Connector Lehi
Date: 4/16/2008
By: BRR

Boring No.: B-5
Sample:
Depth: 7.5
Sample Description: gray clay w/ gravelly sand
Sample type: Undisturbed

Sample height, H (in.)	5.970
Sample diameter, D (in.)	2.870
Sample volume, V (ft^3)	0.0224
Wt. rings + wet soil (g)	1397.41
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1397.41
Moist unit wt., γ_m (pcf)	137.8
Dry unit wt., γ_d (pcf)	**119.2**

Wet soil + tare (g)	824.57
Dry soil + tare (g)	732.05
Tare (g)	140.89
Moisture content, w (%)	**15.7**
Confining stress, $\sigma3$ (psf)	605
Shear rate (in/min)	0.0179
Strain at failure, ε_f (%)	19.93
Deviator stress at failure, $(\sigma1-\sigma3)_f$ (psf)	2406
Shear stress at failure, $q_f = (\sigma1_f-\sigma3_f)/2$ (psf)	1203

Axial Strain (%)	σd σ1-σ3 (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	32.1	16.0
0.10	61.2	30.6
0.15	93.2	46.6
0.20	122.3	61.1
0.25	66.8	33.4
0.30	107.5	53.7
0.35	66.6	33.3
0.40	98.5	49.3
0.45	51.9	26.0
0.70	237.2	118.6
0.95	326.1	163.0
1.20	397.2	198.6
1.45	467.9	233.9
1.70	526.8	263.4
1.95	579.6	289.8
2.20	632.2	316.1
2.45	693.0	346.5
2.70	742.1	371.1
2.95	796.7	398.3
3.20	862.2	431.1
3.45	935.9	468.0
3.70	1012.0	506.0
3.95	1070.8	535.4
4.20	1132.2	566.1
4.45	1137.3	568.7
4.70	1209.3	604.7
4.95	1255.8	627.9
5.45	1342.6	671.3
5.95	1420.2	710.1
6.45	1480.4	740.2
6.95	1548.1	774.1
7.45	1625.7	812.9
7.95	1697.1	848.6
8.45	1756.9	878.5
8.95	1810.5	905.3
9.45	1821.2	910.6
9.95	1847.5	923.8
10.45	1875.8	937.9
10.95	1916.9	958.5
11.45	1962.5	981.3
11.95	2002.2	1001.1
12.45	2031.2	1015.6
12.95	2049.6	1024.8
13.45	2054.9	1027.5
13.95	2080.1	1040.1
14.45	2120.0	1060.0
14.95	2149.2	1074.6
15.45	2182.9	1091.5
15.95	2213.6	1106.8
16.45	2221.8	1110.9
16.95	2237.0	1118.5
17.45	2254.3	1127.2
17.95	2292.8	1146.4
18.45	2335.3	1167.7
18.95	2367.8	1183.9
19.45	2387.9	1194.0
19.93	2405.7	1202.9



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Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

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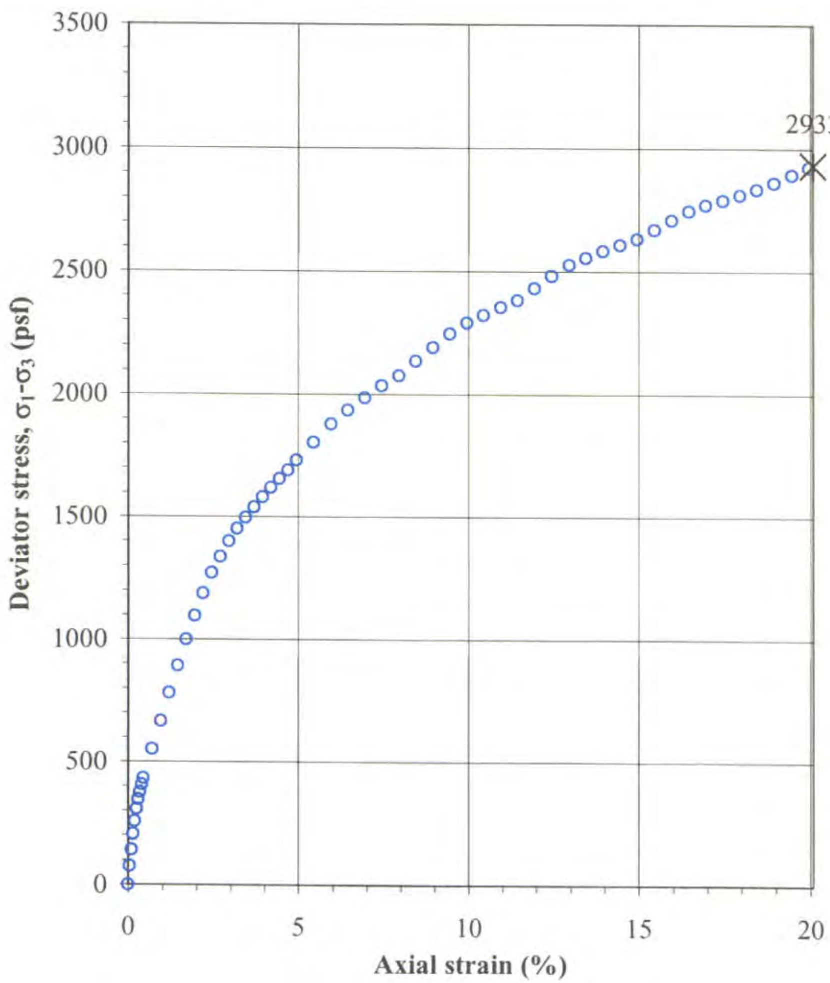
Project: Terracon (61085026)
No: M00385-078
Location: East/ West Connector Lehi
Date: 4/16/2008
By: BRR

Boring No.: B-6
Sample:
Depth: 30
Sample Description: gray clay with silt
Sample type: Undisturbed

Sample height, H (in.)	5.980
Sample diameter, D (in.)	2.840
Sample volume, V (ft^3)	0.0219
Wt. rings + wet soil (g)	1289.88
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1289.88
Moist unit wt., γ_m (pcf)	129.7
Dry unit wt., γ_d (pcf)	**106.4**

Wet soil + tare (g)	818.37
Dry soil + tare (g)	711.17
Tare (g)	221.65
Moisture content, w (%)	**21.9**
Confining stress, σ3 (psf)	1699
Shear rate (in/min)	0.0179
Strain at failure, ε_f (%)	20.08
Deviator stress at failure, $(\sigma1-\sigma3)_f$ (psf)	2933
Shear stress at failure, $q_f = (\sigma1_f-\sigma3_f)/2$ (psf)	1466

Axial Strain (%)	σd σ1-σ3 (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	75.6	37.8
0.10	142.0	71.0
0.15	205.4	102.7
0.20	256.5	128.3
0.25	307.7	153.8
0.30	346.6	173.3
0.35	376.5	188.3
0.40	406.4	203.2
0.45	433.2	216.6
0.70	551.9	275.9
0.95	666.9	333.4
1.20	781.3	390.7
1.45	892.1	446.1
1.70	999.4	499.7
1.95	1097.3	548.7
2.20	1188.6	594.3
2.45	1270.7	635.4
2.70	1337.5	668.8
2.95	1401.1	700.6
3.20	1452.6	726.3
3.45	1497.9	749.0
3.70	1540.1	770.1
3.95	1582.0	791.0
4.20	1620.7	810.4
4.45	1656.4	828.2
4.70	1691.8	845.9
4.95	1732.8	866.4
5.45	1805.4	902.7
5.95	1880.1	940.1
6.45	1936.8	968.4
6.95	1987.1	993.6
7.45	2036.8	1018.4
7.95	2077.3	1038.7
8.45	2136.8	1068.4
8.95	2192.7	1096.4
9.45	2247.9	1124.0
9.95	2291.4	1145.7
10.45	2323.4	1161.7
10.95	2354.9	1177.5
11.45	2383.2	1191.6
11.95	2432.4	1216.2
12.45	2483.5	1241.8
12.95	2528.5	1264.3
13.45	2557.1	1278.6
13.95	2585.2	1292.6
14.45	2610.2	1305.1
14.95	2634.7	1317.4
15.45	2671.5	1335.8
15.95	2710.2	1355.1
16.45	2748.3	1374.2
16.95	2773.2	1386.6
17.45	2792.5	1396.3
17.95	2813.8	1406.9
18.45	2837.2	1418.6
18.95	2864.9	1432.5
19.45	2896.9	1448.5
19.95	2925.9	1463.0
20.08	2932.8	1466.4



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Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

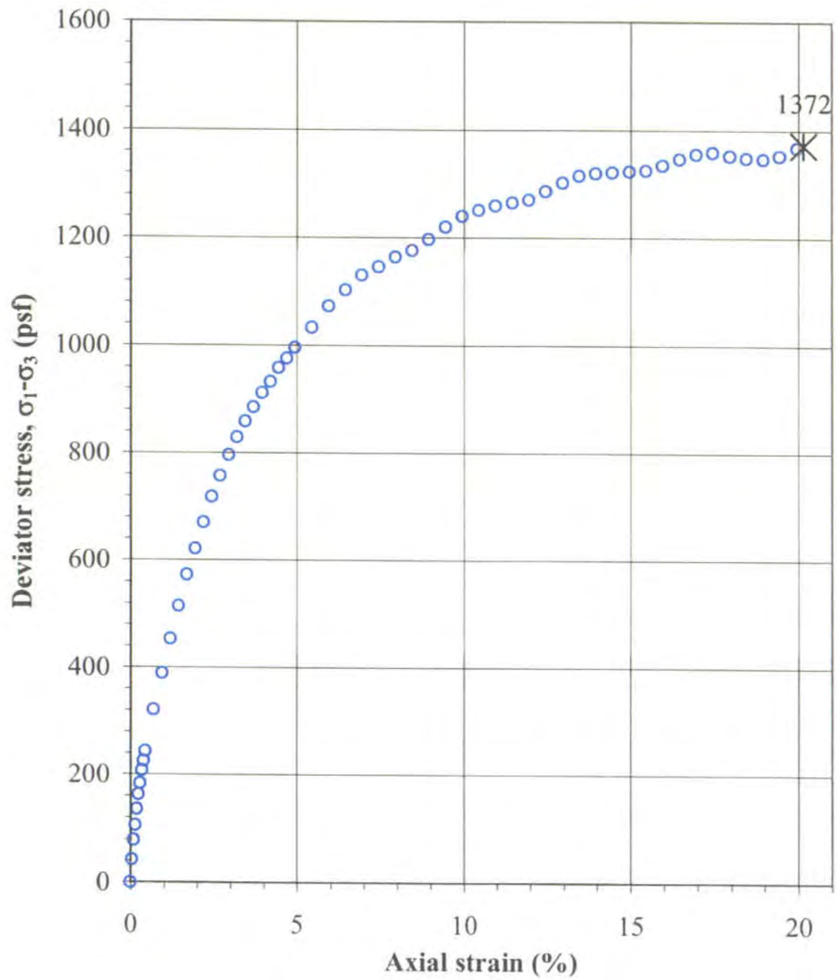
Project: Terracon (61085026)
No: M00385-078
Location: East/ West Connector Lehi
Date: 4/16/2008
By: BRR

Boring No.: B-17
Sample:
Depth: 10
Sample Description: brown clay
Sample type: Undisturbed

Sample height, H (in.)	5.980
Sample diameter, D (in.)	2.840
Sample volume, V (ft^3)	0.0219
Wt. rings + wet soil (g)	1203.11
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1203.11
Moist unit wt., γ_m (pcf)	121.0
Dry unit wt., γ_d (pcf)	**91.8**

Wet soil + tare (g)	369.25
Dry soil + tare (g)	316.98
Tare (g)	152.96
Moisture content, w (%)	**31.9**
Confining stress, $\sigma 3$ (psf)	706
Shear rate (in/min)	0.0179
Strain at failure, ε_f (%)	20.15
Deviator stress at failure, $(\sigma 1-\sigma 3)_f$ (psf)	1372
Shear stress at failure, $q_f = (\sigma 1_f - \sigma 3_f)/2$ (psf)	686

Axial Strain (%)	σd $\sigma 1-\sigma 3$ (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	42.3	21.2
0.10	78.6	39.3
0.15	105.7	52.9
0.20	135.8	67.9
0.25	162.9	81.5
0.30	183.9	92.0
0.35	207.9	104.0
0.40	225.9	112.9
0.45	243.8	121.9
0.70	321.1	160.6
0.95	389.0	194.5
1.20	453.6	226.8
1.45	514.9	257.4
1.70	572.8	286.4
1.95	621.6	310.8
2.20	670.1	335.0
2.45	718.3	359.1
2.70	757.4	378.7
2.95	796.3	398.2
3.20	829.2	414.6
3.45	858.9	429.5
3.70	885.5	442.8
3.95	912.0	456.0
4.20	932.6	466.3
4.45	958.8	479.4
4.70	976.2	488.1
4.95	996.3	498.2
5.45	1033.5	516.8
5.95	1073.0	536.5
6.45	1103.5	551.8
6.95	1130.8	565.4
7.45	1146.5	573.3
7.95	1164.8	582.4
8.45	1177.2	588.6
8.95	1197.7	598.9
9.45	1220.7	610.4
9.95	1240.6	620.3
10.45	1252.0	626.0
10.95	1260.6	630.3
11.45	1266.2	633.1
11.95	1271.7	635.9
12.45	1287.7	643.9
12.95	1303.4	651.7
13.45	1316.3	658.2
13.95	1321.0	660.5
14.45	1323.0	661.5
14.95	1324.9	662.5
15.45	1326.7	663.4
15.95	1335.9	668.0
16.45	1347.5	673.8
16.95	1356.4	678.2
17.45	1360.0	680.0
17.95	1353.6	676.8
18.45	1349.5	674.8
18.95	1347.9	674.0
19.45	1353.5	676.8
19.95	1368.6	684.3
20.15	1372.1	686.1



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Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

IGES

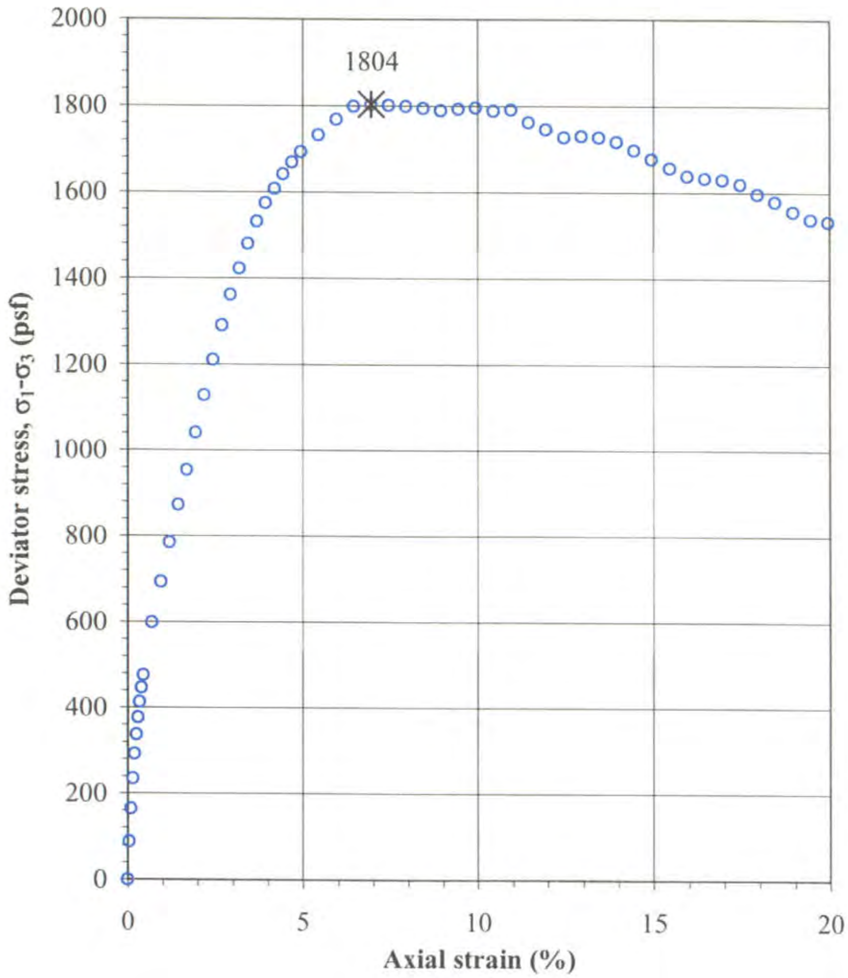
Project: Terracon (61085026)
No: M00385-078
Location: East/ West Connector Lehi
Date: 4/16/2008
By: BRR

Boring No.: B-17
Sample:
Depth: 30
Sample Description: gray clay with silt
Sample type: Undisturbed

Sample height, H (in.)	5.990
Sample diameter, D (in.)	2.830
Sample volume, V (ft^3)	0.0218
Wt. rings + wet soil (g)	1064.24
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1064.24
Moist unit wt., γ_m (pcf)	107.6
Dry unit wt., γ_d (pcf)	**71.0**

Wet soil + tare (g)	772.92
Dry soil + tare (g)	587.07
Tare (g)	226.30
Moisture content, w (%)	**51.5**
Confining stress, σ3 (psf)	1699
Shear rate (in/min)	0.0180
Strain at failure, ε_f (%)	6.95
Deviator stress at failure, $(\sigma 1-\sigma 3)_f$ (psf)	1804
Shear stress at failure, $q_f = (\sigma 1_f-\sigma 3_f)/2$ (psf)	902

Axial Strain (%)	σd σ1-σ3 (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	88.4	44.2
0.10	164.5	82.3
0.15	234.4	117.2
0.20	292.1	146.1
0.25	337.6	168.8
0.30	376.9	188.5
0.35	413.2	206.6
0.40	446.3	223.2
0.45	476.4	238.2
0.70	599.2	299.6
0.95	694.1	347.1
1.20	785.5	392.8
1.45	873.5	436.7
1.70	955.0	477.5
1.95	1042.1	521.0
2.20	1128.7	564.3
2.45	1211.8	605.9
2.70	1291.6	645.8
2.95	1362.0	681.0
3.20	1423.2	711.6
3.45	1481.1	740.5
3.70	1532.8	766.4
3.95	1575.4	787.7
4.20	1609.1	804.5
4.45	1642.5	821.2
4.70	1669.9	834.9
4.95	1694.2	847.1
5.45	1733.8	866.9
5.95	1769.9	884.9
6.45	1799.9	899.9
6.95	1803.9	901.9
7.45	1802.1	901.0
7.95	1800.2	900.1
8.45	1795.4	897.7
8.95	1790.6	895.3
9.45	1793.9	896.9
9.95	1797.2	898.6
10.45	1789.3	894.6
10.95	1792.3	896.1
11.45	1762.7	881.3
11.95	1746.7	873.3
12.45	1728.2	864.1
12.95	1731.0	865.5
13.45	1728.3	864.1
13.95	1717.7	858.8
14.45	1699.2	849.6
14.95	1678.3	839.1
15.45	1657.4	828.7
15.95	1639.3	819.6
16.45	1634.0	817.0
16.95	1631.2	815.6
17.45	1620.7	810.3
17.95	1597.7	798.8
18.45	1579.8	789.9
18.95	1557.1	778.5
19.45	1539.4	769.7
19.95	1534.1	767.0
20.20	1531.4	765.7



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Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

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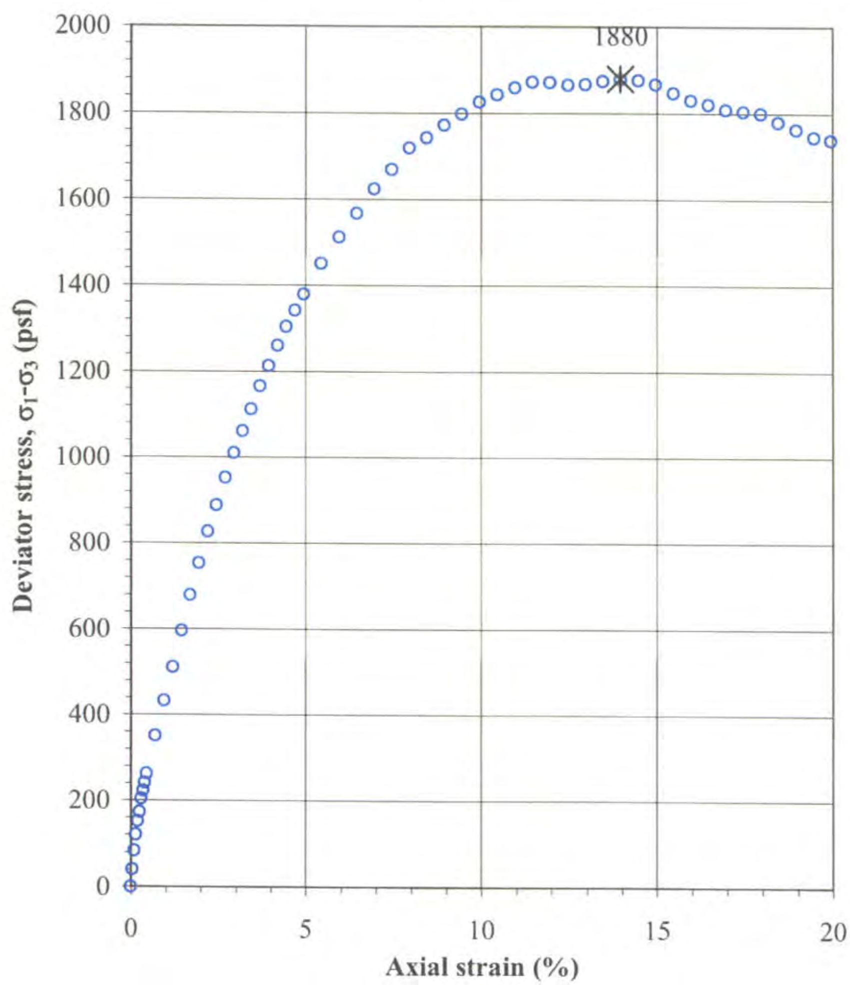
Project: Terracon (61075055)
No: M00385-079
Location: EW Connector
Date: 4/16/2008
By: BRR

Boring No.: B-8
Sample:
Depth: 10
Sample Description: gray silty clay
Sample type: Undisturbed

Sample height, H (in.)	6.000
Sample diameter, D (in.)	2.800
Sample volume, V (ft^3)	0.0214
Wt. rings + wet soil (g)	1109.26
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1109.26
Moist unit wt., γ_m (pcf)	114.4
Dry unit wt., γ_d (pcf)	**83.6**

Wet soil + tare (g)	423.55
Dry soil + tare (g)	358.01
Tare (g)	179.70
Moisture content, w (%)	**36.8**
Confining stress, σ3 (psf)	706
Shear rate (in/min)	0.0180
Strain at failure, ε_f (%)	13.95
Deviator stress at failure, $(\sigma1\text{-}\sigma3)_f$ (psf)	1880
Shear stress at failure, $q_f = (\sigma1_f\text{-}\sigma3_f)/2$ (psf)	940

Axial Strain (%)	σd σ1-σ3 (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	40.4	20.2
0.10	83.9	41.9
0.15	121.1	60.5
0.20	152.0	76.0
0.25	173.6	86.8
0.30	204.5	102.3
0.35	223.0	111.5
0.40	241.4	120.7
0.45	262.9	131.4
0.70	351.5	175.7
0.95	433.5	216.7
1.20	512.0	256.0
1.45	596.2	298.1
1.70	680.0	340.0
1.95	754.1	377.1
2.20	827.9	414.0
2.45	889.1	444.6
2.70	953.1	476.6
2.95	1010.6	505.3
3.20	1061.8	530.9
3.45	1112.8	556.4
3.70	1166.4	583.2
3.95	1213.8	606.9
4.20	1260.9	630.5
4.45	1304.7	652.4
4.70	1342.4	671.2
4.95	1379.8	689.9
5.45	1451.1	725.6
5.95	1512.7	756.4
6.45	1567.8	783.9
6.95	1625.1	812.6
7.45	1670.1	835.1
7.95	1720.3	860.2
8.45	1744.1	872.1
8.95	1773.2	886.6
9.45	1799.1	899.6
9.95	1827.4	913.7
10.45	1844.1	922.1
10.95	1860.5	930.3
11.45	1873.8	936.9
11.95	1873.0	936.5
12.45	1866.8	933.4
12.95	1868.5	934.3
13.45	1875.6	937.8
13.95	1879.7	939.9
14.45	1878.3	939.2
14.95	1868.8	934.4
15.45	1848.7	924.4
15.95	1831.4	915.7
16.45	1822.0	911.0
16.95	1809.9	905.0
17.45	1805.5	902.8
17.95	1801.1	900.6
18.45	1781.2	890.6
18.95	1764.1	882.1
19.45	1747.0	873.5
19.95	1739.9	870.0
20.02	1738.2	869.1



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<u>**Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils**</u>

(ASTM D2850)

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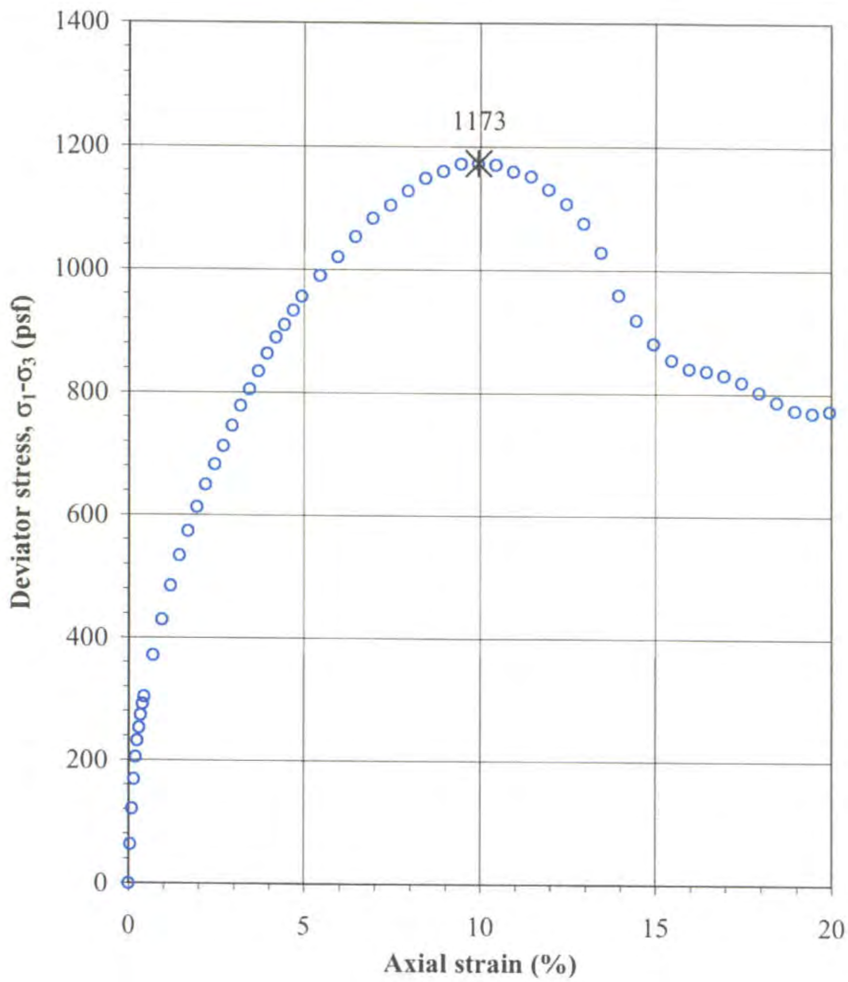
Project: Terracon (61075055)
No: M00385-079
Location: EW Connector
Date: 4/16/2008
By: BRR

Boring No.: B-8
Sample:
Depth: 30
Sample Description: brown clay
Sample type: Undisturbed

Sample height, H (in.)	5.980
Sample diameter, D (in.)	2.840
Sample volume, V (ft^3)	0.0219
Wt. rings + wet soil (g)	1256.48
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1256.48
Moist unit wt., γ_m (pcf)	126.4
Dry unit wt., γ_d (pcf)	**99.5**

Wet soil + tare (g)	445.16
Dry soil + tare (g)	395.31
Tare (g)	210.28
Moisture content, w (%)	**26.9**
Confining stress, $\sigma 3$ (psf)	1699
Shear rate (in/min)	0.0179
Strain at failure, ε_f (%)	9.95
Deviator stress at failure, $(\sigma 1\text{-}\sigma 3)_f$ (psf)	1173
Shear stress at failure, $q_f = (\sigma 1_f\text{-}\sigma 3_f)/2$ (psf)	587

Axial Strain (%)	σd σ1-σ3 (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	63.5	31.7
0.10	120.9	60.4
0.15	169.1	84.5
0.20	205.2	102.6
0.25	232.2	116.1
0.30	253.1	126.5
0.35	274.0	137.0
0.40	291.8	145.9
0.45	303.7	151.8
0.70	371.5	185.8
0.95	430.1	215.0
1.20	485.3	242.7
1.45	534.3	267.1
1.70	574.1	287.0
1.95	613.6	306.8
2.20	650.0	325.0
2.45	683.3	341.6
2.70	713.4	356.7
2.95	746.3	373.1
3.20	779.0	389.5
3.45	805.6	402.8
3.70	835.1	417.6
3.95	864.3	432.2
4.20	890.5	445.3
4.45	910.8	455.4
4.70	933.8	466.9
4.95	956.7	478.4
5.45	990.6	495.3
5.95	1021.3	510.7
6.45	1054.3	527.2
6.95	1084.2	542.1
7.45	1105.2	552.6
7.95	1128.7	564.4
8.45	1149.1	574.6
8.95	1160.9	580.5
9.45	1172.6	586.3
9.95	1173.1	586.6
10.45	1170.8	585.4
10.95	1160.3	580.2
11.45	1152.5	576.3
11.95	1131.4	565.7
12.45	1107.8	553.9
12.95	1076.5	538.3
13.45	1029.7	514.9
13.95	959.9	480.0
14.45	919.3	459.7
14.95	881.8	440.9
15.45	854.8	427.4
15.95	840.8	420.4
16.45	837.0	418.5
16.95	830.7	415.4
17.45	819.3	409.7
17.95	803.1	401.6
18.45	787.0	393.5
18.95	773.4	386.7
19.45	769.7	384.9
19.95	773.3	386.6
20.17	770.5	385.2



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Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

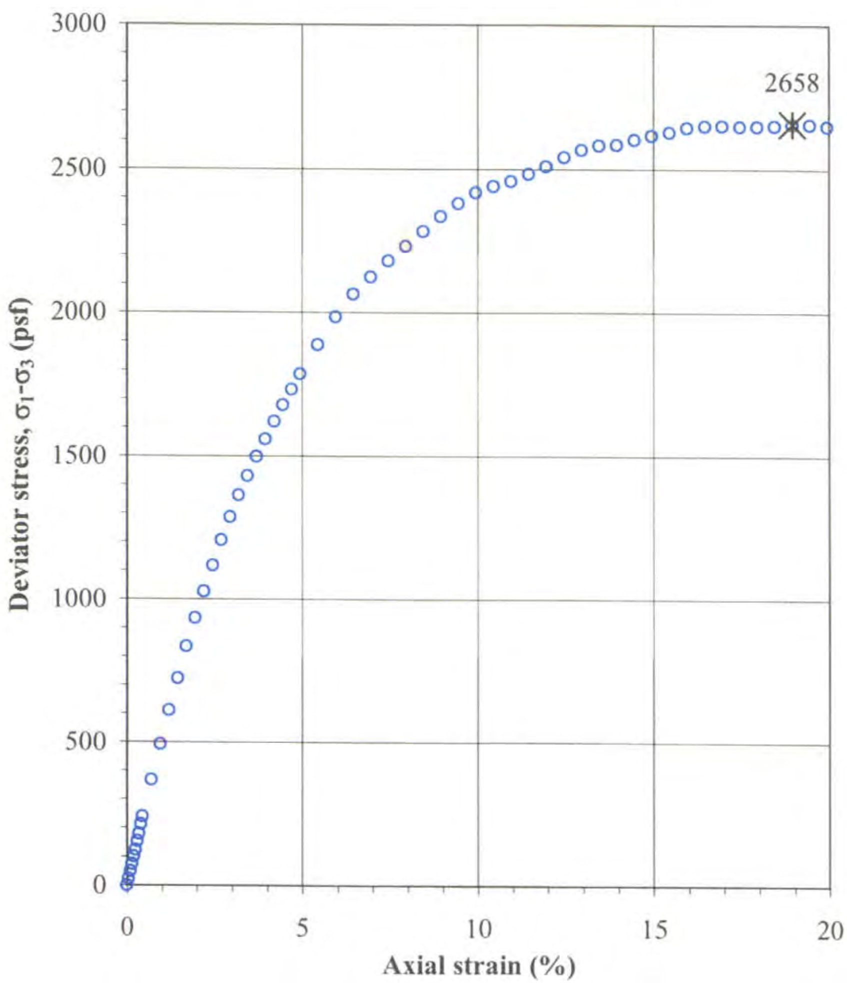
Project: Terracon (61075055)
No: M00385-079
Location: EW Connector
Date: 4/16/2008
By: BRR

Boring No.: B-8
Sample:
Depth: 50
Sample Description: tan clay
Sample type: Undisturbed

Sample height, H (in.)	5.990
Sample diameter, D (in.)	2.870
Sample volume, V (ft^3)	0.0224
Wt. rings + wet soil (g)	1260.63
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1260.63
Moist unit wt., γ_m (pcf)	123.9
Dry unit wt., γ_d (pcf)	**98.6**

Wet soil + tare (g)	424.23
Dry soil + tare (g)	380.79
Tare (g)	211.67
Moisture content, w (%)	**25.7**
Confining stress, σ3 (psf)	2779
Shear rate (in/min)	0.0180
Strain at failure, ε_f (%)	18.95
Deviator stress at failure, $(\sigma1-\sigma3)_f$ (psf)	2658
Shear stress at failure, $q_f = (\sigma1_f-\sigma3_f)/2$ (psf)	1329

Axial Strain (%)	σd σ1-σ3 (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	23.7	11.8
0.10	50.3	25.1
0.15	73.9	36.9
0.20	100.4	50.2
0.25	124.0	62.0
0.30	153.5	76.7
0.35	179.9	90.0
0.40	212.3	106.1
0.45	238.7	119.4
0.70	367.4	183.7
0.95	492.6	246.3
1.20	611.2	305.6
1.45	723.4	361.7
1.70	835.0	417.5
1.95	934.4	467.2
2.20	1027.5	513.7
2.45	1117.1	558.5
2.70	1206.4	603.2
2.95	1286.5	643.2
3.20	1363.3	681.6
3.45	1431.0	715.5
3.70	1498.4	749.2
3.95	1559.8	779.9
4.20	1620.8	810.4
4.45	1678.6	839.3
4.70	1733.2	866.6
4.95	1787.6	893.8
5.45	1889.8	944.9
5.95	1985.2	992.6
6.45	2065.6	1032.8
6.95	2125.7	1062.8
7.45	2182.2	1091.1
7.95	2232.6	1116.3
8.45	2285.0	1142.5
8.95	2336.7	1168.3
9.45	2382.3	1191.1
9.95	2419.3	1209.6
10.45	2442.3	1221.1
10.95	2459.7	1229.8
11.45	2484.6	1242.3
11.95	2511.7	1255.8
12.45	2543.5	1271.7
12.95	2569.7	1284.8
13.45	2585.0	1292.5
13.95	2587.2	1293.6
14.45	2604.5	1302.2
14.95	2618.8	1309.4
15.45	2630.3	1315.1
15.95	2646.5	1323.2
16.45	2652.3	1326.1
16.95	2653.0	1326.5
17.45	2651.0	1325.5
17.95	2651.2	1325.6
18.45	2653.7	1326.8
18.95	2658.3	1329.1
19.45	2657.9	1328.9
19.95	2652.5	1326.2
20.11	2651.7	1325.8



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Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

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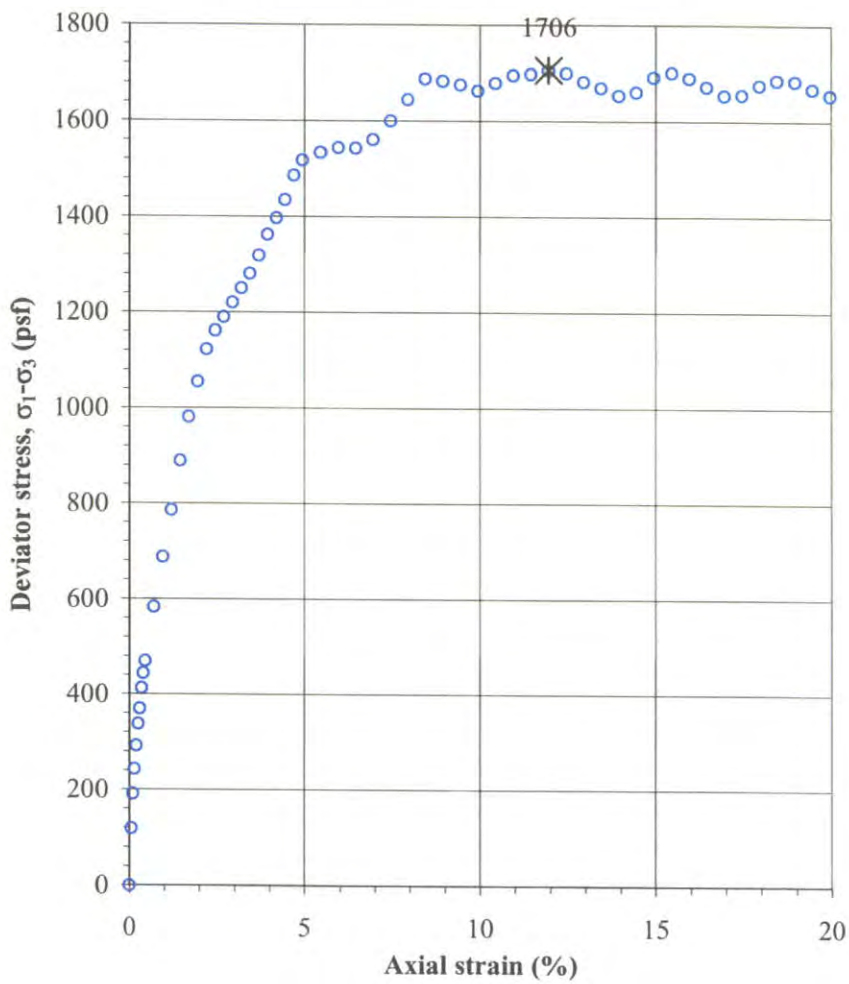
Project: Terracon
No: M00385-080 (61085026)
Location: E.W. Connector
Date: 5/5/2008
By: BRR

Boring No.: B-9
Sample:
Depth: 5
Sample Description: dark brown clay w/ sand & gravel
Sample type: Undisturbed

Sample height, H (in.)	5.980
Sample diameter, D (in.)	2.880
Sample volume, V (ft^3)	0.0225
Wt. rings + wet soil (g)	1313.88
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1313.88
Moist unit wt., γ_m (pcf)	128.5
Dry unit wt., γ_d (pcf)	**107.1**

Wet soil + tare (g)	856.83
Dry soil + tare (g)	739.35
Tare (g)	150.62
Moisture content, w (%)	**20.0**
Confining stress, σ3 (psf)	605
Shear rate (in/min)	0.0179
Strain at failure, ε_f (%)	11.95
Deviator stress at failure, $(\sigma1-\sigma3)_f$ (psf)	1706
Shear stress at failure, $q_f = (\sigma1_f-\sigma3_f)/2$ (psf)	853

Axial Strain (%)	σd σ1-σ3 (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	118.8	59.4
0.10	191.2	95.6
0.15	243.2	121.6
0.20	292.2	146.1
0.25	338.3	169.2
0.30	369.9	185.0
0.35	413.0	206.5
0.40	444.6	222.3
0.45	470.3	235.1
0.70	584.0	292.0
0.95	688.6	344.3
1.20	786.8	393.4
1.45	890.3	445.2
1.70	981.8	490.9
1.95	1055.8	527.9
2.20	1123.7	561.9
2.45	1163.0	581.5
2.70	1190.7	595.4
2.95	1221.1	610.6
3.20	1251.4	625.7
3.45	1281.4	640.7
3.70	1319.7	659.9
3.95	1363.3	681.7
4.20	1398.3	699.2
4.45	1435.9	718.0
4.70	1487.1	743.6
4.95	1518.7	759.4
5.45	1534.8	767.4
5.95	1545.1	772.6
6.45	1544.3	772.2
6.95	1562.3	781.2
7.45	1601.6	800.8
7.95	1645.6	822.8
8.45	1689.1	844.6
8.95	1684.5	842.3
9.45	1677.1	838.6
9.95	1664.5	832.3
10.45	1680.5	840.3
10.95	1696.2	848.1
11.45	1698.8	849.4
11.95	1706.4	853.2
12.45	1701.1	850.6
12.95	1683.0	841.5
13.45	1670.1	835.1
13.95	1654.7	827.4
14.45	1661.7	830.9
14.95	1693.2	846.6
15.45	1702.1	851.1
15.95	1691.2	845.6
16.45	1673.1	836.6
16.95	1655.1	827.6
17.45	1656.3	828.2
17.95	1676.5	838.3
18.45	1686.8	843.4
18.95	1685.0	842.5
19.45	1669.1	834.6
19.95	1655.6	827.8
20.00	1652.3	826.2



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Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

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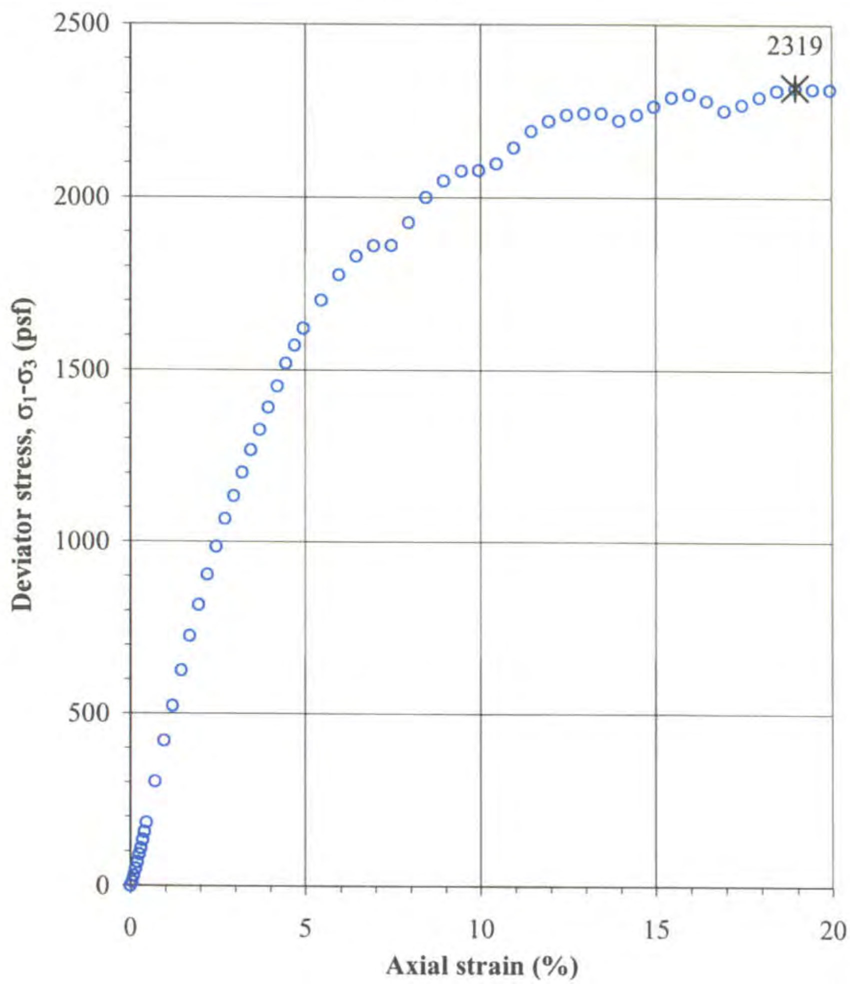
Project: Terracon
No: M00385-080 (61085026)
Location: E.W. Connector
Date: 5/5/2008
By: BRR

Boring No.: B-9
Sample:
Depth: 15
Sample Description:
Sample type: Undisturbed

Sample height, H (in.)	5.990
Sample diameter, D (in.)	2.850
Sample volume, V (ft^3)	0.0221
Wt. rings + wet soil (g)	1236.21
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1236.21
Moist unit wt., γ_m (pcf)	123.2
Dry unit wt., γ_d (pcf)	**95.5**

Wet soil + tare (g)	769.19
Dry soil + tare (g)	627.90
Tare (g)	140.59
Moisture content, w (%)	**29.0**
Confining stress, $\sigma3$ (psf)	1699
Shear rate (in/min)	0.0180
Strain at failure, ε_f (%)	18.95
Deviator stress at failure, $(\sigma1-\sigma3)_f$ (psf)	2319
Shear stress at failure, $q_f = (\sigma1_f-\sigma3_f)/2$ (psf)	1160

Axial Strain (%)	σd $\sigma1-\sigma3$ (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	11.8	5.9
0.10	29.5	14.8
0.15	50.2	25.1
0.20	67.9	33.9
0.25	91.5	45.7
0.30	109.1	54.5
0.35	132.6	66.3
0.40	156.1	78.1
0.45	182.6	91.3
0.70	302.6	151.3
0.95	422.0	211.0
1.20	523.2	261.6
1.45	626.8	313.4
1.70	727.0	363.5
1.95	818.0	409.0
2.20	905.5	452.8
2.45	986.8	493.4
2.70	1067.7	533.9
2.95	1133.8	566.9
3.20	1202.5	601.3
3.45	1267.8	633.9
3.70	1327.1	663.6
3.95	1391.8	695.9
4.20	1453.3	726.7
4.45	1520.1	760.1
4.70	1572.4	786.2
4.95	1621.6	810.8
5.45	1702.4	851.2
5.95	1776.7	888.4
6.45	1830.6	915.3
6.95	1861.8	930.9
7.45	1862.4	931.2
7.95	1928.3	964.2
8.45	2001.6	1000.8
8.95	2049.7	1024.9
9.45	2078.2	1039.1
9.95	2079.7	1039.9
10.45	2099.6	1049.8
10.95	2145.6	1072.8
11.45	2193.5	1096.8
11.95	2222.5	1111.3
12.45	2240.6	1120.3
12.95	2245.5	1122.8
13.45	2245.0	1122.5
13.95	2224.0	1112.0
14.45	2241.1	1120.6
14.95	2265.4	1132.7
15.45	2291.8	1145.9
15.95	2300.2	1150.1
16.45	2281.2	1140.6
16.95	2252.4	1126.2
17.45	2270.2	1135.1
17.95	2292.5	1146.3
18.45	2311.9	1156.0
18.95	2319.0	1159.5
19.45	2316.2	1158.1
19.95	2315.6	1157.8
19.97	2310.4	1155.2



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Reviewed: [illegible]

Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

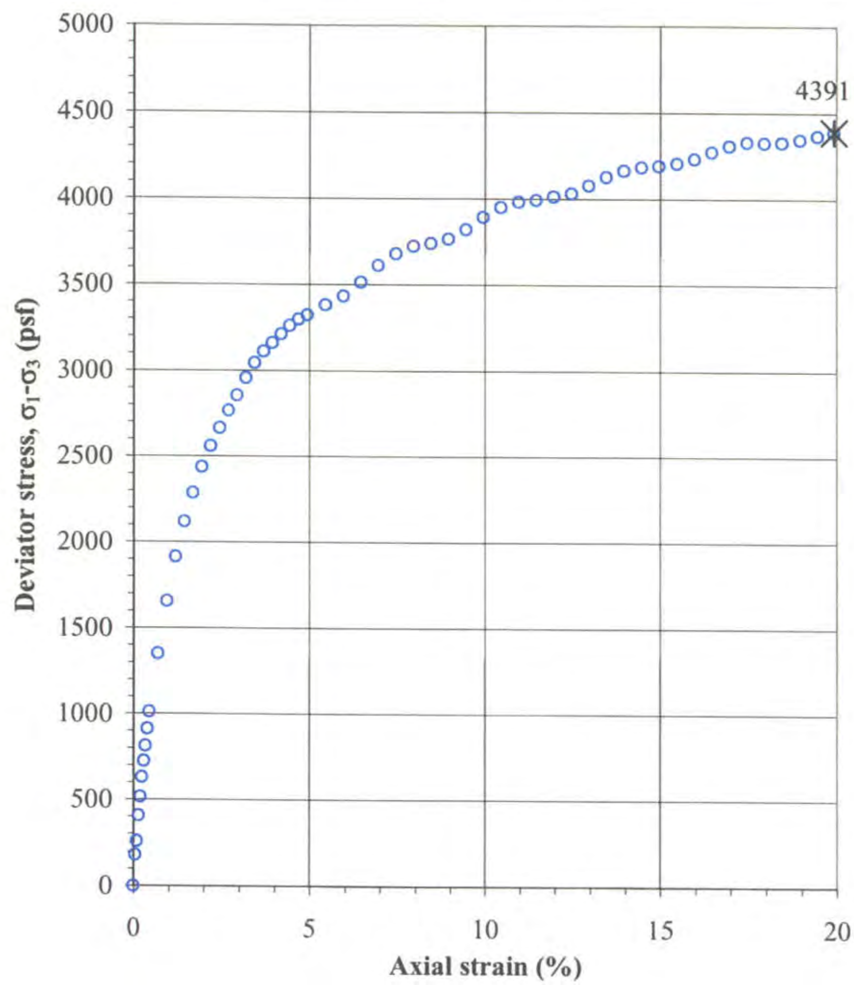
Project: Terracon
No: M00385-080 (61085026)
Location: E.W. Connector
Date: 5/2/2008
By: BRR

Boring No.: B-11
Sample:
Depth: 15
Sample Description: brown clay
Sample type: Undisturbed

Sample height, H (in.)	6.000
Sample diameter, D (in.)	2.850
Sample volume, V (ft^3)	0.0222
Wt. rings + wet soil (g)	1262.82
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1262.82
Moist unit wt., γ_m (pcf)	125.7
Dry unit wt., γ_d (pcf)	**99.8**

Wet soil + tare (g)	703.04
Dry soil + tare (g)	587.27
Tare (g)	140.87
Moisture content, w (%)	**25.9**
Confining stress, σ3 (psf)	1699
Shear rate (in/min)	0.0180
Strain at failure, ε_f (%)	19.92
Deviator stress at failure, $(\sigma 1-\sigma 3)_f$ (psf)	4391
Shear stress at failure, $q_f = (\sigma 1_f - \sigma 3_f)/2$ (psf)	2196

Axial Strain (%)	σd σ1-σ3 (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	180.6	90.3
0.10	260.3	130.2
0.15	408.1	204.0
0.20	517.2	258.6
0.25	632.1	316.1
0.30	726.3	363.1
0.35	814.4	407.2
0.40	914.3	457.1
0.45	1014.0	507.0
0.70	1352.4	676.2
0.95	1659.7	829.8
1.20	1915.6	957.8
1.45	2120.6	1060.3
1.70	2289.6	1144.8
1.95	2440.3	1220.1
2.20	2561.1	1280.5
2.45	2666.9	1333.4
2.70	2769.2	1384.6
2.95	2856.6	1428.3
3.20	2957.8	1478.9
3.45	3047.1	1523.5
3.70	3112.9	1556.4
3.95	3164.2	1582.1
4.20	3215.2	1607.6
4.45	3263.0	1631.5
4.70	3299.2	1649.6
4.95	3323.9	1661.9
5.45	3384.0	1692.0
5.95	3435.0	1717.5
6.45	3515.6	1757.8
6.95	3614.6	1807.3
7.45	3682.0	1841.0
7.95	3726.7	1863.3
8.45	3743.6	1871.8
8.95	3768.2	1884.1
9.45	3824.4	1912.2
9.95	3895.8	1947.9
10.45	3952.9	1976.4
10.95	3985.2	1992.6
11.45	3996.2	1998.1
11.95	4017.0	2008.5
12.45	4034.9	2017.4
12.95	4080.5	2040.2
13.45	4130.5	2065.2
13.95	4166.9	2083.4
14.45	4187.5	2093.7
14.95	4194.8	2097.4
15.45	4209.2	2104.6
15.95	4235.7	2117.8
16.45	4276.4	2138.2
16.95	4311.3	2155.6
17.45	4333.4	2166.7
17.95	4330.5	2165.2
18.45	4332.1	2166.0
18.95	4347.8	2173.9
19.45	4370.1	2185.0
19.92	4391.3	2195.6



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Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

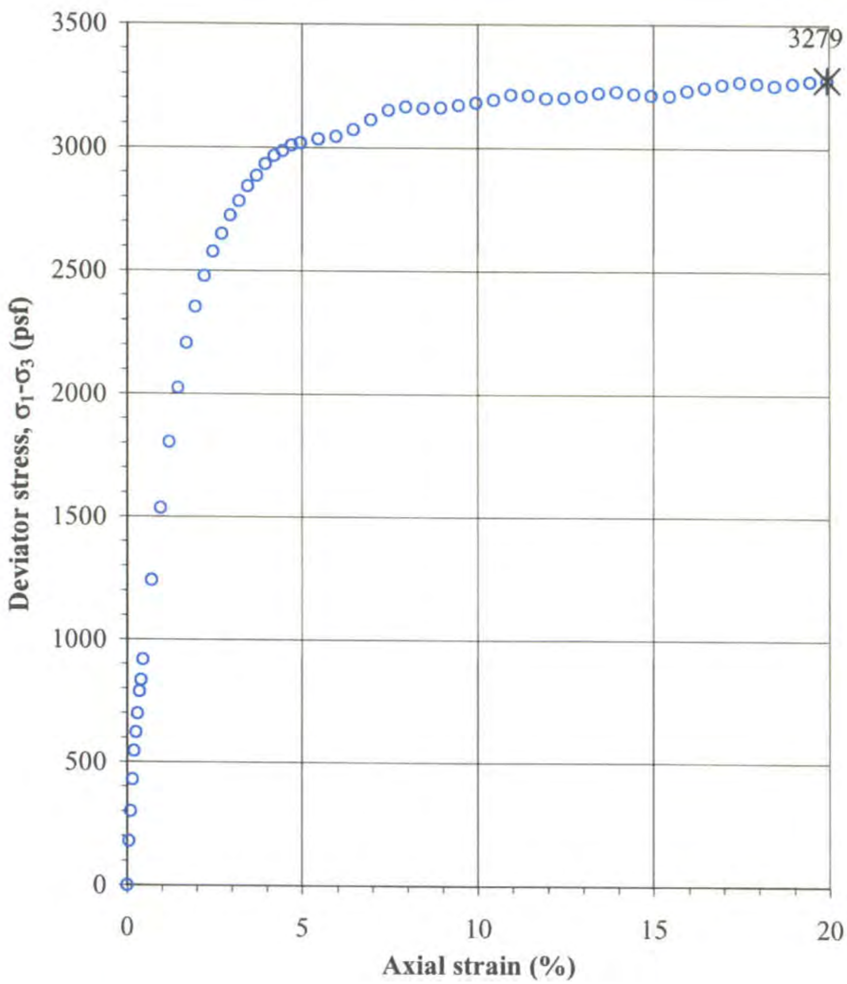
Project: Terracon
No: M00385-080 (61085026)
Location: E.W. Connector
Date: 5/6/2008
By: BRR

Boring No.: B-14
Sample:
Depth: 12.5
Sample Description: brown clay
Sample type: Undisturbed

Sample height, H (in.)	5.980
Sample diameter, D (in.)	2.860
Sample volume, V (ft^3)	0.0222
Wt. rings + wet soil (g)	1241.28
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1241.28
Moist unit wt., γ_m (pcf)	123.1
Dry unit wt., γ_d (pcf)	**96.0**

Wet soil + tare (g)	618.18
Dry soil + tare (g)	512.81
Tare (g)	139.98
Moisture content, w (%)	**28.3**
Confining stress, $\sigma 3$ (psf)	1397
Shear rate (in/min)	0.0179
Strain at failure, ε_f (%)	19.94
Deviator stress at failure, $(\sigma 1-\sigma 3)_f$ (psf)	3279
Shear stress at failure, $q_f = (\sigma 1_f - \sigma 3_f)/2$ (psf)	1640

Axial Strain (%)	σd $\sigma 1-\sigma 3$ (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	179.3	89.6
0.10	299.5	149.8
0.15	428.5	214.2
0.20	545.6	272.8
0.25	621.5	310.7
0.30	697.3	348.6
0.35	787.7	393.8
0.40	834.1	417.0
0.45	918.4	459.2
0.70	1242.6	621.3
0.95	1536.0	768.0
1.20	1804.6	902.3
1.45	2025.4	1012.7
1.70	2207.5	1103.7
1.95	2354.0	1177.0
2.20	2479.5	1239.7
2.45	2578.6	1289.3
2.70	2651.4	1325.7
2.95	2726.6	1363.3
3.20	2784.3	1392.1
3.45	2844.5	1422.2
3.70	2887.3	1443.6
3.95	2935.6	1467.8
4.20	2969.4	1484.7
4.45	2989.0	1494.5
4.70	3011.2	1505.6
4.95	3022.0	1511.0
5.45	3037.9	1518.9
5.95	3047.9	1523.9
6.45	3076.8	1538.4
6.95	3116.2	1558.1
7.45	3155.0	1577.5
7.95	3168.8	1584.4
8.45	3163.4	1581.7
8.95	3165.8	1582.9
9.45	3176.0	1588.0
9.95	3185.9	1592.9
10.45	3198.2	1599.1
10.95	3217.9	1608.9
11.45	3216.4	1608.2
11.95	3204.2	1602.1
12.45	3204.9	1602.4
12.95	3213.0	1606.5
13.45	3225.9	1612.9
13.95	3230.8	1615.4
14.45	3222.8	1611.4
14.95	3217.2	1608.6
15.45	3213.9	1606.9
15.95	3235.0	1617.5
16.45	3248.4	1624.2
16.95	3261.3	1630.6
17.45	3271.4	1635.7
17.95	3266.6	1633.3
18.45	3256.8	1628.4
18.95	3266.0	1633.0
19.45	3274.7	1637.3
19.94	3279.1	1639.5



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Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

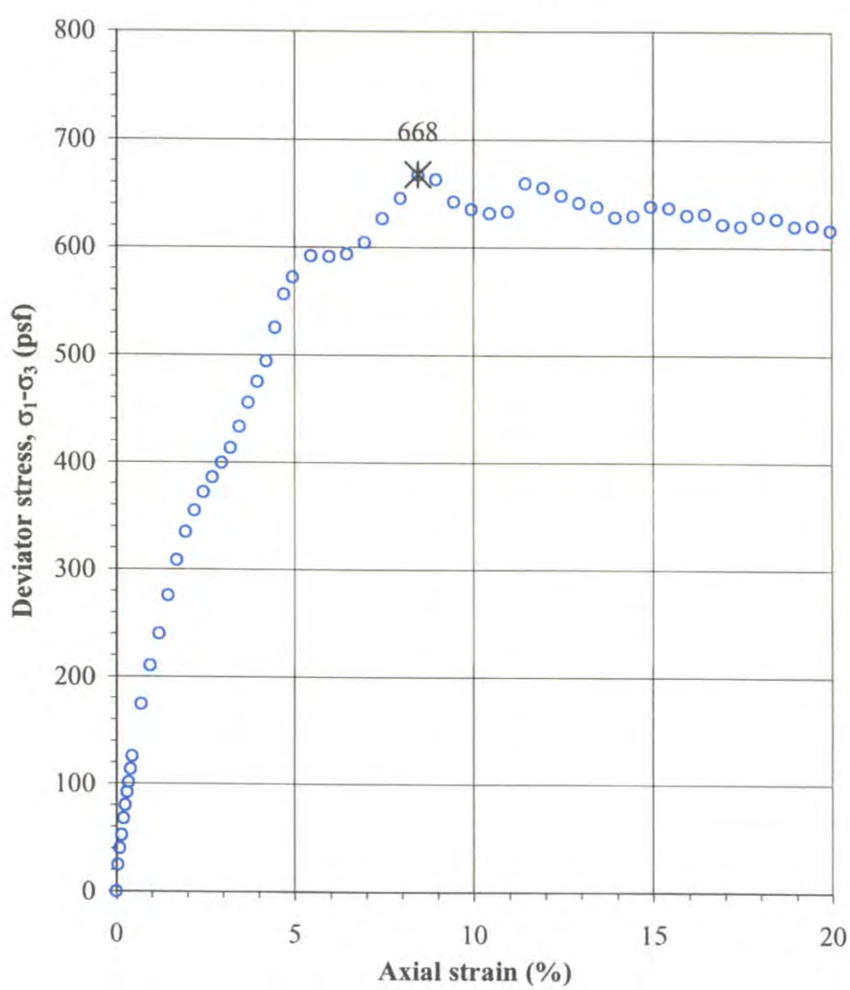
Project: Terracon
No: M00385-080 (61085026)
Location: E.W. Connector
Date: 5/6/2008
By: BRR

Boring No.: B-15
Sample:
Depth: 12.5
Sample Description: gray/ brown clay
Sample type: Undisturbed

Sample height, H (in.)	5.970
Sample diameter, D (in.)	2.790
Sample volume, V (ft^3)	0.0211
Wt. rings + wet soil (g)	1061.25
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1061.25
Moist unit wt., γ_m (pcf)	110.8
Dry unit wt., γ_d (pcf)	**74.9**

Wet soil + tare (g)	717.74
Dry soil + tare (g)	531.03
Tare (g)	140.45
Moisture content, w (%)	**47.8**
Confining stress, σ3 (psf)	1397
Shear rate (in/min)	0.0179
Strain at failure, ε_f (%)	8.45
Deviator stress at failure, $(\sigma1-\sigma3)_f$ (psf)	668
Shear stress at failure, $q_f = (\sigma1_f-\sigma3_f)/2$ (psf)	334

Axial Strain (%)	σd σ1-σ3 (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	24.7	12.3
0.10	40.1	20.0
0.15	52.4	26.2
0.20	67.7	33.9
0.25	80.0	40.0
0.30	92.3	46.1
0.35	101.4	50.7
0.40	113.7	56.8
0.45	125.9	62.9
0.70	174.5	87.3
0.95	210.6	105.3
1.20	240.5	120.2
1.45	276.3	138.1
1.70	308.8	154.4
1.95	335.1	167.6
2.20	355.3	177.6
2.45	372.3	186.1
2.70	386.2	193.1
2.95	400.0	200.0
3.20	413.8	206.9
3.45	433.4	216.7
3.70	455.9	228.0
3.95	475.3	237.7
4.20	494.6	247.3
4.45	525.7	262.8
4.70	556.5	278.3
4.95	572.5	286.3
5.45	592.5	296.2
5.95	591.9	296.0
6.45	594.2	297.1
6.95	605.0	302.5
7.45	627.1	313.6
7.95	646.1	323.1
8.45	667.7	333.8
8.95	663.7	331.8
9.45	642.8	321.4
9.95	636.1	318.0
10.45	632.2	316.1
10.95	633.8	316.9
11.45	659.9	330.0
11.95	655.8	327.9
12.45	648.9	324.5
12.95	642.1	321.1
13.45	638.0	319.0
13.95	628.6	314.3
14.45	629.9	314.9
14.95	638.9	319.4
15.45	637.3	318.7
15.95	630.6	315.3
16.45	631.6	315.8
16.95	622.2	311.1
17.45	620.6	310.3
17.95	629.1	314.5
18.45	627.3	313.7
18.95	620.5	310.3
19.45	621.3	310.6
19.95	616.9	308.5
20.00	616.6	308.3



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Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

IGES

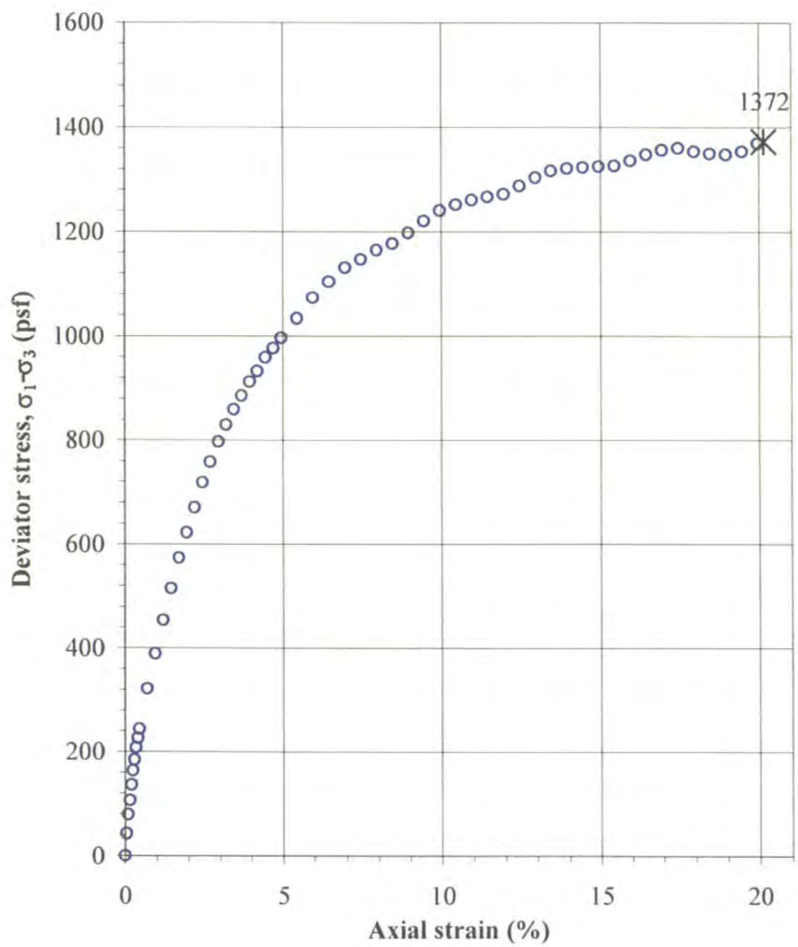
Project: Terracon (61085026)
No: M00385-078
Location: East/ West Connector Lehi
Date: 4/16/2008
By: BRR

Boring No.: B-17
Sample:
Depth: 10
Sample Description: brown clay
Sample type: Undisturbed

Sample height, H (in.)	5.980
Sample diameter, D (in.)	2.840
Sample volume, V (ft^3)	0.0219
Wt. rings + wet soil (g)	1203.11
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1203.11
Moist unit wt., γ_m (pcf)	121.0
Dry unit wt., γ_d (pcf)	**91.8**

Wet soil + tare (g)	369.25
Dry soil + tare (g)	316.98
Tare (g)	152.96
Moisture content, w (%)	**31.9**
Confining stress, σ3 (psf)	706
Shear rate (in/min)	0.0179
Strain at failure, ε_f (%)	20.15
Deviator stress at failure, $(\sigma 1-\sigma 3)_f$ (psf)	1372
Shear stress at failure, $q_f = (\sigma 1_f-\sigma 3_f)/2$ (psf)	686

Axial Strain (%)	σd σ1-σ3 (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	42.3	21.2
0.10	78.6	39.3
0.15	105.7	52.9
0.20	135.8	67.9
0.25	162.9	81.5
0.30	183.9	92.0
0.35	207.9	104.0
0.40	225.9	112.9
0.45	243.8	121.9
0.70	321.1	160.6
0.95	389.0	194.5
1.20	453.6	226.8
1.45	514.9	257.4
1.70	572.8	286.4
1.95	621.6	310.8
2.20	670.1	335.0
2.45	718.3	359.1
2.70	757.4	378.7
2.95	796.3	398.2
3.20	829.2	414.6
3.45	858.9	429.5
3.70	885.5	442.8
3.95	912.0	456.0
4.20	932.6	466.3
4.45	958.8	479.4
4.70	976.2	488.1
4.95	996.3	498.2
5.45	1033.5	516.8
5.95	1073.0	536.5
6.45	1103.5	551.8
6.95	1130.8	565.4
7.45	1146.5	573.3
7.95	1164.8	582.4
8.45	1177.2	588.6
8.95	1197.7	598.9
9.45	1220.7	610.4
9.95	1240.6	620.3
10.45	1252.0	626.0
10.95	1260.6	630.3
11.45	1266.2	633.1
11.95	1271.7	635.9
12.45	1287.7	643.9
12.95	1303.4	651.7
13.45	1316.3	658.2
13.95	1321.0	660.5
14.45	1323.0	661.5
14.95	1324.9	662.5
15.45	1326.7	663.4
15.95	1335.9	668.0
16.45	1347.5	673.8
16.95	1356.4	678.2
17.45	1360.0	680.0
17.95	1353.6	676.8
18.45	1349.5	674.8
18.95	1347.9	674.0
19.45	1353.5	676.8
19.95	1368.6	684.3
20.15	1372.1	686.1



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Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

IGES

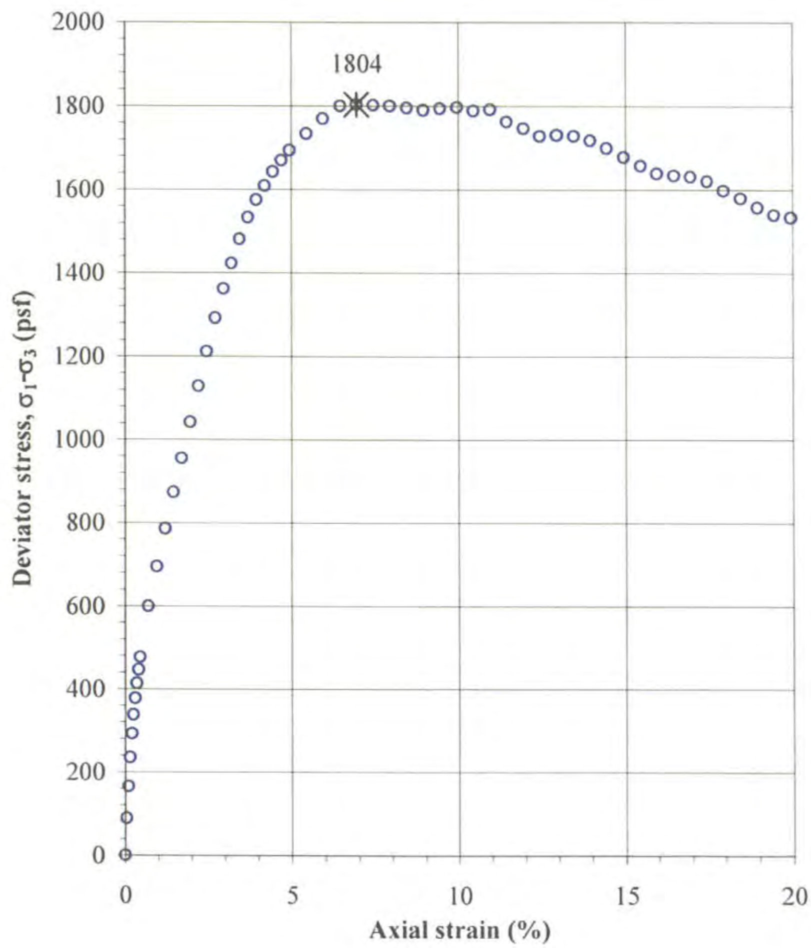
Project: Terracon (61085026)
No: M00385-078
Location: East/ West Connector Lehi
Date: 4/16/2008
By: BRR

Boring No.: B-17
Sample:
Depth: 30
Sample Description: gray clay with silt
Sample type: Undisturbed

Sample height, H (in.)	5.990
Sample diameter, D (in.)	2.830
Sample volume, V (ft^3)	0.0218
Wt. rings + wet soil (g)	1064.24
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1064.24
Moist unit wt., γ_m (pcf)	107.6
Dry unit wt., γ_d (pcf)	**71.0**

Wet soil + tare (g)	772.92
Dry soil + tare (g)	587.07
Tare (g)	226.30
Moisture content, w (%)	**51.5**
Confining stress, σ3 (psf)	1699
Shear rate (in/min)	0.0180
Strain at failure, ε_f (%)	6.95
Deviator stress at failure, $(\sigma 1\text{-}\sigma 3)_f$ (psf)	1804
Shear stress at failure, $q_f = (\sigma 1_f\text{-}\sigma 3_f)/2$ (psf)	902

Axial Strain (%)	σd σ1-σ3 (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	88.4	44.2
0.10	164.5	82.3
0.15	234.4	117.2
0.20	292.1	146.1
0.25	337.6	168.8
0.30	376.9	188.5
0.35	413.2	206.6
0.40	446.3	223.2
0.45	476.4	238.2
0.70	599.2	299.6
0.95	694.1	347.1
1.20	785.5	392.8
1.45	873.5	436.7
1.70	955.0	477.5
1.95	1042.1	521.0
2.20	1128.7	564.3
2.45	1211.8	605.9
2.70	1291.6	645.8
2.95	1362.0	681.0
3.20	1423.2	711.6
3.45	1481.1	740.5
3.70	1532.8	766.4
3.95	1575.4	787.7
4.20	1609.1	804.5
4.45	1642.5	821.2
4.70	1669.9	834.9
4.95	1694.2	847.1
5.45	1733.8	866.9
5.95	1769.9	884.9
6.45	1799.9	899.9
6.95	1803.9	901.9
7.45	1802.1	901.0
7.95	1800.2	900.1
8.45	1795.4	897.7
8.95	1790.6	895.3
9.45	1793.9	896.9
9.95	1797.2	898.6
10.45	1789.3	894.6
10.95	1792.3	896.1
11.45	1762.7	881.3
11.95	1746.7	873.3
12.45	1728.2	864.1
12.95	1731.0	865.5
13.45	1728.3	864.1
13.95	1717.7	858.8
14.45	1699.2	849.6
14.95	1678.3	839.1
15.45	1657.4	828.7
15.95	1639.3	819.6
16.45	1634.0	817.0
16.95	1631.2	815.6
17.45	1620.7	810.3
17.95	1597.7	798.8
18.45	1579.8	789.9
18.95	1557.1	778.5
19.45	1539.4	769.7
19.95	1534.1	767.0
20.20	1531.4	765.7



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Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

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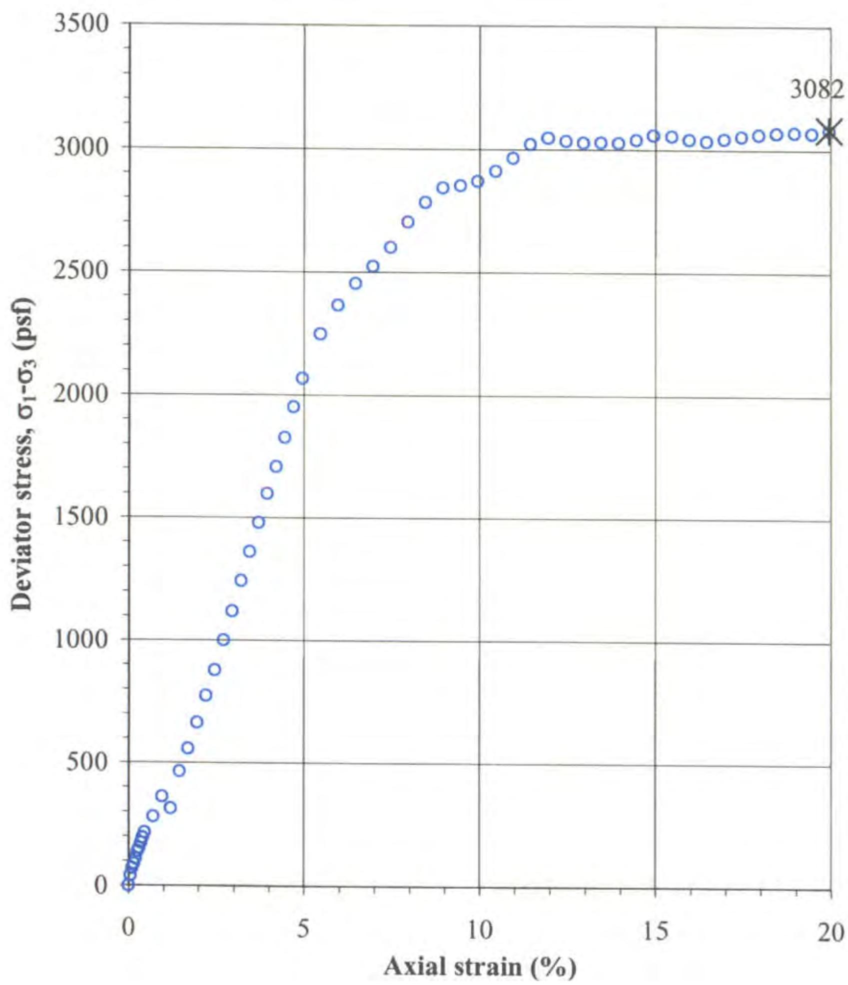
Project: Terracon
No: M00385-080 (61085026)
Location: E.W. Connector
Date: 5/6/2008
By: BRR

Boring No.: B-18
Sample:
Depth: 55
Sample Description: gray clay
Sample type: Undisturbed

Sample height, H (in.)	5.970
Sample diameter, D (in.)	2.840
Sample volume, V (ft^3)	0.0219
Wt. rings + wet soil (g)	1255.43
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1255.43
Moist unit wt., γ_m (pcf)	126.5
Dry unit wt., γ_d (pcf)	**99.4**

Wet soil + tare (g)	866.26
Dry soil + tare (g)	729.23
Tare (g)	225.14
Moisture content, w (%)	**27.2**
Confining stress, σ3 (psf)	2995
Shear rate (in/min)	0.0179
Strain at failure, ε_f (%)	19.95
Deviator stress at failure, $(\sigma1-\sigma3)_f$ (psf)	3082
Shear stress at failure, $q_f = (\sigma1_f-\sigma3_f)/2$ (psf)	1541

Axial Strain (%)	σd σ1-σ3 (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	41.7	20.8
0.10	71.4	35.7
0.15	89.2	44.6
0.20	109.9	55.0
0.25	139.6	69.8
0.30	154.3	77.2
0.35	175.0	87.5
0.40	195.6	97.8
0.45	216.3	108.1
0.70	280.6	140.3
0.95	362.3	181.2
1.20	313.9	157.0
1.45	465.7	232.9
1.70	558.1	279.0
1.95	664.6	332.3
2.20	773.5	386.7
2.45	878.9	439.4
2.70	1001.1	500.5
2.95	1119.9	559.9
3.20	1243.7	621.8
3.45	1361.2	680.6
3.70	1480.9	740.4
3.95	1600.0	800.0
4.20	1709.8	854.9
4.45	1827.6	913.8
4.70	1953.3	976.6
4.95	2069.8	1034.9
5.45	2250.1	1125.0
5.95	2366.6	1183.3
6.45	2456.7	1228.3
6.95	2526.1	1263.0
7.45	2603.0	1301.5
7.95	2706.3	1353.1
8.45	2786.6	1393.3
8.95	2846.8	1423.4
9.45	2854.7	1427.3
9.95	2873.2	1436.6
10.45	2915.3	1457.6
10.95	2967.3	1483.6
11.45	3024.0	1512.0
11.95	3050.9	1525.4
12.45	3038.1	1519.0
12.95	3030.4	1515.2
13.45	3032.8	1516.4
13.95	3030.0	1515.0
14.45	3042.3	1521.1
14.95	3061.8	1530.9
15.45	3058.2	1529.1
15.95	3044.4	1522.2
16.45	3038.0	1519.0
16.95	3046.3	1523.1
17.45	3056.7	1528.3
17.95	3064.4	1532.2
18.45	3071.7	1535.8
18.95	3073.8	1536.9
19.45	3070.9	1535.4
19.95	3082.1	1541.0
19.96	3081.8	1540.9



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Unconsolidated-Undrained Triaxial Compression Test on Cohesive Soils

(ASTM D2850)

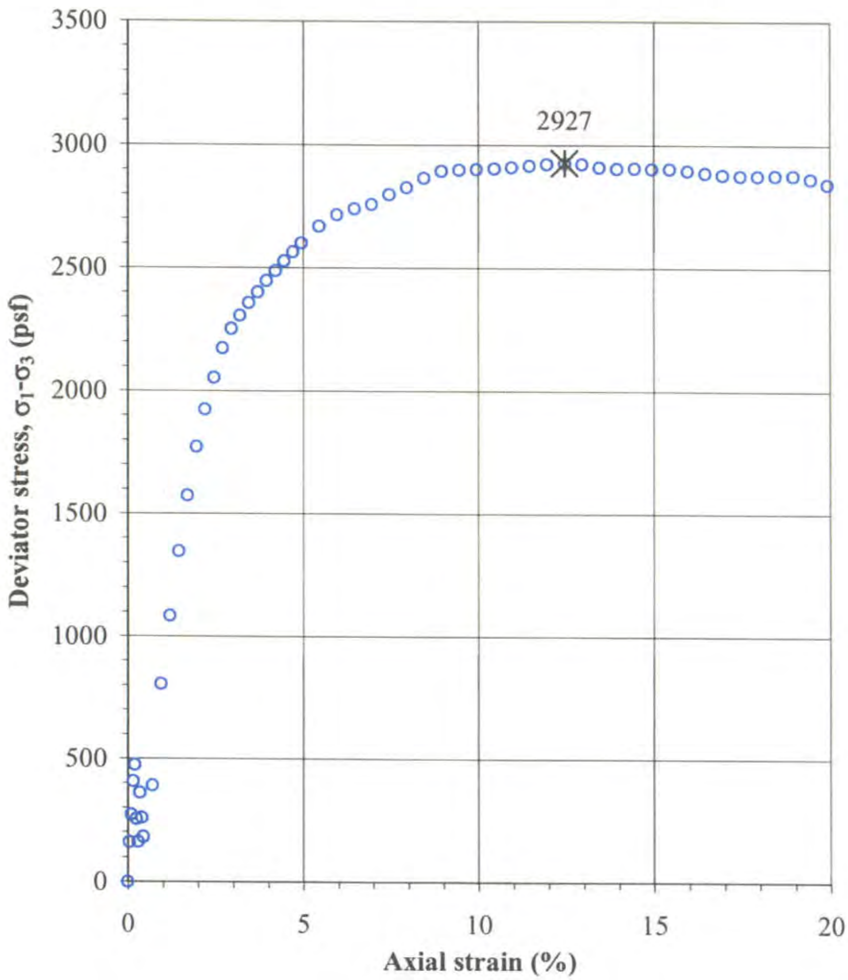
Project: Terracon
No: M00385-080 (61085026)
Location: E.W. Connector
Date: 5/5/2008
By: BRR

Boring No.: B-20
Sample:
Depth: 20
Sample Description: gray/ brown clay
Sample type: Undisturbed

Sample height, H (in.)	5.980
Sample diameter, D (in.)	2.860
Sample volume, V (ft^3)	0.0222
Wt. rings + wet soil (g)	1220.40
Wt. rings/tare (g)	0.00
Moist soil, Ws (g)	1220.40
Moist unit wt., γ_m (pcf)	121.0
Dry unit wt., γ_d (pcf)	**93.5**

Wet soil + tare (g)	383.46
Dry soil + tare (g)	328.20
Tare (g)	140.48
Moisture content, w (%)	**29.4**
Confining stress, σ3 (psf)	1800
Shear rate (in/min)	0.0179
Strain at failure, ε_f (%)	12.45
Deviator stress at failure, $(\sigma 1\text{-}\sigma 3)_f$ (psf)	2927
Shear stress at failure, $q_f = (\sigma 1_f\text{-}\sigma 3_f)/2$ (psf)	1464

Axial Strain (%)	σd σ1-σ3 (psf)	Q 1/2 σd (psf)
0.00	0.0	0.0
0.05	161.5	80.8
0.10	272.9	136.5
0.15	407.7	203.9
0.20	474.8	237.4
0.25	254.2	127.1
0.30	162.9	81.5
0.35	362.0	181.0
0.40	259.1	129.5
0.45	182.5	91.3
0.70	391.3	195.7
0.95	806.0	403.0
1.20	1084.8	542.4
1.45	1347.7	673.9
1.70	1574.5	787.3
1.95	1774.2	887.1
2.20	1926.8	963.4
2.45	2055.6	1027.8
2.70	2175.2	1087.6
2.95	2254.1	1127.1
3.20	2307.0	1153.5
3.45	2359.5	1179.8
3.70	2403.2	1201.6
3.95	2449.5	1224.8
4.20	2489.9	1245.0
4.45	2530.1	1265.1
4.70	2567.1	1283.6
4.95	2603.9	1302.0
5.45	2671.3	1335.7
5.95	2718.5	1359.3
6.45	2742.9	1371.5
6.95	2761.4	1380.7
7.45	2801.4	1400.7
7.95	2829.9	1415.0
8.45	2868.6	1434.3
8.95	2896.0	1448.0
9.45	2901.6	1450.8
9.95	2904.3	1452.2
10.45	2906.7	1453.4
10.95	2911.6	1455.8
11.45	2918.7	1459.4
11.95	2925.7	1462.9
12.45	2927.1	1463.6
12.95	2925.7	1462.9
13.45	2911.5	1455.8
13.95	2907.2	1453.6
14.45	2907.9	1454.0
14.95	2905.8	1452.9
15.45	2905.9	1453.0
15.95	2898.4	1449.2
16.45	2888.3	1444.2
16.95	2880.6	1440.3
17.45	2877.6	1438.8
17.95	2876.8	1438.4
18.45	2878.1	1439.1
18.95	2879.1	1439.6
19.45	2865.7	1432.9
19.92	2843.8	1421.9



Entered by: BRR
Reviewed:

testing services tm

PO Box 572455 / Salt Lake City UT 84157-2455 / USA
TEL (801) 262 2448 - FAX (801) 262 9870

RICK CHESTNUT
TERRACON
12217 LONE PEAK PARKWAY
DRAPER, UT 84020

Analysis No.	TS-A-03538
Report Date	04 April 2008
Date Sampled	18 March 2008
Time Sampled	Unspecified
Where	Unspecified
Sampled By	Client
Phone	(801) 545-8500
Facsimile	(801) 545-8600

This is to certify that we have examined soil sample identified: E W Connector Job Number 61085026 and found when tested to the requirements of:

SW846 pH Test Method 9045C

Ph	
Sample Identification	Results
B-1A @ 5'-7.5'	**7.9**
B-2 @ 7.5'	**7.7**
B-3 @ 10.0'	**8.8**
B-5 @ 7.5'	**8.3**

ASTM G 57 - 95a (2001) "Standard Test Method for Measurement of Soil Resistivity Using the Soil-Box Method"

RESULTS (Saturated using a Soil Box)		
Sample Identification	Resistivity	Conductivity
B-1A @ 5'-7.5'	**502.** ohms-cm	**0.00199** mho/cm
B-2 @ 7.5'	**474.** ohms-cm	**0.00211** mho/cm
B-3 @ 10.0'	**1,040.** ohms-cm	**0.000962** mho/cm
B-5 @ 7.5'	**3,990.** ohms-cm	**0.000251** mho/cm

USEPA Laboratory I.D. UT00930

Merrill Gee

Merrill K. Gee, – Analytical Test Technician

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RICK CHESTNUT
TERRACON
12217 LONE PEAK PARKWAY
DRAPER, UT 84020

Analysis No.	TS-A-03553
Report Date	15 April 2008
Date Sampled	07 April 2008
Time Sampled	Unspecified
Where	Unspecified
Sampled By	Client
Phone	(801) 545-8500
Facsimile	(801) 545-8600

This is to certify that we have examined soil sample identified: EW Connector Job Number 61085026 and found when tested to the requirements of:

SW846 Sulfate (Turbidimetric) Test Method 9038

Sulfate (Soluble)			
Sample Identification	Results		DL[2]
B-4 @ 10'	**17. ppm (mg/kg)**	**0.0017 %[1]**	10 ppm
B-9 @ 7.5'	< 10. ppm (mg/kg)	< 0.0010 %[1]	10 ppm

[1] % Sulfate by weight as received [2] Detection Limit

SW846 Chloride (Turbidimetric) Test Method 8113

Results: Chloride(Soluble)			
Sample ID	Concentration		DL[2]
B-4 @ 10'	**40.8 ppm (mg/kg)**	**0.00408 %[1]**	10 ppm
B-9 @ 7.5'	**93.5 ppm (mg/kg)**	**0.00935 %[1]**	10 ppm

SW846 pH Test Method 9045C

Ph	
Sample Identification	Results
B-4 @ 10'	**7.7**
B-9 @ 7.5'	**8.0**

ASTM G 57 - 95a (2001) "Standard Test Method for Measurement of Soil Resistivity Using the Soil-Box Method"

RESULTS (Saturated using a Soil Box)		
Sample Identification	Resistivity	Conductivity
B-4 @ 10'	**1,110. ohms-cm**	**0.000901 mho/cm**
B-9 @ 7.5'	**4,250. ohms-cm**	**0.000235 mho/cm**

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RICK CHESTNUT
TERRACON
12217 LONE PEAK PARKWAY
DRAPER, UT 84020

Analysis No.	TS-A-03583
Report Date	06 May 2008
Date Sampled	Unspecified
Time Sampled	Unspecified
Where	Unspecified
Sampled By	Client
Phone	(801) 545-8500
Facsimile	(801) 545-8600

This is to certify that we have examined soil sample identified: East West Connector Job Number 61085026 and found when tested to the requirements of:

SW846 Sulfate (Turbidimetric) Test Method 9038

Sulfate (Soluble)			
Sample Identification	Results		DL[2]
B-12 @ 20.0'	< 10. ppm (mg/kg)	< 0.0010 %[1]	10 ppm
B-15 @ 2.5'	**243. ppm (mg/kg)**	**0.0243 %[1]**	10 ppm
B-12 @ 10.0'	< 10. ppm (mg/kg)	< 0.0010 %[1]	10 ppm

[1] % Sulfate by weight as received [2] Detection Limit

SW846 Chloride (Turbidimetric) Test Method 8113

Results: Chloride(Soluble)			
Sample ID	Concentration		DL[2]
B-12 @ 20.0'	**57.9 ppm (mg/kg)**	**0.00579 %[1]**	10 ppm
B-15 @ 2.5'	**34.7 ppm (mg/kg)**	**0.00347 %[1]**	10 ppm
B-12 @ 10.0'	**23.4 ppm (mg/kg)**	**0.00234 %[1]**	10 ppm

SW846 pH Test Method 9045C

Ph	
Sample Identification	Results
B-12 @ 20.0'	**7.3**
B-15 @ 2.5'	**8.0**
B-12 @ 10.0'	**8.0**

ASTM G 57 - 95a (2001) "Standard Test Method for Measurement of Soil Resistivity Using the Soil-Box Method"

RESULTS (Saturated using a Soil Box)		
Sample Identification	Resistivity	Conductivity
B-12 @ 20.0'	**1,310. Ohms-cm**	**0.000763 mho/cm**
B-15 @ 2.5'	**2,130. Ohms-cm**	**0.000469 mho/cm**
B-12 @ 10.0'	**4,630. ohms-cm**	**0.000216 mho/cm**

USEPA Laboratory I.D. UT00930

Merrill Gee

Merrill K. Gee, – Analytical Test Technician

APPENDIX C

General Notes
Unified Classification System
AASHTO Classification System

GENERAL NOTES

DRILLING & SAMPLING SYMBOLS:

SS:	Split Spoon - 1-3/8" I.D., 2" O.D., unless otherwise noted	HS:	Hollow Stem Auger
ST:	Thin-Walled Tube - 2" O.D., unless otherwise noted	PA:	Power Auger
RS:	Ring Sampler - 2.42" I.D., 3" O.D., unless otherwise noted	HA:	Hand Auger
DB:	Diamond Bit Coring - 4", N, B	RB:	Rock Bit
BS:	Bulk Sample or Auger Sample	WB:	Wash Boring or Mud Rotary

The number of blows required to advance a standard 2-inch O.D. split-spoon sampler (SS) the last 12 inches of the total 18-inch penetration with a 140-pound hammer falling 30 inches is considered the "Standard Penetration" or "N-value". For 3" O.D. ring samplers (RS) the penetration value is reported as the number of blows required to advance the sampler 12 inches using a 140-pound hammer falling 30 inches, reported as "blows per foot," and is not considered equivalent to the "Standard Penetration"or "N-value".

WATER LEVEL MEASUREMENT SYMBOLS:

WL:	Water Level	WS:	While Sampling	N/E:	Not Encountered
WCI:	Wet Cave in	WD:	While Drilling		
DCI:	Dry Cave in	BCR:	Before Casing Removal		
AB:	After Boring	ACR:	After Casing Removal		

Water levels indicated on the boring logs are the levels measured in the borings at the times indicated. Groundwater levels at other times and other locations across the site could vary. In pervious soils, the indicated levels may reflect the location of groundwater. In low permeability soils, the accurate determination of groundwater levels may not be possible with only short-term observations.

DESCRIPTIVE SOIL CLASSIFICATION: Soil classification is based on the Unified Classification System. Coarse Grained Soils have more than 50% of their dry weight retained on a #200 sieve; their principal descriptors are: boulders, cobbles, gravel or sand. Fine Grained Soils have less than 50% of their dry weight retained on a #200 sieve; they are principally described as clays if they are plastic, and silts if they are slightly plastic or non-plastic. Major constituents may be added as modifiers and minor constituents may be added according to the relative proportions based on grain size. In addition to gradation, coarse-grained soils are defined on the basis of their in-place relative density and fine-grained soils on the basis of their consistency.

CONSISTENCY OF FINE-GRAINED SOILS

Unconfined Compressive Strength, Qu, psf	Standard Penetration or N-value (SS) Blows/Ft.	Consistency
< 500	0 - 1	Very Soft
500 – 1,000	2 - 4	Soft
1,000 – 2,000	4 - 8	Medium Stiff
2,000 – 4,000	8 -15	Stiff
4,000 – 8,000	15 - 30	Very Stiff
8,000+	> 30	Hard

RELATIVE DENSITY OF COARSE-GRAINED SOILS

Standard Penetration or N-value (SS) Blows/Ft.	Ring Sampler (RS) Blows/Ft.	Relative Density
0 – 3	0-6	Very Loose
4 – 9	7-18	Loose
10 – 29	19-58	Medium Dense
30 – 49	59-98	Dense
> 50	> 99	Very Dense

RELATIVE PROPORTIONS OF SAND AND GRAVEL

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 15
With	15 – 29
Modifier	> 30

RELATIVE PROPORTIONS OF FINES

Descriptive Term(s) of other constituents	Percent of Dry Weight
Trace	< 5
With	5 – 12
Modifiers	> 12

GRAIN SIZE TERMINOLOGY

Major Component of Sample	Particle Size
Boulders	Over 12 in. (300mm)
Cobbles	12 in. to 3 in. (300mm to 75 mm)
Gravel	3 in. to #4 sieve (75mm to 4.75 mm)
Sand	#4 to #200 sieve (4.75mm to 0.075mm)
Silt or Clay	Passing #200 Sieve (0.075mm)

PLASTICITY DESCRIPTION

Term	Plasticity Index
Non-plastic	0
Low	1-10
Medium	11-30
High	> 30

Terracon

UNIFIED SOIL CLASSIFICATION SYSTEM

Criteria for Assigning Group Symbols and Group Names Using Laboratory Tests[A]				Group Symbol	Group Name[B]
Coarse Grained Soils More than 50% retained on No. 200 sieve	Gravels More than 50% of coarse fraction retained on No. 4 sieve	Clean Gravels Less than 5% fines[C]	$Cu \geq 4$ and $1 \leq Cc \leq 3$[E]	GW	Well-graded gravel[F]
			$Cu < 4$ and/or $1 > Cc > 3$[E]	GP	Poorly graded gravel[F]
		Gravels with Fines More than 12% fines[C]	Fines classify as ML or MH	GM	Silty gravel[F,G,H]
			Fines classify as CL or CH	GC	Clayey gravel[F,G,H]
	Sands 50% or more of coarse fraction passes No. 4 sieve	Clean Sands Less than 5% fines[D]	$Cu \geq 6$ and $1 \leq Cc \leq 3$[E]	SW	Well-graded sand[I]
			$Cu < 6$ and/or $1 > Cc > 3$[E]	SP	Poorly graded sand[I]
		Sands with Fines More than 12% fines[D]	Fines classify as ML or MH	SM	Silty sand[G,H,I]
			Fines Classify as CL or CH	SC	Clayey sand[G,H,I]
Fine-Grained Soils 50% or more passes the No. 200 sieve	Silts and Clays Liquid limit less than 50	inorganic	PI > 7 and plots on or above "A" line[J]	CL	Lean clay[K,L,M]
			PI < 4 or plots below "A" line[J]	ML	Silt[K,L,M]
		organic	$\frac{\text{Liquid limit - oven dried}}{\text{Liquid limit - not dried}} < 0.75$	OL	Organic clay[K,L,M,N]
					Organic silt[K,L,M,O]
	Silts and Clays Liquid limit 50 or more	inorganic	PI plots on or above "A" line	CH	Fat clay[K,L,M]
			PI plots below "A" line	MH	Elastic Silt[K,L,M]
		organic	$\frac{\text{Liquid limit - oven dried}}{\text{Liquid limit - not dried}} < 0.75$	OH	Organic clay[K,L,M,P]
					Organic silt[K,L,M,Q]
Highly organic soils	Primarily organic matter, dark in color, and organic odor			PT	Peat

[A] Based on the material passing the 3-in. (75-mm) sieve

[B] If field sample contained cobbles or boulders, or both, add "with cobbles or boulders, or both" to group name.

[C] Gravels with 5 to 12% fines require dual symbols: GW-GM well-graded gravel with silt, GW-GC well-graded gravel with clay, GP-GM poorly graded gravel with silt, GP-GC poorly graded gravel with clay.

[D] Sands with 5 to 12% fines require dual symbols: SW-SM well-graded sand with silt, SW-SC well-graded sand with clay, SP-SM poorly graded sand with silt, SP-SC poorly graded sand with clay

[E] $Cu = D_{60}/D_{10} \quad Cc = \frac{(D_{30})^2}{D_{10} \times D_{60}}$

[F] If soil contains $\geq$ 15% sand, add "with sand" to group name.

[G] If fines classify as CL-ML, use dual symbol GC-GM, or SC-SM.

[H] If fines are organic, add "with organic fines" to group name.

[I] If soil contains $\geq$ 15% gravel, add "with gravel" to group name.

[J] If Atterberg limits plot in shaded area, soil is a CL-ML, silty clay.

[K] If soil contains 15 to 29% plus No. 200, add "with sand" or "with gravel," whichever is predominant.

[L] If soil contains $\geq$ 30% plus No. 200 predominantly sand, add "sandy" to group name.

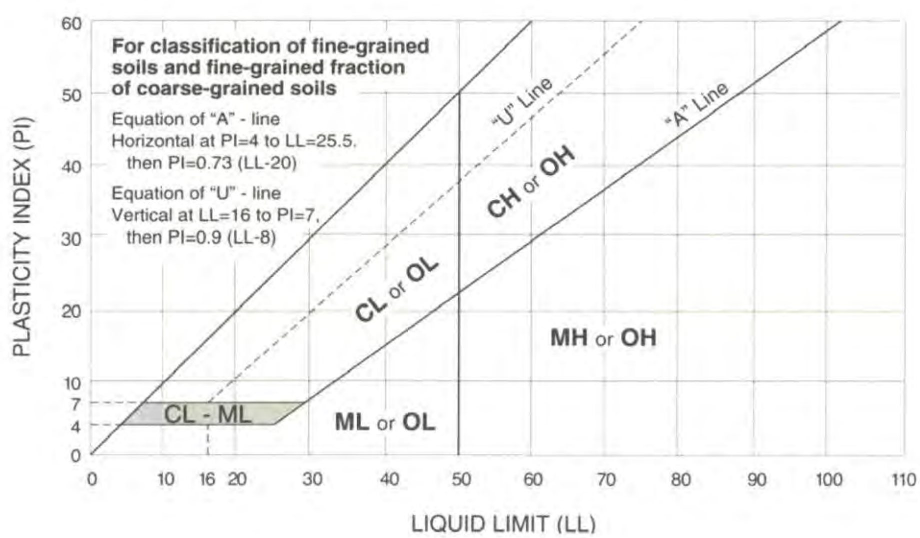
[M] If soil contains $\geq$ 30% plus No. 200, predominantly gravel, add "gravelly" to group name.

[N] PI $\geq$ 4 and plots on or above "A" line.

[O] PI < 4 or plots below "A" line.

[P] PI plots on or above "A" line.

[Q] PI plots below "A" line.



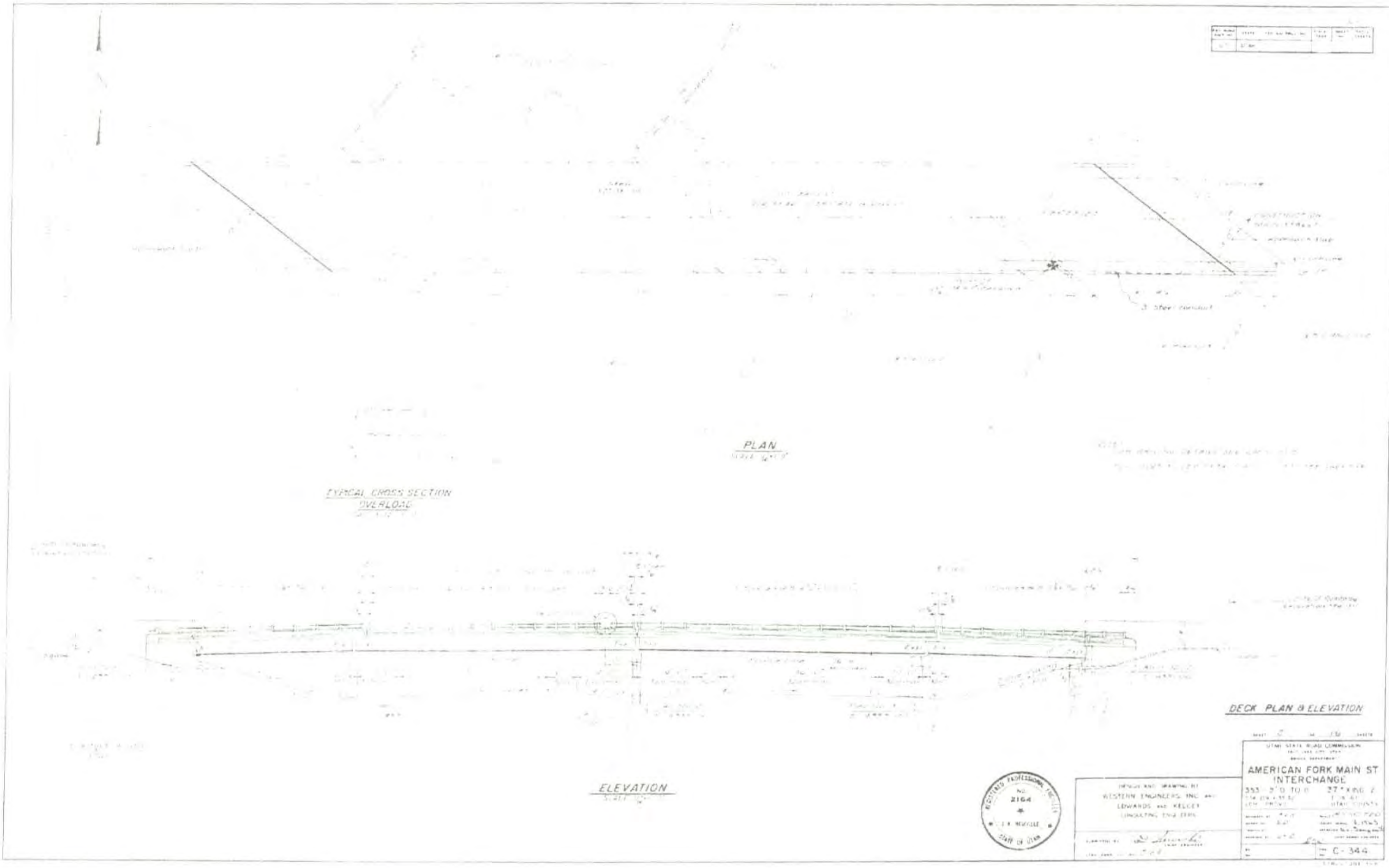
AASHTO Classification of Soils and Soil-Aggregate Mixtures

General Classification	Granular Materials (35 Percent or Less Passing 0.075 mm)							Silt-Clay Materials (More than 35 Percent Passing 0.075 mm)			
	A-1			A-2							A-7
Group Classification	A-1-a	A-1-b	A-3	A-2-4	A-2-5	A-2-6	A-2-7	A-4	A-5	A-6	A-7-5, A-7-6
Sieve analysis, percent passing:											
2.00 mm (No. 10)	50 max	---	---	---	---	---	---	---	---	---	---
0.425 mm (No. 40)	30 max	50 max	51 min	---	---	---	---	---	---	---	---
0.075 mm (No. 200)	15 max	25 max	10 max	35 max	35 max	35 max	35 max	36 min	36 min	36 min	36 min
Characteristics of fraction passing 0.425 mm (No. 40):											
Liquid limit	---		---	40 max	41 min	40 max	41 min	40 max	41 min	40 max	41 min
Plasticity index	6 max		N.P.	10 max	10 max	11 min	11 min	10 max	10 max	11 min	11 min*
Usual types of significant constituent materials	Stone fragments, gravel and sand		Fine sand	Silty or clayey gravel and sand				Silty soils		Clayey soils	
General ratings as subgrade	Excellent to good							Fair to poor			

*Plasticity index of A-7-5 subgroup is equal to or less than LL minus 30. Plasticity index of A-7-6 subgroup is greater than LL minus 30

APPENDIX D

Previous Studies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OVERLOAD
ELEVATION
DECK PLAN & ELEVATION
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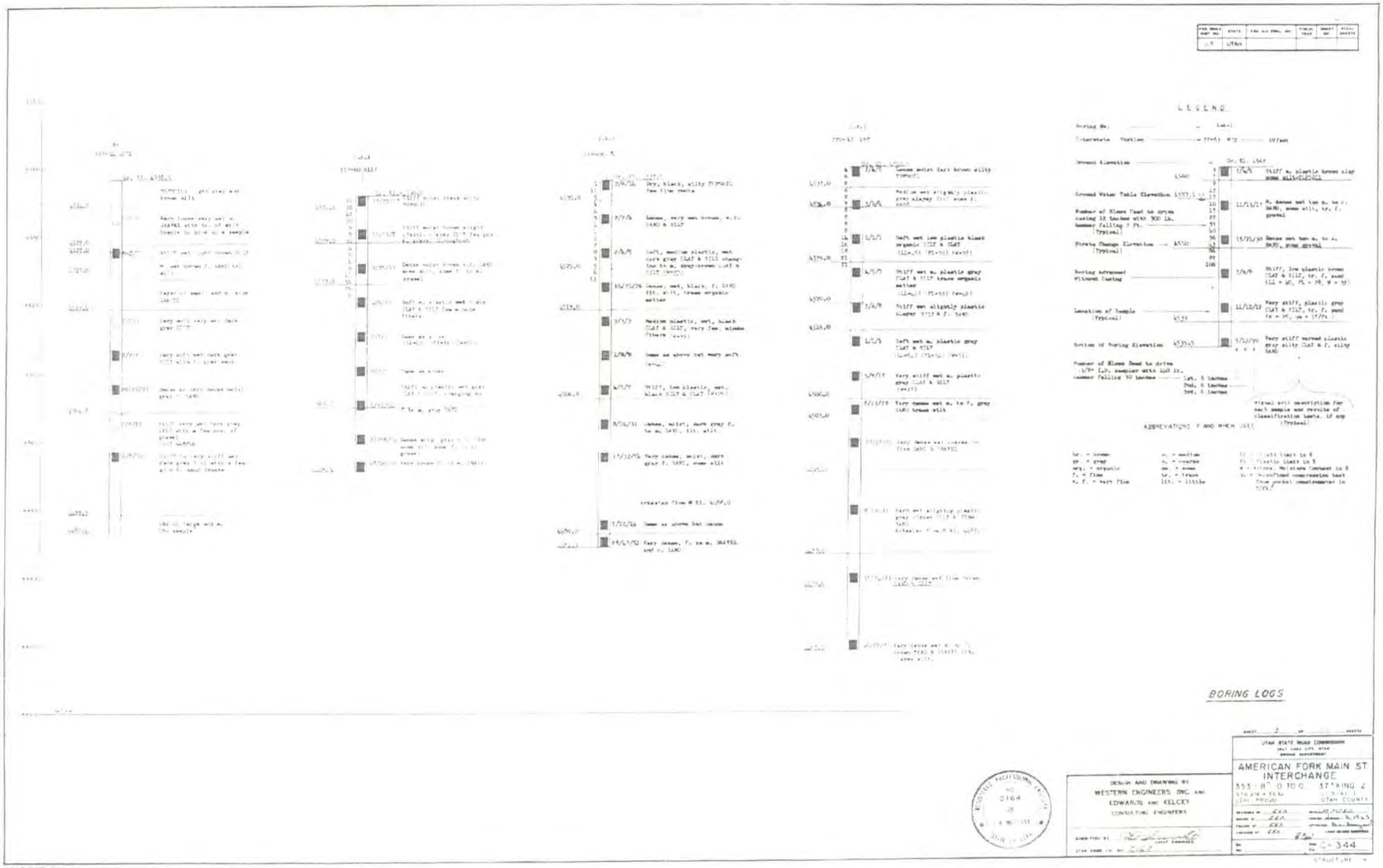
AMERICAN FORK MAIN ST
INTERCHANGE
WESTERN ENGINEERS INC and
EDWARDS and KELCEY
CONSULTING ENGINEERS
REGISTERED PROFESSIONAL ENGINEER
NO. 2164
STATE OF UTAH
C-344


LEGEND
ABBREVIATIONS IF AND WHEN USED
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UTAH STATE ROAD COMMISSION
AMERICAN FORK MAIN ST
INTERCHANGE
UTAH COUNTY
DESIGN AND DRAWING BY
WESTERN ENGINEERS INC AND
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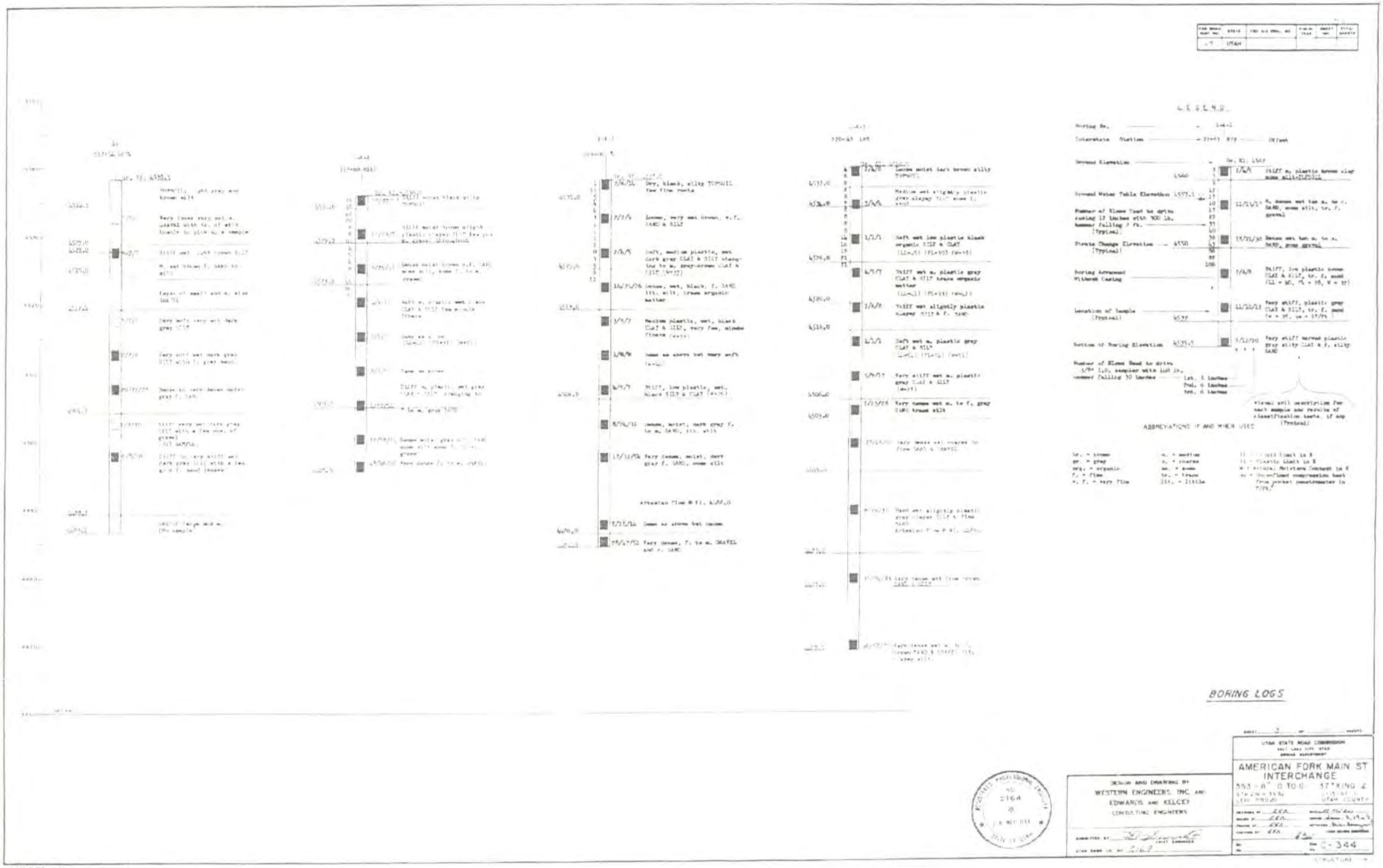
CONSULTING ENGINEERS
C-344



BORING LOGS
UTAH STATE ROAD COMMISSION
AMERICAN FORK MAIN ST
INTERCHANGE
UTAH COUNTY
DESIGN AND DRAWING BY
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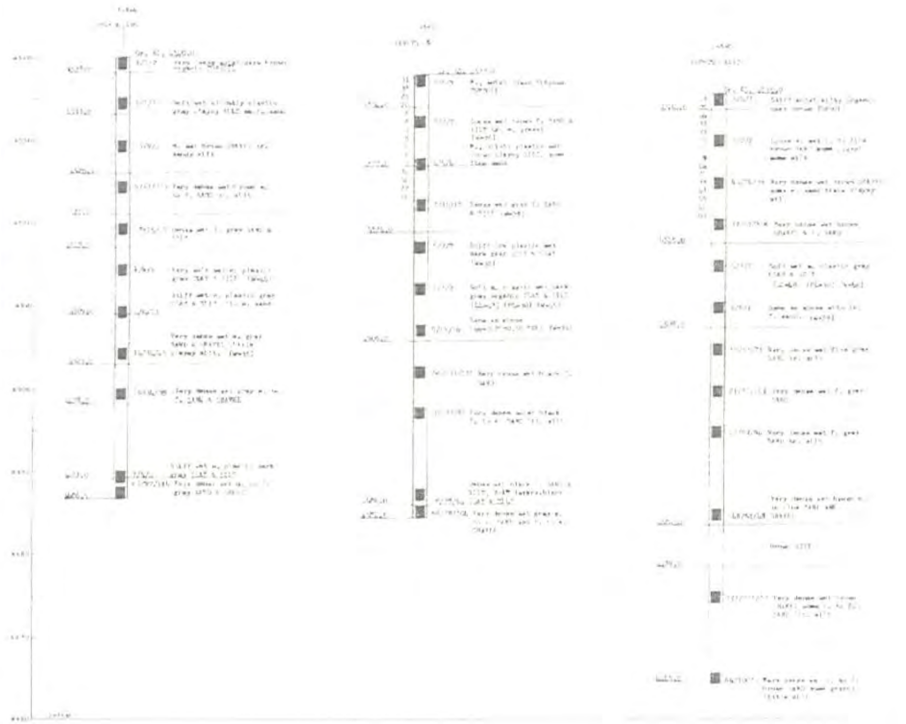
CONSULTING ENGINEERS
ABBREVIATIONS IF AND WHEN USED
C-344

U.T.	UTAH				



LEGEND

Boring No. — 1484.1

Interstate Station — 37+63 RT. Offset

Ground Elevation — Gr. El. 4562

Ground Water Table Elevation 4557.1

Number of Blows Used to drive casing 12 inches with 300 lb. hammer falling 2 ft. (Typical)

Strata Change Elevation — 4550 (Typical)

Boring Advanced Without Casing

Location of Sample (Typical)

Bottom of Boring Elevation 4535.3

Number of Blows Used to drive 1-3/8" I.D. sampler with 140 lb. hammer falling 30 inches — 1st. 6 inches, 2nd. 6 inches, 3rd. 6 inches

Stiff m. plastic brown clay some silt-topsoil

M. dense wet tan m. to c. SAND, some silt, tr. f. gravel

Dense wet tan m. to c. SAND, some gravel

Stiff, low plastic brown CLAY & SILT, tr. f. sand (LL = 40, PL = 28, W = 32)

Very stiff, plastic gray CLAY & SILT, tr. f. sand (W = 35, qu = 1T/Ft.²)

Very stiff warved plastic gray silty CLAY & f. silty SAND

Visual soil description for each sample and results of classification tests, if any (Typical)

ABBREVIATIONS (FAND SOIL USE)

br. = brown	m. = medium	LL = Liquid Limit in %
gr. = gray	c. = coarse	PL = Plastic Limit in %
org. = organic	so. = some	W = Natural Moisture Content in %
f. = fine	tr. = trace	qu = Unconfined compression test from pocket penetrometer in T/Ft.²
v. f. = very fine	lit. = little	

BORING LOGS

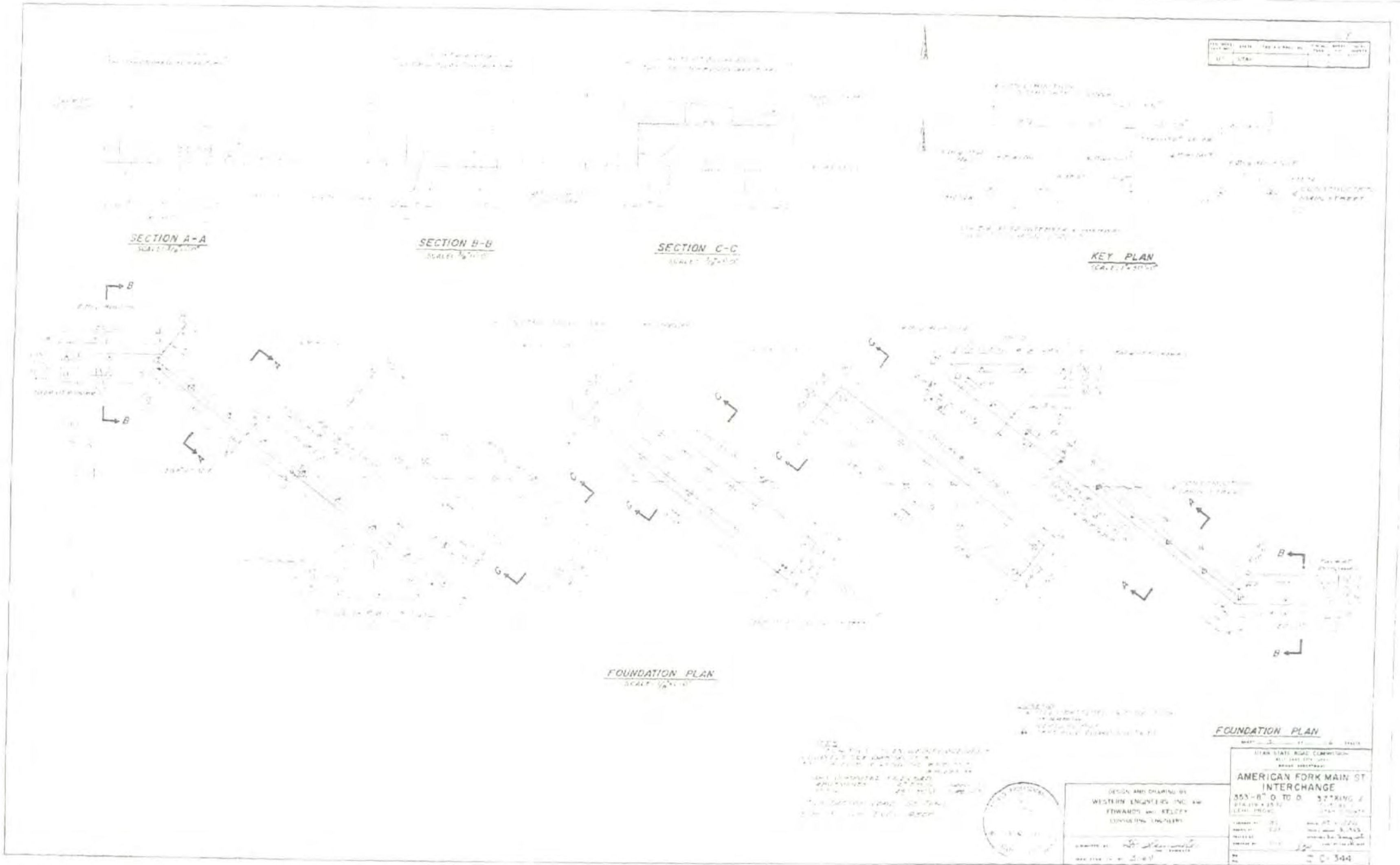
DESIGN AND DRAWING BY
WESTERN ENGINEERS INC. and
EDWARDS and KELCEY
CONSULTING ENGINEERS

UTAH STATE ROAD COMMISSION
SALT LAKE CITY, UTAH

AMERICAN FORK MAIN ST
INTERCHANGE

UTAH COUNTY

C-344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SECTION B-B
SECTION C-C
KEY PLAN
FOUNDATION PLAN
FOUNDATION PLAN
UTAH STATE ROAD COMMISSION
AMERICAN FORK MAIN ST
INTERCHANGE
DESIGN AND DRAWING BY
WESTERN ENGINEERS INC. AND
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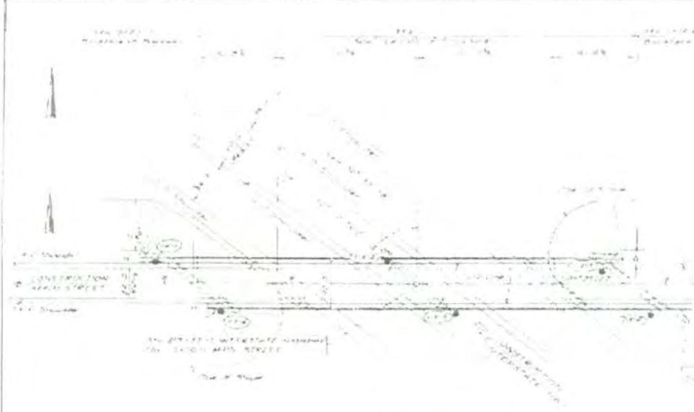
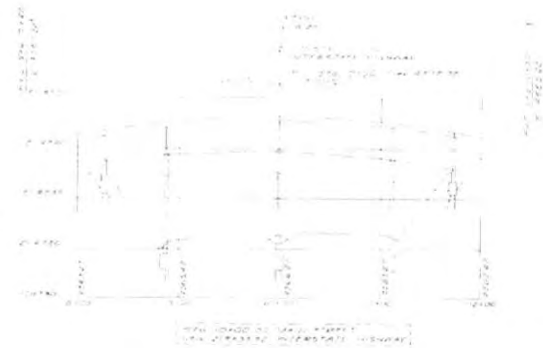
C-344



KEY PLAN



MAIN STREET PROFILE

INTERSTATE HIGHWAY PROFILE

ESTIMATE OF QUANTITIES

NO	ITEM	QUANTITIES	
	EXCAVATION FOR STRUCTURES UNCL.	530 Cu Yd	[illegible]
	STONE CONC. SLOPE PROTECTION	[illegible] Sq Yd	[illegible]
	MECHANICAL TAMPING	78 [illegible]	[illegible]
	CLASS A CONCRETE (AE)	[illegible] Cu Yd	[illegible]
	REINFORCING STEEL	[illegible] Lb	[illegible]
	STRUCTURAL STEEL	[illegible] Lb	[illegible]
	FURNISHING PILE DRIVING EQUIP.	One Lump	
	TEST PILES	35 Lin Ft	63
	LOADING TEST	One Lump	
	PILE (OTHER THAN TIMBER)	[illegible] Lin Ft	5523
	METAL RAILING DOUBLE RAIL	775 Lin Ft	774
	3" RIGID STEEL CONDUIT	390 Lin Ft	402
	ELECTRICAL WORK BRIDGE	1 Lump	

GENERAL NOTES

DESIGN SPECIFICATIONS:

CONSTRUCTION SPECIFICATIONS:

LIVE LOAD:

ALLOWABLE STRESSES:

STRUCTURAL STEEL:

CONCRETE:

REINFORCEMENT:

CHAMFERS:

FOUNDATIONS:

DATUM:

BENCH MARK:

SHOP DRAWINGS:

STANDARD DRAWINGS

DWG. NUMBER	DESCRIPTION
ST-2	[illegible] DETAILS
ST-3	PILE DETAILS
ST-5	LIGHT DETAILS
SS-3	STEEL DETAILS
SS-2	STEEL DETAILS
MR	METAL RAILING STD.

INDEX OF SHEETS

SHEET NUMBER	DESCRIPTION
1	KEY PLAN & INDEX SHEET
2	DECK PLAN & ELEVATION
3-4	BORING LOGS
5	FOUNDATION PLAN
6-7	ABUTMENTS NO. 1 & NO. 2
8	PIERS NO. 1, NO. 2 & NO. 3
9	TRANSVERSE SECTION
10	FRAMING PLAN
11-12	STEEL DETAILS
13	CONCRETE DETAILS
14	REINFORCING SCHEDULE
[illegible]	STONE CONC. SLOPE PROT.
16	SCREED ELEVATIONS
[illegible]	ELECTRICAL WORK BRIDGE

KEY PLAN & INDEX SHEET

UTAH STATE ROAD COMMISSION

AMERICAN FORK MAIN ST. INTERCHANGE

353'-8" O TO O

DESIGN AND DRAWN BY WESTERN ENGINEERS INC. AND EDWARDS AND KELCEY CONSULTING ENGINEERS

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