ARCADIS

Infrastructure, environment, buildings

Georgia-Pacific LLC

Stability Assessment of the Mill Pond Dam

Former Georgia-Pacific Wood Products Facility

Fort Bragg, California

June 2010

ARCADIS
Infrastructure, environment, buildings

Imagine the result

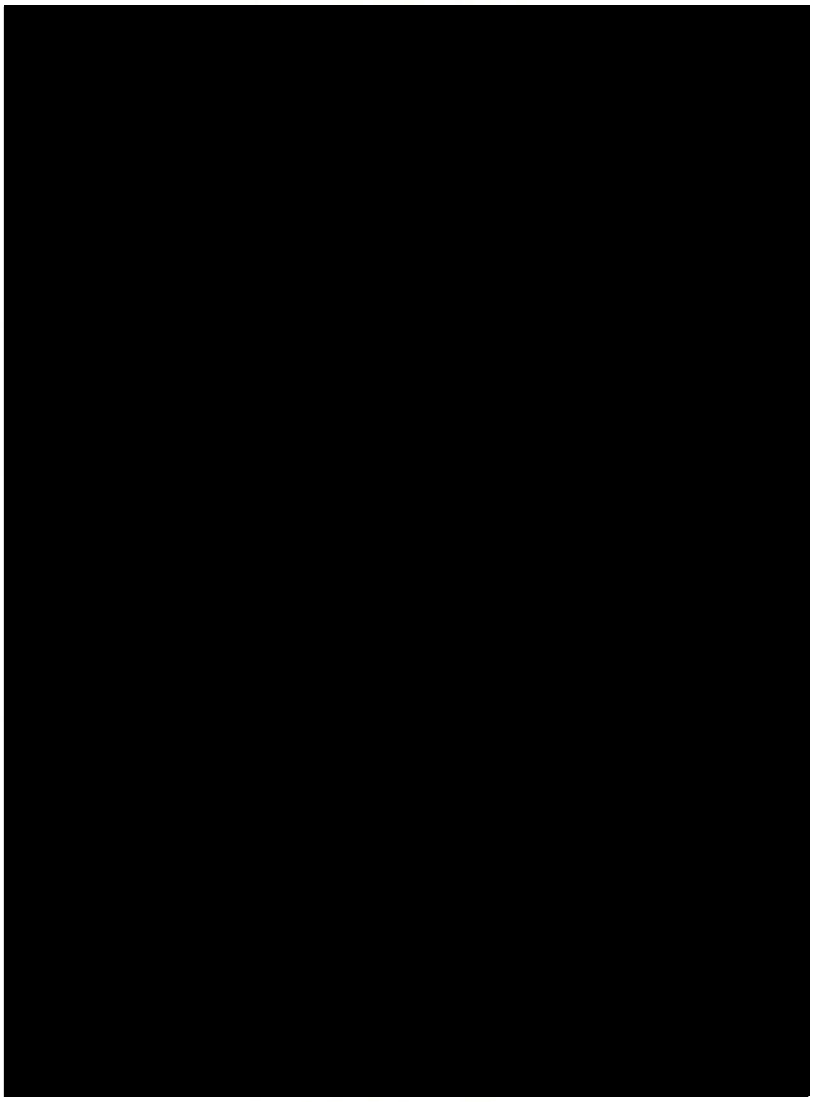
Georgia-Pacific LLC

Stability Assessment of the Mill Pond Dam

Former Georgia-Pacific Wood Products Facility
Fort Bragg, California

June 2010

ARCADIS



Stability Assessment of the Mill Pond Dam

Former Georgia-Pacific Wood Products Facility
Fort Bragg, California

Prepared for:

Georgia-Pacific LLC

Prepared by:

ARCADIS
140 2nd Street
Suite 200
Petaluma
California 94952
Tel 707.776.0865
Fax 707.776.0850

Our Ref.:

B0066116.0000.0004

Date:

June 2010

Table of Contents

1. Introduction 1

1.1 Background and Purpose of Work 1

1.2 Scope of Work 3

1.3 Report Organization 4

2. Site and Mill Pond Dam Description 5

2.1 Mill Pond Dam 5

2.2 Subsurface Conditions 6

2.2.1 Site Geology 6

2.2.2 Generalized Subsurface Soil Conditions 7

2.3 Groundwater 8

2.4 Sediments 8

3. Geotechnical Engineering Analyses 9

3.1 Seismic Source and Seismic Hazard Characterization 10

3.1.1 Faulting 10

3.1.2 Seismic Source Model 10

3.1.3 Historical Seismicity 11

3.2 Seismic Hazard Analysis 11

3.2.1 Attenuation Relationships 12

3.2.2 Deterministic Seismic Hazard Procedure 12

3.2.3 Theoretical Outcrop Response Spectra 12

3.3 Ground Motions 13

3.3.1 Selection of Acceleration Time Histories 13

3.3.2 Spectrally Matched Outcrop Ground Motions 13

3.4 Ground Response Analyses 14

3.4.1 Ground Response Model 14

3.4.2 Peak Accelerations 14

3.5 Soil Liquefaction Analysis 14

3.6 Slope Stability Analyses 15

3.6.1 Assumptions 15

3.6.1.1 Geometry 1 5

3.6.1.2 Soil Properties 15

3.6.1.3 Rock Strength 16

3.6.2 Static Analysis 16

3.6.3 Post-Earthquake Analysis 17

3.6.4 Deformation Analysis 18

3.7 North Wall Assessment 18

4. Geotechnical Engineering Conclusions and Recommendations 21

4.1 Recommendations 22

4.2 Summary 24

5. References 26

Tables

Table 1. Significant Seismic Sources within 100 km of Fort Bragg 11

Table 2 – Material Properties for Slope Stability Analyses 19

Table 3 – Material Properties for Post-earthquake Slope Stability Analyses 20

Figures

1 Site Location Map

2 OU-E Location Map

3 Site and Exploration Plan

4 Cross Sections and Generalized Subsurface Conditions

5 Regional Faulting and Seismicity Map

6 Spectral Accelerations – Comparison of MCE and 475-Year Return Period

7 Spectral Accelerations of Spectrally Matched Input Ground Motions

8 Shear Velocity Profiles for Ground Response Analyis

9 Peak Accelerations – Boring Location OUE-GT-001

10 Peak Accelerations – Boring Location OUE-GT-002

11 Peak Accelerations – Boring Location OUE-GT-003

12 Peak Accelerations – Boring Location OUE-GT-004

13 Liquefaction Analysis Results – Boring Location OUE-GT-001

14 Liquefaction Analysis Results – Boring Location OUE-GT-002

15 Liquefaction Analysis Results – Boring Location OUE-GT-003

16 Liquefaction Analysis Results – Boring Location OUE-GT-004

17 Estimated Extent of Soil Liquefaction During MCE

18 Summary of Slope Stability Analyses – Section B-B'

19 Summary of Slope Stability Analyses – Section D-D'

20 Summary of Slope Stability Analyses – Section E-E'

21 Newmark Analysis Results – Section B-B' – Failure Surface A

22 Newmark Analysis Results – Section B-B' – Failure Surface B

23 Newmark Analysis Results – Section D-D' – Failure Surface A

24 Newmark Analysis Results – Section D-D' – Failure Surface B

Appendices

A Geotechnical Field Exploration Methods and Logs

B Geotechnical Laboratory Test Methods and Results

C North Wall Assessment

D SOD Correspondence

E Photo Log

Acronyms and Abbreviations

CGS	California Geologic Survey
City	City of Fort Bragg

DSHA	deterministic seismic hazard analysis
DSOD	Division of Safety of Dams
Georgia-Pacific	Georgia-Pacific, LLC
km	kilometer(s)
M	magnitude
MCE	maximum credible earthquake
NAVD88	North American Vertical Datum of 1988
OU	operable unit
psf	pounds per square foot
PSHA	probabilistic seismic hazard analyses
site	the former Georgia-Pacific Wood Products Facility
SPT	standard penetration test
TCW	total class weight

1. Introduction

This report was prepared by ARCADIS on behalf of Georgia-Pacific LLC (Georgia-Pacific) and presents the results of a geotechnical engineering study performed to evaluate the seismic stability of the Mill Pond Dam (the dam) completed as part of planning efforts to address short term and long term corrective measures required to ensure the safety and stability of the dam. The dam is currently under the jurisdiction of the (Division of Safety of Dams [DSOD]), No. 2391-000 – Mill Pond Dam, National Dam No. CA01139, located at the Former Georgia-Pacific Wood Products Facility in Fort Bragg, California (the site; Figure 1).

This study was completed in accordance with generally accepted engineering practices for the nature and conditions of work completed in the same or similar localities, at the time that the work was performed.

1.1 Background and Purpose of Work

The MPC (Figure 1-2) comprises a portion of OU E, which includes upland (terrestrial) areas as well as a series of 10 man-made ponds (Ponds 1 through 9 and the North Pond). The Mill Pond (Pond 8, also known as the Log Pond), is a 7.3-acre pond; a cribwall dam, concrete spillway, and north wall form the Mill Pond. When the site was originally developed around 1885, the Mill Pond (also known as the Log Pond or Pond 8, located in Operable Unit [OU] E) was formed by constructing a dam along and on top of the rock that comprises the edge of the coastal bluff (Stetson Engineers Inc., 2005) and directing the flow of Alder Creek to subterranean flow. Site documents indicate that a depression was excavated into the terrace deposits behind the dam to increase the storage capacity of the pond. A more detailed description of the Log/Mill Pond is provided in Section 2.

Due to the height of the dam (i.e., greater than 25 feet high) and its capacity (greater than 15 acre-feet), the dam and pond are under the jurisdiction of the California Division of Safety of Dams (DSOD). In June 2009 and April 2010, the DSOD inspected the dam and notified Georgia-Pacific that the dam condition required corrective action as a result of

- erosion along the toe of the concrete spillway;
- soil has loosened and eroded from the cribwall;
- seepage on the north wall and dam face could lead to failure by piping (internal erosion) ; and
- the dam is more than 100 years old and susceptible to earthquakes (DSOD, 2009).

In addition to the DSOD observations, a preliminary slope stability study performed by Miller Pacific shows that the dam may not be stable under seismic loading conditions (Miller Pacific, 2005). Miller Pacific did not perform a site investigation to obtain data for their study and, therefore, used assumed soil and rock conditions to assess slope stability.

Georgia-Pacific and ARCADIS have been engaged in investigation and remedial planning for Pond 8, the Mill Pond Complex, and other portions of OU-E since 2008 (ARCADIS, 2009, 2010). The investigation, feasibility study, and remedial action planning for the entire OU-E will take several years to complete. Therefore, Georgia-Pacific has proposed a proactive phased approach to address these short and long-term needs for dam stabilization and remediation as independent projects. The results of the preliminary study by Miller-Pacific and prior letters from DSOD warranted a more detailed assessment of the stability of the dam. Georgia-Pacific began the assessment effort for the dam in early 2009 in response to these earlier findings and as part of preliminary work on the remedial planning for the Mill Pond. Georgia-Pacific has spent the last year gathering additional information (ARCADIS, 2010), conducting technical analyses, and evaluating alternatives to address dam safety, necessary stormwater rerouting, and remediation components. As a result of those initial efforts, Georgia-Pacific is currently proposing a Phase I project referred to as the "Interim Corrective Action and Stormwater Rerouting Project," which is an interim project that will reduce the safety hazard associated with the dam by rerouting stormwater to the low-lying areas of the MPC and thus reduce the pressure on the dam.

The second phase, Phase II, is the Dam Stabilization/ Removal and Remediation Project. The purpose of the Phase II project is to provide long-term seismic and geotechnical stability which includes an alternative to stabilize sediments in the western portion of Pond 8 and ultimately remove the dam, spillway, and cribwall and recontour the bluff face.

Objectives for the stability assessment presented herein are to evaluate available geotechnical data, including geotechnical boring data collected in 2009, to assess the overall stability of the dam wall, including the cribwall and spillway portions and the north wall, as detailed in Appendices A through C. This effort was originally initiated to assessment the stability of the dam following removal of several structures from the low-lying portion of OU-E (north of the dam).

The June 2009 DSOD letter (Appendix D) listed five items considered necessary to correct the dam condition:

1. Fill the large void at the timber supported portion of the embankment (cribwall)

2. Patch the damaged base of the spillway structure (spillway)

3. Remove tule growth along the toe the right embankment (northern dam wall)

4. Clear the brush that obstructs embankment, groins and abutments

5. Clear the vegetation that obstructs the spillway entrance

Georgia-Pacific responded to DSOD in a letter dated August 28, 2009, stating that Items 3 through 5 could be completed to enable further inspection of the dam, mainly the north wall, which has now been completed. However, the August 2009 letter (Appendix D) stated that "Due to the ongoing remediation and site planning and local, state, and federal permitting requirements, Georgia-Pacific is faced with certain constraints in its ability to undertake DSOD's recommended dam repair and maintenance activities" but that "Georgia-Pacific is exploring long-term solutions for improving the Mill Pond Dam stability in a manner consistent with the ongoing site remediation and future plans for the site." The investigations described herein were undertaken as part of planning process to determine the corrective measure(s) necessary to respond to the DSOD's June, 2009 letter and ensure overall safety of the dam structure.

1.2 Scope of Work

ARCADIS performed the following tasks as part of the stability assessment:

- Performed a geotechnical subsurface investigation to determine soil stratigraphy and soil properties. The investigation included four soil borings (with rock coring in two of the borings) and geotechnical laboratory testing on soil and rock samples. ARCADIS also performed some limited geologic mapping of the exposed rock face of the dam.

- Performed topographic survey of six cross sections along the dam alignment. The survey was performed by a licensed land surveyor (Laco Associates). The cross sections were used for numerical slope stability modeling.

- Identified the seismic sources within 100 kilometers (km; 62 miles) of the site and estimated the seismic hazard for the maximum credible earthquake (MCE).

- Performed a ground motion database search and selected appropriate ground motions for spectral matching using the seismic hazard due to the MCE. Generated four artificial input ground motions for use in a ground response analysis.

- Performed ground response analyses to generate required input into liquefaction and slope stability analyses.

- Performed static, pseudostatic, and post-earthquake slope stability analyses as well as deformation analyses.

- Evaluated the stability of the north wall and assessed the condition of the north wall following vegetation removal.

- Provided conclusions regarding the stability of the existing dam.

1.3 Report Organization

The remainder of this report is organized as follows:

Section 2, Site and Project Description: provides a brief summary of the site as well as more detailed information regarding the dam.

Section 3, Geotechnical Engineering Analyses: outlines the geotechnical analyses conducted to assess the seismic stability of the dam and the stability of the north wall.

Section 4, Geotechnical Engineering Conclusions: provides the conclusions ascertained from the results of the analyses conducted.

Section 5, References: lists the sources of information relied on in preparation of this report.

Appendix A, Geotechnical Field Exploration Methods and Logs: contains documentation of the field exploration methods and copies of the logs.

Appendix B, Geotechnical Laboratory Test Methods and Results: provides information regarding the laboratory test methods and copies of the laboratory results.

Appendix C, North Wall Assessment: provides the results of evaluations of the safety and stability of the north wall.

Appendix D, DSOD Correspondence: provides copies of the June 2009 DSOD letter and the August 2009 letter response from Georgia-Pacific.

Appendix E, Photo Log: provides photographs taken after the 2009 vegetation clearance activities and recent photos of the dam, cribwall, and north wall conditions.

2. Site and Mill Pond Dam Description

The 415-acre site is located west of Highway 1 along the Pacific Ocean coastline and is bounded by open coastline to the north, Noyo Bay to the south, the City of Fort Bragg (City) to the east, and the Pacific Ocean to the west. According to historical records, Union Lumber Company began sawmill operations of primarily redwood lumber at the site in 1885. Georgia-Pacific acquired the site in 1973 and ceased sawmill operations on August 8, 2002; much of the equipment and structures associated with the lumber production have since been removed.

2.1 Mill Pond Dam

The Mill Pond (and its associated dam) is located in OU-E and is one of a series of ten man-made ponds (Ponds 1 through 9 and the North Pond), each designed and created for various industrial purposes, as well as stormwater management (approximately 70 percent of the stormwater input to the Mill Pond is from the City of Fort Bragg). The location of the ponds is shown in Figure 2. The ponds range in size from 0.1 to 7.3 acres. Water transfer into and among the ponds was an integral part of the operational history of the ponds.

The Mill Pond (also known as the Log Pond or Pond 8) is approximately 7.3 acres and was created in the late 1800s. The pond once extended further to the northeast and southwest from the current configuration. The pond was historically used as a log pond (to float logs into the former Sawmill) and more recently as a source of cooling water for the powerhouse. The pond was also used as part of the treatment process of "scrubber" effluent. The scrubber effluent was discharged into the Mill Pond (Pond 8) after passing through a series of settling ponds (Ponds 6 and 7 in Parcel 4 and Ponds 1 through 4 in Parcel 7). Pond 8 currently receives approximately 40% of the City's stormwater runoff as well as site runoff. The northeastern and southwestern portions of the Mill Pond were filled in the early 1970s using native material and other fill. Water from the Overflow Pond (Pond 3) flowed to Pond 8 through an underground pipeline. Pond 8 also has an outfall that discharges to Soldier Bay. A schematic illustration of water flow at the site and information regarding the operational history of the other OU-E ponds is provided in the *Preliminary Site Investigation Work Plan, Operable Unite E – Ponds* (ARCADIS BBL, 2007).

The dam is under the jurisdiction of the State of California (National ID No. CA01139) (DSOD, 2010). The California Department of Water Resources, DSOD provides the following information regarding the dam (DSOD No. 2381-0000 – Mill Pond Dam):

- Capacity = 72 acre-feet
- Crest Elevation = 51.2 feet
- Height = 33 feet

- Freeboard = 5.2 feet.

The pond is approximately 1,700 feet long and between 120 and 350 feet wide (Figure 3). The dam is located along the northern edge of the pond. DSOD reports that the dam is 200 feet long. Generally, the western portion of the pond located directly along Soldier Bay is approximately 500 feet long. This area includes the spillway and crib wall. In this area, the lower portion of the dam cross section consists of the native rock (Franciscan Formation), which is exposed at the face of the dam along the beach. The upper portion is made up of native and fill soils. The fill soils were placed to create the dam. The rock is not exposed along the eastern portions of the dam. Photographs of the dam, spillway, and crib wall from 2008 are provided in the dam inspections conducted by DSOD in 2008 (Appendix D). Photographs taken in March 2010 are provided in Appendix E.

The most critical portion of the dam appears to be the area near the spillway and crib wall (Figure 3) located at cross section locations B-B' and C-C', respectively. The crib wall is also depicted on Figure 4 – Section B-B'. DSOD also noted in June 2009, relative to the North Wall of the pond, that:

> "Standing water has accumulated within low areas along the right embankment toe. Supply pipes to and from the pond perforate the embankment within this area, and neither the condition of the pipes nor the status of encasement of the pipes is known. I directed Mr. Heitmeyer to investigate the cause of the seepage and to make whatever repairs are necessary to prevent uncontrolled seepage from damaging the embankment."

Recent inspections have since shown that the above conditions also make north wall vulnerable, as discussed in Section 3.

2.2 Subsurface Conditions

Subsurface information presented herein is based on four borings drilled in early 2009 as part of initital investigation efforts (Borings OUE-GT-001 through OUE-GT-004). The approximate locations of the borings are shown on Figure 3. Explorations consisted of four mud rotary borings with standard penetration testing (SPT) and split spoon sample collection. Rock coring was performed in two of the four borings (OUE-GT-001 and OUE-GT-002). The boring logs are presented in Appendix A. Geotechnical laboratory testing results are provided in Appendix B. The generalized subsurface conditions and cross sections through the dam are shown on Figure 4.

2.2.1 Site Geology

Fort Bragg is located along the northern California coastline within the Coast Range geomorphic province. The regional geology consists of complexly folded, faulted, sheared, and altered bedrock. The bedrock of the region is the Franciscan Complex (Complex) of Cretaceous to Tertiary (late Eocene) age (40 to 70 million years old). The Complex comprises a variety of rock types. In the north coast region, the Complex is divided into two units, the Coastal Belt and the Melange. In Mendocino County, the Melange lies inland and is an older portion of the Complex, ranging in age from the Upper Jurassic to the late Cretaceous. The Coastal Belt consists predominantly of greywacke sandstone and shale.

The Coast Range geomorphic province formed at the boundary between the North American and Pacific Plates. The contact between the North American and Pacific Plates is currently the San Andreas Fault Zone and subsidiary faults. Relative to the project site, the San Andreas Fault is located offshore about 6.3 miles (10.2 km; Figure 5). The Coastal Belt has undergone weak to intensive deformation, which has included folding, uplifting, tilting, and overturning.

Other geologic units present in Fort Bragg and the vicinity include surface geologic units containing deposits of beach and dune sands, alluvium, and marine terrace deposits. The most important of these at the site are the marine terrace deposits of the Pleistocene age, which cut bedrock surfaces along the coast and form much of the coastal bluff material overlying bedrock. The marine terrace deposits are massive, semiconsolidated clay, silt, sand, and gravel, ranging from 1 to 140 feet thick.

The site is underlain by Quaternary (less than 1.5 million years old) terrace sediments deposited on wave-cut surfaces parallel to the coast (BCI, 2006). The terrace was created during the Pleistocene Epoch when sea-level fluctuations caused by glaciation created a series of terraces cut into the Franciscan bedrock by wave action (BACE Geotechnical, 2004). The terrace deposits comprise poorly to moderately consolidated marine silts, sands, and gravels and are overlain by topsoil and in some areas by fill. The terrace soils are underlain by Tertiary-Cretaceous marine sediments (approximately 65 million years old) of the Coastal Belt Franciscan Formation, comprising well-consolidated sandstone, shale, and conglomerate.

2.2.2 Generalized Subsurface Soil Conditions

The following geologic units were encountered in the four borings performed for the seismic assessment:

- **Very loose to loose silty sand fill:** The uppermost soil unit within the dam appears to consist of approximately 12 to 17 feet of fill. The fill consists predominantly of very loose to loose, gray to brown. silty sand with various amounts of gravel. Interbedded layers of silt and gravel were encountered in Boring OUE-GT-003. The fill also contains some debris including wood debris.

- **Loose to medium dense silty sand:** This soil unit was generally encountered below the fill and is believed to be part of the marine terrace deposit described above in the site geology section of this report. The marine deposit encountered in the borings is approximately 5 to 13 feet thick and its density appears to increase with depth.

- **Hard silt:** This soil unit may be part of the marine deposit and consists predominantly of hard, non-plastic silt with an interbedded, very dense sand layer. This soil was only encountered in Boring OUE-GT-003 and is approximately 9 feet thick.

- **Bedrock:** Bedrock of the Franciscan formation was encountered in each of the borings at depths ranging from 20 to 30 feet below ground surface. The bedrock generally consists of sandstone. In Boring OUE-GT-002, a relatively thin layer of shale was encountered overlying the sandstone. The sandstone is generally weathered and sometimes highly decomposed and highly fractured near the top of the formation. The quality of the rock generally improves with depth.

More detailed information regarding lithology within the dam is provided on the boring logs in Appendix A. The generalized stratigraphy is shown on the cross sections presented on Figure 4.

In addition to the geologic units above, exploration data from locations within the pond indicate that the native marine deposits are generally overlain by approximately 3 to 4 feet of very soft sediment consisting of organic clayey silt and clayey silt (AME, 2006).

2.3 Groundwater

At the time of drilling, groundwater was encountered at depths ranging from 8 to 10 feet below ground surface (between approximate elevations 34.5 and 36.5 feet [NAVD88]). It should be noted that water levels were measured at the times and under the conditions stated on the boring logs. Fluctuations in the groundwater conditions may occur due to variations in rainfall, temperature, seasons, and other factors. The water level in the pond was at elevation 39.4 feet (NAVD88) at the time of the survey (February 17, 2009).

2.4 Sediments

Previous investigations (ARCADIS 2009, 2010) have characterized pond sediments. The results of these assessment showed elevated concentrations of selected metals (lead, zinc, and sometimes arsenic, and copper) and dioxins/furans in surface and subsurface sediment throughout the Mill Pond, as well as elevated concentrations of petroleum hydrocarbons, largely in the eastern portion of the Mill pond.

3. Geotechnical Engineering Analyses

ARCADIS performed various geotechnical engineering analyses to assess the seismic stability of the dam. The analyses and results are outlined below. The stability assessment was performed in general accordance with the approach outlined in the DSOD guidance documents for seismic stability analyses (Babbitt & Verigin, 1996).

According to DSOD guidance, the dam needs to be capable of withstanding the MCE. ARCADIS characterized the seismic sources relevant to the site, determined the seismic source that represents the MCE, and estimated the associated seismic hazard at the site. The seismic hazard is expressed in terms of ground accelerations. ARCADIS estimated the target response spectrum (spectral accelerations) for the MCE for bedrock outcrop. This is done using published attenuation relationships that take into account factors such as earthquake magnitude, distance, source type, and other parameters. Because the target response spectrum was determined for rock outcrop, the influence of soil overlying the bedrock had to be analyzed. This was done by performing a ground response analysis, which is essentially performed to calculate how the seismic waves travel through the subsurface soils. The ground response analysis can be used to calculate peak accelerations, cyclic stresses, acceleration time histories, and other parameters at various elevations within the soil profile. This analysis requires the use of appropriate acceleration time histories (also referred to as ground motions). Ground motions recorded during actual earthquake events can be used if they meet certain criteria. These ground motions can be retrieved from a number of different strong ground motion databases. ARCADIS used recorded ground motions, but modified them such that they matched the target response spectrum that was previously determined for the MCE. This process is referred to as spectral matching. The results from the ground response analysis were then used in subsequent analyses such as the liquefaction analyses and the slope stability analyses.

The soil liquefaction analysis is generally performed in two steps. First, an assessment is made as to whether the site soils are susceptible to liquefaction. Loose, saturated granular soils are particularly susceptible to liquefaction, but the liquefaction potential also depends on the origin and age of a soil deposit. The likelihood of liquefaction decreases with the age of the deposit. The second step consists of an analysis to determine whether the level of seismic shaking is sufficient to trigger liquefaction. The cyclic stresses were calculated in the ground response analysis. The resistance of the soils to liquefaction is largely a function of density, which was estimated based on the results of the SPT performed during drilling. During liquefaction, soils typically lose a large portion of their strength. For soils that were identified to liquefy during the MCE, residual shear strengths were estimated based on the initial density of the soil in conjunction with published correlations.

The results of the liquefaction analyses and the ground response analyses were then used to perform slope stability analyses. For the slope stability analyses, two-dimensional computer models of the dam were generated and shear strength and other parameters were assigned based on the results of the subsurface investigation and laboratory testing. ARCADIS performed analyses for static and seismic conditions. Slopes with low factors of safety for static conditions are generally not likely to perform well during seismic conditions. ARCADIS performed a few deformation analyses for pre-liquefaction conditions using ground motions calculated in the ground response analysis. However, the liquefaction analyses indicated that liquefaction will be a wide-spread problem during the MCE, and deformations of the dam associated with liquefaction are estimated to be significantly larger than those that occur without liquefaction. It is extremely difficult to estimate deformations that occur due to liquefaction with some accuracy without performing sophisticated and expensive modeling. ARCADIS performed post-earthquake slope stability analyses that take into account reduced shear strength as a result of liquefaction under static conditions. Factors of safety of less than one indicate that flow liquefaction will occur. Flow liquefaction is typically associated with very large deformations. No attempt was made to quantify the deformations in more detail given the results from the post-earthquake slope stability analyses. Soil liquefaction should generally be mitigated for safe performance of the dam.

3.1 Seismic Source and Seismic Hazard Characterization

The following site coordinates were used for the seismic source characterization and seismic hazard analysis:

Latitude: 39.441 ° N

Longitude: 123.813° W

3.1.1 Faulting

According to the California Geologic Survey (CGS, 1999), the site is not located in an Alquist-Priolo zone, and no major active fault crosses the site. There are no known Holocene faults within 200 feet of the site. The site is located about 6.3 miles (10.2 km) from the San Andreas Fault. Seismic activity on the San Andreas Fault could cause significant ground shaking at the site. There are several other seismic sources in the region as shown on Figure 5. Significant faults within 100 km (62 miles) of the site and corresponding fault parameters are shown in Table 1.

3.1.2 Seismic Source Model

The U.S. Geological Survey 2008 Seismic Hazard Map source model (Petersen et al., 2008) was used in the EZ-FRISK software package (Risk Engineering, 2009) to identify significant seismic sources within 100 km (62 miles) of the site. The seismic sources identified for this study are shown in Table 1.

Table 1. Significant Seismic Sources within 100 km of Fort Bragg

Seismic Source Name	Source Type	Moment Magnitude	Closest Distance[1] (km)	Median PGA[2] (g)
San Andreas	strike slip	8.0	10.19	0.451
Maacama-Garberville stri	ke slip	7.4	34.63	0.137
Deep Gridded	intraslab/subduction	7.2	50.00	0.196
Bartlett Springs	strike slip	7.3	61.46	0.073
Collayomi strike	slip	6.5	95.83	0.031

Notes:

[1] Closest distance to potential rupture.

[2] Weighted mean of three attenuation relationships for rock outcrop (refer to Attenuation Relationship section).

g = acceleration of gravity

km = kilometer(s)

PGA = peak ground acceleration

3.1.3 Historical Seismicity

The site is located in a seismically active area. Two significant historical earthquake events occurred within the last 111 years that caused significant seismic shaking at the site. The most significant event was the 1906 Great San Francisco earthquake. Although the epicenter was not located within 100 km of the site, the fault rupture along the San Andreas Fault continued up north to the Mendocino Triple Junction, which is located in the Cape Mendocino Area approximately 128.7 km (80 miles) to the north-northwest of Fort Bragg (refer to Figure 5). The Mendocino Triple Junction is a highly active seismic area where the San Andreas Fault, the Mendocino Fracture Zone, and the Cascadia Subduction Zone meet. Another significant earthquake event occurred in 1898. The epicenter of this magnitude 6.5 (M6.5) event was approximately 37 km (22 miles) south of Fort Bragg. More recent earthquakes include a January 2010 magnitude 6.5 event within the Mendocino Tripe Junction, approximately 150 kilometers to the northwest of the site. Figure 5 shows earthquake events with magnitudes of 5.5 or larger that occurred between 1800 and 2010. The data depicted on Figure 5 was obtained from CGS (2010) and the Northern California Earthquake Data Center (NCEDC, 2010.

3.2 Seismic Hazard Analysis

According to DSOD guidance (Babbitt & Verigin, 1996), the dam needs to be capable of withstanding the MCE. The earthquake event associated with the MCE is selected deterministically rather than using a probabilistic approach. ARCADIS evaluated seismic sources and associated seismic hazards within 100 km

(62 miles) of the site and have defined the MCE as the deterministic median (50th percentile) ground motions resulting from the maximum earthquake on the San Andreas Fault, which is the closest significant fault for this project. The appropriate statistical level of acceleration for deterministic seismic hazard analyses was determined using the DSOD Consequence-Hazard Matrix (Fraser and Howard, 2002). Depending on the total class weight (TCW) of the dam and slip rate of the fault associated with the MCE, accelerations at the site calculated based on a number of attenuation relationships are either calculated at the 50th or 84th percentile statistical level. The TCW is based on the damage potential related to capacity of the dam, its height, estimated number of people that would be placed in peril and would require evacuation in anticipation of dam failure, and the potential downstream property damage (Calzascia and Fitzpatrick, 1989). The damage potential for each of these factors is "low" and the overall TCW for this dam is zero, which is the lowest possible rating. Accordingly, the dam was analyzed for the 50th percentile statistical level.

3.2.1 Attenuation Relationships

The site-specific seismic hazard in terms of spectral accelerations is estimated using attenuation relationships that account for type of seismic source, earthquake magnitude, distance between the site and the earthquake epicenter, and local subsurface conditions. DSOD (Fraser and Howard, 2002) recommends the use of attenuation relationships for shallow crustal earthquake sources (Abrahamson and Silva, 1997; Boore et al., 1997; and Sadigh et al., 1997) and for subduction zone events (Youngs et al., 1997). Although newer attenuation relationships are available and generally result in smaller hazards at the site (i.e., the 2008 Next Generation Attenuation Relationships [PEER, 2009]), ARCADIS used the attenuation relationships recommended in the 2002 DSOD guidance document (Fraser and Howard, 2002). DSOD have not adopted the newer attenuation relationships.

3.2.2 Deterministic Seismic Hazard Procedure

Using the attenuation relationships introduced above and maximum moment magnitudes, peak ground accelerations (PGA) for rock outcrop conditions were calculated for each of the seismic sources identified in Table 1. Table 1 shows that an M8.0 earthquake on the San Andreas Fault produces the largest PGA (0.451 g). Subsequently, a theoretical rock outcrop response spectrum was calculated for the MCE.

3.2.3 Theoretical Outcrop Response Spectra

A theoretical rock outcrop response spectrum was calculated for the MCE (i.e., the event represented by an M8.0 event on the San Andreas Fault) using the computer program EZ-FRISK (Risk Engineering, 2009), assuming a shear wave velocity of 760 meters per second (2,500 feet per second). The rock outcrop spectrum is presented on Figure 6. For comparison, ARCADIS also performed a probabilistic seismic hazard analysis and calculated the response spectrum for the 475-year recurrence interval (10 percent probability of exceedance in 50 years). The response spectrum for the MCE compares well with the 475-year event, which indicates that a similar recurrence interval can be expected for the MCE.

3.3 Ground Motions

Input acceleration time histories (input ground motions) were developed for the ground response analysis. This section describes the input ground motion development process.

3.3.1 Selection of Acceleration Time Histories

ARCADIS performed a database search to find adequate acceleration time histories suitable for use for this project location. The ground motions were obtained from the Pacific Earthquake Engineering Research Center Strong Ground Motion Database (PEER, 2009). The ground motions were selected mainly based on earthquake magnitude, peak ground acceleration, distance, seismic source type, and subsurface conditions at the station location. Because the ground motions were subsequently spectrally matched, selection factors including seismic source type, subsurface conditions, and distance are not as important as the moment magnitude and peak acceleration. Therefore, the selection process focused primarily on magnitude and peak ground acceleration. The following acceleration time histories were selected:

- Chi-Chi, Taiwan, 1999, Station TCU084-N, M7.6
- Kocaeli, Turkey, 1999, Station Sakarya, 90-degree component, M7.4
- Kobe, Japan, 1995, Station KJMA, 90-degree component, M6.9
- Landers, California, 1992, Station 23 Coolwater (TR), M7.4

3.3.2 Spectrally Matched Outcrop Ground Motions

The acceleration time histories selected from the PEER database were modified in the time domain to make them compatible with the target spectrum calculated for the MCE. The modifications were performed using the spectral matching module in the EZ-FRISK computer program (Risk Engineering, 2009). EZ-FRISK uses the spectral matching routine RspMatch developed by Abrahamson (1998). The ground motions were scaled to the PGA of the MCE after each iteration. Following the spectral matching, the ground motions were baseline corrected using the computer software SeismoSignal (SeismoSoft, 2009). The response spectra of the matched outcrop ground motions are shown on Figure 7, along with the target spectrum for the MCE. The input ground motions were named chichi1, kocaeli1, kobe1, and landers1.

3.4 Ground Response Analyses

The site-specific ground response was analyzed using the one-dimensional, equivalent linear computer software SHAKE2000 (Ordonez, 2008). SHAKE2000 was used to calculate ground accelerations and cyclic stresses within the soil profile and to perform soil liquefaction analyses. Additionally, SHAKE2000 was used to calculate acceleration time histories within the soil profile that were used for subsequent slope deformation analyses (Newmark analyses [Newmark, 1965]) also performed in SHAKE2000.

3.4.1 Ground Response Model

Ground response analyses were performed for each of the four boring locations (OUE-GT-001 through OUE-GT-004). Shear wave velocities were estimated based on a number of published correlations. Engineering judgment was used to select appropriate shear wave velocities. The shear wave velocity profiles are shown on Figure 8. The following damping and modulus reduction curves were used:

- Sand: EPRI (1993)
- Cohesive Soil: Vucetic & Dobry (1991)
- Rock: Silva et al. (1997)

3.4.2 Peak Accelerations

Peak accelerations were calculated at various depths for each of the input ground motions. The results of these analyses are presented on Figures 9 through 12. On average no amplification was observed in terms of free field accelerations. Some attenuation of acceleration was observed for boring location OUE-GT-001. A relatively large amount of attenuation was observed at boring location OUE-GT-004.

3.5 Soil Liquefaction Analysis

The soils overlying the bedrock consist predominantly of very loose to loose sands (refer to Figure 4), which are generally considered susceptible to liquefaction. Only the saturated portions of these soils (i.e., the soils below the water table) would liquefy. The site soils consist of fill and marine terrace deposits of the Pleistocene age. Based on its age, the probability of liquefaction to occur within the marine deposit is low (Idriss and Boulanger, 2008).

ARCADIS assessed whether the level of shaking during the MCE would trigger liquefaction in the soils susceptible to liquefaction. This analysis was performed in general accordance with methods presented in Youd et al. (2001). Cyclic stress and the cyclic stress ratio were calculated as part of the ground response analysis in SHAKE2000. The cyclic resistance (cyclic resistance ratio) of the site soils depends on the density of the soils and their fines content. Soil density was assessed based on the SPT that was performed as part of the subsurface investigation. Several correction factors were applied to calculate corrected SPT blow counts. This included a correction for the efficiency of the automatic trip hammer that was used to perform the SPT. Energy measurements were taken during SPTs for three of the borings. The cyclic resistance ratio and factors of safety against liquefaction were calculated in SHAKE2000. The results of the trigger-liquefaction analyses are presented on Figure 13 through 16. Although the analysis indicates that much of the marine deposit would liquefy, the probability of liquefaction in that layer is considered low based on the age of the deposit as mentioned above. It was assumed that only the upper portion of the marine terrace layer would liquefy during the MCE. The probability of liquefaction in the marine terrace deposit should generally decrease with depth as the age and density of the material increases. The assumed extent of liquefaction within the dam is shown on Figure 17.

3.6 Slope Stability Analyses

ARCADIS used the computer program SLOPE/W (GeoSlope International, 2009) to perform numerical slope stability analyses. The program uses limit-equilibrium methods to calculate factors of safety against failure along specified slip surfaces. For the analyses presented herein, the Spencer method of slices was used to calculate factors of safety. Spencer's procedure is generally accepted as an accurate procedure that is applicable to virtually all slope geometries and soil profiles (Duncan & Wright, 2005). It satisfies moment equilibrium as well as horizontal and vertical force equilibriums. It is the simplest equilibrium procedure for computing the factor of safety. Static, post-earthquake, and Newmark deformation analyses were performed for three dam sections. These analyses are discussed in more detail below. The focus of the stability assessment was on relatively large and deep failures that might result in a release of water and potentially release of sediment. The results of the slope stability analyses are summarized on Figures 18 through 20.

3.6.1 Assumptions

3.6.1.1 Geometry

Generalized analysis cross sections were developed for sections B-B', D-D', and E-E' (refer to Figures 18 through 20) based on the survey and subsurface information collected for this study.

3.6.1.2 Soil Properties

The site soils are predominantly granular soils and, therefore, drained strength parameters were used for static loading and for Newmark deformation analyses that did not take into account reduction in soil strength due to liquefaction. Some cohesive soils exist locally, but these soils are generally competent soils that would have higher undrained strength than drained strength. Drained shear strength parameters were generally based on standard penetration test data (corrected blow counts) that were used in conjunction with engineering judgment. An effective cohesion of [redacted] was used for the silty sand. This was done because the existing steep slopes would otherwise experience surficial sliding even at relatively high friction angles. Because the slopes appear to be stable for static conditions, the strength that was based on SPT data was adjusted slightly by adding some cohesion that is provided by the fine material in the silty sands. The [redacted] cohesion value provided just enough strength to prevent surficial failures along the steep slopes. For drained conditions, the Mohr-Coulomb soil strength model was used in Slope/W (Lambe and Whitman, 1969). The residual strength of liquefied soil was modeled as a function of overburden stress (Idriss and Boulanger, 2008). The parameters used in the analyses are provided on Figures 18 through 20.

3.6.1.3 Rock Strength

Field observations during rock coring and laboratory strength testing were used to estimate the strength of the rock mass using the Hoek-Brown strength model (Wyllie & Mah, 2004). When this strength was initially used in the computer model, it was apparent that the in-situ strength of the rock must be higher than the estimated Hoek-Brown strength for the rock mass. While the dam appears to be relatively stable under static conditions, the model indicated failures with very low factors of safety for static conditions (i.e., factors of safety were well below one). It is possible that the intense fracturing that was observed during coring was at least in part a result of the coring process and that this led to false assumptions for the rock mass strength model. Additionally, the visible face of the dam is not intensely fractured. It was concluded that deep failures through the rock are not likely to occur. Some block-type failures may be possible during strong seismic shaking, but these would likely not result in a catastrophic collapse of the dam. Liquefaction within the soil portion of the dam is more likely to cause significant damage as a result of strong seismic shaking. This is discussed in more detail below. For the slope stability model it was, therefore, assumed that the rock is impenetrable (i.e., the computer model was set up such that failures through the rock would not occur). Only for two slope stability analyses for Section B-B' was a Mohr-Coulomb strength assigned and failure surfaces through the rock evaluated for static conditions.

3.6.2 Static Analysis

Static analyses were performed to assess the stability of the existing dam under static conditions. Portions of the dam that do not achieve acceptable static factors of safety will likely experience relatively large deformations under seismic conditions. Additional deformation would occur due to liquefaction. Although the DSOD guidance documents do not recommend a target factor of safety for static conditions, a typical target factor of safety for the design of slopes is 1.5 based on standard engineering practices.

The static stability analyses indicate that portions of the dam are only marginally stable under static conditions. These portions include mainly the steep soil slopes along Soldier Bay. Stability generally increases with the depth of the failure surface (i.e., the surface within the soil mass along which sliding/displacement would occur). Factors of safety for shallow failure surfaces are very low (approximately 1.0 to 1.2). While these slopes appear to be stable at the present time, the low factors of safety under static conditions are an indication that shallow failures are likely during strong seismic shaking. Even some of the deeper failure surfaces have relatively low factors of safety in the range of 1.2 to 1.3. The crib wall shown on Section B-B' (Figure 4) was not modeled as reinforcement and it is unclear what the extent of the logs is within the dam cross section. The crib wall probably provides some stability, but static stability is likely still marginal, particularly in other areas (e.g., Section A-A'). In areas near the spill way (Sections C-C' and D-D'), the deeper failure surfaces appear to be more stable with factors of safety of 1.5 and greater. The static stability of areas to the east (e.g., Section E-E') are generally more stable. Section F-F' was not evaluated in detail. This section contains a wall that provides some stability to the slope. However, the existing concrete and steel sheet pile walls seem to be in a somewhat decrepit condition and may experience distress during seismic loading.

3.6.3 Post-Earthquake Analysis

The liquefaction analyses performed as part of this study indicate that a large portion of the saturated soil will liquefy during the MCE. The assumed extent of liquefaction is presented on Figure 16. Post-earthquake limit-equilibrium slope stability analyses were performed using Slope/W to evaluate the effects of liquefaction. Residual shear strengths were assigned to the liquefied soil for these analyses. The analyses indicate that flow failures will occur during the MCE in Sections A-A' through D-D' with very low factors of safety for large failures. Flow failure generally occurs when the shear stress required for static equilibrium of a soil mass (the static shear stress) is greater than the shear strength of the soil in its liquefied state (Kramer, 1996). Flow liquefaction failures are often characterized by the large distance over which the liquefied materials often move. Section E-E' has factor of safety of about 1.1 to 1.2. It is believed that if some deformation occurs in this section due to liquefaction, it would not be significant enough to constitute a catastrophic failure. However, deformations in Section A-A' to D-D' (i.e., the portion of the dam along Soldier Bay) will likely experience large deformations during the MCE. Along portions of the dam with a significant rock face (Section C-C' and D-D'), soil will fall off the rock and onto the beach below. This will prevent a buttressing effect that usually limits deformation to some extent by rearranging the soil mass into a more stable state. As a result, flow failures could potentially be large enough to constitute a release of water from the dam and possibly sediment toward Soldier Bay. As mentioned above, Section F-F' was not modeled in Slope/W. Liquefaction will likely occur in this area as well. The concrete wall and concrete slabs in front of the wall will likely limit large deformations of the embankment, unless the wall is too decrepit to withstand the lateral loading during seismic shaking. This assessment was not part of this study and would require inspection of the wall by a structural engineer. Ideally, this wall would be replaced by a properly designed new wall or earthen buttress.

3.6.4 Deformation Analysis

Only a limited number of deformation analyses were performed because the stability issues associated with soil liquefaction and the consequential flow failure govern the stability of the dam. Deformations associated with flow failures are extremely difficult to assess and would require sophisticated modeling techniques that do not seem to be warranted by the size of the dam. As mentioned above, it is expected that flow failure deformation will result in significant damage and changes to the geometry of the dam. Displacement of the soil mass due to flow failure is expected to be on the order of several feet to 10s of feet. The Newmark analysis is not suitable for estimating deformation due to flow failure, because it estimates large deformation during earthquakes only..

While the effects of liquefaction will likely occur during the later stages or even the end of seismic shaking, the slopes may experience some deformation due to seismic shaking before the onset of liquefaction. In simplified deformation analyses, it is assumed that each time the acceleration within the slope exceeds the yield acceleration of the slope, some movement occurs. This is simulated in a Newmark analysis that uses the acceleration time histories that were calculated in the ground response analyses at several different depths within the soil profile Yield accelerations were calculated in the slope stability model by varying the seismic coefficient until the factor of safety equaled one. The Newmark analyses were performed in SHAKE2000 using a method developed by Houston et al. (1987). In this analysis, the accelerations that exceed the yield acceleration are double integrated in the time domain to calculate the total deformation of the slope along the failure surface, at the end of seismic shaking. This does not include any deformations that will occur due to liquefaction and associated flow failure.

Calculated deformations for deep failures that might be deep enough to cause overtopping of the dam were negligible. More shallow, but substantial failures would experience about 25 to 30 inches and 5 to 10 inches for Sections B-B' and D-D', respectively. The failure surfaces associated with these deformations are shown on Figures 18 and 19. The analysis output from SHAKE2000 is presented on Figures 21 through 24. The deformations calculated in SHAKE2000 would not cause a major change of the dam geometry and would not result in releases, but might necessitate some repair.

As mentioned above, flow failure during the MCE will likely cause deformation large enough for some water to spill over the damaged dam.

3.7 North Wall Assessment

Additional slope stability analyses were carried out to evaluate the stability of the north wall of the Log Pond Dam in the proximity of Pond 7. The slope of the wall at this location is very steep and shows signs of past and recent movements such as slumps and shallow localized failures. The northern portions of the dam also exhibits seepage anhd Georgia-Pacific was requested to investigate the source of the seepage further..

Standing water has accumulated within low areas along embankment toe. Supply pipes to and from the pond perforate the embankment within this area, and neither the condition of the pipes nor the status of encasement of the pipes is known. These pipes may contribute to the seepage. Remedial activities will be conducted in and around Pond 7 in the future, putting workers and equipment in the proximity of this area. Therefore, a focused stability analysis is deemed necessary in this area.

Limit equilibrium slope stability analyses were carried out on cross section G-G', whose location is shown in Figure 3. The soil stratigraphy was modeled using boring logs data from OUE-GT-002 and OUE-GT-003.

Slope stability limit equilibrium analyses on circular slip surfaces were carried out using the software SLOPE/W 2007 (Geo-Slope International Ltd., Alberta, Canada). The Spencer's method (Spencer 1967), which is a rigorous analysis approach satisfying both force and momentum equilibria, was used to calculate the factor of safety. SLOPE/W software was set to automatically search through thousands of potential slip surfaces to identify the most critical one, i.e. the one with minimum factor of safety. Soil parameters and groundwater profiles were modeled consistently with the analyses carried out on the LPD cross-sections D-D' and E-E', as discussed in section 2.10.2 of this report. The soil strength was modeled using effective stress strength parameters (a friction angle and a cohesion intercept). The geotechnical properties of the soils are listed in Table 2. The sandstone bedrock layer was modeled with a very high (impenetrable) strength.

Table 2 – Material Properties for Slope Stability Analyses

Material	Total Unit Weight (pcf)	Strength	Reference
Silty, Gravelly, Sand	[redacted]		
Upper Silty Sand			
Intermediate Silty Sand			
Lower Silty Sand			
Sandstone bedrock			

A seismic evaluation of the slope was also carried out using post-earthquake soil shear strength properties. The post-earthquake analyses were performed assuming that either the upper silty sand layer would liquefy, the upper and intermediate silty sand layers would liquefy, or that all three layers of silty sand layers would liquefy. Residual shear strengths were assigned to the liquefied native soil for these analyses consistently with the analyses discussed in section 2.10.3. Soil parameters used in the post-earthquake analyses are listed in Table 2.

Table 3 – Material Properties for Post-earthquake Slope Stability Analyses

Material	Total Unit Weight (pcf)	Strength	Reference
Silty, Gravelly, Sand	[redacted]		
Upper Silty Sand	[redacted]		
Intermediate Silty Sand	[redacted]		
Lower Silty Sand	[redacted]		
Sandstone bedrock	[redacted]		

The slope stability analysis results are presented in Appendix C. A factor of safety smaller than 1 was computed for the static analysis. The factor of safety for the post-earthquake analyses was significantly smaller than 1 and decreased with increasing depth of liquefied soils.

4. Geotechnical Engineering Conclusions and Recommendations

This geotechnical engineering study was performed using DSOD guidelines for the assessment of the seismic stability of dams. DSOD guidance requires the identification of the MCE at the site, which is determined by performing a deterministic seismic hazard analysis (DSHA). As part of the DSHA, seismic sources in the vicinity of the site are considered without paying significant attention to the probability of occurrence of earthquakes associated with these sources. Therefore, it is unclear as to what the risk or level of conservatism is of using this approach without using a probabilistic approach in addition to the deterministic approach. ARCADIS performed probabilistic seismic hazard analyses (PSHA) in addition to the DSHA to estimate the level of risk. The MCE was identified as an event with a moment M8.0 on the San Andreas Fault, which is approximately 10 kilometers from the site. Based on the PSHA, the seismic hazard associated with the MCE is approximately equivalent to that of an earthquake event with a return period of 475 years (i.e., a 10 percent probability of exceedance in 50 years). Although the PSHA approach is not required by DSOD, it provides a better idea as to what the level of risk actually is.

Based on the liquefaction analyses that took into account the level of shaking and cyclic stresses in the subsurface soils during the MCE, liquefaction will occur in significant portions of the dam cross section. Soil liquefaction during the MCE is expected to result in flow failures and associated large deformations of the dam. Flow failures will likely be concentrated along the alignment of the dam adjacent to Soldier Bay and in the north wall slope adjacent to Pond 7. The embankment to the east appears to be more stable, but would likely experience some limited deformation due to liquefaction as well. Exact deformations due to flow failure are extremely difficult to estimate and even sophisticated modeling may not provide sufficient confidence in the results. However, deformations associated with flow failure are expected to be on the order of several feet to 10s of feet and the release of some water from the pond toward Soldier Bay appears to be a likely outcome of the occurrence of the MCE. Based on engineering judgment, it generally seems unlikely that sediment would spill over the damaged dam since the deformed dam would still provide some containment. However, in the area of the crib wall and the spillway, where the flow failure can conceivably push the dam to the beach, the containment may be compromised and sediment release may occur. Additional, simple analyses were performed to put the level of risk associated with the MCE in perspective. These analyses are not discussed in detail in this report and are generally not required by DSOD guidelines. The simple analyses consisted of using attenuation relationships to estimate ground accelerations (DSHA approach) and then using these accelerations to perform simplified liquefaction analyses (Youd et al., 2001 – rough estimates of amplification within the soil were used in conjunction with a simplified method of estimating cyclic stresses in the soil profile). Based on these analyses, M4.0 and M5.0 earthquakes on the San Andreas Fault would not result in liquefaction within the dam. Additionally, the USGS seismic hazard maps were used to estimate the seismic hazard associated with an event with a return period of 108 years (PSHA approach). A subsequent simplified liquefaction analysis indicated that liquefaction would occur during this event. This indicates that events smaller than the MCE may trigger liquefaction and substantial deformation within the dam. It should be noted that the ground response analyses performed for the MCE to calculate cyclic stress for the liquefaction assessment are considered a more sophisticated and generally better suited approach (Youd et al., 2001). The selection of different methods for calculating cyclic stress can influence the outcome of the analysis.

Generally, the occurrence of flow failures as a result of strong seismic shaking should be avoided in dams. The large deformations and nearly catastrophic failure experienced by the Lower San Fernando Dam during the 1971 San Fernando earthquake was one of the first examples of the large deformations that can occur as a result of flow failures. While the consequences of a failure of the dam would be less severe than breach of the San Fernando dam, the high probability of wide-spread liquefaction within the dam cross section will likely be considered to be a deficiency by DSOD that will require some form of mitigation.

4.1 Recommendations

This report concludes that the dam wall is built on soils that are prone to liquefaction, and during a Maximum Credible Earthquake event, these soils could undergo significant vertical and lateral deformation, leading to extensive damage to the dam wall, potentially causing release of water from the Mill Pond. It is not expected that sediment retained in the Mill Pond would be released as a result of such an event, with the exception of the cribwall and spillway areas, where sediment release may occur. Based on the above investigations and the various DSOD inspections, there are four main deficiencies associated with the current Mill Pond dam wall, listed as follows:

1. Presence of liquefiable soils

   Liquefiable soils represent a large risk to the overall serviceability of the dam. During ground shaking, the liquefiable soil loses its strength, becomes essentially a flowable liquid with no internal shearing strength. The earthen berm of the dam loses its foundation, and under the horizontal load of the retained water and sediment, the berm (having no foundation) will exhibit large lateral and vertical deformation. In lay terms, the berm will collapse and can potentially lead to the release of the retained water. The berm will provide some ongoing retention, because of its self-weight, but on the western portion of the dam, where the berm is built on the edge of the coastal cliff, large portions of the berm are expected to fall onto the beach. It is likely that the release of water to the beach will occur, and the release of some sediment to the beach cannot be ruled out.

2. Cribwall

   DSOD inspections discuss the need to repair the cribwall on the western (ocean facing) face of the dam as a requirement for ongoing serviceability of the dam (Item 1, DSOD 2009). The cribwall is a relatively small (about 80 feet wide, 12 to 15 feet high) structure made out of redwood logs forming a crib filled most likely with locally available silty materials. The cribwall is on the western face of the wall, directly exposed to the ocean and built on native sandstone (coastal cliff). It appears that the cribwall has lost fill from inside and underneath the structure. Most of the damage is attributed primarily to the wave action on the sandstone coastal cliff. The entire California coastline exhibits erosional processes – the ongoing retreat of coastline due to the constant wave action exhibited by the ocean. The wave action is particularly strong during winter storms. There is abundant visible evidence at the site (and elsewhere in Mendocino County) of the coastal retreat process. According to BACE Geotechnical (2004), the erosion proceeds with 1.5 to 3 inches per annum. ARCADIS opines that over the 120-year life of the dam, the coastal cliff has undergone thousands of winter storms that undermined the sandstone foundation of the cribwall. This could have also led to loss of fill, which in turn, can increase seepage from the dam, further exacerbating the loss of fill in the crib.

While it is technically very difficult to numerically predict the stability of a cribwall, the overall appearance of the wall indicates a low safety margin against failure. There is little doubt that the erosional processes will finally undermine the cribwall causing its failure. The consequence of this failure is relatively limited, but it will impact the coastal zone and could lead to some release of water and sediment.

3. Spillway

The spillway is constructed in two reinforced concrete portions. The low water discharge level is set at an elevation of 39 feet, and it releases water constantly. In lay terms, the spillway works and maintains a steady water level at 39 feet. The spillway structure seems to have undergone structural damage, losing large portions of concrete at the beach. It appears that the spillway was built by placing a concrete layer over the native sandstone, but in time, the coastal erosional processes have washed away the foundation of the spillway and caused its undermining and partial collapse. Similarly, to the cribwall, it is expected that in time the wave action will cause more damage to the spillway. The consequence of a spillway failure would seriously impact the functioning of the dam and would likely cause some release of water and sediment to the beach. DSOD identified this deficiency, Item 2 (Appendix D).

4. Right embankment (northern wall)

The right embankment is built on liquefiable soils. Moreover, this part of the berm (about 15-foot height) has been reworked in the past when various structures were built in the Powerhouse area. It was necessary to provide a footprint for various structures (Bee-Hive Burner and sumps) and, therefore, the sawmill cut into the dam wall and by building a concrete retaining wall, the horizontal area was created to house these structures. There is no documentation regarding the structural details of the vertical retaining wall. Visual inspection during the 2006 demolition activities suggested that the retaining wall was of limited structural capacity, some parts were missing and eroding.

Therefore, demolition was terminated at least 20 feet from the retaining wall to prevent damage to the wall (and the dam) from vibrations caused by concrete-breaking equipment.

This portion of the dam wall represents a relatively small risk. Without clearly understanding the structural details of the retaining walls, it is difficult to ascertain the level of risk, but overall it seems that this portion of the wall is in marginal condition. The reservoir is leaking very likely along and around various pipes and outlets, which in the past conveyed water to the Powerhouse complex, as well as from groundwater under the dam. (DSOD identified this deficiency as Items 3 and 4; DSOD 2009).

At [redacted] a portion of the retaining wall is missing; also there are signs of slope movement but no large-scale failure is expected in this region (unless during a seismic event). Furthermore, if there is structural distress in this area, discharge of water and failed dam wall material would remain on Georgia-Pacific property. To reduce this risk, it is recommended to flatten the dam slope in this area, by filling and grading the downstream slope.

The northern dam wall and the flat area in front of it were previously overgrown by vegetation. This did not allow detailed inspection of the dam wall previously. After vegetation was removed in late 2009, a site visit was conducted and it was observed that the dam wall is leaking and there are several features protruding through the north wall. It is recommended that these features be individually assessed and appropriate repair measures be developed to reduce leaking.

The dam wall above Pond 7 seems oversteepened. Following the removal of vegetation, surficial sloughing seems widespread in this area. As discussed in Section2.11, the factor of safety of the dam wall is very low in this area. It is recommended that the downstream slope face in this area be regraded to a flatter slope face including the filling of Pond 7.

4.2 Summary

The liquefaction issue discussed herein is addressed by large-scale soil improvement only. Although, repairs to the cribwall and the spillway would greatly improve the serviceability of the wall, they will not address the presence of liquefiable soils under a large portion of the dam wall. Similarly, the restoration of the northern wall will reduce the deficiency along this portion of the wall, but it does not address the liquefaction potential. It is also important to note that the repairs to the cribwall and spillway while useful, would not indefinitely prolong the life of the dam. The wave action along the coast will continue and any temporary measures, such as placement of a rock berm and filling voids beneath the cribwall, would have to be inspected on a regular basis and repaired as needed.

As discussed previously, Georgia-Pacific has proposed a proactive phased approach to address these short and long-term needs for dam stabilization as well as associated remediation as independent projects. Georgia-Pacific is currently proposing a Phase I project referred to as the "Interim Corrective Action and Stormwater Rerouting Project," which is an interim project that will reduce the safety hazard associated with the dam by rerouting stormwater to the low-lying areas of the MPC and thus reduce the pressure on the dam. The Phase I project will also provide some stabilization of the north wall. The Phase II project, the Dam Stabilization/ Removal and Remediation Project, will result in complete removal of the spillway and cribwall and complete stabilization of the north wall.

5. References

Abrahamson, N.A. and Silva, W.J. 1997. Empirical response spectral attenuation relations for shallow crustal earthquakes. *Seismological Research Letters* 68(1)94-127.

Abrahamson, N.A. 1998. *Non-Stationary Spectral Matching Program RSPMATCH.* Pacific Gas & Electric Company, Internal Report, February 1998.

AME. 2006. *Data Transmittal Report, Georgia-Pacific California Wood Products Manufacturing Facility, 90 West Redwood Avenue, Fort Bragg, California.* Prepared for Georgia-Pacific Corporation. Project No. 16017.08. Acton•Mickelson•Environmental, Inc., August 14.

ARCADIS BBL. 2007. *Preliminary Site Investigation Work Plan, Operable Unit E - Ponds, Former Georgia-Pacific Wood Products Facility, Fort Bragg, California.* Prepared for Georgia-Pacific LLC. December.

ARCADIS. 2009. *Data Summary Report Operable Unit E Pond Sediment. Former Georgia-Pacific Wood Products Facility, 90 West Redwood Avenue, Fort Bragg, California.* Prepared for Georgia-Pacific, LLC. May.

ARCADIS. 2010. *Data Summary Report – Additional Investigation Pond 8 Sediment. Former Georgia-Pacific Wood Products Facility, 90 West Redwood Avenue, Fort Bragg, California.* Prepared for Georgia-Pacific, LLC. April.

BACE Geotechnical. 2004. *Engineering Geologic Reconnaissance Report, Planned Blufftop Access Trail, Georgia-Pacific Property, Fort Bragg, California*. Prepared for the City of Fort Bragg. Project No. 11886.1. BACE Geotechnical (a division of Brunsing Associates, Inc.). September.

Babbitt, D.H. and S.W. Verigin. 1996. *General Approach to Seismic Stability Analysis of Earth Embankment Dams.* California Division of Safety of Dams.

BCI. 2006. Letter from Mr. Rick Sowers, PE, CEG, Senior Project Manager, and Mr. Tom Blackburn, GE, Principal, to Mr. John Mattey, Acton•Mickelson•Environmental, Inc., re: Geotechnical Evaluation, Bearing Support for Heavy Equipment Loads, Georgia-Pacific Mill Site, Fort Bragg, California. BCI File 924.1. Blackburn Consulting, Inc. February.

Boore, D.M., W.B. Joyner, and T.E. Fumal. 1997. Equations for estimating horizontal response spectra and peak acceleration from Western North American earthquakes: A summary of recent work. *Seismological Research Letters* 68(1)128-153.

Calzascia E.R., and Fitzpatrick, J.A. 1989. *Hydrologic Analysis with California's Dam Safety Program.* Department of Water Resources – Department of Safety of Dams (DSOD).

CGS. 1999. *Regional Geologic Hazards and Mapping Program – Index to Earthquake Fault Zone Maps – Figure 4A*. Available online at the California Geologic Survey internet site: http://www.conservation.ca.gov/cgs/rghm/ap/Map_index/Pages/F4A.aspx.

CGS. 2010. *Historical Earthquake Data between 1800 and 1999.* State of California Department of Conservation – California Geological Survey (CGS). The data is available at http://www.conservation.ca.gov/cgs/rghm/quakes/Pages/Index.aspx.

DSOD, 2010. *Dams Within the Jurisdiction of the State of California.* California Department of Water Resources – Division of Safety of Dams (DSOD). The data is available at http://www.water.ca.gov/damsafety/damlisting/index.cfm.

Duncan, J.M. and S.G. Wright. 2005. *Soil Strength and Slope Stability.* John Wiley & Sons. Hoboken, New Jersey.

EPRI. 1993. *Guidelines for Site Specific Ground Motions.* Palo Alto, California, Electric Power Research Institute (EPRI), November, TR-102293.

Fraser, W.A. and J.K. Howard. 2002. *Guidelines for Use of the Consequence-Hazard Matrix and Selection of Ground Motion Parameters.* Department of Water Resources – Division of Safety of Dams. October 4, 2002.

GeoSlope International. 2009. *Slope/W*, GeoStudio 2007, Version 7.13, Build 4419.

Houston, S.L., W.N. Houston, and J.M. Padilla. 1987. Microcomputer-aided evaluation of earthquake-induced permanent slope displacements. *Microcomputer in Civil Engineering*, 207-222.

Idriss, I.M. and R.W. Boulanger. 2008. *Soil Liquefaction During Earthquakes.* Earthquake Engineering Research Institute.

Kramer, S.L. 1996. *Geotechnical Earthquake Engineering.* Prentice-Hall International Series in Civil Engineering and Engineering Mechanics. Prentice Hall, Upper Saddle River, New Jersey.

Lambe, T.W, and Whitman, R.V. 1969. *Soil Mechanics.* Series in Soil Engineering. John Whiley & Sons, New York.

Miller Pacific. 2005. *Geotechnical Evaluation – Georgia-Pacific Mill Pond and Dam – Fort Bragg, California.* Letter report to Stetson Engineers, dated March 21, 2005.

NCEDC. 2010. *Historical Earthquake Data between 1967 and 2010.* Northern California Earthquake Data Center (NCEDC). Data source: Northern California Seismic Network, U.S. Geological Survey, Menlo Park. The data is available at http://www.ncedc.org/ncedc/catalog-search.html.

Newmark, N.M. 1965. Effects of earthquakes on dams and embankments. *Geotechnique* 15(2): 139-160.

Ordonez, G.A. 2008. *SHAKE2000, Version 3.5.0.* GeoMotions LLC. January 2008.

PEER. 2009. *Strong Ground Motion Database.* Pacific Earthquake Engineering Research Center. The data is available online at: http://peer.berkeley.edu/products/strong_ground_motion_db.html.

Petersen, M.D., A.D. Frankel, S.C. Harmsen, C.S. Mueller, K.M. Haller, R.L. Wheeler, R.L. Wesson, Y. Zeng, O.S. Boyd, D.M. Perkins, N. Luco, E.H. Field, C.J. Wills, and K.S. Rukstales. 2008. *Documentation for the 2008 Update of the United States National Seismic Hazard Maps*: U.S. Geological Survey Open-File Report 2008–1128, 61 p.

Risk Engineering. 2009. *EZ-FRISK*, Version 7.31 – Build 005.

Sadigh, K., C.Y. Chang, J. Egan, F. Makdisi, and R. Youngs. 1997. Attenuation relationships for shallow crustal earthquakes based on California strong motion data. *Seismological Research Letters* 68(1):180-189.

SeismoSoft. 2009. Computer program *SeismoSignal* Version 3.2.0, Build 100.

Silva, W.J., N. Abrahamson, G. Toro, and C. Costantino. 1997. *Description and Validation of the Stochastic Ground Motion Model.* Report Submitted to Brookhaven National Laboratory, Associated Universities, Inc. Upton, New York 11973, Contract No. 770573.

Spencer, E. 1967. A Method of Analysis of Embankments Assuming Parallel Interslice Forces. *Geotechnique*, 17(1): 11-26.

Stetson Engineers Inc. 2005. *Mill Pond Technical Study – Technical Memorandum No. 1.* Prepared for Georgia-Pacific LLC, dated January 14, 2005.

TRC. 2004. *Phase I Environmental Site Assessment, Georgia-Pacific California Wood Products Manufacturing Division, 90 West Redwood Avenue, Fort Bragg, California.* Prepared for Georgia-

Pacific Corporation, 133 Peachtree Street, NE, Atlanta, Georgia. Project No. 41-041901. TRC Companies, Inc. March.

USGS. 2002. *Documentation for the 2002 Update of the National Seismic Hazard Maps.* United States Geological Survey (USGS), Open-File Report 02-420. Various online and java tools are available online at: http://earthquake.usgs.gov/research/hazmaps/

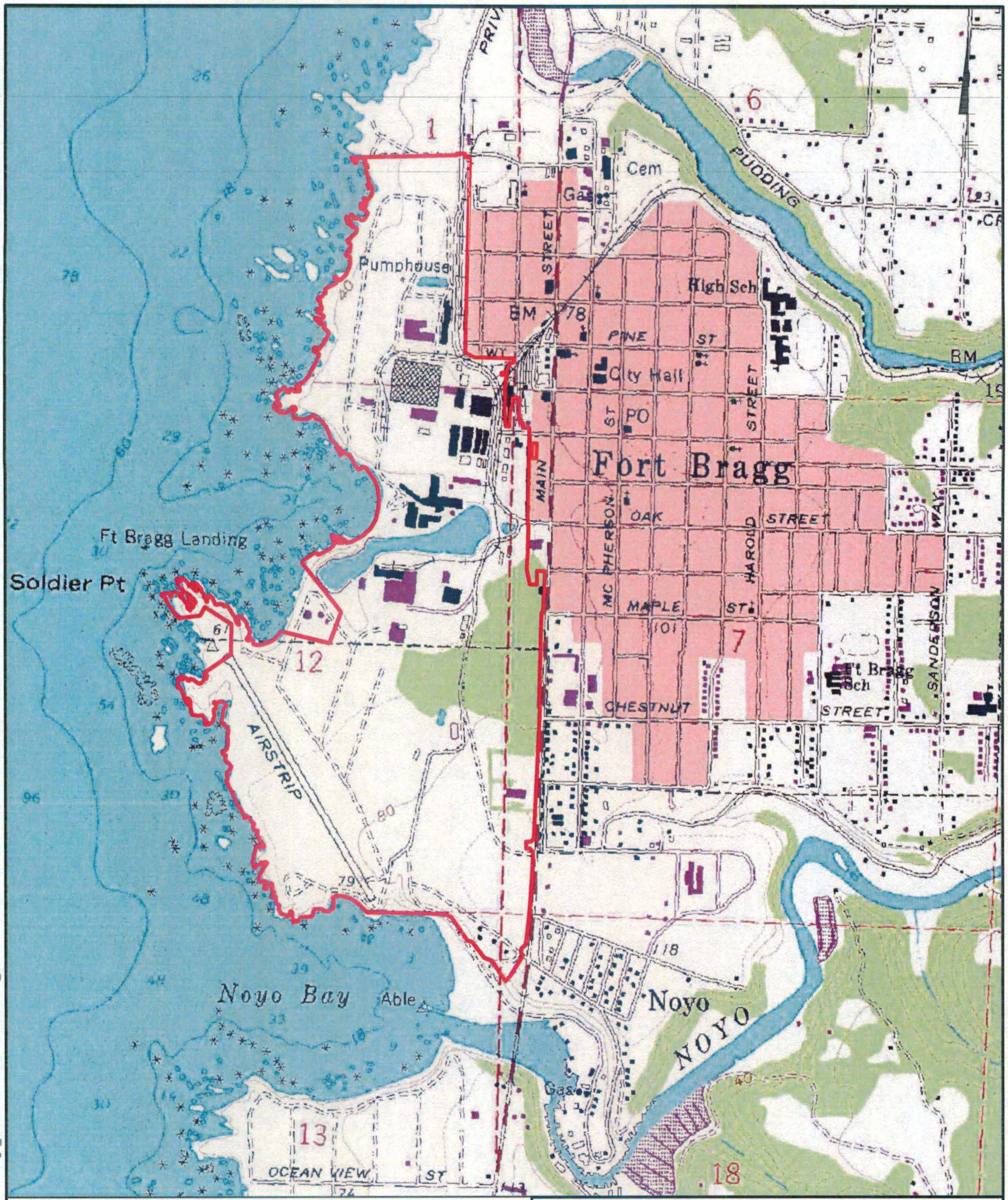
Vucetic, M. and R. Dobry. 1991. Effect of soil plasticity on cyclic response. *Journal of Geotechnical Engineering* 117(1):89-107. American Society of Civil Engineers (ASCE).

Wyllie, D.C. and C.W. Mah. 2004. *Rock Slope Engineering – Civil and Mining.* 4th Edition. Based on "Rock Slope Engineering" by Evert Hoek and John Bray.

Youd, T.L., I.M. Idriss, R.D. Andrus, I. Arango, G. Castro, J.T. Christian, R. Dobry, W.D.L. Finn, L.F. Harder, Jr., M.E. Hynes, K. Ishihara, J.P. Koester, S.S.C. Liao, W.F. Marcuson III, G.R. Martin, J.K. Mitchell, Y. Moriwaki, M.S. Power, P.K. Robertson, R.B. Seed, and K.H. Stokoe II. 2001. *Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils. Journal of Geotechnical and Geoenvironmental Engineering.* October.

Youngs, R.R., S.-J. Chiou, W.J. Silva, and J.R. Humphrey. 1997. Strong ground motion attenuation relationships for subduction zone earthquakes. *Seismological Research Letters* 68(1)58-73.

FIGURES



LEGEND:

SITE BOUNDARY

0 720 1,440 Feet

GRAPHIC SCALE

FORMER GEORGIA-PACIFIC WOOD PRODUCTS FACILITY
FORT BRAGG, CALIFORNIA

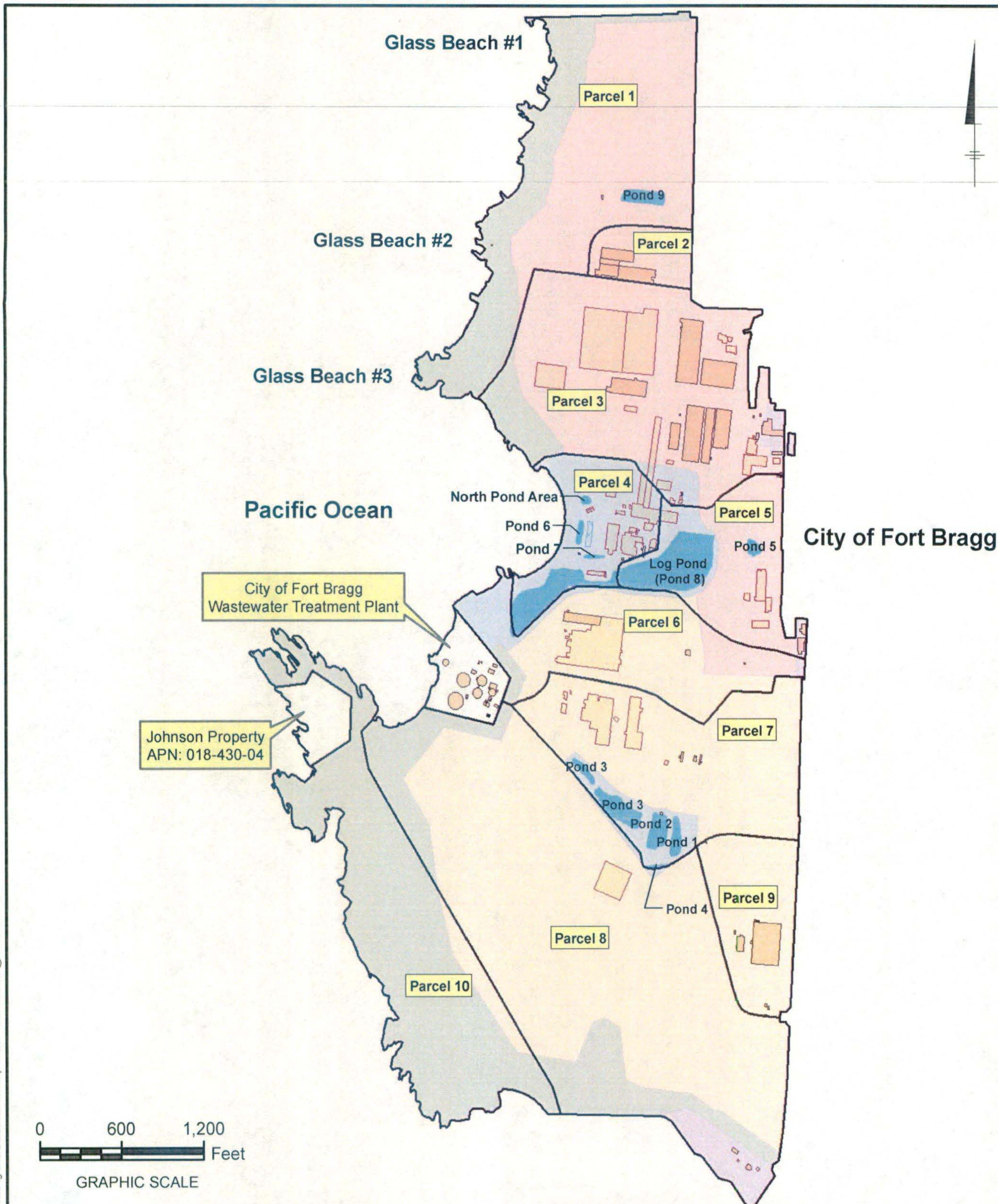
STABILITY ASSESSMENT OF THE MILL POND DAM

SITE LOCATION MAP

ARCADIS

FIGURE
1

CITY: HR DIV/GROUP: IM GIS DB: BCG
Project B0066116.0000
I:\FortBragg\MXD\MillPondDam\StabilityAssessment\Fig1_SiteLocation.mxd - 6/21/2010 @ 12:58:30 PM



LEGEND:

- POND
- FORMER POND
- STRUCTURE
- FORMER STRUCTURE
- FACILITY PARCEL

OPERABLE UNITS

- COASTAL TRAIL/PARK ACQUISITION (OU-A)
- "OFFSITE" NON-INDUSTRIAL (OU-B)
- NORTHERN (OU-C)
- SOUTHERN (OU-D)
- PONDS/PARK (OU-E)

FORMER GEORGIA-PACIFIC WOOD PRODUCTS FACILITY
FORT BRAGG, CALIFORNIA

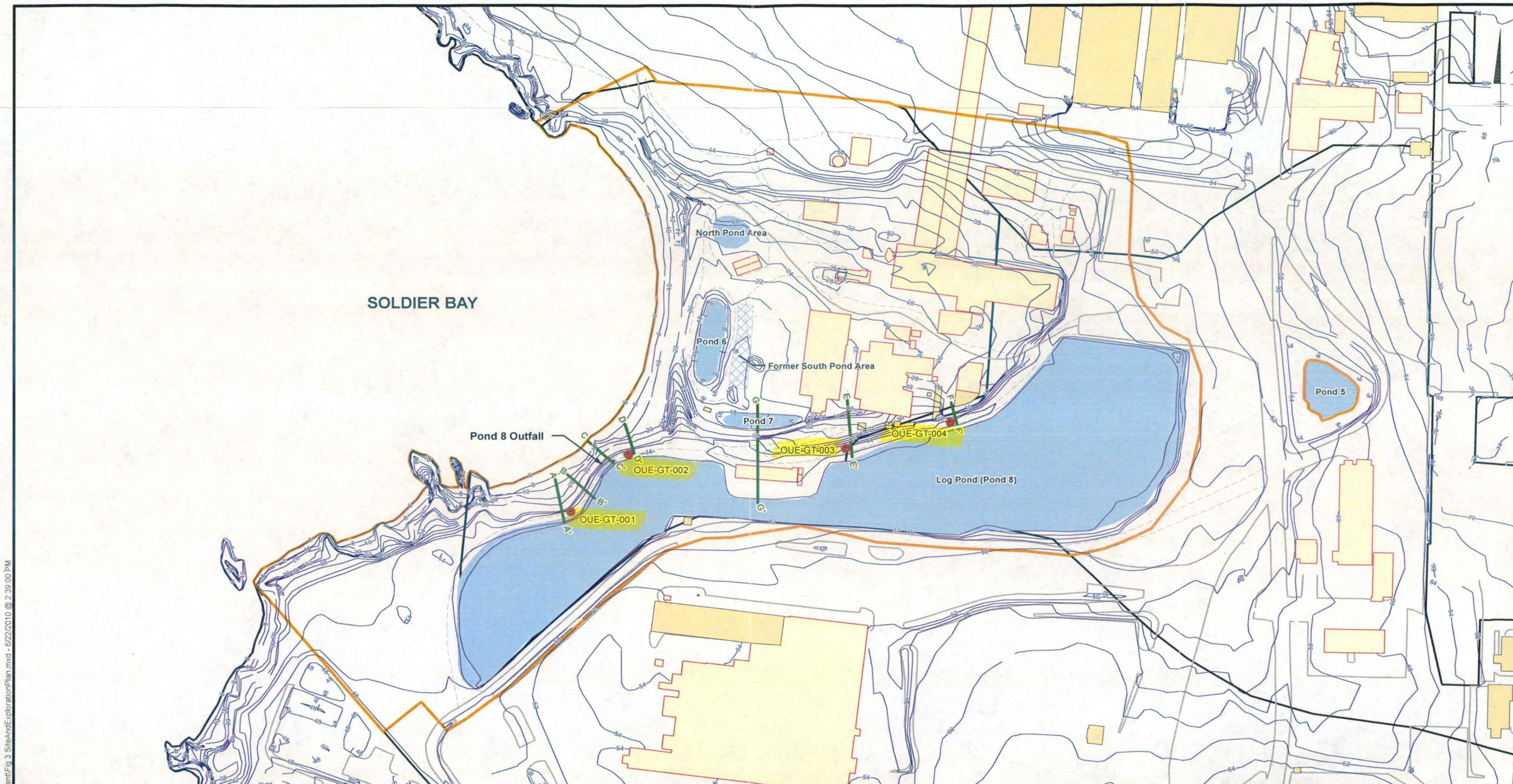
STABILITY ASSESSMENT OF THE MILL POND DAM

OU-E LOCATION MAP

ARCADIS

FIGURE 2

CITY: San Francisco DIV/GROUP:SER IV AIT GIS DB:ME LD: PM: TM:
Project#: 66116.0004
I:\FortBragg\MXD\MillPondDam\StabilityAssessment\Fig 2 OUELocationMap .mxd - 6/21/2010 @ 1:19:04 PM



LEGEND:

OUE-GT-002 BORING ID AND LOCATION	UNPAVED ROADWAY
CROSS-SECTION LINE	PAVED ROADWAY
SURFACE CONTOUR LINE (2FT INTERVAL)	EXISTING STRUCTURE
OU-E BOUNDARY	FORMER STRUCTURE
FACILITY PARCEL	POND
	FORMER POND

NOTES:

1. THE CONTOUR LINES SHOWN ON THIS FIGURE ARE BASED ON TOPOGRAPHIC SURVEY PERFORMED BY DAVID EVANS AND ASSOCIATES INC. IN JUNE 2005.
2. VERTICAL DATUM IS IN NAVD88.

0 200 400 Feet

GRAPHIC SCALE

FORMER GEORGIA-PACIFIC WOOD PRODUCTS FACILITY
FORT BRAGG, CALIFORNIA

STABILITY ASSESSMENT OF THE MILL POND DAM

SITE AND EXPLORATION PLAN

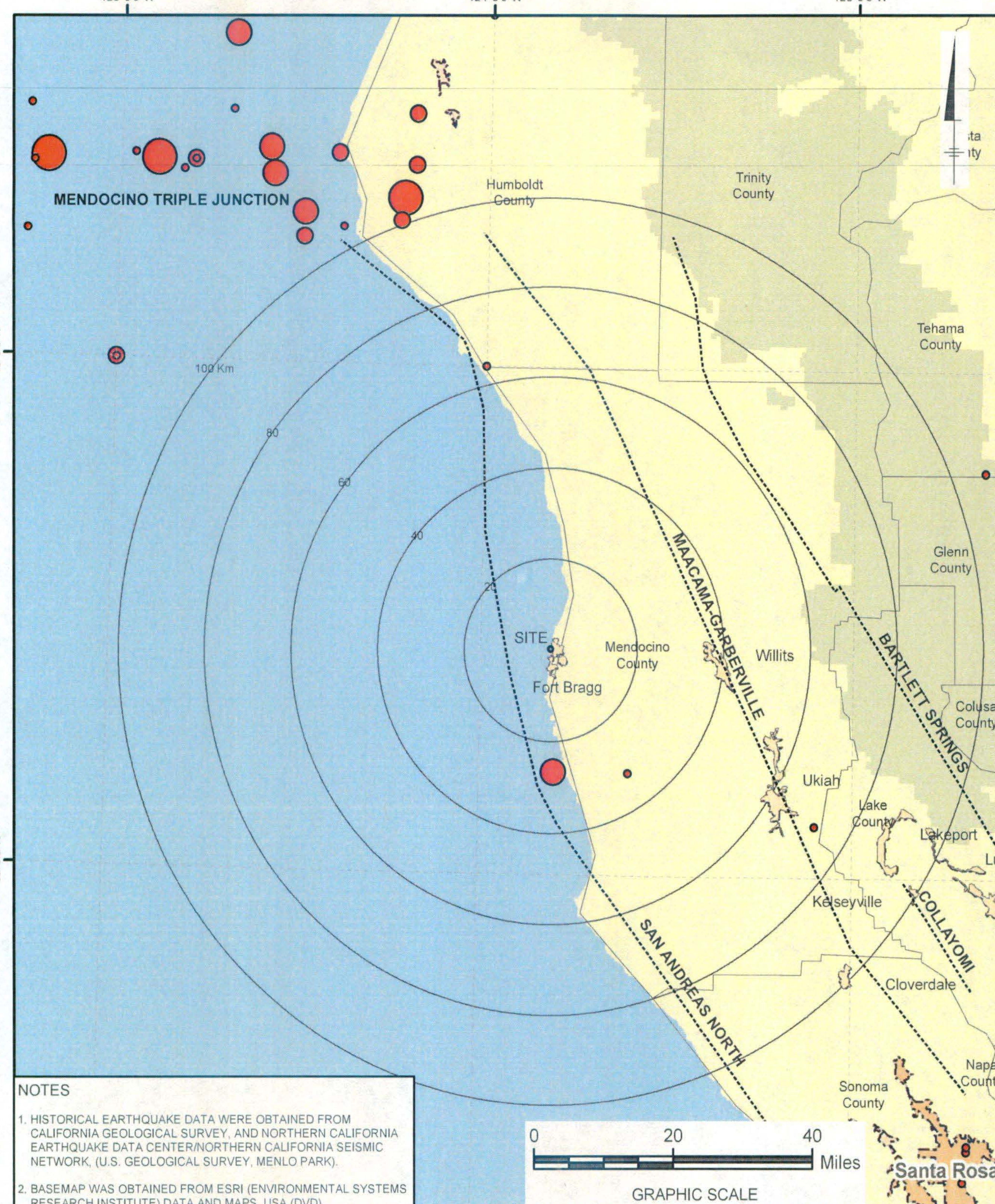
ARCADIS

FIGURE 3

CITY: HR DIV/GROUP: IM GIS DB: BCG
Project #: 66116.0000.0009
I:\FortBragg\MXD\MillPondDam\StabilityAssessment\Fig 3 SiteAndExplorationPlan.mxd - 6/22/2010 @ 2:39:00 PM

[redacted]

4



NOTES

1. HISTORICAL EARTHQUAKE DATA WERE OBTAINED FROM CALIFORNIA GEOLOGICAL SURVEY, AND NORTHERN CALIFORNIA EARTHQUAKE DATA CENTER/NORTHERN CALIFORNIA SEISMIC NETWORK, (U.S. GEOLOGICAL SURVEY, MENLO PARK).
2. BASEMAP WAS OBTAINED FROM ESRI (ENVIRONMENTAL SYSTEMS RESEARCH INSTITUTE) DATA AND MAPS, USA (DVD).

LEGEND:

EARTHQUAKE EPICENTERS (HISTORICAL EARTHQUAKES BETWEEN 1800 AND 2010)

MAGNITUDE

- 5.5 - 5.9
- 6.0 - 6.4
- 6.5 - 6.9
- ≥ 7

SAN ANDREAS FAULT

FORMER GEORGIA-PACIFIC WOOD PRODUCTS FACILITY
FORT BRAGG, CALIFORNIA

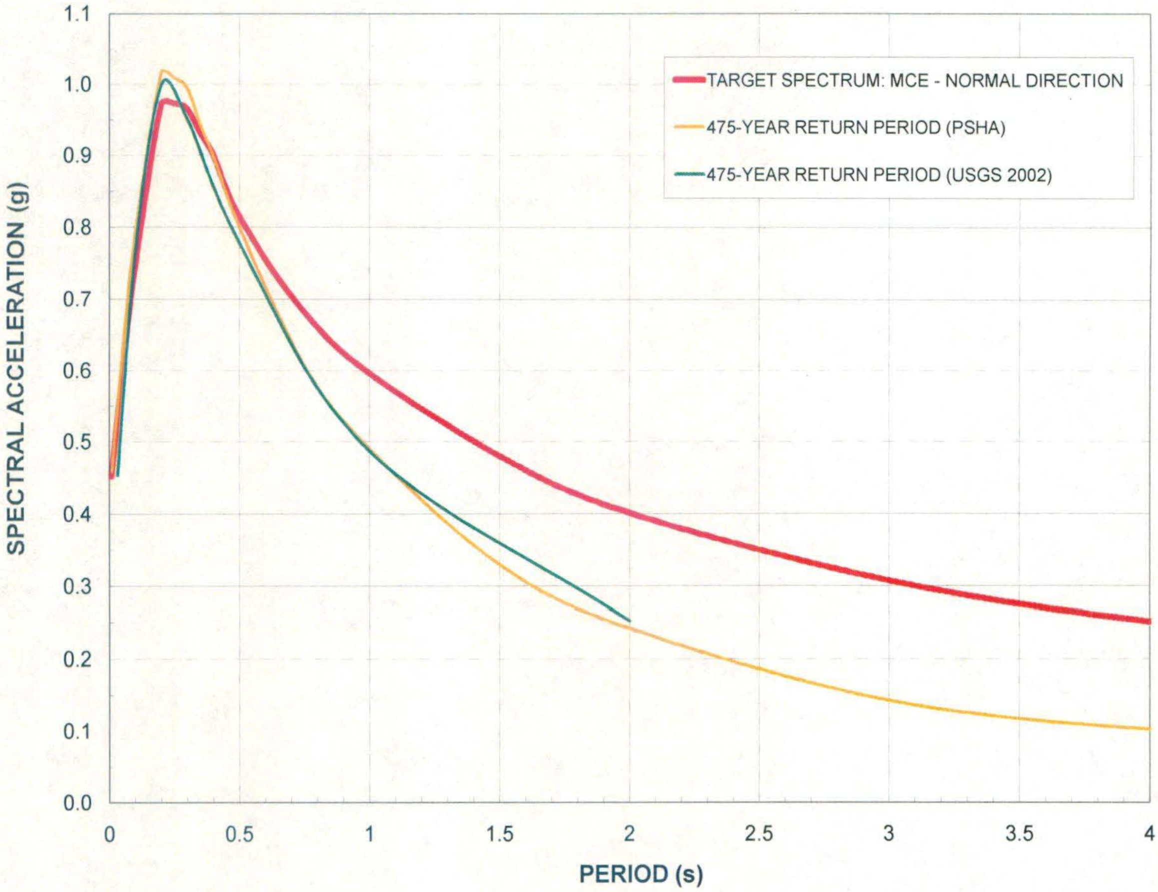
STABILITY ASSESSMENT OF THE MILL POND DAM

REGIONAL FAULTING AND SEISMICITY MAP

ARCADIS

FIGURE **5**

CITY: San Francisco GROUP: 85 DB: ME LD: PIC: PM: TM: TR:
Project #: 66116.0000.00004
I:\FortBragg\MXD\MillPondDam\StabilityAssessment\Fig 5 SeismicStabilityAssessment.mxd - 6/21/2010 @ 2:46:19 PM



NOTES:

1. SPECTRAL ACCELERATIONS REPRESENT ROCK OUTCROP CONDITIONS.
2. MCE = MAXIMUM CREDIBLE EARTHQUAKE; s = SECONDS; g = ACCELERATION OF GRAVITY; PSHA = PROBABILISTIC SEISMIC HAZARD ANALYSIS
3. SPECTRAL ACCELERATIONS FOR THE MCE WERE CALCULATED TAKING INTO ACCOUNT DIRECTIVITY EFFECTS. ACCELERATIONS WERE CALCULATED FOR THE NORMAL DIRECTION.
4. SPECTRAL ACCELERATIONS FOR THE 475-YEAR RETURN PERIOD ARE BASED ON A PROBABILISTIC SEISMIC HAZARD ANALYSIS (PSHA) AND DATA AVAILABLE FROM UNITED STATES GEOLOGIC SURVEY (USGS, 2002).
5. SPECTRAL ACCELERATIONS FOR THE 475 YEAR RETURN PERIOD WERE ONLY PLOTTED FOR COMPARISON.

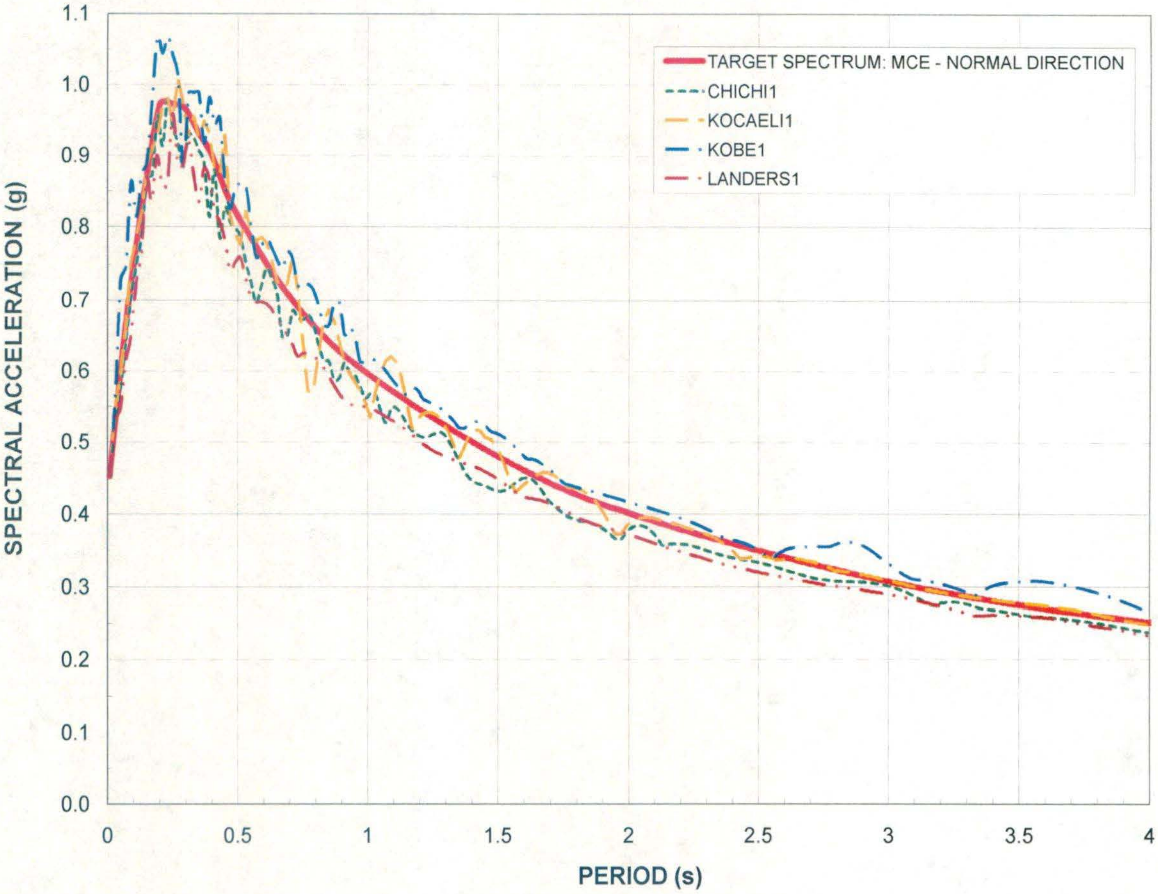
FORMER GEORGIA-PACIFIC WOOD PRODUCTS FACILITY
FORT BRAGG, CALIFORNIA

STABILITY ASSESSMENT OF THE MILL POND DAM

SPECTRAL ACCELERATIONS - COMPARISON OF MCE AND 475-YEAR RETURN PERIOD

ARCADIS

FIGURE **6**



NOTES:

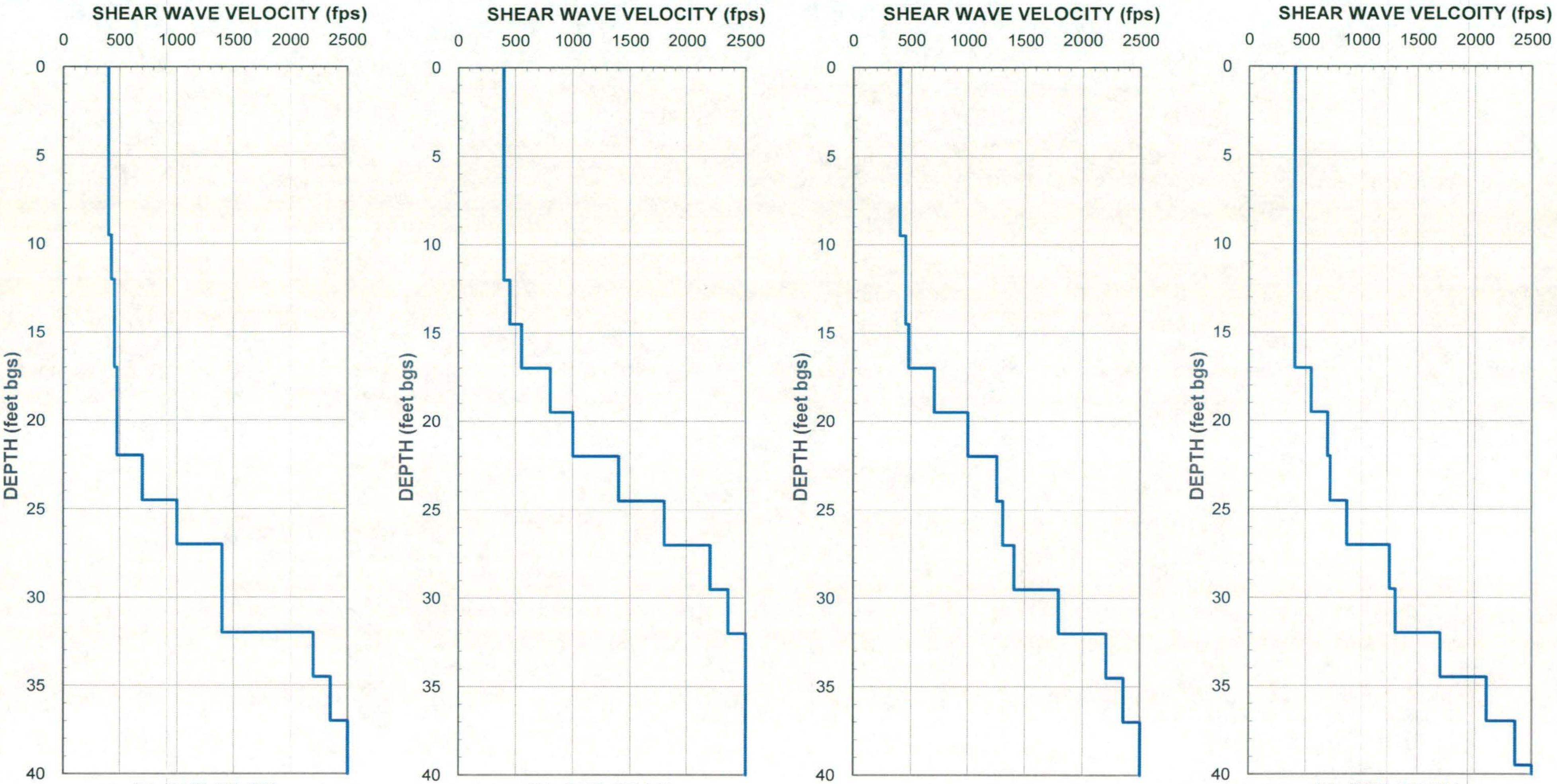
1. SPECTRAL ACCELERATIONS REPRESENT ROCK OUTCROP CONDITIONS.
2. MCE = MAXIMUM CREDIBLE EARTHQUAKE; s = SECONDS; g = ACCELERATION OF GRAVITY
3. SPECTRAL ACCELERATIONS FOR THE MCE WERE DETERMINED DETERMINISTICALLY.
4. CHICHI1, KOCAELI1, KOBE1, AND LANDERS1 ARE GROUND MOTIONS THAT WERE GENERATED USING ACCELERATION TIME HISTORIES FROM ACTUAL RECORDED EARTHQUAKE EVENTS, WHICH WERE SPECTRALLY MATCHED USING THE TARGET SPECTRUM FOR THE MCE.

FORMER GEORGIA-PACIFIC WOOD PRODUCTS FACILITY
FORT BRAGG, CALIFORNIA
STABILITY ASSESSMENT OF THE MILL POND DAM

SPECTRAL ACCELERATIONS OF SPECTRALLY MATCHED INPUT GROUND MOTIONS

ARCADIS

FIGURE
7



(a) OUE-GT-001 (b) OUE-GT-002 (c) OUE-GT-003 (d) OUE-GT-004

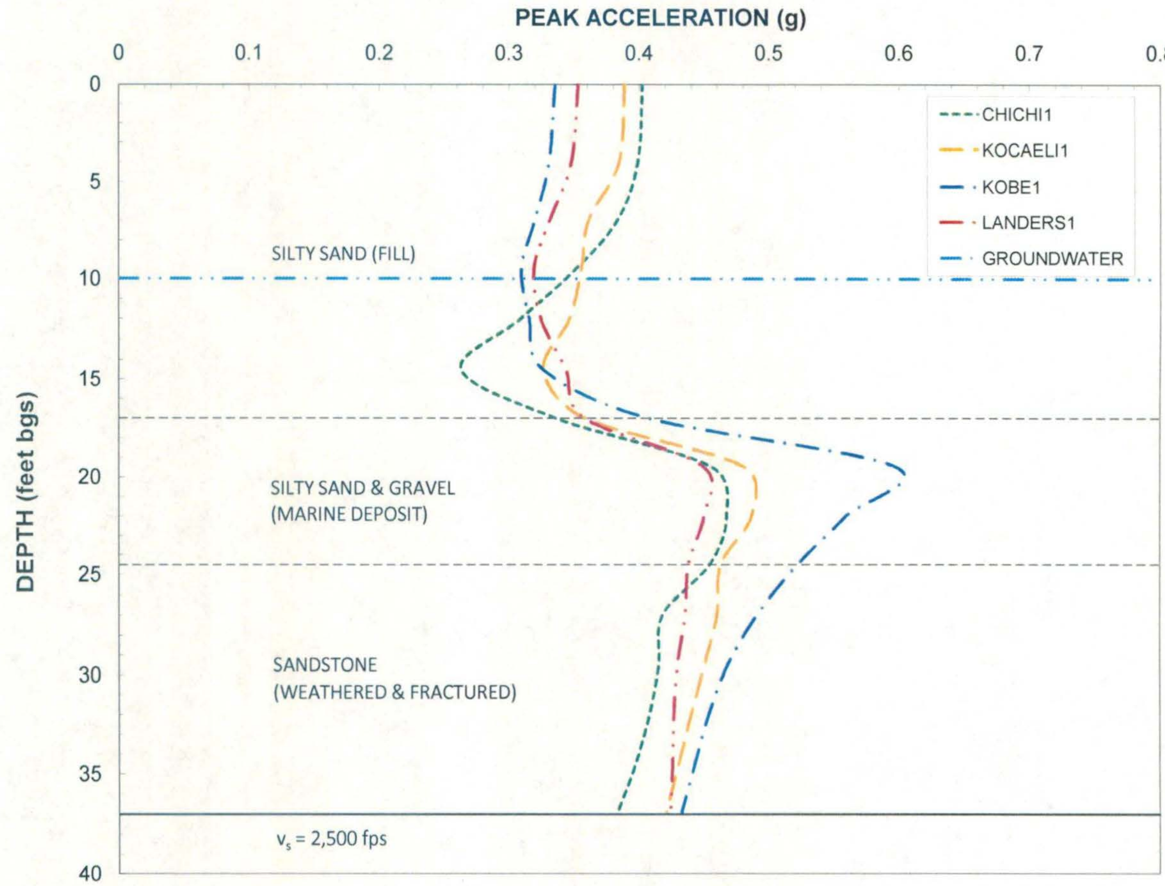
Notes:
1. fps = FEET PER SECOND; bgs = BELOW GROUND SURFACE

FORMER GEORGIA PACIFIC WOOD PRODUCTS FACILITY
FORT BRAGG, CALIFORNIA
STABILITY ASSESSMENT OF THE MILL POND DAM

SHEAR WAVE VELOCITY PROFILES FOR GROUND RESPONSE ANALYSIS

ARCADIS

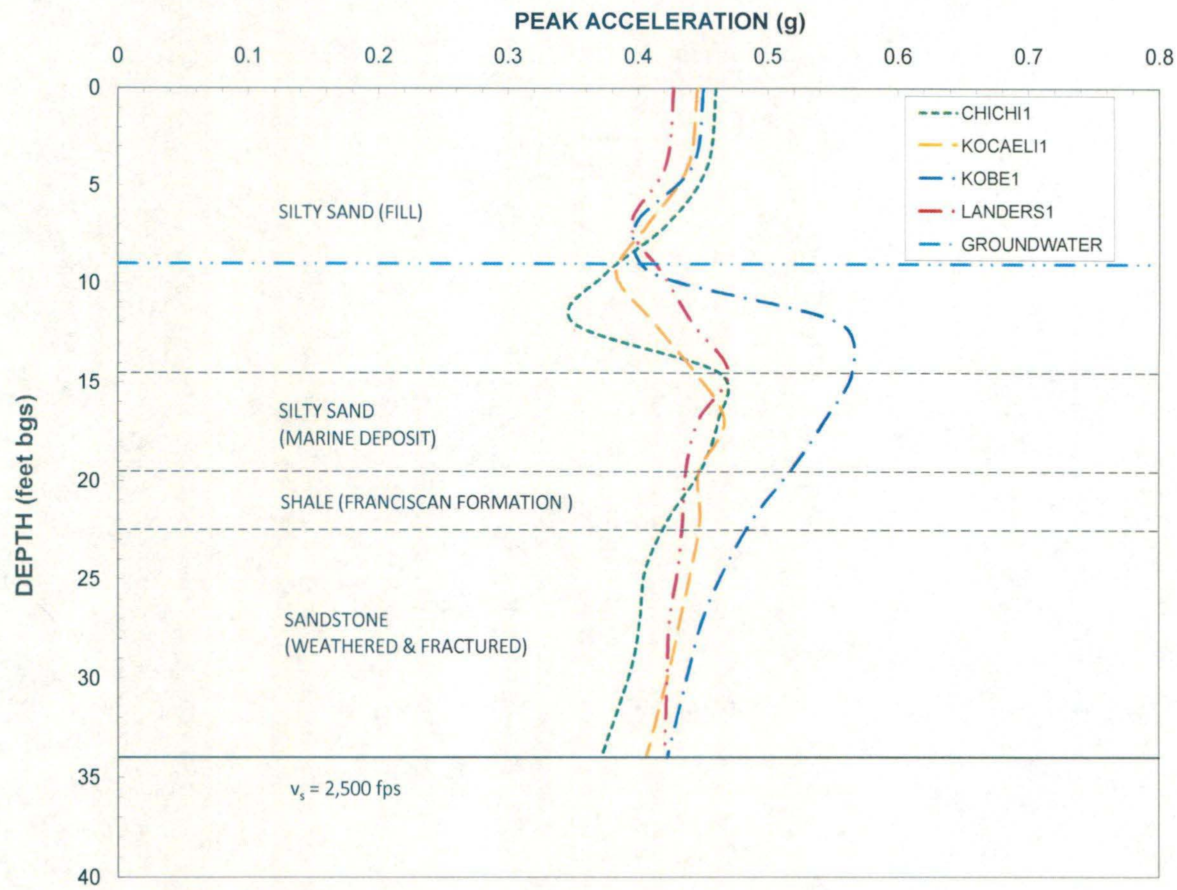
FIGURE
8



NOTES:

1. bgs = BELOW GROUND SURFACE; v_s = SHEAR WAVE VELOCITY; fps = FEET PER SECOND; MCE = MAXIMUM CREDIBLE EARTHQUAKE
2. CHICHI1, KOCAELI1, KOBE1, and LANDERS1 ARE GROUND MOTIONS THAT WERE GENERATED USING ACCELERATION TIME HISTORIES FROM ACTUAL RECORDED EARTHQUAKE EVENTS, WHICH WERE SPECTRALLY MATCHED USING THE TARGET SPECTRUM FOR THE MCE.

FORMER GEORGIA-PACIFIC WOOD PRODUCTS FACILITY FORT BRAGG, CALIFORNIA **STABILITY ASSESSMENT OF THE MILL POND DAM**	
PEAK ACCELERATIONS BORING LOCATION OUE-GT-001	
ARCADIS	FIGURE **9**



NOTES:

1. bgs = BELOW GROUND SURFACE; v_s = SHEAR WAVE VELOCITY; fps = FEET PER SECOND; MCE = MAXIMUM CREDIBLE EARTHQUAKE
2. CHICHI1, KOCAELI1, KOBE1, and LANDERS1 ARE GROUND MOTIONS THAT WERE GENERATED USING ACCELERATION TIME HISTORIES FROM ACTUAL RECORDED EARTHQUAKE EVENTS, WHICH WERE SPECTRALLY MATCHED USING THE TARGET SPECTRUM FOR THE MCE.

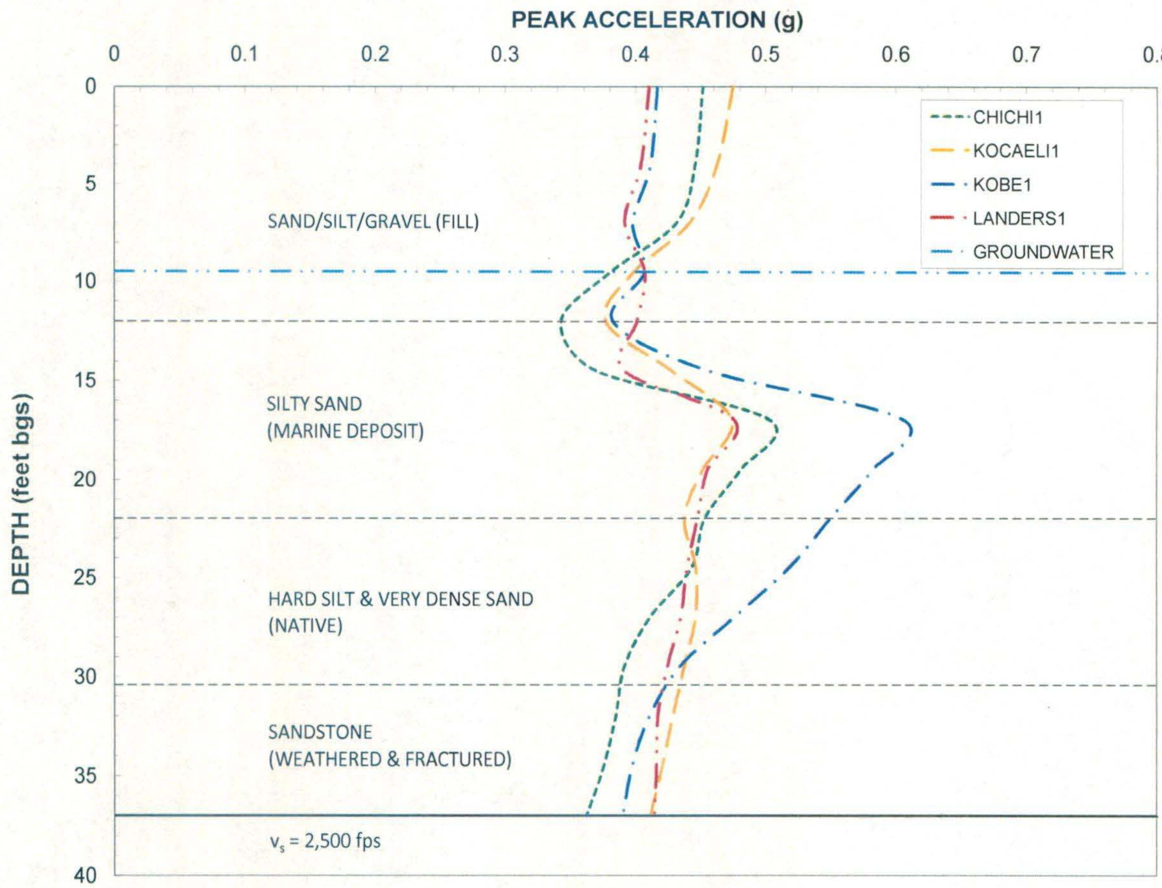
FORMER GEORGIA-PACIFIC WOOD PRODUCTS FACILITY
FORT BRAGG, CALIFORNIA

STABILITY ASSESSMENT OF THE MILL POND DAM

PEAK ACCELERATIONS
BORING LOCATION OUE-GT-002

ARCADIS

FIGURE
10



NOTES:

1. bgs = BELOW GROUND SURFACE; v_s = SHEAR WAVE VELOCITY; fps = FEET PER SECOND; MCE = MAXIMUM CREDIBLE EARTHQUAKE
2. CHICHI1, KOCAELI1, KOBE1, and LANDERS1 ARE GROUND MOTIONS THAT WERE GENERATED USING ACCELERATION TIME HISTORIES FROM ACTUAL RECORDED EARTHQUAKE EVENTS, WHICH WERE SPECTRALLY MATCHED USING THE TARGET SPECTRUM FOR THE MCE.

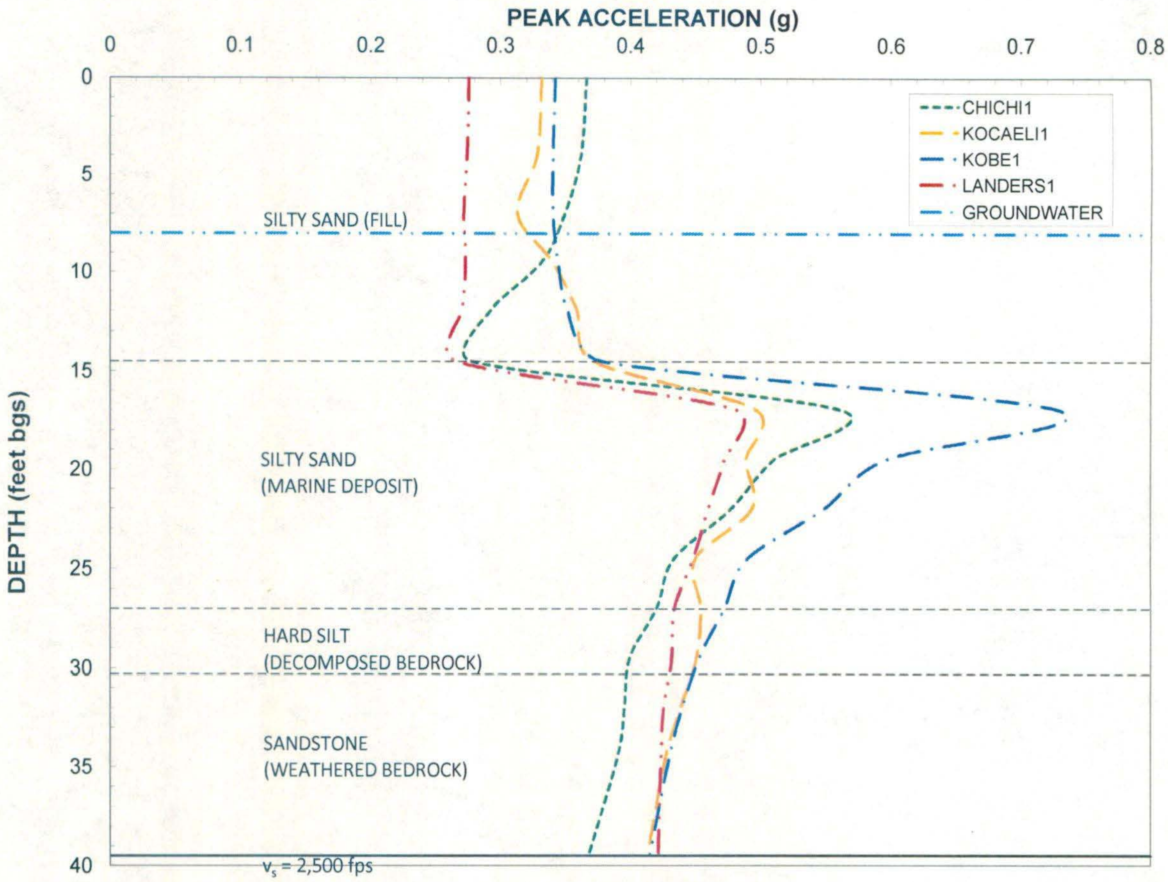
FORMER GEORGIA-PACIFIC WOOD PRODUCTS FACILITY
FORT BRAGG, CALIFORNIA

STABILITY ASSESSMENT OF THE MILL POND DAM

PEAK ACCELERATIONS
BORING LOCATION OUE-GT-003

ARCADIS

FIGURE 11



NOTES:

1. bgs = BELOW GROUND SURFACE; v_s = SHEAR WAVE VELOCITY; fps = FEET PER SECOND; MCE = MAXIMUM CREDIBLE EARTHQUAKE
2. CHICHI1, KOCAELI1, KOBE1, and LANDERS1 ARE GROUND MOTIONS THAT WERE GENERATED USING ACCELERATION TIME HISTORIES FROM ACTUAL RECORDED EARTHQUAKE EVENTS, WHICH WERE SPECTRALLY MATCHED USING THE TARGET SPECTRUM FOR THE MCE.

FORMER GEORGIA-PACIFIC WOOD PRODUCTS FACILITY FORT BRAGG, CALIFORNIA **STABILITY ASSESSMENT OF THE MILL POND DAM**	
PEAK ACCELERATIONS BORING LOCATION OUE-GT-004	
ARCADIS	FIGURE **12**

ms_Figures 9 thr 12 (Peak Accelerations).xlsx_Figure 12_6/18/2010

[redacted]

ARCADIS

FIGURE

13

[redacted]

ms_F[redacted]

[redacted]

15

ms_Figures 13 thr 16 (Liquefaction).xlsx_Figure 15_6/18/2010

[redacted]

CITY: SYRACUSE, NY GROUP: ENVCAD DB: P. LISTER PM: B. DESHIELDS TM: C. BECKER TR: M. HANNA LYR: ON=*;OFF=REF, (FRZ)
G:\ENVCAD\SYRACUSE\ACT\B0066116\0000\00004\DWG\66116V01.dwg LAYOUT: 17 SAVED: 6/21/2010 8:25 AM ACADVER: 17.0S (LMS TECH) PAGESETUP: ---- PLOTSTYLETABLE: PLTFULL.CTB PLOTTED: 6/21/2010 8:27 AM BY: LISTER, PAUL

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

[redacted]

APPENDICES

APPENDIX A

ARCADIS

Appendix A

Geotechnical Field Exploration
Methods and Logs

SOIL DESCRIPTION

Soil descriptions on the exploration logs are based on visual observations and laboratory testing on selected samples. The samples were visually classified in general accordance with ASTM D 2488.

Soil descriptions generally consist of the following:
Color, MAJOR CONSTITUENT, minor constituents, moisture, density/consistency, additional observations

MINOR CONSTITUENTS

Description	Estimated Percentage
Trace	Less than 5%
Few	5 to 10%
Little	15 to 25%
Some	30 to 45%

MOISTURE

Dry	Little perceptible moisture
Damp	Below optimum moisture for compaction
Moist	Likely near optimum moisture content
Wet	Likely wet of optimum moisture content
Saturated	Probably below water table or in perched groundwater

DENSITY/CONSISTENCY

Soil density/consistency descriptions on boring logs are primarily based on Standard Penetration Resistance. Density/consistency descriptions on exploration logs are provided in parentheses if they are based on visual observations rather than correlations with Standard Penetration Resistance (N-values) and other test results.

Granular Soils Density	Standard Penetration Resistance (N) in Blows/Foot	Cohesive Soils Consistency	Standard Penetration Resistance(N) in Blows/Foot	Approximate Undrained Strength in TSF
Very loose	0 to 4	Very soft	0 to 2	<0.125
Loose	4 to 10	Soft	2 to 4	0.125 to 0.25
Medium dense	10 to 30	Medium stiff	4 to 8	0.25 to 0.5
Dense	30 to 50	Stiff	8 to 15	0.5 to 1.0
Very dense	>50	Very stiff	15 to 30	1.0 to 2.0
		Hard	>30	>2.0

ROCK DESCRIPTION

Rock descriptions on the exploration logs are based on visual observations and generally consist of the following:
Color, ROCK TYPE, field strength, structure, decomposition, disintegration, fracture density, fracture type, fracture infilling, fracture uneveness, moisture condition, additional observations.

TEST SYMBOLS

MC	Moisture Content	UC	Unconfined Compression
GS	Grain Size	TX	Triaxial Compressive Strength
AL	Atterberg Limits	DS	Direct Shear
SG	Specific Gravity	PL	Point Load Index
DT	Density Test	K	Permeability
OG	Organic Content	PP	Pocket Penetrometer in tons/ft^2
CN	Consolidation	TV	Torvane in tons/ft^2
UU	Unconsolidated Undrained Triaxial	PID	Photoionization Detector Reading
CU	Consolidated Undrained Triaxial	CA	Chemical Analysis

SAMPLE TYPE SYMBOLS

- ⊠ Split Spoon
- ⧅ Shelby Tube
- Cuttings
- Core Run
- P Tube pushed, not driven

FORMER GEORGIA PACIFIC WOOD PRODUCTS FACILTY, FORT BRAGG,CALIFORNIA
STABILITY ASSESSMENT OF THE MILL POND DAM

KEY TO EXPLORATION LOGS

ARCADIS

FIGURE
A-1

Boring Log *OUE-GT-001*

Former Georgia Pacific Wood Products Facility, Fort Bragg, California

N2291676.7 E6049675.5 NAD 83 California Zone II

Approximate Ground Surface Elevation: 44.5 feet

Depth (ft.)	Sample No.	Sample Type	SPT Results	Lab Tests	TCR (%)	SCR (%)	RQD (%)	Water Level	Lithology	Description
0										Dark grayish brown, fine to medium SAND, little to some Silt, trace to some Gravel, damp, loose to very loose. (Fill)
	S-1			MC						
5	S-2		N=4	MC						
	S-3		N=1	MC, GS						Sand becomes moist.
10	S-4		N=4					ATD		Sand becomes wet.
	S-5		N=6	MC, GS, AL						
15	S-6		N=5	MC, GS						
	S-7		N=3							Dark grayish brown, coarse GRAVEL and fine SAND, some Silt, wet, loose to very loose. (Marine Terrace Deposit)
20	S-8		N=5	MC, GS, AL						
	S-9		N=17	MC						Dark grayish brown, fine SAND, wet, medium dense. (Marine Terrace Deposit)
25	S-10		N=50	MC						Gray SANDSTONE, decomposed, highly weathered, friable. (FRANCISCAN FORMATION)
	S-11		N=50/3"	DS						Gray SANDSTONE, strong, massive, intensely fractured zones are highly decomposed, fractured pieces are competent moderately to intensely fractured, wet. (FRANCISCAN FORMATION)
30	CR-1				50	32	13			
										Gray SANDSTONE, very weak, structure unknown, highly decomposed, intensely disintegrated, very intensely fractured.Recoverd material appears to be sandy soil with gravel. (FRANCISCAN FORMATION)
35	CR-2				16	0	0			
										Bottom of rock coring at 37.25 feet. Completed 01/28/2009
40										

1. Refer to Figure A-1, Key to Exploration Logs, for explanation of symbols and definitions.
2. The stratum lines represent the approximate boundaries between soil units. Actual changes may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the subsurface conditions.
4. ATD = at time of drilling; TCR = total core recovery; SCR = solid core recovery; RQD = rock quality designation

ARCADIS

G:\Div 11\Rockware\Logplot 2001\Logplot\Logfiles\B0016810

Boring Log *OUE-GT-002*

Former Georgia Pacific Wood Products Facility, Fort Bragg, California

N2291799.4 E6049792.3 NAD 83 California Zone II

Approximate Ground Surface Elevation: 44.3 feet

Depth (ft.)	Sample No.	Sample Type	SPT Results	Lab Tests	TCR (%)	SCR (%)	RQD (%)	Water Level	Lithology	Description
0										Dark grayish and yellowish brown, fine SAND, little to some Silt, trace to few Gravel, moist, loose to very loose. (Fill)
	S-1			MC						
5	S-2		N=9	MC						
	S-3		N=1	MC, GS, AL				ATD		
10	S-4		N=1	MC, GS, AL						Sand becomes wet.
	S-5		N=4	MC, GS						
15	S-6		N=9	MC, GS						Dark grayish/yellowish brown, fine to medium SAND, trace to few Gravel, few to some Silt, wet, loose to very dense. (Marine Terrace Deposit)
	S-7		N=59	MC						
20	S-8		N=50/3"	MC						Very dark gray SHALE, weak, very friable, massive, highly decomposed, intensely fractured, joint at 10 degrees, clean, rough, wet. (FRANCISCAN FORMATION)
	CR-1			PL	50	40	40			
	CR-2			PL	90	25	25			Gray SANDSTONE, strong, massive, fresh, intensely fractured, numerous veins, joints= 10 degrees,15 degrees, clean, rough to stepped, wet. (FRANCISCAN FORMATION)
25	CR-3			PL, PL, PL	100	94	31			Gray SANDSTONE, strong, massive, fresh, competent, moderately to intensely fractured, moderately hard rock, multiple quartz veins (core pieces that seem intact fall apart under their own weight), wet. (FRANCISCAN FORMATION)
30	CR-4			TX, DS	100	60	37			Gray SANDSTONE, massive, fresh, competent, intensely fractured, moderately hard rock, shattered in broken zones, highly decomposed. (FRANCISCAN FORMATION)
	CR-5				97	0	0			Gray SANDSTONE, strong, massive, fresh, competent, intensely fractured, moderately hard rock, shattered throughout. (FRANCISCAN FORMATION)
35	CR-6			UC, PL, TX, PL	100	98	90			Gray SANDSTONE, strong, massive,fresh, competent, moderately fractured joints show disclination 1' into rock (FRANCISCAN FORMATION)
40										Bottom of rock coring at 38.0 feet. Completed 01/28/2009

1. Refer to Figure A-1, Key to Exploration Logs, for explanation of symbols and definitions.
2. The stratum lines represent the approximate boundaries between soil units. Actual changes may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the subsurface conditions.
4. ATD = at time of drilling; TCR = total core recovery; SCR = solid core recovery; RQD = rock quality designation

ARCADIS

Figure A-3 Page 1 of 1

G:\Div 11\Rockware\Logplot 2001\Logplot\Logfiles\B0016810

Boring Log *OUE-GT-003*

Former Georgia Pacific Wood Products Facility, Fort Bragg, California
N2291811.9 E6050242.5 NAD 83 California Zone II
Approximate Ground Surface Elevation: 44.3 feet

Depth (ft.)	Sample No.	Sample Type	SPT Results	Lab Tests	Water Level	Lithology	Description
0							Very dark gray, fine SAND, little Gravel, little Silt, dry, medium dense. (Fill)
	S-1			MC			
5	S-2		N=8	MC			Mottled gray and reddish brown SILT, some Sand, little Gravel, moist, medium stiff to stiff. (Fill)
	S-3		N=3	MC, GS			Very dark gray, fine to medium SAND, few Gravel, little Silt, wet, very loose. (Fill)
10	S-4		N=7	MC, GS	ATD		Very dark gray, fine to coarse GRAVEL, little Sand, few Silt, wet, loose. (Fill)
	S-5		N=5	MC, GS, AL			Dark to greenish gray, fine SAND, little to some Silt, trace to some Gravel, wet, loose. (Marine Terrace Deposit)
15	S-6		N=5	MC, GS			
	S-7		N=8	MC, GS			
20	S-8		N=19	MC			SAND becomes medium dense.
	S-9		N=77	MC, AL			Very dark gray SILT, little Gravel, moist, hard. (Native)
25	S-10		N=96	MC			Dark gray, fine SAND, few Gravel, few Silt, moist, very dense. (Native)
	S-11		N=86	MC, AL			Dark gray, SILT, some Gravel, moist, hard. (Native)
30	S-12		N=50/4"	MC			Bottom of boring at 30.4 feet bgs. Refusal on bedrock. Completed 01/27/2009
35							
40							

Fill

Terrace

Rock?

1. Refer to Figure A-1, Key to Exploration Logs, for explanation of symbols and definitions.
2. The stratum lines represent the approximate boundaries between soil units. Actual changes may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the subsurface conditions.
4. ATD = at time of drilling

ARCADIS

Figure A-4 Page 1 of 1

G:\Div 11\Rockware\Logplot 2001\Logplot\Logfiles\B0016810

Boring Log *OUE-GT-004*

Former Georgia Pacific Wood Products Facility, Fort Bragg, California

N2291867.4 E6050457.4 NAD 83 California Zone II

Approximate Ground Surface Elevation: 44.7 feet

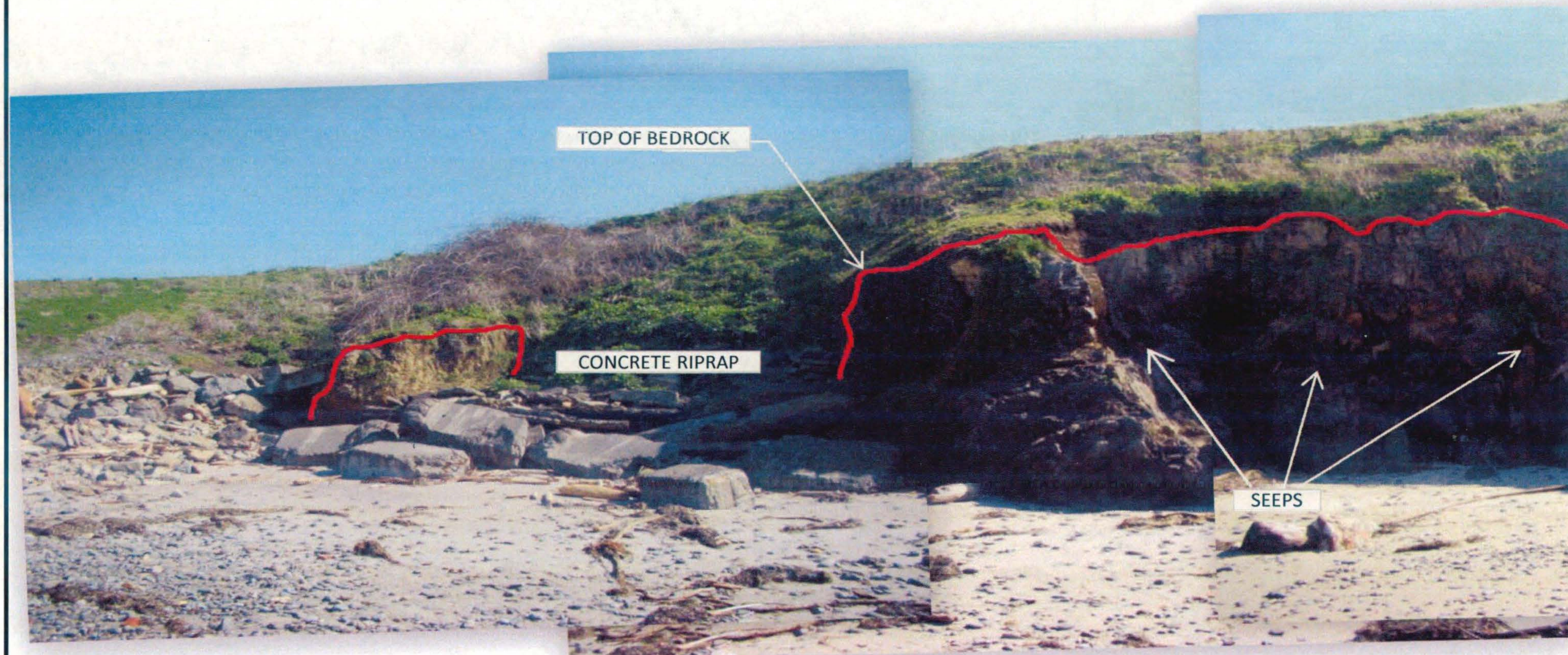
Depth (ft.)	Sample No.	Sample Type	SPT Results	Lab Tests	Water Level	Lithology	Description
0							Very dark grayish brown, fine SAND, trace to little angular Gravel, little to some Silt, moist, loose to very loose. (Fill)
	S-1			MC			
5	S-2		N=6	MC			
	S-3		N=4	MC, GS, AL	ATD		
10	S-4		N=55				Wood in sampler (did not recover soil).
	S-5		N=3	MC, GS			
15	S-6		N=3				Dark brown, fine to coarse SAND, trace to little Gravel, trace to little Silt, wet, very loose. (Marine Terrace Deposit)
	S-7		N=11				Sand becomes medium dense.
20	S-8		N=20	MC, GS			
	S-9		N=23				
25	S-10		N=36	MC			Sand becomes dense.
	S-11		N=85	MC, GS, AL			Very dark grayish/yellowish brown SILT, some Gravel, little Sand, wet, hard. (Decomposed bedrock/ Franciscan Formation, sandstone pebbles in a sheared claystone matrix)
30	S-12		N=50/4"	MC			Bottom of boring at 30.3 feet. Refusal on bedrock. Completed 01/27/2009
35							
40							

1. Refer to Figure A-1, Key to Exploration Logs, for explanation of symbols and definitions.
2. The stratum lines represent the approximate boundaries between soil units. Actual changes may be gradual.
3. The discussion in the text of this report is necessary for a proper understanding of the subsurface conditions.
4. ATD = at time of drilling

ARCADIS

Figure A-5

G:\Div 11\Rockware\Logplot 2001\Logplot\Logfiles\B0016810



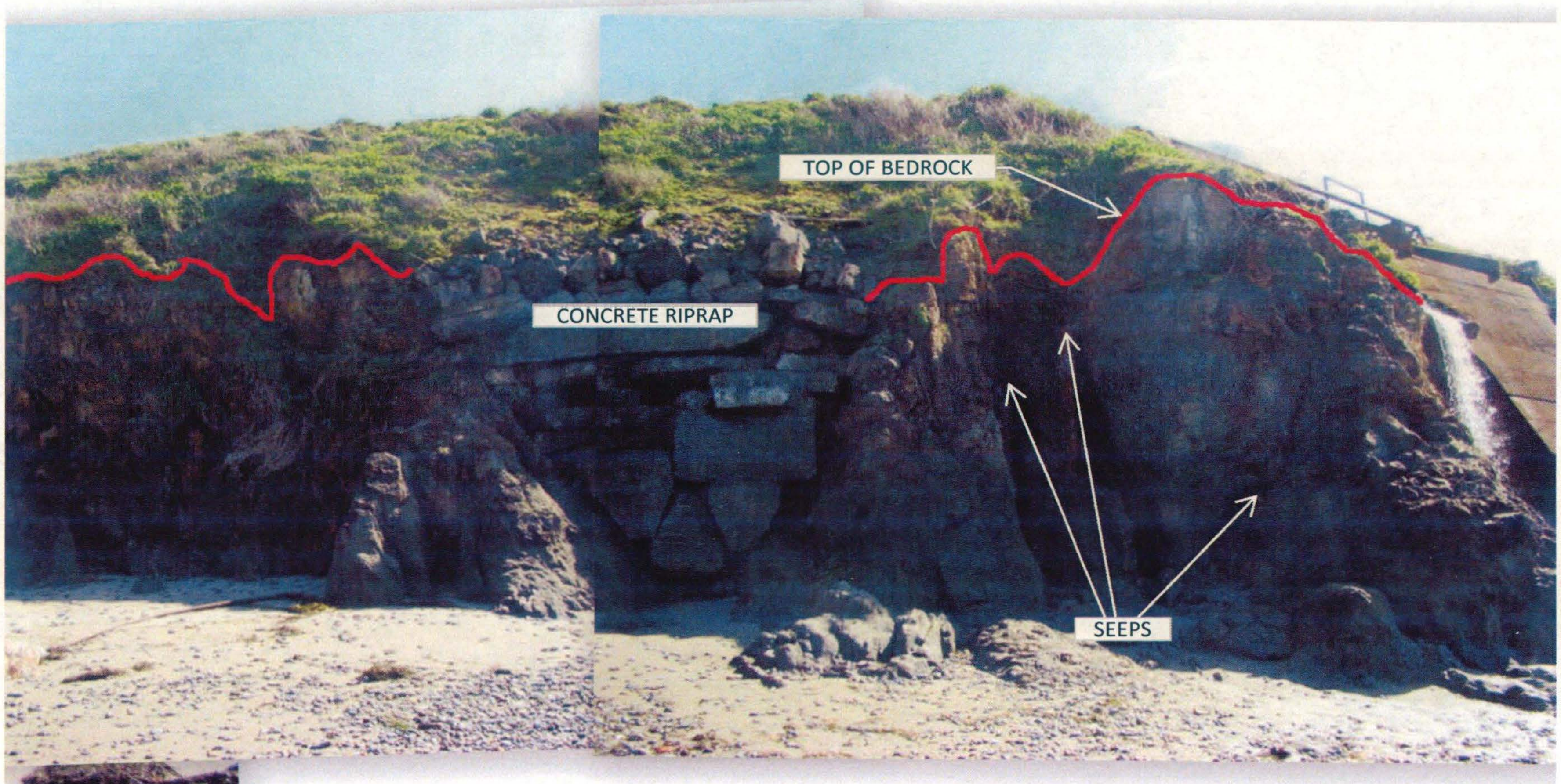
NOTES:
PHOTOS WERE TAKEN FROM SOLDIER BAY BEACH, JUST TO THE EAST OF SECTION D-D' (REFER TO FIGURE 3 OF THE REPORT).

FORMER GEORGIA PACIFIC WOOD PRODUCTS FACILITY
FORT BRAGG, CALIFORNIA
STABILITY ASSESSMENT OF THE MILL POND DAM

EXPOSED ROCK FACE (1)

ARCADIS

FIGURE
A-6



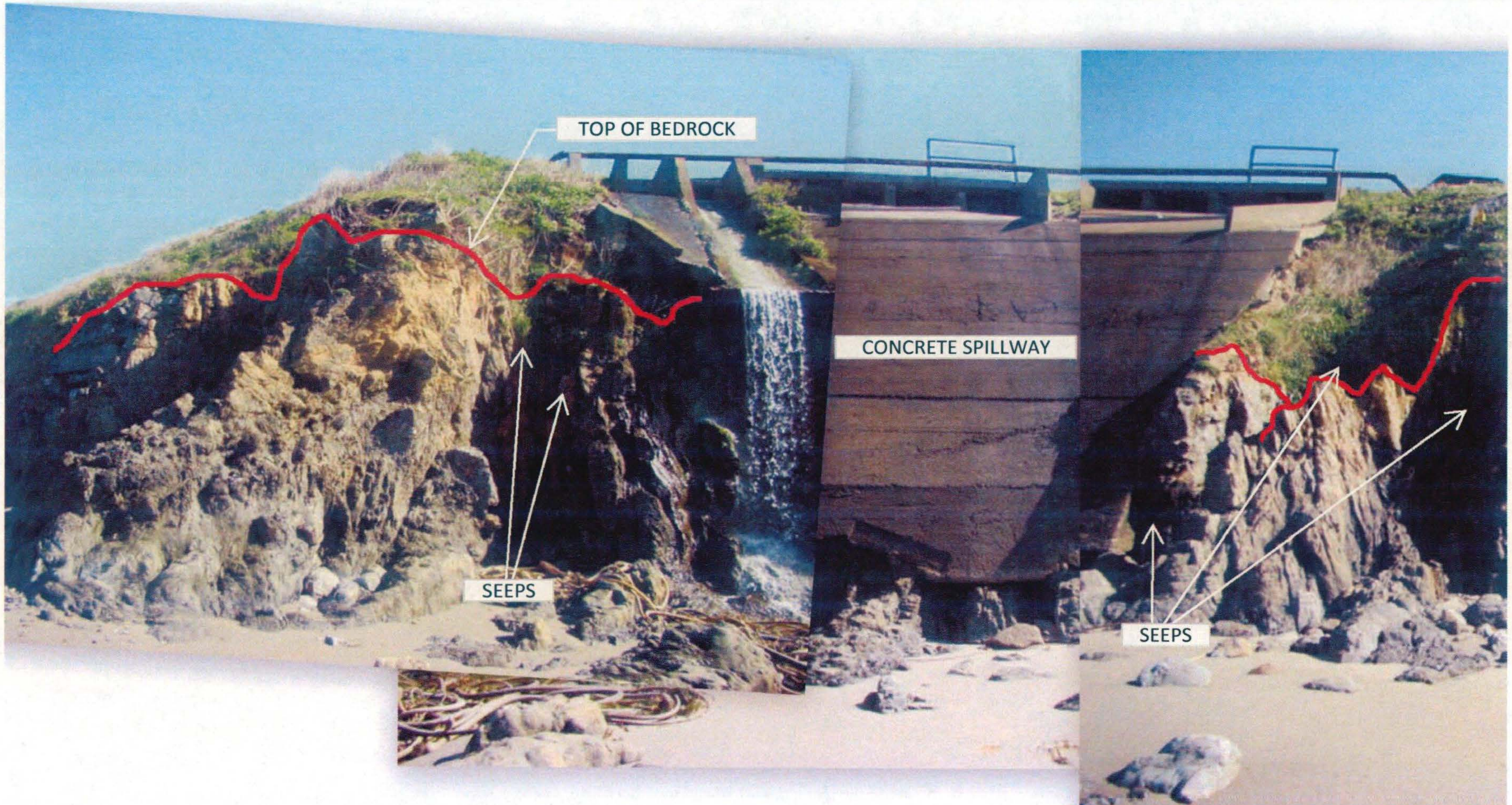
NOTES:
PHOTOS WERE TAKEN FROM SOLDIER BAY BEACH, NEAR SECTION D-D'
(REFER TO FIGURE 3 OF THE REPORT).

FORMER GEORGIA PACIFIC WOOD PRODUCTS FACILITY
FORT BRAGG, CALIFORNIA
STABILITY ASSESSMENT OF THE MILL POND DAM

EXPOSED ROCK FACE (2)

ARCADIS

FIGURE
A-7



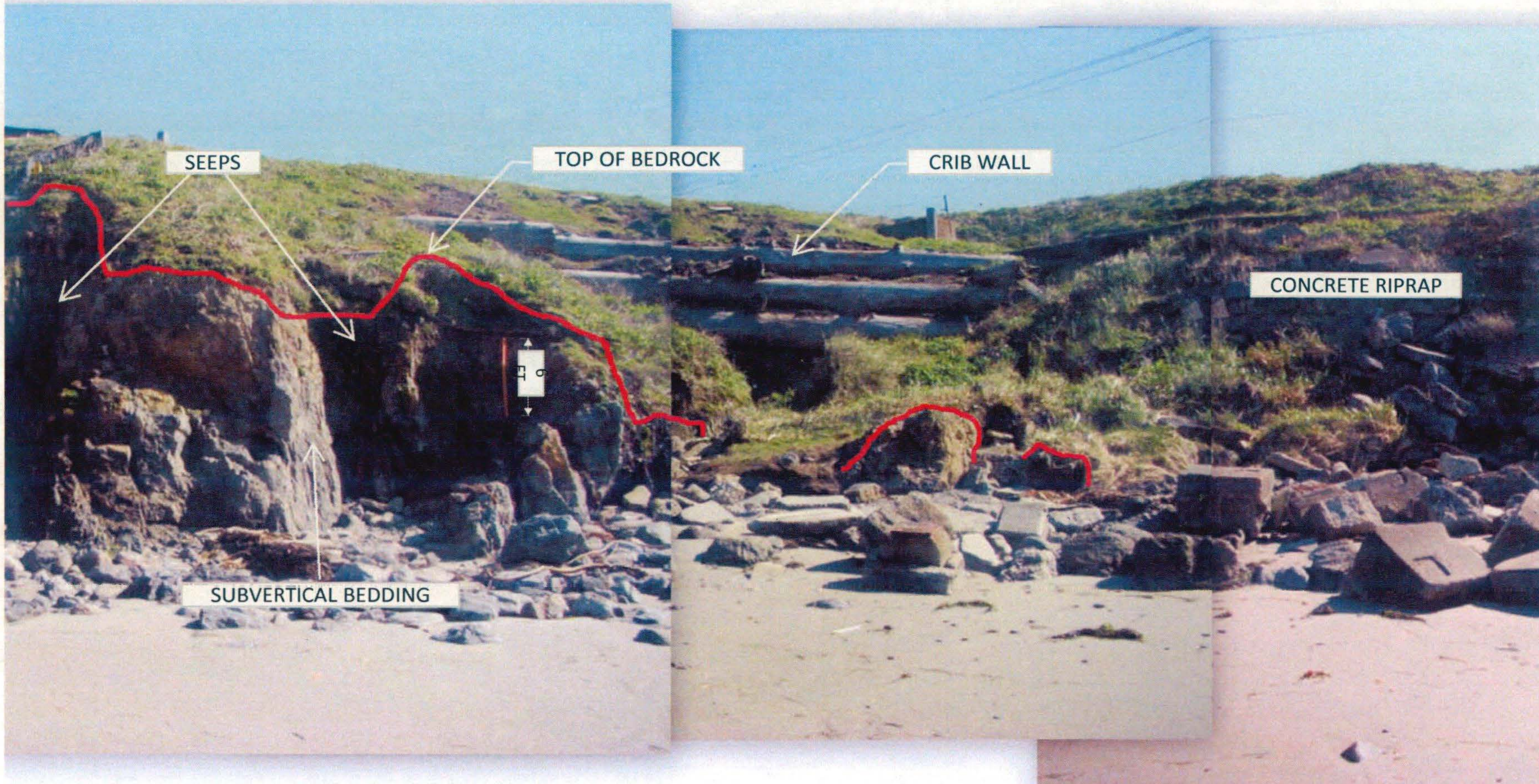
NOTES:
PHOTOS WERE TAKEN FROM SOLDIER BAY BEACH, NEAR SECTION C-C' AND THE SPILLWAY (REFER TO FIGURE 3 OF THE REPORT).

FORMER GEORGIA PACIFIC WOOD PRODUCTS FACILITY
FORT BRAGG, CALIFORNIA
STABILITY ASSESSMENT OF THE MILL POND DAM

EXPOSED ROCK FACE (3)

ARCADIS

FIGURE
A-8



NOTES:
PHOTOS WERE TAKEN FROM SOLDIER BAY BEACH, NEAR SECTIONS A-A' AND B-B' AND THE CRIB WALL (REFER TO FIGURE 3 OF THE REPORT).

FORMER GEORGIA PACIFIC WOOD PRODUCTS FACILITY
FORT BRAGG, CALIFORNIA
STABILITY ASSESSMENT OF THE MILL POND DAM

EXPOSED ROCK FACE (4)

ARCADIS

FIGURE
A-9

ORIGIN ≡ 1

$i := 1..200 \quad S_i := i - 100 \quad W_i := \sqrt{10000 - (S_i)^2} \quad w_i := -W_i$

$° := deg$

D :=

	1	2
1	357	47
2	313	52
3	149	67
4	315	75
5	85	57
6	78	60
7	152	50
8	146	68
9	155	70
10	203	0
11	86	30
12	235	70
13	232	70
14	293	65
15	217	70
16	217	83
17	312	60
18	128	55
19	25	80
20	234	60
21	30	80
22	14	82
23	38	85
24	349	80
25	164	45
26	234	75
27	237	55
28	236	85
29	90	80
30	346	85
31	280	15
32	140	50
33	236	80
34	20	90
35	338	85
36	7	75
37	252	80
38	86	80
39	238	75
40	40	85
41	310	30
42	349	65
43	295	75
44	344	60

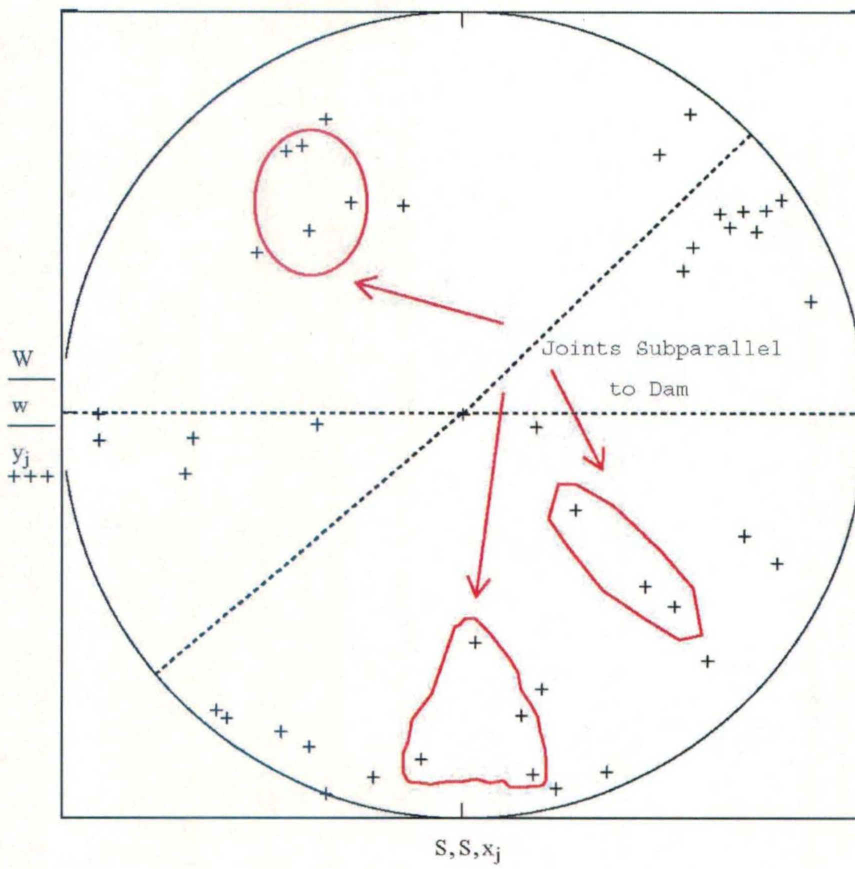
$j := 1..44 \quad ddr_j := D_{j,1} \quad dip_j := D_{j,2}$

Note: set j limit to n-th data row.

$$x_j := 100\cdot\sqrt{2}\cdot\sin\left(\frac{dip_j}{2}\cdot°\right)\cdot\sin\left[\left(ddr_j - 180\right)\cdot°\right]$$

$$y_j := 100\cdot\sqrt{2}\cdot\sin\left(\frac{dip_j}{2}\cdot°\right)\cdot\cos\left[\left(ddr_j - 180\right)\cdot°\right]$$

L.-Hem. Stereonet - Franciscan Formation



Notes:
1. DIP AND DIP DIRECTION DATA WERE COLLECTED ON MARCH 12, 2009.
2. STRUCTURES MAPPED INCLUDE BEDDING, JOINTS, AND FRACTURES.

FORMER GEORGIA PACIFIC WOOD PRODUCTS FACILITY
FORT BRAGG, CALIFORNIA
STABILITY ASSESSMENT OF THE MILL POND DAM

STEREONET PLOT OF POLES

ARCADIS

FIGURE **A-10**

$F :=$

	1	2
1	357	47
2	313	52
3	149	67
4	315	75
5	85	57
6	78	60
7	152	50
8	146	68
9	155	70
10	203	0
11	86	30
12	235	70
13	232	70
14	293	65
15	217	70
16	217	83
17	312	60
18	128	55
19	25	80
20	234	60
21	30	80
22	14	82
23	38	85
24	349	80
25	164	45
26	234	75
27	237	55
28	236	85
29	90	80
30	346	85
31	280	15
32	140	50
33	236	80
34	20	90
35	338	85
36	7	75
37	252	80
38	86	80
39	238	75
40	40	85
41	310	30
42	349	65
43	295	75
44	344	60

$k := 1 .. 44 \qquad ddr_k := F_{k,1} \qquad dip_k := F_{k,2}$

Note: set k limit to n-th data row.

$$l_k := \sin\left(dip_k \cdot {}^\circ\right) \cdot \cos\left[\left(ddr_k + 180\right) \cdot {}^\circ\right]$$

$$m_k := \sin\left(dip_k \cdot {}^\circ\right) \cdot \sin\left[\left(ddr_k + 180\right) \cdot {}^\circ\right]$$

$$n_k := \cos\left(dip_k \cdot {}^\circ\right) \qquad Suml := \sum_k l_k \qquad Summ := \sum_k m_k \qquad Sumn := \sum_k n_k$$

$$R := \sqrt{Suml^2 + Summ^2 + Sumn^2} \qquad R = 17.666$$

Mean Vector: $l_v := \frac{1}{R} \cdot Suml \qquad l_v = -0.21417$

$$m_v := \frac{1}{R} \cdot Summ \qquad m_v = 0.20664$$

$$n_v := \frac{1}{R} \cdot Sumn \qquad n_v = 0.95469$$

Mean-Vector Plane: $MV_{dip} := \operatorname{acos}(n_v) \cdot 57.2958 \qquad \boxed{MV_{dip} = 17.3}$ deg.

$$MV_{ddr} := \begin{cases} 180 - \operatorname{acos}\left(\dfrac{l_v}{\sin(MV_{dip} \cdot deg)}\right) \cdot 57.2958 & \text{if } m_v < 0 \\ 180 + \operatorname{acos}\left(\dfrac{l_v}{\sin(MV_{dip} \cdot deg)}\right) \cdot 57.2958 & \text{otherwise} \end{cases}$$

$\boxed{MV_{ddr} = 316}$ deg.

****

Notes:
1. DIP AND DIP DIRECTION DATA WERE COLLECTED ON MARCH 12, 2009.
2. STRUCTURES MAPPED INCLUDE BEDDING, JOINTS, AND FRACTURES.

FORMER GEORGIA PACIFIC WOOD PRODUCTS FACILITY
FORT BRAGG, CALIFORNIA
STABILITY ASSESSMENT OF THE MILL POND DAM

FRACTURE SET CALCULATIONS

ARCADIS

FIGURE
A-11

APPENDIX B

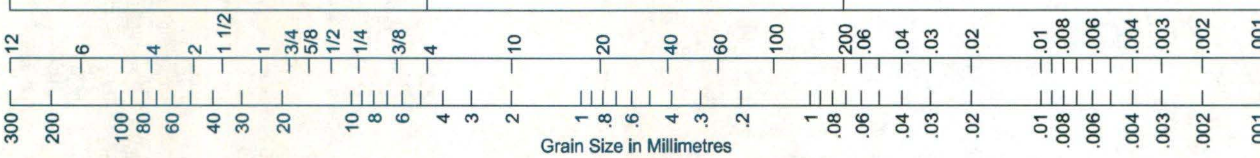
ARCADIS

Appendix B

Geotechnical Laboratory Test Methods and Results

Grain Size

Size of Opening in Inches	U.S. Standard Sieve Size	Grain Size in Millimetres



COBBLES	GRAVEL	SAND	SILT and CLAY
Coarse-Grained Soils			Fine-Grained Soils

Coarse-Grained Soils

G W	G P	G M	G C	S W	S P	S M	S C
GRAVEL with <5% fines		* GRAVEL with >12% fines		GRAVEL with <5% fines		* GRAVEL with >12% fines	
More than 50% (by weight) of coarse fraction larger than No. 4				More than 50% (by weight) of coarse fraction smaller than No. 4			
More than 50% (by weight) larger than No. 200 sieve							

G W and S W: $\left(\frac{D_{60}}{D_{10}}\right)$ >4 for G W, >6 for S W & $1 \leq \left(\frac{(D_{30})^2}{D_{10} \times D_{60}}\right) \leq 3$

G P and S P: GRAVEL or SAND not meeting requirements for G W and S W

G M and S M: Atterberg limits below A line or PI <4

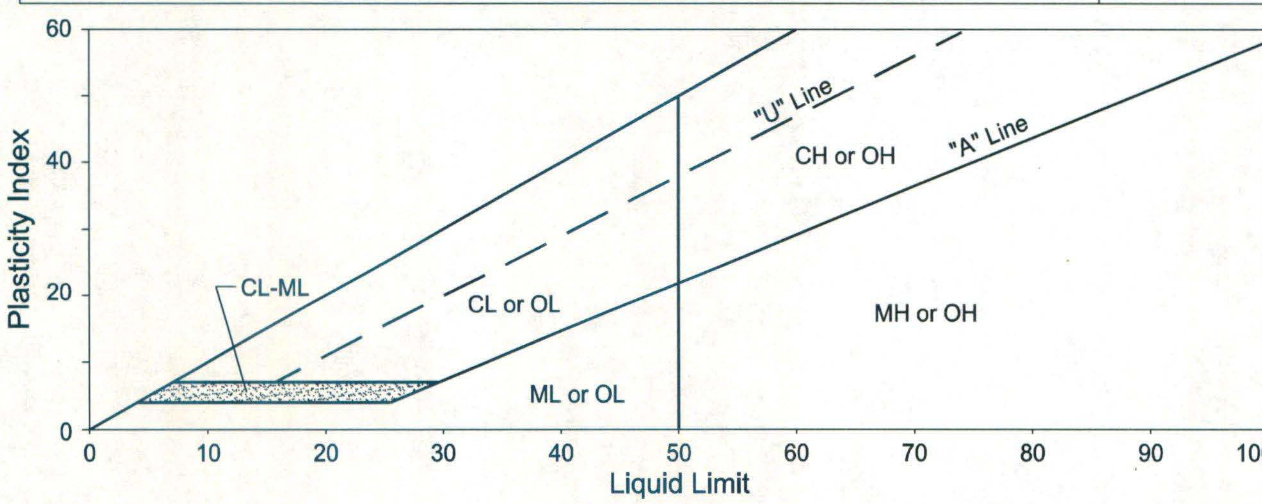
G C and S C: Atterberg limits above A line and PI >7

* Coarse-grained soils with fines contents between 5 and 12% require dual classification using two group symbols.

D_{10}, D_{30}, and D_{60} are the particle-size diameters corresponding to 10, 30, and 60% passing on the cumulative particle-size distribution curve.

Fine-Grained Soils

M L	C L	O L	M H	C H	O H	Pt
SILT	CLAY	Organic	SILT	CLAY	Organic	Peat
Liquid Limit <50%			Liquid Limit >50%			
More than 50% (by weight) smaller than No. 200 sieve						



FORMER GEORGIA PACIFIC WOOD PRODUCTS FACILTY, FORT BRAGG,CALIFORNIA

STABILITY ASSESSMENT OF THE MILL POND DAM

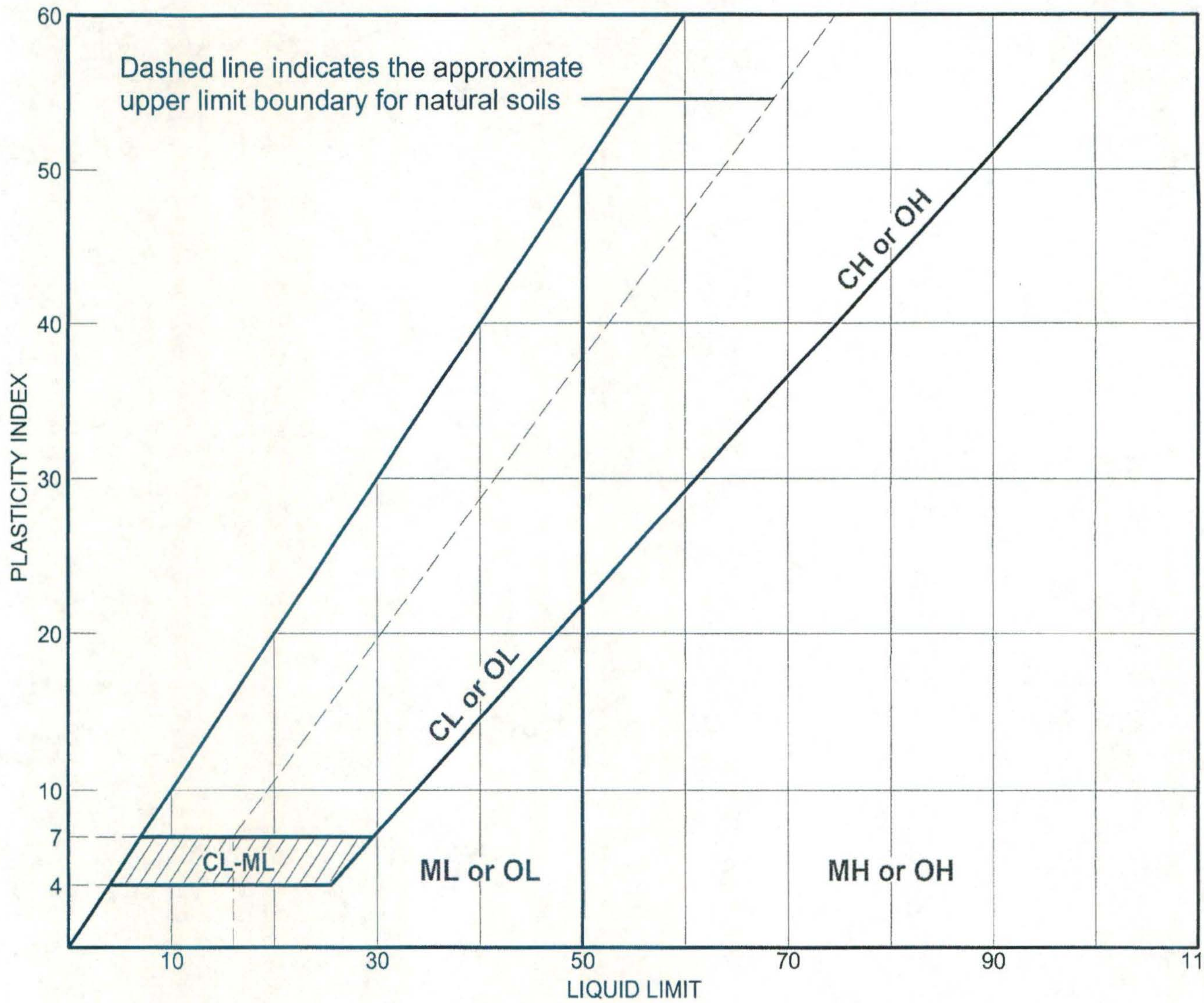
UNIFIED SOIL CLASSIFICATION SYSTEM

ARCADIS

FIGURE

B-1

LIQUID AND PLASTIC LIMITS TEST REPORT



SOIL DATA								
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	NATURAL WATER CONTENT (%)	PLASTIC LIMIT (%)	LIQUID LIMIT (%)	PLASTICITY INDEX (%)	USCS
●	OUE-GT-003	9	22.5' to 24.0	13.6	NP	NV	NP	
■	OUE-GT-003	11	27.5' to 29.0'	8.3	NP	NV	NP	

LIQUID AND PLASTIC LIMITS TEST REPORT

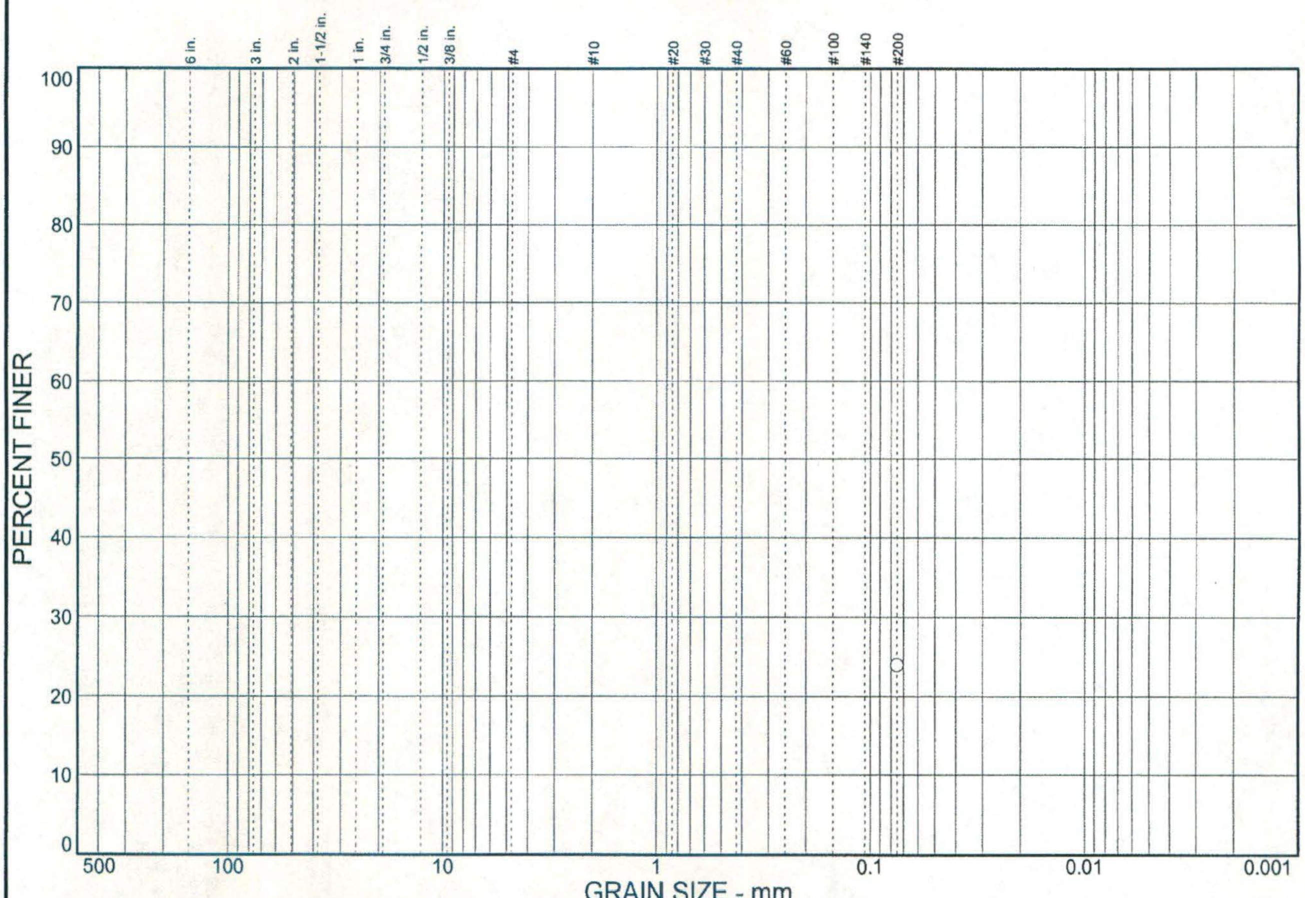
KLEINFELDER, INC.

Client: ARCADIS

Project: Mill Pond Dam
Fort Bragg, CA

Project No.: 102522

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
						24	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	24		

* (no specification provided)

Soil Description

Silty sand
Moisture Content: 28.9%
Laboratory No.: 8638

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

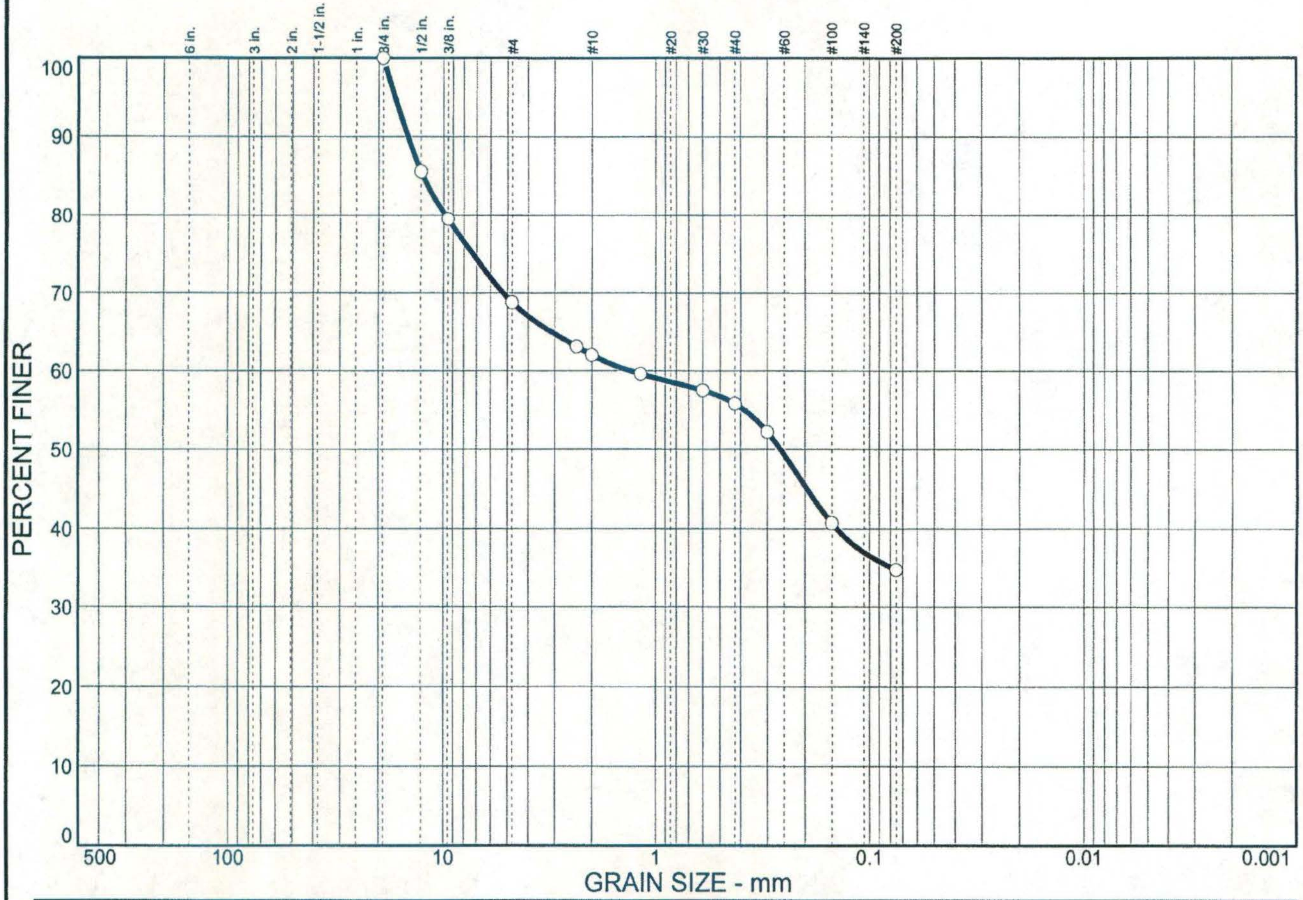
Sampled By: Bill Copeland (ARCADIS)
Tested By: A. Stirbys
Reviewed By: J. Revard, CET

Sample No.: 3 **Source of Sample:** OUE-GT-001 **Date:** 3/13/09
Location: Boring ID: OUE-GT-001 **Elev./Depth:** 7.5' to 9.0'

KLEINFELDER, INC.

Client: ARCADIS
Project: Mill Pond Dam
Fort Bragg, CA
Project No: 102522

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0	0	31	7	6	21	35	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4 in.	100		
1/2 in.	86		
3/8 in.	80		
#4	69		
#8	63		
#10	62		
#16	60		
#30	58		
#40	56		
#50	52		
#100	41		
#200	35		

* (no specification provided)

Soil Description

Silty sand with gravel
Moisture Content: 17.3%
Laboratory No.: 8639

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 12.4 D_{60}= 1.32 D_{50}= 0.260
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO= A-2-4(0)

Remarks

Sampled By: Bill Copeland (ARCADIS)
Tested By: A. Stirbys
Reviewed By: J. Revard, CET

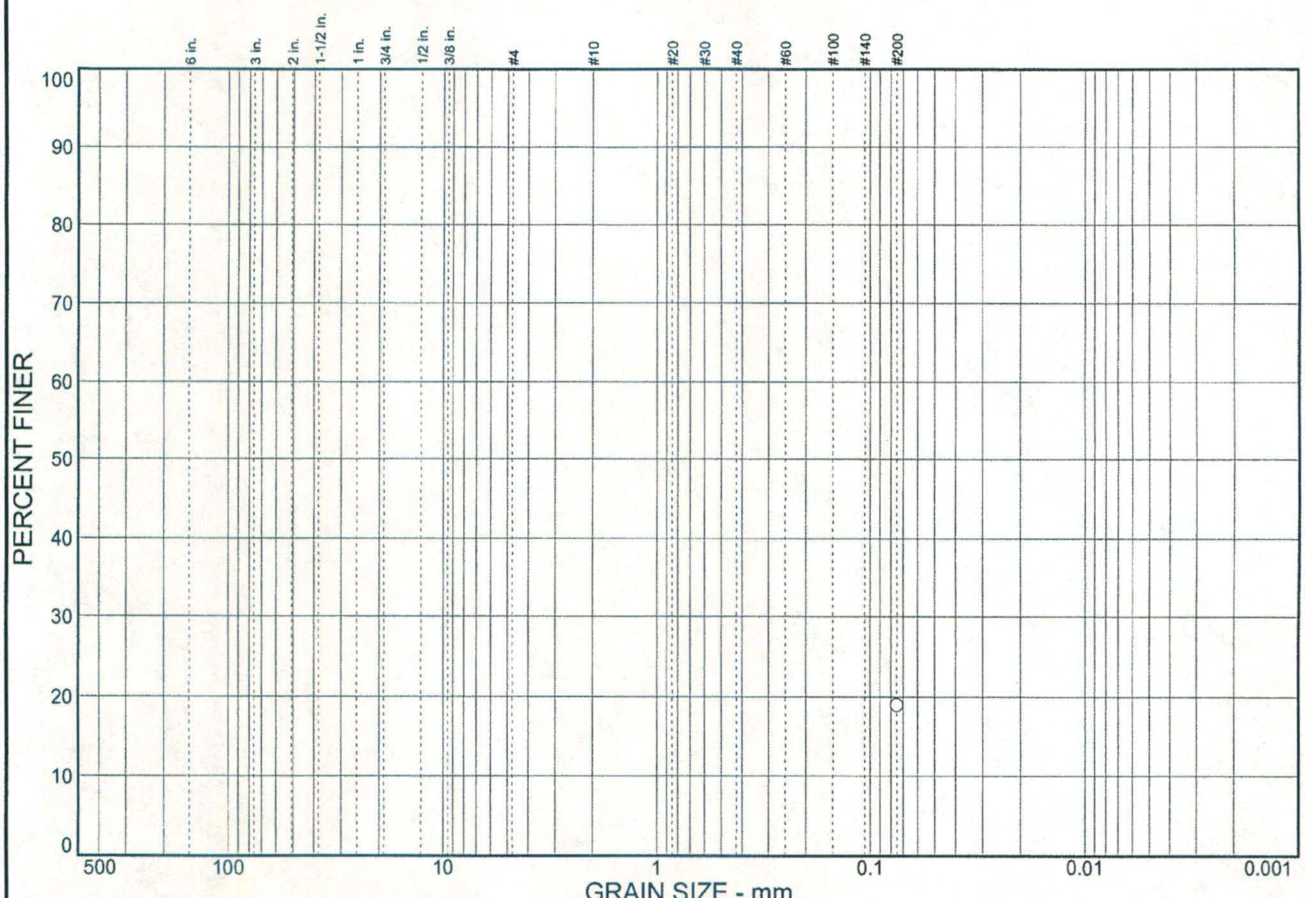
Sample No.: 5 **Source of Sample:** OUE-GT-001 **Date:** 3/13/09
Location: Boring ID: OUE-GT-001 **Elev./Depth:** 12.5' to 14.0'

KLEINFELDER, INC.

Client: ARCADIS
Project: Mill Pond Dam
Fort Bragg, CA
Project No: 102522

FIGURE B-4

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
						19	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	19		

* (no specification provided)

Soil Description

Silty sand
Moisture Content: 25.8%
Laboratory No.: 8640

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

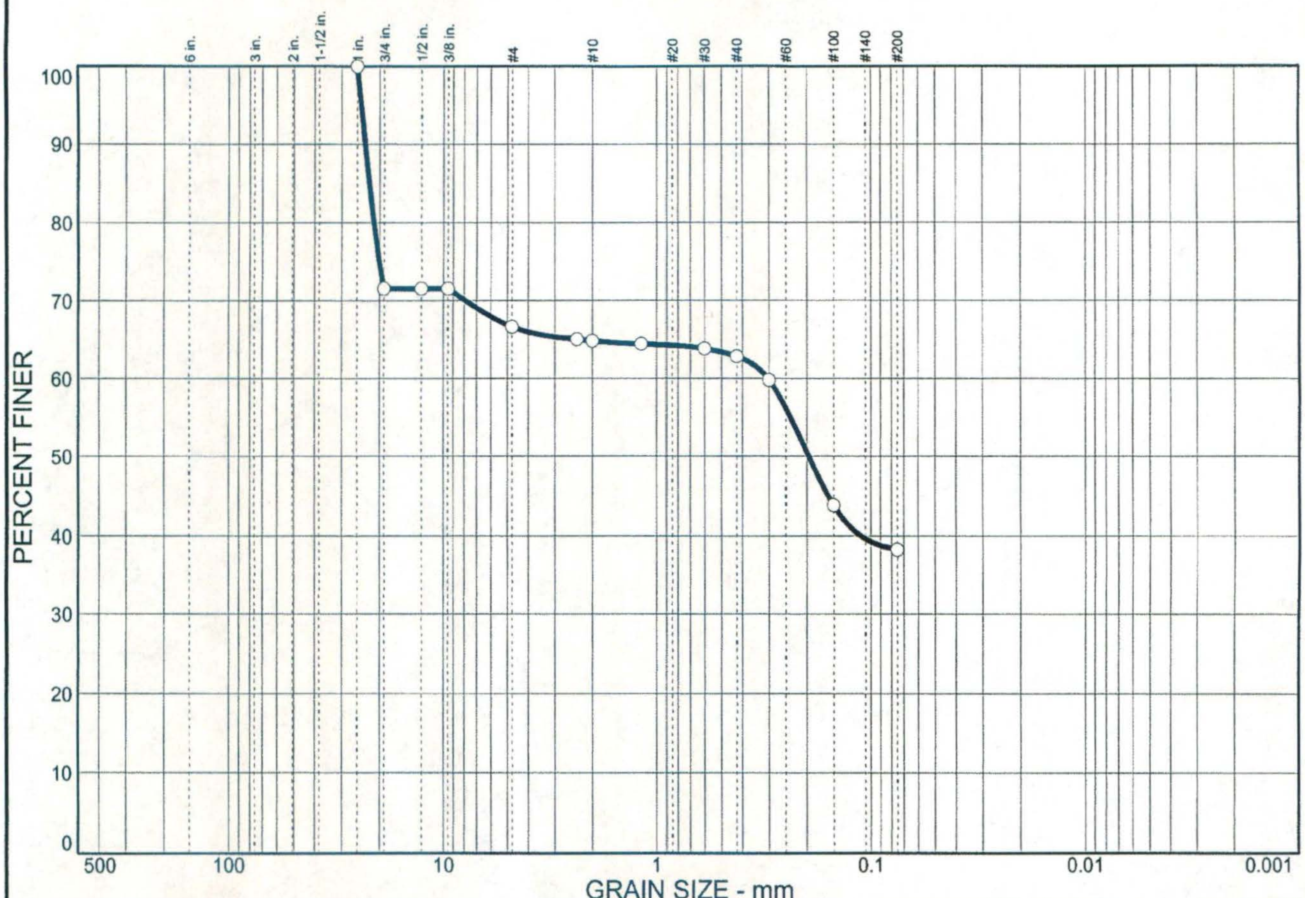
Sampled By: Bill Copeland (ARCADIS)
Tested By: A. Stirbys
Reviewed By: J. Revard, CET

Sample No.: 6 **Source of Sample:** OUE-GT-001 **Date:** 3/13/09
Location: Boring ID: OUE-GT-001 **Elev./Depth:** 15.0' to 16.5'

KLEINFELDER, INC.

Client: ARCADIS
Project: Mill Pond Dam
Fort Bragg, CA
Project No: 102522

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0	29	4	2	2	24	38	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100		
3/4 in.	72		
1/2 in.	72		
3/8 in.	72		
#4	67		
#8	65		
#10	65		
#16	64		
#30	64		
#40	63		
#50	60		
#100	44		
#200	38		

* (no specification provided)

Soil Description

Silty gravel with sand
Moisture Content: 26.0%
Laboratory No.: 8641

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 22.3 D_{60}= 0.304 D_{50}= 0.196
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= GM AASHTO= A-4(0)

Remarks

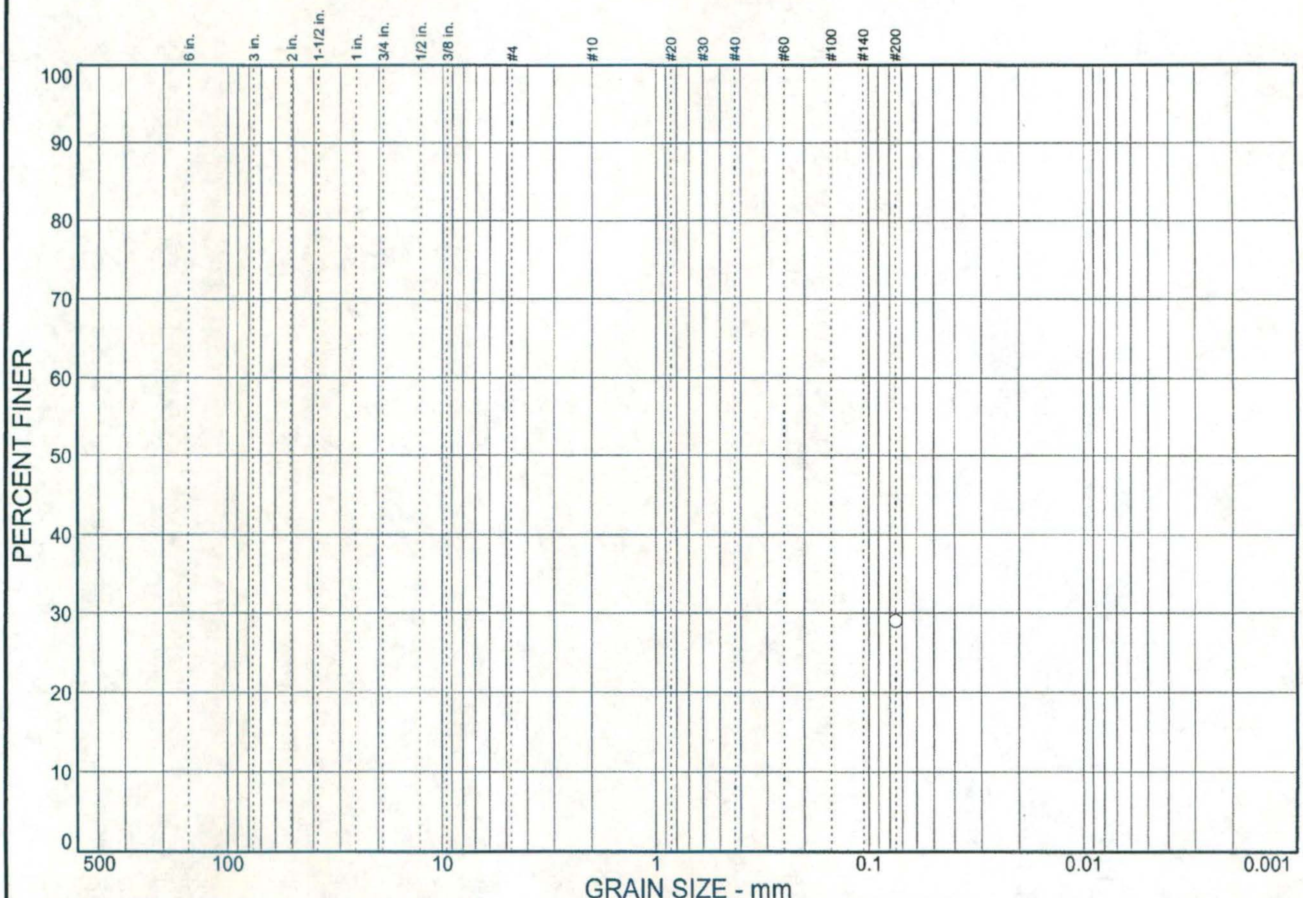
Sampled By: Bill Copeland (ARCADIS)
Tested By: A. Stirbys
Reviewed By: J. Revard, CET

Sample No.: 8 **Source of Sample:** OUE-GT-001 **Date:** 3/13/09
Location: Boring ID: OUE-GT-001 **Elev./Depth:** 20.0' to 21.5'

KLEINFELDER, INC.

Client: ARCADIS
Project: Mill Pond Dam
Fort Bragg, CA
Project No: 102522

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
						29	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	29		

* (no specification provided)

Soil Description

Silty sand
Moisture Content: 25.8%
Laboratory No.: 8646

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

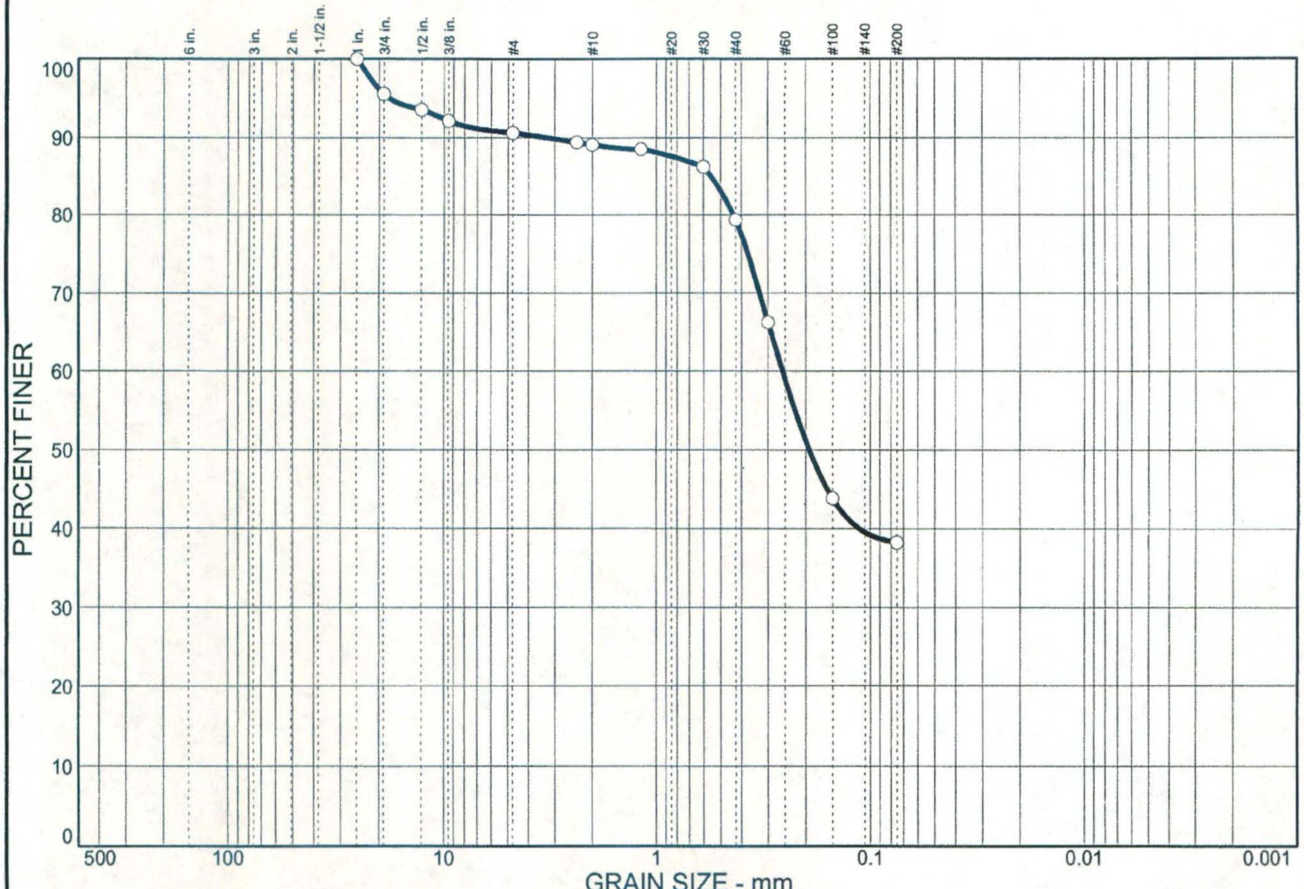
Sampled By: Bill Copeland (ARCADIS)
Tested By: A. Stirbys
Reviewed By: J. Revard, CET

Sample No.: 3 **Source of Sample:** OUT-GT-002 **Date:** 3/13/09
Location: Boring ID: OUE-GT-002 **Elev./Depth:** 7.5' to 9.0'

KLEINFELDER, INC.

Client: ARCADIS
Project: Mill Pond Dam
Fort Bragg, CA
Project No: 102522

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0	5	4	2	10	40	38	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100		
3/4 in.	96		
1/2 in.	94		
3/8 in.	92		
#4	91		
#8	89		
#10	89		
#16	89		
#30	86		
#40	79		
#50	66		
#100	44		
#200	38		

* (no specification provided)

Soil Description

Silty sand
Moisture Content: 35.0%
Laboratory No.: 8647

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.555 D_{60}= 0.257 D_{50}= 0.192
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO= A-4(0)

Remarks

Sampled By: Bill Copeland (ARCADIS)
Tested By: A. Stirbys
Reviewed By: J. Revard, CET

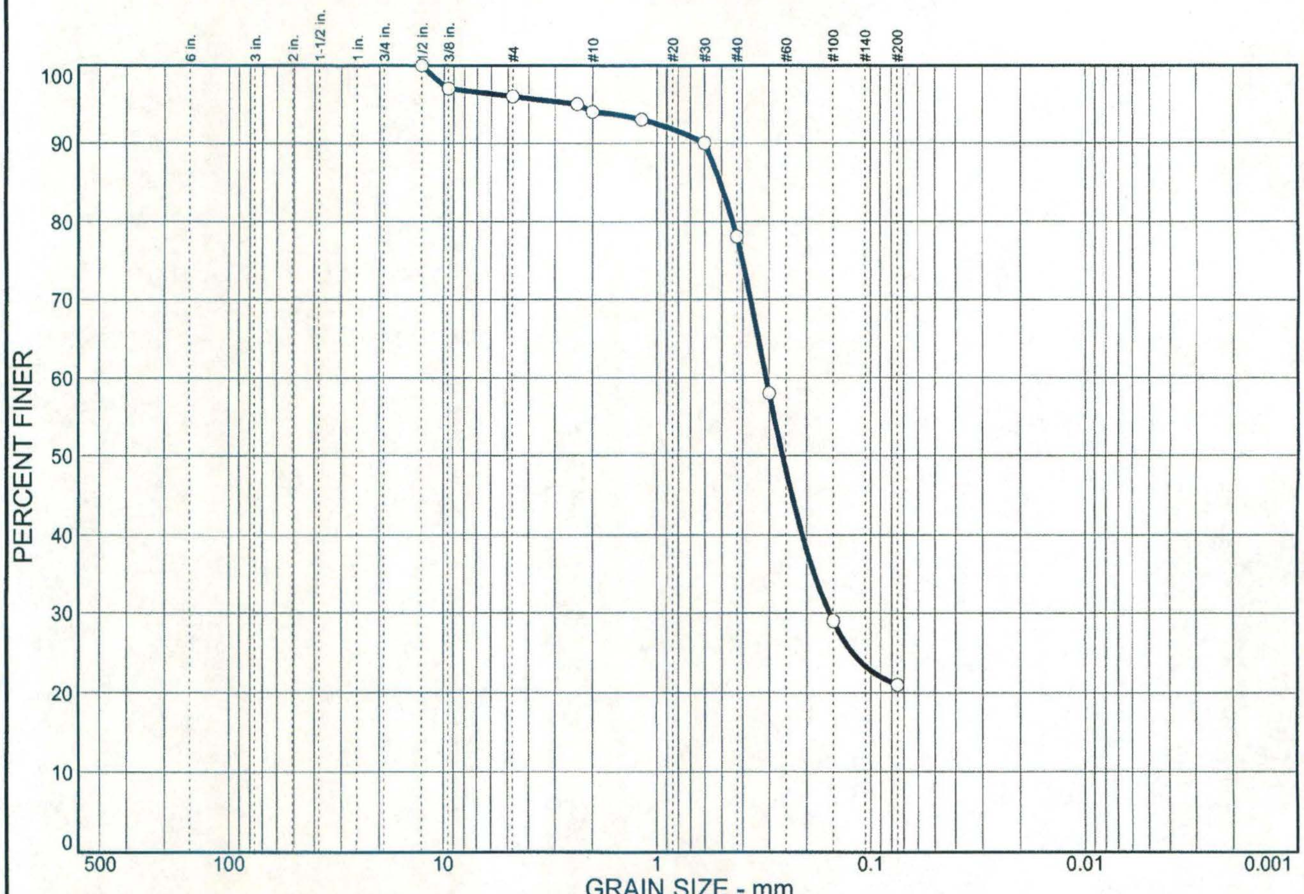
Sample No.: 4 **Source of Sample:** OUT-GT-002 **Date:** 3/13/09
Location: Boring ID: OUE-GT-002 **Elev./Depth:** 10.0' to 11.5'

KLEINFELDER, INC.

Client: ARCADIS
Project: Mill Pond Dam
Fort Bragg, CA
Project No: 102522

FIGURE B-8

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0	0	4	2	16	57	21	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1/2 in.	100		
3/8 in.	97		
#4	96		
#8	95		
#10	94		
#16	93		
#30	90		
#40	78		
#50	58		
#100	29		
#200	21		

* (no specification provided)

Soil Description

Silty sand
Moisture Content: 29.8%
Laboratory No.: 8648

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.507 D_{60}= 0.310 D_{50}= 0.260
D_{30}= 0.156 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO= A-2-4(0)

Remarks

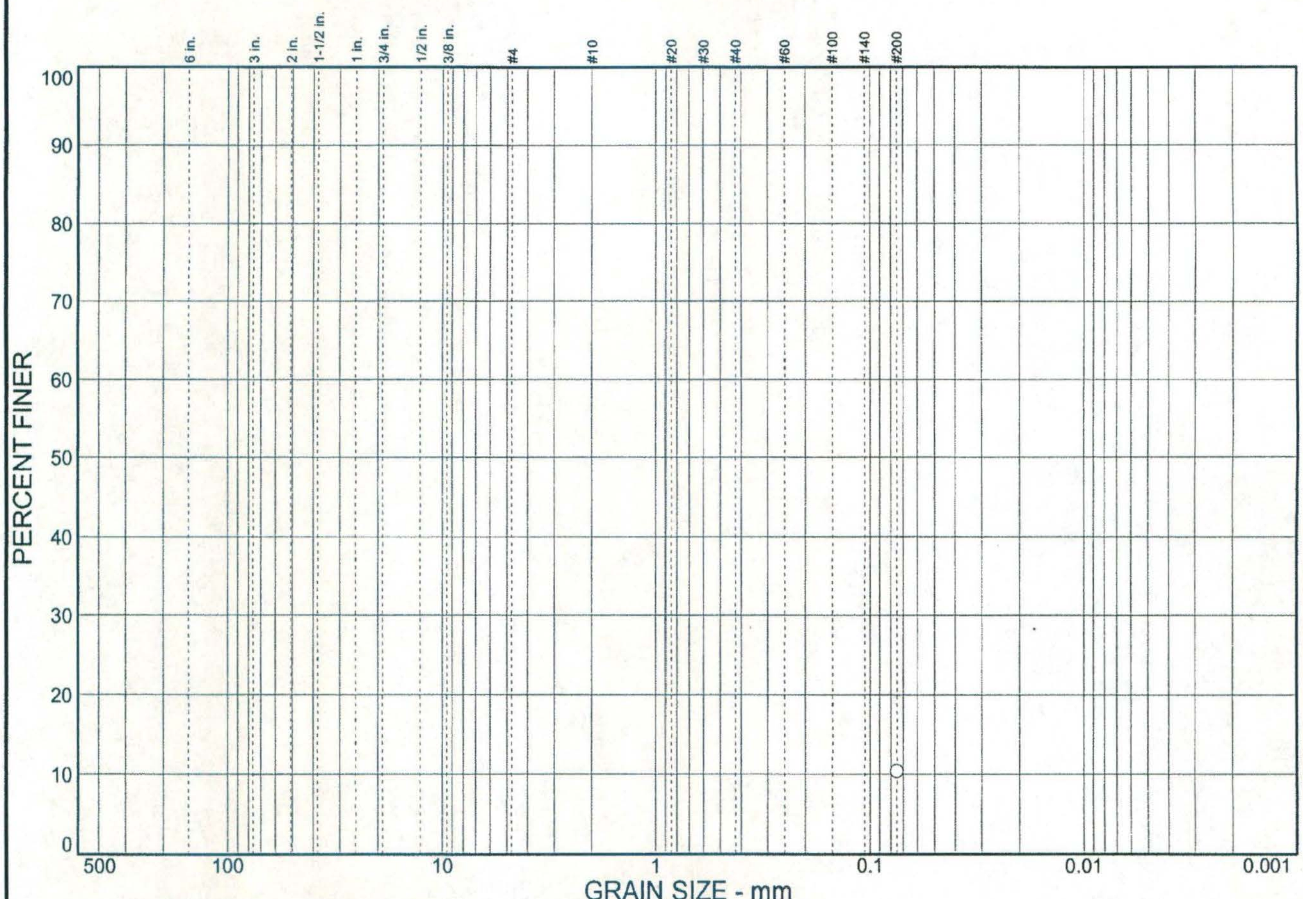
Sampled By: Bill Copeland (ARCADIS)
Tested By: A. Stirbys
Reviewed By: J. Revard, CET

Sample No.: 5 **Source of Sample:** OUT-GT-002 **Date:** 3/13/09
Location: Boring ID: OUE-GT-002 **Elev./Depth:** 12.5' to 14.0'

KLEINFELDER, INC.

Client: ARCADIS
Project: Mill Pond Dam
Fort Bragg, CA
Project No: 102522

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
						10	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	10		

* (no specification provided)

Soil Description

Poorly graded sand with silt
Moisture Content: 19.5%
Laboratory No.: 8649

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SP-SM AASHTO=

Remarks

Sampled By: Bill Copeland (ARCADIS)
Tested By: A. Stirbys
Reviewed By: J. Revard, CET

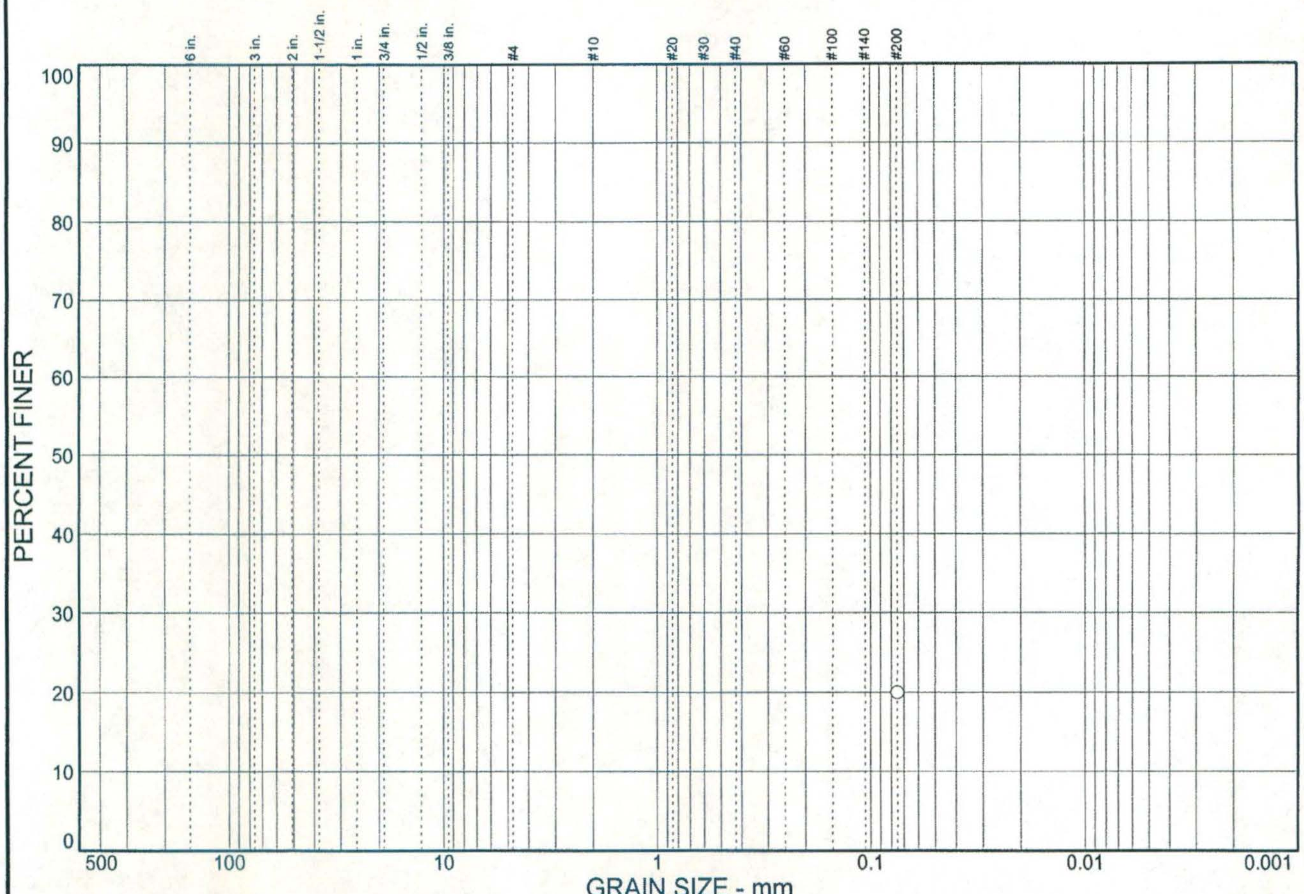
Sample No.: 6 **Source of Sample:** OUT-GT-002 **Date:** 3/13/09
Location: Boring ID: OUE-GT-002 **Elev./Depth:** 15.0' to 16.5'

KLEINFELDER, INC.

Client: ARCADIS
Project: Mill Pond Dam
Fort Bragg, CA
Project No: 102522

FIGURE B-10

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
						20	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	20		

* (no specification provided)

Soil Description

Silty sand
Moisture Content: 14.1%
Laboratory No.: 8654

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

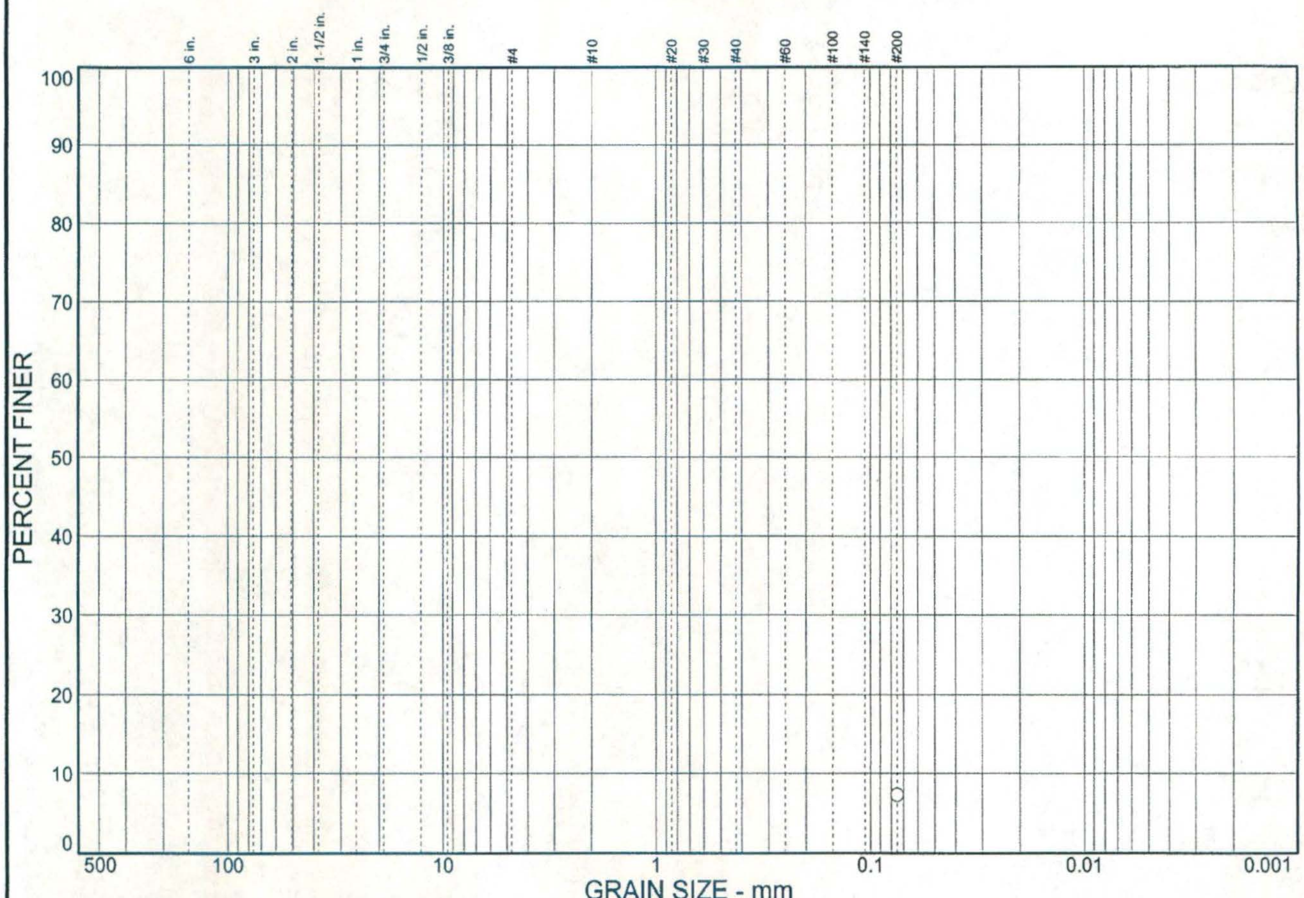
Sampled By: Bill Copeland (ARCADIS)
Tested By: A. Stirbys
Reviewed By: J. Revard, CET

Sample No.: 3 **Source of Sample:** OUE-GT-003 **Date:** 3/13/09
Location: Boring ID: OUE-GT-003 **Elev./Depth:** 7.5' to 9.0'

KLEINFELDER, INC.

Client: ARCADIS
Project: Mill Pond Dam
Fort Bragg, CA
Project No: 102522

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
						7	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	7.3		

* (no specification provided)

Soil Description

Poorly graded sand with silt
Moisture Content: 19.5%
Laboratory No.: 8655

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SP-SM AASHTO=

Remarks

Sampled By: Bill Copeland (ARCADIS)
Tested By: A. Stirbys
Reviewed By: J. Revard, CET

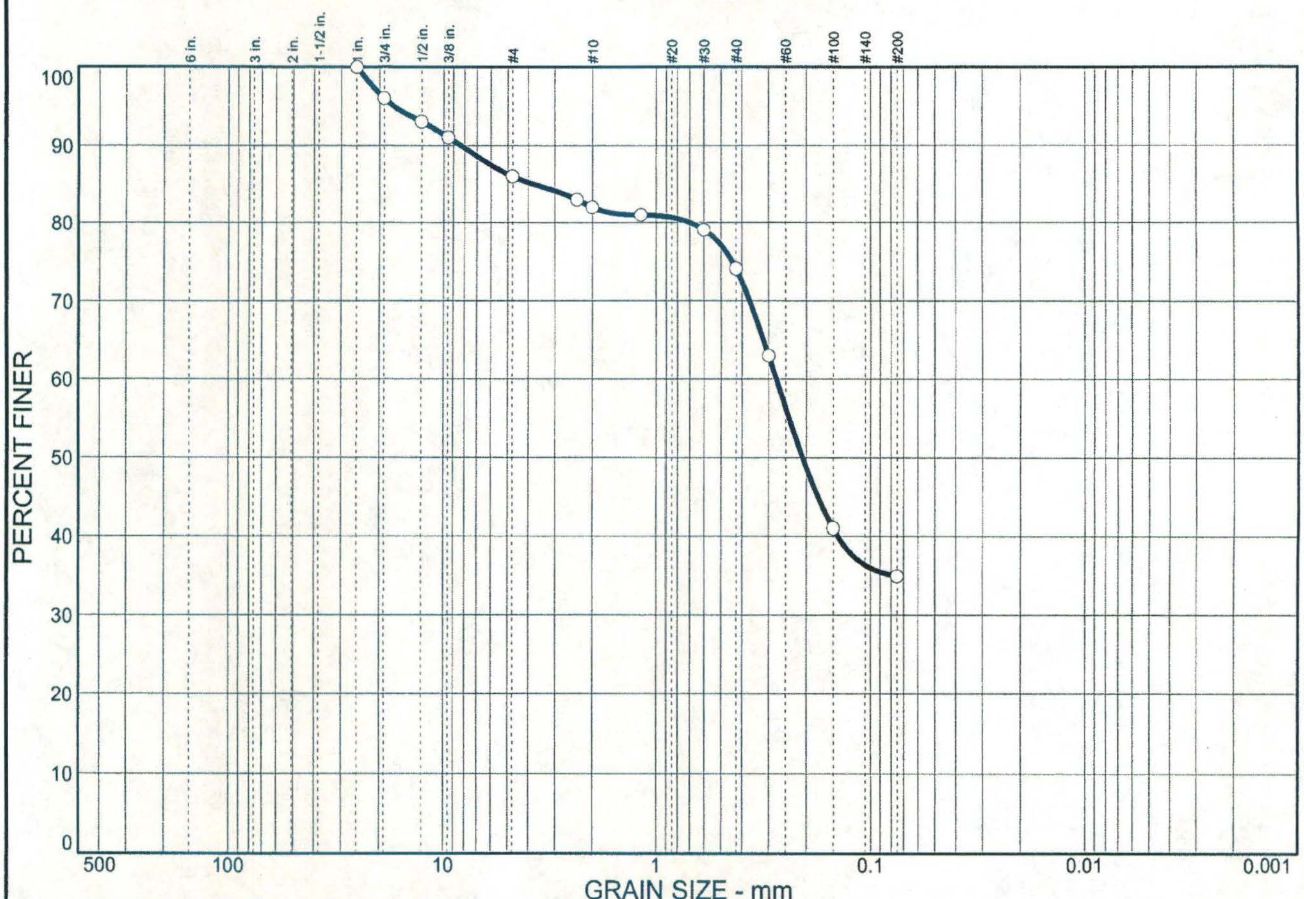
Sample No.: 4 **Source of Sample:** OUE-GT-003 **Date:** 3/13/09
Location: Boring ID: OUE-GT-003 **Elev./Depth:** 10.0' to 11.5'

KLEINFELDER, INC.

Client: ARCADIS
Project: Mill Pond Dam
Fort Bragg, CA
Project No: 102522

FIGURE B-12

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0	4	10	4	8	39	35	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
1 in.	100		
3/4 in.	96		
1/2 in.	93		
3/8 in.	91		
#4	86		
#8	83		
#10	82		
#16	81		
#30	79		
#40	74		
#50	63		
#100	41		
#200	35		

* (no specification provided)

Soil Description

Silty sand
Moisture Content: 23.4%
Laboratory No.: 8656

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 3.78 D_{60}= 0.276 D_{50}= 0.208
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO= A-2-4(0)

Remarks

Sampled By: Bill Copeland (ARCADIS)
Tested By: A. Stirbys
Reviewed By: J. Revard, CET

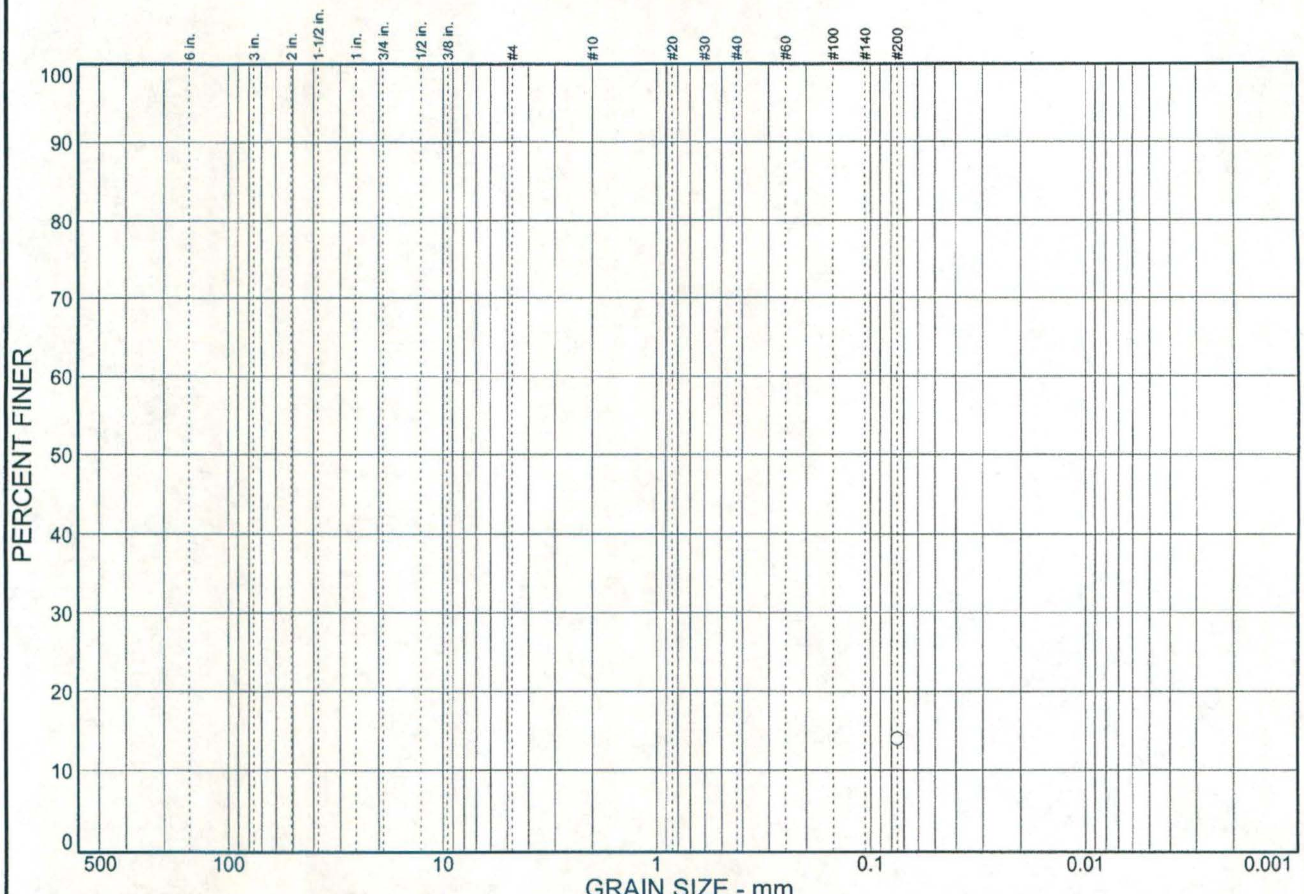
Sample No.: 5 **Source of Sample:** OUE-GT-003 **Date:** 3/13/09
Location: Boring ID: OUE-GT-003 **Elev./Depth:** 12.5' to 14.0'

KLEINFELDER, INC.

Client: ARCADIS
Project: Mill Pond Dam
Fort Bragg, CA
Project No: 102522

FIGURE B-13

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
						14	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	14		

* (no specification provided)

Soil Description

Silty sand
Moisture Content: 32.3%
Laboratory No.: 8657

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

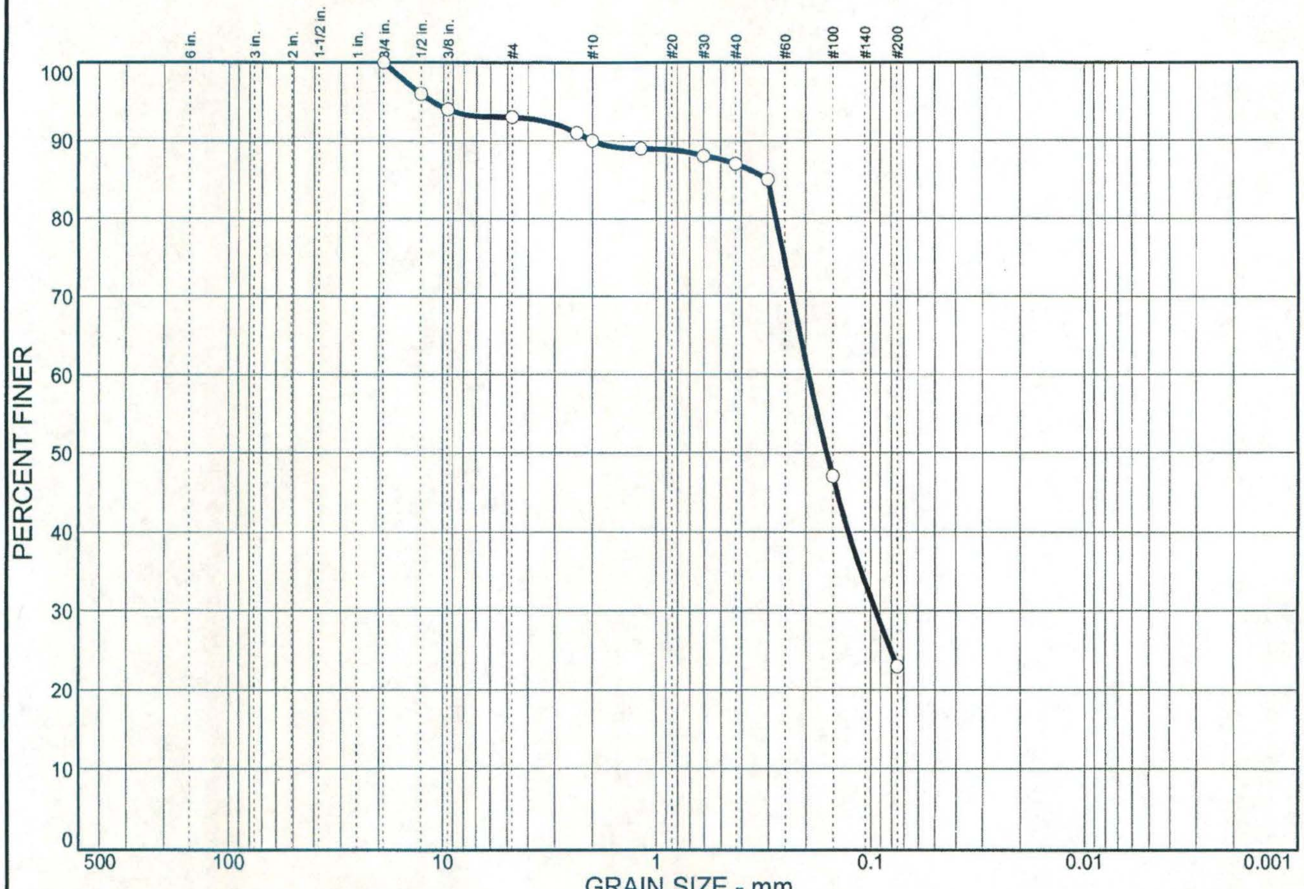
Sampled By: Bill Copeland (ARCADIS)
Tested By: A. Stirbys
Reviewed By: J. Revard, CET

Sample No.: 6 **Source of Sample:** OUE-GT-003 **Date:** 3/13/09
Location: Boring ID: OUE-GT-003 **Elev./Depth:** 15.0' to 16.5'

KLEINFELDER, INC.

Client: ARCADIS
Project: Mill Pond Dam
Fort Bragg, CA
Project No: 102522

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0	0	7	3	3	64	23	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4 in.	100		
1/2 in.	96		
3/8 in.	94		
#4	93		
#8	91		
#10	90		
#16	89		
#30	88		
#40	87		
#50	85		
#100	47		
#200	23		

* (no specification provided)

Soil Description

Silty sand
Moisture Content: 23.4%
Laboratory No.: 8658

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 0.300 D_{60}= 0.194 D_{50}= 0.160
D_{30}= 0.0948 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO= A-2-4(0)

Remarks

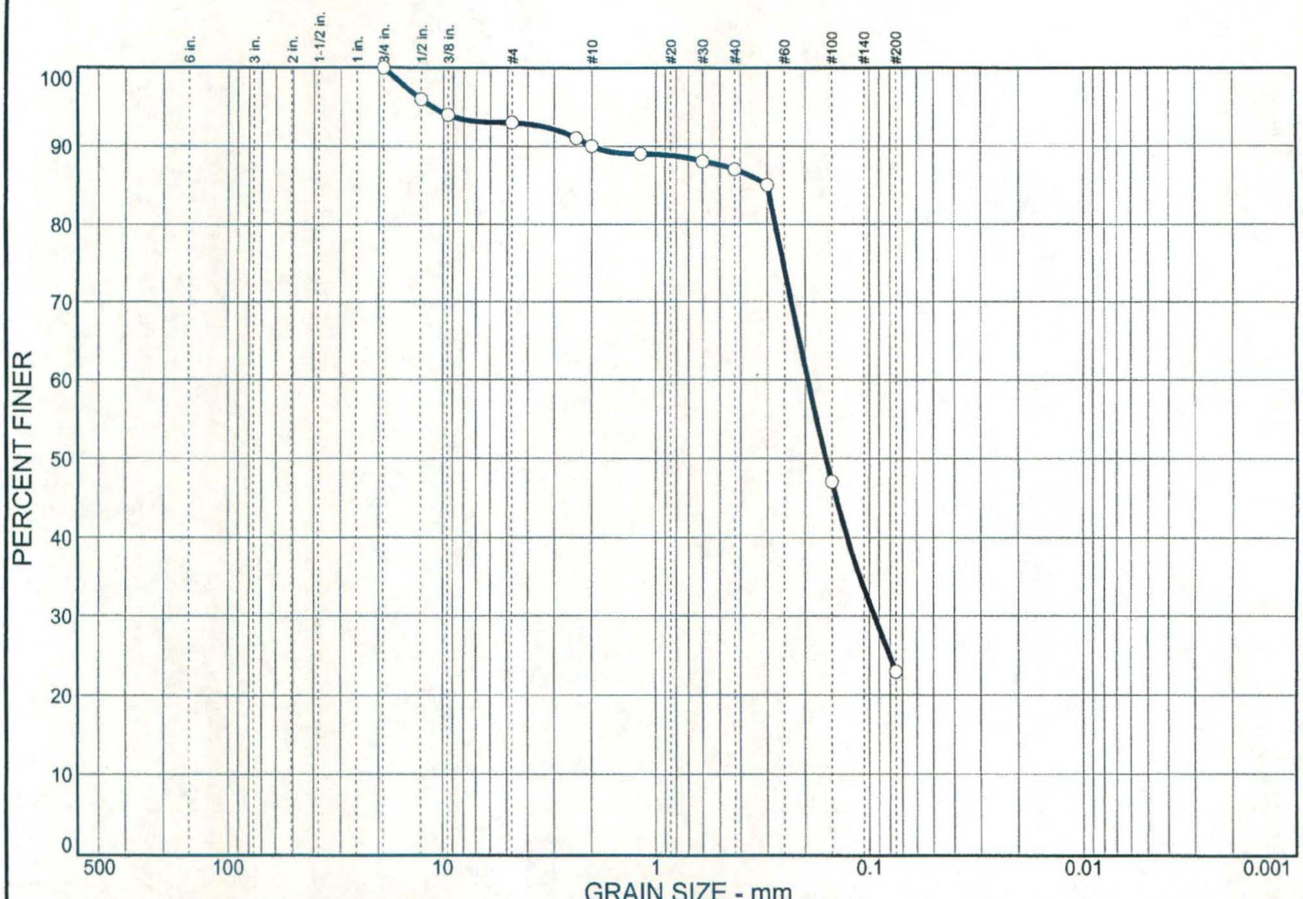
Sampled By: Bill Copeland (ARCADIS)
Tested By: A. Stirbys
Reviewed By: J. Revard, CET

Sample No.: 7 **Source of Sample:** OUE-GT-003 **Date:** 3/13/09
Location: Boring ID: OUE-GT-003 **Elev./Depth:** 17.5' to 19.0'

KLEINFELDER, INC.

Client: ARCADIS
Project: Mill Pond Dam
Fort Bragg, CA
Project No: 102522

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0	0	7	3	3	64	23	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/4 in.	100		
1/2 in.	96		
3/8 in.	94		
#4	93		
#8	91		
#10	90		
#16	89		
#30	88		
#40	87		
#50	85		
#100	47		
#200	23		

* (no specification provided)

Soil Description

Silty sand
Moisture Content: 26.9%
Laboratory No.: 8666

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= 0.300 D_{60}= 0.194 D_{50}= 0.160
D_{30}= 0.0948 D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO= A-2-4(0)

Remarks

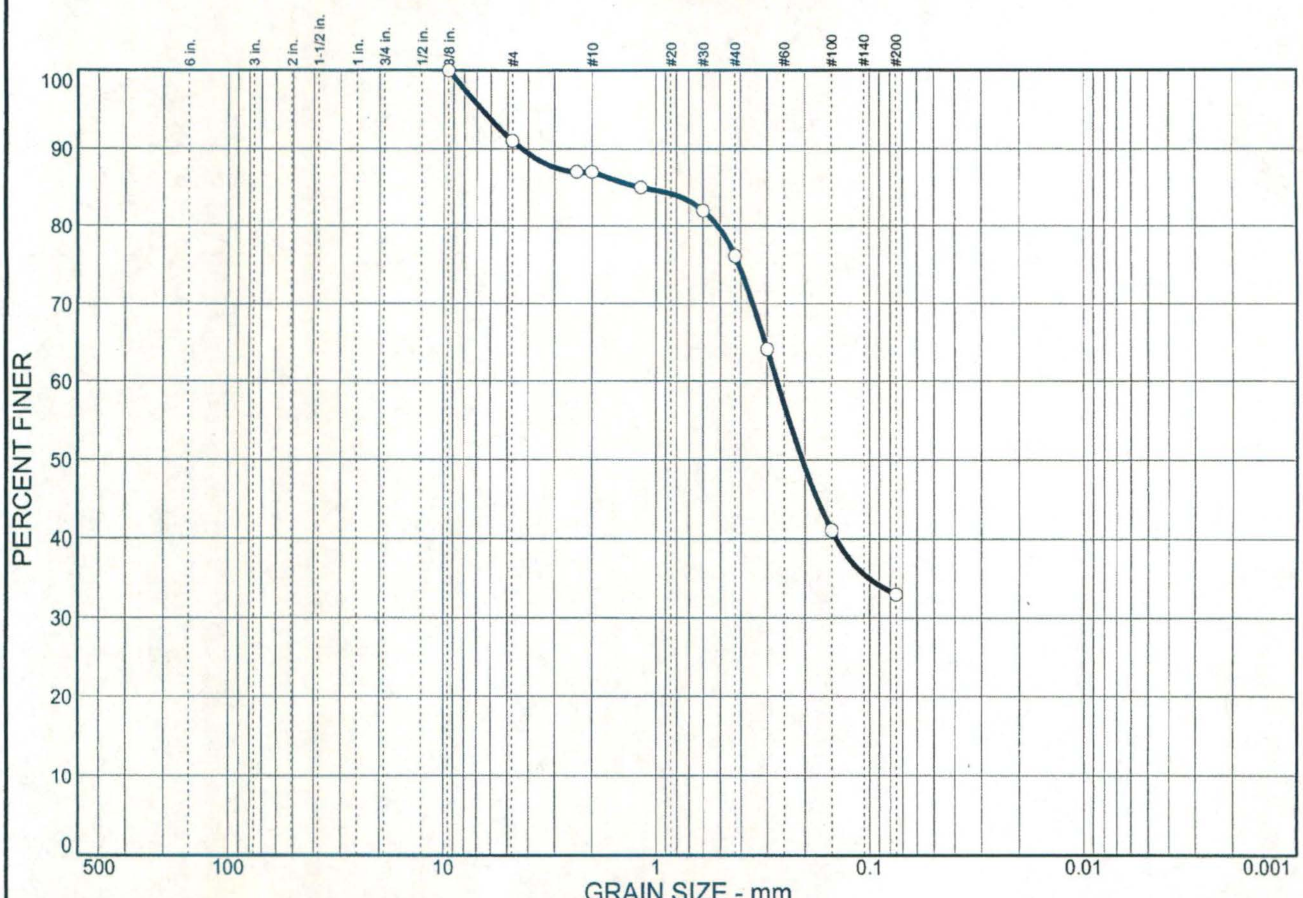
Sampled By: Bill Copeland (ARCADIS)
Tested By: A. Stirbys
Reviewed By: J. Revard, CET

Sample No.: 3 **Source of Sample:** OUE-GT-004 **Date:** 3/13/09
Location: Boring ID: OUE-GT-004 **Elev./Depth:** 7.5' to 9.0'

KLEINFELDER, INC.

Client: ARCADIS
Project: Mill Pond Dam
Fort Bragg, CA
Project No: 102522

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
0	0	9	4	11	43	33	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
3/8 in.	100		
#4	91		
#8	87		
#10	87		
#16	85		
#30	82		
#40	76		
#50	64		
#100	41		
#200	33		

* (no specification provided)

Soil Description

Silty sand
Moisture Content: 37.9%
Laboratory No.: 8667

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= 1.18 D_{60}= 0.270 D_{50}= 0.205
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO= A-2-4(0)

Remarks

Sampled By: Bill Copeland (ARCADIS)
Tested By: A. Stirbys
Reviewed By: J. Revard, CET

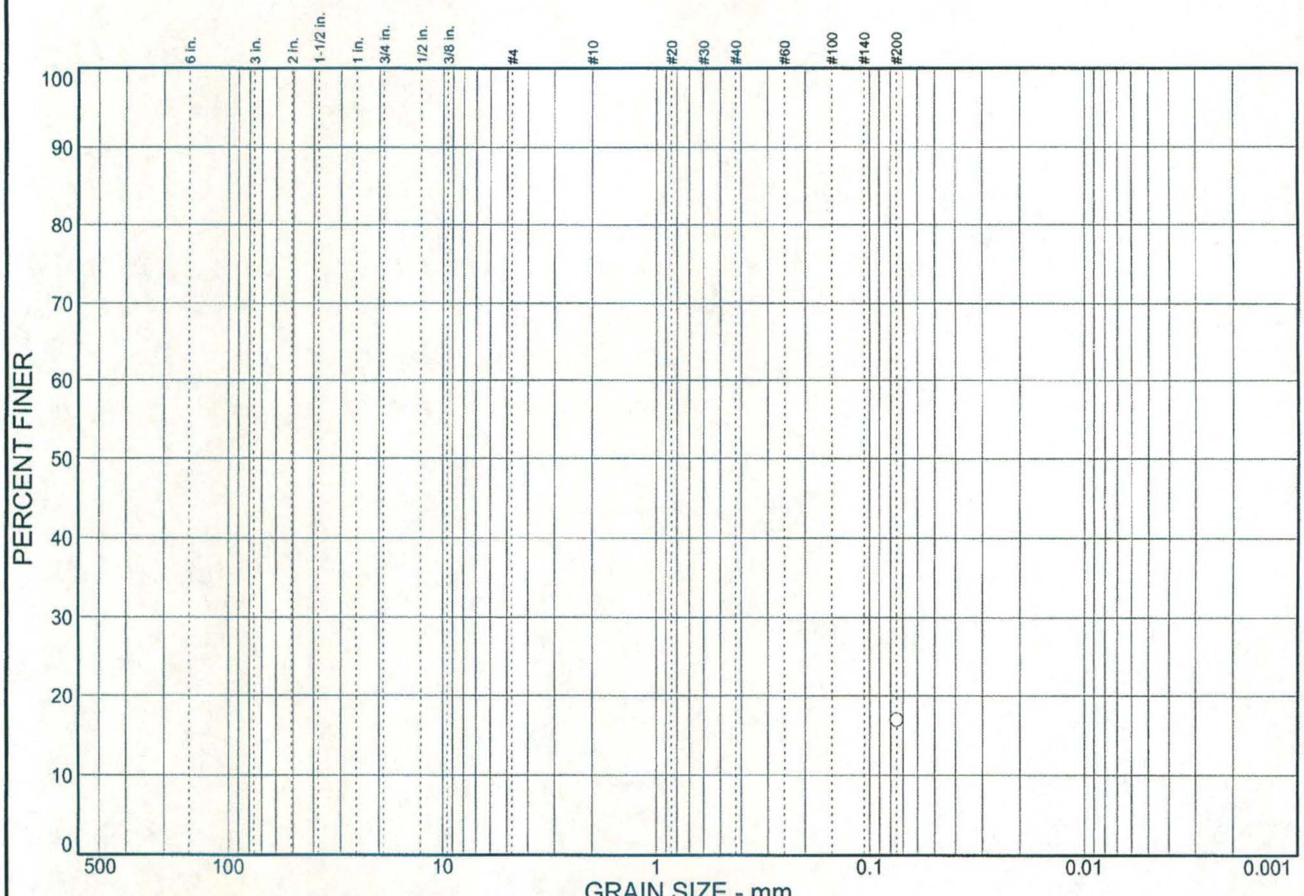
Sample No.: 5 **Source of Sample:** OUE-GT-004 **Date:** 3/13/09
Location: Boring ID: OUE-GT-004 **Elev./Depth:** 12.5' to 14.0'

KLEINFELDER, INC.

Client: ARCADIS
Project: Mill Pond Dam
Fort Bragg, CA
Project No: 102522

FIGURE B-17

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
						17	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	17		

* (no specification provided)

Soil Description

Silty sand
Moisture Content: 24.6%
Laboratory No.: 8668

Atterberg Limits

PL= LL= PI=

Coefficients

D_{85}= D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= SM AASHTO=

Remarks

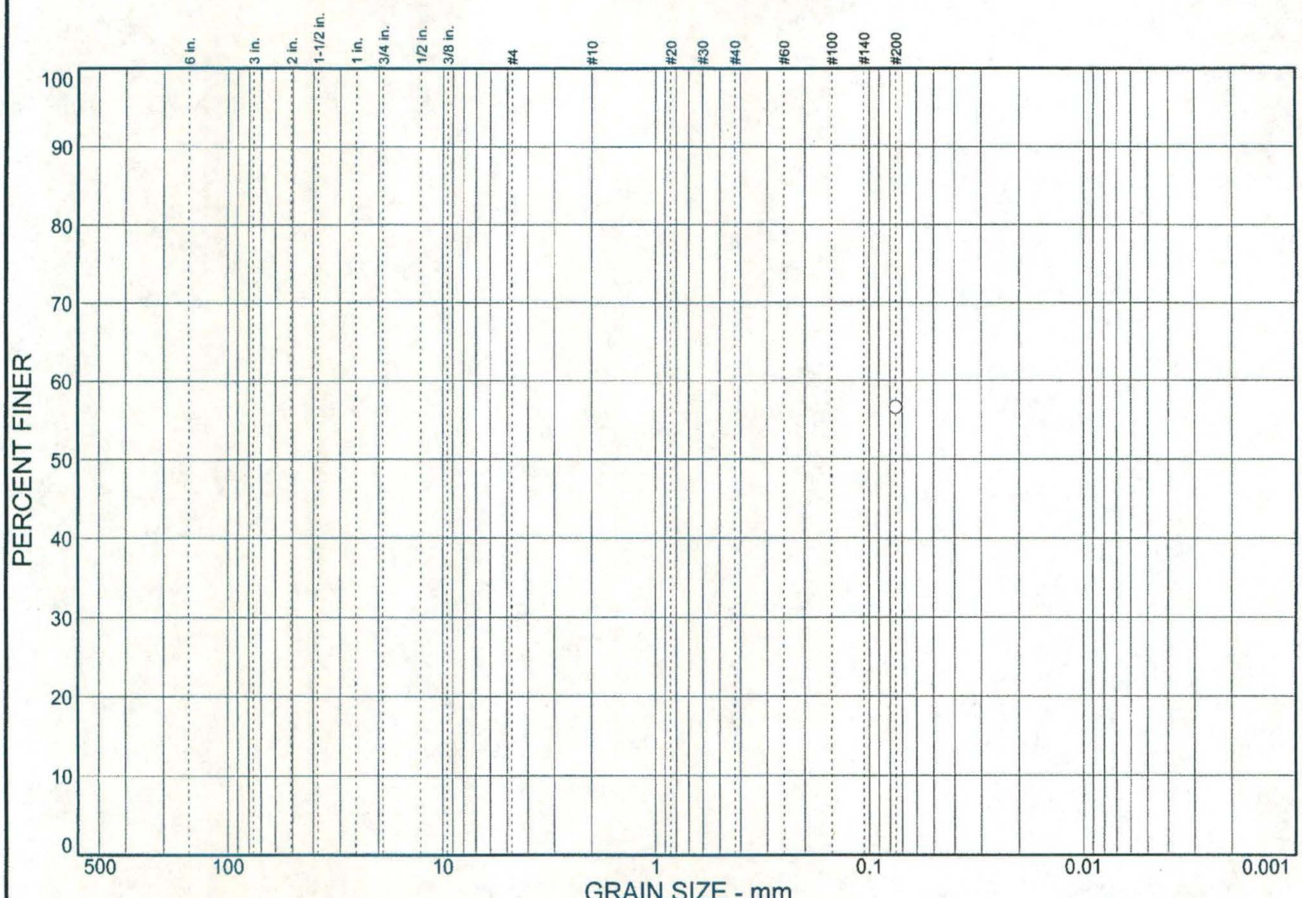
Sampled By: Bill Copeland (ARCADIS)
Tested By: A. Stirbys
Reviewed By: J. Revard, CET

Sample No.: 8 **Source of Sample:** OUE-GT-004 **Date:** 3/13/09
Location: Boring ID: OUE-GT-004 **Elev./Depth:** 20.0' to 21.5'

KLEINFELDER, INC.

Client: ARCADIS
Project: Mill Pond Dam
Fort Bragg, CA
Project No: 102522

Particle Size Distribution Report



% COBBLES	% GRAVEL		% SAND			% FINES	
	CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
						57	

SIEVE SIZE	PERCENT FINER	SPEC.* PERCENT	PASS? (X=NO)
#200	57		

* (no specification provided)

Soil Description

Sandy silt
Moisture Content: 14.4%
Laboratory No.: 8670

Atterberg Limits

PL= NP LL= NV PI= NP

Coefficients

D_{85}= D_{60}= D_{50}=
D_{30}= D_{15}= D_{10}=
C_u= C_c=

Classification

USCS= ML AASHTO=

Remarks

Sampled By: Bill Copeland (ARCADIS)
Tested By: A. Stirbys
Reviewed By: J. Revard, CET

Sample No.: 11 **Source of Sample:** OUE-GT-004 **Date:** 3/13/09
Location: Boring ID: OUE-GT-004 **Elev./Depth:** 27.5' to 29.0'

KLEINFELDER, INC.

Client: ARCADIS
Project: Mill Pond Dam
Fort Bragg, CA
Project No: 102522

KLEINFELDER

Data checked by: JJL Date: 3/23/09

Point Load Strength Index Test Results ASTM D-5731

PROJECT:	Mill Pond Dam	LAB SAMPLE NO.:	8674
PROJECT NO.:	102522	SAMPLE NO.:	OUE-GT-002 20.7'-21.0'
PROJECT LOCATION:	Fort Bragg, CA	SAMPLE DESCRIP:	Shale
SAMPLED BY:	Bill Copeland (ARCADIS)	DATE TESTED:	3/10/2009
DATE SAMPLED:	3/5/2009	REPORTED BY:	A. Stirbys

Boring No.	Depth (ft)	Test Number	Test Type*	Rock Type	Width, W (in)	Depth or Diameter, D (in)	Failure Load, P (lbs)	De^2 (in^2)	Point Load Strength Index, $I_{s(50)}$ (PSI)	Uniaxial Compressive Strength, UCS (PSI)	Valid/ Invalid
OUE-GT-002	20.7-21.0	1	d	Shale	2.38	2.38	0.0	5.66	0	0	Invalid

$I_{s(50)}$ 0 psi or 0 psf or 0.00 MPa

σ_c= 0 psi or 0 psf or 0 MPa

σ_c = uniaxial compressive strength

*Test Type
d = diametral
a = axial
b = block
i= irregular lump

Point Load Strength Index Test Results ASTM D-5731

KLEINFELDER

Data checked by: JR Date: 3/23/09

Point Load Strength Index Test Results ASTM D-5731

PROJECT:	Mill Pond Dam	LAB SAMPLE NO.:	8675
PROJECT NO.:	102522	SAMPLE NO.:	OUE-GT-002 22.5'-23.0'
PROJECT LOCATION:	Fort Bragg, CA	SAMPLE DESCRIP:	Shale
SAMPLED BY:	Bill Copeland (ARCADIS)	DATE REPORTED:	3/10/2009
DATE SAMPLED:	3/5/2009	REPORTED BY:	A. Stirbys

Boring No.	Depth (ft)	Test Number	Test Type*	Rock Type	Width, W (in)	Depth or Diameter, D (in)	Failure Load, P (lbs)	De^2 (in^2)	Point Load Strength Index, $I_{s(50)}$ (PSI)	Uniaxial Compressive Strength, UCS (PSI)	Valid/ Invalid
OUE-GT-002	22.5-23.0	1	i	Shale	2.38	1.82	1.2	5.52	244	5990	Valid
OUE-GT-002	22.5-23.0	2	i	Shale	2.38	1.72	1.3	5.21	268	6567	Valid

$I_{s(50)}$ 256 psi or 36,864 psf or 1.77 MPa

σ_c= 6,279 psi or 904,176 psf or 43 MPa

σ_c = uniaxial compressive strength

*Test Type
d = diametral
a = axial
b = block
i = irregular lump

KLEINFELDER INC. 2405 140th AVE NE, Suite A101 Bellevue, WA 98005 Office (425) 562-4200 Fax (425) 562-4201

FIGURE B-22

KLEINFELDER

Point Load Strength Index Test Results ASTM D-5731

KLEINFELDER

Data checked by: JR Date: 3/23/09

Point Load Strength Index Test Results ASTM D-5731

PROJECT:	Mill Pond Dam	LAB SAMPLE NO.:	8676
PROJECT NO.:	102522	SAMPLE NO.:	OUE-GT-002 24.5'-24.8'
PROJECT LOCATION:	Fort Bragg, CA	SAMPLE DESCRIP:	Sandstone
SAMPLED BY:	Bill Copeland (ARCADIS)	DATE TESTED:	3/10/2009
DATE SAMPLED:	3/5/2009	REPORTED BY:	A. Stirbys

Boring No.	Depth (ft)	Test Number	Test Type*	Rock Type	Width, W (in)	Depth or Diameter, D (in)	Failure Load, P (lbs)	De^2 (in^2)	Point Load Strength Index, $I_{s(50)}$ (PSI)	Uniaxial Compressive Strength, UCS (PSI)	Valid/ Invalid
OUE-GT-002	24.5-24.8	1	i	Sandstone	2.39	1.41	1.2	4.29	276	6,764	Valid
OUE-GT-002	24.5-24.8	2	i	Sandstone	1.03	1.56	0.6	2.05	237	5,806	Valid
OUE-GT-002	24.5-24.8	3	i	Sandstone	2.35	1.14	0.6	3.41	174	4,252	Valid
OUE-GT-002	24.5-24.8	4	i	Sandstone	1.86	1.26	0.6	2.98	204	4,995	Valid

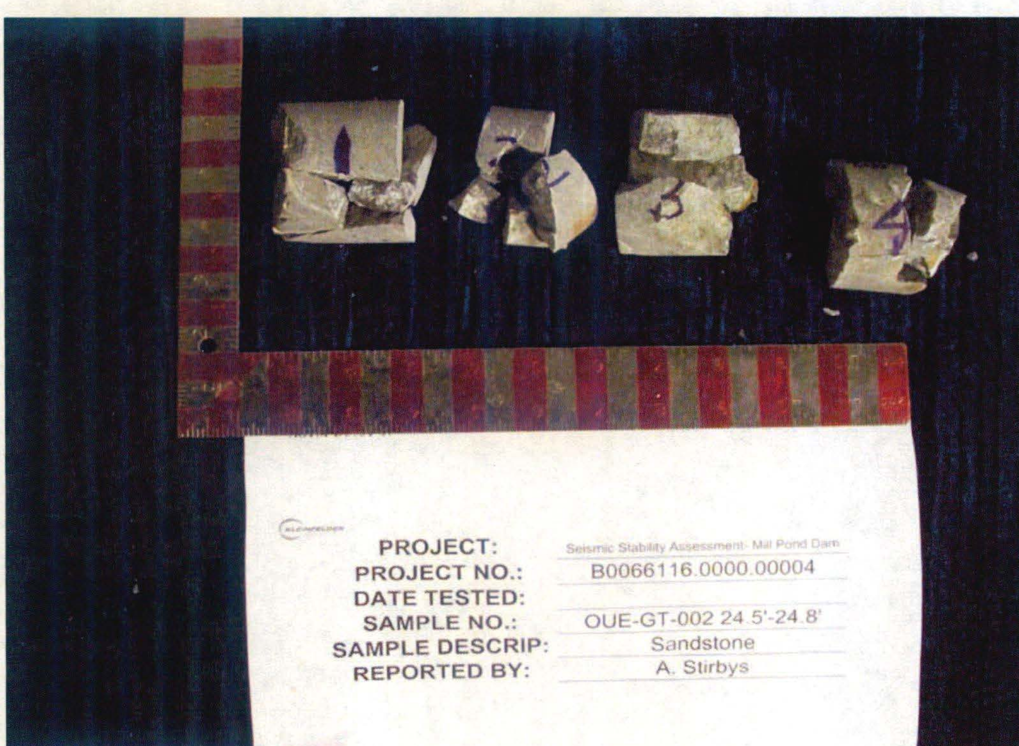
$I_{s(50)}$ 223 psi or 32,112 psf or 1.54 MPa

σ_c= 5,454 psi or 785,376 psf or 38 MPa

σ_c = uniaxial compressive strength

*Test Type
d = diametral
a = axial
b = block
i = irregular lump

Point Load Strength Index Test Results ASTM D-5731



KLEINFELDER

Data checked by: JR Date: 3/23/09

Point Load Strength Index Test Results ASTM D-5731

PROJECT:	Mill Pond Dam	LAB SAMPLE NO.:	8677
PROJECT NO.:	102522	SAMPLE NO.:	OUE-GT-002 25.7'-26.1'
PROJECT LOCATION:	Fort Bragg, CA	SAMPLE DESCRIP:	Sandstone
SAMPLED BY:	Bill Copeland (ARCADIS)	DATE REPORTED:	3/10/2009
DATE SAMPLED:	3/5/2009	REPORTED BY:	A. Stirbys

Boring No.	Depth (ft)	Test Number	Test Type*	Rock Type	Width, W (in)	Depth or Diameter, D (in)	Failure Load, P (lbs)	De^2 (in^2)	Point Load Strength Index, $I_{s(50)}$ (PSI)	Uniaxial Compressive Strength, UCS (PSI)	Valid/ Invalid
OUE-GT-002	25.7-26.1	1	i	Sandstone	2.38	1.17	0.6	3.55	167	4096	Valid
OUE-GT-002	25.7-26.1	2	i	Sandstone	1.56	1.09	1.0	2.17	407	9976	Valid
OUE-GT-002	25.7-26.1	3	i	Sandstone	2.25	1.08	0.8	3.09	256	6261	Valid

$I_{s(50)}$	277	psi	or	39,888	psf	or	1.91	MPa
σ_c=	6,778	psi	or	976,032	psf	or	47	MPa

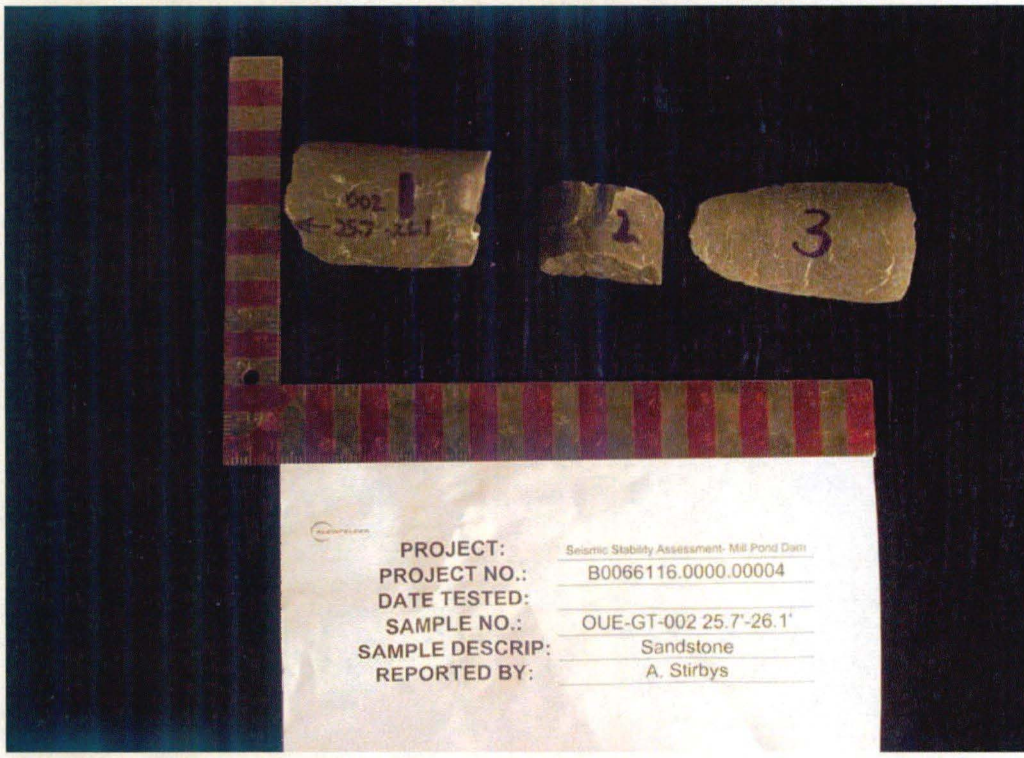
σ_c = uniaxial compressive strength

*Test Type

d = diametral
a = axial
b = block
i = irregular lump

KLEINFELDER

Point Load Strength Index Test Results ASTM D-5731



KLEINFELDER

Data checked by: JJR Date: 3/23/09

Point Load Strength Index Test Results ASTM D-5731

PROJECT:	Mill Pond Dam	LAB SAMPLE NO.:	8678
PROJECT NO.:	102522	SAMPLE NO.:	OUE-GT-002 26.7'-27.3'
PROJECT LOCATION:	Fort Bragg, CA	SAMPLE DESCRIP:	Sandstone
SAMPLED BY:	Bill Copeland (ARCADIS)	DATE TESTED:	3/10/2009
DATE SAMPLED:	3/5/2009	REPORTED BY:	A. Stirbys

Boring No.	Depth (ft)	Test Number	Test Type*	Rock Type	Width, W (in)	Depth or Diameter, D (in)	Failure Load, P (lbs)	De^2 (in^2)	Point Load Strength Index, $I_{s(50)}$ (PSI)	Uniaxial Compressive Strength, UCS (PSI)	Valid/ Invalid
OUE-GT-002	26.7-27.3	1	i	Sandstone	2.37	2.24	1.1	6.75	187	4574	Valid
OUE-GT-002	26.7-27.3	2	i	Sandstone	1.53	1.06	0.6	2.06	265	6482	Valid
OUE-GT-002	26.7-27.3	3	i	Sandstone	2.37	1.30	0.7	3.92	180	4421	Valid

$I_{s(50)}$	211	psi	or	30,384	psf	or	1.46	MPa
σ_c=	5,159	psi	or	742,896	psf	or	36	MPa

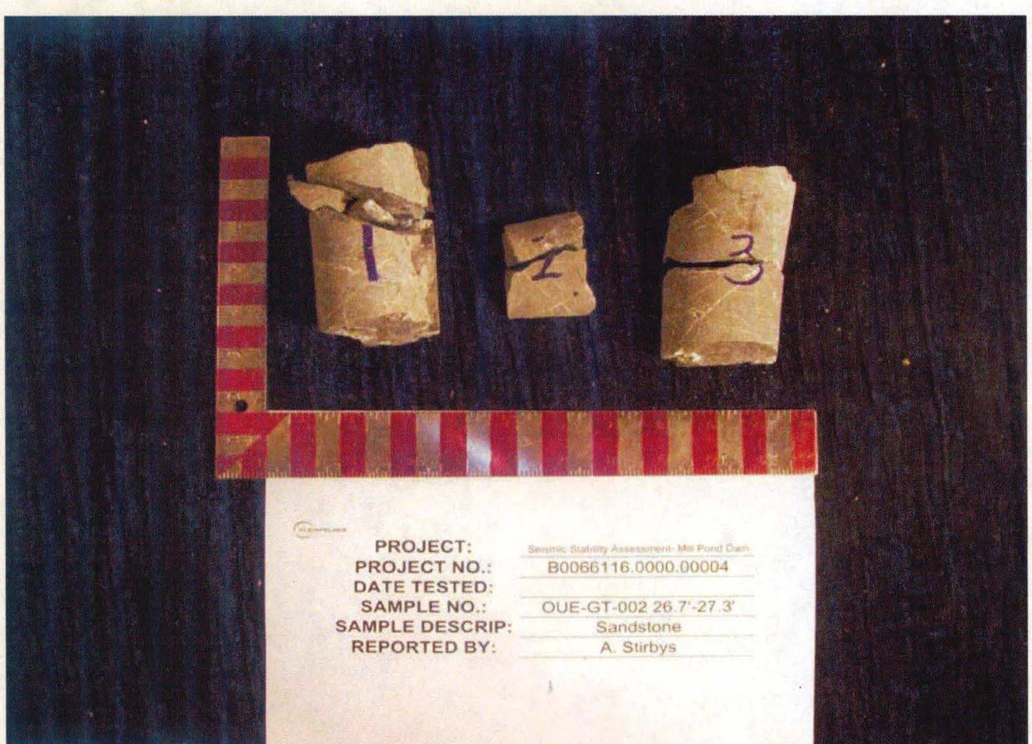
σ_c = uniaxial compressive strength

*Test Type
d = diametral
a = axial
b = block
i= irregular lump

KLEINFELDER INC. 2405 140th AVE NE, Suite A101 Bellevue, WA 98005 Office (425) 562-4200 Fax (425) 562-4201 ©October 2007

FIGURE B-28

Point Load Strength Index Test Results ASTM D-5731



KLEINFELDER

Data checked by: JR Date: 3/23/09

Point Load Strength Index Test Results ASTM D-5731

PROJECT:	Mill Pond Dam	LAB SAMPLE NO.:	8680
PROJECT NO.:	102522	SAMPLE NO.:	OUE-GT-002 34.1'-35.2'
PROJECT LOCATION:	Fort Bragg, CA	SAMPLE DESCRIP:	Sandstone
SAMPLED BY:	Bill Copeland (ARCADIS)	DATE TESTED:	3/12/2009
DATE SAMPLED:	3/5/2009	REPORTED BY:	A. Stirbys

Boring No.	Depth (ft)	Test Number	Test Type*	Rock Type	Width, W (in)	Depth or Diameter, D (in)	Failure Load, P (lbs)	De^2 (in^2)	Point Load Strength Index, $I_{s(50)}$ (PSI)	Uniaxial Compressive Strength, UCS (PSI)	Valid/ Invalid
OUE-GT-002	34.1-35.2	1	d	Sandstone	2.37	1.03	0.0	0.91	0	0	Invalid
OUE-GT-002	34.1-35.2	2	i	Sandstone	1.31	1.16	0.2	1.93	71	1729	Valid
OUE-GT-002	34.1-35.2	3	i	Sandstone	2.38	2.38	0.1	6.85	23	558	Valid

$I_{s(50)}$ 47 psi or 6,768 psf or 0.32 MPa

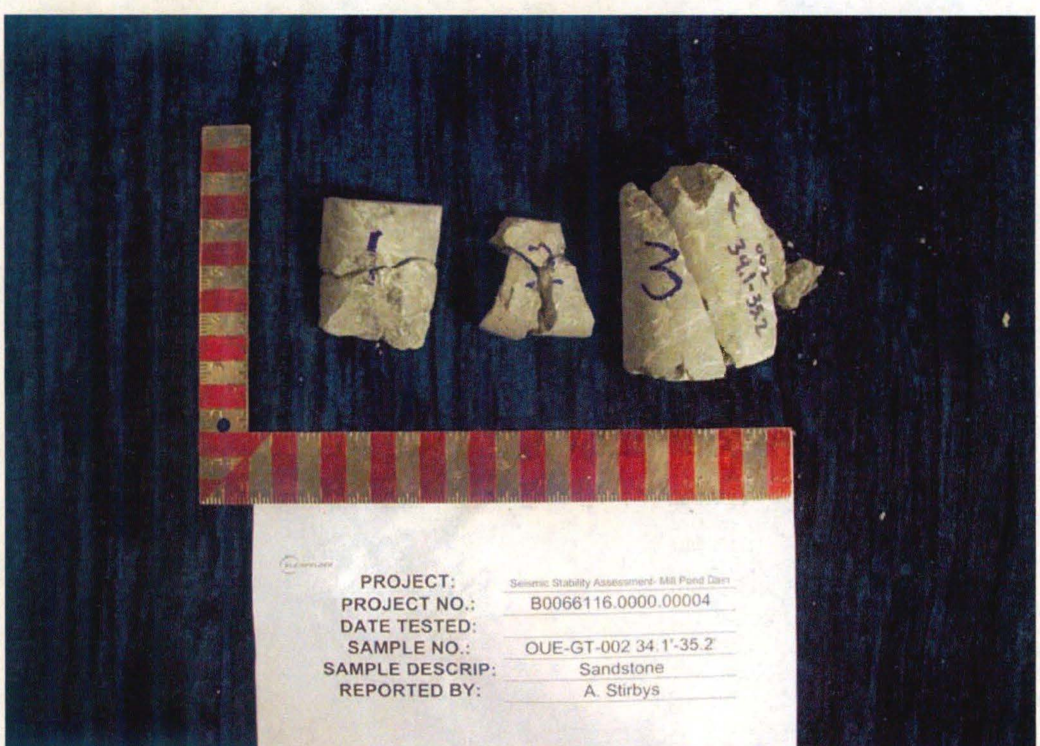
σ_c= 1,143 psi or 164,592 psf or 8 MPa

σ_c = uniaxial compressive strength

*Test Type
d = diametral
a = axial
b = block
i= irregular lump

KLEINFELDER

Point Load Strength Index Test Results ASTM D-5731



KLEINFELDER

Data checked by: JR Date: 3/23/09

Point Load Strength Index Test Results ASTM D-5731

PROJECT:	Mill Pond Dam	LAB SAMPLE NO.:	8681
PROJECT NO.:	102522	SAMPLE NO.:	OUE-GT-002 37.2'-37.9'
PROJECT LOCATION:	Fort Bragg, CA	SAMPLE DESCRIP:	Sandstone
SAMPLED BY:	Bill Copeland (ARCADIS)	DATE TESTED:	3/10/2009
DATE SAMPLED:	3/5/2009	REPORTED BY:	A. Stirbys

Boring No.	Depth (ft)	Test Number	Test Type*	Rock Type	Width, W (in)	Depth or Diameter, D (in)	Failure Load, P (lbs)	De^2 (in^2)	Point Load Strength Index, $I_{s(50)}$ (PSI)	Uniaxial Compressive Strength, UCS (PSI)	Valid/ Invalid
OUE-GT-002	37.2-37.9	1	i	Sandstone	2.38	1.48	0.4	4.48	91	2,221	Valid
OUE-GT-002	37.2-37.9	2	d	Sandstone	2.96	2.38	0.2	5.66	32	773	Valid

$I_{s(50)}$	61	psi	or	8,784	psf	or	0.42	MPa
σ_c=	1,497	psi	or	215,568	psf	or	10	MPa

σ_c = uniaxial compressive strength

*Test Type
d = diametral
a = axial
b = block
i = irregular lump

KLEINFELDER

Point Load Strength Index Test Results ASTM D-5731

KLEINFELDER

Data checked by: JR Date: 3/23/09

Unconfined Compressive Strength Test Results ASTM D7012-04

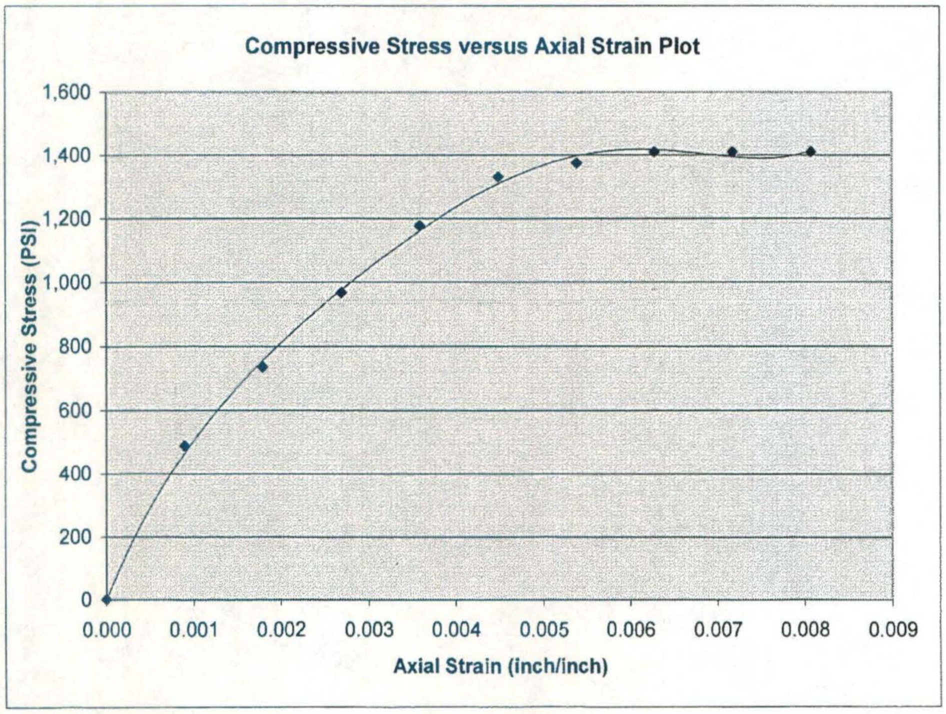
PROJECT:	Mill Pond Dam	LAB SAMPLE NO.:	8680
PROJECT NO.:	102522	SAMPLE NO.:	OUE-GT-002 34.1'-35.2'
PROJECT LOCATION:	Fort Bragg, CA	SAMPLE DESCRIP:	Sandstone
SAMPLED BY:	Bill Copeland (ARCADIS)	DATE TESTED:	3/12/2008
DATE SAMPLED:	3/5/2009	REPORTED BY:	A. Stirbys

Tare Weight (grams)	0.00	Diameter (in)	2.38
Wet Specimen Weight + tare (g)	1007.20	Area (in^2)	4.43
Dry Speicmen Weight + tare (g)	1007.20	Height (in)	5.57
Weight of Water (g)	0.00	Volume (in^3)	24.68
Weight of Dry Specimen (g)	1007.20	Maximum Load, P (lbs)	6,090
Weight of Wet Specimen (g)	1007.20	**Compressive Strength (PSI)**	1,410
Water Content (%)	0.0		
Unit Weight Wet (pcf)	155.5	Specific Gravity	2.49
Unit Weight Dry (pcf)	155.5	Sample Break Type: Intact	
Unit Weight of Water (pcf)	62.43	Sample Break Type: Fracture	

Other Break Comments: Gypsum capped, numerous fractures throughout sample. Saturated for 8-12 hours.

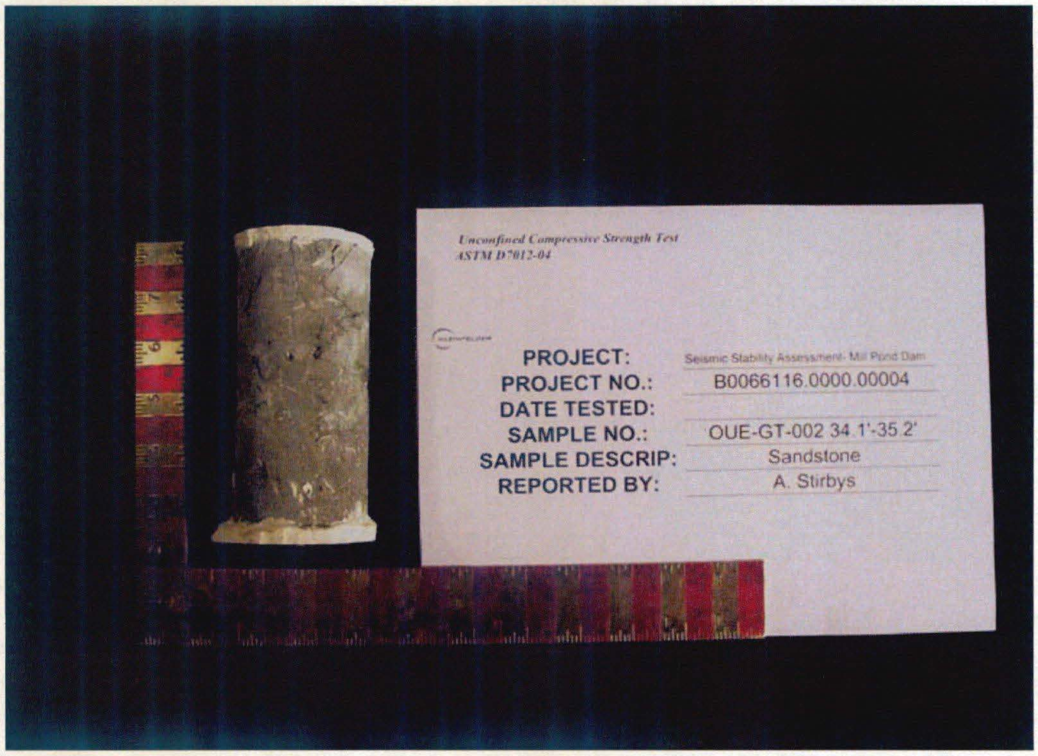
NOTE: L/D > 2.0 ASTM states that the failure stress must have a correction factor applied.

Axial Load (lbs)	Deflection (inch)	Axial Strain (inch/inch)	Compressive Stress (PSI)
0	0.000	0.00000	0
2,110	0.005	0.00090	488.67
3,170	0.010	0.00180	734.16
4,170	0.015	0.00269	965.75
5,080	0.020	0.00359	1,176.51
5,740	0.025	0.00449	1,329.36
5,930	0.030	0.00539	1,373.36
6,090	0.035	0.00628	1,410.42
6,090	0.040	0.00718	1,410.42
6,090	0.045	0.00808	1,410.42



KLEINFELDER

Unconfined Compressive Strength Test Results ASTM D7012-04



KLEINFELDER

PROJECT:	Mill Pond Dam	**LAB SAMPLE NO.:**	8673
PROJECT NO.:	102522	**SAMPLE NO.:**	OUE-GT-001-28.5'-29.1'
PROJECT LOCATION:	Fort Bragg, CA	**SAMPLE DESCRIP.:**	
SAMPLED BY:	Bill Copeland (ARCADIS)	**DATE SAMPLED:**	3/5/2009
SAMPLE PREP.:	A. Stirbys	**DATE REPORTED:**	3/13/2009
		REPORTED BY:	R.Hogg

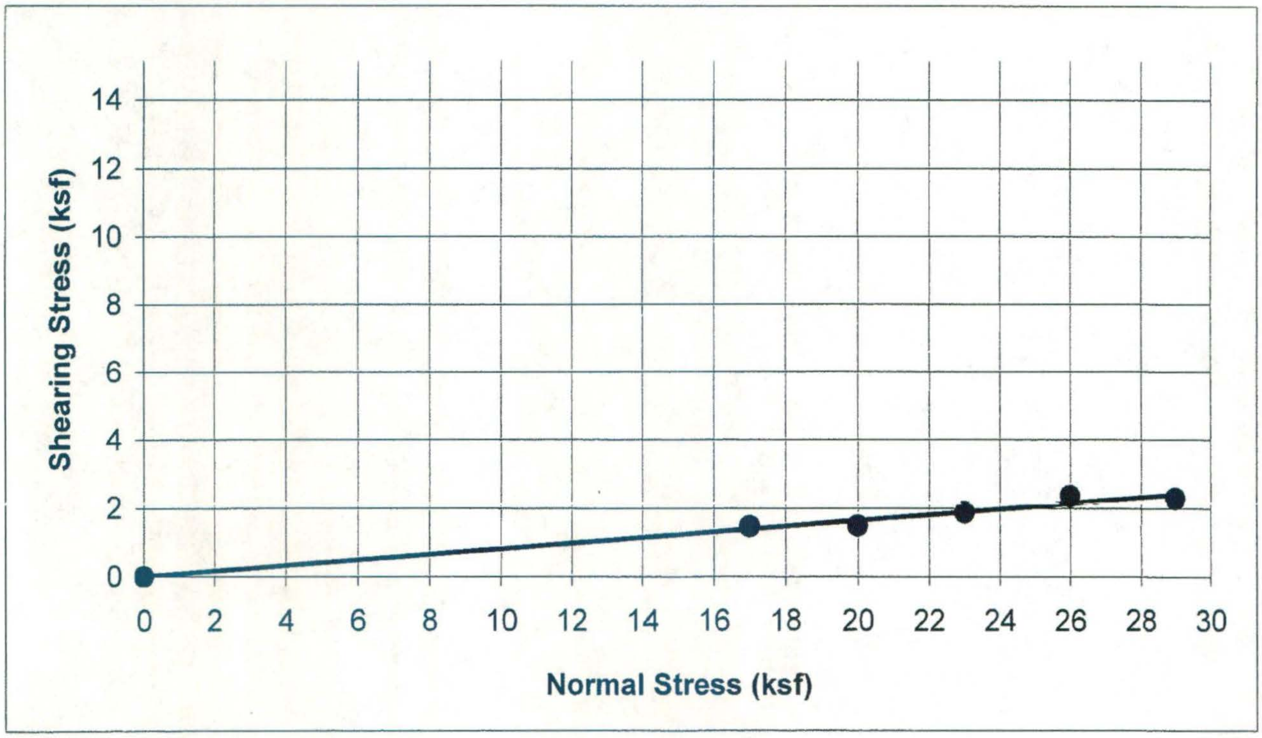
DIRECT SHEAR TEST OF ROCK SPECIMENS UNDER CONSTANT NORMAL LOAD (ASTM D5607-02)

Initial thickness of specimen (in.):	2.00
Initial diameter of specimen (in.):	2.17
Rate of deformation (in/min):	0
Dry mass of specimen (g):	292.9
Insitu Dry Density (lb per cu.ft):	150.9
Shearing device used:	

Created by DigiShear Version 3.1.3; Copyright 2004, GEOTAC

Normal Stress (kps):		Max Shearing Stress (kps):	
Point 1	17	Point 1	1.47
Point 2	20	Point 2	1.49
Point 3	23	Point 3	1.88
Point 4	26	Point 4	2.34
Point 5	29	Point 5	2.27
Vert Deformation @ Max Shear:		**Horiz Deformation @ Max Shear:**	
Point 1	0.0025 in.	Point 1	0.6999 in.
Point 2	0.0053 in.	Point 2	0.6705 in.
Point 3	0.0042 in.	Point 3	0.6224 in.
Point 4	0.0030 in.	Point 4	0.5381 in.
Point 5	0.0024 in.	Point 5	0.4569 in.

Slope of line, m	
Preliminary Friction Angle (degrees)	4.7
Apparent Friction Angle (degrees)	0.0



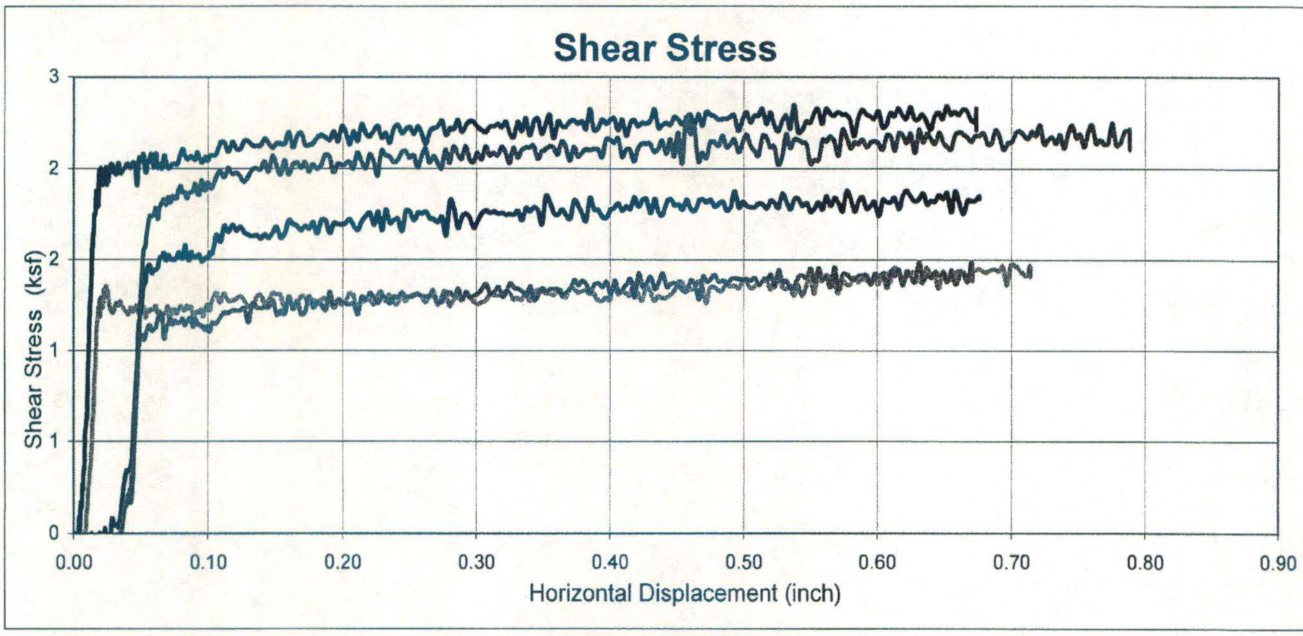
KLEINFELDER INC. 2405 140th AVE NE, Suite A101 Bellevue, WA 98005
Office (425) 562-4200 Fax (425) 562-4201

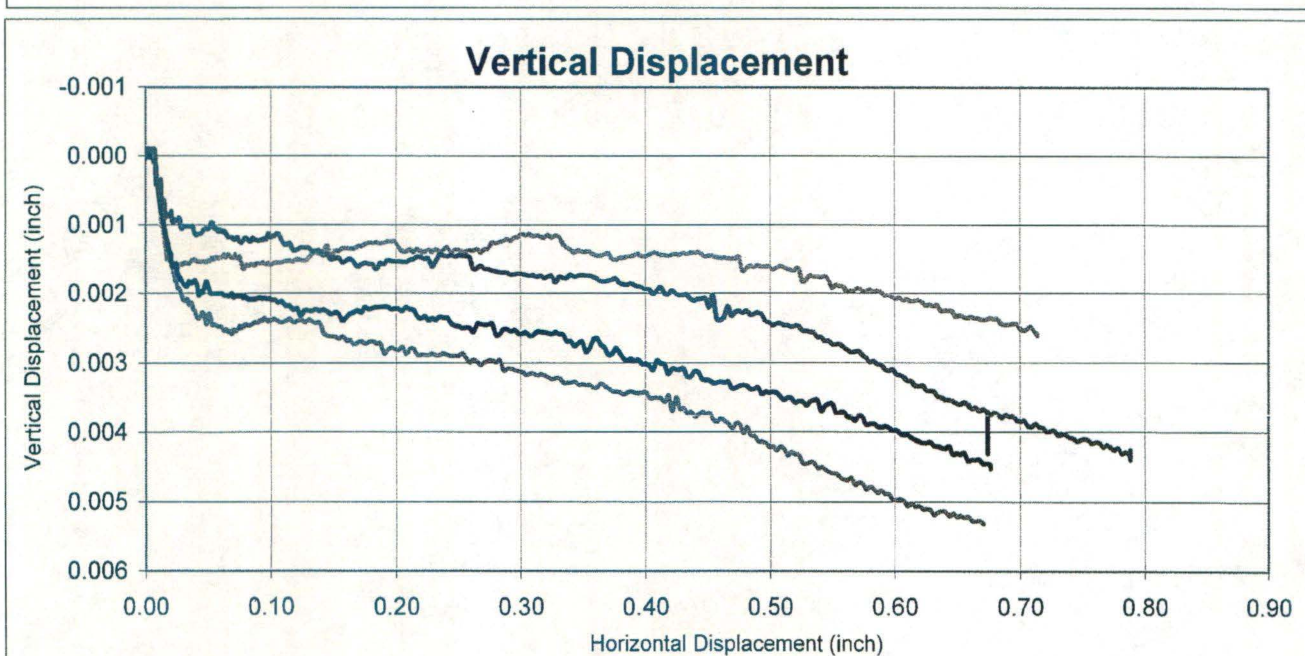
FIGURE B-36

KLEINFELDER

PROJECT:	Mill Pond Dam	**LAB SAMPLE NO.:**	8673
PROJECT NO.:	102522	**SAMPLE NO.:**	OUE-GT-001-28.5'-29.1'
PROJECT LOCATION:	Fort Bragg, CA	**SAMPLE DESCRIP.:**	
SAMPLED BY:	Bill Copeland (ARCADIS)	**DATE SAMPLED:**	3/5/2009
SAMPLE PREP.:	A. Stirbys	**DATE REPORTED:**	3/13/2009
		REPORTED BY:	R.Hogg

DIRECT SHEAR TEST OF ROCK SPECIMENS UNDER CONSTANT NORMAL LOAD (ASTM D5607-02)

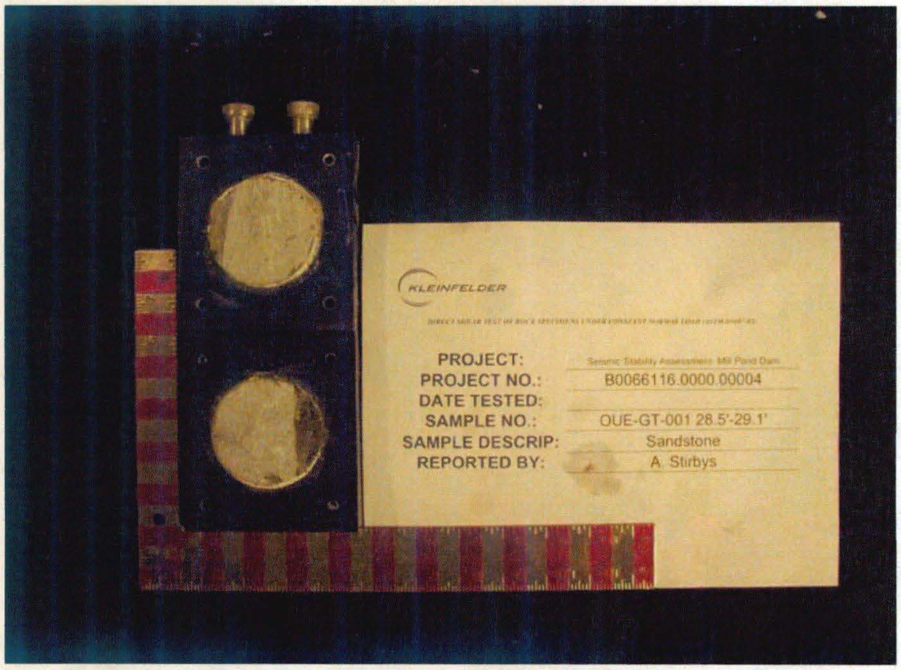


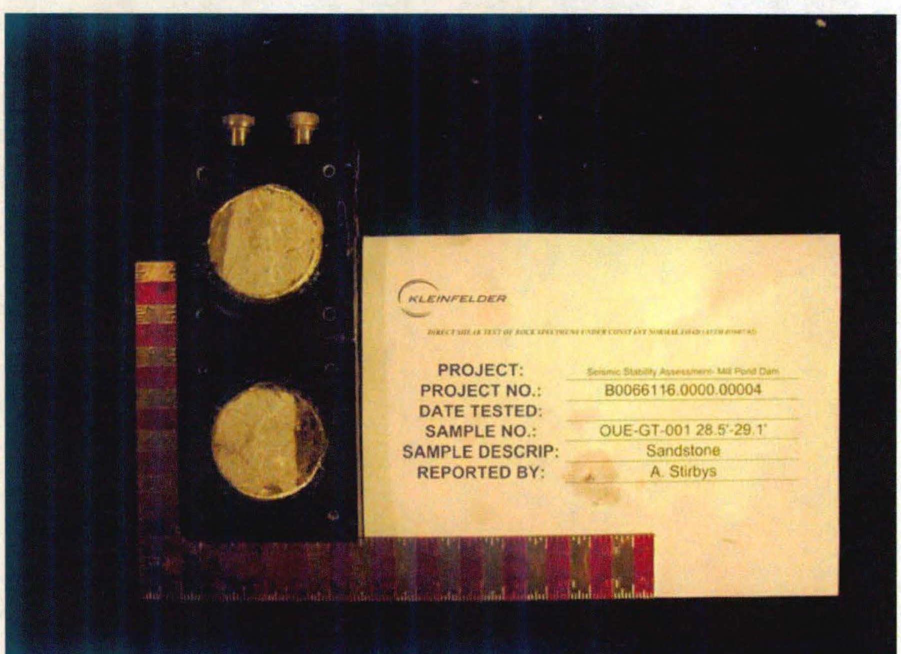


KLEINFELDER INC. 2405 140th AVE NE, Suite A101 Bellevue, WA 98005
Office (425) 562-4200 Fax (425) 562-4201

KLEINFELDER

DIRECT SHEAR TEST OF ROCK SPECIMENS UNDER CONSTANT NORMAL LOAD (ASTM D5607-02)





KLEINFELDER

PROJECT:	Mill Pond Dam	**LAB SAMPLE NO.:**	8679
PROJECT NO.:	102522	**SAMPLE NO.:**	OUE-GT-002-28.2'-29.1'
PROJECT LOCATION:	Fort Bragg, CA	**SAMPLE DESCRIP.:**	
SAMPLED BY:	Bill Copeland (ARCADIS)	**DATE SAMPLED:**	3/5/2009
SAMPLE PREP.:	B.Kochanski/ A.Stirbys	**DATE REPORTED:**	3/13/2009
		REPORTED BY:	R.Hogg

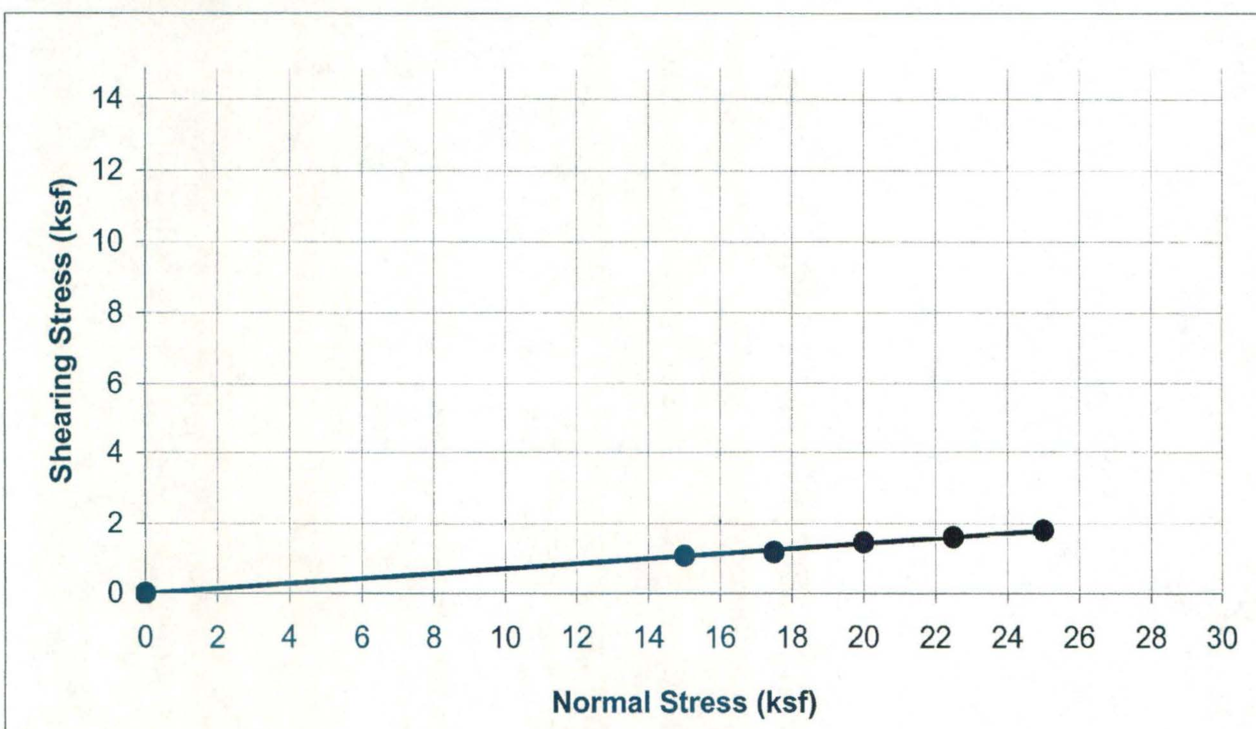
DIRECT SHEAR TEST OF ROCK SPECIMENS UNDER CONSTANT NORMAL LOAD (ASTM D5607-02)

Initial thickness of specimen (in.):	2.02
Initial diameter of specimen (in.):	2.38
Rate of deformation (in/min):	0
Dry mass of specimen (g):	356.5
Insitu Dry Density (lb per cu.ft):	151.2
Shearing device used:	

Created by DigiShear Version 3.1.3; Copyright 2004, GEOTAC

Normal Stress (kps):		Max Shearing Stress (kps):	
Point 1	15	Point 1	1.07
Point 2	17.5	Point 2	1.18
Point 3	20	Point 3	1.46
Point 4	22.5	Point 4	1.61
Point 5	25	Point 5	1.80
Vert Deformation @ Max Shear:		**Horiz Deformation @ Max Shear:**	
Point 1	-0.0019 in.	Point 1	0.5476 in.
Point 2	-0.0012 in.	Point 2	0.5980 in.
Point 3	-0.0021 in.	Point 3	0.5979 in.
Point 4	-0.0014 in.	Point 4	0.5620 in.
Point 5	-0.0023 in.	Point 5	0.7656 in.

Slope of line, m	
Preliminary Friction Angle (degrees)	4.1
Apparent Friction Angle (degrees)	0.0

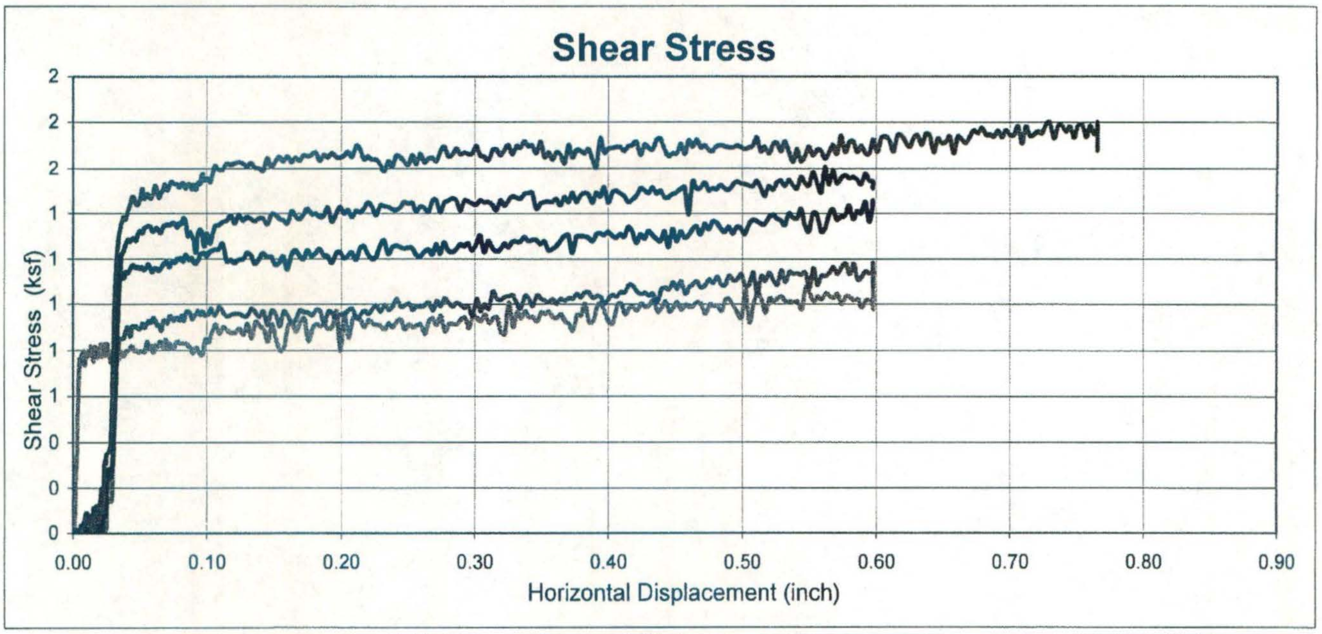


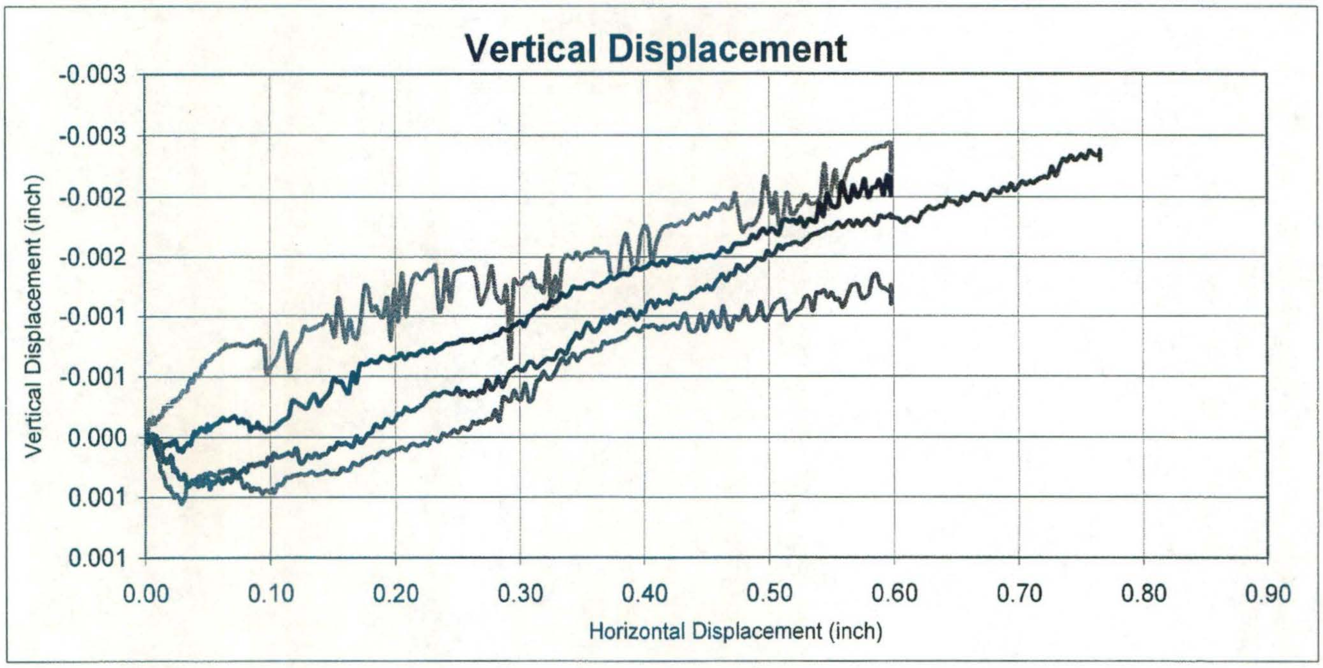
KLEINFELDER INC. 2405 140th AVE NE, Suite A101 Bellevue, WA 98005
Office (425) 562-4200 Fax (425) 562-4201

KLEINFELDER

PROJECT:	Mill Pond Dam	**LAB SAMPLE NO.:**	8679
PROJECT NO.:	102522	**SAMPLE NO.:**	OUE-GT-002-28.2'-29.1'
PROJECT LOCATION:	Fort Bragg, CA	**SAMPLE DESCRIP.:**	
SAMPLED BY:	Bill Copeland (ARCADIS)	**DATE SAMPLED:**	3/5/2009
SAMPLE PREP.:	B.Kochanski/ A.Stirbys	**DATE REPORTED:**	3/13/2009
		REPORTED BY:	R.Hogg

DIRECT SHEAR TEST OF ROCK SPECIMENS UNDER CONSTANT NORMAL LOAD (ASTM D5607-02)



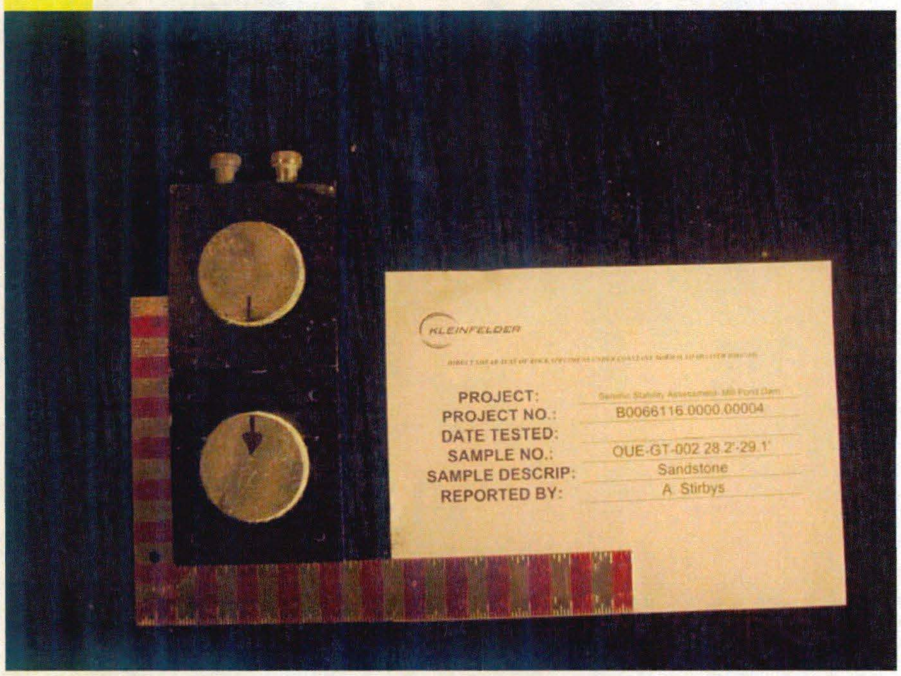


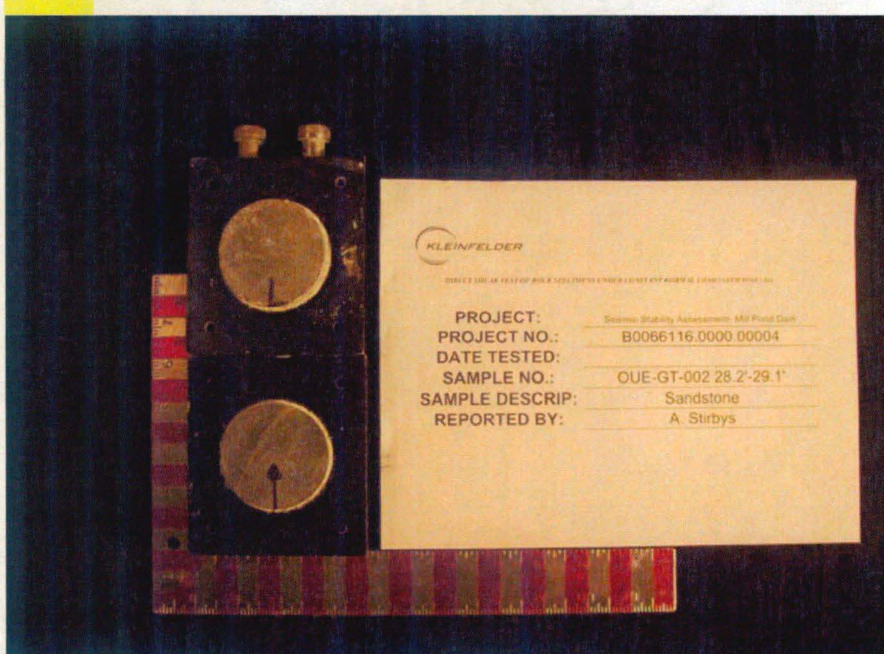
KLEINFELDER INC. 2405 140th AVE NE, Suite A101 Bellevue, WA 98005
Office (425) 562-4200 Fax (425) 562-4201

FIGURE B-40

KLEINFELDER

DIRECT SHEAR TEST OF ROCK SPECIMENS UNDER CONSTANT NORMAL LOAD (ASTM D5607-02)





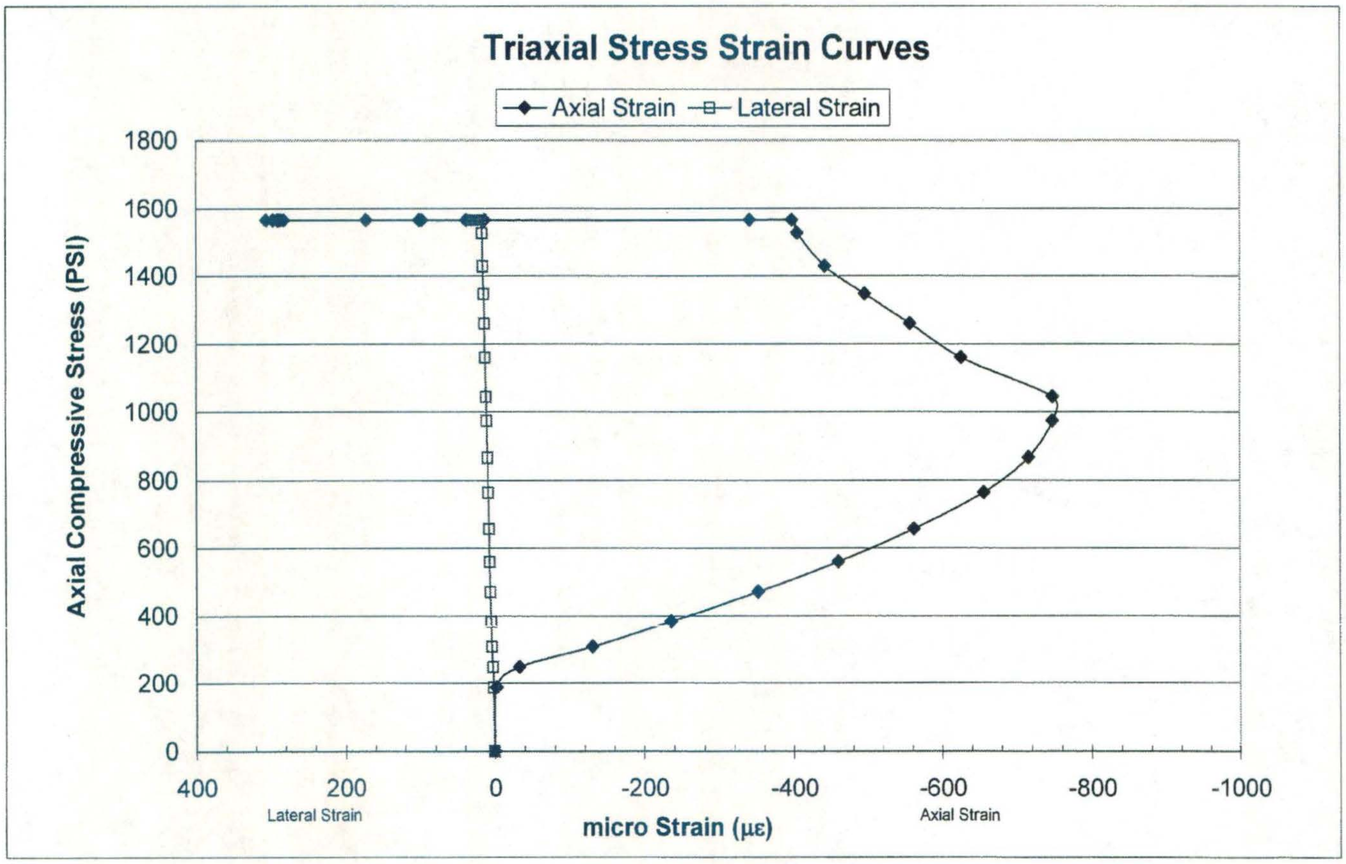
KLEINFELDER

Triaxial Compressive Strength Test Results ASTM D7012-04

PROJECT:	Mill Pond Dam	LAB SAMPLE NO.:	8679
PROJECT NO.:	102522	SAMPLE NO.:	OUE-GT-002-28.2'-29.1'
PROJECT LOCATION:	Fort Bragg, CA	SAMPLE DESCRIP:	
SAMPLED BY:	Bill Copeland (ARCADIS)	DATE REPORTED:	3/13/2009
DATE SAMPLED:	3/5/2009	REPORTED BY:	R.Hogg

RH

Water Content (%)	0.0	Young's Modulus (PSI)	Not Reported
Unit Weight Wet (pcf)	158.6	Poisson's Ratio	Not Reported
Unit Weight Dry (pcf)	158.6	Confining Stress, σ_3 (PSI)	5
		Total Failure Stress, σ_1 (PSI)	1,015
		Triaxial Compressive Strength, σ (PSI)	**1,010**



KLEINFELDER

Data checked by: JN Date: 3/23/09

Triaxial Compressive Strength Test Results ASTM D7012-04

PROJECT:	Mill Pond Dam	LAB SAMPLE NO.:	8679
PROJECT NO.:	102522	SAMPLE NO.:	OUE-GT-002-28.2'-29.1'
PROJECT LOCATION:	Fort Bragg, CA	SAMPLE DESCRIP:	
SAMPLED BY:	Bill Copeland (ARCADIS)	DATE REPORTED:	3/13/2009
DATE SAMPLED:	3/5/2009	REPORTED BY:	R.Hogg

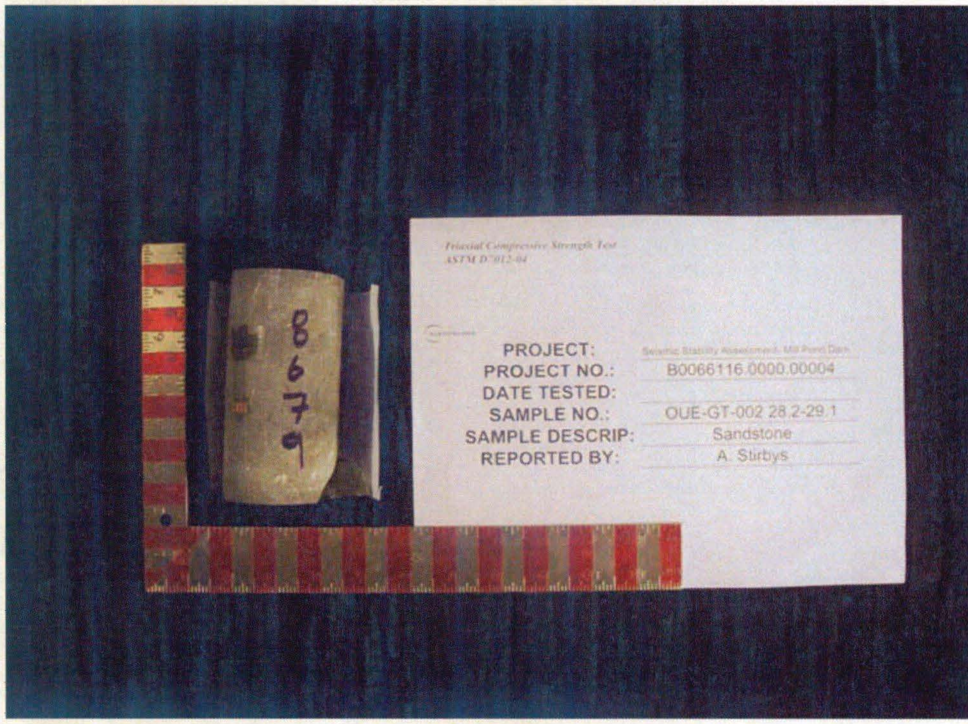
Tare Weight (grams)	0.00	Diameter (in)	2.38	Sample Break Type	Intact	X
Wet Specimen Weight + tare (g)	899.60	Area (in^2)	4.45		Fracture	
Dry Speicmen Weight + tare (g)	899.60	Height (in)	4.86	Young's Modulus (PSI)		
Weight of Water (g)	0.00	Volume (in^3)	21.61	Poisson's Ratio		
Weight of Dry Specimen (g)	899.60	Specific Gravity	2.54	Confining Stress, σ_3 (PSI)		5
Weight of Wet Specimen (g)	899.60			Failure Load, P (lbs)		6,880
Water Content (%)	0.0			Total Failure Stress, σ_1 (PSI)		1,015
Unit Weight Wet (pcf)	158.6			**Triaxial Compressive Strength, σ (PSI)**		**1,010**
Unit Weight Dry (pcf)	158.6					

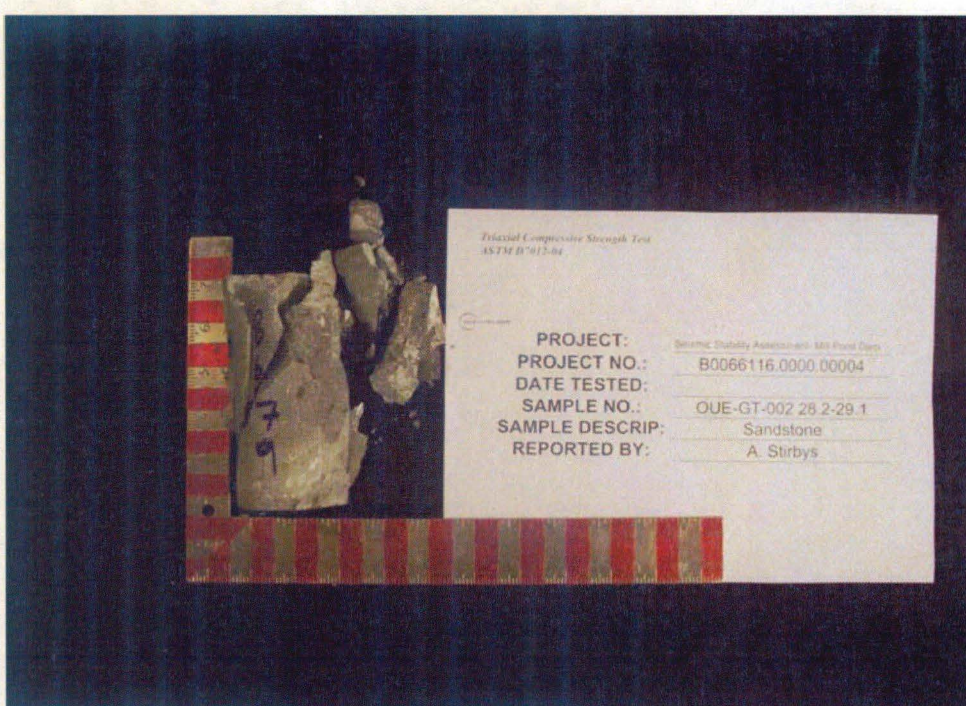
Other Break Comments: **Saturated sample for 8-12 hours before testing. Air dried prior to sample prep then tested. Visual micro fractures.**

NOTE: L/D > 2.0 ASTM states that the failure stress must have a correction factor applied.

Axial Load (lbs)	Axial Strain (inch/inch)	Lateral Strain (inch/inch)	Compressive Stress (PSI)	Axial Load (lbs)	Axial Strain (inch/inch)	Lateral Strain (inch/inch)	Compressive Stress (PSI)
0.00	0.00	0.00	0.00				
820.00	-2		187				
1090.00	-33	0	248				
1360.00	-131	-15	309				
1690.00	-236	-45	384				
2070.00	-352	-80	471				
2460.00	-460	-121	560				
2890.00	-562	-177	657				
3360.00	-656	-219	764				
3810.00	-716	-249	867				
4280.00	-748	-249	974				
4590.00	-748	-269	1,044				
5100.00	-626	-313	1,160				
5540.00	-557	-326	1,260				
5920.00	-496	-331	1,347				
6280.00	-442	-331	1,428				
6710.00	-405	-333	1,526				
6880.00	-398	-340	1,565				
6880.00	-341	-367	1,565				
6880.00	101	-303	1,565				
6880.00	172	-284	1,565				
6880.00	38	-282	1,565				
6880.00	13	-278	1,565				
6880.00	98	-251	1,565				
6880.00	306	-120	1,565				
6880.00	297	-121	1,565				
6880.00	291	-119	1,565				
6880.00	289	-118	1,565				
6880.00	286	-118	1,565				
6880.00	282	-118	1,565				
	279	-119					
	275	-120					

Triaxial Compressive Strength Test Results ASTM D7012-04





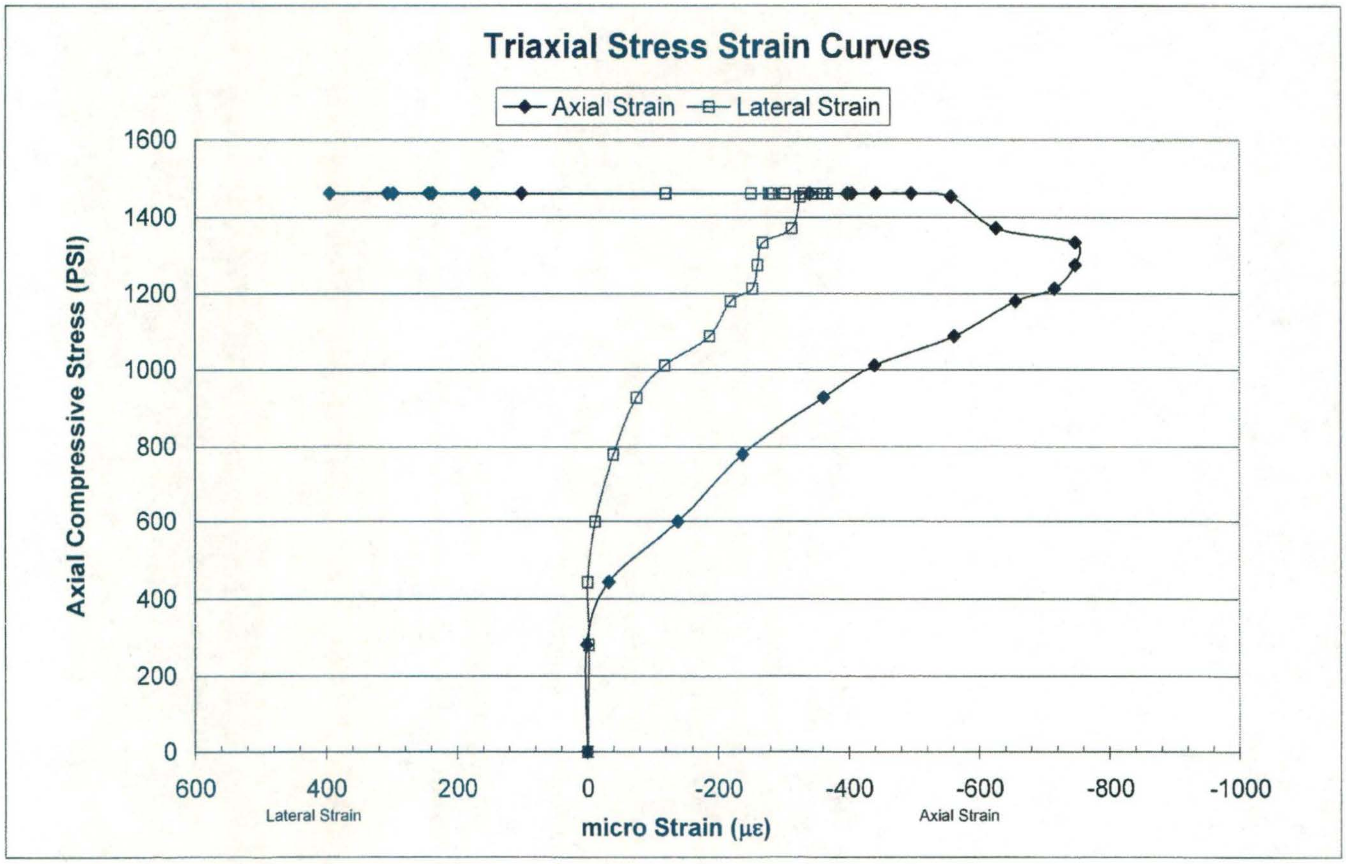
KLEINFELDER

Triaxial Compressive Strength Test Results ASTM D7012-04

PROJECT:	Mill Pond Dam	**LAB SAMPLE NO.:**	8681
PROJECT NO.:	102522	**SAMPLE NO.:**	OUE-GT-002-37.2'-37.9'
PROJECT LOCATION:	Fort Bragg, CA	**SAMPLE DESCRIP:**	
SAMPLED BY:	Bill Copeland (ARCADIS)	**DATE REPORTED:**	3/13/2009
DATE SAMPLED:	3/5/2009	**REPORTED BY:**	R.Hogg

JM

Water Content (%)	0.0	Young's Modulus (PSI)	Not Reported
Unit Weight Wet (pcf)	151.3	Poisson's Ratio	Not Reported
Unit Weight Dry (pcf)	151.3	Confining Stress, σ_3 (PSI)	7
		Total Failure Stress, σ_1 (PSI)	1,332
		Triaxial Compressive Strength, σ (PSI)	**1,325**



KLEINFELDER

Data checked by: JN Date: 3/23/09

Triaxial Compressive Strength Test Results ASTM D7012-04

PROJECT:	**Mill Pond Dam**	**LAB SAMPLE NO.:**	8681
PROJECT NO.:	**102522**	**SAMPLE NO.:**	OUE-GT-002-37.2'-37.9'
PROJECT LOCATION:	Fort Bragg, CA	**SAMPLE DESCRIP:**	
SAMPLED BY:	Bill Copeland (ARCADIS)	**DATE REPORTED:**	3/13/2009
DATE SAMPLED:	3/5/2009	**REPORTED BY:**	R.Hogg

Tare Weight (grams)	0.00	Diameter (in)	2.38	**Sample Break Type**	Intact	X
Wet Specimen Weight + tare (g)	902.60	Area (in^2)	4.45		Fracture	
Dry Speicmen Weight + tare (g)	902.60	Height (in)	5.11	Young's Modulus (PSI)		
Weight of Water (g)	0.00	Volume (in^3)	22.72	Poisson's Ratio		
Weight of Dry Specimen (g)	902.60	Specific Gravity	2.42	Confining Stress, σ_3 (PSI)		7
Weight of Wet Specimen (g)	902.60			Failure Load, P (lbs)		6,400
Water Content (%)	0.0			Total Failure Stress, σ_1 (PSI)		1,332
Unit Weight Wet (pcf)	151.3			**Triaxial Compressive Strength, σ (PSI)**		**1,325**
Unit Weight Dry (pcf)	151.3					

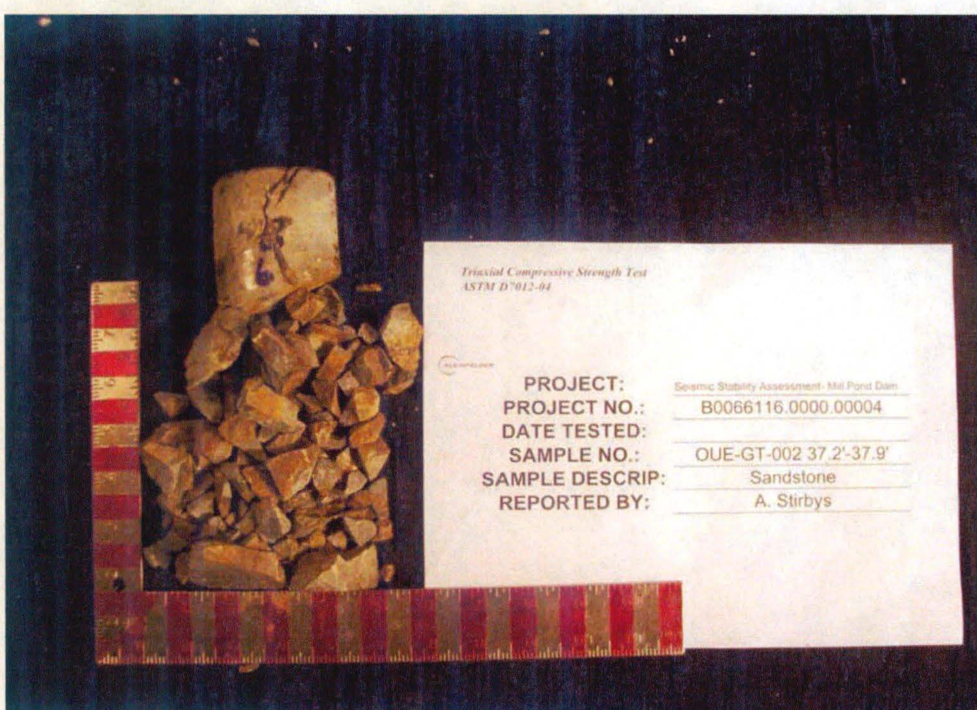
Other Break Comments: **Saturated sample for 8-12 hours before testing. Air dried prior to sample prep then tested. Visual micro fractures.**

NOTE: L/D > 2.0 ASTM states that the failure stress must have a correction factor applied.

Axial Load (lbs)	Axial Strain (inch/inch)	Lateral Strain (inch/inch)	Compressive Stress (PSI)	Axial Load (lbs)	Axial Strain (inch/inch)	Lateral Strain (inch/inch)	Compressive Stress (PSI)
0.00	0.00	0.00	0.00				
1230.00	1	-2	281				
1940.00	-33	0	444				
2620.00	-139	-12	599				
3410.00	-238	-40	780				
4050.00	-362	-76	926				
4420.00	-440	-119	1,011				
4760.00	-562	-187	1,089				
5160.00	-656	-219	1,180				
5300.00	-716	-253	1,212				
5570.00	-748	-261	1,274				
5830.00	-748	-269	1,333				
6000.00	-626	-313	1,372				
6360.00	-557	-326	1,454				
6400.00	-496	-331	1,464				
6400.00	-442	-352	1,464				
6400.00	-405	-359	1,464				
6400.00	-398	-362	1,464				
6400.00	-341	-367	1,464				
6400.00	101	-303	1,464				
6400.00	172	-284	1,464				
6400.00	238	-282	1,464				
6400.00	243	-278	1,464				
6400.00	298	-251	1,464				
6400.00	306	-120	1,464				
6400.00	394	-121	1,464				

KLEINFELDER

Triaxial Compressive Strength Test Results ASTM D7012-04



APPENDIX C

ARCADIS

Appendix C

North Wall Assessment

C1. Slope Stability

Slope stability analyses were carried out to estimate the factor of safety of the north wall of the Log Pond Dam (LPD) area in the proximity of Pond 7.

C1.1 Approach

The analyses were carried out on cross section G-G' of the LPD, as shown on Figure 3 in the main text. The soil stratigraphy was modeled using boring logs data from OUE-GT-002 and OUE-GT-003 as discussed in section 2.4 of the main text.

Slope stability limit equilibrium analyses on circular slip surfaces were carried out using the software SLOPE/W 2007 (Geo-Slope International Ltd., Alberta, Canada). The Spencer's method (Spencer 1967), which is a rigorous analysis approach satisfying both force and momentum equilibriums, was used to calculate the factor of safety. SLOPE/W software was set to automatically search through thousands of potential slip surfaces to identify the most critical one (i.e., the one with minimum factor of safety).

C1.2 Calculations

Slope stability analyses were carried out to evaluate the static factor of safety of the cross-section G-G'. Soil parameters and groundwater profile were modeled consistently with the analyses carried out on the LPD cross-sections D-D' and E-E. The soil strength was modeled using effective stress strength parameters (a friction angle and a cohesion intercept). The geotechnical properties of the soils are listed in Table C-1. The sandstone bedrock layer was modeled with a very high (impenetrable) strength.

Table C-1 Material Properties for Slope Stability Analyses from Previous Analyses

Material	Total Unit Weight (pcf)	Strength	Reference
Silty, Gravelly, Sand	[redacted]		
Upper Silty Sand			
Intermediate Silty Sand			
Lower Silty Sand			
Sandstone Bedrock			

C1.3 Results

The slope stability analysis for section G-G' resulted in a factor of safety less than 1. SLOPE/W output files are presented in Figures C1-A and C1-B. A factor of safety less than 1 indicates an unstable slope. However, this factor of safety contradicts the physical evidence. Therefore, a back-calculation analysis was carried out to refine the soil parameters values. Back-calculation is a standard process when soil parameters are adjusted/refined based on physical evidence. Assuming that the slope is marginally safe, as it shows only minor sloughing (i.e., the factor of safety is slightly greater than 1), the soil parameters were adjusted, and for subsequent analyses the adjusted soil parameters were carried forward.

Table C-2 Back-Calculated Material Properties for Slope Stability Analyses

Material	Total Unit Weight (pcf)	Strength	Reference
Silty, Gravelly, Sand	[redacted]		
Upper Silty Sand			
Intermediate Silty Sand			
Lower Silty Sand			
Sandstone Bedrock			

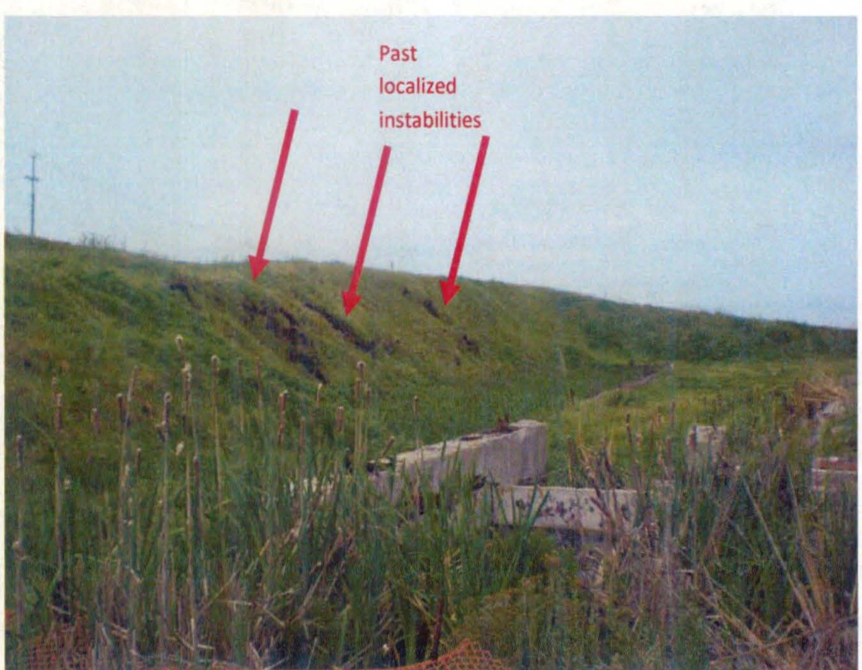
The slope is marginally stable, as a factor of safety slightly greater than 1 indicates, which is confirmed by visual indications of past localized slumps and shallow slope failures, as illustrated in the photographs presented in Figure C2.

C2. Slope Stability

Spencer, E. 1967. A Method of Analysis of Embankments Assuming Parallel Interslice Forces. *Geotechnique*, 17(1): 11-26.

[redacted]


Past
Localized
Instabilities


Past
localized
instabilities

APPENDIX D

ARCADIS

Appendix D

DSOD Correspondence

Georgia-Pacific

Georgia-Pacific LLC
909 Harris Avenue, Ste 201D
Bellingham, WA 98225
(360) 733-2482

August 28, 2009

Mr. David A. Gutierrez, Chief
Division of Safety of Dams
Department of Water Resources
1416 Ninth Street, P. O. Box 94236
Sacramento, CA 94236-001

Re: Mill Pond Dam, No. 2381
Mendocino County

Dear Mr. Gutierrez,

Georgia-Pacific LLC (Georgia-Pacific) is in receipt of the Division of Safety of Dams (DSOD) letter dated June 23, 2009 to [redacted] regarding the findings of the June 2, 2009 Mill Pond Dam inspection. Thank you for your recommendations concerning the Mill Pond Dam.

Georgia-Pacific is conducting further evaluation of DSOD's recommendations to determine the appropriate solution and timing for implementing the identified corrective actions and maintenance activities in light of current planning efforts for the Mill Site. You may be aware that Georgia-Pacific currently is in the middle of the remediation process for the Mill Site which is under the oversight of the Department of Toxic Substances Control (DTSC) through a Site Investigation and Remediation Order. Recently, the City of Fort Bragg and Georgia-Pacific also initiated a specific plan process for the redevelopment of the property. As part of these efforts, Georgia-Pacific is exploring long-term solutions for improving the Mill Pond Dam stability in a manner consistent with the ongoing site remediation and future plans for the site. Due to the ongoing remediation and site planning and local, state, and federal permitting requirements, Georgia-Pacific is faced with certain constraints in its ability to undertake DSOD's recommended dam repair and maintenance activities at this time, as further discussed below.

<u>DSOD Item #1 - Dam Repair</u>

While we agree with the identified corrective actions in Item #1, it appears that DSOD's recommended actions will require certain local, state and federal permits (e.g., coastal development permit, Section 404 permit, etc.) before we can initiate the work. In order to plan and initiate the necessary work, Georgia-Pacific also intends to complete further investigations to determine the scope and extent of such repairs (which we plan to conduct during the development review process), as well as ongoing site characterization work with respect to hazardous materials contamination in the Mill Pond. Consequently, given the need for further studies and the time necessary to prepare the engineering documents required to support the permits, we are concerned that even if we were able to obtain the permits in time to commence the work this year (which is unlikely), we would not be able to *complete* the dam repair and associated maintenance activities before October 1, 2009. We also would not want to initiate any work (and would likely be denied approval to do so based on Local Coastal Plan [LCP] requirements) during the rainy season.

DSOD Item #2 - Spillway Structure

For the same reasons indicated in Item #1 above (permitting and planning requirements), we are unable to patch the base of the reinforced concrete spillway structure before October 1, 2009. Georgia Pacific currently is evaluating a long-term solution for improving the spillway structure, but this too, will require further studies, the necessary permits and coordination with the Mill Site development and remediation processes.

DSOD Items #3, 4 & 5 – Vegetation Removal

We understand from our engineers' initial review of DSOD's recommendations that removal of the existing vegetation within the embankment (see e.g., Items 3, 4 and 5) is necessary. You letter is not specific in terms of methods, but complete removal of vegetation (i.e., stripping and removal of root structures) would potentially accelerate erosion on the north embankment, thereby contributing to a potentially hazardous condition. However, ,more limited vegetation removal on that embankment (e.g., mowing, trimming) is possible and would allow for inspection of those features thereafter.

Although we will need to postpone the dam repair and some of the recommended maintenance activities at this time, Georgia-Pacific intends to clear the vegetation located along the northern embankment of the Mill Pond, including some removal of tules at the toe of the slope near a small pond (see Items 3 and 4). This vegetation removal will enable further inspection of this embankment, while minimizing the potential for erosion. Georgia-Pacific can also clear some vegetation from near the spillway structure (see Item 5). We anticipate that an emergency Coastal Development Permit (CDP) can be granted by the City of Fort Bragg for this work in time to complete it by October 1, 2009.

Georgia-Pacific will continue to keep DSOD apprised of our schedule for completion of the repair items and maintenance items indicated above. We appreciate your assistance with this matter. Please feel free to contact me if you have any questions or need further information regarding our work plan.

Sincerely yours,

[redacted]

Senior Director, Environmental Affairs
GEORGIA-PACIFIC LLC

cc: [redacted], Georgia-Pacific West, Inc.

STATE OF CALIFORNIA – CALIFORNIA NATURAL RESOURCES AGENCY

ARNOLD SCHWARZENEGGER, Governor

DEPARTMENT OF WATER RESOURCES

1416 NINTH STREET, P.O. BOX 942836
SACRAMENTO, CA 94236-0001
(916) 653-5791

JUN 23 2009

[redacted], Plant Operator
Georgia Pacific Corporation
Fort Bragg Wood Products Manufacturing Facility
90 West Redwood Avenue
Fort Bragg, California 95437

Mill Pond Dam, No. 2381
Mendocino County

Dear [redacted]:

On June 2, 2009, Field Engineer Jim Lowe inspected Mill Pond Dam. The enclosed report documents his observations, conclusions, and recommendations with regard to the safety of the dam.

The erosion created void behind and beneath the timber supported portion of the embankment, first noted in the June 5, 2003 inspection report, has not been repaired as requested. The following deficiency requires immediate corrective action:

1. Fill the large void at the timber supported portion of the embankment with compacted fill, and protect the restored surface with rock or other suitable material.

As noted in the report, several longstanding maintenance items also require immediate attention:

2. Patch the damaged base of the reinforced concrete spillway structure.

3. Remove the tule growth along the downstream toe of the right embankment. If live flow is detected afterwards, evaluate the source and significance of this seepage.

4. Clear the brush and dense vegetation that obstructs the embankment, groins, and abutments.

5. Clear the vegetation that obstructs the spillway entrance.

Complete this work by October 1, 2009. Also, please keep us apprised of your work schedule for the repair items described in the first two items.

If you have any questions or need additional information, you may contact Area Engineer Dave Borger at (916) 227-4629 or Regional Engineer Y-Nhi Enzler at (916) 227-4604.

Sincerely,

David A. Gutierrez, Chief
Division of Safety of Dams

Enclosure

STATE OF CALIFORNIA
THE RESOURCES AGENCY
DEPARTMENT OF WATER RESOURCES
DIVISION OF SAFETY OF DAMS

INSPECTION OF DAM AND RESERVOIR IN CERTIFIED STATUS

Name of Dam Mill Pond Dam No. 2381 County Mendocino
Type of Dam ERTH Type of Spillway Concrete flashboard structure
Water is 0.1 feet above spillway crest (no flashboard in place).
Weather Conditions Partly cloudy and mild.
Contacts made Doug Heitmeyer during the inspection.
Reason for Inspection Periodic evaluation.

Important Observations, Recommendations or Actions Taken

As with other periodic inspections over the past several years, no substantive maintenance or repairs have been performed on the deteriorated timber supported portion of the embankment dam. Similar to those past inspections, loss of soil from behind the timbers is evident, and transported soils can be observed behind and beneath the increasingly poorly supported base of the timber cribbing. I directed Mr. Heitmeyer to repair the deteriorated timber supported portion of the embankment, and to restore the compacted fill behind and beneath the timber cribbing.

Standing water has accumulated within low areas along the right embankment toe. Supply pipes to and from the pond perforate the embankment within this area, and neither the condition of the pipes nor the status of encasement of the pipes is known. I directed Mr. Heitmeyer to investigate the cause of the seepage and to make whatever repairs are necessary to prevent uncontrolled seepage from damaging the embankment.

Vegetation control is unsatisfactory, and tall and dense vegetation throughout the embankment makes a thorough inspection for seepage, sliding, and other defects impractical. Tall and dense vegetation must be removed from the embankment, groins, and abutments.

The spillway entrance is partially impeded by dense vegetation. Dense vegetation must be removed from in front of the spillway entrance and measures taken to prevent clogging of the spillway by vegetation during storm flows.

There are no prior outstanding administrative requirements. The total class weight of 0 appears satisfactory at this time.

Conclusions

From the known information and the visual inspection, the dam, reservoir, and the appurtenances are judged satisfactory for continued use pending repair of the timber wall and demonstration of satisfactory control of seepage along the downstream toe of the right embankment.

Observations and Comments

Embankment: In the absence of any substantive maintenance, the timber-cribbing wall to the left of the spillway remains in very poor condition and continues to deteriorate. Embankment material continues to be transported from behind the wall, opening voids and causing a loss of contact and support between the structural timbers and the supported embankment.

The visible portions of the remainder of the upstream and downstream faces, including the crest, the abutment contacts, and the stacked concrete retaining wall to the right of the spillway, remain in what appears to be marginally satisfactory condition. The generally haphazard construction of the pond embankments, combined with little to no substantive maintenance over long periods of time, means the dam is in poorer condition than is either desirable or sustainable over time. Maintenance and repairs must be made in a timely fashion because further deterioration of the embankment will likely render the dam unsatisfactory for continued use in the near future.

Vegetation control is unsatisfactory, and tall and dense vegetation throughout the embankments makes a thorough inspection for seepage and other defects impractical.

Rodent control cannot be ascertained because of the tall and dense vegetation throughout the embankment.

Typed by J.A. Lowe
Date 4 June 2009
cc for Owner/Book

Inspected by J.A. Lowe
Date of Inspection 2 June 2009
Date of Report 3 June 2009
Photos taken? Yes [X] No [ ]

INSPECTION OF DAM AND RESERVOIR IN CERTIFIED STATUS

Name of Dam: Mill Pond   Dam No. 2381

Date of Inspection: 2 June 2009

Spillway: The spillway entrance is partially impeded by dense vegetation. Abundant floating vegetation throughout the pond, including immediately adjacent to the spillway entrance, makes the possibility of clogging of the spillway with floating vegetation and other debris during a storm a concern. The control section and exit channel were open and clear, and all flashboards have been removed from the spillway section. The "Petro Barrier" was in place in front of the spillway entrance.

Total freeboard is 5.2 feet and the residual freeboard for the design storm is 0.7 feet. Freeboard is marginal.

Outlet: There is no outlet.

Seepage and Drainage: Clear seepage from the base of the timber cribbing retaining wall was flowing at a rate of approximately 2 to 3 gpm. The rate of flow observed is similar to the 2 gpm reported by John Leonhardt during his May 18, 2001 inspection, but is less than the 10 to 15 gpm reported following the February 20, 2002 inspection. The reduced seepage rate is probably the result of lowering the reservoir by removal of the stop logs from within the spillway control section.

Standing water occupies the downstream toe of the right embankment at the former location of a water treatment plant. While several supply and feed pipes from the facility are known to perforate the embankment in this area, neither the condition of the pipes nor the status of pipe encasements is known. It appears likely that seepage from, or around, the pipes are a likely source of the seepage observed. While the seepage appears to be clear, tall and dense vegetation within the standing water prevents a thorough evaluation of seepage conditions.

Instrumentation: There is no instrumentation and none is believed necessary at this time.

The timber-cribbing wall to the left of the spillway remains in very poor condition and continues to deteriorate. Embankment material continues to be transported from behind the wall, opening voids and causing a loss of contact and support between the structural timbers and the supported embankment.

Author/Typist: J.A. Lowe

INSPECTION OF DAM AND RESERVOIR IN CERTIFIED STATUS

Name of Dam Mill Pond Dam No. 2381

Date of Inspection 2 June 2009

The concrete spillway (top photograph, above) is also deteriorating, and rebar is exposed beneath and behind the concrete facing (bottom photograph, below.

Author/Typist: J.A. Lowe

INSPECTION OF DAM AND RESERVOIR IN CERTIFIED STATUS

Name of Dam Mill Pond Dam No. 2381

Date of Inspection 2 June 2009

The spillway entrance is partially impeded by dense vegetation (top photograph, above). Abundant floating vegetation throughout the pond, including immediately adjacent to the spillway entrance, makes the possibility of clogging of the spillway with floating vegetation and other debris during a storm a concern.

Standing water occupies the downstream toe of the right embankment at the former location of a water treatment plant (bottom photograph, above). While several supply and feed pipes from the facility are known to perforate the embankment in this area, neither the condition of the pipe nor the status of pipe encasements is known. It appears likely that seepage from, or around, the pipes are a likely source of the seepage observed.

Author/Typist: J.A. Lowe

APPENDIX E

ARCADIS

Appendix E

Photo Log

Crib wall on the western dam wall exhibiting soil loss and cavity developing amongst the cribs

View of the western wall including the spillway and the cribwall showing beach erosion and shoreline loss

View of the cribwall and the spillway discharging directly on the beach

Spillway showing beach erosion and damage to the spillway concrete flow structure

View of the spillway discharging directly on to the beach

View of the north wall looking towards west, showing surficial sloughing

View of the oversteepened slope of the northern dam wall above Pond 7

Eroding concrete retaining wall along the north wall showing extensive leakage