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DRAFT Mill Pond Dam Supplemental Site Investigation Report

Former Georgia-Pacific Wood Products Facility

Fort Bragg, California

March 2015

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1. Introduction

This Supplemental Site Investigation Report was prepared by ARCADIS U.S., Inc. (ARCADIS) on behalf of Georgia-Pacific LLC (Georgia-Pacific) and describes field and laboratory data, and analyses of field conditions, liquefaction potential, and stability performed to support evaluation and design of mitigation measures for the Mill Pond Dam (Division of Safety of Dams [DSOD] No. 2391-000 – Mill Pond Dam, National Dam No. CA01139). The dam is located at the former Georgia-Pacific Wood Products Facility (Site) at 90 West Redwood Avenue, Fort Bragg, Mendocino County, California (Figure 1).

The Mill Pond is a 7.3-acre impoundment located in the central portion of the Site that is about 100 to 350 feet wide and about 1,600 feet long. The pond was formed by an earth fill dam that extends along the ocean-side boundary of the pond. Although the alignment of the dam trends southwest to northeast, the alignment has conventionally been referenced as a north to south structure and this convention is retained in this report. The dam is approximately 15 feet high with its crest at an elevation of about 43 to 44 feet above mean sea level. The dam includes a concrete spillway and crib wall near its southern end (Figure 2 and Figure 3).

2. Objectives and Scope of Work

In 2010, ARCADIS (2010a) completed an evaluation of the Mill Pond Dam that concluded portions of the Mill Pond Dam are built of, and on, soils susceptible to liquefaction under ground shaking associated with an earthquake on the nearby San Andreas fault. In the event of liquefaction, the evaluations showed that deformation of the dam structures or foundation could lead to release of water from the Mill Pond. Since the 2010 evaluation was completed, ARCADIS performed additional analyses (described in more detail below) to better characterize the Mill Pond dam conditions and to develop conceptual measures to mitigate the potential effects of liquefaction. These analyses indicated the following additional information was necessary to demonstrate the feasibility of the conceptual mitigation measures being considered at that time:

- The interpretation of geologic conditions along the alignment of the dam in the 2010 report was based on four, relatively widely spaced soil borings and a bedrock outcrop mapping along the face of the bluff in the vicinity of the spillway/overflow structure and crib wall. As a result, the characteristics and occurrence of the dam fill material and underlying native materials was uncertain.

- Little data were available to evaluate subsurface conditions at the toe and immediately downstream of the northern earth embankment. This information was judged to be necessary for the evaluation of potential mitigation measures along that portion of the structure.

- No data were available to evaluate the properties or extent of borrow soil potentially available for construction of a stabilization buttress (one of the preliminary mitigation concepts);

- Groundwater levels in the native unconsolidated deposits below and downgradient of the dam were not well characterized. This information is necessary to better characterize liquefaction potential as well as construction feasibility; and

- The number and the significance of abandoned pipe penetrations through the existing earthen embankment were not well characterized. This information was judged important to the further evaluation of mitigation measures that required excavation or modification of the embankment, crib wall, or spillway structures.

To address these data gaps, a field and laboratory testing program was completed that included geophysical exploration, cone penetration tests (CPTs), soil borings and standard penetration testing (SPT), installation of groundwater piezometers, completion of a borrow study, and laboratory testing of selected samples recovered from the borings. This information was then used as the basis for additional analyses used to update the overall understanding of the site conceptual model and to further evaluate potential mitigation measures for the dam.

3. Background

3.1 History and Setting

The Mill Pond was originally installed in 1885 by constructing a dam along and on top of the rock that comprises the edge of the coastal bluff (Stetson Engineers Inc., 2005). As part of this work, a section of dam was constructed by placing a redwood timber crib wall across the Alder Creek drainage channel and backfilling it with site soil. Site documents indicate a depression was excavated into the terrace deposits behind the dam to increase the storage capacity of the pond. The pond is approximately 1,700 feet long and between 120 and 350 feet wide. The pond location is shown on Figure 2.

The westernmost portion of the pond is located immediately adjacent to Soldier Bay, is about 500 feet long, and includes the concrete spillway/overflow structure and the crib wall. Franciscan Formation bedrock forms the abutments and foundation of the concrete spillway/overflow structure and portions of the left and right abutments of the crib wall. The approximately 1,200-foot-long northern section of the dam consists of an earth embankment variably founded on bedrock, native unconsolidated deposits, and possibly fill soils. Site studies (ARCADIS, 2009) show that a considerable amount of sediment is present behind the dam.

As described in more detail below, a former log deck was identified as a potential borrow area for stabilization buttress soil. The log deck was constructed concurrently with the initial development of the Site in the 1880s, presumably by importing large diameter, well-graded gravel and placing it to provide a relatively flat storage area for raw logs and finished lumber. The size of the log deck was expanded several times and it currently encompasses approximately 40 acres (Figure 2).

3.2 DSOD Inspection Findings

The Mill Pond Dam falls under the jurisdiction of DSOD and is identified as Dam Number 2381. DSOD periodically inspects the dam and in 2005 expressed “some concerns about the condition and the stability of the dam” (Stetson Engineers Inc., 2005). In 2007, DSOD re-inspected the dam and issued a report stating that repair was required. The inspection report also indicated that the spillway capacity of 585 cubic feet per second should be increased to 690 cubic feet per second to manage flow associated with the 1,000-year storm event.

In 2009, DSOD identified several deficiencies associated with portions of the dam, including the crib wall, spillway, right embankment (i.e., the northern earth embankment), along with concerns regarding a lack of general maintenance of the dam (DSOD, 2009). In April 2010, the DSOD notified Georgia-Pacific that the dam condition required corrective action to mitigate erosion at the toe of the concrete spillway and erosion of the crib wall backfill. DSOD also noted that seepage on the northern embankment could lead to piping and that the dam was more than 100 years old and was susceptible to damage from earthquakes (DSOD, 2010).

3.3 2010 Stability Assessment

ARCADIS completed a stability evaluation of the Mill Pond Dam in 2010 to address the DSOD inspection findings, results, and conclusions. The objectives of the 2010 stability assessment were to collect and evaluate relevant site geotechnical data and to use this information to assess the overall stability of the dam, including the crib wall, spillway, and northern embankment. Work performed for this study included advancing borings OUE-GT-001, OUE-GT-002, OUE-GT-003, and OUE-GT-004 at the locations shown on Figure 3. Figure 3 shows that all of the boring and CPT identifiers are preceded with “OUE-“ In the interest of brevity, this area-specific identifier is not used in the text of this report. As part of this investigation, selected soil samples obtained from the borings were tested in the laboratory for index properties, grain size, and shear strength.

The results of engineering evaluations indicated that portions of the dam are built on soils that are susceptible to liquefaction under ground shaking associated with the maximum credible earthquake (MCE) occurring on the San Andreas fault, which is located about 10 kilometers (km) from the Site. In the event of liquefaction, deformations of the dam structures and/or foundation are possible, which in turn, could lead to a release of water from the Mill Pond. The report further concluded that with the exceptions of the crib wall and spillway areas, little to no sediment would be released from the Mill Pond if liquefaction and deformation were to occur (ARCADIS, 2010a).

3.4 2010 Maintenance Project

In response to DSOD requirements and the findings of the 2010 Stability Assessment, Georgia-Pacific and ARCADIS completed maintenance activities between September 25, 2010 to October 20, 2010 that included filling crevices in the dam wall beneath the spillway and overflow structure with shotcrete, filling voids in the timber crib wall with flowable concrete fill, and installation of articulating block concrete mats and rip rap at the toe of the spillway to minimize erosion and scour. A summary of the maintenance work is presented in the *Mill Pond Dam Maintenance Completion Report* (ARCADIS, 2010b). The dam is currently inspected by Georgia-Pacific or ARCADIS on a monthly basis by walking the length of the dam, taking observational notes and photos. A summary report of observations is submitted to DSOD following each inspection.

3.5 2012 Pond Sediment Investigation

In March 2012, ARCADIS conducted an additional investigation in the Mill Pond to better characterize the distribution and geotechnical properties of the pond sediment. As part of this work, the thickness of sediment was measured at over 300 locations and sediment cores were collected at eight locations within the pond. Probes of the sediment in the pond indicate that the sediment is of variable thickness between approximately 6 and 9 feet thick near the western end, 10 and 12 feet thick at the narrow central area, and 13 and 24 feet near the northeastern end. The sediment generally thins toward the edges of the pond but generally thickens towards the pond’s northern edge.

The pond sediment is generally described as loose fine grain material with wood chips (the organic content of samples that were tested ranged between 20 and 50 percent). Vane shear tests at depths of 5 and 10 feet below the muck line indicates undrained shear strengths as low as 52 pounds per square foot (psf) to as high as 668 psf. The average shear strength at 5 feet was 131 psf and the average shear strength at 10 feet was 486 psf.

3.6 2013-2014 Geologic Characterization

Beginning in early 2013, additional work was performed to better characterize the geologic conditions affecting (or likely to affect) the long-term performance of the Mill Pond Dam. This work included: (i) a survey of rock outcroppings along the shoreline from just north of the concrete spillway/overflow structure to just south of the crib wall; (ii) review of geologic information from site borings and monitoring wells in the vicinity of the pond; (iii) review of the 2010 Stability Assessment and 2012 Pond Sediment Investigation information; (iv) preparation of additional geologic cross sections through the dam; (v) development of conceptual mitigation measures to stabilize different sections of the dam; and (vi) development of recommendations for additional field and laboratory testing to further develop and design mitigation measures for the dam structures.

During the course of this work, two meetings were held with DSOD to discuss the updated geological characterization, the conceptual mitigation measures, the uncertainties associated with the site data, and the general field investigation procedures that were being developed to support design.

3.7 DSOD Alteration Application

ARCADIS submitted a work plan and Alteration Application in accordance with DSOD requirements for the proposed additional investigation activities (ARCADIS 2014). The work plan and Alteration Application No. 2381 were approved DSOD in a letter dated October 17, 2014. The DSOD approval included several conditions including adjustment of the proposed boring locations, increased frequency of SPT, reduced frequency of sample collection, additional piezometer installations, and the results of the bedrock outcropping survey performed in 2013. The rock outcropping survey data are included in Appendix A.

4. Permits & Pre-Field Activities

4.1 Utility Clearance

The utility clearance included a notification to Underground Service Alert (USA), a visual site inspection, and a private utility locator. USA was notified 48 hours prior to the start of invasive activities. Utilities were cleared by NorCal Geophysical Consultants, Inc. (NorCal) of Cotati, California, on October 15-17, 2014. Buried utilities identified during the utility clearance were marked with paint on the ground surface and on maps and/or figures for use by field staff during the work. The results of NorCal's clearance are summarized in Appendix B.

4.2 Coastal Development Permit

An Administrative Coastal Development Permit was acquired from the City of Fort Bragg. A copy of the Notice of Final Action is included in Appendix C. ARCADIS complied with the conditions listed on the permit, particularly special conditions 1 through 3. ARCADIS notified Native American monitors in advance of work performed south of the spillway. ARCADIS field staff installed orange construction fencing around environmentally sensitive habitat areas located within 50 feet of the work areas prior to the start of invasive field activities. Best management practices (BMPs) including straw waddles were implemented during the fieldwork.

4.3 Boring Permit

A boring and well permit was obtained from Mendocino County, Health and Human Services Agency prior to implementing the fieldwork. A copy of the permit is included in Appendix C.

5. Investigation Activities

The field activities were completed in October of 2014 in substantial accordance with the ARCADIS (2014) Work Plan that was submitted as part of the Alteration Application and approved by DSOD prior to fieldwork. Activities included: geophysical exploration, CPTs, soil borings, SPT, installation of groundwater piezometers, completion of a borrow study, and laboratory testing.

5.1 Field Modifications to Work Plan

In several cases, it was necessary to modify the scope of work due to conditions encountered in the field. Modifications to the ARCADIS (2014) Work Plan included:

- Planned CPT-12-11 was not advanced due to inclement weather that created soft ground and prevented access of the CPT truck to this portion of the Site. As a result, an additional boring (GT-009) was advanced at the downstream toe of the earth embankment to provide additional data in this area. The boring was advanced to a depth of approximately 10 feet below ground surface (bgs) where wood debris was encountered. Shortly after refusal at the wood debris, the boring was terminated due to adverse weather conditions and potential safety hazards;
- Concrete coring on the spillway was not completed due to safety concerns; and
- Rock coring was performed to a depth of 5 feet below top of rock in borings located along the central and northern embankment portions of the dam (GT-007 and GT-008) due to drilling conditions. Rock coring was performed to depths of 25 and 20 feet below top of rock in borings GT-005 and GT-006, respectively.

5.2 Geophysical Exploration

NorCal performed geophysical exploration activities at the Site on October 15-17 and October 23, 2014. The utility clearance included electromagnetic line locating, metal detecting, electromagnetic profiling, and ground penetrating radar to clear the boring and test pit locations and to identify buried pipes located within the dam embankment. The location of buried pipes identified within the dam embankment were recorded using a hand-held GPS and are presented on Plate 4, Plate 5A, and Plate 5B in the NorCal Geophysical Investigation Report (Appendix B).

Seismic refraction (SR) and electrical resistivity (ER) surveys were performed to assist in identifying bedrock contact in the vicinity of the timber crib wall and spillway, and to assist in evaluating the extent of the timber crib wall. The SR and ER profiles are presented on Plate 1 through Plate 3 of the NorCal Geophysical Investigation Report (Appendix B). The NorCal Geophysical Investigation Report also includes a description of bedrock contact and the timber crib wall location as interpreted from the geophysical data.

5.3 Cone Penetrometer Tests

Eleven CPTs were advanced at the Site from October 22 through October 24, 2014 at the locations shown on Figure 3. CPT-12-1 through CPT-12-9 were advanced along the crest of the dam to provide a centerline profile. CPT-12-10 and CPT-12-12 were advanced at the downstream toe of the earth embankment.

The CPTs were advanced by Gregg Drilling & Testing, Inc. (Gregg Drilling) of Martinez, California, in general accordance with ASTM D3441. Data gathering included measurements of cone tip resistance, sleeve friction, and pore water pressure. Each completed CPT was advanced to bedrock. The CPT data, including the total depth of each CPT, the interpreted soil behavior type, and interpreted $(N_1)_{60}$ values, is provided in Appendix D. One pore water dissipation test was performed in each of the CPTs advanced along the crest of the dam (CPT-12-1 through CPT-12-10). The results of the pore water dissipation testing are provided in Appendix D.

5.4 Soil Borings

Five soil borings were advanced at the Site by Gregg Drilling from October 27 through October 31, 2014 at the locations shown on Figure 3. The soil borings were completed using mud rotary drilling techniques. Borings GT-005 through GT-008 were advanced along the crest of the dam as indicated in the work plan. Boring GT-009 was advanced to refusal at a depth of 10 feet bgs near the toe of the existing embankment. Borings GT-005, GT-006, GT-007, and GT-008 were paired with CPT-12-4, CPT-12-5, CPT-12-7, and CPT-12-8, respectively, to allow relatively direct comparison of physical soil boring information and soil property information inferred from the CPT data. The borings were completed to the total depths shown in the boring logs (Appendix E).

5.4.1 Soil Sampling

Soil samples collected from each boring were logged by a qualified ARCADIS staff member in general accordance with ASTM D2488 (visual-manual) procedures. The upper 5 feet of each boring were advanced by hand auger as part of the utility clearance procedures. The hand auger spoils were continuously logged. Soil samples were generally obtained using a split spoon sampler from borings GT-005, GT-007, and GT-008. Soil samples were generally obtained using a Modified California sampler from boring GT-006 to obtain relatively undisturbed samples of the terrace deposit materials. One Shelby Tube sample was collected from each boring at depths of less than approximately 12 feet to obtain relatively undisturbed samples of the embankment fill material. The boring logs are provided in Attachment E.

5.4.2 Rock Coring

Rock coring was completed in borings GT-005 through GT-008 to depths of between 5 and 25 feet below top of rock. Cored rock was backfilled with coated bentonite pellets to top of rock, at the request of the Mendocino County Environmental Health Department field inspector.

5.4.3 Piezometers

On completion, four of the CPTs (CPT-12-6, CPT-12-9, CPT-12-10, and CPT-12-12) and four of the soil borings (GT-005 through GT-008) were converted to piezometers. The remaining CPT locations and soil borings were backfilled with neat cement grout, in accordance with Mendocino County requirements. Two CPTs (CPT-12-6 and CPT-12-9) and four borings (GT-005 through GT-008) were completed as permanent piezometers with traffic-rated surface completion well boxes, and two CPTs (CPT-12-10 and CPT-12-12) were completed as temporary piezometers with above-grade PVC completions. The temporary piezometers were surrounded with delineators and caution tape for protection. It is anticipated that the temporary piezometers will be removed during subsequent remedial efforts at the Dam. Construction details for the piezometers are provided in Table 1, groundwater levels measured in the piezometers on October 31, 2014 shortly after installation and on February 5, 2015 are provided in Table 2, and piezometer locations are shown on Figure 4.

5.4.4 SPT & Energy Measurements

SPT was performed concurrently with split spoon sampling at approximately 2.5-foot intervals in each boring. SPT was performed in general accordance with ASTM D6066 procedures using an auto-hammer.

Gregg Drilling measured the hammer/drill rod transfer energy in general accordance with ASTM D4633 from all SPT intervals in borings GT-005 and GT-008. As described by Gregg Drilling, the average energy transfer ratio in borings GT-005 and GT-008 was 79 percent and 76 percent, respectively. A more detailed discussion of the energy transfer methods and computations are provided in a letter report from Gregg Drilling, included in Appendix F.

The raw N-values obtained from the SPT were converted to normalized N-values, called $(N_1)_{60}$ values, using methods developed by Seed et al. (1985) and Skempton (1986). The blow counts obtained from the Modified California sample intervals were converted to raw SPT N-values using a ratio of 0.55, after Rogers (2006). The SPT results are summarized in Appendix G.

5.5 Borrow Study

The former log deck at the Site has been identified as a potential borrow source for stabilization buttress material. The location of the former log deck is shown on Figure 2. Ten test pits were excavated in the former log deck by Gary A. Swanson Excavating, Inc. of Fort Bragg, California, on October 22, 2014. Soils from each test pit were logged by a qualified ARCADIS staff member in general accordance with ASTM D2488 (visual-manual) procedures and soil samples were collected from select locations for laboratory analysis. The test pits were backfilled with native material and compacted by tamping with the excavator bucket and track-walking to provide a firm and generally non-yielding surface. Each test pit location was surveyed using hand-held GPS equipment by NorCal. The location of each test pit is shown in the NorCal Geophysical Investigation Report (Appendix B) and test pit logs are provided in Appendix E.

5.6 Laboratory Testing

Selected samples taken from the borings and test pits were submitted to Cooper Testing Laboratory, Inc., of Palo Alto, California. Soil samples were analyzed for a suite of soil properties including:

- Atterberg limits in accordance with ASTM D2487, procedures to characterize the samples with respect to the Unified Soil Classification System (USCS).

- Grain size in accordance with ASTM D422, methods to characterize the distribution of particle sizes. Distribution of particle sizes larger than 75 micrometers (µm; No. 200 sieve) were measured by sieving and distribution of particle sizes smaller than 75 µm were measured by hydrometer.

- Shear strength tests were performed on relatively undisturbed samples of embankment fill soil and the underlying terrace deposits. Consolidated-undrained triaxial (CUTX) tests were performed in accordance with ASTM D4767 procedures to provide information regarding total and effective strengths of the materials tested.

- Compaction tests were performed on bulk samples recovered from the test pits in accordance with ASTM D1557 (Modified Proctor) procedures.

The laboratory reports are included in Appendix H and the results of testing are summarized in Table 3.

5.7 Surveying

DobleThomas & Associates, Inc., of Healdsburg, California, surveyed the top-of-casing and ground surface elevations at each boring and CPT location on November 18, 2014. The horizontal coordinate system used during the survey was NAD 83 and the vertical coordinate system was NAVD88. Several previously surveyed points were re-measured to verify the current survey data was comparable to historical measurements. The piezometer top-of-casing and ground surface elevations are summarized in Table 1 and the surveyed boring, CPT, and piezometer locations are shown on Figure 3 and Figure 4.

5.8 Investigation Results and Interpretation

Results obtained from the investigation activities were used to update ARCADIS' understanding of subsurface conditions at the Site. The geologic and seismologic setting are discussed in the 2010 stability assessment (ARCADIS, 2010a). The boring logs, laboratory test results, and CPT data were used to construct profiles of the dam embankment centerline. A section location map is provided on Figure 5, and profiles of the crib wall-spillway portion of the dam and the northern embankment portion of the dam with exaggerated vertical scales are presented on Figures 6 and 7 and profiles with equivalent horizontal and verticals are presented in Appendix I. Profiles of the dam centerline that include overlays of the geophysical ER and SR results are presented on Figures 8 and 9 and in Appendix I.

5.8.1 Soil Fill

5.8.1.1 Embankment

The upper 12 to 23 feet of material along the alignment of the dam consists of undocumented fill that consists of silty sand, sandy silt, fine-grained clays and silt, and occasional gravel. The CPT data indicated more fine-grained soil was present than was logged by field personnel in the boring that was paired with selected CPTs. Similar differences in the fill are noted when laboratory data are compared to the fines content estimated based on the CPT data. These differences are shown in the dam profiles where the paired CPT/borings are plotted next to each other (Figure 6, Figure 7, and Appendix I) and in the comparison of laboratory and CPT fines content plots (Figure 10). As described in more detail below, the liquefaction analyses varied the soil content and Soil Behavior Index to assess the sensitivity of the analysis on varying fines content.

The CPT and SPT based (N1)60 values correlate well in the embankment fill layer (Figure 11). This soil layer is loose and is characterized by lower (N1)60 values than deeper soil layers (Figure 11). The fill material consists of interbedded silty sand which has a fines content that ranges from 20 to 36 percent finer than the number 200 sieve, and poorly graded sand that has a fines content that ranges from 2 to 5 percent finer than the number 200 sieve. The silty sand appears in both samples from boring GT-005 near the crib wall-spillway portion of the dam, and interbedded silty sand and poorly graded sand appear in samples from borings GT-006 and GT-007 in the central portion of the dam. One sample from boring GT-007 was classified as SW.

The moist unit weight of the fill ranges from 95 to 107 pounds per cubic foot in borings located in the crib wall-spillway portion and central portion of the dam (GT-005, GT-006, and GT-007). The laboratory results indicate that the fill material moist unit weight is higher in one sample collected in boring GT-008 in the northern embankment portion of the dam (116 pounds per cubic foot). Field and laboratory soil descriptions indicate that the fill material is non-plastic, except for one sample each from borings GT-005 and GT-008, which are listed as clayey sand in the laboratory descriptions.

A CUTX test with pore pressure measurements was performed in the laboratory to quantify the strength of the fill material. The laboratory-measured soil strength is higher than the strength inferred from the low (N1)60 values measured in this material. The laboratory strength measurement may overestimate the actual soil strength. The reason for this discrepancy may be due to the low density of the material in situ. A shear strength lower than the laboratory-measured value was assumed for this material in the stability analysis, as described in more detail below.

5.8.1.2 Toe and Downstream of Embankment

Shallow soils along the toe of the northern dam embankment appear to consist of undocumented fill that consists of soft silty sand with minor amounts of gravel and wood debris based on field descriptions. The strength of this fill material is low based on measured SPT (N1)60 values between 4 and 7 blows per foot in GT-009 (Appendix G), and CPT (N1)60 values generally less than 10 blows per foot (Appendix D).

The extent of this fill material was evaluated from CPT-12-10, CPT-12-12, GT-009, as well as several borings previously completed at the Site (DP-077, DP-078, and HA-013). This fill material appears to extend to bedrock at the toe of the northern dam embankment (CPT-12-12) and to a depth of approximately 8 feet bgs in CPT-12-10. Fill material was encountered to the termination depth of 10 feet bgs in GT-009 at the toe of the central portion of the dam. The interpreted extent of this fill material is summarized on Figure 12.

5.8.2 Marine Deposits

This soil unit was encountered below the fill material in the crib wall-spillway section of the dam, and beneath portions of the northern embankment section of the dam at thicknesses of 0 to 8 feet. The marine terrace deposits generally consist of sand with occasional gravel, silt, and clay. Two samples from boring GT-008 were classified as SM and SC-SM. The SPT (N1)60 values and CPT (N1)60 values correlate well in the marine deposits (Figure 11). The marine deposits are dense and are characterized by (N1)60 values greater than approximately 15 blows per foot and soil behavior types that classify as sand. Marine deposits do not appear in GT-007, CPT-12-7, or CPT-12-9 and are not continuous across the profile of the central and northern embankment portions of the dam. Geologic interpretations of marine deposits are shown on Figures 6 through 9 and in Appendix I.

The laboratory-measured unit weight and strength of the marine deposits is based on several samples collected in a Modified California sampler from boring GT-006. One of the samples was described as weathered sandstone by the laboratory, and was not used in characterization of the marine deposits. The laboratory-measured moist unit weight of this material is within the range 101 to 108 pounds per cubic foot. Field soil descriptions indicate that the material is non-plastic, as confirmed by one laboratory test from boring GT-008.

The fines content of the marine deposits in the crib wall-spillway portion of the dam (GT-005 and GT-006) are lower (4.1 to 13.8 percent finer than the No. 200 sieve) than the fines content in the marine deposits in the northern embankment portion of the dam (GT-008; 19.8 to 45.4 percent finer than the No. 200 sieve).

The strength of the marine deposits were quantified in the laboratory by a CUTX test. The laboratory-measured effective soil strength is comparable to the strength inferred from the moderate (N1)60 values measured in this material. The strength was calculated from samples collected in a Modified California sampler from two different depths in boring GT-006. The Modified California samples used in the testing were collected from the 17.5-19.0 feet bgs depth interval and the 22.5 to 24.0 feet bgs depth interval. A third sample of material from the 20.0-21.5 feet bgs depth interval was tested in the lab, but the results of this test were not used to calculate a friction angle because the sample consisted of weathered sandstone. The laboratory test results are summarized in Appendix H.

5.8.3 Bedrock

Bedrock of the Franciscan Formation was encountered in each of the borings at depths of approximately 15 to 30 feet bgs. The bedrock formation encountered at the Site consisted of a mélange of marine-deposited sandstone and shale in various states of weathering. The bedrock observed in cores was heavily weathered and friable when first encountered and generally became more competent 10 to 15 feet into the formation.

A bedrock outcrop survey was performed in 2013 to measure the elevation of the top of bedrock along the shoreline in the vicinity of the crib wall and spillway. The bedrock outcrop survey data are provided in Appendix A. Some of the bedrock outcropping survey data are depicted on the crib-wall/spillway profile (Figure 6) and on the EM and SR geophysical profile overlays on Figure 8 and Figure 9, respectively. The bedrock outcropping survey correlates well with the bedrock contact elevation inferred from the borings, CPTs, and geophysical results. These profiles indicate that the spillway is likely founded on bedrock, and that the former stream channel is likely located between OUE-CPT-12-3 and OUE-CPT-12-4.

5.8.4 Groundwater

Groundwater depths and elevations measured in the field area summarized in Table 2. At the completion of the drilling activities on October 31, 2014, groundwater depths varied from 2.5 feet bgs at the toe of the northern embankment (PZ-16) to 16 feet bgs in the central portion of the dam (PZ-6). The depths to groundwater were measured again on February 5, 2015 to assess whether the levels fluctuated following the initial measurements. As shown in Table 2, the water level measured in PZ-6 was approximately 12 feet higher on February 5. Fluctuations on the order of 0 feet (PZ-9) to about 1.4 feet (PZ-10) were measured in the other piezometers.

5.8.5 Log Deck

Soils in the former log deck area generally consist of coarse well-graded gravel that is underlain by coarse sand. The coarse gravel was apparently imported during the original construction of the log deck and the underlying sand likely consists of native deposits. The gravel was very dense and cobbles greater than 12 inches in their maximum dimension were occasionally encountered in the test pits. The gravel layer varies in thickness from about 0.5 feet to more than 4 feet thick. Little fine-grained material was observed in the gravel layer, and was confirmed by laboratory tests (8 and 14 percent finer than the number 200 sieve). The coarse sand underlying the gravel was dense and contained few fines (21 percent finer than the number 200 sieve).

Maximum dry densities of 137 and 142 pounds per cubic foot at optimum moisture contents of 6 to 7 percent were measured in the laboratory. The results of laboratory testing on the log deck sand and gravel materials are summarized in Table 3.

6. Geotechnical Analysis

6.1 Liquefaction

Principal factors that influence liquefaction potential are soil type and depth, grain size and the percentage of fine-grain soil, soil plasticity, soil relative density, groundwater level and degree of saturation, and the intensity and duration of ground shaking. In-situ CPT and SPT data are the two most widely used indices for evaluating the liquefaction characteristics of soils. The SPT was used first in developing liquefaction correlations. The evaluations summarized below were based primarily on CPT data that were calibrated based on laboratory test results and the paired CPT/SPT locations along the crest of the Mill Pond Embankment.

6.1.1 Analysis Methods

A deterministic, semi-empirical, stress-based approach for evaluating the potential for liquefaction triggering was initially developed by Seed and Idriss (1967) to compare the earthquake-induced cyclic stress ratio (CSR) with the cyclic resistance ratio (CRR) of the soil horizon under consideration[1]. Although the Seed and Idriss (1967) procedure has been refined and updated based on additional case history data and evaluations, the CSR/CRR ratio still provides the fundamental basis for evaluating whether or not a soil is likely to liquefy in an earthquake. The stress-based liquefaction analysis framework for cohesionless soil includes four relationships that describe fundamental aspects of dynamic site response, penetration resistance, and soil characteristics and behavior. These four relationships, along with the major factors affecting each, are:

- The shear stress reduction coefficient (r_d) that is a function of depth; earthquake and ground motion characteristics; and dynamic properties of the soil;
- The SPT or CPT overburden correction factor (CN) that is a function of the vertical effective stress (σ'_v), relative density of the soil (D_R), and the fines content (FC) of the soil;
- The overburden correction factor (K_σ) that is a function of σ'_v, D_R; and FC; and
- The magnitude scaling factor (MSF) that is a function of earthquake and ground motion characteristics; D_R, and FC.

For the purposes of the current investigation, the potential for liquefaction of the Mill Pond embankment fill and the underlying deposits was evaluated by the following methods:

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[1] The CSR at a given depth in a soil profile is usually defined as 65 percent of the ratio of maximum shear stress to the effective overburden stress for a specific earthquake magnitude and in-situ effective stress. The choice of the reference stress level (i.e., the factor 0.65) was selected by Seed and Idriss (1967) and has been in use since that time. Selecting a different reference stress level would alter the values of certain parameters and relationships but would have no net significant effect on the final outcome of the derived liquefaction evaluation procedure, as long as this same reference stress level is used throughout, including forward calculations. The soil's CRR is typically correlated to an in-situ parameter such as CPT penetration resistance or SPT blow count after the application of procedural and correction factors that account for overburden stress effects, duration of shaking, sustained static shear stresses, and the fines content of the soil.

- The SPT data were evaluated in general accordance with the recommendations and procedures included in the 1998 National Center for Earthquake Engineering Research and National Science Foundation Workshops on Evaluation of Liquefaction Resistance of Soils (Youd et al., 2001) as updated by Idriss and Boulanger (2008); and

- The CPT data were evaluated in general accordance with the recommendations and procedures included in the Boulanger and Idriss (2014) CPT and SPT Based Liquefaction Triggering Procedures monograph. One of the principal modifications recommended by Boulanger and Idriss was a revised MSF that incorporates functional dependency on the soil characteristics and the earthquake magnitude. The revised MSF was found to improve the agreement between CPT-based and SPT-based liquefaction triggering correlations and their respective case history databases.

As described in more detail below, the results of the CPT-based liquefaction analysis results were adjusted to approximately calibrate to the SPT-based analytical results where laboratory fines content information was available. In most cases, this resulted in a more conservative assessment of liquefaction potential.

6.1.2 Input Parameters

6.1.2.1 Ground Motion

A deterministic evaluation of potential ground motions associated with known faults within 100 km of the Site was completed using the most recent California Geological Survey and U.S. Geological Survey fault model for California (Field et al., 2008; Petersen et al., 2008). The potential ground motions for each source were then calculated as the equally weighted average of four commonly accepted Next Generation Attenuation (NGA) relationships for relatively shallow crustal faults in California and other active tectonic regions. The most recent (2014) NGA relationships were assumed for analysis (Abrahamson et al., 2014; Boore et al., 2014; Campbell and Bozornia, 2014; Chiou and Youngs, 2014) because these relationships represent an update of previous (2008) NGA models by the same authors and considered analysis of an expanded ground motion database and the results of numerical simulations.

The results of this analysis are summarized in Table 4 and are included in Appendix J. As shown in this table and appendix, ground motions controlled by the Northern San Andreas fault, which at its closest point is located about 10 km west of the Site. The

Northern San Andreas fault is broken into north-to-south segments identified as the Offshore (SAO), North Coast (SAN), Peninsula (SAP), and Santa Cruz (SAS) segments. For the purpose of this evaluation, a design earthquake of moment magnitude 8.05 was assumed based on the simultaneous rupture of all segments of the fault at a distance of 10.2 km from the Site (i.e., the source was assumed to include rupture of the SAO+SAN+SAP+SAS segments). Based on the referenced NGA relationships, this earthquake would result in a median site PGA of 0.34g.[2]

Previous site-specific response analyses (ARCADIS, 2010a) showed that no amplification of free-field bedrock ground motions at the Site would be expected for an earthquake of this magnitude and that some attenuation of the ground motions were likely in the terrace deposits. However, the amount and magnitude of attenuation was inconsistent between the different locations selected for the ARCADIS (2010a) response analyses. As a result, a uniform PGA within the terrace deposits and overlying soil fill was assumed for the current liquefaction analyses.

6.1.2.2 Soil Properties

Comparison of the CPT data (Appendix D) with the logs of paired borings (Appendix E) shows that both the CPTs and boring logs identify the terrace deposits predominantly as sand and gravel, with occasional silty sand. However, the boring logs identify the overlying fill as loose sand with occasional gravel and the CPTs identify the fill soil behavior type as variably consisting of fine-grained soil at most locations or as a layered sequence of fine-grained soil and sand, sandy silt, and clayey silt at other locations. These lithological differences notwithstanding, Figure 11 shows normalized SPT blow

[2] As described in ARCADIS (2010a), the statistical level of acceleration for deterministic seismic hazard analyses was based on 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 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). For the case of the Mill Pond embankment, the damage potential for each of these factors is “low” and the overall TCW for this dam is zero. Accordingly, the dam was analyzed for the 50th percentile statistical level.

counts (SPT $[N_1]_{60}$) calculated for the CPT profiles compare relatively well with the energy-corrected SPT $(N_1)_{60}$ values obtained from the paired borings. The sensitivity of the analysis to the potential clay content of the soil was evaluated by varying and correlating the soil behavior index parameter with SPT and laboratory test results.

6.1.2.3 Fines Content

In general, the published correlations between I_c and fines content are poor (Idriss and Boulanger, 2008). As a result, selected samples recovered from the paired borings were analyzed in the laboratory (Appendix H) and these results are compared with the fines content inferred from the CPTs on Figure 10. As shown on this figure, although the data are limited, the CPT fines content for the terrace deposits is generally consistent with the laboratory fines content measured in the laboratory. The CPT-inferred fines content for the overlying soil fill is less consistent with the laboratory data and there is more uncertainty regarding the fines content of this layer. As a result, the sensitivity of the analysis results to the fines content was addressed in accordance with Boulanger and Idriss (2014) recommendations summarized below.

6.1.2.4 Age Effects

A number of researchers have shown that the liquefaction resistance and the liquefied residual shear strength of sand increases with the age of the deposit (e.g., Arango et al., 2000; Leon et al., 2006). This effect is not entirely reflected by CPT or SPT penetration resistances and liquefaction evaluations for pre-Holocene sands using these data may be overly conservative. The terrace deposits at the Site are Pleistocene in age. The Pleistocene time period spanned from 2.6 million to 11,700 years ago and estimating the actual age of the terrace deposits would require laboratory age-dating that is beyond the scope of this investigation. Therefore, for the purposes of analysis, the terrace deposits were conservatively assumed to be deposited at the end of the Pleistocene epoch (i.e., they are about 11,700 years old). Strength gain factors for 10,000 year old materials reported by different researchers (Lewis et al., 1999; Arango and Migues, 1996) range from about 1.3 to 3 and Leon et al. (2006) indicate a value of 2.3 is consistent with the available data. For the purposes of this analysis, a conservative aging factor of 1.5 was assumed for the terrace deposits (as described in more detail below, applying this factor makes little difference to the overall results and conclusions of this study).

6.1.2.5 Sensitivity and Calibration

Parametric sensitivity analyses were performed to address the uncertainty in fines content and the soil behavior soil indices calculated from the CPTs. The sensitivity analyses included comparison of liquefaction triggering safety factors for the following conditions at the paired CPT/SPT locations:

- A base case, where the fines parameter C[FC] was set to 0 and the soil behavior index I_c was set to be 2.4;
- A case where C[FC] parameter was set to be -0.29;
- A case where the I_c parameter was lowered to from 2.6 to 2.4;
- A case where the I_c parameter was increased from 2.6 to 2.8;
- A case where the aging factor for the terrace deposits was assumed to be 0, 1.5, and 2; and
- A case where the most critical combination of C[FC] and I_c were incorporated into the analysis.

These analyses were then compared against the liquefaction triggering safety factors calculated for paired soil borings using the boring SPT blow counts and the results of laboratory grain size testing. The results of these analyses are shown on Figure 13 and indicate adjusting the soil behavior index up to 2.8 from the default value of 2.6 provides the best fit to the SPT and laboratory test data. It is significant to note that varying the different parameters “shift” the liquefaction points to the left or the right depending on parameter but do not alter the fundamental determination of liquefaction or no liquefaction at any location.

6.1.3 Liquefaction Results

The results of the analyses are summarized on Figure 14. As shown on this figure, both the embankment fill and the underlying terrace deposits contain zones of liquefiable material. However, the continuity of these zones is variable and it is usually not possible to trace any one zone across multiple borings or CPT locations. This is illustrated in the extended cross sections shown on Figure 15, Figure 16, and in Appendix I.

6.2 Slope Stability Analyses

6.2.1 Cross Sections Analyzed

Slope stability analyses were performed for the following three representative cross sections through the dam at the locations shown on Figure 5:

- Section A-A' is located through the approximate line of the crib wall and is adjacent to the spillway. This section was selected for analysis because it includes the relatively greatest thickness of fill and marine terrace deposits over bedrock. As described in more detail below, the contribution of the crib wall to stability of the embankment was neglected for the purposes of analysis. As a result, the analytical results for this section are very likely conservative;

- Section F-F' is located through the central portion of the dam where the dam section is the thickest. This section was selected: (i) to assess whether or not the overall thickness of this section was sufficient to preclude failure affecting the reservoir; and (ii) because the field investigation program indicate relatively high liquefaction potential of the marine terrace deposits in the portion of the Site in comparison to adjacent sections of the dam; and

- Section G-G' is located through the northern portion of the dam where the dam section is the thinnest and where the results of analyses indicated the potential for liquefaction of the embankment fill and some of the underlying marine terrace deposits is relatively high.

Generalized geologic interpretations and zones of potential liquefaction for these sections are shown on Figure12. The actual profiles used for analysis are shown with the stability analysis plots in Appendix K.

6.2.2 Geologic Conditions and Material Properties Assumed for Analysis

Native materials underlying the Mill Pond Dam include marine terrace deposits of variable thickness that are underlain by Franciscan Formation bedrock. As shown in the site cross sections and profiles, site data indicate the marine terrace deposits locally pinch out and are not present at all locations. The cross sections and profiles also show that the top of the bedrock surface is irregular. The dam was constructed of fill that includes varying amounts of debris and a crib wall was placed across Alder Creek to provide support at the time the dam was completed. Materials behind the dam include unconsolidated sediments and water. Materials at the toe of the dam include native terrace deposits and bedrock adjacent to the crib wall and spillway section and undocumented fill along the remainder of the alignment. Observations and limited subsurface data indicate the fill adjacent to the northern section of the dam contains appreciable debris in the form of wood, timbers, concrete, and occasional steel pipe.

Material properties assumed for analysis are summarized in Table 5 and were based on data from the 2010 Stability Evaluation, the 2012 Sediment Survey, the field and laboratory test results from the current investigation, and engineering judgment. As indicated in Table 5, the principal properties and assumptions associate with each material were:

- **Embankment Fill.** The unit weight is based on the lab testing data for this material. The effective strength of the material is based on the lab testing data (CU test) and correlations from field data (CPTs and SPTs). The total strength of the material is based on the lab testing data (CU test). The undrained strength of the material uses the effective strength value. The liquefied strength (residual strength) is based on correlation between residual shear strength ratio and equivalent clean-sand SPT-corrected blow counts using average $(N1)_{60\text{-}cs}$ =10.

- **Marine Terrace Deposits.** The unit weight is based on the lab testing data for this material. The effective strength of the material is based on the lab testing data (CU test) and correlations from field data (CPTs and SPTs). The total strength of the material is based on the lab testing data (CU test). The undrained strength of the material uses the effective strength value. The liquefied strength (residual strength) is based on correlation between residual shear strength ratio and equivalent clean-sand SPT-corrected blow counts using average $(N1)_{60\text{-}cs}$ =24.

- **Sediment Behind Dam**. The unit weight is based on experience with similar material. The effective strength and undrained strength of the material are selected assuming the material is normally consolidated.

The bedrock was assumed to be impenetrable for the purposes of analysis.

6.2.3 Groundwater Conditions

Groundwater conditions assumed for analysis were based on the groundwater levels measured in the piezometers installed at the locations shown on Figure 4. As shown in Table 2, the groundwater elevation measured in PZ-6 on February 5, 2015 was approximately 12 feet higher than the first measurement on October 31, 2014. Fluctuations on the order of 0 feet (PZ-9) to about 1.4 feet (PZ-10) were measured in the other piezometers. The most conservative (highest) water levels were assumed for the analyses.

6.2.4 Analysis Methods

Static stability analyses for the embankment were based on Spencer's Method of Analysis using the computer program SLOPE/W (v7.23; GeoStudio, 2007). Search routines were used to evaluate both rotational and non-circular failure mechanisms and to calculate the static safety factor, pseudostatic safety factor, and yield acceleration for the different cross sections. For the purposes of analysis, the bedrock was assumed to be impenetrable and failure surfaces were not allowed to pass through this material.

For the non-liquefied condition, seismic deformations were calculated for each section using the commonly accepted Makdisi and Seed (1978) procedure and the more recent Bray and Travasarou (2007) procedure. The deformation analyses assumed a M_w 8.05 earthquake on the San Andreas fault at a distance of 10 km from the Site and a site PGA of 0.34g (Table 4). The potential for liquefaction to affect the dam was evaluated by calculating the static safety factor for failure surfaces judged likely to result in unacceptable dam performance. Both front-facing and reservoir-facing failure surfaces were considered for the purposes of assessing the potential effects of liquefaction.

6.2.5 Results of Analyses

The results of the analyses are summarized in Table 6 and the stability model output and seismic stability calculations are included in Appendix K. As indicated in Table 6:

- **Section A-A (Southern Section and Crib Wall).** The analyses indicate a static safety factor of 1.28 for this section. However, the analyses did not incorporate the crib wall, and as a result, are conservative because the crib probably provides appreciable support for the dam at this location. Potential seismic deformations assuming no liquefaction of the underlying deposits indicate permanent deformation on the order of 5 inches for both the Makdisi and Seed (1978) and Bray and Travasarou (2007) procedures. This is also conservative because the analyses did not consider the crib wall. If the subsurface materials susceptible to liquefaction are assumed to liquefy, the analyses indicate a safety factor less than 1 and that the most critical failure surface would compromise most of the dam and a portion of the underlying marine terrace deposits. Although not incorporated into the analysis, the crib wall is judged unlikely to provide sufficient support to mitigate the effects of liquefaction.

- **Section G-G' (Central Section)**. The analyses indicate a static safety factor of 1.43 for this section. Although lower than 1.5, the failure surface occurs on the outboard, downstream face of the dam and does not come within more than 150 feet from the reservoir. Deformation analyses under non-liquefied seismic conditions indicate displacement of less than 1 inch for both procedures. Analyses under liquefied conditions indicate that the most critical failure surface would be located more than 100 feet from the reservoir and that safety factors on the order of 3 are associated with failure surface that propagate to within 20 feet or less of the reservoir.

- **Section F-F' (Northern Section)**. The analyses indicate static safety factors on the order of 2.1 for this section of the dam. Deformation analyses indicate less than 1 inch of displacement for the non-liquefied condition. If the various layers within embankment and underlying materials are assumed to liquefy, the analyses indicate a safety factor of less than 1 and that the failure surface compromises most of the embankment fill and reaches the reservoir. The analyses indicate that slumping back into the reservoir is possible but less critical than sliding in a downstream direction.

7. Findings

The field investigation, laboratory testing, and engineering analyses summarized above and presented in the tables, figures, and appendices of this report were performed to better characterize conditions along and adjacent to the Mill Pond Dam. This work was approved by DSOD because the current understanding of the dam and the need for mitigation measurements was based largely on four, relatively widely spaced soil borings and limited laboratory test data. As a result, there was considerable uncertainty regarding potentially applicable mitigation measures for the structure. Overall findings from this evaluation are summarized below.

7.1 Geologic and Site Conditions

The soil borings and CPTs advanced for this study indicate the dam consists of relatively loose undocumented fill overlying marine terrace deposits. The fill is predominantly fine-grained silty sand but also contains lenses and layers of fine-grained soil and gravel. Occasional wood and other debris is present in the fill and a number of penetrations (mostly pipes) penetrate the fill at various locations. Most of the pipelines detected by the geophysical survey trend parallel or sub-parallel to the dam axis, although several pipelines were identified that appear to pass through the dam at a high angle to the dam axis. The density of the fill appears to be relatively greater at the southern end of the fill (south of the crib wall) and does not appear to be susceptible to liquefaction at this location.

The fill is underlain by marine terrace deposits that also consist primarily of sandy material. The transition from the overlying fill to the terrace deposits was typically observed in the CPTs as a relatively distinct increase in penetration resistance. The thickness of the terrace deposits is variable and it pinches out and is not present at some locations along the dam. Layers with the terrace deposits are susceptible to liquefaction although these layers do not appear to be continuous and generally cannot be correlated between the borings and CPTs.

The terrace deposits are underlain by Tertiary-Cretaceous marine sediments of the Coastal Belt Franciscan Formation that typically consist of well-consolidated sandstone, shale, and conglomerate in the vicinity of the dam. Seismic velocity profiles indicate the bedrock is moderately weathered to depths of about 2 to 20 feet. Geophysical data and site observations indicate the bedrock surface elevation varies over relatively short distances.

Undocumented fill is present at the toe of the dam beginning north of the spillway and extending to the northern end of the dam. The fill was difficult to drill due to the presence of debris that stopped or appreciably slowed drilling progress. Limited data indicate the fill may be 10 or more feet thick on the downstream side of the dam. Groundwater was encountered within several feet of the ground surface in the fill and readily flowed into the drill holes and shall hand-dug excavations used to clear debris prior to drilling.

The reservoir on the upstream side of the dam is largely filled with saturated, relatively soft sediment that varies in thickness from about 6 and 9 feet thick near the western end, about 10 and 12 feet thick at the narrow central area, and about 13 and 24 feet thick near the northern end. The pond sediment consists of loose fine-grain material with wood chips (the organic content of samples that were tested ranged between 20 and 50 percent). In-situ vane shear data indicate the sediment is relatively weak. The water column on top of the sediment is typically several feet or less.

7.2 Crib Wall

The crib wall was constructed using interlaced wood timbers and appears to be wedge shaped with a flat top and downward taper. The crib wall is seated in what is interpreted to be a form channel that is incised into bedrock. The electrical resistivity survey performed for this study indicates the crib wall occurs at a depth of about 10 to 30 feet below the ground surface and may be about 70 feet long. The results of the geophysical surveys indicate the crib wall is seated in a drainage channel in the bedrock. The extent that the cribs extend back into the dam could not be determined from the geophysical survey, although the locations of the seismic refraction and resistivity survey lines indicate the timbers may extend at least to the centerline of the dam.

7.3 Liquefaction and Stability

Previous (ARCADIS, 2010a) analyses were based on very limited information and made the conservative assumption that saturated fill overlying the terrace deposits and the upper several feet of the terrace deposits would liquefy in response to strong ground shaking at the Site. Potentially liquefiable material was assumed to be extensive and continuous along the dam and throughout the vertical profile of the saturated fill and terrace deposits. While the results of the current study show that the potentially liquefiable layers are present in the terrace deposits and appear to extend below the fill on the downstream side of the dam, these layers are thinner and less continuous than previously assumed. Liquefaction is only likely within relatively thin and generally discontinuous layers and lenses of sandy material within the fill and terraced deposits.

Although the potentially liquefiable zones are thinner and less continuous than indicated in the earlier studies, the results of stability analyses indicate if liquefaction occurs, the integrity of the northern and southern sections of the dam will likely be compromised. The analyses indicate the central section is sufficiently wide that liquefaction and sliding in this area are unlikely to result in a failure of the dam in this area.

7.4 Finding with Respect to Conceptual Mitigation

Mitigation concepts developed prior to initiating the work described in this report focused on constructing a stabilizing buttress on the outboard side of the northern section of the dam and excavating the existing crib wall and replacing it with a reinforced concrete gravity structure that would also act as a new spillway for the dam. The results of the current study support these concepts and show that the log deck would provide sufficient quantities and quality of borrow material to construct the soil buttress. However, the stabilizing buttress would be deeper and longer than originally envisioned and foundation preparation in the vicinity of the crib wall would likely be more extensive than assumed.

The results also support the use of less intrusive ground improvement techniques that could sufficiently mitigate potential liquefaction and achieve acceptable factors of safety at the crib wall and north section. These techniques would significantly reduce the need for construction dewatering of deeper excavations in the vicinity of the stabilizing buttress and eliminate foundation preparation in irregular bedrock conditions in the vicinity of the crib wall. Implementing ground improvements would reduce implementation and cost uncertainty, effects on marine and wetland systems adjacent to the dam, and associated permitting and mitigation requirements.

Ground improvements techniques that are likely to achieve acceptable performance include deep soil mixing (DSM), jet grouting, diaphragm walls, vibroflotation, stone columns, and other similar techniques. One of these would be used to stabilize the existing embankment and underlying deposits, leaving the crib wall in place. Experience indicates DSM is likely to be the simplest and most cost effective to implement. Additional measures, such as rock fill, would be implemented to protect the ocean side of the crib wall from future wave action and would offer aesthetic and ecological benefits over a reinforced concrete structure. All alternatives will consider improving or relocating the existing spillway. Following a discussion of acceptable approaches, a recommended alternative will be included in the final Alteration Application Report to DSOD.

7.5 Limitations

This report has been prepared with a standard of care generally accepted in the geotechnical engineering practice. No other warranty, express or implied, is made. The analyses and recommendations contained in this report are based on the data obtained from soil borings and field observations. The boring logs and related information depict subsurface conditions only at the specific locations and times indicated. Subsurface conditions and water levels at other locations may differ from conditions occurring at the indicated locations. Also, the passage of time may result in a change in the conditions at these locations. The boring logs indicate subsurface conditions only at specific locations and times, and only to the depths penetrated. They do not necessarily reflect strata variations that may exist between such locations. If variations in subsurface condition from those described are noted during construction, recommendations in this report must be re-evaluated.

In the event that any changes in the nature, design, or location of the facilities are planned, the conclusions and recommendations contained in this report should not be considered valid unless the changes are reviewed and conclusions of this report modified or verified in writing by ARCADIS. ARCADIS is not responsible for any claims, damages, or liability associated with interpretation of subsurface data or reuse of the subsurface data or engineering analyses without the express written authorization of ARCADIS.

8. References

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Tables

Table 1
Piezometer Construction Details
Georgia-Pacific - Mill Pond Dam
Fort Bragg, California

Piezometer ID	Casing Diameter *(inches)*	Screen Interval *(feet bgs)*	Total Depth *(feet bgs)*	Ground Surface Elevation *(feet NAVD88)*	Top of Casing Elevation *(feet NAVD88)*	Comments *Well Set in Boring:*
PZ-4	1.5	5.0 - 20.0	20.0	43.73	43.50	OUE-GT-005
PZ-5	1.5	6.0 - 26.0	26.0	43.25	43.37	OUE-GT-006
PZ-6	0.75	3.5 - 28.5	28.5	43.92	44.24	OUE-CPT-12-6
PZ-7	1.5	4.5 - 19.5	19.5	43.75	43.84	OUE-GT-007
PZ-8	1.5	5.0 - 25.0	25.0	43.93	43.91	OUE-GT-008
PZ-9	0.75	5.0 - 20.0	20.0	43.71	43.70	OUE-CPT-12-9
PZ-10	0.75	4.6 - 14.6	14.6	19.35	20.01	OUE-CPT-12-10
PZ-12	1	2.0 - 22.0	22.00	28.66	28.82	OUE-CPT-12-12

Notes:
bgs = below ground surface
Depth to water measured from top of casing. Water levels taken within 12 hours of rain storm event.
Elevation based on a survey conducted by Doble Thomas & Associates, Inc., on November 18, 2014.
Elevations recorded in NAVD 88 coordinate system.

Table 2
Historical Piezometer Groundwater Elevation
Georgia-Pacific - Mill Pond Dam
Fort Bragg, California

Piezometer ID	Date Measured	Depth to Water *(feet below TOC)*	Top of Casing Elevation *(feet NAVD88)*	Groundwater Elevation *(feet NAVD88)*	Comments *Well Set in Boring:*
PZ-4	10/31/2014	14.42	43.50	29.08	OUE-GT-005
	2/5/2015	14.31	43.50	29.19	
PZ-5	10/31/2014	11.54	43.37	31.83	OUE-GT-006
	2/5/2015	12.76	43.37	30.61	
PZ-6	10/31/2014	16.15	44.24	28.09	OUE-CPT-12-6
	2/5/2015	4.10	44.24	40.14	
PZ-7	10/31/2014	5.24	43.84	38.60	OUE-GT-007
	2/5/2015	4.95	43.84	38.89	
PZ-8	10/31/2014	5.50	43.91	38.41	OUE-GT-008
	2/5/2015	5.93	43.91	37.98	
PZ-9	10/31/2014	5.26	43.70	38.44	OUE-CPT-12-9
	2/5/2015	5.26	43.70	38.44	
PZ-10	10/31/2014	12.30	20.01	7.71	OUE-CPT-12-10
	2/5/2015	10.94	20.01	9.07	
PZ-12	10/31/2014	2.48	28.82	26.34	OUE-CPT-12-12
	2/5/2015	2.35	28.82	26.47	

Notes:
Depth to water measured from top of casing (TOC).
10/31/2014 water levels taken within 12 hours of rain storm event.
Elevation based on a survey conducted by Doble Thomas & Associates, Inc., on November 18, 2014.
Elevations recorded in NAVD 88 coordinate system.

Table 3
Summary of Laboratory Test Results
Georgia-Pacific - Mill Pond Dam
Fort Bragg, California

Sample Identification	Sampler Type	Moisture (%)	In-Situ Dry Unit Weight (pcf)	Maximum Dry Unit Weight[a] (pcf)	Optimum Moisture Content (%)	PI	C_U	P200 (% finer)	Soil Classification	Soil Color	Soil Description	CU Triaxial Φ (°)	CU Triaxial Φ' (°)	Comments
OUE-GT-005-7.5-10	Shelby Tube	23.8	96.6	--	--	--	55	35.8	--	Black	Clayey Sand	--	--	
OUE-GT-005-14.5-15	Split-Spoon	21.1	--	--	--	--	55	22.1	--	Black	Silty Sand	--	--	
OUE-GT-005-18-18.5	Split-Spoon	16.1	--	--	--	--	28	9.1	--	Gray	Poorly Graded Sand with Silt & Gravel	--	--	
OUE-GT-006-9-11.5	Shelby Tube	--	--	--	--	--	33	25.3	--	Dark Brown	Silty Sand	15.3	39.3	
OUE-GT-006-12.5-14	Mod Cal	18.5	96.3	--	--	--	4	3.8	--	Gray	Poorly Graded Sand	--	--	
OUE-GT-006-15-16.5	Mod Cal	13.6	106.7	--	--	--	24	8.8	--	Olive	Poorly Graded Sand with Silt & Gravel	--	--	
OUE-GT-006-17.5-19	Mod Cal	19.4	107.6	--	--	--	2	4.1	--	Greenish Gray grading to Brown	Poorly Graded Sand	27.8	38.0	
OUE-GT-006-20-21.5	Mod Cal	13.9	121.7	--	--	--	--	--	--	Dark Grayish Green	Weathered Sandstone	--	--	
OUE-GT-006-22.5-24	Mod Cal	23.1	101.4	--	--	--	5	13.8	--	Greenish Gray	Silty Sand	--	--	
OUE-GT-007-7.5-10	Shelby Tube	5.8	100.6	--	--	--	17	2.3	--	Gray	Poorly Graded Sand with Gravel	--	--	Top of Shelby Tube
OUE-GT-007-7.5-10	Shelby Tube	21.4	94.5	--	--	--	25	20.2	--	Dary Gray	Silty Sand	--	--	Bottom of Shelby Tube
OUE-GT-007-16-16.5	Split-Spoon	36.0	--	--	--	--	3	5.1	--	Dark Gray	Poorly Graded Sand with Silt	--	--	
OUE-GT-007-18.5-19	Split-Spoon	16.6	--	--	--	NP	9	4.4	SW	Very Dark Gray	Well-Graded Sand with Gravel	--	--	
OUE-GT-008-9-11.5	Shelby Tube	15.9	116.1	--	--	--	54	25.7	--	Dary Greenish Gray	Clayey Sand	--	--	
OUE-GT-008-18.5-19	Split-Spoon	9.2	--	--	--	--	58	19.8	--	Olive	Silty Sand with Gravel	--	--	
OUE-GT-008-21-21.5	Split-Spoon	14.4	--	--	--	NP	44	45.4	SM	Olive	Silty Sand	--	--	
OUE-GT-008-23.5-24	Split-Spoon	15.6	--	--	--	6	30	40.9	SC-SM	Olive	Silty, Clayey Sand	--	--	
TP-2	Bulk	--	--	136.10	7.0	--	--	14.4	--	Dark Brown	Silty Gravel with Sand	--	--	Composite Sample (0-4 feet bgs)
TP-4-4.0	Bulk	--	--	--	--	--	25	20.9	--	Very Dark Brown	Silty Sand	--	--	
TP-10-0.5	Bulk	--	--	141.9	6.0	--	124	7.7	--	Dark Gray	Well-Graded Gravel with Silt and Sand	--	--	

Notes:
pcf = pounds per cubic foot
PI = plasticity index
Cu = coefficient of uniformity
P200 = percent of soil passing the No. 200 sieve by weight
CU = consolidated undrained (with pore pressure measurements)
a = by ASTM D1557
NP = not plastic
bgs = below ground surface

Table 4
Summary of Holocene Active Faults Within 100 Kilometers and Associated Peak Ground Acceleration at the Site
Georgia-Pacific Mill Pond
Fort Bragg, California

Fault	Approximate Distance (km)	Characteristic Moment Magnitude	Equally Weighted Mean PGA	Attenuation Relationship PGA (g)			
				a	b	c	d
N. San Andreas (SAO+SAN+SAP+SAS)	10	8.05	0.341	0.33	0.33	0.33	0.37
Maacama-Garberville	35	7.40	0.124	0.13	0.10	0.14	0.13
Bartlett Springs	61	7.30	0.075	0.09	0.06	0.08	0.07
Collayomi	96	6.70	0.027	0.02	0.03	0.03	0.02

Notes:

1. Fault distances and MCE magnitudes based on USGS 2008 Seismic Hazard Mapping Program (Field et al., 2008; Petersen et al., 2008).
2. Attenuation relationships:
   a. Abrahamson et al. (2014) NGA West 2
   b. Boore et al. (2014) NGA West 2
   c. Campbell-Bozorgnia (2014) NGA West 2
   d. Chiou and Youngs (2014) NGA West 2
3. Northern San Andreas Nested Fault Segments:
   SAN - North Coast Segment
   SAP - Peninsula Segment
   SAS - Santa Cruz Segment
   SAO - Offshore Segment
4. See Appendix F for 0.5 fractile (mean) spectral accelerations.

km = kilometers
MCE = maximum credible earthquake

Table 5
Summary of Material Properties Assumed for Analysis
Georgia-Pacific Mill Pond Dam
Fort Bragg, California

MATERIAL	UNIT WEIGHT (lb/ft^3)	SHEAR STRENGTH						BASIS FOR PROPERTIES
		Undrained (lb/ft^2)	Total Cohesion (lb/ft^2)	Total Friction Angle (degrees)	Effective Cohesion (lb/ft^2)	Effective Friction Angle (degrees)	Liquefied Strength (lb/ft^2)	
Sediment Behind Dam	70	0.22*σ'	Not Used		0	26	N/A	1. Unit Weight - assumed value based on experience with similar material 2. Undrained Shear Strength - assume sediment is normally consolidated 3. Total Shear Strength - not used 4. Effective Shear Strength - assume material is normally consolidated 5. Liquefied Shear Strength - N/A
Embankment Fill	115	Effective Strength Used	900	15	100	34	0.10*σ'	1. Unit Weight - lab test data 2. Undrained Shear Strength - granular material, use effective strength 3. Total Shear Strength - lab test data 4. Effective Shear Strength - lab test and field test data 5. Liquefied Shear Strength - correlation between residual shear strength ratio and equivalent clean-sand SPT-corrected blow counts
Marine Terrace Deposits	120	Effective Strength Used	5,000	27	0	38	0.24*σ'	1. Unit Weight - lab test data 2. Undrained Shear Strength - granular material, use effective strength 3. Total Shear Strength - lab test data 4. Effective Shear Strength - lab test and field test data 5. Liquefied Shear Strength - correlation between residual shear strength ratio and equivalent clean-sand SPT-corrected blow counts
Bedrock	Assumed to be impenetrable (infinite strength) and failure surfaces not allowed to pass through layer							

Notes:

lb/ft^2 = pound(s) per square foot

lb/ft^3 = pound(s) per cubic foot

N/A = not applicable

SPT = standard penetration testing

Table 6
Summary of Stability Analysis Results
Georgia-Pacific Mill Pond Dam
Fort Bragg, California

CROSS SECTION	LOCATION	STATIC SAFETY FACTOR	SEISMIC STABILITY (NO LIQUEFACTION)				POTENTIAL EFFECTS OF LIQUEFACTION	
			Pseudostatic Safety Factor (K_h=0.15)	Yield Acceleration (g)	Estimated Deformation (in)		Failure Surface (Safety Factor)	Integrity of Dam and Potential Release
					Makdisi-Seed (1978)	Bray-Travasarou (2007)		
A-A'	Crib Wall - Southern Section (Note - the crib itself was not incorporated into the analysis)	1.28	1.04	0.18	5	4.8	Front Face (0.45)	Failure surface compromises most of embankment fill and reaches reservoir.
							Reservoir Side (2.04)	The analyses indicate a failure surface back into the reservoir is not likely.
F-F'	Northern Embankment	2.1	1.52	0.41	<1	<1	Front Face (0.56)	Failure surface compromises most of embankment fill and reaches reservoir.
							Reservoir Side (1.06)	The analyses that sliding back into the reservoir is possible but less likely than sliding in a downstream direction.
G-G'	Central Section	1.43	1.2	0.34	<1	<1	Front Face (1.03)	The analysis indicates some slumping and deformation of the face of the dam is likely; failure surface does not come within 100 feet of reservoir.
							Front Face to within 20 Feet of Reservoir (2.97)	This analysis incorporated a search routine to assess failure surfaces within 20 feet of the reservoir under liquefied conditions.
							Reservoir Side (1.57)	The analysis indicates that failure back into the reservoir is not likely under liquefied conditions.

Notes:
1. See Table 5 for material properties used for analysis.
2. See Figure 5 for locations of cross sections.
3. See Appendix F for static stability analysis output and seismic stability calculations and results.
4. Estimated deformations based on M_w = 8.05 earthquake occurring 10 km from site and a site PGA of 0.34g.
5. Bray-Travasarou deformations were calculated at 50 percent probability of exceedance.
6. Yield acceleration is the horizontal coefficient that results in a static safety factor of 1.0.