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PRELIMINARY GEOTECHNICAL EVALUATION
DB5 – PHOENIX LAKE DAM & RESERVOIR
WATERSHED FLOOD DAMAGE REDUCTION & CREEK MANAGEMENT STUDY
MARIN COUNTY, CALIFORNIA

May 6, 2010

Project 960.05

Prepared For:
Marin County Flood Control and
Water Conservation District, Zone 9
c/o Stetson Engineers
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CERTIFICATION

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I INTRODUCTION

This report presents the results of our preliminary geotechnical evaluation for Detention Basin 5 (Phoenix Lake Dam and Reservoir) as part of the Watershed Flood Damage Reduction and Creek Management Study, Marin County, California. The location of the project site is shown on Figure 1, Site Location Map. This report is intended for the exclusive use of Marin County Flood Control and Water Conservation District, Stetson Engineers and their consultants on this project. No other use is authorized without the express written consent of Miller Pacific Engineering Group. The purpose of our current services is to review available data, evaluate geologic and geotechnical conditions, and provide our opinion regarding the feasibility of using of Phoenix Lake as a flood control reservoir.

In accordance with our agreement dated January 5, 2010, the scope of our geotechnical services includes the following:

- Review of geologic and geotechnical data available from the design team and local government sources (County of Marin, Marin Municipal Water District, local city files and Division of Safety of Dams (DSOD)), as well as, review of published USGS and state geologic data, and relative Miller Pacific Engineering Group reference data,

- Site reconnaissance to observe the site conditions, project features, constraints and site access. Examination of the slopes and general reservoir area for existing landslides, rock outcrops, structure and stratigraphy,

- Air photo examination for evaluation of geologic surface features suggestive of instability, faulting or shear zones,

- Review topographic mapping provided by the design team,

- Attendance at project meetings to consult with project team regarding project status, detention basin storage capacity, drawdown rates and reservoir levels,

- Opinion of rim slope stability associated with use of the reservoir as a detention basin during flood events,

- Consult with DSOD regarding design requirements and determine probabilistic ground shaking accelerations at the project site for use in pseudo-static slope stability,

- Develop a model of the dam from information available in the project files and from the previous geotechnical exploration and laboratory testing performed at the site. This may include some estimated soil properties based on the soil type and construction practices,

- Perform preliminary static and pseudo-static stability analyses using a cross-section near the center of the dam. We will evaluate dam stability for various reservoir levels, sudden drawdown conditions and potential seismic deformations using procedures published by Bray and Travasarou, and

- Prepare technical memorandum describing the geologic and geotechnical evaluation, site seismicity, dam stability and geotechnical feasibility of using Phoenix Lake as a flood control reservoir.

Our current scope of services did not include any subsurface exploration or laboratory testing. Theses services may be performed as part of a more detailed investigation and design of the flood control improvements at Phoenix Lake Dam.

II. PROJECT DESCRIPTION

Phoenix Lake is a recreational reservoir in southern Marin County owner by Marin Municipal Water District. We understand the Marin County Flood Control and Water Conservation District would like to utilize this reservoir for short term storage of storm water to aid in flood management of Corte Madera and San Anselmo Creek. Phoenix Lake Dam is an earth fill dam constructed in 1907 utilizing the construction techniques of that time. The existing Phoenix Dam is approximately 94-feet in height (crest elevation 189-feet), 350-feet in length and has a crest width of approximately 22-feet with slopes varying between approximately 1.5:1 (horizontal:vertical) to 3:1. From the early 1900's until the mid 1980's the reservoir water level was maintained at approximately elevation +180 feet[1]. The dam was modified in the late 1960's to improve performance during future seismic events.

As part of the spillway retrofit work performed in the 1980's, the spillway elevation was lowered to elevation + 174-feet. The purpose of this study is to evaluate the potential to temporarily increase the water level for storm water storage. In consultation with the project team, we have evaluated the potential for temporary increases in the reservoir level to elevations of +180 and +184 to allow greater water storage during significant rain events to reduce the potential for flooding of downstream properties.

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[1] All elevations given in this report are relative to the NGVD29 datum.

III. REFERENCE DATA

We reviewed various geotechnical reports and data on file at the Division of Safety of Dams (DSOD) that have been performed regarding Phoenix Lake Dam. The relevant reports reviewed are listed below:

- Dames & Moore "Stability Evaluation Phoenix Lake Dam," October 1959,
- Leeds, Hill & Jewett, Inc., "Methods of Strengthening Phoenix Lake Dam," March 1966,
- Marin Municipal Water District, "Specifications for the Rehabilitation of Phoenix Lake Dam," Undated (1968?),
- Earth Sciences Associates, "1977-78 Evaluation of Seismic Stability Phoenix Lake Dam," March 1978,
- Marin Municipal Water District, "Phoenix Dam Spillway Seismic Analysis," March 1980,
- Division of Safety of Dams, Department of Water Resources, "Phase 1 Inspection Report for Phoenix Lake Dam," May 1981, and
- Earth Sciences Associates, "Phoenix Lake Spillway Reconstruction Geotechnical Report," February 1984.

The Dames & Moore (1959), Leeds, Hill & Jewett, Inc. (1966), and Earth Sciences Associates (1978) reports analyzed various stability conditions of Phoenix Dam including seismic and rapid drawdown conditions. The remaining reference reports included design specifications, seismic analysis of the spillway structure, DSOD inspection report, geotechnical report for the spillway structure reconstruction, and various general correspondences. The three pertinent geotechnical analysis reports are outlined below:

Dames & Moore – The 1959 report was issued prior to the retrofit of Phoenix Dam. Dames & Moore analyzed seismic conditions utilizing a seismic load of "10% gravity", or 0.10 g, and rapid drawdown conditions (completely draining the reservoir) on both the upstream and downstream slopes. Based on the results of the slope stability analyses, Dames & Moore concluded the downstream seismic and static factors of safety were 1.2 and were "adequate". Additionally, Dames & Moore concluded the computed upstream factor of safety of 1.15 during rapid drawdown was "not high" and the drawdown of the dam should be controlled.

Leeds, Hill, & Jewett – The 1966 report prepared by Leeds, Hill, & Jewett, Inc. (LHJ) provided Marin Municipal Water District (MMWD) with three conceptual options to retrofit the existing dam to strengthen the dam. The conceptual plans included:

Plan I – Repair Plan I was originally developed by MMWD that included flattening the slopes by adding an impervious “blanket” on the upstream bank, buttressing the downstream side with a semi-impervious material on the downstream bank, constructing a new outlet tunnel, and extending the existing spillway.

Plan II – Repair Plan II included flattening the upstream bank by excavation, flattening the downstream bank by filling, construction of a hydraulically operated gate controlled inlet, and the construction of a new spillway.

Plan III – Repair Plan III included flattening the upstream bank by adding an impervious “blanket”, constructing a drain on the downstream toe of the dam, and constructing an earth buttress on the downstream toe to confine the drain and provide additional support.

Leeds, Hill, & Jewett performed seismic slope stability analyzes utilizing a peak ground acceleration (PGA) of 0.15 g on two of the conceptual plans (Plans II and III), LHJ did not analyze Plan I due to the relatively high costs of implementing Plan I. The results of the stability analyses performed on Plan II indicate the upstream and downstream factors of safety were approximately 1.70 and 1.20, respectively. LHJ concluded the factors of safety for Plan II indicated “ample stability”. The results of the stability analyses for Plan III indicated the upstream factor of safety was approximately 1.25 and downstream factor of safety was lower. However, LHJ concluded that the downstream results were “not believed valid” due to the implementation of a drain at the toe of the dam and the placement of impervious blanket on the upstream slope. They concluded the phreatic surface would be “significantly lowered” with Plan III and therefore would “be much more effective in stabilizing the existing stratified, somewhat pervious dam than simply by the addition of a pervious stabilizing fill downstream only, as in Plan II”. Therefore, LHJ ultimately recommended that MMWD construct Plan III and based on the current configuration of the Phoenix Dam it appears that Plan III was designed and constructed.

Earth Sciences Associates – The report prepared by Earth Sciences Associates (ESA) was performed after Phoenix Dam was retrofitted as described in the report issued by LHJ (Plan III). ESA performed a SHAKE analysis utilizing a design earthquake event of the San Andreas Fault rupturing with magnitude 8.4. The results of ESA’s SHAKE analyses provided the predicted seismic acceleration throughout the height to the dam (0.56 g at the base to 1.0+ g at the crest). These accelerations were utilized to conservatively estimate the potential upslope deformation during the design seismic event and concluded a potential crest settlement of 5.3-feet and down slope movement of 11-feet.

IV. SITE CONDITIONS

A. Regional and Local Geology

The site is located within the Coast Range Geomorphic Province of California. The regional bedrock geology mostly consists of complexly folded, faulted, sheared, and altered sedimentary, igneous, and metamorphic rock of the Jurassic-Cretaceous age (65-190 million years ago) Franciscan Complex. The Franciscan is characterized by a diverse assemblage of greenstone, sandstone, shale, chert, and mélange, with lesser amounts of conglomerate, calc-silicate rock, schist and other metamorphic rocks.

The regional topography is characterized by northwest-southeast trending mountain ridges and intervening valleys that were formed by compressive movement between the North American and the Pacific Plates. Continued deformation and erosion during the late Tertiary and Quaternary Age (the last several million years) formed the prominent coastal ridges and the inland depression that is now the San Francisco Bay. The more recent seismic activity within the Coast Range Geomorphic Province is concentrated along the San Andreas Fault zone, a complex group of generally north to northwest trending faults.

Additional geologic mapping was performed by Earth Sciences Associates (1978) and indicate Phoenix Lake is predominately surrounded by greywacke sandstone. Minor inclusions of serpentinite, chert, and greenstone are mapped within the greywacke. The drainage swales surrounding Phoenix Lake contain colluvial and alluvial deposits. The mapping also indicates three landslides are located on the surrounding rim of Phoenix Lake. The largest mapped landslide is located on the northwestern tip of Phoenix Lake. A Site Geology Map is presented on Figure 2.

B. Seismicity

1. Active Faults in the Region – The project property is located within the seismically active California Coast region and will therefore experience the effects of future earthquakes. Such earthquakes could occur on any of several active faults within the region. The California Geological Survey (CGS)–formerly California Division of Mines and Geology (2000)–has mapped various active and inactive faults in the region. Active faults are defined by the CGS as those that show evidence of movement in the past 11,000 years and have reported slip rates of >0.1 mm/year.

   Based on the CGS information (1999) there are no known active faults passing through or in the immediate proximity of the property. The closest known active fault is the San Andreas Fault, which is located about 6.4 miles (10.3 kilometers) to the west. The locations of the active faults relative to the project site are shown on Figure 3.

2. Historical Fault Activity - Numerous earthquakes have occurred in the region within historical times. The results of our computer database search indicate that 70 earthquakes (Richter Magnitude 5.0 or larger) have occurred within 100 kilometers (62 miles) of the site between 1735 and 2010. Significant earthquakes to affect the project site are summarized in Table A.

TABLE A
SIGNIFICANT EARTHQUAKE ACTIVITY
PHOENIX LAKE DAM
ROSS, CALIFORNIA

Epicenter (Latitude, Longitude)	Magnitude	Fault	Year	Distance
37.80, -122.20	6.8	Hayward	1836	37 km
37.60, -122.40	7.0	San Andreas	1838	42 km
37.70, -122.10	6.8	Hayward	1868	50 km
38.20, -122.40	6.2	Rodgers Creek	1898	31 km
		Post Construction		
37.70, -122.50	8.2	San Andreas	1906	29 km
37.67, -122.48	5.3	San Andreas	1957	32 km
38.46, -122.69	5.7	Hayward	1969	56 km
37.85, -121.82	5.8	San Gregorio	1980	67 km
37.91, -121.69	4.5	San Andreas	1999	10 km
37.43, -121.77	5.6	Calaveras	2007	91 km

Reference: USGS (2010)

Probability of Future Earthquakes – The historical records do not directly indicate either the maximum credible earthquake or the probability of such a future event. To evaluate earthquake probability in this region, the USGS has assembled a group of researchers into the "Working Group on California Earthquake Probabilities" to estimate the probabilities of earthquakes on active faults. Potential sources were analyzed considering fault geometry, geologic slip rates, geodetic strain rates, historic activity, and micro-seismicity, to arrive at estimates of probabilities of earthquakes with a Moment Magnitude greater than 6.7 by 2037.

The probability studies focused on seven "fault systems" within the Bay Area. Fault systems are composed of different, interacting fault segments capable of producing earthquakes within the individual segment or in combination with other segments of the same fault system. The probabilities for a magnitude 6.7 or greater earthquake before 2032 on fault segments within the

San Francisco Bay Area are presented on Figure 3.

In addition to the seven fault systems, the studies included probabilities of "background earthquakes." These earthquakes are not associated with the identified fault systems and may occur on lesser faults (i.e., West Napa) or previously unknown faults (i.e., the 1989 Loma Prieta and 2000 Napa/Mt. Veeder Earthquake). When the probabilities on all seven fault systems and the background earthquakes are combined mathematically, there is a 62 percent chance for a magnitude 6.7 or larger earthquake to occur in the Bay Area by the year 2032. Smaller earthquakes (between magnitudes 6.0 and 6.7), capable of considerable damage depending on proximity to urban areas, have about an 80 percent chance of occurring in the Bay Area by 2032 (USGS, 2002). Additional studies by the USGS regarding the probability of large earthquakes in the Bay Area are on going. These current evaluations include data from additional active faults and updated geological data.

C. <u>Aerial Photograph Review</u>

We reviewed several aerial photographs obtained from Pacific Aerial Surveys of Oakland, California. The photographs reviewed are summarized below:

- September 06, 1946, AV9-2-1 (1:23,600) – This aerial photograph is the earliest available that shows Phoenix Lake and Phoenix Lake Dam. At the time of the photograph, Phoenix Lake Dam appears to be operating at a relatively high water elevation. The dam is highly vegetated with grasses. A significant number of trees have encroached onto the southwestern abutment and toe of the dam.

- July 02, 1970, AV957-03-25 (1:12,000) – This areal photograph was taken in the summer of 1970 and the vegetation on the dam appears to be dead/dormant and lighter in color. The light color and bright summer sun caused the photo to "wash-out", concealing the finer details of the dam. However, it appears trees have been removed from the southeastern abutment. It also appears the additional buttress and bench has been constructed on the downstream side of the dam.

- April 01, 1980, AV1840-03-29 (1:12,000) – The aerial photograph is relatively unchanged from the previous photograph. The spillway is more detailed in the 1980 photograph and it appears to be a covered structure.

- May 03, 1982, AV2140-03-25 (1:12,000) – The aerial photograph is relatively unchanged from the previous photograph. However, it appears some erosion has occurred at the spillway outlet.

- April 19, 1986, AV2860-10-18 (1:12,000) – The aerial photograph is relatively unchanged from the previous photograph However the water level of Phoenix Lake has reached capacity and the spillway is operating.

- March 15, 1990, AV3766-8-29 (1:12,000) – The aerial photograph is relatively unchanged from the previous photograph. However, it appears the erosion at the spillway outlet has been repaired.
- August 14, 1995, AV4890-16-54 (1:12,000) – The aerial photograph is relatively unchanged from the previous photograph.

- March 06, 2005, KAV9010-19-1 (1:12,000) – The aerial photograph is relatively unchanged from the previous photograph.

D. Site Reconnaissance and Surface Conditions

We performed a site inspection on February 17, 2010 to observe existing conditions and identify any significant visual threats that could preclude use of Phoenix Lake as a flood-control detention basin. Our geologic and geotechnical site reconnaissance is summarized below with our observations noted on the attached Figure 2, Geologic Map.

The reservoir is surrounded by rugged terrain, characterized by steep slopes and deeply incised drainage channels. Bedrock typically is composed of Franciscan sandstone and shale, often interbedded in discontinuous layers, are visible in outcrops along most of the shoreline and adjacent trails. Bedrock typical of this portion of Mount Tamalpais is especially well-exposed just east of the dam along the shoreline, and in a large cut slope along the rim trail approximately ¼ miles west of the dam. Locally, bedrock may be thin- to thick-bedded and relatively fresh. In general, bedrock is massive and highly altered through physical and chemical weathering processes.

The slopes surrounding the lake are generally steep, with inclinations ranging from 0.5:1 (horizontal:vertical) to 3:1 or shallower. In general, slopes consist of a few feet of colluvial and residual soil over relatively competent bedrock. On slopes where colluvial deposits are present, some soil creep is suspected due to the overall “terraced” appearance. On slopes where vegetation is more prevalent and soil deposits are thicker, small landslides and debris flows are common. Drainage channels are typically filled with debris, including soil, rock, and vegetation. Cut slopes along adjacent hiking trails commonly exhibit evidence of instability, including sloughing, raveling, and debris flows.

Two larger landslides were noted during our reconnaissance. Both are on the north shore of the lake, and have been mapped previously by Rice (1976). The main rim trail has been graded across both slides. One slide toes into the lake near its western end, while the other toes into a tributary which discharges at the north end of the dam. Both landslides deposited soil and rock debris into the reservoir which has likely reduced the storage capacity.

Phoenix Lake Dam appears to be in good condition. We observed erosion channels at various locations along the downstream edges the dam, incised to depths of less than 1 foot. Some surface rills were present on both upstream and downstream faces of the dam to a maximum depth of approximately 3-inches, though they were uncommon. We did not observe any signs of seepage through the dam or visible damage to the spillway walls, floor, or piers.

E. Interpreted Subsurface Conditions and Laboratory Testing

Our scope of services did not include performing a subsurface exploration. However, subsurface explorations were performed by Dames & Moore (1959) and Earth Sciences Associates (1978). The approximate boring locations of the previous subsurface explorations are shown on Figure 4. The subsurface explorations performed by the aforementioned firms observed silty sands (SM) and silty clays (CL) within the upper 20-feet of the embankment. The lower portions of the embankment consisted of gravely sandy clay (CL) and clayey sandy gravels (GC). The observed bedrock below the earth dam is graywacke sandstone with minor inclusions of shale and metagraywacke. The boring logs performed by Dames & Moore and Earth Sciences Associates are presented in Appendix A.

Both Dames & Moore and Earth Sciences Associates performed laboratory testing on select soil samples to determine the pertinent engineering soil properties. The tests performed included moisture content, dry density, unconfined compression, and consolidated undrainded triaxial tests with pore pressure measurements (TXCU-pp). The test results were utilized to develop a strength profile for Phoenix Dam. The summarized results of the laboratory tests and outlined the strength data developed by Dames & Moore and Earth Sciences Associates are presented on Figure 5.

We plotted the existing laboratory shear strength data versus depth to identify trends in strength values versus depth of the dam. Considering the variability of the laboratory data, we developed a shear strength versus depth profile for use in our analyses, as shown on Figure 6. For comparison, we also plotted the shear strength profiles developed and utilized by Dames & Moore and Earth Sciences Associates.

V. GEOLOGIC HAZARDS EVALUATION

A. General

This section identifies potential geologic hazards at the project site, their significant adverse impacts, and recommended mitigation measures. The significant geologic hazards at the project site are strong seismic ground shaking, potential slope instability, and erosion. We judge that other geologic/seismic hazards are of lesser concern.

B. Fault Surface Rupture

Pursuant to the Alquist-Priolo Special Studies Zone Act of 1972, the California Geological Survey (CGS) (formerly California Division of Mines and Geology (CDMG)) produced 1:24,000 scale maps showing all known active faults and delineating boundaries to either side of these faults called "Special Studies Zones." Within these zones, the Act requires that a fault investigation be undertaken. The intent of the Act and required investigation is to assure that structures for human habitation are not located astride an active fault trace. Our review of the Special Studies maps (CGS, 2000) and our aerial photograph interpretation indicate that the closest active fault trace is the San Andreas Fault is located about 10 km west of the site. The site is not within the special studies zone and the potential for surface fault rupture through the property is low.

No mitigation measures anticipated.

C. Seismic Shaking

The site will likely experience seismic ground shaking from future earthquakes in the San Francisco Bay Area. Earthquakes along several active faults in the region, as shown on Figure 3, could cause moderate to strong ground shaking at the site.

Deterministic Seismic Hazard Analysis – Deterministic Seismic Hazard Analysis (DSHA) predicts the intensity of earthquake ground motions by analyzing the characteristics of nearby faults, distance to the faults and rupture zones, earthquake magnitudes, earthquake durations, and site-specific geologic conditions. Empirical relations (Abrahamson and Silva, Boore and Atkinson, Campbell and Borzognia, Chiou and Youngs, and Idriss (2008)) for bedrock were utilized to provide approximate estimates of median peak site accelerations. A summary of the principal active faults affecting the site, their closest distance, moment magnitude of characteristic earthquake and probable peak ground accelerations (PGA), which an earthquake on the fault could generate at the site are shown in Table B.

TABLE B
DETERMINISTIC PEAK GROUND ACCELERATION
PHOENIX LAKE DAM
ROSS, CALIFORNIA

Fault	Moment Magnitude	Distance	Median PGA	84th% PGA
San Andreas	7.8	10 km	0.31 g	0.53 g
San Gregorio	7.2	21 km	0.16 g	0.29 g
Hayward	6.9	19 km	0.16 g	0.28 g
Point Reyes	6.9	22 km	0.13 g	0.25 g

References: Sources: USGS (2009), Abrahamson and Silva (2008), Boore and Atkinson (2008), Campbell and Borzognia (2008), Chiou and Youngs (2008), Idriss (2008)

Probabilistic Seismic Hazard Analysis – Probabilistic Seismic Hazard Analysis (PSHA) analyzes all possible earthquake scenarios while incorporating the probability of each individual event to occur. The probability is determined in the form of the recurrence interval, which is the average time for a specific earthquake acceleration to be exceeded. The design earthquake is not solely dependent on the fault with the closest distance to the site and/or the largest magnitude, but rather the probability of given seismic events occurring on both know and unknown faults.

Probabilistic seismic hazard analyses (PSHA) ground motions are determined in the form of recurrence intervals such as 10% chance of exceedance in 100 years. Each recurrence interval converts to a return period, for instance the return period for a probabilistic ground motion with a 10% chance of exceedance in 100 years is 949 years. Common PSHA recurrence intervals are 2% chance exceedance in 50 years (2,475 year return period) and 10% chance of exceedance in 50 years (475 year return period). Predicted accelerations for the common recurrence intervals are given below on Table C.

TABLE C
PROBABILISTIC PEAK GROUND ACCELERATION
PHOENIX LAKE DAM
ROSS, CALIFORNIA

Recurrence Interval	Return Period	PGA, g
10% in 50 years	475 years	0.42
2% in 50 years	2,475 years	0.72

References: National Seismic Hazard Map Program (USGS, 2008)

The potential for strong seismic shaking at the project site is high. Due to their close proximity and historical seismic activity, the San Andreas and Hayward Faults present the highest potential for severe ground shaking. The most significant adverse impact associated with strong seismic shaking is embankment or slope instability, seismic displacements, and potential damage to structures and improvements.

Evaluation: *Less than significant with mitigation.*

Mitigation: *Embankment slopes shall be stable under static conditions and provide acceptable levels of deformation during the anticipated levels of strong ground shaking. Preliminary slope stability analyses indicate the performance of the dam during strong seismic shaking may be better than previously estimated. Mitigation measures include checking the dam stability and calculated displacements using various water level and seismic ground motions to confirm appropriate levels of safety are maintained. Any dam modification or ancillary structures for the project should be designed and constructed in accordance with the seismic provisions of the most recent version of the California Building Code (CBC). Consultation, review and approval of the any dam modifications need to be performed by the California Division of Safety of Dams.*

D. Liquefaction Potential

Liquefaction refers to the sudden, temporary loss of soil shear strength during strong ground shaking. Liquefaction-related phenomena include liquefaction-induced settlement, flow failure, and lateral spreading. These phenomena can occur where there are saturated, loose, granular (non-clayey) deposits. These conditions have not been identified at the project site. Therefore, the potential for liquefaction to occur appears low.

No mitigation measures anticipated.

E. Seismic Induced Ground Settlement

Ground shaking can induce settlement of loose granular soils above the water table. Based on previous explorations, the dam is primarily composed of clayey gravel and gravelly clay. Loose granular deposits were not observed. Therefore, the potential for seismic induced ground settlement is low.

No mitigation measures anticipated.

F. Lurching, Lateral Spreading, and Ground Cracking

Lurching and associated ground cracking can occur during strong ground shaking. Ground cracking generally occurs along the tops of slopes where stiff soils are underlain by soft deposits or along essentially flat terrain that is fronted by a free face, such as a channel bank. These conditions are not present at the project site. Lurching and ground cracking can damage structures or utilities located close to the top of slopes.

No mitigation measures anticipated.

G. Slope Stability

Active and dormant landslides exist around the reservoir as shown on Figure 2. These landslides range in size from small "pop-outs" to large dormant features. There is the potential for re-activation of existing landslides due to seismic shaking or significant saturation. We did not observe any landslide features that could significantly impact the dam. However, surficial sloughing was reported when the reservoir was drained to perform the 1960's improvements. Based on our preliminary slope stability analyses, Phoenix Lake Dam is most susceptible to slope instability and deformation during large seismic events. The results of our analyses indicate deformation during a strong seismic event would be less than calculated deformation from the previous reports. A more detailed discussion of slope instability of Phoenix Lake Dam is presented later in this report. Given the steep slopes and erosive nature of colluvial soil deposits, the potential for landsliding and slope instability around the reservoir is moderate to high.

Evaluation:	*Potentially Significant.*
Mitigation:	*Phoenix Lake Dam has been in place for nearly a century and has reportedly not experienced any significant instability or displacements over its lifetime. Mitigation measures performed in the 1960's and 1980's included lowering the spillway to account for displacements and crest settlement. New analyses indicate less displacement. Planned modification to dam should be analyzed to confirm adequate dam safety and freeboard are maintained after potential seismic deformation.*

H. Erosion

Sandy soils on moderate slopes or clayey soils on steep slopes are susceptible to erosion and gullying when exposed to concentrated surface water flow. Erosion is increased on slopes subjected to concentrated runoff by outfall from drainage facilities and on long slopes without surface drainage control. Currently the inboard slope of Phoenix Dam is covered in a "blanket" of rip-rap to reduce the erosion caused by wave action. The existing downstream slopes are significantly covered with low grasses. Additionally, we did not observe any evidence of excess erosion during our site visit. Therefore, the potential for significant erosion is low.

Evaluation: *Less than significant with mitigation.*

Mitigation: *The vegetation and rip-rap on the slopes of the dam should be maintained. Re-establishing vegetation on disturbed areas will minimize erosion. Erosion control measures during and after construction should conform to the most recent version of the Erosion and Sediment Control Field Manual (San Francisco Branch, Regional Water Quality Control Board, 2002).*

I. Seiche and Tsunami

Seiches and tsunamis are short duration earthquake or landslide generated water waves in large, enclosed bodies of water and the open ocean, respectively. The extent and severity of a seiche would be dependent upon the ground motion and the fault offset from nearby active faults. There is some potential for seiches to occur after an earthquake, especially when water levels are high. Additionally, there are landslides mapped around the reservoir. Based on the topography surrounding this area, it appears that landslides have impacted the reservoir in the past. If a landslide were to remobilize and flow into the reservoir, it could displace a sizable volume of water that could create a seiche. It is not likely that an earthquake or landslide induced seiche will damage the dam provided adequate freeboard is maintained and the spillway can release the excess water. Given the low risk of damage, mitigation measures do not appear warranted.

Evaluation: *Less than significant with mitigation.*

Mitigation: *Maintain adequate dam freeboard above the lake water level to prevent a seiche from over-topping Phoenix Dam. For preliminary design, we recommend that a minimum 4-foot freeboard should be maintained.*

J. Flooding

The adverse impact from flooding is water overtopping the earthen dam creating excess erosion of the dam. Phoenix Lake is surrounded by a watershed that will divert surface water runoff into the reservoir raising the water surface elevation. However, the existing dam is equipped with a spillway that can release excess water as it approaches its maximum elevation. Additionally, reservoir water can be released prior to storms to provide additional storage capacity during heavy rain events. Therefore, provided flood management procedure are developed, the spillway is operating and Phoenix Dam is monitored during heavy rain events, flooding is not considered a significant geologic hazard. A detailed flood management study including the Phoenix Lake is being prepared by Stetson Engineers.

No mitigation measures anticipated.

K. Settlement

New surface loads can cause consolidation of soft clays or compression of loose soils. These conditions do not appear to exist at the dam site. Since the dam has been in place for nearly 100 years and construction of new heavy structures or fills is not expected as part of this project, settlement does not appear to be an issue.

No mitigation measures anticipated.

L. Expansive Soil

Expansive soil conditions occur when clay particles interact with water, causing volume changes in the clay with a resultant reduction in strength. The clayey soils swell when saturated and shrink when dry. Such physical changes may damage lightly loaded foundations, flatwork, and pavement. Expansive soil problems generally decrease in magnitude with increased confinement pressure at depth. Highly expansive soils most likely do not exist at the site, and do not play a role in the pertinent issues of this report.

No mitigation measures anticipated.

VI. DISCUSSION AND EVALUATION

A. General

Based on our research, geologic reconnaissance and initial investigation, we conclude that the proposed use of Phoenix Lake Dam and Reservoir as a flood control detention basin is feasible. Based on our analyses, increasing the water level within the dam during short term storm events has a minor impact on the overall stability. The primary geologic and geotechnical issues include verification of the preliminary slope stability analyses and deformation estimates based on additional exploration and lab data.

B. DSOD Jurisdictional Determination

We have consulted with the California Department of Water Resources, Division of Safety of Dams (DSOD) regarding the potential to utilize Phoenix Lake as a storm water storage detention basin for flood management. Based on our conversation, DSOD would allow Phoenix Lake to be utilized as a storm water detention basin. Phoenix Lake is currently certified for operation at reservoir level +174. Without submitting supplemental analyses, the reservoir could be utilized for flood management by drawing down the reservoir prior to storm events. Temporary impoundment of storm water at higher reservoir levels is feasible provided that supplemental analyses are performed and documentation provided showing adequate stability and freeboard is maintained. For pseudo-static (seismic) analyses, recommended ground motions are the higher of the 84th percentile of the deterministic motions or probabilistic analyses with a reasonable return period. The analyses do not need to consider a worst case earthquake and worst case storm occurring at the same time. DSOD requires a 4-foot minimum freeboard to be maintained for the dam.

C. Stability Analyses

We performed slope stability analyses for static, pseudo-static, and rapid draw down conditions using Spencer’s Method with the computer program Slide version 6.0, produced by RocScience. We evaluated an idealized cross section that corresponded to the differing geometries of Phoenix Dam. Strength parameters for the materials were determined from a compilation of all available data, as shown on Figure 7.

We performed slope stability analyses on various scenarios including static, pseudo-static (seismic), and rapid drawdown. The static and seismic analyses were performed on the downstream slopes only. The additional weight of Phoenix Lake increases the stability of the upstream slope; therefore the downstream slopes are more critical than the upstream. The rapid drawdown analyses were performed on the upstream slopes because the water level within the dam would be higher in the upstream slopes subsequently causing higher pore pressure when the reservoir is lowered.

Based on deterministic analyses (NGA 2008), the seismic response of the site due to a seismic event on the San Andreas Fault is 0.53g for the 84th percentile. We utilized the probabilistic peak ground accelerations (0.49 and 0.78 g's) for our seismic slope stability analyses. The results of our analyses are summarized below on Table E and are presented on Figures 7 through 9.

Minor sloughs may occur on the downstream side during a rapid drawdown conditions. It is difficult to determine the stability of minor sloughs.

TABLE E
SLOPE STABILITY FACTORS OF SAFETY
PHOENIX LAKE DAM
ROSS, CALIFORNIA

	Static Conditions		Rapid Drawdown	
Water Level	Downstream	Upstream	Half	Full
174 feet	1.37	2.22	1.55	1.40
180 feet	1.36	2.28	1.58	1.38
184 feet	1.36	2.41	1.76	1.38

Pseudo-Static (Seismic) Analyses

	DSHA[1]		PSHA[2]			
	84th Percentile (0.53g)		10% in 50 years (0.49g)		2% in 50 yrs (0.78g)	
Water Level	Downstream	Upstream	Downstream	Upstream	Downstream	Upstream
174 feet	0.63	0.62	0.66	0.66	0.48	0.46
180 feet	0.61	0.62	0.65	0.65	0.47	0.47
184 feet	0.61	0.62	0.65	0.66	0.46	0.49

Notes:
1) Deterministic Seismic Hazard Analyses
2) Probabilistic Seismic Hazard Analyses

As shown in Table E, raising the water level from 174 feet to 184 feet does not significantly influence the calculated factors of safety. Additionally, the global stability factors of safety under static and rapid drawdown conditions are above 1.3. However, some localized surficial instability may occur during rapid drawdown. The factors of safety under seismic conditions are below 1.0 which indicates deformation of the dam may occur during strong seismic shaking. A slope stability output file is presented in Appendix B.

D. Seismic Slope Displacement

Due to factors of safety below 1.0 under seismic conditions, the slopes of Phoenix Lake Dam will likely deform during the strong seismic shaking. The previous 1978 deformation analyses by Earth Science estimated 11 feet of elastic deformation along the slip plane which results in 5.3 feet of vertical settlement of the crest. We analyzed the potential slope displacement based on the procedures outlined by Bray & Travasarou (2007). The results of our analyses indicate that the anticipated range of displacements along the slip plane between 1 and 35 inches, depending on the seismic acceleration used in the analyses. The calculated potential dam displacements are shown on Table F.

TABLE F
PREDICTED DAM DISPLACEMENT
PHOENIX LAKE DAM
ROSS, CALIFORNIA

	Predicted Slope Displacement		
Water Level	DSHA[1] 84th Percentile	PSHA[2] 10% in 50 yrs.	PSHA[3] 2% in 50 yrs.
174 feet	1.2 – 5.7 inches	1.9 – 7.7 inches	7.8 – 29.1 inches
180 feet	1.7 – 7.0 inches	2.4 – 9.3 inches	9.1 – 33.9 inches
184 feet	1.7 – 7.2 inches	2.5 – 9.6 inches	9.3 – 34.7 inches

Notes:
1. DSHA – Deterministic Seismic Hazard Analysis, spectral acceleration = 0.58g
2. PSHA – Probabilistic Seismic Hazard Analysis, spectral acceleration = 0.65g
3. PSHA – Probalistic Seismic Hazard Analysis, spectral acceleration = 1.15g

---

The predicted displacement listed above is considered the total displacement along the slope of the dam. The total displacement can be broken down into horizontal and vertical components based on the existing slope inclination and basic geometric principles. For example an 18-inch displacement on the dam's 2:1 (26.6°) slope would break down to 16-inches of horizontal displacement and 8-inches of vertical displacement.

At all reservoir levels analyzed (+174, +180 and +184), the estimated displacements are significantly less than the previous estimates. Based on the preliminary analyses, overtopping of the dam should not occur during a strong earthquake event when the water level in the reservoir is elevated (up to elev. +184) during temporary storage of storm waters. Supplemental exploration, laboratory testing and more sophisticated deformation analyses should be performed to confirm the preliminary results. Supplemental services should be performed in consultation DSOD.

E. Operating Constraints

From a geotechnical and geologic standpoint there appear to be only a few operating constraints. As mentioned above, the factors of safety during strong seismic conditions are below 1.0 and therefore the dam may experience seismic displacements. Following a seismic event, a thorough inspection of the dam and reservoir should be performed.

Minor sloughing may continue to be an occurrence during a drawdown event. When possible, drawdown should be performed at a slow rate (i.e., 1-foot/day) to reduce the potential for upstream slope failures. Higher rates of drawdown (i.e., 10 to 15-ft/day) will increase the potential for minor sloughing to occur. Placement of additional rip-rap and a seepage collection system on the upstream face of the dam would reduce the potential for shallow sloughing during sudden drawdown.

As with all earth fill dams, inspection of the slopes and surrounding area should be performed on a periodic basis to identify and remediate geotechnical conditions that can lead to larger slope instability problems. Some of these conditions are expected to include signs of wave erosion, surface “rilling”, piping, seepage areas or ground cracking.

VII. SUPPLEMENTAL SERVICES

Provided that the concept of using Phoenix Lake Dam and Reservoir as a flood control measure is approved for use by the owner/operators, our supplemental services should include exploration and laboratory testing to provide additional data on the engineering properties of the soil and rock that comprise the dam. This phase of work will involve consultation with the Division of Safety of Dams (DSOD) to determine project specific analyses to be performed. More refined stability and deformation analyses should be performed utilized the additional data collected. In addition, we recommend consultation with an independent geotechnical peer reviewer during the design level investigation regarding the planned dam improvements.

We should review the plans and specifications for the project when they near completion to review the geotechnical aspects, confirm that the intent of our geotechnical recommendations has been incorporated and provide supplemental recommendations, if needed. During construction, inspection and testing of the geotechnical portions of the project should be performed under the direction of a registered geotechnical engineer.

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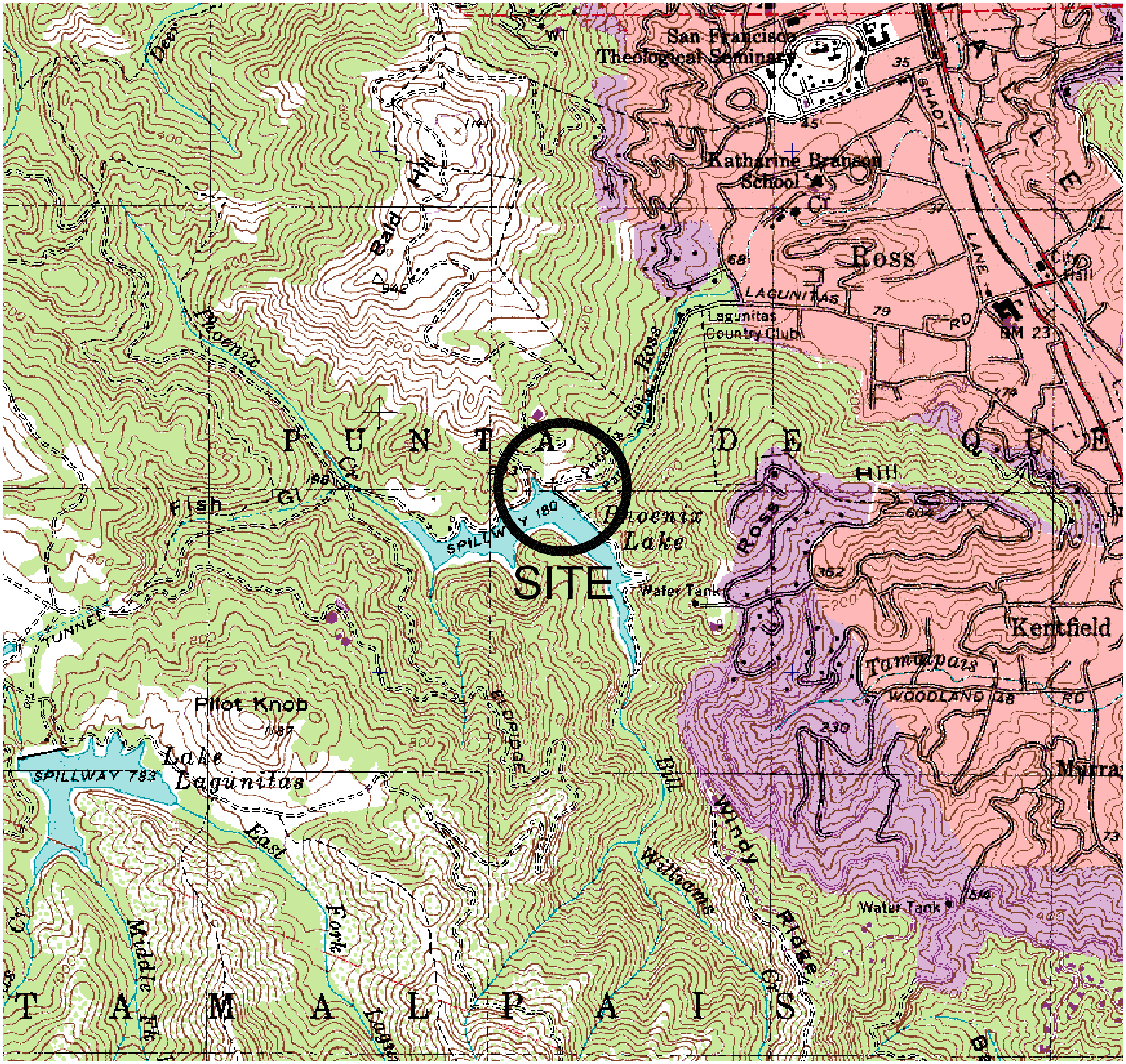

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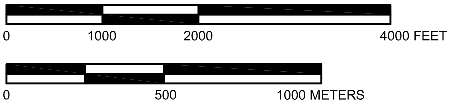

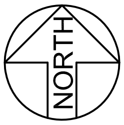

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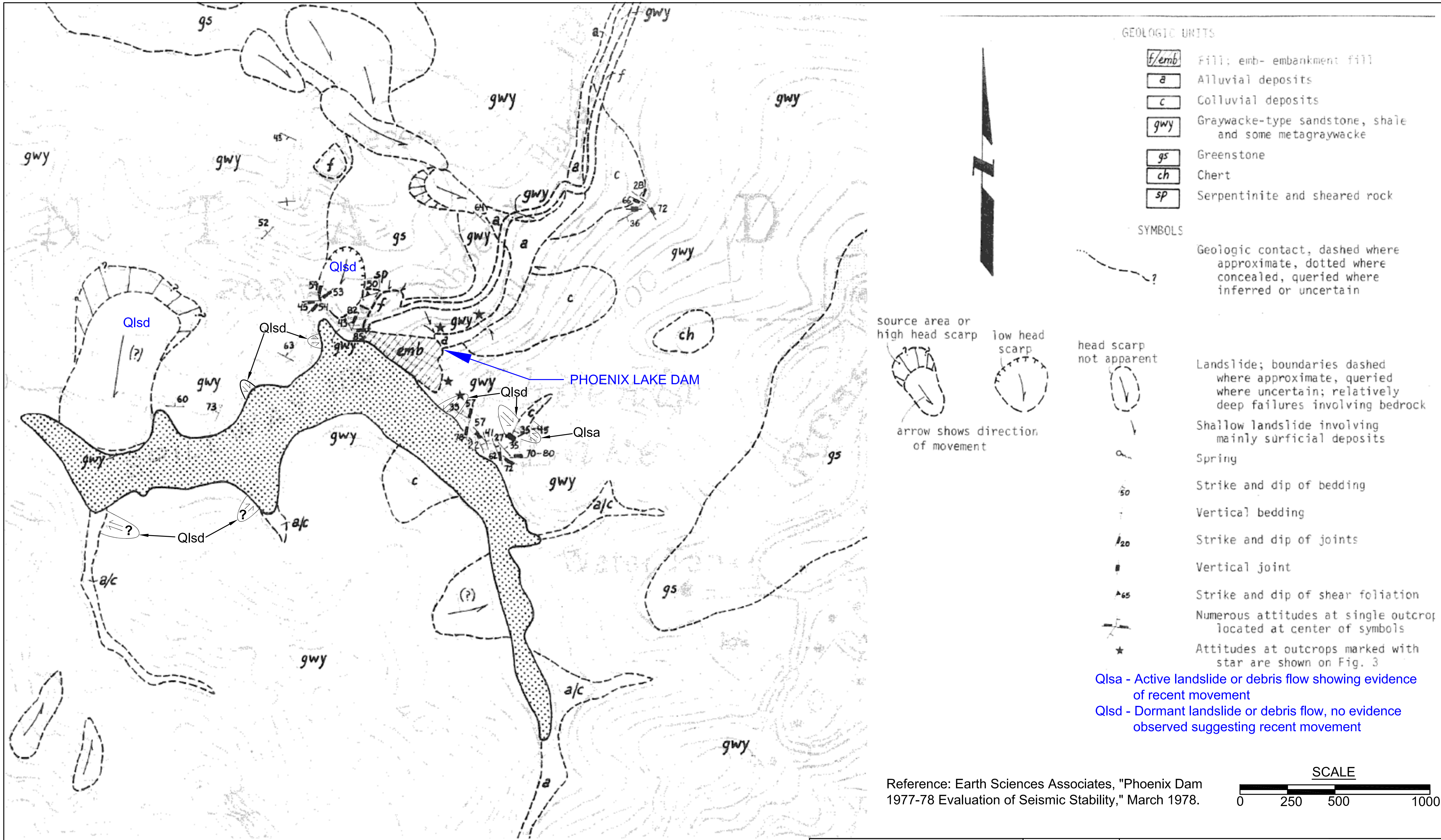
SITE LOCATION MAP

Stetson - MC Flood Protection
Phoenix Lake Dam
Marin County, California
Project No. 960.05 Date: 1/4/10

Drawn JSC
Checked

1
FIGURE



GEOLOGIC UNITS

f/emb	Fill; emb- embankment fill
a	Alluvial deposits
c	Colluvial deposits
gwy	Graywacke-type sandstone, shale and some metagraywacke
gs	Greenstone
ch	Chert
sp	Serpentinite and sheared rock

SYMBOLS

Geologic contact, dashed where approximate, dotted where concealed, queried where inferred or uncertain

Landslide; boundaries dashed where approximate, queried where uncertain; relatively deep failures involving bedrock

Shallow landslide involving mainly surficial deposits

Spring

Strike and dip of bedding

Vertical bedding

Strike and dip of joints

Vertical joint

Strike and dip of shear foliation

Numerous attitudes at single outcrop located at center of symbols

Attitudes at outcrops marked with star are shown on Fig. 3

Qlsa - Active landslide or debris flow showing evidence of recent movement

Qlsd - Dormant landslide or debris flow, no evidence observed suggesting recent movement

Reference: Earth Sciences Associates, "Phoenix Dam 1977-78 Evaluation of Seismic Stability," March 1978.

SCALE

0 250 500 1000 FEET

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GEOLOGIC MAP

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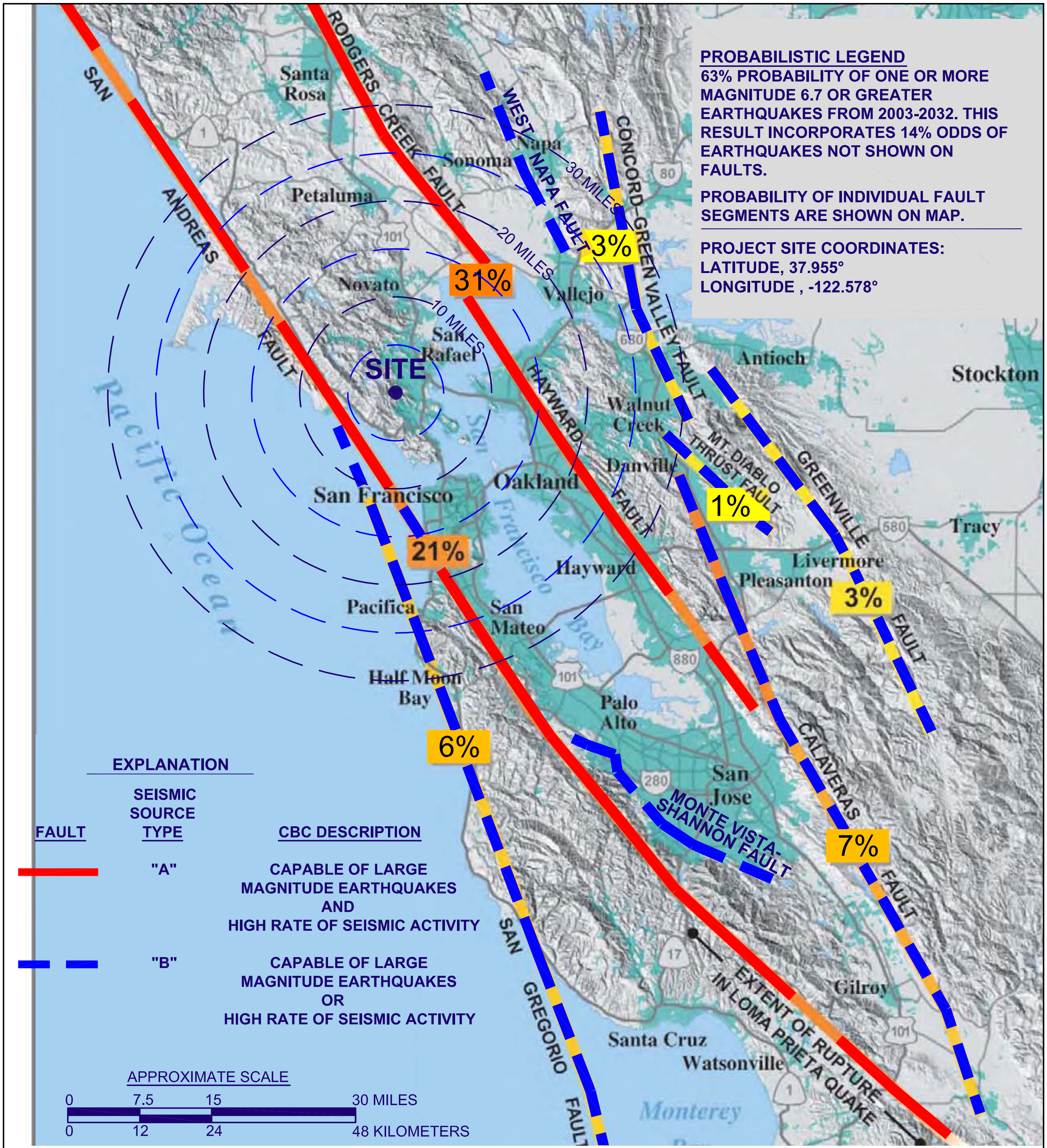
2
FIGURE



PROBABILISTIC LEGEND

63% PROBABILITY OF ONE OR MORE MAGNITUDE 6.7 OR GREATER EARTHQUAKES FROM 2003-2032. THIS RESULT INCORPORATES 14% ODDS OF EARTHQUAKES NOT SHOWN ON FAULTS.

PROBABILITY OF INDIVIDUAL FAULT SEGMENTS ARE SHOWN ON MAP.

PROJECT SITE COORDINATES:
LATITUDE, 37.955°
LONGITUDE , -122.578°

EXPLANATION

FAULT	SEISMIC SOURCE TYPE	CBC DESCRIPTION
▬	"A"	CAPABLE OF LARGE MAGNITUDE EARTHQUAKES AND HIGH RATE OF SEISMIC ACTIVITY
▬ ▬	"B"	CAPABLE OF LARGE MAGNITUDE EARTHQUAKES OR HIGH RATE OF SEISMIC ACTIVITY

APPROXIMATE SCALE

0 7.5 15 30 MILES
0 12 24 48 KILOMETERS

REFERENCES:

1) REVISED 2002 CALIFORNIA PROBABILISTIC SEISMIC HAZARD MAPS, CGS, 2003.

2) ACTIVE FAULT MAP MODIFIED FROM SUMMARY OF EARTHQUAKE PROBABILITIES IN THE S.F. BAY REGION, 2002 - 2031, THE WORKING GROUP ON CALIFORNIA EARTHQUAKE PROBABILITIES, 2010.

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ACTIVE FAULT MAP

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Project No. 960.05 Date: 1/4/10

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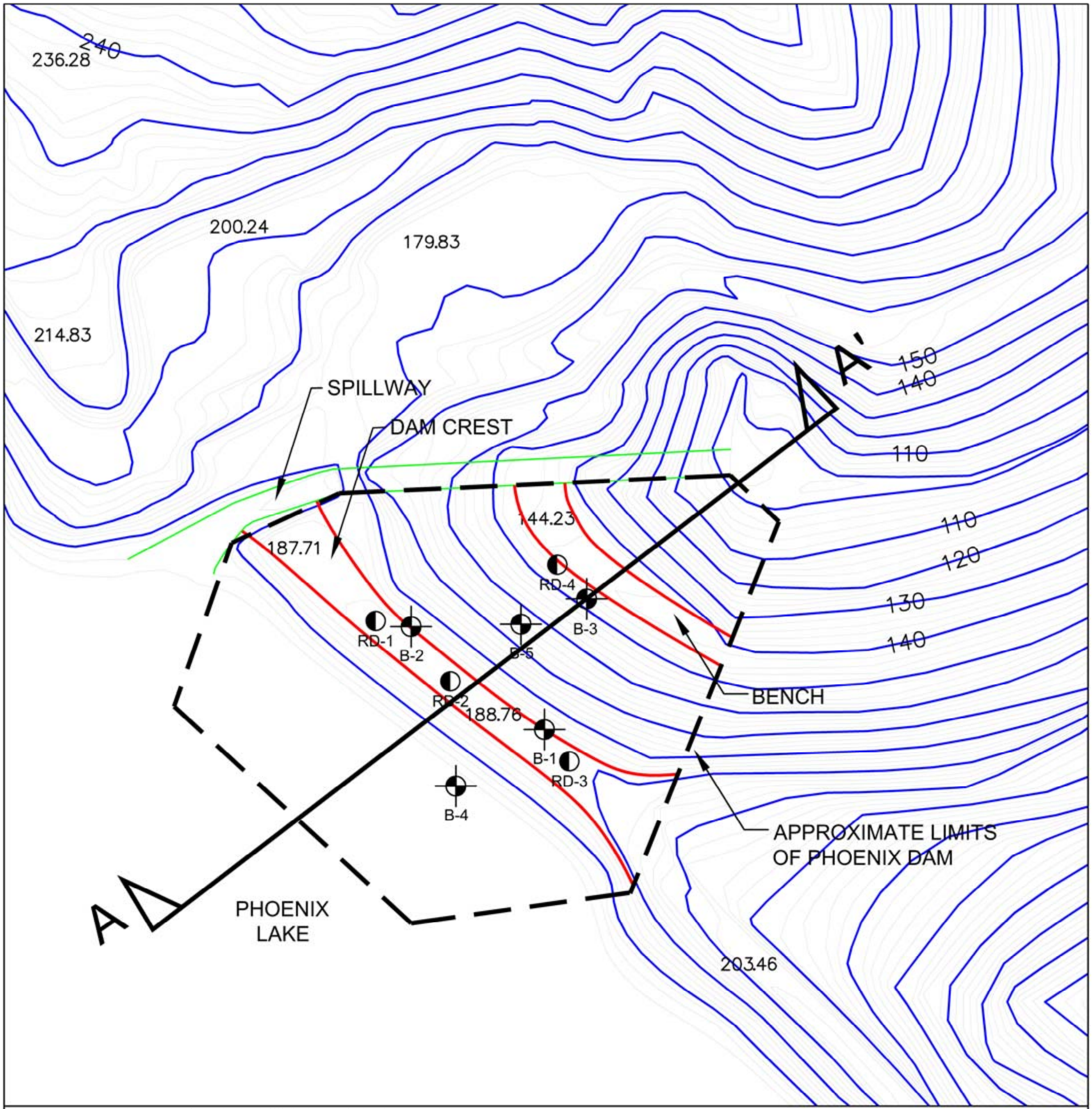
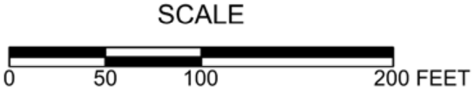
3
FIGURE



Reference: Marin Municipal Water District, 2004

◐ RD-4 Boring by Earth Sciences Associates, 1977

⊕ B-2 Boring by Dames & Moore, 1958



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BORING LOCATIONS

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Project No. 960.05 Date: 2/3/2010

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4
FIGURE

PREVIOUS LABORATORY TEST RESULTS

							TXUU	TXCU-pp				
Firm	Boring	Elev. (ft)	Elev. (ft)	Soil Type	γ_D (pcf)	MC (%)	UC (psf)	φ' (deg)	c' (psf)	P200 (%)	LL	PI
D&M	B-3	146	2	CL	111	17.2	1100					
D&M	B-3	132	16	SC	115		1200					
D&M	B-3	126	22	SC			3400					
D&M	B-3	116	32	SC-GC	114	17.1	500					
D&M	B-4	158	6	SC	130	11.5	1500					
D&M	B-4	152	12	SC	117	15.1	2100					
D&M	B-4	147	17	SC	120	14.6	1200					
D&M	B-4	147	17	SC	117	10.5	1800					
D&M	B-4	133	31	SC	128	14.1	2750					
D&M	B-4	133	31	SC	111	19.6	3300					
D&M	B-5	162	1	SC	121	14.7	990					
D&M	B-5	156	7	SC	115	19.5	2200					
D&M	B-5	145	18	SC	120	14.4	2800					
D&M	B-5	141	22	GP-GC	110	22.3	4100					
D&M	B-5	132	31	CL	112	22.6	1400					
ESA	RD-3	158	27	CL	108.8	17.2						
ESA	RD-3	147	38	GC-CL	115.6	17.0	1700			34.9		
ESA	RD-3	143	42	GC-CL	120.7	13.1						
ESA	RD-3	138	47	GC-CL	114.5	15.9						
ESA	RD-3	130	55	GC-CL	124.2	13.6	3600			26.0	40	21
ESA	RD-3	125	60	GC-CL	117.1	16.3	3900			43.5	38	19
ESA	RD-3	(remolded sample)			111.0	12.8		31	144			
				Average Properties =	**116.8**	**16.0**	**2196.7**	**31**	**144**	**34.8**	**39**	**20**

VALUES UTILIZED IN PREVIOUS ANALYSES

DAMES & MOORE

Levee Strength

		Static		Seismic	
γ_{SAT} (pcf)	γ_{SUB} (pcf)	φ' (deg)	c' (psf)	φ'	c' (psf)
137.4	75.0	25	500	30	650

Rock Strength

γ_T (pcf)	φ' (deg)	c' (psf)
150	84.3	10000

EARTH SCIENCE

UPPER 25 FEET: Estimated Ultimate Shear Strength = 1500 psf

UPPER 25 FEET: Estimated Cyclic Shear Strength = 1000 psf

BELOW 25 FEET: Ultimate Shear Strength $\tau/\sigma'_{vo} = 0.5$

BELOW 25 FEET: Cyclic $\tau/\sigma'_{vo} = 0.33$

Undrained Shear Strength 25-50 feet = 2200 (@25 ft. depth) + 52pcf/foot

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LABORAATORY DATA SUMMARY

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Phoenix Lake Dam
Marin County, California

Project No. 960.05 Date: 2/04/10

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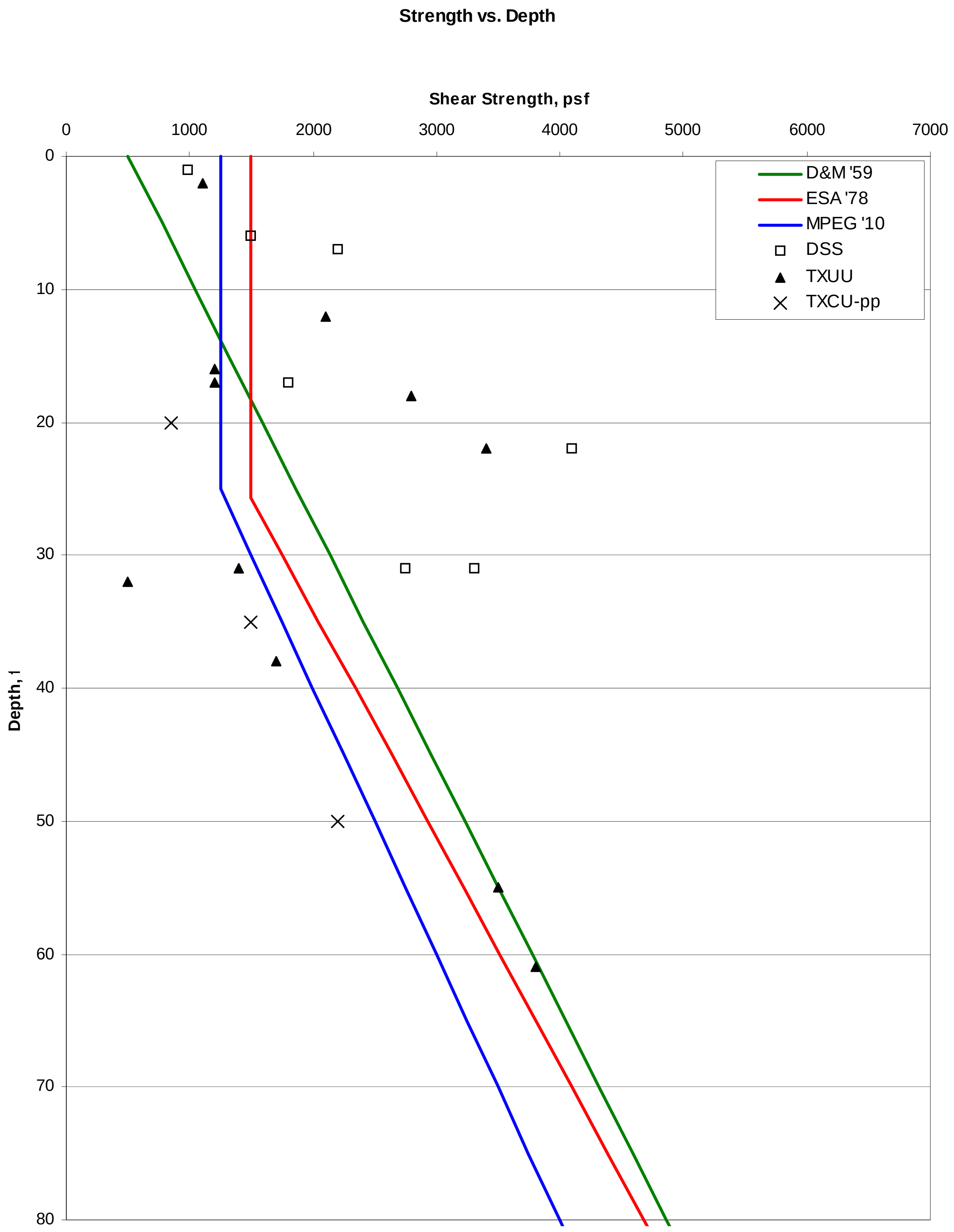
5
FIGURE



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SHEAR STRENGTH vs. DEPTH PROFILE

Stetson - MC Flood Control
Phoenix Lake Dam
Marin County, California
Project No. 960.05 Date: 2/04/10

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6
FIGURE

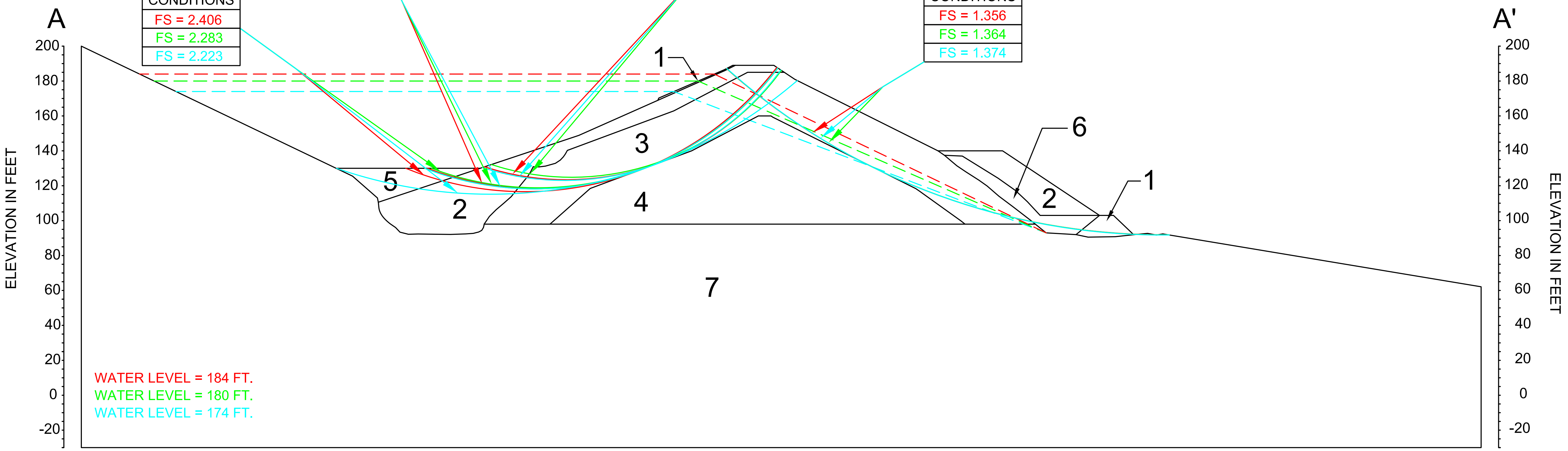
PHOENIX LAKE DAM
STATIC CONDITIONS

RAPID DRAWDOWN TO EMPTY RESERVOIR		
INITIAL WATER LEVEL	FINAL WATER LEVEL	FACTOR OF SAFETY
184 FT.	130 FT.	1.376
180 FT.	130 FT.	1.384
174 FT.	130 FT.	1.398

RAPID DRAWDOWN TO HALF RESERVOIR		
INITIAL WATER LEVEL	FINAL WATER LEVEL	FACTOR OF SAFETY
184 FT.	157 FT.	1.592
180 FT.	155 FT.	1.576
174 FT.	152 FT.	1.551

STATIC CONDITIONS
FS = 2.406
FS = 2.283
FS = 2.223

STATIC CONDITIONS
FS = 1.356
FS = 1.364
FS = 1.374



MATERIAL PROPERTIES

LAYER	MATERIAL TYPE	γ_{TOTAL} (pcf)	γ_{SAT} (pcf)	FRICTION ANGLE	COHESION (psf)	τ/σ
1	Rock Fill	135.0	145.0	40°	200	---
2	Dam Butress	125.0	140.0	0°	1500	---
3	Dam 0 - 25 ft	130.0	137.4	0°	1250	---
4	Dam 25+ ft	130.0	137.4	0°	0	0.427
5	Silt Deposits	100.0	100.0	0°	100	---
6	Drain Blanket	135.0	130.0	35°	0	---
7	Bedrock	140.0	145.0	38°	2000	---

SCALE
0 30 60 120 FEET

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SLOPE STABILITY ANALYSES RESULTS

Stetson - MC Flood Control
Phoenix Lake Dam
Marin County, California
Project No. 960.05
Date: 02/03/2010

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7
FIGURE

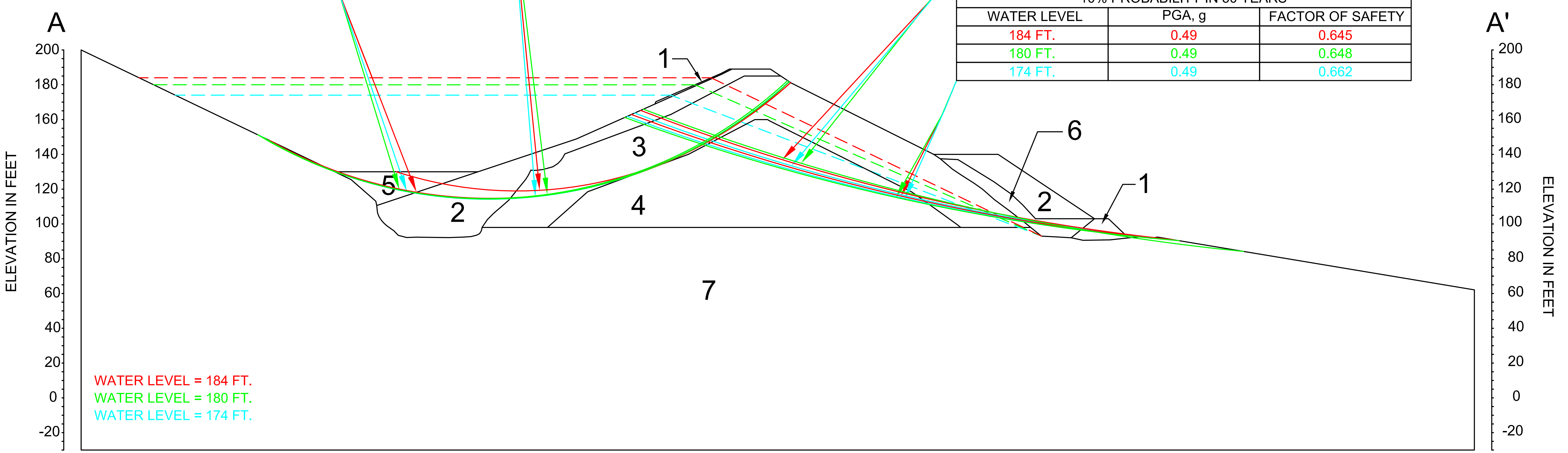
PHOENIX LAKE DAM
SEISMIC CONDITIONS
PROBABILISTIC ANALYSIS

2% PROBABILITY IN 50 YEARS		
WATER LEVEL	PGA, g	FACTOR OF SAFETY
184 FT.	0.78	0.486
180 FT.	0.78	0.467
174 FT.	0.78	0.461

10% PROBABILITY IN 50 YEARS		
WATER LEVEL	PGA, g	FACTOR OF SAFETY
184 FT.	0.49	0.664
180 FT.	0.49	0.652
174 FT.	0.49	0.659

2% PROBABILITY IN 50 YEARS		
WATER LEVEL	PGA, g	FACTOR OF SAFETY
184 FT.	0.78	0.464
180 FT.	0.78	0.468
174 FT.	0.78	0.477

10% PROBABILITY IN 50 YEARS		
WATER LEVEL	PGA, g	FACTOR OF SAFETY
184 FT.	0.49	0.645
180 FT.	0.49	0.648
174 FT.	0.49	0.662



MATERIAL PROPERTIES

LAYER	MATERIAL TYPE	γ_{TOTAL} (pcf)	γ_{SAT} (pcf)	FRICTION ANGLE	COHESION (psf)	τ/σ
1	Rock Fill	135.0	145.0	40°	200	---
2	Dam Butress	125.0	140.0	0°	1500	---
3	Dam 0 - 25 ft	130.0	137.4	0°	1250	---
4	Dam 25+ ft	130.0	137.4	0°	0	0.427
5	Silt Deposits	100.0	100.0	0°	100	---
6	Drain Blanket	135.0	130.0	35°	0	---
7	Bedrock	140.0	145.0	38°	2000	---

SCALE
0 30 60 120 FEET

Miller Pacific
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FILE: 960.05F8.dwg

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SLOPE STABILITY ANALYSES RESULTS

Stetson - MC Flood Control
Phoenix Lake Dam
Marin County, California
Project No. 960.05
Date: 02/03/2010

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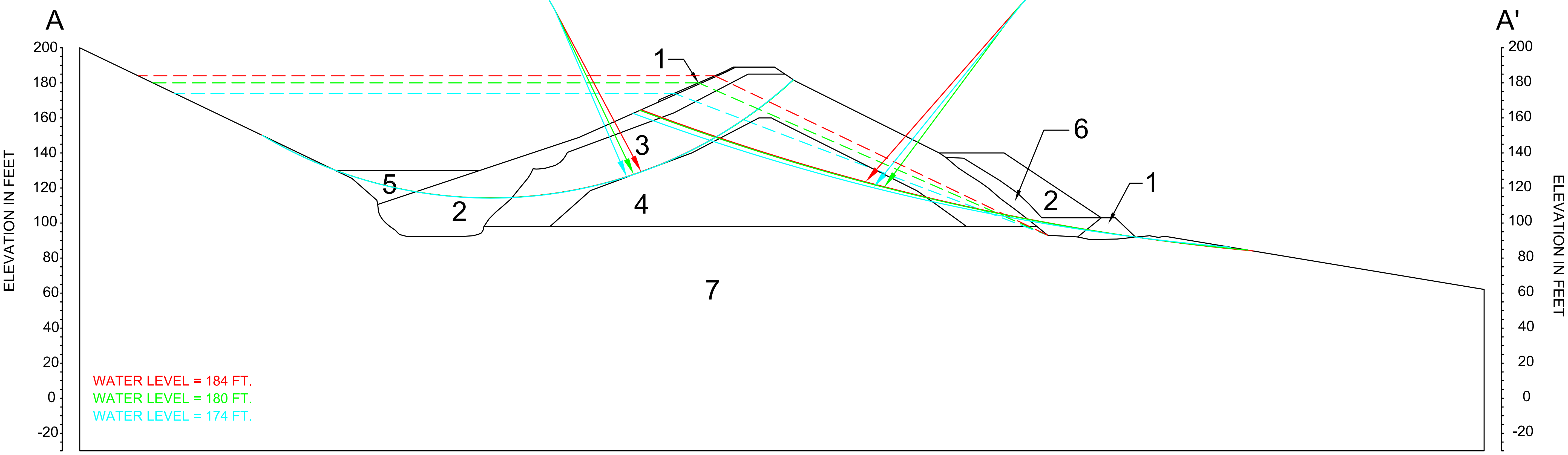
8
FIGURE

PHOENIX LAKE DAM
SEISMIC CONDITIONS
DETERMINISTIC ANALYSIS

DSHA 84th PERCENTILE		
WATER LEVEL	PGA, g	FACTOR OF SAFETY
184 FT.	0.53	0.623
180 FT.	0.53	0.620
174 FT.	0.53	0.621

DSHA 84th PERCENTILE		
WATER LEVEL	PGA, g	FACTOR OF SAFETY
184 FT.	0.53	0.605
180 FT.	0.53	0.611
174 FT.	0.53	0.626



MATERIAL PROPERTIES

LAYER	MATERIAL TYPE	γ_{TOTAL} (pcf)	γ_{SAT} (pcf)	FRICTION ANGLE	COHESION (psf)	τ/σ
1	Rock Fill	135.0	145.0	40°	200	---
2	Dam Butress	125.0	140.0	0°	1500	---
3	Dam 0 - 25 ft	130.0	137.4	0°	1250	---
4	Dam 25+ ft	130.0	137.4	0°	0	0.427
5	Silt Deposits	100.0	100.0	0°	100	---
6	Drain Blanket	135.0	130.0	35°	0	---
7	Bedrock	140.0	145.0	38°	2000	---

SCALE
0 30 60 120 FEET

Miller Pacific
ENGINEERING GROUP

FILE: 960.05F9.dwg

1333 N. McDowell Blvd.
Suite C
Petaluma, CA 94947
T 707 / 765-6140
F 707 / 765-6222
www.millerpac.com

SLOPE STABILITY ANALYSES RESULTS

Stetson - MC Flood Control
Phoenix Lake Dam
Marin County, California
Project No. 960.05 Date: 02/03/2010

Drawn MFJ
Checked

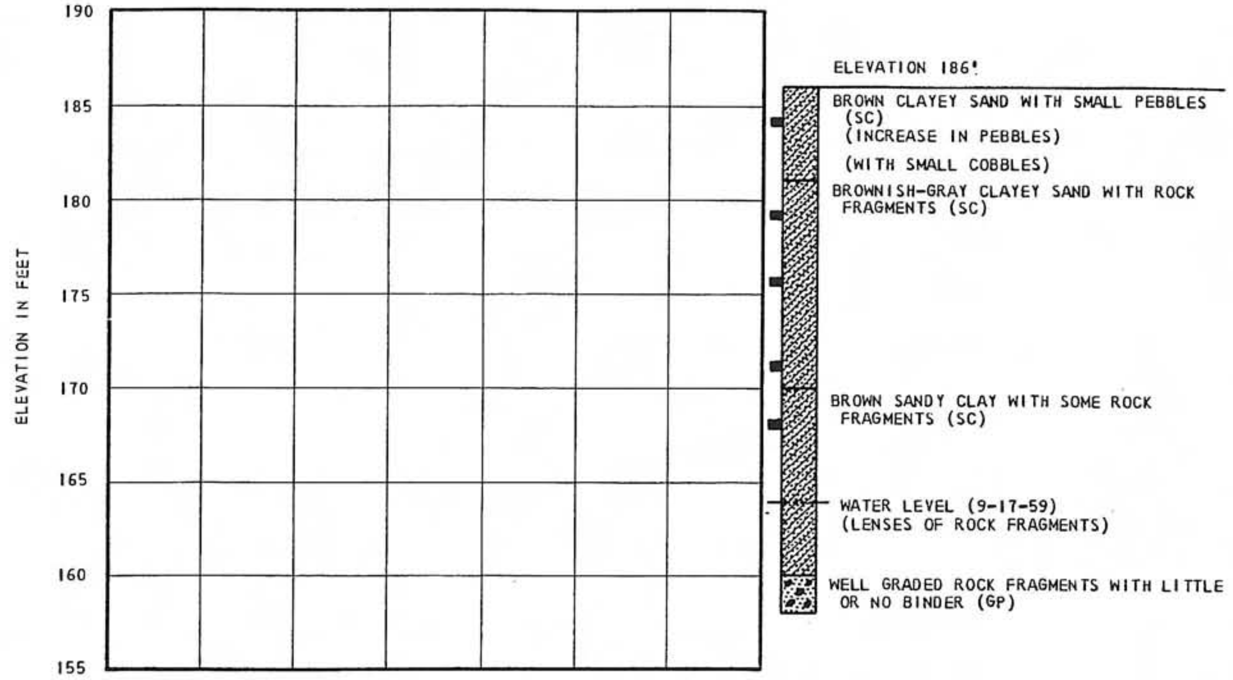
9
FIGURE

APPENDIX A
PREVIOUS BORING LOGS

REVISIONS
BY______DATE______
BY______DATE______
PLATE______OF______

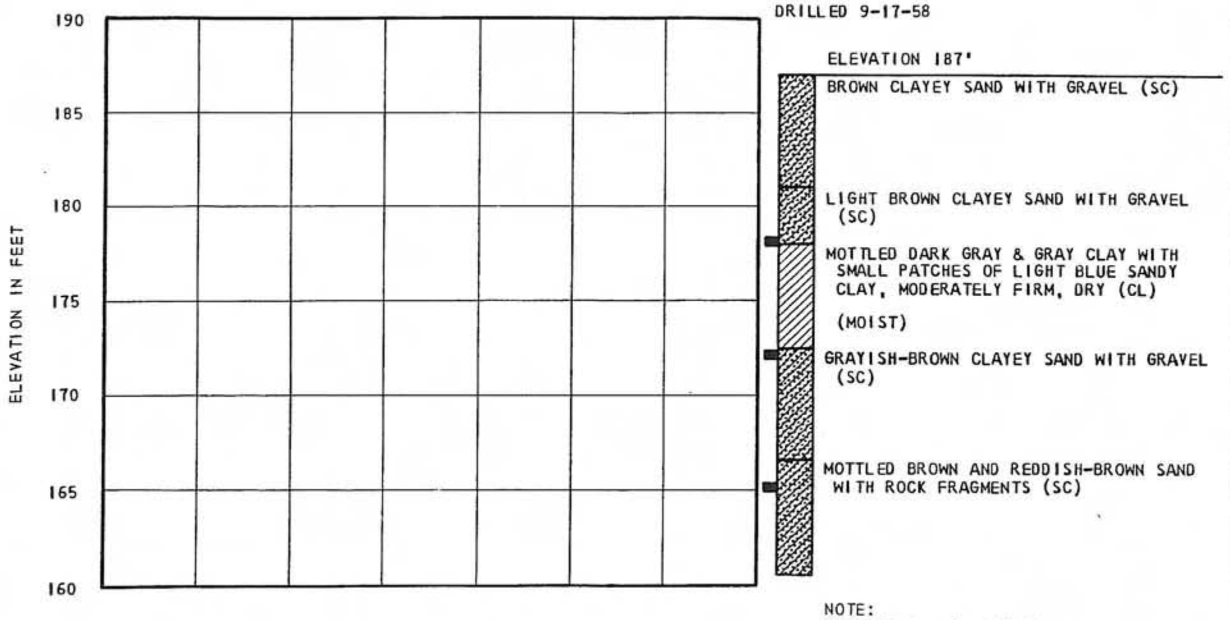
BORING 1

DRILLED 9-17-58



BORING 2

DRILLED 9-17-58



NOTE:
BORINGS 1 & 2, 16" DIAM, DRILLED WITH AUGER TYPE DRILLING EQUIPMENT.
BORINGS 3,4 & 5, 4" DIAM, DRILLED WITH ROTARY-WASH DRILLING EQUIPMENT.
ELEVATIONS REFER TO GROUND SURFACE CONTOURS SHOWN ON PLOT PLAN.

LOG OF BORINGS

FILE 118[illegible] [illegible] Water
BY [illegible] DATE [illegible]
CHECKED BY [illegible] DATE 10/20/[illegible]

DAMES & MOORE
SOIL MECHANICS ENGINEERS

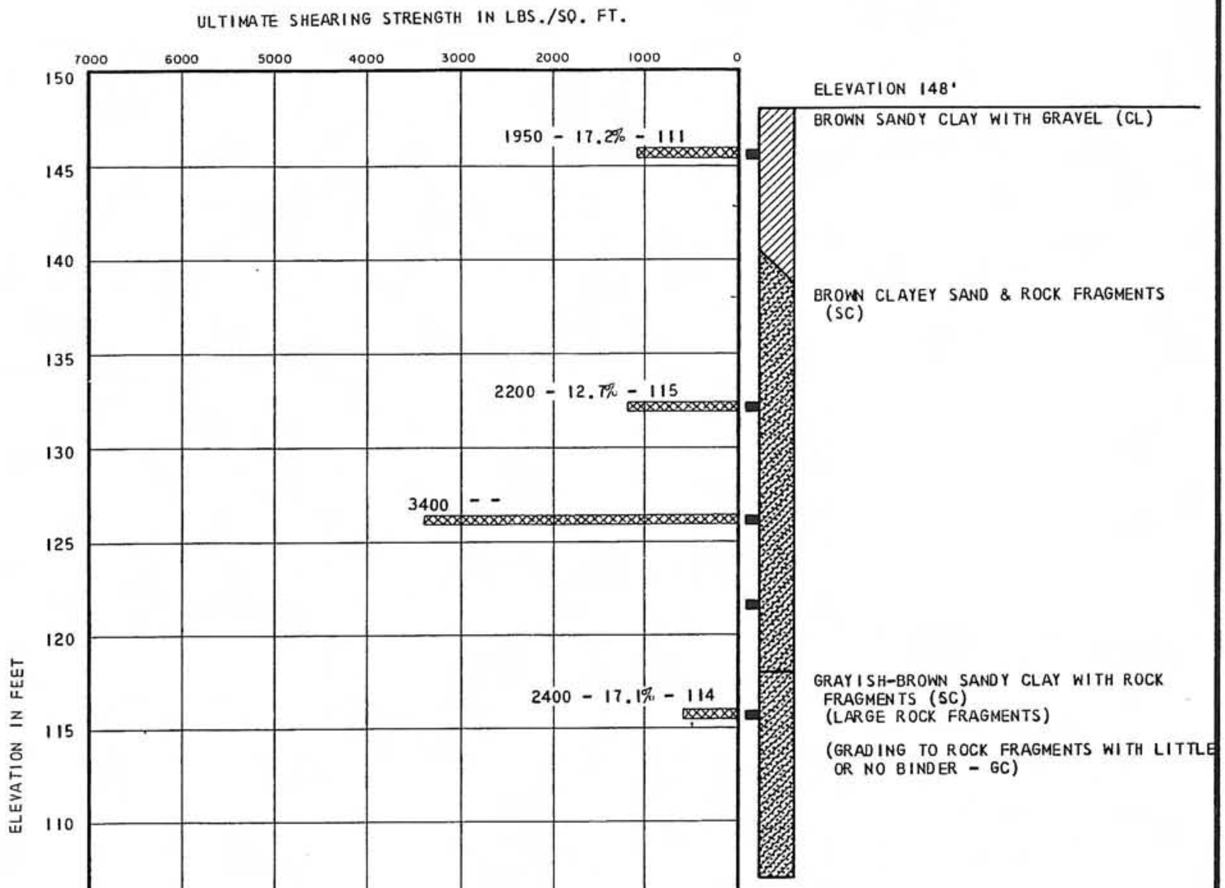
PLATE 3A

REVISIONS
BY ____ DATE ____
BY ____ DATE ____
PLATE ____ OF ____

FILE 1184.E
M M W D
BY HST- KT DATE 10-21-59
CHECKED BY DATE 10/30/59

BORING 3

DRILLED 4-20-59 TO 4-24-59



LOG OF BORING

DAMES & MOORE
SOIL MECHANICS ENGINEERS

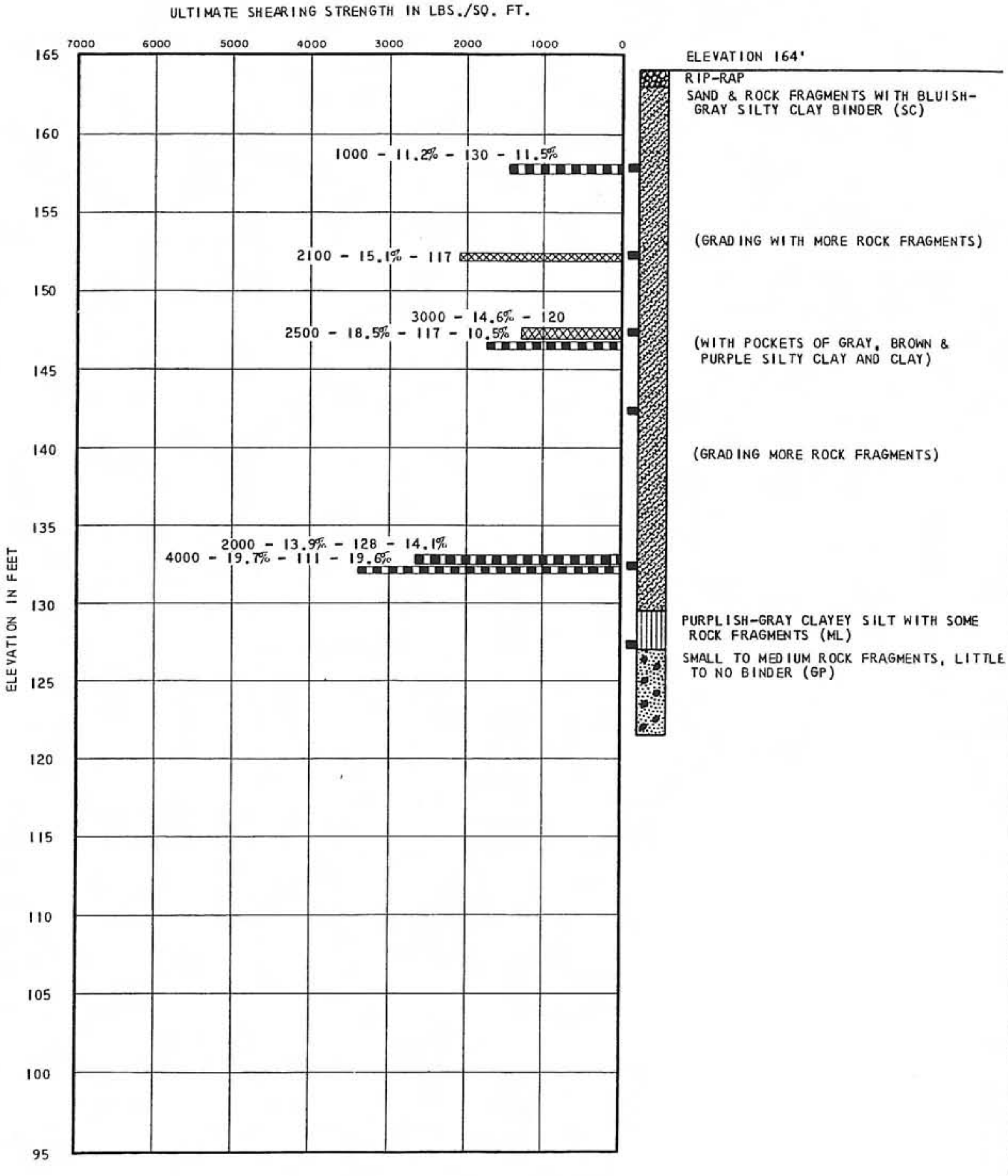
PLATE 3B

REVISIONS
BY____DATE____
BY____DATE____
PLATE____OF____

FILE 1184.E
M.M.N.D.
BY H.C.T. DATE 10-21-59
CHECKED BY C.-Y.L. DATE 10/30/59

BORING 4

DRILLED 4-23-59 TO 4-24-59



LOG OF BORING

DAMES & MOORE
SOIL MECHANICS ENGINEERS

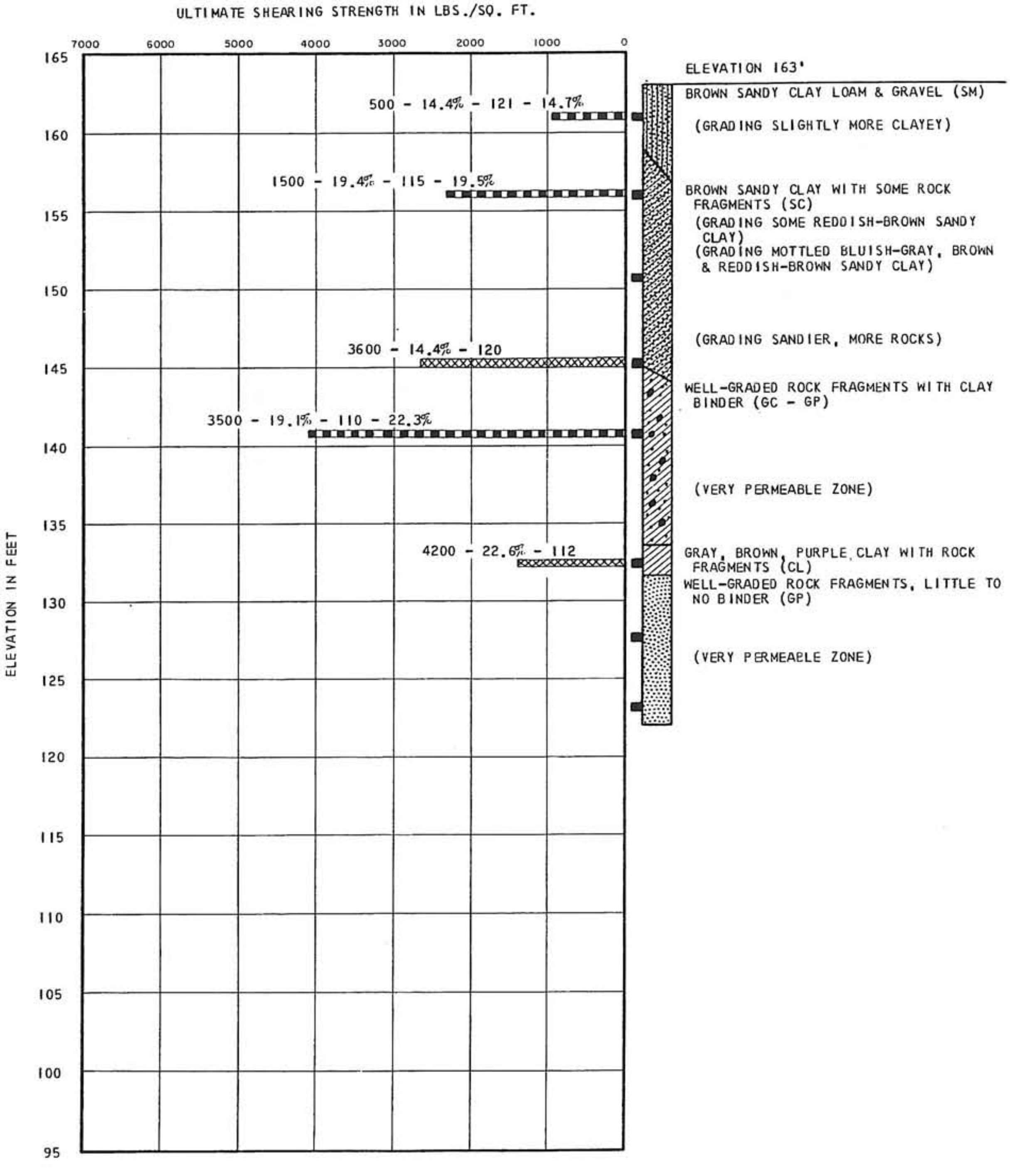
PLATE 3C

REVISIONS
BY______ DATE______
BY______ DATE______
PLATE______ OF______

FILE 184-E
M.M.W.P.
BY HCT - RJ DATE 10-21-59
CHECKED BY ____ DATE 10/20/59

BORING 5

DRILLED 4-24-59 TO 4-28-59



LOG OF BORING

DAMES & MOORE
SOIL MECHANICS ENGINEERS

PLATE 3D

EARTH SCIENCES ASSOCIATES

DRILLING AND SAMPLING LOG

PROJECT PHOENIX DAM 1890 DATE DRILLED 11:22 HOLE NO. RD-1
LOCATION GROUND SURFACE ELEV. 189±
RILLING CONTRACTOR J.N. PITCHER LOGGED BY MTD DEPTH TO GROUND WATER
TYPE OF RIG FAILING 750 HOLE DIAMETER 4 7/8" HAMMER WEIGHT AND FALL 140lb - 30 in.
SURFACE CONDITIONS BASE ROCK WEATHER CLOUDY - COOL

Piezometer Installation

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE Penetrometer TSF Torvane TSF	MODE	REMARKS
0.0	SM	0.0-5.0 <u>GRAVELLY SAND:</u> Blue; 10-20% predominantly silt fines; fine to medium sand; fine angular gravel; dense; moist. Base Rock. 0.5 color grades to light moderate brown. ~2 small lense of soft, wet clayey silt.		AD	
2.0					
4.0				AD DR	Drove Standard split spoon
	SM CL	5.0-22.5 <u>SILT CLAY:</u> Light olive grey and light grey; medium plasticity; contains scattered rounded gravel; scattered roots and wood; firm; moist.		0.5/1.5	4.0 - 5.5 12/0.5 19/0.5 18/0.5
6.0				RD	Hit rock at 5' with auger. 5.0 Set 6' of casing - began rotary drilling.
8.0				PB 0/2.4	Pitcher Barrel 7.0 - 9.4 Rocks block tube.
10.0				RD	
12.0				PB 0/2.5	Pitcher Barrel 11.0-13.5
14.0			J-1 0.39 0.36 0.38 | 0.8 2.7 2.0	DR 1.0/1.5	Drove California Sampler 13.5-15.0 12/1.5
16.0				PB 0/2.5	Pitcher Barrel 15.0-17.5 Took barrel apart to see if ball was in place - it wasn't, no replacement.
18.0				RD PB 0/2.5	2.30 stopped for day. Pitcher Barrel 18.0-20.5
20.0	CL				

SHEET 1 OF 4

PROJECT 1890 DATE DRILLED 11:23 HOLE NO. RD-1

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE Penetrometer TSF	MODE	REMARKS
20.0	CL	5.0-22.5 SILTY CLAY: cont.	Torvane TSF	PB	
				RD	Appear to be pushing rock in front of bit - Tried to push it aside with drag bit.
22.0					
	CL (GC)	22.5-38.0 GRAVELLY SANDY CLAY: Purple with numerous olive green and yellowish brown mottles; low plasticity clay; fine to coarse sand; 20% fine-coarse gravel (serpentine?); firm to stiff ; moist		PS	Pushed Shelby Tube 23.0-25.5
24.0			>0.45 >.45	0.5 2.5	300 psi Tube bent at end.
			J-2		
26.0				RD	
				PB	Pitcher Barrel 26.0-28.0
				1.5 2.5	
28.0			J-3		
		clay binder appears to get softer.		PB	Pitcher Barrel 28.0-30.5
30.0				1.7 2.5	Samples when extruded are generally broken into 4-6" Lengths
			J-4		
		clay binder has strengths from drive sample 0.7 → 1.5 TSF		DR 0.8 1.5	Drove Standard Split Spoon 30.5 - 32.0
32.0			J-5	RD	3/0.5 3/0.5 5/0.5
	Piezoseal			PB	Pitcher Barrel
34.0	CL CL-CH CL	34.4-34.3 blue grey silty clay; contains scattered gravel 34.3-36.0 Moderate brown sandy clay; contains some 10-15% gravel.	J-6 0.6 0.6 1.2 1.0 1.4 J-7 2.5 2.7	1.4 2.5	33.0-35.5
36.0	GC-CL	grades back to same as 22.5-38.0 except gravel content increases.		PB	3" serpentine gravel in bottom of tube Pitcher Barrel 35.5-38.0
			J-8	0.9 2.5	Clay binder washes out unless sample is forced.
38.0	Sand back fill GC	38.0-61.0 CLAYEY GRAVELS Moderate brown with purple discolorations clay binder; 5-20% low plasticity clay; some sand; angular to subrounded gravel up to 2"; dense ; moist to wet. Clay binder is soft to firm.		DR 0.7 1.5	Drove Standard Split Spoon 38.0 - 39.5
40.0				RD	5/0.5 9/0.5 11/0.8 Sample broken up - impossible to test.
Piezometer #1 →			J-9	PB 0.4/0.5	Pitcher Barrel 41.0-41.5 (Refusal)
42.0		(Sample J-10, includes loose "clean" gravel that are probably cuttings)	J-10	DR 0.3 1.5	Drove Standard Split Spoon. 41.5-43.0 9/0.5 11/0.5 15/0.5
44.0	GC	clay content increases		RD	

SHEET 2 OF 4

PROJECT 1890 DATE DRILLED 11:23 HOLE NO. RD-1

DEPTH	CLASS.		FIELD DESCRIPTION	SAMPLE	Penetrometer	MODE	REMARKS
44.0		GC (CL)	33.0-67.0 CLAYEY GRAVEL: cont.		TSF	DR	Drove Standard Split spoon 44.0-45.5
		CL		J-11		0.6/1.5	15/0.5 12/0.5 13/0.5
46.0	sand backfill					RD	
		GC-GP				PB	Pitcher Barrel 46.5-49.0
48.0		GC	clay binder moderate brown and purple.			0/2.5	47-48 Rig chatters strongly. Sample washed out?
						RD	
50.0	Peltoseal					PB 0.6/2.5	Pitcher Barrel 50.0-52.5
52.0				J-12			Rig chatters sporadically. Large rock in bottom
						DR	Drove Standard Split Spoon 52.5-54.0
54.0				J-13	[illegible] 2.0 2.5.	0.5/1.5	5/0.5 15/0.5 13/0.5
						RD	54.5 Rig chatters violently. →55.0
56.0							
58.0							
60.0	Sand backfill					RD	Drove Standard Split Spoon 60.0-61.5
						DR 0.0/1.5	7/0.5 9/0.5 11/0.5
62.0						RD	Stopped for Day 11:28:78 Hole caved to 31.5 ft.
		(GC-GP)	clay content decreases.				
64.0						PB 0.4/2.5	Pitcher Barrel 64.0-66.0 Continuous loss of water
66.0				J-14		DR	Drove Standard Split Spoon 66.0-67.5
Piezometer #2 →			67.0 - BEDROCK:	J-15		0.5/1.5	15/0.5 15/0.5 50/0.5
68.0						RD	

SHEET 3 OF 4

PROJECT 1890 DATE DRILLED 11:28:77 HOLE NO. 2D-1

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
68.0		67.0 – <u>BEDROCK:</u> Cont. Dark olive green shale and Franciscan melange; contains some roots in top 5 feet of unit; varying quantities of clay, decreasing with depth; hard.		RD	Rig chattering continuously
70.0					
72.0					
74.0			J-16	RD DR 0.5/0.5 RD	Drove Standard Split Spoon 74.0 – 74.5 50/0.5
78.0					
80.0		B.H. 80.2		RD	Drove Standard Split Spoon 80.0 – 80.2. 30/0.2 Rods bouncing
82.0					Backfilled with: sand – 80.2 – 55 piezoseal – 55 – 50 piezometer @ 69'.
84.0					sand 50 – 36 piezoseal 36 – 31 piezometer 43' (graphical display of piezometer installations in left-hand CLASS. column of log)
86.0					
88.0					
90.0					
92.0					
94.0					

SHEET 4 OF 4

EARTH SCIENCES ASSOCIATES

DRILLING AND SAMPLING LOG

PROJECT PHOENIX DAM 1890 DATE DRILLED 11:29 - HOLE NO. RD-2

LOCATION DAM CREST NR. MAXIMUM SECTION. GROUND SURFACE ELEV. 159±

DRILLING CONTRACTOR J.N. PITCHER LOGGED BY MTD DEPTH TO GROUND WATER

TYPE OF RIG FAILING 750 HOLE DIAMETER 4 7/8" HAMMER WEIGHT AND FALL 140 lb - 30 inch.

SURFACE CONDITIONS DAM CREST ROAD - BASE ROCK WEATHER CLEAR - COOL.

Piezometer Installation

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	Penetrometer	MODE	REMARKS
			Torvane TSF	TSF		
0.0	GP SM	0.0-1.0 <u>BASE ROCK:</u> Greyish blue; less than 5% fines; fine to coarse sand; fine gravel; moist; dense.			AD	Augered to 5 ft. Set 7'. 6"ϕ casing.
2.0	CL	1.0-5.5 (?)1963 FILL: Moderate yellowish brown with dark olive green and purple discoloration; generally a silty sand with layers of silty and gravelly clay; silty sand generally mod. yell. brown; moist; dense to very dense.			AD	
4.0			J-1		DR $\frac{0.8}{1.5}$	Drove Standard Split Spoon 3.5-5.0 7/0.5 24/0.5 26/0.5 Rock in bottom of shoe.
	CL				RD	
6.0	CL-GC	5.5-11.5 CLAYEY GRAVEL TO GRAVELLY CLAY: Purple and dark olive grey; variable amount of clay fines; some sand; fine to coarse angular gravel; some cobbles; moist; dense.	J-2	2.0 1.8 1.9	PB $\frac{1.7}{2.5}$	Pitcher Barrel 6.0-8.5 Bottom of tube badly bent.
8.0			J-3		PB 0.5/0.5	Pitcher Barrel 8.5-9.0 Refusal - losing all circulation. Tube Bent, rock in Bottom.
10.0			J-4		DR $\frac{0.2}{1.5}$ RD	Drove Standard split Spoon 9.0-10.5 15/0.5 8/0.5 7/0.5 Rocks in shoe, some clay binder.
12.0	CL-GC CL	11.5-19.4 SILTY CLAY: Olive green and light grey; low to moderate plasticity; contains scattered roots; firm; moist to wet. some black organic(?) discoloration. contains scattered gravel up to 2"	0.61 0.52	1.0 .9 1.0 1.0	PB $\frac{1.2}{2.5}$	10.0 Added half a bag of revert, could not get circulation. Mixed up ½ bag of bentonite. Drove another 5' of casing.
14.0			J-5	1.4 0.7 1.75	PB	Pitcher Barrel 12.0-14.5
16.0			0.55 0.45 0.51 J-6	0.5 0.8 1.2 1.1 2.0 2.25 2.1 *	$\frac{1.0}{2.5}$	Pitcher Barrel 14.5-17.0
18.0			0.4 0.25 J-7	1.0 1.2 0.9 1.2 1.4 1.3	PB $\frac{1.0}{2.5}$	Pitcher Barrel 17.0-19.5
20.0	CL CL	19.4-26.0 SANDY GRAVELLY CLAY:			PB	SHEET 1 OF 4

* Test on sample end.

PROJECT 1390 DATE DRILLED 11:29:77 HOLE NO. RD-2

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
20.0	CL	19.4-86.0 SANDY GRAVELLY CLAY: cont. Reddish brown; medium plasticity clay; 20-30% predominantly coarse sand; up to 20% coarse gravel; medium dense to dense; moist to wet.		PB 1.2/2.5	Pitcher Barrel 19.5 - 22.0 Tube Bent at bottom. Rock gouged side of sample.
22.0			J-8	PB 0.5/2.0	Pitcher Barrel 22.0 - 24.0 Steady loss of water.
24.0			J-9	PB 0.3/2.5	Pitcher Barrel 24.0 - 26.5
26.0			J-10	DR 0.7/1.5	Drove Standard Split Spoon 26.5 - 28.0 4/0.5 4/0.5 7/0.5
28.0	(GC) bentonite seal CL GC-GP	28.5 Cuttings become cleaner, fines could be washing out or material is cleaner.	J-11	RD	stopped for night 11:29:77
				PB	Pitcher Barrel
30.0		grades to clayey gravel; with lenses of clean gravel. Binder is medium plasticity clay.		0/2.5	29.0 - 31.5 Tube bent
32.0				PB 0/2.5	Pitcher Barrel 31.5 - 34.0 Tube bent
Piezometer #1					
34.0				RD	Loosing water continuously.
				DR	Drove Standard split spoon
36.0	sand backfill		J-12.	0*/1.5	35.0 - 36.5 5/0.5 4/0.5 8/0.5
				RD	* 6" of loose clean gravel in tube, could be cuttings or native.
	GC-GP CL	36.7-38.5 lense of light olive grey sandy clay, contains scattered fine gravel; stiff to very stiff; moist.		DR 0.5/1.5	Drove Standard split spoon
38.0	GC		J-13	RD	37.0 - 38.5 9/0.5 8/0.5 12/0.5
		color grades to dark grey			
40.0	bentonite seal			PS PB 0/1.0	Pushed Shelby Tube 40.0-41.0 Pitcher Barrel 40.0-42.5
42.0			J-14	0.5/2.5	Tube Bent.
				PB	Pitcher Barrel 42.5-45.0
44.0					

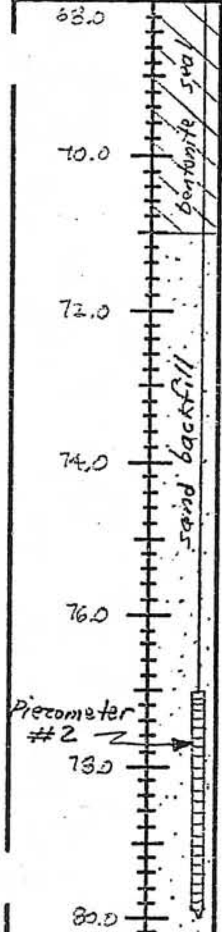
SHEET 2 OF 4

PROJECT 1890 DATE DRILLED 11:30 HOLE NO. RD-2

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE Penetrometer?	MODE	REMARKS
44.0	GC GP GC-SC	19.4-86.0 SANDY GRAVELLY CLAY: cont.	Torvane ↓ TSF	PB	
			J-15	1.0/1.5	Tube Bent.
		44.6 sand content increases.	TSF	DR	Drove Standard split spoon
46.0		matrix becomes sandy clay.		0.6/1.5	45.0-46.5
			J-16		9/0.5 19/0.5 22/0.5
				RD	Rock in shoe.
48.0					
	GC-SC CL	? ~49.0 grades to sandy clay with up to 20% fine to coarse gravel in lenses and throughout unit.			
50.0				RD DR	Drove Standard split Spoon
		sand content 30%? variable.		0.5/1.5	50.0-51.5
			J-17		11/0.5 7/0.5 13/0.5
52.0				RD	
54.0					
56.0				RD	
	CL			PB	Pitcher Barrel
		57.7-58.2 clayey silt to silty clay with some gravel and sand.	0.50 1.7		
58.0	CL-ML		J-18		57.0-59.5
	SM-ML	58.2-58.6 Sandy silt to silty sand w. gravel.	J-19 0.45 1.75	1.3/2.5	
	SM(SP) CL	58.6-59.0 silty sand w. gravel	J-20 0.83		
	CL-ML	59.0-59.5 Clayey silt to silty clay, w. gravel.	J-21 2.3		
60.0	CL (SC)			PB	Pitcher Barrel 59.5-62.0.
				0.7/1.5	
62.0			J-22	DR	Drove standard split Spoon 62.0-63.5
	bentonite seal		J-23 1.2	0.5/2.5	8/0.5 14/0.5 24/0.5
64.0				RD	
66.0	(SC) CL ? GC				
68.0					67.0 Stopped for night 11:30

SHEET 3 OF 4

PROJECT 1890 DATE DRILLED 12:1 HOLE NO. RD-2



DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE Penetrometer	MODE	REMARKS
68.0	GC	19.4-86.0 SANDY GRAVELLY CLAY; CONT.	√ TSF	PB 0/2.5	Pitcher Barrel 68.0-70.5
70.0					12:1 Hole caved to 53' overnight.
				RD	300 gals of water + 1½ sacks of mud to clean out and run tube - still losing circulation.
72.0				PB 0/2.5	Pitcher Barrel 72.0 - 74.5
74.0					Drove an extra 5' of casing to try and control water loss.
				DR 0/1.5	Drove Standard split-Spoon 74.5-76.0 3/0.5 4/0.5 6/0.5
76.0				RD	sample of cuttings. Hole was well cleaned out no obvious reason for lack of sample. Pushing rock?
		cleaned out for long time; cuttings of rock with bits of purple clay.		DR 2/1.5 *	
78.0			J-24	RD †	Drove Standard Split Spoon 77.0-78.5 3/0.5 3/0.5 12/0.5
		cuttings of sand and gravel; some clay.			* Cuttings.
80.0		hole caves before sampler gets back into hole, can't get thick mud because of leakage.		PB 0/2.5	Pitcher Barrel 80.0-82.5
82.0					† 78.5-80.0 Rig chatters moderately.
	CL-SC	? 83.0 grey sandy clay to clayey sand.		RD	
84.0				DR 0.6/2.5	Drove Standard Split Spoon 84.0-86.5 5/0.5 6/0.5 12/0.5
			J-25	1.75 1.4 1.25 1.0	
86.0	CL-SC	86.0-88.4 BEDROCK Olive green shale with some clay binder, clay decreases with depth.		RD	86.0-87.5 Rig chatters violently.
88.0			J-26	DR 0.3/0.4	Drove standard split spoon 88.0-88.4 30/0.2 rods bouncing.
		B.H. 88.4			Sand B.H-71 Piezo 80 Bentonite Balls 71-63. Hole caved overnight 63-43. Bent. Balls 43-40 sand 40-30 piezo. 35'. Bent. balls 30-27'
90.0					
92.0					

SHEET 4 OF 4

(graphical display of piezometer installations in left-hand CLAS column of log)

EARTH SCIENCES ASSOCIATES

DRILLING AND SAMPLING LOG

PROJECT PHOENIX DAM 1890 DATE DRILLED 12:5: HOLE NO. RD-3
LOCATION DAM CREST GROUND SURFACE ELEV. 1891
DRILLING CONTRACTOR J.N. PITCHER LOGGED BY MTD DEPTH TO GROUND WATER
TYPE OF RIG FAILING 750 HOLE DIAMETER 9" HAMMER WEIGHT AND FALL 140 lb-30"
SURFACE CONDITIONS BASE ROCK WEATHER MISTY

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE Penetrometer		MODE	REMARKS
			Torvane TSF	√ TSF		
0.0	GM	0.0-0.5 BASE ROCK:			RD	Rotary Drill with 9" bit.
	CL	0.5-9.0 SILTY SANDY CLAY: Moderate yellowish brown with numerous dark olive green and purple discolorations; sandy clay with pockets of silty clay; contains gravel up to 2"; moist; stiff to very stiff.				
2.0						
4.0						
6.0					RD	
			B-1	>4.5	PB 0.6/2.5	220 psi hydraulic pressure while cutting cored with 6" pitcher barrel most of sample dropped out near top of hole
		7.0 gravel content increases; color grades to purple.				
8.0	(GC) CL					Cleaned out hole to 8.5'
	GC-CL	9.0-14.2 CLAYEY GRAVEL TO GRAVELLY CLAY: Dark olive green; varying amounts of sandy and silty clay;			PB 2.5/2.5	Pitcher Barrel 8.5-11.0 9.5 Rig chatters 10.5 Rig chatters.
10.0	GC-CL		B-2			
	CL			1.1, 1.3 1.2, 1.5		
	GC-CL CL CL	11.4-11.6 light olive green silty clay. 11.6-11.8 purple sandy clay.	B-3		PB 0.8/2.0	losing circulation used 1½ bags of bentonite for first 11 feet. Water stands at 9 feet.
12.0	GC-CL (?)					
	GC-GP	clay binder is purple, predominantly sandy clay; boulders up to 5"; mod. clean	B-4		PB 2.5/2.5	Pitcher Barrel 13.0-15.5 13.0-14.2 rig chatters strongly, losing circulation.
14.0	GC-GP					
	CL	14.2-21.6 SILTY CLAY: Light olive grey and light grey with rust red discolorations; medium plasticity; contains scattered gravel up to 2"; moist, locally wet; firm.	B-5 0.73 0.65	0.8 1.0 1.75 1.9	RD	
16.0					PB 2.5/2.5	Pitcher Barrel 16.0-18.5
			B-6 0.37 0.45 0.63 0.63	0.5 0.70 0.75 1.0 0.85 0.8 0.75 1.75 0.9 1.9		
18.0					RD	
20.0					RD	

SHEET 1 OF 4

PROJECT 1830 DATE DRILLED 12:6:77 HOLE NO. RD-3

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	Penetration	MODE	REMARKS
20	CL	14.2-21.6 SILTY CLAY: cont.		¼ TSF	RD	
		21.6-70.5 GRAVELLY SANDY CLAY:				21.5 Stopped for night. Water dropped 3' overnight.
22	CL	Purple with rust brown and light olive green discoloration; sandy clay (soft to firm) matrix; fine to coarse sand; pred. fine gravel; 20 - 30 % coarse sand and gravel; firm; moist to wet.			RD PB	Pitcher Barrel 22.0 - 22.5
24			B-7	1.25 1.3	2.4/2.5	Continuous moderate chatter.
					RD	25.0 Plate loading Test.
26					RD PB	Pitcher Barrel 26.5 - 27.8
	CL GP	27.5-28.5 Rig chattering violently; boulder(?)	B-8		0.5/1.3	Strong Chattering, Refusal.
28	CL				RD	Bottom of sample fell out of tube. Clean cuttings(?) in tube.
					DR	Drove Standard Split Spoon 29.0 - 31.5
30	CL GC-CL	30.0 Dark olive grey gravelly clay. 30.5 color grades to purple; gravel and sand content increases to 50-60%; clay binder is firm.	B-9		0.3/1.5 PB	5/0.5 4/0.5 7/0.5 Pitcher Barrel
32			B-10		1.1/2.5 RD	
34		color grades to medium grey; with brick red and turquoise discolorations.				35.0 Plate Loading Test. 35.0 stopped for Day 12:6:77.
36		37.5 Pockets of turquoise silty clay; firm. 37.8 - 40.2 layer of yellowish brown gravelly sandy clay; appears less dense than surrounding material.			RD	
38			B-11		PB 2.5/2.5	Pitcher Barrel 37.5 - 40.0 37.5-38.0 Rig chatters strongly
		gravel content is about 25%	B-12	1.1 1.5		38.0-40.0 " " moderately
40					RD	40.0 Plate loading Test. DMY logging hole ↓
		42.1 - 42.4 large grn. gray graywacke cobble; washed out void ~3" max.				
42		42.4 - 43.0 med. to dk. ol gray silty clay, scattered med. sand; small patches yel. brown clay; gravel content ~ 20-25%	B-13	1.7(?) 1.2 1.25 1.30(?)	RD PB 0.9/1.0	Cored w/ Pitcher barrel sampler Have been coring w/200-300 psi down pressure
					RD	swivel elbow on mud line broke—pulled sampler
44	GC-CL				RD	

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SHEET 2 OF 4

PROJECT 1890-Phoenix Dam DATE DRILLED 12/7/77 HOLE NO. RD-3

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	Penetrometer	MODE	REMARKS
44	GCCL	21.6-70.5 GRAVELLY SANDY CLAY: cont.		↓ TSF	RD	44.0 Plate loading test
46		color grades dark greenish grey; gravel content 25-30%.	B-14		RD $PB\frac{0.5}{0.5}$ RD	Cored w/Pitcher barrel sampler Barrel plugged up - no circulation 12:50 p.m. - Line on pump blew; down 10 minutes
48		48-48.6 2-3 large graywacke cobbles, ~3" max. dimension 48.5-49.5 - brn. gray gravelly clay 49.5-50 purplish gray sandy silty clay, scattered gravel to 15-20% 49.6 - layer of clayey med. sand; ~1/2" thick	B-15	1.25 1.4 1.35 2.3 1.7	PB $\frac{1.9}{2.5}$ RD	Cored with Pitcher Barrel 48.0-50.5.
52		54.8 - 55.7 - greenish to mod. grey gravelly sandy clay some stringers of purple clay, less well compacted				
54		55.7 - 56.5 - several pockets yel-grn. silty clay, clayey silt; also purple silty clay gravel ~35%	B-16	1.7	RD PB $\frac{1.7}{2.5}$	Cored with Pitcher Barrel 54 - 56.5
56		56.5 - small pocket of mod. brown clay.	B-17	1.5 1.7 2.0 1.5	PB RD	
58						
60			B-18	1.5 1.3 1.1	RD PB $\frac{1.3}{1.5}$	cored with Pitcher Barrel 60 - 61.5 61.5 - 62.8 hard, rig chattering
		62.8 - purple & grey brn clay in cuttings		2.3	PB RD	62.5 - softer, drilling faster
62						
64		64 - lot of clay coming up in cuttings				
66		67 - high amount of clay in cuttings				67 - drilling smooth
68						

SHEET 3 OF 4

PROJECT 1890 - Phoenix Dam DATE DRILLED 12/7/77 HOLE NO. RD-3

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
68		21.6-70.5 <u>GRAVELLY SANDY CLAY</u>: con't.		RD	68.7 - bit of chattering
70		~70 - 60%-70% of cuttings is clay <u>Bedrock</u>			70.5 - heavy chattering
		70.5 - 71.5 - <u>GRAYWACKE</u> hard to very hard little to mod weathered judging by cuttings and drill rate			
	B.H. 71.5'				B.H. 71.5

SHEET 4 OF 4

EARTH SCIENCES ASSOCIATES
DRILLING AND SAMPLING LOG

PROJECT 1890 - Phoenix Dam DATE DRILLED 12/5-6/77 HOLE NO. RD-4
LOCATION Downstream berm between existing holes 3 & 4 GROUND SURFACE ELEV. 140' (Topo)
DRILLING CONTRACTOR J.N. Pitcher Co. LOGGED BY DMY DEPTH TO GROUND WATER —
TYPE OF RIG Failing 250 HOLE DIAMETER 5" HAMMER WEIGHT AND FALL 140 lb., 30"
SURFACE CONDITIONS grassy embankment berm WEATHER clear, mild
(track mounted crawler rig)

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	Pocket Penetrom	MODE	REMARKS
2 4 new toe berm	CL	0.0-4.5' (GRAVELLY) SANDY CLAY: dk. yel. brn. (5 YR 3/4); mod. plastic, slow dilatancy silty-clay fines (75-85%); scattered (15-20%) fine to coarse qtz., lithic sand grains; fine gravel (5-10%), pred. Fran. gwy, some ch, grnst., sh., ang. to subrnd. frags, vary hd. to friable, weath. stained; firm to stiff (mod. dense); slightly moist.			AD RD	Spin augered to 3½' set up mud tub-1½ sk. bentonite to seal collar, start circulation Drilled down w/ drag bit set surface casing to 4.0'
6 8 chimney drain	GP	4.5-9.0' GRAVEL: med. grn. gray (5 GS 5/1), some mod. brn. (5 YR 4/2) weath. stains; v. minor fines (<< 5%); scattered coarse, ang. graywacke sand grains (1-3%); pred. med. gravel (½-1" avg.), ang., fresh to little weathered, hd., strong frags. (Probably a drain chimney)	B-1		PB 0.4/2.5 RD	Cored w/ Pitcher Bbl. Assem. (only clean gravel w/some sand in tube, no clay binder) Drilled down with drag bit Add 2½ sks. bentonite to maintain circulation set casing to 9.0' to reduce water loss, maintain circulation
10 12 14 16 hole caved to here 12/5 after drilling to 19.0' Original embankment	GC	9.0-22.0' CLAYEY GRAVEL: dk. yel. brn. (10 YR 4/2), mod brn. (5 YR 4/4); 10-15% mod plastic silty clay fines; 5-10% scattered coarse, ang. sandst, volc. sand grains; 75-85% fine to med. gravel, pred. ang. to sub rnd. graywacke, meta graywacke, metavolc. frags. consistency uncertain, probably dense in-situ; moisture uncertain in-situ. 14.0-15.0 grades more sandy (15-20%), more clayey (20-30%)	J-1 J-2	19(?)	PB 1.5/2.5 RD PB 1.2/2.5	Cored with Pitcher barrel assembly Drilled down w/ drag bit Cored with Pitcher barrel assembly 0.5' gravel cave on top of sample
18	GP	16.5-19.0 amount of caving and appearance of sample indicate little clay binder	B-2		PB 0.3/2.5	cored with Pitcher barrel assembly 0.5' cave on top of 3-4" diam. cobble wedged in end of tube
12/5 20	GC	19-20 cuttings suggest material similar to 14.0-15.0 interval			RD	Drilled down w/ tricone bit

SHEET 1 OF 2

RD to 19' 3 times, hole caves to 15' each time, add ½ sk bentonite, core to 16'

PROJECT 1890 - Phoenix Dam DATE DRILLED 12/5-6/77 HOLE NO. RD-4

add 1/4 sack bentonite

cased to 20', add 1/4 sk. bentonite, flush to 22' after RD

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS	N, blows/ft
	original embankment GC			DR 0.0/1.5	Drove standard splitspoon 13/0.5 12/0.5 18/0.5	3
				RD	Drilled down with tricone bit	
22	GC (GP)	22.0-43.5' <u>CLAYEY SANDY GRAVEL:</u> mod. brn. (5 YR 4/4); generally 20-25% mod. plastic silty clay fines, locally to 40-45% in clayey pockets; sand (30-40%) fine to predom. coarse, mainly angular rock frags; 35-45% gravel, avg. size 3/4-1", pred. ang. to subang., chert, serpentinite, metavolc., graywacke, frags gen. hard, strong. gravel seems to be in loose (to soft) clayey sand matrix in places, blow counts suggest mod. dense material; probably moist in-situ.	J-3	PB 1.7/2.5	Cored with Pitcher barrel assembly Tube had 2.7' sample + voids before extrusion	
24			cave	PB 0.3/2.5	Cored w/ Pitcher barrel assembly minor slough/cave on top of 2½" cobble in end of tube	
26			cave	RD	Drilled down w/ tricone bit	
			B-3	DR 0.7/1.5	Drove standard split spoon 16/0.5 12/0.5 17/0.5	2
28				RD	Drilled down w/ tricone bit add 3/4 sk. bentonite to flush cuttings	
	SC (GC)	29.0-29.9 grades more coarse sand (clayey gravelly sand to gravelly clayey sand)	B-4	DR 0.9/1.5	Drove standard split spoon 11/0.5 8/0.5 9/0.5	17
30		30.5-34.0' cuttings suggest more coarse-fine sand, less gravel, more fines		RD	Drilled down w/ tricone bit add 1 sk bentonite to flush cuttings; water loss <u>not</u> significant (have used total of ~5 sks. bentonite in hole to here)	
32						
34	GC (GP)	34.2-34.6 distinctive gray-brown clayey sandy gravel	J-4 J-5	PB 1.6/2.5	Cored w/ Pitcher barrel assembly samples appear relatively undisturbed after extrusion	
36				RD	Drilled down w/ tricone bit	
	GP?	37.0-39.0' cuttings suggest relatively little clay binder				
38						
	GC (GP)			DR 0.0/1.5	Drove standard split spoon 11/0.5 11/0.5 11/0.5	2.
40				RD	Drilled down w/ tricone bit	
42		<u>Bedrock</u> 43.5-44.0 <u>METAGRAYWACKE:</u> cuttings suggest ol. to gray grn meta-graywacke sandstone; little clayey binder in cuttings; hard drilling			mod. chatter, rig slows at 43.5'	
44	rock				SHEET 2 OF 2	

B.H. - 44.0'

Terminate hole at 44.0

Important Information About Your

Geotechnical Engineering Report

Subsurface problems are a principal cause of construction delays, cost overruns, claims, and disputes.

The following information is provided to help you manage your risks.

Geotechnical Services Are Performed for Specific Purposes, Persons, and Projects

Geotechnical engineers structure their services to meet the specific needs of their clients. A geotechnical engineering study conducted for a civil engineer may not fulfill the needs of a construction contractor or even another civil engineer. Because each geotechnical engineering study is unique, each geotechnical engineering report is unique, prepared *solely* for the client. No one except you should rely on your geotechnical engineering report without first conferring with the geotechnical engineer who prepared it. *And no one — not even you* — should apply the report for any purpose or project except the one originally contemplated.

Read the Full Report

Serious problems have occurred because those relying on a geotechnical engineering report did not read it all. Do not rely on an executive summary. Do not read selected elements only.

A Geotechnical Engineering Report Is Based on A Unique Set of Project-Specific Factors

Geotechnical engineers consider a number of unique, project-specific factors when establishing the scope of a study. Typical factors include: the client's goals, objectives, and risk management preferences; the general nature of the structure involved, its size, and configuration; the location of the structure on the site; and other planned or existing site improvements, such as access roads, parking lots, and underground utilities. Unless the geotechnical engineer who conducted the study specifically indicates otherwise, do not rely on a geotechnical engineering report that was:

- not prepared for you,
- not prepared for your project,
- not prepared for the specific site explored, or
- completed before important project changes were made.

Typical changes that can erode the reliability of an existing geotechnical engineering report include those that affect:

- the function of the proposed structure, as when it's changed from a parking garage to an office building, or from a light industrial plant to a refrigerated warehouse,
- elevation, configuration, location, orientation, or weight of the proposed structure,
- composition of the design team, or
- project ownership.

As a general rule, *always* inform your geotechnical engineer of project changes—even minor ones—and request an assessment of their impact. *Geotechnical engineers cannot accept responsibility or liability for problems that occur because their reports do not consider developments of which they were not informed.*

Subsurface Conditions Can Change

A geotechnical engineering report is based on conditions that existed at the time the study was performed. *Do not rely on a geotechnical engineering report* whose adequacy may have been affected by: the passage of time; by man-made events, such as construction on or adjacent to the site; or by natural events, such as floods, earthquakes, or groundwater fluctuations. *Always* contact the geotechnical engineer before applying the report to determine if it is still reliable. A minor amount of additional testing or analysis could prevent major problems.

Most Geotechnical Findings Are Professional Opinions

Site exploration identifies subsurface conditions only at those points where subsurface tests are conducted or samples are taken. Geotechnical engineers review field and laboratory data and then apply their professional judgment to render an opinion about subsurface conditions throughout the site. Actual subsurface conditions may differ—sometimes significantly—from those indicated in your report. Retaining the geotechnical engineer who developed your report to provide construction observation is the most effective method of managing the risks associated with unanticipated conditions.

A Report's Recommendations Are *Not* Final

Do not overrely on the construction recommendations included in your report. *Those recommendations are not final,* because geotechnical engineers develop them principally from judgment and opinion. Geotechnical engineers can finalize their recommendations only by observing actual subsurface conditions revealed during construction. *The geotechnical engineer who developed your report cannot assume responsibility or liability for the report's recommendations if that engineer does not perform construction observation.*

A Geotechnical Engineering Report Is Subject to Misinterpretation

Other design team members' misinterpretation of geotechnical engineering reports has resulted in costly problems. Lower that risk by having your geotechnical engineer confer with appropriate members of the design team after submitting the report. Also retain your geotechnical engineer to review pertinent elements of the design team's plans and specifications. Contractors can also misinterpret a geotechnical engineering report. Reduce that risk by having your geotechnical engineer participate in prebid and preconstruction conferences, and by providing construction observation.

Do Not Redraw the Engineer's Logs

Geotechnical engineers prepare final boring and testing logs based upon their interpretation of field logs and laboratory data. To prevent errors or omissions, the logs included in a geotechnical engineering report should *never* be redrawn for inclusion in architectural or other design drawings. Only photographic or electronic reproduction is acceptable, *but recognize that separating logs from the report can elevate risk.*

Give Contractors a Complete Report and Guidance

Some owners and design professionals mistakenly believe they can make contractors liable for unanticipated subsurface conditions by limiting what they provide for bid preparation. To help prevent costly problems, give contractors the complete geotechnical engineering report, *but* preface it with a clearly written letter of transmittal. In that letter, advise contractors that the report was not prepared for purposes of bid development and that the report's accuracy is limited; encourage them to confer with the geotechnical engineer who prepared the report (a modest fee may be required) and/or to conduct additional study to obtain the specific types of information they need or prefer. A prebid conference can also be valuable. *Be sure contractors have sufficient time* to perform additional study. Only then might you be in a position to give contractors the best information available to you, while requiring them to at least share some of the financial responsibilities stemming from unanticipated conditions.

Read Responsibility Provisions Closely

Some clients, design professionals, and contractors do not recognize that geotechnical engineering is far less exact than other engineering disciplines. This lack of understanding has created unrealistic expectations that have led to disappointments, claims, and disputes. To help reduce the risk of such outcomes, geotechnical engineers commonly include a variety of explanatory provisions in their reports. Sometimes labeled "limitations" many of these provisions indicate where geotechnical engineers' responsibilities begin and end, to help others recognize their own responsibilities and risks. *Read these provisions closely.* Ask questions. Your geotechnical engineer should respond fully and frankly.

Geoenvironmental Concerns Are Not Covered

The equipment, techniques, and personnel used to perform a *geoenvironmental* study differ significantly from those used to perform a *geotechnical* study. For that reason, a geotechnical engineering report does not usually relate any geoenvironmental findings, conclusions, or recommendations; e.g., about the likelihood of encountering underground storage tanks or regulated contaminants. *Unanticipated environmental problems have led to numerous project failures.* If you have not yet obtained your own geoenvironmental information, ask your geotechnical consultant for risk management guidance. *Do not rely on an environmental report prepared for someone else.*

Obtain Professional Assistance To Deal with Mold

Diverse strategies can be applied during building design, construction, operation, and maintenance to prevent significant amounts of mold from growing on indoor surfaces. To be effective, all such strategies should be devised for the *express purpose* of mold prevention, integrated into a comprehensive plan, and executed with diligent oversight by a professional mold prevention consultant. Because just a small amount of water or moisture can lead to the development of severe mold infestations, a number of mold prevention strategies focus on keeping building surfaces dry. While groundwater, water infiltration, and similar issues may have been addressed as part of the geotechnical engineering study whose findings are conveyed in this report, the geotechnical engineer in charge of this project is not a mold prevention consultant; ***none of the services performed in connection with the geotechnical engineer's study were designed or conducted for the purpose of mold prevention. Proper implementation of the recommendations conveyed in this report will not of itself be sufficient to prevent mold from growing in or on the structure involved.***

Rely, on Your ASFE-Member Geotechncial Engineer for Additional Assistance

Membership in ASFE/The Best People on Earth exposes geotechnical engineers to a wide array of risk management techniques that can be of genuine benefit for everyone involved with a construction project. Confer with you ASFE-member geotechnical engineer for more information.

ASFE
The Best People on Earth

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