APPENDIX E

GEOTECH REPORT FOR WASTEWATER FACILITY

GEOCON
CONSULTANTS, INC
GEOTECHNICAL
ENVIRONMENTAL
MATERIALS

GEOTECHNICAL AND GEOLOGICAL RECONNAISSANCE STUDY

PROPOSED IONE CASINO AND HOTEL TREATED WATER SEASONAL STORAGE RESERVOIR AMADOR COUNTY, CALIFORNIA

PREPARED FOR

ANALYTICAL ENVIRONMENTAL SERVICES
SACRAMENTO, CALIFORNIA

DRAFT DATE: AUGUST 2004
REVISED APRIL 2005

Project No. S8874-06-01
Draft Date: August 20, 2004
Revised April 29, 2005

Mr. Josh Ferris
Analytical Environmental Services
2021 "N" Street, Suite 200
Sacramento, California 95814

Subject: PROPOSED IONE CASINO AND HOTEL
TREATED WATER SEASONAL STORAGE RESERVOIR
AMADOR COUNTY, CALIFORNIA
GEOTECHNICAL AND GEOLOGICAL RECONNAISSANCE STUDY

Dear Mr. Ferris:

In accordance with your authorization of our proposal dated July 15, 2004, we have performed a geotechnical and geological reconnaissance study for the proposed treated water seasonal storage reservoir at the project Site. The Site is located on the east side of State Route 49, just south of the town of Plymouth, Amador County, California.

The accompanying report presents the findings of our study, and our initial conclusions and recommendations pertaining to the geotechnical and geological feasibility (on a reconnaissance level) of developing the project as presently proposed. From a geotechnical and geological standpoint, construction of an earth or rock-fill embankment dam at the Site appears to be feasible provided the recommendations of this report are followed and subsequent design-level geotechnical studies are performed to confirm that geologic and geotechnical conditions are as initially indicated. However, constructing a liner system for the reservoir to control leakage and seepage will likely be very difficult. Discussion of this issue is included in the report.

This report has been revised based on the review comments of the United States Bureau of Indian Affairs (BIA) contained in a memorandum prepared by Tom Dang, Regional Engineer/Program Manager, Natural Resources, dated March 2, 2005.

If there are any questions concerning the contents of this report, or if Geocon may be of further service, please contact the undersigned at your convenience.

Sincerely,

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TABLE OF CONTENTS

GEOTECHNICAL AND GEOLOGIC RECONNAISSANCE STUDY

FIGURES

TABLE OF CONTENTS (Continued)

GEOTECHNICAL AND GEOLOGIC RECONAISSANCE STUDY

1.0 PURPOSE AND SCOPE

This report presents the results of a geotechnical and geologic reconnaissance study for the proposed Ione Casino and Hotel Treated Water Seasonal Storage Reservoir Project. The project consists of constructing an embankment dam-impounded reservoir to provide up to approximately 43 acre-feet of seasonal storage capacity for tertiary treated wastewater (reclaimed water) at the Site. The Site is located on the east side of State Route (SR) 49, just south of the city of Plymouth, Amador County, California (Vicinity Map, Figure 1).

The purpose of the investigation was to provide an initial geotechnical engineering and geological assessment of the Site and based on conditions encountered, provide reconnaissance-level conclusions and recommendations regarding the geotechnical and geological feasibility of developing the project as presently proposed. It will be necessary to perform an in-depth geotechnical investigation, including additional extensive subsurface exploration and laboratory testing, at the reservoir Site to adequately characterize the embankment dam foundation and abutment areas as well as appurtenant structures.

Geocon performed the following scope of services:

1.1 Pre-field Activities

- Reviewed the following literature, maps and stereo-pair aerial photographs regarding the geologic, seismic and geotechnical setting of the Site:
    - Applied Engineering and Geology, Inc., *Results of Soil Mantle and Percolation Tests, Ione Rancheria, Plymouth, Amador County, California,* March 2, 2004.
    - Cartwright Aerial Survey, 2001, scale: 1:2,400, Black and White, Flight AC 01 – 24, Frames 101, 11.
    - California Division of Safety of Dams, *Guidelines for the Design and Construction of Small Embankment Dams*, 1993 reprint.
    - California Division of Mines and Geology, *Fault Evaluation Report FER-148, Southern Foothills Fault System, Amador, Calaveras, El Dorado, and Tuolumne Counties*, William A. Bryant, December 30, 1983.
    - Fraser, W. A., 1995, *DSOD Fault Activity Guidelines*, California Division of Safety of Dams.
    - HydroScience Engineers, Inc., *Ione Casino and Hotel Feasibility Study, Wastewater Treatment and Disposal Alternative A, Phase 1* (Figure 4-2), undated.
    - Jennings, C.W. (compiler), 1977, *Geologic Map of California*, California Division of Mines and Geology.
    - Loyd, R.C., 1983, *Mineral Land Classification of the Sutter Creek 15 minute Quadrangle, Amador and Calaveras Counties, California*, California Division of Mines and Geology.

- U.S. Army Corps of Engineers, 1994, *Earth and Rock-Fill Dams – General Design and Construction Considerations*, Engineer Manual (EM 1110-2-2300).
- Wagner, D.L., Jennings, C.W., Bedrossian, T.L. and Bortugno, E.J., 1987, *Geologic Map of the Sacramento Quadrangle*, California Division of Mines and Geology.

- Reviewed preliminary design plans (HydroScience, 2004) to determine exploratory test pit locations.

1.2 Geologic Reconnaissance

- Performed a geologic reconnaissance by a California Certified Engineering Geologist.
- Prepared a reconnaissance-level geologic map presenting the geologic/soils conditions observed at the reservoir Site.

1.3 Field Exploration Program

- Performed six exploratory test pits (TP1 through TP6) within the foundation and abutment areas for the dam. The test pits were excavated with a John Deere 710D backhoe equipped with an 18-inch bucket with rock teeth. The approximate exploratory test pit locations are shown on the Site Plan/Geologic Maps, Figures 2a and 2b. A detailed discussion of the field investigation program including logs of the exploratory test pits is included in Appendix A.
- Obtained bulk samples from the exploratory test pits for subsequent laboratory testing.
- Backfilled the exploratory test pits with the excavated material upon completion.

1.4 Laboratory Testing Program

- Performed geotechnical laboratory tests on selected samples to determine pertinent geotechnical parameters. A description of the laboratory testing program, including test results, is included in Appendix B.

1.5 Seismic Evaluation

- Performed a preliminary seismic evaluation of the proposed reservoir Site. The assessment included review of published seismic literature, maps and computer analysis.

1.6 Feasibility Analysis and Report Preparation

- Prepared this report summarizing our reconnaissance level conclusions and recommendations regarding the geotechnical and geological feasibility of designing and constructing the reservoir improvements.

2.0 SITE AND PROJECT DESCRIPTION

The site description presented herein is based on a site reconnaissance performed on July 29, 2004. The project description is based on information provided by Analytical Environmental Services (AES) and discussions with HydroScience Engineers, Inc. (HSe, project water/wastewater design consultant).

The proposed reservoir site is located within a primarily undeveloped 160-acre (approximate) parcel (parent parcel) located east of SR 49 just south of the city of Plymouth, in Amador County, California (Vicinity Map, Figure 1). The largely undeveloped parent parcel is currently utilized for cattle grazing and rock/aggregate mining. Existing improvements include a domestic water well, unpaved haul roads and an abandoned gold mine (Pioneer Mine). Wire fencing generally delineates the parcel boundaries. Natural features of the parent parcel include rock outcroppings, annual grassland, oak and pine woodland, and unnamed intermittent drainages. The topography of the parcel varies from gentle (2%) to very steep (100%) slopes, with elevations ranging from approximately 780 to 1,150 feet above mean sea level.

The proposed seasonal storage reservoir Site (the Site) is located within an unnamed, deeply incised canyon which contains an intermittent drainage that conveys seasonal flows. The canyon is located within the southeast portion of the parent parcel. The canyon is a tributary to Dry Creek located south-southeast of the Site. At the Site, the canyon is heavily vegetated with annual grasses, brush and oaks. The canyon drains to the south at an approximate slope of 5%. The inclination of the canyon walls range from approximately 3:1 (horizontal to vertical) to 1:1 or steeper. The elevation differential between the bottom of the canyon and the surrounding ridges ranges from approximately 100 to 200 feet. The location and topography of the Site are depicted on the Site Plan/Geologic Maps, Figures 2a and 2b. The downstream watershed area below the proposed dam is shown on the Downstream Watershed Map, Figure 3. The Watershed Map shows the drainage course from the Site, along Dry Creek to the terminus with the Mokelumne River in the Sacramento/San Joaquin Delta Region approximately 42 miles southwest of the Site.

Two unpaved roads traverse the canyon at the Site. The primary road crosses the drainage over an embankment fill approximately 10 feet high. The drainage is conveyed through the embankment fill via a corrugated metal pipe (CMP) culvert. The secondary road, presumably constructed as a material haul road, crosses the drainage at-grade. The roads are shown on Photo No. 1 on Figure 4. Downstream from the roads, within the likely footprint of the proposed embankment dam, the canyon has been partially filled with rock and soil. A CMP culvert was placed in the bottom of the canyon to covey drainage through the fill. The fill extends longitudinally approximately 225 feet and is approximately 40 to 50 feet wide. The thickness of the fill varies from approximately 8 to 12 feet. The fill area is shown on Photo No. 2 on Figure 4.

Based on the preliminary design developed by HSe, the project wastewater treatment and disposal system requires approximately 43 acre-feet (14 million gallons) of wet weather seasonal storage for reclaimed water. During wet weather periods, the reclaimed water will be detained in the proposed reservoir until conditions are suitable for subsurface percolation or spray-field disposal. HSe has indicated that all of the detained water stored in the reservoir will need to be disposed of using onsite disposal facilities which consist of subsurface leachfields and spray irrigation systems. Therefore, inflow of additional water, including precipitation, storm runoff and groundwater, will need to be minimized. Additionally, leakage of the detained water, which could possibly lead to offsite seepage, will need to be minimized. Thus, HSe has indicated that the reservoir will likely need to be lined.

The proposed embankment dam will consist of an earth or rock-fill dam structure. The dam will likely be constructed of native materials excavated from cuts or borrow areas within the project. Given the topography of the canyon at the Site and the storage requirements, the embankment dam and reservoir is expected to have the following characteristics:

- Reservoir Surface Area: 2.84 acres
- Dam Footprint Area: 1.13 acres
- Maximum Dam Crest Height: 75 feet
- Minimum Freeboard: 10 feet
- Maximum Pool Elevation: 865 feet

The preliminary dam location and reservoir pool area are shown on the Site Plan/Geologic Maps, Figures 2a and 2b. This preliminary configuration was developed by HSe to meet the storage requirements and to locate the dam wholly within the parent parcel. The dam will likely include an overflow spillway, a pump station and associating piping for conveyance of the reclaimed water.

If the project Site is acquired in Trust for the Ione Band, the proposed embankment dam would be on Trust Land and would be on the United States Bureau of Indian Affairs (BIA) inventory of dams. Therefore, the facility must be designed and constructed in accordance with the U.S. Bureau of Reclamation (USBR) and/or U.S. Army Corps of Engineers requirements and other applicable California Division of Safety of Dams (DSOD) requirements.

3.0 SITE OBSERVATIONS, GEOLOGY, AND HYDROGEOLOGY

3.1 Field Observations

Geocon performed a field reconnaissance of the Site on July 29, 2004, and excavated exploratory test pits at the Site on August 3, 2004. The reconnaissance was supported by review of a topographic site plan (provided by HSe), review of the referenced geologic literature and maps, and stereoscopic aerial photograph analysis. Features noted during the site reconnaissance are indicated on the Site Plan/Geologic Maps, Figures 2a and 2b.

At the time of our reconnaissance, the drainage within the bottom of the canyon was dry. The sideslopes of the canyon were covered with a thick growth of annual grasses, brush and trees. The canyon walls are blanketed with a thin soil cover. In-place soil that has weathered from the parent rock is termed "residual soil", while soil that has experienced gravity-driven deposition or movement is termed "colluvium." For the purposes of this report, we refer to both types generally as "residual soil." Soil within the channel of the drainage that has been deposited by the action of water is termed "alluvium." The residual soil and alluvium are not mapped on the Site Plan/Geologic Maps. The thickness of the residual soil varied from non-existent to approximately 2 to 3 feet, based on our observations. In areas devoid of residual soil, exposures and outcrops of bedrock were noted throughout the Site. We did not observe springs or seeps within the canyon at the Site. Deep seated slope failures or evidence of landsliding were also not observed. However, localized soil creep was observed within the residual soil cover blanketing the canyon sidewalls. The creep was evidenced by the presence of several "pistol butted" trees.

Three mining adits and associated mine tailings were noted within the reservoir pool area. The approximate locations of the adits are mapped on the Site Plan/Geologic Maps, Figures 2a and 2b. The adits were caved to the entrance. Based on the size of the tailings piles adjacent to the adits, the mines are likely not very extensive. AES has indicated that these features are not considered historic structures. The abandoned Pioneer Mine was observed on the crest of a ridge approximately 1,200 feet north-northwest of the Site. HSe has indicated that there is a lateral shaft that extends from the Pioneer Mine into the canyon, upstream from the Site and not within the proposed reservoir pool area.

3.2 Aerial Photograph Interpretation

Analysis of stereoscopic aerial photographs did not detect evidence of landsliding at the Site. A prominent lineation corresponding to the main mapped trace of the Melones Fault is visible within the Dry Creek drainage located to the east-northeast of the Site. The abandoned Pioneer Mine was noted on the crest of a ridge approximately 1,200 feet north-northwest of the Site. Several elongated piles of tailings from the mine trail down gullies leading to the drainage upstream from the project Site.

3.3 Regional Geology

The Site is located within the foothills of the Sierra Nevada geomorphic province. Bedrock within the region ranges in age from Paleozoic to Cenozoic and includes sedimentary, igneous and metamorphic rocks. Numerous faults trend generally northwesterly through the area. The dominant group of faults belongs to the Foothills Fault System. In the vicinity of the Site, the Foothills Fault System consists of the Bear Mountains Fault Zone on the west and the Melones Fault Zone on the east. Portions of the Foothills Fault System are considered to be potentially active. Further discussion of faulting and seismicity is included in Section 4.1 of this report. The Site is located within the "Mother Lode" belt of the Sierra Nevada. This belt is notable for containing large deposits of precious metals and has been mined extensively.

3.4 Site Soil and Geologic Conditions

The general soil and rock types observed during the reconnaissance and field investigation include: fill material, alluvium, residual soil and slate rock identified by the California Geological Survey (CGS) as the Mariposa Formation. The materials are described below:

3.4.1 Fill Material

As previously discussed, fill material has been placed in the bottom of the canyon within the proposed footprint of the embankment dam. The approximate lateral extent of the fill material is shown on the Site Plan/Geologic Map, Figure 2a. We excavated an exploratory test pit (TP1) within the central portion of the fill to evaluate the thickness and consistency. The fill generally consisted of excavated and weathered Mariposa Formation (local bedrock). The material consisted of slightly clayey, silty, sandy angular gravel with cobble-sized, angular rock fragments. The relative density of the fill material is generally loose as evidenced by the instability (caving) of trench sidewalls between depths of approximately 5 to 12 feet below the ground surface. The fill was placed directly over the alluvium within the bottom of the drainage. The maximum thickness of the fill observed within our excavation was approximately 12 feet.

3.4.2 Alluvium

A layer of alluvium approximately 1.5 feet thick was encountered beneath the fill material within the bottom of the canyon. The alluvium generally consisted of sandy, silty clay (CL) with gravel and cobble-sized rock. The alluvium also contained varying concentrations of plant rootlets and decomposed plant organic material.

3.4.3 Residual Soil

Long-term weathering of the parent rock has produced a thin mantle of residual soil within the upland portions of the Site (canyon walls and beyond). The residual soil generally consists of loose to stiff, sandy, silty clay (CL) with gravel. The residual soil also contains varying amounts of plant roots and other decomposed plant organic material. The thickness of the residual soil varies from approximately 6 inches to 4.5 feet within our exploratory test pits. Locally, the residual soil grades downward into intensely weathered friable rock, approximately 1 to 2 feet thick, before less weathered rock is encountered. The gradational contact between the residual soil and the underlying Mariposa formation generally parallels the natural topography.

3.4.4 Mariposa Formation

We encountered bedrock in each of the six exploratory test pits excavated at the Site. Bedrock was also exposed within road cuts at the Site. Based on mapping by the CGS, the bedrock is composed of Jurassic age Mariposa Formation. These rocks consist of dark brittle slate that in some cases has been moderately fractured. Based on laboratory testing, pulverized rock material (powder) consists of sandy lean clay (CL) and sandy silt (ML). The beds have been folded and as a result strike and dip attitudes vary considerably. Measured strike and dip attitudes within the proposed dam and reservoir location are noted on the Site Plan/Geologic Maps, Figures 2a and 2b. In addition, fractures with a variety of attitudes are also present. Vein quartz was observed as float on the surface of the unpaved roads at the Site. The upper 1 to 2 feet of rock is generally severely weathered and is excavatable with a standard backhoe excavator. Below this depth, the rock becomes less weathered as evidenced by excavation refusal conditions.

3.5 Surface Water and Groundwater

No surface water, groundwater, springs or seeps were observed at the Site at the time of our reconnaissance and field exploration. In December 2003, Applied Engineering and Geology (AEG) performed a site reconnaissance to identify springs or seeps within the parent parcel. AEG identified eight springs throughout the parent parcel. AEG did not identify any springs within the drainage of the Site. HSe performed a similar wet weather reconnaissance to identify springs or seeps. HSe has indicated that numerous springs and seeps are present at different elevations within the project area during precipitation events. Based on the geologic conditions at the Site, these springs and seeps are likely the result of precipitation traveling along the contact between the residual soil and bedrock or within the bedding planes or fractures of the bedrock.

One water well (identified as DW-1 in the referenced AEG report) was identified on the parent parcel. The well is located approximately 3,100 feet northwest of the Site at an approximate elevation of 1,082 feet. The total depth and completion details of the well were not identified. AEG indicates in their report that the groundwater level in the well varies from approximately 38 to 43 feet below the ground surface. This indicates a potentiometric groundwater surface elevation of approximately 1,044 feet at the well location. AEG also reports that springs within the eastern portion of the parent parcel are located below an elevation of 1,000 feet. Therefore, AEG concludes that the areal direction of groundwater flow is toward Dry Creek, southeast of the Site.

4.0 GEOLOGIC HAZARDS

4.1 Faulting

Based on the site reconnaissance and review of geologic maps, reports and interpretation of stereoscopic aerial photographs, the Site is not located on any known active or potentially active fault traces. In addition, the Site is not contained within a Special Studies Earthquake Fault Zone (formerly referred to as an Aliquist-Priolo Special Studies Zone). In order to determine the distance of known faults to the site, the computer program *EQFAULT*, (Blake, 2000), was utilized. Principal references used within *EQFAULT* in selecting faults to be included are Jennings (1975), Anderson (1984) and Wesnousky (1986). In addition to fault location, *EQFAULT* was used to deterministically estimate peak ground accelerations at the Site for the Maximum Considered Earthquake (MCE). The MCE is defined as the maximum earthquake that is possible of occurring under the presently known tectonic framework.

The Site is located between the Bear Mountains Fault Zone on the west and the Melones Fault Zone on the east. The Bear Mountains Fault Zone is located approximately 3.7 miles west of the Site and the Melones Fault Zone is located approximately 2,500 feet east of the Site. These fault zones with numerous splays and traces make up the Foothills Fault System. Deterministic estimation of peak site acceleration for the MCE models the Foothills Fault System as an areal source considering potential activity from the Bear Mountains Fault Zone as well as the Melones Fault Zone. Table 4.1 summarizes the estimated moment magnitudes (M_W) and peak site accelerations based on attenuation relationships developed by Sadigh et al (1997) for the faults considered most likely to contribute to ground shaking at the Site.

TABLE 4.1
MAXIMUM EARTHQUAKE MAGNITUDE AND PEAK SITE ACCELERATION

Fault Name	Distance From Site (miles)	Maximum Earthquake Magnitude, M_W	Peak Site Acceleration (g)
Foothills Fault System	0.5 to 3.7	6.5	0.51
Mohawk – Honey Lake Zone	54.9	7.3	0.04
Great Valley 5	55.7	6.5	0.02
Genoa	57.9	6.9	0.03
Great Valley 4	58.0	6.6	0.02
Great Valley 6	58.5	6.7	0.03

The estimated peak site accelerations shown in Table 4.1 are *deterministic* estimates of the MCE ground motion, which is defined to be the level of ground motion with a **2% chance of exceedance in 50 years** with a statistical return period of approximately 2,475 years (International Building Code 2000, §1615.2.1). Thus, these estimated ground motions are more conservative than the probabilistic ground motions used for design of essential services buildings in California. These structures are designed to resist collapse when subjected to the Upper Bound Earthquake (UBE) ground motion which is defined to have a **10% chance of exceedance in 100 years** with a statistical return period of approximately 949 years. Design of other structures in California are based on a less conservative level of ground motion termed the Design Basis Earthquake (DBE) which is defined as having a 10% chance of exceedance in 50 years with a statistical return period of approximately 475 years. The estimated DBE acceleration at the Site is approximately 0.11g (CGS PSHA Model). The dam designer should determine the appropriate level of ground motion for design.

While listing peak accelerations is useful for comparison of potential effects of fault activity in a region, other considerations are important in seismic design, including the frequency and duration of motion and the soil conditions underlying the Site. The Site could be subjected to moderate ground shaking in the event of a major earthquake along the faults mentioned above or other area faults. However, the seismic risk at the Site is not considered to be significantly greater than that of other developments in Amador County.

4.2 Liquefaction

Liquefaction of granular soils can be caused by strong vibratory motion due to earthquakes. Soils that are highly susceptible to liquefaction are loose, granular and saturated. The Site is underlain by dense, hard rock at relatively shallow depths. Consequently, the potential for liquefaction at the Site is negligible.

4.3 Slope Stability and Landsliding

Soil "creep" was observed within the residual soil cover blanketing the canyon sidewalls. The creep was evidenced by the presence of several "pistol butted" trees. The creep is the result of gravity-driven colluvial movement of the thin residual soil veneer on top of the Mariposa Formation bedrock. Deep seated slope failures or evidence of landsliding was not observed at the Site or on aerial photographs of the Site.

Although landsliding does not appear to be currently occurring at the Site, stable conditions may be changed by slope alterations due to cuts or fills, saturation of slope materials from impounded water and/or changes to drainage patterns. Variable and erratic strike and dip attitudes of the folded bedrock suggest that adverse bedding conditions (out-of-slope bedding orientations) may be present within planned excavations. However, given the extensive folding of the bedrock, such conditions are likely localized and are not expected to create large-scale landslide hazards.

5.0 CONCLUSIONS AND RECOMMENDATIONS

The following discussion, conclusions and recommendations are based on the results of our reconnaissance, field exploration and experience with similar soil and geologic conditions. The conclusions and recommendations are preliminary and are intended for planning purposes only. Once the location of the embankment dam and reservoir are determined, a detailed, design-level geotechnical investigation will need to be performed. Recommendations for the design-level geotechnical investigation are presented herein.

5.1 Soil, Geologic and Groundwater Conditions

- The Site is underlain by fill material, residual soil, alluvium and Mariposa Formation bedrock.
- The maximum thickness of the fill material (at the time of our reconnaissance and field exploration) is approximately 12 feet. The fill soil is in a relatively loose condition and overlies alluvial material with a significant organic content.
- The alluvium within the canyon bottom is approximately 1 to 2 feet thick. The alluvium contains gravel and cobble-sized rock and a significant organic content.
- The residual soil variably blankets the upland portions of the Site and ranges up to a thickness of approximately 5 feet.
- In general, the soil cover at the Site is relatively thin and, therefore, bedrock is relatively close to the ground surface. We consider this condition to be favorable for constructing an embankment dam at the Site.
- Based on the geology of the Site and parent parcel, suitable material, either soil or sound rock, for construction of an embankment dam should be available onsite or close to the Site. However, the onsite material is generally more silty than clayey, therefore, the material will likely not be suitable for "impervious" portions of the embankment dam. Geotechnical investigation and laboratory testing of proposed borrow areas will be necessary to identify suitable materials.
- Surface water, groundwater, springs or seeps were not observed during our site reconnaissance and field exploration. However, these conditions have been observed by others during wetter parts of the year and should be expected at the Site. These conditions will impact the design and operation of the embankment dam and reservoir facility, specifically the reservoir liner system, if required.
- The Site may be subjected to moderate ground shaking as a result of an earthquake on regional active faults. It is our opinion that an embankment dam can be designed and constructed to withstand the anticipated seismic conditions at the Site. The dam designer should determine the appropriate level of ground motion (deterministic MCE, UBE, DBE) for design of the dam structure in accordance with jurisdictional oversight authority.

5.2 Topography

The narrow canyon with high, rocky walls is considered well suited for construction of an embankment dam. There should be adequate room to construct haul roads and other temporary construction facilities close to the proposed dam footprint. Since the reservoir rim is high and unbroken, a chute or tunnel spillway may be necessary.

5.3 Embankment Dam – General Design Considerations

5.3.1 Embankment Design – Type and Materials

The two principal types of embankment dams are earth and rock-fill dams, depending on the predominant fill material used. In general, earth dams can be constructed on weaker foundation materials while rock-fill dams require a more competent foundation material. Based on the anticipated onsite materials available, either type may be constructed. Since the onsite materials likely do not contain a significant portion of clay fines, an earth or rock-fill dam with an impervious core consisting of imported impervious material will likely be necessary. Ultimately, the dam designer should determine the type of dam based on future design-level studies at the Site.

5.3.2 Foundation and Abutment Areas

Proper preparation of the foundation and abutment areas is critical to the performance of the embankment dam structure. The following recommendations are considered applicable to the Site. Additional foundation and abutment area characterization will be necessary to determine the strength properties of the rock materials.

- The fill, alluvium and residual soil are not considered suitable foundation materials for the embankment dam. The materials should be completely removed within the footprint of the embankment. With the exception of the fill material, the depth or thickness of these materials is expected to be on the order of 5 feet or less. Localized areas could be thicker.
- The upper, weathered portion of the Mariposa formation should be removed to expose sound, relatively unweathered bedrock. Based on the conditions observed within our exploratory test pits, the depth of the weathered portion of rock is on the order of 1 to 2 feet thick. However, localized areas could be thicker.
- The exposed rock surface should be cleaned of all loose fragments, including semi-detached surface blocks of rock spanning relatively open crevices. Projecting knobs of rock should be removed to facilitate operation of compaction equipment and to avoid differential settlement.
- Cracks, joints, and openings should be filled with mortar or lean concrete according to the width of opening. The treatment of rock defects should not result in layers of grout or gunite that cover surface areas of sound rock, since they might crack under subsequent fill placement and compaction operations.
- Some adverse bedding conditions may be exposed within foundation and abutment excavations. If these conditions are exposed, additional pinning and grouting may be necessary.
- A cut-off trench to control under-seepage may be necessary depending on the design of the dam. The cutoff trench would likely be excavated 4 to six 6 into the relatively unweathered portion of the Mariposa formation. The width of the cut-off trench is typically half the height of the dam, with a 14-foot minimum (DSOD).

5.3.3 Embankment Design - Geometry

- If the total height of the dam is on the order of 75 feet, the top width of the dam should be between 25 and 45 feet (U.S. Army Corps of Engineers). Narrower top widths may be suitable, if approved by the dam designer.
- Assuming an earth or rock-fill dam with an impermeable core, the upstream embankment shell should be inclined at 3:1 (horizontal to vertical) or flatter. The downstream embankment shell may be inclined at 2:1 or flatter. These inclinations are considered conservative estimates for planning purposes. Final inclinations should be based on reservoir design operating conditions, material source laboratory test results and detailed slope stability analyses. Other conditions, such as required widths of the core, filter and transition zones may dictate flatter slopes.

5.4 Surface Water Diversion, Seepage/Leakage Control

HSe has indicated that in order to reduce the capacity requirements of the reservoir and the ultimate disposal quantity of water, the inflow of surface and subsurface water should be minimized. In addition, subsurface leakage, which could lead to offsite seepage, should be avoided. In order to achieve these criteria, surface water within the drainage, upstream from the reservoir will need to be diverted and the reservoir will need to be lined. The liner system would need to be designed to both prevent groundwater seepage from entering the reservoir and prevent reclaimed water within the reservoir from leaking out. It is our opinion that, given the site topography, soil and geologic conditions, constructing and maintaining a suitable reservoir liner will be extremely difficult. The following sections address these issues.

5.4.1 Surface Water Diversion

During wet weather periods, the canyon at the Site can carry a significant volume of surface water. To avoid filling the reservoir with this water, it would need to be diverted upstream from the reservoir. A likely solution is to construct an upstream headwall and longitudinal culvert that would extend down the length of the reservoir and daylight below the dam. The headwall and culvert would need to be designed to convey the maximum anticipated flow (based on hydrologic modeling) within the canyon during a design storm event. Additionally, surface water runoff that sheetflows into the reservoir from the perimeter slopes could be collected in a perimeter french drain system that is connected to the longitudinal bottom culvert drain. Such a drainage system would require cleanouts and other features that would allow for cleaning and maintenance. It is our opinion that, if designed properly (including redundant facilities) and carefully constructed, such a system is feasible.

5.4.2 Reservoir Lining

Two basic lining technologies commonly utilized for reservoirs include (1) compacted clay liners (CCLs) and (2) geosynthetic liners. Geosynthetic liners include a variety of materials and technologies designed to meet site-specific conditions. It is our opinion that lining the reservoir with either technology will be very difficult for the following reasons:

- Constructing a liner system within the reservoir pool area would require thorough clearing, grading and smoothing of the reservoir bottom and canyon sideslopes. In some areas within the reservoir pool area, the sideslopes are inclined at a gradient of 1:1 or steeper. At this steep inclination, grading equipment would need to be tethered from the top during grading operations. These conditions may prove to be unsafe and uneconomical.
- Construction of CCLs is generally limited to side slopes of 2:1 or flatter. At steeper inclinations, it becomes virtually impossible to place a uniform layer of clay soil and perform proper compaction. Additionally, since the reservoir will be empty for the majority of the year, especially the hot, dry summer season, the CCL would be subject to severe damage from desiccation.
- Although geosynthetic liners can be constructed on steeper slopes, these systems are subject to installation/puncture damage and long-term ultraviolet (UV) degradation. Therefore, the surface to be lined needs to be meticulously graded smooth, which will likely be very difficult at the Site. For UV protection, the geosynthetic liner would need to protected with a soil or vegetated cover. Construction of such a cover would be subject to the same installation difficulties as a CCL.
- Either liner system would need to be underlain by a drainage layer to collect any groundwater seepage and direct it to a central subdrainage system for collection. If a drainage layer is not provided, the liner would be subject to localized hydraulic uplift forces (when the reservoir is empty) that could damage or "blow out" portions of the liner. The drainage layer could consist of a lined crushed rock system or a prefabricated geosynthetic drainage composite material. The crushed rock system would be subject to the same installation difficulties as a CCL. The geosynthetic composite would be difficult to anchor to the steep slopes and would be subject to installation damage.

Based on the above discussion, we recommend that the reservoir system be designed without a liner. If the surface water flow is diverted from the reservoir with a culvert and french drain system as discussed above, the additional contribution of inflow from subsurface seepage is likely to be very small with respect to the capacity of the reservoir. Additionally, as the age of the reservoir increases, siltation will occur. This siltation will aid in "plugging" potential drainage paths that could cause offsite seepage.

If it is determined that a liner system is absolutely necessary, the most feasible technology is a geosynthetic liner system rather than a CCL. We recommend consultation with a geosynthetic designer with experience in reservoir design to develop a system to meet the physical and operational conditions of the Site.

5.5 Recommended Design-Level Geotechnical Investigation Scope

Due to the risks associated with the failure of a dam facility, the scope of the design-level geotechnical investigation is critical to produce a design that will result in a dam structure that is safe and stable during its entire life, including construction. Specific investigative recommendations are outlined in the referenced documents by the Bureau of Reclamation, U.S. Army Corps of Engineers, and DSOD. Based on our knowledge of the Site, we recommend the following general scope of services.

5.5.1 Field Investigation and In-Situ Testing

A detailed subsurface investigation program including in-situ testing is required to determine strength properties of the foundation and abutment materials and to determine required foundation treatment. In addition, the borrow sources need to be fully evaluated for strength and quality of material. At a minimum, the field investigation should include the following:

- Detailed geologic mapping within the dam footprint and reservoir pool area. Exploratory trench excavations and large-diameter borings may be necessary to aid in geologic mapping. The mapping should attempt to identify surficial materials, geologic contacts and structural attitudes. Features such as rock jointing and other discontinuities should be measured with respect to orientation, continuity, spacing, roughness, mineral coating/joint infilling and other characteristics which may affect the shear strength of the rock.
- Exploratory adits in abutments, test pits, test trenches, large-diameter calyx holes, and large-diameter core boring to satisfactorily investigate foundation and abutment conditions. Exploratory borings (rock coring) within the foundation/abutment areas, canyon walls close to the dam, spillway and outlet channel locations. Coring will help identify foundation and abutment rock characteristics such as depth of bedding, solution cavities, fissures, orientation of joints, clay seams, gouge zones, and faults which may affect the stability of rock foundations and slopes, particularly in association with seepage. Rock Quality Designation (RQD) provides an assessment of the engineering qualities of bedrock. The exploratory excavation locations should be identified in cooperation with a DSOD representative or other applicable regulatory oversight official.
- In-situ water pressure testing (packer testing) to determine permeability characteristics of the foundation and abutment rock. This information will aid in designing the seepage control features of the embankment dam.
- Surface and downhole geophysical testing, including borehole photography, to further define subsurface stratigraphy and excavation characteristics.

5.5.2 Laboratory Testing

A broad suite of geotechnical laboratory testing is necessary to evaluate the index and engineering properties of the foundation and abutment materials as well as the borrow sources. Such testing would likely include (but not be limited to):

- In-situ Moisture/Density
- Maximum Density/Optimum Moisture
- Grain Size Distribution
- Atterberg Limits
- Aggregate Quality Tests: Durability Index, Friable Particles, Sulfate Soundness
- Corrosion Potential: pH, Sulfate Content, Chloride Content
- Triaxial Shear Strength, Unconsolidated Undrained (UU) and Consolidated Undrained (CU)

- Direct Shear
- Unconfined Compressive Strength of Rock

5.5.3 Engineering Analysis

The field and laboratory data should be analyzed to develop practical design recommendations for the embankment dam and reservoir with regard to:

- Borrow area characterization, selection of embankment materials, and properties of embankment materials
- Properties of foundation and abutment materials
- Excavatability
- Slope stability – embankment and reservoir side slopes, under static, seismic and rapid-drawdown conditions
- Seismic analysis and development of site-specific ground motion parameters in accordance with the level of ground motion selected by the dam designer and jurisdictional oversight authority
- Site preparation, clearing and grubbing
- Preparation of embankment foundations and abutments
- Design embankment sections: geometry, crest design, materials, filter design, slope protection
- Seepage control including cut-off trench details and toe drains
- Inlet/outlet works and control structures
- Spillway design
- Embankment construction, including material placement and compaction recommendations, moisture control and final finish grading

5.6 Other Recommendations

Based on the review comments of the BIA dated March 2, 2005, the following additional tasks should be part of design-level studies for the project:

- Further and continued monitoring of seasonal seeps and springs within the proposed reservoir pool area to confirm that they are the result of precipitation traveling along the contact between residual soil, and bedrock or within the bedding planes or fractures of the bedrock.
- Prepare a hydrologic study for the design of the drainage facilities at the Site, which could include open channels and culverts.
- Perform a Costs Evaluation to facilitate the selection of whether a reservoir liner system or a culvert and french drain system should be installed.

6.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

The analyses, conclusions, and recommendations contained in this report are based on site conditions as they existed at the time of our site reconnaissance and field investigation. This report is applicable only for the project and site studied.

Only cursory subsurface explorations consisting of excavating test-pits with a backhoe were performed as a part of this report. Therefore, all opinions, conclusions and recommendations presented herein are preliminary in nature and are based on the limited field reconnaissance and referenced published and unpublished literature and maps.

The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally accepted geological and geotechnical engineering principles and practices used in the Western Amador County area in August 2004. This warranty is in lieu of all other warranties, either expressed or implied.

APPENDIX A

FIELD INVESTIGATION

The field reconnaissance and investigation were performed on July 29 and August 3, 2004, respectively. The field investigation consisted of excavating six exploratory test pits (TP1 through TP6) at the approximate locations shown on the Site Plan/Geologic Maps, Figures 2a and 2b.

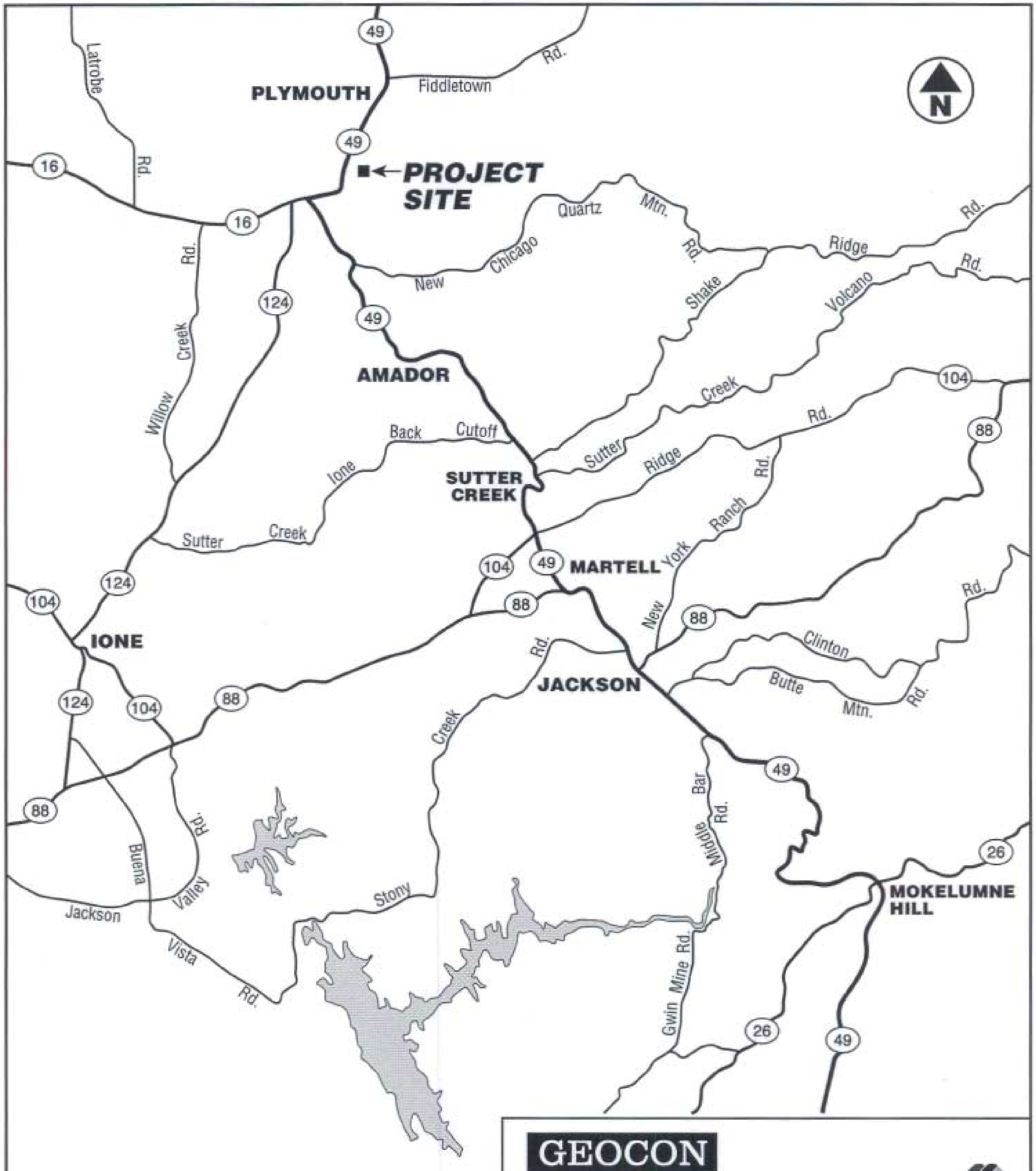
The test pits were excavated with a rubber tire John Deere 710 backhoe equipped with an 18-inch bucket. Bulk soil and rock samples were obtained from the test pits. Upon completion, the test pits were backfilled with the excavated material.

The soil conditions encountered in the exploratory test pits were visually examined, classified and logged in general accordance with American Society for Testing and Materials (ASTM) practice for Description and Identification of Soils (Visual-Manual Procedure D2844). Logs of the exploratory test pits are presented on Figures A1 through A6. The logs depict the soil and geologic conditions encountered and the depth at which samples were obtained.

APPENDIX B

LABORATORY TESTING

Laboratory tests were performed in accordance with generally accepted test methods of the American Society for Testing and Materials (ASTM) or other suggested procedures. Selected samples were tested for classification and plasticity characteristics. The laboratory test worksheets are included herein.



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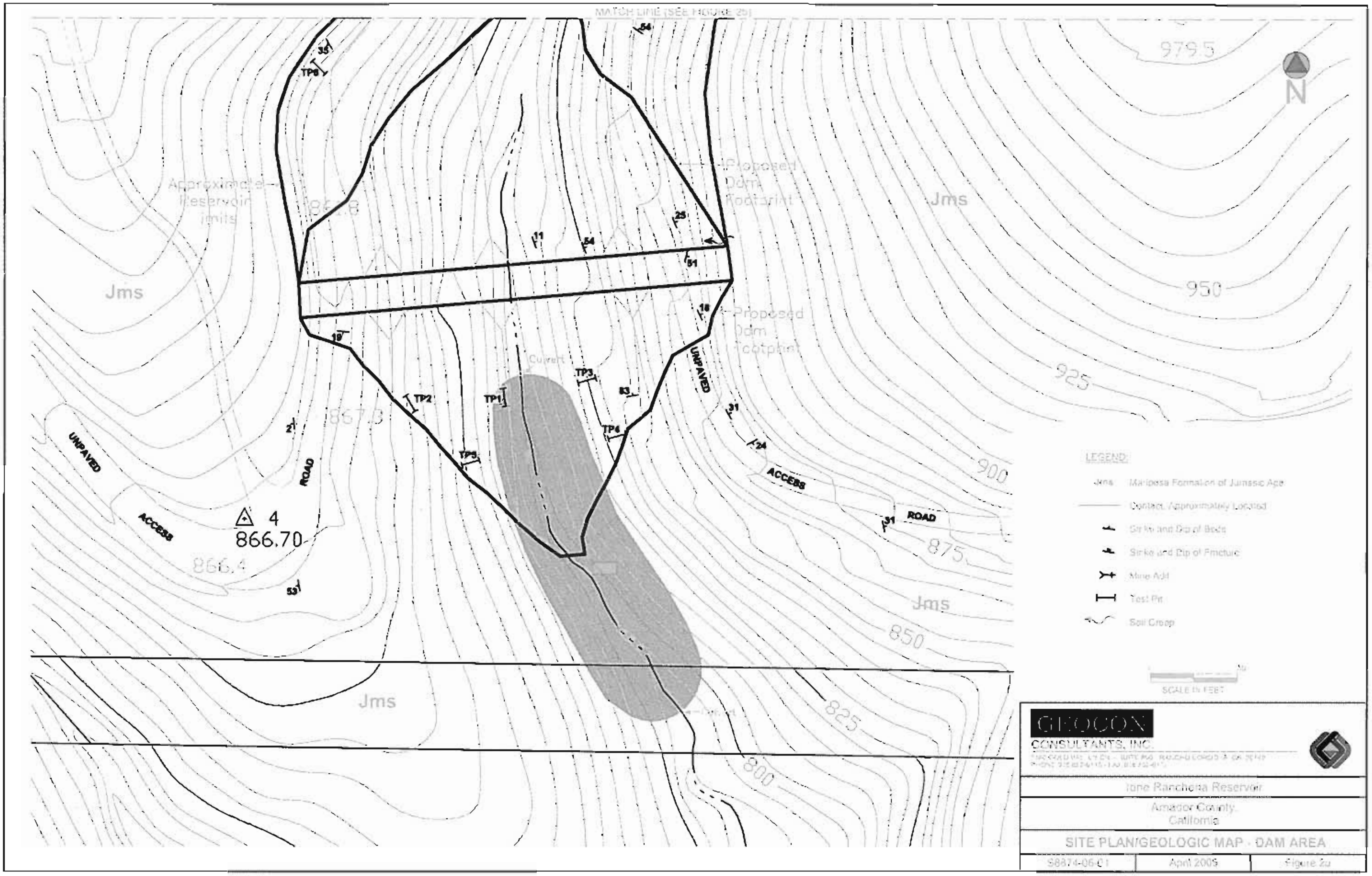
Amador County, California

VICINITY MAP

S8874-06-01	April 2005	Figure 1



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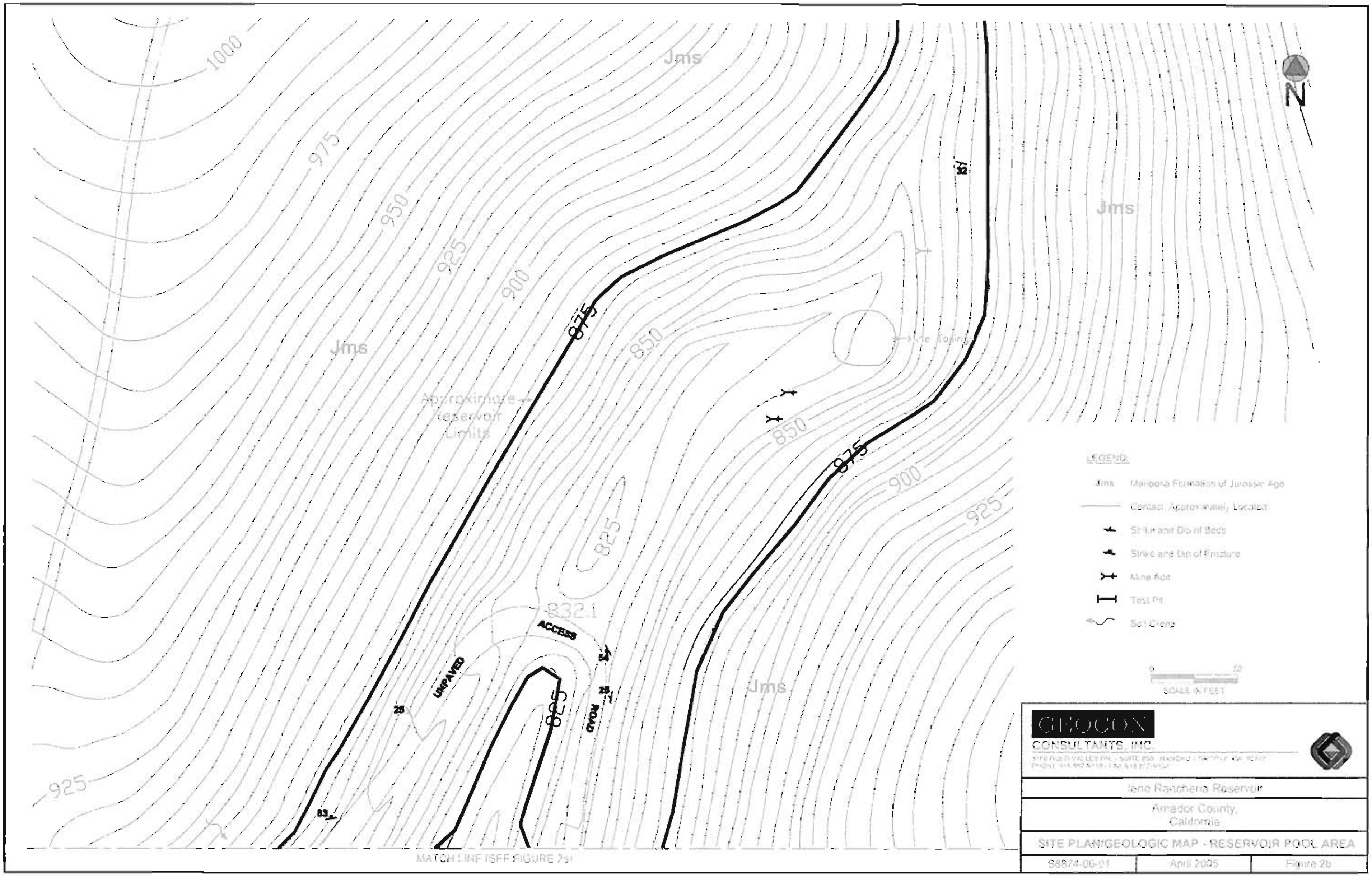
Amador County, California

SITE PLAN/GEOLOGIC MAP - DAM AREA

S8874-06-01	April 2005	Figure 2a


N
Jms
1000
975
950
925
900
875
850
825
832.1
Approximate Reservoir Limits
Mine Adit
UNPAVED
ACCESS
ROAD
25
63
MATCH LINE (SEE FIGURE 2a)
LEGEND:
Jms Mariposa Formation of Jurassic Age
Contact, Approximately Located
Strike and Dip of Beds
Strike and Dip of Fracture
Mine Adit
Test Pit
Soil Creep
0
50
SCALE IN FEET
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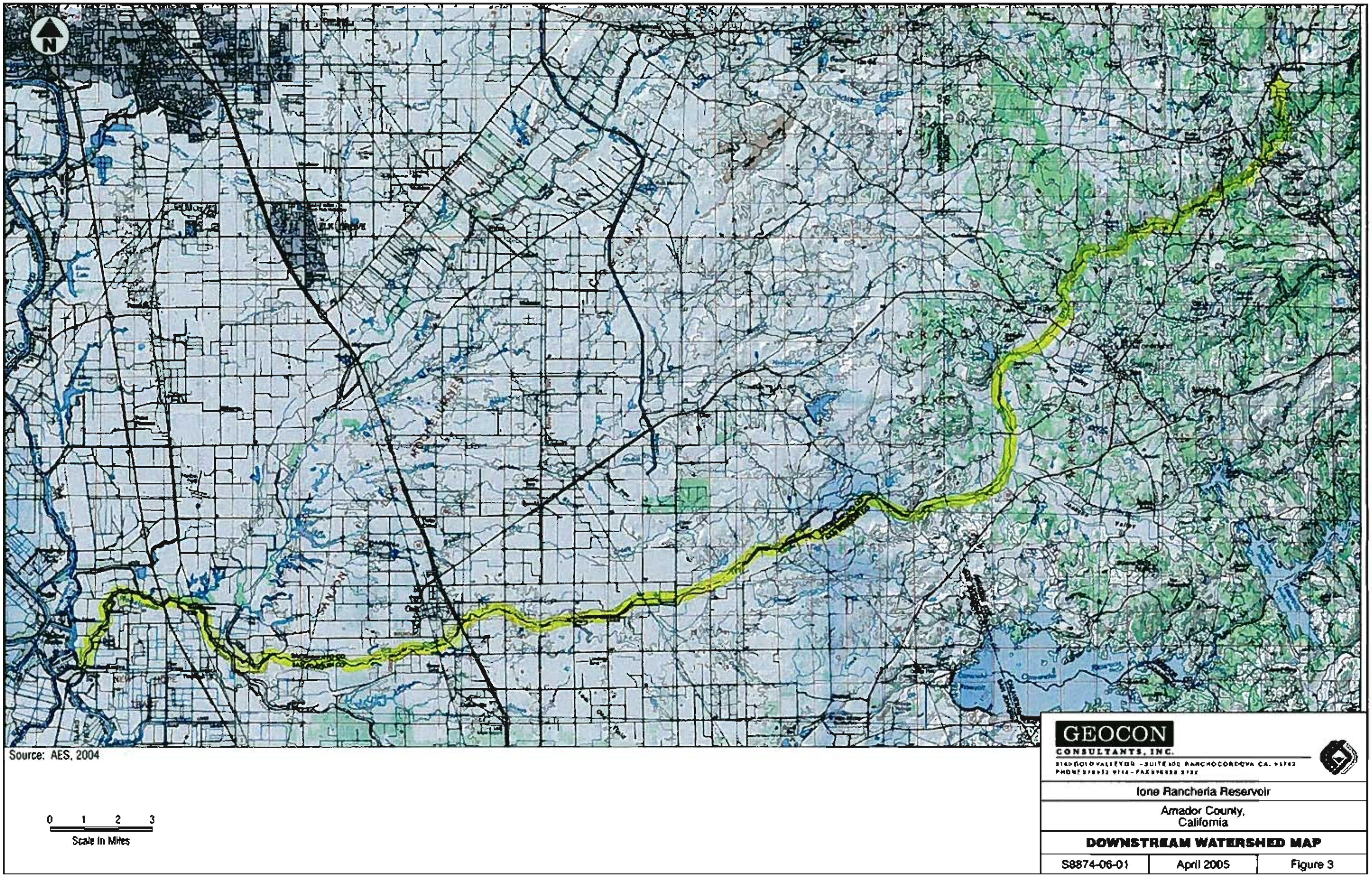
SITE PLAN/GEOLOGIC MAP - RESERVOIR POOL AREA
S8874-06-01
April 2005
Figure 2b



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Ione Rancheria Reservoir

Amador County, California

DOWNSTREAM WATERSHED MAP

S8874-06-01	April 2005	Figure 3

Photo No. 1 Unpaved Access Roads Traversing the Site

Photo No. 2 Fill Material Placed in Canyon at the Site (Looking Downstream)

SITE PHOTOS NO. 1 & 2

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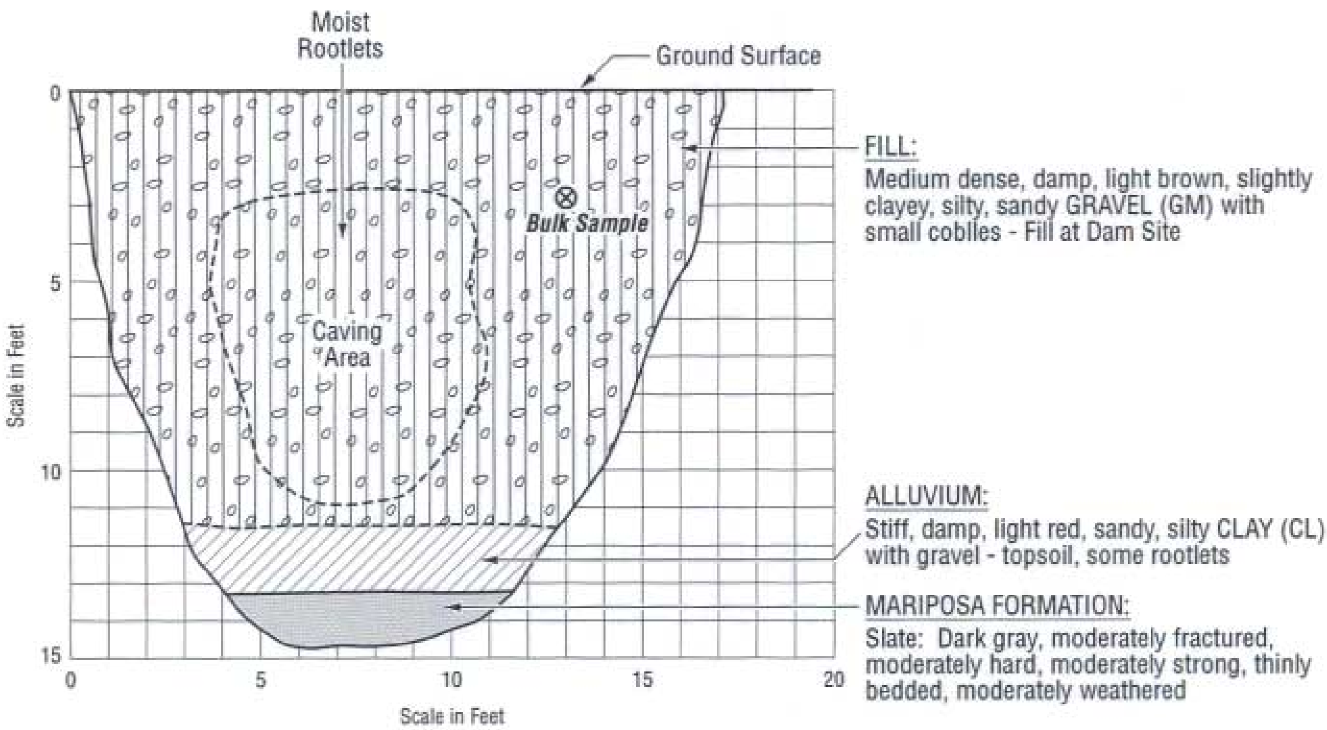
Ione Rancheria Reservoir		
Amador County, California		
S8874-06-01	April 2005	Figure 4

APPENDIX A

TP1
West Wall Profile



LEGEND:

⊗ Sample Location

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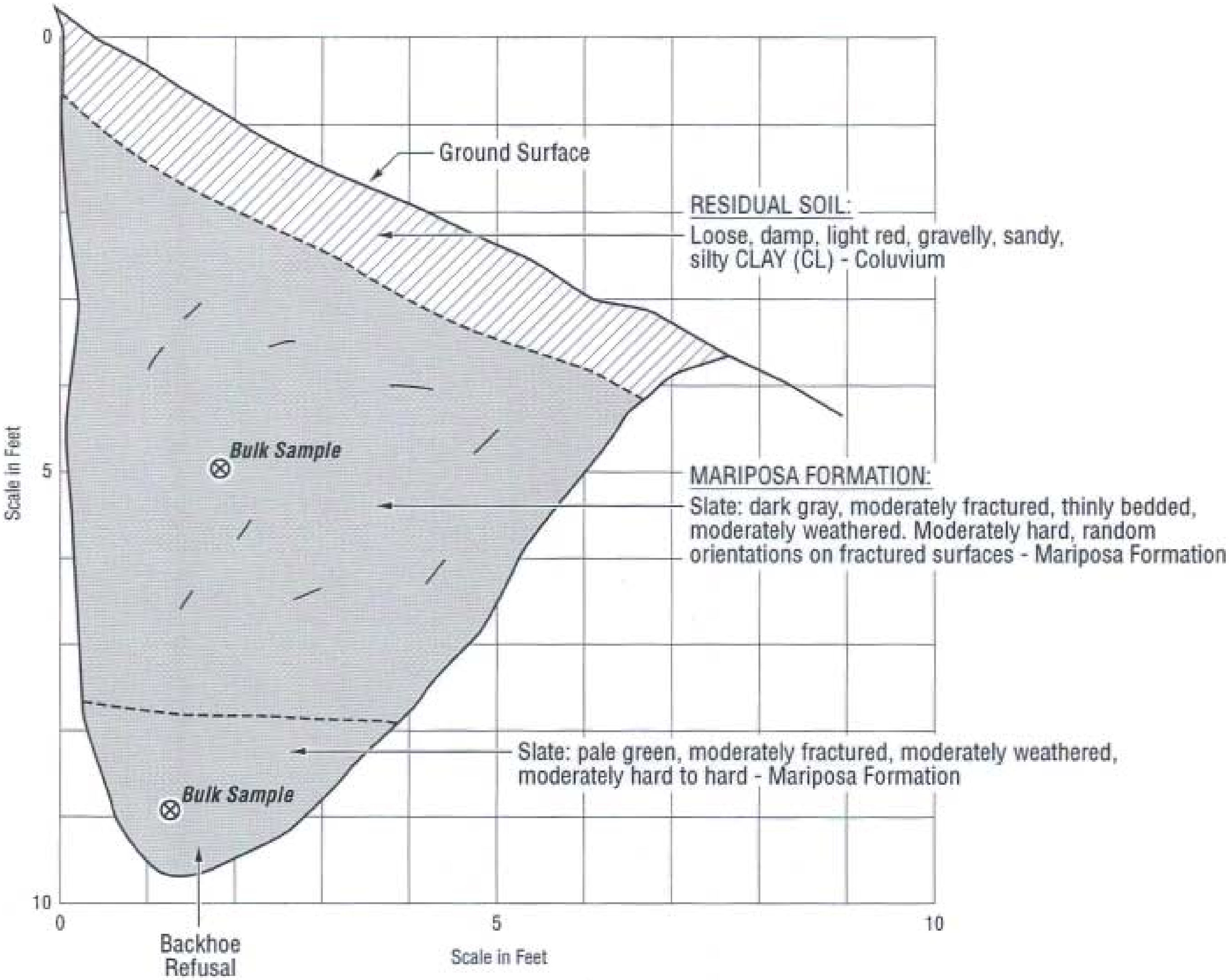
Ione Rancheria Reservoir		
Amador County, California		
TEST PIT TP1		
S8874-06-01	April 2005	Figure A1

TP2

Northeast Wall Profile



LEGEND:

⊗ Sample Location

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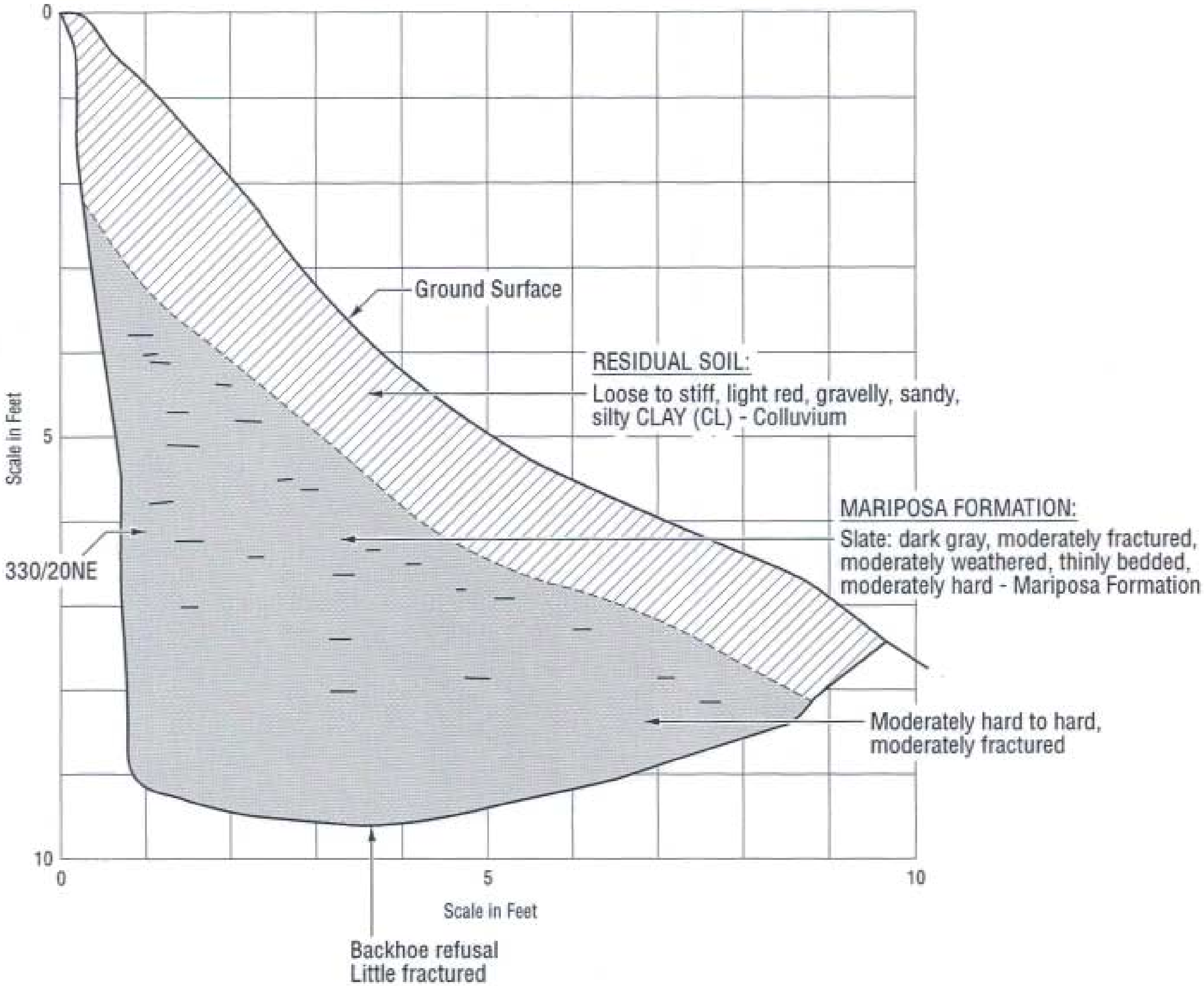
Ione Rancheria Reservoir		
Amador County, California		
TEST PIT TP2		
S8874-06-01	April 2005	Figure A2

TP3
Southwest Wall Profile



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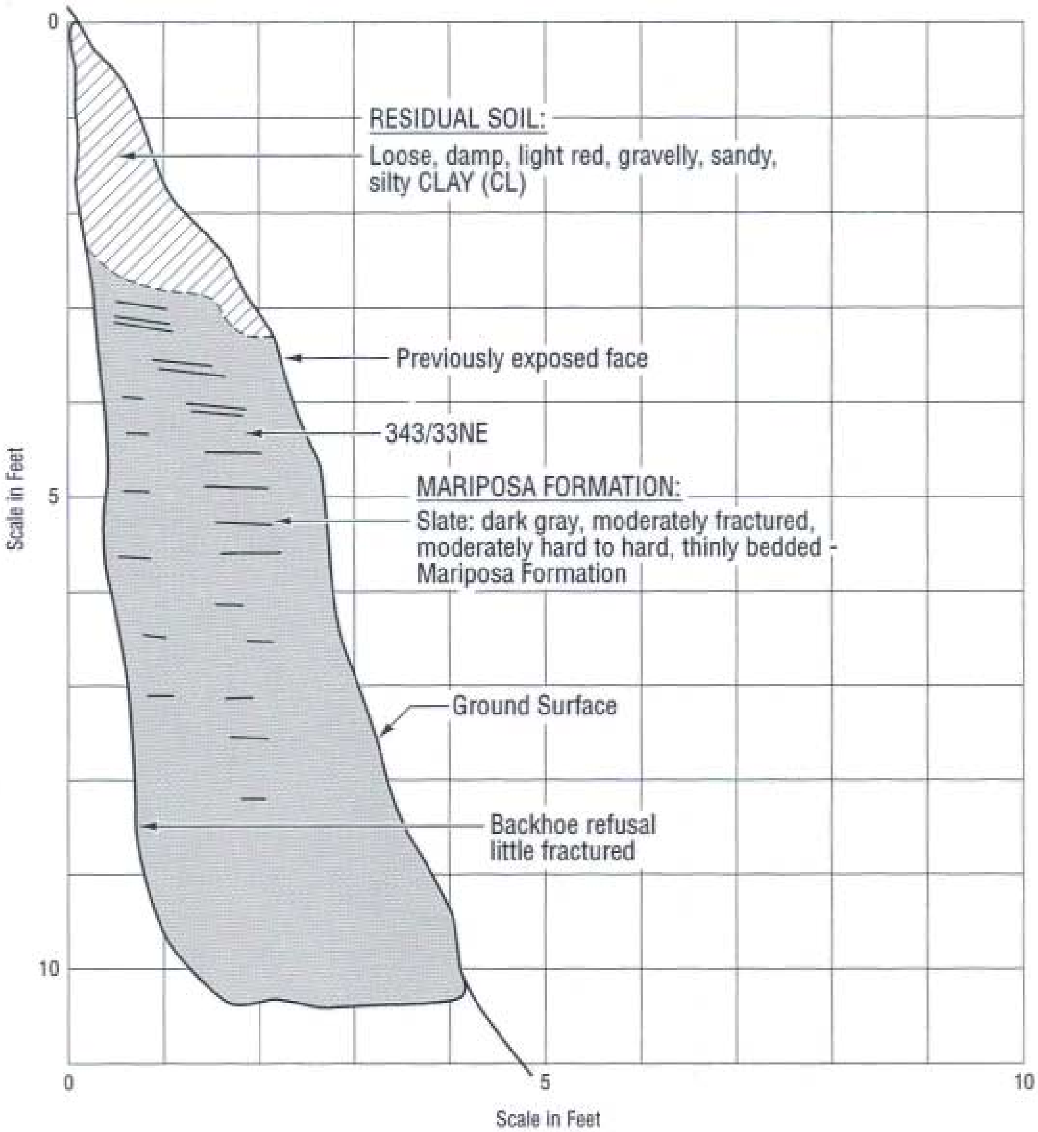
Ione Rancheria Reservoir		
Amador County, California		
TEST PIT TP3		
S8874-06-01	April 2005	Figure A3

TP4
Southwest Wall Profile



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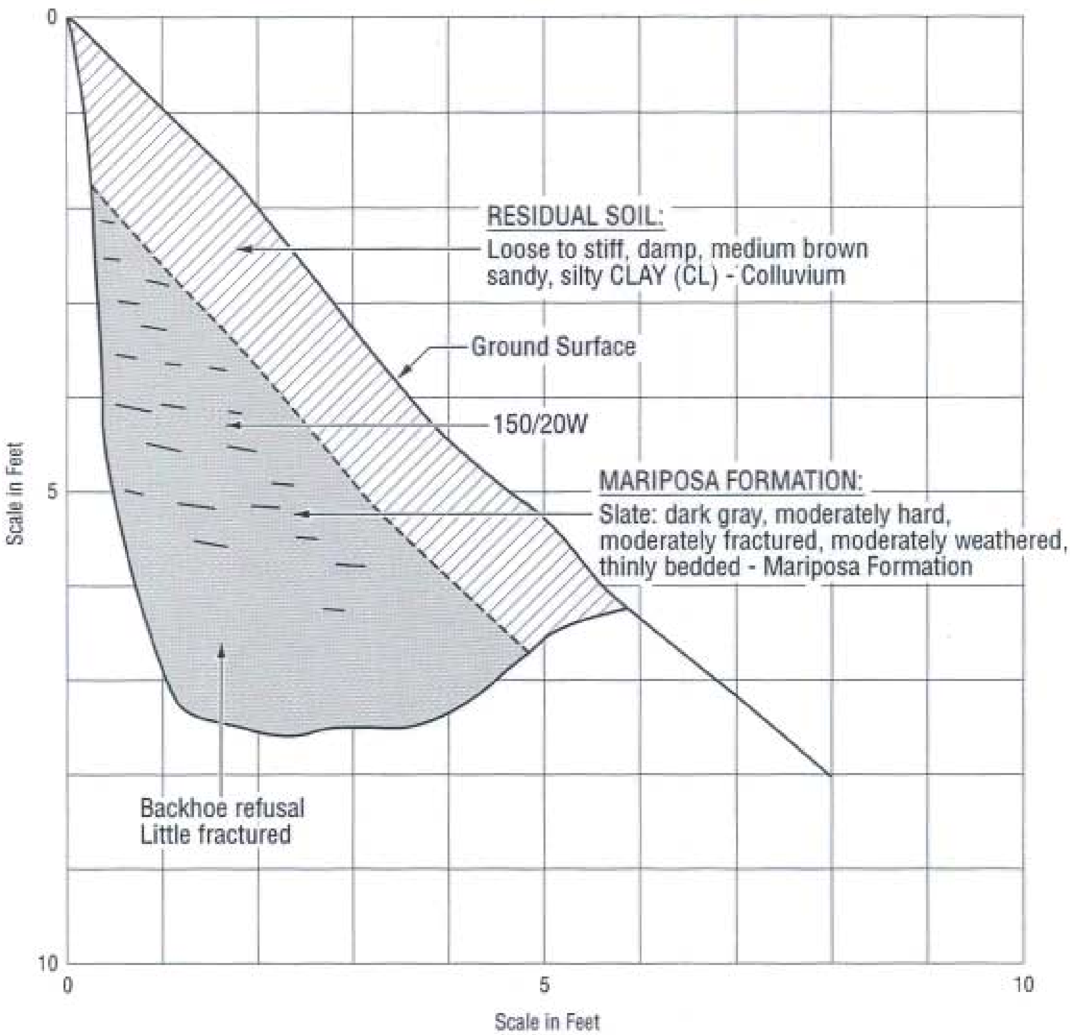
Ione Rancheria Reservoir
Amador County, California
TEST PIT TP4

S8874-06-01	April 2005	Figure A4

TP5

Northwest Wall Profile



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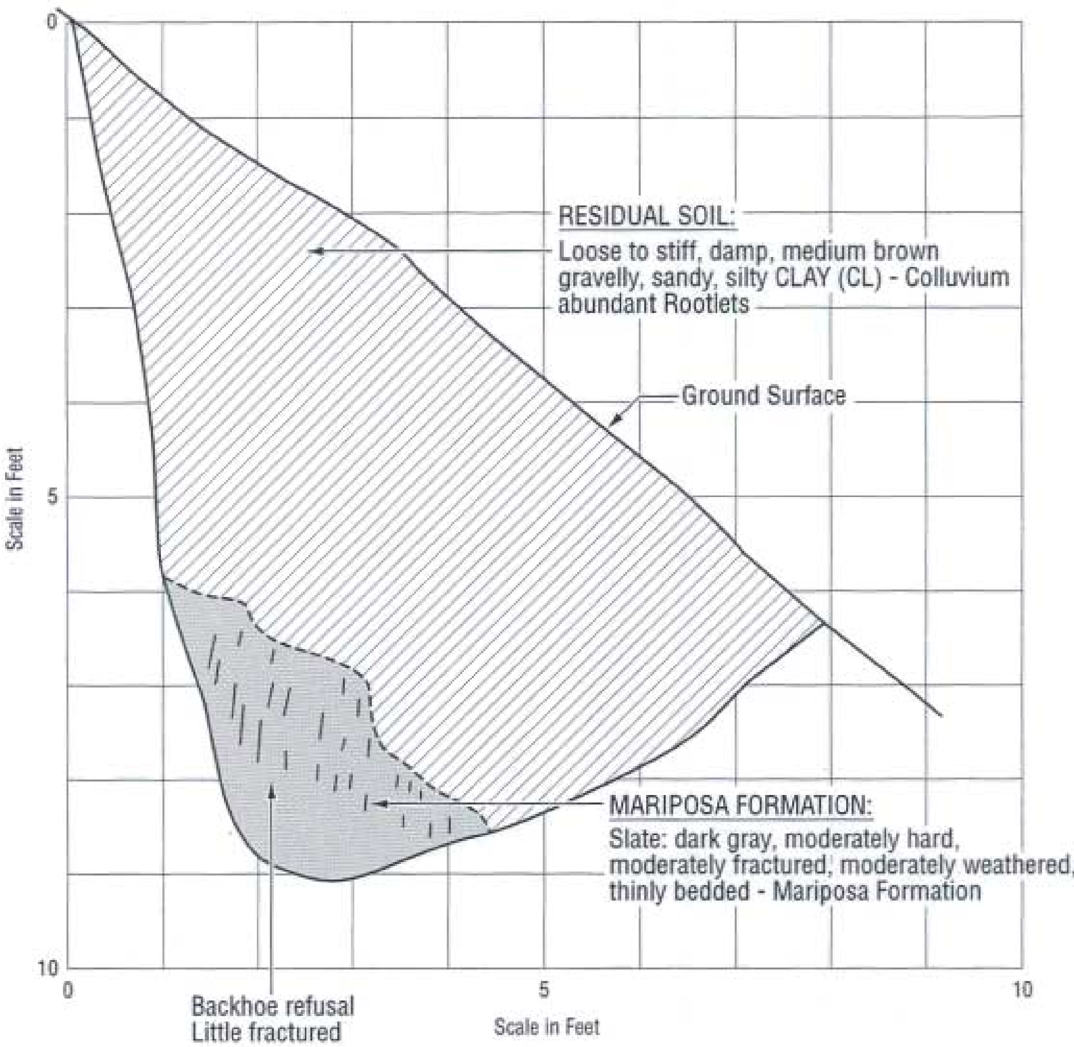
Ione Rancheria Reservoir		
Amador County, California		
TEST PIT TP5		
S8874-06-01	April 2005	Figure A5

TP6
Northeast Wall Profile



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Ione Rancheria Reservoir

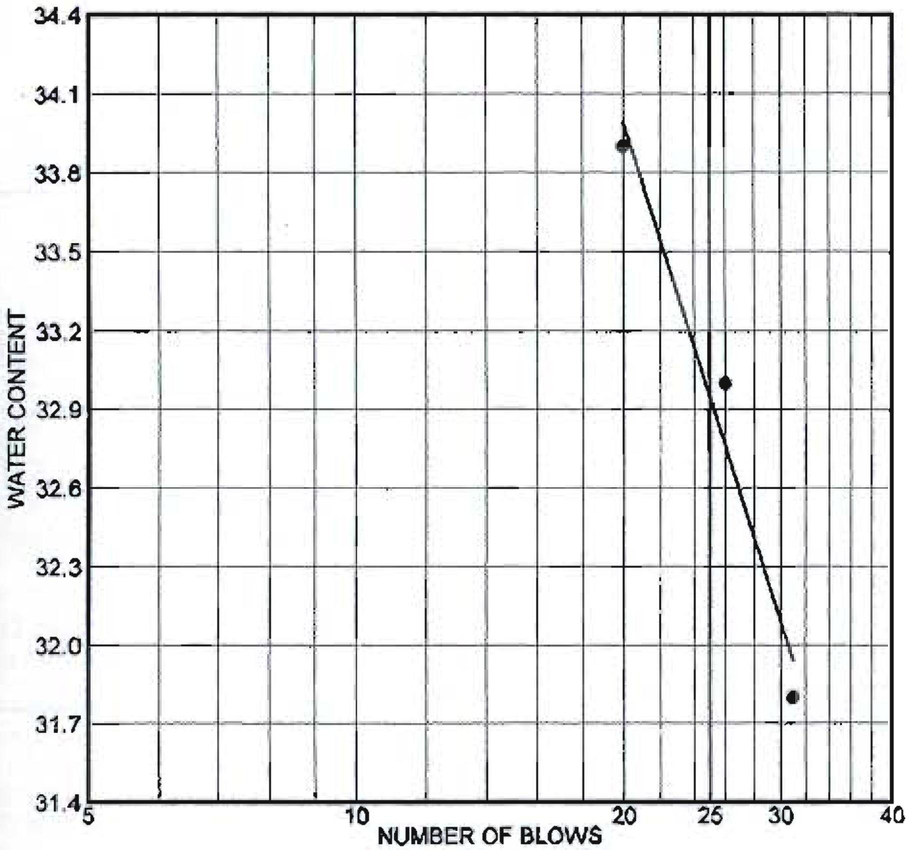
Amador County,
California

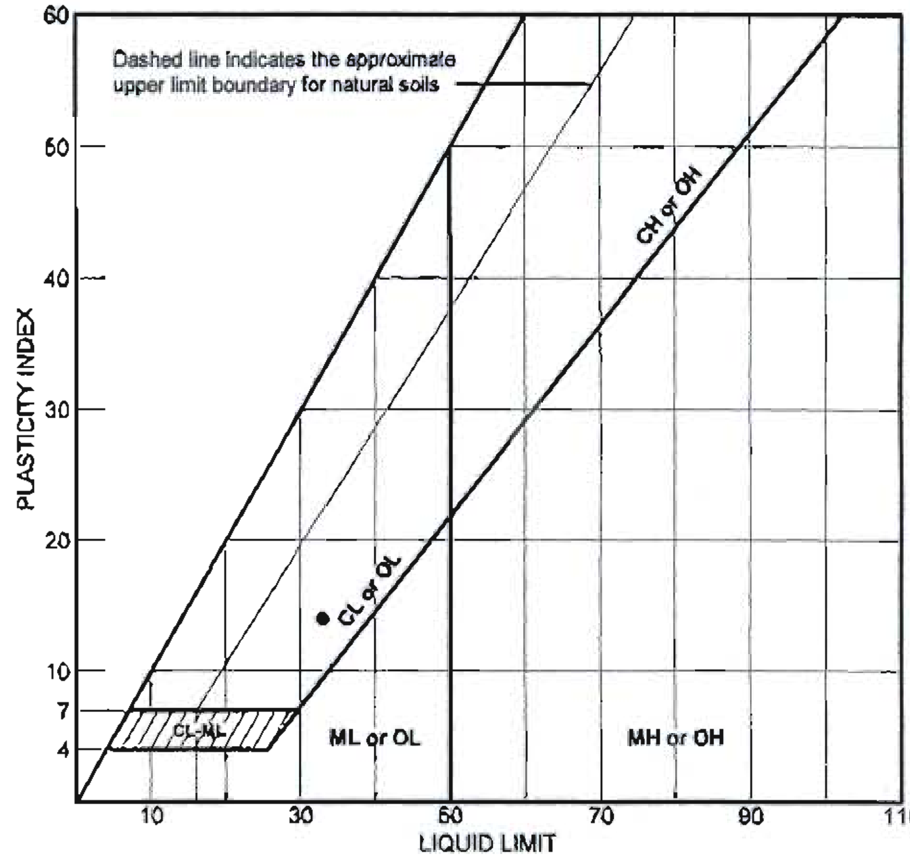
TEST PIT TP6

S8874-06-01	April 2005	Figure A6

APPENDIX B

LIQUID AND PLASTIC LIMITS TEST REPORT





	SOURCE	SAMPLE #	DEPTH/ELEV.	DATE SAMPLED	USCS	MATERIAL DESCRIPTION	NM %	LL	PI
●		TP-1		8/9	CL	Light olive brown sandy lean CLAY		33	14

Client

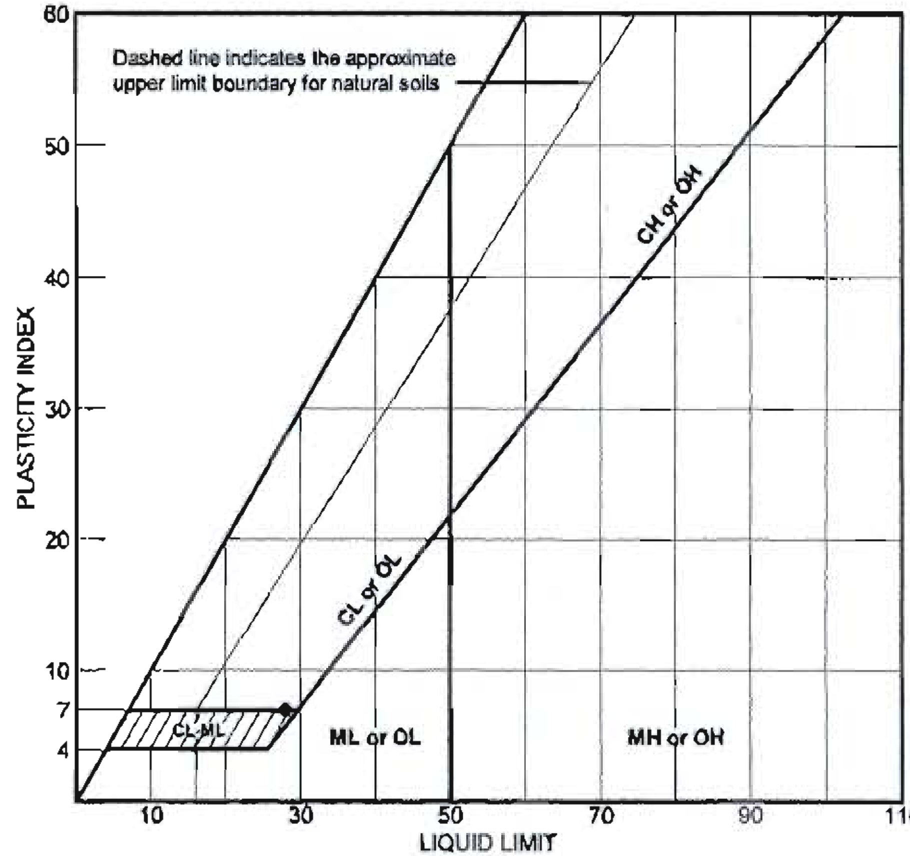
Project Ione Resevoir

Project No. S8874-06-01 Figure

GEOCON CONSULTANTS, INC.

● Test performed on <#4 fraction.
No grain size test performed on sample to date.

LIQUID AND PLASTIC LIMITS TEST REPORT



	SOURCE	SAMPLE #	DEPTH/ELEV.	DATE SAMPLED	USCS	MATERIAL DESCRIPTION	NM %	LL	PI
●		#2		8/9	CL-ML	Strong brown sandy SILT		28	7

Client	GEOCON CONSULTANTS, INC.	● Test performed on <#4 fraction. No grain size test performed on sample to date.
Project Ione Resevoir		
Project No. S8874-06-01 Figure		