Appendix J

GEOTECHNICAL INVESTIGATION

FAIRFAX VINEYARDS
RESERVOIR AND SUMP POND
35147 ANNAPOLIS ROAD
ANNAPOLIS, CALIFORNIA

11673.2

BACE GEOTECHNICAL
A Division of Brunsing Associates, Inc.

October 29, 2009

Brunsing Associates, Inc.

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35147 ANNAPOLIS ROAD
ANNAPOLIS, CALIFORNIA

11673.2

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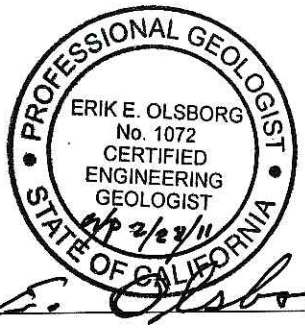

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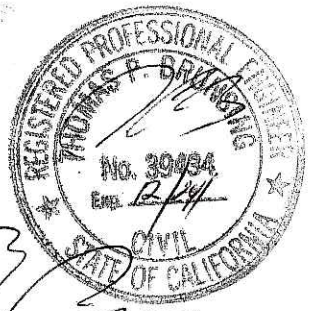

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1.0 INTRODUCTION

This report presents the results of the Geotechnical Investigation performed by BACE Geotechnical (BACE), a division of Brunsing Associates, Inc., for the planned reservoir and sump on Artesa Vineyards and Winery's Fairfax Vineyard, 35147 Annapolis Road, Annapolis, California. The property, A.P.N. 123-040-32, is located on the south side of Annapolis Road approximately 0.25 miles west of the intersection of Soda Springs Road and Annapolis Road, as shown on the Vicinity Map, Plate 1.

BACE previously performed a geotechnical investigation for a 36.5 acre-foot reservoir and sump in approximately the same location as those currently planned. The results of that investigation were presented in a report dated June 25, 2002. Based on discussions with Mr. Lee Erickson, P.E. of Erickson Engineering, Inc. (EEI), and our review of the revised plans dated 02.03.2009, by Erickson Engineering Inc., we understand that the new reservoir will have a capacity of 73 acre-feet. The reservoir will have a bottom elevation of 798 feet above Mean Sea Level (MSL) and the top of the embankment at elevation 822 feet above MSL. The sump pond capacity will be 2 acre-feet with a bottom elevation of 688 feet and the top of embankment at an elevation of 696 feet. The reservoir and sump will have either artificial or compacted soil liners. Both the reservoir and the sump pond will have exterior and interior slopes of two and one half horizontal to one vertical (2-1/2H:1V). The planned reservoir and sump pond locations are shown on the Site Plan & Geologic Map, Plate 2.

The purpose of our investigation was to evaluate the site soil/rock conditions in order to determine the feasibility of the larger reservoir, and to provide geotechnical conclusions and recommendations regarding site grading, embankment stability, compacted soil liner construction, suitability of on-site soils for use as liner material, and the need for sub-drainage, keying and benching. The scope of our services, as outlined in our Professional Services Agreement dated August 11, 2009, consisted of reconnaissance and subsurface exploration, laboratory testing, geologic and engineering analysis, and the preparation of this report.

2.0 INVESTIGATION

2.1 Research

As part of our investigation, we initially reviewed the following published geologic references:

- Blake, M.C., Jr., Terry-Smith, J., Wentworth, C.M., and Wright, R.H., 1971, Preliminary Geologic Map of Western Sonoma County and Northernmost Marin County, California: United States Geological Survey San Francisco Bay Region Environment and Resources Planning Study, Basic Data Contribution 12.
- California Division of Mines and Geology (CDMG), 1998, Uniform Building Code, Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada: International Conference of Building Officials (ICBO).

- Fuller, M.S., Haydon, W.D., Purcell, M.G., and Custis, K, 2002, Geologic and Geomorphic Features Related to Landsliding, Gualala River Watershed, Sonoma and Mendocino Counties, California: California Geological Survey, CD 2002-08.
- Higgins, C.G., Ohlson Ranch Formation, Pliocene, Northwestern Sonoma County, California. University of California Publications in Geological Sciences: vol. 36, no. 3: 199-231.
- Huffman, M.E., and Armstrong, C.F., 1980, Geology for Planning in Sonoma County: CDMG Special Report 120.
- Jennings, C.W., 1960, Geologic Map of California, Ukiah Sheet: CDMG.
- Slosson, J.E., 1974, State of California Special Studies Zones: CDMG, Annapolis 7.5-minute Quadrangle.

2.2 Field Reconnaissance and Exploration

Our field reconnaissance consisted of observations of site topography, vegetation, and near surface soils, and interpretation of geomorphic expressions within the property and vicinity. Site Photographs A and B on Plate 3 show the areas of the planned reservoir and sump. Site Photograph C on Plate 4 shows a closer view of the planned sump area. The photograph locations are shown on Plate 2.

Our field exploration consisted of excavating, logging, and sampling five test pits to depths ranging from 8.5 to 15.0 feet on September 29, 2009. The test pits were excavated with a Takeuchi TB175 excavator using a 30-inch bucket. The locations of the test pits, TP-8 through TP-12, as well as the test pits and borings from our previous investigation (TP-1 through TP-7 and test Borings B-1 and B-2) are shown on Plate 2.

Our senior engineer and staff geologist logged the test pits and obtained both relatively undisturbed (tube) and loose bulk samples of the soil and rock materials encountered for visual classification and laboratory testing. The relatively undisturbed tube samples were obtained using a 3-inch outside diameter Modified California split-barrel sampler, pushed vertically with the excavator bucket. The geologist performed pocket penetrameter (PP) readings on the pit walls in TP-8 through TP-12; PP readings give a relative indication of strength and are presented on the logs.

The logs of the recent and previous test pits and test borings, showing the various soil and rock materials encountered and the depths at which samples were obtained, are presented on Plates 5 through 13. The soils are classified in accordance with the Unified Soil Classification System (ASTM D2487) outlined on Plate 14. The various descriptive properties used to describe the soil and rock is listed on Plates 15 and 16, respectively.

2.3 Laboratory Testing

Selected samples were tested in our laboratory to determine their pertinent geotechnical engineering characteristics. Laboratory testing consisted of moisture content/dry density, grain size distribution, Atterberg Limits, intact and remolded triaxial compressive strength, direct shear, remolded permeability (triaxial cell), and maximum dry density

(compaction). The laboratory test data are summarized on the test boring logs opposite the samples tested. A Key to Test Data is provided on Plate 14. In addition, grain size distribution test results are presented on Plates 17 and 18, Triaxial Compressive Strength Data are presented on Plates 19 and 20, direct shear data is presented on Plate 21, compaction test data are presented on Plates 22 and 23, and earlier permeability test results are summarized on Plate 24. The most recent permeability test results completed due to expansion of the reservoir are summaries on Plate 25. ASTM has modified the test procedures subsequent to the earlier permeability test completed (Plate 24). Consequently the results summarized on Plates 24 and 25 are not directly comparable.

3.0 SITE CONDITIONS

The Fairfax Vineyard site is located in the coastal hills east of Annapolis. The reservoir is to be located on the crest of Beatty Ridge, which has a general east-west trend in the reservoir location. The sump will be located near the easterly ridge toe, just upslope of the Patchett Creek ravine. The site is currently undeveloped and is accessed by a dirt driveway from Annapolis Road. The reservoir site is covered with a moderate growth of seasonal grasses and a few scattered trees. To the south of the planned reservoir and to the east and south of the sump pond is a thick wooded area. The reservoir site is near level to very gently-sloping, with gradients from ten horizontal to one vertical (10H:1V) to 25H:1V. The southeasterly slopes, downslope of the planned sump, are moderately steep, about 4H:1V.

4.0 SITE GEOLOGY AND SOIL CONDITIONS

4.1 Regional Geologic Setting

According to the published geologic references we reviewed for this investigation, the bedrock of this part of Sonoma County is comprised of shallow marine sediments of the Quaternary-Tertiary Period (Pleistocene-Pliocene) Ohlson Ranch Formation. The Ohlson Ranch Formation is generally massive (no distinct bedding) fine-grained sandstone with occasional pebbly layers, forming gently sloping caps on the coastal ridges. It unconformably overlies the Tertiary-Cretaceous Period Coastal Belt Franciscan Complex, marine sandstone, which crops out on the lower steeper slopes and in the valleys.

The seismicity and tectonics of the Sonoma and Mendocino coastal region are controlled by a network of generally northwest-trending strike-slip faults of the San Andreas Fault system. The active San Andreas Fault (north coast segment) is located within the South Fork Gualala River Valley, approximately 3.4 miles (5.4 kilometers) southwest of the site. The active Maacama Fault (south segment) is located approximately 20.9 miles (33.7 kilometers) northeast of the site. Future, large magnitude earthquakes originating on these, or other nearby faults are expected to cause strong ground shaking at the site.

4.2 Site Geology and Soils

The Ohlson Ranch Formation was encountered in 10 of the 12 test pits, and in both of the test borings. As observed in these test pits and test borings, the Ohlson Ranch Formation consists generally of yellow-brown, orange-brown, and olive-brown, silty fine-grained sandstone that is little to occasionally fractured, soft to very hard, and little to moderately weathered. At deeper levels, the sandstone is generally dark greenish gray or dark bluish gray, hard, and little weathered.

Test Pits TP-5 and TP-8 were excavated in the area of the planned sump, on the western slope of the Patchett Creek ravine. At these lower elevations, we encountered orange-brown shale and sandstone of the Franciscan Complex. The shale and sandstone were generally intensely to closely fractured, low in hardness to very hard with depth, with little to moderate weathering.

We constructed a geologic map and cross section based on our observations within the test pits in conjunction with our published reference research. The geologic map is shown on Plate 2, and Geologic Cross Section A-A' is presented on Plate 26. The orientation (strike and dip) of the basal contact between the two geologic units at the property was determined using map interpretation methods. On the published geologic map (Basic Data Contribution 12), we measured the elevation change and lateral width of the outcrop pattern in four directions from the reservoir. Using this method, we determined an approximate north-south strike and 0.7 degree dip to the west.

Fossilized corals were discovered within the Ohlson Ranch Sediments during both the current and previous investigations. Corals are generally an indication of a relatively shallow, warm marine depositional environment. The Ohlson Ranch Formation is thought to have been deposited in such an environment during the Pliocene, whereas the Franciscan Complex is comprised of older, deep-water deposits.

The excavator and backhoe equipment were able to excavate the Ohlson Ranch sandstone with little difficulty in most locations. However, the excavator did encounter practical excavation refusal in hard sandstone below 8.5 feet in Test Pit TP-12. Excavation was also difficult within the Franciscan Complex shale. Backhoe refusal was encountered at 11.3 and 8.75 feet in Test Pits TP-5 and TP-8, respectively.

As observed in our test pits and test borings, one to four feet, or more, of silt to silty sand residual/colluvial soil, blankets the bedrock. The soils are generally soft to medium stiff/medium dense, porous, and contain roots.

No groundwater was encountered during our current, September 2009 investigation. Moderate groundwater seepage was encountered in most of the previous test pits and Boring B-1 during winter of 2001-2002. The ground water observed at that time was mostly perched within the upper soils and weathered bedrock 2 to 3.5 feet below the ground surface (bgs) at each test pit or boring location. Ground water was encountered at

6 feet in Test Boring B-1 and rose to 3 feet below the ground surface approximately 2 hours after completion of drilling in January of 2002.

Minor caving occurred within the upper two feet of Test Pits TP-1 and TP-2; no caving occurred during our current investigation.

No evidence of faulting or landsliding was observed at the site, and none of the published geologic maps that we reviewed show faults or landslides in the reservoir or sump areas.

5.0 DISCUSSIONS AND CONCLUSIONS

5.1 General

Based on the results of our field exploration and laboratory testing, we conclude that the site is geotechnically suitable for the planned reservoir and sump pond. If properly constructed, the reservoir and sump fill embankments and cut slopes should be stable. With the use of an artificial or soil compacted liner the main geotechnical constraints that should be considered in design and construction for the reservoir and sump include the presence of weak/porous surface soils, isolated seepage areas, excavation characteristics, slope stability, and strong seismic shaking from future earthquakes. These considerations and possible mitigation measures are discussed below.

5.2 Weak-Porous Surface Soils

The existing porous natural soils encountered in the upper 1 to 4 feet of our test pits and borings are weak, and are not suitable for support of new fill embankments. These soils may collapse when loaded with embankment fill material, and may also be more prone to erosion on cut slopes. Therefore, these soils should be removed and replaced as properly compacted fill within embankment areas and within reservoir cut areas where not removed by planned excavation.

5.3 Seepage and Pervious Zones

Depending on the time of year of constructions, isolated seepage and pervious zones may be encountered during planned excavations for the reservoir and sump. Therefore, to prevent leakage from the reservoir and sump sides and bottoms, a compacted soil liner at least three feet in perpendicular thickness needs to be provided or an artificial liner needs to cover the reservoir and sump. Detailed recommendations for compacted soil liner are presented in the following sections of this report. The results from permeable testing of samples from the original planned reservoir excavation were marginally suitable for reuse as a compacted soil liner. Based on the permeability test data from soil samples collected from the current reservoir planned excavation, we conclude that the on-site soils from the area are <u>not</u> suitable for reuse as a compacted soil liner.

Because ASTM has improved the methodology of the permeability and compaction test procedures, the old data should not be relied upon. Consequently we recommend either

augmentation of the on-site material or use of a synthetic liner to reduce seepage from the reservoir and sump.

We understand that an artificial liner may be used for the reservoir and sump. Suggested artificial liner design considerations are presented in the recommendations section of the report.

It should be noted that artificial liners may become slippery and can be dangerous to people and animals. A covering mat or other means of escape should be added to prevent accidental drowning.

Where seepage is encountered, sub-drains will need to be provided to intercept the subsurface water beneath the liner, whether compacted soil or artificial. Sub-drain recommendations are further discussed below.

5.4 Soil Moisture Conditioning

To minimize moisture conditioning (drying out) of the fill soils, site grading should take place during the dry summer months. The soils within the deeper excavation will be wet, regardless of the season. A relatively large, adjacent work area will be needed for soil moisture conditioning prior to placing the soils as fill.

5.5 Excavation Characteristics

As mentioned above and noted on the test pit logs, refusal to the backhoe was encountered in hard shale at a depth of about 11.3 and 8.75 feet in Test Pits TP-5 and TP-8, and refusal to the excavator was encountered in hard sandstone at a depth of about 8.5 feet in Test Pit TP-12. However, based on our observation of drilling equipment during exploration, and dozer/backhoe/excavator performance at sites in the vicinity, we conclude that the majority of planned excavations can be achieved using conventional heavy excavation equipment, such as a Caterpillar D8R tractor equipped with a single-tooth ripper. Localized, more massive hard rock zones could possibly be encountered in the deeper cut areas. Excavations may become more difficult due to hard rock masses, especially within the planned sump.

5.6 Slope Stability

5.6.1 Natural Slopes

No evidence of recent landsliding or gross instability of the existing natural slopes was observed within the planned reservoir and sump sites. No landslides are shown at the property on the published geologic maps we reviewed for this investigation. Surficial slope failures can occur during seasons of heavy rainfall and from concentrated drainage conditions, such as within the Patchett Creek ravine.

The current standard of practice in geotechnical engineering makes it possible to identify most areas of existing instability, and/or to make recommendations which lower the risk of instability to generally acceptable levels. However, current standard of practice cannot make total assurances of avoiding possible future instability.

5.6.2 Cut and Fill Slopes

The finished interior and exterior cut and fill embankment slopes for the entire perimeter of the reservoir will be inclined at a 2.5H:1V gradient. The maximum cut slope height is 24 feet on the west side of the reservoir. The maximum fill slope height of 39 feet occurs at the southeastern corner of the reservoir.

Our slope stability analyses were performed to correspond to the guidelines by (1) American Society of Civil Engineers (ASCE) and Southern California Earthquake Center (SC/EC) "Recommended Procedures for Implementation of DMG Special Publication 117 Guidelines for Analyzing and Mitigating Landslide Hazards in California", dated June 2002 and (2) California Geological Survey (CGS) "Guidelines for Evaluating and Mitigating Seismic Hazards in California" dated 2008. These publications suggest that slopes should have a factor of safety greater than or equal to 1.5 for static conditions and 1.1 for seismic conditions, with a resulting permanent displacement between 5 and 15 centimeters (cm) and a horizontal seismic coefficient of 0.15. The CGS report also suggests that slopes that have a pseudo-static factor of safety greater than 1.0 using a seismic coefficient derived from the screening analysis procedure can be considered stable.

The locations of Cross Sections B-B' and C-C' used for our stability analysis are shown on attached Plate 2. Three soil/rock "units", with different density and strength parameters, were delineated within the embankments for our stability analysis. Unit "1" is the silty sand compacted fill, Unit "2" is the natural weak topsoils, and Unit "3" is the Quaternary-Tertiary Ohlson Ranch Formation sandstone.

For our stability analysis Unit "1" was assigned a wet density of 117 pounds per cubic foot (pcf), and the following strength parameters; cohesion (C) of 2000 pounds per square foot (psf), and a friction angle (phi) of 28 degrees. Unit "2" was assigned a wet density of 110 pcf, and strength parameters; C of 500 psf and phi of 0. Unit "3" was assigned a wet density of 125 pcf, and the following strength parameters; C of 350 psf and phi of 44 degrees. These are considered as minimum parameters.

The above assigned strengths were determined from strength test results obtained from this site, as well as from back-analysis of the slope stability calculations. The stability of the slope was analyzed using the computer program SLIDE 5.0 version 5.039 by Rocscience Inc. The results of our stability analyses are presented in Appendix A.

5.7 Seismic Hazards/Ground Shaking

5.7.1 Ground Shaking

As is typical of the Sonoma County coastal region the proposed reservoir and sump will be subject to strong ground shaking from future earthquakes. In general, the intensity of ground shaking at the site will depend on the distance to the causative earthquake epicenter, the magnitude of the shock, and the response characteristics of the underlying earth materials. With the embankments founded upon firm weathered bedrock, and with interior and exterior slopes of 2 1/2 H:1V; the embankments should be well suited to resist the effects of ground shaking.

5.7.2 Liquefaction

The natural topsoils encountered in our test pits and borings at the site are only approximately 1 to 4 feet in thickness, and are expected to be removed during planned excavation or fill embankment construction. The deeper site soils and bedrock are predominately silty sandstone in the reservoir area and silty sandstone and shale in the sump area. No clean sands (little or no clay or silt) were encountered in our test pits or borings. Since the underlying bedrock has moderate to high strength and relatively high blow counts, this material should not be subject to liquefaction. The densely compacted fill embankments should also not be subject to liquefaction. We consider this site, with this project, to have a very low potential for liquefaction during seismically induced strong ground shaking.

5.7.3 Fault Rupture

The nearest known active fault (the San Andreas Fault) is located approximately 5.4 kilometers southwest of the property. The active Maacama Fault (south segment) is located about 33.7 kilometers northeast of the site. Considering that these faults are more than 5 kilometers from the site, we conclude that the potential for surface fault rupture at this site is very low.

5.8 Soil Liner

Based upon the results of our current permeability tests, the silty sand soils (Ohlson Ranch Formation) have an estimated permeability of 1 x 10^{-5} centimeters per second (cm/sec) as compacted soil liner material, if compacted to 90 percent relative compaction. Because these soils have very little clay they are not suitable as a liner material unless they are augmented with imported material.

If a compacted soil liner is used, it may be practical to put a sump and pump at the underdrain outlet location. The collected underdrain water could then be pumped back into the reservoir, as needed. If minimal seepage is desired, the use of an artificial liner may be more practical.

The sump soils/weathered bedrock materials within the sump area do not appear suitable for use as compacted soil liner. Therefore, soils from the reservoir area (Ohlson Ranch Formation) will be needed for the sump liner material.

6.0 RECOMMENDATIONS

6.1 Site Preparation and Grading

6.1.1 Fill Area Preparation

Areas to be graded for placement of fill soils should be cleared of surface vegetation, and stripped to remove surface soils containing roots. We anticipate the depth of stripping would generally be about two to four inches. Deeper stripping and grubbing may be required to remove isolated concentrations of organic matter and tree roots. The cleared strippings should not be used as fill soil, but may be used as topsoil cover for the exterior embankments, provided that erosion-resistant vegetation is planted.

Weak porous surface soils (approximately one to four feet in thickness) should be removed from fill embankment areas and the zone extending at least five feet beyond the exterior embankment toe. The excavated soils, free of remaining organic matter and over-size rocks (greater than six inches in largest dimension), can be stockpiled for later use as embankment fill material.

6.1.2 Fill Slope Keyway Construction

At the toe of embankment fill slopes, an initial keyway should be excavated a minimum of two feet into firm supporting material on the downhill side, in accordance with the Keyway/Bench Drainage Detail, Plate 27. Keyway depths are anticipated to be approximately 3 to 6 feet in depth on the downslope side, for both the reservoir and sump, as observed by BACE. The initial keyway excavation should have an inward gradient of about two-percent into the slope.

The initial excavation (keyway) should have a perforated pipe and gravel drain placed on the uphill side, as shown on Plate 27. The perforated pipe should be a 4-inch diameter, SDR 35, or other non-corrosive equivalent pipe with a pipe stiffness of at least 40 pounds per square inch. The pipe should be placed with the perforations down. The gravel should consist of clean, free-draining gravel or crushed rock graded from 1½ inches maximum to ½ inch minimum in size, or Class 2 Permeable Material per Section 68 of Caltrans Standard Specifications. Drain rock, excluding the Class 2 Permeable Material, should be completely wrapped in geotextile filter fabric (Mirafi 140N, or equivalent) so that there is no natural soil/drain rock contact.

Intermediate benches under the fill slopes should extend through the weak, porous surface soils into firm supporting soils. Intermediate bench sub-drains may also be necessary based on the conditions observed in the field by BACE during grading, and

should be installed according to the details on the Keyway/Bench Drainage Detail, Plate 27.

6.1.3 Fill Materials, Placement and Compaction

In embankment areas on gentle slopes, after BACE observes the soils exposed by the recommended excavations, the exposed soil/rock materials should be scarified to a depth of six inches, moisture conditioned to at least 2 percent above optimum moisture content (OMC) and compacted to at least 90 percent relative compaction (RC), per latest edition of ASTM D1557 test method, using sheepsfoot rollers; smooth-wheel rollers should not be used except for final surface grading. These moisture conditioning and compaction procedures should be observed by BACE.

Based on our test pit, test boring and laboratory test data, we anticipate that the existing Ohlson Ranch formation at the site, obtained from on-site cuts and excavations, will be suitable for use as fill during fill embankment construction, provided that it is approved by BACE prior to placement. On-site fill materials should be in a "clean" condition (free of organics and debris) and free of over-size rocks greater than six inches in largest dimension.

Import fill material that may be brought to the site should be a select fill material that meets the following requirements:

Select Fill Material:

Free of organics and other deleterious materials
Maximum particle size: 3 inches
Percent passing 1-inch sieve: 90 percent minimum by weight
Percent passing #200 sieve: 8 percent to 40 percent by weight

We recommend that representative samples of materials proposed for use as compacted fill at the site be submitted to BACE for testing and approval at least three week prior to the start of grading.

Areas to receive compacted fill material should be approved by BACE prior to the placement of fill material. Fill should be placed in thin horizontal lifts (six to eight inches depending on compaction equipment), conditioned to at least 2 percent above OMC, and compacted to at least 90 percent RC as determined by the ASTM D 1557 test procedure, latest edition, to achieve planned grades. Before compaction begins, the fill should be brought to a water content that will permit proper compaction by either: 1) aerating the material if it is too wet; or 2) spraying the material with water if it is too dry. Each lift should be thoroughly mixed before compaction to insure a uniform distribution of water content.

Field density tests to determine the compaction of the fill should be made at the locations recommended by BACE. If compacted fill materials are allowed to dry out prior to

placing subsequent lifts, so that shrinkage cracks appear on the surface of the fill, then the compacted fill must be scarified, moisture-conditioned, and re-compacted before further construction.

6.1.4 Finished Slope Grading and Spillway Trench

Fill embankment slopes should be compacted by rolling and trimming, or overfilled and trimmed back to planned grade, to expose a firm, smooth surface free of loose material. Cut and fill slopes should be planted with erosion-resistant vegetation, or protected from erosion by other measures upon completion of grading. Where possible, surface runoff should be intercepted and diverted away from finished slope surfaces.

The planned spillway pipe trench on the southern fill embankment corner should be "broom-cleaned" and observed by BACE prior to backfill. The trench should be backfilled with concrete around the pipe according to the specifications outlined on the plans. The grading contractor will be responsible for vibrating the concrete during placement. A BACE representative should observe placement of the concrete.

6.2 Liners

6.2.1 Compacted Soil Liner

Soil liner material should consist of silty/clayey soils and should meet the following gradation:

Sieve/Screen Size	Percent Passing (by dry weight)
3-inch	100
1 inch	90 - 100
No. 4	70 - 100
No. 200	30 - 100

In general, the on-site soils encountered in our test pits and borings should meet the above criteria. Soil suitability for liner material should be determined in the field by BACE. The compacted soil liner should be at least 3 feet in thickness, perpendicular to the reservoir or sump surface. Liner soils should be placed and compacted as described in Section 6.1 of this report.

Based on our current test data, we conclude that the on-site soils in the reservoir areas are not suitable for reuse as a compacted soil liner, see permeability test data on Plate 25.

6.2.2 Artificial Liner

If used, the artificial liner should be at least 60 mils thick high-density polyethylene (HDPE). The liner should be installed, and each joint and penetration sealed, per the manufacturer's recommended procedures. The liner installation should be inspected and tested in accordance with the manufacturer's requirements.

6.2.3 Liner Under-Drainage

Subdrains will be required beneath the artificial and/or the compacted soil reservoir liner. A perimeter drain should be constructed in the reservoir bottom. Under-liner drains should consist of a 2-feet wide by 18-inch deep trench with a 4-inch perforated pipe, SDR35, or equal. The trench should be backfilled with Class 2 permeable material, or clean, free draining ¾-inch or 1 ½-inch gravel wrapped in filter fabric (Mirafi 140N or equal). See the Typical Trench Subdrain Detail on Plate 28 for a conceptual drawing.

Finger drains may be required in the cut slope. If used, the finger drains should consist of a 2-feet wide by 18-inch deep trench with a 4-inch perforated pipe, SDR35, or equal. The trench should be backfilled with ¾-inch or 1 ½-inch (crushed, not rounded) drain rock wrapped in filter fabric (Mirafi 140N or equal), or Caltrans Class 2 permeable material (without the filter fabric), see Plate 28. BACE should determine the final number and location of finger drains during construction.

6.3 Riprap

Riprap should be used at outlets for the spillway and overflow pipe. The riprap section should be about two feet thick and placed over geotextile fabric (Mirafi FW500 or equal). Riprap rock should be sound, resistant to abrasion and reasonably free from cracks, seams, and other defects that would tend to increase their destruction by water action. Riprap rock should be between six inches and two feet in size and carefully fitted together to provide a tight interlock. As an alternative, the use of erosion control and turf reinforcement could be use such as North American Green Vmax C350 or equal.

6.4 Surface and Subsurface Drainage

Surface water runoff should be intercepted and directed away from the top and toe of cut and fill slopes. Drainage ways should be maintained to prevent water from ponding along the top or toe of the slopes.

Depending upon the time of year, ground water seepage may be encountered during embankment excavation operations. If seepage is severe, an upstream cutoff trench may be needed. Ground water collected in the cutoff trench should be directed to the vicinity of the existing creek on site.

We anticipate that sub-drains will most likely be needed to intercept subsurface water beneath the compacted soil or artificial liner. We recommend installing a sub-drain as illustrated on the Trench Subdrain Detail presented on Plate 28. Finger sub-drains or extended gravel blankets may be required up the slopes where concentrated seepage is encountered. Beneath artificial liner, composite prefabricated drains (such as Miradrain or equivalent) can be used as an alternative subdrain system. The composite sub-drainage should be installed in accordance with manufacturer's suggested procedures.

Surface and subsurface water should be collected in solid pipes and outlet into the existing or established drainage system(s) or creek channels on site. Alternatively, the collected water can be pumped back into the reservoir and sumps.

7.0 ADDITIONAL SERVICES

Prior to construction, BACE should review the final grading and reservoir/sump construction plans, and related specifications, for conformance with our recommendations. During construction, BACE should be retained to provide periodic observations, together with field and laboratory testing, during site preparation, keyway excavation, subdrain installation, placement and compaction of fills for embankment construction and soil liner installation, and artificial liner installation (if used). Reservoir and sump excavations should be reviewed by BACE while the excavation operations are being performed. Our observations and tests will allow us to verify conformance of the work to project guidelines, determine that soil/bedrock conditions are as anticipated, and to modify our recommendations, if necessary.

8.0 LIMITATIONS

This geotechnical investigation and review of the proposed reservoir/sump development were performed in accordance with the usual and current standards of the profession, as they relate to this and similar localities. No other warranty, expressed or implied, is provided as to the conclusions and professional advice presented in this report. Our conclusions are based upon reasonable geologic and engineering interpretation of available data. A soil corrosion study was not included in our scope of services for this project.

The samples taken and tested, and the observations made, are considered to be representative of the site; however, soil and geologic conditions may vary significantly between test pits, borings and across the site. As in most projects, conditions revealed during construction excavation may be at variance with preliminary findings. If this occurs, the changed conditions must be evaluated by BACE and revised recommendations be provided as required.

This report is issued with the understanding that is the responsibility of the owner, or of his/her representative, that the information and recommendations contained herein are brought to the attention of all other design professionals for the project, and incorporated into the plans, and the Contractor and Subcontractors implement such recommendations in the field. The safety of others is the responsibility of the Contractor. The Contractor should notify the Owner and BACE if he/she considers any of the recommended actions presented herein to be unsafe or otherwise impractical.

Changes to the conditions of a site can occur with the passage of time, whether they are due to natural events or to human activities on this, or adjacent sites. In addition, changes in applicable or appropriate codes and standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, this report may become

invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and revision as changed conditions are identified.

The recommendations contained in this report are based on certain specific project information regarding type of construction and reservoir/sump locations, which has been made available to us. If any conceptual changes are undertaken during final project design, we should be allowed to review them in light of this report to determine if our recommendations are still applicable.