Appendix 2-7

Geotechnical Design Criteria

THE TWINING LABORATORIES, INC.
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ANALYTICAL CHEMISTRY · ENVIRONMENTAL SERVICES
GEOTECHNICAL ENGINEERING · SAMPLING SERVICES
CONSTRUCTION INSPECTION & MATERIALS TESTING

PRELIMINARY GEOTECHNICAL ENGINEERING INVESTIGATION AND GEOLOGIC HAZARD EVALUATION

KINGS RIVER CONSERVATION DISTRICT

PROPOSED POWER PLANT

NORTHEAST OF THE INTERSECTION OF EAST DINUBA AVENUE AND SOUTH BETHEL AVENUE

FRESNO COUNTY, CALIFORNIA

Twining Project Number: F08306.02-01

For:

Kings River Conservation District
4886 East Jensen Avenue
Fresno, CA 93725

February 28, 2007

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Kings River Conservation District
4886 East Jensen Avenue
Fresno, CA 93725

Attention: Mr. Jim Richards

Subject: Preliminary Geotechnical Engineering Investigation and
Geologic Hazard Evaluation
Proposed Power Plant
Northeast of the Intersection of East Dinuba Avenue and South Bethel Avenue
Fresno County, California

Dear Mr. Richards:

We are pleased to submit this preliminary geotechnical engineering investigation report and geologic hazard evaluation prepared for the proposed power plant to be located northeast of the intersection of East Dinuba Avenue and East Bethel Avenue in Fresno County, California. The contents of this report include the purpose of the investigation, scope of services, background information, investigative procedures, our findings, evaluation, conclusions, and recommendations.

The purpose of the investigation was to conduct a subsurface exploration and a laboratory testing program, evaluate the data collected during the field exploration and laboratory testing, and provide preliminary geotechnical engineering recommendations for use in the preliminary assessment of the project. It is our understanding that the details of the proposed power plant are being developed (design loads, foundation sizes, etc.), and the actual locations of the structures have not been determined. Accordingly, the evaluations, conclusions, and recommendations provided in this report are preliminary and should be used for preliminary design and planning purposes only. Additional field exploration and preparation of a design level geotechnical report based on the final proposed site configuration, loads, and structural details, etc. will be required prior to final design, bidding and construction of the project.

It is recommended that those portions of the plans and specifications that pertain to earthwork, pavements, and foundations be reviewed by The Twining Laboratories, Inc. (Twining) to determine if they are consistent with our recommendations. This service is not a part of this current contractual agreement. The client should retain our firm to review these documents a prior to their issuance for bidding and construction.

We appreciate the opportunity to be of service to the Kings River Conservation District. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience.

Sincerely,
THE TWINING LABORATORIES, INC.

Zubair Anwar
Staff Engineer
Geotechnical Engineering Division

EXECUTIVE SUMMARY

The Twining Laboratories, Inc. (Twining) was authorized by Mr. Jim Richards with the Kings River Conservation District (KRCD) to prepare this preliminary geotechnical engineering investigation report and geologic hazard evaluation for the proposed power plant to be located northeast of the intersection of East Dinuba Avenue and South Bethel Avenue in Fresno County, California.

The purpose of the investigation was to conduct a subsurface exploration and a laboratory testing program, evaluate the data collected during the field exploration and laboratory testing, and provide preliminary geotechnical engineering recommendations for use in the preliminary assessment of the project. It is our understanding that the details of the proposed power plant are being developed (design loads, foundation sizes, etc.). The evaluations, conclusions, and recommendations provided in this report should be used for preliminary design and planning purposes only. Additional field exploration and preparation of a design level geotechnical report based on the final proposed site configuration, loads, and structural details, etc. will be required prior to final design, bidding and construction of the project.

The project site plan shows the general arrangement of the proposed structures. Based on our review of the site plan, it is anticipated that the proposed construction will include turbine generators, exhaust stacks, transformers, storage tanks, pre-engineered buildings, and miscellaneous structures including various equipment structures (equipment pads, cooling tower, pumps, etc.). Appurtenant construction is also anticipated to include asphaltic concrete and Portland cement concrete parking and driveways, and underground utilities.

The location of test borings drilled for the field exploration were selected by KRCD. Prior to drilling, the boring locations were surveyed by Provost and Prichard Engineering Group. Between October 31, 2006, and November 3, 2006, seventeen (17) test borings were drilled at the approximate locations selected.

At the time of the field exploration, the site was in agricultural use with a grape vineyard and three structures were located near the western boundary of the site. An agricultural water well was also noted near the western boundary of the site. Dirt access roads traversed the northern, southern, eastern, and central portions of the site.

The near surface soils generally consisted of loose to dense silty sands and medium stiff to stiff sandy silts. The near surface silty sand and sandy silt soils were encountered from the ground surface to depths ranging from about five (5) to twenty-five (25) feet BSG. Below the near surface silty sand and sandy silt soils, various interbedded layers of very stiff to hard sandy silt, loose to dense silty sands, and loose to medium dense poorly graded sands were encountered to the maximum depth explored of 60 feet BSG.

Based on laboratory data and field exploration (N-value) test results, the soils are anticipated to exhibit low shear strength, moderate compressibility and low to moderate collapse characteristics, and good to excellent pavement support characteristics.

EXECUTIVE SUMMARY (cont.)

Groundwater was encountered at depths ranging from 35 to 40 feet BSG in ten (10) of the seventeen (17) borings at the time of our field exploration. The Phase I Environmental Report indicated that groundwater was reported to be 35 feet BSG. Well data, provided by the California Department of Water Resources, indicated a water well with a similar elevation to the project site had a groundwater level of 20 feet BSG during the last measurement taken in 1981.

The primary geotechnical issues that will potentially impact design and construction of the project are: 1) the potential for excessive collapse settlement of soils in the upper 5 feet BSG, 2) the potential for excessive static settlements of some of the proposed structures and tanks with relatively large foundations and bearing loads, 3) the potential for post construction seismic settlements, and 4) the potential for excessive organics in the near surface soils resulting from the current and previous agricultural crops.

The near surface soils exhibit the potential for soil collapse (hydroconsolidation); thus, the near surface site soils will not provide adequate support for foundations and interior slabs-on-grade. Therefore, recommendations for over-excavation and compaction of the near surface soils are included in this report to reduce the potential for hydroconsolidation.

Based on the subsurface data and the results of laboratory testing performed as part of this investigation, static settlement calculations indicated the potential for excessive static settlements. Based on the anticipated and assumed structural loads, and the potential for excessive static settlements, it is recommended that the foundations be supported on a minimum thickness of engineered fill.

Shallow spread foundations are anticipated for the pre-engineered buildings (relatively light structures). Settlement calculations indicate that shallow spread foundations placed directly on undisturbed native soils could experience total static settlements in excess of 1 inch. To reduce the total static settlement to 1 inch and the differential static settlements to ½ inch in 40 linear feet, the foundations should be placed entirely on at least 3 feet of engineered fill, engineered fill which extends to a minimum depth of 4 feet below preconstruction site grades, or to undisturbed native soils, whichever is deeper.

For structures supported on mat foundations, over-excavation should be conducted to a depth of at least 5 feet below preconstruction site grades, or to 3 feet below the bottom of the foundation, whichever provides the deeper fill. The exposed subgrade should be scarified to a depth of 8 inches, moisture conditioned and compacted as recommended in this report prior to backfilling the excavation with engineered fill.

For pavement areas and exterior slabs beyond the overbuild zone for structures, over-excavation should be conducted to a depth of at least 12 inches below preconstruction site grades, the depth to remove 12 inches below soils disturbed during the removal of vineyard, or to 12 inches below the aggregate base, whichever provides the deepest fill. The exposed subgrade should be scarified to a depth of 8 inches, moisture conditioned and compacted as recommended in this report prior to backfilling the excavation with engineered fill.

EXECUTIVE SUMMARY (cont.)

Given the magnitude of the foundation loads anticipated for some of the heavier structures (i.e., the HRSG, GSU, steam turbine and combustion turbine, etc.) and the storage tanks, the estimated static settlements for some of the structures exceed the allowable settlements, even when foundations are supported on the minimum depth of engineered fill recommended in this report. The remaining proposed structures will meet the allowable static settlements when over-excavated and compacted as recommended in this report. Alternate measures will be required for these heavier loaded structures to reduce the potential static settlements. The alternatives include surcharging the building areas until a sufficient amount of settlement has occurred, supporting the foundations on rammed aggregate piers (i.e., Geopiers), supporting the structures on a deep foundation system, or conducting a deep soil mixing program are recommended to meet the allowable static settlements for these structures. Based on our evaluations and the foundation loads provided for the other improvements, over-excavation and compaction of engineered fill below foundations is recommended to reduce the potential for static settlement to meet the allowable settlements. As part of the supplemental geotechnical engineering investigation, additional borings should also be drilled and supplemental laboratory testing should be conducted when site plans are finalized.

The site is not located in an earthquake fault zone designated pursuant to the Alquist-Priolo Earthquake Fault Zoning Act (1972). The potential for fault rupture at the site is low. The site is not located in a Seismic Hazard Zone delineated in response to the Seismic Hazards Mapping Act (1990) for liquefaction and landslide hazards. The nearest known active or potentially active fault is the Clovis fault, located about 21.8 kilometers northeast of the site.

The peak horizontal site acceleration with a 10 percent probability of being exceeded in 100 years (Upper Bound Earthquake ground motion) was determined to be 0.17g. The ground motion of 0.17g was used for the liquefaction and seismic settlement analyses.

Total seismic settlements of ¾ inch were estimated as a result of shaking from the Upper Bound Earthquake ground motion. Differential seismic settlements of ⅜ inch in 40 linear feet should also be anticipated. Seismic settlements should be considered for design in addition to the estimated static settlements. Based on the magnitude of the estimated static and seismic settlements and the depth of influence of the proposed foundation loads, a supplemental investigation is recommended using Cone Penetrometer Testing (CPT) methods to provide subsurface data for use in preparation of final design recommendations. The results of CPT analyses are also helpful in evaluation of alternative ground improvement and other foundation support types such as soil mixing, rammed aggregate piers and deep foundations.

Based on the Flood Insurance Rate Maps, distributed by the Federal Emergency Management Agency, the site is located in Zone X. Zone X denotes "Areas of 500-year flood; areas of 100-year flooding with average depths less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 100 year flood."

Results of chemical testing conducted on three (3) soil samples collected from the ground surface to a depth of 3 feet BSG indicated pH values of 6.2, 6.2, and 6.6; and minimum resistivity values of 11,000; 16,000; and 37,000 ohm-centimeters, respectively. Based on the resistivity values, the soils exhibit corrosion potential in the range of "mildly corrosive" to "relatively less corrosive."

EXECUTIVE SUMMARY (cont.)

Corrosion of concrete due to sulfate attack is not anticipated based on a low detected concentration of sulfates determined for the near-surface soils. According to Table 19-A-4 of the 2001 California Building Code, the concentration of sulfates falls in the negligible classification.

This executive summary should not be used for design or construction and should be reviewed in conjunction with the attached report.

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GEOLOGIC HAZARD EVALUATION

KINGS RIVER CONSERVATION DISTRICT

PROPOSED POWER PLANT

NORTHEAST OF THE INTERSECTION OF

EAST DINUBA AVENUE AND SOUTH BETHEL AVENUE

FRESNO COUNTY, CALIFORNIA

Twining Project Number: F08306.02-01

1.0 INTRODUCTION

This report presents the results of a preliminary geotechnical engineering investigation and geologic hazard evaluation for the proposed power plant to be located northeast of the intersection of East Dinuba Avenue and South Bethel Avenue in Fresno County, California. The Twining Laboratories, Inc. (Twining) was authorized by Mr. Jim Richards with Kings River Conservation District (KRCD) to conduct this preliminary geotechnical engineering investigation.

The purpose of the investigation was to conduct a subsurface exploration and a laboratory testing program for the proposed project, evaluate the data collected during the field exploration and laboratory testing, and provide preliminary geotechnical engineering design parameters for use in the preliminary assessment of the project. It is our understanding that the details of the proposed power plant are being developed (loads, foundation sizes, etc.) and the final locations of the structures have not been determined. Accordingly, the evaluations, conclusions, and recommendations provided in this report are preliminary and should be used for preliminary design and planning purposes. Additional field exploration and preparation of a design level report based on the final proposed site configuration, loads, and structural details, etc. will be required prior to final design, bidding and construction of the project.

The contents of this report include the purpose of the investigation and the scope of services provided. The site history, previous studies, existing site features, and proposed construction are discussed. In addition, a description of the investigative procedures used and the subsequent findings obtained are presented. Finally, the report provides an evaluation of the findings, general conclusions, and related recommendations. The three report appendices contain the drawings (Appendix A), the logs of test borings (Appendix B), and the results of laboratory tests (Appendix C).

The Geotechnical Engineering Division of Twining, headquartered in Fresno, California, performed the investigation.

2.0 PURPOSE AND SCOPE OF INVESTIGATION

2.1 Purpose: The purpose of this preliminary geotechnical engineering investigation was to conduct a field exploration and laboratory testing program, evaluate the field and laboratory data, and provide the following:

2.1.1 Geotechnical parameters for use in preliminary design and planning of foundations, slabs-on-grade, and development of lateral resistance;

2.1.2 Preliminary evaluation of liquefaction potential and seismic settlement at the site;

2.1.3 Evaluation of the potential for surface fault rupture at the site;

2.1.4 Evaluation of the potential for geologic hazards such as flooding, landslides, lurching, seiches and tsunamis, and volcanic activity to impact the site;

2.1.5 Recommendations for site preparation including placement, moisture conditioning, and compaction of engineered fill soils;

2.1.6 Preliminary recommendations for the design and construction of new asphaltic concrete (AC) and Portland cement concrete (PCC) pavements;

2.1.7 Preliminary recommendations for temporary excavations and trench backfill; and

2.1.8 Preliminary conclusions regarding soil corrosion potential.

This preliminary geotechnical engineering investigation report is provided specifically for Kings River Conservation District for the proposed power plant project referenced in the "Proposed Construction" section of this report. This preliminary geotechnical engineering investigation report is not a design level report and should not be used for final design of the project.

This investigation did not include a floodplain investigation, compaction tests, environmental investigation, or environmental audit.

2.2 Scope: The following scope of work was performed for this investigation:

2.2.1 A site plan entitled "General Arrangement," undated, provided by Utility Engineering Corporation, was reviewed and will be referred to as the site plan.

2.2.2 A proposed boring location map titled "New Plant Study Boring Location Plan," undated, provided by Utility Engineering Corporation, was reviewed and will be referred to as the KRCD boring location map.

2.2.3 A Phase I Report, entitled, "Phase I Environmental Site Assessment, 31.25 Acre Property, 9664 South Bethel Avenue, Fresno County, California." dated September 29, 2006, prepared by The Twining Laboratories, Inc. (herein referred to as the "Twining Phase I Environmental Report," was reviewed.

2.2.4 A site reconnaissance and the initial field exploration were conducted.

2.2.5 Laboratory tests were conducted to determine selected physical and engineering properties of the subsurface soils.

2.2.6 Mr. Jim Richards (KRCD), Mr. Taylor Matteson (KRCD), and Mr. Erik McElwain (Utility Engineering Corporation) were consulted during the investigation.

2.2.7 The data obtained from the investigation were evaluated to develop an understanding of the subsurface conditions and engineering properties of the subsurface soils.

2.2.8 The data obtained from the investigation were evaluated to develop an understanding of the subsurface soil conditions and engineering properties of the subsurface soils.

It should be noted that the effects of vibrating equipment, such as residual dynamic settlement, were not evaluated within the scope of this investigation.

3.0 BACKGROUND INFORMATION

The previous studies, site history, existing site features, and the proposed construction are summarized in the following subsections.

3.1 **Previous Studies:** A Phase I Environmental Site Assessment, prepared by Twining, dated September 29, 2006, herein referred to as the Twining Phase I Environmental Report, was reviewed.

The Phase I Environmental Report included a site area of approximately 31.25 acres planned for the power plant development. According to the Phase I Environmental Report, the parcel was used for agricultural purposes and the west-central boundary of the site was noted to be occupied by three (3) vacant structures. The structures noted in the Phase I Environmental Report consist of a vacant residence, a vacant wooden barn with metal roof, and a vacant detached wood-frame garage. Additionally, the Phase I Environmental Report stated that refuse items consisting of scrap metal, wood, tires, etc. were present within the vicinity of the vacant structures.

According to the Phase I Environmental Report, the site has been agriculturally developed, with the three (3) structures located in the central-western portion of the site, since 1937 or earlier. The Phase I Report also stated that a canal traversed the southern boundary of the site from 1950, or earlier, to at least 1987. Additionally, a 500-gallon underground storage tank (UST) was reportedly removed from the site approximately 15 years ago; however, the location of the tank was unknown.

The Phase I Report also stated that two active electric powered water wells and a septic system were located on the site.

No other previous geotechnical engineering, geological, or environmental studies conducted for this site were provided for review during this investigation. If available, these reports should be provided for review and consideration for this project.

3.2 <u>Site History</u>: At the time of the field exploration, the site was agriculturally developed as a grape vineyard and three structures were located near the western boundary of the site. The site has been agriculturally developed with the three (3) structures located near the western boundary of the site since 1937 or earlier. Additionally, a canal traversed the southern boundary of the site from 1950, or earlier, to at least 1987.

3.3 <u>Site Description</u>: The project site comprises approximately 31.25 acres located northeast of the intersection of East Dinuba Avenue and South Bethel Avenue in Fresno County, California. A site location map is presented on Drawing No. 1 in Appendix A. Based on our visual site reconnaissance on October 31, 2006, the site was agriculturally developed (grape vineyard) and three structures were located near the west-central boundary of the site. A water well, used for agricultural purposes, was noted near the western boundary of the site. Dirt access roads traversed the northern, southern, eastern, and central portions of the site.

The site was bound to the north by an existing water treatment plant and agricultural land; to the south by agricultural land; to the west by South Bethel Avenue and agricultural land beyond; and to the east by agricultural land and vacant land.

According to the 7½ minute series topographic map (Selma, California), produced by the United States Geological Survey (USGS), the site is located at 119.5727 degrees west longitude and 36.5951 degrees north latitude, with an elevation of approximately 325 feet above mean sea level (AMSL).

At the time of the field exploration, the topography of the project site was relatively flat. The site slopes slightly to the southwest and is approximately 2 to 4 feet higher on the east.

3.4 <u>Proposed Construction</u>: It is our understanding that the design details of the proposed power plant facility such as design loads and foundation types are not yet known. The project site plan shows the general arrangement of the proposed structures. It is also our understanding that the final locations of the structures have not been determined. Based on our review of the site plan it is anticipated that the proposed construction will include turbine generators, exhaust stacks, transformers, water tanks, pre-engineered buildings, and miscellaneous structures including various equipment structures (equipment pads, cooling tower, pumps, electrical boxes, etc.). Appurtenant construction is also anticipated to include asphaltic concrete and Portland cement concrete parking and driveways, and underground utilities.

Mr. McElwain (Utility Engineering Corporation) provided Twining with the following preliminary required bearing capacities and preliminary estimates of total dead and live loads for the proposed power plant improvements:

Table No. 1

Proposed Improvement	Preliminary Required Bearing Capacities (pound per square foot)	Preliminary Estimate of Total Dead and Live Loads (kips)
Warehouse (pre-engineered)	1,500	1.7 kips per linear foot for continuous foundations and 20 kips for column foundations
Generator Step-Up (GSU)	2,000	1,400
Steam Turbine	4,000	6,000
Combustion Turbine Generator (CTG)	5,000	5,000
Heat Recovery Steam Generator (HRSG)	4,000	20,000
Recycle Water Holding Tank	2,500	12,650
Cooling Towers	2,000	25,000
Water Treatment Building	4,000	4,000
Zero Liquid Discharge Treatment Area (ZLD)	3,000	3,000
Brine Holding Tank	3,000	9,950

Given the preliminary nature of the loading and foundation types at the time this report was prepared, the actual loads and bearing values may change during design. When final design loads are available they should be provided to our firm for review as soon as they are available.

Storage tanks are proposed for the brine holding tank (65 feet in diameter and 34 feet high), recycled water holding tank (80 feet in diameter and 40 feet high), demineralized water tank (35 feet in diameter and 32 feet high), and service/fire water (40 feet in diameter and 40 feet high). Based on the information provided by Mr. McElwain, the dead and live loads for the brine holding tank, recycled water holding tank, demineralized water tank, and service/fire water tank were estimated to be 9,950 kips, 12,560 kips, 2,885 kips, and 3,768 kips, respectively.

Elevated utility and electrical pipe racks are also planned as part of this project. These pipes will be attached to columns supported by shallow drilled concrete shafts. Structural loads were not provided for the pipe racks anticipated to be supported on pier foundations. For the purpose of this report, the anticipated loads for pipe racks were assumed to be about 30 kips axial compression, 4 kips uplift, and 4 kips lateral loading.

Grading plans were not available at the time this report was prepared. For the purposes of this report, maximum cuts and fills are anticipated to be on the order of 2 to 4 feet in order to achieve pad grades and provide positive drainage. In addition, over-excavation is recommended in this report to reduce the potential for static settlement. Twining should be provided with the preliminary grading plans as soon as they are available. If the proposed cuts and fills vary from those assumed for this report, additional recommendations may be required to supplement this report.

4.0 INVESTIGATIVE PROCEDURES

The field exploration and laboratory testing program conducted for this investigation are summarized in the following subsections.

4.1 Field Exploration: The field exploration consisted of a site reconnaissance, drilling test borings, soil sampling, and standard penetration tests.

4.1.1 Site Reconnaissance: The site reconnaissance consisted of walking the site and noting visible surface features. The reconnaissance was conducted by Mr. Zubair Anwar (Staff Engineer) on October 31, 2006. The features noted are described in the background information section.

4.1.2 Drilling Test Borings: The location of test borings drilled for the field exploration were selected by KRCD, based on the anticipated locations of some of the proposed structures, and presented in the Request for Proposal. Prior to drilling, the boring locations were surveyed by Provost and Prichard Engineering Group. Between October 31, 2006 and November 3, 2006, seventeen (17) test borings were drilled at the approximate locations selected by KRCD. The test boring locations are shown on Drawing No. 2 in Appendix A.

Under the direction of a Twining staff engineer, the test borings were drilled to depths of 20 to 60 feet below site grades (BSG) using a CME-75 drill rig equipped with 6⅝ inch outside diameter (O.D.) hollow stem augers. The soils encountered in the test borings were logged. The field soil classification was in accordance with the Unified Soil Classification System and consisted of particle size, color, and other distinguishing features of the soil.

The presence and elevation of free water, if any, in the borings were noted and recorded during drilling and immediately following completion of borings.

The initial sixteen (16) test boring locations were marked with wood stakes by KRCD prior to our field exploration. The location of the seventeenth (17^{th}) boring drilled was determined in the field by our firm. The boring locations, as described, should be considered accurate to within about 5 feet. Elevations of the test borings were not measured as a part of the investigation. The test borings were loosely backfilled with material excavated during the drilling operations; thus, some settlement should be anticipated.

4.1.3 **Soil Sampling:** Standard penetration tests were conducted in conjunction with the test borings, and both disturbed and undisturbed soil samples were obtained.

The standard penetration resistance, N-value, is defined as the number of blows required to drive a standard split barrel sampler into the soil. The standard split barrel sampler has a 2-inch O.D. and a 1⅜ inch inside diameter (I.D.). The sampler is driven by a 140-pound weight free falling 30 inches. The sampler is lowered to the bottom of the bore hole and set by driving it an initial 6 inches. It is then driven an additional 12 inches and the number of blows required to advance the sampler the additional 12 inches is recorded as the N-value.

Relatively undisturbed soil samples for laboratory tests were obtained by pushing or driving a California modified split barrel ring sampler into the soil. The soil was retained in brass rings, 2.5 inches O.D. and 1 inch in height. The lower 6-inch portions of the samples were placed in close-fitting, plastic, airtight containers which, in turn, were placed in cushioned boxes for transport to the laboratory.

Soil samples obtained were taken to Twining's laboratory for classification and testing.

4.2 **Laboratory Testing:** The laboratory testing was programmed to determine selected physical and engineering properties of the soils underlying the site. The tests were conducted on disturbed and undisturbed samples representative of the subsurface material.

The results of laboratory tests on samples obtained from the test borings are summarized on Figure Numbers 1 through 21 in Appendix C. These data, along with the field observations, were used to prepare the final test boring logs in Appendix B.

5.0 FINDINGS AND RESULTS

The findings and results of the field exploration and laboratory testing are summarized in the following subsections.

5.1 **Surface Conditions:** At the time of the field exploration, the site was in agricultural use with a grape vineyard and three structures located near the western boundary of the site. A water well, used for agricultural purposes, was also noted near the western boundary of the site. Dirt access roads traversed the northern, southern, eastern, and central portions of the site.

5.2 Soil Profile: The near surface soils generally consisted of loose to dense silty sands and medium stiff to stiff sandy silts. The near surface silty sand and sandy silt soils were encountered from the ground surface to depths ranging from about five (5) to twenty-five (25) feet BSG. Below the near surface silty sand and sandy silt soils, various interbedded layers of very stiff to hard sandy silt, loose to dense silty sands, and loose to medium dense poorly graded sands were encountered to the maximum depth explored of 60 feet BSG. Additionally, a loose clayey sand layer was encountered in one (1) of the seventeen (17) borings.

The foregoing is a general summary of the soil conditions encountered in the test borings drilled for this investigation. Detailed descriptions of the soils encountered at each test boring are presented on the logs of borings in Appendix B. The stratification lines shown on the logs represent the approximate boundary between soil types; the actual in-situ transition may be gradual.

5.3 Soil Engineering Properties: The following is a description of the soil engineering properties as determined from our field exploration and laboratory testing.

Near Surface Silty Sands and Sandy Silts: The near surface sandy silts, encountered in the majority of the borings, were medium stiff to stiff as indicated by standard penetration resistance, N-values, ranging from 4 to 11 blows per foot. The near surface silty sands were loose to dense as indicated by standard penetration resistance, N-values, ranging from 4 to 40 blows per foot. The near surface silty sands (between depths of 3½ and 6½ feet BSG) exhibited moderate compressibility characteristics as indicated by the results of three (3) consolidation tests (5.4, 5.3, and 5.4 percent consolidation under a load of 8 kips per square foot). Upon inundation (wetting), the samples exhibited moderate collapse potential (2.4, 2.3, and 2.4 percent consolidation when wetted under a load of 2.0 kips per foot). The near surface sandy silt soils (between depths of 3½ and 5 feet BSG) exhibited moderate compressibility characteristics as indicated by the results of one (1) consolidation test (7.0 percent consolidation under a load of 8 kips per square foot). Upon inundation (wetting), the sample exhibited moderate collapse potential (2.4 percent consolidation when wetted under a load of 2.0 kips per square foot). The deeper silty sand soils (between depths of 10 and 11½ feet BSG) had low compressibility characteristics as indicated by the results of one (1) consolidation test (3.3 percent consolidation under a load of 8 kips per square foot). Upon inundation (wetting), the sample exhibited low collapse potential (1.3 percent consolidation when wetted under a load of 2.0 kips per square foot). The results of one (1) expansion index test performed on a near surface sandy silt sample indicated a very low expansion potential (based on an expansion index value of 0). Two (2) direct shear tests performed on near surface samples indicated internal angles of friction values of 26 and 27 degrees with cohesion values of 0 and 10 pounds per square foot, respectively.

Deeper Soils: The deeper soils consisted of various interbedded layers of loose to dense silty sands, as indicated by standard penetration resistance, N-values, ranging from 6 to 49 blows per foot, stiff to hard sandy silts, as indicated by standard penetration resistance, N-values, ranging from 9 to 51 blows per foot, and loose to medium dense poorly graded sands, as indicated by standard penetration resistance, N-values, ranging from 4 to 16 blows per foot to the maximum depth explored of 60 feet BSG.

R-Value Tests: R-value tests were conducted on four (4) near surface soil samples collected between the ground surface and 3 feet BSG. The results of the tests indicated R-values of 57, 58, 63, and 65.

Chemical Tests: The results of chemical testing conducted on three (3) soil samples collected between the ground surface and a depth of 3 feet BSG indicated pH values of 6.2, 6.2, and 6.6; and minimum resistivity values of 11,000; 16,000; and 37,000 ohm-centimeters, respectively. The results also indicated 0.0016, 0.00074, and 0.00066 percent by weight concentrations of sulfate, and "none detected" (reporting limit of 0.00060) percent by weight concentrations of chloride.

5.4 Groundwater Conditions: Groundwater was encountered at depths ranging from 35 to 40 feet BSG, in ten (10) of the seventeen (17) borings drilled at the time of our field investigation. The Phase I Environmental Report indicated that groundwater was reported to be at 35 feet BSG.

Water well data reviewed on the California Department of Water Resources online data base, indicated a water well with a similar elevation to the project site had a groundwater level of 20 feet BSG during the last measurement taken in 1981.

It should be recognized, however, that water table elevations fluctuate with time, since they are dependent upon seasonal precipitation, irrigation, land use, and climatic conditions as well as other factors. Therefore, water level observations at the time of the field investigation may vary from those encountered both during the construction phase and the design life of the project. The evaluation of such factors was beyond the scope of this investigation and report.

6.0 EVALUATION

The data and methodology used to develop conclusions and recommendations for preliminary design are summarized in the following subsections. The evaluation was based upon the subsurface conditions determined from the investigation and our understanding of the proposed construction.

The primary geotechnical issues that could impact the design and construction of the proposed project are: 1) the potential for excessive collapse settlement of soils in the upper 5 feet BSG, 2) excessive static settlements (estimated to occur as a result of the consolidation in deeper soils) of some of the proposed structures with relatively large foundations and bearing loads, 3) the potential for post construction seismic settlements, and 4) the potential for excessive organics in the near surface soils resulting from the current and previous agricultural crops. These issues are discussed in further detail in the following section. It should be noted that the effects of vibrating equipment, such as the residual dynamic settlement, are dependent on the characteristics of the machine vibrations and were not evaluated within the scope of this investigation.

6.1 Surface Conditions: At the time of the field exploration, a portion of the site was in agricultural use consisting of a grape vineyard and a portion was occupied by three (3) structures located near the western boundary of the site. An existing water well, used for agricultural purposes, was also noted near the western boundary of the site. Dirt access roads traversed the northern, southern, eastern, and central portions of the site.

Based on the existing grape vineyards and structures, removal of organics, roots, existing structures, foundations, septic systems, underground utilities, etc. will be required as part of site preparation. As part of site preparation, stripping should be conducted to remove the organic rich soils, vegetation and root structures from previous agricultural crops. The stripping depths should be sufficient to remove existing vegetation, roots, residual limbs, organic matter, etc. and remove those soils containing more than 3 percent by dry weight organic material as determined by loss-on-ignition testing (ASTM D2974). For estimating purposes, a minimum stripping depth of 6 inches should be used. The actual depth of stripping should be reviewed by our firm at the time of construction.

Roots, organic matter, etc., should not be disced into the near-surface soils. These materials should be raked and hand-picked, as necessary, to ensure proper removal.

Based on our experience, vineyard removal will cause disturbance of the surface soils to a depth of 3 to 4 feet BSG. Methods of vineyard removal that cause deeper soil disturbance should be avoided if possible. Our firm should be contacted prior to commencement of vineyard removal to discuss with the contractor the proposed removal methods. Twining should observe and document the removal of the existing vineyard. Based on the anticipated depth of soils to be disturbed, over-excavation and compaction are recommended in this report in the areas occupied by vineyards.

The Phase I Environmental report indicates up to two (2) water wells are present on the site and that a septic system is also present on the site. In addition to the existing structures, subsurface irrigation piping related to agricultural activity was noted on the site. Accordingly, existing foundations and subsurface improvements that are present on the site will need to be removed and the resulting excavations backfilled as engineered fill as part of the site preparation.

6.2 **Soil Collapse and Static Settlements:** Native soil samples collected from the upper 5 feet BSG exhibited a moderate potential for collapse when saturated under the approximate effective stress anticipated from the assumed foundation loads. Since the near surface subgrade soils are relatively permeable to the downward migration of moisture, and the in-place moisture contents of the soils were relatively low, it is likely that these soils could be subjected to higher moisture conditions over the design life of the project. Laboratory testing indicates that, if soil moisture contents increase considerably after development, collapse-type settlement could occur. Structures supported on collapsible soils are likely to settle excessively and unequally once the soils are exposed to excessive moisture due to surface runoff, landscape irrigation, etc. Cracking and distress to the structures could develop and would become more pronounced as the moisture content of the soils increases. In addition, large settlements of structures located on collapsible soils have been caused by the introduction of water from poor drainage, defective drains and sumps, broken water pipes, and leaky sewers.

Considering the potential for soil collapse (hydroconsolidation), the near surface soils in their native condition will not provide adequate support for foundations and interior slabs-on-grade. Accordingly, the near surface native soils are not considered suitable to provide direct support for the proposed foundations. Therefore, recommendations for over-excavation and compaction of the near surface soils are included in this report to reduce the potential for hydroconsolidation.

6.3 **Liquefaction and Seismic Settlement:** Preliminary evaluations of liquefaction and seismic settlement were conducted based on the results of the preliminary field exploration, including SPT N-values. Liquefaction in this instance describes a phenomenon in which a saturated, cohesionless soil loses strength during an earthquake as a result of induced shearing strains. Lateral and vertical movements of the soil mass, combined with loss of bearing usually results. Research has shown that liquefaction potential of soil deposits induced by earthquake activity depends on soil types, void ratio, groundwater conditions, duration of shaking, and confining pressure over the potentially liquefiable soil mass. Fine, well sorted, loose sand, high groundwater conditions, higher intensity earthquakes, and relatively long duration of groundshaking are the requisite conditions for liquefaction.

One of the most common phenomena that occurs during seismic shaking is the induced settlement of loose, unconsolidated sediments. This can occur in unsaturated and saturated granular soils, however, seismic settlements are typically largest where liquefaction occurs (saturated soils). The results of the analyses indicate liquefaction would not occur as a result of the Upper Bound Earthquake horizontal ground acceleration of 0.17g (ground motion with a 10 percent chance of being exceeded in a 100-year period) and a predominant earthquake magnitude of 6.3 (see subsection 6.10 for further description of the seismic conditions). However, potential seismic settlement of dry soils would be anticipated in the event of moderate to strong ground shaking and should also be considered in design.

Groundwater was encountered in ten (10) of the seventeen (17) borings at the time of our field exploration, at depths ranging from 35 feet to 40 feet BSG. Water well data, provided by the California Department of Water Resources, indicated a water well with a similar elevation to the project site had a groundwater level of 20 feet BSG during the last measurement taken in 1981. Accordingly a groundwater depth of 20 feet was used for the liquefaction and seismic settlement analyses.

Seismic settlement analysis was conducted based on soil properties revealed by test borings and the results of laboratory testing. The analyses were conducted using a spread sheet developed based on the computer program LIQUEFYPRO by CivilTech. An Upper Bound Earthquake horizontal ground acceleration of 0.17g (ground motion with a 10 percent chance of being exceeded in a 100-year period) and a predominant earthquake magnitude of 6.3 were used. The N-values generated from the standard penetration test (SPT) results and soil parameters, such as wet unit weight, N-value, and fines content were input for the soil layers encountered throughout the depths explored (see test boring logs, Appendix B).

Based on our evaluations, a preliminary total seismic settlement of ¾ inch was estimated for the site. Differential seismic settlements are preliminarily estimated to be on the order of about ⅜ inch over a horizontal distance of 40 feet. These settlements should be considered for preliminary design in addition to the estimated static settlements discussed in this report for foundation design.

It has been our experience that additional field testing in the form of cone penetration tests (CPT) may generate data which would support refined estimates of total and differential seismic settlements. In contrast to the hollow-stem auger borings, CPT testing provides N-value (soil density) data at nearly continuous intervals and a more detailed soil density and penetration resistance profile. Since soil density strongly impacts seismic settlements, evaluation of CPT test results generally produce refined seismic settlement estimates when compared with data from hollow stem auger borings. The results of CPT analyses are also helpful in evaluation of alternative ground improvement and other foundation support types such as soil mixing, rammed aggregate piers (i.e., Geopiers) and deep foundations. Accordingly, it is recommended that a supplemental investigation be conducted to perform CPT soundings in order to produce refined seismic settlement estimates prior to the selection of final design recommendations for the stuctures where higher settlements are estimated. CPT soundings will help provide a more detailed soil density and penetration resistance profile to better evaluate the seismic settlement potential of the soils.

6.4 **Bearing Capacity:** The potential for excessive total and differential static settlements of foundations, structures and slabs-on-grade is a geotechnical concern that was evaluated for this building site. The increases in effective stress to underlying soils which can occur from new foundations and structures, placement of fill, withdrawal of groundwater, etc. can cause vertical deformation of the soils, which can result in damage to the overlying structure and improvements. The differential component of the settlement is often the most damaging. In addition, the allowable bearing pressures of the soils supporting the foundations were evaluated for shear and punching type failure of the soils resulting from the imposed foundation loads.

Net allowable soil bearing pressure is the additional contact pressure at the base of the foundations caused by the structure. The weight of the soil backfill may be neglected and the weight of concrete in the footings and slabs below grade may be neglected. The net allowable soil bearing pressure presented was selected to satisfy both the settlement criteria and Terzaghi bearing capacity equations for spread foundations. A minimum factor of safety of 3 was used to determine the allowable bearing capacity based on the Terzaghi equations.

6.5 **Foundations - General:** Proposed foundation types for this project include shallow spread foundations (to be used with conventional slabs-on-grade), mat foundations, drilled shaft foundations, and mat or ring foundations for support of storage tanks. A summary of the preliminary evaluations for the individual foundation types are provided in the following subsections and assume that the subgrade soils have been prepared in accordance with the recommendations provided in this report.

Based on the subsurface data and the results of laboratory testing performed as part of this investigation, preliminary static settlement calculations were performed. Considering the anticipated foundation loads discussed in this report, and the potential for static settlement (including collapse), the foundations should be supported on a near uniform thickness of engineered fill to reduce the static settlements. Given the magnitude of the foundation loads anticipated for some of the heavier structures (i.e., the HRSG, GSU, steam turbine and combustion turbine, etc.) and the storage tanks, the estimated static settlements for some of the structures exceed the allowable settlements, when foundations are supported on the minimum depth of engineered fill recommended in this report. Accordingly, alternate measures will be required to reduce the potential static settlements for some of these structures. The alternatives include surcharging the building areas until a sufficient amount of settlement has occurred, supporting the foundations on rammed aggregate piers (i.e., Geopiers), supporting the structures on a deep foundation system, or conducting a deep soil mixing program to reduce the compressibility of the native soils.

The net allowable soil bearing pressure is the additional contact pressure at the base of the foundations caused by the structure. The weight of the soil backfill may be neglected and the weight of concrete in the footings and slabs below grade may be neglected. The net allowable soil bearing pressures were selected to satisfy the Terzaghi bearing capacity equations for spread foundations. A factor of safety of 3 was used to determine the net allowable bearing capacity based on Terzaghi equations. The Schmertmann's method for static settlement of granular soils were used to estimate foundation settlements.

6.5.1 <u>Shallow Spread Foundations</u>: Shallow spread foundations are anticipated for the proposed pre-engineered buildings (i.e., relatively light structures). The preliminary design loads provided are 20 kips and 1.7 kips per foot for column and continuous foundations, respectively. Preliminary settlement calculations (including estimates of collapse settlement) indicate that shallow spread foundations placed directly on undisturbed native soils could experience a total static settlement in excess of the allowable settlements. To reduce total static settlements of shallow foundations, and differential static settlement to ½ inch in 40 feet, the foundations should be placed entirely on at least 3 feet of engineered fill, engineered fill which extends to a minimum depth of 4 feet below preconstruction site grades, or to undisturbed native soils, whichever is deeper.

Based on the assumed foundation loads, shallow spread foundations may be designed for an allowable soil bearing pressure of 1,500 pounds per square foot (net) for dead-plus-sustained live loads. The foundation bearing capacity may be increased by one-third for short duration wind or seismic loads. Total and differential static settlements of 1-inch and ½-inch, respectively, should be anticipated for design.

6.5.2 <u>Mat Foundations</u>: It is our understanding mat foundations are proposed for the CTG units, exhaust stacks, GSU transformers and Heat Recovery Steam Generator (HRSG). Use of a mat foundation in lieu of a conventional foundation and slabs-on-grade generally offers lower angular distortion across slabs subjected to relatively heavy loads, due to the greater stiffness of the mat foundations over conventional foundations.

Mr. McElwain (Utility Engineering Corporation) provided our firm with the estimated required bearing capacities and estimated total dead and live loads for the proposed power plant improvements, which are presented in Table No. 1 in this report under the section entitled, "Anticipated Construction."

Mr. McElwain (Utility Engineering Corporation) provided Twining with the following total allowable static and differential settlements noted in Table No. 2 for the proposed power plant improvements

Table No. 2

Proposed Improvement	Total Allowable Static / Differential Settlement
Generator Step-Up (GSU)	1 inch/ 1 inch
Steam Turbine	1 inch / ½ inch
Combustion Turbine Generator (CTG)	2 inches / 1 inch
Heat Recovery Steam Generator (HRSG)	2 inches / 1 inch
Cooling Towers	2 inches / 1 inch
Water Treatment Building	2 inches / 1 inch
Zero Liquid Discharge Treatment Area (ZLD)	2 inches / 1 inch

In general, in order to reduce the estimated static settlement, and the potential for hydroconsolidation, over-excavation and compaction should be conducted in areas to receive mat foundations to at least 5 feet below preconstruction site grades, or to 3 feet below the bottom of the foundations, whichever provides the deepest fill.

The magnitudes of differential settlements will be predominantly influenced by the distribution of vertical load across the foundation and the lateral variations in soil conditions. Based on the predominantly granular nature of the soils, it should be noted the majority of the static settlements would be anticipated to occur from application of the dead loads during construction. However, the design settlements estimated after the minimum recommended site preparation (over-excavation and compaction), for the GSU, HRSG, steam turbine, and combustion turbine exceed the allowable settlement criteria provided by Utility Engineering. Twining evaluated a deeper over-excavation (up to 15-20 feet) and compaction of engineered fill below the foundations for these improvements; however, our evaluations indicated conventional over-excavation and compaction would not sufficiently reduce the estimated static settlements to meet the allowable settlements. Accordingly, in addition to the minimum over-excavation recommended in this report, the following additional measures may be utilized to reduce the total static and differential settlements to meet the allowable settlement criteria for these improvements: 1) applying a soil surcharge to induce the majority of the static settlements to within acceptable levels prior to foundation construction; 2) support the improvements on a deep foundation system (i.e., drilled pier, driven pile foundations); 3) supporting the foundations on rammed aggregate piers (i.e., Geopiers), or 4) other ground improvement techniques such as deeper over-excavation or deep soil mixing. The estimated static settlements for the remaining planned improvements (i.e., improvements excluding the GSU, HRSG, steam turbine, combustion turbine and tanks) supported on the thickness of engineered fill recommended in this report will meet the allowable settlements noted in Table No. 2.

Based on the magnitude of the estimated static settlements and the depth of influence of the proposed loads, a supplemental investigation is recommended using Cone Penetrometer Testing (CPT) methods to provide subsurface data for use in preparation of final design recommendations and selection of the preferred measure to reduce the settlements to meet the allowable settlement criteria. Final estimates for differential settlement will need to be developed based on the final site and foundation loads.

Due to the close proximity of the heavy structures such as HRSG, Steam Turbine, and Combustion Turbine, differential settlements between structures which may be connected are also a potential concern that needs to be evaluated as a part of the final design level investigation. Similar measures such as those described above may be employed to reduce the potential for differential settlement between these improvements.

Mat foundation design is dependent on the k-value (modulus of subgrade reaction) of the slab subgrade soils, the loads and load distribution. Considering that the mat slab load distributions were not known at the time of preparation of this report, a preliminary modulus of subgrade reaction was estimated based on our experience with similar foundations at sites with similar soil conditions. A preliminary modulus of subgrade reaction of 30 pounds per square inch per inch should be used for preliminary design of the mat foundations for the project. It is recommended that the project structural engineers provide our firm with load distribution diagrams for all of the mat foundations showing the loads at the bottom of the foundations based on a k-value of 30. Based on the load distribution, a refined modulus of subgrade reaction value can be provided for use in final design.

6.5.3 <u>Drilled Shaft Pier Foundations</u>: It is anticipated that drilled shaft foundations will be required for the elevated pipe racks. In addition, deep foundations may also be considered as one of the options to mitigate static settlement of the heavier structures (i.e., HRSG, Steam Turbine and Combustion Turbine) as noted above.

Structural loads were not provided for the pipe racks at the time of preparation of this report. Based on the anticipated soil and groundwater conditions, and load estimates based on our experience with similar structures, it is anticipated drilled shaft foundations can provide adequate support for the proposed pipe racks provided the design parameters presented herein are utilized in design.

In determining the total allowable vertical load capacity for each pier foundation, skin friction may be considered. End bearing should not be considered due to the difficulty in ensuring that all loose soils (slough) are removed from the bottom of the drilled shaft. The uplift capacity should be based on the sum of the weight of the concrete in the piers and one-half (½) of the allowable downward skin friction.

The soils anticipated to be encountered in the deep foundation excavations are considered to have a low "stand-up" capacity, and it is anticipated that caving will occur in these open excavations. Therefore, it should be anticipated that casing will be required to prevent these soils from caving into the excavations. The casing should be slowly removed as the concrete is placed.

Preliminary design capacities are provided in the Recommendations section of this report. The allowable values may be increased by one third for short duration live loads such as wind and earthquake loads. A total static settlement of 1 inch and a differential static settlement of ½ inch between columns for the pipe racks should be considered for design considering the preliminary design parameters provided in this report.

The contractor hired to conduct the drilling operation for the proposed drilled shaft pier foundations should have a minimum of five years experience in the construction of drilled shaft foundations and should have conducted at least five similar projects in the past three years. After selection of the contractor and subcontractors has been completed and before construction begins, a preconstruction meeting is recommended prior to construction of the drilled shaft foundations to discuss drilling and construction procedures.

6.6 **Tank Foundations:** It is understood that tanks may be supported on either mat or ring foundations. The tank structures and/or foundations should be designed and reinforced for the anticipated total and differential settlements. Settlements were estimated based on the preliminary anticipated loads and assuming the recommended site preparation (over-excavation and compaction) is conducted in accordance with this report.

A structural engineer experienced in tank and foundation design should recommend the foundation design details based on the following anticipated settlements.

Table No. 3

Proposed Improvement	Total Allowable static / differential settlement
Recycle water holding tank	3 inches / 2 inches
Brine holding tank	3 inches / 2 inches
Demineralized water tank	3 inches / 2 inches
Service/fire water tank	3 inches / 2 inches

Maximum allowable contact pressures for the tank bottoms of 2,500 psf, 3,000 psf, 3,000 psf, and 3,000 psf for the recycle water holding tank, brine holding tank, demineralized water tank, and service/fire water tank, respectively, were used for preliminary design. The estimated settlements exceed the allowable settlements for the proposed tanks when supported on native soils and/or engineered fill. Accordingly, alternate methods of site preparation will be required to reduce the anticipated settlements to meet the allowable criteria noted in Table No. 3. These methods include specialty ground improvement methods (i.e., deep soil mixing, etc.), surcharging the areas of the proposed tanks, supporting the tanks on rammed aggregate piers (i.e., Geopiers), designing additional camber into the bottom of the tank shell to accommodate the estimated settlements, or a combination of these methods. As part of this report, Twining evaluated a deeper over-excavation (up to 15-20 feet in depth) and compaction of engineered fill below the tanks; however, our evaluations indicated additional over-excavation and compaction of engineered fill below the tanks would not reduce the estimated static settlements to meet the allowable settlements. Based on the magnitude of the estimated static settlements and the depth of influence of the proposed loads, a supplemental investigation is recommended using Cone Penetrometer Testing (CPT) methods to provide subsurface data for use in preparation of final design recommendations. In our opinion, the above methods may be used to support the proposed tanks and meet the required bearing capacity and allowable settlement criteria.

Net allowable soil bearing pressure is the additional contact pressure at the base of the foundations caused by the structure. The weight of the soil backfill may be neglected. The net allowable soil bearing pressure presented was selected to satisfy both the settlement criteria and Terzaghi bearing capacity equations for spread foundations. A factor of safety of 3 was used to determine the allowable bearing capacity based on Terzaghi equations. Schmertmann's method was used to estimate foundation settlements.

Damage to connections between exterior locations and the tank pad due to differential settlements can be reduced by pre-loading the tank for about two to three weeks to induce up to about 50 percent of the anticipated settlements. Flexible connections for inlet/outlet pipes should be designed to account for the estimated differential movement without damage (includes a safety factor). After one year, with at least 75 percent of the maximum load, the flexible connections can be replaced with connections designed to withstand ½ inch of differential movement.

It is recommended that survey targets be placed and elevations established at four locations around the tanks before the tanks are loaded so that elevations can be monitored during loading. During the initial loading, it is recommended the elevations be surveyed on a regular basis. After initial loading, elevations should be checked at a regular frequency, such as quarterly. The data should be provided to our firm and the structural engineer as soon as it is available. Additional recommendations should be made based on the elevation measurements.

6.7 <u>Construction of Slabs-On-Grade</u>: Several issues need to be considered to limit the potential for damage to slabs during construction. These issues include: 1) differential slab movement at interior columns of buildings; 2) aggregate base sections below slabs, and 3) crane and construction equipment loads on the slabs/foundations.

Depending on the sequence of slab loading and the location of wall construction and erection of structures, damage to slabs from differential loading conditions could occur. Therefore, provisions for settlement between foundation and slabs should be provided for in design and construction to prevent damage.

The method of interior column construction for buildings can also potentially damage the overlying slabs. In some cases, the subgrade preparation for the slab is not continuous across the top of spread footings. Often the zone above the top of structural footings is backfilled with concrete during slab placement. This results in a differential slab support condition which often causes cracking at the soil/base-to-concrete transition. This crack appears as an outline of the underlying footing at the floor surface. The potential for this type of slab cracking can be reduced by backfilling the zone above the top of the footing and below the bottom of slabs with an approved backfill material and/or an aggregate base section below the floor slab. This procedure will provide more uniform support for the slabs which should reduce the potential for cracking.

It has been our experience that placing concrete for the concrete slabs-on-grade by the tailgating method can cause subgrade instability due to the high frequency of concrete trucks which travel across the prepared subgrade. Compacted subgrade can experience instability under high traffic loads resulting in heaving and depressions in the subgrade during critical pours. This condition becomes more critical during wet winter and spring months. A layer of aggregate base (AB) can reduce the potential for instability under the high frequency loading of concrete trucks. Also, the improved support characteristics of the AB can be used in the design of the slab sections. Therefore, it is recommended to utilize a slab design with at least 6 inches of AB for constructability and design purposes.

6.8 <u>Faulting and Seismicity</u>: The site is not located in an earthquake fault zone designated pursuant to the Alquist-Priolo Earthquake Fault Zoning Act (1972). The potential for fault rupture at the site is low. The site is not located in a Seismic Hazard Zone delineated in response to the Seismic Hazards Mapping Act (1990) for liquefaction and landslide hazards. The nearest known active or potentially active fault is the Clovis fault, located about 21.8 kilometers northeast of the site.

The 2001 California Building Code (CBC), Figure 16.2 (Seismic Zone Map of the United States) indicates that the site is in seismic zone 3. Estimates of seismic ground motion were developed and were used in the liquefaction hazard analyses. The "Upper Bound Ground Motion," as defined in Section 1627 of the California Building Code (CBC), is the seismic ground motion having a 10 percent probability of being exceeded in a 100-year period. The probabilistic analyses described in this section were used to determine the Upper Bound Earthquake ground motion.

Probabilistic ground motion evaluation requires use of a seismicity model and ground motion attenuation functions to approximate the modification of seismic waves between the earthquake hypocenter (source) and the site. The seismicity model, including the location and fault parameters (such as slip rate, fault length, magnitude and rupture area) of faults capable of impacting the site, was based on published geologic papers and corresponds with those listed in the California Geological Survey (CGS) database entitled "California Fault Parameters" (Revised 2002). Multiple probabilistic evaluations were conducted using the FRISKSP computer program and the faults indicated as those active and potentially active faults listed in the "California Fault Parameters" (Revised 2002) database.

Our evaluation considered the average of the predicted design basis ground motions for three separate analyses incorporating the ground motion attenuation relationships of Boore (1997), Sadigh (1997), and Abrahamson and Silva (1997) for the faults within 160 kilometers (100 miles) of the site. The average of the Upper Bound Earthquake ground accelerations (based on the above attenuation relationships) was determined to be 0.17g. Deaggregation of the probabilistic analyses was conducted to determine the predominant earthquake magnitude contributing to the Upper Bound Earthquake ground acceleration. An earthquake magnitude of 6.3 represents the predominant earthquake magnitude for the site. This ground motion represents a value not weighted for magnitude. Magnitude weighting was conducted in the liquefaction analysis.

6.9 Seismic Coefficients: It is anticipated that the 2001 CBC will be used for structural design, and that seismic site coefficients are needed for design. Based on the 2001 CBC Table 16-J, the site is classified as a stiff soil S_D site with standard penetration resistance, N-values, generally between 15 and 50 blows per foot in the upper 100 feet BSG.

The site coefficients for acceleration and velocity are based on the distance and activity of the local faults. Digitized seismic models published by the CGS indicate that no active faults are located within 15 kilometers of the site. The site does not require near-source corrections (CBC Tables 16-S and 16-T) based on a seismic source Type C. Therefore, the values of the near-source acceleration factor, Na, and the near-source velocity factor, Nv, may be taken as 1.0. Based on these values, the seismic acceleration coefficient, Ca (Table 16-Q), would be 0.36, and the seismic velocity coefficient, Cv (Table 16-R), would be 0.54.

6.10 Flooding: Based on the Flood Insurance Rate Maps, distributed by the Federal Emergency Management Agency, the site is located in Zone X. Zone X denotes "Areas of 500-year flood; areas of 100-year flooding with average depths less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 100 year flood."

The site is located within the Dam Inundation area from the Pine Flats Dam. The potential for breach of the existing dam is unknown.

6.11 Landslides: Due to the relatively flat relief at the site, the potential for landslides on this site is considered to be low.

6.12 Seiches and Tsunamis: A seiche is a wave generated by the periodic oscillation of a body of water whose period is a function of the resonant characteristics of the containing basin as controlled by its physical dimensions. These periods generally range from a few minutes to an hour or more. Since no significant bodies of water are located near the site, seiches are not considered a potential hazard at the site.

Tsunamis are waves generated in oceans from seismic activity. Due to the inland location of the site, tsunamis are not considered a potential hazard.

6.13 Volcanic Activity: The closest areas of Quaternary volcanism are the Mono Lake and Mammoth Mountain area (including Sadler Peak) located approximately 80 miles (129 km) northeast of the site and the Owens Valley area located approximately 75 miles (120 km) east of the site, (Jennings, 1994). These areas contain cinder cones and volcanic flows dated as young as approximately 100 to 200 years before present (Jennings, 1994). No other areas of volcanism are known to be located within 100 miles (160 km) of the site (Jennings, 1994). Based on the distance of the aforementioned volcanic areas from the site and age of activity, the prospect for lava flows or significant ash falls at the site during the design life of the structures is considered low.

6.14 Asphaltic Concrete Pavements: Asphaltic concrete pavement structural sections are presented in the Recommendations section of this report. The structural sections were designed using the gravel equivalent method in accordance with Chapter 600 of the California Department of Transportation Highway Design Manual. The analysis was based on traffic indices ranging from 5 to 8 at half point increments. It should be noted that if pavements are constructed prior to the building construction, the above traffic index values may be too low. If the pavements are placed prior to construction, or if more frequent truck traffic is anticipated Twining should re-evaluate the traffic index values.

The anticipated subgrade soils are silty sands and sandy silts. The subgrade support characteristics of the native silty sand soils were evaluated by Resistance (R)-value tests. The results of the tests indicated R-values of 57, 58, 63, and 65. In order to account for potential soil variability, for the purpose of design, an R-value of 40 was used for preliminary design.

6.15 Corrosion Protection: The risk of corrosion of construction materials relates to the potential for soil-induced chemical reaction. The rate of deterioration depends on soil resistivity, texture, acidity, and chemical concentration. The evaluation was based on the results of the analyses of a near-surface soil sample collected from the depths of 0 to 3 feet BSG.

Results of chemical testing conducted on three (3) soil samples collected from the ground surface to a depth of 3 feet BSG indicated pH values of 6.2, 6.2, and 6.6; and minimum resistivity values of 11,000; 16,000; and 37,000 ohm-centimeters, respectively. Based on the resistivity values, the soils exhibit corrosion potential in the range of "mildly corrosive" to "relatively less corrosive." The results also indicated 0.0016, 0.00074, and 0.00066 percent by weight concentrations of sulfate, and not detected (reporting limit of 0.00060) percent by weight concentrations of chloride, respectively. The results also indicated 0.0016, 0.00074, and 0.00066 percent by weight concentrations of sulfate, and not detected (reporting limit of 0.00060) percent by weight concentrations of chloride, respectively, for the three (3) samples analyzed. Based on these concentrations, the soils exhibited a low potential for sulfate exposure to concrete. If piping or concrete are placed in contact with deeper soils or engineered fill, these soils should be analyzed to evaluate the corrosion potential of these soils.

If the manufacturers or suppliers cannot determine if materials are compatible with the soil corrosion conditions, a professional consultant, i.e., a corrosion engineer, with experience in corrosion protection should be consulted to provide design parameters. Twining does not provide corrosion consulting services.

7.0 CONCLUSIONS

Based on the data collected during the field and laboratory investigations, our geotechnical experience in the vicinity of the project site, and our understanding of the proposed construction, the following preliminary general conclusions are presented.

7.1 This preliminary report includes evaluation of the estimated settlements for the proposed conventional shallow spread, mat, and/or drilled pier foundations based on assumed foundation loading. Given the magnitude of the foundation loads anticipated for some of the heavier structures (i.e., the HRSG, GSU, steam turbine and combustion turbine, etc.) and the storage tanks, the estimated static settlements for some of the structures exceed the allowable settlements, when foundations are supported on the minimum depth of engineered fill recommended in this report. Accordingly, alternate measures will be required for these structures to reduce the potential static settlements. The alternatives include surcharging the building areas until a sufficient amount of settlement has occurred, supporting the foundations on rammed aggregate piers (i.e., Geopiers), supporting the structures on a deep foundation system, or conducting a deep soil mixing program are recommended to meet the allowable static settlements for these structures. Based on our evaluations and the foundation loads provided for the other improvements, over-excavation and compaction of engineered fill below foundations is recommended to reduce the potential for static settlement to meet the allowable settlements. Based on the data obtained, the site is considered geotechnically suitable for the proposed construction when the recommendations in this report are followed. It should be noted this report is considered preliminary in nature. As part of the supplemental geotechnical engineering investigation, additional borings should also be drilled and supplemental laboratory testing should be conducted when site plans are finalized.

7.2 Based on the anticipated static settlements and the magnitude of the anticipated loading for some of the structures, a supplemental geotechnical investigation using cone penetrometer testing (CPT) is recommended to provide design-level recommendations for the proposed improvements.

7.3 The near surface soils consisted of loose to dense silty sands and medium stiff to stiff sandy silts. The near surface silty sand and sandy silt soils were encountered to depths ranging from about five (5) to twenty-five (25) feet BSG. Below the near surface silty sand and sandy silt soils, various interbedded layers of very stiff to hard sandy silt, loose to dense silty sands, and loose to medium dense poorly graded sands were to maximum depth explored of 60 feet BSG.

7.4 Based on laboratory data and field exploration (N-value) test results, the soils are anticipated to exhibit low shear strength, moderate compressibility and low to moderate collapse characteristics, and good to excellent pavement characteristics.

7.5 The site is not located in a mapped Seismic Hazard Zone for liquefaction. Liquefaction is not anticipated to occur as a result of the Upper Bounds Earthquake ground motion. Accordingly, the potential for lateral spreading and/or sand boils (surface manifestations of liquefaction) is considered low.

7.6 A total seismic settlement of ¾ inch was estimated as a result of shaking from the Upper Bound Earthquake ground motion. A differential seismic settlement of ⅜ inch in 40 linear feet should also be anticipated. Seismic settlements should be considered for design in addition to the predicted static settlements.

7.7 Considering the potential for soil collapse (hydroconsolidation), the near surface site soils will not provide adequate support for foundations and interior slabs-on-grade. Thus, minimum recommendations for over-excavation and compaction of the near surface soils are presented in this report to reduce the potential for hydroconsolidation. However, as noted above, due to the relatively high structural loads required for this project, and our estimates of static settlement occurring even after the over-excavation of the near surface subgrade soils, it is anticipated that some of the heavier structures will require additional measures to mitigate the anticipated settlements. It is recommended that these measures be evaluated in conjunction with the results of the future CPT testing.

7.8 Groundwater was encountered ranging from depths of 35 feet to 40 feet BSG, in ten (10) of the seventeen (17) borings at the time of our field investigation. The Phase I Environmental Report indicated that groundwater was reported to be at 35 feet BSG. Well data, provided by the California Department of Water Resources, indicated a water well with a similar elevation to the project site had a groundwater level of 20 feet BSG during the last measurement taken in 1981.

7.9 The site is not located in an Alquist-Priolo Earthquake Fault Zone for fault rupture and the potential for fault rupture is low.

7.10 The potential for landslides or ground lurching at the project site is low.

7.11 Based on the Flood Insurance Rate Maps, distributed by the Federal Emergency Management Agency, the site is located in Zone X. Zone X denotes "Areas of 500-year flood; areas of 100-year flooding with average depths less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 100 year flood." The site is located within the Dam Inundation area from the Pine Flats dam. The potential for breach of the existing dam is unknown.

8.0 RECOMMENDATIONS

Based on the evaluation of the field and laboratory data and our geotechnical experience in the vicinity of the project, the following recommendations are provided for preliminary project design and planning. However, this report should be considered in its entirety. When applying the recommendations for design, the background information, procedures used, findings, evaluation, and conclusions should be considered. The recommended design consultation and observation of construction activities by Twining are integral to the proper application of the recommendations.

8.1 General

8.1.1 Evaluations, conclusions, and recommendations provided in this report are preliminary and should be used for preliminary design and planning purposes. Additional field exploration and preparation of a design level report based on the final proposed site configuration, loads, and structural details, etc. will be required prior to final design, bidding and construction of the project.

8.1.2 The design settlements estimated after the minimum recommended site preparation (over-excavation and compaction), for the GSU, HRSG, steam turbine, and combustion turbine exceed the allowable settlement criteria provided by Utility Engineering. Accordingly, the following measures may be utilized to reduce the total static and differential settlements to meet the allowable settlement criteria for these improvements: 1) applying a soil surcharge to induce the majority of the static settlements to within acceptable levels prior to foundation construction; 2) support the improvements on a deep foundation system (i.e., drilled pier, driven pile foundations); 3) supporting the foundations on rammed aggregate piers (i.e., Geopiers), or 4) other ground improvement techniques such as deeper over-excavation or deep soil mixing. In our opinion, proper application of these methods can be used to reduce the estimated settlements to meet the allowable settlement criteria for the proposed structures.

8.1.3 In general, if a soil surcharge is used, the surcharge should be left in place until the settlement monitoring indicates an acceptable amount of static settlement has been achieved. Acceptance of the soil surcharge shall be determined based on survey monitoring of the settlements during the soil surcharge activity.

8.1.4 Based on the magnitude of the estimated static and seismic settlements and the depth of influence of the proposed foundation loads, a supplemental investigation is recommended using Cone Penetrometer Testing (CPT) methods to provide subsurface data for use in preparation of final design recommendations. The results of CPT analyses are also helpful in evaluation of alternative ground improvement and other foundation support types such as soil mixing, rammed aggregate piers (i.e., Geopiers) and deep foundations.

8.1.5 Our firm should review the grading plans as soon as they are available. It is anticipated that the recommendations presented in this report may change depending on the proposed site grading.

8.1.6 A demolition plan should be prepared prior to commencement of earthwork. The plan should show the site structures, including buildings, buried pipes, water wells, septic systems, etc. scheduled for removal. The plan should outline the methods to be used to demolish and remove the existing features. All excavations made during demolition and removal of the existing improvements will need to be backfilled as engineered fill.

8.1.7 A preconstruction meeting including, as a minimum, the owner, Contractor, foundation, earthwork and paving subcontractors, civil engineer, structural engineer, and Twining should be scheduled by the general contractor at least one week prior to the start of clearing and grubbing. The purpose of the meeting should be to discuss critical project issues, concerns and scheduling. Twining should be contacted prior to commencement of vineyard removal to discuss with the contractor the proposed removal methods.

8.1.8 If any county, state and/or federal standards are cited on the plans or specifications, all of these standards should be in addition to the recommendations of this report.

8.1.9 The contractor is responsible to conduct grading in compliance with the applicable building codes, the project geotechnical report, the project plans, the project specifications, and the County of Fresno requirements, whichever is most stringent.

8.1.10 The contractor is responsible for compliance with the Storm Water Pollution Prevention Program (SWPPP) requirements specified in the project plans, the project specifications, the County of Fresno (as applicable) and the agencies having jurisdiction, whichever is most stringent.

8.1.11 The contractor is responsible for protecting existing facilities from damage including but not limited to existing foundations, slabs-on-grade, pavements, utilities, buildings, streets, etc. Any damage shall be repaired by the contractor at no cost to the owner(s).

8.1.12 The proposed foundations should not surcharge any existing utilities (if they exist) and associated trench backfill. This can be achieved by deepening the proposed foundations to a depth such that a 1 horizontal to 1 vertical plane from the bottom of the foundations extends below the bottom of the trench backfill and existing utility. If deepening of the foundations is not possible or desired, existing utilities located within the influence of the foundations should be relocated.

8.1.13 If existing utility lines within a horizontal distance of about 10 feet from new foundations are to remain in service, the soil backfill associated with the utility trenches should be tested to determine if the backfill is properly compacted and capable of supporting the improvements proposed to be constructed in these areas (i.e., pavements, etc.). These areas should be designated on the project civil drawings. In addition, the project civil engineer should determine if the existing utilities (if any) are capable of supporting the proposed improvements and overburden and if the existing utilities will impact future utility crossings. It is recommended that, for the purpose of bid, the Contractor should assume excavation and compaction of the backfill above existing underground utilities will be required. For bidding purposes, the Contractor should assume the backfill of all existing utility trenches located within 10 feet of the proposed new foundations will require over-excavation and compaction.

8.2 <u>Site Grading and Drainage</u>

8.2.1 It is critical to develop and maintain site grades which will drain surface and roof runoff away from foundations, and floor slabs - both during and after construction. Adjacent exterior finished grades which are not covered by pavements or walkways should be sloped a minimum of two (2) percent for a distance of at least six (6) feet away from the structure, or as necessary to preclude ponding of water adjacent to foundations, whichever is more stringent. Adjacent exterior surfaces which are paved should be sloped at least 1 percent away from the foundations.

8.2.2 Surface water must not be allowed to pond adjacent to the building or structure foundations. To reduce this potential, it is recommended that rain gutters be installed to direct all water from roof drains into closed conduits that are connected to an acceptable discharge area away from the building foundations. It is also recommended that exterior grades adjacent to the building and structures be paved and sloped to drain to the site storm drain system. Excessive irrigation must be avoided, and low volume sprinklers or irrigation by hand is highly recommended. Owing to the potential for hydroconsolidation, establishing and maintaining positive drainage away from structures is considered integral to limiting potential settlements.

8.2.3 Landscape and planter areas should be irrigated using low flow irrigation (such as drip, bubblers or mist type emitters). It is recommended to use plants with low water requirements.

8.2.4 It is not recommended to place landscape or planted areas adjacent to the building foundations. Trees should be setback from proposed structures at least 10 feet or a distance equal to the anticipated drip line radius of the mature tree. For example, if a tree has an anticipated drip-line diameter of 30 feet, the tree should be planted at least 15 feet away (radius) from proposed or existing buildings.

8.2.5 Landscape and planter areas should be irrigated using low flow irrigation (such as drip, bubblers or mist type emitters). The use of plants with low water requirements is recommended.

8.3 Site Preparation

8.3.1 All topsoil, grape vines, other vegetation and organic material, as well as irrigation lines and any other utility lines or debris that may be encountered should be removed from the areas of the proposed development. The general depth of stripping should be sufficiently deep to remove the root systems and organic material. For estimating purposes, a minimum stripping depth of 6 inches should be used. The actual depth of stripping should be reviewed by Twining at the time of construction. It is possible that deeper stripping may be required if any roots larger than ¼-inch are encountered during grading and in localized areas, such as low areas where water may pond. Stripping should extend laterally a minimum of 5 feet outside the building and pavement perimeters. These materials will not be suitable for use as engineered fill; however, stripped topsoil may be stockpiled and reused in landscape areas at the discretion of the owner. All underground utility lines, pipes and other structures associated with the existing improvements (not to be used) should be removed.

8.3.2 Stripping should be observed by our firm. Roots larger than ¼-inch, and any accumulation of roots that result in an organic content greater than 3 percent by weight as determined by loss-on-ignition tests, should be removed. The exposed subgrade in the excavations should be scarified and compacted as engineered fill to a minimum depth of 8 inches and the excavation backfilled with engineered fill.

8.3.3 As a minimum, the existing features to be removed include existing structures, irrigation lines, water wells, septic systems and grape vines. The resulting excavations should be cleaned of all loose or organic material, the exposed native soils should be scarified to a depth of 8 inches, moisture conditioned, then compacted as engineered fill, and the excavation then backfilled with engineered fill.

8.3.4 Based on our experience, removal of the vineyard will cause disturbance to a depth of about 3 to 4 feet BSG. Recommendations for over-excavation provided in this report are based on the anticipation that removal of the vineyard will cause disturbance in the upper 4 feet BSG. Methods used to remove the vineyard that cause deeper soil disturbance should be avoided. Our firm should be contacted prior to commencement of removal of the vineyard to discuss with the Contractor the proposed removal methods. Our firm should observe and document vineyard removal during these activities.

8.3.5 It is recommended that soils disturbed during the removal vineyard be over-excavated and compacted as engineered fill. The exposed native soils should be scarified to a depth of 8 inches, moisture conditioned, then compacted as engineered fill.

8.3.6 Water wells were observed during the field investigation. All wells scheduled for demolition should be abandoned per state and local requirements. The contractor should obtain an abandonment permit from the local environmental health department, and issue certificates of destruction to the owner and Twining upon completion. At a minimum, wells in building areas (and within 5 feet of building perimeters), should have their casings removed to a depth of at least 10 feet below site grade or finished pad grade. In parking lot or landscape areas, the casings should be removed to a depth of at least 7 feet below site grade or finished grade. The wells should be capped with concrete and the resulting excavations should be backfilled as engineered fill. If the wells are related to an environmental investigation regulated by a local, state, or federal agency, the removal of these wells should be performed under their oversight.

8.3.7 Existing underground utilities may be located on the site. All underground pipelines not to be used after construction should be excavated and removed. Water lines, sewer lines or irrigation pipes should not be crushed and left in place. The resulting excavations should be cleaned of all loose or organic material, the exposed native soils should be scarified to a depth of 8 inches then compacted as engineered fill, and the excavation backfilled with engineered fill.

8.3.8 Over-excavation: After stripping and removal of subsurface structures over-excavation should be conducted within the proposed building areas, structure areas, tank areas and to a distance of 5 feet beyond the building/structure perimeters and a minimum of 3 feet beyond adjacent walkways, or to a distance at least equal to the depth of the fill below the foundations, whichever is further. Over-excavation should be conducted throughout the entire areas of the structures and overbuild zones as follows:

Conventional shallow spread foundations: Over-excavation of at least 4 feet below preconstruction site grades, or to 3 feet below the bottom of foundations, whichever provides the deeper fills, is recommended. The exposed subgrade should be scarified to a depth of 8 inches, moisture conditioned and compacted as recommended in this report prior to backfilling the excavation with engineered fill. The overbuild zone over-excavation should extend a minimum of five (5) feet beyond foundations, 3 (feet) beyond adjacent concrete walkways, or by a horizontal distance equal to the depth of fill at the edge foundations.

Mat and Ring Foundations: Over-excavation should be conducted to a depth of at least 5 feet below preconstruction site grades, or to 3 feet below the bottom of the foundation, whichever provides the deeper fill. The exposed subgrade should be scarified to a depth of 8 inches, moisture conditioned and compacted as recommended in this report prior to backfilling the excavation with engineered fill. The overbuild zone over-excavation should extend a minimum of five (5) feet beyond foundations, 3 (feet) beyond adjacent concrete walkways, or by a horizontal distance equal to the depth of fill at the edge foundations.

Exterior Slabs: Over-excavation should be conducted to a depth of at least 12 inches below preconstruction site grades, the depth to remove 12 inches below soils disturbed during the removal of vineyard, or to 12 inches below the aggregate base, whichever provides the deepest fill. The exposed subgrade should be scarified to a depth of 8 inches, moisture conditioned and compacted as recommended in this report prior to backfilling the excavation with engineered fill.

Asphaltic Concrete Pavements: Over-excavation should be conducted to a depth of at least 12 inches below preconstruction site grades, depth to remove 12 inches below soils disturbed during the removal of the vineyard, to 12 inches below the proposed aggregate base section, or to undisturbed native soils, whichever provides the deepest fills. The exposed soils following the over-excavation should be scarified to a minimum depth of 8 inches, moisture conditioned, and compacted as engineered fill before placement and compaction of additional engineered fills.

Portland Cement Concrete Pavements: Over-excavation should be conducted to depth of at least 12 inches below preconstruction site grades, a depth to remove 12 inches below soils disturbed during the removal of vineyard, to 12 inches below the bottom of the aggregate base, or to undisturbed native soils, whichever provides the deepest fill. The exposed subgrade should be scarified to a depth of 8 inches, moisture conditioned and compacted as recommended in this report prior to backfilling the excavation with engineered fill.

8.3.9 Mat foundations and interior floor slabs should be supported on a minimum of 6 inches of Class 2 aggregate base compacted to a minimum of 95 percent relative compaction over the depth of engineered fill recommended below foundations.

8.3.10 It is recommended that extra care be taken by the contractor to ensure that the horizontal and vertical extent of the excavation and compaction conform to the site preparation recommendations presented in this report. The geotechnical engineer is not responsible for measuring and verifying the horizontal and vertical extent of excavation and compaction. The contractor should verify in writing to the owner and to the geotechnical engineer that the horizontal and vertical excavation limits were completed in conformance with the recommendations of the geotechnical engineer, the project plans, and the specifications, whichever is the most stringent applies). It is recommended that this verification be performed by a licensed surveyor. This verification should be provided prior to requesting pad certification from the geotechnical engineer or excavating for foundations.

8.3.11 It is the responsibility of the contractor to provide safe working conditions with respect to excavation slope stability. The contractor is responsible for site slope safety, classification of materials for excavation purposes, and maintaining slopes in a safe manner during construction. The grades classification and height recommendations presented for temporary slopes are for consideration in preparing budget estimates and evaluating construction procedures.

8.3.12 Shoring systems, if used, should be designed by a structural engineer with experience in designing shoring systems and registered in the State of California.

8.3.13 All fill required to bring the site to final grades should be placed as engineered fill. In addition, all native soils over-excavated should be compacted as engineered fill.

8.3.14 The moisture content and density of the compacted soils should be maintained until the placement of concrete. If soft or unstable soils are encountered during excavation or compaction operations, our firm should be notified so the soils conditions can be examined and additional recommendations provided to address the pliant areas.

8.3.15 The Contractor is responsible for the disposal of concrete, asphaltic concrete, soil, spoils, etc. that must be exported from the site. Individuals, facilities, agencies, etc. may require analytical testing and other assessments of these materials to determine if these materials are acceptable. The Contractor is responsible to perform the tests, assessments, etc. to determine the appropriate method of disposal. In addition, the Contractor is responsible for all costs to dispose of these materials in a legal manner.

8.4 Engineered Fill

8.4.1 The on-site native soils encountered are predominantly silty sands. These soils should be suitable for use as engineered fill material to a depth of 6 inches below interior slabs on grade and mat foundations, provided they are free of organics and debris and the moisture content of the soil is between optimum and 3 percent above the optimum moisture content. If soils other than those considered in this report are encountered, Twining should be notified to provide alternate recommendations.

8.4.2 The compactability of the native soils is dependent upon the moisture content, subgrade conditions, degree of mixing, type of equipment, as well as other factors. The evaluation of such factors was beyond the scope of this report; therefore, they should be evaluated by the Contractor during design and construction of the project.

8.4.3 Import fill soil should be non-corrosive, non-expansive and granular in nature and contain enough fine grained material (binder) to allow cutting "neat" footing trenches with the following acceptance criteria recommended:

Percent Passing 3-Inch Sieve	100
Percent Passing No. 4 Sieve	50 - 100
Percent Passing No. 200 Sieve	10 - 30
Plasticity Index	Less than 10
Expansion Index (UBC 18-2)	Less than 10
R-value	Minimum 50
Soil Resistivity	Greater than10,000 ohm-cm
Sulfates	< 0.05 percent by weight

Prior to importing fill, the Contractor shall submit test data to KRCD and Twining that demonstrates that the proposed import material complies with the recommended geotechnical criteria. Also, prior to being transported to the site, the import material shall be certified by the Contractor and the supplier (to the satisfaction of KRCD and Twining) that the soils do not contain any environmental contaminates regulated by local, state or federal agencies having jurisdiction. This certification shall consist of, as a minimum, recent analytical data, including appropriate chain-of-custody documentation, specific to the source of the import material. After receipt and approval, by KRCD and Twining, of the data for geotechnical and environmental compliance of the proposal import material, Twining will sample and test the proposed import material. Prior to being transported to the site, the import fill material should be tested and approved by Twining. The Contractor shall allow a minimum of seven (7) working days for each import source to be tested.

8.4.4 Onsite native and imported engineered fill soils should be placed in loose lifts approximately 8 inches thick, moisture-conditioned to within optimum to 3 percent above the optimum moisture content, and compacted to a dry density of at least 95 percent of the maximum dry density as determined by ASTM Test Method D1557. Additional lifts should not be placed if the previous lift did not meet the required dry density or if soil conditions are not stable.

8.4.5 Open graded gravel and rock material such as ¾-inch crushed rock or ½-inch crushed rock should not be used as backfill including trench backfill. In the event gravel or rock is required by a regulatory agency for use as backfill (Contractor to obtain a letter from the agency stating the requirement for rock and/or gravel as backfill), all open graded materials shall be fully encased in a geotextile filter fabric, such as Mirafi 140N, to prevent migration of fine grained soils into the porous material. Gravel and rock cannot be used without the written approval of Twining.

8.4.6 Aggregate base shall comply with Class 2 aggregate base per State of California Standard Specifications. Aggregate base shall be compacted to a minimum relative compaction of 95 percent.

8.4.7 Recycled materials (AC materials, construction materials, etc.) should not be used within 10 feet of any improvement without approval by the owner, and Twining. Contractors should not assume that recycled materials (AC construction materials, etc.) can be used in preparing bids for the project without approval by the owner, and/or architect.

8.5 Shallow Spread Foundations for Buildings

8.5.1 Structural loads for the proposed pre-engineered buildings may be supported on spread or continuous footings placed entirely on at least 3 feet of engineered fill, engineered fill which extends to a minimum depth of 4 feet below preconstruction site grades, or to undisturbed native soils, whichever is deeper. Site preparation should be conducted in accordance with subsection 6.4 and the Recommendations section of this report.

8.5.2 Continuous and spread footings may be designed for an allowable soil bearing pressure of 1,500 pounds per square foot (net) for dead-plus-live loads, and maximum column and line loads of 20 kips and 1.7 kips per foot, respectively. The bearing capacity may be increased by one-third for short duration wind or seismic loads. The foundations should be designed and evaluated by the project structural engineer.

8.5.3 The shallow spread footings should have a minimum depth of 12 inches below finish pad grades or adjacent exterior grades, whichever is lower. Footings should have a minimum width of 12 inches, regardless of load.

8.5.4 Shallow spread foundations should be continuous around the perimeter of the structure to reduce moisture migration beneath the structure. Continuous perimeter foundations should be extended through doorways and/or openings that are not needed for support of loads.

8.5.5 Depending of wall heights and configuration of retaining structures, seismic increments may be needed for design of retaining walls. Twining should be contacted to review wall plans when plans are available and can provide seismic increments for wall design upon request.

8.5.6 The shallow spread foundations should be designed and reinforced for the anticipated settlements. A structural engineer experienced in foundation design should recommend the thickness, design details and concrete specifications for the foundations based on: 1) a total anticipated static settlement of 1 inch; 2) a differential static settlement of ½ inch in 40 linear feet of continuous footings; 3) a total seismic settlement of ¾ inch and a differential seismic settlement of ⅜ inch in 40 feet, 4) and a differential anticipated static settlement of ½-inch between isolated column footings.

8.5.7 The following factors were developed based on the tables in Chapter 16 of the 2001CBC and the digitized active fault locations published by the California Geological Survey (CGS).

Seismic Factor	CBC Value
Soil Type	S_D
Seismic Zone Factor	Z=0.3
Near Source Acceleration Factor, Na	1.0
Near Source Velocity Factor, Nv	1.0
Seismic Acceleration Coefficient, Ca	0.36
Seismic Velocity Coefficient, Cv	0.54

8.6 Mat Foundations for Heavily Loaded Structures (HRSG, CTG, STG, GSU)

8.6.1 As discussed in this report, the estimated total static settlements for the GSU, HRSG, steam turbine, and combustion turbine exceed the total allowable settlement criteria provided by Utility Engineering. Accordingly, the following measures may be utilized to reduce the total static and differential settlements to meet the allowable settlement criteria: 1) applying a soil surcharge to induce the majority of the static settlements to within acceptable levels; 2) support the improvements on a deep foundation system, 3) support the proposed improvements on rammed aggregate piers, or 4) other ground improvement techniques such as deep soil mixing. These mitigation measures will need to be evaluated further as a part of the design level geotechnical engineering investigation report.

8.6.2 The allowable soil bearing capacities for mat foundations may be increased by one-third for short duration wind or seismic loads.

8.6.3 A total seismic settlement of ¾ inch and a differential seismic settlement of ⅜ inch in 40 feet, should be anticipated for design of all foundations. The estimated seismic settlement is in addition to the estimated static settlements.

8.6.4 The bottoms of the mat foundations should be at least 3 feet below adjacent finished grade level. If the mat foundations are planned at shallower depths, Twining should be requested to evaluate the allowable bearing capacities based on the proposed depths of the mat foundations.

8.6.5 Over-excavation and compaction of engineered fill should be conducted in areas to receive mat foundations, such as the Combustion Turbine, Steam Turbine, and HRSG as recommended in the site preparation section of this report.

8.6.6 The mat foundations should be underlain by at least 6 inches of compacted Class 2 aggregate base (minimum 95 percent relative compaction), which is underlain by engineered fill extending to the depths of engineered fill recommended below foundations.

8.6.7 A preliminary modulus of subgrade reaction of 30 pounds per square inch per inch should be used for preliminary design of the mat foundations for the project. It is recommended that the project structural engineers provide Twining with load distribution diagrams for all of the mat foundations showing the loads at the bottom of the foundations based on a k value of 30. Based on the load distribution, a refined modulus of subgrade reaction value can be provided for final design.

8.7 Drilled Shaft Foundation Design

Structural loads were not provided for the pipe racks anticipated to be supported on pier foundations. For the purpose of this preliminary report, loads for pipe racks of 30 kips axial compression, 4 kips uplift, and 4 kips lateral loading were assumed in design.

8.7.1 The total axial load on the shaft should be determined by the project structural engineer. The weight of the concrete below grade may be ignored in design. A shaft depth should be selected that will provide a total allowable capacity in excess of the design axial load with acceptable safety factors.

8.7.2 The piers should be placed no closer together than three pile diameters, center-to-center.

8.7.3 The allowable downward resistance values presented in this section may be increased by one-third for short duration live loads such as wind and seismic loads.

8.7.4 Soils to a depth of 1 pier diameter should be neglected in determining the axial capacity and lateral passive resistance.

8.7.5 For caissons, an allowable skin friction of 200 psf may be used for preliminary design. End bearing capacity should not be considered for preliminary design.

8.7.6 The capacity of the cast-in-drilled hole concrete pier in uplift (tension) may be taken as the total dead weight of the pier plus ½ of the allowable axial load.

8.7.7 The lateral capacity of the piers may be computed using an allowable passive resistance equal to an equivalent fluid pressure of 200 pounds per cubic foot. For piers spaced at least three pile diameters (center to center), twice the caisson diameter may be used for computing passive resistance.

8.7.8 Total and differential static settlements of 1 inch and ½ inch, respectively should be anticipated for design of the cast-in-drilled hole concrete pier/grade beam foundations.

8.7.9 A civil or structural engineer registered in the state of California should design the dimension of the cast-in-drilled hole concrete piers and reinforcement cages to resist shear, moment, and axial (tension and compression) loads.

8.7.10 It should be anticipated that casing of the drilled shafts may be required.

8.7.11 Twining should observe the drilling of the shafts to insure that the materials encountered are consistent with those evaluated during our geotechnical engineering investigation. This inspection should be prior to placement of reinforcing steel and concrete.

8.7.12 The bottoms of the drilled shafts should be cleaned of all loose soils, cobbles, gravel, or other materials prior to installation of steel. The bottoms of the foundations should be observed by Twining to verify removal of these materials.

8.7.13 Before construction, the Contractor and project structural engineer should review and evaluate the final design shaft dimensions based on the proposed loads to verify that the minimum recommendations presented in this report have been interpreted correctly.

8.7.14 The type and strength of the concrete used for construction of the shafts should be specified by the project structural engineer.

8.8 Drilled Shaft Construction

8.8.1 The contractor hired to conduct the drilling operation for the proposed drilled shaft pier foundations should have a minimum of five (5) years experience in the construction of drilled shaft foundations and should have conducted at least five similar projects in the past three years. The contractor should be experienced in the use of casing to stabilize sidewalls during the construction of the piers. After selection of the contractor and subcontractors has been completed and before construction begins, a preconstruction meeting is recommended prior to construction of the drilled shaft foundations to discuss drilling and construction procedures.

8.8.2 The soils encountered in the borings are considered to have a limited "stand-up" capacity, and caving of soils into the drilled shaft excavation should be anticipated. Therefore, Contractors bids shall include temporary casing of the drilled shaft excavations during construction by the Contractor to provide temporary support of the excavations. The temporary casing should be slowly removed from the shaft excavation during placement of concrete to ensure the casing is not raised above the level of the concrete during shaft construction, to prevent sidewall soils from sloughing into the shaft excavation. Drilling slurry shall not be used.

8.8.3 Twining should observe the drilling of the shafts to assess whether the soils encountered are consistent with those evaluated during our geotechnical engineering investigation. A Twining engineer should provide continuous inspection of shaft drilling and concrete placement. If sidewall sloughing is noted, temporary casing should be available on-site to be used at any time. If the contractor chooses not to use temporary casing during drilling or decides to use drilling slurry in place of casing, the contractor shall assume the responsibility for redrilling or relocating the drilled shaft, if sloughing occurs.

8.8.4 Loose soils should be removed from the drilled shaft excavation prior to placement of reinforcing steel and concrete. The drilled shaft excavation, reinforcing steel, and concrete placement should be inspected by Twining during construction.

8.8.5 Concrete should be placed in the drilled shaft as soon as possible following drilling. In no case should the excavations be left open longer than six hours. Drilling slurry, if used, should not remain in the shaft excavation for more than six hours.

8.8.6 Temporary casing should be able to withstand the external pressures of the caving soils. The outside diameter of the casing should not be less than the diameter of the drilled shaft. In addition, casing should not be left in the ground.

8.8.7 The rebar cage should be designed (i.e., tied) with adequate space between the bars to allow concrete to flow. In addition, the rebar cage should be designed to withstand the hydrostatic forces of the concrete as the casing is pulled.

8.8.8 The design slump of the concrete at the time of placement should not be less than four inches.

8.8.9 The concrete should be placed using tremie methods which can place concrete at the bottom of the boring. The tremie may be lifted slowly as the concrete is placed; however the tremie should be kept at least 5 feet below the top of the concrete after 5 feet of concrete has been placed in the excavation.

8.8.10 Casing should be lifted slowly as the concrete is deposited, while the bottom of the casing is kept at least five feet below the top of the concrete.

8.8.11 Shaft excavation should be drilled within 2 degrees of vertical. This condition should be verified and documented by the contractor.

8.8.12 The rebar cage should be suspended within 2 degrees of vertical in the center of the excavation. This condition should be verified and documented by the contractor. Minimum concrete cover should be maintained throughout the length of the excavation.

8.9 Mat Foundations for Tanks and Pads and Ring Foundations

8.9.1 The estimated static settlements exceed the allowable settlements provided for the proposed tanks. Therefore, alternate methods of site preparation will be required to reduce the anticipated settlements. These may include specialty ground improvement methods (i.e., deep soil mixing), surcharging the site, supporting tanks on rammed aggregate piers (i.e., Geopiers), or designing additional camber into the bottom of the tank shell to accommodate the estimated settlements.

8.9.2 The tank structural loads may be supported on mat or ring footings placed entirely on engineered fill which extends to at least 48 inches below preconstruction grades, or to at least 24 inches below the bottom of foundations, whichever provides the deeper fill, as recommended in the Site Preparation section of this report.

8.9.3 A maximum allowable contact pressure for the tank bottoms of 2,500 psf, 3,000 psf, 3,000 and 3,000 psf for the recycle water holding tank, brine holding tank, demineralized water tank, and service/fire water tank, respectively, may be used for preliminary design. Exceeding these pressures will increase the predicted total and differential settlements.

8.9.4 If a ring foundation is used, the perimeter concrete ring foundation should be extended to a depth of at least 24 inches below adjacent exterior grades or below interior finish floor elevation, whichever is deeper.

8.9.5 If a mat foundation is used, a thickened edge or a separate concrete cut-off (minimum 8 inch wide, 2-sack sand-cement slurry) should be extended to a depth of at least 24 inches below adjacent exterior grades or below top of the slab, whichever is deeper.

8.9.6 If isolated spread foundations are used to support roof supports, the bottom of the foundation can be supported at a minimum depth of 12 inches below the pad subgrade elevation.

8.9.7 It is assumed that the tanks will be constructed on a concrete slab (mat foundation or conventional slab) overlying aggregate base, or the tank will be placed directly on Class 2 aggregate base (with concrete ringwalls). A minimum of 6 inches of Class 2 aggregate base compacted to 95 percent relative compaction, over the depth of engineered fill recommended in the site preparation section of this report is recommended below the tanks (with or without the use of a slab) to provide a uniform bearing surface.

8.9.8 It is recommended, concrete or asphaltic concrete should be constructed at the ground surface, from the outer edge of the tank for a distance of at least 5 feet away from the tanks, and completely around the tanks to prevent water flowing over the tank surface from entering the near surface granular soils below the tanks.

8.9.9Damage to connections between exterior locations and the tank due to differential settlements can be reduced by pre-loading the tank for two to three weeks to induce up to about 50 percent of the estimate settlements. However, flexible connections for inlet/outlet pipes should be designed to account for as much as 3-inches of differential movement without damage. After one year, with at least 75 percent of the maximum load, the flexible connections can be replaced with connections designed to withstand ½ inch of differential movement.

8.9.10 Survey targets should be placed and elevations established at four locations around the tanks before the tanks are loaded so that elevations can be monitored during loading. During the initial loading, it is recommended the elevations be surveyed on a regular basis. After initial loading, elevations should be checked at a regular frequency, such as quarterly. The data should be provided to our firm and the structural engineer as soon as it is available. Additional recommendations should be made based on the elevation measurements. Additional recommendations should be made based on the elevation measurements. Also, the tank structural engineer should be requested to provide limits of settlement that is a concern with respect to the tank structure.

8.9.11 It will be critical to moisture condition the engineered fill subgrade below aggregate base section to within optimum to 3 percent above optimum moisture content immediately prior to placement of the aggregate base. The moisture content and the relative compaction of the engineered fill below the aggregate should be confirmed in writing by in-situ moisture tests and density tests prior to placement of the non-expansive fill.

8.10 <u>Frictional Coefficient and Earth Pressures</u>

8.10.1 The values provided below may be used for preliminary foundation and retaining wall design. Retaining wall plans should be provided to Twining for review to assess whether the plans incorporate the geotechnical data and recommendations in this report.

8.10.2 The bottom surface area of concrete footings or concrete slabs in direct contact with engineered fill can be used to resist lateral loads. An allowable coefficient of friction of 0.30 can be used for design. In areas where slabs are underlain by a synthetic moisture barrier, an allowable coefficient of friction of 0.10 can be used for design.

8.10.3 The ultimate passive resistance of the native soils and engineered fill may be assumed to be equal to the pressure developed by a fluid with a density of 275 pounds per cubic foot. An appropriate factor of safety should be applied.

8.10.4 The passive pressure was calculated based on a minimum soil unit weight of 100 pounds per cubic foot. The soils within the passive zone at the foot of basement and retaining walls (one footing width in front of the wall to a depth equal to the footing depth) should be tested to verify that the soils have the minimum unit weight of 100 pounds per cubic foot (with moisture). If the soils have a unit weight of less than 100 pounds per cubic foot, the soils within this zone should be over-excavated and replaced as engineered fill. These soils should be tested prior to backfilling behind the wall.

8.10.5 A minimum factor of safety of 1.5 should be used if the frictional and passive resistance of the soil are combined in determining the total lateral resistance. The upper 12 inches of subgrade should be neglected in determining the total passive resistance.

8.10.6 The active and at-rest pressures of the native soils and engineered fill may be assumed to be equal to the pressures developed by a fluid with a density of 45 and 68 pounds per cubic foot, respectively. These pressures assume level ground surfaces and do not include the surcharge effects of construction equipment, loads imposed by nearby foundations and roadways and hydrostatic water pressure.

8.10.7 The active and at-rest pressures were calculated based on a maximum soil unit weight of 135 pounds per cubic foot. The compacted soils behind the retaining walls should not have a compacted unit weight above 135 pounds per cubic foot (with moisture). If the soils have a unit weight of greater than 135 pounds per cubic foot, the soils should be over-excavated and replaced at a lower degree of compaction. If the backfill soils must be placed at a unit weight of more than 135 pounds per cubic foot to achieve minimum compaction requirements the material should not be used as backfill behind basement and retaining walls.

8.10.8 The at-rest pressure should be used in determining lateral earth pressures against walls which are not free to deflect. For walls which are free to deflect at least one percent of the wall height at the top, the active earth pressure may be used.

8.11 Interior Building and Lightly Loaded Slabs-on-Grade

8.11.1 The recommendations provided herein are intended only for the design of interior conventional or light equipment concrete slabs-on-grade and their proposed uses, which do not include construction traffic (i.e., cranes, concrete trucks, heavy rotating equipment, and rock trucks, etc.). The recommendations do not apply to mat slabs. The Contractor should assess the slab section and determine its adequacy to support any proposed construction traffic.

8.11.2 A structural engineer experienced in slab-on-grade design should recommend the thickness, design details and concrete specifications for the proposed slabs-on-grade for the combined static and seismic settlements estimated for design in this report.

8.11.3 The floor slab should be reinforced for the anticipated temperature and shrinkage stresses. A structural engineer experienced in slab-on-grade design should recommend the thickness, design details and concrete specifications for the proposed slab-on-grade for a differential vertical movement between the exterior walls and interior footings, and the adjacent floor slab of ½ inch.

8.11.4 In areas where interior concrete slabs-on-grade are anticipated, over-excavation should be conducted to provide the minimum depth of engineered recommended in the site preparation section of this report.

8.11.5 The moisture content of the subgrade or engineered fill below the aggregate base section should be verified to be optimum to 3 percent above optimum moisture content prior to placing non-expansive fill, and also within 48 hours of placement of the vapor retarding membrane or the concrete for the slab-on-grade if a vapor retarding membrane is not used. The moisture content of the upper 12 inches of the subgrade soils should be tested and confirmed prior to placement of the base section, vapor retarding membrane or slab-on-grade.

8.11.6 Concrete should be placed by pump to reduce the potential for creating an unstable subgrade during placement operations.

8.11.7To aid in uniform curing of the slabs, the slabs and underlying subgrade should be constructed in accordance with current American Concrete Institute (ACI) standards.

8.11.8 ACI recommends that the interior slab-on-grade should be placed directly on a vapor retarding membrane when the potential exists that the underlying subgrade or sand layer could be wet or saturated prior to placement of the slab-on-grade. It is recommended that Stegowrap 15 or equivalent should be used where moisture sensitive floor coverings, such as carpet and tile, are anticipated or where moisture could permeate into the interior and create problems. The layer of Stegowrap 15 should overlay a minimum of 6 inches of compacted Class 2 AB. It should be noted that placing the PCC slab directly on the vapor retarding membrane will increase the potential for cracking and curling; however, ACI recommends the placement of the vapor retarding membrane directly below the slab to reduce the amount vapor emission through the slab-on-grade. Based on discussions with Mr. Eric Gerst with Stego Industries, L.L.C. (telephone 949-493-5460), the Stegowrap can be placed directly on the Class 2 AB and the concrete can be placed directly on the Stegowrap. It is recommended that the design professional obtain written confirmation from Stego Industries that this product is suitable for the specific project application. It is recommended that the slab be moist cured for a minimum of 7 days to reduce the potential for excessive cracking. It should be noted that moist curing can also increase the moisture due to migration through joints, cracks, etc. This may require addition time for the slab to dry. The underslab membrane should have a high puncture resistance (minimum of approximately 2,400 grams of puncture resistance), high abrasion resistance, rot resistant, and mildew resistant. It is recommended that the membrane be selected in accordance with ASTM C 755-02, Standard Practice For Selection of Vapor Retarder For Thermal Insulation and conform to ASTM E 154-99 Standard Test Methods for Water Vapor Retarders Used in Contact with Earth Under Concrete Slabs, on Waters, or as Ground Cover. It is recommended that the vapor retarding membrane selection and installation conform to the ACI Manual of Concrete Practice, Guide for Concrete Floor and Slab Construction (302.1R-96), Addendum, Vapor Retarder Location and ASTM E 1643-98, Standard Practice for Installation of Water Vapor Retarders Used In Contact with Earth or Granular Fill Under Concrete Slabs. In addition, it is recommended that the manufacturer of the floor covering and floor covering adhesive be consulted to determine if the manufacturers have additional recommendations regarding the design and construction of the slab-on-grade, testing of the slab-on-grade, slab preparation, application of the adhesive, installation of the floor covering and maintenance requirements.

8.11.9 The membrane should be installed so that there are no holes or uncovered areas. All seams should be overlapped and sealed with the manufacturer approved tape, continuously at the laps so they are vapor tight. All perimeter edges of the membrane, such as pipe penetrations, interior and exterior footings, joints, etc., should be sealed per manufacturer's recommendations.

8.11.10 Tears or punctures that may occur in the membrane should be repaired prior to placement of concrete per the manufacturer's recommendations. Once repaired, the membrane should be inspected by the contractor and the owner to verify adequate compliance with manufacture's recommendations.

8.11.11 The manufacturer's requirements vary regarding the surface and cover material around the placed membrane. Vapor retarding membranes should be installed in accordance with the manufacturers' specifications.

8.11.12 The membrane is not required beneath exposed concrete floors provided that moisture intrusion into the structure is permissible for the design life of the structure.

8.11.13 Additional measures to reduce moisture migration should be implemented if moisture sensitive floor coverings (such as wood or vinyl) are used. These include: 1) constructing a less pervious concrete floor slab by maintaining a low water-cement ratio as recommended by ACI in the concrete for slabs-on-grade; 2) moist cure the slab for at least 7 days; 3) ensuring that all seams and utility protrusions are sealed with tape to create a "water tight" moisture retarding membrane; 4) placing concrete walkways or pavements adjacent to the structure; 5) locating lawns and flower beds away from the structure; and 6) providing adequate drainage away from the structure at a minimum two percent slope. In addition, water should not be allowed to pond adjacent to the structure.

8.11.14It should be noted that the placement and compaction of the Class 2 AB, the vapor retarding membrane installation, protection, etc., and the placement, curing, etc. of concrete should be in accordance with the project geotechnical engineering report, applicable ACI requirements, the manufacturer's requirements, the project plans, the project specifications, whichever is most stringent.

8.11.15The moisture vapor transmission through the slab, pH and relative humidity should be tested by the Contractor at a frequency and method as specified by the flooring manufacturer. Vapor transmission, pH, internal relative humidity, etc. results should be within floor manufacturers' specifications prior to placing floor coverings. The floor covering and adhesive manufacturers may specify additional requirements relative to installation of floor covering and adhesives. The project architect should provide the specifics relative to the floor system in the project documents.

8.12 Exterior Slabs-On-Grade

The recommendations for exterior slabs provided below are not intended for use for slabs subjected to vehicular traffic, equipment, or forklift loads, rather lightly loaded slabs for sidewalks, curbs, and planters, etc. Recommendations for asphaltic concrete and PCC slabs subjected to vehicular traffic are provided in subsequent sections of this report.

8.12.1 Exterior slabs should be underlain by 4 inches of Class 2 aggregate base over the depth of engineered fill recommended below exterior slabs. Areas to receive exterior slabs-on-grade should be over-excavated to a minimum depth of 12 inches below preconstruction site grades, 12 inches below the elevation of subgrade level to receive aggregate base, and 12 inches below the depth of soils disturbed during removal of the vineyard, whichever provides the deeper fill. Upon approval of the over-excavation by Twining, the bottom of the over-excavation should be scarified a minimum of 8 inches in depth, moisture conditioned to within optimum to three (3) percent over optimum and compacted to a minimum of 92 percent relative compaction. If any city, county, and/or state standards are cited on the plans or specifications, these standards should be in addition to the recommendations in this report.

8.12.2 The moisture content of the subgrade should be verified to be between optimum and 3 percent above optimum moisture content prior to placing non-expansive fill, and also within 48 hours of placement of the slab-on-grade. If necessary to achieve the recommended moisture content, the subgrade could be over-excavated, moisture conditioned as necessary and compacted as engineered fill.

8.12.3 To reduce the effects of drying around the edges of the flatwork and reduce the potential for infiltration of water into the granular fill, lateral cutoffs which extend to a depth of 4 inches below the adjacent grade at the edges of flatwork such as inverted curbs are recommended.

8.12.4 Exterior improvements that subject the subgrade soils to a sustained load greater than 150 pounds per square foot should be prepared in accordance with recommendations presented in this report for foundations and floor slabs. Twining can provide alternative design recommendations for exterior slabs, if requested.

8.12.5 Since exterior sidewalks, curbs, etc., are typically constructed at the end of the construction process, the moisture conditioning conducted during earthwork can revert to natural dry conditions. Placing non-expansive materials and/or concrete walks and finish work over dry or slightly moist native subgrade should be avoided. Twining should be notified to conduct in-place moisture and density tests prior to placing non-expansive fill and concrete flatwork. Written test results indicating passing density and moisture tests should be in the contractor's possession prior to placing concrete for exterior flatwork.

8.13 <u>Asphaltic Concrete (AC) Pavements</u>

8.13.1 In AC pavement areas, over-excavation should be conducted to a depth of at least 12 inches below preconstruction site grades, to 12 inches below the proposed aggregate base section, or to a depth of 12 inches below the depth of soils disturbed during removal of the vineyard, whichever provides the deepest fills. The exposed soils following the over-excavation should be scarified to a minimum depth of 8 inches, moisture conditioned, and compacted as engineered fill before placement and compaction of additional engineered fills.

8.13.2 After fill placement, the pavement section subgrade should be proof-rolled (using rubber tire equipment) under the observation of a Twining representative. If soft or unstable areas are identified, these soils should be removed and replaced with properly moisture conditioned and compacted engineered fill prior to placement of pavements.

8.13.3 The following pavement sections were designed using an R-value of 40 and a range of traffic indices of 5.0 to 8.0. The traffic index should be selected by the project civil engineer based on traffic loading and the governing municipalities minimum requirements. It should be noted that if pavements are constructed prior to the building construction, the traffic index values may be too low and need to be increased. If the pavements are placed prior to construction, or if more frequent truck traffic is anticipated, Twining should be contacted to re-evaluate the traffic index values.

Traffic Index	AC thickness, inches	AB thickness, inches	Compacted Subgrade, inches
5.0	3.0	5.0	12
5.5	3.0	5.0	12
6.0	3.0	6.0	12
6.5	3.5	7.0	12
7.0	3.5	8.0	12
7.5	4.0	8.5	12
8.0	4.5	8.5	12

AC - Asphaltic Concrete
AB - Class 2 Aggregate Base compacted to at least 95 percent relative compaction (ASTM D-1557)
Subgrade - Subgrade soils compacted to at least 95 percent relative compaction (ASTM D-1557)

8.13.4 The curbs where pavements meet irrigated landscape areas or uncovered open areas should be extended to the bottom of the aggregate base. This should reduce subgrade moisture from irrigation and runoff from migrating into the base section and reducing the life of the pavements.

8.13.5 Alternative pavement sections, such as equivalent asphaltic concrete sections or full depth asphaltic concrete sections, may be used. Twining should be contacted for adjusted AC sections and AB sections, if needed.

8.13.6 If actual pavement subgrade materials are significantly different from those tested for this study due to unanticipated grading or soil importing, the pavement section should be re-evaluated for the changed subgrade conditions.

8.13.7 If the paved areas are to be used during construction, or if the type and frequency of traffic are greater than assumed in design, the pavement section should be re-evaluated for the anticipated traffic.

8.13.8 Pavement section design assumes that proper maintenance, such as sealing and repair of localized distress, will be performed on an as needed basis for longevity and safety.

8.13.9 Pavement materials and construction method should conform to Sections 25, 26, and 39 of the Caltrans Standard Specifications Requirements.

8.13.10The asphaltic concrete pavements should include a bottom binder course which complies with Caltrans Standard Specifications Section 39 for Type B, ¾ inch maximum medium, and the upper 2 inch wearing course which complies with the Caltrans Standard Specifications Section 39 for Type B, ½ inch maximum medium. The asphaltic concrete, both mat and joints, shall be compacted to a minimum relative compaction of 95 percent based on State of California Test Method.

8.13.11The asphalt concrete should comply with Type "B" asphalt concrete as described in Section 39 of the State of California Standard Specification Requirements. It is recommended that an asphalt concrete mix design be prepared and signed by a registered civil engineer and reviewed and approved by the Owner and Twining prior to construction.

8.14 Portland Cement Concrete (PCC) Pavements

Recommendations for Portland Cement Concrete pavement structural sections are presented in the following subsections. These recommendations should be used for design and construction of concrete slabs subject to vehicular traffic. The PCC pavement design assumes a minimum modulus of rupture of 550 psi. A qualified design professional should specify where heavy duty and standard duty slabs are used based on the anticipated type and frequency of traffic.

8.14.1 In PCC pavement areas, over-excavation should be conducted to a depth of at least 12 inches below preconstruction site grades, to 12 inches below the proposed aggregate base section, or to a depth of 12 inches below the depth of soils disturbed during removal of the vineyard, whichever provides the deepest fills. The exposed soils following the over-excavation should be scarified to a minimum depth of 8 inches, moisture conditioned, and compacted as engineered fill before placement and compaction of additional engineered fills.

8.14.2 The vehicular drive pavement section was designed based on a traffic index of 6.0. A design k-value of 200 psi/in was used considering a recommended 6-inch layer of Class 2 aggregate base material (R-value of 78) over the native compacted soils.

Pavement Component	Thickness, Inches
Portland Cement Concrete	6.5
Class 2 Aggregate Base (95% Minimum Relative Compaction)	6.0
Compacted Subgrade (95% Minimum Relative Compaction)	12.0

8.14.3 The truck traffic pavement section was designed based on a traffic index of 7.5 and a k-value of 200 psi/in considering a recommended 6-inch layer of Class 2 aggregate base material (R-value of 78).

Pavement Component	Thickness, Inches
Portland Cement Concrete	7.0
Class 2 Aggregate Base (95% Minimum Relative Compaction)	6.0
Compacted Subgrade (95% Minimum Relative Compaction)	12.0

8.14.4 Stresses are anticipated to be greater at the edges and construction joints of the pavement section. A thickened edge is recommended on the outside of slabs subjected to wheel loads.

8.14.5 Joint spacing in feet should not exceed twice the slab thickness in inches, e.g., 12 feet by 12 feet for a 6-inch slab thickness. Regardless of slab thickness, joint spacing should not exceed 15 feet.

8.14.6 Lay out joints to form square panels. When this is not practical, rectangular panels can be used if the long dimension is no more than 1.5 times the short.

8.14.7 Control joints should have a depth of at least one-fourth the slab thickness, e.g., 1-inch for a 4-inch slab.

8.14.8 Isolation (expansion) joints should extend the full depth and should be used only to isolate fixed objects abutting or within paved areas. Construction joint location should be determined by the contractor's equipment and procedures.

8.14.9 Pavement section design assumes that proper maintenance such as sealing and repair of localized distress will be performed on a periodic basis.

8.14.10Pavement construction should conform to Sections 40 and 80 of the State of California Standard Specifications.

8.15 Temporary Excavations and Shoring

8.15.1 It is the responsibility of the Contractor to provide safe working conditions with respect to excavation slope stability. The contractor if responsible for site slope safety, classification of materials for excavation purposes, and maintaining slopes in a safe manner during construction. The grades classification and height recommendations presented for temporary slopes are only for consideration in preparing budget estimates and evaluating construction procedures. The contractor is responsible to determine the OSHA requirements for the site.

8.15.2 Based on OSHA excavation guidelines, the upper granular soils (fill and native) should be temporarily sloped based on a Type C condition, at 1½H to 1V or flatter. If excavations cannot meet these criteria, the temporary excavations should be shored.

8.15.3 Shoring should be designed by an engineer with experience in designing shoring systems and registered in the State of California. Twining should be provided with the shoring plan to assess whether the plan incorporates the recommendations in the geotechnical report.

8.15.4 In no case should excavations extend below a 2H to 1V zone below existing utilities, foundations and/or floor slabs which are to remain after construction. Excavations which are required to be advanced below the 2H to 1V envelope should be shored to support the soils, foundations, and slabs.

8.15.5 Excavation stability should be monitored by the contractor. Slope gradient estimates provided in this report do not relieve the contractor of the responsibility for excavation safety. In the event that tension cracks or distress to the structure occurs, during or after excavation, the owners and Twining should be notified immediately and the contractor should take appropriate actions to minimize further damage or injury.

8.16 Utility Trenches

8.16.1 The trench width, type of pipe bedding, the type of initial backfill, and the compaction requirements of bedding and initial backfill material for utility trenches (storm drainage, sewer, water, electrical, irrigation, etc.) should be specified by an applicable design professional compliance with the manufacturer's requirements, governing requirements and this report, whichever is more stringent. For flexible polyvinylchloride (PVC) pipes, these requirements should be in accordance with the manufacturer's requirements or ASTM D-2321, whichever is more stringent. The width of the trench should provide sufficient space between the sidewall of the trench and the pipe to allow testing with a nuclear density gage (minimum 12 inches). The bottom of the trench should be compacted prior to placing the pipe bedding. As a minimum, the pipe bedding should consist of 4 inches of compacted (95 percent relative compaction) ASTM C-33 sand. The haunches and initial backfill (12 inches above the top of pipe) should consist of ASTM C-33 sand that is placed in maximum 6-inch thick lifts compacted to a minimum relative compaction of 95 percent using hand equipment. The final fill (12 inches above the pipe to the surface) should be non-expansive material compacted to a minimum of 95 percent relative compaction. All materials should be placed at optimum moisture content to 3 percent above the optimum moisture content. The Contractor should take measures to control migration of moisture in the trenches such as slurry collars, etc.

8.16.2 If ribbed or corrugated HDPE or metal pipes are used on the project, then the backfill should consist of ASTM C-33 sand or Class 2 Aggregate Base that is placed in maximum 6-inch thick lifts, extending to at least 1 foot above the top of pipe, and compacted to a minimum relative compaction of 92 percent using hand equipment. Prior to placement of the pipe, as a minimum, the pipe bedding should consist of a minimum of 4 inches of compacted (92 percent relative compaction) ASTM C-33 sand, Caltrans sand bedding or Class 2 Aggregate Base. As a minimum, the width of the trench should meet the requirements below to provide sufficient space between the sidewall of the trench and the pipe to allow compaction testing of the haunches and initial backfill with a nuclear density gage (minimum 12 inches). As an alternative to the trench width recommended above and the use of ASTM C-33 sand, Caltrans sand bedding or Class 2 Aggregate Base, a lesser trench width for HDPE pipes may be used if the entire trench is backfilled with a 2-sack sand-cement slurry from the bottom of the trench to a minimum of 1 foot above the top of the pipe.

Table No. 4
Minimum Trench Widths for HDPE Pipe with
ASTM C-33, Caltrans Sand Bedding or Class 2 AB Initial Backfill

Diameter of HDPE Pipe (inches)	Minimum Trench Width (inches) per ASTM D2321
12	36
18	42
24	48
36	60
48	72

8.16.3 Utility trench backfill placed in or adjacent to building areas, exterior slabs or pavements should be moisture conditioned to within optimum to 3 percent above the optimum moisture content and compacted to at least 92 percent of the maximum dry density as determined by ASTM Test Method D1557. The contractor should use appropriate equipment and methods to avoid damage to utilities and/or structures during placement and compaction of the backfill materials.

8.16.4 Trench backfill should be placed in 8 inch lifts, moisture conditioned to within optimum to 3 percent above optimum and compacted to achieve the minimum relative compaction.

8.16.5 On-site soils and approved imported engineered fill may be used as final backfill (the zone 12 inches above the top of pipe). It should be noted that granular type materials can be difficult to compact in narrow trenches.

8.16.6 Jetting of trench backfill is not recommended to compact the backfill soils.

8.16.7 Where utility trenches extend from the exterior to the interior limits of a building, lean concrete should be used as backfill material for a minimum distance of 2 feet laterally on each side of the exterior building line to prevent the trench from acting as a conduit to exterior surface water.

8.16.8 Storm drains and/or utility lines should be designed to be "watertight." If encountered, leaks should be immediately repaired. Leaking storm drain and/or utility lines could result in trench failure, sloughing and/or soil heave causing damage to surface and subsurface structures, pavements, flatwork, etc. In addition, landscaping irrigation systems should be monitored for leaks. It is recommended that the pipelines be inspected prior to placement of foundations, slabs-on-grade or pavements to verify that the pipelines are constructed properly and are "watertight."

8.16.9 The plans should note that utility trenches for electrical lines, irrigation lines, etc. should be compacted to a minimum relative compaction of 95 percent per ASTM D-1557, as required.

8.16.10Utility trenches should not be constructed within a zone defined by a line that extends at an inclination of 2 horizontal to 1 vertical downward from the bottom of building foundations.

8.16.11The project Civil Engineer should include slurry type cutoff collars along utility trenches at critical locations to prevent the migration of surface water into the trench and along the trench backfill material. The collars should extend a minimum of 6 inches into the trench sidewall and at least 5 feet above the top of pipe (as permitted based on the depth of the pipe).

8.17 Corrosion Protection

8.17.1 Based on the ASTM Special Technical Publication 741 and the analytical results of three (3) soil sample analyses, the soils are "mildly corrosive" to "relativity less corrosive" to ferrous alloy pipes. Buried metal objects should be protected in accordance with the manufacturer's recommendations based on the "moderately corrosive" corrosion potential of the soil. The evaluation was limited to the effects of soils to metal objects; corrosion due to other potential sources, such as stray currents and groundwater, was not evaluated.

8.17.2 Corrosion of concrete due to sulfate attack is not anticipated based on a low detected concentration of sulfates determined for the near-surface soils. According to Table 19-A-4 of the 2001 California Building Code, the concentration of sulfates falls in the negligible classification (0.00 to 0.10 percent by weight) for concrete. Therefore, restrictions are not required regarding the type, water-to-cement ratio, or strength of the concrete used for foundation and slabs due to the sulfate content.

8.17.3 These soil corrosion data should be provided to the manufacturers or suppliers of materials that will be in contact with soils (pipes or ferrous metal objects, etc.) to provide assistance in selecting the protection and materials for the proposed products or materials. If the manufacturers or suppliers cannot determine if materials are compatible with the soil corrosion conditions, a professional consultant, i.e., a corrosion engineer, with experience in corrosion protection should be consulted to design parameters. Twining is not a corrosion engineer; thus, cannot provide recommendations for mitigation of corrosive soil conditions. It is recommended that a corrosion engineer be consulted for the site specific conditions.

9.0 DESIGN CONSULTATION

9.1 Twining should be provided the opportunity to review those portions of the contract drawings and specifications that pertain to earthwork operations and foundations prior to finalization to determine whether they are consistent with our recommendations. This service is part of this current contractual agreement.

9.2 It is the client's responsibility to provide plans and specification documents for our review prior to their issuance for construction bidding purposes.

9.3 If Twining is not afforded the opportunity for review, we assume no liability for the misinterpretation of our conclusions and recommendations. This review is documented by a formal plan/specification review report provided by Twining.

10.0 CONSTRUCTION MONITORING

10.1 Twining should be retained to conduct the necessary observation, field-testing services and provide results so that action necessary to remedy indicated deficiencies can be taken in accordance with the plans and specifications. Upon completion of the work, Twining will provide a written summary of the observations, field testing and conclusions regarding the conformance of the completed work to the intent of the plans and specifications.

10.2 In the event that the earthwork operations for this project are conducted prior to the construction of the individual structures such that the construction sequence is not continuous, (or if construction operations disturb the surface soils) It is recommended that the exposed subgrade to receive floor slabs be tested to verify adequate compaction. If adequate compaction is not verified, the disturbed subgrade should be over-excavated, scarified, and compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM Test Method D1557. This condition should be verified prior to installation of plumbing, footing excavation, and construction of the slabs-on-grade.

10.3 Compaction tests should be conducted at a minimum frequency of:

Area	Minimum Test Frequency
Mass Fills or Subgrade	1 test per 2,500 square feet per compacted 6-inch lift
Pavement Subgrade	1 test per 5,000 square feet per compacted 6-inch lift
Utility Lines	1 test per 150 feet per 6-inch lift

The above testing frequencies are suggested rates for tests. Testing frequency should be adjusted by the field technician and Twining as needed based on continuous earthwork observation considering the methods used for compaction and the soil conditions.

10.4 The construction monitoring is an integral part of this investigation. This phase of the work provides Twining the opportunity to verify the subsurface conditions interpolated from the soil borings and make alternative recommendations if the conditions differ from those anticipated.

10.5 If Twining is not afforded the opportunity to provide engineering observation and field-testing services during construction activities related to earthwork, foundations, pavements and trenches; then, Twining will not be responsible for compliance of any aspect of the construction with our recommendations or performance of the structures or improvements if the recommendations of this report are not followed. We recommend that the geotechnical engineer selected to conduct these services that they provide evidence of professional liability insurance and review this report. After their review, the firm should, in writing, state that they understand and agree with the conclusions and recommendations of this report and agree to conduct sufficient observations and testing to ensure the construction complies with this report's recommendations.

10.6 Upon the completion of work, a final report should be prepared by Twining per the requirements of the 2001 California Building Code, Chapter 33, "Excavation and Grading," Section 3318.1, "Final Reports." This report is essential to ensure that the recommendations presented are incorporated into the project construction, and to note any deviations from the project plans and specifications. The client should notify Twining upon the completion of work to provide this report. This service is not, however, part of this current contractual agreement.

11.0 NOTIFICATION AND LIMITATIONS

11.1 The conclusions and recommendations presented in this report are based on the information provided regarding the proposed construction, and the results of the field and laboratory investigation, combined with interpolation of the subsurface conditions between boring locations.

11.2 The nature and extent of subsurface variations between borings may not become evident until construction.

11.3 If variations or undesirable conditions are encountered during construction, Twining should be notified promptly so that these conditions can be reviewed and the recommendations reconsidered where necessary. It should be noted that unexpected conditions frequently require additional expenditures for proper construction of the project.

11.4 If the proposed construction is relocated or redesigned, or if there is a substantial lapse of time between the submission of our report and the start of work (more than 12 months) at the site, or if conditions have changed due to natural cause or construction operations at or adjacent to the site, the conclusions and recommendations contained in this report should be considered invalid unless the changes are reviewed and our conclusions and recommendations modified or approved in writing.

11.5 Changed site conditions, or relocation of proposed structures, may require additional field and laboratory investigations to determine if our conclusions and recommendations are applicable considering the changed conditions or time lapse.

11.6 The conclusions and recommendations contained in this report are valid only for the project discussed in Section 3.4, Anticipated Construction. The use of the information and recommendations contained in this report for structures on this site not discussed herein or for structures on other sites not discussed in Section 3.3, Site Description is not recommended. The entity or entities that use or cause to use this report or any portion thereof for another structure or site not covered by this report shall hold Twining, its officers and employees harmless from any and all claims and provide Twining's defense in the event of a claim.

11.7 This report is issued with the understanding that it is the responsibility of the client to transmit the information and recommendations of this report to the Contract so that the steps necessary to carry out these recommendations in the design, construction and maintenance of the project are taken by the appropriate party.

11.8 This report presents the results of a geotechnical engineering investigation only and should not be construed as an environmental audit or study.

11.9 Our professional services were performed, our findings obtained, and our recommendations prepared in accordance with generally-accepted engineering principles and practices. This warranty is in lieu of all other warranties either expressed or implied.

11.10 This investigation report should not be used in the preparation of a Storm Water Pollution Prevention Plan (SWPPP). Use of this report or any data included in the report in preparation of a SWPPP would be at the owner's sole risk.

11.11 Reliance on this report by a third party (i.e., one who is not a party to our written agreement) is at the party's sole risk. If the project and/or site are purchased by another party, the purchaser must obtain written authorization and sign an agreement with Twining in order to rely upon the information provided in this report for design or construction of the project.

We appreciate the opportunity to be of service to Kings River Conservation District. If you have any questions regarding this report, or if we can be of further assistance, please contact us at your convenience.

Sincerely,
THE TWINING LABORATORIES, INC.

Zubair Anwar

Zubair Anwar, EIT
Staff Engineer
Geotechnical Engineering Division

REGISTERED GEOLOGIST
KENNETH JAMES CLARK
No. EG 1864
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA
Exp 5-31-07

Kenneth J. Clark, CEG
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REGISTERED PROFESSIONAL ENGINEER
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Read L. Andersen, RCE
Manager
Geotechnical Engineering Division

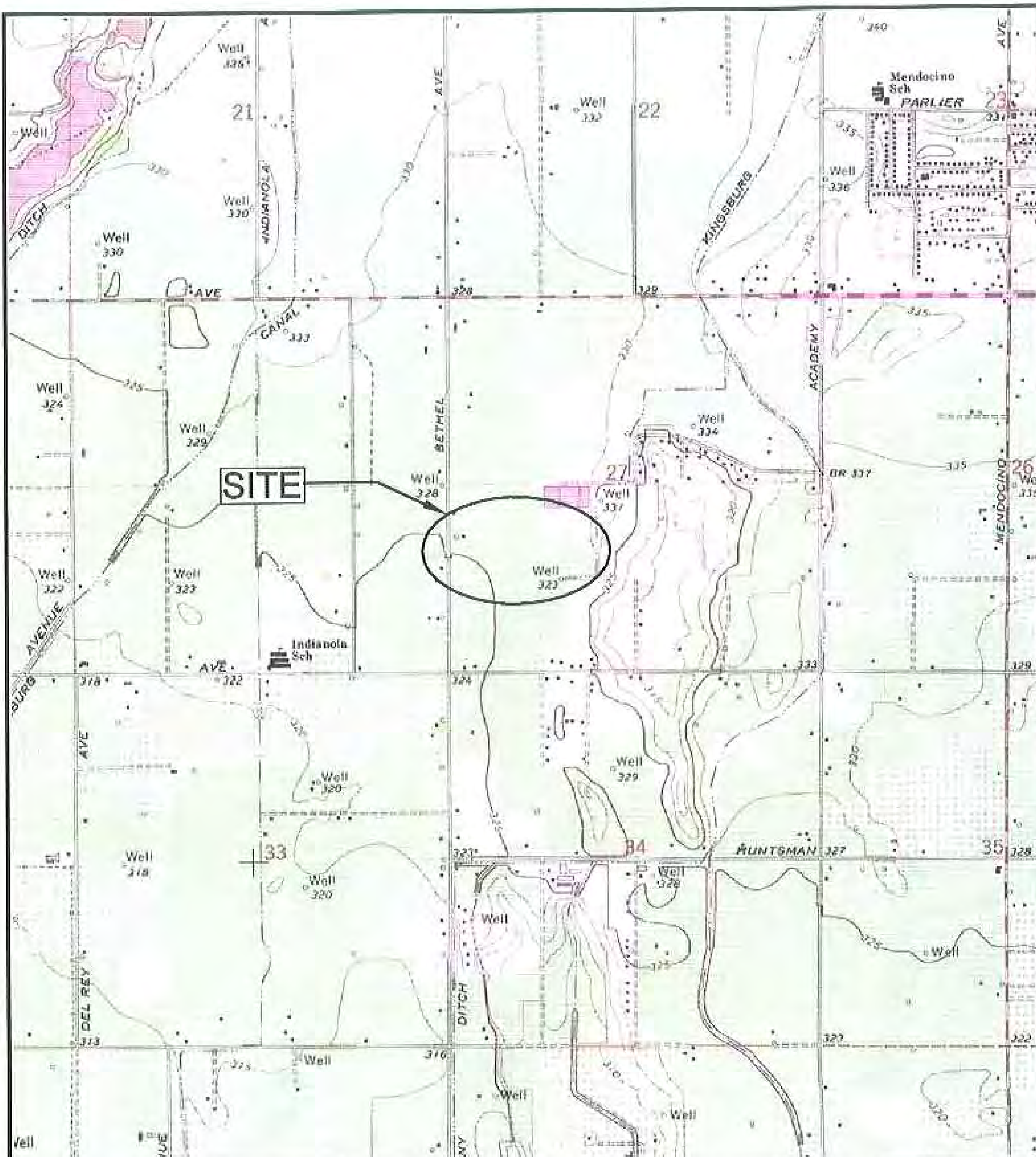
ZA/KJC/RLA/vv

APPENDIX A

DRAWINGS

Drawing No. 1 - Site Location Map

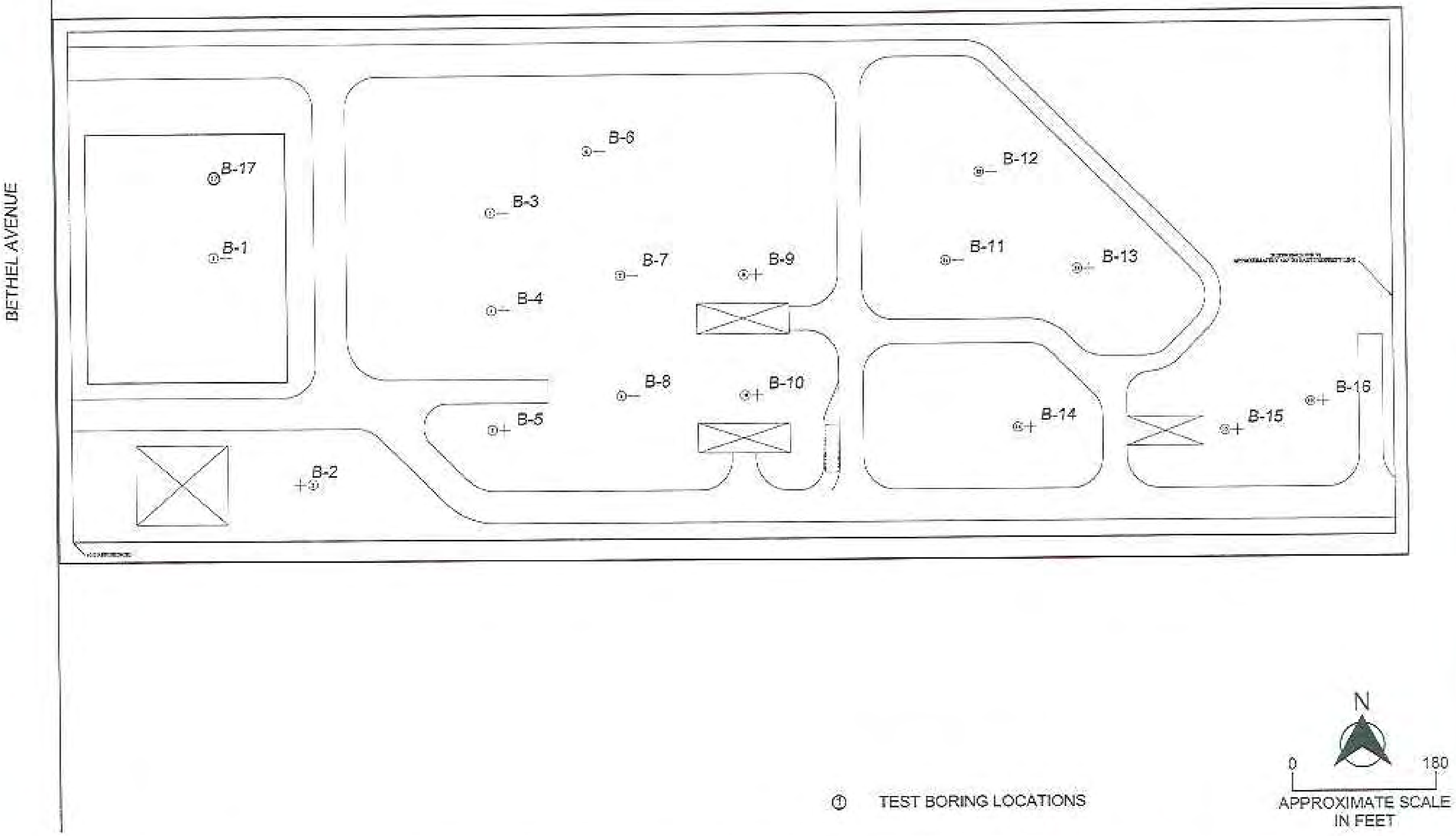
Drawing No. 2 - Site Plan



SOURCE: U.S.G.S. TOPOGRAPHIC MAP, 7 ½ MINUTE SERIES
SELMA, CALIFORNIA QUADRANGLE, PHOTOREVISED 1981

0 2000
APPROXIMATE SCALE
IN FEET

SITE LOCATION MAP KINGS RIVER CONSERVATION DISTRICT BETHEL AND DINUBA AVENUE SELMA, CALIFORNIA	FILE NO.: 08306-02-01	DATE: 12/18/06	THE TWINING LABORATORIES, INC. EST.1898 FRESNO/MODESTO/SACRAMENTO/CORONA/BAKERSFIELD/MONTEREY
	DRAWN BY: RM	APPROVED BY:	
	PROJECT NO. A08306.02	DRAWING NO. 1	



TEST BORING LOCATION MAP KINGS RIVER CONSERVATION DISTRICT BETHEL AND DINUBA AVENUE SLEMA, CALIFORNIA	FILE NO. 08306-02-02	DATE DRAWN: 12/18/06
	DRAWN BY: RM	APPROVED BY:
	PROJECT NO. F08306.02	DRAWING NO. 2

THE TWINING LABORATORIES, INC.

SOURCE: UTILITY ENGINEERING, D012659-100L101, SHEET 6, REV P DATED: 9/1/2006

APPENDIX B

LOGS OF BORINGS

This appendix contains the final logs of test borings. The logs represent our interpretation of the contents of the field logs and the results of the field and laboratory tests.

The test logs and related information depict subsurface conditions only at these locations and at the particular time designated on the logs. Soil conditions at other locations may differ from conditions occurring at these test boring locations. Also, the passage of time may result in changes in the soil conditions at these test boring locations.

In addition, an explanation of the abbreviations used in the preparation of the logs and a description of the Unified Soil Classification System are provided at the end of Appendix B.

THE TWINING LABORATORIES, INC. EST. 1898

SOIL TEST BORING SYMBOLIC LOG

BORING B-1

Project: Kings River Conservation District
Location: Selma, CA
Logged By: H.E.
Drilled By: T.B.
Drill Type: CME 75
Auger Type: HSA 6 5/8" O.D.

Project Number: TL F08306.02
Date: 10/31/06
Elevation: N/A
Depth to Groundwater: 40 feet
Cased to Depth: N/A
Hammer Type: TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
0	2/6 3/6 1/6	SM	SAND, Silty; Loose, dry, fine to medium, brown		4	2.8
					--	3.0
5	2/6 1/6 2/6		Damp, very loose	DD=104.3	3	3.4
10	3/6 3/6 4/6		Loose, increase in silt		7	8.4
15	4/6 8/6 7/6		Medium dense, reddish brown		15	8.1
20	5/6 5/6 6/6	SP	Poorly graded sand, medium dense, damp		11	3.5
25	3/6 5/6 6/6		Loose		11	3.8
30	1/6 7/6 7/6				14	3.6

Notes:

Figure Number

THE TWINING LABORATORIES, INC. EST. 1898

SOIL TEST BORING SYMBOLIC LOG

BORING B-1

Project: Kings River Conservation District

Location: Selma, CA

Logged By: H.E.

Drilled By: T.B.

Drill Type: CME 75

Auger Type: HSA 6 5/8" O.D.

Project Number: TL F08306.02

Date: 10/31/06

Elevation: N/A

Depth to Groundwater: 40 feet

Cased to Depth: N/A

Hammer Type: TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
35	4/6 5/6 4/6		Loose, interbedded Silt layer		9	4.9
40	2/6 5/6 4/6		Groundwater encountered		9	20.0
45	3/6 6/6 6/6	SM	SAND, Silty; Medium dense, wet, fine to medium, brown		12	13.6
50	4/6 5/6 8/6				13	14.3
			Bottom of boring at 51.5 feet			
55						
60						
65						

Notes:

Figure Number

THE TWINING LABORATORIES, INC. EST. 1898

SOIL TEST BORING SYMBOLIC LOG

BORING B-2

Project: Kings River Conservation District
Location: Selma, CA
Logged By: H.E.
Drilled By: T.B.
Drill Type: CME 75
Auger Type: HSA 6 5/8"O.D.

Project Number: TL F08306.02
Date: 10/31/06
Elevation: N/A
Depth to Groundwater: N/E
Cased to Depth: N/A
Hammer Type: TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
0		SM	SAND, Silty; Loose, moist, fine to medium, brown	ø=27° C=10psf		
	2/6 2/6 2/6				4	6.9
5	1/6 2/6 2/6				4	11.0
					--	10.7
10	9/6 18/6 22/6		Dense		40	5.9
15	7/6 9/6 9/6		Medium dense		18	
20	6/6 7/6 9/6	SP	SAND, Poorly Graded; Moist, fine to medium, brown		16	3.6
			Bottom of boring at 21.5 feet			
25						
30						

Notes:

Figure Number

THE TWINING LABORATORIES, INC. EST. 1938

SOIL TEST BORING SYMBOLIC LOG

BORING B-3

Project: Kings River Conservation District
Project Number: TL F08306.02

Location: Selma, CA
Date: 10/31/06

Logged By: H.E.
Elevation: N/A

Drilled By: T.B.
Depth to Groundwater: N/E

Drill Type: CME 75
Cased to Depth: N/A

Auger Type: HSA 6 5/8"O.D.
Hammer Type: TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
0		SM	SAND, Silty; Loose, moist, fine to medium, brown			
	2/6 2/6 2/6				4	4.3
5				DD=95.7pcf	--	4.7
	3/6 4/6 6/6				10	5.0
10	14/6 14/6 24/6		Dense, seams of silt		38	10.4
15	10/6 4/6 11/6		Medium dense		15	6.8
20	3/6 4/6 7/6	SP	SAND, Poorly Graded; Medium dense, moist, fine to medium, brown		11	3.6
			Bottom of boring at 21.5 feet			
25						
30						

Notes:

Figure Number

SOIL TEST BORING SYMBOLIC LOG

THE TWINING LABORATORIES, INC. EST. 1898

BORING B-4

Project: Kings River Conservation District

Location: Selma, CA

Logged By: H.E.

Drilled By: T.B.

Drill Type: CME 75

Auger Type: HSA 6 5/8"O.D.

Project Number: TL F08306.02

Date: 10/31/06

Elevation: N/A

Depth to Groundwater: N/E

Cased to Depth: N/A

Hammer Type: TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
0	2/6 3/6 3/6	SM	SAND, Silty; Loose, moist, fine to medium, brown		6	3.8
	2/6 2/6 2/6				4	4.7
5						
					--	19.3
10	9/6 14/6 12/6	ML	SILT, Sandy; Very stiff, moist, non plastic, gray		26	21.8
		SM	SAND, Silty; Medium dense, moist, fine to medium, brown			
15	7/6 7/6 7/6				14	7.9
20	3/6 5/6 7/6	SP	SAND, Poorly Graded; Medium dense, moist, fine to medium, brown		12	3.2
			Bottom of boring at 21.5 feet			
25						
30						

Notes:

Figure Number B-4

SOIL TEST BORING SYMBOLIC LOG

THE TWINING LABORATORIES, INC.
EST. 1898

BORING B-5

Project: Kings River Conservation District
Project Number: TL F08306.02

Location: Selma, CA
Date: 10/31/06

Logged By: H.E.
Elevation: N/A

Drilled By: T.B.
Depth to Groundwater: N/E

Drill Type: CME 75
Cased to Depth: N/A

Auger Type: HSA 6 5/8"O.D.
Hammer Type: TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
0	1/6 2/6 2/6	SM	SAND, Silty; Loose, moist, fine to medium, brown		4	5.4
				DD=102.7pcf	--	7.6
5	2/6 2/6 2/6				4	8.7
10	5/6 5/6 4/6	ML	SILT, Sandy; Stiff, moist, non plastic, gray		9	20.0
15	5/6 9/6 9/6	SM	SAND, Silty; Medium, dense, moist, fine to medium, brown		18	4.8
20	3/6 5/6 7/6	SP	SAND, Poorly Graded; Medium dense, moist, fine to medium, brown		12	3.0
			Bottom of boring at 21.5 feet			
25						
30						

Notes:

Figure Number B-5

SOIL TEST BORING SYMBOLIC LOG

THE TWINING LABORATORIES, INC.

EST. 1898

BORING B-6

Project: Kings River Conservation District

Location: Selma, CA

Logged By: H.E.

Drilled By: T.B.

Drill Type: CME 75

Auger Type: HSA 6 5/8"O.D.

Project Number: TL F08306.02

Date: 10/31/06

Elevation: N/A

Depth to Groundwater: 40 feet

Cased to Depth: N/A

Hammer Type: TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
0	1/6 2/6 4/6	SM	SAND, Silty; Loose, damp, fine to medium, brown		6	3.2
				DD=111.3pcf	--	4.1
5	4/6 2/6 2/6		Increase in sand		4	4.6
10	3/6 5/6 7/6		Medium dense, reddish brown		12	7.9
15	6/6 4/6 11/6				15	6.9
20	4/6 6/6 8/6		Sharp increase in sand, gray brown		14	3.0
25	5/6 5/6 8/6	SP	SAND, Poorly Graded; Medium dense, dry, gray		13	2.9
30	14/6 10/6 6/6	ML	SILT; Very stiff, dry, non plastic, brown		16	17.4

Notes:

Figure Number B-6

THE TWINING LABORATORIES, INC. EST. 1898

SOIL TEST BORING SYMBOLIC LOG

BORING B-6

Project: Kings River Conservation District
Location: Selma, CA
Logged By: H.E.
Drilled By: T.B.
Drill Type: CME 75
Auger Type: HSA 6 5/8"O.D.

Project Number: TL F08306.02
Date: 10/31/06
Elevation: N/A
Depth to Groundwater: 40 feet
Cased to Depth: N/A
Hammer Type: TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
35	4/6 7/6 11/6	SM	SAND, Silty; Medium dense, damp, fine to medium, interbedded clay layer		18	18.4
40	4/6 4/6 5/6		Loose, wet		9	14.7
45	2/6 3/6 5/6				8	18.2
50	2/6 2/6 4/6		Sharp increase in sand		6	23.8
			Bottom of boring at 51.5 feet			
55						
60						
65						

Notes:

Figure Number B-6

SOIL TEST BORING SYMBOLIC LOG

THE TWINING LABORATORIES, INC. EST. 1898

BORING B-7

Project: Kings River Conservation District	**Project Number:** TL F08306.02
Location: Selma, CA	**Date:** 10/31/06
Logged By: H.E.	**Elevation:** N/A
Drilled By: T.B.	**Depth to Groundwater:** 35 feet
Drill Type: CME 75	**Cased to Depth:** N/A
Auger Type: HSA 6 5/8"O.D.	**Hammer Type:** TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
0	2/6 3/6 3/6	SM	SAND, Silty; Loose, moist, fine to medium, brown		5	4.3
					--	8.8
5	5/6 4/6 4/6	ML	SILT, Sandy; Stiff, moist, non plastic, gray		8	21.7
10	7/6 12/6 10/6	SM	SAND, Silty; Medium dense, moist, fine to medium, brown, trace of clay		22	10.5
15	4/6 5/6 7/6		Brown, no clay		12	5.6
20	4/6 5/6 5/6	ML	SILT, Sandy; Stiff, moist, nonplastic, gray	-200=84.3%	10	16.0
25	4/6 5/6 8/6	SP	SAND, Poorly Graded; Medium dense, dry, gray		13	3.9
30	3/6 3/6 7/9	ML	SILT, Sandy; Stiff, moist, non plastic, gray		10	20.9

Notes:

Figure Number B-7

THE TWINING LABORATORIES, INC. EST. 1898

SOIL TEST BORING SYMBOLIC LOG

BORING B-7

Project: Kings River Conservation District

Location: Selma, CA

Logged By: H.E.

Drilled By: T.B.

Drill Type: CME 75

Auger Type: HSA 6 5/8"O.D.

Project Number: TL F08306.02

Date: 10/31/06

Elevation: N/A

Depth to Groundwater: 35 feet

Cased to Depth: N/A

Hammer Type: TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
35	3/6 5/6 5/6	SM	SAND, Silty; Medium dense, wet, fine, gray		10	25.0
40	3/6 3/6 5/6				8	16.0
45	3/6 3/6 6/6				9	19.2
50	4/6 7/6 8/6		Medium dense, seams of poorly graded sand		15	18.7
55			Bottom of boring at 55 feet			
60						
65						

Notes:

Figure Number B-7

SOIL TEST BORING SYMBOLIC LOG

THE TWINING LABORATORIES, INC. EST. 1898

BORING B-8

Project: Kings River Conservation District

Location: Selma, CA

Logged By: H.E.

Drilled By: T.B.

Drill Type: CME 75

Auger Type: HSA 6 5/8" O.D.

Project Number: TL F08306.02

Date: 10-31-06

Elevation: N/A

Depth to Groundwater: 40 feet

Cased to Depth: N/A

Hammer Type: TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
0	2/6 2/6 3/6	ML	SILT; Medium stiff, damp, non plastic	C=0psf ø=26° EI=0	5	14.7
	2/6 2/6 4/6				8	12.5
5						
10		SM	SAND, Silty; Loose, damp, fine to medium, reddish brown		--	7.4
	10/6 6/6 3/6				9	7.1
15	3/6 4/6 5/6				9	5.4
20	9/6 5/6 6/6	SP	SAND, Poorly Graded; Medium dense, damp, gray to brown		11	3.6
25	4/6 4/6 6/6				10	3.8
30	18/6 24/6 25/6	SM	SAND, Silty; Dense, damp, fine, brown to gray		49	10.5

Notes:

Figure Number B-8

SOIL TEST BORING SYMBOLIC LOG

THE TWINING LABORATORIES, INC. EST. 1898

BORING B-8

Project: Kings River Conservation District
Location: Selma, CA
Logged By: H.E.
Drilled By: T.B.
Drill Type: CME 75
Auger Type: HSA 6 5/8" O.D.

Project Number: TL F08306.02
Date: 10-31-06
Elevation: N/A
Depth to Groundwater: 40 feet
Cased to Depth: N/A
Hammer Type: TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
35	18/6 22/6 29/6		Very dense	-200=49.1%	51	12.5
40	2/6 5/6 5/6	ML	SILT, Sandy; Very stiff, moist, non plastic, brown		10	23.2
45	11/6 16/6 20/6		Very stiff	-200=59.4%	36	20.5
50	6/6 10/6 15/6	SM	SAND, Silty; Medium dense, wet, fine to medium, brown		25	15.2
			Bottom of boring at 51.5 feet			
55						
60						
65						

Notes:

Figure Number B-8

SOIL TEST BORING SYMBOLIC LOG

THE TWINING LABORATORIES, INC. EST. 1898

BORING B-9

Project: Kings River Conservation District
Location: Selma, CA
Logged By: H.E.
Drilled By: T.B.
Drill Type: CME 75
Auger Type: HSA 6 5/8" O.D.

Project Number: TL F08306.02
Date: 10-31-06
Elevation: N/A
Depth to Groundwater: 40 feet
Cased to Depth: N/A
Hammer Type: TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
0	2/6 2/6 3/6	ML	SILT, Sandy; Medium stiff, damp, non plastic, brown	-200=62.9%	5	7.7
5	2/6 3/6 3/6				6	10.1
10	3/6 7/6 12/6	SM	SAND, Silty; Medium dense, fine to medium, reddish brown		19	8.4
15	5/6 10/6 15/6				25	5.5
20	3/6 5/6 5/6	SP	SAND, Poorly Graded; Medium dense, dry, gray		10	2.6
25	5/6 5/6 5/6	SM	SAND, Silty; Medium dense, damp, fine to medium, brown		10	6.9
30	8/6 8/6 10/6	ML	SILT, Sandy; Very stiff, damp, low plasticity, brown	-200=56.1%	18	18.0

Notes:

SOIL TEST BORING SYMBOLIC LOG

THE TWINING LABORATORIES, INC. EST. 1898

BORING B-9

Project: Kings River Conservation District

Location: Selma, CA

Logged By: H.E.

Drilled By: T.B.

Drill Type: CME 75

Auger Type: HSA 6 5/8" O.D.

Project Number: TL F08306.02

Date: 10-31-06

Elevation: N/A

Depth to Groundwater: 40 feet

Cased to Depth: N/A

Hammer Type: TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
35	8/6 8/6 11/6		Interbedded layer of poorly graded sand		19	20.0
40	8/6 4/6 6/6	SM	SAND, Silty; Medium dense, very moist, fine to medium, brown		10	14.0
45	7/6 8/6 12/6	ML	SILT, Sandy; Very stiff, wet, non plastic, brown		20	17.0
50	5/6 7/6 11/6	SM	SAND, Silty; Medium dense, wet, fine to medium, brown		18	13.7
			Bottom of boring at 51.5 feet			
55						
60						
65						

Notes:

THE TWINING LABORATORIES, INC.

EST. 1906

SOIL TEST BORING SYMBOLIC LOG

BORING B-10

Project: Kings River Conservation District

Location: Selma, CA

Logged By: H.E.

Drilled By: T.B.

Drill Type: CME 75

Auger Type: HSA 6 5/8" O.D.

Project Number: TL F08306.02

Date: 10-31-06

Elevation: N/A

Depth to Groundwater: 40 feet

Cased to Depth: N/A

Hammer Type: TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
0		ML	SILT; Medium stiff, damp, non plastic, brown			
	3/6 2/6 3/6				5	9.7
5	3/6 3/6 3/6				6	15.8
10	7/6 14/6 10/6	SM	SAND, Silty; Medium dense, damp, fine, reddish brown	DD=42.3%	24	11.7
15	8/6 8/6 9/6		Sharp increase in sand		17	5.5
20	5/6 4/6 5/6	SP	SAND, Poorly Graded; Loose, damp, gray, very fine sand		9	7.9
25	4/6 11/6 12/6		Medium dense, decrease in fines		23	2.9
30	19/6 21/6 20/6	SM	Silt; Damp, very stiff, non plastic, gray		41	18.8

Notes:

Figure Number B-10

SOIL TEST BORING SYMBOLIC LOG

THE TWINING LABORATORIES, INC. EST. 1889

BORING B-10

Project: Kings River Conservation District
Location: Selma, CA
Logged By: H.E.
Drilled By: T.B.
Drill Type: CME 75
Auger Type: HSA 6 5/8" O.D.

Project Number: TL F08306.02
Date: 10-31-06
Elevation: N/A
Depth to Groundwater: 40 feet
Cased to Depth: N/A
Hammer Type: TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
35	12/6 11/6 21/6		Sharp increase in in silt		32	22.7
40	7/6 7/6 9/6	SM	SAND, Silty; Medium dense, wet, fine to medium, brown		16	13.9
45	18/6 20/6 21/6	ML	SILT, Very stiff, damp, non plastic, gray		41	19.4
50	4/6 12/6 15/6	SM	SAND, Silty; Medium dense, wet, fine to medium, brown		27	14.5
55	4/6 6/6 7/6				13	19.9
60			Heaving sands up to 7 feet into the auger Bottom of boring at 60 feet			
65						

Notes:

Figure Number B-10

THE TWINING LABORATORIES, INC.

EST. 1898

SOIL TEST BORING SYMBOLIC LOG

BORING B-11

Project: Kings River Conservation District

Location: Selma, CA

Logged By: H.E.

Drilled By: T.B.

Drill Type: CME 75

Auger Type: HSA 6 5/8" O.D.

Project Number: TL F08306.02

Date: 10-31-06

Elevation: N/A

Depth to Groundwater: 40 feet

Cased to Depth: N/A

Hammer Type: TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
0	1/6 2/6 2/6	SM	SAND, Silty; Loose, damp, fine, gray to brown		4	4.9
5	4/6 4/6 4/6	ML	SILT, Sandy; Stiff, damp, non plastic, brown		8	1.8
10	6/6 5/6 6/6		Increase in silt content		11	17.8
15	5/6 7/6 12/6	SM	SAND, Silty; Medium dense, damp, fine to medium, reddish brown		19	10.4
20	5/6 7/6 9/6	ML SP	SILT; Very stiff, Very moist, non to low plasticity, brown SAND, Poorly Graded; Medium dense, damp, gray		16	26.0 2.0
25	6/6 12/6 12/6	ML	SILT, Sandy; Very stiff, moist, non plastic, brown		24	13.4
30	12/6 12/6 14/6				26	21.6

Notes:

Figure Number B-11

THE TWINING LABORATORIES, INC. EST. 1898

SOIL TEST BORING SYMBOLIC LOG

BORING B-11

Project: Kings River Conservation District
Location: Selma, CA
Logged By: H.E.
Drilled By: T.B.
Drill Type: CME 75
Auger Type: HSA 6 5/8" O.D.

Project Number: TL F08306.02
Date: 10-31-06
Elevation: N/A
Depth to Groundwater: 40 feet
Cased to Depth: N/A
Hammer Type: TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
35	6/6 8/6 9/6				17	
40	4/6 6/6 8/6	SP-SM	Poorly Graded sand, medium dense, wet, gray	-200=6.0%	14	22.0
45	4/6 6/6 7/6	ML	SILT, Stiff, wet, low to non plastic, brown		13	23.0
50	4/6 6/6 14/6		Very stiff, interbedded layer of sand		20	20.4
55			Heaving sands up to 7 feet			
60						
65						

Notes:

THE TWINING LABORATORIES, INC. EST. 1898

SOIL TEST BORING SYMBOLIC LOG

BORING B-12

Project: Kings River Conservation District
Location: Selma, CA
Logged By: H.E.
Drilled By: T.B.
Drill Type: CME 75
Auger Type: HSA 6 5/8" O.D.

Project Number: TL F08306.02
Date: 10-31-06
Elevation: N/A
Depth to Groundwater: N/E
Cased to Depth: N/A
Hammer Type: TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
0		ML	SILT, Sandy; Medium stiff, damp, non plastic, brown			
	2/6 2/6 3/6				5	
5	2/6 2/6 2/6		Increase in sand		4	
10	2/6 2/6 5/6	SM	SAND, Silty; Stiff, damp, non plastic, brown		7	
15	3/6 6/6 10/6		Medium dense		16	
20	2/6 5/6 6/6	ML	SILT, Sandy; Stiff, damp, non plastic, brown		11	
25	3/6 3/6 5/6	SM	SAND, Silty; loose, damp, fine to medium, gray to brown		8	
30	3/6 6/6 6/6	ML	SILT, Very stiff, moist, low to non plastic, brown		12	
			Bottom of boring at 31.5 feet			

Notes:

Figure Number B-12

SOIL TEST BORING SYMBOLIC LOG

THE TWINING LABORATORIES, INC. EST. 1898

BORING B-13

Project: Kings River Conservation District

Location: Selma, CA

Logged By: H.E.

Drilled By: T.B.

Drill Type: CME 75

Auger Type: HSA 6 5/8" O.D.

Project Number: TL F08306.02

Date: 10-31-06

Elevation: N/A

Depth to Groundwater: 40 feet

Cased to Depth: N/A

Hammer Type: TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
0		ML	SILT, Sandy; Medium stiff, damp, non plastic, brown			
	3/6 3/6 2/6				5	
5					--	
	5/6 3/6 3/6				6	
10	3/6 3/6 3/6	SM	SAND, Silty; Medium dense, damp, fine to medium, gray to brown		6	
15	4/6 8/6 20/6		Medium dense, reddish brown		28	
20	4/6 9/6 12/6	ML	SILT, Sandy; Very stiff, damp, non plastic, black brown		21	
25	4/6 5/6 4/6		Stiff, increase in sand with depth		9	
30	3/6 4/6 5/6				9	

Notes:

SOIL TEST BORING SYMBOLIC LOG

THE TWINING LABORATORIES, INC. EST. 1898

BORING B-13

Project: Kings River Conservation District

Location: Selma, CA

Logged By: H.E.

Drilled By: T.B.

Drill Type: CME 75

Auger Type: HSA 6 5/8" O.D.

Project Number: TL F08306.02

Date: 10-31-06

Elevation: N/A

Depth to Groundwater: 40 feet

Cased to Depth: N/A

Hammer Type: TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
35	7/6 6/6 7/6	SM	SAND, Silty; Medium dense, damp, fine, gray brown		13	
40	4/6 6/6 8/6		Wet, sharp increase in sand		14	
45	4/6 6/6 8/6		Wet, interbedded silt layers		14	
50	12/6 20/6 25/6		Dense		45	
			Bottom of boring at 51.5 feet			
55						
60						
65						

Notes:

Figure Number B-13

THE TWINING LABORATORIES, INC. EST. 1896

SOIL TEST BORING SYMBOLIC LOG

BORING B-14

Project: Kings River Conservation District
Location: Selma, CA
Logged By: H.E.
Drilled By: T.B.
Drill Type: CME 75
Auger Type: HSA 6 5/8" O.D.

Project Number: TL F08306.02
Date: 10-31-06
Elevation: N/A
Depth to Groundwater: 40 feet
Cased to Depth: N/A
Hammer Type: TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
0	3/6 2/6 2/6	ML	SILT; Medium stiff, damp, non plastic, gray		4	
					--	
5	4/6 6/6 11/6	SM	SAND, Silty; Medium dense, damp, fine to medium, reddish brown		17	
10	15/6 13/6 16/6	ML	SILT, Sandy; Very stiff, damp, non plastic, brown		29	
15	5/6 5/6 6/6	SM	SAND, Silty; Medium dense, damp, fine to medium, reddish brown		11	
20	5/6 6/6 7/6	SP	SAND, Poorly Graded; Medium dense, damp, gray		13	
25	5/6 4/6 7/6	ML	SILT, Stiff, damp, non plastic, gray		11	
30	6/6 5/6 5/6		Medium stiff, increase in sand		10	

Notes:

Figure Number B-14

THE **TWINING** LABORATORIES, INC. EST. 1898

SOIL TEST BORING SYMBOLIC LOG

BORING B-14

Project: Kings River Conservation District

Location: Selma, CA

Logged By: H.E.

Drilled By: T.B.

Drill Type: CME 75

Auger Type: HSA 6 5/8" O.D.

Project Number: TL F08306.02

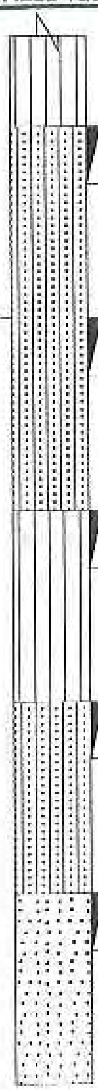
Date: 10-31-06

Elevation: N/A

Depth to Groundwater: 40 feet

Cased to Depth: N/A

Hammer Type: TRIP



ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
35	3/6 2/6 2/6	SM	SAND, Silty; Loose, wet, fine to medium, brown		4	
40	12/6 6/6 7/6		Medium dense, very moist, increase in fine sand		13	
45	9/6 11/6 14/6	ML	SILT, Very stiff, wet, low to non plastic, brown		25	
50	2/6 4/6 5/6	SM	SAND, Silty; Loose, wet, fine to medium, brown		9	
55	2/6 2/6 2/6	SP	SAND, Poorly Graded; Loose wet, gray		4	
60			Heaving sand Bottom of boring at 60 feet			
65						

Notes:

THE TWINING LABORATORIES, INC. EST. 1898

SOIL TEST BORING SYMBOLIC LOG

BORING B-15

Project: Kings River Conservation District

Location: Selma, CA

Logged By: H.E.

Drilled By: T.B.

Drill Type: CME 75

Auger Type: HSA 6 5/8" O.D.

Project Number: TL F08306.02

Date: 10-31-06

Elevation: N/A

Depth to Groundwater: N/E

Cased to Depth: N/A

Hammer Type: TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
0	1/6 2/6 3/6	SM	SAND, Silty; Loose, damp, fine to medium, brown		5	3.1
					--	
5	1/6 3/6 3/6				6	4.5
10	2/6 3/6 3/6	SP	SAND, Poorly Graded; Loose, damp, gray		6	3.6
15	3/6 3/6 3/6				6	3.7
20	3/6 4/6 5/6	ML	SILT; Stiff, damp, low plasticity, brown, trace of clay		9	15.0
			Bottom of boring at 21.5 feet			
25						
30						

Notes:

THE TWINING LABORATORIES, INC.

EST. 1898

SOIL TEST BORING SYMBOLIC LOG

BORING B-16

Project: Kings River Conservation District

Location: Selma, CA

Logged By: H.E.

Drilled By: T.B.

Drill Type: CME 75

Auger Type: HSA 6 5/8" O.D.

Project Number: TL F08306.02

Date: 10-31-06

Elevation: N/A

Depth to Groundwater: 40 feet

Cased to Depth: N/A

Hammer Type: TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
0	3/6 3/6 3/6	SM	SAND, Silty; Loose, damp, fine to medium, brown		6	3.9
5					--	
	5/6 5/6 6/6		Medium dense, increase in sand		11	3.9
10	6/6 6/6 9/6	SP	SAND, Poorly Graded; Medium dense, damp, gray		15	2.8
15	1/6 3/6 5/6				8	3.6
20	3/6 4/6 3/6	ML	SILT; Stiff, damp, non plastic, brown		7	20.2
25	7/6 7/6 8/6	SM	SAND, Silty; Medium dense, damp, fine, gray		16	13.3
30	7/6 12/6 12/6	ML	SILT, Sandy; Stiff, damp, non plastic, brown		24	12.2

Notes:

SOIL TEST BORING SYMBOLIC LOG

THE TWINING LABORATORIES, INC. EST. 1898

BORING B-16

Project: Kings River Conservation District
Location: Selma, CA
Logged By: H.E.
Drilled By: T.B.
Drill Type: CME 75
Auger Type: HSA 6 5/8" O.D.

Project Number: TL F08306.02
Date: 10-31-06
Elevation: N/A
Depth to Groundwater: 40 feet
Cased to Depth: N/A
Hammer Type: TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
35	13/6 11/6 14/6	SM	SAND, Silty; Medium dense, fine to medium, light gray		25	16.6
40			Dense, increase in silt			19.8
45	20/6 15/6 18/6	ML	SILT; Very stiff, wet, non plastic, brown		32	14.4
50	10/6 8/6 10/6				18	14.8
			Bottom of boring at 51.5 feet			
55						
60						
65						

Notes:

SOIL TEST BORING SYMBOLIC LOG

THE TWINING LABORATORIES, INC. EST. 1898

BORING B-17

Project: Kings River Conservation District
Project Number: TL F08306.02

Location: Selma, CA
Date: 10-31-06

Logged By: H.E.
Elevation: N/A

Drilled By: T.B.
Depth to Groundwater: 40 feet

Drill Type: CME 75
Cased to Depth: N/A

Auger Type: HSA 6 5/8" O.D.
Hammer Type: TRIP

ELEVATION/ DEPTH (feet)	SOIL SYMBOLS SAMPLER SYMBOLS AND FIELD TEST DATA	USCS	Soil Description	Remarks	N-value	Moisture Content %
0	2/6 3/6 3/6	SM	SAND, Silty; Loose, moist, fine to medium, brown		6	3.0
					--	4.2
5	6/6 4/6 5/6				9	
	3/6 4/6 5/6	SP	SAND, Poorly Graded; Loose, moist, fine to medium, brown		9	3.4
10						
	3/6 3/6 6/6	SC	SAND, Clayey; Loose, moist, fine to medium, brown		9	12.3
15						
	3/6 5/6 6/6	SM	SAND, Silty; Medium dense, moist, fine to medium, brown		11	5.9
20						
	3/6 4/6 6/6	SP	SAND, Poorly Graded; Medium dense, moist, fine to medium, brown		10	3.2
25						
	3/6 4/6 5/6		Loose		9	3.8
30			Bottom of boring at 30 feet			

Notes:

Figure Number **B-17**

KEY TO SYMBOLS

Symbol	Description
Strata symbols	
	SAND, Silty (SM)
	SAND, Poorly Graded (SP)
	SILT, Sandy (ML)
	SAND, Poorly Graded with Silt (SP-SM)
	SAND, Clayey (SC)
0-Misc. Symbols	
	Boring continues
	Water table during drilling

Symbol	Description
Soil Samplers	
	Standard penetration test
	California Modified split barrel ring sampler

Notes:

1. Soil borings were drilled on 10/23/06 using a CME-75 portable Hollow Stem Auger drill rig equipped with 8" O.D. Hollow Stem auger.

2. Groundwater was not encountered during drilling activities at the site.

3. Boring locations were located with reference to the existing site features.

4. These logs are subject to the limitations, conclusions, and recommendations in this report.

5. Results of tests conducted on samples recovered are reported on the logs. Abbreviations used are:

N/A =	Not applicable
N/E =	None Encountered
bsg =	Below Site Grade
PID =	Photo Ionization Detector
ppm =	Parts per Million

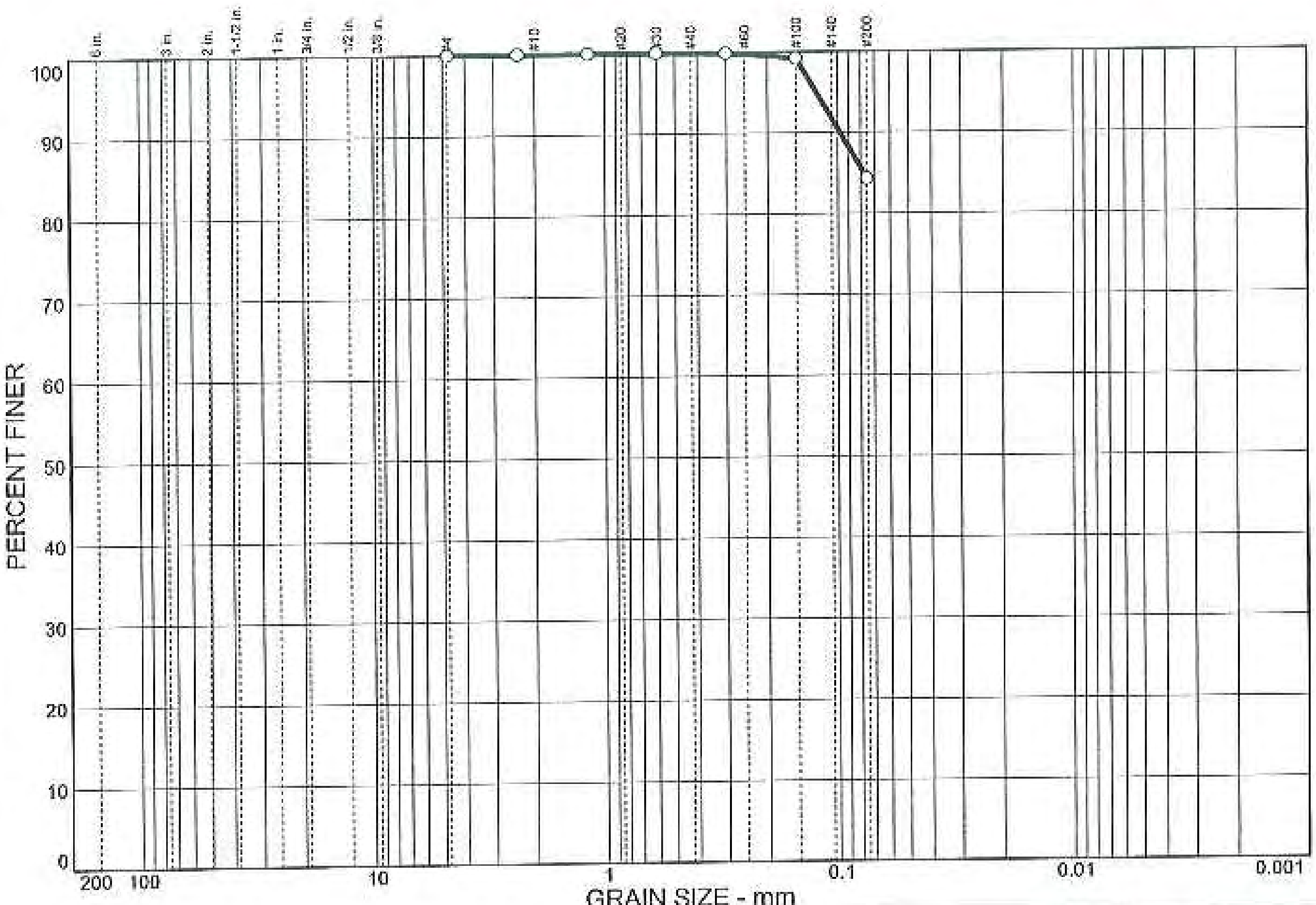
APPENDIX C

RESULTS OF LABORATORY TESTS

This appendix contains the individual results of the following tests. The results of the moisture content and dry density tests are included on the test boring logs in Appendix B. These data, along with the field observations, were used to prepare the final test boring logs in Appendix B.

These Included:	To Determine:
Dry Density (ASTM D2216)	Dry unit weight of sample representative of in-situ or in-place undisturbed condition.
Grain-Size Distribution (ASTM D422)	Size and distribution of soil particles, i.e., clay, silt, sand, and gravel.
Direct Shear (ASTM D3080)	Soil shearing strength under varying loads and/or moisture conditions.
Consolidation (ASTM D2435)	The amount and rate at which a soil sample compresses when loaded, and the influence of saturation on its behavior.
R-Value (CTM 301)	The capacity of a subgrade or subbase to support a pavement section designed to carry a specified traffic load.
Expansion Index (UBC 18-2)	Swell potential of soil with increases in moisture content.
Moisture-Density Relationship (ASTM D1557)	The optimum (best) moisture content for compacting soil and the maximum dry unit weight (density) for a given compactive effort.

Sulfate Content (ASTM D4327)	Percentage of water-soluble sulfate as (SO_4) in soil samples. Used as an indication of the relative degree of sulfate attack on concrete and for selecting the cement type.
Chloride Content (ASTM D4327)	Percentage of soluble chloride in soil. Used to evaluate the potential attack on encased reinforcing steel.
Resistivity (ASTM D1125)	The potential of the soil to corrode metal.
pH (ASTM D4972)	The acidity or alkalinity of subgrade material.



	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	0.0	0.0	0.0	0.0	15.7	84.3	

SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	DESCRIPTION	USCS
○		B-7	20-21.5'		ML

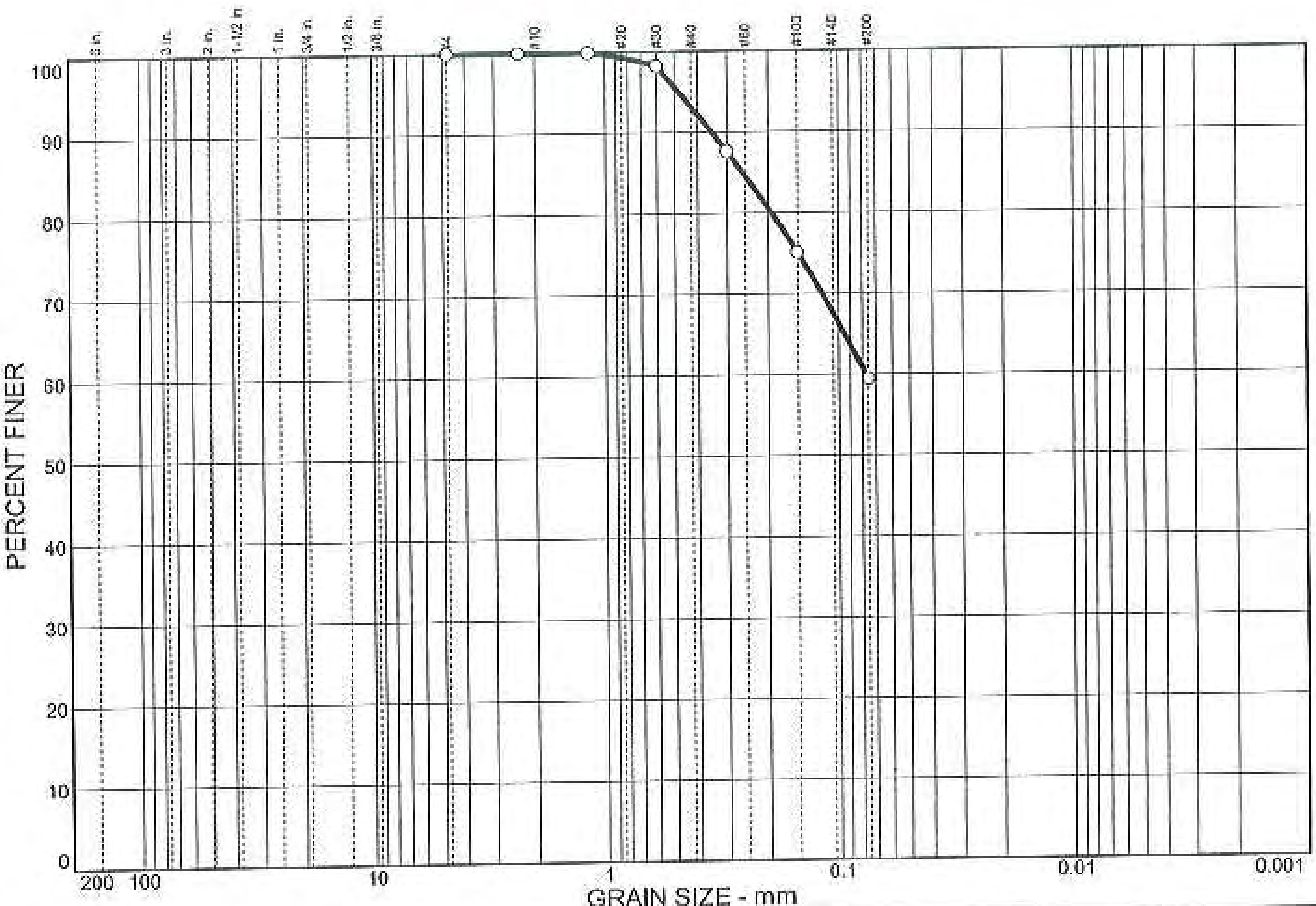
THE TWINING LABORATORIES, INC.

Client: Kings River Conservation District

Project: Kings River Conservation District

Project No.: F08306.02

Figure No.



	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	0.0	0.0	0.0	6.8	33.8	59.4	

SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	DESCRIPTION	USCS
○		B-8	45-46.5'		ML

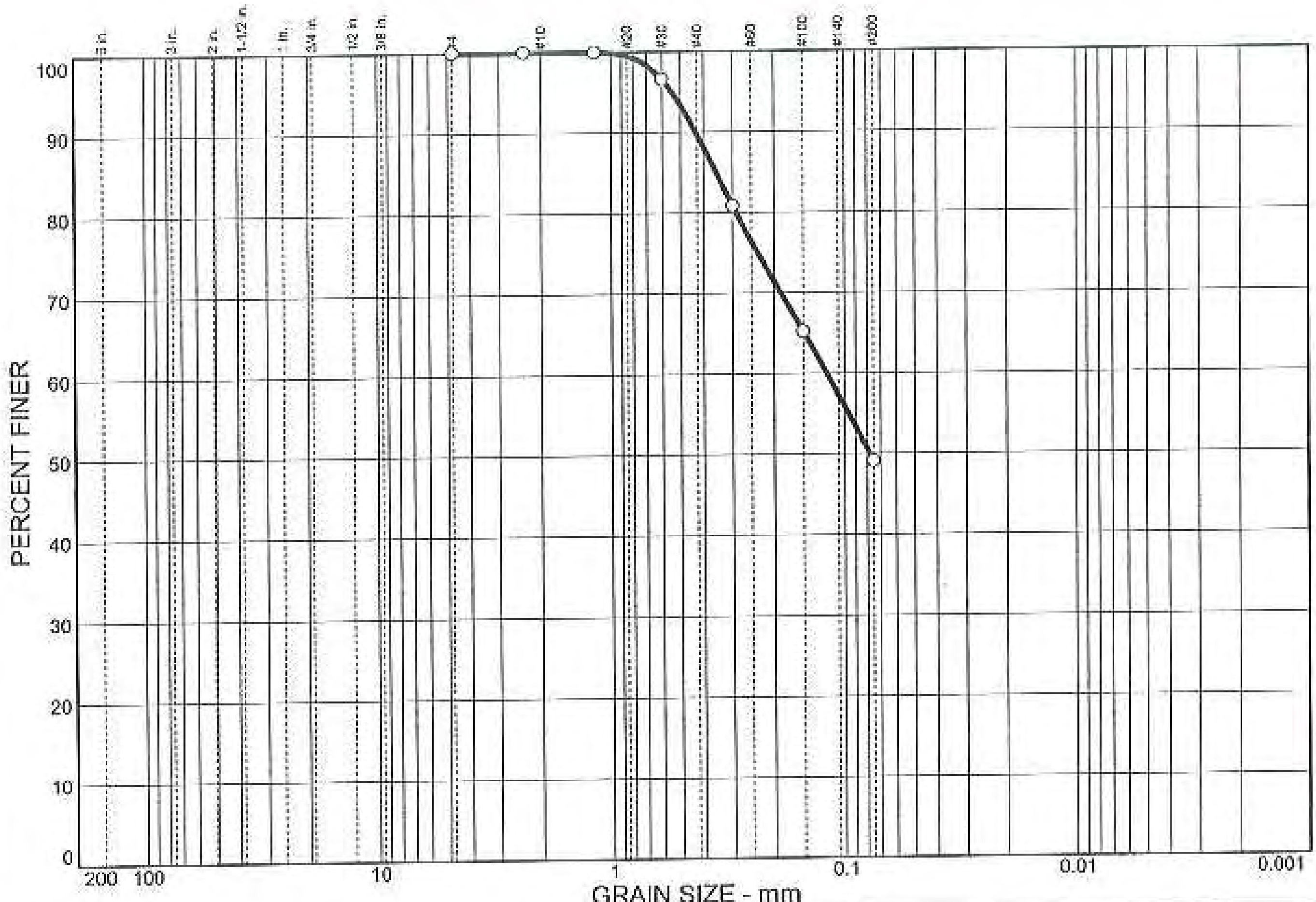
THE TWINING LABORATORIES, INC.

Client: Kings River Conservation District

Project: Kings River Conservation District

Project No.: F08306.02

Figure No.



	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
O	0.0	0.0	0.0	0.0	10.4	40.5	49.1	

SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	DESCRIPTION	USCS
O		B-8	35-36.5'		SM

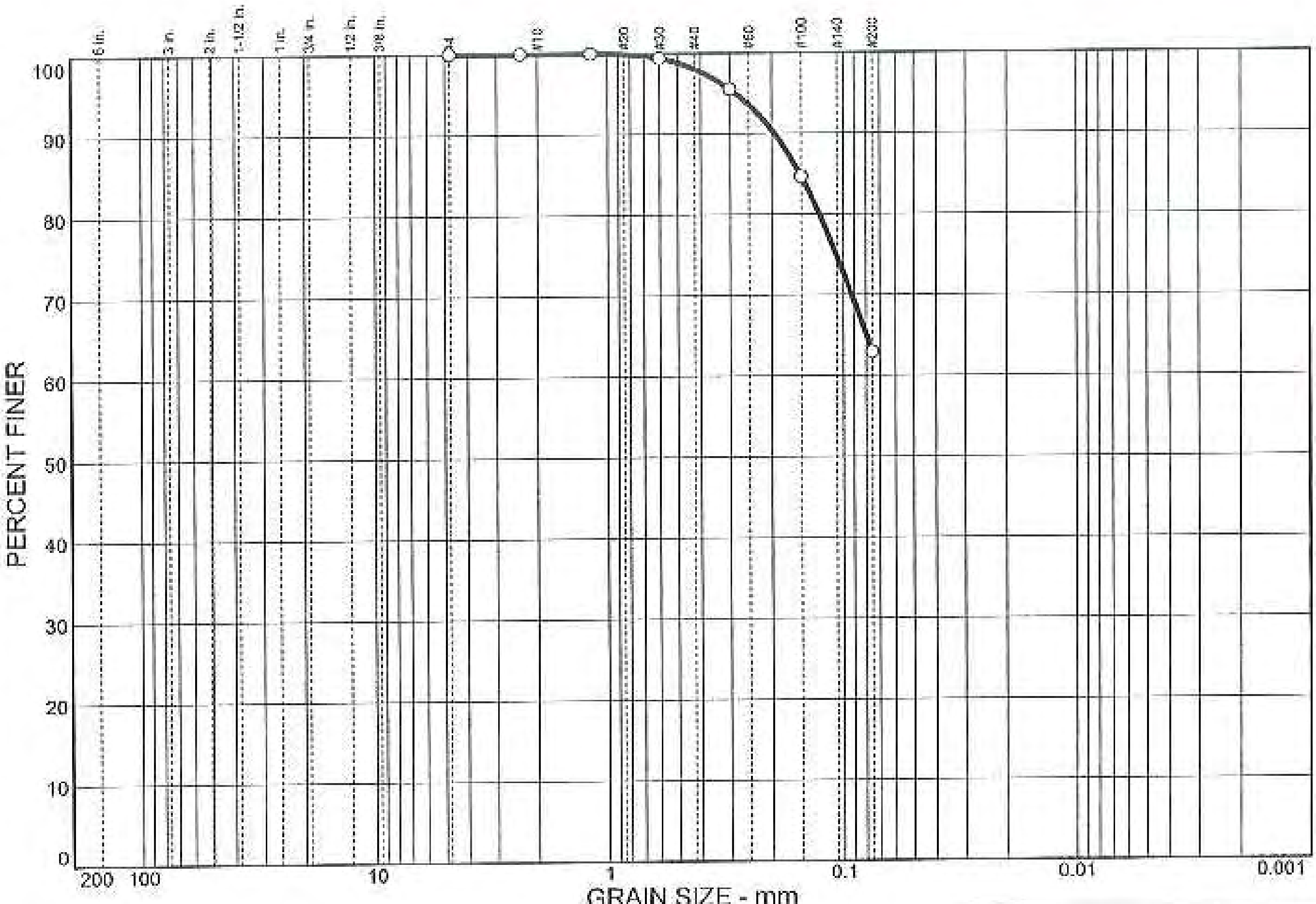
THE TWINING LABORATORIES, INC.

Client: Kings River Conservation District

Project: Kings River Conservation District

Project No.: F08306.02

Figure No.



	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
O	0.0	0.0	0.0	0.0	2.0	35.1	62.9	

SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	DESCRIPTION	USCS
O		B-9	1-2.5'		ML

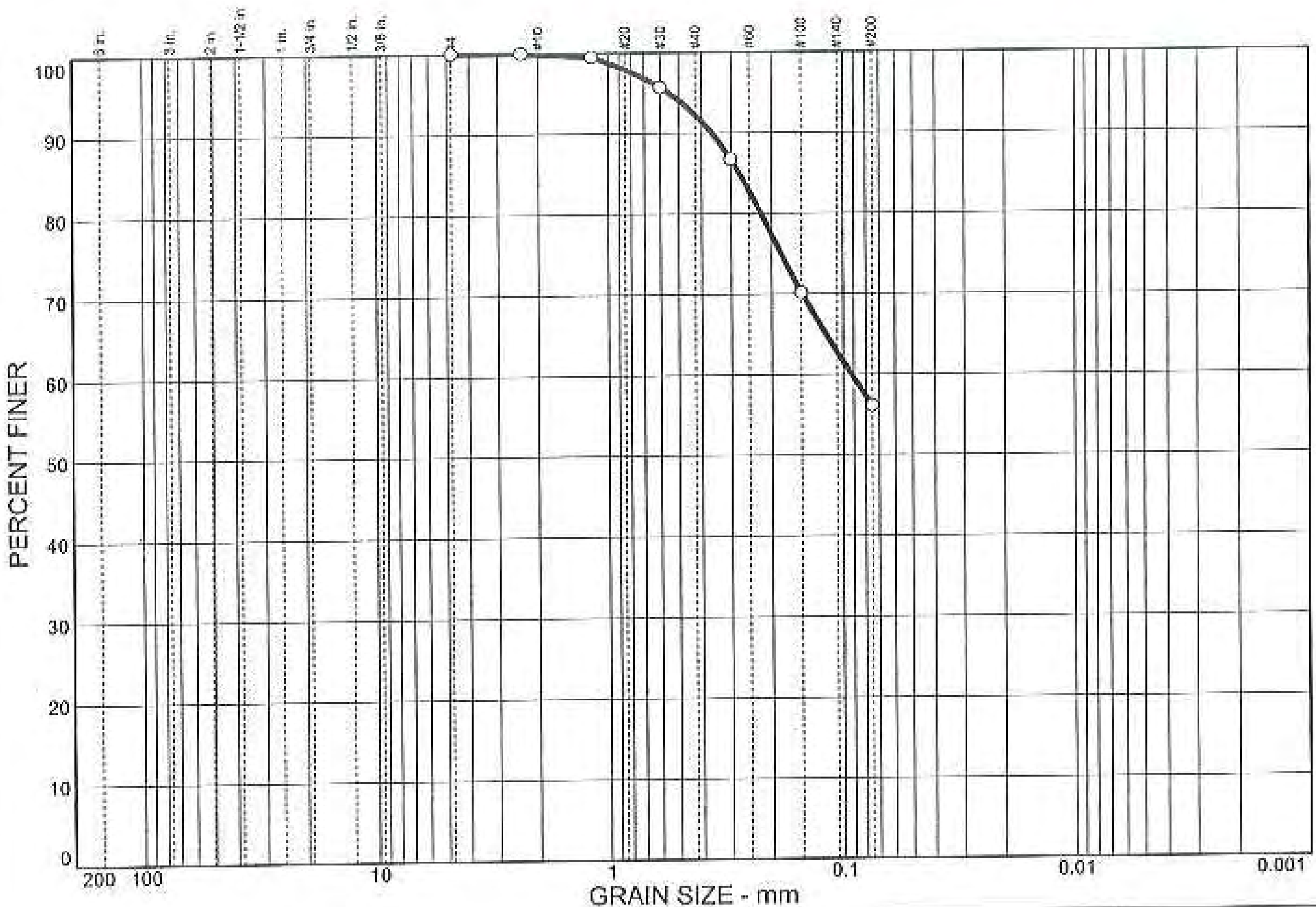
THE TWINING LABORATORIES, INC.

Client: Kings River Conservation District

Project: Kings River Conservation District

Project No.: F08306.02

Figure No.



	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	0.0	0.0	0.1	7.6	36.2	56.1	

SOIL DATA

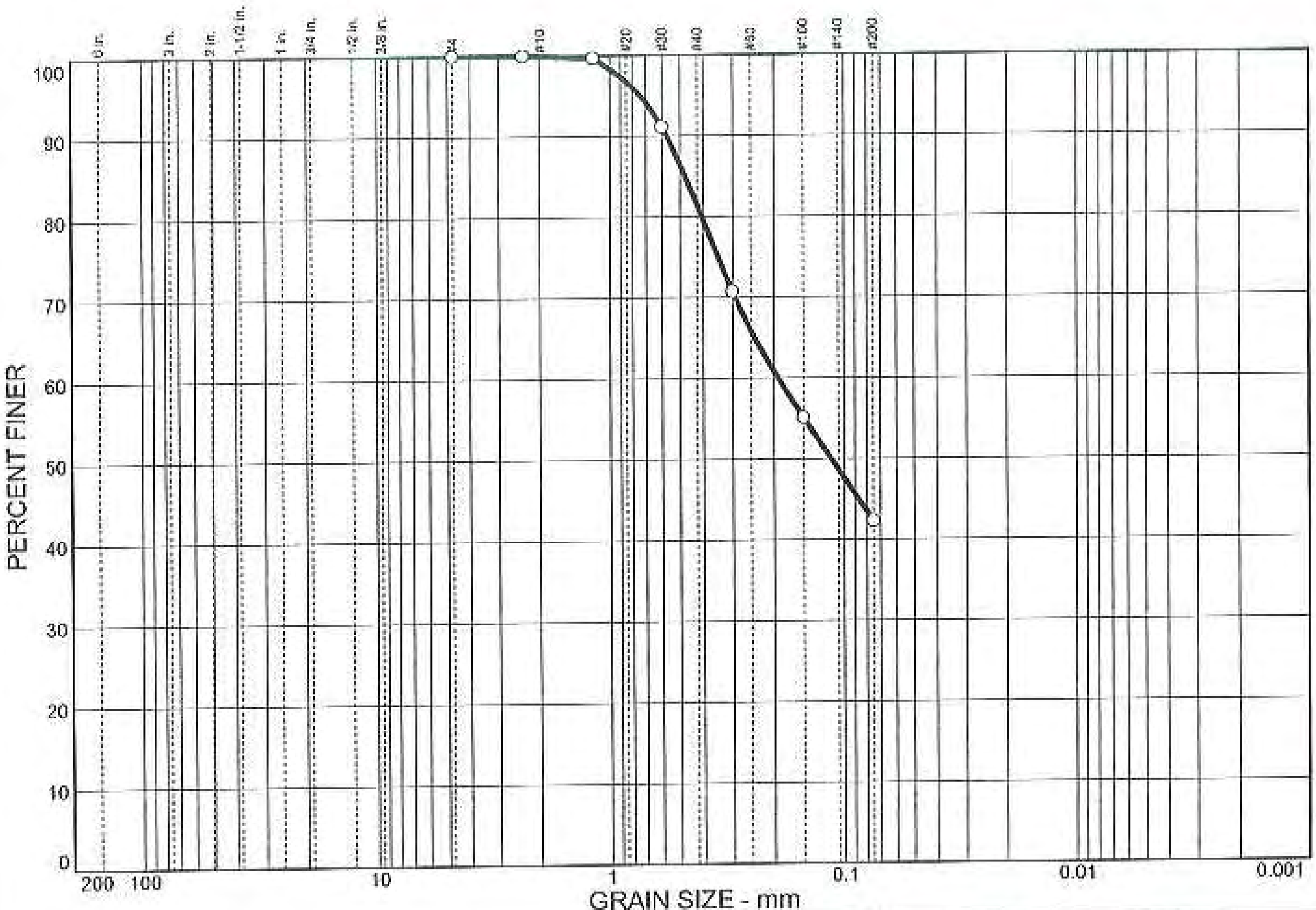
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	DESCRIPTION	USCS
○		B-9	30-31.5'		ML

THE TWINING LABORATORIES, INC.

Client: Kings River Conservation District

Project: Kings River Conservation District

Project No.: F08306.02 **Figure No.**



	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
O	0.0	0.0	0.0	0.1	18.4	39.2	42.3	

SOIL DATA

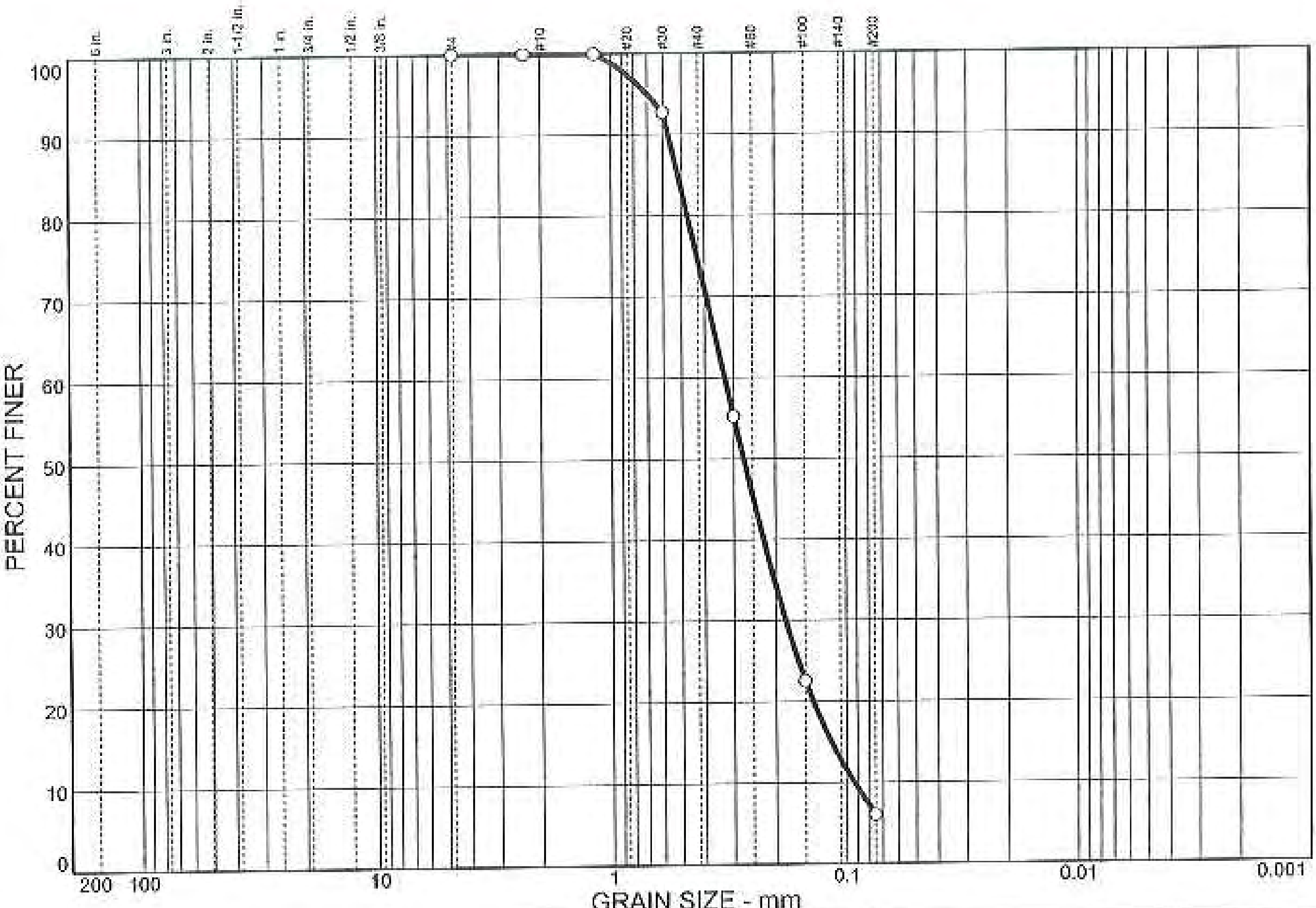
SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	DESCRIPTION	USCS
O		B-10	10-11.5'		SM

THE TWINING LABORATORIES, INC.

Client: Kings River Conservation District

Project: Kings River Conservation District

Project No.: F08306.02 **Figure No.**

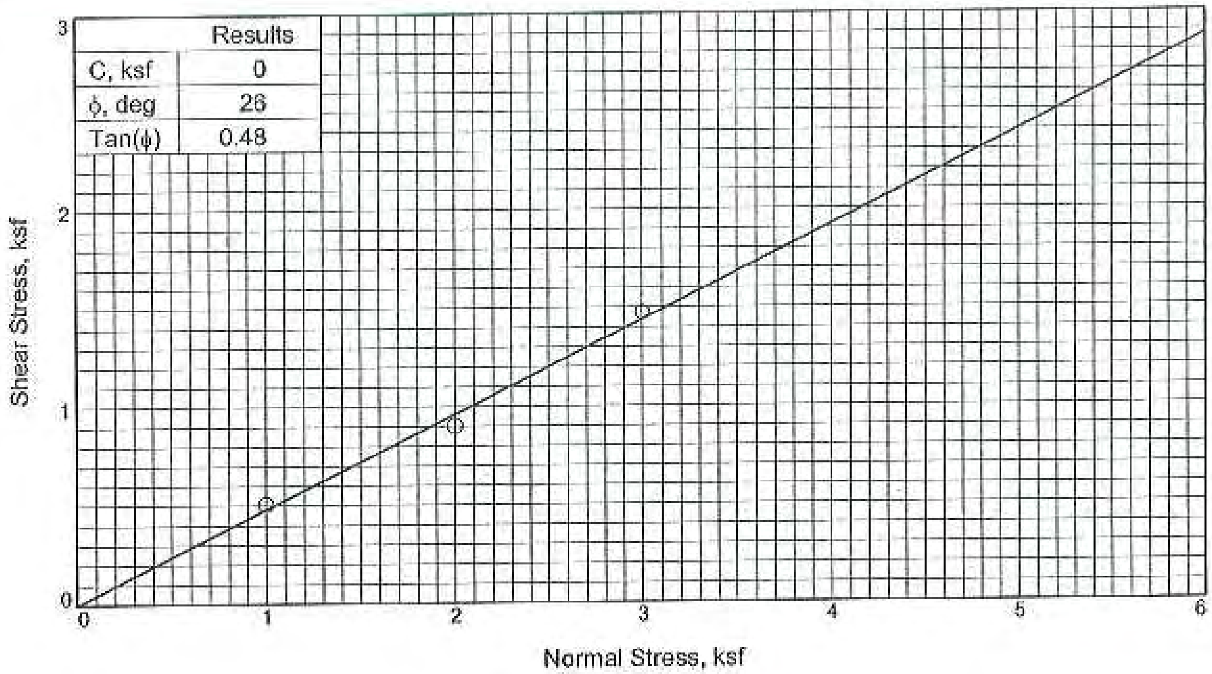


	% + 3"	% GRAVEL		% SAND			% FINES	
		CRS.	FINE	CRS.	MEDIUM	FINE	SILT	CLAY
○	0.0	0.0	0.0	0.0	25.7	68.3	6.0	

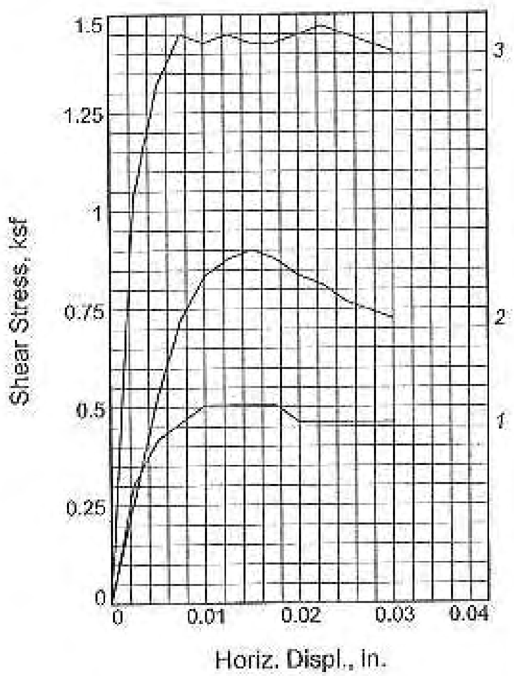
SOIL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	DESCRIPTION	USCS
○		B-11	40-41.5		SP-SM

THE TWINING LABORATORIES, INC.	**Client:** Kings River Conservation District **Project:** Kings River Conservation District **Project No.:** F08306.02 **Figure No.**



Results	
C, ksf	0
ϕ, deg	26
Tan(ϕ)	0.48



	Sample No.	1	2	3
Initial	Water Content, %	19.2	19.0	18.3
	Dry Density, pcf	103.3	104.6	104.2
	Saturation, %	84.8	86.4	82.4
	Void Ratio	0.6013	0.5820	0.5883
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	19.2	19.1	18.4
	Dry Density, pcf	104.8	107.2	107.2
	Saturation, %	88.0	93.2	89.9
	Void Ratio	0.5792	0.5434	0.5435
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.99	0.98	0.97
	Normal Stress, ksf	1.00	2.00	3.00
	Shear Stress, ksf	0.50	0.90	1.47
	Displacement, in.	0.01	0.02	0.02
	Ult. Stress, ksf			
	Displacement, in.			
	Strain at peak, %	0.4	0.6	0.9

Sample Type:

Description:

Assumed Specific Gravity= 2.65

Remarks:

Figure No. ______

Client: Kings River Conservation District

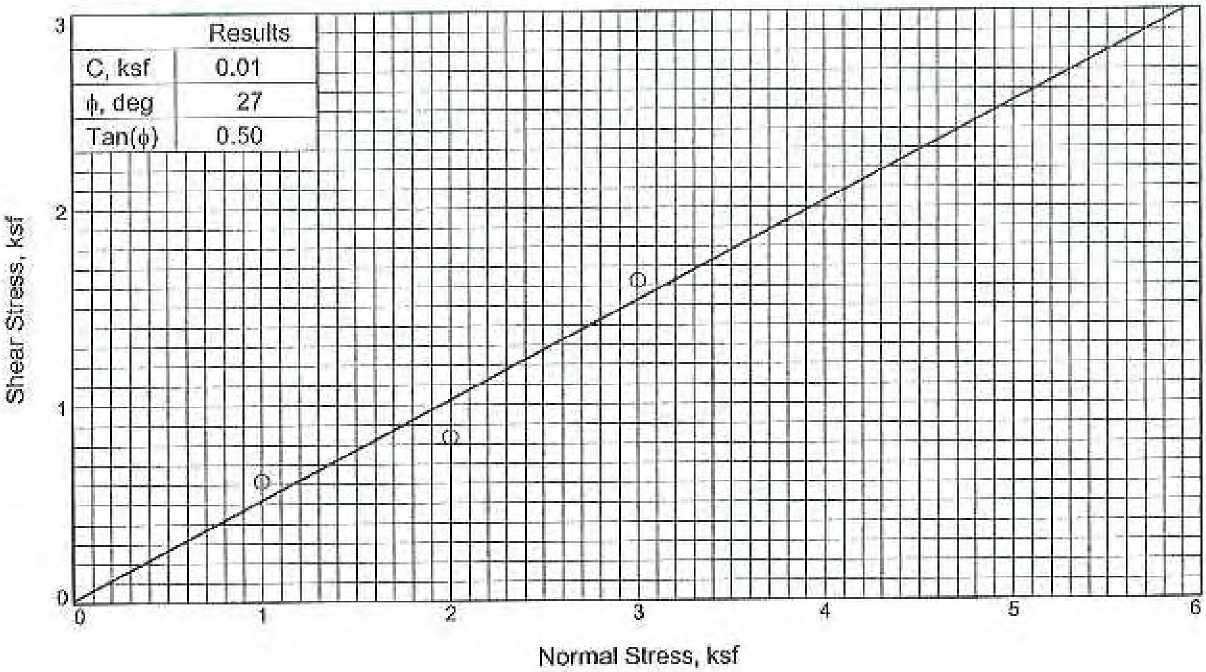
Project: Kings River Conservation District

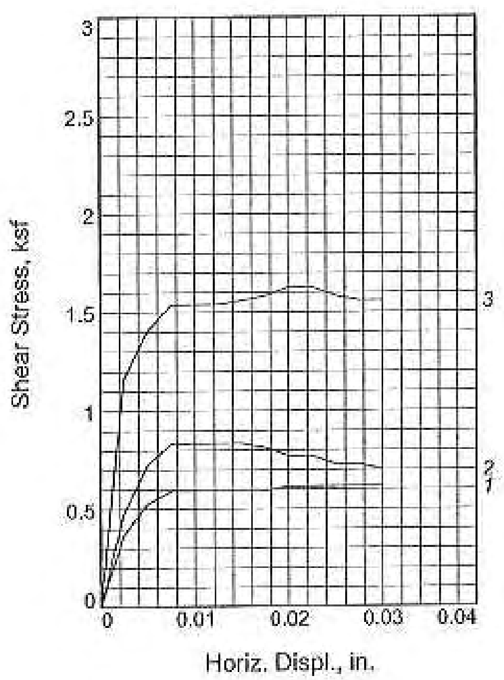
Sample Number: B-8 **Depth:** 0-3'

Proj. No.: F08306.02 **Date:**

DIRECT SHEAR TEST REPORT

THE TWINING LABORATORIES, INC





Sample No.		1	2	3
Initial	Water Content, %	17.5	19.7	17.2
	Dry Density, pcf	104.8	103.2	102.3
	Saturation, %	80.4	86.6	73.8
	Void Ratio	0.5784	0.6035	0.6170
	Diameter, in.	2.42	2.42	2.42
	Height, in.	1.00	1.00	1.00
At Test	Water Content, %	17.0	18.5	17.0
	Dry Density, pcf	105.6	106.9	103.8
	Saturation, %	79.7	89.4	75.8
	Void Ratio	0.5672	0.5479	0.5941
	Diameter, in.	2.42	2.42	2.42
	Height, in.	0.99	0.97	0.99
Normal Stress, ksf		1.00	2.00	3.00
Shear Stress, ksf		0.61	0.83	1.62
Displacement, in.		0.02	0.01	0.02
Ult. Stress, ksf				
Displacement, in.				
Strain at peak, %		0.8	0.3	0.8

Sample Type:
Description:

Assumed Specific Gravity= 2.65
Remarks:

Figure No. _______

Client: Kings River Conservation District

Project: Kings River Conservation District

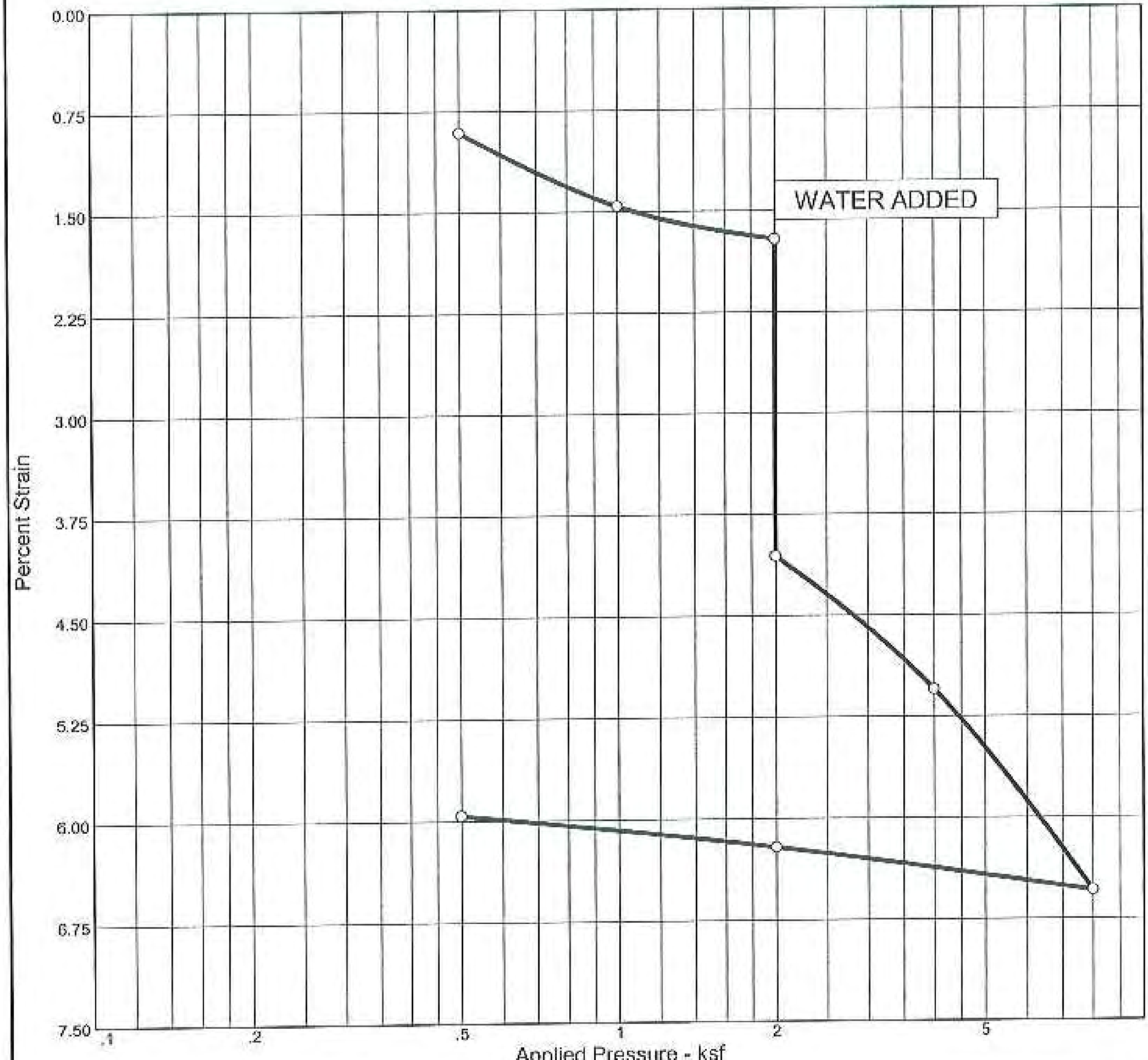
Sample Number: B-2 **Depth:** 0-3'

Proj. No.: F08306.02 **Date:**

DIRECT SHEAR TEST REPORT

THE TWINING LABORATORIES, INC

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Clpse. %	e_o
Sat.	Moist.											
10.9 %	3.0 %	104.5			2.65		4.20	0.09	0.01		2.4	0.74

MATERIAL DESCRIPTION	USCS	AASHTO

Project No. F08306.02 **Client:** Kings River Conservation District

Project: Kings River Conservation District

Source: **Sample No.:** B-1 **Elev./Depth:** 3.5-5'

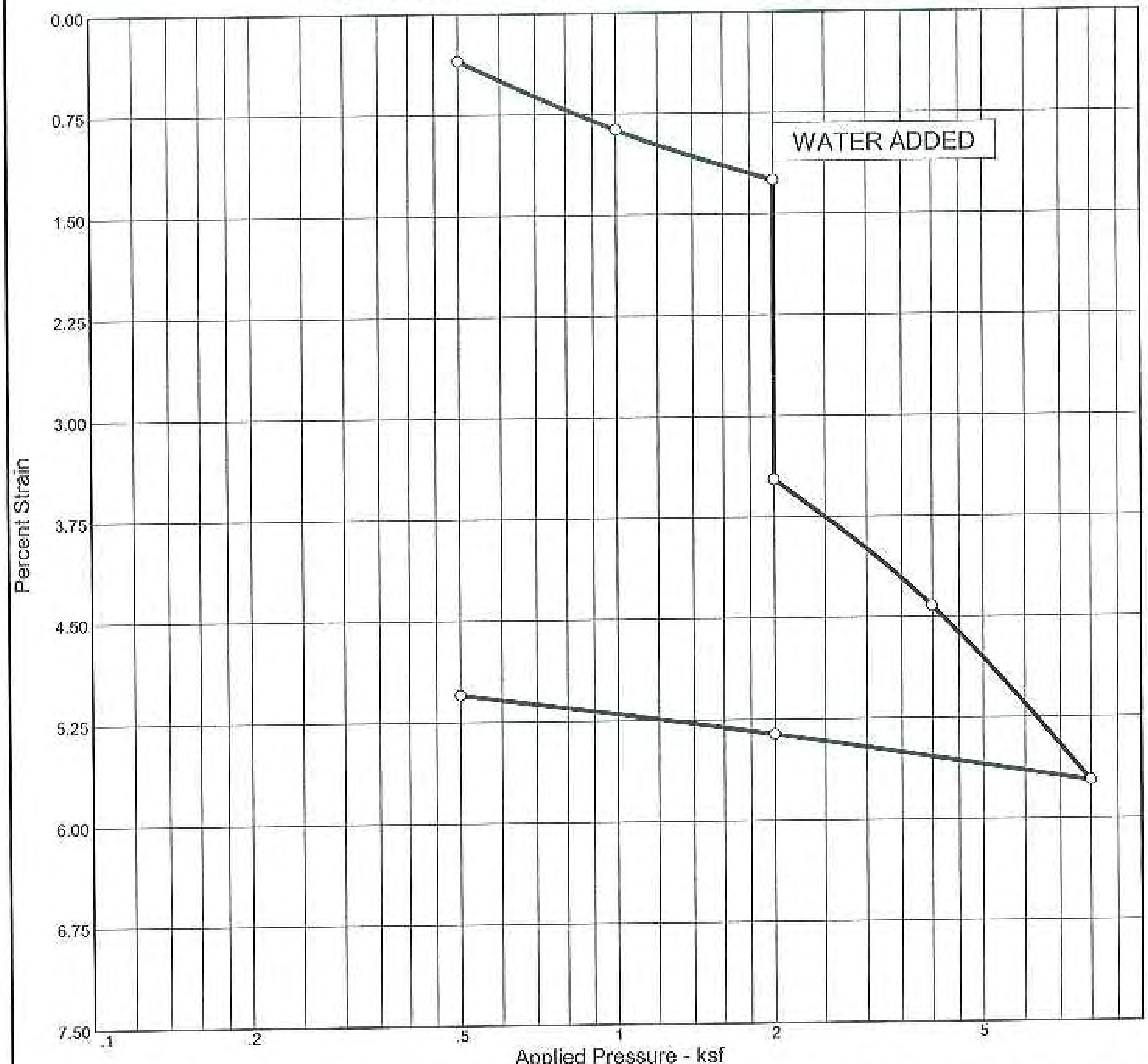
Remarks:

CONSOLIDATION TEST REPORT

THE TWINING LABORATORIES, INC.

Figure No.

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Clpse. %	e_o
Sat.	Moist.											
33.3 %	9.3 %	103.2			2.65		4.16	0.08	0.01		2.3	0.74

MATERIAL DESCRIPTION	USCS	AASHTO

Project No. F08306.02 **Client:** Kings River Conservation District

Project: Kings River Conservation District

Source: **Sample No.:** B-5 **Elev./Depth:** 3.5-5'

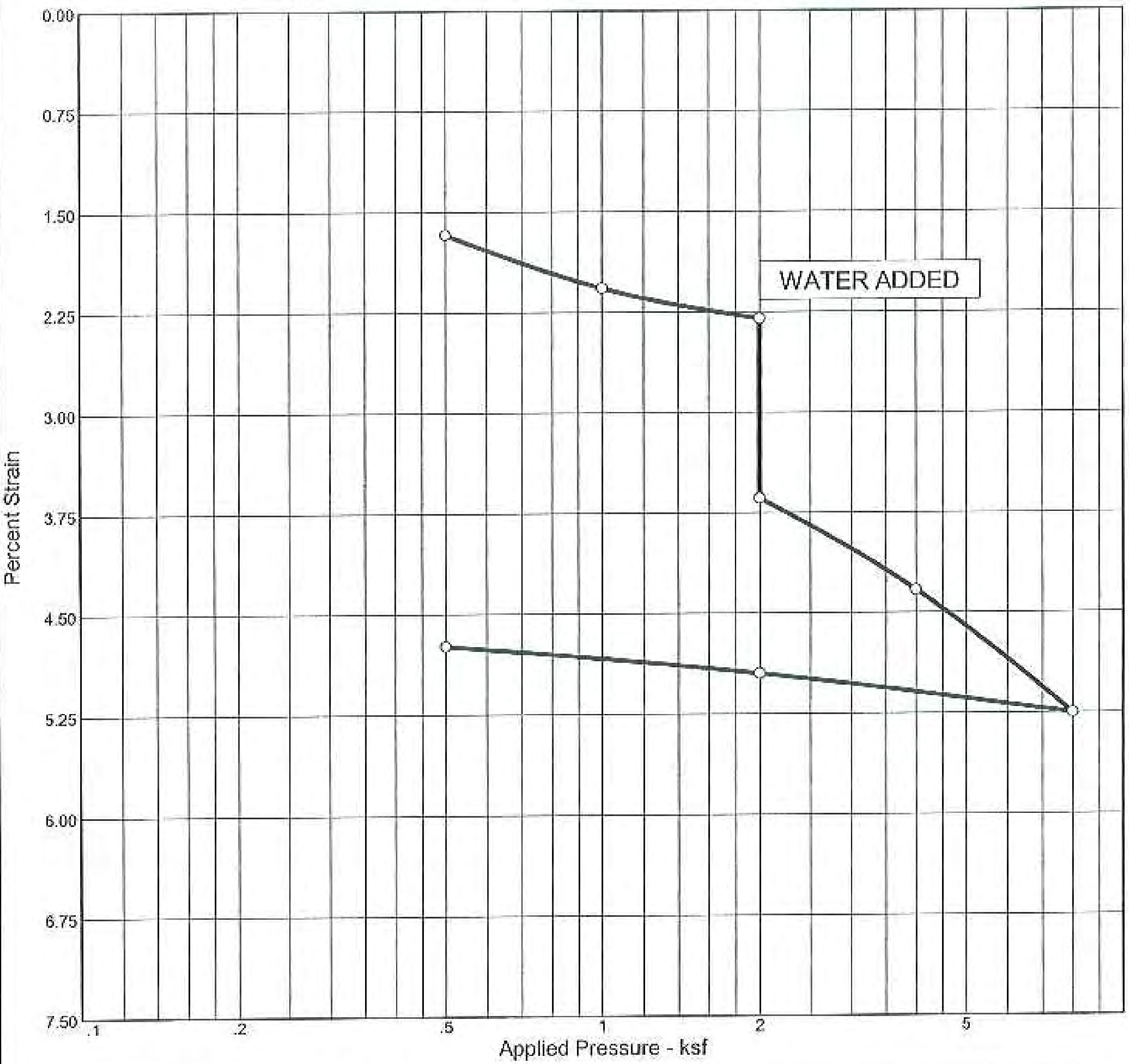
CONSOLIDATION TEST REPORT

THE TWINING LABORATORIES, INC.

Remarks:

Figure No.

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Clpse. %	e_o
Sat.	Moist.											
56.1 %	9.0 %	121.8			2.65		4.15	0.04	0.01		1.3	0.42

MATERIAL DESCRIPTION	USCS	AASHTO

Project No. F08306.02 **Client:** Kings River Conservation District

Project: Kings River Conservation District

Source: **Sample No.:** B-8 **Elev./Depth:** 10-11.5'

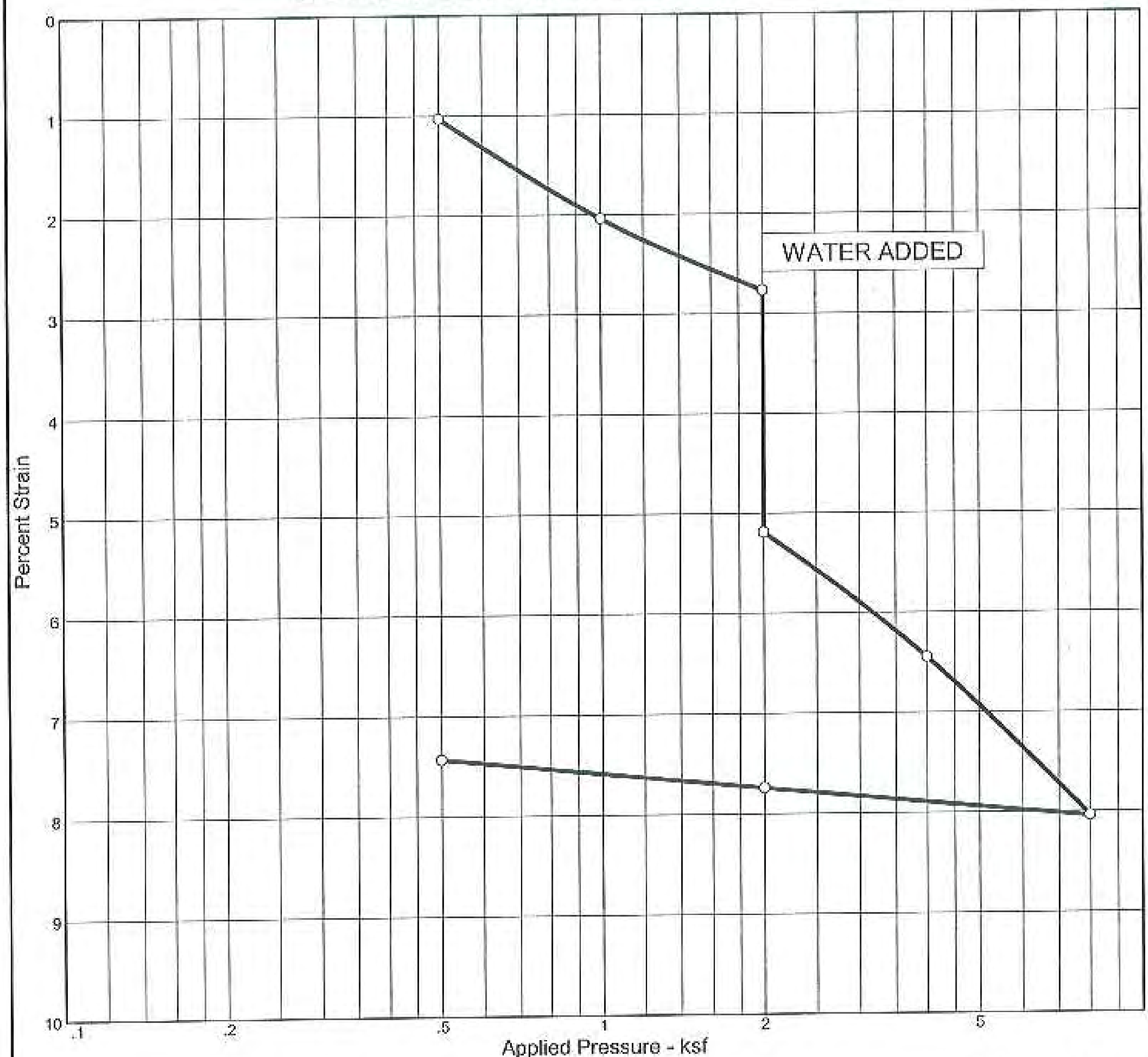
Remarks:

CONSOLIDATION TEST REPORT

THE TWINING LABORATORIES, INC.

Figure No.

CONSOLIDATION TEST REPORT



Natural		Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Clpse. %	e_o
Sat.	Moist.											
28.2 %	8.7 %	101.9			2.65		0.51	0.10	0.01		2.4	0.81

MATERIAL DESCRIPTION	USCS	AASHTO

Project No. F08306.02 **Client:** Kings River Conservation District

Project: Kings River Conservation District

Source: **Sample No.:** B-14 **Elev./Depth:** 3.5-4'

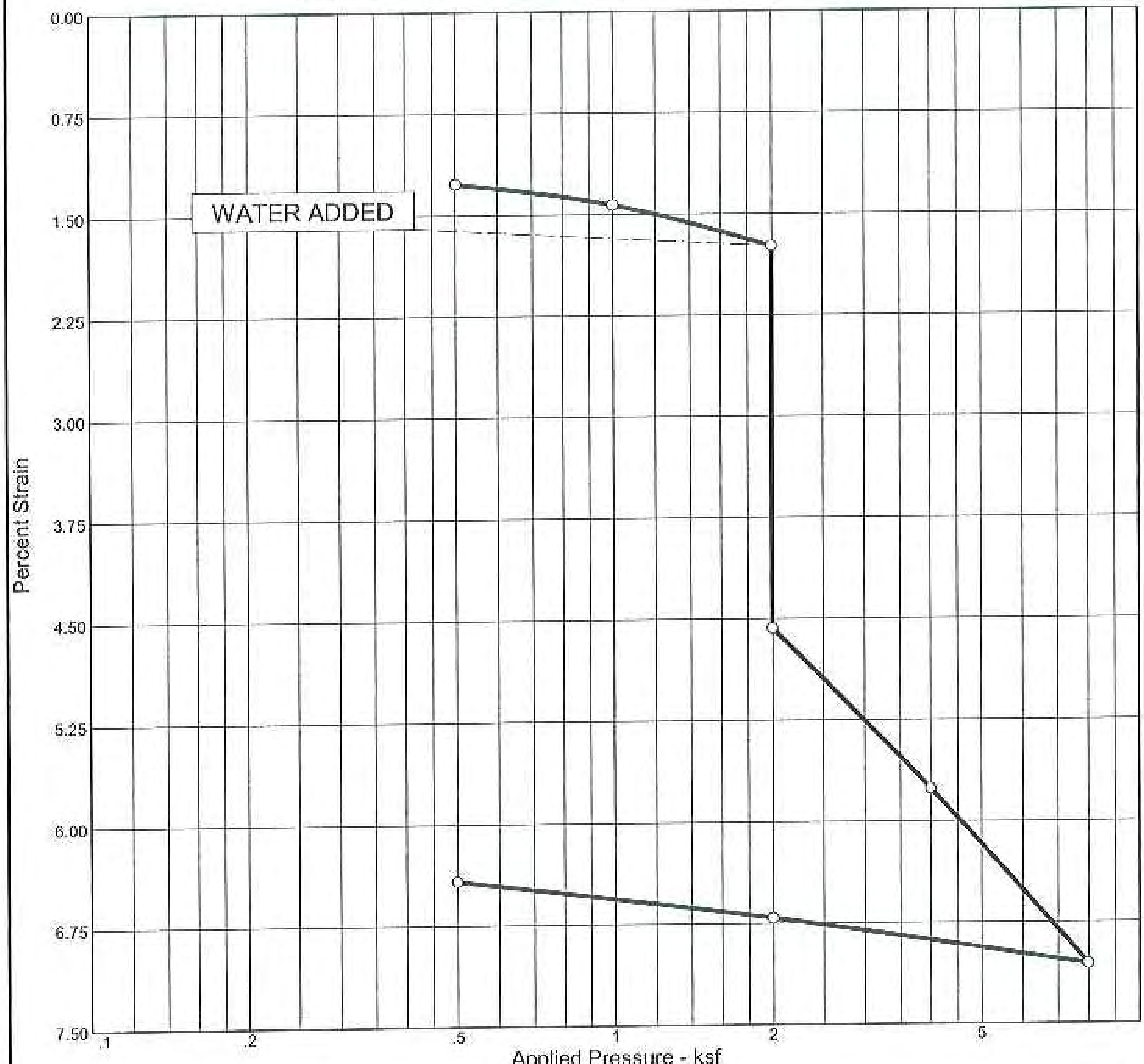
Remarks:

CONSOLIDATION TEST REPORT

THE TWINING LABORATORIES, INC.

Figure No.

CONSOLIDATION TEST REPORT



Natural Sat.	Natural Moist.	Dry Dens. (pcf)	LL	PI	Sp. Gr.	Overburden (ksf)	P_c (ksf)	C_c	C_s	Swell Press. (ksf)	Clpse. %	e_o
26.2 %	6.6 %	108.9			2.65		0.37	0.07	0.01		2.9	0.66

MATERIAL DESCRIPTION	USCS	AASHTO

Project No. F08306.02 **Client:** Kings River Conservation District

Project: Kings River Conservation District

Source: **Sample No.:** B-16 **Elev./Depth:** 5-6.5'

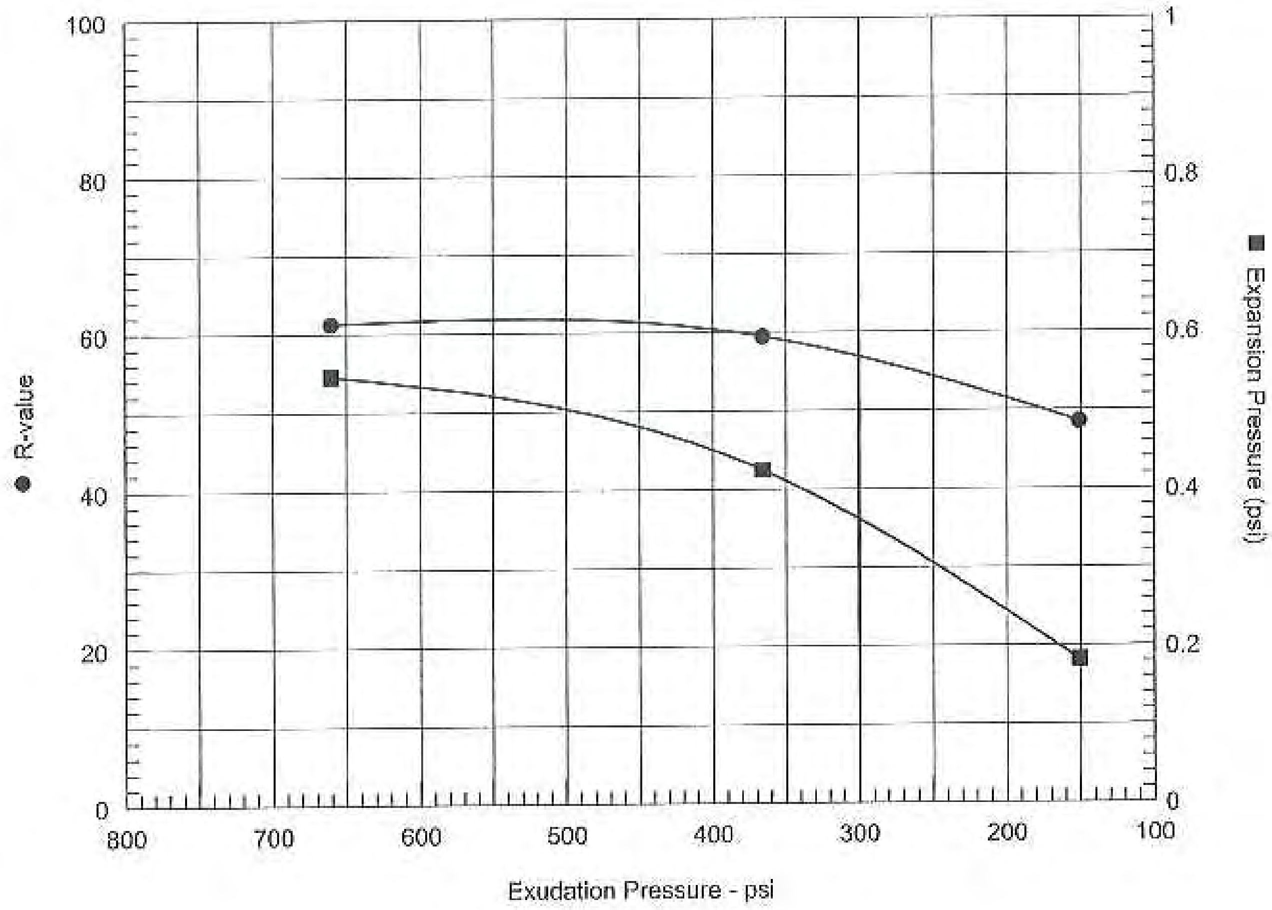
Remarks:

CONSOLIDATION TEST REPORT

THE TWINING LABORATORIES, INC.

Figure No.

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Valu Cor
1	250	118.0	10.7	0.18	56	2.66	151	44	48
2	250	118.1	8.1	0.55	34	2.48	660	61	61
3	250	119.6	9.4	0.42	39	2.62	366	56	59

Test Results

R-value at 300 psi exudation pressure = 57

Exp. pressure at 300 psi exudation pressure = 0.36 psi

Material Description

Project No.: F08306.02

Project: Kings River Conservation District

Sample Number: B-2 **Depth:** 0-3'

Date: 3/2/2007

Tested by: 981

Checked by: 871

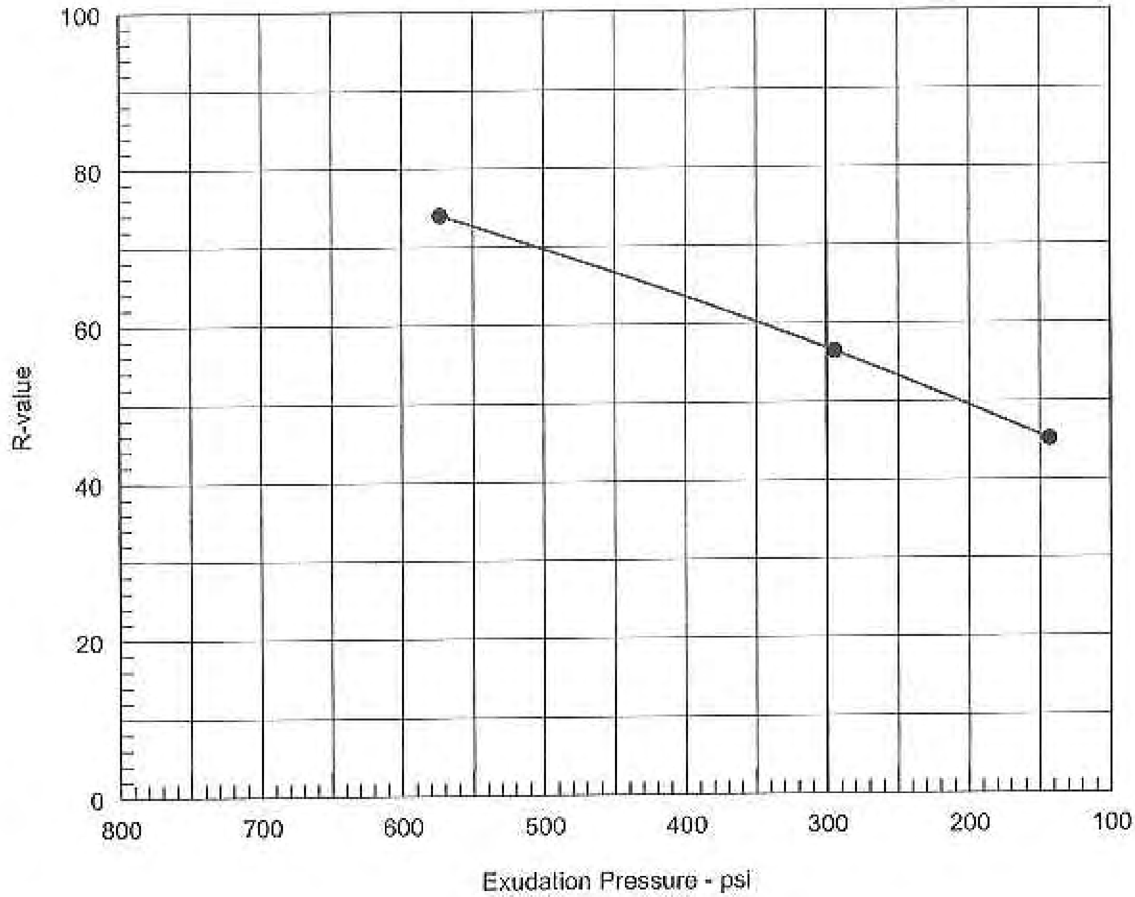
Remarks:

R-VALUE TEST REPORT

THE TWINING LABORATORIES, INC.

Figure No.

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Valu Cor
1	250	115.3	10.7	0.00	29	2.68	573	70	74
2	250	113.3	12.5	0.00	48	2.63	294	53	56
3	250	107.1	13.4	0.00	52	2.49	143	45	45

Test Results	Material Description
R-value at 300 psi exudation pressure = 57	

Project No.: F08306.02

Project: Kings River Conservation District

Sample Number: B-8 **Depth:** 0-3'

Date: 12/18/2006

Tested by: 981

Checked by: 871

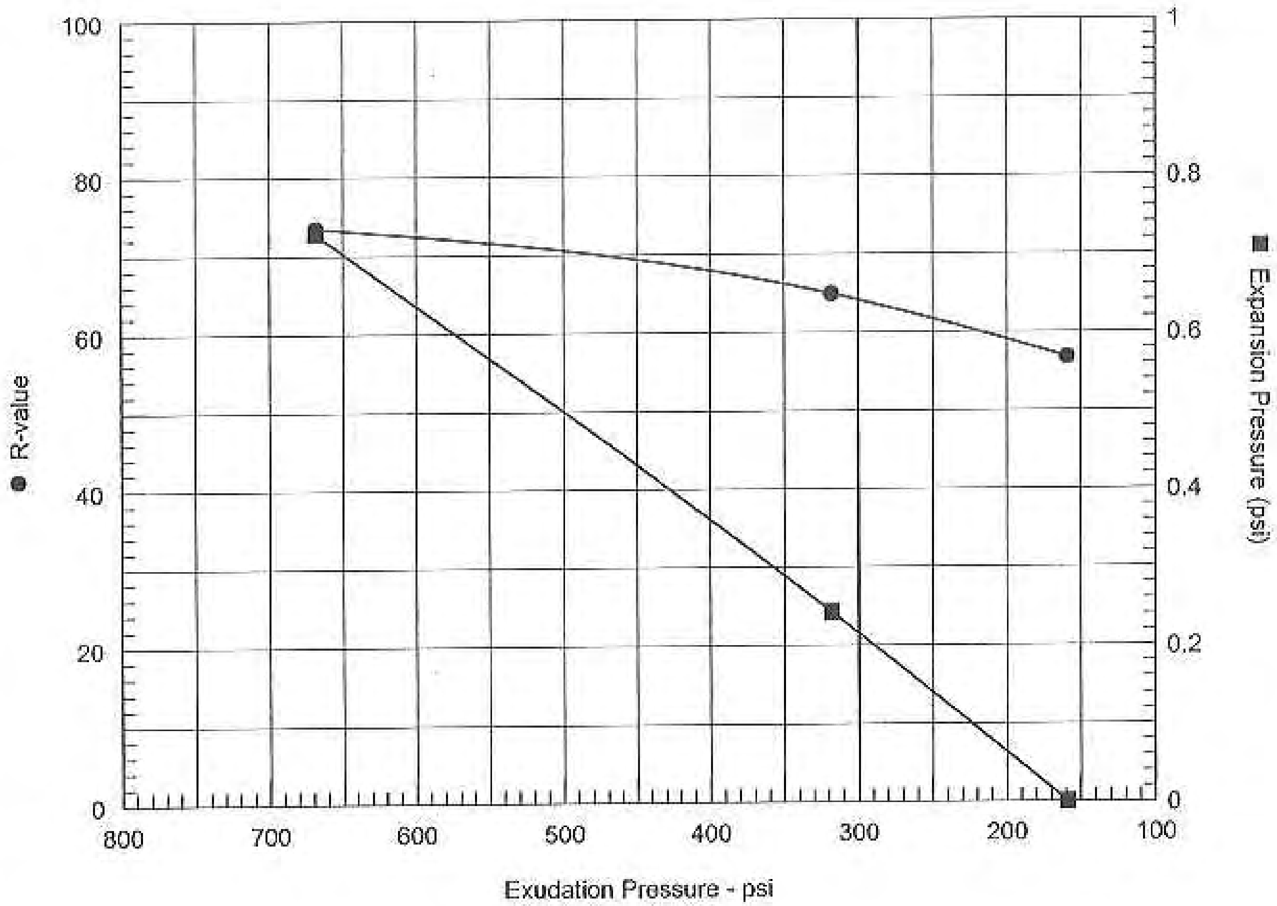
Remarks:

R-VALUE TEST REPORT

THE TWINING LABORATORIES, INC.

Figure No. ____

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Valu Cor
1	250	120.2	8.3	0.73	25	2.66	668	70	73
2	250	118.9	9.6	0.24	34	2.64	318	61	65
3	250	117.7	10.5	0.00	39	2.53	159	57	57

Test Results	Material Description
R-value at 300 psi exudation pressure = 64 Exp. pressure at 300 psi exudation pressure = 0.22 psi	

Project No.: F08306.02

Project: Kings River Conservation District

Sample Number: B-15 **Depth:** 0-3'

Date: 12/18/2006

Tested by: 981

Checked by: 871

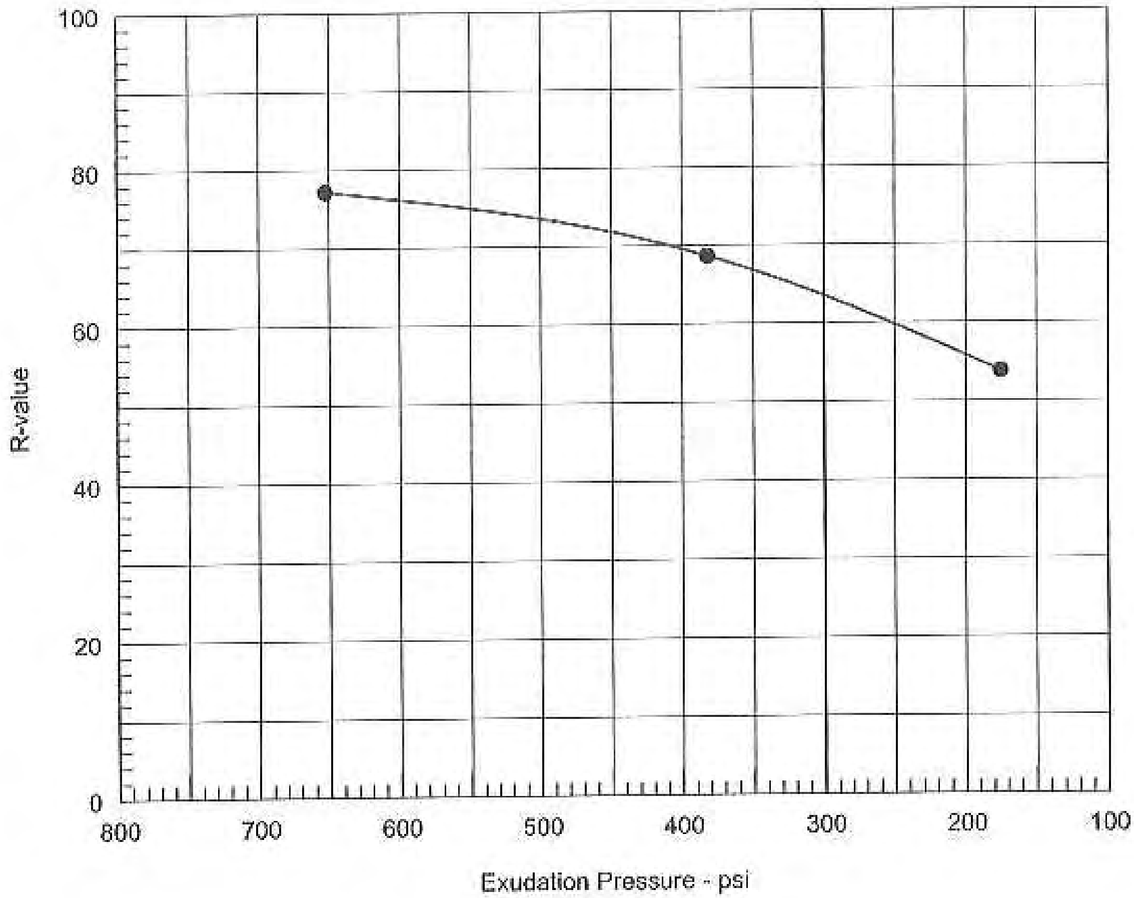
Remarks:

R-VALUE TEST REPORT

THE TWINING LABORATORIES, INC.

Figure No. ____

R-VALUE TEST REPORT



Resistance R-Value and Expansion Pressure - Cal Test 301

No.	Compact. Pressure psi	Density pcf	Moist. %	Expansion Pressure psi	Horizontal Press. psi @ 160 psi	Sample Height in.	Exud. Pressure psi	R Value	R Valu Cor
1	250	113.3	7.5	0.00	22	2.64	653	75	77
2	250	110.2	8.8	0.00	32	2.66	382	65	69
3	250	109.6	9.6	0.00	48	2.62	175	51	54

Test Results	Material Description
R-value at 300 psi exudation pressure = 63	

Project No.: F08306.02

Project: Kings River Conservation District

Sample Number: B-17 **Depth:** 0-3'

Date: 12/18/2006

Tested by: 981

Checked by: 871

Remarks:

R-VALUE TEST REPORT

THE TWINING LABORATORIES, INC.

Figure No. ______

EXPANSION INDEX TEST

Uniform Building Code (UBC) 18-2

Project Number: F08306.02

Project: KRCD Power Plant

Sample Location: B-8

Depth: 0-3'

Date Sampled: 10/31/06

Sampled by: H.E.

Sample Number	Molding Moisture Content	Final Moisture Content	Dry Density (γ d)
B-8	8.7	15.6	114.0

Initial Thickness: 1.0000

Final Thickness: 1.0000

Expansion Index (EI): **0**

Expansion Soil Classification: **Very Low**

TABLE NUMBER 18-2
EXPANSIVE SOIL CLASSIFICATION

Expansion Index	Potential Expansion
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
Above 130	Very High

Figure No.

PROCTOR TEST REPORT

Test specification: ASTM D 1557-00 Method A Modified

Elev/ Depth	Classification		Nat. Moist.	Sp.G.	LL	PI	% > No.4	% < No.200
	USCS	AASHTO						
0-3'								

TEST RESULTS	MATERIAL DESCRIPTION
Maximum dry density = 119.8 pcf Optimum moisture = 9.0 %	

Project No. F08306.02 **Client:** Kings River Conservation District

Project: Kings River Conservation District

Date:

● **Source:** **Sample No.:** B-8 **Elev./Depth:** 0-3'

Remarks:

PROCTOR TEST REPORT

THE TWINING LABORATORIES, INC.

Figure No.

THE TWINING LABORATORIES, INC.
EST. 1898
CALIFORNIA ELAP CERTIFICATE #1371

2527 Fresno Street
Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

November 16, 2006

Work Order #: 6K07047

Zubair Anwar
Twining Geotechnical Department
2527 Fresno Street
Fresno, CA 93721

RE: KRCD

Enclosed are the analytical results for samples received by our laboratory on 11/07/06 . For your reference, these analyses have been assigned laboratory work order number 6K07047.

All analyses have been performed according to our laboratory's quality assurance program. All results are intended to be considered in their entirety, The Twining Laboratories, Inc. (TL) is not responsible for use of less than complete reports. Results apply only to samples analyzed.

If you have any questions, please feel free to contact us at the number listed above.

Sincerely,

The Twining Laboratories, Inc.

Ronald J. Boquist
Director of Analytical Chemistry

THE TWINING LABORATORIES, INC.

EST. 1898

CALIFORNIA ELAP CERTIFICATE #1371

2527 Fresno Street
Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

Twining Geotechnical Department 2527 Fresno Street Fresno CA, 93721	Project: KRCD Project Number: F08306.02 Project Manager: Zubair Anwar	**Reported:** 11/16/2006

B-8 @ 0-3
6K07047-01 (Soil)

Analyte	Result	Reporting Limit	Units	Batch	Prepared	Analyzed	Method	Qualifier
Inorganics								
Chloride	ND	6.0	mg/kg	T6K1403	11/14/06	11/14/06	ASTM D-4327-84	
Chloride	ND	0.00060	% by Weight	[CALC]	11/14/06	11/14/06	ASTM D4327-84	
Sulfate as SO4	**0.0016**	0.00060	% by Weight	[CALC]	11/14/06	11/14/06	ASTM D4327-84	
pH	**6.2**	0.30	pH Units	T6K1403	11/14/06	11/14/06	ATSM D4972-89 Mod	
Resistivity	**11000**		ohms/cm	T6K1403	11/14/06	11/14/06	ASTM D1125-82	
Sulfate as SO4	**16**	6.0	mg/kg	T6K1403	11/14/06	11/14/06	ASTM D4327-84	

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Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

Twining Geotechnical Department 2527 Fresno Street Fresno CA, 93721	Project: KRCD Project Number: F08306.02 Project Manager: Zubair Anwar	**Reported:** 11/16/2006

B-11 @ 0-3
6K07047-02 (Soil)

Analyte	Result	Reporting Limit	Units	Batch	Prepared	Analyzed	Method	Qualifier
Inorganics								
Chloride	ND	6.0	mg/kg	T6K1403	11/14/06	11/14/06	ASTM D-4327-84	
Chloride	ND	0.00060	% by Weight	[CALC]	11/14/06	11/14/06	ASTM D4327-84	
Sulfate as SO4	**0.00077**	0.00060	% by Weight	[CALC]	11/14/06	11/14/06	ASTM D4327-84	
pH	**6.2**	0.30	pH Units	T6K1403	11/14/06	11/14/06	ATSM D4972-89 Mod	
Resistivity	**16000**		ohms/cm	T6K1403	11/14/06	11/14/06	ASTM D1125-82	
Sulfate as SO4	**7.7**	6.0	mg/kg	T6K1403	11/14/06	11/14/06	ASTM D4327-84	

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2527 Fresno Street
Fresno, CA 93721
(559) 268-7021 Phone
(559) 268-0740 Fax

Twining Geotechnical Department 2527 Fresno Street Fresno CA, 93721	Project: KRCD Project Number: F08306.02 Project Manager: Zubair Anwar	**Reported:** 11/16/2006

B-17 @ 0-3
6K07047-03 (Soil)

Analyte	Result	Reporting Limit	Units	Batch	Prepared	Analyzed	Method	Qualifier
Inorganics								
Chloride	ND	6.0	mg/kg	T6K1403	11/14/06	11/14/06	ASTM D-4327-84	
Chloride	ND	0.00060	% by Weight	[CALC]	11/14/06	11/14/06	ASTM D4327-84	
Sulfate as SO4	**0.00066**	0.00060	% by Weight	[CALC]	11/14/06	11/14/06	ASTM D4327-84	
pH	**6.6**	0.30	pH Units	T6K1403	11/14/06	11/14/06	ATSM D4972-89 Mod	
Resistivity	**37000**		ohms/cm	T6K1403	11/14/06	11/14/06	ASTM D1125-82	
Sulfate as SO4	**6.6**	6.0	mg/kg	T6K1403	11/14/06	11/14/06	ASTM D4327-84	

Notes and Definitions

ND	Analyte NOT DETECTED at or above the reporting limit
NR	Not Reported
RPD	Relative Percent Difference

Quality Control Data Available Upon Request