Pico Power Project

Appendix 10-G
Geotechnical Report

October 2002

GEOTECHNICAL INVESTIGATION
PROPOSED PICO POWER PLANT
SANTA CLARA, CALIFORNIA

1 INTRODUCTION

This report presents the results of our geotechnical investigation for the proposed Pico Power Plant in Santa Clara, California. The project site is located west of Lafayette Street and south of Duane Avenue. The approximate location of the site is shown on the Site Vicinity Map, Plate 1. A layout of the proposed improvements is shown on the Site Plan, Plate 2. This investigation has been performed for PB Power, Inc. and the City of Santa Clara Silicon Valley Power.

This report presents our conclusions and geotechnical recommendations for project design and construction. These conclusions and recommendations are based on the subsurface conditions encountered at the locations of our exploration and the provisions and requirements outlined in the Additional Services and Limitations section of this report. The conclusions and recommendations presented herein should not be extrapolated to other areas or used for other projects without our review.

A geotechnical investigation report prepared by Terratech, Inc., titled, "Geotecncial Investigation, Lafayette Street Substation, Santa Clara, California," (Project Number 3953) dated July 1986 was provided to us. We understand this report was for the existing substation south of the Pico Power Plant project site.

1.1 PROJECT DESCRIPTION

The proposed structures and their estimated foundation loads provided by PB Power are tabulated below.

Proposed Structures	Estimated Equipment Loads
Combustion turbine generator (CTG)	Dead load = 1300 kips, Estimated foundation pressure due to seismic = 3.2 ksf
Heat recovery steam generator (HRSG)	Dead load = 3320 kips, Estimated foundation pressure due to seismic = 3.2 ksf
Steam turbine generator (STG)	Dead load = 2130 kips, Estimated foundation pressure due to seismic = 4.0 ksf
Cooling towers	Dead load = 2100 kips, Estimated foundation pressure due to seismic = 1 ksf
Water tanks	Dead load = 5700 kips, Estimated foundation pressure due to seismic = 2 ksf
Pipe rack	Unit load = 1000 pounds per foot; for a typical 20 foot span, total load per span = 20 kips

Other proposed structure includes the plant operations building, switchyard relay house, water sample and analysis lab, various pumps and a paved parking area. The plant operations building, which will be constructed in the eastern portion of the site, will be a pre-fabricated building with a concrete slab-on-grade floor. The other buildings are anticipated to be relatively lightly-loaded pre-fabricated structures with concrete slab-on-grade floors.

1.2 Scope of Services

The objective of this geotechnical investigation, as presented in our proposal dated June 18, 2002, document number SJ02P128, was to explore and evaluate the subsurface soil conditions at the site. Based on the results of our investigation, this report provides recommendations for the geotechnical aspects of the design and construction of foundations, concrete slabs-on-grade, flexible and rigid pavements, retaining structures, site grading and underground utility trench backfill. The scope of our services included a site reconnaissance, subsurface exploration, field resistivity testing, laboratory testing of selected soil samples, engineering analysis, preparation of a draft report, and preparation of this report. In addition to the geotechnical laboratory soil testing, four selected soil samples were sent to CERCO Analytical for preliminary corrosivity testing.

Environmental services such as evaluation and chemical analysis of the soil and groundwater for hazardous materials were not included in our scope of services.

2 SITE INVESTIGATION

2.1 SITE DESCRIPTION

The irregular-shaped site is located west of Lafayette Street and south of Duane Avenue in the city of Santa Clara. The project area is north of the existing Lafayette Substation facility and is undeveloped. The site is occupied by several overhead transmission towers and poles, stacks of cables and other electrical equipment, small stockpile of sand, and wood power poles lying on the ground. In the eastern portion of the site is the abandoned Pico Way oriented in a north-south direction. There are also isolated trees and bushes on the site. A depressed area, which appears to have been used as a wash area, is in the southeastern portion of the project area.

2.2 FIELD INVESTIGATION

Our field investigation consisted of surface reconnaissance and subsurface exploration program. On June 27 and July 1, 2002, five exploratory borings (Borings B-1, B-2, B-6, B-7 and B-8), were drilled to a depth of about 40 feet below the existing ground surface. The borings were drilled using a truck-mounted drill rig equipped with 6-inch diameter hollow-stem augers. On June 27 and 28, 2002, five Cone Penetrometer Test (CPT) holes (CPTs 3, 4, 5, 9 and 10) were advanced to depths between 31 and 77 feet below the existing ground surface. In CPTs 4, 5 and 10, the holes encountered refusal to advancement and were terminated above the planned depths.

The borings and the CPT holes were located in the field by our representative based on rough measurement from existing features. As such, the locations of the borings and the CPT holes are approximate and should be considered accurate only to the degree implied by the methods used. These approximate locations are shown on the Site Plan, Plate 2.

Prior to the start of our fieldwork, Underground Services Alert (USA) was notified of our exploration work and we met with representatives of PB Power and Silicon Power Company. Upon completion of the exploration, the borings and the CPT holes were backfilled with cement grout as required by the Santa Clara Valley Water District. The soil cuttings were placed in 55-gallon drums and the drums were left on the site.

The soils encountered in our exploratory borings were visually classified in the field in general accordance with the Unified Soil Classification System (ASTM D2488) by our engineering staff. The results of our laboratory tests were used to refine the field classifications based on ASTM D2487. A key for classification of the soils is presented on the Boring Log Legend, Plate A-1. The logs of the borings are presented on Plates A-2 through A-6.

Soil samples were obtained from the borings at selected depths by driving a 2-inch inside diameter Modified California sampler or a 1-3/8 inch inside diameter split-spoon SPT sampler. The Modified California and SPT samplers were driven up to a depth of 18 inches into the underlying soil using a 140-pound hammer falling 30 inches. The number of blows required to drive the samplers was recorded for each 6-inch penetration interval. The number of blows required to drive the samplers the last 12 inches, or the penetration interval indicated if higher resistance was encountered, is noted on the boring logs. Samples collected from the borings were returned to our laboratory for further evaluation and testing.

The cone penetrometer tests were performed by John Sarmiento & Associates. The tip resistance, side friction, and pore pressure measured by the cone as it was pushed through the soil strata were recorded electronically every 0.05 meters (approximately 2 inches). The CPT data include the following with respect to depth:

Qc	Tip Resistance,
Fs	Local Friction,
Rf	Friction Ratio
SPT(N)	Equivalent Standard Penetration N-value
SPT (N')	Corrected Equivalent Standard Penetration N-value
TotVtStr	Total Overburden Stress
PHI	Internal friction angle for granular soils
Su	Undrained Shear Strength for cohesive soils
Soil Behavior type	
Density Range	

The CPT data and graphic presentations of some of the CPT data are included in Appendix A after the boring logs.

2.3 FIELD RESISTIVITY TESTING

Field resistivity testing was performed by JDH Corrosion Consultants, Inc. at the ten boring and CPT locations (See Plate 2). The field tests were performed on June 27, 2002. A report prepared by JDH Corrosion Consultants summarizing their findings is included in Appendix C.

According to Darby Howard of JDH Corrosion Consultants, Inc., their testing complies with ANSI/IEEE Standard 81.

2.4 LABORATORY TESTING

Laboratory testing was performed on selected soil samples collected from the borings to evaluate their natural moisture content, in-place density, grain size distribution, unconfined compressive strength, and plasticity (Atterberg limits). An R-value test was performed on a bulk sample of near-surface soil collected from near Borings B-6 and B-8. Most of the laboratory test results are presented on the boring logs. Graphic presentations of the results of the Atterberg Limits, sieve analysis, unconfined compressive strength, and R-value tests are presented in Appendix B.

Four selected soil samples were submitted to CERCO Analytical for corrosivity testing. A report prepared by CERCO Analytical summarizing the results of their tests and a brief evaluation of the results are included in Appendix D.

2.5 SUBSURFACE CONDITIONS

Below the ground surface, our borings encountered a layer of clay/fat clay extending to depths of 8 to 10.5 feet below ground surface. This clay is very stiff to hard in consistency and has intermediate to high plasticity and high expansion potential. Below this surface clay layer, the subsurface soils generally consist of layers of clay and sandy clay of intermediate plasticity, with interbedded layers of sand, silty sand, clayey sand and poorly graded sand. The deeper clay layers are generally stiff to very stiff in consistency, and contain variable amounts of sand. The sand layers are generally medium dense to locally very dense in relative density, and contain variable amounts of fines and gravel.

The soil behavior types interpreted from the CPT soundings compare well with the soil types encountered in our borings.

Groundwater was encountered in four borings (not in B-2) at the time of drilling, between depths of 8 and 18 feet below the existing ground surface. Groundwater was also encountered in all CPT holes, with estimated groundwater depth ranging between approximately 9 and 14 feet below the ground surface. Because groundwater was encountered in all other exploratory holes and in all CPT holes, it is reasonable to conclude that groundwater would have been encountered in B-2 if the hole was left open long enough.

Groundwater was reported in the 1986 Terratech report at depths ranging from 11½ to 13½ feet below ground surface. The groundwater levels were measured in geotechnical borings which were advanced for the existing Lafayette Substation south of the subject site.

It should be noted that fluctuations in groundwater level could occur due to variations in rainfall, temperature, pumping from wells, and other factors that were not evident at the time of our investigation. If significant variations in the groundwater level are encountered during construction, it may be necessary for Kleinfelder to review the recommendations and recommend adjustments as necessary.

The above is a general description of the subsurface soil conditions encountered in the borings and inferred from the CPT soundings advanced for this investigation. For a more detailed description of the soil conditions encountered, refer to the boring and CPT logs presented in Appendix A.

3 GEOLOGIC SETTING AND SEISMIC CONSIDERATIONS

3.1 Geologic Setting

Over about the last 40 years, the geology of Santa Clara County has been extensively studied and mapped by the United States Geological Survey (USGS), California Division of Mines and Geology (CDMG), and other investigators. Numerous published geologic maps, including *Brabb and Pampeyan (1972, 1983), Brabb et al. (1998), Hall (1965), Wentworth, et al. (1975), Bonilla (1964, 1965, 1971, 1989), Lajoie et al. (1974), and Helley et al. (1994)* as well as numerous consultants' reports are available. The geologic summary of the site and vicinity that is presented in this report is based on review of pertinent maps.

The San Francisco Bay Area lies within the Coast Ranges Geomorphic Province, a discontinuous series of northwest-southeast trending mountain ranges, ridges, and intervening valleys characterized by complex folding and faulting. The general geologic framework of the Central Coast Area of California is illustrated in studies by Page (1966), as well as in studies by Schlocker, 1971, Wagner, and others, (1991), and other investigators.

Geologic structures within the Coast Ranges Province are generally controlled by a major tectonic transform plate boundary. This right-lateral strike-slip fault system extends from the Gulf of California, in Mexico, to Cape Mendocino, off the coast of Humboldt County in northern California and forms a portion of the boundary between two tectonic plates. In this portion of the Coast Ranges Province, the Pacific plate moves north relative to the North American plate, which is located east of the transform boundary. Deformation across this plate boundary is distributed across a wide fault zone, which includes the San Andreas, Hayward, Calaveras, and San Gregorio faults. Together, these and other faults are referred to as the San Andreas Fault System. The general trend (about N 30° W) of the faults within this system is responsible for the strong northwest-southeast structural grain of most geologic and geomorphic features in the Coast Ranges Province.

The project site is located on the broad alluvial-covered plain lying between the Santa Cruz Mountains, forming the backbone of the San Francisco peninsula to the northwest, and the Diablo Range to the east. The inland valleys as well as the structural depression, within which San Francisco Bay is located, are filled with unconsolidated to semi-consolidated deposits of Quaternary Age (less than 1.6 million years to present). Continental deposits (alluvium) consist of unconsolidated to semi-consolidated sand, silt, clay, and gravel, and the bay deposits typically consist of very soft organic rich silt and clay or sand.

The project site is located in the alluvial basin situated near the center of the San Francisco bay plain southeast of the San Francisco Bay. Local studies by the United States Geological Survey (USGS) that describe the Quaternary alluvium in the vicinity of the project area include Helley et al. (1972), Helley et al. (1979), Helley and Graymer (1997), and Helley et al. (1994). The alluvial fill includes the semi-consolidated San Jose Formation of Pliocene and Pleistocene age (5.3 million years to 11,000 years ago) and the overlying unconsolidated alluvial and bay deposits of Pleistocene and Holocene age (less than 1.6 million years old). This fill is as much as 1,500 feet thick (Poland, 1971). Rogers and Williams (1974) show the alluvium in the general site area is underlain by bedrock at depths estimated to be in excess of 450 feet.

As described in Helley, et al. (1994), the area is underlain by Holocene age Basin Deposits (Qhb). These deposits typically consist of unconsolidated, very fine silty clay to clay deposits occupying on flat-floored basins at the distal edge of alluvial fans adjacent to the bay mud. CDMG (2001) shows the site to be located in an area of potential liquefaction.

3.2 FAULTING AND SEISMICITY

The San Francisco Bay Area is considered to be one of the most seismically active regions in the United States. The area is seismically dominated by the San Andreas Fault System, which includes, among others, the Hayward, Calaveras and San Andreas faults. The site is located approximately 14.2 km southwest of the Hayward fault, 10 km southwest of the Hayward fault (southeast extension), 14.7 km southwest of the Calaveras fault (south), 12.6 km northeast of the Monte Vista-Shannon fault, and 18.6 km northeast of the San Andreas fault. These distances are map distances to the surface projections of the respective faults. The site is not located within any of the State of California Earthquake Fault Zones.

3.3 SITE CHARACTERIZATION

The project site is located in Seismic Zone 4. The soil conditions encountered in our exploratory borings may be characterized as a stiff soil profile S_D according to Table 16-J in the 1997 Uniform Building Code. S_D is defined as a soil profile consisting of soft soil with shear wave velocity of between 600 and 1,200 m/s or SPT-N between 15 and 50, or undrained shear strength (Su) between 1,000 and 2,000 psf for the upper 100 feet or 30 meters.

3.4 NEAR-FAULT ISSUES IN STRUCTURAL DESIGN

In recent years, many modern structures located near the seismic source have collapsed or been severely damaged. The severe damage and/or collapse are attributed to near-fault motions that are characterized by energetic unidirectional velocity pulses (Singh 1984, 1985). What makes these motions particularly damaging is the duration of the impulse (area under the acceleration curve multiplied by the mass). A structural system that yields during a long duration pulse (impulse loading) may experience very large permanent deformations and/or collapse. The extent of these actions depend on the strength and natural period of the structure, and on the ability of the structure to articulate, as well as the amplitude, duration, and shape of the pulse. The near-fault pulse type motion can be particularly damaging because they can result in the accumulation of inelastic deformations in one direction.

Because the proposed structures will be designed based on the 1997 Uniform Building Code (UBC), we have included information addressing near-fault effects for use by the project structural engineer. Structures with strength discontinuities, soft stories, plan irregularities, discontinuous shear walls and ductile moment frames are particularly vulnerable to this type of motion, and should either be avoided or properly evaluated.

For a code equivalent lateral force design based on procedures in the 1997 UBC, the near-source factors N_a and N_v are incorporated into the seismic coefficients C_a and C_v. Both of these factors are used to determine the design lateral force or shear at the base of the structures. The values of these factors depend on the distance of the site from the fault and the fault type. Type A faults located within 15 km and Type B faults located within 10 km of the site are to be considered for near-source factors. For this site, the Hayward fault is the closest Type A fault at 14.2 km and Hayward fault (southeast extension) is the closest Type B fault at 10 km from the site. Values for the Near-Source Factors N_a and N_v obtained from Tables 16-S and 16-T of the 1997 UBC, are therefore both 1.0. Alternatively, consideration may be given to dynamic analyses utilizing site-specific response spectra that better account for the type of near-source effects observed in the recent Northridge, California and Kobe, Japan earthquakes.

4 CONCLUSIONS AND DISCUSSION

Based on the results of our engineering analysis and the information provided to us, it is our opinion the site may be developed as discussed in this report. This is provided the recommendations presented in this report are incorporated into the geotechnical aspects of the design and construction of the project. Our opinions, conclusions and recommendations are based on our field and office studies, the properties of soils encountered in our borings, the results of the laboratory testing program, and our understanding of the proposed development.

4.1 GROUND RUPTURE AND SEISMIC SHAKING

Because no known faults have been mapped across the site, ground rupture should not be a concern at the site. However, based on our knowledge of the seismicity of the region and on historical information, the site will be subject to seismic shaking from at least one moderate to severe earthquake. Periodic slight to moderate earthquakes will also occur during the design life of the proposed project. Some degree of structural damage due to strong seismic shaking at the site should be expected, but the risk can be reduced through adherence to seismic design codes.

4.2 LIQUEFACTION POTENTIAL AND DYNAMIC COMPACTION

Soil liquefaction is a phenomenon in which saturated, cohesionless or granular soils undergo a substantial loss in strength due to excess build-up of pore water pressure during cyclic loading such as that induced by earthquakes. The primary factors affecting the liquefaction potential of soil include: (1) intensity and duration of seismic shaking; (2) soil type and relative density; (3) overburden pressure; and (4) depth to groundwater. Soils most susceptible to liquefaction are generally clean, loose, fine-grained sands that are saturated and uniformly graded. Silty sands have also been known to be susceptible to liquefaction.

The subsurface soils encountered in our borings and CPT holes generally consist of cohesive soil with localized layers of sand, silty sand and clayey sand. The sand layers are medium dense to very dense in relative density. Based on our analysis, the liquefaction potential for the sands is low.

Dynamic compaction (or seismically – induced settlement) is the densification of unsaturated, loose sands due to strong seismic shaking resulting from earthquakes. The soils above groundwater are generally clay soils. Because no loose sands were encountered above groundwater, the potential for dynamic compaction is low.

4.3 EXPANSIVE SOIL

Based on the results of our field investigation, the surficial layer of clay soil across the site can be characterized as having a high expansion potential. Expansive soils have the ability to undergo significant volume change (shrink or swell) due to variations in moisture content. Changes in soil moisture content can result from rainfall, landscape irrigation, perched groundwater, drought or other factors. Changes in soil moisture may result in unacceptable settlement or heave of structures, concrete slabs or pavements supported on the expansive soil. Depending on the extent and location below finished subgrade, the expansive soil could have a detrimental effect on the proposed construction.

For this project, we have recommended the use "non-expansive" fill under building concrete slabs-on-grade and exterior slabs where differential movements of the slabs is not desired. In addition to the use of "non-expansion" fill, maintaining surface drainage away from the slabs and providing a relatively uniform soil moisture content year-round through controlled irrigation will aid in mitigating the adverse effects of expansive soils; but will not eliminate them completely. Some differential ground movement due to the expansive soils is unavoidable and maintenance of such areas would be necessary. Refer to the following sections of this report for recommendations.

4.4 HIGH GROUNDWATER CONDITIONS

During our field investigation in June 2002, groundwater was encountered as shallow as approximately 9 feet below ground surface (in CPT-3). It is possible that the groundwater table would be higher during the rainy seasons. High groundwater table should be considered in the design and construction of the project, especially for underground utilities and foundations such as drilled piers. Where excavations extend into groundwater, dewatering will be required.

Special considerations such as casing of pier holes and "tremie" placement of concrete should be utilized for drilled piers.

We understand none of the proposed structures will extend below ground surface.

5 GEOTECHNICAL RECOMMENDATIONS

5.1 EARTHWORK

5.1.1 Site Clearing and Stripping

Prior to grading, the construction areas should be cleared of all obstructions and deleterious materials, including designated structures, foundations, abandoned or designated utility lines, and other below grade obstacles encountered during the clearing operation. Depressions, excavations and holes that extend below the proposed finish grades should be cleaned and backfilled with engineered fill compacted to the requirements given under Item 5.1.5 "Fill Placement and Compaction."

After clearing, any areas containing surface vegetation should be stripped to sufficient depth to remove all vegetation and organic laden topsoil. Stripped material may be stockpiled for use in landscape areas if approved by the project landscape architect, or otherwise removed from the site. The required stripping depth should be determined in the field by the Geotechnical Engineer at the time of construction. But for planning purposes, an average stripping depth of 3 inches may be assumed in vegetated areas.

5.1.2 Subgrade Preparation

After site clearing and stripping, and after excavation to achieve design grades in cut areas, the exposed soil surface in areas to receive engineered fills, mat foundations, water tanks, concrete slabs-on-grade and pavements should be scarified to a depth of 12 inches. The scarified subgrade should be moisture conditioned and compacted in accordance with the recommendations given in Section 5.1.5, "Fill Placement and Compaction." In building areas to receive concrete slabs-on-grade, subgrade preparation should extend at least 5 feet beyond the limits of the proposed buildings and any adjoining flatwork. In areas to receive mat foundations or water tanks, subgrade preparation should extend a minimum of 3 feet beyond the limits of the proposed structures. In proposed pavement areas and for exterior flatwork not connected to buildings or structures, subgrade preparation should extend at least 2 feet beyond the back of the curbs or outside limits of flatwork.

Prepared soil subgrades should be non-yielding when proof-rolled by a fully loaded water truck or similar weighted piece of equipment. Moisture conditioning of subgrade soils should consist of adding water if the soils are too dry and allowing the soils to dry if the soils are too wet. After the subgrades are properly prepared, the areas may be raised to design grades by placement of engineered fill.

Wet and/or soft soils encountered during earthwork should be stabilized prior to placement of new fill and further construction. A representative of Kleinfelder should evaluate the method of stabilization at the time of construction.

5.1.3 Non-expansive Fill

Because of the high expansion potential of the near-surface soil, we recommend all concrete slabs-on-grade be constructed on a layer of "non-expansive" fill meeting the requirements presented in Section 5.1.4, "Materials for Fill." In areas of proposed buildings and adjoining flatwork, the "non-expansive" fill layer should be at least 18 inches thick and should extend at least 5 feet horizontally beyond the limits of the proposed buildings and adjoining flatwork. Where capillary break material or Class 2 aggregate base will be used under concrete slabs (see Section 5.2.5), this material may be considered as the upper portion of the "non-expansive" fill.

For exterior concrete slabs-on-grade not connected to buildings or structures, the "non-expansive" fill should be a minimum of 12 inches thick.

No "non-expansive" fill is required under the mat foundations.

5.1.4 Material for Fill

In general, on-site soils with an organic content of less than 3 percent by weight or without visible organic matter deemed excessive by Kleinfelder, and free of deleterious materials or hazardous substances may be used as engineered fill except where special material is recommended. A layer of "non-expansive" fill is recommended under concrete slabs-on-grade, and a layer of capillary break rock is recommended under floor slabs with moisture sensitive flooring.

All import fill material should be predominantly granular, should not contain any rocks or lumps larger than 3 inches in greatest dimension, and should not contain more than 15 percent of the material larger than 1-1/2 inches. The material should contain sufficient fines to allow excavations to be made without caving, and should have a low expansion potential (as indicated by Atterberg Limits, expansion index or other appropriate test).

In additional to the above requirements, material for use as "non-expansive" fill should be predominantly granular, should have a Plasticity Index of 12 or less, and should contain 10 to 40 percent passing a U. S. Standard No. 200 sieve.

All fills should be approved by the project geotechnical engineer prior to delivery to the site. At least five (5) working days prior to importing to the site, a representative sample of each proposed import fill should be delivered to our laboratory for evaluation.

5.1.5 Fill Placement and Compaction

Fill materials should be placed and compacted in horizontal lifts each not exceeding 8 inches in uncompacted thickness. Compaction of fill should be performed by mechanical means only. Due to equipment limitations, thinner lifts may be necessary to achieve the recommended degree of compaction. Relative compaction or compaction is defined as the in-place dry density of the compacted soil divided by the laboratory compacted maximum dry density as determined by ASTM Test Method D1557-latest edition, expressed as a percentage.

Engineered fills consisting of expansive clay soil should be compacted to between 88 and 93 percent relative compaction at soil moisture content of between 3 and 5 percent above the laboratory optimum moisture content. Imported soils with low expansion potential, including the "non-expansive" fill, should be compacted to at least 90 percent relative compaction at soil moisture content of between 1 and 3 percent above the laboratory optimum moisture content. In pavement areas, the upper 12 inches of subgrade should be compacted to at least 95 percent relative compaction at soil moisture content of 1 to 3 percent above optimum value. Aggregate base materials in pavement areas should be compacted at slightly above the optimum moisture content to at least 95 percent relative compaction.

5.1.6 Trench Excavation and Backfill

We anticipate that excavation of utility trenches can be readily made with conventional excavation equipment. The walls of utility trenches in the near-surface clayey soils and less than 5 feet in height should be able to stand near vertical with minimal bracing, provided proper moisture content in the soil is maintained. Where excavations extend into sandy soils with little or no cohesion, or into groundwater, shoring or sloping of the sidewalls at a safe inclination will be required to increase slope stability. In addition, excavations should be located so that no structures, existing or new, are located above a plane projected 45 degrees upward from any point in the excavation, regardless of whether the trenches are shored or not. All excavations should be constructed in accordance with current OSHA safety standards. Safety in and around the site is the responsibility of the general contractor.

Where utility trenches extend into groundwater table, dewatering will be required so construction can proceed in a dry condition. The underground contractor is responsible for design, installation, maintenance and removal of the dewatering system. The dewatering system should be capable to draw the groundwater to at least 2 feet below the bottom of the excavation. If wet and softer soils are encountered at the bottom of the excavations, the underground contractor is responsible for over-excavation of the wet and softer soils to a sufficient depth and replacing them with ¾-inch minus clean crushed rock to create a firm platform for construction.

Pipe zone backfill, extending from the bottom of the trench to 1 foot above the top of pipe, should consist of free-draining sand unless concrete is specified. The sand should be compacted to at least 90 percent relative compaction. Above the pipe zone, underground utility trenches should be backfilled with free-draining sand, on-site soil or imported soil. The trench backfill should be compacted to at least 90 percent for on-site or imported soil backfill. Trench backfill should be capped with at least 12 inches of compacted, on-site soil similar to that of the adjoining subgrade. The upper 12 inches of trench backfill in areas to be paved should be compacted to at least 95 percent relative compaction. The backfill material should be placed in lifts not exceeding 6 inches in uncompacted thickness. Thinner lifts may be necessary to achieve the recommended level of compaction of the backfill due to equipment limitations. Compaction should be performed by mechanical means only. Water jetting or flooding to attain compaction of backfill should not be permitted.

Where under groundwater, granular backfill such as pea gravel may be considered as backfill to above groundwater level, subject to approval of the project Geotechnical Engineer depending on site conditions.

5.1.7 Surface Drainage

Final site grading should provide surface drainage away from the proposed structures and slabs-on-grade to reduce the percolation of water into the underlying soils. Surface water should not be allowed to collect adjacent to structures and along edges of concrete slabs or pavements. Grades should be sloped away from the structures a minimum of 4 percent in landscaped areas and 2 percent in paved areas for a horizontal distance of at least 5 feet. Surface water should be directed away from exposed soil slopes. Rainwater on the roof of the buildings should be conveyed through gutters, downspouts and closed pipes which discharge directly into the site storm water collection system or pavement. If discharging onto the pavement, safety of pedestrian traffic should be considered.

5.1.8 Seepage Control

Where utility lines extend through or beneath perimeter footings or curbs at pavement areas, permeable backfill should be terminated at least 1 foot from the footings or curbs. Concrete or compacted clayey soil should be used around the pipes to act as a seepage cutoff. Beneath footings, the pipes should be "sleeved" through concrete cutoffs, and the annular space around the pipes should be filled with waterproof caulk. This will help reduce the amount of water seeping through the pervious trench backfill and collecting under the building or pavements.

Where slabs or pavements abut against landscaped areas, the base rock and subgrade soil should be protected against saturation. If landscape water or surface runoff is allowed to seep into the pavement section or subgrade, the service life of the pavement will be reduced. Subdrains behind curbs in landscape areas or vertical cut-off structures are recommended to reduce lateral seepage under pavements or slabs from adjacent landscaped areas. Vertical cut-off structures may consist of deepened curb sections, or equivalent, extending at least 3 inches below the baserock/subgrade interface. Subdrains should discharge to a proper outlet as determined by the project civil engineer. Cut-off structures should be carefully constructed such that they extend below the base section and are poured neat against undisturbed native soil or compacted clayey fill. The cut-off structures should be continuous. Utility trenches (irrigation lines, electrical conduit, etc.) that extend through or under the curbs should be sealed with compacted clayey soil or poured in-place concrete. In addition, care should be taken to prevent over-watering of landscaped areas.

5.1.9 Wet Weather Construction

If site grading and construction is to be performed during the winter rainy months, the owner and contractors should be fully aware of the potential impact of wet weather. Rainstorms can cause delay to construction and damage to previously completed work by saturating compacted pads or subgrades, or flooding excavations.

Earthwork during rainy months will require extra effort and caution by the contractors. The grading contractor should be responsible to protect his work to avoid damage by rainwater. Standing pools of water should be pumped out immediately. Construction during wet weather conditions should be addressed in the project construction bid documents and/or specifications. We recommend the grading contractor submit a wet weather construction plan outlining procedures they will employ to protect their work and to minimize damage to their work by rainstorms.

5.1.10 Construction Observation

Variations in soil types and conditions are possible and may be encountered during construction. To permit correlation between the soil data obtained during this investigation and the actual soil conditions encountered during construction, we recommend that Kleinfelder be retained to provide observation and testing services during site earthwork and foundation construction. This will allow us the opportunity to compare actual conditions exposed during construction with those encountered in our investigation and to provide supplemental recommendations if warranted by the exposed conditions. All earthwork should be performed in accordance with the recommendations presented in this report, or as recommended by Kleinfelder during construction. Kleinfelder should be notified at least 2 working days prior to the start of construction and prior to when observation and testing services are needed.

We also recommend that Kleinfelder be retained to review your final foundation and grading plans and specifications. It has been our experience that this review provides an opportunity to detect misinterpretation or misunderstanding prior to the start of construction.

5.2 FOUNDATIONS

To maintain foundation support, foundations located near utility trenches or other foundations should be deepened so that their bearing surfaces are below an imaginary plane having an inclination of 1.5 horizontal to 1.0 vertical. This imaginary plane should be drawn extending upward from the bottom edge of the adjacent foundations or utility trench.

It is important that soils in the foundation excavations not be allowed to dry before placement of concrete. If shrinkage cracks appear in the foundation excavations, the soils should be thoroughly moisture conditioned for at least two days prior to concrete placement to close all cracks. Water should not be allowed to pond in the bottom of foundation excavations. Areas that become water damaged should be over-excavated to a firm base.

The foundation excavations should be observed by a representative of Kleinfelder to assess the moisture content and to confirm the adequacy of the bearing materials. We recommend that

Kleinfelder be retained to observe the foundation excavations prior to placing reinforcing steel or concrete to check that foundations are founded in the anticipated bearing soil.

5.2.1 Conventional Shallow Foundations

The proposed lightly loaded buildings may be supported on conventional shallow footing foundations bearing on undisturbed native soil or engineered fill. Load bearing walls should be supported on continuous footings and columns may be supported on isolated or continuous footings. For buildings with no perimeter load bearing walls (loads supported by perimeter columns), the perimeter column footings should be structurally tied with grade beams to provide a barrier against moisture infiltration into interior building areas.

Continuous footings (or grade beams) should have a minimum width of 12 inches and isolated spread footings should have a minimum dimension of 12 inches. The bottom of the footings should be at least 18 inches below pad grade or lowest adjacent finished grade, whichever provides a deeper embedment. Pad grade is defined as the bottom of the capillary break material or the top of the "non-expansive" fill.

Footings constructed in accordance with the recommendations above may be designed for a net allowable soil bearing pressure of 2,500 pounds per square foot (psf) due to dead plus live loads. This pressure may be increased by one-third when including transient loads such as seismic or wind.

After completion of construction, total foundation settlements are anticipated to be 1 inch or less. Differential settlements are expected to be about ½ inch or less between adjacent isolated spread footings and over a distance of 30 feet for continuous footings. The estimated settlement values due to liquefaction and dynamic compaction are in addition to these static settlement values.

Lateral loads may be resisted by a combination of friction between the bottom of foundations and the supporting subgrade, and by passive resistance acting against the vertical sides of the foundations. An ultimate friction coefficient of 0.3 may be used for friction between the foundations and supporting subgrade. Ultimate passive resistance equal to an equivalent fluid weight of 350 pounds per cubic foot (pcf) acting against the embedded sides of the foundations may be used for design purposes. The passive pressure can be assumed to act starting at the top of the lowest adjacent grade in paved areas. In unpaved areas, the passive pressure can be assumed to act starting at a depth of one foot below grade. It should be noted that the passive resistance value discussed above is only applicable where the concrete is either placed directly against undisturbed soil. Voids created by the use of forms should be backfilled with soil compacted to the requirements given in this report or with concrete.

5.2.2 Mat Foundations

The proposed generators, steam engines, boiler stacks, cooling tower, pumps, and switch board may be supported on structural mat foundations constructed on properly moisture-conditioned and compacted soil subgrade. For dead plus live loading, the mats may be designed for a net allowable bearing pressure of 3,000 psf. This value may be increased by one-third when transient wind and seismic loads are included. The mat slabs should be designed to distribute the structure loads uniformly over the entire area of the mats and should have a thickness of at least 18 inches. Structural design may require a thicker mat. The bottom of the mats should extend to a depth of at least 24 inches below the lowest adjacent finish grade.

For mat foundations constructed on properly prepared soil subgrade, the following parameters may also be used in the design of the mats.

Modulus of subgrade reaction, Kv_1 = 100 tons per cubic foot

Dynamic shear modulus, Gmax = 600 kips per square foot

Dynamic Poisson's ratio = 0.45

After completion of construction, total foundation settlement are anticipated to be 1 inch or less. Differential settlements are expected to be about 1/2 inch between the center and the perimeter of the mat.

The parameters presented in Section 5.2.1 "Shallow Foundations" for calculation of resistance to lateral loads may be used for calculation of resistance to lateral loads for the mat foundations.

5.2.3 Drilled Piers Foundations

The columns for the pipe rack may be supported on drilled pier foundations. Drilled, straight-shaft, cast-in-place, reinforced concrete piers should be designed to derive their vertical load-supporting capacity by skin friction between the pier shaft and the surrounding soils. Piers should have a minimum diameter of 24 inches and a minimum depth of 7 feet. Piers should be reinforced throughout their entire length and should be spaced at least three pier diameters apart. Piers spaced closer than this will have a reduced load capacity due to interference. For design under dead plus live loads, the following net allowable skin friction/adhesion value may be used.

Depth Below Existing Grade, feet	Type of Soil	Net Allowable Adhesion Value, psf
0 to 15 feet	Clay	600
15 to 25 feet	Sand	250
25 to 35 feet	Clay	500

Note: Consult Kleinfelder for specific recommendations for piers extending deeper than 35 feet below existing ground surface.

The above adhesion values may be increased by one-third when including transient loads such as wind or seismic. Resistance to uplift loads would be provided by the weight of the pier and skin friction along the pier shafts. We recommend a maximum of 60 percent of the allowable vertical compressive capacity be used as uplift capacity.

Relatively clean sands are present below the site. Piers extending into the sand layers will require casing to prevent caving of the pier holes. In addition, pier holes deeper than 10 feet or extending into groundwater should be constructed by placing the concrete into the holes from the bottom up using the tremie method.

Where the pier holes are cased, the casing should be slowly raised as the pier shaft is filled with concrete, with the bottom of the casing maintained at least 3 feet below the level of the concrete. Any accumulated water in the bottom of the holes should be pumped out prior to the placement of the concrete unless the water is displaced when the concrete is placed by the tremie method. The tremie pipe should be raised slowly as the pier shaft is filled with concrete, with the bottom of the tremie pipe maintained at least 3 feet below the level of the concrete. The use of the tremie pipe should be continued until the concrete is brought to the required height to promote the displacement of water and laitance (undesirable concrete-mud-water mixture) out the top of the hole. Improper placement of concrete in piers may result in either contaminated and/or weak concrete, or voids. Such defective piers will have a greatly reduced support capacity.

5.2.4 L-Pile Analysis on Drilled Pier Foundations

L-Pile analysis was performed on the drilled pier foundations for the subject project. Two soil profiles, Profile A and Profile B, were developed in our analysis. The soil profiles were developed based on subsurface soil conditions encountered in our five borings and five Cone Penetrometer Test holes advanced at the site. Soil Profile A was developed based on Borings B-1, B-2, CPT-3, and CPT-4. Soil Profile B was developed based on Borings B-6, B-7, B-8, CPT-5, CPT-9, and CPT-10. The loads on the piers were provided by PB Power, Inc. The results of our analyses performed for each given pier type are tabulated below. Pile types P1, P2 and P3 are referenced from proposed pier diameters of 24, 30, and 36 inches, respectively.

Soil Profile A (B-1, B-2, CPT-3 and CPT-4)

Soil Behavior Type	Depth to Bottom of Soil Layer, feet	Total Unit Weight, pcf	Angle of Internal Friction, degrees	Su, Undrained Shear Strength, psf	K, Subgrade Modulus, pci	ε_{50}, %
Clay	10	126	N/A	3000	1000	0.0052
Clay	19	129	N/A	1800	500	0.0066

Sand	22	135	34	N/A	60	----
Clay	30	122	N/A	1600	450	0.0069

Note: Groundwater table at 10 feet below ground surface.

Soil Profile B (B-6, B-7, B-8, CPT-5, CPT-9, and CPT-10)

Soil Behavior Type	Depth to Bottom of Soil Layer, feet	Total Unit Weight, pcf	Angle of Internal Friction, degrees	Su, Undrained Shear Strength, psf	K, Subgrade Modulus, pci	ε_{50}, %
Clay	11	110	N/A	3000	1000	0.0052
Clay	15	132	N/A	2000	600	0.0063
Sand	22	135	36	N/A	60	----
Clay	30	129	N/A	1900	550	0.0064

Notes:

1. Groundwater table at 10 feet below ground surface

Design Parameters:

Given: Maximum Applied Axial Load (Compression) = 70.5 kips
Maximum Applied Axial Load (Tension) = 34 kips
Maximum Applied Lateral Load = 10 kips
Maximum Applied Moment = 0 kip-ft
Concrete Strength = 4000 psi

Summary of LPile Analysis Based on Soil Profile A

Pier Designation	P1	P2	P3
Design Diameter (in)	24	30	36
Design Length (in)	300	192	180
Loading: Compression			
Pier Head Deflection (in)	0.01	0.003	0.002
Max. Moment in Pier (k-ft)	15	15.5	16
Max. Shear in Pier (kips)	10	10	10
Depth to Max. Moment (ft)	3	3.2	3.3
Depth to Point of Fixity (ft)	5.4	5.7	5.9
Loading: Tension			
Pier Head Deflection (in)	0.01	0.003	0.002
Max. Moment in Pier (k-ft)	15	15.5	16
Max. Shear in Pier (kips)	10	10	10
Depth to Max. Moment (ft)	3.25	3.2	3.3
Depth to Point of Fixity (ft)	5.4	5.7	5.9

Summary of LPile Analysis Based on Soil Profile B

Pier Designation	P1	P2	P3
Design Diameter (in)	24	30	36
Design Length (in)	300	192	180
Loading: Compression			
Pier Head Deflection (in)	0.01	0.003	0.002
Max. Moment in Pier (k-ft)	15	15.5	16
Max. Shear in Pier (kips)	10	10	10
Depth to Max. Moment (ft)	3	3.2	3.3
Depth to Point of Fixity (ft)	5.4	5.7	5.9
Loading: Tension			
Pier Head Deflection (in)	0.01	0.003	0.002
Max. Moment in Pier (k-ft)	15	15.5	16
Max. Shear in Pier (kips)	10	10	10
Depth to Max. Moment (ft)	3.25	3.2	3.3
Depth to Point of Fixity (ft)	5.4	5.7	5.9

5.2.5 Ring Wall Foundations for Water Storage Tanks

We recommend each water storage tank be supported on a reinforced concrete ring wall foundation founded on undisturbed native soil or properly compacted engineered fill. The bottom of the ring wall foundation should be at least 18 inches below the lowest adjacent finished grade. The width of the foundation should be such that the ring wall imposes about the same level of stress on the underlying soil as the tank and its contents impose on the subgrade soil within the ring wall. This level of stress should not exceed 3,000 pounds per square foot (psf), with an allowable one-third increase when including short-term transient wind or seismic loads.

The ring wall foundation should be reinforced to resist hoop stresses, which may be calculated by assuming an outward lateral pressure on the ring wall equal to one-half the veritcal pressure acting on the adjacent subgrade inside the ring wall.

The bottom of the steel tank should be protected from corrosion as recommended by the tank manufacturer. One typical means to reduce the potential for corrosion is to place, a cushion of granular material within the ring wall directly beneath the base of the tank. The cushion should consist of compacted, lightly oiled, clean sand (or clean, fine crushed rock), and its surface should be shaped to provide a crown of 1 inch per 10 feet of diameter from the center of the tank. Oil and sand should be thoroughly mixed either in a concrete mixer or by hand. Sufficient oil should be used to thoroughly wet but not saturate the sand.

The tank should be bolted to the ring wall to provide an anchorage for resisting lateral or uplift forces that might be developed during an earthquake. A layer of sand-cement grout mix should be placed to fill the gap between the top of the ring wall and the underside of the tank base plate to provide uniform support around the periphery of the tank bottom.

5.2.6 Building Floor Slab

The proposed building floor slabs should be constructed on a layer of properly moisture-conditioned and compacted “non-expansive” fill as recommended in the "Earthwork" section. The required thickness and reinforcement for the slab should be determined by the project structural engineer. As a minimum, we suggest a slab thickness of 4 inches with reinforcement consisting of No. 3 steel reinforcing bars at 18 inches on center each way. Care must be taken during construction to keep the reinforcement from being pushed to the bottom of the slab.

The suggested minimum steel reinforcement will not prevent the development of slab cracks but will aid in keeping the construction joints relatively tight and reduces the potential for differential movement between adjacent panels. Slab control joints should be spaced in accordance with the recommendations presented in the ACI Manual of Concrete Practice.

Where the risk of moisture penetration through the slab is to be reduced, the slab should be constructed on a layer of capillary break material covered by a continuous impermeable membrane vapor barrier. The capillary break material should be at least 4 inches thick, and should consist of free-draining crushed rock or gravel graded such that 100 percent will pass the 1-inch sieve and none will pass the No. 4 sieve. The impermeable membrane should consist of 10 or 20-mil polyethylene sheeting or similar moisture barrier. Lapped joints and perforations in the vapor barrier should be kept to a minimum, and should be sealed. Use of large sheets is recommended wherever possible to reduce the potential for moisture vapor to seep through the vapor barrier (at joints). To provide protection for the membrane, 2 inches of slightly moistened clean fine sand should be placed on top of the membrane prior to placement of concrete. Where crushed rock is used as the capillary break material, seating of the rock with a vibratory plate compactor may aid in reducing the potential for damage to the vapor barrier as the reinforcing steel and the concrete are placed. The potential for punctures in the polyethylene sheeting due to foot traffic during construction activities would be much lower where 20-mil sheeting is used. This combined 6-inch thick layer of capillary rock, vapor barrier and sand may be considered as the upper 6 inches of the recommended "non-expansive" section.

To further reduce the potential for soil moisture to migrate through the slabs-on-grade as a vapor, and to reduce concrete shrinkage, consideration should be given to the use of a concrete mix with a low water/cement ratio. By specifying a concrete mix with a water/cement ratio of 0.45 to 0.49 for use at the slab-on-grade floors, the degree of porosity of the concrete will be reduced. This will also reduce the amount of entrapped water within the fresh concrete and lessen the potential for moisture vapor distress to flooring products. The use of Flyash, water reducing admixtures and/or plasticizers will increase the workability of a mix with relatively low water to cement ratio. In addition to controlling the water/cement ratio at the time of batching, the addition of water to the ready-mix concrete at the site should be strictly controlled. Consolidation of the concrete will also reduce the degree of vapor that can pass through the slab.

Where concrete slabs will be subject to vehicle traffic, forklift loads or vibratory loads, and if moisture penetration through the slabs is not crucial, at least 6 inches of Class 2 Aggregate Base should be placed and compacted to at least 95 percent relative compaction under the slabs.

5.3 EXTERIOR CONCRETE SLABS-ON-GRADE

Proper moisture conditioning and compaction of subgrade soils is very important. All exterior slabs should be constructed on a layer of "non-expansive" fill as recommended under the "Non-expansive Fill" section of this report. Even with proper site preparation, there will still be some effects of soil moisture change on concrete flatwork. Exterior flatwork will be subjected to edge effects due to the drying out or wetting of subgrade soils where adjacent to landscape or vacant areas. To help reduce edge effects, lateral cutoffs such as an inverted curb are suggested. Control joints should be spaced on a maximum of 10-foot centers to reduce the potential for unsightly panel cracks as a result of soil displacement. The use of steel reinforcement will aid in keeping the control joints and any other cracks tightly closed.

Exterior concrete slabs-on-grade should be cast free from adjacent footings or other non-heaving edge restraint. This may be accomplished by using a strip of 1/2-inch asphalt impregnated felt divider material between the slab edges and the adjacent structure. Frequent construction or control joints should be provided in all concrete slabs where cracking is objectionable. Continuous reinforcing or dowels at the construction and control joints will also aid in reducing uneven slab uplift.

5.4 Retaining Walls

Retaining walls must be designed to resist static earth pressures due to the adjacent soil, surcharge pressures induced by loads close to the walls, and seismic pressures induced during an earthquake. For this project, we recommend the walls be designed using the lateral pressures presented below, which are expressed as equivalent fluid weights for on-site soil backfill.

Lateral Earth Pressures For Walls Up To 15 Feet High	
Active Soil Pressure	45 pcf
At-rest Soil Pressure	65 pcf
Passive Soil Pressusre	350 pcf (ultimate)

For static loading conditions, the walls may be designed using at-rest or active soil pressure as discussed herein. At-rest soil pressure should be used for retaining walls where movement at the top of walls is restrained or undesirable. Wall movement could cause settlement of backfill and structures supported on the backfill. Active soil pressure may be used for retaining walls where the top of walls is free to deflect and resulting movement of the backfill is acceptable. The at-rest and active soil pressures given above are for level backfill and do not include hydrostatic pressures that might be caused by groundwater or water trapped behind the walls.

For seismic loading conditions, the walls may be evaluated using active soil pressure plus a horizontal seismic line force of $17H^2$ pounds per lineal foot (where H is the height of the vertical design plane from the wall base to the ground surface above). The resultant of the active soil pressure should be applied at H/3 above the wall base and the resultant of the seismic line force should be applied at 2/3H above the wall base. A reduced factor of safety for overturning and sliding may be used in seismic design.

The effects of surcharge loads close to the walls should be included in the wall design, including foundation and floor loads from adjacent buildings, traffic loads from adjacent streets and parking, etc. To simulate the effect of adjacent occasional passenger cars or light pickup trucks, a horizontal uniform pressure of 50 pounds per square foot may be assumed to act against the full height of the walls. For other uniform loads behind the walls, such as floor loads from the adjacent buildings or equipment loads, the additional lateral surcharge pressure should be 50 percent of the vertical surcharge loads. For adjacent foundation loads and other line loads, point loads, strip loads, heavy truck loads, etc., Kleinfelder should be consulted for specific recommendations.

For static loading conditions, a minimum factor of safety of 1.5 should be used for overturning and sliding. For seismic loading conditions, a minimum factor of safety of 1.2 should be used for overturning and sliding.

Backfill against structures should be compacted as discussed in the "Earthwork" Section of our report. Over-compaction should be avoided because increased compaction effort can result in lateral pressures significantly higher than those recommended above. Backfill placed within 5 feet of the walls should be compacted with hand-operated equipment.

Retaining walls less than 15 feet in height for this project may be supported on shallow footings or mat foundations founded on undisturbed native material or engineered fills as presented under the "Foundations" Section of this report.

Retaining walls should be well drained to reduce the potential for built-up of hydrostatic pressure. A typical drainage system consists of a 1 to 2 foot wide zone of crushed, free draining gravel (with less than 5 percent fines) wrapped in a geotextile filter fabric (Mirafi 140N or equivalent) or Caltrans Class 2 Permeable Material (Caltrans Standard Specifications, Section 68) immediately adjacent to the walls. Geotextile filter fabric is not required if Class 2 Permeable material is used. As an alternative, a prefabricated drainage board such as Mirdrain G100W or equivalent may be used in lieu of the Class 2 Permeable Material or filter wrapped drain rock. A minimum 4-inch diameter, rigid, perforated pipe should be placed in the lower portion of the drainage material to collect discharge water to a storm drain or other discharge facility. The pipe should be PVC Schedule 40 or ABS with an SDR of 35 or better. The pipes should be sloped to drain by gravity to the sump pump system/outlets.

We recommend the retaining wall design showing height of wall, backfill material type, drainage details and the earth pressures used be reviewed by Kleinfelder for conformance to the recommendations given.

5.5 PAVEMENTS

Pavements for this project will consist of parking and driveway areas for light passenger cars and pickup trucks, with heavier traffic areas for equipment and maintenance trucks.

A bulk sample of the near surface soil were obtained and the laboratory test measured an R-value of 10. For design purposes, an R-value of 10 was used in our pavement section calculation to take into consideration the expansive clay soil encountered at the site.

5.5.1 Flexible Pavements

For this project, we have included flexible pavement sections for Traffic Indices (TIs) of 4.5 to 7.5. The recommended pavement sections are presented in the table below.

FLEXIBLE PAVEMENT SECTION ALTERNATIVES R-VALUE = 10				
Traffic Index	**Asphalt Concrete (inches)**	**Class 2 Aggregate Base (inches)**	**Class 2 Aggregate Sub-base (inches)**	**Total Thickness (inches)**
4.5	2.5	9.0	----	11.5
	3.0	8.0	----	11.0
5.0	2.5	10.0	----	12.5
	3.0	9.0	---	12.0
5.5	3.0	11.0	----	14.0
6.0	4.0	11.0	----	15.0
6.5	4.0	13.0	----	17.0
	4.0	6.0	7.0	17.0
7.0	4.0	15.0	----	19.0
	4.0	6.0	9.0	19.0
7.5	4.0	16.0	----	20.0
	4.0	6.0	11.0	21.0

The anticipated traffic and alternate pavement sections presented in this section should be reviewed by the project civil engineer in consultation with the owner during the development of the final grading plans. We have made our pavement designs based on the pavement subgrade soil consisting of a fat clay soil. If site grading exposes soil other than that utilized in our analysis, we should perform additional tests to confirm or revise the recommended pavement sections to reflect the actual field conditions.

Subgrade preparation should extend a minimum of 2 feet laterally beyond the face of the curb and should comply with the requirements under Sections 5.1 "Earthwork" and its subsections. Compacted pavement subgrade should be non-yielding. Removal and subsequent replacement of some material (i.e., areas of excessively wet materials, unstable subgrades, or yielding soils) may be required to obtain the minimum 95 percent compaction to the recommended depth.

Asphalt Concrete should meet the requirements for 1/2- or 3/4-inch maximum, medium Type B asphalt concrete in vehicle areas, Section 39, Caltrans Standard Specifications, 1992 edition. The Class 2 Aggregate Base material should conform to Section 26 of the Caltrans Standard Specifications. The Class 2 Aggregate Subbase should comply with Section 25 of the Caltrans Standard Specifications. ASTM test procedures should be used to assess the percent relative compaction of soils, aggregate base and asphalt concrete. Asphalt concrete should be compacted to a minimum of 96 percent of the maximum laboratory compacted (Hveem) unit weight.

5.5.2 Portland Cement Concrete Pavements

Using the Portland Cement Association guidelines for concrete parking areas, we have developed the following recommendations for parking areas and access driveways.

AREA	MINIMUM THICKNESS
Automobile and light pickup trucks (single axle load of less than 2,500 lbs, front and rear)	4 inches
Two-axle trucks (single load of less than 9,000 lbs front and less than 18,000 lbs rear)	6 inches (up to 40 passes per week) 7 inches (unlimited passes per week)
Three-axle trucks (single front axle load of less than 8,000 lbs and tandem rear axle load of less than 32,000 lbs)	6 inches (up to 20 passes per week) 7 inches (unlimited passes per week)

Concrete pavements should be constructed on at least 6 inches of Class 2 Aggregate Base compacted to a minimum of 95 percent relative compaction based on ASTM D1557 – latest edition. Concrete should have a minimum compressive strength of 4,000 psi (28-day).

Concrete curbs or shoulders, and construction and expansion joints should be provided, as designed by the project Civil or Structural Engineer.

6 ADDITIONAL SERVICES AND LIMITATIONS

6.1 ADDITIONAL SERVICES

The review of plans and specifications, and the observation and testing by Kleinfelder of earthwork related construction activities, are an integral part of the conclusions and recommendations made in this report. If Kleinfelder is not retained for these services, the client will be assuming our responsibility for any potential claims that may arise during or after construction. The required testing, observation, and consultation by Kleinfelder during construction includes, but is not limited to:

- Review of plans and specifications;
- Observation of site clearing and stripping;
- Construction observation and density testing during subgrade preparation, placement and compaction of fill material, backfilling of utility trenches, and pavement construction; and
- Observation of foundation excavations and foundation construction.

6.2 LIMITATIONS

The services provided under this contract as described in this report include professional opinions and conclusions based on the data collected. These services have been performed according to generally accepted geotechnical engineering practices that exist in the project area at the time the report was written. This report is issued with the understanding that the owner chooses the risk they wish to bear by the expenditures involved with the construction alternatives and scheduling that is chosen. No warranty is expressed or implied.

This report may be used only by PB Power, Inc., City of Santa Clara and their consultants and contractors for the project, only for the purposes stated, and within a reasonable time from its issuance. Land use, site conditions (both on site and off site) or other factors may change over time, and additional work may be required with the passage of time. Any party other than PB Power, Inc., City of Santa Clara and their authorized consultants and contractors who wishes to use this report shall notify Kleinfelder of such intended use. Based on the intended use of the report, Kleinfelder may require that additional work be performed and that an updated report be issued. Non-compliance with any of these requirements by the clients or anyone else will release Kleinfelder from any liability resulting from the use of this report by any unauthorized party.

The conclusions and recommendations in this report are for the proposed improvements at the building additions project at the Pico Power Plant in Santa Clara, California, as described in the text of this report. The conclusions and recommendations in this report are invalid if:

- The anticipated structure loads or the proposed structure locations change;
- The report is used for adjacent or other property;
- The Additional Services section of this report is not followed, particularly the observation of subgrade preparation and placement and compaction of engineered fills;
- If changes of grades occur between the issuance of this report and construction, or
- Any other change is implemented which materially alters the project from that proposed at the time this report was prepared.

The conclusions and recommendations presented in this report are based on information obtained from the following:

- Five borings and five CPT holes advanced within the site;
- The observations of our engineer;
- The results of laboratory tests; and
- Our experience on similar projects with similar soil conditions.

The logs of the exploratory borings and CPT holes do not provide a warranty as to the conditions that may exist beneath the entire site. The extent and nature of subsurface soil and groundwater variations may not become evident until construction begins. It is possible that variations in soil conditions and depth to groundwater could exist beyond the points of exploration that may require additional studies, consultation, and possible design revisions. If conditions encountered in the field during construction are different from those described in this report, our firm should be contacted immediately to provide any necessary revisions to these recommendations.

GEOTECHNICAL INVESTIGATION
PROPOSED PICO POWER PLANT
LAFAYETTE STREET AND DUANE AVENUE
SANTA CLARA, CALIFORNIA

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GEOTECHNICAL INVESTIGATION
PROPOSED PICO POWER PLANT
LAFAYETTE STREET AND DUANE AVENUE
SANTA CLARA, CALIFORNIA

PREPARED FOR: PB Power, Inc.
303 Second Street, Suite 700
San Francisco, California 94107

ATTENTION: Mr. Colin McRae

This document was prepared for use only by PB Power, Inc., City of Santa Clara and their consultants and contractors on the subject project, for the purposes stated and within a reasonable time from its issuance. Please read the "Limitations" section of this report. Unauthorized use or copying of this document is strictly prohibited. See "Application for Authorization to Use" located at the end of this document if use or copying is desired by anyone other than those indicated for the project identified above.

August 26, 2002

18234 (SJO2R459) bl

APPLICATION FOR AUTHORIZATION TO USE

Proposed Pico Power Plant
Lafayette Street and Duane Avenue
Santa Clara, California
18234
August 26, 2002

Kleinfelder, Inc.
1362 Ridder Park Drive
San Jose, CA 95131
408-436-1155 408-436-1771
(Telephone) *(Fax)*

To whom it may concern:

Applicant understands and agrees that the Geotechnical Investigation Report for the subject site is a copyrighted document, that Kleinfelder, Inc., is the copyright owner and that unauthorized use or copying of the Report for the subject site is strictly prohibited without the express written permission of Kleinfelder, Inc. Applicant understands that Kleinfelder, Inc., may withhold such permission at its sole discretion, or grant permission upon such terms and conditions as it deems acceptable.

Applicant agrees to accept the contractual terms and conditions between Kleinfelder, Inc., and General Growth Properties, Inc. originally negotiated for preparation of this Geotechnical Investigation Report. Use of this Report without permission releases Kleinfelder, Inc. from any liability that may arise from use of this report.

To be Completed by Applicant

____
(company name)

____
(address)

____
(city, state, zip)

____________________ ____________________
(telephone) *(FAX)*

By: ____________________________

Title: ____________________________

Date: ____________________________

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________ disapproved, report needs to be updated

By: ____________________________
(Kleinfelder, Inc. project manager)

Date: ____________________________

August 26, 2002
File: 18234

Mr. Colin McRae
PB Power, Inc.
303 Second Street, Suite 700
San Francisco, California 94107

SUBJECT: **Geotechnical Investigation for the Proposed Pico Power Plant in Santa Clara, California**

Dear Mr. McRae,

Kleinfelder, Inc. is pleased to present our geotechnical investigation report for the subject project. The accompanying report provides the results of our field investigation, laboratory testing and engineering analyses. Design recommendations for site earthwork, foundations, concrete slabs-on-grade, site drainage and flexible and rigid pavements are provided. The report has been updated from our draft report dated July 31, 2002, to include your review comments.

The primary geotechnical considerations with respect to the proposed construction are the highly expansive surface soils and high groundwater table encountered at the site. The recommendations contained in our report address these conditions and provide recommendations intended to reduce the effects of these conditions on the proposed construction. We have included recommendations for non-expansive fill in conjunction with moisture conditioning of the expansive subgrade soils at the site. Recommendations for the design of conventional footings, mats and drilled pier foundations are presented.

As noted in our report, Kleinfelder should be engaged to review project plans and specifications as well as to observe the earthwork and construction of the foundations. If you have any questions regarding the information or recommendations presented in our report, please do not hesitate to contact us at your convenience.

If you have any questions regarding the information or recommendations presented in our report, please do not hesitate to contact us at (408) 436-1155 at your convenience.

Sincerely,
KLEINFELDER, INC.

Chalerm "Beeson" Liang, CE, GE
Geotechnical Department Manager

Michael Clark, CEG
Senior Engineering Geologist

Copies: Addressee (10)