CGI Technical Services Inc.

Geotechnical Report

SPI COGENERATION FACILITY

ANDERSON, CALIFORNIA

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Mr. Cedric Twight, R.P.F.
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P.O. Box 496014
Redding, California 96049

Subject: **Geotechnical Investigation Report**
Proposed SPI Cogeneration Plant
Shasta County, California

Dear Mr. Twight,

CGI Technical Services, Inc., is pleased to submit this report for the above referenced project. The purpose of this investigation was to explore and evaluate the surface and near surface conditions and provide subsurface and laboratory data to assist Sierra Pacific Industries with the design and construction of the proposed cogeneration plant. This report presents findings during our field investigation, laboratory testing, and our recommendations for the proposed project.

We appreciate the opportunity to perform this study and look forward to continued participation during the design and construction phases of this project. If you have any questions pertaining to this report, or if we may be of further service, please contact us at (530) 244-6277 at your earliest convenience.

Regards,

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SHASTA COUNTY, CALIFORNIA

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1.0 GENERAL

This report presents the results of our geotechnical investigation for a proposed cogeneration facility within the Sierra Pacific Industries' saw mill property, Shasta County, California. CGI Technical Services, Inc. (CGI), prepared this report at the request of Sierra Pacific Industries, Inc. (SPI) and in general accordance with our proposal, CG07P065, dated August 29, 2007. The following sections present the project understanding, purpose of our study, and the findings, conclusions, and recommendations of this study. Our scope of work only includes items specifically addressed within this report. Plate 1 – Site Location Map, presents the general site location of this project.

1.1 PROJECT UNDERSTANDING

The proposed project consists of the design and construction of a new cogeneration facility. The proposed facility is to be located adjacent to an existing cogeneration plant that is located centrally within the SPI Anderson sawmill property. The new facility will consist of several structures, including a fuel shed, boiler, cooling tower, turbine, and an electro static precipitator (ESP). The anticipated column loads are estimated to be on the order of 25-50 kips, with some loads reportedly as high as 750 kips on the boiler. The heights of the proposed structures range from about 38 feet for the cooling tower and the turbine to as high as about 105 feet for the boiler.

It is anticipated that the supporting elements of the proposed structures will consist of a combination of shallow, slab-on-grade foundations and deep, cast-in-place pier or driven pile foundations.

1.2 STUDY PURPOSE

The purpose of this study was to characterize selected geotechnical and subsurface information in the vicinity of the proposed structures and to provide geotechnical recommendations for the design and construction of the structure foundations. In addition, geologic hazards that might affect the project were assessed and geotechnical recommendations provided for those geologic hazards that were identified to potentially adversely affect the project.

1.3 SCOPE OF SERVICES

Our scope of services included the following:

- Reconnaissance of the site surface conditions, topography and existing drainage features.
- Exploration of the subsurface conditions within the footprint of the proposed facility using 5 test pits and 5 drill holes. Information regarding the subsurface exploration and logs of test pits and drill holes are presented in Appendix A – Subsurface Exploration, and on Plate 2 - Test Pit Locations and Proposed Improvements.
- Limited laboratory testing on selected samples obtained during our field investigation. Results of the laboratory tests are presented in Appendix B – Laboratory Testing
- Preparation of this report, which includes:
    - A description of the proposed project;
    - A map showing field exploration locations is presented in Plate 2 – - Test Pit and Drill Hole Locations and Proposed Improvements.
    - A summary of our field exploration and laboratory testing programs;

- A description of site surface and subsurface conditions encountered during our field investigation;
- An evaluation of geohazards that might affect the proposed improvements;
- A description of ground shaking conditions expected at the site, including CBC seismic design criteria;
- Recommendations for:
    - Liquefiable and compressible fill;
    - Site grading;
    - Dewatering;
    - Temporary excavations;
    - Engineered fill;
    - Shallow foundation design;
    - Deep foundation design;
    - Retaining wall design; and
    - Preliminary geotechnical recommendations for pavement section design; and
- Appendices, which include summaries of our field investigation procedures and laboratory testing programs, and engineering analysis.

2.0 FINDINGS

2.1 SITE CONDITIONS

2.1.1 Surface Conditions

The proposed facility is located on property that has been used as a sawmill and wood processing facility for decades. As a result, there are past and present infrastructure improvements located throughout the area. Remnants of past improvements adjacent to the proposed facility include large log ponds that are located to the northwest and southeast of the site and large concrete footings associated with a dismantled boiler located northwest of the site. Current improvements located to the immediate northeast and southeast of the project facility include an existing fuel shed and cogeneration plant, respectively. Other ancillary improvements include asphalt-paved areas, above ground and below ground utilities, overhead conveyor belts, and two large fuel silos. The footprint of the proposed facility is mostly within undeveloped open space that has a gravel surface. However, a portion of the proposed fuel shed overlaps the location of the existing fuel silos, which are to be dismantled.

2.1.2 Topography & Drainage

The site resides within a flood terrace of the Sacramento River, which is located immediately northeast of the site and parallels the site's northeast border. The topographic expression of the site has a gentle slope of about 1%-2% to the northeast with an average elevation of about 420 feet above mean sea level. Disrupting the relatively planar and low gradient topographic expression across the site are depressions formed by old log ponds and several surface drainage ditches.

Surface drainage across the site occurs as sheet flow into the surrounding log ponds and drainage ditches, where it is eventually conveyed to the Sacramento River.

2.2 GEOLOGIC CONDITIONS

2.2.1 Regional Geologic Setting

The project site is located in the northern Sacramento Valley at the northern margin of the Great Valley Physiographic province. The Great Valley province is bordered to the north by the Klamath and Cascade Physiographic provinces, to the east by the Cascade and Sierra Nevada Physiographic provinces, to the west by the Klamath and Coast Ranges Physiographic provinces, and to the south by the Transverse Ranges Physiographic province.

The Great Valley Physiographic province is about 50 miles wide and 400 miles long. The Sacramento Valley, which forms the northern portion of the province, is about 150 miles long and 40 miles wide (Hinds, 1952). According to Hackel (1966), "The Great Valley is a large elongate northwest-trending asymmetric structural trough that has been filled with a tremendously thick sequence of sediments ranging from Jurassic to recent." Sediment thicknesses of up to 10 miles are reported within the Sacramento Valley; however, in the project area, being at the northern margin of the valley, those thicknesses have been projected to be less than one mile (Hackel, 1966). Sediments within the Great Valley consist of both marine and continental deposits, with most of the sediments underlying the project area consisting of continental deposits.

2.2.2 Local Geologic Setting and Subsurface Conditions

The footprint of the proposed cogeneration facility and associated improvements is located on top of a prominent flood terrace adjacent to the Sacramento River. According to Fraticelli, et al (1987), this flood terrace is composed of sediment belonging to the Modesto Formation. The Modesto Formation is of Pleistocene age, is commonly found bordering river channels in the area, and is generally described as being composed of tan to light gray gravely sand, silt and clay.

To better characterize the local geology and subsurface soil conditions, we performed a subsurface exploration of the site consisting of excavating and logging a total of 5 test pits and 5 drill holes (Plate 2). A full discussion outlining the work completed in our subsurface exploration and the results obtained is provided in Appendix A – Subsurface Exploration. However, for brevity, the following is a summery of the general conditions encountered below the site.

Depth below ground surface	**Description**
0-15 feet	**Fill:** Predominantly wood debris with up to 2-foot thick interbeds of clayey SILT with gravel to silty SAND with gravel and cobbles. The estimated lateral distribution and thickness of fill is provided in Plate 3.
4-20 feet	**Modesto Formation**: Sandy GRAVEL with some cobbles, moist, gray, dense to very dense, well-graded, rounded clasts up to 8 inches in diameter.
18+ feet	**Riverbank Formation (?):** Silty to Clayey GRAVEL with some cobbles, moist, tan, very dense, weakly cemented, rounded clasts up to 8 inches in diameter. Based on the color change and cementation, we interpret this unit to possible belong to the Riverbank Formation.

Depths below about 22 feet were not directly sampled due to drilling refusal. Thus, material properties of soils beneath this depth were extrapolated from adjacent subsurface information provided by Caltrans and their investigation of a bridge spanning the Sacramento River (Bridge number 06-0128) located approximately 1,000 yards southeast of the site.

2.2.3 Geologic Structure

The project is located within thick sequences of alluvium derived by the Sacramento River and adjacent watercourses. No near-surface faults or folds have been mapped at the project site (Strand, 1977). The closest mapped fault is located about 6 miles southeast of the site and is known as the Bear Creek Fault that trends away from the project site with a southwest to northeast orientation.

Based on our subsurface investigation, the structural orientation of local bedding is roughly horizontal, with minor internal structures consisting of dipping cross beds.

2.2.4 Faults & Seismicity

The State of California designates faults as active, potentially active, and inactive depending on the recency of movement that can be substantiated for a fault. Fault activity is rated as follows:

Fault Activity Ratings

Fault Activity Rating	Geologic Period of Last Rupture	Time Interval (Years)
Active	Holocene	Within last 11,000 Years
Potentially Active	Quaternary	11,000 to 1.6 Million Years
Inactive	Pre-Quaternary	Greater than 1.6 Million Years

The California Geologic Survey (CGS) evaluates the activity rating of a fault in fault evaluation reports (FER). FERs compile available geologic and seismologic data and evaluate if a fault should be zoned as active, potentially active, or inactive. If an FER evaluates a fault as active, then it is typically incorporated into a Special Studies Zone in accordance with the Alquist-Priolo Earthquake Hazards Act (AP). AP Special Studies Zones require site-specific evaluation of fault location and require a structure setback if the fault is found traversing a project site.

No faults are known to project through the project site (Jennings, 1994; Hart & Bryant, 1997; Strand, 1977). However, a number of regional and local faults traverse the project region. The most significant of these faults is the potentially active Battle Creek fault, located about 9 miles south of the site (Jennings, 1994). The closest fault mapped to the site is the inactive Bear Creek fault, located about 6 miles to the southeast (Jennings, 1994). The closest active fault, as zoned by the State, is the Foothill Fault System, located about 19 miles south-southeast of the site.

In addition to the continental faulting noted above, the project area rests above the Cascadia subduction zone. West of the site, off the coast of California, the oceanic crust of the Gorda plate is being subducted beneath the continental crust of the Pacific Plate, in an area known as the Gorda Escarpment. The descending ramp caused by that subduction, called the Cascadia Subduction zone, extends beneath the project area at a depth of about 20 to 25 miles. That ramp is capable of storing elastic stress that periodically causes earthquakes that could affect the project area.

The following table presents fault location and information data collected from the CGS database (Blake, 1999a).

Fault Information

Fault Name	Fault Activity Rating[1]	Distance From Site		Upper Bound Earthquake (M_w)
		Miles	Kilometers	
Battle Creek	PA	9.3	15.0	6.5
Foothills Fault System	A	18.5	29.8	6.5
Rate for NE California	A	29.3	47.2	7.3
Hat Creek-MacArthur	A	48.2	77.8	6.7
Great Valley 1	PA	55.1	88.6	6.7
Lake Mountain	A	61.1	98.3	6.8

[1] - A= active, PA = potentially active, per Peterson et al. (1996).

2.2.5 Historical Seismicity

Northern California is a seismically active area that has been subjected to numerous historical earthquakes. A search of historical earthquakes occurring between 1800 and 1999, listed in the CGS catalog, was performed for a 100-mile radius around the project site (Blake, 1999b). That search found that 207 earthquakes have occurred within that area. Of those earthquakes, only 44 with moment magnitudes (M_W) of 5 or greater, and 2 with M_W 6 and one with M_W 6.5 or greater have occurred in the search area. The largest earthquake to affect the area was a M_W 6.5 that occurred on December 21, 1954. The closest earthquake to affect the site was a M_W 4.5 that occurred approximately 6.2 miles from the site on April 16, 1904. The most recent significant earthquake to affect the project area was a M_L 5.2 earthquake that occurred on November 26, 1998.

2.2.6 Groundwater

Groundwater was encountered in explorations made at the site at an average depth of about 10 feet below ground surface. However, the depth to groundwater is expected to vary throughout the year and from year to year. Intense and long duration precipitation, modification of topography, and cultural land uses, such as water well usage, on site waste disposal systems, and water diversions can contribute to fluctuations in groundwater levels. If groundwater is encountered during construction, it is the Contractor's responsibility to install mitigation measures to address the adverse impacts caused encountering groundwater.

3.0 GEOLOGIC HAZARDS

3.1 FLOODING

The site is located on a flood terrace of the Sacramento River. According to the Federal Emergency Management Agency (FEMA), the terrace encompassing the site has a 0.2% annual chance of flooding (equivalent to the 500 year flood). The depth of inundation associated with such a flood is unknown.

In addition to the large magnitude floods identified by FEMA, small isolated areas of ponding water and flooding are expected in closed depressions and along drainage ditches located throughout the site, respectively.

3.2 SEISMIC SHAKING

This section provides California Building Code (CBC) seismic design criteria for the site as well as an estimate of the peak horizontal ground acceleration. The peak horizontal ground acceleration that could affect the project site was estimated using a deterministic evaluation. If required, probabilistic evaluations of horizontal strong ground motion can be provided for structural design purposes upon request.

Latitude and longitude values used for seismic evaluations of the site are as follows:

- Latitude: N40º 28' 20" (N40.4723º)
- Longitude: W122º 25' 31" (W122.3237º)

3.2.1 CBC Design Recommendations

At a minimum, structures should be designed in accordance with the California Building Code (CBC) criteria. CBC-based design requires the definition of a Seismic Zone Factor (Z), a Soil Profile Type (S), Seismic Source Type, Near-Source Factors (N_a and N_v), Seismic Coefficient (C_a and C_v), Site Coefficient Factor (S) and an Importance Factor (I).

The Structural Engineers Association of California (SEAOC) Commentary to the CBC indicates that "the primary function of the CBC design requirements are to provide minimum standards for use in building design regulations to maintain public safety in the extreme earthquakes….not to limit damage, maintain function, or provide for easy repair". The owner should note that in the event of severe ground motions, structures designed per the CBC may be subject to structural damage.

CBC Seismic Zone Factor. The design of structures for seismic loading conditions, in accordance with the 2001 edition of the CBC, should be based on a Seismic Zone Factor, Z, equal to 0.30. The UBC's Seismic Zone Factor should not be used as an estimate of peak ground acceleration.

CBC Soil Profile Type. The CBC Soil Profile Type, S, is a function of the soil conditions and subsurface stratigraphy. Ignoring the soft, unclassified fill in the upper 5 to 15 feet (See section 4.0), the site is underlain by stiff soils having a site profile S_D.

CBC Seismic Source Type. The CBC Seismic Source Type is based upon the estimated maximum moment magnitudes and slip rates of faults in the project region. As discussed above, a number of seismic sources are present in the project region. Based upon the estimated slip rates and moment magnitudes of the controlling fault, the Battle Creek fault, it is estimated that the Seismic Source Type conforms to a Type "B" (Peterson et al., 1996). Seismic Source Type B encompasses faults that have the potential to generate moment magnitudes of greater than or equal to 6.5 with a slip rate of less than 2 millimeters per year.

CBC Near-Source Factors. The CBC Near-Source Factors, N_a and N_v, for Zone 3 are:

- N_a – 1.0
- N_v – 1.0

CBC Seismic Coefficient. The CBC seismic coefficients, C_a and C_v, are based upon the Seismic Zone Factor, Z, and Soil Profile Type, S. As discussed above, the Seismic Zone Factor is estimated to be 0.3 and the Soil Profile Type is estimated to be S_D. Using those criteria, the Seismic Coefficients are estimated to be the following:

- C_a – 0.36N_a
- C_v – 0.54N_v

3.2.2 Deterministic estimates of strong ground motion

Peak horizontal ground accelerations were estimated for the project site using attenuation relations from Boore et al. (1997) and the computer program EQFAULT (Blake, 1999a). The results of those estimates are shown in the following table:

Deterministic Ground Motion Data

Fault Name	Maximum Credible Magnitude (M_W)	Distance From Site (mi)	Fault Data Length (km)	Fault Data Slip Rate (mm/yr) [A]	Deterministically Estimated Peak Ground Acceleration (g) M [B]	M+δ [B]
Battle Creek	6.5	9.3	29	0.5±0.4	0.25	0.41
Foothills Fault System	6.5	18.5	360	0.05±0.03	0.12	0.19
Rate for NE California	7.3	29.3	230	4±2	0.10	0.17
Hat Creek-MacArthur	6.7	48.2	96	1.5±1	0.05	0.09
Great Valley 1	6.7	55.1	33	6±3	0.03	0.06

[A] – From Peterson et al. (1996). [B] – M = indicates estimated mean peak horizontal ground acceleration. **M+δ** = peak horizontal ground acceleration utilizing mean plus at least one standard deviation (84th percentile) for seismicity data.

Soil conditions modeled in the deterministic studies consisted of stiff and deep soils having shear wave velocities averaging 250 meters per second. Based on these evaluations, the site could be subjected to an estimated mean peak horizontal ground accelerations of about 0.25g. The causative fault that is responsible for that peak horizontal ground acceleration is the Battle Creek fault, located about 9.3 miles south of the project site. It should be noted that probability and exposure periods are not considered during deterministic evaluations and that, typically, deterministic estimates of strong ground motion for a site generate relatively conservative horizontal ground acceleration values.

3.3 LIQUEFACTION

Liquefaction is described as the sudden loss of soil shear strength due to a rapid increase of soil pore water pressures caused by cyclic loading from a seismic event. In simple terms, it means that a liquefied soil acts more like a fluid than a solid when shaken during an earthquake.

Near surface, low-density wood debris interbedded with silty sand layers and a relatively shallow groundwater table were encountered during exploration at the project site. To estimate if the underlying soils have the potential to liquefy during a seismic event, we used methods described by Youd et al (2001). For our analyses, we assumed groundwater is present at a depth of 2 feet and performed the analysis using a maximum credible earthquake magnitude of 6.5 and a horizontal ground acceleration of 0.25g, which corresponds to the mean peak ground acceleration determined deterministically and presented in Section 3.2.2 of this report. A factor of safety (FOS) against liquefaction occurring of 1.3 or less is typically considered a potentially liquefiable layer. If the FOS exceeds 1.3 then liquefaction is not considered as potentially impacting the project.

The liquefaction analyses indicate that there is a potential for liquefaction to occur within the loose fill across the site with reported FOS of less than 1.3. Because the fill material beneath the site consists mostly of wood debris with strength properties that do not precisely fit traditional soil strength conditions used in the model, there is some uncertainty surrounding the magnitude of liquefaction. The depth of liquefiable fill located across the site ranges between about 5 and 15 feet. Results of the liquefaction analyses are presented in Appendix C - Liquefaction.

In general, the effects of liquefaction on the proposed project could include:

- Lateral spreading;
- Vertical settlement; and/or
- The soils surrounding key utilities can lose their strength and those key utilities can become damaged or severed.

Methods to reduce the liquefaction potential and its related effects on the project are discussed in Section 4.2 of this report.

3.4 LANDSLIDES

The site is located in a relatively flat area with only isolated steep slopes occurring along the banks of the Sacramento River and the local log ponds. Due to the low height of the steep banks, their distance from any proposed improvement, and the lack of any significant instability along the banks, it is CGi's opinion that the potential for landslides to adversely affect the project is low.

Man-induced slope failures might occur within temporary excavations made during the course of construction of the project. Measures to reduce the risk of trench and temporary slope failures are discussed in greater detail in the Temporary Excavations section of this report.

3.5 EXPANSIVE SOILS

The site is primarily underlain by coarse alluvial materials with very little fines. Moreover, with a plasticity index of about 4, the fines that are present beneath the site at shallow depths have a low expansion potential (Day, 1999). Consequently, expansive soil conditions are unlikely to impact the proposed facility.

3.6 SOIL COROSIVITY

Selected samples of the near-surface soils encountered at the site were subjected to chemical analysis for the purpose of assessment of corrosion and reactivity with concrete. The samples were tested for soluble sulfates and soluble chlorides by Basic Laboratory of Redding and the results are presented below.

Boring No.	**Sample Depth**	**Chloride (ppm)**	**Sulfates (ppm)**
DH-4	7-10'	5.09	24.0
TP-1	2-14'	12.6	258

According to Table 19-A-4 of the 1997 Uniform Building Code and Table 2-2 of Portland Cement Associations Design and Control of Concrete Mixtures (PCA, 1988), the sulfate levels are below thresholds that require corrosion resistant (Type V) cement types. In addition, other tested constituents appear to pose a low relative corrosive potential to concrete. Thus, Type II cement can be utilized for this project.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL

Based on the results of our investigation, it is our opinion that the site may be developed for the proposed improvements utilizing a combination of both shallow and deep foundation construction techniques. The site is impacted by shallow groundwater, and by potentially compressible and liquefiable fill material. Possible mitigation measures for those adverse conditions are discussed below.

Specific comments and recommendations regarding the geotechnical aspects of project design and construction are presented in the following sections of this report. Geologic hazards and seismic design recommendations are discussed in previous sections.

Recommendations presented herein are based on our understanding of the proposed improvements. If the improvements significantly vary from those discussed in Section 1.0, then the conclusions and recommendations presented in this report may need to be supplemented.

4.2 LIQUEFIABLE AND COMPRESSIBLE FILL

Liquefaction analyses indicate that the low-density, organic rich fill material within the upper 5 to 15 feet across the site has the potential to liquefy when subjected to regional earthquakes. Moreover, due to the high organic content, the same fill material has the possibility to settle over time, regardless of seismic shaking. If not mitigated, the results can include damage to the proposed structure foundations and improvements.

To reduce the adverse effects of liquefaction and compressible fill material on the proposed facility, we recommend that the fill material be over excavated and replaced with compacted engineered fill. Because of the close proximity of some improvements (particularly the proposed fuel shed) to existing structures that are founded on artificial fill, over excavation cannot be performed without undercutting existing structures. As a result, where there is a conflict with an existing structure, or if it is unfeasible to over excavate and replace due to encountering excessively thick zones of artificial fill, we recommend deep foundations be used to support the proposed improvements.

Recommendations for over excavation, backfilling with engineered fill, and deep foundations are provided in subsequent sections.

4.3 SITE GRADING

Preparation of the exposed subgrade and requirements for engineered fill should be in accordance with recommendations provided below in the Scarification and Compaction, and Engineered Fill sections.

4.3.1 Existing Utilities, Wells, and/or Foundations

Due to the long history of the site as an active log processing facility, the presence of underground utilities and other improvements is likely. Thus, if any below-grade utility lines, septic tanks, cesspools, wells, on-site waste disposal fields and tanks, irrigation ponds and/or foundations are encountered within the area of construction, they should be removed and disposed of off-site.

Water wells should be destroyed in accordance with applicable regulatory agency requirements. Buried tanks should be removed in compliance with applicable regulatory agency requirements. Existing, below-grade utility pipelines that extend beyond the limits of the proposed construction and will be abandoned in-place should be plugged with lean concrete or grout to prevent migration of soil and/or water. All excavations resulting from removal and demolition activities should be cleaned of loose or disturbed material prior to placing any fill or backfill.

4.3.2 Over excavation

Artificial fill materials were encountered during exploration for this project, as discussed in Section 2.2.2. It is recommended that all artificial fill materials be removed from the area beneath and at least 15 feet horizontally outside of the footprint of all structures within the improvement area. However, no excavation should be performed adjacent to existing structures or improvements inside an imaginary line starting 5 feet set back from the bottom of the structure foundation and projecting down and away from the structure at a 45-degree angle. The depth of over excavation should be approved by a representative of CGI and stop in dense native sandy gravels of the Modesto Formation.

4.3.3 Scarification & Compaction

Following any required over excavation, all areas to receive engineered fill should be scarified to a depth of 8 inches, uniformly moisture-conditioned to near optimum moisture content, and compacted to at least 90 percent of the maximum dry density as determined using standard test method ASTM D1557[1]. Oversize cobbles and boulders (particles having a maximum dimension of greater than 6 inches) encountered during scarification and preparation of subgrade should be removed prior to placement of fill materials unless those materials can be mechanically reduced to dimensions smaller than 6 inches.

4.3.4 Wet/Unstable Soil Conditions

Groundwater in this area is relatively shallow and at various times of the year could be near the ground surface. Some of the soils encountered at this site may be sensitive to disturbances caused by construction traffic and to changes in moisture content.

If site preparation or grading is performed in the winter or spring season, or shortly after significant precipitation, near-surface on-site soils may be significantly over optimum moisture content. In addition, if grading is performed in areas where near surface soils have been removed and groundwater is near the surface, the soils may be significantly over optimum moisture content. These conditions could hinder equipment as well as efforts to compact site soils to a specified level of compaction. If over optimum soil moisture content conditions are encountered during construction, disking to aerate, replacement with imported material, chemical treatment, stabilization with a geotextile fabric or grid, and/or other methods will likely be required to facilitate earthwork operations. The applicable method of stabilization is the contractor's responsibility and will depend on the contractor's capabilities and experience, as well as other project-related factors beyond the scope of this study. Therefore, if over-optimum soil conditions are encountered during construction, CGi should review these conditions (as well as the contractor's capabilities) and, if requested, provide suggestions for their treatment.

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[1] This test procedure applies wherever relative compaction, maximum dry density, or optimum moisture content is referenced within this report.

4.4 DEWATERING

Shallow groundwater is present across the site and will likely need to be addressed during over excavation and backfill placement, particularly at depths below about 10 feet. The method of dewatering is at the discretion of the contractor. However, possible methods may include installing perimeter slit trenches to channel groundwater to flow towards collection points so it can be pumped out. Shoring may be necessary to hold the slit trench open. Alternatively, slotted pipes could be buried vertically across the site and fitted with sump pumps to lower the groundwater table. All groundwater should be discharged into one of the adjacent log ponds or in an approved area located outside the construction zone. Upon request, CGi can perform in-place hydrologic tests prior to construction to assess the subsurface hydrologic conditions and develop a detailed dewatering plan.

4.5 TEMPORARY EXCAVATIONS

4.5.1 General

All temporary excavations must comply with applicable local, state, and federal safety regulations, including the current OSHA Excavation and Trench Safety Standards. Construction site safety generally is the responsibility of the contractor, who should be solely responsible for the means, methods, and sequencing of construction operations so that a safe working environment is maintained.

4.5.2 Construction Considerations

Heavy construction equipment, building materials, excavated soil, and vehicular traffic should not be allowed within a 1:1 (horizontal to vertical) projection from the toe of an excavation to the ground surface. Where the stability is endangered by excavation operations, support systems such as shoring, bracing, or underpinning may be required to provide structural stability and to protect personnel working within the excavation.

During wet weather, earthen berms or other methods should be used to prevent runoff and/or flowing water from entering excavations. All water entering the excavation(s) should be collected and disposed of outside the construction limits.

4.6 ENGINEERED FILL

4.6.1 On-Site Soil Materials

It is our opinion that most of the near-surface <u>soils</u> encountered during exploration can be used for engineered fill. All low-density, organic rich material and highly plastic clayey should be segregated and excluded from engineered fill.

All on-site material proposed to be used as fill should be analyzed and approved by CGi prior to placement.

4.6.2 Imported Fill Materials - General

If imported fill materials are used for this project, they should consist of soil and/or soil-aggregate mixtures generally less than 3 inches in maximum dimension, nearly-free of organic or other deleterious debris, and essentially non-plastic. Typically, well-graded mixtures of gravel, sand, non-plastic silt, and <u>small</u> quantities of clay are acceptable for use as imported engineered fill within foundation areas. Imported fill materials should be sampled and tested prior to importation to the project site to verify that those materials meet recommended material criteria noted below. Specific requirements for imported fill materials, as well as applicable test procedures to verify material suitability are as follows:

ENGINEERED FILL - GENERAL

Gradation		Test Procedures	
Sieve Size	***Percent Passing***	***ASTM***	***AASHTO***
3-inch	100	D 422	T 88
¾-inch	70 - 100	D 422	T 88
No. 200	5 - 20	D 422	T 88
Plasticity			
Liquid Limit	***Plasticity Index***		
Less than 30	Less than 12	D 4318	T 89, T 90
Organic Content			
Less than 3%		D 2974	
Maximum Dry Density			
Greater than 125 pcf		D 1557	T 180

4.6.3 Materials - Granular

All granular fill should consist of imported soil mixtures generally less than 4 inches in maximum dimension, nearly-free of organic or other deleterious debris, and essentially non-plastic. Specific requirements for granular fill, as well as applicable test procedures to verify material suitability are as follows:

ENGINEERED FILL - GRANULAR

Gradation		Test Procedures	
Sieve Size	***Percent Passing***	***ASTM***	***AASHTO***
3-inch	100	D 422	T 88
¾-inch	70 – 100	D 422	T 88
No. 200	<5	D 422	T 88
Plasticity Index			
Nonplastic		D 4318	T 89, T 90
Organic Content			
Less than 3%		D 2974	
Maximum Dry Density			
Greater than 110 pcf		D 1557	T 180

4.6.4 Placement & Compaction

Soil and/or soil-aggregate mixtures used for fill beneath the proposed structures should be uniformly moisture-conditioned to within 3 percent of optimum moisture content, placed in horizontal lifts less than 8 inches in loose thickness, and compacted to at least 90 percent relative compaction. Testing should be performed to verify that the relative compactions are being obtained as recommended herein. Compaction testing, at a minimum, should consist of one test per every 500 cubic yards of soil being placed or at every 1.5-foot vertical fill interval, whichever comes first.

In general, a "sheep's foot" or "wedge foot" compactor should be used to compact fine-grained fill materials. A vibrating smooth drum roller could be used to compact granular fill materials and final fill surfaces.

4.6.5 Site Drainage

Finished grading should be performed in such a manner that provides positive surface gradients away from all structures and slopes. The ponding of water should not be allowed adjacent to the proposed structures. Surface runoff should be directed toward engineered collection systems or suitable discharge areas.

4.7 UTILITY TRENCHS AND TRENCH BACKFILL

4.7.1 Trenches & Dewatering

Utility trenches greater than 5 feet deep should be braced or shored in accordance with good construction practices and all applicable safety ordinances. In general, soils having a tendency to run or flow were observed during our study and shallow unshored trenches excavated with sidewalls steeper than 1:1 could locally cave. The actual construction of the trench walls and worker safety is the sole responsibility of the contractor.

Heavy construction equipment, building materials, excavated soil, and vehicular traffic should not be allowed within a 1:1 (horizontal to vertical) projection from the toe of the trench excavation to the ground surface. Where the stability of project improvements is endangered by excavation operations, support systems such as shoring, bracing, or underpinning may be required to provide structural stability and to protect personnel working within the excavation.

Perched groundwater is likely to be encountered within the depths of typical trench excavations and could enter utility trenches excavated for this project. If groundwater is encountered during construction, it is recommended that the contractor install measures to capture and/or divert groundwater from entering the excavation.

4.7.2 Materials

Pipe zone backfill (i.e., material placed from the trench bottom to a minimum of 6 inches over the pipeline crown) should consist of imported soil having a Sand Equivalent (SE) of no less than 30 and having a particle size no greater than ½-inch in maximum dimension. On site soils will likely not meet this recommendation. Trench zone backfill (i.e., material placed between the pipe zone backfill and finished subgrade) may consist of on-site soil that meets the material requirements previously provided for engineered fill with 100% passing the ¾-inch sieve.

If imported material is used for pipe or trench zone backfill, we recommend it consist of fine-grained sand. In general, use of coarse-grained sand and/or gravel is not recommended due to the potential for soil migration into and water seepage along trenches backfilled with this type of material.

Recommendations provided above for pipe zone backfill are minimum requirements only. More stringent material specifications may be required to fulfill local codes and/or bedding requirements for specific types of pipe. We recommend the project Civil Engineer develop these material specifications based on planned pipe types, bedding conditions, and other factors beyond the scope of this study.

4.7.3 Placement & Compaction

Trench backfill should be placed and compacted in accordance with recommendations previously provided for engineered fill. Mechanical compaction is strongly recommended; ponding or jetting should not be allowed unless specifically reviewed and approved by CGi prior to construction. It should be noted that in the rare instances that ponding or jetting are allowed, the pipe zone backfill materials should have an SE of 50 or greater and should be less than ½-inch in maximum dimension. In addition, a number of additional conditions for collection and removal of excess ponded or jetted water will be required if ponding or jetting are utilized. Special care should be given to ensuring that adequate compaction is made beneath the haunches of the pipeline (that area from the pipe springline to the pipe invert) and that no voids remain in this space.

4.7.4 Trench Subgrade Stabilization

Soft and yielding trench subgrade could be encountered along the bottom of trench excavations, particularly in soils with a high organics content. It is recommended that the bottom of trenches be stabilized prior to placement of the pipeline bedding so that, in the judgment of the geotechnical engineer, the trench subgrade is firm and unyielding. The Contractor should have the sole responsibility for design and implementation of trench subgrade stabilization techniques. Some methods that we have observed used to stabilize trench subgrades include the following:

- Use of ¾–inch to 1½-inch floatrock worked into the trench bottom and covered with a geotextile fabric such as Mirafi 500X;
- Placement of a geotextile fabric, such as Mirafi 500X, on the trench bottom and covered with at least one foot of compacted processed miscellaneous base (PMB) conforming to the requirements of Section 200-2.5 of the Greenbook, latest edition;
- Overexcavation of trench subgrade and placement of two-sack sand-cement slurry; and
- In extreme conditions, injection grouting along the trench alignment.

If floatrock is used, typically sand with an SE of 50 or more should be used to fill the voids in the rock prior to placement of pipe bedding materials.

4.8 SHALLOW FOUNDATIONS

4.8.1 Minimum Footing Embedment and Dimensions

We recommend that shallow isolated and continuous wall footings be founded entirely on engineering fill. Minimum embedment depths, widths, and thicknesses should conform to Table 18-I-C of the CBC, but should be determined by the Structural Engineer and should be no less than 12 inches wide and 12 inches deep. The footing thickness should be determined by the Structural Engineer.

It should be noted that frost heave is not typically a hazard in the area and is generally not considered in design of foundation systems. Therefore, no recommendations for frost protection have been provided herein.

4.8.2 Allowable Bearing Pressures

All shallow structure foundations for the proposed facility must rest on compacted engineered fill. Isolated and continuous footing elements should be proportioned for dead loads plus probable maximum live loads assuming a maximum allowable bearing pressure of 2,500 pounds per square foot (psf). The allowable bearing pressure can be increased by 400 psf for each additional foot of footing depth above the minimum recommended. An increase of 250 psf can be added to the allowable bearing pressure for each additional one-foot increase in footing width. However, the maximum allowable bearing pressure should not exceed 4,000 psf.

The allowable bearing pressure provided is a net value. Therefore, the weight of the foundation (which extends below finished subgrade) may be neglected when computing dead loads. The allowable bearing pressure applies to dead plus live loads, includes a calculated factor of safety of at least 3, and may be increased by 1/3 for short-term loading due to wind or seismic forces. The allowable bearing value is for vertical loads only; eccentric loads may require adjustment to the values recommended above.

4.8.3 Minimum Footing Reinforcement

Footing reinforcement should be designed by a Structural Engineer and should conform to pertinent structural code requirements. Minimum footing reinforcement should not be less than that required for shrinkage, temperature control, and structural integrity, but should consist of at least four No. 4 bars with two placed at the top and two placed at the bottom of the footing.

4.8.4 Estimated Settlements

The anticipated total settlement for the proposed structure foundations, if construction occurs as recommended within this report, should be less than one inch. Differential settlement for the structure foundations resting on engineered fill is anticipated to be less than 1/2-inch in 20 feet. Due to the granular nature of the subsurface materials and the engineered backfill, settlement is anticipated to occur soon (if not immediately) after the load of the proposed structures is applied.

4.8.5 Construction Considerations

Prior to placing steel or concrete, foundation excavations should be cleaned of all debris, loose or disturbed soil, and any water. A representative of CGi should observe all foundation excavations prior to concrete placement.

4.9 SLIDING AND PASSIVE RESISTANCE

4.9.1 Sliding Resistance

Ultimate sliding resistance generated through a compacted soil/concrete interface can be computed by multiplying the total dead weight structural loads by the friction coefficient of 0.35 for imported granular engineered fill.

4.9.2 Passive Resistance

Ultimate passive resistance developed from lateral bearing of shallow foundation elements bearing against compacted engineered fill surfaces for that portion of the foundation element extending below a depth of 1 foot below the lowest adjacent grade can be estimated using an equivalent fluid weight of 300 pcf.

4.9.3 Safety Factors

Sliding resistance and passive pressure may be used together without reduction in conjunction with recommended safety factors outlined below. A minimum factor of safety of 2 is recommended for foundation sliding, where sliding resistance and passive pressure are used together. The safety factor for sliding can be reduced to 1.5 if passive pressure is neglected.

4.10 INTERIOR CONCRETE FLOOR SLABS SUPPORTED ON-GRADE

4.10.1 General

All ground-supported slabs should be designed by a Civil Engineer to support the anticipated loading conditions but as a minimum should be at least 4 inches thick. Reinforcement for floor slabs should be designed by a Civil Engineer to maintain structural integrity, and should not be less than that required to meet pertinent code, shrinkage, and temperature requirements. Reinforcement should be placed at mid-thickness in the slab with provisions to ensure it stays in that position during construction and concrete placement.

The mat can be designed using a flat slab on an elastic half-space analog. A modulus of subgrade reaction (k_{s1}) of 50 kcf is recommended for design of mat-type foundations. That modulus of subgrade reaction value represents a presumptive value based on soil classification. No plate-load tests were performed as part of this study. The modulus value is for a 1-foot-square plate and must be corrected for mat size and shape, assuming a cohesionless subgrade.

4.10.2 Subgrade Preparation

Subgrade soils supporting interior concrete floor slabs should be scarified to a minimum depth of 8 inches, uniformly moisture-conditioned to near the optimum moisture content, and compacted to at least 95 percent relative compaction.

4.10.3 Rock Capillary Break/Vapor Barrier

Interior concrete floor slabs supported-on-grade should be underlain by a capillary break consisting of a blanket of compacted, free-draining, durable rock at least 4 inches thick, graded such that 100 percent passes the 1-inch sieve and less than 5 percent passes the No. 4 sieve[2]. Furthermore, a vapor barrier should be placed beneath all interior concrete floor slabs supported-on-grade that will be covered with moisture-sensitive floor coverings. This barrier may consist of a plastic or vinyl membrane placed directly over the rock capillary break. The vapor barrier should be sealed around all penetrations, including utilities. If a vapor barrier is not installed, there is a risk of moisture vapors and salts penetrating the slab-on-grade. To promote more uniform curing of the slab and provide protection of the membrane during construction, a 1- to 2-inch thick sand blanket should be placed on top of the membrane prior to placing slab concrete.

Sand placed above the membrane may be moistened just prior to concrete placement to aid in curing. Concrete should not be placed if sand overlying the vapor barrier has been allowed to become saturated (due to precipitation or excessive moistening) or if standing water is present above the membrane. Excessive water beneath interior floor slabs could result in significant vapor transmission through the slab, adversely affecting moisture-sensitive floor coverings. A capillary break and/or vapor barrier may not be

[2] In general, Caltrans Class 2 aggregate base (or similar material) does not meet the requirements provided above for a capillary break. Therefore, we recommend this material <u>not</u> be used for a capillary break beneath interior concrete slabs supported-on-grade.

required for some types of construction (such as equipment buildings, warehouses, garages, and other uninhabited structures insensitive to water intrusion and/or vapor transmission through the slab). For these types of structures, the gravel capillary break and/or vapor barrier recommended above may be omitted and the slab placed directly on the prepared subgrade or other approved surface. In the event a capillary break and/or vapor barrier is not to be used, CGi should review the planned structure in order to assess the applicability of the approach and provide (if necessary) additional recommendations regarding subgrade preparation and/or support.

4.11 EXTERIOR CONCRETE SLABS SUPPORTED-ON-GRADE

Subgrade soils supporting exterior concrete slabs[3] should be scarified to a minimum depth of 8 inches, uniformly moisture-conditioned to near the optimum moisture content and compacted to at least 95 percent relative compaction. In the event the exposed subgrade is dense and uniformly compacted, scarification and compaction may be omitted if approved by CGi during construction.

4.12 RETAINING WALLS

4.12.1 Lateral Earth Pressures

If retaining walls are utilized in this project, they should be designed to resist earth pressures exerted by the retained, compacted backfill plus any additional lateral force that will be applied to the wall due to surface loads placed at or near the wall. The recommended equivalent fluid weights presented below are for static conditions of compacted engineered fill with the backfill materials placed level behind walls.

Lateral Earth Pressures Under Static Conditions

Lateral Earth Pressure Condition	Equivalent Fluid Weight (pcf)
At-Rest	60
Active	40

The resultant force of the static lateral force prism should be applied at a distance of 30 percent of the wall height above the bottom of the foundation on the back of the wall.

The tabulated values are based on a soil unit weight of 130 pounds per cubic foot (pcf), and do not provide for surcharge conditions resulting from foundations, vehicle traffic, or compaction equipment. The drained values do not provide for hydrostatic forces (for example, standing water in the backfill materials). Foundation loads not considered as surcharges should bear behind a 1:1 (horizontal to vertical) line projected upward from the base of the wall. If conditions such as surcharge resulting from footings or hydrostatic forces are expected, CGi should be advised so that we can provide additional recommendations as needed.

Surcharge loads induce additional pressures on earth retaining structures. An additional lateral load on non-yielding walls equal to 0.5 times the applied surcharge pressure should be included in the design for uniform area surcharge pressures. Lateral pressures for other surcharge loading conditions can be provided, if required.

3 Within this report, exterior concrete slabs supported-on-grade refers to walkways, driveways, patios, etc. and specifically excludes roadway pavements.

4.12.2 Drainage Measures

Drainage measures should be constructed behind the proposed retaining walls to reduce the potential for groundwater accumulation. To help reduce the potential for the buildup of hydrostatic forces behind walls, a granular free-draining backfill, at least 2 feet thick, should be placed behind the wall. The free-draining backfill should consist of clean, coarse-grained material with no more than 5 percent passing the No. 200 sieve. Acceptable backfill would be:

- Pervious Backfill conforming to Item 300-3.5.2 of the *Standard Specifications for Public Works Construction* (Greenbook), most current edition;
- Permeable Material (Class 2) conforming to Item 68-1.025 if the *Caltrans Standard Specifications*, most current edition;
- Pea gravel having a nominal diameter or ¼-inch; or
- Crushed stone sized between ¼-inch and ½-inch.

In lieu of free-draining backfill materials of the types suggested above, manufactured (geosynthetic) drainage systems (for example MiraDrain manufactured by TC Mirafi, Inc., or equivalent) can be used against retaining or below-grade walls. Manufacturer recommendations for the installation and maintenance of these products should generally be followed, although they should be reviewed by CGi for approval. In addition, manufactured drainage systems should be attached to the retaining wall face as opposed to the excavated slope face. This implies that provisions to protect the integrity of the drainage panels will need to be made while fill materials are placed behind the walls.

A perforated drainpipe system should be installed at the base of the wall to collect water from the free-draining material and/or geosynthetic drainage system. The drainpipe system should allow gravity drainage of the collected water away from the buried wall or, as a less preferred option, should be tied into a sump and pump system to remove the water to an acceptable outlet facility.

Finish surface grades should be sloped away from the retaining walls and designed to channel water to an acceptable collection and offsite disposal system. Provisions should be included for removal of surface runoff that may tend to collect behind the backs of walls and for drainage of water away from the fronts of walls. Also, provisions should be included to mitigate the infiltration of surface water into the below-ground, free-draining backfill/geosynthetic drainage system by placing a minimum of 18-inches of low permeability compacted soil over the top of those materials.

4.12.3 Dynamic Earth Pressures

For unrestrained walls, the increase in lateral earth pressure acting on the wall resulting from earthquake loading can be estimated using the approach of Seed and Whitman (1970). That theory is based on the assumption that sufficient wall movement occurs during seismic shaking to allow active earth pressure conditions to develop. For restrained walls, the increase in lateral earth pressure resulting from earthquake loading also can be estimated using these relations. Because that theory is based on the assumption that sufficient movement occurs so that active earth pressure conditions develop during seismic shaking, the applicability of the theory to restrained or basement walls is not direct; however, there have been studies (Nadim and Whitman, 1992) that suggest the theory can be used for such walls.

In the Seed and Whitman (1970) approach, the total dynamic pressure can be divided into static and dynamic components. The estimated dynamic lateral force increase (based on seismic loading conditions) for either unrestrained or restrained walls, could be taken as the following:

$$P_E=3/8*pga*\gamma_t*H^2$$

Where:

P_E	=	Seismically-induced horizontal force (lbs per lineal foot of wall)
Pga	=	Peak Ground Acceleration (g)
γ_t	=	Total unit weight of backfill (pcf)
H	=	Height of the wall below the ground surface (ft)

We recommend that a *pga* value of 0.25 be applied for this project. The centroid of the dynamic lateral force increment should be applied at a distance of 0.6*H above the base of the wall.

To estimate the total dynamic lateral force, the dynamic lateral force increase should be added to the static earth pressure force computed using recommendations for active lateral earth pressures presented above. That recommendation is based on the concept that during shaking, earth pressures recommended for permanent conditions will be reduced to those more closely approximating active conditions.

4.12.4 Compaction Adjacent to Walls

Backfill within 5 feet, measured horizontally, behind retaining walls should be compacted with relatively lightweight, hand-operated compaction equipment to reduce the potential for creation of relatively large compaction-induced stresses. If large or heavy compaction equipment is used, compaction-induced stresses could result in increased lateral earth pressures on the retaining walls in addition to those presented in this report.

Backfill material should be brought up uniformly behind retaining walls (in other words, the backfill should be at about the same elevation behind the retaining wall as the backfill is placed and compacted). The elevation difference of the backfill surface behind the wall should not be greater than about 2 feet, unless the walls are designed for those differences.

4.13 DEEP FOUNDATIONS

As discussed, it is anticipated that structural and economic considerations preclude the use of shallow spread foundations for support in areas of excessively thick fill and/or adjacent to existing structures. Thus, it is anticipated that deep foundations will be required to support portions of the proposed fuel shed and, possibly, the proposed cooling towers. The anticipated vertical loads provided for these structures range between 25 to 50 kips.

A preliminary cost comparison analysis was performed to determine the type of deep foundation system that would best fit the site conditions. Results of this analysis suggest that driven H-piles will be the most economical and will provide the most benefit, namely: 1) they can be driven through dense gravels and cobbles present below the site, 2) they can be easily installed with a batter to resist lateral loads and uplift forces, and 3) they can be easily spliced in the field. The main drawback for H-piles at the site, however, is that ground disturbance and ground vibrations during construction may damage adjacent structures. A solution to this problem could be a deep foundation system composed of cast-in-drill-hole (CIDH) piers. The primary advantage of the CIDH piers is the limited ground disturbance and vibration during construction that could damage adjacent buildings. However, the draw backs of CIDH piers at the site are far greater than H-piles, and include: 1) higher anticipated drilling costs due the presence of dense, cobble rich gravels below the site, 2) the presence of sloughing soils that may require casing to be advances or heavy drilling fluids to be used, and 3) the added challenges of the piers to be installed with a batter to assist in resisting lateral loads. For these reasons, the following deep-foundation analysis was performed using H-piles. If desired, however, CIDH piers or other pile types, such as pipe piles, can also be assessed upon request.

Subsurface soils information were obtained using the standard penetrating test (SPT) to a depth of about 22 feet below existing ground, where refusal of the drill stem was encountered in a very dense, weakly cemented, clayey gravel. Rather than mobilizing a second drill rig capable of obtaining deeper depths (such as a mud-rotary drill rig), the client requested that subsurface data below 22-feet be extrapolated from adjacent borehole data. The added uncertainty and risk associated with performing this type of extrapolation was discussed with the client and acknowledged.

4.13.1 Vertical Capacity

We have recommended specified shaft lengths on the basis of estimated frictional and end resistances for HP 14x73 piles driven into dense Modesto and Riverbank (?) Formation materials. In CGi's opinion, HP piles can be driven into the underlying soils; however, gravel, cobble, and boulder zones could pose difficult pile driving conditions.

The capacity of HP piles was estimated on the basis of frictional resistance and limited end bearing following the FHWA method. The actual analyses were facilitated by use of the computer program APILE, Version 4.0 (Ensoft, 2004). That program is specifically used for driven piles and incorporates commonly-accepted design procedures for multiple types of geo-materials, such as the sand and gravels beneath the site. The capacities of the piles, and, thus, the recommended lengths for specific loading conditions, were estimated assuming that: 1) the upper 5 to 15 feet of unclassified fill provides no resistance and is neglected; 2) the recommended supporting pile lengths are entirely within dense materials; 3) end bearing without plugging of the H-pile web is present; and 4) good construction practices are used. It is expected that HP piles constructed to the specified lengths (presented below) will provide an allowable axial load capacity of 30 tons (60 kips), which roughly equates to the anticipated loads required for the project. The allowable pile capacity was calculated using a factor of safety of 3 for static loading conditions. A factor of safety of 2 can be applied provided load tests are performed.

CGI's recommended shaft lengths are presented below based on the minimum and maximum anticipated fill at the surface.

HP 14x73 Pile Design Recommendations			
Minimum and maximum Depth of Unclassified Fill on Surface	**Compressive Pile Capacity (Tons)**		**Minimum Specified Length (feet)**
	Allowable	**Ultimate**	
5 feet	30	90	23
15 feet	30	90	30

Note that for the above stated compressive capacities, the recommended supporting pile lengths are from the surface down and discount the thickness of underlying fill.

4.13.2 Negative Skin Friction

Negative skin friction or downdrag may occur when sediments located directly adjacent to piles move downward (i.e., compress) and induce additional forces on the pile. The downward movement of sediments usually is the result of additional fill materials being placed above compressible, fine-grained sediments that are located either below the tip or along the sides of a pile. More specifically, at the project site, most of the encountered sediments below the compressible fill consist of dense to very dense granual material with an anticipated low compressibility. It is CGi's opinion that these materials have a very low potential for inducing negative skin friction on installed piles.

There is a possibility, however, that the unclassified fill overly the dense granular soils may move downward and could possibly induce some negative skin friction on piles installed through those materials. It is recommended that the potential for negative skin friction within those materials be compensated by adding an extra 2 feet to the recommended pile lengths. This added pile length is already included in the above minimum specified lengths under vertical capacity.

Alternatively, the piles could be driven in holes drilled through the fill. The hole should have a diameter of not less than the greatest dimension of the pile cross section plus 6 inches. The holes should be cased or, after driving the pile, the space around the pile should be filled to the ground surface with dry sand or pea gravel. Load tests may also be applied to assess the load caring capacity of the piles to insure that sufficient capacity is available to counteract the effects of negative skin friction.

4.13.3 Uplift Capacity

The uplift capacity of H-piles is a function of the frictional force on the perimeter of the pile plus the pile weight. The following table provides both ultimate and allowable uplift pile capacities for the two extreme conditions based on the anticipated fill thickness. These values ignore all frictional forces within the upper 5 to 15 feet, depending on fill thickness, and are based on the following: 1) the uplift frictional capacity of the pile is approximately 0.75 that of the downward loaded capacity (Coduto, 2001); and 2) a factor of safety of 3 is applied to obtain allowable conditions. Note a factor of safety of 2 can be applied provided load tests are performed.

HP 14x73 Pile Design Recommendations			
Minimum and maximum Depth of Unclassified Fill on Surface	**Uplift Pile Capacity (Tons)**		**Minimum Specified Length (feet)**
	Allowable	**Ultimate**	
5 feet	11	32	23
15 feet	12.3	35	30

4.13.4 Lateral Deflection

Lateral movement of the piles due to wind and seismic forces can be substantial and could affect the integrity of the structures. Lateral deflection is a function of several factors including the lateral and vertical forces on the piles, pile batter, and the type of pile cap design, all of which are unknown at this time.

As a preliminary estimate of lateral deflection of a single, vertical pile under the worst anticipated conditions, which include the pile being loaded to the maximum allowable vertical capacity of 60 kips and ignoring the upper 15 feet of low density fill, the maximum horizontal load applied to the top of the pile at ground surface is 2.7 kips to stay under 1-inch of lateral deflection.

Because of the uncertainties surrounding the type of loads and foundation conditions present, we recommend performing lateral deflection analyses on the proposed deep foundations once a working design has been developed and the boundary conditions of the loading and pile cap are better known.

4.13.5 Settlement

We estimate that settlement of piles under static loading conditions, when installed using good construction techniques, should not exceed 0.5-inch total and 0.25-inch differential between adjacent piles.

4.13.6 Spacing and Group Effects

The recommended shafts lengths (noted above for certain loads) are for piles that have a minimum center-to-center spacing of 3 feet. We note that actual spacing may be controlled by construction conditions and requirements to limit disturbance of adjacent piles.

The ultimate capacity of a group of piles can be estimated by multiplying the sum of the capacities of all the piles in the group by a group efficiency factor. The group efficiency factor is defined as the ratio of the ultimate load capacity of the group to the sum of the ultimate capacities of the individual piles. Group efficiency factor of 1.0 is recommended for center-to-center spacing of 3 feet or higher. We note that, because of the presence of over-sized materials (e.g., cobbles and boulders) beneath the project site, center-to-center spacing should be maximized to reduce the potential for disturbance to adjacent piles during driving.

4.14 PAVEMENT DESIGN

Due to the variable, near surface conditions across the site, structural pavement sections are provided in the following for only those areas where the compressible fill materials will be over excavated and replaced with engineered fill.

4.14.1 R-Values

The material to be placed as engineered fill in the areas that are to be over excavated was undetermined at the time this report was published. However, in order for the material to meet the specifications of engineered fill, we anticipate that the material will have a minimum R value of 55. Because the actual subgrade materials that will be present at finish subgrade are unknown at this time, we recommend that confirmatory R-value tests be obtained during construction. If the construction R-values are significantly different than the R-value reported above, then we can modify the pavement design at that time to reflect the constructed conditions.

4.14.2 Subgrade Preparation

All subgrade soils should be scarified to a minimum depth of 1-foot, moisture conditioned as necessary to near optimum moisture conditions and compacted to a minimum of 95 percent of the maximum dry density as determined by ASTM 1557 Test Method. The subgrade should be smooth and unyielding prior to the placement of aggregate base rock. Density testing and proof rolling of the subgrade using a loaded water truck should be performed with satisfactory results prior to placement of the aggregate base rock.

Concrete curbs, if applicable, that border pavement sections should be embedded into the subgrade soils a minimum of 2 inches to reduce the migration of meteoric and irrigation water into the pavement section.

4.14.3 Aggregate Base

The aggregate base materials (AB) should be of such quality as to meet or exceed Caltrans specifications for Class 2 AB and should have a minimum R-value of 78. The AB should be spread in thin lifts restricted to 8 inches in loose thickness or less, moisture conditioned as necessary to near optimum moisture content and compacted to a minimum of 95 percent of the maximum dry density as determined by AASHTO T-180. Density testing and proof rolling should be performed prior to placement of the asphalt paving.

4.14.4 Asphalt Concrete Paving

Assuming Traffic Indices ranging from 8 to 10 for the proposed project, we have used Caltrans pavement design standards to define the following structural pavement sections:

Structural Pavement Sections

Material Type	Traffic Index (TI)		
	8.0	9.0	10.0
Full Depth Asphalt (ft)	0.59	0.64	0.74
Asphaltic Concrete (ft)	0.39	0.44	0.49
Aggregate Base (ft)	0.34	0.39	0.54

Asphalt paving materials and equipment should meet or exceed current Caltrans specifications.

5.0 ADDITIONAL SERVICES

We recommend CGi review final grading and foundation plans, and specifications to evaluate that recommendations contained herein have been properly interpreted and implemented during design.

6.0 LIMITATIONS

This report has been prepared in substantial accordance with the generally accepted geotechnical engineering practice, as it existed in the site area at the time our services were rendered. No other warranty, either express or implied, is made. Conclusions and recommendations contained in this report were based on the conditions encountered during our field investigation and are applicable only to those project features described herein (see the Proposed Project section). It is possible subsurface conditions could vary between or beyond the points explored. If conditions are encountered during construction that differ from those described in this report, or if the scope or nature of the proposed construction changes, we should be notified immediately in order to review and, if deemed necessary, conduct additional studies and/or provide supplemental recommendations.

This report may be used only by our client and their agents and only for the purposes stated herein, within a reasonable time from its issuance. Land use, site conditions, and other factors may change over time that may require additional studies. In the event a significant period of time elapses between the issuance date of this report and construction, CGi shall be notified of such occurrence in order to review current conditions. Depending on that review, CGi may require that additional studies be conducted and that an updated or revised report is issued.

Any party other than our client who wishes to use all or any portion of this report shall notify CGi of such intended use. Based on the intended use as well as other site-related factors, CGi may require that additional studies be conducted and that an updated or revised report be issued. Failure to comply with any of the requirements outlined above by the client or any other party shall release CGi from any liability arising from the unauthorized use of this report.

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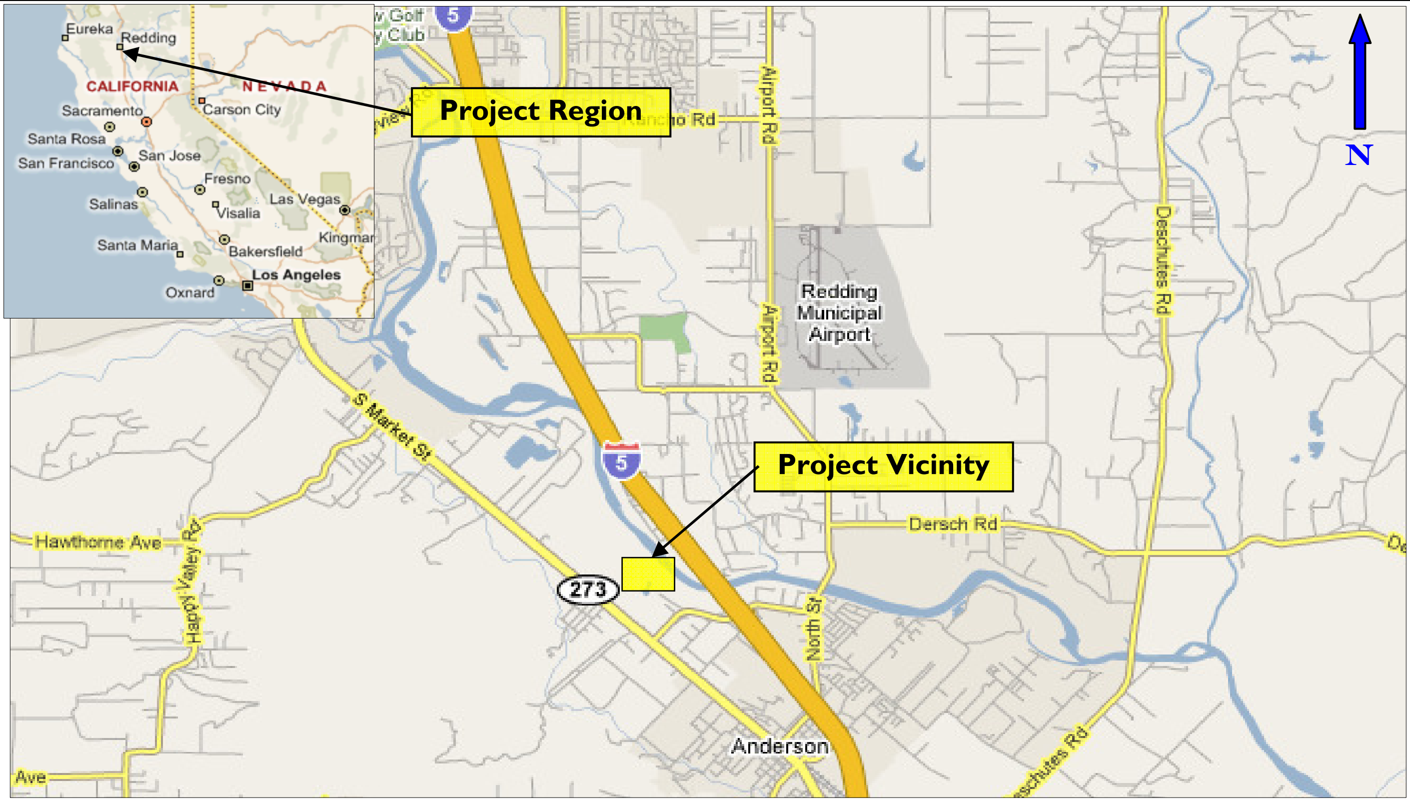

NOT TO SCALE

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CGI Technical Services Inc.	SITE LOCATION GEOTECHNICAL STUDY SPI COGENERATION FACILITY ANDERSON, CALIFORNIA	Plate 1
Project No. 07-1588.05		

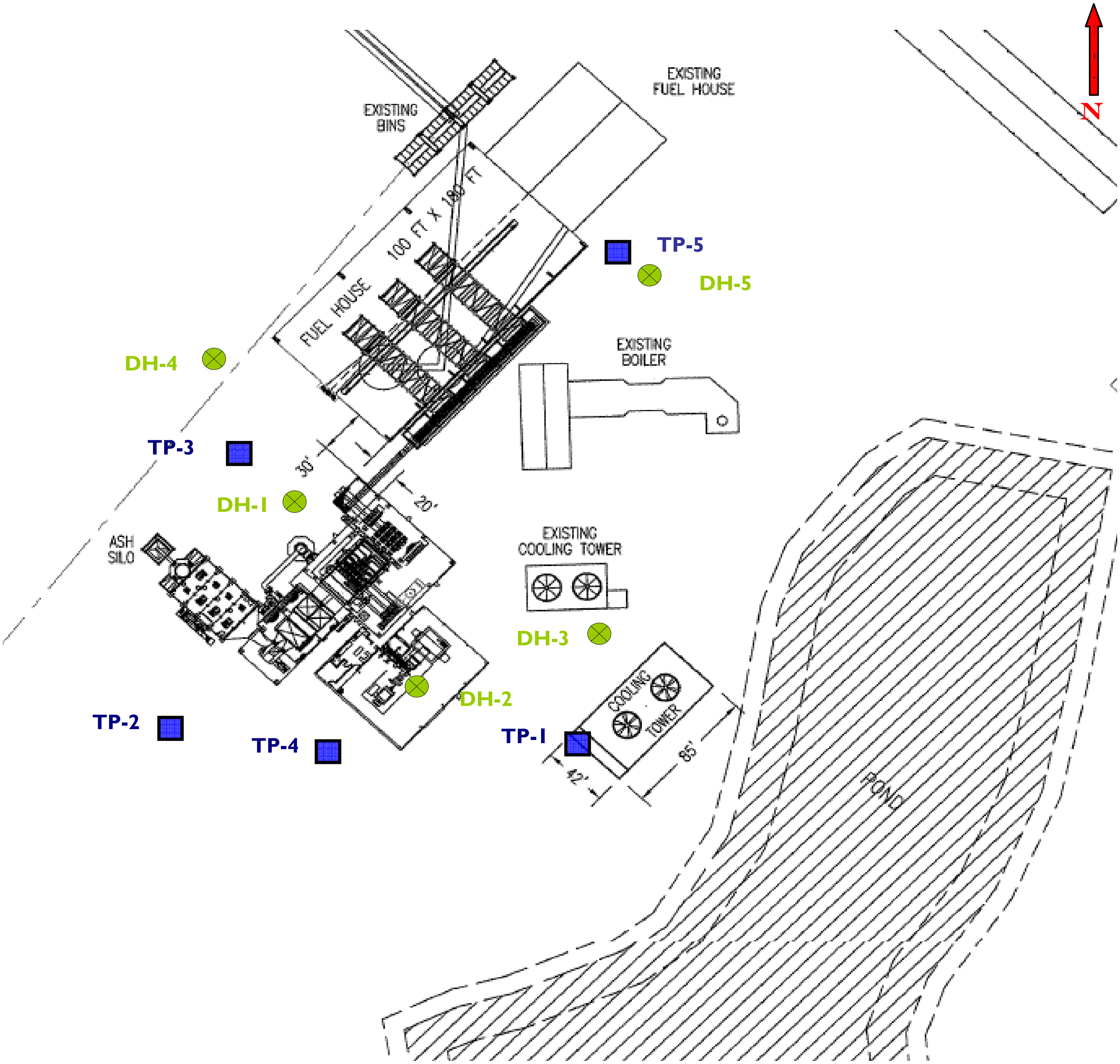


Image from Site Plan for Co-Gen Facility

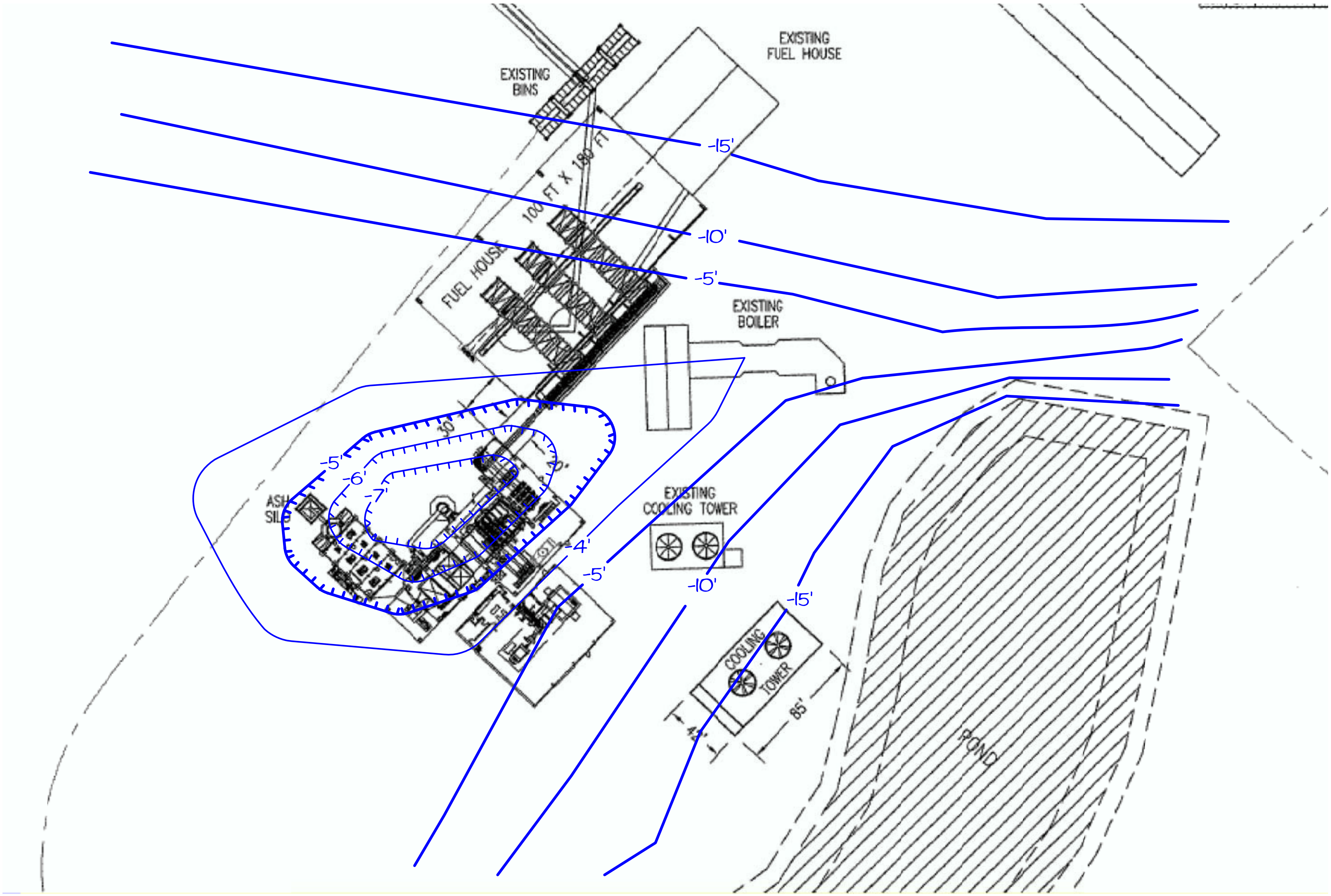
Legend

DH-5: Drill Hole Location

TP-5: Test Pit Location

NTS: Not To Scale

CGI Technical Services Inc.	SITE MAP, DRILL HOLE, & TEST PITS LOCATIONS GEOTECHNICAL STUDY SPI COGENERATION FACILITY ANDERSON, CALIFORNIA	Plate 2
Project No. 07-1588.05		



CGI TECHNICAL SERVICES INC.	EXTENT OF FILL DEPTHS (APPROXIMATE) GEOTECHNICAL STUDY SPI COGENERATION FACILITY ANDERSON, CALIFORNIA	Scale Not to Scale	Plate 3
Project No. 07-1588.05			

APPENDIX A
SUBSURFACE EXPLORATION

The subsurface exploration program for the proposed project consisted of the excavation and logging of five test pits and five drill holes. Locations of the explorations are shown on Plates 2 in the report text.

The test pits were excavated to a maximum depth of about 15 feet using a Hitachi UH 122 excavator equipped with a 4-foot wide bucket. The drill holes were advanced using a Mobil B-61 drill rig using hollow-stem auger methods. The drill rig was provided by PC Exploration. The drill holes were advanced to depths ranging from 20 to 22 feet below the existing ground surface. Bulk samples of the materials encountered were recovered from the test pits and drill holes for laboratory classification and testing. The results of the testing procedures are noted on the Logs of Borings and attached within Appendix B.

The exploration logs describe the earth materials encountered, sampling methods used, and laboratory tests performed. The logs also show the location, exploration number, date of exploration, and the names of the logger and equipment used. A CGi engineer or geologist, using ASTM 2488 for visual soil classification and Caltrans methods for logging rock materials, logged the explorations. The boundaries between soil and rock types shown on the logs are approximate because the transition between different soil and rock layers may be gradual and may change with time. Logs of drill holes advanced for this study are presented as Plated A-2.1 through A-3.5. Legends to the logs are noted on Plated A-1.1 and A-1.2

The test pits were backfilled with native, on-site material, and the drill holes were backfilled using bentonite chips and a concrete slurry mix. No efforts were made to densify the excavated soils within the test pits. As a result, they should be over excavated during construction and backfilled with compacted engineered fill.

MAJOR DIVISIONS			USCS SYM.	DESCRIPTION
COARSE-GRAINED SOILS MORE THAN 50% OF SAMPLE OR MATERIAL IS LARGER THAN THE #200 SIEVE (0.0029 INCHES)	**GRAVELS** MORE THAN 50% OF THE COARSE FRACTION IS RETAINED ON THE #4 SIEVE (0.1870 INCHES)	**GRAVELS** CLEAN GRAVELS WITH LITTLE OR NO FINES	**GW**	WELL GRADED GRAVELS AND SAND MIXTURES WITH LITTLE TO NO FINES
			GP	POORLY GRADED GRAVELS & GRAVEL/ SAND MIXTURES W/LITTLE TO NO FINES
		GRAVELS WITH FINES IN APPRECIABLE AMOUNTS	**GM**	SILTY GRAVELS AND POORLY GRADED GRAVEL/ SAND/ SILT MIXTURES
			GC	CLAYEY GRAVELS AND POORLY GRADED GRAVEL/ SAND/ CLAY MIXTURES
	SANDS MORE THAN 50% OF THE COARSE FRACTION PASSES THE #4 SIEVE (0.1870 INCHES)	**SANDS** CLEAN SANDS WITH LITTLE OR NO FINES	**SW**	WELL GRADED SANDS AND GRAVELLY SANDS WITH LITTLE TO NO FINES
			SP	POORLY GRADED SANDS AND GRAVELLY SANDS WITH LITTLE TO NO FINES
		SANDS WITH FINES IN APPRECIABLE AMOUNTS	**SM**	SILTY SANDS AND POORLY GRADED SAND/ GRAVEL/ SILT MIXTURES
			SC	CLAYEY SANDS AND POORLY GRADED SAND/ GRAVEL/ CLAY MIXTURES
FINE-GRAINED SOILS MORE THAN 50% OF SAMPLE OR MATERIAL IS SMALLER THAN THE #200 SIEVE (0.0029 INCHES)	**SILTS AND CLAYS** LIQUID LIMIT LESS THAN 50		**ML**	INORGANIC SILTS AND VERY FINE SANDS, SILTY AND/OR CLAYEY FINE SANDS, CLAYEY SILTS WITH SLIGHT PLASTICITY
			CL	INORGANIC CLAYS WITH LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	ORGANIC SILTS AND CLAYS WITH HIGH PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		**MH**	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDS OR SILTS
			CH	INORGANIC CLAYS WITH HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC SILTS AND CLAYS WITH HIGH PLASTICITY
HIGHLY ORGANIC SOILS			**PT**	PEAT, HUMUS, SWAMP SOIL WITH HIGH ORGANIC CONTENT

GENERAL NOTES

- Dual symbols (such as ML/CL or SM/SC) are used to indicate borderline soil classifications.
- In general, USCS designations shown on the logs were evaluated utilizing visual methods. Actual designations (based on laboratory tests) may vary.
- Logs represent general soil conditions observed on the date and locations indicated. No warranty is provided regarding soil continuity between locations.
- Lines separating soil strata's on the logs are approximate. Actual transitions may be gradual and vary in depth.

CGI TECHNICAL SERVICES INC.

Project No. 07-1588.05

UNIFIED SOIL CLASSIFICATION SYSTEM
GEOTECHNICAL STUDY
SPI COGENERATION FACILITY
ANDERSON, CALIFORNIA

Plate
A-1.1

SAMPLERS AND SAMPLE SYMBOLS

Symbol	Blow Count Representation	Samplers and Sample Types
	45	Split-Spoon Sampler (SPT): 1-3/8" ID, 2" OD, Driven
	(35)	California Modified S 2
	NA	Bulk Sample (bag, grab sample, etc.)
	NA	No Sample Recovery
	NA	Shelby Tube, 2-7/8" ID, 3" OD, Pushed

Blow counts are recorded as the number of blows required for one foot of sampler penetration using a 140-lb. hammer falling 30 inches. Typically, a sampler is driven 18" and the initial 6" of sample is discarded.

SYMBOLS SHOWN ON LOGS

Symbol	Represents
	Separation between geologic formations
	Separation between lithologic units within a geologic formation
	Initial water level measurement**
	Water level after initial measurement**

**(both measurements may not represent stabilized water levels)

LABORATORY TEST ABBREVIATIONS

DS - Direct Shear; Consol - Consolidation; GS - Grain-Size Distribution; EI - Expansion Index; UC - Unconfined Compression; TC - Triaxial Compression; SC - Soil-Chemistry; AL - Atterberg Limits; SE - Sand Equivalent; R - R-value; S - Swell; Proctor - Curve; PP - Pocket Penetrometer; TV - Torevane

NOTES FOR ALL LOGS

The data presented on the logs are a generalization of actual geologic conditions present at the site of the exploration at the time and location the exploration was performed. Actual subsurface geologic and geotechnical conditions may vary at the exploration site and other locations, with the passage of time. In addition, lines separating strata and geologic units are approximate boundaries only and those contacts might be gradual. No Warranty is provided as to the continuity of soil conditions between sample locations. In general, the USCS designations presented on the logs were estimated by visual methods only and actual designations (based on laboratory tests) may vary.

CGI Technical Services Inc.	UNIFIED SOIL CLASSIFICATION SYSTEM GEOTECHNICAL STUDY SPI COGENERATION FACILITY ANDERSON, CALIFORNIA	Plate A-1.2
Project No. 07-1588.05		

Exploration Date	Approx. Elevation Ft.	Logged By:	Final Exploration Depth	Excavation No.
September 19, 2007	*428*	*DNL*	*22 feet*	**DH-1**
Exploration Equipment			Excavation	
Mobil B-61 with Auto-hammer Drill Rig			*SPT*	

Approximate Location

See Plate 2 in Text

Approx. Groundwater El. (at time of investigation)

Encountered at 11 feet

DEPTH (feet)	*SAMPLE (Location)*	*SAMPLE NO.*	*USCS SYMBOL*	Description	*DRY DENSITY (PCF)*	*MOISTURE (%)*	*PASSING #200 (%)*	*LIQUID LIMIT*	*PLASTICITY INDEX*	*REMARKS*
				FILL						
1.5			SP-ML	**0' - 2' Sandy SILT with Cobbles,** brown, dry, slightly plastic, hard consistency						
3			PT	**2' - 5' Wood Debris**, black, moist, soft						
4.5		1A	ML-CL	**5' - 7' Clayey SILT with sand and gravels**, grey, moist, medium stiff to soft, wood debris present						SPT: 6, 5, 3
6										
				MODESTO FORMATION (Qm)						
7.5			SP-SM	**7' - 10' Silty SAND with Gravels and Cobbles**, grey, moist, dense, rounded						
9										
10.5		1B	GP	**10' - 15' Sandy Gravels with Cobbles**, grey, wet, dense, rounded, groundwater encountered at 11'						SPT: 17, 25, 21
12										
13.5										
15		1C	GP-SP	**15' - 20' Gravelly SAND with Cobbles,** grey, wet, very dense, non-plastic, up to 6" diameter clasts Note: SPT blow irrelative due to cobble in shoe						SPT: 22, 30, 50 limited recovery
16.5										
18										
				RIVERBANK FORMATION (Qr)						
19.5		1D	GP-GM	**20' - 22' Clayey GRAVEL to Silty GRAVEL with Cobbles,** brown, very dense, partially cemented, slightly plastic						SPT: 50 - 3" limited recovery
21										
22.5				Refusal Encountered, Bottom of DH at 22' Backfilled with wet cement and bentonite						

CGI TECHNICAL SERVICES INC.

Project No. 07-1588.05

LOG OF DRILL HOLES
GEOTECHNICAL STUDY
SPI COGENERATION FACILITY
ANDERSON, CALIFORNIA

Plate **A-2.1**

Exploration Date	Approx. Elevation Ft.	Logged By:	Final Exploration Depth	Excavation No.
September 19, 2007	*428*	*DNL*	*21 feet*	**DH-2**
Exploration Equipment			Excavation	
Mobil B-61 with Auto-hammer Drill Rig			*SPT & CAL*	

Approximate Location: *See Plate 2 in Text*

Approx. Groundwater El. (at time of investigation): *Encountered at 11 feet*

DEPTH (feet)	SAMPLE (Location)	SAMPLE NO.	USCS SYMBOL	Description	DRY DENSITY (PCF)	MOISTURE (%)	PASSING #200 (%)	LIQUID LIMIT	PLASTICITY INDEX	REMARKS
				FILL						
1.5			ML	**0' - 2' Sandy SILT with Cobbles,** brown, dry, slightly plastic, hard consistency						
3			PT	**2' - 5' Wood Debris**, black, moist, soft						
4.5		2A	SP-SM	**5' - 7' Clayey, Silty SAND with trace gravels**, grey, very soft, slightly plastic, fill or previous pond bottom			33.6	21	5	SPT: 2, 2, 2
6										
7.5			SP-SM	**7' - 9' Silty SAND with Gravels and Cobbles**, grey, moist, medium dense						
9				**MODESTO FORMATION (Qm)**						
10.5		2B	SP-SC	**9' - 15' Clayey SAND with Gravel,** grey, moist, slightly plastic, dense, rounded, groundwater encountered at 11'						SPT: 11, 17, 45
12										
13.5										
15		2C	GW	**15' - 20' Sandy GRAVEL with Cobbles,** grey, wet, dense, non-plastic, up to 6" diameter clasts			2.5			CAL: 15, 30, 25
16.5										
18										
19.5				**RIVERBANK FORMATION (Qr)**						
21		2D	GC-GM	**20' - 21' Clayey GRAVEL to Silty GRAVEL with Cobbles,** brown, very dense, partially cemented, slightly plastic						SPT: 50 - 6" limited recovery
22.5				Refusal Encountered, Bottom of DH at 21' Backfilled with wet cement and bentonite						

CGI TECHNICAL SERVICES INC.

Project No. 07-1588.05

LOG OF DRILL HOLES
GEOTECHNICAL STUDY
SPI COGENERATION FACILITY
ANDERSON, CALIFORNIA

Plate **A-2.2**

Exploration Date	Approx. Elevation Ft.	Logged By:	Final Exploration Depth	Excavation No.
September 19, 2007	*428*	*DNL*	*20 feet*	**DH-3**
Exploration Equipment			Excavation	
Mobil B-61 with Auto-hammer Drill Rig			*SPT*	

Approximate Location

See Plate 2 in Text

Approx. Groundwater El. (at time of investigation)

Encountered at 11 feet

DEPTH (feet)	*SAMPLE (Location)*	*SAMPLE NO.*	*USCS SYMBOL*	Description	*DRY DENSITY (PCF)*	*MOISTURE (%)*	*PASSING #200 (%)*	*LIQUID LIMIT*	*PLASTICITY INDEX*	*REMARKS*
				FILL						
1.5			ML	**0' - 2' Sandy SILT with Cobbles,** brown, dry, slightly plastic, hard consistency						
3			PT	**2' - 12.5' Wood Debris**, black, moist, soft						
4.5										
6										
7.5										
9										
10.5										
12				Groundwater encountered at 11'						
				MODESTO FORMATION (Qm)						
13.5										
15		3A	GP	**15' - 20' Gravelly SAND with Cobbles,** grey, wet, very dense, non-plastic, up to 6" diameter clasts						SPT: 16, 27, 32
16.5										
18										
		3B		**RIVERBANK FORMATION (Qr)**						
19.5			GC-GM	**20' Clayey GRAVEL to Silty GRAVEL with Cobbles,** brown, wet, very dense, partially cemented, slightly plastic						SPT: 50 - 6" limited recovery
21				Refusal Encountered at 20'						
22.5				Backfilled with wet cement and bentonite						

CGI Technical Services Inc.

Project No. 07-1588.05

LOG OF DRILL HOLES
GEOTECHNICAL STUDY
SPI COGENERATION FACILITY
ANDERSON, CALIFORNIA

Plate

A-2.3

Exploration Date	Approx. Elevation Ft.	Logged By:	Final Exploration Depth	Excavation No.
September 19, 2007	*428*	*DNL*	*20 feet*	**DH-4**
Exploration Equipment			Excavation	
Mobil B-61 with Auto-hammer Drill Rig			*SPT*	

Approximate Location

See Plate 2 in Text

Approx. Groundwater El. (at time of investigation)

Encountered at 10 feet

DEPTH (feet)	*SAMPLE (Location)*	*SAMPLE NO.*	*USCS SYMBOL*	Description	*DRY DENSITY (PCF)*	*MOISTURE (%)*	*PASSING #200 (%)*	*LIQUID LIMIT*	*PLASTICITY INDEX*	*REMARKS*
				FILL						
1.5			SW-SM	**0' - 7' Silty SAND with Gravel,** brown, dry,non-plastic, dense						
3										
4.5		4A	SW-SM	At 5', same as above with interbeds of wood chips						SPT: 10, 30, 27
6										
				MODESTO FORMATION (Qm)						
7.5			GW-SW	**7' - 19' Sandy GRAVEL with Cobbles**, grey, moist, medium dense, rounded clasts						
9				Groundwater encountered at 10'						
10.5		4B								SPT: 4, 6, 8
12										
13.5										
15		4C	GW-SW	**15' - 19' Sandy GRAVEL with Cobbles,** grey, wet, dense, non-plastic, up to 6" diameter clasts						SPT: 27, 19, 19
16.5										
18										
				RIVERBANK FORMATION (Qr)						
19.5		4D	GC-GM	**19' - 20' Clayey GRAVEL to Silty GRAVEL with Cobbles,** brown, very dense, partially cemented, slightly plastic						SPT: 37, 50 at 6"
21				Hole Terminated at 20'						
22.5				Backfilled with wet cement and bentonite						

CGI TECHNICAL SERVICES INC.

Project No. 07-1588.05

LOG OF DRILL HOLES
GEOTECHNICAL STUDY
SPI COGENERATION FACILITY
ANDERSON, CALIFORNIA

Plate **A-2.4**

CG07GS029

Exploration Date	Approx. Elevation Ft.	Logged By:	Final Exploration Depth	Excavation No.
September 19, 2007	428	DNL	20 feet	**DH-5**
Exploration Equipment			Excavation	
Mobil B-61 with Auto-hammer Drill Rig			SPT	

Approximate Location: See Plate 2 in Text

Approx. Groundwater El. (at time of investigation): Encountered at 7 feet

DEPTH (feet)	SAMPLE (Location)	SAMPLE NO.	USCS SYMBOL	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE (%)	PASSING #200 (%)	LIQUID LIMIT	PLASTICITY INDEX	REMARKS
				FILL						
1.5			SP-SM	**0' - 2.5' Silty SAND with Gravel,** brown, dry,non-plastic, dense, interbeds of wood chips						
3			PT	**2.5' - 14' Wood Debris,** black, moist, soft						
4.5										
6										
7.5				Groundwater encountered at 7'						
9										
10.5										
12										
13.5										
15				**MODESTO FORMATION (Qm)**						
16.5		5A	GP-GC	**14' - 19' Sandy GRAVEL with Clay,** grey, slightly plastic, dense, rounded clasts, cobbles present			5			SPT: 9, 18, 13
18				**RIVERBANK FORMATION (Qr)**						
19.5		5B	GC-GM	**19' - 20' Clayey GRAVEL to Silty GRAVEL with Cobbles,** brown, very dense, partially cemented, slightly plastic						SPT: 50 at 1"
21				Hole Terminated at 20' Backfilled with wet cement and bentonite						
22.5										

CGI TECHNICAL SERVICES INC.

Project No. 07-1588.05

LOG OF DRILL HOLES
GEOTECHNICAL STUDY
SPI COGENERATION FACILITY
ANDERSON, CALIFORNIA

Plate **A-2.5**

Exploration Date	Approx. Elevation Ft.	Logged By:	Final Exploration Depth	Excavation No.
September 19, 2007	*428*	*DNL*	*15 feet*	**TP-1**
Exploration Equipment			Excavation	
Hitachi UH-122			*4' Bucket*	

Approximate Location: *See Plate 2 in Text*

Approx. Groundwater El. (at time of investigation): *11 feet below surface*

DEPTH (feet)	SAMPLE (Location)	SAMPLE NO.	USCS SYMBOL	Description	DRY DENSITY (PCF)	MOISTURE (%)	PASSING #200 (%)	LIQUID LIMIT	PLASTICITY INDEX	REMARKS
				FILL						
1			GW-GM	**0' - 1.5' Sandy to Silty GRAVEL,** brown, dry, slightly plastic, rounded, up to 5" cobbles						
2			GW-GM	**1.5' - 4.5'** Same as above, with wood debris, recent, well preserved						
3										
4			PT	**4.5' to 14.5' Wood Debris,** black, moist to wet, soft, metal scraps present						
5										
6										
7										
8										
9										
10										
11				Groundwater Encountered at 11'						▼
12										
13										
14			GW-SW	**MODESTO FORMATION (Qm)** **14.5' - 15 Gravelly SAND with Cobbles,** grey, wet, dense, non-plastic, clasts up to 12"						
15				Test Pit Terminated at 15'						

CGI TECHNICAL SERVICES INC.	LOG OF TEST PITS GEOTECHNICAL STUDY SPI COGENERATION FACILITY ANDERSON, CALIFORNIA	Plate **A-3.1**
Project No. 07-1588.05		

Exploration Date	Approx. Elevation Ft.	Logged By:	Final Exploration Depth	Excavation No.
September 19, 2007	*428*	*DNL*	*12 feet*	**TP-2**
Exploration Equipment			Excavation	
Hitachi UH-122			*4' Bucket*	

Approximate Location: *See Plate 2 in Text*

Approx. Groundwater El. (at time of investigation): *11 feet below surface*

DEPTH (feet)	SAMPLE (Location)	SAMPLE NO.	USCS SYMBOL	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE (%)	PASSING #200 (%)	LIQUID LIMIT	PLASTICITY INDEX	REMARKS
				FILL						
1			GP-GM	**0' - 1.5' Sandy to Silty GRAVEL,** brown, dry, slightly plastic, rounded, up to 5" cobbles, geotextile fabric present at base						
2				**1.5' - 2.5' Wood Debris,** black, moist, soft						
3			SP-SM	**2.5' - 4' Silty SAND with Gravels**, moist, grey, weakly cemented, ammended soils						
4				**4' - 4.5' Wood Debris,** black, moist, stiff						
5	X	1A	SP-SC	**4.5' - 6.5' Clayey SILT to Clayey SAND with Gravel,** moist, brown, stiff, partially cemented, poorly sorted						
				MODESTO FORMATION (Qm)						
6	X		SP	**6.5' - 12' Gravelly SAND with Cobbles,** dense, up to 8" in diameter, alluvial						
7										
8										
9	X	2A								
10										
11				Groundwater Encountered at 11'						▼
12				Test Pit Terminated at 12'						
13										
14										
15										

CGI Technical Services Inc.

Project No. 07-1588.05

LOG OF TEST PITS
GEOTECHNICAL STUDY
SPI COGENERATION FACILITY
ANDERSON, CALIFORNIA

Plate **A-3.2**

Exploration Date	Approx. Elevation Ft.	Logged By:	Final Exploration Depth	Excavation No.
September 19, 2007	*428*	*DNL*	*12 feet*	**TP-3**
Exploration Equipment			Excavation	
Hitachi UH-122			*4' Bucket*	

Approximate Location

See Plate 2 in Text

Approx. Groundwater El. (at time of investigation)

10 feet below surface

DEPTH (feet)	SAMPLE (Location)	SAMPLE NO.	USCS SYMBOL	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE (%)	PASSING #200 (%)	LIQUID LIMIT	PLASTICITY INDEX	REMARKS
				FILL						
1			GP-GM	**0' - 1.5' Sandy to Silty GRAVEL,** brown, dry, slightly plastic, rounded, up to 5" cobbles, geotextile fabric present at base						
2			GP-GC	**1.5' - 5.5' Clayey GRAVEL,** brown, moist, soft, wood & metal debris present						
3										
4										
5										
				MODESTO FORMATION (Qm)						
6				**5.5' - 12' Sandy GRAVEL with Cobbles**, grey, moist, dense, rounded						
7										
8										
9										
10				Groundwater Encountered at 10'						
11										
12				Test Pit Terminated at 12'						
13										
14										
15										

CGI TECHNICAL SERVICES INC.

Project No. 07-1588.05

LOG OF TEST PITS
GEOTECHNICAL STUDY
SPI COGENERATION FACILITY
ANDERSON, CALIFORNIA

Plate **A-3.3**

Exploration Date	Approx. Elevation Ft.	Logged By:	Final Exploration Depth	Excavation No.
September 19, 2007	*428*	*DNL*	*4.5 feet*	**TP-4**
Exploration Equipment			Excavation	
Hitachi UH-122			*4' Bucket*	

Approximate Location: *See Plate 2 in Text, near west end of turbine*

Approx. Groundwater El. (at time of investigation): *Groundwater not encountered*

DEPTH (feet)	SAMPLE (Location)	SAMPLE NO.	USCS SYMBOL	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE (%)	PASSING #200 (%)	LIQUID LIMIT	PLASTICITY INDEX	REMARKS
				FILL						
1			GP-GM	**0' - 1.5' Sandy to Silty GRAVEL,** brown, dry, slightly plastic, rounded, up to 5" cobbles, fabric present						
2			PT	**1.5' - 3' Wood Debris,** black, moist, soft						
3				**MODESTO FORMATION (Qm)**						
4			GW-SW	**3' - 4.5' Sandy GRAVEL,** with some cobbles, moist, grey, dense, well-graded, rounded clasts up to 8" diameter						
5				Test Pit Terminated at 4.5'						
6										
7										
8										
9										
10										
11										
12										
13										
14										
15										

CGI Technical Services Inc.

Project No. 07-1588.05

LOG OF TEST PITS
GEOTECHNICAL STUDY
SPI COGENERATION FACILITY
ANDERSON, CALIFORNIA

Plate **A-3.4**

Exploration Date	Approx. Elevation Ft.	Logged By:	Final Exploration Depth	Excavation No.
September 19, 2007	428	DNL	10 feet	**TP-5**
Exploration Equipment			Excavation	
Hitachi UH-122			4' Bucket	

Approximate Location: See Plate 2 in Text, near fuel house

Approx. Groundwater El. (at time of investigation): Groundwater encountered at 7 feet

DEPTH (feet)	SAMPLE (Location)	SAMPLE NO.	USCS SYMBOL	DESCRIPTION	DRY DENSITY (PCF)	MOISTURE (%)	PASSING #200 (%)	LIQUID LIMIT	PLASTICITY INDEX	REMARKS
				FILL						
1	X	3		**0' - 4" Asphalt Concrete**						
			SP-SM	**4" - 2' Silty SAND with Gravel,** brown, dry, dense, underlain by geotextile fabric						
2			PT	**2' - 3' Wood Debris,** black, moist, soft						
3			SP-SM	**3' - 5' Clayey, Silty SAND with trace Gravels,** grey, moist, soft, slightly plastic						
4										
5				**5' - 10' Wood Debris,** black, moist to wet, soft						
6										
7				Groundwater encountered at 7 feet						▼
8										
9				Encountered some loose gravels and cobbles						
10				Test Pit Terminated at 10 feet						
11										
12										
13										
14										
15										

CGI TECHNICAL SERVICES INC.

Project No. 07-1588.05

LOG OF TEST PITS
GEOTECHNICAL STUDY
SPI COGENERATION FACILITY
ANDERSON, CALIFORNIA

Plate **A-3.5**

CG07GS029

APPENDIX B
LABORATORY TESTING

Laboratory Analyses

Laboratory tests were performed on selected undisturbed and bulk soil samples to estimate engineering characteristics of the various earth materials encountered. Laboratory testing was performed by CurryGroup, Inc. Testing was performed under procedures described in one of the following references:

- ASTM Standards for Soil Testing, latest revision;
- Lambe, T. William, Soil Testing for Engineers, Wiley, New York, 1951;
- Laboratory Soils Testing, U.S. Army, Office of the Chief of Engineers, Engineering Manual No. 1110-2-1906, November 30, 1970.

Atterberg Limits Test

Atterberg Limits test was performed on a selected soil sample to estimate the plasticity index, plastic limit, and liquid limit of the soils tested. The test was conducted in general accordance with standard test method ASTM D4318. The results of the tests are presented on the boring logs and on the attached Atterberg Limits Tests figure.

Grain-Size Evaluations

Three grain-size evaluations were performed in accordance with standard test method ASTM D422 to estimate the general distribution of grain sizes in the samples tested. The results of the grain-size distribution tests are shown on the attached Laboratory Sieve Analysis figures.

Limited Soil-Chemistry

Tests were performed on selected soil samples to evaluate chloride and sulfate contents. Basic Laboratory in Redding, California performed the tests and the results are presented on the attached *Soil Chemistry* sheet.

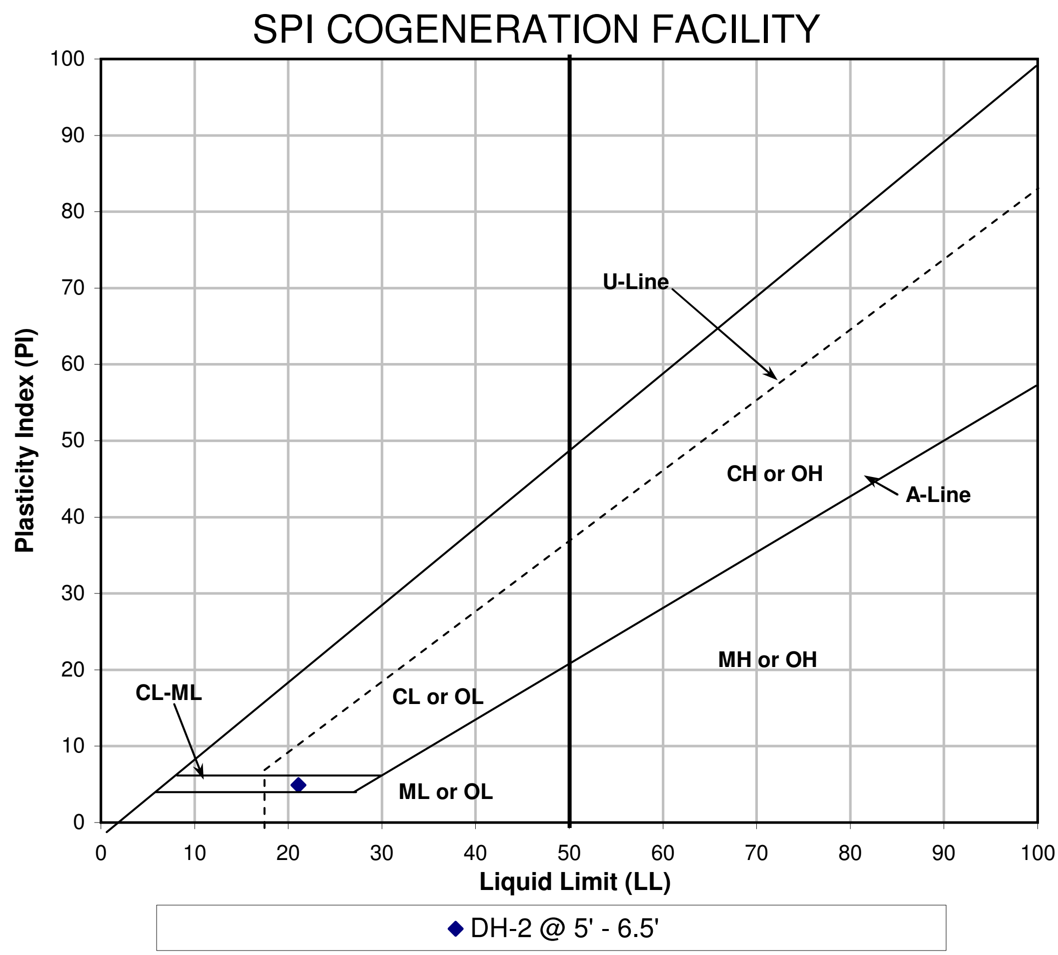
ATTERBERG LIMITS TESTS

CGI TECHNICAL SERVICES INC.

Client: SIERRA PACIFIC INDUSTRIES
Project: SPI COGENERATION FACILITY
Location: ANDERSON, CALIFORNIA
Sampled By: DNL
Received By: JC
Tested By: JC
Reviewed By: ME

Job No.: 07-1588.05
Lab No.: 2060

Date Sampled: 19-Sep-07
Date Received: 19-Sep-07
Date Tested: 26-Sep-07
Date Reviewed: 24-Oct-07



LEGEND			CLASSIFICATION	ATTERBERG LIMITS TEST RESULTS		
Location	Depth, ft	Sample No.		Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)
DH-2	5' - 6.5'	1	Silty Clay (ML-CL)	21	16	5

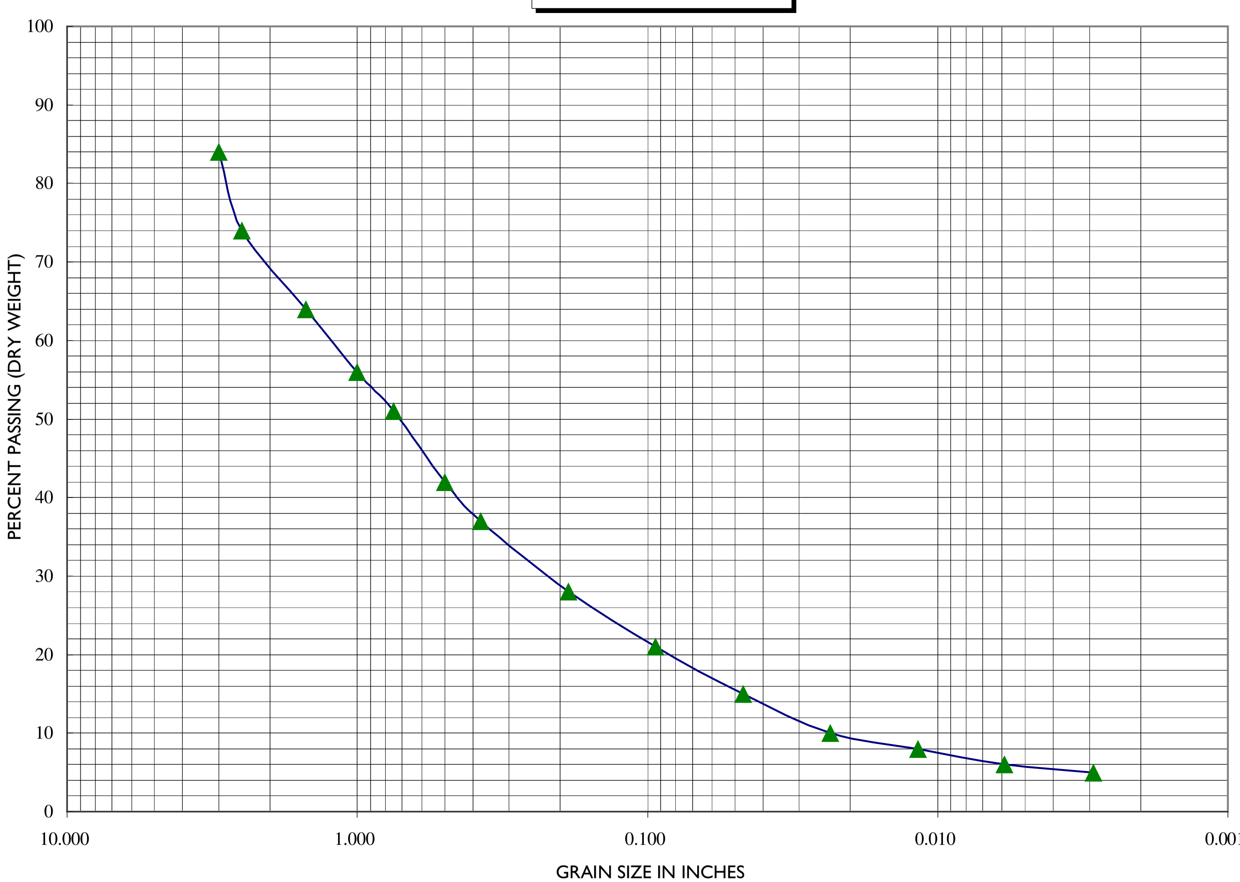
ASTM D4318 & D2487

CGI TECHNICAL SERVICES INC.

LABORATORY TEST RESULTS

Client:	SIERRA PACIFIC INDUSTRIES	**Job No.:**	07-1588.05
Project:	SPI COGENERATION FACILITY	**Lab No.:**	2060
Location:	ANDERSON, CALIFORNIA	**Date Sampled:**	19-Sep-07
Material Type:	NATIVE SOIL	**Sampled By:**	DNL
Test Procedure:	ASTM D422	**Tested By:**	SP
		Date Tested:	26-Sep-07
		Date Reviewed:	24-Oct-07

SPI CO-GEN
DH-5 @ 15' to 16.5'



SIEVE ANALYSIS

Grain Size (inches)	Grain Size (mm)	Percent Passing	Specified
3.0000	76.00	84	
2.5000	63.50	74	
1.5000	51.00	64	
1.0000	25.00	56	
0.7500	19.00	51	
0.5000	12.50	42	
0.3750	9.50	37	
0.1870	4.75	28	
0.0937	2.36	21	
0.0469	1.18	15	
0.0234	600um	10	
0.0117	300um	8	
0.0059	150um	6	
0.0029	75um	5.0	

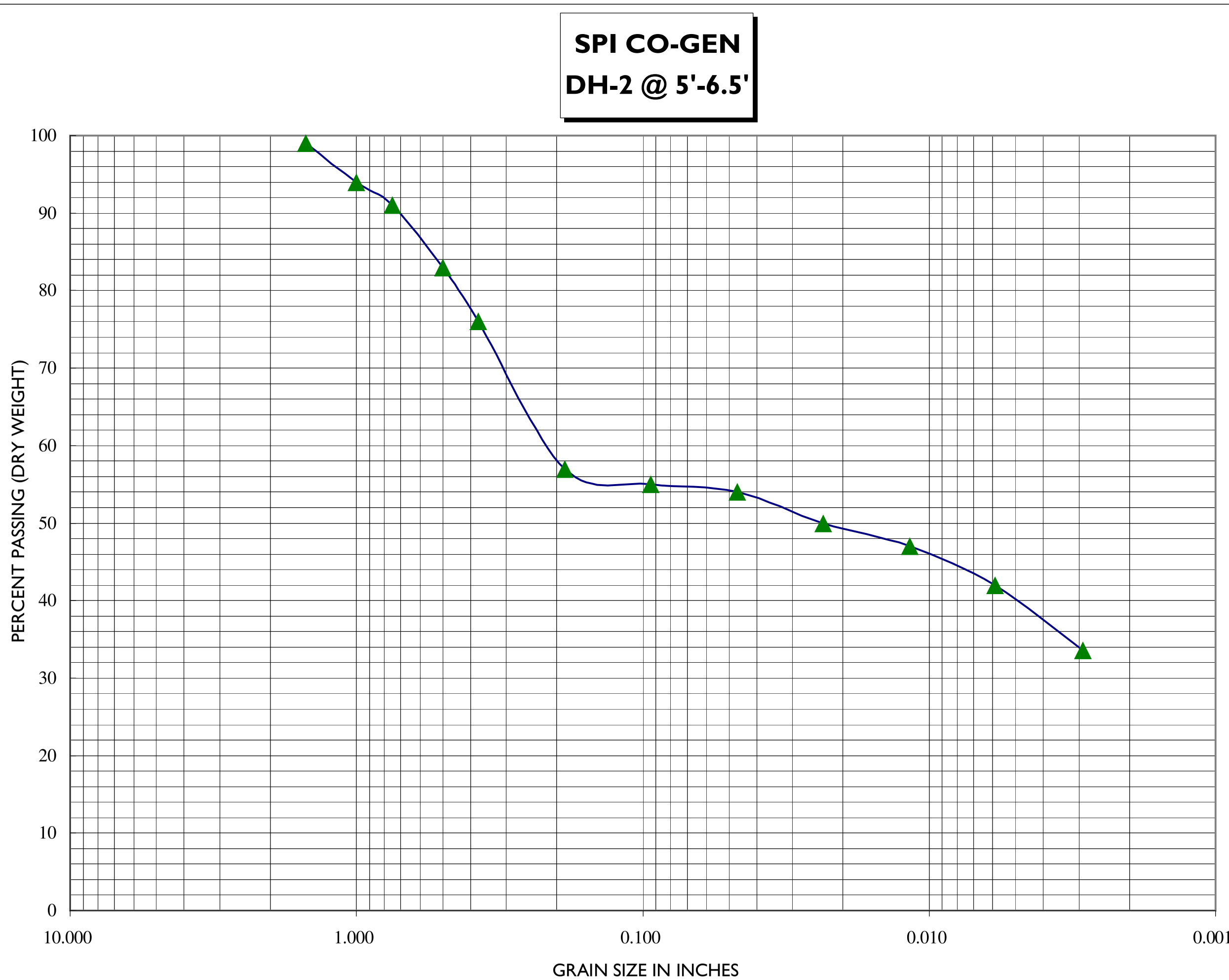
ATTERBERG LIMIT

Liquid Limit	
Plastic Limit	
Plasticity Index	
Specification	GP

LABORATORY TEST RESULTS

Client: SIERRA PACIFIC INDUSTRIES
Project: SPI COGENERATION FACILITY
Location: ANDERSON, CALIFORNIA
Material Type: NATIVE SOIL **Sampled By:** DNL
Test Procedure: ASTM D422 **Tested By:** SP

Job No.: 07-1588.05
Lab No.: 2060
Date Sampled: 19-Sep-07
Date Tested: 26-Sep-07
Date Reviewed: 24-Oct-07



SIEVE ANALYSIS			
Grain Size (inches)	Grain Size (mm)	Percent Passing	Specified
3.0000	76.00		
2.5000	63.50		
1.5000	51.00	99	
1.0000	25.00	94	
0.7500	19.00	91	
0.5000	12.50	83	
0.3750	9.50	76	
0.1870	4.75	57	
0.0937	2.36	55	
0.0469	1.18	54	
0.0234	600um	50	
0.0117	300um	47	
0.0059	150um	42	
0.0029	75um	33.6	

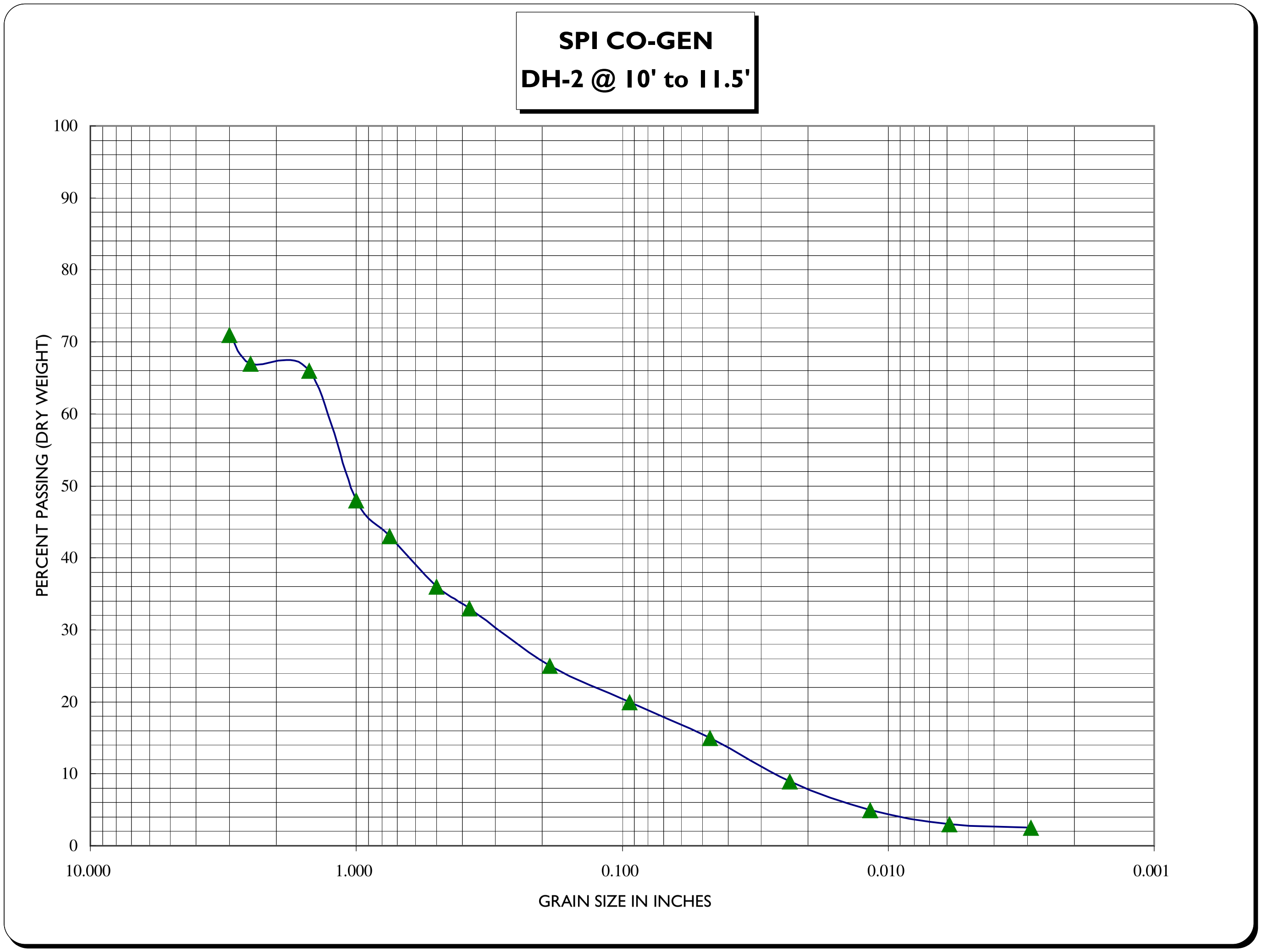
ATTERBERG LIMIT	
Liquid Limit	
Plastic Limit	
Plasticity Index	
Specification	GM

CGI Technical Services Inc.

LABORATORY TEST RESULTS

Client:	SIERRA PACIFIC INDUSTRIES	**Job No.:**	07-1588.05
Project:	SPI COGENERATION FACILITY	**Lab No.:**	2060
Location:	ANDERSON, CALIFORNIA	**Date Sampled:**	19-Sep-07
Material Type:	NATIVE SOIL	**Date Tested:**	26-Sep-07
Test Procedure:	ASTM D422	**Date Reviewed:**	24-Oct-07

Sampled By: DNL
Tested By: SP



SIEVE ANALYSIS

Grain Size (inches)	Grain Size (mm)	Percent Passing	Specified
3.0000	76.00	71	
2.5000	63.50	67	
1.5000	51.00	66	
1.0000	25.00	48	
0.7500	19.00	43	
0.5000	12.50	36	
0.3750	9.50	33	
0.1870	4.75	25	
0.0937	2.36	20	
0.0469	1.18	15	
0.0234	600um	9	
0.0117	300um	5	
0.0059	150um	3	
0.0029	75um	2.5	

ATTERBERG LIMIT	
Liquid Limit	
Plastic Limit	
Plasticity Index	
Specification	GW

basic laboratory
www.basiclab.com
voice 530.243.7234 fax 530.243.7494
2218 Railroad Avenue Redding, California 96001

Report To: CGI TECHINCAL SERVICES (CURRY GROUP)
1612 WEDDING WAY
REDDING, CA 96003
Attention: DON LINDSAY
Project: GENERAL TESTING

Lab No: 7100317
Reported: 10/24/07
Phone: 244-6277
P.O. #

General Chemistry - Solid

Analyte	Units	Results	Qualifier	MDL	RL	Method	Analyzed	Prepared	Batch
DH-4 @ 7'-10' Soil (7100317-01)	**Sampled:10/08/07 00:00**		**Received:10/09/07 12:03**						
Chloride	mg/kg	**5.09**		0.40	2.00	EPA 300.0	10/14/07	10/14/07	B7J0342
Sulfate as SO4	"	**24.0**		0.80	4.00	"	"	"	"
TP-1 @ 2'-14' Soil (7100317-02)	**Sampled:10/08/07 00:00**		**Received:10/09/07 12:03**						
Chloride	mg/kg	**12.6**		0.40	2.00	EPA 300.0	10/14/07	10/14/07	B7J0342
Sulfate as SO4	"	**258**		0.80	4.00	"	"	"	"

Notes and Definitions

DET	Analyte DETECTED
ND	Analyte NOT DETECTED at or above the detection limit
NR	Not Reported
dry	Sample results reported on a dry weight basis
RPD	Relative Percent Difference
<	Less than reporting limit
≤	Less than or equal to reporting limit
>	Greater than reporting limit
≥	Greater than or equal to reporting limit
MDL	Method Detection Limit
RL/ML	Minimum Level of Quantitation
MCL/AL	Maxium Contaminant Level/Action Level
mg/kg	Results reported as wet weight
TTLC	Total Threshold Limit Concentration
STLC	Soluble Threshold Limit Concentration
TCLP	Toxicity Characteristic Leachate Procedure

Approved By

Basic Laboratory, Inc.
California D.O.H.S. Cert #1677

APPENDIX C
LIQUEFACTION

To estimate if the underlying soils have the potential to liquefy during a seismic event, we used methods described by Youd et al (2001). For our analyses, we assumed groundwater is present at a depth of 2 feet and performed the analysis using a maximum credible earthquake magnitude of 6.5 and a horizontal ground acceleration of 0.25g, which corresponds to the mean peak ground acceleration determined deterministically. A factor of safety (FOS) against liquefaction occurring of 1.3 or less is typically considered a potentially liquefiable layer. If the FOS exceeds 1.3 then liquefaction is not considered as potentially impacting the project.

Project:	SPI Cogeneration Plant
Project No.:	07-1588.05
Drill Hole Number	

Depth to Water for Analysis (ft)	**2**
Depth to Water at Drilling (ft)	**10**
Earthquake Magnitude (Mw)	**6.5**
Peak Ground Acceleration (g)	**0.25**

Hammer Energy Correction C_E:	1.0
Drill Hole Diameter (in):	8.5
SPT liner used? (Yes/No):	No

Soil Log

Begin Depth (ft)	End Depth (ft)	Moist Density	Saturated Density	Description
0	2	-		Silty Sand with Gravel
2	7	-		Organics
7	12	-		Silty Sand with Gravel
12	19	-		Sandy Gravel with cobbles
19	40	-		Silty Sand with Gravel
40		-		
0		-		

Depth		Ave. Density above sample	Total Stress		Effective Stress		r_d	SPT	Corrections to SPT (Youd 2002)					Cal. Mod. Correction	$(N_1)_{60}$	Fines Content	α	β	$(N_1)_{60CS}$	CSR	CRR	MSF	Factor of Safety
ft	m	pcf	psf	kPa	psf	kPa		Blows/Ft	C_N	C_E	C_B	C_S	C_R			%							
5	1.5	100	500.0	23.9	313	14.98	0.99	8	1.38	1.0	1.15	1.2	0.75	1.0	11	20	3.6	1.1	16	0.26	0.17	1.44	0.95
10	3.0	100	1000.0	47.9	501	23.98	0.98	8	1.31	1.0	1.15	1.2	0.80	1.0	12	5	0.0	1.0	12	0.32	0.13	1.44	0.58
15	4.6	100	1500.0	71.8	689	32.98	0.97	8	1.24	1.0	1.15	1.2	0.85	1.0	12	5	0.0	1.0	12	0.34	0.13	1.44	0.54
20	6.1	126	2520.0	120.7	1397	66.88	0.96	52	1.04	1.0	1.15	1.2	0.95	1.0	71	5	0.0	1.0	71	0.28	N.L.	1.44	N.L.
40	12.2	128	5120.0	245.1	2749	131.61	0.85	41	0.80	1.0	1.15	1.2	1.00	1.0	45	20	3.6	1.1	52	0.26	N.L.	1.44	N.L.

Project:	SPI Cogeneration Plant
Project No.:	07-
Drill Hole Number	1

Depth to Water for Analysis (ft)	**2**
Depth to Water at Drilling (ft)	**10**
Earthquake Magnitude (Mw)	**6.5**
Peak Ground Acceleration (g)	**0.25**

Hammer Energy Correction C_E:	1.0
Drill Hole Diameter (in):	8.5
SPT liner used? (Yes/No):	No

Soil Log

Begin Depth (ft)	End Depth (ft)	Moist Density	Saturated Density	Description
0	2	-		Silty Sand with Gravel
2	7	-		Organics
7	12	-		Silty Sand with Gravel
12	19	-		Sandy Gravel with cobbles
19	40	-		Silty Sand with Gravel
40		-		
0		-		

Depth		Ave. Density above sample	Total Stress		Effective Stress		r_d	SPT	Corrections to SPT (Youd 2002)					Cal. Mod. Correction	$(N_1)_{60}$	Fines Content	α	β	$(N_1)_{60CS}$	CSR	CRR	MSF	Factor of Safety
ft	m	pcf	psf	kPa	psf	kPa		Blows/Ft	C_N	C_E	C_B	C_S	C_R			%							
5	1.5	100	500.0	23.9	313	14.98	0.99	8	1.38	1.0	1.15	1.2	0.75	1.0	11	20	3.6	1.1	16	0.26	0.17	1.44	0.95
10	3.0	117.5	1175.0	56.3	676	32.36	0.98	21	1.25	1.0	1.15	1.2	0.80	1.0	29	5	0.0	1.0	29	0.28	0.41	1.44	2.12
15	4.6	123	1845.0	88.3	1034	49.50	0.97	29	1.14	1.0	1.15	1.2	0.85	1.0	39	5	0.0	1.0	39	0.28	N.L.	1.44	N.L.
20	6.1	126	2520.0	120.7	1397	66.88	0.96	52	1.04	1.0	1.15	1.2	0.95	1.0	71	5	0.0	1.0	71	0.28	N.L.	1.44	N.L.
40	12.2	128	5120.0	245.1	2749	131.61	0.85	41	0.80	1.0	1.15	1.2	1.00	1.0	45	20	3.6	1.1	52	0.26	N.L.	1.44	N.L.