GEOTECHNICAL REPORT
Big Bend Hot Springs Project
Big Bend, Shasta County, California

Prepared For:

Big Bend Hot Springs Project, LLC

CGI Technical Services Inc.

BIG BEND Hot Springs Project

CGI TECHNICAL SERVICES INC.

October 3, 2011
CGI: 01-1995.01

Mr. Brook Leaf
BIG BEND HOT SPRINGS PROJECT, LLC
c/o Barrett Ecological Services, LLC
1534 SE 40th Avenue
Portland, Oregon 97214

Subject: **Geotechnical Report**
Big Bend Hot Springs Project
Big Bend, Shasta County, California

Dear Mr. Leaf,

CGI Technical Services, Inc. (CGI), is pleased to submit this geotechnical report for the proposed improvements at Big Bend Hot Springs located in the Big Bend area of Shasta County, California. This report presents our findings, conclusions, and recommendations for design of the proposed development.

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 Jim Bianchin at (530) 244-6277 at your earliest convenience.

Regards,

CGI TECHNICAL SERVICES, INC.

REGISTERED GEOLOGIST
JAMES A. BIANCHIN
EG NO. 1644
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

REGISTERED PROFESSIONAL ENGINEER
AZEDDINE BAHLOUL
No. C63022
Exp. 06/12
CIVIL
STATE OF CALIFORNIA

James A. Bianchin, C.E.G.
Senior Engineering Geologist

Azeddine Bahloul, P.E.
Senior Engineer

Copies: One (1) electronic file (PDF) to Big Bend Hot Springs Project, LLC
Three (3) hardcopies and one (1) electronic file (PDF) to Barrett Ecological Services, LLC

Redding Office
1612 Wedding Way
Redding, CA 96003
Ph: 530.244.6277
Fax: 530.244.6276

TABLE OF CONTENTS
GEOTECHNICAL REPORT
BIG BEND HOT SPRINGS PROJECT
BIG BEND, SHASTA COUNTY, CALIFORNIA

TABLE OF CONTENTS (continued)
GEOTECHNICAL REPORT
BIG BEND HOT SPRINGS PROJECT
BIG BEND, SHASTA COUNTY, CALIFORNIA

PLATES

APPENDICES

1.0 GENERAL

This report presents the results of our geotechnical study for proposed improvements at the Big Bend Hot Springs project, located in Shasta County, California. CGI Technical Services, Inc. (CGI), has prepared this report at the request of Big Bend Hot Springs Project, LLC (BBHS). The project location is shown on Plate 1 – Site Location Map. The following sections present our understanding of the project, the purpose of our study, and the findings, conclusions, and recommendations of this study. Our services were performed in general compliance with our proposal dated April 27, 2011.

1.1 PROJECT UNDERSTANDING

The project, as we understand it, consists of the design of a number of new structures and soaking tubs, and improvement of numerous existing facilities to create a unique, sustainable hot springs, camping, and meeting facility. We understand that the project includes the development of the following:

- New soaking tubs and changing rooms;
- A manager's residence with a proposed basement;
- A meeting space structure,
- A kitchen;
- An ADA accessible cabin and a number of camping areas dispersed on the property.

In addition, we understand that an existing foot bridge crossing an unnamed creek that bisects the property may be replaced or widened as part of the improvement plans. The proposed improvements are shown on Plate 2 – Project Elements.

We understand that the proposed structures will generally be single-story and wood framed, including the straw-bale manager's residence. As such, it is anticipated that the structures will be relatively lightly loaded and will be supported on shallow foundation systems. The foundation loads for those structures are unknown but assumed to not exceed 3 kips per lineal foot and 15 kips for continuous and isolated foundations, respectively.

1.2 STUDY PURPOSE

The purpose of our geotechnical study was to explore and evaluate selected site surface and subsurface conditions in order to provide geotechnical engineering recommendations related to the design and construction of the project, and to identify potential geologic hazards that could impact the project. The subsurface characterization was primarily intended to estimate the depth, profile, consistency, strength, and grain-size distribution of the soils that might be encountered during project construction, along with the general depth to groundwater.

1.3 PREVIOUS WORK PERFORMED

We know of no previous geotechnical studies that have been performed at the project site. Regional geological maps and studies have been performed in the project area. Selected regional geological studies and maps referred to in this study are cited in References Section of this report.

1.4 SCOPE OF SERVICES

Services performed for this study are in general conformance with the proposed scope of services presented in our April 27, 2011 proposal. Our scope of services included:

- Reconnaissance of the site surface conditions, topography, and existing drainage features;
- Attempted acquisition of existing, available geotechnical data for the project site;
- Review of pertinent, selected regional geological data;
- Exploration of the subsurface conditions within the project site using test pits. Exploration locations are shown on Plate 3 – Geotechnical Map. Exploration procedures and test pit logs are presented in Appendix A – Subsurface Exploration;
- Performance of laboratory testing on selected samples obtained during our field investigation. Laboratory test procedures and results of those tests are presented in Appendix B – Laboratory Testing;
- Preparation of this report, which includes:
  - A description of the proposed project;
  - A summary of our field exploration and laboratory testing programs;
  - A description of site surface and subsurface conditions encountered during our field investigation;
  - A description of ground shaking conditions expected at the site, including CBC seismic design criteria;
  - Recommendations for:
    - Site preparation, engineered fill, site drainage, and subgrades;
    - Suitability of on-site materials for use as engineered fill;
    - Construction of keyways, benches, and subdrains;
    - 2010 CBC seismic design criteria;
    - Concrete slabs on-grade;
    - Temporary excavations, shoring, and trench backfill;
    - Lateral earth pressures for retaining wall design; and
    - Allowable bearing capacities for foundation design.
  - Appendices that present a summary of our field investigation procedures and laboratory testing programs.

2.0 FINDINGS

2.1 FIELD INVESTIGATION

CGI conducted a geotechnical field investigation to evaluate subsurface soil conditions, and to provide subsurface data for evaluation of the proposed development. Our field geotechnical investigation was limited to reconnaissance-level geologic mapping of the project site and subsurface exploration through excavation of four test pits. The test pits, designated TP-1 through TP-4, were excavated on June 21, 2011. Test pit locations are shown on Plate 3. Detailed descriptions of soils encountered are presented on the test pit logs included in Appendix A. The soils encountered within the test pits were logged in general accordance with the Unified Soil Classification System (USCS). Surficial and subsurface soil samples were collected and transported to our laboratory for testing. Laboratory test results are included with this report.

2.2 SITE CONDITIONS

2.2.1 Surface Conditions

The general topography of the site consists of an emergent river terrace located adjacent to the Pit River and separated from an elevated bench or older terrace (herein referred to as upper bench) by an unnamed drainage and moderately inclined slope. The emergent terrace is relatively flat and slightly inclined towards the east at general inclinations of less than about 5 degrees. The emergent terrace is located at an elevation range of about 965 to 1,000 feet above mean sea level (Northstar, 2010). This area is currently developed with a restroom facility, unpaved access roads and paths, a yurt-type structure, wells and a well house, storage shed, yoga deck, a shallow hot springs pool, a footbridge over the unnamed drainage, and the existing hot springs structure perched above the creek. The terrace has numerous, locally dense stands of oaks and some confers, and sparse to dense seasonal grasses, shrubs, and perennial vegetation. Drainage occurs as sheet flow to the north and west into the Pit River and unnamed drainage.

The upper bench is located in the southwest portion of the proposed improvement area. We understand that this area was historically developed with hotel and residence structures, which are no longer present. The presence of numerous fruit trees is evidence of the prior historical development. This area is relatively narrow and elongate in a northwest direction. It is relatively flat and inclined downward towards the northeast at an inclination of about 5 to 10 degrees. Elevations of the upper bench range from about 1,005 to 1,045 feet above MSL (Northstar, 2010). Currently, a barn and three accessory structures are present and serviced by unpaved access roads. This area is covered with moderate to dense stands of conifers and oaks with scattered fruit trees. Drainage occurs as sheetflow towards the northeast into the Pit River and unnamed drainage.

Relatively steep slopes separate the upper bench and the lower emergent terrace, are present along the banks of the unnamed drainage and along the Pit River. Those slopes range in height from about 15 to 30 feet and are inclined as steep as ½:1 (horizontal to vertical). The riverbank located along the northern margin of the proposed improvement area ranges in height from about 15 to 25 feet and is inclined as steep as about ½:1. Natural hot springs discharge from the western creek bank near the Pit River and from the river bank west of the creek channel.

2.2.2 Subsurface Conditions

Subsurface conditions encountered during this study vary based on whether the explorations were advanced in the emergent river terrace or in the upper bench. Subsurface conditions encountered in the emergent terrace consisted of coarse-grained alluvial sediments. Those sediments consisted of moderate to dark brown, damp, medium dense to dense, fine to coarse sand with subrounded to rounded fine to coarse gravel, cobbles, and boulder with maximum dimensions of at least 36 inches.

Few to moderate amounts of fine to medium roots were encountered in the upper 18 inches of the soil profile. Locally, minor slightly plastic clay was encountered within the alluvium.

Colluvial soils were encountered in explorations advanced in the upper bench area. Those materials consisted of moderate brown, damp, stiff to medium dense sandy clay to clayey sand that was slightly plastic, and contained fine sand, minor subrounded fine to medium gravel, cobbles and boulders up to a maximum dimension of at least 24 inches. Moderate to abundant fine to medium roots were encountered in the upper 12 inches of the soil profile.

2.3 GEOLOGIC CONDITIONS

2.3.1 Regional Geology

The project site is located at the eastern margin of the Klamath Mountains geomorphic/geologic province of California (Irwin, 1994). The Klamath Mountains province extends from the northern end of the California Coast Ranges north into Oregon. It is bounded to the east by the Cascade Range province, to the south by the Coast Ranges and Great Valley provinces, to the west by the Pacific Ocean, and to the north by Coast Ranges of Oregon. The Klamath Mountains province is predominately composed of pre-Paleozoic and Paleozoic sedimentary, volcanic, intrusive, and metamorphic rocks that have been locally intruded by Mesozoic-age rocks (Hinds, 1952). Rock materials within this province have been accreted during tectonic processes into differing terrains or differing ages.

The project site is situated in the Eastern Klamath Terrane of the Klamath Mountains (Irwin, 1994). According to Irwin (1994), the project region is underlain predominately by the Arvison Formation with a minor component of the Potem Formation. The Arvison Formation consists of marine andesitic, pyroclastic beds, flows, breccia, and conglomerate and interbedded tuff, tuffaceous sandstone and minor limestone (Dupras, 1997; Irwin, 1994; Sanborn, 1960). The Potem Formation consists of marine argillite, tuffaceous sandstone, and minor limestone and coarse pyroclastic deposits (Dupras, 1997; Irwin, 1994; Sanborn, 1960). It is thought that the Bragdon Formation, consisting of andesite, is a volcanic facies within the Potem Formation (Lydon and O'Brien, 1974; Sanborn, 1960).

Dupras (1997) has the entire project region in the Big Bend area mapped as underlain by the Montgomery Creek Formation. The Montgomery Creek Formation is a nonmarine deposit of thickly bedded, weakly indurated arkosic sandstone, conglomerate, and shale.

An undated local geologic map was prepared for the Floyd Fowler property (located south of the subject project site) by Wessley Paulsen and Roger Hail. That map encompasses the project property and has a divergent geologic interpretation as compared to Sanborn (1960) and Irwin (1994). According to Paulsen and Hail, the project region is dominated by landslide deposits nested on the Montgomery Creek Formation. They mapped a number of subservient faults extending through the Montgomery Creek Formation and a more significant fault projecting along or beneath the northern edge of the landslide deposits. Paulsen and Hail also map the Bagley Formation along the Pit River just west of the project property.

Based on our observations at the site, it is our opinion that the mapped geological conditions of Paulsen and Hail map closely reflect regional geological conditions. It is unlikely that the Bagley Formation mapped by Paulsen and Hail is present and could be an exposure of the underlying Arvison Formation. The Montgomery Creek Formation likely overlays the Arvison Formation within the project region and is the likely source for the landslide deposits.

2.3.2 Local Geologic Setting

Alluvium, colluvium and older landslide deposits are present at the project site, as shown on Plate 3. The alluvium is present as uplifted terrace deposits located beneath the majority of the project site. The terrace deposits are composed of granular soils containing abundant gravel, cobbles and boulders. The thicknesses of the terrace deposits are unknown, were not fully penetrated by our explorations, and are exposed for the entire cut bank height along the Pit River and unnamed creek.

Colluvial soils consist of soil materials moved and deposited by gravity, slope creep, and assisted by sheet-flow drainage. Colluvium is generally composed of weathered byproducts of underlying formational materials and materials located upslope. They are located beneath the upper bench and locally on slopes leading down to the emergent terrace, creek, and Pit River. Those materials are generally finer grained relative to the terrace deposits and contain some gravel, cobbles and boulders.

Older landslide deposits were not encountered in our explorations but are, in our opinion, located in slopes above and below he upper bench. It is likely that older landslide deposits are composed of weathered byproducts of the Montgomery Creek Formation from which it appears they were derived. It should be noted that the older landslide deposits located on site are older deposits and that no signs of recent or incipient landsliding were observed. They appear to have been deposited then eroded and incised and now are overlain by alluvium and colluvium.

2.3.3 Groundwater

Groundwater was not encountered within the test pits. It is anticipated that shallow groundwater might be present locally beneath the site, especially near the confluence between the creek and Pit River. That water could occur as hot springs or it could be at ambient temperature, depending on the location of encounter.

It is anticipated that groundwater elevations will fluctuate over time. The depth to groundwater can vary throughout the year and from year to year. Intense and long duration precipitation, modification of topography, and cultural land uses, such as irrigation, water well usage, on site waste disposal systems, and water diversions can contribute to fluctuations in groundwater levels. Localized saturated conditions or perched groundwater conditions near the ground surface should be anticipated during and following periods of heavy precipitation and snowmelt. If groundwater is encountered during construction, it is the Contractor's responsibility to install mitigation measures for adverse impacts caused by groundwater encountered in excavations.

3.0 GEOLOGICAL HAZARDS

3.1 GEOLOGIC HAZARD ZONES

No mapped geologic hazards zones are known for the project region.

3.2 FAULTING & SEISMICITY

3.2.1 Seismic Setting

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.

The site is not located within an Alquist-Priolo Earthquake Fault Zone and no active faults are known to pass through the project site (Jennings, 1994; Hart & Bryant, 1997). The faults mapped by Paulsen and Hail project through the project site. The largest of those faults forms an escarpment separating the emergent terrace from the upper bench. As mapped, that fault projects beneath the terrace and older landslide deposits. It is our opinion that under a worst case, the fault is potentially active but more likely is inactive. That opinion is based on the oblique orientation of that fault relative to active faults in the region and the lack of surface geomorphology that would imply recency of movement. In addition, the subservient faults that project onto the property are mapped within the Montgomery Creek Formation and are mapped as being covered by the older landslide deposits. Those faults are also, in our opinion, potentially active to inactive.

The closest fault to the site (other than those mapped by Paulsen and Hail) recognized by CGS is the inactive Willow Springs fault located about 2.5 miles northwest of the site (Jennings, 1994). The closest active fault is the Hat Creek fault, located about 18 miles southeast of the site (Jennings, 1994).

Historically over the last approximately 200 years, 25 earthquakes with local magnitudes (ML) equal or greater than 5.5 have occurred within approximately 50 kilometers of the site, based on a search of selected earthquake catalogs (Toppozada and Branum, 2002). The most recent significant earthquake to affect the project area was an earthquake with a moment magnitude (Mw) of 6.4 that occurred on April 19, 1892 approximately 90 miles (144 km) from the site.

Local earthquakes can also be expected from Lassen Peak if it enters a phase nearing eruption or if subsurface migration of magma occurs. Those earthquakes, similar to earthquakes experienced prior to eruption of Mt. St. Helens or at Mammoth Mountain (without eruption), typically occur as swarms with earthquake magnitudes of low to moderate intensity.

3.2.2 CBC Design Recommendations

At a minimum, structures should be designed in accordance with the 2010 California Building Code (CBC) seismic design criteria. CBC-based design requires the definition of the following seismic parameters: Site class designation; site coefficients (F_a and F_v); mapped spectral accelerations for short periods (S_s); and mapped spectral accelerations for a 1-second period (S_1).

CBC SEISMIC DESIGN PARAMETERS	
Parameter	**CBC Designation**
Site Class Designation	D
Mapped Spectral Acceleration, S_s	0.842g
Mapped Spectral Acceleration, S_1	0.282g
Site Coefficient, F_a	1.163
Site Coefficient, F_v	1.836

3.3 LANDSLIDES

No signs of landsliding, either recent or incipient, were observed on or adjacent to the project property. It is our opinion that natural landslides pose a low risk to the project. Potential man-made slope failures are discussed in greater detail in Sections 5.6.8, 5.7, and 5.13 of this report.

3.4 LIQUEFACTION AND LATERAL SPREADING

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. In order for liquefaction to occur, the following are needed:

- Granular soils (sand, silty sand, sandy silt, and some gravels);
- A high groundwater table; and
- A low density in the granular soils underlying the site.

If those criteria are present, then there is a potential that the soils could liquefy during a seismic event.

The adverse effects of liquefaction include local and regional ground settlement, ground cracking and expulsion of water and sand, the partial or complete loss of bearing and confining forces used to support loads, amplification of seismic shaking, and lateral spreading. In general, the effects of liquefaction on the proposed project could include:

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

Lateral spreading is defined as lateral earth movement of liquefied soils, or soil riding on a liquefied soil layer, down slope toward an unsupported slope face, such as a creek bank, or an inclined slope face. In general, lateral spreading has been observed on low to moderate gradient slopes, but has been noted on slopes inclined as flat as one degree.

The earth materials that underlie the project site are stiff to medium dense or dense. Earth materials with those characteristics pose a low potential of liquefaction.

3.5 EXPANSIVE POTENTIAL

There is a direct relationship between plasticity of a soil and the potential for expansive behavior, with expansive soil generally having a high plasticity. Thus, granular soils typically have a low potential to be expansive, where as, clay-rich soils can have a low to high potential to be expansive. Atterberg limit testing performed on two selected samples recorded plasticity indices (PI) of approximately 5 and 10. These PIs correlate to material having a very low to low expansion potential (Day, 1999).

3.6 SOIL CHEMISTRY

Two selected samples 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 chlorides. Testing was conducted by HDR/Schiff Associates of Claremont and results are presented below, as well as included in the appendix of laboratory results.

Sample	Sample Depth	Sulfates (ppm)	Chlorides (ppm)	pH	Resistivity (ohms-cm)
TP-1	1'–3'	25	6.1	5.4	5,600
TP-4	1'–4'	23	4.4	6.1	6,000

According to the ACI-318, a sulfate concentration below 0.10 percent by weight (1,000 ppm) is negligible. A chloride content of less than 500 ppm is generally considered non-corrosive to reinforced concrete. Minimum resistivity testing performed on the soil sample indicated the soils are considered to be moderately corrosive to buried metal objects. A commonly accepted correlation between soil resistivity and corrosivity towards ferrous metals (NACE Corrosion Basics, 1984) is provided below:

Minimum Resistivity (ohm-cm)	Corrosion Potential
0 to 1000	Severely Corrosive
1,000 to 2,000	Corrosive
2,000 to 10,000	Moderately Corrosive
Over 10,000	Mildly Corrosive

Thus, according to the table above, the soils are estimated to be mildly to moderately corrosive to mildly corrosive based upon the soil resistivity.

4.0 ENGINEERING PROPERTIES OF SIGNIFICANT EARTH MATERIALS

The following section discusses selected engineering properties of critical earth materials that could be encountered during construction of the proposed project. The discussions are based on field observations made during exploration and on laboratory test results. Those data are presented on the exploration logs located in Appendix A. Laboratory test results are presented in Appendix B.

4.1 TERRACE DEPOSITS (ALLUVIUM)

Terrace deposits consist of medium dense to dense, damp, silty sand with gravel, cobbles, and boulders. Cobbles and boulders up to at least 36 inches in maximum dimension were observed during this study. The terrace deposits were encountered to depths of up to 8 feet deep and are anticipated to extend to depths of up to at least 15 feet. Atterberg limit testing performed on terrace deposits yielded a PI of approximately 5 with a Liquid Limit (LL) of about 28, correlating to a low plasticity soil. Grain-size distribution testing found that between 22 and 27 percent of the sample was larger than 1.5" in diameter and that 2.8 to 12.4 percent of the sample consisted of silt and clay. Maximum density and optimum moisture content ranged from about 123.3 pcf and 12 percent, respectively.

4.2 COLLUVIUM

Colluvial deposits consist of stiff to medium dense sandy clay to clayey sand with trace to moderate gravel, cobbles, and boulders. Cobbles and boulders up to at least 24 inches in maximum dimension were observed during this study. The colluvium was encountered to depths of up to 6.5 feet deep and is anticipated to extend to depths of up to at least 10 feet. Atterberg limit testing performed on colluvial deposits yielded a PI of approximately 10 with a Liquid Limit (LL) of about 38, correlating to a medium plasticity soil.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

Based on the results of our investigation, it is our opinion that the site is suitable for the proposed improvements provided recommendations presented, herein, are utilized during design and construction of the project. Specific comments and recommendations regarding the geotechnical aspects of project design and construction are presented in the following sections of this report.

Recommendations presented, herein, are based upon the draft Site Concept plan prepared by communitecture, inc. and Barrett Ecological (2011) and discussions with Brook Leaf of Big Bend Hot Springs Project, LLC. Changes in the configuration from those studied during this investigation may require supplemental recommendations.

5.2 FAULTING

No known active faults pass through the project site. Several faults have been mapped in the vicinity of the project area, including at the project site. The site does not lie within the boundaries of an Alquist-Priolo Earthquake Fault Zone; therefore, it is our opinion that surface rupture potential is low.

5.3 LANDSLIDES

No signs of landsliding, either recent or incipient, were observed on or adjacent to the project property. It is our opinion that naturally occurring landslides pose a low risk to the project. See Sections 5.6.8, 5.7, and 5.13 of this report regarding temporary and man-made slope stability issues.

5.4 LIQUEFACTION POTENTIAL

Based on our observations and material exposed during the investigation, it our opinion that liquefaction and lateral spreading have a relatively low risk of adversely affecting the proposed improvements.

5.5 EXPANSIVE POTENTIAL

Atterberg limit testing performed on selected samples recorded plasticity indices of approximately 5 and 10. These material correlates to material having a very low to low expansion potential (Day, 1999).

5.6 SITE PREPARATION AND GRADING

5.6.1 Stripping

Prior to general site grading and/or construction of planned improvements, existing vegetation, trees, organic topsoil, debris, and deleterious materials should be stripped and disposed of off-site or outside the construction limits. It is anticipated that stripping depths will extend 2 to 6 inches deep, depending on the vegetative cover density and types. In addition, there are a number of trees and shrubs that may have relatively dense accumulations of roots that are laterally and vertically extensive. These root balls could extend deeper than 3 feet below grade and should be removed during stripping. CGI should be allowed to observe stripped areas to confirm that adequate removal of organic, deleterious, and unsuitable materials have been properly stripped and removed from the site.

5.6.2 Existing Utilities, Wells, and/or Foundations

Below-grade utility lines, septic tanks, cesspools, wells, on-site waste disposal fields and tanks, irrigation ponds and/or foundations that are encountered during construction should be removed and disposed of off-site. Buried tanks, if present, should be removed in compliance with applicable regulatory agency requirements. Existing, below-grade utility pipelines (if any) 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.

5.6.3 Scarification and Compaction

Following site stripping and overexcavation, areas to receive engineered fill should be scarified to a minimum 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].

5.6.4 Keying and Benching

The proposed developments are located on relatively flat ground that does not have slope inclinations exceeding 20 percent (5:1, horizontal to vertical). Because of this, it is anticipated that keying and benching will not be required for this project. If improvements are proposed on slopes having gradients steeper than 20 percent, then CGI can provide keying and benching details to use for grading on those slopes.

5.6.5 Wet/Unstable Soil Conditions

If site preparation or grading is performed in the winter, spring, or early summer seasons, shortly after significant precipitation, or in areas having shallow groundwater, near-surface on-site soils may be significantly over optimum moisture content. This condition could hinder equipment access as well as efforts to compact site soils to a specified level of compaction. In addition, perched water can be present in subsurface layers throughout the year and contribute to wet soil conditions. 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 investigation. Therefore, if over-optimum moisture within the soil is encountered during construction, CGI should review these conditions (as well as the contractor's capabilities) and, if requested, provide recommendations for their treatment.

---

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

5.6.6 Site Drainage

Finished grading should be performed in such a manner that provides a minimum of 10 horizontal feet of positive surface gradient away from all structures. The ponding of water should not be allowed adjacent to structures, retaining walls, or the top of fill sections. Interceptor drains should be constructed above all cut and fill slopes to prevent water from flowing over those slopes. Surface runoff should be directed toward engineered collection systems or suitable discharge areas and not allowed to flow onto or over slopes. Discharge from roof downspouts should also be collected, conveyed in solid (unperforated) pipelines, and discharged away from all structures and into engineered systems, such as storm drains. Landscape plantings around structures should be avoided or be dry climate tolerant and require minimal irrigation. Care should be taken to avoid overwatering all landscaping.

5.6.7 Excavation Characteristics & Bulking

Explorations for this project were advanced using a relatively light-duty rubber-tired backhoe. In general, earth materials encountered during this study were penetrated with relatively minimal to moderate effort using this equipment. It is our opinion that soils present at the site should be excavatable using conventional heavy grading equipment operated by experienced personnel. Large cobbles and boulders, if encountered within those soils, could pose difficult excavation conditions and should not be anticipated to be mechanically reduced in size unless a rock crusher is utilized.

Bulking or shrinkage of excavated materials at the project site can be estimated using the following information:

SHRINKAGE & BULKING FACTORS		
Material	**Bulking**	**Shrinkage**
Colluvium & Terrace Deposits	-	2% to 5%[a]

[a] This value does not account for exclusion of large size material

These factors should be included in volume calculations for on-site soils that are excavated then compacted per recommendations within this report.

5.6.8 Temporary & Permanent Slopes

This section explicitly excludes trench slopes for buried utilities. Temporary trench excavations are discussed in Section 5.7.1 of this report.

Temporary construction slopes for keyway and bench construction, and for clean-outs of swales, drainages, and canyons, can be constructed at ½:1 inclinations if the temporary cut slopes are less than 6 feet in height. All other temporary slopes should be constructed no steeper than 1:1.

Permanent slopes should be constructed at inclinations of 2:1 or flatter. In isolated areas where a cut slope is less than 8 feet tall, is adequately protected from erosion, and is not intended to support structures or surcharges, then the cut slope can be constructed at inclinations of 1.5:1 or flatter, per Section J106 of the 2010 CBC.

In order to comply with CBC regulations, minimum setbacks for proposed structures should be equivalent to the height of the slope divided by 3, but need not exceed 40 feet. Minimum setbacks for proposed in-ground hot springs tubs and other pools should be equivalent to height of the slope divided by 6, but not to exceed 20 feet. If the desired setbacks are less than these requirements, then the foundations of the structures should be deepened or opt for alternate setbacks in accordance with requirements of section 1808.7.5 of 2010 CBC.

5.6.9 Overexcavation & Subdrains

Areas of overexcavation were not identified during this study. If, during construction, areas of uncertified fill, or having high concentrations or organics or deleterious materials, are encountered, those materials should be overexcavated and removed. Prior to placement of engineered fill materials within the overexcavations, a CGI engineer or geologist should observe and approve the depth and horizontal extent of overexcavation. Engineered fill placed within the overexcavations should be placed and compacted in accordance with recommendations provided in Section 5.6.13 of this report.

5.6.10 On-Site Soil Materials

It is our opinion that most of the near-surface soils encountered at the site can be used for general engineered fill provided it is free of organics, debris, oversized particles (>3") and deleterious materials. If highly plastic clayey materials (materials having a plasticity index exceeding 30 and a liquid limit in excess of 50) are encountered during grading, those materials should be segregated and excluded from engineered fill, where possible, or thoroughly mixed with granular materials to reduce the plasticity of the soil. If potentially unsuitable soil is considered for use as engineered fill, CGI should observe, test, and provide recommendations as to the suitability of the material prior to placement as engineered fill.

5.6.11 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 small 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:

IMPORTED FILL RECOMMENDATIONS				
GRADATION				
Sieve Size	General Fill	Granular Fill	Test Procedures	
	Percent Passing		ASTM	AASHTO
3-inch	100	100	D422	T88
¾-inch	70 – 100	70 – 100	D422	T88
No. 200	0 - 30	<5	D422	T88
PLASTICITY				
Liquid Limit	<30	NA	D4318	T89
Plastic Index	<12	Nonplastic	D4318	T90
ORGANIC CONTENT	<3%	<3%	D2974	NA

5.6.12 Materials - Granular

All granular fill should consist of imported soil mixtures generally less than 3 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 presented in Section 5.6.11 of this report.

5.6.13 Placement & Compaction

Soil and/or soil-aggregate mixtures used for fill 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[2]. 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. We recommend that CGI be retained to perform compaction testing to verify compliance with our recommendations.

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.

5.7 UTILITY TRENCHS AND TRENCH BACKFILL

5.7.1 Trenches and 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 not observed during our study; however, there is a potential that shallow un-shored trenches excavated with sidewalls steeper than 1:1 could locally slough and/or 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

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

ground surface. Where the stability of adjoining buildings, walls, buried utilities within the trench sidewalls, or other structures 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.

Groundwater might 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. If this is not possible, then the Contractor should channel groundwater to flow towards collection points to be removed from the trench and disposed of at an approved area.

5.7.2 Pipe Zone Backfill

The pipe zone, as discussed herein, is that cross sectional area that extends from the bottom of the trench to 6 inches over the crown of the pipeline, and from trench wall to trench wall. Pipe zone backfill materials should consist of imported soil having an SE of no less than 30 and having a particle size no greater than ½-inch in maximum dimension, per Section 306-1.2.1 of the Greenbook. On-site soils will likely not meet these recommendations.

5.7.3 Trench Zone Backfill

Trench zone backfill (i.e., material placed between the pipe zone backfill and finished subgrade) may consist of on-site soils or imported materials. If on-site soils are used, then those materials should be screened of deleterious materials, organic debris, highly plastic clay, and oversized materials having dimensions of greater then 3 inches in any direction prior to placement within the trench.

Alternatively, imported soils can be used as trench zone backfill. We recommend that imported trench zone materials conform to recommendations presented for imported general engineered fill materials presented in Section 5.6.11 of this report. Those imported materials should be free of deleterious materials, organic debris, or clasts exceeding 3 inches in diameter in any direction.

5.7.4 Controlled Low Strength Backfill

An alternative to the use of pipe zone and trench zone backfill materials noted above is the use of controlled low strength material (CLSM) as pipe and/or trench zone backfill. CLSM consists of a fluid, workable mixture of aggregate, cement, and water that is of limited strength as to allow future excavation and maintenance of buried improvements yet capable of supporting the proposed pipeline and backfill. If CLSM is used in the pipe zone or trench zone, we recommend that those materials conform and be placed according to specifications presented in Section 19-3.062 of the 2006 Caltrans Standard Specifications. Care should be taken during placement of CLSM materials to prevent the pipeline from floating.

5.7.5 Placement & Compaction

Trench backfill should be placed and compacted in accordance with recommendations previously provided for engineered fill. Mechanical compaction should be the means in which compaction is achieved. Jetting should not be allowed as a means of compaction. According to Section 306-1.3.3 of the Greenbook, jetting is not allowed if the trench sidewalls have an SE of less than 15. Most on site soils are clay-rich and should have an SE that is less than 15. Thus, compaction jetting for most of the proposed pipeline alignment would not conform to Greenbook specifications.

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. Compaction tests of pipe zone backfill should be performed at horizontal intervals of no more than 300 feet and vertical intervals of no more than 18 inches. Within the pipe zone, compaction tests should be performed near springline and near the top of the pipe zone backfill. Assessment of the potential presence of voids within the haunch area should be performed following completion of those compaction tests. If voids are observed, then the Contractor should be required to rework the pipe zone materials to eliminate the presence of voids in the pipeline haunches. Retesting of the pipe zone materials should then be performed. All areas of failing compaction tests should be reworked and retested until the specified relative compaction is achieved.

Compaction of trench zone backfill should be performed at horizontal intervals of no more than 300 feet and vertical intervals of no more than 18 inches. If imported trench zone backfill materials are used, then periodic compaction testing services will be required by the geotechnical engineer in order to comply with the testing recommendations noted above.

Placement of CLSM materials should be performed in accordance with specifications presented in Caltrans Standard Specification 19-3.062. If CLSM is used, then compaction tests are not required; however, a minimum of four hours should be allowed between placement of CLSM and placement of engineered fill materials above the CLSM, as noted in Caltrans Standard Specification 19-3.062.

5.7.6 Trench Subgrade Stabilization

Soft and yielding trench subgrade could be encountered along the bottom of trench excavations. 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.

5.7.7 Trench Plugs

The use of relatively clean sand and crushed rock within utility trench backfill can form a path of migration of groundwater through these materials, since they tend to have a higher permeability than the native soils and engineered fill materials. If these materials are used as backfill, a lower permeability plug should be placed and compacted within the trench at regular intervals. We recommend that the lower permeability material consist of grout or a well graded soil with greater than 30 percent passing the No. 200 sieve. The plug should be placed for a length of 3 feet at an interval of about every 300 feet along the length of the trench.

If livable spaces or structures with floors sensitive to moisture are situated at elevations lower than utility pipeline servicing the structures, there is a potential that water might migrate along the pipeline bedding and beneath the structures. We recommend that if such conditions are present, a lower permeability plug should be placed and compacted within the trench at the service stub from the main utility. We recommend that the lower permeability material consist of grout or bentonite. The plug should be keyed into the trench sidewall and bottom a minimum of 6 inches, extend from the trench bottom to the top of trench, and be a minimum of 2 feet wide.

5.8 FOUNDATIONS

5.8.1 Minimum Footing Embedment and Dimensions

Minimum embedment depths, widths, and thicknesses should conform to Table 1809.7 of the CBC, but should be determined by the Structural Engineer. Transition lots, where structures span across engineered fill with variable thicknesses or both native cut materials and engineered fills, can lead to differential settlement issues. **Foundations should not span both cuts and fills unless engineered fill thicknesses are less than one foot thick beneath the bottom of footings**.

Where proposed foundations span both cuts and fills, we recommend that:

- The area of cuts supporting the proposed foundations should be overexcavated below the planned bottom of footings to a depth of at least 3 times the width of the foundation. CGI should observe and approve the overexcavated area once exposed. Overexcavation limits should extend throughout the cut area and to a minimum of five horizontal feet past the perimeter foundations of the structure, as illustrated on Plate 4 – Transition Lot Details. The overexcavated area should then be backfilled in accordance with recommendations presented in Section 5.6.13 of this report; or

- Proposed foundations should be deepened to extend through engineered fill materials to be supported on competent undisturbed native soils, so that the entire foundation system for the structure rests on undisturbed native soils, as illustrated on Plate 4. If this depth is less than 5 feet below the planned bottom of the foundation, then a two-sack sand-cement slurry can be used as backfill in lieu of structural concrete, from the excavation bottom up to the planned bottom of the proposed foundation. CGI should observe and approve the deepened foundation excavation prior to placement of slurry or structural concrete.

Deepened footing excavations should extend below any observed yielding material. If soft, yielding, or unsuitable soil is encountered during construction, CGI should review these conditions (as well as the contractor's capabilities) and, if requested, provide recommendations for their treatment.

Frozen ground was not encountered during our exploration at the site; however, the depth of freezing in the soil for the region, is estimated to be up to 12 inches.

5.8.2 Allowable Bearing Capacity

It is assumed that all structure foundations for the proposed buildings will rest entirely on cut or entirely on engineered fill. The foundations must not be constructed partially on fill and partially on cut. Isolated and continuous footing elements should be proportioned for dead loads plus probable maximum live load, and a maximum allowable bearing pressure of the following:

ALLOWABLE BEARING CAPACITIES			
Material	**Allowable Bearing Capacity (psf)**	**Increase per Foot of Embedment (psf)**	**Maximum Allowable Bearing Capacity (psf)**
Alluvium	2,000	250	3,000
Colluvium	1,500	150	2,250
Engineered Fill	1,500	150	2,250

The allowable bearing pressures provided are net values. 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 and includes a calculated factor of safety of at least 3. An increase of allowable bearing pressure by one-third for short-term loading due to wind or seismic forces should NOT be incorporated unless an alternative load combination, as described in Section 1605.3.2 of the 2010 CBC, is applied. The allowable bearing value is for vertical loads only; eccentric loads may require adjustment to the values recommended above.

5.8.3 Lateral Earth Pressures

Subsurface structures should be designed to resist the earth pressure exerted by the retained, compacted backfill plus any additional lateral force that will be applied due to surface loads placed at or near the wall or below-grade structure. Recommended design criteria for subsurface structures are presented below:

The recommended equivalent fluid weights presented below are for static (non-earthquake) conditions with the ground level or inclined at 2:1 behind the shoring system.

CGI: Copyright 2011

LATERAL EARTH PRESSURES UNDER STATIC CONDITIONS		
Lateral Earth Pressure Condition	**Slope Inclination Above Structure**	**Equivalent Fluid Weight (pcf) Moist to Wet Conditions**
At-Rest	Flat	75
	2:1	90
Active	Flat	50
	2:1	65

The resultant force of the static lateral force prism should be applied at a distance of 30 percent of the wall height above the soil elevation on the toe side of the wall. The tabulated values are based on a soil unit weight of 125 pounds per cubic foot (pcf), and do not provide for surcharge conditions resulting from construction materials, equipment, or vehicle traffic. Loads not considered as surcharges should bear behind a 1:1 (horizontal to vertical) line projected upward from the base of the shoring. If surcharges are expected, CGI should be advised so that we can provide additional recommendations as needed.

5.8.4 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.

5.8.5 Estimated Settlements

The proposed structures should not rest partially on fill and partially on cut. All foundations are anticipated to rest on native soils or engineering fill. 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 is anticipated to be less than ½ -inch in 20 feet.

5.8.6 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.

5.9 SLIDING AND PASSIVE RESISTANCE

5.9.1 Sliding Resistance

Ultimate sliding resistance generated through a compacted soil/concrete interface can be computed by:

- Multiplying the soil/concrete adhesion (150 psf) by the footing contact area for cohesive soils. In no case shall the lateral sliding resistance exceed one-half the dead load; or
- Multiplying the total dead weight structural loads by the friction coefficient of 0.30 for imported granular engineered fill.

5.9.2 Passive Resistance

Ultimate passive resistance developed from lateral bearing of shallow foundation elements bearing against compacted soil 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 150 pcf. Passive resistance of the upper one foot of the soil column should be neglected.

5.9.3 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.

5.10 INTERIOR CONCRETE FLOOR SLABS SUPPORTED ON-GRADE

5.10.1 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 slab 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.

5.10.2 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 90 percent relative compaction.

5.10.3 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.[3] 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. For this project, flooring materials on slabs-on-grade

[3] 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.

are unknown. It is our recommendation that American Concrete Institute (ACI) guidelines ACI 302 and ACI 360 be referred to regarding installation of vapor barriers based on the anticipated flooring materials to be installed.

A capillary break and/or vapor barrier may not be 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.

5.11 EXTERIOR CONCRETE SLABS SUPPORTED-ON-GRADE

Subgrade soils supporting exterior concrete slabs[4] should be scarified to a minimum depth of 1-foot, uniformly moisture-conditioned to near the optimum moisture content and compacted to at least 90 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.

5.12 RETAINING WALLS

5.12.1 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 are presented in section 5.8.3 of this report.

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 125 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.

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

5.12.2 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, as shown on Plate 5 – Retaining Wall Details. The two-foot thick layer can be decreased to one foot in thickness if wrapped with a geosynthetic filter fabric, as discussed on Plate 6; however, the structural engineer should be consulted to confirm that the retaining wall is design to withstand potential increased stresses due to compaction closer to 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.

5.12.3 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:

$$P_E = 3/8 * pga * Y_t * H^2$$

Where:

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

Peak ground acceleration (pga) values for the site are provided in Section 3.2.2 of this report. 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 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.

5.12.4 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.

5.12.5 Retaining Wall Differential Settlement

Retaining walls that span across cut-fill lines have the potential to experience differential settlement much like structures, as discussed in Section 5.8.1 of this report. Differential settlement of walls can result in cracking and deformation of the walls, whether they consist of concrete cantilever, segmental block, or other retaining wall systems. Where proposed retaining wall foundations span both cuts and fills, we recommend that either: 1) recommendations made in Section 5.8.1 be performed; 2) control joints be established in the retaining walls at the cut-fill daylight line location; or 3) the retaining wall be designed by a structural engineer to be sufficiently rigid to resist stresses induced by anticipated differential settlement along the retaining wall.

5.13 SHORING CONSIDERATIONS

If shoring systems are utilized in this project, they should be designed to resist earth pressures exerted by the retained soils plus any additional lateral force that will be applied to the shoring due to surface loads placed at or near the excavation. Retaining systems that are free to rotate or translate laterally (for example, cantilevered retaining walls) through a horizontal distance to shoring height ratio of no less than 0.004 are referred to as unrestrained or yielding retaining structures. Such shoring systems can generally move enough to develop active conditions. Retaining systems that are unable to rotate or deflect laterally (for example, restrained basement walls) are referred to as restrained or non-yielding. If such shoring systems cannot move or translate very much, then at-rest conditions develop.

Recommended equivalent fluid weights for active and at-rest conditions are presented in Section 5.8.3.

6.0 REVIEW OF PLANS AND SPECIFICATIONS

We recommend CGI conduct a general review of final plans and specifications to evaluate that recommendations contained herein have been properly interpreted and implemented during design. In the event that CGI is not retained to perform this recommended review, we will assume no responsibility for misinterpretation of our recommendations.

7.0 GEOTECHNICAL OBSERVATION AND TESTING DURING GRADING

This report was based, in part, upon review of data obtained from a limited number of observations, site visits, soil excavations, samples, and tests. Such information is, by necessity, incomplete. The nature of many sites is such that differing soils or geologic conditions can be experienced within small distances and under varying climatic conditions. Changes in subsurface conditions can and do occur over time. Therefore, the findings, conclusions, and recommendations presented in this report are only valid if CGI has the opportunity to observe subsurface conditions during grading in order to confirm that our collected data are representative for the site.

Geotechnical observation and testing should be conducted at the following stages:

- Upon completion of clearing and grubbing;
- During all phases of rough grading, including removals, benching and fill operations, keyway excavation, material and pad overexcavation, and cut slope excavation;

- During installation of subdrains and filter materials;
- During excavation of footings for foundations and retaining walls;
- During trench and retaining wall backfill operations;
- During roadway subgrade and aggregate base placement and compaction; and
- When any conditions are encountered during grading that vary from the conditions described in this report.

8.0 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. The recommendations provided in this report are based on the assumption that an adequate program of tests and observations, as described in Section 7.0, will be conducted by CGI during the construction phase in order to evaluate compliance with our recommendations.

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 Section 1.1 – Project Understanding). Soil and rock deposits can vary in type, strength, and other geotechnical properties between points of observation and exploration. Additionally, groundwater and soil moisture conditions can also vary seasonally and for other reasons. Therefore, we do not and cannot have a complete knowledge of the subsurface conditions underlying the project site. The conclusions and recommendations presented in this report are based upon the findings at the point of exploration, and interpolation and extrapolation of information between and beyond the points of observation, and are subject to confirmation based on the conditions revealed by construction. If conditions encountered during construction 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.

The scope of services provided by CGI for this project did not include the investigation and/or evaluation of toxic substances, or soil or groundwater contamination of any type. If such conditions are encountered during site development, additional studies may be required. Further, services provided by CGI for this project did not include the evaluation of the presence of critical environmental habitats or culturally sensitive areas.

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 significant 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.

-- ◆ --

REFERENCES

Barrett Ecological (2011), Site Concept, Big Bend Hot Springs Project, 25322 Hot Springs Road, Big Bend, CA 96011, draft figures labeled XX.X and dated March 11, 2 sheets.

BNI Building News (2009), *Standard Specifications for Public Works Construction "Greenbook"*, Anaheim.

California Building Code (2010), International Code Council.

California Department of Transportation (2006), Standard Specifications.

Communitecture, inc. (2011), Site Concept, Big Bend Hot Springs Project, 25322 Hot Springs Road, Big Bend, CA 96011, draft figures labeled XX.X and dated March 11, 2 sheets.

Day, R. (1999), Geotechnical and Foundation Engineering, Design and Construction, McGraw – Hill, New York, NY 10121-2298.

Dupras, D. (1997), Mineral Land Classification of Alluvial, Sand and Gravel, Crushed Stone, Volcanic Cinders, Limestone, and Diatomite Within Shasta County, California, California Division of Mines and Geology Open File Report 97-03, p. 14 plus plates.

Hart, E.W. and Bryant, W.A. (1997), Fault-Rupture Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zone Maps, California Division of Mines and Geology Special Publication 42, with supplements 1 and 2 added in 1999, 38 p.

Helley, E.J. and Hardwood, D.S. (1985), Geologic Map of the Late Cenozoic Deposits of the Sacramento Valley and Northern Sierran Foothills, California, United States Geological Survey, Miscellaneous Field Studies Map MF-1790, scale 1:62,500.

Irwin, W.P. (1994), Geologic Map of the Klamath Mountains, California and Oregon, U.S. Geological Survey Miscellaneous Investigations Series Map I-2148, Scale 1:500,000.

Hinds, N.E. (1952), Evolution of the California Landscape, California Division of Mines and Geology Bulletin 158, pp 145-152.

Jennings, C.W. (1994), Fault Activity Map of California and Adjacent Area, with Locations and Ages of Recent Volcanic Eruptions, California Division of Mines and Geology, Geologic Data Map No. 6, Scale 1:750,000.

Lydon, P.A., and O'Brien, J.C. (1974), Mines and Mineral Resources of Shasta County, California, California Division of Mines and Geology County Report 6, p. 12.

NACE (1984), Corrosion Basics – An Introduction, National Association of Corrosion Engineers (NACE), Houston, TX.

Nadim, F., and Whitman, R.V. (1992), Coupled Sliding and Tilting of Gravity Retaining Walls During Earthquakes, in Proceedings of 8th World Conference on Earthquake Engineering, San Francisco.

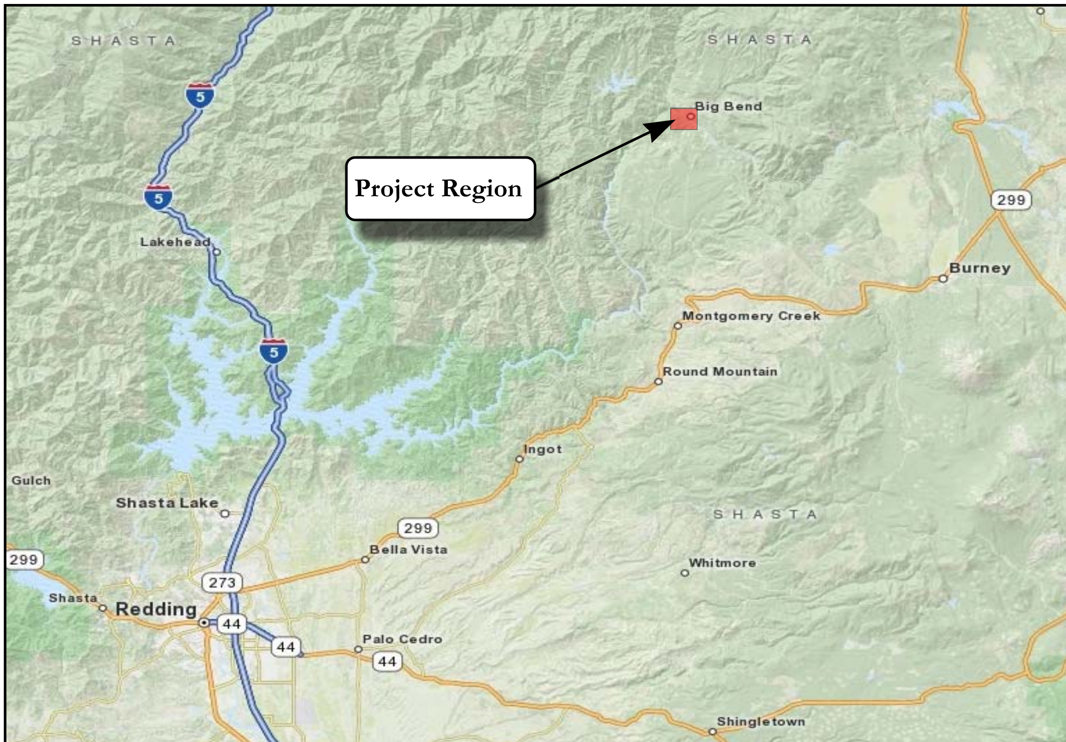
Northstar (2010), Big Bend Hot Springs, Topographic Survey, Sheet 1 of 1, dated November 5.

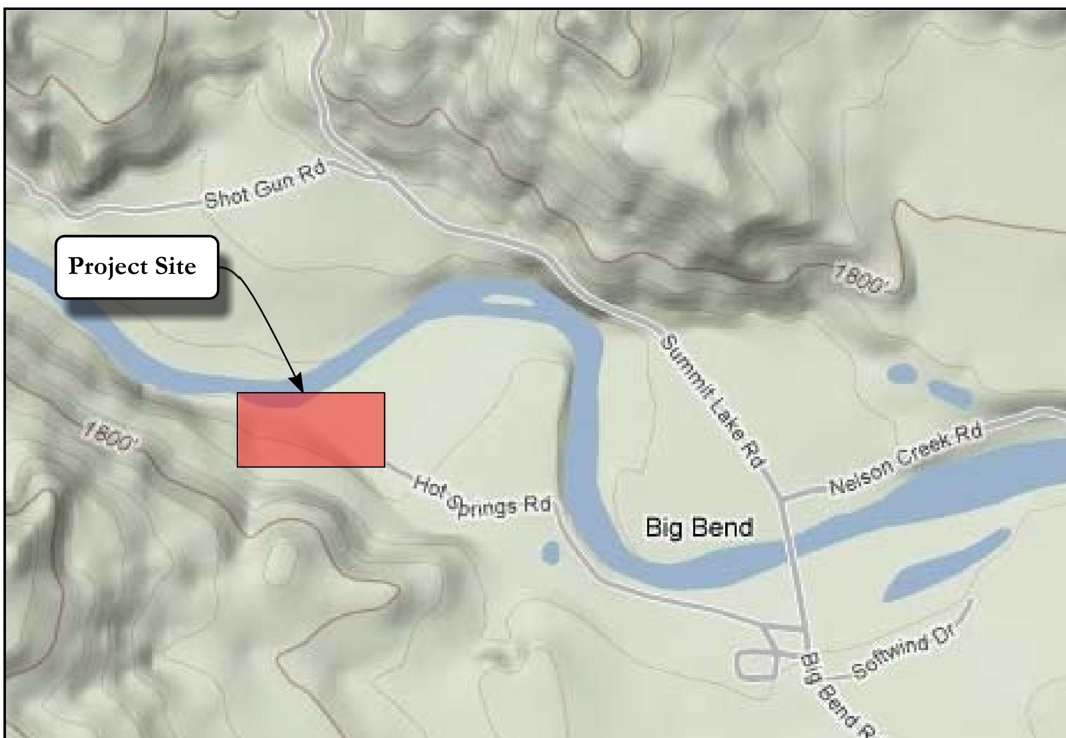
Sanborn, A.F. (1960), Geology and Paleontology of a Part of the Big Bend Quadrangle, Shasta County, California, California Division of Mines and Geology Special Report 63, 26 p.

Seed, H.B., and Whitman, R. (1970), Design of Earth Retaining Structures for Dynamic Loads, ASCE Specialty Conference on Lateral Stresses in the Ground and Design of Earth Retaining Structures, p. 103-147.

Toppozada and Branum (2002), *Bulletin of the Seismological Society of America*; October 2002; v. 92; no. 7; p. 2555-2601.

Toppozada, T. R. and D. Branum (2002), *California M >= 5.5 earthquakes, history and areas damaged*, in Lee, W. H., Kanamori, H. and Jennings, P., International Handbook of Earthquake and Engineering Seismology, International Association of Seismology and Physics of the Earth's Interior.





CGI TECHNICAL SERVICES INC. Project No.: 11-1995.01	SITE LOCATION MAP BIG BEND HOT SPRINGS PROJECT SHASTA COUNTY, CALIFORNIA	Plate 1

Scale Not Determined

Existing Structures

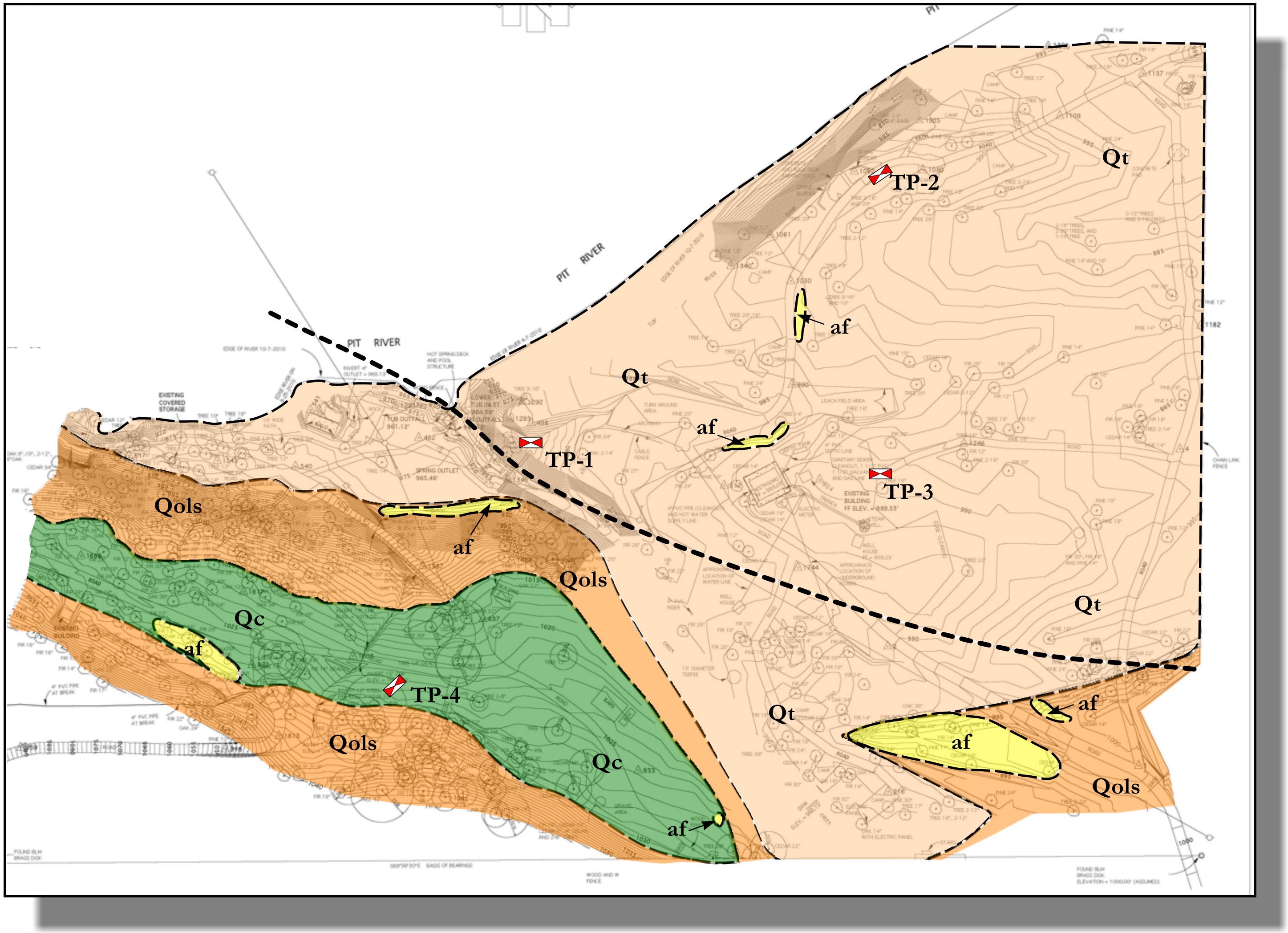
Proposed Structures

CGI TECHNICAL SERVICES INC.

Project No.: 11-195.01

PROJECT ELEMENTS
BIG BEND HOT SPRINGS PROJECT
SHASTA COUNTY, CALIFORNIA

Plate 2



Scale Not Determined

af Artificial Fill

Qt Terrace Deposits

Qc Colluvium

Qols Older Landslide Deposits

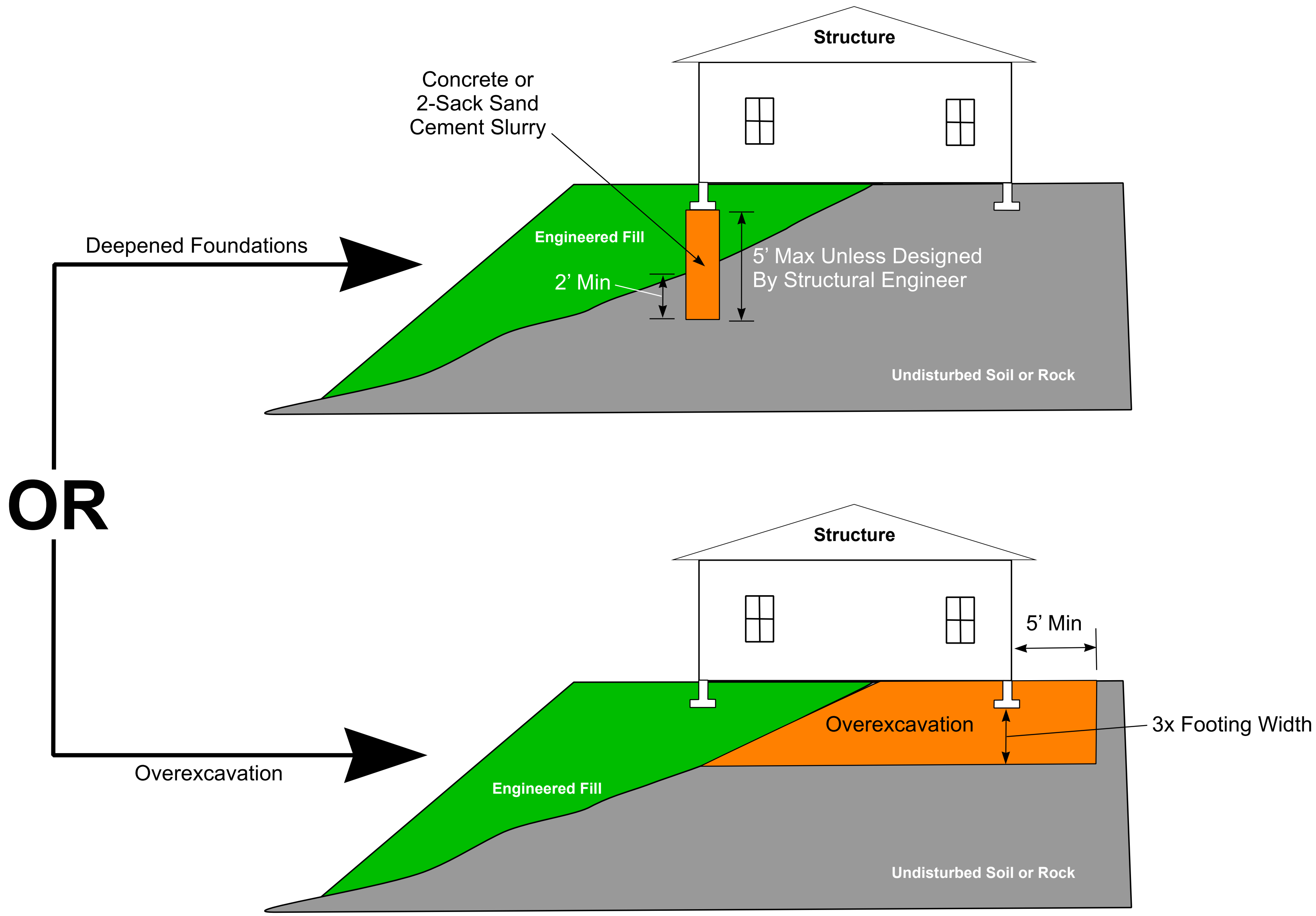
Geologic Contact: dashed where approximate, dotted where covered, queried where uncertain

Fault: showing dip of fault and trend of striae on fault surface (arrow); bar and ball on downthrown side; dashed where approximate, dotted where concealed; queried where uncertain

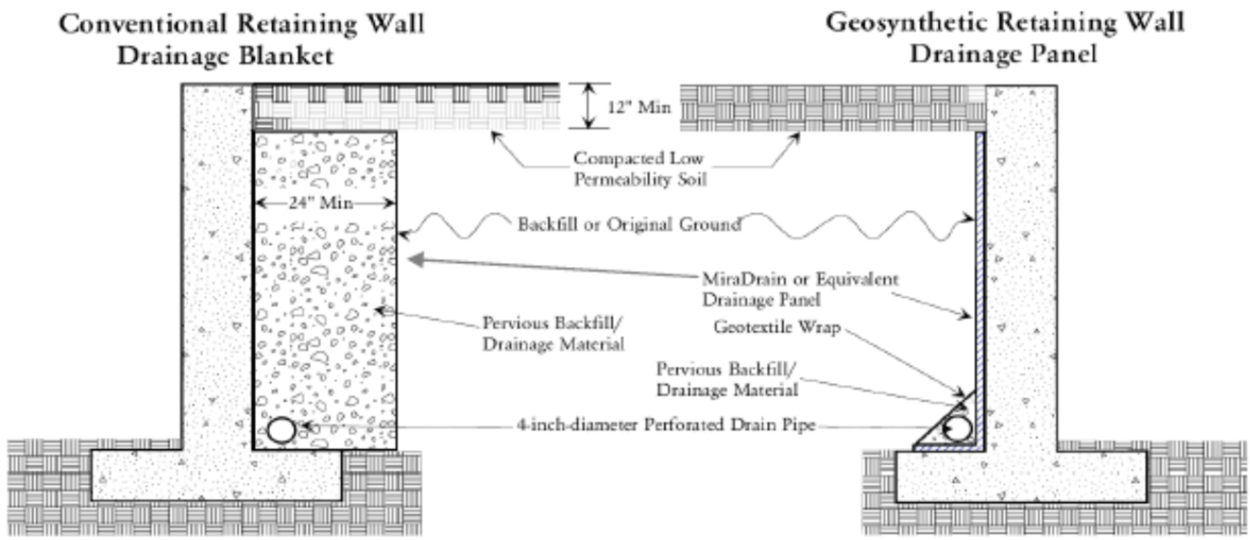
TP-4 Test Pit Locations

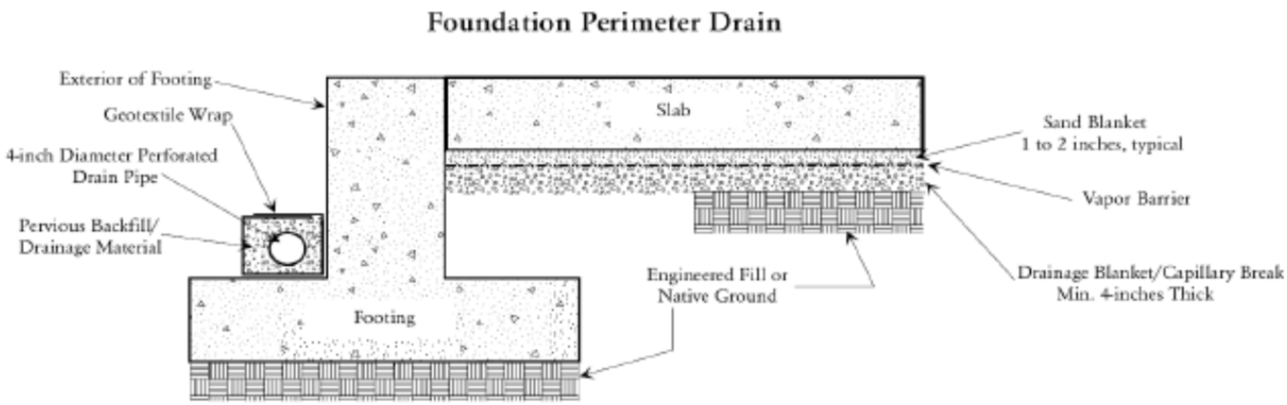
CGI TECHNICAL SERVICES INC.	GEOTECHNICAL MAP BIG BEND HOT SPRINGS PROJECT SHASTA COUNTY, CALIFORNIA	Plate 3
Project No.: 11-195.01		

Methods to Reduce Potential for Differential Settlement on Transition Lots



CGI TECHNICAL SERVICES INC. Project No.: 11-1995.01	TRANSITION LOT DETAILS BIG BEND HOT SPRINGS PROJECT SHASTA COUNTY, CALIFORNIA	Plate 4





General Notes

Pervious backfill/drainage material should conform to Pervious Backfill per Greenbook specifications, Class 2 Permeable Material per Caltrans Standard Specifications, pea gravel having a nominal 1/4-inch diameter, or crushed stone sized between 1/4-inch and 1/2-inch.

Geosynthetic wrapping material should conform to Caltrans Standard Specifications Section 88, placed per manufacturer's specifications.

Perforated drain pipe should consist of 4-inch diameter Schedule 40 PVC, with two sets of 1/4-inch (maximum) diameter perforations drilled axially at 90 degrees to each other, with at least one perforation per line spaced at 12 inches, and the perforations installed facing downward.

Drainage should be collected in a solid conduit and diverted to a proper, approved drainage facility.

CGI Technical Services Inc.	WALL AND DRAIN DETAILS BIG BEND HOT SPRINGS PROJECT SHASTA COUNTY, CALIFORNIA	Plate 5
Project No.: 11-1995.01		

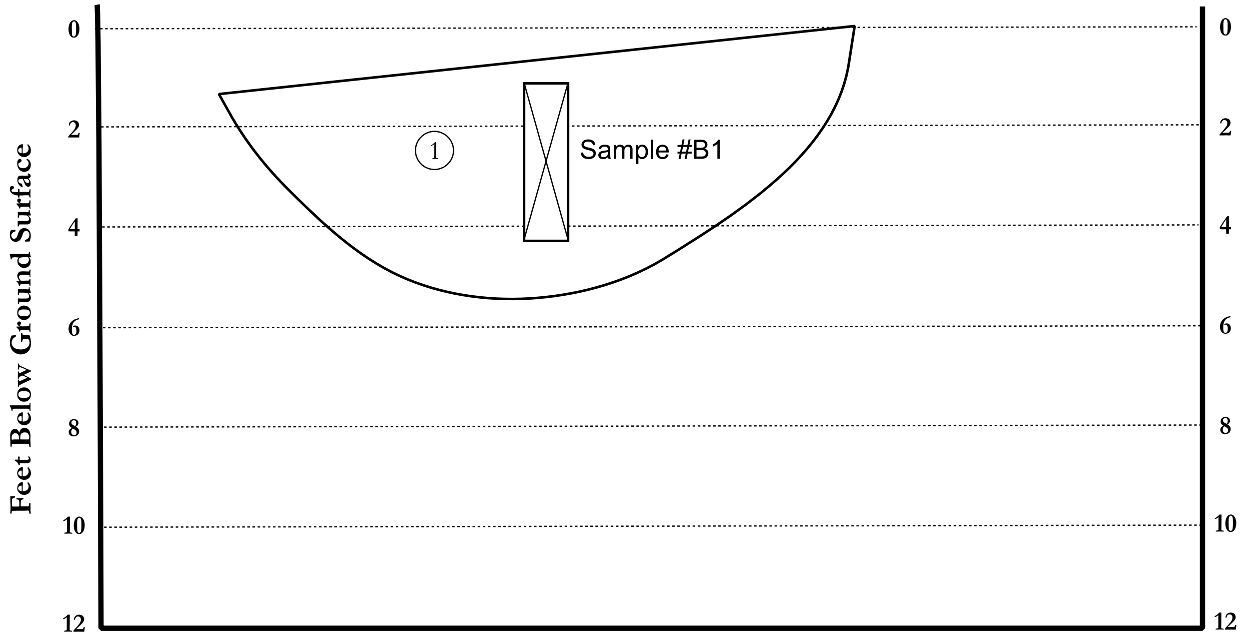
APPENDIX A
SUBSURFACE EXPLORATION

The subsurface exploration program for the proposed project consisted of excavating and logging of four exploratory test pits. Test pit locations are shown on Plate 3.

The test pits were excavated on June 21, 2011 using a Kubota C39 tractor equipped with a backhoe attachment. The test pits were excavated to depths ranging from approximately 4.5 to 6.5 feet below the existing ground surface. Select samples of surficial soils were collected from the test pits for laboratory classification and testing. The results of the testing procedures are attached within Appendix B.

The exploration logs describe the earth materials encountered. The logs also show the location, exploration number, date of exploration, and the names of the logger and equipment used. A CGI geotechnical engineer, using ASTM 2488 for visual soil classification, logged the explorations. The boundaries between soil types shown on the log are approximate because the transition between different soil layers may be gradual and may change with time. Excavation logs for this study are presented as Plate A-1.1 through A-1.4. A legend to the test pits logs is presented as Plate A-2.

LOG OF TEST PIT



Soil Descriptions	
1	TERRACE DEPOSITS (Qt) Silty SAND with Gravel (SM), dark brown, damp, medium dense to dense, with fine to coarse sand, moderate subrounded fine to coarse gravel, cobbles, and boulders up to at least 36 inches in maximum dimension and with minor slightly plastic clay. Fine to medium roots to a depth of 18 inches.

Date Logged:	June 21, 2011	Excavated With:	Kubota C39
Logged by:	Jim Bianchin	Backfilled With:	Excavated Soils
Excavator:	NA	Depth to Water (ft):	Not Encountered

CGI TECHNICAL SERVICES INC.

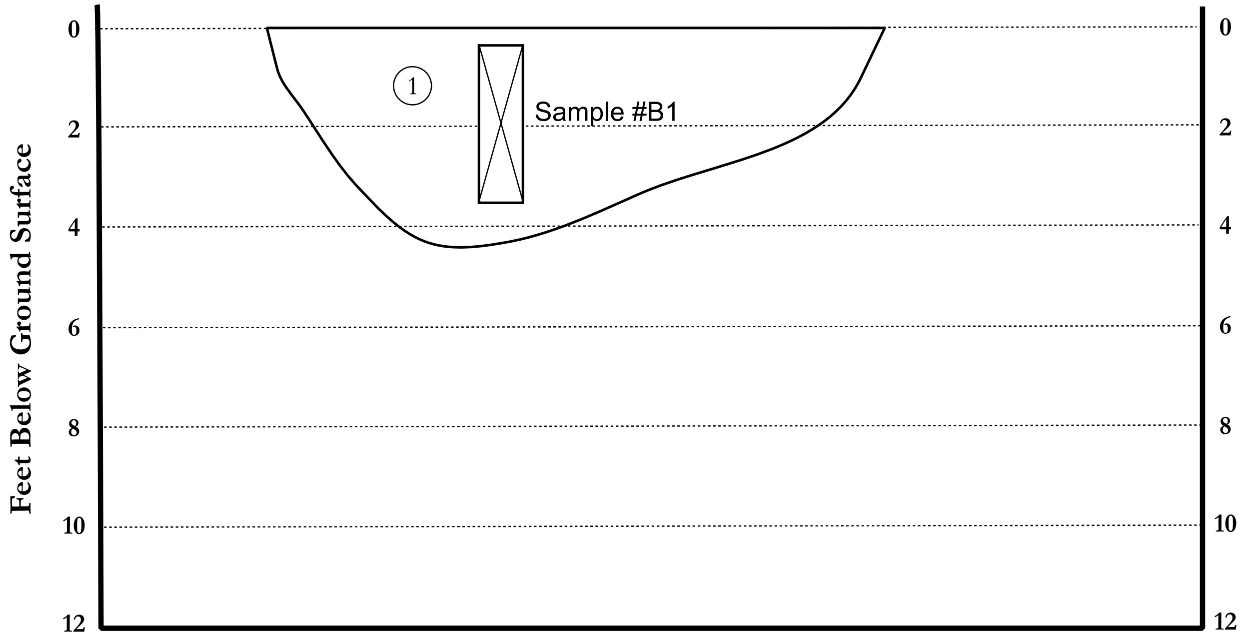
Project No.: 11-1995.01

TEST PIT TP-1
BIG BEND HOT SPRINGS PROJECT
SHASTA COUNTY, CALIFORNIA

Plate No.

A-1.1

LOG OF TEST PIT



Soil Descriptions	
1	TERRACE DEPOSITS (Qt) Silty fine SAND to fine Sandy SILT (ML), moderate brown, damp, medium dense to dense, with subrounded to rounded fine to coarse gravel, cobbles, and boulder up to at least 32 inches in maximum dimension.

Date Logged:	June 21, 2011	Excavated With:	Kubota C39
Logged by:	Jim Bianchin	Backfilled With:	Excavated Soils
Excavator:	NA	Depth to Water (ft):	Not Encountered

CGI TECHNICAL SERVICES INC.

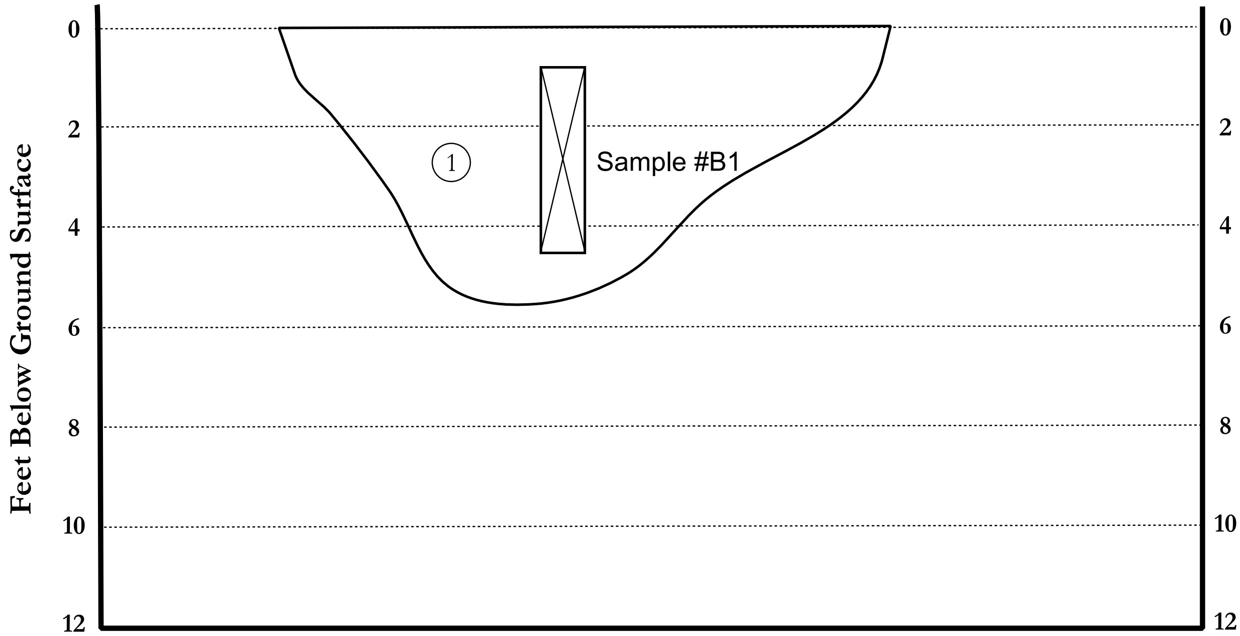
Project No.: 11-1995.01

TEST PIT TP-2
BIG BEND HOT SPRINGS PROJECT
SHASTA COUNTY, CALIFORNIA

Plate No.

A-1.2

LOG OF TEST PIT



Soil Descriptions	
1	TERRACE DEPOSITS (Qt) Silty SAND with Gravel to Sandy GRAVEL (SM/GP), moderate dark brown, damp, medium dense to dense, with fine to coarse sand, moderate subangular to subrounded fine to coarse gravel, cobbles, and boulders up to at least 28 inches in maximum dimension. Fine to medium roots to a depth of 18 inches.

Date Logged:	June 21, 2011	Excavated With:	Kubota C39
Logged by:	Jim Bianchin	Backfilled With:	Excavated Soils
Excavator:	NA	Depth to Water (ft):	Not Encountered

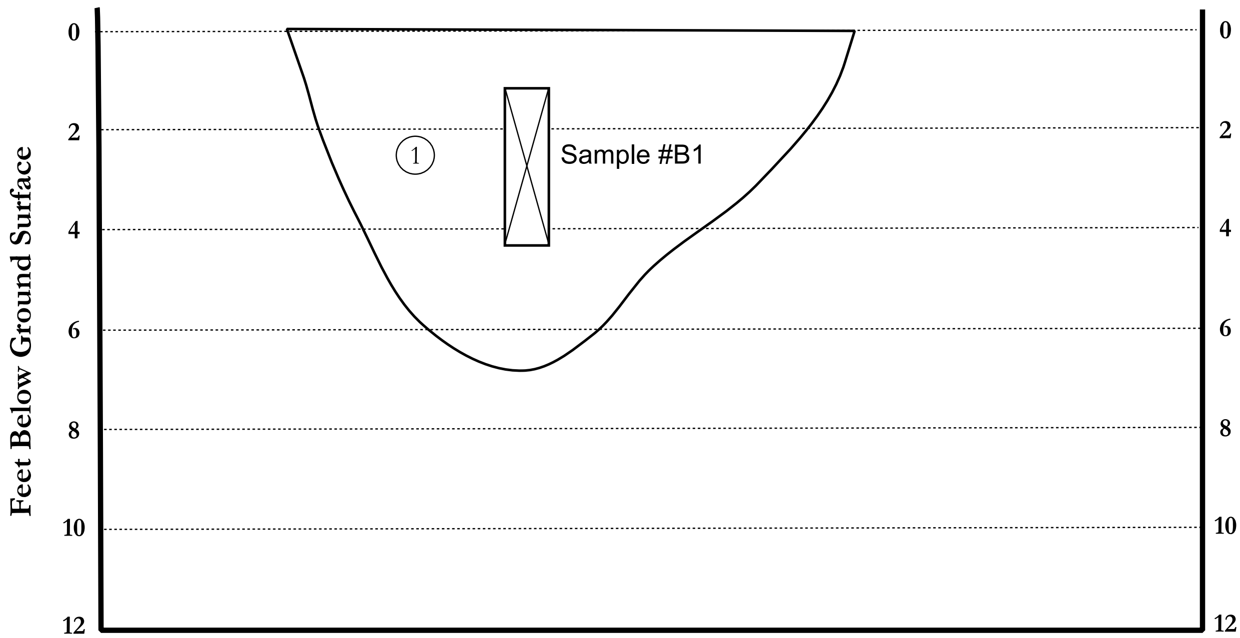
CGI TECHNICAL SERVICES INC.

Project No.: 11-1995.01

TEST PIT TP-3
BIG BEND HOT SPRINGS PROJECT
SHASTA COUNTY, CALIFORNIA

Plate No.
A-1.3

LOG OF TEST PIT



Soil Descriptions	
1	COLLUVIUM (Qc) Sandy CLAY to Clayey SAND (CL/SC), moderate brown, damp, stiff to medium dense, slightly plastic, with fine sand, minor subrounded fine to medium gravel, cobbles, and boulders up to at least 24 inches in maximum dimension. At 5 feet: moderate yellowish brown, increased clay content.

Date Logged:	June 21, 2011	Excavated With:	Kubota C39
Logged by:	Jim Bianchin	Backfilled With:	Excavated Soils
Excavator:	NA	Depth to Water (ft):	Not Encountered

CGI TECHNICAL SERVICES INC.

Project No.: 11-1995.01

TEST PIT TP-4
BIG BEND HOT SPRINGS PROJECT
SHASTA COUNTY, CALIFORNIA

Plate No.
A-1.4

Major Divisions			USCS Symbol	Description
COARSE-GRAINED SOILS More than 50% of sample or material is larger than the No. 200 Sieve (0.0029 inches)	GRAVELS More than 50% of the coarse fraction is retained on No. 4 sieve (0.187 inches)	GRAVELS Clean Gravels, few fines	GW	Well graded gravels and sand mixtures with little to no fines
			GP	Poorly graded gravels & gravel/sand mixtures with little to no fines
		GRAVELS With appreciable fines	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 No. 4 sieve (0.187 inches)	SANDS Clean Sands, few 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 appreciable fines	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 No. 200 Sieve (0.0029 inches)	SILTS & CLAYS Liquid limit less than 50		ML	Inorganic silts with 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 low plasticity
	SILTS & 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	Orgainic silts and clays with high plasticity
HIGHLY ORGANIC SOIL			PT	Peat, humus, swamp soil with high organic content

Samples

☒ Bulk or disturbed sample

■ Relatively undisturbed sample

Symbols

▼ Groundwater

------ Contact Between Soil/Rock Layers

Caving

GENERAL NOTES
Dual symbols (such as ML/CL or SM/SC) are used to indicate borderline classifications.
In general, USCS designations shown on the logs were evaluated using 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 on logs are approximate. Actual transitions may be gradual and vary with depth.

CGI TECHNICAL SERVICES INC.	LEGEND TO TEST PIT LOGS BIG BEND HOT SPRINGS PROJECT SHASTA COUNTY, CALIFORNIA	Plate No. A-2
Project No.: 11-1995.01		

APPENDIX B
LABORATORY TESTING

Laboratory Analyses

Laboratory tests were performed on selected bulk soil samples to estimate engineering characteristics of the various earth materials encountered. 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.

Plasticity Index Tests

Atterberg Limits (plastic limit, liquid limit, and plasticity index) tests were performed on two selected samples in accordance with standard test method ASTM D4318. Results of the Atterberg Limits tests are presented in the report text and on the attached plate labeled *Plasticity Index Tests*.

Grain Size Distribution

Grain size distribution was determined for two selected soil samples in accordance with standard test method ASTM D1140. The grain size distribution data are shown on the attached plates labeled *Laboratory Sieve Analysis*.

Corrosion Testing

Soil chemistry tests were performed to evaluate the resistivity, pH, chloride, and sulfate concentrations within two samples of on-site soils. The results of the test are attached to this appendix.

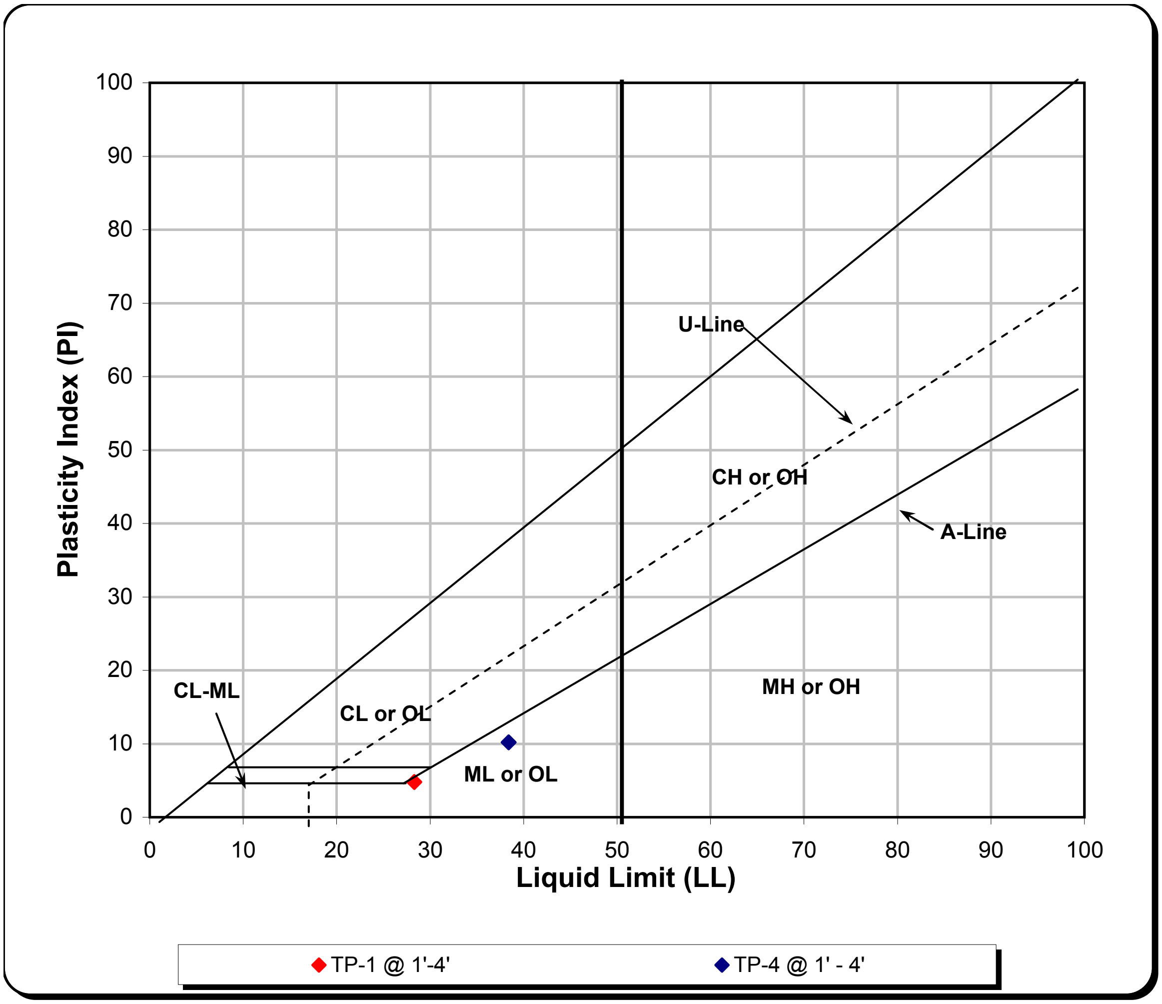
Moisture Density Relations

The compaction characteristics of one selected bulk soil sample were estimated in accordance with standard test method ASTM D1557. The results of the compaction test are shown on the attached plate labeled *Moisture Density Relationship*.

CGI TECHNICAL SERVICES INC.

ATTERBERG LIMITS TESTS

Client:	Big Bend Hot Springs Project LLC	**Job No.:**	11-1995.01
Project:	Big Bend Hot Springs	**Lab No.:**	4774
Location:	Sjhasta County, CA		
Sampled By:	JAB	**Date Sampled:**	21-Jun-11
Received By:	JS	**Date Received:**	21-Jun-11
Tested By:	JS	**Date Tested:**	7-Jul-11
Reviewed By:	JAB	**Date Reviewed:**	7-Jul-11



LEGEND			CLASSIFICATION	ATTERBERG LIMITS TEST RESULTS		
Location	Depth, ft	Sample No.		Liquid Limit (LL)	Plastic Limit (PL)	Plasticity Index (PI)
TP-1	1' - 4'	B1	Silty Sand	28.3	23.5	4.8
TP-4	1' - 4'	1	Clayey Silt	38.4	28.2	10.2

ASTM D4318 & D2487

CGi: Copyright 2009

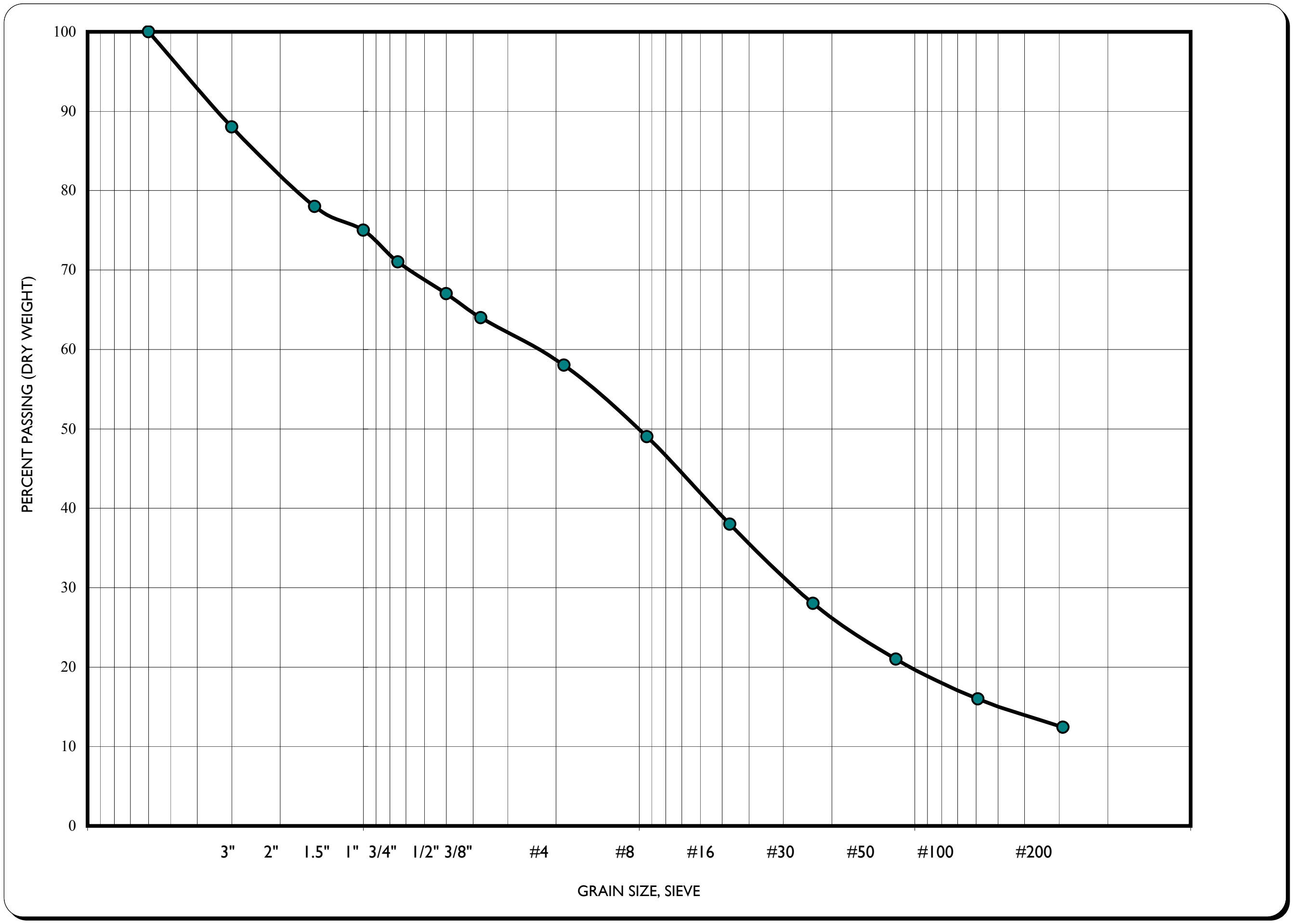
CGi Technical Services, Inc.

LABORATORY TEST RESULTS

Client: Big Bend Hot Springs Project LLC
Project: Big Bend Hot Springs
Material Type: Silty sand with gravel
USCS: SM

Material Source: Native
Sample Location: TP-1
Sampled By: JAB
Tested By: JS

Job No.: 11-1995.01
Lab No.: 4774
Date Received: 21-Jun-11
Date Tested: 20-Jul-11
Date Reviewed: 20-Jul-11



SIEVE ANALYSIS		
Sieve Size Standard	Grain Size (mm)	Percent Passing
6	150.00	100
3	75.00	88
1.5	37.50	78
1"	25.00	75
3/4"	19.00	71
1/2"	12.50	67
3/8"	9.50	64
#4	4.75	58
#8	2.36	49
#16	1.18	38
#30	600um	28
#50	300um	21
#100	150um	16
#200	75um	12.4

CGi: Copyright 2009

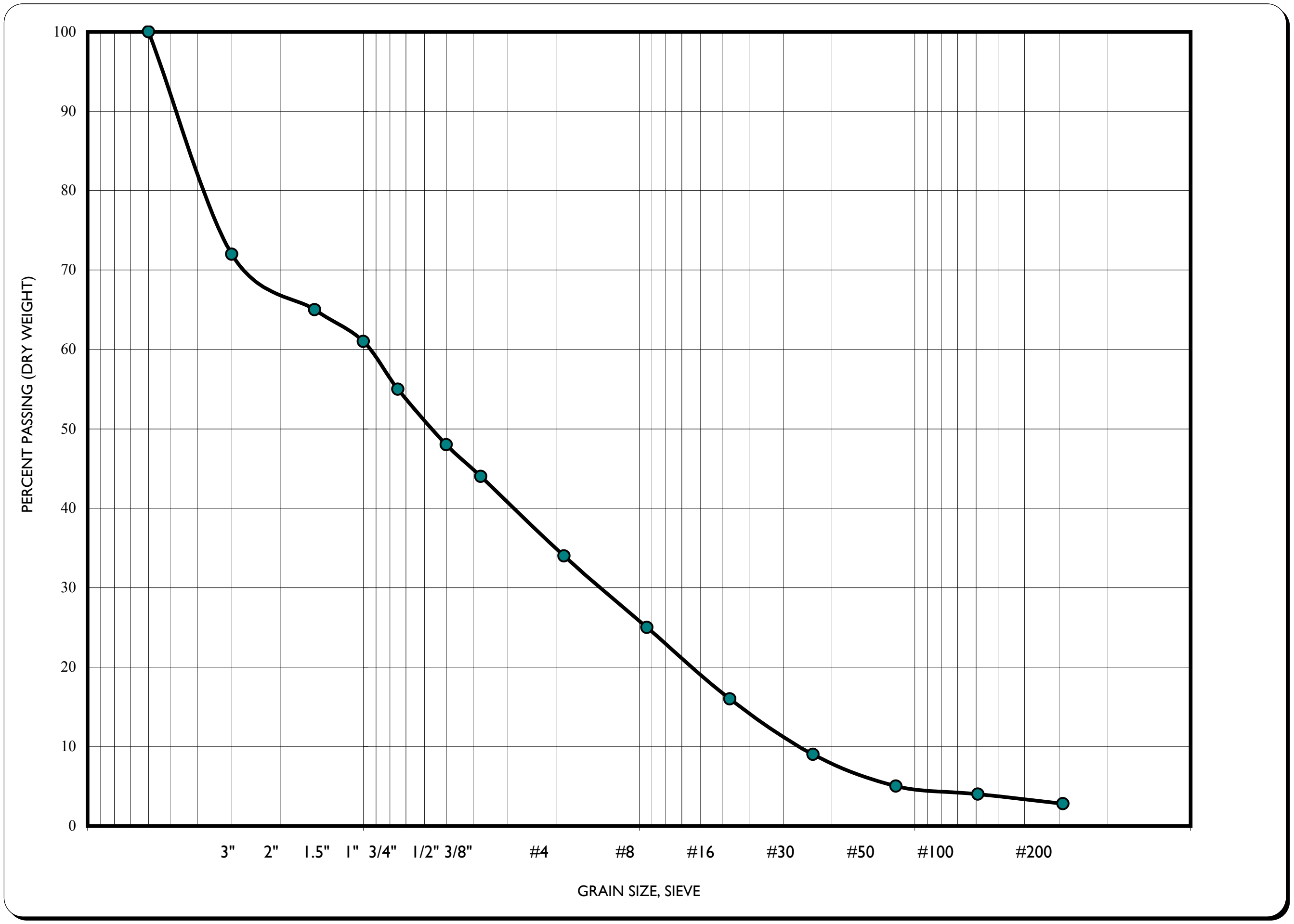
CGi Technical Services, Inc.

LABORATORY TEST RESULTS

Client: Big Bend Hot Springs Project LLC
Project: Big Bend Hot Springs
Material Type: Sand with gravel
USCS: SW

Material Source: Native
Sample Location: TP-3
Sampled By: JAB
Tested By: JS

Job No.: 11-1995.01
Lab No.: 4774
Date Received: 21-Jun-11
Date Tested: 20-Jul-11
Date Reviewed: 20-Jul-11



SIEVE ANALYSIS		
Sieve Size Standard	Grain Size (mm)	Percent Passing
6	150.00	100
3	75.00	72
1.5	37.50	65
1"	25.00	61
3/4"	19.00	55
1/2"	12.50	48
3/8"	9.50	44
#4	4.75	34
#8	2.36	25
#16	1.18	16
#30	600um	9
#50	300um	5
#100	150um	4
#200	75um	2.8

CGi: Copyright 2009

CGI Technical Services Inc.

1612 Wedding Way
Redding, California 96003
530-244-6277
530-244-6276 FAX
www.CurryGroup.com

MOISTURE DENSITY RELATIONSHIP

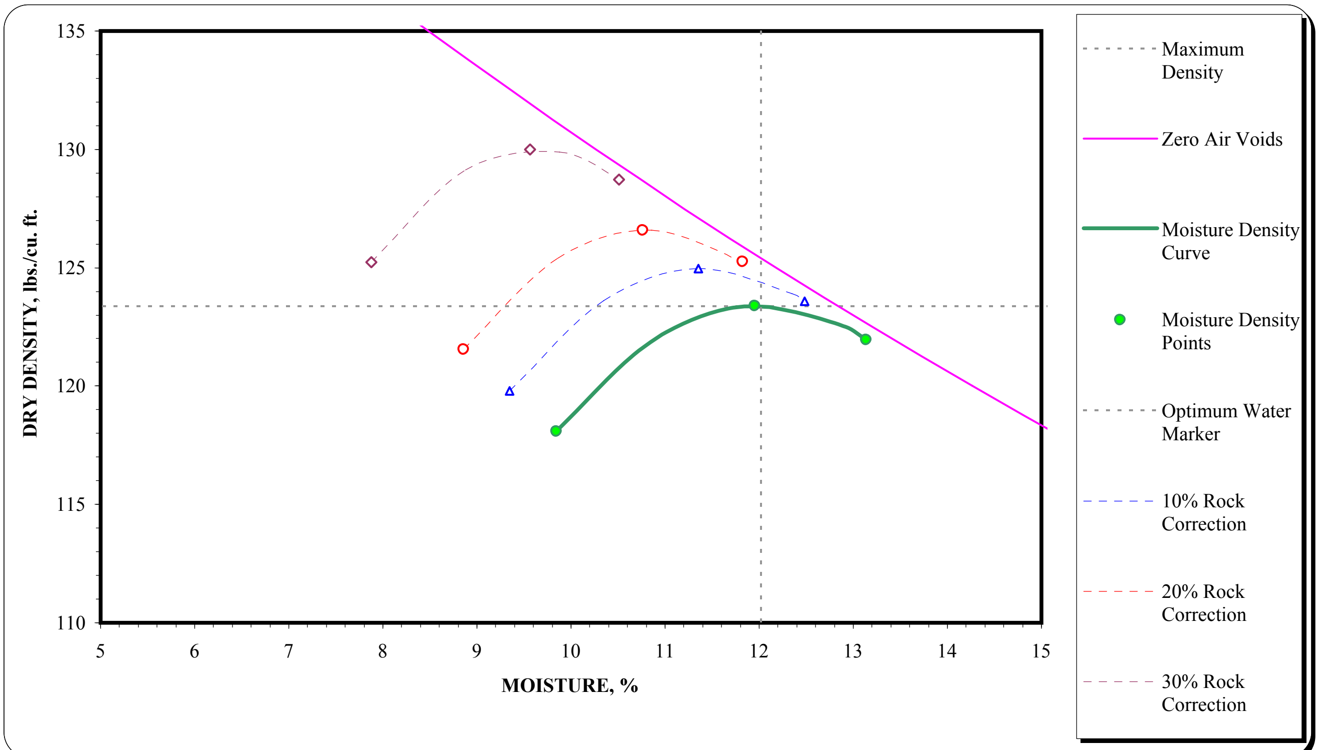
Client: Big Bend Hot Springs Project LLC
Project: Big Bend Hot Springs
Location: Shasta County, California
Material Type: Brown Silty Sand to Gravelly Sand
Sample Location: TP-3
Material Description: Native
Material Supplier: NA
Sampled By: JAB
Received By: JS
Tested By: TJ
Reviewed By: JAB
Test Procedure: ASTM

Job No.: 11-1995.01
Lab No.: 4774
CURVE NO.: 1

Date Sampled: 21-Jun-11
Date Received: 21-Jun-11
Date Tested: 5-Jul-11
Date Reviewed: 5-Jul-11
Method: D-1557

Oversized Material (%), +3/4": 43.6%

Correction Required: X YES NO



SPECIMEN	A	B	C
DRY DENSITY	118.1	123.4	122.0
MOISTURE AT TEST, %	9.8	12.0	13.1

Maximum Dry Density, PCF 123.4 **@ Optimum Moisture, %** 12.0

With 5% Rock Correction	125.0	Corrected Moisture Content	11.4
With 10% Rock Correction	126.6	Corrected Moisture Content	10.8
With 20% Rock Correction	130.0	Corrected Moisture Content	9.6

CGi: Copyright 2010

Table 1 - Laboratory Tests on Soil Samples

CGI Technical Services, Inc.
Big Bend Hot Springs
HDR|Schiff #11-0632LAB
6-Jul-11

Sample ID			Sample-1 (TP 1)	Sample-2 (TP 4)
Resistivity		**Units**		
as-received		ohm-cm	12,800	8,000
saturated		ohm-cm	5,600	6,000
pH			5.4	6.1
Electrical Conductivity		mS/cm	0.10	0.05
Chemical Analyses				
Cations				
calcium	Ca^{2+}	mg/kg	59	30
magnesium	Mg^{2+}	mg/kg	6.2	5.7
sodium	Na^{1+}	mg/kg	24	19
potassium	K^{1+}	mg/kg	28	14
Anions				
carbonate	CO_3^{2-}	mg/kg	ND	ND
bicarbonate	HCO_3^{1-}	mg/kg	79	46
fluoride	F^{1-}	mg/kg	ND	ND
chloride	Cl^{1-}	mg/kg	6.1	4.4
sulfate	SO_4^{2-}	mg/kg	25	23
phosphate	PO_4^{3-}	mg/kg	2.7	ND
Other Tests				
ammonium	NH_4^{1+}	mg/kg	2.5	1.7
nitrate	NO_3^{1-}	mg/kg	59	5.1
sulfide	S^{2-}	qual	na	na
Redox		mV	na	na

Electrical conductivity in millisiemens/cm and chemical analysis were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

431 West Baseline Road · Claremont, CA 91711
Phone: 909.626.0967 · Fax: 909.626.3316