GEOTECHNICAL INVESTIGATION

SARATOGA BRIDGES
RODEO CREEK AND SARATOGA CREEK
SARATOGA, CALIFORNIA

Prepared for:
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NOVEMBER 2006

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November 28, 2006
E0476

Ms. Iveta Harvancik
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13777 Fruitvale Avenue
Saratoga, California 95070

SUBJECT: **Geotechnical Investigation**
RE: Saratoga Bridges, Rodeo Creek and Saratoga Creek
Saratoga, California

Dear Ms. Harvancik:

With this letter, we are pleased to submit the following report describing the findings, conclusions, and recommendations of our geotechnical investigation for the proposed pedestrian/bicycle bridges over Rodeo Creek and Saratoga Creek in Saratoga, California. This investigation was performed in accordance with our Proposal for Geotechnical Services dated August 15, 2006.

Included in this report is our engineering geologic characterization and assessment of the geologic and subsurface conditions at the bridge sites, as well as our conclusions and recommendations, based on our geotechnical investigation of the surface and subsurface conditions in the vicinity of the proposed bridge abutments.

We appreciate the opportunity to have been of service to you on this project. If you have any questions regarding this report, please contact our office.

Very truly yours,
COTTON, SHIRES AND ASSOCIATES, INC.

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Saratoga, California

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Saratoga, California

1.0 INTRODUCTION

In this report we present the results of our geotechnical investigation for the proposed pedestrian/bicycle bridges to be located across Rodeo Creek and Saratoga Creek in Saratoga, California. The proposed bridge site at Rodeo Creek is to be located between Saratoga-Sunnyvale Road and Cox Avenue, and adjacent to the southwestern side of the railroad tracks (Figure 1). The proposed bridge site at Saratoga Creek is to be located between Glen Brae Drive and Saratoga Avenue, near the southeastern end of Congress Springs Park, and adjacent to the southwestern side of the railroad tracks (Figure 1).

We understand that both bridges will consist of prefabricated steel truss structures supported by concrete abutments. We also understand that the planned pedestrian/bicycle bridges will be part of the Saratoga de Anza Trail, and that the bridges will be designed for relatively light loads associated with pedestrians and bicycles.

We performed this supplemental investigation between October 19, 2006 and November 21, 2006, for the City of Saratoga in accordance with our proposal dated August 15, 2005.

1.1 Purpose and Scope of Work

The purpose of our investigation was to develop geotechnical data for the project design. Our objectives were to: 1) evaluate and characterize surface and subsurface conditions; and 2) develop conclusions and recommendations regarding: geotechnical hazards; site grading; and foundation type and design criteria.

The specific scope of work performed for our investigation included the following tasks:

1) Review of in-house geologic data;
2) Subsurface exploration;
3) Laboratory testing;
4) Engineering analysis; and
5) Preparation of this report.

2.0 GEOLOGIC SETTING

2.1 Terrain

Both bridge sites are located in a relatively level area of Saratoga, just west of Highway 85. According to the topographic cross section provided to us, the existing ground surface elevation at the Rodeo Creek bridge site is roughly 322 feet, and the elevation at the Saratoga Creek bridge site is approximately 321 feet.

2.2 Geologic Setting

Both sites are located in the Santa Clara Valley, east of the Santa Cruz Mountains. The Rodeo Creek site is mapped as being underlain, at depth, by older alluvial fan deposits while the Saratoga Creek site is mapped as being underlain by younger alluvial terrace materials (Lettis, 1994).

Locally the Santa Cruz Mountains consist of Franciscan Complex sedimentary and metamorphic bedrock materials that have been uplifted and thrust over younger bedrock and alluvial soil materials of the Santa Clara Valley. The Santa Clara Valley is an elongated, northwest-trending extension of the San Francisco Bay structural trough, bounded on the west by the Santa Cruz Mountains and on the east by the Diablo Range.

2.3 Seismic Setting

Based on our review of regional geologic maps of the area, a trace of the potentially active Monta Vista (locally known as the Shannon fault) is located approximately 1,000 feet (0.2 miles/0.3 kilometers), and 2,000 feet (0.4 miles/0.6 kilometers) southwest, respectively, of the Rodeo Creek and Saratoga Creek Bridge sites (Figure 2). The Lettis (1994) map depicts several topographic lineaments, identified as traces of the Monta

Vista fault, that pass very close to, or through, the proposed Rodeo Creek bridge site and just south of the Saratoga Creek site. The other significant active fault located close to the bridge sites is the San Andreas fault (4 to 4.5 miles/6.5 to 7.2 kilometers toward the southwest).

2.3.1 Deterministic Analysis - The following table summarizes pertinent fault information for the three closest active faults to the Rodeo Creek and Saratoga Creek Sites, including lists of the major earthquake sources, the distance from the sources to the site, the faulting style, the Maximum Credible Earthquake (MCE) Moment Magnitudes and the results of our deterministic Peak Horizontal Ground Accelerations analysis that are anticipated at the two sites (the Rodeo Creek site is listed first followed by the Saratoga Creek Site).

Fault Source	Fault-to-Site Distance (mi/km)[1]	Faulting Style	MCE Moment Magnitude[2]	Peak Horizontal Accelerations (g)[3]
Monta Vista (Shannon)	0.2/0.3 and 0.6/0.6	Reverse-oblique	6.5	**0.66** and **0.65**
San Andreas	4/6.5 and 4.5/7.2	Strike-slip	7.9	0.48 and 0.47
Zayante-Vergeles	14.1/22.7 and 13.4/21.5	Strike-slip	7.0	0.17 and 0.18

[1]Based on UBCSEIS Computer Program Vesion 1.03 by T.F. Blake, updated 2004;

[2] Based on "Probabilistic Seismic Hazard Assessment for the State of California" by CDMG, DMG Open-File Report 96-08;

[3] Based on attenuation relationships developed by Bozorgnia, Campbell & Niazi 1999, (horizontal component, Pleistocene, corrected); as determined using the computer program EQFAULT Version 3.00b by T.F. Blake, 1989, and updated 2004.

2.3.2 **Probabilistic Analysis** - We also performed a probabilistic analysis employing the computer program FRISKSP (by T.F. Blake, 1988 and updated 2004) and incorporated moment magnitudes from the California Division of Mines and Geology (CDMG) publication "Probabilistic Seismic Hazard Assessment For The State of

California" (DMG Open-File Report 96-08), and attenuation relationships by Bozorgnia, Campbell, and Niazi 1999 (horizontal – Pleistocene soil, corrected). The results of our probabilistic analysis indicate an appropriate Design Basis Earthquake (10 percent probability of exceedance in 50 years or a 475-year return interval, which is generally used for residential and commercial buildings) peak horizontal ground acceleration is **0.56g** for the Rodeo Creek site and **0.55g** for the Saratoga Creek site.

Taking into account the above earthquake Moment Magnitudes, the 1997 Uniform Building Code (UBC) coefficients presented in Section 5.6, and the results of the deterministic and probabilistic approaches, it is our opinion that the Rodeo Creek and Saratoga Creek project areas could experience peak horizontal ground acceleration (PGA) as high as **0.66g** to **0.65g** (equal to the deterministic acceleration calculated for an earthquake on the Monta Vista fault for the sites), respectively.

3.0 SITE CONDITIONS

In the following section we summarize the results of our interpretation of the surface and subsurface conditions observed and encountered at the Rodeo Creek and Saratoga Creek Bridge Sites.

3.1 Surface Conditions

The Rodeo Creek bridge site is a located in a residential area situated toward the northern side of the Town of Saratoga. Vegetation in the area includes several large, mature trees adjacent to the creek bank. The bridge would be located adjacent to the southern side of the railroad tracks between Saratoga Sunnyvale Road and Cox Avenue. The existing improvements in the immediate area consist of an existing culvert (with trash rack) that conveys the creek under the tracks, adjacent high power utility lines and a tower, and various vaults and manholes for underground utilities. The incised creek channel at the proposed bridge location is approximately 40 to 45 feet wide and roughly 7 feet deep.

The Saratoga Creek bridge site is located just beyond the southeastern end of Congress Springs Park, adjacent to the Highway 85 and Saratoga Avenue intersection. We observed several large, mature trees adjacent to the creek bank. The bridge would be located adjacent to the southern side of the railroad tracks between Glen Brae Drive and Saratoga Avenue. The existing improvements in the immediate area consist of a railroad track bridge over the creek, adjacent high power utility lines and a tower, and various vaults and manholes for underground utilities. The incised creek channel at this proposed bridge location is approximately 70 to 75 feet wide and roughly 14 to 15 feet deep.

We did not observe any indications of landsliding or significant grading at the two proposed bridge sites.

3.2 Subsurface Conditions

We explored subsurface conditions at the two bridge sites by means of four (4) exploratory borings (one boring located on each bridge abutment) drilled to depths of 35.5 to 36 feet at the locations shown on Figures 3 and 4. In the two borings (CSA/SD-3 and CSA/SD-4) located at the Rodeo Creek bridge site, we generally encountered loose (in CSA/SD-3 at 10 feet) to very dense silty, sandy alluvial soil materials (with gravels and several interbedded clay lenses) to the depths explored (Figure 5). In the two borings (CSA/SD-1 and CSA/SD-2) located at the Saratoga Creek bridge site, we generally encountered very loose (in CSA/SD-2 at 4 feet) to very dense silty, sandy fill and alluvial soil materials (with gravels and one interbedded clay lense) to the depths explored (Figure 6). In Borings CSA/SD-1 and CSA/SD-2, we encountered 8.5 feet and 5.5 feet of fill material, respectively, overlying the alluvial soil material. The fill material generally consisted of silty sand; however, in CSA/SD-2, we encountered two feet thickness of clean sand. We interpreted the clean sand material as trench backfill, possibly for one of the nearby underground utilities.

Based on the results of our laboratory testing, it appears that the native, sandy material has high shear strengths and has low plasticity, at both bridge sites.

A detailed description of the exploration program, logs of the borings, and the results of laboratory tests performed on representative samples are presented on the boring logs in Appendix A and in Appendix B.

3.3 Groundwater Conditions

Groundwater was encountered during drilling of the borings adjacent to Saratoga Creek at the following depths: in CSA-1 at a depth of 20 feet; and in CSA-2 at a depth of 35 feet. Fluctuations in the groundwater level could occur from variations in rainfall, flooding and other factors, and groundwater levels may be different at different times and locations.

4.0 POTENTIAL GEOTECHNICAL HAZARDS

In the following section, we list identified potential geotechnical hazards at the proposed bridge sites, along with the corresponding degrees of determined potential risk, and recommendations for possible mitigation measures.

4.1 Creek Bank Erosion and Scour

Based on our field reconnaissance and mapping, we judge that the potential for undermining of the bridge foundations and abutments from creek bank erosion and scour to be **moderate to high**. In order to reduce the potential adverse effects of creek scour undermining the proposed bridge foundations, we have provided recommendations for a cast-in-place drilled pier foundation where the piers extend well below the current creek channel bottom. We have also recommended that the drilled piers be located at least 6 feet from the top of the creek bank, and this is consistent with the City's request.

We recommend that the project hydrologist determine the following: peak creek flows and corresponding return intervals to ensure that the proposed pedestrian/bicycle bridges will be able to convey the design flows; the capacity of the adjacent culvert and bridge; and potential adverse upstream and downstream effects due to the proposed new structures.

4.2 Seismic Hazards

Seismic ground shaking associated with a large earthquake on either the San Andreas or Monta Vista/Shannon faults is considered to be a hazard in the project area. Peak ground accelerations between 0.55 g and 0.66 g should be anticipated at the bridge sites (see report Section 2.3).

Seismically-induced ground failure mechanisms include fault rupture, differential compaction, liquefaction, and landsliding. Our review of several geologic maps of the area indicates that one map (Lettis, 1994) projects a trace or a "photolineament" of the Monta Vista/Shannon fault approximately through the proposed Rodeo Creek bridge site area (Figure 7). Consequently, the potential for fault rupture at the proposed Rodeo Creek bridge site is considered to be **moderate to high** if the local trace is presumed to be active, while the potential for fault rupture at the proposed Saratoga Creek bridge site is considered to be **low**. The geological activity of this fault trace is not well defined. If the City determines that additional investigation for identifying the location of fault traces in the area is warranted, then a fault investigation could be considered; however, due to the relatively low slip rates associated with the Monta Vista/Shannon fault system, and that the structure is uninhabitable and not a "life line", it is possible that such an investigation would not significantly modify the bridge design or location.

The potential for lurching and differential compaction due to earthquake shaking is considered to be **moderate to high,** and could result in differential settlements, while the potential for seismically induced landsliding of the creek banks is also considered **moderate** to **high**. The potential for deep landsliding, which could impact deep foundations, is considered to be **low**. The recommended foundation systems should mitigate the potential adverse effects of earthquake-induced creek bank failures, as well as seismically induced differential settlement and/or lurching.

Soil liquefaction is a phenomenon in which a saturated, cohesionless, near-surface soil layer loses strength during cyclic loading, such as is caused by earthquakes. During the loss of strength, the soil acquires a "mobility" sufficient to permit both horizontal and vertical movements. Soils that are most susceptible to liquefaction are clean, loose saturated, uniformly graded, fine-grained sands. We note that the Saratoga Creek site is located in the liquefaction hazard zone as mapped by the California Geological Survey, while the Rodeo Creek site is not (Figure 8).

Based on an assumed conservative high groundwater level of 4 feet below the ground surface, we encountered potentially liquefiable sandy soil materials at both the Saratoga Creek Bridge and the Rodeo Creek Bridge sites.

We determined the factors of safety against triggering liquefaction (FS_l) by calculating the ratio of (1) the horizontal cyclic shear stress necessary to trigger liquefaction to (2) the average horizontal cyclic shear stress induced by the design earthquake. When this ratio is 1.3 or less (i.e., F.S.$\leq$1.3), liquefaction is predicted to occur or could potentially be a problem.

We calculated **high** liquefaction potentials (i.e., low factor of safety), at the Saratoga Creek Bridge site in Boring CSA/SD-1, between 4 and 6.5 feet and in Boring CSA/SD-2, between 4 and 18.5 feet. We also calculated **high** liquefaction potentials at the Rodeo Creek Bridge site in Boring CSA/SD-3, between depths of 9 and 13 feet. The potential consequences of liquefaction include dynamic settlement, sand boils, ground fissures, and lateral deformations that could damage structures and other improvements.

We calculated that total or differential settlement of the ground surface due to liquefaction could be up to 2-1/4 inches (Tokimatsu and Seed, 1987) at the Saratoga Creek site and 1-1/2 inches at the Rodeo Creek site.

Due to the near-surface potentially liquefiable materials, we judge that liquefaction at both sites could result in surficial expressions such as sand boils (Ishihara, 1995). We also judge that the potential for lateral spreading is **high** because liquefiable soils are located at or above the Saratoga Creek channel bottom; however, the proposed foundation systems should help to reduce the potential adverse affects to the bridge abutments from lateral spreading and sand boils.

In order to reduce the adverse effects of liquefaction, we have provided recommendations for a cast-in-place drilled pier foundation system to support the bridge abutments below the zone of liquefiable material. While the recommended foundation system should resist ground distress from liquefaction to the extent necessary to prevent foundation failure, the level of damage that occurs from earthquake shaking will depend upon a number of factors, including the strength and duration of shaking, variations in soil conditions between borings, the design of the lateral force-resisting system, and the care taken during construction.

We estimate that differential settlements due to static loading (excluding liquefaction) will be less than 1 inch across the bridge, at either site.

4.3 Surficial Erosion

Based on our experience, the underlying alluvial soil and terrace deposits are **moderately** susceptible to surficial erosion. We have provided recommendations for erosion control and surface drainage.

5.0 RECOMMENDATIONS

Based on the results of our investigation of the proposed bridge sites, we have developed the following recommendations and design criteria.

5.1 Foundation Design Criteria

5.1.1 Cast-in-Place Drilled Piers/Bridge Abutment - The proposed new bridge abutments should be supported by drilled, cast-in-place reinforced concrete piers that derive vertical support from adhesion (skin friction) in firm alluvium material beginning at a depth of 13 feet for both sites. The piers should also be located no closer than 6 feet from the top of the creek bank. The piers should be sized according to the following criteria:

Vertical Capacity – minimum three (3) pier diameter spacing
Minimum pier diameter....................18 inches
Minimum pier penetration below a depth of 13 feet............10 feet

Allowable adhesion (skin friction), for reinforced concrete dead plus live loads:
0 to 13 feet into alluvium and fill material .... 0 psf
Below a depth of 13 feet .... 550 psf

Lateral Passive Resistance - piers [equivalent fluid pressure applied over an effective width of two (2) pier diameters]
0 to 13 feet into alluvium and fill material .... 0 pcf
Below a depth of 13 feet .... 450 pcf

The above adhesion value (skin friction) can be increased by 1/3 for seismic loading and should be decreased by 1/2 for uplift. The upper portion of the piers should be formed to create vertical surfaces, and "mushrooming" of pier tops and overpours around grade beams should be prevented. Drilled pier holes should be machine cleaned of all loose material prior to the placement of steel and concrete. Piers should be steel reinforced with a minimum of 4, No. 5 bars vertical (or with greater reinforcement as required by the project Structural Engineer).

If water is present in the pier holes prior to placing concrete (and we anticipate that this will be the case), the water should be pumped out until the pier holes are dry, or the concrete should be poured by tremie methods to displace the water. Casing will likely be necessary to prevent the cohesionless materials encountered in our borings from caving.

5.2 Site Grading

Based on our field investigation, grading excavations should be within the capabilities of moderate to heavy conventional excavation equipment (i.e., excavators, drill rigs and dozers).

5.2.1 **Site Preparation** - All loose material, vegetation, debris, and other deleterious material should be stripped and removed from the areas to be developed. This material should be disposed of in a suitable location off-site.

Excavation should proceed as necessary for planned grades, and soft and/or yielding materials in the location of the planned structures should be over excavated and replaced with engineered fill. Areas to be filled, should be scarified to at least an 8-inch depth, moisture conditioned to at least optimum moisture content and compacted to at least 90 percent relative compaction based on ASTM D-1557-00. If fill is to be placed at the Saratoga Creek site, then the existing 5.5 to 8.5 feet of fill material should be removed and replaced with engineered fill.

5.2.2. Compacted Fill - Excavated on-site material is suitable for re-use as compacted fill provided it is free of organic material and other debris and rocks greater than 4 inches in maximum dimension. Imported fill should be free of organic material, should contain no material larger than 4 inches and should have a plasticity index of less than 16. Fill should be placed in horizontal lifts not exceeding 8 inches in loose thickness, moisture conditioned to at least optimum moisture content, and compacted in lifts to at least 95 percent relative compaction beneath all structures, and within 18 inches of the aggregate base rock for pavements, and 90 percent relative compaction elsewhere.

5.2.3 Cut Slope Design - Any new permanent cut slopes in alluvium should not exceed an inclination of 2 horizontal to 1 vertical (2:1).

During the dry season, temporary cut slopes of 1.5 horizontal to 1 vertical (1.5:1) in alluvium, should be satisfactory for construction purposes, provided that they are inspected and approved by our field representative at the time of construction and monitored daily during construction. Excavation methods and safety are ultimately the responsibility of the contractor. All excavations should comply with applicable Local, State and Federal safety regulations.

5.2.4 Fill Slope Design - All permanent fill slopes should have a maximum inclination of 2 horizontal to 1 vertical (2:1).

5.2.5 Keyway Design - Fill materials placed on slopes steeper than 6:1 should be continuously keyed and benched at least 1 foot into firm in-place material. The resulting subgrade should be inspected for firmness prior to placement of any new fill materials.

5.2.6 **Pipelines, Utility Trench and Retaining Wall Backfill** – Planned pipelines should be placed at least 3 feet below final ground surface. Bedding materials for pipes should be in accordance with the pipe manufacturer's recommendations. Trenches should be backfilled with either on site or approved import fill material compacted to a minimum of 90% of maximum dry density in non-structural areas and a minimum of 95% of maximum dry density beneath structures and the upper 18 inches of pavement subgrades. Equipment and methods should be used that are suitable for work in confined areas without damaging the walls or conduits.

Where a pipeline crosses a bridge abutment, it should be equipped with a flexible connection capable of withstanding up to 6 inches of deflection in all directions (including elongation).

Retaining walls should be backfilled with material that meets the requirements for compacted fill. Compaction equipment and methods should be used that are suitable for work in confined areas and that will not damage the backdrain pipe and filter fabric.

5.2.7 **Trail Subgrade Preparation** - After general compaction and compaction of the utility trench backfills, trail subgrade surfaces should be checked for yielding areas by proof-rolling with a loaded water truck or equivalent. Any yielding areas should be excavated and replaced with compacted fill. Then the upper 8 inches should be moisture conditioned to at least optimum moisture content and compacted to at least 95 percent relative compaction.

5.3 Retaining Wall Design

The following section provides our recommendations for the bridge abutments, wing walls, and site retaining walls.

5.3.1 **Retaining Walls** – The bridge abutments, wing walls, and site retaining walls should be supported on drilled piers designed according to the Foundation Design Criteria provided above. Retaining walls and abutments free to rotate should be designed to resist an active lateral fluid pressure of 50 pounds per cubic foot (pcf) for horizontal backfill, 60 pcf for 3:1 sloping backfill, and 75 pcf for 2:1 sloping backfill. The above active lateral fluid pressures should be increased by 50% for walls that are restrained from rotation. The lateral loads on the retaining wall can be resisted by passive pressure against the side of the piers using the Lateral Passive Resistance provided in Foundation Design Criteria, above. For seismic loading, a dynamic resultant force should be applied to the wall acting at 0.6H up from the bottom of the wall, and equal to $20H^2$ (where H is the height of the wall).

For walls which will support vehicle loads (bridge abutments), a traffic surcharge of 250 psf should be included and applied against the top 10 feet of the retaining wall.

5.3.2 **Backdrain** - Backdrains should be constructed behind all retaining walls. The backdrain should be a minimum 12-inch wide continuous blanket of either Caltrans Class 2 Permeable Material or 3/4-inch x 1/2-inch clean crush drainrock enclosed in Mirafi 140N (or approved equivalent) filter fabric, and extended to within 1 to 1-1/2 feet of the ground surface where an impervious fill and/or asphaltic concrete cap should be placed. A minimum 4-inch diameter PVC Schedule 40 perforated drain pipe should be placed near the bottom of the drainrock (perforations down), surrounded by a minimum of 4 inches of drainrock with at least 2 inches of drainrock underlying the pipe. All backdrain pipes should be sloped to drain at a minimum of 1/2 percent and collected in 4-inch diameter non-perforated Schedule 40 PVC pipes which are sloped a minimum of 1 percent and discharged either into the City storm drainage system, the creek, or to a suitable location away from structures, or onto an impermeable surface.

5.4 Drainage

Because of the detrimental influence of water as it interacts with soil, bedrock, foundations, pavements, and cut and fill slopes, it is important that surface water be controlled. Retaining walls supporting cutslopes should be equipped with concrete lined ditches which discharge into area drains. Grades should be sloped to drain at a minimum of 2% for a distance of at least 10 feet out from structures with runoff directed into an appropriate catch basin/storm drain system.

5.5 Seismic Design

Peak ground accelerations of 0.66g and 0.65g for the Rodeo Creek and Saratoga Creek sites should be anticipated for design purposes. For the 1997 Uniform Building Code

(UBC), assume that the site is located within Seismic Zone No. 4, and use the following values for design at both sites: Seismic Coefficients Ca and Cv of 0.57 and 1.02, respectively; and Na and Nv values of 1.3 and 1.6, respectively.

5.6 Erosion Control

All graded slopes higher than eight (8) feet, and steeper than 20 percent (5:1) should be covered with a securely staked erosion control blanket consisting of straw and coconut fiber and treated with hydroseed prior to exposure to rain. All other grounds disturbed by construction activities should be treated with hydroseed prior to exposure to rain.

An approved Storm Water Pollution Prevention Plan (SWPPP) should be implemented in accordance with Caltrans Standard Specifications. If freshly graded slopes are exposed to rain, this plan should include properly keyed and staked straw bale barriers at the base of the slopes higher than eight feet and steeper than 20 percent.

5.7 Technical Review

Supplemental geotechnical design recommendations should be provided by our firm based on specific design needs developed by the other project design professionals. This report, and any supplemental recommendations, should be reviewed by the contractor as part of the bid process. It is strongly recommended that no construction be started nor grading undertaken until the final drawings, specifications, and calculations have been reviewed and approved in writing by a representative of **our firm.**

5.8 Earthwork Construction Inspection and Testing

All excavations including foundations and keyways should be inspected by a representative of **our firm** prior to placing rebar, backfilling, and/or pouring concrete foundations. Any grading should also be inspected and tested as appropriate to confirm adequate stripping, subgrade preparation, and compaction. Our office should be contacted with a minimum of 48 hours advance notice of construction activities requiring inspection and/or testing services.

6.0 INVESTIGATION LIMITATIONS

Our services consist of professional opinions and recommendations made in accordance with generally accepted engineering geology and geotechnical engineering principles and practices. No warranty, expressed or implied, or mer chantability of fitness, is made or intended in connection with our work, by the proposal for consulting or other services, or by the furnishing of oral or written reports or findings.

Any recommendations and/or design criteria presented in this report are contingent upon our firm being retained to review the final drawings and specifications, to be consulted when any questions arise with regard to the recommendations contained herein, and to provide testing and inspection services for earthwork and construction operations. Unanticipated soil and geologic conditions are commonly encountered during construction which cannot be fully determined from existing exposures or by limited subsurface investigation. Such conditions may require additional expenditures during construction to obtain a properly constructed project. Some contingency fund is recommended to accommodate these possible extra costs.

This report is issued with the understanding that it is the responsibility of the owner, or of his representative, to ensure that the information and recommendations contained herein are called the attention of the project engineer and incorporated into the plans. Furthermore, it is also the responsibility of the owner, or of his representative, to ensure that the contractor and subcontractors carry out such recommendations in the field.

7.0 REFERENCES

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APPENDIX A

Field Investigation
Logs of Exploratory Borings

APPENDIX A
FIELD INVESTIGATION

The subsurface conditions of the subject sites were investigated using truck-mounted equipment provided by Cenozoic Exploration, Felton, California to drill a total of 4, small-diameter borings, one at each abutment. The engineering geologist logged the borings and visually classified the soils in accordance with ASTM D-2487. We obtained relatively undisturbed samples of the materials encountered at selected depths. These samples were obtained in brass liners that were 2.5 inches in outside diameter and 6 inches long; the liners were inside a 3-inch diameter modified split-barrel California Sampler. The sampler was driven with a 140-pound hammer that was raised by rope and cathead and allowed to freely fall about 30 inches. We also performed Standard Penetration Tests at selected depths. The depths of the sampling (and penetration testing) are shown on the boring logs. The bold number at the conclusion of the sampling interval represents the corrected blow count from a modified California sampler to Standard Penetration Test value accomplished by multiplying the blow count by 0.68.

Descriptive logs of the borings are presented in this appendix (see Logs for CSA/SD-1 through CSA/SD-4, A-1 through A-8). These logs depict our interpretation of the subsurface conditions at the dates and locations indicated based on representative samples collected at roughly a two-and-half to five-foot sampling intervals. It is not warranted that they are representative of subsurface conditions at other times and locations. The contacts on the logs represent the approximate boundaries between earth materials, and the transitions between these materials may be gradual. Representative samples were collected for subsequent laboratory testing and identification (see Appendix B).

APPENDIX B

Laboratory Testing

Summary of Triaxial Compression Shear Strength Testing

APPENDIX B
LABORATORY TESTING

The laboratory analysis performed consisted of limited testing of the principal soil types sampled during the field investigation to evaluate index properties and strength parameters of subsurface materials. The soil descriptions and the field and laboratory test results were used to assign parameters to the various materials at the site. The results of the laboratory testing program are presented in this appendix and on the boring logs (Figures B-1 through B-3).

The following laboratory tests were performed as part of this investigation:

1. Detailed soil description: ASTM D 2487;
2. Natural moisture content of the soil: ASTM D 2216;
3. In-situ unit weight of the soil (wet and dry);
4. Triaxial compression shear strength (unconsolidated, undained) ASTM D 2850;
5. Triaxial compression shear strength (consolidated, undrained) ASTM D 4767;
6. Percent minus the No. 200 sieve, ASTM D 1140; and
7. Atterberg limits determination: ASTM D 4318.

COTTON, SHIRES, AND ASSOCIATES, INC.
LOG OF EXPLORATORY DRILLING

Project: SARATOGA CREEK BRIDGES — Boring No.: CSA/SD-1

Location: West side of Saratoga Creek — Project No.: E0476

Drilling Contractor/Rig: CENOZOIC — Date of Drilling: 10/19/06

Ground Surface Elev.: 320.14′ — Logged By: AM — Hole Diameter: 3 1/4″

Surface Conditions: Gravel Fill Pad — Weather: Clear / Warm

Depth (feet)	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Bl./ft.	Sample Type	Recov.	Remarks
	SM	FILL 0′-8.5′							Start at 10:06 am
		0′-8.5′ Silty sand with gravels; moderate yellowish brown, medium dense, dry to moist, gravels up to 2″ diameter				7	MC		
			T-1			14			
2			T-2	107	7.1	13			
						18			
4						12	MC		
			T-3			12			
			T-4	114	7.9	13			
6						17			
						14	MC		
8			T-5			7			
			T-6	110	7.1	32			
	SM	ALLUVIUM 8.5′-BOH				27			
10		8.5′-16′ Silty sand with gravels; moderate brown to orange brown, dense, moist to wet				23	MC		
			T-7	118	7.1	30			
			T-8			32			10:33 am
12						42			
14									
						15	MC		
16			T-9			41			
	SC	16′-36′ Clayey sand with gravels; moderate brown, dense, moist to wet	T-10	126	12.5	50			TX/UU 8320 (3000)
						62			10:46 am
18									
20		- below 20′ material becomes gray, less fines				40	MC		
			T-11			29			
			T-12			39			
22						46			
24									
						28	MC		
26			T-13			50/5″			11:27 am
						50/5″			
28									

COTTON, SHIRES & ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

Project SARATOGA CREEK BRIDGES / E0476 Date 10/19/06 Boring No. CSA/SD-1

Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Bl./ft.	Sample Type	Recov. (%)	Remarks
				T-14			27 50/5" 50/5"	MC		
32										
34										
				T-15			42 53/6" 53/6"	MC		end at 12:00 pm
36			Total Depth = 36' Water encountered at 20' Creek at 15'							
38										
40										
42										
44										
46										
48										
50										
52										
54										
56										
58										
60										
62										

COTTON, SHIRES, AND ASSOCIATES, INC.
LOG OF EXPLORATORY DRILLING

Project SARATOGA CREEK BRIDGES | Boring No. CSA/SD-2

Location East side of Saratoga Creek | Project No. E0476

Drilling Contractor/Rig CENOZOIC | Date of Drilling 10/19/06

Ground Surface Elev. ______ Logged By AM | Hole Diameter 3 1/4"

Surface Conditions Gravel Fill Pad | Weather Clear / Warm

Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Bl./ft.	Sample Type	Recov.	Remarks
		SM	**FILL 0'-5.5'**							Start at 1:07 pm
2			0'-4' Silty sand with gravels; moderate yellowish brown, medium dense, dry to moist, gravels up to 2" diameter	T-1 T-2	114	6.4	25 20 15 **24**	MC		
4		SP	4'-5.5' Clean Sand trench backfill	T-3			1 2 4 **4**	MC		
6		SM	**ALLUVIUM 5.5'-BOH**							
8			5.5'-20' Silty sand with gravels; moderate brown to orange brown, loose to medium dense, moist to wet, gravels up to 2"	B-1			6 12 12 **16**	MC		
10				T-4 T-5	116	2.3	1 2 6 **5**	MC		TX/UU 4055 (2000)
12										
14										
16				T-6 T-7	127	4.8	15 20 13 **22**	MC		
18										
20		SC	20'-35.5' Clayey sand with gravels; moderate brown, dense, moist to wet	T-8 T-9	131	10.4	13 21 32 **36**	MC		TX/UU 7300 (3000)
22										
24										
26				T-10 T-11	122	6.8	13 37 34 **48**	MC		
28										2:55 pm

Project SARATOGA CREEK BRIDGES / E0476 Date 10/19/06 Boring No. CSA/SD-2

Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Bl./ft.	Sample Type	Recov. (%)	Remarks
							20	MC		
				T-12			38			
				T-13			50/3"			
32							50/5"			
34										
				T-14			50/4"	MC		
36			Total Depth = 35.5' Free Water on tip of sampler @ 35' Creek at 15'				50/4"			end at 12:00 pm
38										
40										
42										
44										
46										
48										
50										
52										
54										
56										
58										
60										
62										

COTTON, SHIRES & ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

COTTON, SHIRES, AND ASSOCIATES, INC.
LOG OF EXPLORATORY DRILLING

Project: SARATOGA CREEK BRIDGES
Boring No.: CSA/SD-3
Location: East side of Rodeo Creek
Project No.: E0476
Drilling Contractor/Rig: CENOZOIC
Date of Drilling: 10/20/06
Ground Surface Elev.:
Logged By: AM
Hole Diameter: 3 1/4"
Surface Conditions:
Weather: Clear / Warm

Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Bl./ft.	Sample Type	Recov.	Remarks
		SM	**ALLUVIUM 0'-BOH**							Start at 8:55 am
			0'-25' Silty sand with gravels; moderate yellow brown to orange brown, loose to medium dense, moist to wet, gravels up to 2"				21	MC		
				T-1	108	6.3	32			
2				T-2			50/5"			
							50/5"			
4							10	MC		
				T-3			11			
				T-4	111	6.1	15			9:07 am
							18			
6										
							7	MC		
				T-5			20			
8				T-6			50/5"			11% Passing #200 sieve
							50/5"			
10							4	MC		
				T-7			3			
				T-8	103	10.8	7			TX/UU 10,029 (7000)
							7			
12										
14										9:30 am
							27	MC		
				T-9			42			
16			- Samples become wet at 15.5'	T-10	130	9.4	50/5"			
							50/5"			
18										
20							26	MC		
				T-11			50/3"			
							50/3"			
22										
24										10:09 am
		SC	25'-36' Clayey sand with gravels; moderate brown, dense, moist to wet				12	MC		
26				T-12	127	12.1	31			TX/UU 12,786 (10,000)
				T-13			37			
							46			
28										

Project SARATOGA CREEK BRIDGES / E0476 Date 10/20/06 Boring No. CSA/SD-3

Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Bl./ft.	Sample Type	Recov. (%)	Remarks
							23	MC		
				T-14			42			
				T-15			50/3"			
32							50/5"			
34										
							25	MC		
				T-16			50/4"			end at 10:55 am
36			Total Depth = 36′ No Water Encountered				50/4"			
38										
40										
42										
44										
46										
48										
50										
52										
54										
56										
58										
60										
62										

COTTON, SHIRES & ASSOCIATES, INC.
CONSULTING ENGINEERS AND GEOLOGISTS

COTTON, SHIRES, AND ASSOCIATES, INC.
LOG OF EXPLORATORY DRILLING

Project SARATOGA CREEK BRIDGES — Boring No. CSA/SD-4
Location East Side of Rodeo Creek — Project No. E0476
Drilling Contractor/Rig CENOZOIC — Date of Drilling 10/20/06
Ground Surface Elev. ______ Logged By AM — Hole Diameter 3 1/4"
Surface Conditions ______ — Weather Clear / Warm

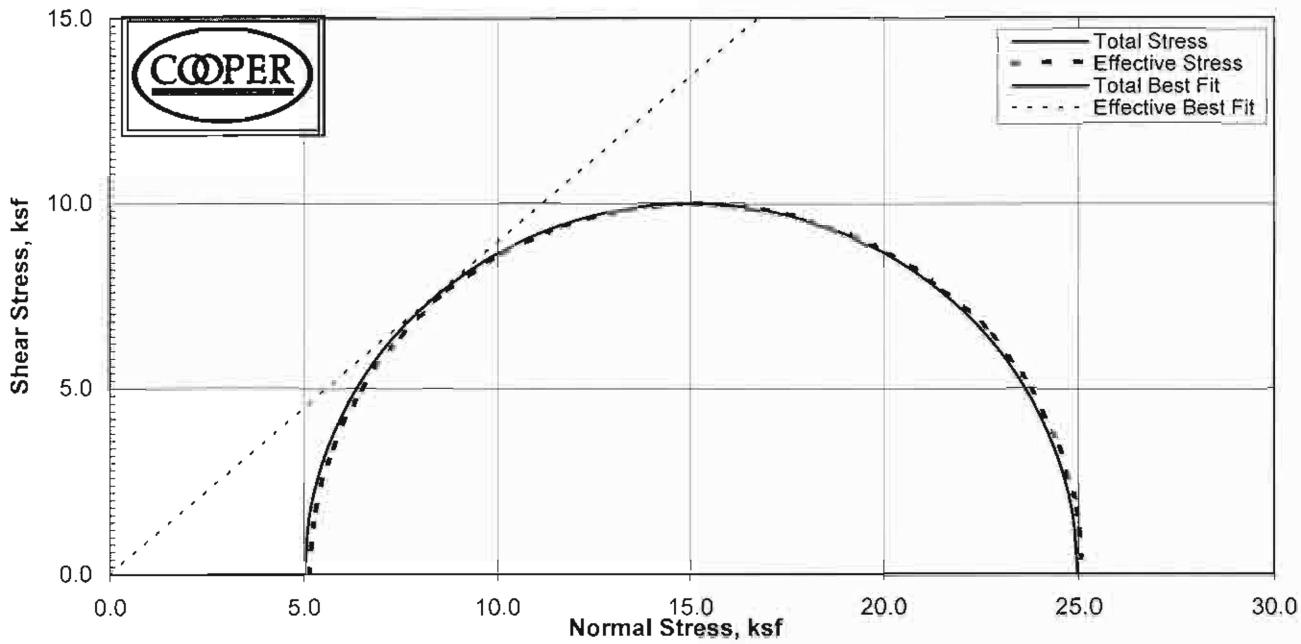
Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Bl./ft.	Sample Type	Recov.	Remarks
0		SM	ALLUVIUM 0'-BOH							Start at 12:00 pm
			0'-36' Silty sand with gravels; moderate yellow brown to orange brown, loose to medium dense, moist to wet, gravels up to 2"				27	MC		
2				T-1			27			LL = 21, PI = 4
				T-2			25			
							35			
4							8	MC		
				T-3			32			
				T-4	127	5.8	51			
6							56			
							19	MC		
				T-5			32			
8				T-6	127	6.7	51			TX/CU C=0 Ø = 42°
							56			
10							20	MC		12:27 pm
				T-7			50/5"			
							50/5"			
12										
14										
							34	MC		
16				T-8			43			
				T-9	123	11.6	50/5"			
							50/5"			
18										
20							23	MC		12:55 pm
				T-10	124	9.8	26			
				T-11			34			
22							41			
24										
							9	MC		
26				T-12			15			
				T-13			22			
							25			
28										

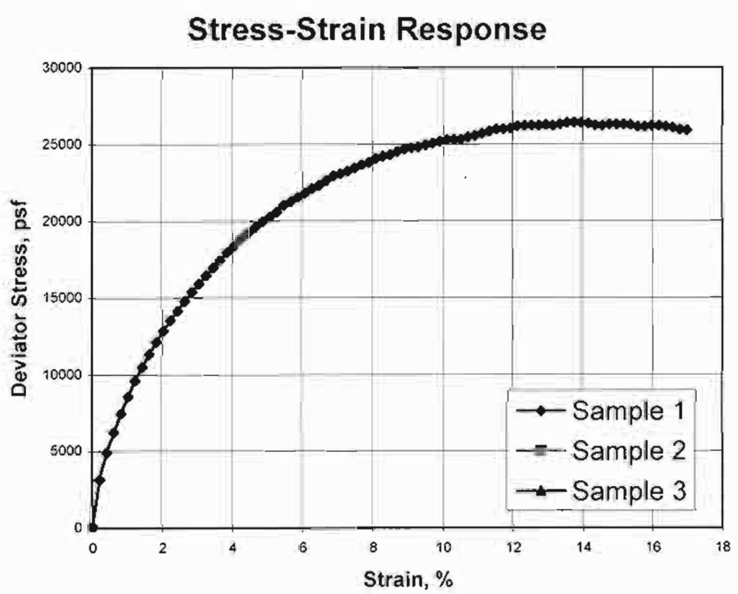
Project SARATOGA CREEK BRIDGES / E0476 Date 10/20/06 Boring No. CSA/SD-4

Depth (feet)	Graphic Log	USCS Class.	Geotechnical Description	Sample Desig.	Dry Density (pcf)	Moisture Content (%)	SPT Bl./ft.	Sample Type	Recov. (%)	Remarks
				T-14			50/3"	MC		
							50/5"			
32										
34										
							25	MC		
				T-15			45			
36				T-16			50/5"			end at 2:07 pm
			Total Depth = 36' No Water Encountered				50/5"			
38										
40										
42										
44										
46										
48										
50										
52										
54										
56										
58										
60										
62										

Triaxial Consolidated Undrained

(ASTM D4767)





Sample:	1	2	3	4
MC, %	6.7			
Dry Dens, pcf	127.4			
Sat. %	56.1			
Void Ratio	0.323			
Diameter in	2.42			
Height, in	5.00			
	Final			
MC, %	11.6			
Dry Dens, pcf	128.2			
Sat. %	100.0			
Void Ratio	0.314			
Diameter, in	2.42			
Height, in	4.95			
Cell, psi	73.2			
BP, psi	38.4			
	Effective Stresses At:			
Strain, %	5.0			
Deviator ksf	19.958			
Excess PP	-0.130			
Sigma 1	25.098			
Sigma 3	5.141			
P, ksf	15.120			
Q, ksf	9.979			
Stress Ratio	4.882			
Rate in/min	0.002			
Total C	N/A	ksf		
Total Phi	N/A	Degrees		
Eff. C	0.0	ksf		
Eff. Phi	41.9	Degrees		

Job No.: 026-350 **Date:** 11/27/2006

Client: Cotton-Shires BY:MD/DC

Project: E0476

Sample 1) SD-4, T-6 @ 8.0' Brown Silty GRAVEL w/Sand

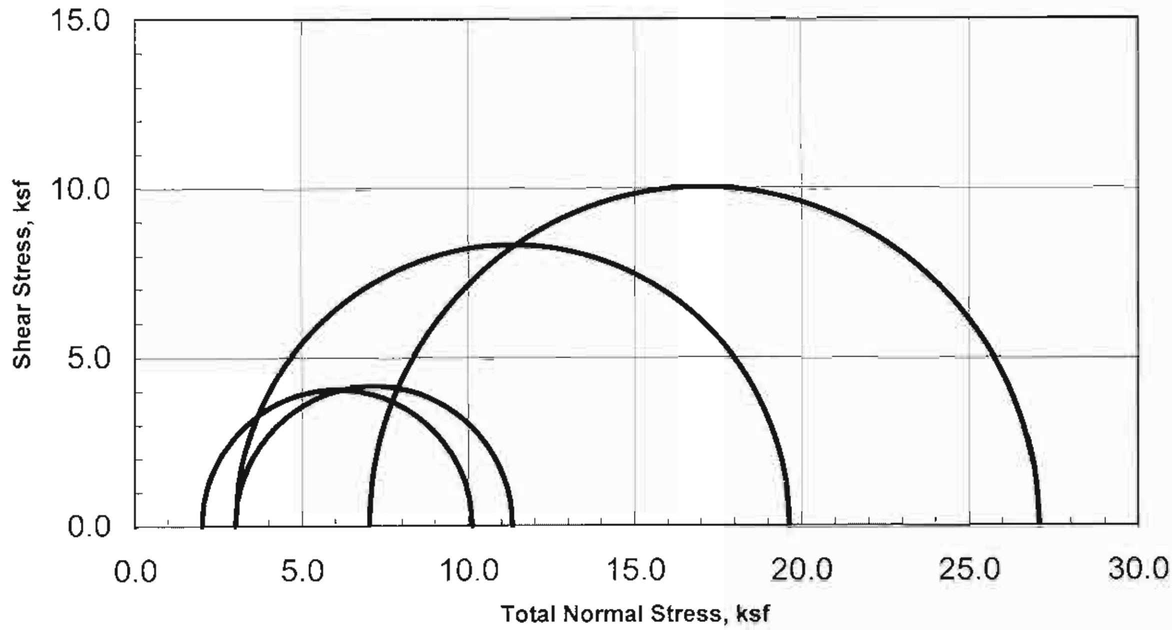
Sample 2)

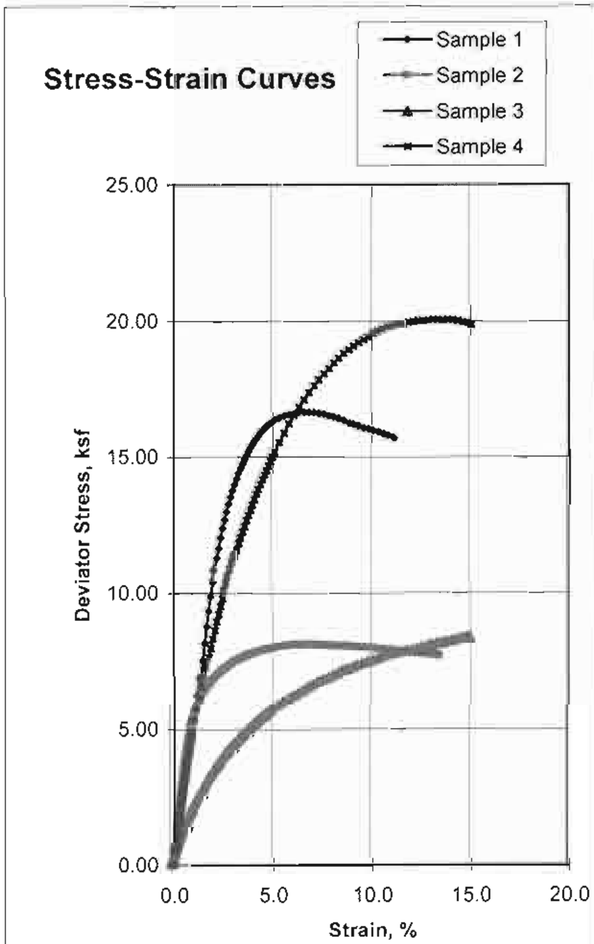
Sample 3)

Sample 4)

REMARKS: Strengths picked at 5% strain.

Unconsolidated-Undrained Triaxial Test
ASTM D-2850





Sample Data	1	2	3	4
Moisture %	12.5	2.3	10.4	10.8
Dry Den,pcf	125.7	116.2	131.1	103.1
Void Ratio	0.341	0.451	0.286	0.635
Saturation %	98.9	13.6	98.4	46.0
Height in	5.00	4.99	4.98	5.00
Diameter in	2.43	2.43	2.42	2.42
Cell psi	20.8	13.9	20.8	48.6
Strain %	6.60	6.80	14.60	13.10
Deviator, ksf	16.641	8.109	8.384	20.058
Rate %/min	1.00	1.00	1.00	1.00
in/min	0.050	0.050	0.050	0.050
Job No.:	026-350a			
Client:	Cotton, Shires & Associates			
Project:	Saratoga Bridges - E0476			
Boring:	SD-1	SD-2	SD-2	SD-3
Sample:	T-10	T-5	T-9	T-8
Depth ft:	16	11	21	11

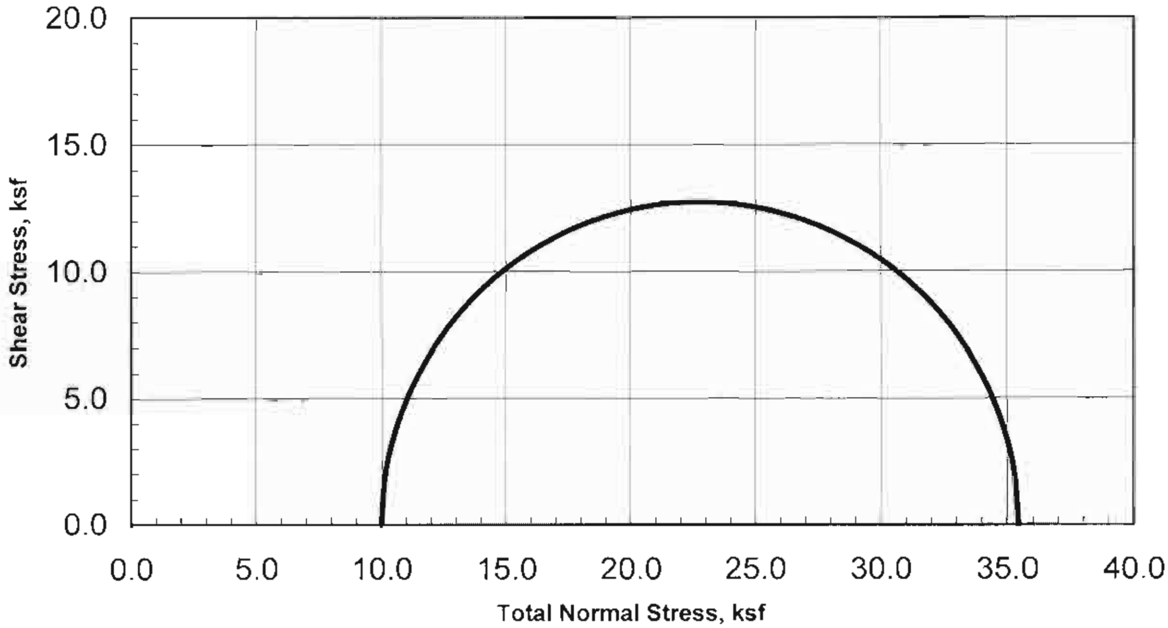
Visual Soil Description

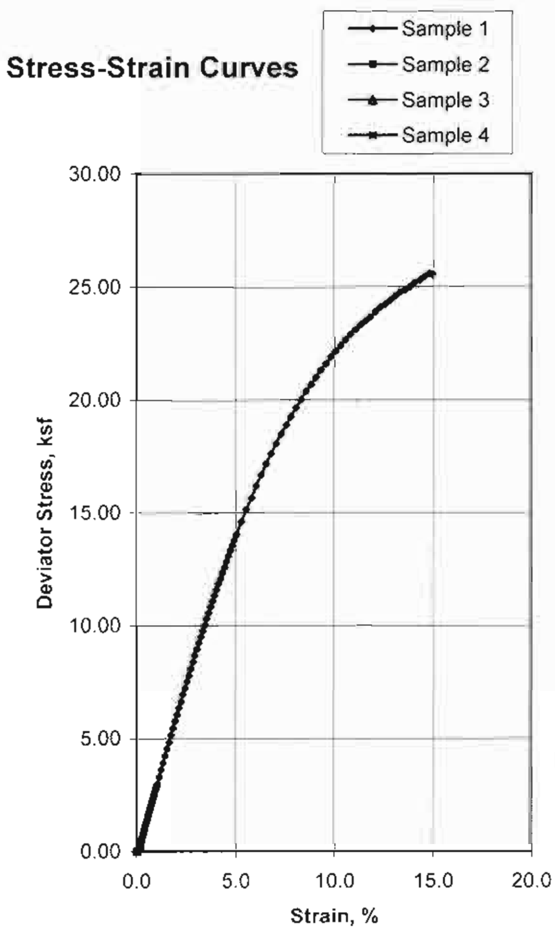
Sample #	
1	Brown Clayey SAND w/ Gravel
2	Brown Clayey SAND w/ Gravel
3	Brown Clayey SAND w/ Gravel
4	Brown Clayey SAND w/ Gravel

Remarks:

Unconsolidated-Undrained Triaxial Test

ASTM D-2850





Sample Data	1	2	3	4
Moisture %	12.1			
Dry Den,pcf	127.0			
Void Ratio	0.327			
Saturation %	99.8			
Height in	5.01			
Diameter in	2.41			
Cell psi	69.4			
Strain %	14.80			
Deviator, ksf	25.572			
Rate %/min	1.00			
in/min	0.050			
Job No.:	026-350b			
Client:	Cotton, Shires & Associates			
Project:	Saratoga Bridges - E0476			
Boring:	SD-3			
Sample:	T-12			
Depth ft:	26			

Visual Soil Description	
Sample #	
1	Brown Clayey SAND w/ Gravel
2	
3	
4	

Remarks: