Cal Land Engineering, Inc.
dba Quartech Consultants
Geotechnical, Environmental and Civil Engineering

October 17, 2013

Kam Sang Company
411 E. Huntington Drive, #305
Arcadia, California 91006

Attention: Mr. John Hicks

Subject: Report of Geotechnical Engineering Investigation, Proposed Residential (Phase I) and Commercial (Phase II) Development, 12791 Brookhurst street, (Brookhurst Triangle) APN: 0899-661-03 to 06, 089-071-06, 07, 11, to 14, 24 and 25, Garden Grove, California. QCI Project Number: 13-082-002GE

Gentlemen:

In accordance with your request, Quartech Consultants (QCI) is pleased to submit this Geotechnical Engineering Report for the subject site. The purpose of this report was to evaluate the subsurface conditions and provide recommendations for foundation designs and other relevant parameters of the proposed construction.

Based on the findings and observations during our investigation, the proposed construction of the subject site for the intended use is considered feasible from the geotechnical engineering viewpoints, provided that specific recommendations set forth herein are followed.

This opportunity to be of service is sincerely appreciated. If you have any questions pertaining to this report, please call the undersigned

Respectfully submitted,
Cal Land Engineering, Inc. (CLE)
dba Quartech Consultants (QCI)

Jack C. Lee, GE 2153	Abe Kazemzadeh
Principal Engineer	Project Engineer

Dist: (4) Addressee

REPORT OF GEOTECHNICAL ENGINEERING INVESTIGATION

Proposed Residential (Phase I) and Commercial (Phase II) Development

At

12791 Brookhurst Street (Brookhurst Triangle)
APN: 0899-661-03 to 06, 089-071-06, 07, 11, to 14, 24 and 25
Garden Grove, California

Prepared by

QUARTECH CONSULTANTS (QCI)

Project No.: 13-032-008 GE

October 17, 2013

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1.0 INTRODUCTION

1.1 Purpose

This report presents a summary of our geotechnical engineering investigation related to the proposed residential and commercial development at the subject site. The purpose of this investigation was to evaluate the subsurface conditions and provide relevant geotechnical information pertinent to grading, foundation design and other relevant parameters of the development.

1.2 Scope of Services

Our scope of services included:

- Review of available soil and geologic data of the area.
- Logging and sampling of five 8-inch diameter hollow stem auger borings to a maximum depth of 51.5 feet below the existing ground surface. Boring logs are presented in Appendix "A"
- Laboratory testing of representative samples collected from the proposed construction area to establish engineering characteristics of the on-site soils. The laboratory test results are presented in Appendix B (Laboratory Testing) and on the boring logs (Appendix A).
- Engineering analyses of the geotechnical data obtained from our background studies, field investigation, and laboratory testing.
- Preparation of this report presenting our findings, conclusions, and recommendations for the proposed development.

1.3 Proposed Construction

Based on the provided information, it is our understanding that the subject site development will be divided into several phases. Phase I development will be the multi-family residential complex. The main structure of the Phase I building is anticipated to be five stories (maximum) in height above the ground level with one level of semi-subterranean garage. The lowest garage floor will be approximately 5 feet below the existing ground surface. The semi-subterranean garage is anticipated to occupy the entire building site. No detailed design plans and structural loads were available during our preparation of this report. It is estimated the maximum column loads will be on the order of 750 kips and the approximate continuous wall loads will be on the order 10 kips per linear foot. Phase II and/or remaining phases will be developed into residential/commercial mixed use or commercial complex. However, no detail architectural plans or foundation plans are available during our preparation of this report.

1.4 Site Conditions

The subject site is located at the north side of Garden Grove Boulevard between Brookhurst Street and Brookhurst Way in the City of Garden Grove, California. The approximate regional location is shown on the attached Site Location Map (Figure 1). The site is relatively flat and no major surface erosions were observed during our subsurface investigation.

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2.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

2.1 Subsurface Exploration

Our subsurface exploration consisted of drilling five 8-inch diameter hollow stem auger borings to a maximum depth of 51.5 feet below the existing ground surface. The drilling of the boring was supervised and logged by a QCI's engineer. Relatively undisturbed and bulk samples were collected for laboratory testing. In addition, Standard Penetration Tests (SPT) was also conducted during drilling of the boring. Boring logs are presented in Appendix A.

2.2 Laboratory Testing

Representative samples were tested for the following parameters: in-situ moisture content and density, consolidation, direct shear strength, grain size analysis, expansion and corrosion potential of soils. The results of our laboratory testing along with a summary of the testing procedures are presented in Appendix B. In-situ moisture and density test results are provided on the boring logs (Appendix A).

3.0 SUMMARY OF GEOTECHNICAL CONDITIONS

3.1 Soil Conditions

The onsite near surface soils consist predominantly of silty sand (SM). In general, these soils exist in a medium dense and slightly moist to moist condition. Underlying the surface soils consists of silty sand (SM), poorly graded sand (SP), sand and silty sand (SP-SM), sandy clay (CL), clayey silt (ML) and silty clay (CH) to the depth explored (51.5 feet). These soils exist in medium dense to dense, very stiff to hard and moist to saturated conditions.

3.2 Groundwater

Ground water level was encountered at the depth of about 15 feet during our subsurface investigation. Based on our review of the "Historically Highest Ground Water Contours and Borehole Log Data Locations, Anaheim 7.5–minute Quadrangle", by CDMG, it is estimated that the highest ground water level is approximately 15 feet below the existing grade. It should be noted that the CDMG ground water map is obtained by evaluating technical publications, geotechnical borehole data, water-well logs dating back to the "turn-of-the-century". This report also indicated that ground water levels in the areas from 1960-1997 data are generally 5 to 50 feet deeper than the earlier measured data. No specific date was provided pertaining to the high ground water level.

4.0 FAULTING AND SEISMICITY

4.1 Seismicity

The subject site is located in southern California, which is a tectonically active area. The type and magnitude of seismic hazards affecting the site are dependent on the distances to causative faults, the intensity, and the magnitude of the seismic event. Table 1 indicates the distances of the fault zones and the associated maximum magnitude earthquake that can be produced by nearby seismic events. As indicated in Table 1, the Compton Thrust fault zone, which is located 1.5 miles from the site, is considered to have the most significant affect at the site from a design standpoint.

TABLE 1

Characteristics and Estimated Earthquakes for Regional Faults

Fault Name	Approximate Distance to Site (mile)	Maximum Magnitude Earthquake (Mw)
Compton Thrust	1.5	6.8
Elysian Park Thrust	5.4	6.7
San Joaquin Hills	6.0	7.0
Puente Hills Blind Thrust	6.3	7.1
Newport-Inglewood (LA Basin)	6.8	7.1
Whittier	12.0	6.8
Newport-Inglewood (Offshore)	13.0	7.1
Chino-Central Ave. (Elsinore)	15.7	6.7
Palos Verdes	16.1	7.3
San Jose	18.7	6.5
Elsinore - Glen Ivy	19.2	6.8
Sierra Madre	25.8	7.2

***Based on EQ Fault by "Thomas Blake"**

4.2 Estimated Earthquake Ground Motions

In order to estimate the seismic ground motions at the subject site, QCI has utilized the seismic hazard map published by California Geological Survey. According to this report, the peak ground Alluvium acceleration at the subject site for a 10% probability of exceeding in 50 years is about 0.38g (2008 NSHMP PSHA Interactive Deaggregation and Seismic Parameters S_{DS}). This report also indicates that the subject site is located within a zone where the magnitude range is 6.5-7.0.

5.0 SEISMIC HAZARDS

5.1 Liquefaction Potential

Liquefaction is the transformation of a granular material from a solid to a liquid state as a result of increase pore-water pressure. The material will then loses strength and can flow if unrestrained, thus leading to ground failure. Liquefaction can be triggered in saturated cohesionless material by short-term cyclic loading, such as shaking due to an earthquake. Ground failure that results from liquefaction can be manifested as flow landsliding, lateral spread, loss of bearing capacity, or settlement.

The potential for liquefaction at the site's sandy soil was evaluated using the computer program "LIQUEFY 2" and the data from Boring B-3 and B-4. The design earthquake (M=7.0) and ground acceleration of 0.38g are discussed in the previous Section. The total unit weight used for the onsite soils is 120 pcf. The ground water level imputed is raised to the depth of 10 feet below the existing ground surface. Based on our analyses, it is concluded that the factor of safety is less than 1.30 for the onsite soils at the depth of 27 to 32 feet for B-3 and B-4.

Based on the laboratory test results on clayey soils, the saturated moisture content of the encountered clayey soils is less than 85 percent of liquid limit with PI less than 12 (Bray and Sancio 2006, if PI<12, w_c/LL<0.85 not susceptible to liquefaction), the saturated moisture content of the encountered clayey soils is less than 80 percent of liquid limit with PI between 12 and 18 (Bray and Sancio 2006, if PI is between 12 to 18, w_c/LL<0.80 not susceptible to liquefaction) and the saturated moisture content of the encountered clayey soils is less than 80 percent of liquid limit with PI >18 and the soil is insensitive (Bray and Sancio 2006, if PI>18 and w_c/LL<0.80 and insensitive it is not susceptible to liquefaction). According to procedures referenced in SP117A, (Guideline for Evaluating and Mitigating Seismic Hazards in California), our laboratory Atterberg Limits and saturated moisture content of clayey soils material, it is our opinion that the encountered clayey soil is not susceptible to liquefaction.

5.2 Liquefaction Induced Settlement

The sandy soils tend to settle and densify when they are subjected to earthquake shaking. Should the sand be saturated and there is no possibility for drainage so that constant volume conditions are maintained, the primary effect of the shaking is the generation of excess pore water pressures. Settlement then occurs as the excess pore pressures dissipate. The primary factors controlling seismic induced settlement are the cyclic stress ratio, maximum shear strain induced by earthquake, the strength and density of the sand, and the magnitude of the earthquake.

Based on the procedures developed by Tokimatsu and Seed on 1987, it is estimated that seismic induced settlement of the underlying sandy soils is 0.75 and 0.66 inches and differential settlement of about 0.49 and 0.43 inches for B-3 and B-4 respectively at the site under the design earthquake.

5.3 Surface Manifestation of Liquefaction

One of the most dramatic causes of damage to structures during earthquakes has been the development of liquefaction in saturated sandy soils, manifested either by the formation of boils and mud-spouts at the ground surface, by seepage of water through ground cracks. Based on the procedures suggested by the Ishihara (1985), it is unlikely the surface manifestation would occur at the subject site under the design earthquake even.

5.4 Lurching

Soil lurching refers to the rolling motion on the surface due to the passage of seismic surface waves. Effects of this nature are not considered significant on the subject site where the thickness of alluvium does not vary appreciably under structures.

5.5 Surface Rupture

Surface rupture is a break in the ground surface during or as a consequence of seismic activity. The potential for surface rupture on the subject site is considered negligible due to the absence of known active faults at the site.

5.6 Ground Shaking

Throughout southern California, ground shaking, as a result of earthquakes, is a constant potential hazard. The relative potential for damage from this hazard is a function of the type and magnitude of earthquake events and the distance of the subject site from the event. Accordingly, proposed structures should be designed and constructed in accordance with applicable portions of the building code.

6.0 CONCLUSIONS

The following is a summary of the geotechnical design and construction factors that may affect development of the site.

6.1 Seismicity

Based on our studies on seismicity, there are no known active faults crossing the property. However, the site is located in a seismically active region and is subject to seismically induced ground shaking from nearby and distant faults, which is a characteristic of all Southern California areas.

6.2 Liquefaction

Based on our liquefaction evaluation, it is estimated potential seismic induced settlement of the underlying sandy soils is about 0.75 and 0.66 inches and differential settlement of about 0.49 and 0.43 inches for B-3 and B-4, respectively, at the site under the design earthquake.

6.3 Groundwater

Ground water level was encountered at the depth of about 15 feet during our subsurface investigation. In our opinion, groundwater will not be a problem during the near surface construction

7.0 RECOMMENDATIONS

Based on the subsurface conditions exposed during field investigation and laboratory testing program, it is recommended that the following recommendations be incorporated in the design and construction phases of the project.

7.1 Grading

7.1.1 Site Preparation

Prior to initiating grading operations, any existing vegetation, trash, debris, over-sized materials (greater than 8 inches), and other deleterious materials within construction areas should be removed from the subject site.

7.1.2 Surficial Soil Removals

In order to provide a uniform support for the foundation and concrete slab, it is recommended that the subterranean garage areas be cut to grade then over-excavated to a minimum depth of 3 feet below the final pad grade, then replaced with compacted fill to the design grade. The bottom of the excavation shall be observed by a representative of this office to verify the sub-grade soil conditions. Outside the building areas, the near surface soils are loose and weathered and should be removed to expose competent natural soils.

7.1.3 Seepage

Our subsurface investigation encountered the groundwater at the depth of 15 feet below the existing grade. Perched water or seepage water might be encountered during the onsite grading and construction. Should the water be encountered during grading/construction, the water should be drained and/or pumped away from the construction area. Any loose/soft soils within the construction area should be removed under the direction of the project geotechnical consultants. It is also recommended that geotexile be placed at the bottom of the excavation. Approximate two feet of 3/4 inches crushed rock may be placed on the top of the geotexile and an additional layer of geotexile shall be placed on the top of the gravel follow by the compacted fill to the design grade.

7.1.4 Treatment of Removal Bottoms

Soils exposed within areas approved for fill placement should be scarified to a depth of 6 inches, conditioned to near optimum moisture content, then compacted in-place to minimum project standards.

7.1.5 Structural Backfill

The onsite soils may be used as compacted fill provided they are free of organic materials and debris. Fills should be placed in relatively thin lifts (6 to 8 inches), brought to near optimum moisture content, and compacted to at least 90 percent relative compaction based on laboratory standard ASTM D-1557-09.

7.2 Semi-Subterranean Garage Excavation

The required excavation for the proposed semi-subterranean garage will extend to a maximum of approximately 6 to 10 feet below the existing ground surface. The criteria for sloped excavations and/or shoring method for the alignments required vertical cuts, depends on many factors, which include depth of excavation, soil conditions, types of shoring, distance to the existing structures or public improvement, consequences of potential ground movement, and construction procedures.

7.2.1 Sloping Excavation

Should the space be available at the site, the required excavation may be made with sloping banks. Based on materials encountered in the test borings, it is our opinion that sloped excavations may be made no steeper than 3/4:1 (horizontal to vertical) for the underlying native soils. Flatter slope cuts may be required if loose soils encountered during excavation. No heavy construction vehicles, equipment, nor surcharge loading should be permitted at the top of the slope. A representative of this office should inspect the temporary excavation to make any necessary modifications or recommendations.

7.2.2 Shoring

Shoring will be required for temporary excavation made vertically or near vertically of more than 5 feet. An active earth pressure of 26 pound per cubic foot may be used for the temporary cantilever shoring system. Any surcharged loads resulting from the adjacent building or the traffic in the adjacent street or alley should be considered as an added loads to the above recommended. Soldier piles or beams should be spaced at the required distance specified by the project structural/shoring engineer. Lagging may be required to span between soldier piles to support the lateral earth pressure.

The shoring and bracing should be designed and constructed in accordance with current requirements of CAL/OSHA and all other public agencies having jurisdiction. Careful examination of the soil excavation and inspection of on-site installation of the shoring system by a representative of this office is recommended to verify the conditions or to make recommendations as are pertinent if different conditions are disclosed during excavation.

7.3 Foundation Design

Grading and foundation plans are not available during our preparation of this report. Based on our subsurface investigation, it is our opinion that the proposed building may be supported on shallow foundation or mat foundation founded on competent soils. For fill composed of the onsite soil materials and graded in accordance with the recommendations of this report, construction of concrete slab-on-grade with conventional shallow foundation structures is feasible from the geotechnical engineering viewpoint. The following presents our preliminary recommendations:

7.3.1 Conventional Shallow Foundation - Miscellaneous

An allowable bearing value of 2000 pounds per square foot (psf) may be used for design of continuous or pad footings with a minimum of 18 or 24 inches in width, respectively. All footings should be a minimum of 24 inches deep and founded on compacted soils approved by the project geotechnical engineer. This value may be increased by one third (1/3) when considering short duration seismic or wind loads.

7.3.2 Mat Foundation – Building

In order to provide a uniform support for the proposed mat foundation, it is recommended that the existing soils should be removed to a minimum depth of 3 feet below the bottom of the proposed grade, then replaced with compacted fill to the design grade. Selected sandy or gravelly soils may be used for the fill. The fill should be brought to near optimum, then compacted to at least 92 percent of the ASTM D-1557-09.

The foundation should have sufficient stiffness and thickness to distribute the column loads to the foundation. An allowable bearing value of 3000 pounds per square foot (psf) may be used for design of mat foundation. The thickness and reinforcement of the proposed mat foundation should be designed by the project structural engineer. However, it is recommended that the thickness of the mat should be at least 12 inches from the geotechnical engineering viewpoint.

The mat foundation should provide sufficient strength for any potential negative moment and shear. Should the elastic method be used for the mat foundation design, the allowable subgrade modulus of 120 pounds per cubic inch may be used. The construction of the mat foundation should avoid excessive shrinkage cracks. Should the construction joints be utilized, the joints should be carefully located at sections of low shear stress. Reinforcing bars should be continuous across the joints. If reinforcing bar splicing is needed, the lap of the bars should be provided in accordance with the structural engineer’s design or other applicable specification.

7.3.3 Lateral Pressures

Active earth pressure from horizontal backfill may be computed as an equivalent fluid weighting of 35 pounds per cubic foot for cantilever retaining wall and 60 pcf for restrained retaining wall. This value assumes free-draining conditions.

The effect of surcharge, such as traffic loads, adjacent building loads, and etc. within a 1 to 1 projection from the inner edge of the foundation should be included in the design of the retaining walls. For a uniformly disturbed load behind the wall, a corresponding uniformly distributed lateral soil pressure equal to 30 percent of the surcharged should be added to the equivalent fluid pressure.

Resistance to the lateral loads can be assumed to be provided by the passive earth pressure and the friction between the concrete and competent soils. Passive earth pressure may be computed as an equivalent fluid pressure of 300 pcf, with a maximum earth pressure of 3000 psf. An allowable coefficient of friction between soil and concrete of 0.30 may be used with the dead load forces. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one third (1/3).

7.4 Foundation Construction

It is anticipated that the entire structure will be underlain by onsite soils of very low expansion potential. The following presented our recommendations for the foundation construction.

7.4.1 Shallow Foundation

All footings should be founded at a minimum depth of 18 inches below the lowest adjacent ground surface. All continuous footings should have at least two No. 4 reinforcing bars placed both at the top and two No. 4 reinforcing bars placed at the bottom of the footings.

7.4.2 Mat Foundation

The thickness and reinforcement of the mat foundation should be designed by the project structural engineer. However, it is recommended that the thickness of the mat should be at least 12 inches from the geotechnical engineering viewpoint. The construction of the mat foundation should avoid excessive shrinkage cracks. Should the construction joints be utilized, the joints should be carefully located at sections of low shear stress. Reinforcing bars should be continuous across the joints. If reinforcing bar splicing is needed, the lap of the bars should be provided in accordance with the structural engineer’s design or other applicable specification.

7.5 Concrete Slab

Concrete slab should be founded on properly placed compacted fill or competent natural soils approved by the project geotechnical consultant. All un-certified fill or disturbed soils within the concrete slab areas should be removed to exposed competent natural soils then backfill with compacted fills to the design grade.

Concrete slabs should be a minimum of 4 inches thick and reinforced with a minimum of No. 4 bars at 18-inches in center both way or its equivalent. All slab reinforcement should be supported to ensure proper positioning during placement of concrete. Concrete slabs in moisture sensitive areas should be underlain with a vapor barrier consist of a minimum of 10 mil polyvinyl chloride membrane with all laps sealed. A minimum of one inch of sand should be placed over the membrane to aid in uniform curing of concrete.

The above foundation and concrete flatwork reinforcement recommendations are presented in accordance with the geotechnical engineering viewpoint. Additional reinforcement may be required in the concentrated column and/or traffic loading areas. Final reinforcement should be designed by the project structural engineer.

7.6 Retaining Wall Drainage

Walls should be backfilled with compacted fill. A free-drainage, selected backfill materials (Sand Equivalent of 30 or greater), at least 2 feet wide should be used against the wall. Onsite soil materials should be used for the upper 24 inches of the wall backfill.

A drainage system should be placed around the perimeter of the foundation or the basement walls. The system should be consist of a four-inch diameter perforated ABS SDR-35 or PVC Schedule 40, and similar non-perforated outlet pipe. The perforated portion of the pipe should be embedded in at least one cubic foot per linear foot of 3/4 inch crushed rock or its equivalent and wrapped in filter fabric. The installation of the subdrainage system should be observed by the project geotechnical engineer.

The bottom of the recommended drainage system should not be higher than the bottom of the base under the basement floor. The subdrain pipe should discharge by gravity or mechanical means into the approved drainage system that compiled with the current plumbing code in accordance with the current Los Angeles Building Code. Specific gradients, pipe routing and outlet locations, should be designed by the project civil engineer.

7.7 Temporary Excavation and Backfill

All trench excavations should conform to CAL-OSHA and local safety codes. All utilities trench backfill should be brought to near optimum moisture content and then compacted to obtain a minimum relative compaction of 90 percent of ASTM D-1557-09. All temporary excavations should be observed by a field engineer of this office so as to evaluate the suitability of the excavation to the exposed soil conditions.

8.0 INSPECTION

As a necessary requisite to the use of this report, the following inspection is recommended:

- Temporary excavations.
- Removal of surficial and unsuitable soils.
- Backfill placement and compaction.

- Utility trench backfill.

The geotechnical engineer should be notified at least 1 day in advance of the start of construction. A joint meeting between the client, the contractor, and the geotechnical engineer is recommended prior to the start of construction to discuss specific procedures and scheduling.

9.0 CORROSION POTENTIAL

Chemical laboratory tests were conducted on the existing onsite near surface materials sampled during QCI's field investigation to aid in evaluation of soil corrosion potential and the attack on concrete by sulfate soils. The testing results are presented in Appendix B.

According to CBC and ACI 318, Table 4.3.1, a "negligible" exposure to sulfate can be expected for concrete placed in contact with the onsite soils. Therefore, Type II cement or its equivalent may be used for this project. Based on the resistivity test results, it is estimated that the subsurface soils are corrosive to buried metal pipe. It is recommended that any underground steel utilities be blasted and given protective coating. Should additional protective measures be warranted, a corrosion specialist should be consulted.

10.0 SEISMIC DESIGN

Based on our studies on seismicity, there are no known active faults crossing the property. However, the subject site is located in southern California, which is a tectonically active area. Based on 2010 California Building Code (Chapter 16) the following seismic related values may be used:

Seismic Parameters (Latitude: 33.776913, Longitude: -117.957778)	
Mapped 0.2 Sec Period Spectral Acceleration **S_S**	1.396g
Mapped 1.0 Sec Period Spectral Acceleration **S_1**	0.500g
Site Coefficient for Site Class "D", **F_a**	1.0
Site Coefficient for Site Class "D", **F_v**	1.5
Maximum Considered Earthquake Spectral Response Acceleration Parameter at 0.2 Second, **S_{MS}**	1.396g
Maximum Considered Earthquake Spectral Response Acceleration Parameter at 1.0 Second, **S_{M1}**	0.750g
Design Spectral Response Acceleration Parameters for 0.2 sec, **S_{DS}**	0.931g
Design Spectral Response Acceleration Parameters for 1.0 Sec, **S_{D1}**	0.500g

The Project Structural Engineer should be aware of the information provided above to determine if any additional structural strengthening is warranted.

11.0 INVESTIGATION LIMITATIONS

The materials encountered on the subject site and utilized in our laboratory testing program are believed to be representative of the area. However, soil may vary in characters between the exploratory borings. Since our investigation is based on the site materials observed, selected laboratory testing, and engineering analyses, the conclusions and recommendations are professional opinion. These opinions have been derived in accordance with current standard of practice, and no warranty is expressed or implied.

12.0 REFERENCES

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www.conservation.ca.gov/cgs/rghm/psha/fault_parameters/pdf/Documents/B_flt.pdf

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APPENDIX A

FIELD INVESTIGATION

Our subsurface conditions were explored by drilling five 8-inch diameter hollow stem auger borings to a maximum depth of 51.5 at approximate location shown on the enclosed Site Plan (Figure 2). Upon completion of excavating, the boreholes were backfilled with onsite soils that were removed from the excavations.

The drilling of the test boring was supervised by an engineer of this office, who continuously logged the borings and visually classified the soils in accordance with the Unified Soil Classification System. Ring and SPT samples were taken at frequent intervals. These samples, taken by the hollow stem auger, were obtained by driving a ring and SPT sampler with successive blows of 140-pound hammer dropping from a height of 30 inches.

Representative undisturbed samples of the subsurface soils were retained in a series of brass rings, each having an inside diameter of 2.42 inches and a height of 1.00 inch. All ring samples were transported to our laboratory. Bulk surface soil samples were also collected for additional classification and testing.

APPENDIX B
LABORATORY TESTING

During the subsurface exploration, QCI personnel collected relatively undisturbed ring samples and bulk samples. The following tests were performed on selected soil samples:

Moisture-Density

The moisture content and dry unit weight were determined for each relatively undisturbed soil sample obtained in the test borings in accordance with ASTM D2937 standard. The results of these tests are shown on the boring logs in Appendix A.

Shear Tests

Shear tests were performed in a direct shear machine of strain-control type in accordance with ASTM D3080 standard. The rate of deformation was 0.010 inch per minute. Selected samples were sheared under varying confining loads in order to determine the Coulomb shear strength parameters: internal friction angle and cohesion. The shear test results are presented in the attached Figures.

Consolidation Tests

Consolidation tests were performed on selected undisturbed soil samples in accordance with ASTM D2435 standard. The consolidation apparatus is designed for a one-inch high soil filled brass ring. Loads are applied in several increments in a geometric progression and the resulting deformations are recorded at selected time intervals. Porous stones are placed in contact with the top and bottom of each specimen to permit addition and release of pore fluid. The samples were inundated with water at a load of two kilo-pounds (kips) per square foot, and the test results are shown on the attached Figures.

Corrosion Potential

Chemical laboratory tests were conducted on the existing onsite near surface materials sampled during QCI's field investigation to aid in evaluation of soil corrosion potential and the attack on concrete by sulfate soils. These tests are performed in accordance with California Test Method 417, 422, 532, and 643. The testing results are presented below:

Sample Location	PH	Chloride (ppm)	Sulfate (% by weight)	Min. Resistivity (ohm-cm)
B-1 @ 0' - 3'	7.39	90	0.0195	1,600

Grain Size Analysis

Grain size distribution was determined for selected soil samples in accordance with ASTM D422 standard. The test results are presented in the attached plate.

Atterberg Limits

Liquid and plastic limits was determined for selected clayey samples in accordance with ASTM D4318 standard. The test results are presented in the attached grain size distribution curve.

Expansion Index

Expansion Index test was conducted on the existing onsite near surface materials sampled during QCI's field investigation. The test is performed in accordance with ASTM D-4829. The testing results are presented below:

Sample Location	Expansion Index	Expansion Potential
B-1 @ 0-3'	5	Very Low

Percent of Fine

Percent of fine was determined for selected soil samples in accordance with ASTM D1140 standard. The test results are presented in the attached table.

B-3

Sample Location	Percent of Fine
B-3 @ 5' & 10'	18.1
B-3 @ 25'	18.4
B-3 @ 30'	10.1
B-3 @ 35'	26.0
B-3 @ 40'	48.9

B-4

Sample Location	Percent of Fine
B-4 @ 5'	16.8
B-4 @ 10'	11.1
B-4 @ 25'	27.3
B-4 @ 30'	5.0
B-4 @ 35'	55.1

Grain Size Analyses

Sieve analyses were determined for selected soil samples in accordance with ASTM D422. The test results are presented in the attached plates.

Atterberg Limits

Laboratory Atterberg Limit tests were conducted on the existing onsite materials sampled during QCI's field investigation to aid in evaluation of soil liquefaction potential. These tests are performed in accordance with ASTM D4318. The testing results are presented below:

Sample Location	USCS Class. ASTM D2488	Natural Moisture Content MC%	Liquid Limit ASTM D4318 LL	Plastic Limit ASTM D4318 PL	Plasticity Index ASTM D4318 PI
B-3 @15'	CL	17.3	40	23	17
B-3 @20'	CH	30.4	52	26	26
B-3 @ 40'	CL	27.1	43	22	21
B-3 @ 45'	CH	30.8	53	28	25
B-3 @ 50'	CH	31.8	54	28	26
B-4 @ 15'	CL	26.0	39	23	16
B-4 @ 20'	CL	24.0	41	24	17
B-4 @ 35'	ML	26.9	38	27	11
B-4 @ 40'	CL	30.6	42	24	18
B-4 @ 45'	CH	31.0	55	28	27
B-4 @ 50'	CH	31.3	53	27	26

APPENDIX C

RESULTS OF LIQUEFACTION ANALYSIS