# APPENDIX E
# Geotechnical Investigation

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


April 17, 2019

Arroyo Development , LLC
2409 Strozier Avenue, suite A
South El Monte, California 91733

Attention: Mr. Frank Lac

Subject: Report Update, 235 S. Arroyo Drive, APN: 5346-011-004, San Gabriel, California, Project No.: 14-010-024a

References: "Report of Geotechnical Engineering Investigation, Proposed 4-Story Residential Development, 235 S. Arroyo Drive, APN: 5346-011-004, San Gabriel, California" by Cal Land Engineering, Inc., Project No. 14-010-024a dated June 11, 2015.

Gentlemen:

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 the ASCE 7-10 Standard, CBC 2016, the following seismic related values may be used:

| Seismic Parameters (Latitude: 34.099906 Longitude: -118.113176) | Site Class "D" |
|---|---|
| Mapped 0.2 Sec Period Spectral Acceleration **$S_S$** | 2.806g |
| Mapped 1.0 Sec Period Spectral Acceleration **$S_1$** | 0.928g |
| Site Coefficient for Site Class "D", **Fa** | 1.0 |
| Site Coefficient for Site Class "D", **Fv** | 1.5 |
| Maximum Considered Earthquake Spectral Response Acceleration Parameter at 0.2 Second, **$S_{MS}$** | 2.806g |
| Maximum Considered Earthquake Spectral Response Acceleration Parameter at 1.0 Second, **$S_{M1}$** | 1.392g |
| Design Spectral Response Acceleration Parameters for 0.2 sec, **$S_{DS}$** | 1.871g |
| Design Spectral Response Acceleration Parameters for 1.0 Sec, **$S_{D1}$** | 0.928g |

Peak ground acceleration (PGA), corresponding to USGS Design Map Summary Report, ASCE 7-10 Standard is 1.068g. The Project Structural Engineer should be aware of the information provided above to determine if any additional structural strengthening is warranted.

This opportunity to be of service is sincerely appreciated. Should you have any questions pertaining to this addendum, please call us.

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

REGISTERED PROFESSIONAL ENGINEER
JACK C. LEE
No. 2153
Exp. 3/31/21
GEOTECHNICAL
STATE OF CALIFORNIA

Jack C. Lee, GE 2153

Abe Kazemzadeh


576 East Lambert Road, Brea, California 92821; Tel: 714-671-1050; Fax: 714-671-1090

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

June 11, 2015

Arroyo Development , LLC
2409 Strozier Avenue, suite A
South El Monte, California 91733

Attention: Mr. Frank Lac

Subject: Report of Geotechnical Engineering Investigation, Proposed 4-Story Residential Development, 235 S. Arroyo Drive, APN: 5346-011-004, San Gabriel, California. QCI Project No.: 14-010-024aGE

Gentlemen:

In accordance with your request, Quartech Consultants (QCI) has prepared this geotechnical engineering report for the proposed development at the subject site. The purpose of this report was to evaluate the subsurface conditions and to provide recommendations for foundation designs and other relevant parameters for the proposed construction.

Based on the findings and observations during our investigation, it is concluded that the subject site is suitable for its intended use from the geotechnical engineering viewpoint, provided that 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,
**CalLand Engineering, Inc. (CLE)**
**dba Quartech Consultants (QCI)**

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

Dist: (4) Addressee

# REPORT OF GEOTECHNICAL ENGINEERING INVESTIGATION AND

**Proposed**
**4-Story Residential Development**

**At**

**APN: 5346-011-004**
**235 S. Arroyo Drive**
**San Gabriel, California**

Prepared by

QUARTECH CONSULTANTS (QCI)

Project No.: 14-010-024aGE

June 11, 2015

## TABLE OF CONTENT

# 1.0 INTRODUCTION

## 1.1 Purpose

This report presents a summary of our preliminary geotechnical engineering investigation for the proposed development at the subject site. The purposes of this investigation were to evaluate the subsurface conditions at the area of proposed construction and to provide recommendations pertinent to grading, foundation design and other relevant parameters.

## 1.2 Scope of Services

Our scope of services included the followings:

- Review of available soil and geologic data of the subject site and its vicinity.
- Subsurface exploration consisting of logging and sampling of two 8-inch diameter hollow stem auger borings to a maximum depth of 51.5 feet below the existing grade at the subject site. The exploration was logged by a QCI engineer. Boring logs are presented in Appendix A.
- Laboratory testing of representative samples obtained from the subject site to investigate engineering characteristics of the onsite 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 to present our findings, conclusions, and recommendations for the proposed construction.

## 1.3 Proposed Construction

Based on the provided information, it is our understanding that the subject site will be developed for construction of a 46 condominium units. The main structure of the building is anticipated to be four stories in height above the ground level with one level of subterranean garage. The lowest garage floor will be approximately 10 feet below the existing ground surface. The subterranean garage will occupy the entire building site. No detail design structural loads were available at the time when this report was prepared.

## 1.4 Site Location

The project site is located at the west of Arroyo Drive and Alhambra Wash, between southwest Hampton Court and northeast Vega street in the City of San Gabriel, California. The approximate location of the site is presented in the attached Site Location Map (Figure 1). The existing drainage channel “Alhambra Wash” is located east of the site and west of Arroyo Drive. Detailed configuration of the site is presented in the attached Site Plan, Figure 2.

## 2.0 FIELD EXPLORATION AND LABORATORY TESTING

### 2.1 Field Exploration

Our subsurface exploration consisted of excavating two 8-inch diameter hollow stem auger borings to a maximum depth of 51.5 feet below the existing ground surface at the subject site. Approximate locations of the borings are shown on the attached Site Plan (Figure 2). The purpose of the explorations was to assess the engineering characteristics of the onsite soils with respect to the proposed development.

The borings were logged by a representative of this office. Relatively undisturbed and bulk samples were collected during drilling for laboratory testing. Natural soil was encountered in the borings to the depths explored. 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, Atterberg Limits, percent fines, expansion and corrosion potential. 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 medium dense and slightly moist condition. Underlying the surface soils, silty sand (SM), and sand/silty sand mixtures (SP-SM), were disclosed in the borings to the depths explored (51.5 feet below the existing ground surface). These soils exist in the slightly moist to moist conditions. The soils become denser as depth increases.

### 3.2 Groundwater

No groundwater was encountered in the borings to the depths explored (51.5 ft.). Based on our review of the “Historically Highest Ground Water Contours and Borehole Log Data Locations, El

Monte Quadrangle", by CDMG, it is estimated that the highest ground water level is approximately 140 to 150 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 SEISMICITY

### 4.1 Faulting

Based on our study, there are no known active faults crossing the property. The nearest known active regional fault is the Raymond Fault zones located approximately 1.6 miles from the site.

### 4.2 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 depend on the distance to causative faults, the intensity, and the magnitude of the seismic event. Table 1 indicates the distance of the fault zones and the associated maximum magnitude earthquake that can be produced by nearby seismic events. As indicated in Table 1, the Raymond Fault zones are considered to have the most significant effect to 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) |
|---|---|---|
| Raymond | 1.6 | 6.8 |
| Elysian Park (Upper) | 2.3 | 6.7 |
| Verdugo | 3.2 | 6.9 |
| Sierra Madre | 5.8 | 7.2 |
| Hollywood | 6.9 | 6.7 |
| Elsinore-W | 8.1 | 7.0 |
| Clamshell-Sawpit | 8.4 | 6.7 |
| Puente Hills (LA) | 9.1 | 7.0 |
| Santa Monica | 9.9 | 7.4 |
| Puente Hills (Santa Fe Spring) | 12.7 | 6.7 |
| San Jose | 14.0 | 6.7 |
| Puente Hills (Coyote Hills) | 14.8 | 6.9 |
| Newport-Inglewood,Conn. alt 2 | 15.8 | 7.5 |

References: 2008 National Seismic Hazard Maps-Source Parameters

### 4.3 Estimated Earthquake Ground Motion

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 2% and 10% probability of exceedance in 50 years is about 1.034g and 0.595g respectively (NSHMP, 2008 Deaggregation of Seismic Hazards).

## 5.0 CONCLUSIONS

Based on the results of our subsurface investigation, it is our opinion that the proposed construction is feasible from a geotechnical standpoint, provided the recommendations contained herein are incorporated in the design and construction. The following is a summary of the geotechnical design and construction factors that may affect the development of the site:

### 5.1 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.

### 5.2 Seismic Induced Hazards

Based on our review of the “Seismic Hazard Zones, El Monte Quadrangle” by California Department of Conservation, Division of Mines and Geology, it is concluded that the site is not located in the mapped potential liquefaction areas.

### 5.3 Excavatability

Based on our subsurface investigation, excavation of the subsurface materials should be accomplished with conventional earthwork equipment.

### 5.4 Surficial Soil Removal

The near surface soils are relatively dry and vary in density. In order to provide a uniform support for the foundation, it is recommended the existing soil be removed and backfilled with compacted fill to a minimum depth of 4 feet below the existing grade to provide a uniform support of the structures.

### 5.5 Groundwater

Groundwater was not encountered during our field exploration. Groundwater is not anticipated to be encountered during the near surface construction.

## 6.0 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.

### 6.1 Grading

6.1.1 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.

6.1.2 Surficial Soil Removals

It is anticipated that most unsuitable or and loose near surface soils will be removed by excavation for the subterranean parking structures. It is recommended that the subterranean garage areas be cut to grade then 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.

6.1.3 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.

6.1.4 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.

**6.2 Subterranean Garage Excavation**

The required excavation for the proposed subterranean garage will extend to a maximum of approximately 10 to 12 feet below the existing ground surface. The excavation will have minor impact to the adjacent structures. 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.

6.2.1 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.

6.2.2 Shoring

Shoring will be required for temporary excavation made vertically or near vertically. 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. The upper 10 feet of the shoring is recommended to be designed to resist an additional pressure of 200 pounds per square ft. resulting from the traffic in the adjacent street. 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.

**6.3 Foundation Design**

Based on our subsurface investigation, it is our opinion that the proposed building may be supported on shallow foundation. 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:

6.3.1 Conventional Shallow Foundation

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 18 inches deep and founded on soils approved by the project geotechnical engineer. This bearing value may be increased by 200 psf for each additional foot of depth or width to a maximum value of 2500 psf. This value may be increased by one third (1/3) when considering short duration seismic or wind loads.

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 2000 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).

6.3.2 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.

6.3.3 Seismic Loading

Active earthquake earth pressure distribution on retaining walls retaining more than 6 feet of soils when the slope of the backfill behind the wall is level may be computed as an inverted right triangle with 32H psf at the top (where H is the height of the walls). The earthquake induced earth pressure may be applied as an inverted triangle (inverted equivalent fluid pressure) with largest dynamic earth pressure occurring at the top of the wall (upper ground surface). Resultant seismic earth force may be applied at approximately 0.6xH from the bottom of the wall.

**6.4 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.

It is anticipated that the entire structure will be underlain by onsite soils of very low expansion potential. 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 bar placed both at the top and two No. 4 reinforcing bar placed at the bottom of the footings.

**6.5 Concrete Slab**

Concrete slab should be founded on properly placed compacted fill or competent natural soils approved by the project geotechnical consultant. All 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. 3 reinforcing bar spaced 18-inch each way or its equivalent. All slab reinforcement should be supported to ensure proper positioning during placement of concrete.

In order to comply for the moisture sensitive area with the requirements of the 2013 CalGreen Section 4.505.2.1, a minimum of 4-inch thick base of ½ inch or larger clean aggregate should be provided with a vapor barrier in direct contact with concrete. A 10-mil Polyethylene vapor retarder, with joints lapped not less than 6 inches, should be placed above the aggregate and in direct contact with the concrete slab. 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.

### 6.6 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 18 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 City Building Code. Specific gradients, pipe routing and outlet locations, should be designed by the project civil engineer.

### 6.7 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.

## 7.0 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.

## 8.0 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 ASCE 7 –10 Standard (CBC 2013), the following seismic related values may be used:

| Seismic Parameters (Latitude: 34.099906, Longitude:-118.113176) | |
|---|---|
| Mapped 0.2 Sec Period Spectral Acceleration **$S_S$** | 2.806g |
| Mapped 1.0 Sec Period Spectral Acceleration **$S_1$** | 0.928 |
| 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, **$S_{MS}$** | 2.806 |
| Maximum Considered Earthquake Spectral Response Acceleration Parameter at 1.0 Second, **$S_{M1}$** | 1.392 |
| Design Spectral Response Acceleration Parameters for 0.2 sec, **$S_{DS}$** | 1.871 |
| Design Spectral Response Acceleration Parameters for 1.0 Sec, **$S_{D1}$** | 0.928 |

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

## 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 318-11, 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 moderately 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 REMARKS

The conclusions and recommendations contained herein are based on the findings and observations at the exploratory locations. However, soil materials may vary in characteristics between locations of the exploratory locations. If conditions are encountered during construction, which appear to be different from those disclosed by the exploratory work, this office should be notified so as to recommend the need for modifications.

This report has been prepared in accordance with generally accepted professional engineering principles and practice. No warranty is expressed or implied. This report is subject to review by controlling public agencies having jurisdiction.

## 11.0 REFERENCES

California Division of Mines and Geology, 1998, Seismic Hazard Zone Report for the El Monte 7.5-minutes Quadrangle, Los Angeles County, California Seismic Hazard Zone report 024.

www.conservation.ca.gov/cgs/rghm/psha/fault_parameters/pdf/Documents/B_flt.pdf
https://geohazards.usgs.gov/deaggint/2008/
http://eqint.cr.usgs.gov/deaggint/2008/index.php
http://earthquake.usgs.gov/research/software/
http://earthquake.usgs.gov/hazards/qfaults/
“http://geohazards.usgs.gov/deaggint/2008/Earthquake Hazards Program, Seismic Design Maps and tools”, ASCE 7-10 Standard
http://www.conservation.ca.gov/cgs/shzp/pages/index.aspx

# APPENDIX A
# FIELD INVESTIGATION

Subsurface conditions were explored by drilling two 8-inch diameter hollow stem auger borings to a maximum depth of 51.5 feet below the existing grade at the subject site. The approximate boring location is shown on the enclosed Site Plan, Figure 2.

The drilling of the boring was supervised by a QCI's engineer, 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 from hollow stem drilling rig were obtained by driving a 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 plates.

**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 @ 0-5’ | 6.79 | 164 | 0.0010 | 4,600 |

**Expansion Index**

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

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

**Percent Passing #200 Sieve**

Percent of soil passing #200 sieve were determined for selected soil samples in accordance with ASTM D1140 standard. The test results are presented in the following table:

| Sample Location | % Passing #200 |
|---|---|
| B-1 @ 5’ | 27.2 |
| B-1 @ 10’ | 10.8 |
| B-1 @ 15’ | 35.6 |
| B-1 @ 20’ | 38.2 |
| B-1 @ 25’ | 14.3 |
| B-1 @ 30’ | 47.7 |
| B-1 @ 35’ | 30.6 |
| B-1 @ 40’ | 14.9 |
| B-1 @ 45’ | 8.5 |
| B-1 @ 50’ | 10.8 |

**Atterberg Limits**

Laboratory Atterberg Limits 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 | Liquid Limit %ASTM D4318 | Plastic Limit %ASTM D4318 | Plasticity Index %ASTM D4318 | % Passing #200 |
|---|---|---|---|---|---|
| B-1 @ 15’ | SM | 25 | NP | NP | 35.6 |