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

November 8, 2019

Taiwan Center
3001 Walnut Grove,
Rosemead, CA 91770

Attention: Mr. Alan Thian

Subject: Report of Geotechnical Engineering Investigation, Proposed 4-Story Mixed Use Development, with One-Level of Subterraneous Garage, 3001 Walnut Grove Avenue and nearby lots, APN: 5288-001-040, 041, 042, 043, Rosemead, California. QCI Project No.: 19-221-001GE

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)

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REPORT OF GEOTECHNICAL ENGINEERING INVESTIGATION

Proposed

4-Story Mixed Used Development
With 1 level Subterranean Garage

At

3001 Walnut Grove and nearby lots
APN: 5288-001-040, 041, 042, 043
Rosemead, California

Prepared by

QUARTECH CONSULTANTS (QCI)

Project No.: 19-221-001GE

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1.0 INTRODUCTION

1.1 Purpose

This report presents a summary of our preliminary geotechnical engineering investigation for the proposed construction 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 of the development.

1.2 Scope of Services

Our scope of services included:

- Review of available soil engineering data of the area.
- Our subsurface investigation consisted of excavation 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 to establish engineering characteristics of the on-site soil. The laboratory test results are presented in Appendices A and B.
- 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.

1.3 Proposed Construction

Based on the 16-scale architectural plan by SLA Architects dated August 26, 2019, it is our understanding that the subject site will be developed for construction of a commercial and residential mixed used building. 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.

1.4 Site Location

The project site is located on the northwest corner of Garvey Avenue and Walnut Grove Avenue, in the City of Rosemead, California. The approximate location of the site is presented in the attached Site Location Map (Figure 1). The site is relatively flat and is currently occupied by a commercial building and associated improvements. No major surface erosions were observed during our subsurface investigation.

2.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

2.1 Subsurface 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, percent of fines, expansion, Atterburg limits, and corrosion potential. 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 presented on the boring logs in Appendix A.

3.0 SUMMARY OF GEOTECHNICAL CONDITIONS

3.1 Soil Conditions

The onsite near surface soils consist predominantly of clayey sand (SC). In general, these soils exist in the loose and moist condition. Underlying the surface soils, fine grained clayey sand (SC), silty sand (SM), poorly graded sand (SP) and sandy clay (CL) were disclosed in the borings to the depths explored (51.5 feet below the existing ground surface). These soils exist in medium dense to very dense and very stiff and slightly moist to very moist conditions. In general, the soils become denser as depth increases.

3.2 Groundwater

Ground water level was not encountered to the depth explored (approximately 51.5 feet below the existing grade) during our subsurface investigation. In our opinion, groundwater will not be a problem during the near surface construction. Based on our review of the “Historically Highest Ground Water Contours and Borehole Log Data Locations, El Monte Quadrangle”, by CGS (formerly CDMG), it is estimated that the highest historical ground water level is approximately 10 feet below the existing grade.

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 Upper Elysian Park fault located at 1.1 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 Upper Elysian Park fault is 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 the Site (mile)	Maximum Magnitude Earthquake (Mmax)
Elysian Park (Upper)	1.1	6.7
Raymond	4.4	6.8
Elsinore;W+GI+T+J+CM	5.1	7.8
Verdugo	6.2	6.9
Puente Hills (LA)	7.0	7.0
Sierra Madre Connected	7.8	7.3
Clamshell-Sawpit	9.2	6.7
Hollywood	9.4	6.7
Puente Hills (Santa Fe Springs)	9.8	6.7
San Jose	11.8	6.7
Puente Hills (Coyote Hills)	12.0	6.9
Santa Monica Connected alt 2	12.0	7.4
Newport Inglewood Connected alt 2	15.4	7.5
Newport-Inglewood, alt 1	15.7	7.2
Newport Inglewood Connected alt 1	15.7	7.5
Santa Monica, alt 1	18.9	6.6
Santa Monica Connected alt 1	18.9	7.3
Sierra Madre (San Fernando)	19.2	6.7
Chino, alt 2	19.4	6.8

Reference: 2008 National Seismic Hazard Maps-Source Parameters

4.3 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 2% and 10% probability of exceedance in 50 years is about 0.863g and 0.517g, respectively (USGS, 2008 Deaggregation of Seismic Hazards).
Site modified peak ground acceleration (PGA_M), corresponding to USGS Seismic Design Maps, ASCE 7-10 Standard, is 0.949g.

5.0 SEISMIC HAZARDS

5.1 Liquefaction

Liquefaction is the transformation of a granular material from a solid to a liquid state as a result of increasing 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 "LIQUEFY2" by Thomas Blake, the subsurface data from Boring B-1, the design earthquake (M =7.0), and ground acceleration of 0.863g (2% probability of exceedance in 50 years). The total unit weight used for the onsite soils is 120 pcf. The calculated ground water level is raised to the depth of 10 feet below the existing ground surface. Conversion from California modified split spoon to field SPT blow counts is 0.7 (County of L.A. GS045.0 October 1, 2014). The analyses presented on the enclosed Appendix C indicated that the factor of safety is less than 1.30 for the onsite soils at the depth of 37 to 42 feet.

Based on the laboratory test results on clayey soils, for B-1 @ 30 @ 50 feet, the saturated moisture content of the encountered clayey soils is less than 85 percent of liquid limit when PI is less than 12 (County of L.A., GS045.0, October 1, 2014 and Bray and Sancio 2006, if PI is less than 12 and wc/LL is less than 0.85, the clayey soil 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 soils are not susceptible to liquefaction.

5.2 Earthquake 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 our opinion that total seismic induced settlement and differential settlement of saturated sand are **0.70 inches** and **0.47 inches** respectively.

5.3 Landsliding

A potential for landsliding is often suggested in areas of moderate to steep terrain that is underlain by weak or un-favorably oriented geological conditions. Neither of these conditions exists at the site. Due to the relatively flat nature of the site, it is our opinion that the potential for landslide is remote.

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 low due to the absence of known active faults at the site.

5.6 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 evaluation procedures suggested by the Ishihara (1985), it is concluded that surface manifestation of liquefaction is unlikely at the subject site under the design earthquake event.

5.7 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

Based on the results of our subsurface investigation, it is our opinion that the proposed improvements are 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:

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.

6.2 Liquefaction Potential

Based on our field investigation, liquefaction analyses and laboratory testing, the analyses presented on the enclosed Appendix C indicated that the factor of safety is less than 1.30 for the onsite soils at the depth of 37 to 42 feet.

6.3 Excavatability

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

6.4 City of Rosemead Fault Hazard Management Zones (HFMZ)

The site is not located within the designated City of Rosemead Safety Element as shown on Figure 1a, City of Rosemead Fault Hazard Management Zones (FHMZ). Based on examining the exploratory borings, B-1 and B-2 and nearby site’s exploratory borings, it was determined that the soil borings are similar within the site underlying soil materials.

6.5 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 near surface structures.

6.6 Groundwater

No groundwater was encountered in the borings to the depths explored. 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 Site 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

It is anticipated that most unsuitable or and loose near surface soils will be removed by excavation for the basement structures. It is QCI's opinion that no additional removal will be necessary within the basement areas. However, it is recommended that the basement areas be cut to grade then observed by a representative of this office to verify the sub-grade soil conditions for any potential needs of removal loose soils and replacement with compacted fill. This may be also necessary due to difference in expansion characteristics of foundation materials beneath a structure.

Outside the basement areas, the near surface soils should be removed to expose competent natural soils. Based on our field exploration and laboratory data obtained to date, it is recommended that the surficial soils be removed to a depth of at least 2 feet below existing grade for the uniform concrete flatworks support. Locally deeper removals may be necessary to expose competent natural ground. The actual removal depths should be determined in the field as conditions are exposed. Visual inspection and/or testing may be used to define removal requirements.

7.1.3 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.

7.1.4 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, then compacted to at least 90 percent relative compaction based on laboratory standard ASTM D-1557-12.

7.2 Subterranean Garage Excavation

The required excavation for the proposed subterranean parking garage will be around 10 to 12 feet below ground. 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 1:1 (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 35 pound per cubic foot may be used for the temporary cantilever shoring system. Any surcharge loads resulting from adjacent buildings or traffic should be considered as an added load to the design. Based on the existing on site conditions, it is recommended that a uniform lateral surcharge pressure of 70 psf may be used for the traffic loads along Walnut Grove and Garvey Avenue. Soldier piles or beams should be spaced at the specification 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.

7.2.3 Slot Cut

Should the ABC slot cut method be used for the onsite vertical excavation of more than 5 feet in height, the following presents the slot cut recommendations. The slot cut stability analysis is presented in the attached plate.

1. Excavate to the design elevation at the side slopes no steeper than 1:1, horizontal to vertical.

2. Excavate in alternative slots with each slot no wider than the design width (i.e. 10 feet)

3. Excavate the footings at each slot, pour the footings and construct the walls per project standard. The depth of vertical cut should be limited to no more than 10 feet.

4. After completion of the slope construction, excavate the adjacent slots and repeated the above procedures to complete the adjacent slope.

5. All excavations should be made under the inspection and testing of the project geotechnical consultant.

6. Care should be taken to prevent surcharge loads above un-shored slots within a horizontal distance from the top of cut equal to depth of excavation.

7. Provisions for drainage should be implemented to prevent saturation of un-shored excavations.

8. Once vertical excavations are completed. The basement/retaining wall should be constructed without delay.

7.3 Foundation Design

Based on our subsurface investigation, it is our opinion that the proposed building may be supported on shallow foundation founded on the competent nature soil at the depth of 10 to 12 feet below the existing grade. The following presents our preliminary recommendations:

7.3.1 Conventional Foundation (Building)

An allowable bearing value of 2500 pounds per square foot (psf) may be used for design of continuous or pad footings with a minimum of 18 or 36 inches in width, respectively. All footings should be a minimum of 24 inches deep. This value may be increased by one third (1/3) when considering short duration seismic or wind loads. This bearing value may be increased by 300 psf for each additional foot of depth or width to a maximum value of 3500 psf. This value may be increased by one third (1/3) when considering short duration seismic or wind loads.

7.3.2 Settlement

Settlement of the footings placed as recommended, and subject to no more than allowable loads is not expected to exceed 1/2 inch. Differential settlement between adjacent columns is not anticipated to exceed 1/4 inch for the adjacent column spaced at a distance of about 30 feet. Additionally, the foundation should also be designed to resist the potential seismic induced total settlement and differential settlement of **0.70 inches** and **0.47 inches**, respectively.

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. Based on the existing on site conditions, it is recommended that a uniform lateral surcharge pressure of 70 psf for the traffic loads for be added in the basement wall design along Walnut Grove and Garvey Avenue.

Based on our review of the 10-scale tentative map, it is understood that the proposed basement will be located at least 5 feet away from the existing adjacent neighbor 1-story commercial structures. The lateral surcharged load from the adjacent foundation is then calculated based on the adjacent building located at 6 feet away from the wall and the basement wall is 10 feet below the existing grade.

Reference: NAVFAC DM 7.02, Figure 11, Page 7.2-74

Foundation Line Load Q = 2000 lbs
Distance Between Wall to Foundation X = 6 feet
Depth of Basement Wall H = 10 feet

m = 6/10 = 0.6 > 0.4 n = 0.6 from bottom of the wall

Resultant P_H = 0.64 x Q / (mxm +1) = 941 pounds < 1000 pounds

Based on our calculations, it is our opinion that the recommended horizontal surcharge of 1000 pounds per square feet act at approximately 0.6xH (H: height of wall) from the bottom of the wall.

7.3.4 Lateral Resistance Pressures

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 350 pcf, with a maximum earth pressure of 3500 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).

7.3.5 Wall Seismic Loading

Earthquake earth pressure distribution on cantilever 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 31H psf at the base. Resultant seismic earth force may be applied at approximately 0.6xH from the top of the footing. H should be measured from top of footing to the top of wall. The earthquake-induced pressure should be added to the static earth pressure. Design of walls less than 6 feet in height may neglect the additional seismic pressure.

7.3.6 Wall Drainage

Any proposed retaining walls at the site should be provided with backdrains to reduce the potential for the buildup of hydrostatic pressure. Backdrains should consist of 4-inch (minimum) diameter perforated PVC pipe surrounded by a minimum of 1 cubic foot per lineal foot of clean coarse gravel wrapped in filter fabric (Mirafi 140 or the equivalent) placed at the base of the wall.

The drain should be covered by no less than 18 inches (vertical) of compacted wall backfill soils. The backdrain should outlet through non-perforated PVC pipe or weepholes. Alternatively, commercially available drainage fabric (i.e., J-drain) could be used. The fabric manufacturer's recommendations should be followed in the installation of the drainage fabric backdrain. If there is not enough room for placing the above mentioned drainage systems, an alternative system such as pre-fabricated drainage system AQUADRAIN 100 BD with a 3-inch drain pipe set in gravel behind the wall, to prevent the buildup of hydrostatic pressure. This drainpipe may be connected to a 3-inch drain collector pipe connected to a sump pump.

7. 4 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 24 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. 5 Concrete Flatwork

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. 4 reinforcing bar spaced 18-inch each way or its equivalent. All slab reinforcement should be supported to ensure proper positioning during placement 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.

In order to comply with the requirements of the 2016 CalGreen Section 4.505.2.1 within the moisture sensitive concrete slabs, 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. As an alternate method, 2 inches of sand then 10 mil polyethylene membrane and another 2 inches of sand over the membrane and under the concrete may be used, provided this request for an alternative method is approved by City Building Officials.

7.6 Temporary Trench Excavation and Backfill

All trench excavations should conform to CAL-OSHA and local safety codes. All utility trenches backfill should be brought to near optimum moisture content and then compacted to obtain a minimum relative compaction of 90 percent of ASTM D-1557-12.

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 2016, the following seismic related values may be used:

Seismic Parameters (Latitude: 34.063155, Longitude: -118.082399)	Site Class “D”
Mapped 0.2 Sec Period Spectral Acceleration, S_S	2.542g
Mapped 1.0 Sec Period Spectral Acceleration, S_1	0.881g
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}	2.542g
Maximum Considered Earthquake Spectral Response Acceleration Parameter at 1.0 Second, S_{M1}	1.322g
Design Spectral Response Acceleration Parameters for 0.2 sec, S_{DS}	1.694g
Design Spectral Response Acceleration Parameters for 1.0 Sec, S_{D1}	0.881g

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

9.0 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.

10.0 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 2016 CBC and ACI 318-14, 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.

11.0 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.

12.0 REFERENCES

Seed, H.B., Tokimatsu, K., Harder, L.F., and Chung, R.M., (1985), "Influence of SPT Procedures in Soil Liquefaction Resistance Evaluations," Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol. 111, No. GT12, pp. 1425-1445.

Tokimatsu, K., and Seed, H.B., (1987), "Evaluation of Settlements in Sands Due to Earthquake Shaking," Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol. 113, No. 8, pp. 861-878.

Ishihara, K. and Yoshimine, M., (1992), "Evaluation of Settlements in Sand Deposits Following Liquefaction During Earthquakes", Japanese Society of Soil Mechanics and Foundation Engineering, Vol. 32, No. 1, pp. 173-188

Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117, Adopted by California State Mining and Geology Board in accordance with the Seismic Hazards Mapping Act of 1990, Revised and Re- adopted September 11, 2008 by the State Mining and Geology Board**.** http://www.conservation.ca.gov/cgs/shzp/pages/index.aspx

T.Y. Loud, I.M. Idriss, and et. al. (2001), "Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshops on Evaluation of Liquefaction Resistance of Soils", Journal of the Geotechnical Engineering Division, American Society of Civil Engineers, Vol. 127, No. GT10, pp. 817-833.

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 98-15. http://www.conservation.ca.gov/cgs/shzp/pages/index.aspx

EERC, "Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework", EERC Report No. 2003-06, 26th Annual ASCE Geotechnical Spring Seminar, Long Beach, April 30, 2003

City of Rosemead General Plan Update, 2008, Chapter 5: Public Safety adopted October 14, 2008. http://www3.cityofrosemead.org:8081/weblink7/ElectronicFile.aspx?docid=264&dbid=1

City of Rosemead General Plan Update, 2010, Chapter 5, Adopted April 13, 2010

Bray, J. D. and Sancio, R.B., (2006) "Assessment of liquefaction susceptibility of fine-graind soils," Jornal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 132, No. 9, pp. 1165-1177

"Earthquake Hazards Program, Seismic Design Maps and tools", ASCE 7-10 Standard

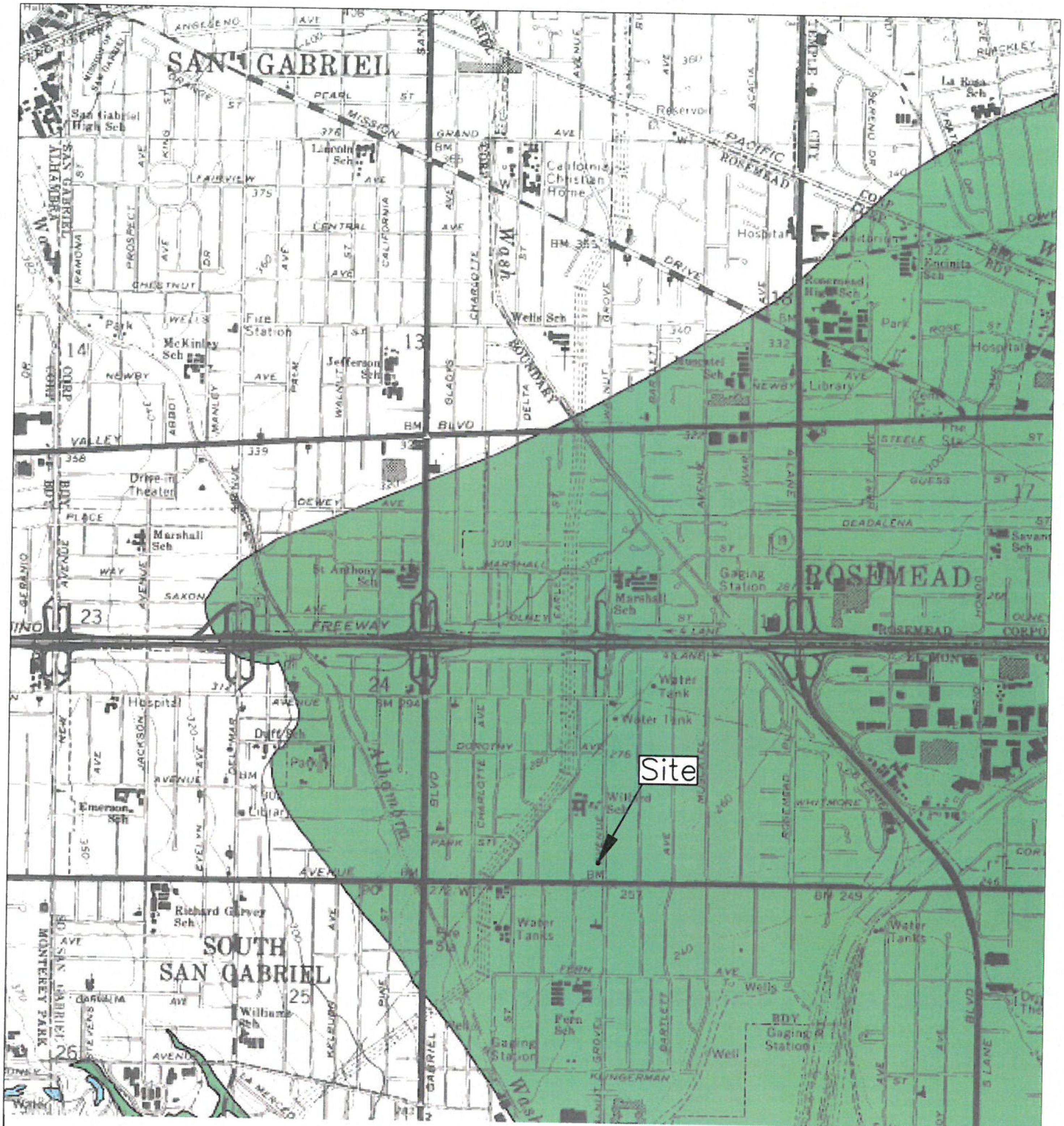
Thomas F. Blake, Liquefy 2, Version 1.50.

"County of Los Angeles, Department of Public Works", GS 045.0, Revised 10/1/2014

"Guideline for Evaluating and Mitigating Seismic Hazards in California" SP-117A

https://earthquake.usgs.gov/designmaps/us/application.php

https://earthquake.usgs.gov/hazards/interactive/
https://earthquake.usgs.gov/cfusion/hazfaults_2008_search/query_main.cfm
https://geohazards.usgs.gov/deaggint/2008/
www.conservation.ca.gov/cgs/rghm/psha/fault_parameters/pdf/Documents/B_flt.pdf
http://earthquake.usgs.gov/research/software/
http://earthquake.usgs.gov/hazards/qfaults/



SCALE: 1"=2000'

LEGEND

Maps modified from "Seismic Hazard Zones, El Monte Quadrangle" by CDMG

Area Subjected to Liquefaction Induced Settlement

CalLand Engineering, Inc.
dba Quartech Consultants

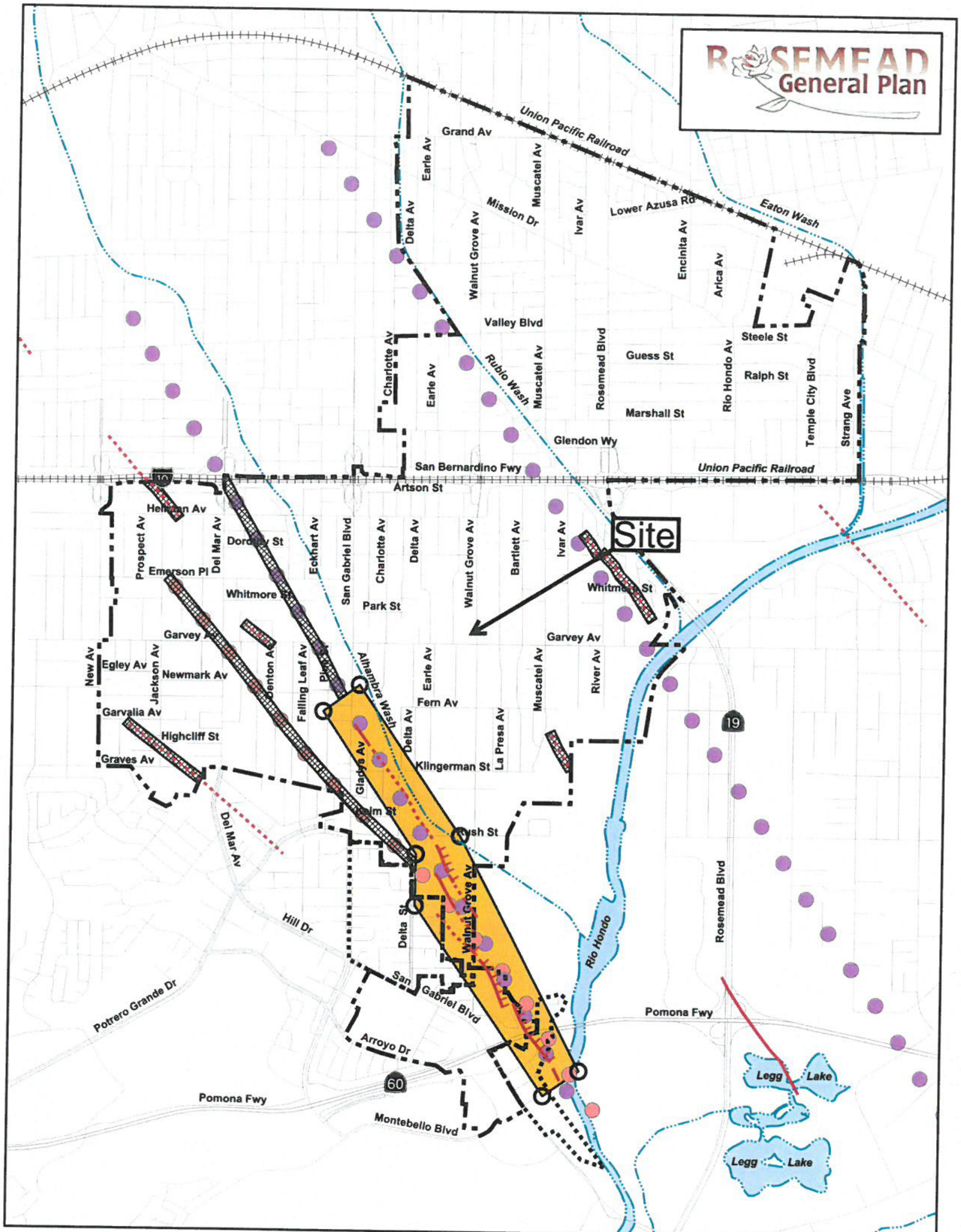
Geotechnical, Enviromental & Civil Engineering Services

Project Address:
APN: 5288-001-040, 041, 042, 043
3001 Walnut Grove
Rosemead, California

Site Location Map

11/19

FIGURE 1



Fault Hazard Management Zone (FHMZ) for Important Facilities

Alquist-Priolo Earthquake Fault Zone

Approximate location of escarpment of Bullard and Lettis (1992)

Inferred faults from California Department of Water Resources (1966)

Photolineaments defining Probable or Possible fault

Well-defined

Less well-defined

Indicate downside of scarp

Sources: CDMG, 1991, Treiman, FER-222; Bullard and Lettis, 1992; CDWR, 1966.

Feet

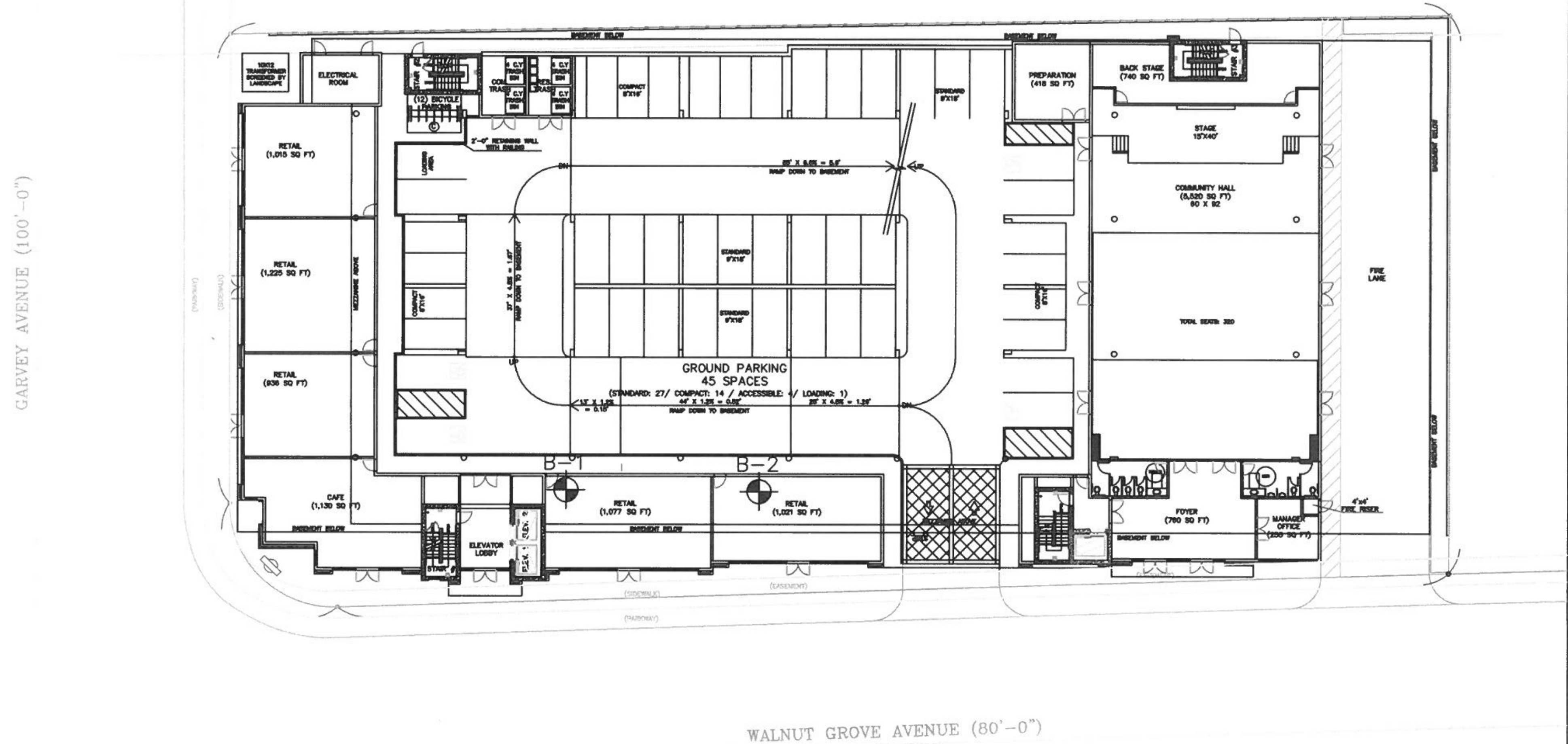
0 1,000 2,000 3,000 4,000

Fault Hazard Management Zones (FHMZ)

City of Rosemead
General Plan Update

June 2008

Figure 1a



LEGEND

B-1 Approximate boring location

SCALE: 1" = 40'

CalLand Engineering, Inc. dba Quartech Consultants Geotechnical, Environmental & Civil Engineering Services	Project Address: APN: 5288-001-040, 041, 042,043 3001 Walnut Grove Rosemead, California

SITE PLAN

11/19

FIGURE 2

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 at approximate locations shown on the enclosed Site Plan, Figure 2.

The drilling of the test borings was supervised by a QCI engineer, who continuously logged the borings and visually classified the soils in accordance with the Unified Soil Classification System. Ring samples were taken at frequent intervals. These samples 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.

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BORING LOG B-1

PROJECT LOCATION: 3001 Walnut Grove Ave., Rosemead, CA
PROJECT NO.: 19-221-001

DATE DRILLED: 10/3/2019
SAMPLE METHOD: Hollow Stem
ELEVATION: N/A
LOGGED BY: MW

B: Bulk Bag
S: Standard Penetration Test
R: Ring Sample

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/6"	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Description of Material
							5" Concrete thicness.
	B		2	SC		14.2	Clayey sand, fine grained, dark brown, moist, loose
2		R	3 8	SC	101.1	17.3	Clayey sand, fine grained, dark brown, moist, loose to medium dense Percent of Fines:48.5
5		R	3 10 14	SC	104.5	15.2	Clayey sand, fine grained, dark brown, moist, medium dense Percent of Fines: 41.2
10	B	S	8 12 17	SM		6.8	Silty sand, medium grained, medium brown, slightly moist, medium dense Percent of Fines: 12.9
15		R	11 22 29	SP	106.4	2.9	Gravelly sand, coarse grained, reddish brown, slightly moist, dense Percent of Fines: 4.2
20		S	9 17 20	SP-SM		3.9	Sand and silty sand, medium grained, grayish brown, slightly moist, dense Percent of Fines:9.2
25		R	11 19 24	SC	121.7	16.3	Clayey sand, fine grained, dark brown, very moist, dense Percent of Fines:35.4
30		R	7 11 19	CL	105.3	21.2	Sandy clay, medium brown, very moist, very stiff Percent of Fines: 72.0,LL= 32, PL= 22, PI= 10
35		S	11 18 22	SC		20.6	Clayey sand, fine grained, medium brown, very moist, dense Percent of Fines: 32.3

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BORING LOG B-1

PROJECT LOCATION: 3001 Walnut Grove Ave., Rosemead, CA
PROJECT NO.: 19-221-001

DATE DRILLED: 6/6/2018
SAMPLE METHOD: Hollow Stem
ELEVATION: N/A
LOGGED BY: MW

B: Bulk Bag
S: Standard Penetration Test
R: Ring Sample

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/6"	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Description of Material
40		S	9 13 18	SM		14.1	Silty sand, fine grained, yellowish brown, moist, dense Percent of Fines: 40.6
45		S	22 41 50/5"	SM		7.9	Silty sand, fine grained, grayish brown, slightly moist to moist, very dense Percent of Fines: 32.2
50		R	9 14 23	CL	101.8	23.5	Sandy clay, grayish brown, very moist, very stiff Percent of Fines: 75.1. LL = 33, PL = 22, PI = 11
55							Total Depth: 51.5 feet No Groundwater Hole Backfilled
60							Hammer Driving Weight: 140 lbs Hammer Driving Height: 30 inches
65							
70							

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BORING LOG B-2

PROJECT LOCATION: 3001 Walnut Grove Ave., Rosemead, CA
PROJECT NO.: 19-221-001

DATE DRILLED: 10/3/2019
SAMPLE METHOD: Hollow Stem
ELEVATION: N/A
LOGGED BY: MW

B: Bulk Bag
S: Standard Penetration Test
R: Ring Sample

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/6"	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Description of Material
							5" Concrete thickness
2		R	4 5 10	SC	103.2	15.1	Clayey sand, fine grained, dark brown, moist, loose to medium dense
5		S	4 9 9	SM		12.2	Silty sand, fine grained, medium brown, moist, medium dense
10		R	10 18 20	SM	108.2	7.7	Silty sand, medium grained, medium brown, slightly moist, medium dense
15		S	12 18 21	SP-SM		4.1	Sand and silty sand, medium grained, brown, slightly moist, dense
20		S	10 17 22	SM		7.3	Silty sand, medium grained, medium brown, slightly moist, dense
25							Total Depth: 21.5 feet No Groundwater Hole Backfilled Hammer Driving Weight: 140 lbs Hammer Driving Height: 30 inches
30							
35							

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.

Expansion Index

Laboratory Expansion Index test was conducted on the existing onsite near surface materials sampled during QCI's field investigation to aid in evaluation of soil expansion potential. The test is performed in accordance with ASTM D-4829. The testing result is presented below:

Sample Location	Expansion Index	Expansion Potential
B-1 @ 0-4'	10	Very Low
B-1 @ 10'-11'	2	Very Low

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'	8.70	120	0.0260	1,800

Percent Passing #200 Sieve

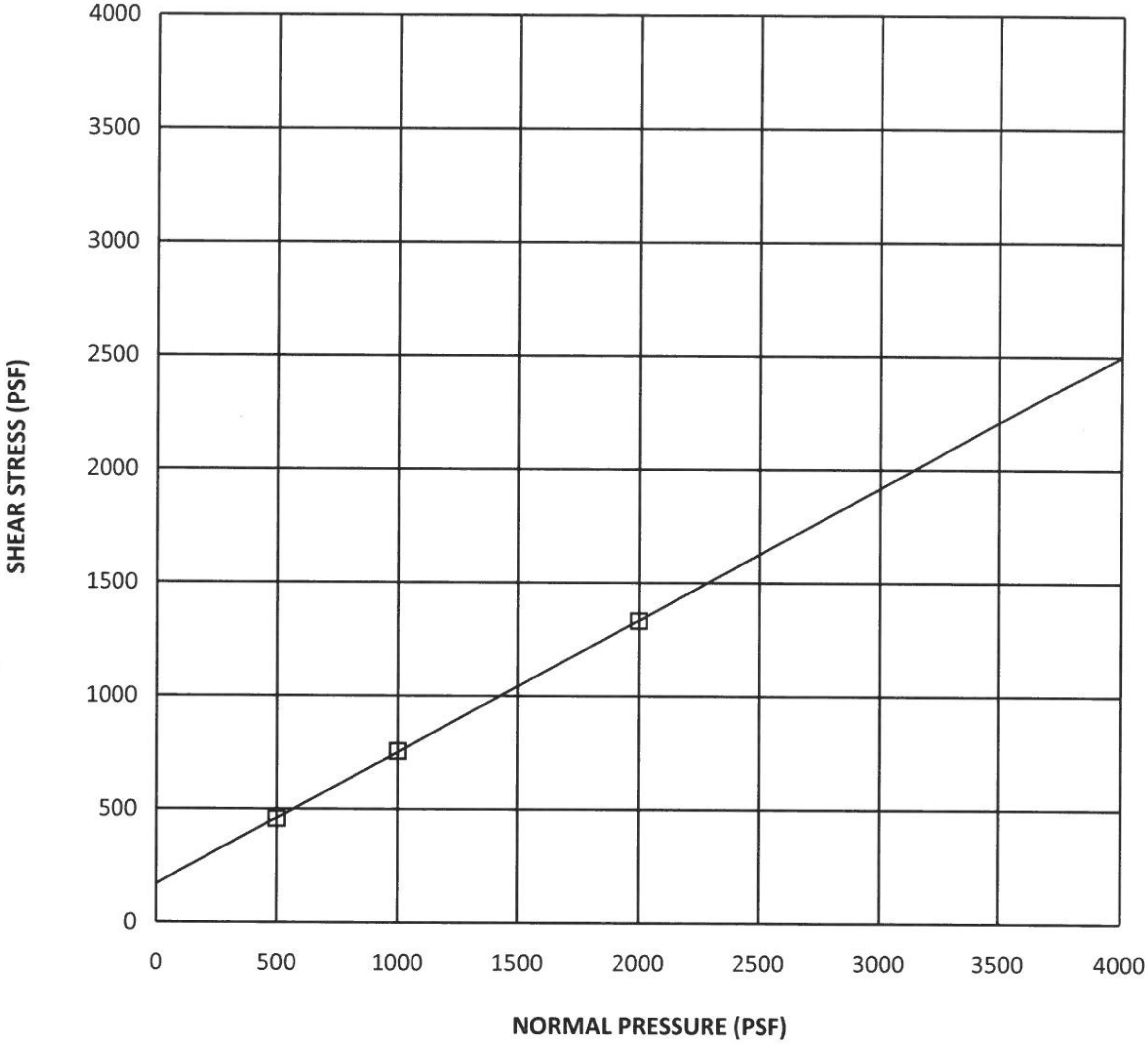
Percent of soil passing #200 sieve was determined for selected soil samples in accordance with ASTM D1140 standard. The test results are presented in the following table:

Sample Location	% Passing #200
B-1 @ 0-4'	48.5
B-1 @ 5'	41.2
B-1 @ 10'	12.9
B-1 @ 15'	4.2
B-1 @ 20'	9.2
B-1 @ 25'	35.4
B-1 @ 30'	72.0
B-1 @ 35'	32.3
B-1 @ 40'	40.6
B-1 @ 45'	32.2
B-1 @ 50'	75.1

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
B-1 @ 30'	CL	32	22	10
B-1 @ 50'	CL	33	22	11



SYMBOL	BORING NO.	SAMPLE NO.	DEPTH (FT)	SAMPLE TYPE	SOIL TYPE	COHESION (PSF)	FRICTION ANGLE (DEG)
□	B-1	N/A	2.0	RING	SC	170	30

Vertical Loads (PSF)	Moisture Content Before Test (%)	Moisture Content After Test (%)
500	17.3	24.4
1000	17.3	23.9
2000	17.3	23.7

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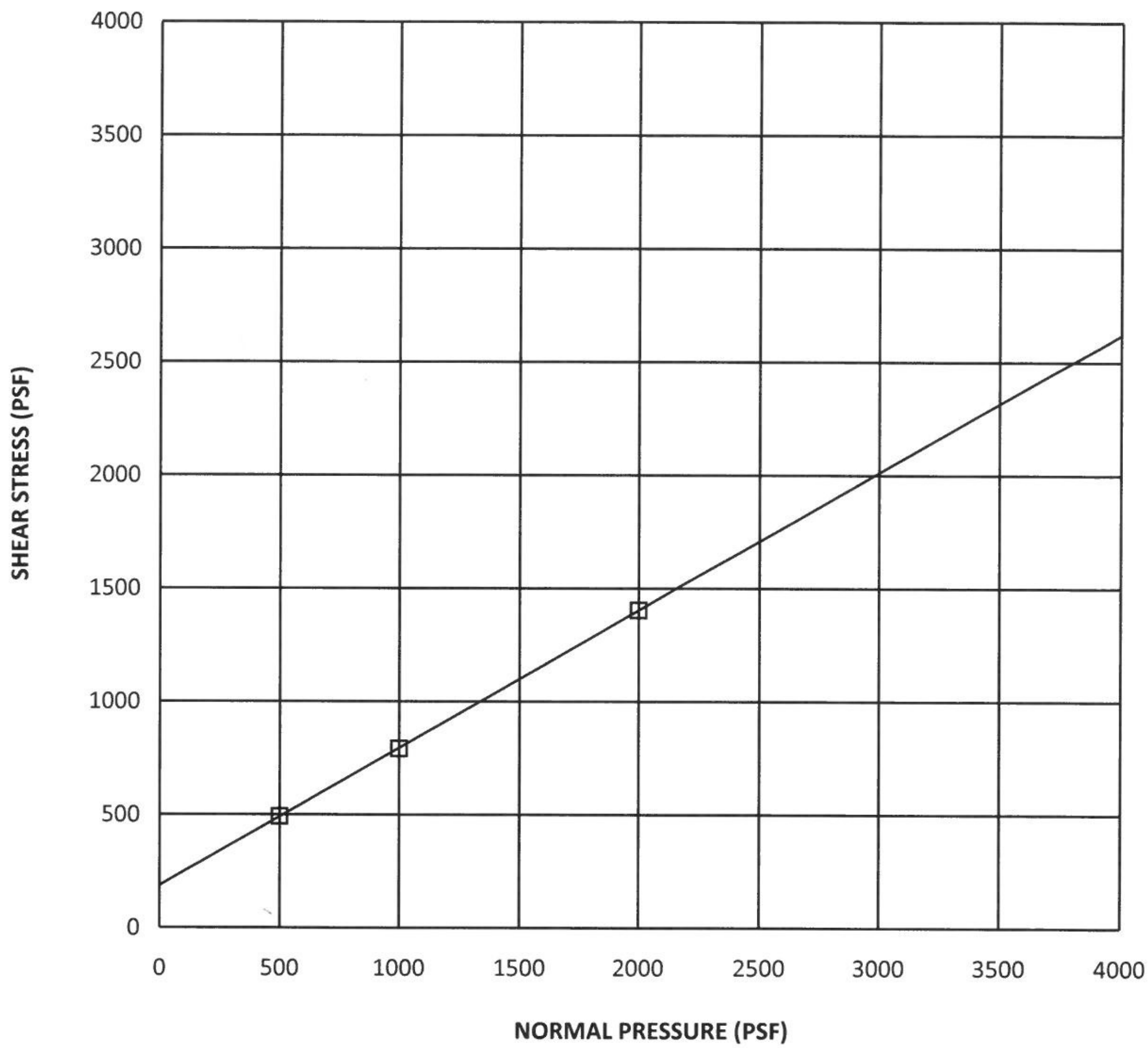
DIRECT SHEAR

(ASTM D3080)

11/19

FIGURE 3



SYMBOL	BORING NO.	SAMPLE NO.	DEPTH (FT)	SAMPLE TYPE	SOIL TYPE	COHESION (PSF)	FRICTION ANGLE (DEG)
□	B-1	N/A	5.0	RING	SC	190	31

Vertical Loads (PSF)	Moisture Content Before Test (%)	Moisture Content After Test (%)
500	15.2	22.2
1000	15.2	21.8
2000	15.2	21.3

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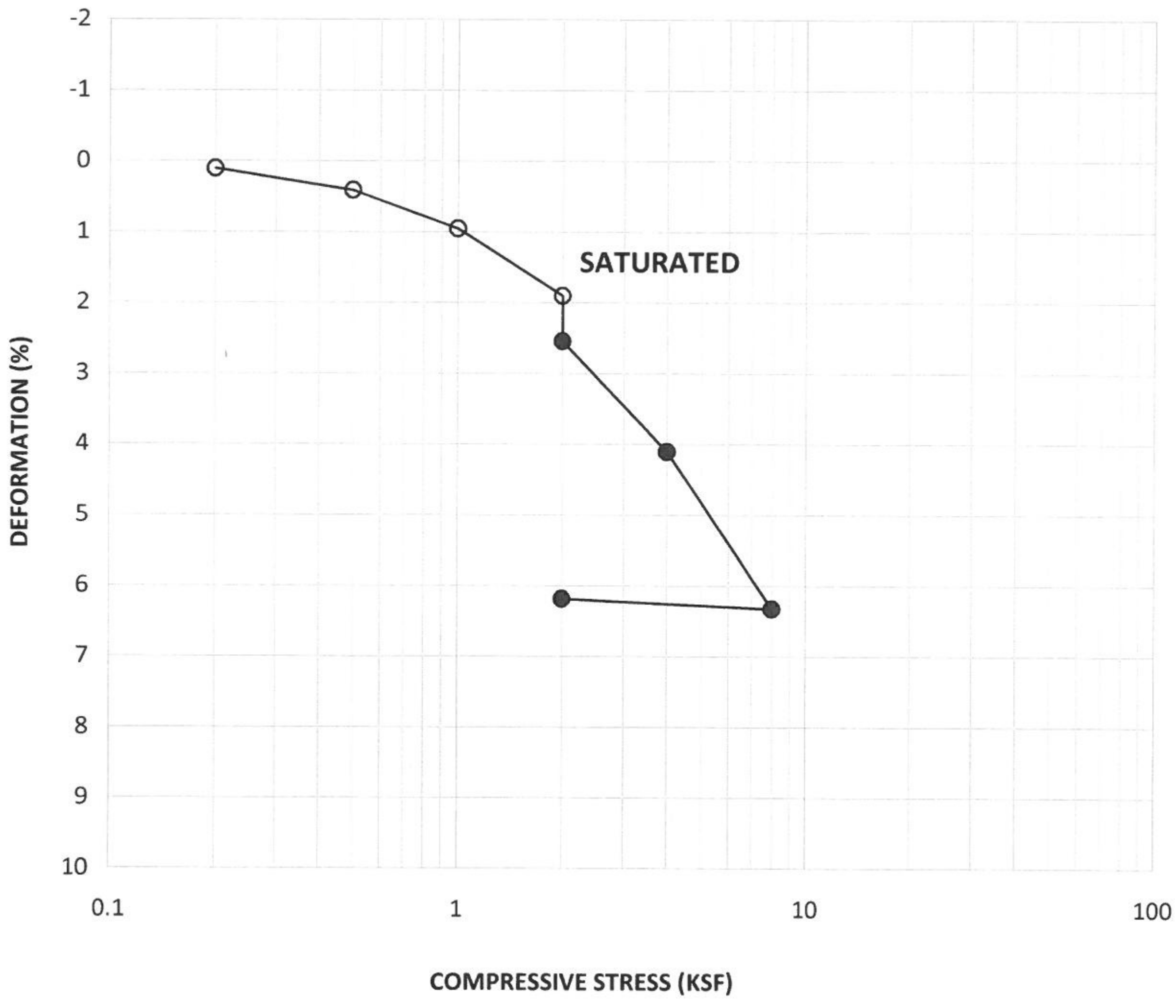
DIRECT SHEAR

(ASTM D3080)

11/19

FIGURE 4



SYMBOL	BORING NO.	SAMPLE NO.	DEPTH (FT)	SOIL TYPE	INIT. MOISTURE CONTENT (%)	INIT. DRY DENSITY (PCF)	INIT. VOID RATIO
○	B-1	N/A	5	SC	15.2	104.5	0.612

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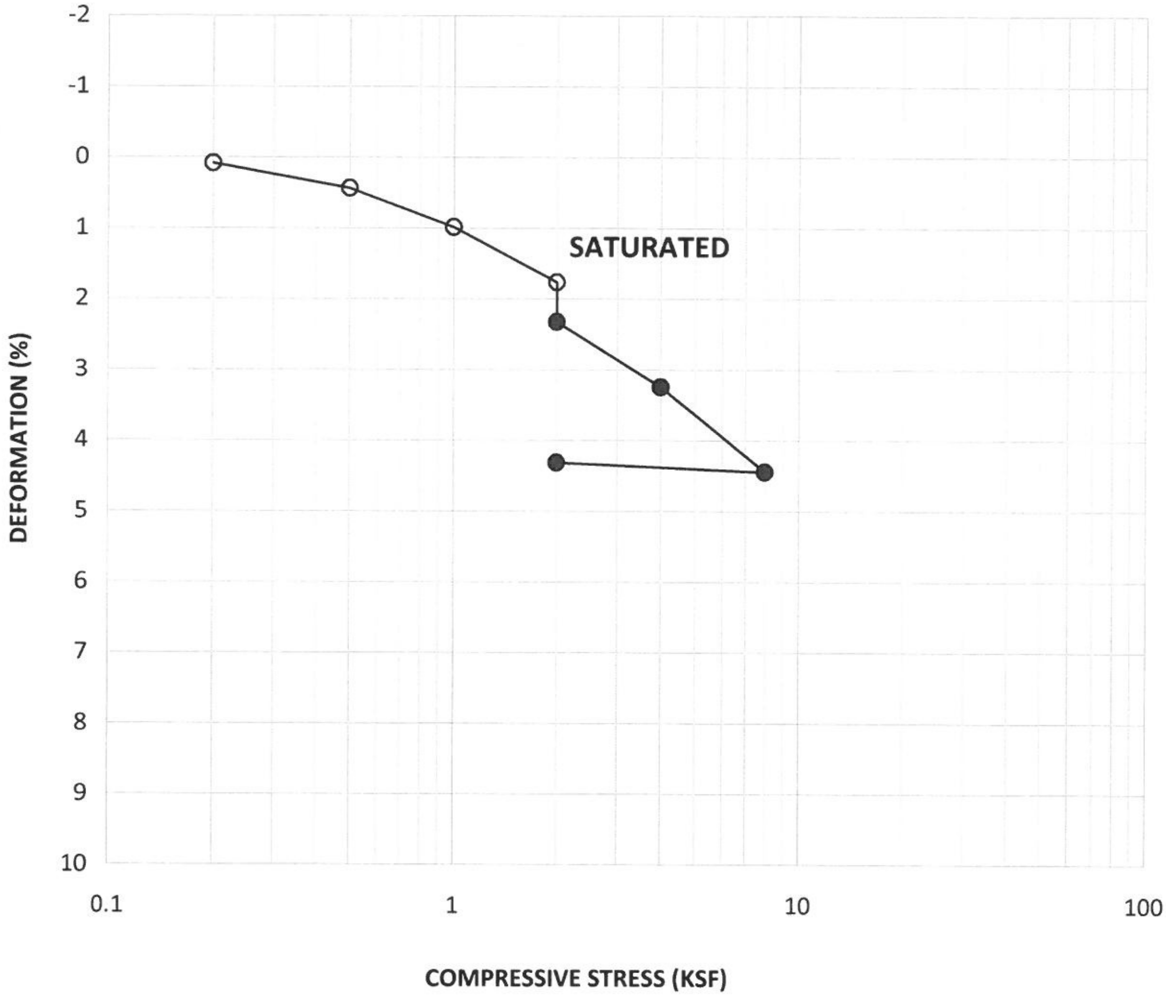
CONSOLIDATION

(ASTM D2435)

11/19

FIGURE 5



SYMBOL	BORING NO.	SAMPLE NO.	DEPTH (FT)	SOIL TYPE	INIT. MOISTURE CONTENT (%)	INIT. DRY DENSITY (PCF)	INIT. VOID RATIO
○	B-1	N/A	15	SP	2.9	106.4	0.583

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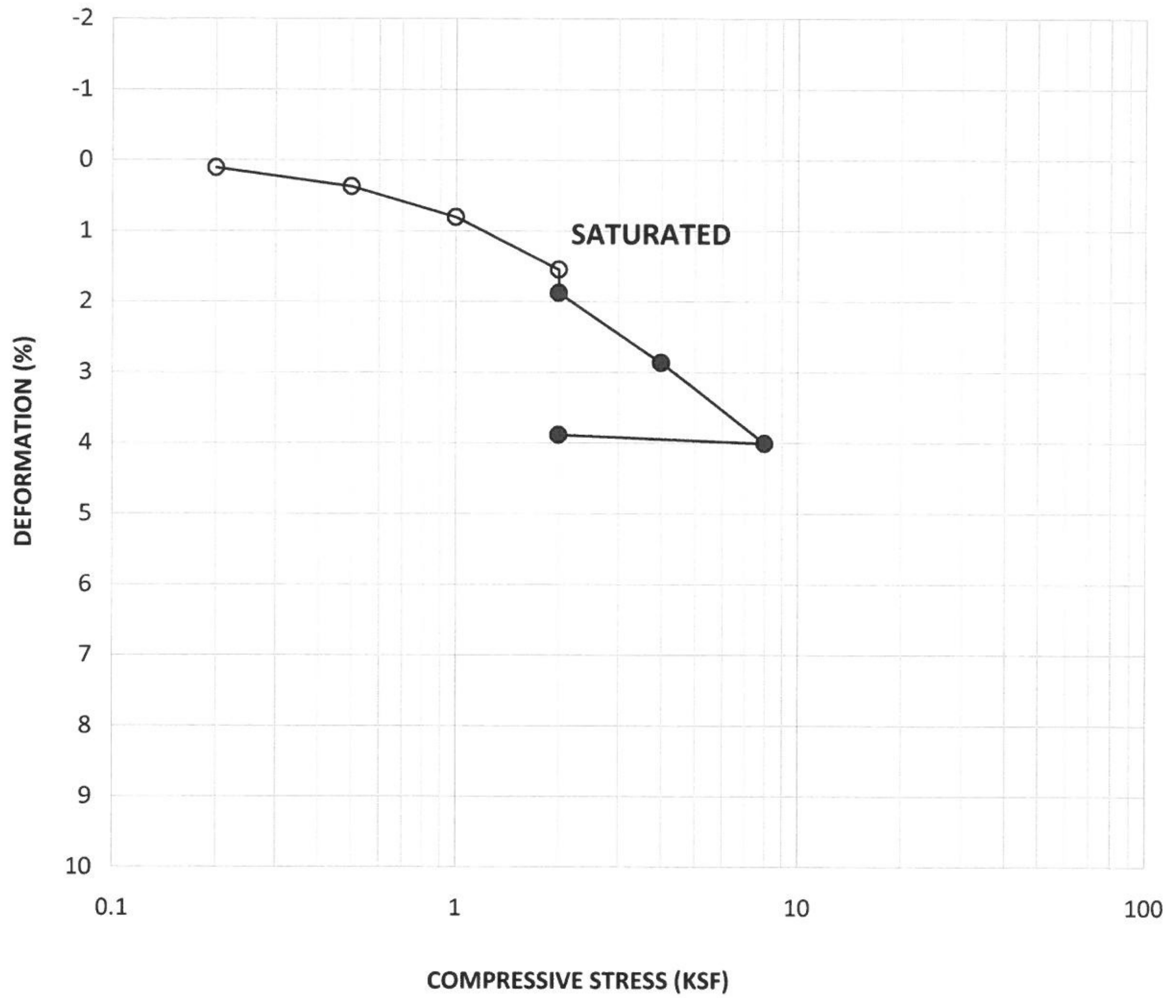
11/19

FIGURE 6



SYMBOL	BORING NO.	SAMPLE NO.	DEPTH (FT)	SOIL TYPE	INIT. MOISTURE CONTENT (%)	INIT. DRY DENSITY (PCF)	INIT. VOID RATIO
○	B-1	N/A	25	SC	16.3	121.7	0.384

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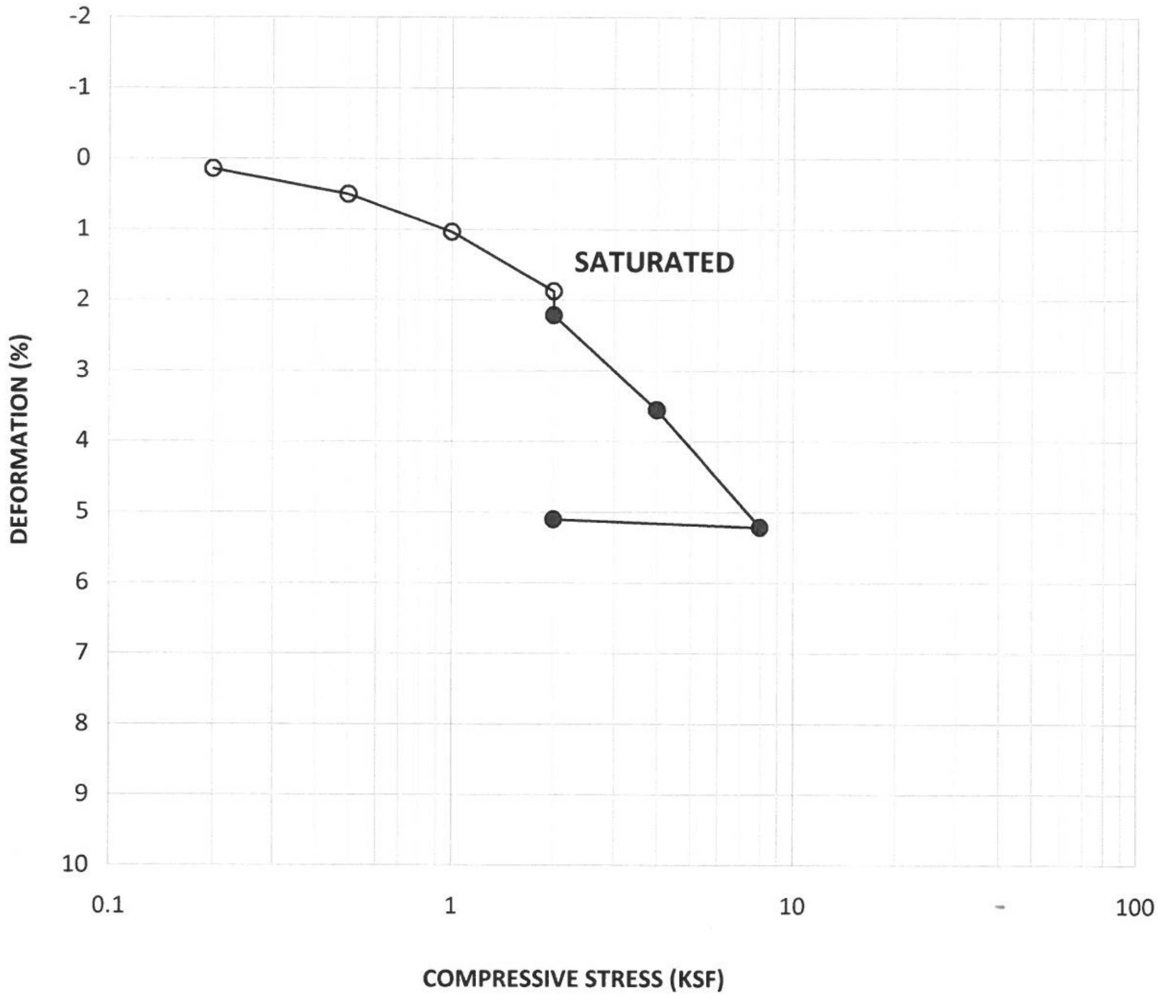
CONSOLIDATION

(ASTM D2435)

11/19

FIGURE 7



SYMBOL	BORING NO.	SAMPLE NO.	DEPTH (FT)	SOIL TYPE	INIT. MOISTURE CONTENT (%)	INIT. DRY DENSITY (PCF)	INIT. VOID RATIO
O	B-2	N/A	10	SM	7.7	108.2	0.557

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CONSOLIDATION

(ASTM D2435)

11/19

FIGURE 8

LIQUEFY2 - LIQUEFACTION (NCEER, 1997)

Begin Calculations | Exit

Job/Analysis Name (20 Characters Max): 3001 Walnut Gro

Job Number (15 Characters Max): 19-221-001

rd-Factor: Seed (1985) [selected] / NCEER (1997) / Idriss (1998) *

Rod Length Corr.?: Yes [selected] / No — Rod Stick Up (ft): 3.0

Magnitude Scaling Factor (MSF): Idriss (1997) [selected] / Idriss (1998) * / Andrus/Stokoe / Y / N 20% / Y / N 32% / Y / N 50% / R.Seed (1998) *

Sampler Corr. 1.000 Help

Borehole Dia. Corr. 1.000 Help

Hammer Energy Corr. 1.000 Help

Calc. Water Depth (ft): 10.0

EQ. Mag. (Mw): 7.0

Peak Acc. (g): 0.863

--------- Surcharge Fill Loading ---------

Fill Unit Wt. (pcf): 0.0 — Fill Ht. (ft): 0.0

Output File: WALNUT.OUT — WIN Notepad

Soil Profile Log Name (*.LDW): WALNUT.LDW

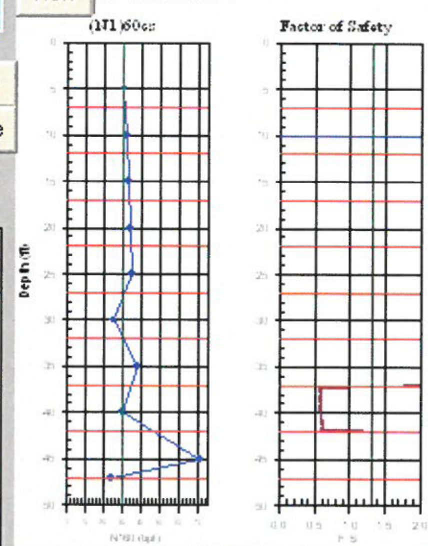
Update Page From File

Write Table to LDW File

Liquefaction Results File (*.LAR): WALNUT.LAR

Boring Water (ft): 50.0

Base	Field N	0/1	Unit Wt.	Fines	D50	SPT Depth
7	15	1	120	41	0.2	5
12	29	1	120	13	0.25	10
17	35	1	120	4	0.35	15
22	37	1	120	9	0.3	20
27	30	1	120	35	0.2	25
32	21	0	120	72	0.05	30
37	40	1	120	32	0.2	35
42	31	1	120	41	0.2	40
47	91	1	120	32	0.2	45
50	25	0	120	75	0.05	47



(4) Adobe Reader

start | 4 Adobe Reader | Report and Plates | Scan$ on 'Newserv... | Settlement

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*                           *
*      L I Q U E F Y 2      *
*                           *
*       Version 1.50        *
*                           *
*****************************
```

EMPIRICAL PREDICTION OF
EARTHQUAKE-INDUCED LIQUEFACTION POTENTIAL

JOB NUMBER: 19-221-001 DATE: 11-08-2019

JOB NAME: 3001 Walnut Gro

SOIL-PROFILE NAME: WALNUT.LDW

BORING GROUNDWATER DEPTH: 50.00 ft

CALCULATION GROUNDWATER DEPTH: 10.00 ft

DESIGN EARTHQUAKE MAGNITUDE: 7.00 Mw

SITE PEAK GROUND ACCELERATION: 0.863 g

BOREHOLE DIAMETER CORRECTION FACTOR: 1.00

SAMPLER SIZE CORRECTION FACTOR: 1.00

N60 HAMMER CORRECTION FACTOR: 1.00

MAGNITUDE SCALING FACTOR METHOD: Idriss (1997, in press)

Magnitude Scaling Factor: 1.193

rd-CORRECTION METHOD: Seed (1985)

FIELD SPT N-VALUES ARE CORRECTED FOR THE LENGTH OF THE DRIVE RODS.

Rod Stick-Up Above Ground: 3.0 ft

CN NORMALIZATION FACTOR: 1.044 tsf

MINIMUM CN VALUE: 0.6

File Name: WALNUT.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
1	0.25	0.015	0.015	15	9.11	*	*	*	*	*	**
1	0.75	0.045	0.045	15	9.11	*	*	*	*	*	**
1	1.25	0.075	0.075	15	9.11	*	*	*	*	*	**
1	1.75	0.105	0.105	15	9.11	*	*	*	*	*	**
1	2.25	0.135	0.135	15	9.11	*	*	*	*	*	**
1	2.75	0.165	0.165	15	9.11	*	*	*	*	*	**
1	3.25	0.195	0.195	15	9.11	*	*	*	*	*	**
1	3.75	0.225	0.225	15	9.11	*	*	*	*	*	**
1	4.25	0.255	0.255	15	9.11	*	*	*	*	*	**
1	4.75	0.285	0.285	15	9.11	*	*	*	*	*	**
1	5.25	0.315	0.315	15	9.11	*	*	*	*	*	**
1	5.75	0.345	0.345	15	9.11	*	*	*	*	*	**
1	6.25	0.375	0.375	15	9.11	*	*	*	*	*	**
1	6.75	0.405	0.405	15	9.11	*	*	*	*	*	**
2	7.25	0.435	0.435	29	2.95	*	*	*	*	*	**
2	7.75	0.465	0.465	29	2.95	*	*	*	*	*	**
2	8.25	0.495	0.495	29	2.95	*	*	*	*	*	**
2	8.75	0.525	0.525	29	2.95	*	*	*	*	*	**
2	9.25	0.555	0.555	29	2.95	*	*	*	*	*	**
2	9.75	0.585	0.585	29	2.95	*	*	*	*	*	**
2	10.25	0.615	0.607	29	2.95	1.319	31.6	Infin	0.979	0.556	NonLiq
2	10.75	0.645	0.622	29	2.95	1.319	31.6	Infin	0.978	0.569	NonLiq
2	11.25	0.675	0.636	29	2.95	1.319	31.6	Infin	0.977	0.581	NonLiq
2	11.75	0.705	0.650	29	2.95	1.319	31.6	Infin	0.976	0.593	NonLiq
3	12.25	0.735	0.665	35	0.04	1.077	32.6	Infin	0.974	0.604	NonLiq
3	12.75	0.765	0.679	35	0.04	1.077	32.6	Infin	0.973	0.615	NonLiq
3	13.25	0.795	0.694	35	0.04	1.077	32.6	Infin	0.972	0.625	NonLiq
3	13.75	0.825	0.708	35	0.04	1.077	32.6	Infin	0.971	0.635	NonLiq
3	14.25	0.855	0.722	35	0.04	1.077	32.6	Infin	0.970	0.644	NonLiq
3	14.75	0.885	0.737	35	0.04	1.077	32.6	Infin	0.969	0.653	NonLiq
3	15.25	0.915	0.751	35	0.04	1.077	32.6	Infin	0.968	0.661	NonLiq
3	15.75	0.945	0.766	35	0.04	1.077	32.6	Infin	0.967	0.670	NonLiq
3	16.25	0.975	0.780	35	0.04	1.077	32.6	Infin	0.966	0.677	NonLiq
3	16.75	1.005	0.794	35	0.04	1.077	32.6	Infin	0.965	0.685	NonLiq
4	17.25	1.035	0.809	37	1.10	0.933	33.3	Infin	0.964	0.692	NonLiq
4	17.75	1.065	0.823	37	1.10	0.933	33.3	Infin	0.963	0.699	NonLiq
4	18.25	1.095	0.838	37	1.10	0.933	33.3	Infin	0.961	0.705	NonLiq
4	18.75	1.125	0.852	37	1.10	0.933	33.3	Infin	0.960	0.711	NonLiq
4	19.25	1.155	0.866	37	1.10	0.933	33.3	Infin	0.959	0.717	NonLiq
4	19.75	1.185	0.881	37	1.10	0.933	33.3	Infin	0.958	0.723	NonLiq
4	20.25	1.215	0.895	37	1.10	0.933	33.3	Infin	0.956	0.728	NonLiq
4	20.75	1.245	0.910	37	1.10	0.933	33.3	Infin	0.955	0.733	NonLiq
4	21.25	1.275	0.924	37	1.10	0.933	33.3	Infin	0.954	0.738	NonLiq

File Name: WALNUT.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
4	21.75	1.305	0.938	37	1.10	0.933	33.3	Infin	0.952	0.743	NonLiq
5	22.25	1.335	0.953	30	9.83	0.834	34.4	Infin	0.951	0.747	NonLiq
5	22.75	1.365	0.967	30	9.83	0.834	34.4	Infin	0.949	0.752	NonLiq
5	23.25	1.395	0.982	30	9.83	0.834	34.4	Infin	0.948	0.756	NonLiq
5	23.75	1.425	0.996	30	9.83	0.834	34.4	Infin	0.946	0.759	NonLiq
5	24.25	1.455	1.010	30	9.83	0.834	34.4	Infin	0.945	0.763	NonLiq
5	24.75	1.485	1.025	30	9.83	0.834	34.4	Infin	0.943	0.766	NonLiq
5	25.25	1.515	1.039	30	9.83	0.834	34.4	Infin	0.941	0.770	NonLiq
5	25.75	1.545	1.054	30	9.83	0.834	34.4	Infin	0.939	0.773	NonLiq
5	26.25	1.575	1.068	30	9.83	0.834	34.4	Infin	0.938	0.776	NonLiq
5	26.75	1.605	1.082	30	9.83	0.834	34.4	Infin	0.936	0.778	NonLiq
6	27.25	1.635	1.097	21	~	~	~	~	~	~	~~
6	27.75	1.665	1.111	21	~	~	~	~	~	~	~~
6	28.25	1.695	1.126	21	~	~	~	~	~	~	~~
6	28.75	1.725	1.140	21	~	~	~	~	~	~	~~
6	29.25	1.755	1.154	21	~	~	~	~	~	~	~~
6	29.75	1.785	1.169	21	~	~	~	~	~	~	~~
6	30.25	1.815	1.183	21	~	~	~	~	~	~	~~
6	30.75	1.845	1.198	21	~	~	~	~	~	~	~~
6	31.25	1.875	1.212	21	~	~	~	~	~	~	~~
6	31.75	1.905	1.226	21	~	~	~	~	~	~	~~
7	32.25	1.935	1.241	40	9.65	0.705	37.9	Infin	0.909	0.795	NonLiq
7	32.75	1.965	1.255	40	9.65	0.705	37.9	Infin	0.906	0.795	NonLiq
7	33.25	1.995	1.270	40	9.65	0.705	37.9	Infin	0.903	0.796	NonLiq
7	33.75	2.025	1.284	40	9.65	0.705	37.9	Infin	0.899	0.796	NonLiq
7	34.25	2.055	1.298	40	9.65	0.705	37.9	Infin	0.896	0.796	NonLiq
7	34.75	2.085	1.313	40	9.65	0.705	37.9	Infin	0.893	0.795	NonLiq
7	35.25	2.115	1.327	40	9.65	0.705	37.9	Infin	0.889	0.795	NonLiq
7	35.75	2.145	1.342	40	9.65	0.705	37.9	Infin	0.886	0.794	NonLiq
7	36.25	2.175	1.356	40	9.65	0.705	37.9	Infin	0.882	0.794	NonLiq
7	36.75	2.205	1.370	40	9.65	0.705	37.9	Infin	0.878	0.793	NonLiq
8	37.25	2.235	1.385	31	9.01	0.660	29.5	0.390	0.874	0.792	0.59
8	37.75	2.265	1.399	31	9.01	0.660	29.5	0.390	0.871	0.790	0.59
8	38.25	2.295	1.414	31	9.01	0.660	29.5	0.390	0.866	0.789	0.59
8	38.75	2.325	1.428	31	9.01	0.660	29.5	0.390	0.862	0.788	0.59
8	39.25	2.355	1.442	31	9.01	0.660	29.5	0.390	0.858	0.786	0.59
8	39.75	2.385	1.457	31	9.01	0.660	29.5	0.390	0.854	0.784	0.59
8	40.25	2.415	1.471	31	9.01	0.660	29.5	0.390	0.849	0.782	0.60
8	40.75	2.445	1.486	31	9.01	0.660	29.5	0.390	0.845	0.780	0.60
8	41.25	2.475	1.500	31	9.01	0.660	29.5	0.390	0.840	0.778	0.60
8	41.75	2.505	1.514	31	9.01	0.660	29.5	0.390	0.836	0.775	0.60
9	42.25	2.535	1.529	91	14.51	0.622	71.1	Infin	0.831	0.773	NonLiq
9	42.75	2.565	1.543	91	14.51	0.622	71.1	Infin	0.826	0.770	NonLiq
9	43.25	2.595	1.558	91	14.51	0.622	71.1	Infin	0.822	0.768	NonLiq

File Name: WALNUT.OUT

SOIL NO.	CALC. DEPTH (ft)	TOTAL STRESS (tsf)	EFF. STRESS (tsf)	FIELD N (B/ft)	FC DELTA N1_60	C N	CORR. (N1)60 (B/ft)	LIQUE. RESIST RATIO	r d	INDUC. STRESS RATIO	LIQUE. SAFETY FACTOR
9	43.75	2.625	1.572	91	14.51	0.622	71.1	Infin	0.817	0.765	NonLiq
9	44.25	2.655	1.586	91	14.51	0.622	71.1	Infin	0.812	0.762	NonLiq
9	44.75	2.685	1.601	91	14.51	0.622	71.1	Infin	0.807	0.759	NonLiq
9	45.25	2.715	1.615	91	14.51	0.622	71.1	Infin	0.802	0.756	NonLiq
9	45.75	2.745	1.630	91	14.51	0.622	71.1	Infin	0.797	0.753	NonLiq
9	46.25	2.775	1.644	91	14.51	0.622	71.1	Infin	0.792	0.750	NonLiq
9	46.75	2.805	1.658	91	14.51	0.622	71.1	Infin	0.787	0.746	NonLiq
10	47.25	2.835	1.673	25	~	~	~	~	~	~	~~
10	47.75	2.865	1.687	25	~	~	~	~	~	~	~~
10	48.25	2.895	1.702	25	~	~	~	~	~	~	~~
10	48.75	2.925	1.716	25	~	~	~	~	~	~	~~
10	49.25	2.955	1.730	25	~	~	~	~	~	~	~~
10	49.75	2.985	1.745	25	~	~	~	~	~	~	~~

LIQUEFACTION INDUCED SETTLEMENT

References:

1. "Evaluation of Settlement in Sands Due to Earthquake Shaking", Seed, et. al., ASCE Journal of Geotechnical Engineering, Page 861 – 878, Vol. 113, No. 8, August 1987

2. "Manual for Evaluation and Mitigation of Liquefaction Hazard for Foundation Design", Workshop on Seismic Hazards Mapping Act, Los Angeles, January 1998

Recommended Fine Correction (Ref 2)

Percent of Fine	N_{corr}(Blow/ft)
10	1
25	2
50	4
75	5

Job # 19-221-001GE Address: 3001 Walnut Grove Ave, Rosemead, California
November 8, 2019

From Liquefaction Study: **B-1**

Soil Layer (4) with FS < 1.30

From Reference 1: Earthquake Magnitude Correction
$r_m = 0.65/(0.1(M-1)) = 0.65/(0.1(7.0-1)) = 1.08$

Layer	Depth (feet)	τ/δ	Percent of Fine	(N1)	$(N1)_{60,\,corr}$	Volumetric Strain ε	Settlement (inch)
1	10-12	0.569	12.9	29	31.6	0.55%	0.13
2	12-17	0.644	4.2	34	32.6	0.20%	0.12
3	17-22	0.733	9.2	36	33.3	0.10%	0.06
4	37-42	0.788	40.6	31	29.5	0.65%	0.39

Soil Layers (1, 2 & 3) FS> 1.30

Soil Layer (4) FS< 1.30

Total Settlement: 0.70 inches

Differential Settlement
67% x total settlement = 0.67 x 0.70= 0.47 Inches

Lateral Pressure Calculations

Soil Properties:

Depth 0 - 10 ft.   Unit Weight r = 120 pcf,. Cohesion C = 170 psf, Friction Angle $\phi = 30^o$

Surcharge at 10 ft. q=120x10=1200, Strength at 10ft. t=170+1200x tan(30)= 862.8 psf.

Equivalent Friction Angle ϕ'= Arc tan(862.8/1200)=35.7deg. Use ϕ=35 degrees

Lateral Pressure (Ref: Geotechnical Engineering Analysis and Evaluation", Roy Hunt, McGraw Hill Book Company, 1986)

For Cantilever Retaining Wall

Active Earth Pressure Pa = r x Ka $Ka = \tan^2(45 - \phi/2) = 0.271$

Pa = 120x 0.271 = 32.5 pcf use 35 pcf

For Restrained Retaining Wall

At Rest Earth Pressure Pa = r x Ko $Ko = 1 - \sin(\phi) = 0.426$

Pa = 120 x 0.426 = 51.1 pcf   use 60 pcf

Seismic Lateral Pressure

Ref.1: Foundation & Earth Structures, Naval Design Manual, DM 7.02, September 1986

Ref.2: Seismic Earth Pressures on Deep Building Basement, SEAOC 2010 Convention Proceedings

Ref.3: County of Los Angeles, Department of Public Works, Manual for Preparation of Geotechnical Reports

Ref 4: City of Los Angeles, "Seismic Lateral Earth Pressures on Basement and Retaining Walls", July 16, 2014

$P_E = 3/8 \times r \times H^2 \times k_h$ $PGA_M = 0.949g$

Maximum Ground Acceleration $k_h = 0.949 \times 0.5 \times 2/3 = 0.316g$

$P_E = 3/8 \times 120 \times H^2 \times 0.316 = 14.2 \times H^2$ use $14.2 \times H^2$ or P_E (EFP) = 29H

Passive Earth Pressure (Ref: Geotechnical Engineering Analysis and Evaluation", Roy Hunt, McGraw Hill Book Company, 1986)

Earth Pressure Pp = r x Kp $Kp = \tan^2(45 + \phi'/2) = 3.69$

Pp = 120 x 3.69 = 442.8 pcf > 350 pcf OK

Friction $\mu = 0.67 \times \tan(\phi) = 0.47 > 0.30$ OK

The retaining walls should be designed for the applicable factor of safety against lateral sliding and overturning in accordance with the current building code.

BEARING CAPACITY EVALUATION

Soil Properties:
Depth 0 – 10 feet, Unit Weight r = 120 pcf
Average Cohesion C = 170 psf
Average Friction Angle ϕ = 30°

Reference: "Foundation Analysis and Design", by Joseph E. Bowles, Second Edition, 1977
"Principles of Foundation Engineering", by Braja M. Das, PWS Publishers, 1984

Equation:
$Qult = C \times N_c + r \times D \times N_q + 0.5 \times r \times B \times N_r$

C : Cohesion of Soil
R : Unit Weight of Soil
B : Width of Foundation
D : Depth of Foundation
Φ : Friction Angle of Soil
N_c, N_q, N_r = Bearing Capacity Coefficient

Condition:

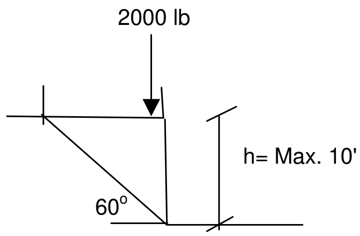
C : 110 psf
r : 120 pcf
Φ : 35
B : 12 inches
D : 18 inches
$N_c = 37.2$ $N_q = 22.5$ $N_r = 19.7$

Q = 170 x 37.2 + 120 x 1.5 x 22.5 + 0.5 x 120 x 1 x 19.7
= 6324 + 4050 + 1182 psf

SF = 3

Qall = Q/3 = (6324 + 4050 + 1182) / 3
= 2108 +1350 + 394 = 3852 > 3500 psf

SLOT CUT CALCULATIONS
Proposed Residential Development, 3001 Walnut Grove Avenue, Rosemead, California



Surcharge =	2000	lb	
α (Failure Surface inclination) =	60	deg	
γ m =	120.0	pcf	
ϕ =	30	deg	
C =	170	psf	
Ko =	1-SIN(ϕ)	0.50	
H (Height) =	10	ft	
d (Slot Width) =	8	ft	
b=	Height/TAN(α)	5.8	ft
A (Side Area) =	1/2(H)(b)	28.9	ft^2
Δ F = Side Shear =	A(1/2*γ_{m*H*} K_o*TAN(ϕ)+C) =	9907.5	lb
W (weight of soil + surcharge) =	A*γ_m + Surcharge =	5464.1	lb

F.S. = $\dfrac{d*[W*COS^2\alpha TAN(\phi) + Cb] + 2\,\Delta f}{d*(WSIN\alpha COS\alpha)}$ = **1.8**