GEOTECHNICAL INVESTIGATION REPORT
PROPOSED RESIDENTIAL BUILDINGS
10460 SLATER AVENUE

Fountain Valley, California

Prepared for:
MR. DAVID NGUYEN

Prepared by:
GEOBODEN INC.
Irvine, CA 92620

July 28, 2018

Project No. Slater-1-01

GEOBODEN INC.

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PROPOSED RESIDENTIAL BUILDINGS
10460 SLATER AVENUE
FOUNTAIN VALLEY, CALIFORNIA

MR. DAVID NGUYEN

Prepared by:

GEOBODEN INC.
5 Hodgenville
Irvine, California 92620

July 28, 2018

J.N. Slater-1-01

GEOBODEN, INC.
Geotechnical Consultants

July 28, 2018 Project No. Slater-1-01

Attention: Mr. David Nguyen

Subject: Geotechnical Investigation Report
Proposed Residential Buildings
10460 Slater Avenue
Fountain Valley, California

GeoBoden, Inc. is pleased to provide you with this report on our geotechnical investigation report for the proposed Residential Buildings on the subject site.

This report presents the results of our field investigation, laboratory testing and our engineering judgment, opinions, conclusions and recommendations pertaining to the proposed Residential Buildings.

Should you have any questions regarding the contents of this report, or should you require additional information, please contact us at (949) 872-9565.

Respectfully submitted,
GEOBODEN, INC.

Cyrus Radvar,
Principal Engineer, G.E. 2742
Expires: 06/30/2020

REGISTERED PROFESSIONAL ENGINEER
SHAHROKH E. RADVAR
2742
GEOTECHNICAL
STATE OF CALIFORNIA

Copies: 2/Addressee

GEOTECHNICAL INVESTIGATION REPORT

PROPOSED RESIDENTIAL BUILDINGS
10460 SLATER AVENUE
FOUNTAIN VALLEY, CALIFORNIA

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GEOTECHNICAL INVESTIGATION REPORT
PROPOSED RESIDENTIAL BUILDINGS
10460 SLATER AVENUE
FOUNTAIN VALLEY, California

1.0 INTRODUCTION

This report presents the results of our geotechnical study performed by GeoBoden, Inc. (GeoBoden) for the proposed Residential Buildings to be constructed on the subject site. The general location of the project is shown on Figure 1, “Site Vicinity Map”.

The purposes of this study were to determine the geotechnical properties of subsurface soil conditions, to evaluate their in-place characteristics, evaluate site seismicity, and to provide geotechnical recommendations with respect to design and construction of the proposed improvements.

The scope of the authorized investigation included performing a site reconnaissance, conducting field exploration and laboratory testing programs, performing engineering analyses, and preparing this Geotechnical Investigation Report. Evaluation of environmental issues or the potential presence of hazardous materials was not within the scope of services provided.

2.0 SITE LOCATION AND DESCRIPTION

The subject site is located at 10460 Slater Avenue in Fountain Valley, California, and is occupied by a church building, vacant land and surface parking. The site is enclosed by Slater Avenue on the north, by Ward Street on the east, by existing residential properties on the south and west.

The proposed residential buildings will include construction of 12 residential buildings. The new buildings will be two to three-story wood-frame construction with slabs on-grade.

3.0 GEOTECHNICAL INVESTIGATION

Our geotechnical investigation included a field exploration program and a laboratory testing programs. These programs were performed in accordance with our scope of services. The field exploration and laboratory testing programs are described below.

3.1 FIELD EXPLORATION PROGRAM

The field exploration program was performed and involved drilling of one hollow-stem auger boring to depth of 51.5 feet below existing ground surface. Soil materials encountered were visually classified and logged in accordance with the Unified Soil Classification System. The approximate location of the boring is shown on Figure 2.

Log of subsurface conditions encountered in the boring was prepared in the field by an engineer. Soil samples consisting of relatively undisturbed brass ring samples and Standard Penetration Tests (SPT) samples were collected at approximately 2 and 5-foot depth intervals and were returned to the laboratory for testing. One bulk sample was collected at depths of 1 to 5 feet below ground surface (bgs). The SPTs were performed at selected depth in accordance with ASTM D-1586. Final boring log was prepared from the field log and is presented in Appendix A.

3.2 LABORATORY TESTING

Selected samples collected during drilling activities were tested in the laboratory to assist in evaluating controlling engineering properties of subsurface materials at the site. Physical tests performed included moisture and density determination, direct shear, No. 200 Wash, Atterberg limits, and corrosion. The results of the laboratory testing are presented in Appendix B.

4.0 DISCUSSION OF FINDINGS

The following discussion of findings for the site is based on the results of the field exploration and laboratory testing programs.

4.1 SITE AND SUBSURFACE CONDITIONS

Generally, the near surface soil conditions encountered in our boring consisted of sand. Deeper soils were interlayers of silty sand, sandy clay and clay.

The native sandy soils were found medium dense. Clayey soils were found firm to stiff. For more detailed descriptions of the subsurface materials refer to the boring log in Appendix A.

4.2 GROUNDWATER CONDITIONS

Groundwater was encountered within our exploratory boring B-1 at an approximate depth of 20 feet bgs. Based on information from the California Geological Survey (California Division of Mines and Geology, 1997), the historic high ground water level in the site vicinity is at a depth of approximately 5 feet beneath the existing ground surface.

Fluctuations of the groundwater level, localized zones of perched water, and soil moisture content should be anticipated during and following the rainy season. Irrigation of landscaped areas on or adjacent to the site can also cause a fluctuation of soil moisture content and local groundwater levels.

4.3 SOIL ENGINEERING PROPERTIES

Physical tests were performed on the relatively undisturbed samples to characterize the engineering properties of the native soils. Moisture content and dry unit weight determinations were performed on the samples to evaluate the in-situ unit weights of the different materials. Moisture content and dry unit weight results are shown on the boring logs in Appendix A.

5.0 STRONG GROUND MOTION POTENTIAL

The project site is located in a seismically active area typical of Southern California and likely to be subjected to a strong ground shaking due to earthquakes on nearby faults.

5.1 CBC DESIGN PARAMETERS

To accommodate effects of ground shaking produced by regional seismic events, seismic design can, at the discretion of the designing Structural Engineer, be performed in accordance with the 2016 edition of the California Building Code (CBC). Table below, 2016 CBC Seismic Parameters, lists (next) seismic design parameters based on the 2016 CBC methodology, which is based on ASCE/SEI 7-10:

2016 CBC Seismic Design Parameters	Value
Site Latitude (decimal degrees)	33.7081
Site Longitude (decimal degrees)	-117.9464
Site Class Definition (ASCE 7 Table 20.3-1)	D
Mapped Spectral Response Acceleration at 0.2s Period, S_s (Figure 1613.3.1(1))	1.520
Mapped Spectral Response Acceleration at 1s Period, S_1 (Figure 1613.3.1(2))	0.565
Short Period Site Coefficient at 0.2s Period, F_a (Table 1613.3.3(1))	1.0
Long Period Site Coefficient at 1s Period, F_v (Table 1613.3.3(2))	1.5
Adjusted Spectral Response Acceleration at 0.2s Period, S_{MS} (Eq. 16-37)	1.520
Adjusted Spectral Response Acceleration at 1s Period, S_{M1} (Eq. 16-38)	0.847
Design Spectral Response Acceleration at 0.2s Period, S_{DS} (Eq. 16-39)	1.013
Design Spectral Response Acceleration at 1s Period, S_{D1} (Eq. 16-40)	0.565

6.0 LIQUEFACTION POTENTIAL

For liquefaction to occur, all of three key ingredients are required: liquefaction-susceptible soils, groundwater within a depth of 50 feet or less, and strong earthquake shaking. Soils susceptible to liquefaction are generally saturated loose to medium dense sands and non-plastic silt deposits below the water table.

We have used the risked based peak acceleration PGA_M = 0.583g in our liquefaction analysis for the subject site. The predominant modal earthquake magnitude 6.89 used in our liquefaction analysis is based on the results of PSHA Deaggregation on NEHRP "D" soil. The SPT consists of driving a standard sampler, as described in the ASTM 1586 Standard Method, using a 140 pound hammer falling 30 inches. An Automatic Trip Hammer was used to drive samplers 18 inches into the soil. For an automatic hammer, the energy ratio value of 1.27 was used in our analysis. SPT hammer was raised 30 inches utilizing an Automatic Trip Hammer. A correction factor of 1.0 for borehole correction was used in our revised liquefaction evaluation.

The screening criteria of Bray and Sancio (2006) were used to determine if fine-grained soils within boring B-1 is susceptible to liquefaction. To determine if soils are susceptible to liquefaction, the Plasticity Index (PI) and in-situ moisture content were determined. For screening analysis purposes, all soil samples above and below the groundwater table were soaked and saturated, and then tested for moisture content. Bray and Sancio (2006) found loose soils with a PI < 12 and moisture content> 85% of the liquid limit are susceptible to liquefaction. For PI greater than 12 and moisture content less than 85 percent of liquid limit, clayey soils are not susceptible to liquefaction. Based on the results of Atterberg limits testing, clayey soils are not susceptible to liquefaction.

Computer program LiquefyPro developed by CivilTech Software was used for evaluation of liquefaction at the site. The program is based on the most recent publications of NCEER Workshop and SP117a Implementation. The results of our liquefaction analyses are attached.

In order to estimate the amount of post-earthquake settlement, methods proposed by Tokimatsu and Seed (1987) were used for the settlement calculations of silty sand layer. Based on our analysis and under the current site conditions, the maximum total liquefaction-induced ground settlements at the site is 1.22 inches. Differential settlement of 0.81 or less is anticipated over a span of 40 feet. It is our opinion that potential for liquefaction at the site will not adversely impact the foundation performance of the residential buildings provided recommendations in this report are incorporated in the design.

7.0 DESIGN RECOMMENDATIONS

Based upon the results of our investigation, the proposed residential buildings are considered geotechnically feasible provided the recommendations presented herein are incorporated into the design and construction. If changes in the design of the structure are made or variations or changed conditions are encountered during construction, GeoBoden should be contacted to evaluate their effects on these recommendations. The following geotechnical engineering recommendations for the proposed Residential Buildings are based on observations from the field investigation program and the physical test results.

7.1 EARTHWORK

All earthworks, including excavation, backfill and preparation of subgrade, should be performed in accordance with the geotechnical recommendations presented in this report and applicable portions of the grading code of local regulatory agencies. All earthwork should be performed under the observation and testing of a qualified geotechnical engineer.

7.2 SITE AND FOUNDATION PREPARATION

All weeds, grasses, brush, shrubs, trees and similar vegetation existing within areas to be graded should be stripped and removed from the site. Trees and large shrubs, when removed, should be grubbed out so as to include their stumps and major root systems, and these organic materials removed from the site. During site grading, laborers should clear from onsite soils any roots, tree branches and other deleterious materials missed during initial clearing and grubbing operations.

The construction area should be cleared of any vegetation and stripped of miscellaneous debris and other deleterious material. Organic matter and all other material that may interfere with the completion of the work should be removed from the limits of the construction area. Vegetation, construction debris, and organic matter should not be incorporated into engineered fill.

All active or inactive utilities within the construction limits should be identified for relocation, abandonment, or protection prior to grading. Any subsurface piping encountered should be abandoned in-place by being filled with sand/cement slurry. The adequacy of existing backfill around utilities to remain in place under new structures should be evaluated; loose or dumped trench backfill should be removed and replaced with properly compacted backfill.

In general, all loose soils within the proposed buildings footprints should be overexcavated and replaced with engineered fill. As a minimum, removals should extend to competent native soils. At least 3 feet of compacted fill should be provided underneath all spread footings and floor slabs. The compacted fill should extend laterally a minimum of 5 feet beyond the foundation footprints, where possible. All existing low-density, near-surface soils will require removal to competent material from areas to receive newly compacted fill. The basis for establishing a competent exposed surface on which to place fill should consist of competent materials exhibiting an in-place relative compaction of at least 85 percent. Prior to placing structural fill, exposed bottom surfaces in each removal area approved for fill should first be scarified to a depth of at least 6 inches, water or air dried as necessary to achieve near optimum moisture conditions, and then recompacted in place to a minimum relative compaction of 90 percent.

Based on the observations made in our borings and the results of pertinent laboratory tests, anticipated depths of removal of unsuitable soils will be about 5 feet. However, actual removal depths will have to be determined during grading on the basis of in-grading observations and testing performed by a representative of geotechnical consultants.

7.3 FILL PLACEMENT AND COMPACTION REQUIREMENTS

Material for engineered fill should be select free of organic material, debris, and other deleterious substances, and should not contain fragments greater than 3 inches in maximum dimension. On-site excavated soils that meet these requirements may be used to backfill the excavated Residential Buildings area.

All fill should be placed in 6-inch-thick maximum lifts, watered or air dried as necessary to achieve a few percent above optimum conditions, and then compacted in place to a maximum relative compaction of 90 percent. The laboratory maximum dry density and optimum moisture content for each change in soil type should be determined in accordance with Test Method ASTM D 1557. A representative of the project consultant should be present on-site during grading operations to verify proper placement and compaction of all fill, as well as to verify compliance with the other geotechnical recommendations presented herein.

Imported soils, if any, should consist of clean materials exhibiting a VERY LOW expansion potential (Expansion Index less than 20). Soils to be imported should be approved by the project geotechnical consultant prior to importation.

7.4 GEOTECHNICAL OBSERVATIONS

Exposed bottom surfaces in each removal area should be observed and approved by the project geotechnical consultant prior to placing fill. No fill should be placed without prior approval from the geotechnical consultant.

The project geotechnical consultant should be present on site during grading operations to verify proper placement and compaction of fill, as well as to verify compliance with the recommendations presented herein.

7.5 POST-GRADING CONSIDERATIONS

Positive drainage devices such as concrete flatwork, graded swales, and area drains should be provided around the new construction to collect and direct all water to a suitable discharge area. Neither rain nor excess irrigation water should be allowed to collect or pond against building foundations.

7.6 UTILITY TRENCH BACKFIL

All utility trench backfill should be compacted to a minimum relative compaction of 90 percent. Trench backfill materials should be placed in lifts no greater than approximately 6 inches in thickness, watered or air-dried as necessary to achieve a few percent above optimum conditions, and then mechanically compacted in place to a minimum relative compaction of 90 percent. A representative of the project geotechnical consultant should probe and test the backfills to verify adequate compaction.

As an alternative for shallow trenches where pipe or utility lines may be damaged by mechanical compaction equipment, such as under building floor slabs, imported clean sand exhibiting a sand equivalent (SE) value of 30 or greater may be utilized. The sand backfill materials should be watered to achieve a few percent above optimum conditions and then tamped into place. No specific relative compaction will be required; however, observation, probing, and if deemed necessary, testing should be performed by a representative of the project geotechnical consultant to verify an adequate degree of compaction and that the backfill will not be subject to settlement.

Where utility trenches enter the footprint of the buildings, they should be backfilled through their entire depths with on-site fill materials, sand-cement slurry, or concrete rather than with any sand or gravel shading. This "Plug" of less- or non-permeable materials will mitigate the potential for water to migrate through the backfilled trenches from outside of the buildings to the areas beneath the foundations and floor slabs.

7.7 SHALLOW FOUNDATIONS

Following the site and foundation preparation recommended above, foundation for load bearing walls and interior columns if any may be designed as discussed below.

7.7.1 Bearing Capacity and Settlement

Load bearing walls and interior columns may be supported on continuous spread footings and isolated spread footings, respectively, and should bear entirely upon properly engineered fill or competent native soils. Continuous and isolated footings should have a minimum width of 14 inches and 18 inches, respectively. All footings should be embedded a minimum depth of 24 inches measured from the lowest adjacent finish grade. Continuous and isolated footings placed on such materials may be designed using an allowable (net) bearing capacity of 2,000 pounds per square foot (psf). Allowable increases of 200 psf for each additional 1 foot in width and 200 psf for each additional 6 inches in depth may be utilized, if desired. The maximum allowable bearing pressure should be 3,000 psf. The maximum bearing value applies to combined dead and sustained live loads. The allowable bearing pressure may be increased by one-third when considering transient live loads, including seismic and wind forces.

Based on the allowable bearing value recommended above, total settlement of the shallow footings are anticipated to be less than one inch, provided foundation preparations conform to the recommendations described in this report. Differential settlement is anticipated to be approximately half the total settlement for similarly loaded footings spaced up to approximately 30 feet apart.

7.7.2 Lateral Load Resistance

Lateral load resistance for the spread footings will be developed by passive soil pressure against sides of footings below grade and by friction acting at the base of the concrete footings bearing on compacted fill. An allowable passive pressure of 250 psf per foot of depth may be used for design purposes. An allowable coefficient of friction 0.35 may be used for dead and sustained live load forces to compute the frictional resistance of the footings constructed directly on compacted fill. Safety factors of 2.0 and 1.5 have been incorporated in development of allowable passive and frictional resistance values, respectively. Under seismic and wind loading conditions, the passive pressure and frictional resistance may be increased by one-third.

7.7.3 Footing Reinforcement

Reinforcement for footings should be designed by the structural engineer based on the anticipated loading conditions. Footings for lightly loaded wood-frame structures that are supported in very low expansive soils should have No. 4 bars, two top and two bottom.

7.8 CONCRETE SLAB ON-GRADE

Concrete slabs will be placed on undisturbed natural soils or properly compacted fill as outlined in Section 7.2. Moisture content of subgrade soils should be maintained near the optimum moisture content.

At the time of the concrete pour, subgrade soils should be firm and relatively unyielding. Any disturbed soils should be excavated and then replaced and compacted to a minimum of 90 percent relative compaction. Slabs should be designed to accommodate low expansive fill soils. The structural engineer should determine the minimum slab thickness and reinforcing depending upon the expansive soil condition intended use. Slabs should be at least 4 inches thick and have minimum reinforcement of No. 3 bars placed at mid-height of the slabs and spaced 18 inches on centers, in both directions. The structural engineer may require thicker slabs with more reinforcement depending on the anticipated slab loading conditions.

Per CALGreen Code, a 4-inch thick base of ½ -inch or larger clean aggregate shall be provided with a vapor retarder in direct contact with concrete and a concrete mix design which will address bleeding, shrinkage and curling shall be used. Alternatively, concrete floor slabs should be underlain with a moisture vapor retarder consisting of a polyvinyl chloride membrane such as 10-mil Visqueen, or equivalent. All laps within the membrane should be sealed, and at least 2 inches of clean sand should be placed over the membrane to promote uniform curing of the concrete. To reduce the potential for punctures, the membrane should be placed on a pad surface that has been graded smooth without any sharp protrusions. If a smooth surface can not be achieved by grading, consideration should be given to placing a 1-inch thick leveling coarse of sand across the pad surface prior to placement of the membrane.

7.9 SOLUBLE SULFATES AND SOIL CORROSIVITY

Minimum resistivity test on one near surface bulk sample from the site indicated that on-site soils are mildly corrosive when in contact with ferrous materials. The preliminary chemical test results are included in Appendix B. Typical recommendations for mitigation of the corrosive potential of the soil in contact with building materials are the following:

- Below grade ferrous metals should be given a high quality protective coating, such as an 18 mil plastic tape, extruded polyethylene, coal tar enamel, or Portland cement mortar.

- Below grade ferrous metals should be electrically insulated (isolated) from above grade ferrous metals and other dissimilar metals, by means of dielectric fittings in utilities and exposed metal structures breaking grade.

- Steel and wire reinforcement within concrete in contact with the site soils should have at least two inches of concrete cover.

It is also recommended that additional sampling and analysis be conducted during the final stages of site grading to provide a complete assessment of soil corrosivity. GeoBoden does not practice corrosion engineering. Therefore, we recommend that on-site soils be tested and analyzed near or at the completion of precise grading by a qualified corrosion engineer to evaluate the general corrosion potential of the on-site soils and any impact on the proposed construction.

Corrosion test results also indicate that the surficial soils at the site have negligible sulfate attack potential on concrete. No special sulfate-resistant cement will be necessary for concrete placed in contact with the on-site soils.

8.0 CONSTRUCTION CONSIDERATIONS

Based on our field exploration program, earthwork can be performed with conventional construction equipment.

8.1 TEMPORARY DEWATERING

Groundwater was encountered within our exploratory boring at 20 feet. Based on the anticipated excavation depths, it is unlikely that dewatering will be required during construction.

8.2 CONSTRUCTION SLOPES

The temporary excavation side walls may be cut vertically to a maximum height of 3 feet. Surcharge loads should be kept away from the top of temporary excavations a horizontal distance equal to at least one-half the depth of excavation. Surface drainage should be controlled along the top of temporary excavations to preclude wetting of the soils and erosion of the excavation faces.

8.3 POST INVESTIGATION SERVICES

Final project plans and specifications should be reviewed prior to construction to confirm that the full intent of the recommendations presented herein have been applied to design and construction. Following review of plans and specifications, observation should be performed by the geotechnical engineer during construction to document that foundation elements are founded on/or penetrate onto the recommended soils, and that suitable backfill soils are placed upon competent materials and properly compacted at the recommended moisture content.

9.0 CLOSURE

The conclusions, recommendations, and opinions presented herein are: (1) based upon our evaluation and interpretation of the limited data obtained from our field and laboratory programs; (2) based upon an interpolation of soil conditions between and beyond the borings; (3) are subject to confirmation of the actual conditions encountered during construction; and, (4) are based upon the assumption that sufficient observation and testing will be provided during construction.

If parties other than GeoBoden are engaged to provide construction geotechnical services, they must be notified that they will be required to assume complete responsibility for the geotechnical phase of the project by concurring with the findings and recommendations in this report or providing alternate recommendations.

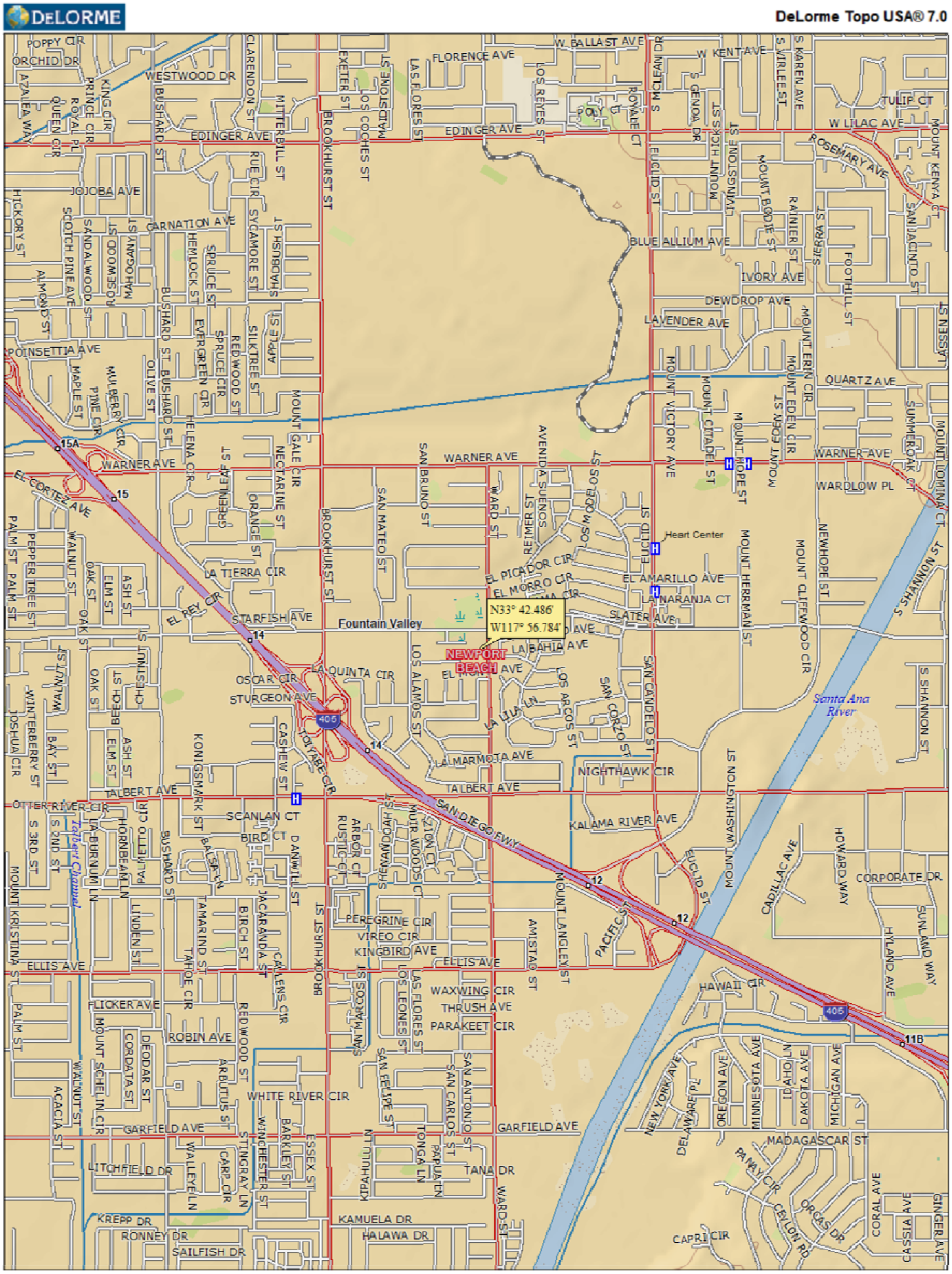
If pertinent changes are made in the project plans or conditions are encountered during construction that appear to be different than indicated by this report, please contact this office. Significant variations may necessitate a re-evaluation of the recommendations presented in this report.

10.0 REFERENCES

California Building Code, 2016 Volume 2.

Department of Conservation, Division of Mines and Geology. 1997. "Seismic Hazard Zone Report for the Anaheim and Newport Beach 7.5-Minute Quadrangles, Orange County, California, Seismic Hazard Zone Report 03.

FIGURES



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MN (11.8° E)

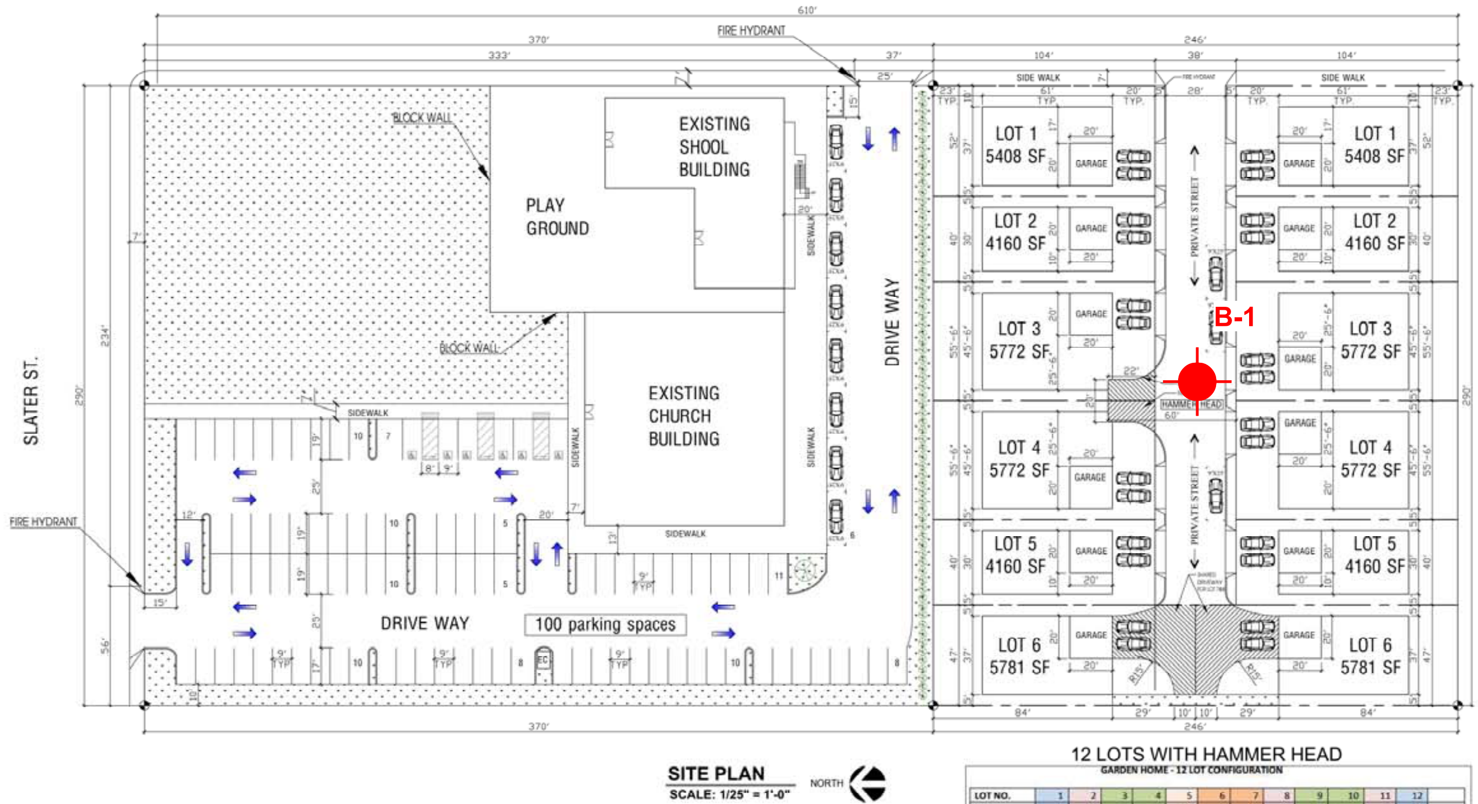
0 800 1600 2400 3200 4000 ft

Data Zoom 13-0

GEOBODEN INC. Geotechnical Consultants	**SITE VICINITY MAP** **Proposed Residential Buildings** **10460 Slater Avenue** **Fountain Valley, California**	Figure By S.R.	Project No. Slatter-1-01
		Map No. XX	Figure No. 1
		Date 07-29-18	

COASTAL COMMUNITY FELLOWSHIP

CONTACT: Kene Panas. kene@fvccf.com. 714.756.1311



LOT SIZE: 107,300 S.F (2.46 acres)

TOTAL NUMBER OF GUEST PARKINGS: 100

CURRENT DRIVEWAY FROM WARD: 30 FT, REDUCED TO 25 FT TO MAKE ROOM FOR LANDSCAPE SETBACK

LANDSCAPE SIDE SETBACK TO RESIDENTIAL BLOCKWALL: REQUIRED 10 FT. PROPOSED 10 FT

NEW DRIVEWAY FROM SLATER: 25 FT.

12 LOTS WITH HAMMER HEAD

GARDEN HOME - 12 LOT CONFIGURATION

LOT NO.	1	2	3	4	5	6	7	8	9	10	11	12	
Plan	B	D	A	A	D	C	C	D	A	A	D	B	
Lot Width	52	40	55.5	55.5	40	47	47	40	55.5	55.5	40	52	290
Lot Length	104	104	104	104	104	123	123	104	104	104	104	104	
Lot Size	5408	4160	5772	5772	4160	5781	5781	4160	5772	5772	4160	5408	
House Width	37	30	45.5	45.5	30	37	37	30	45.5	45.5	30	37	
FAR	0.52	0.52	0.52	0.52	0.52	0.52	0.52	0.52	0.52	0.52	0.52	0.52	
Max Living SF	2812	2163	3001	3001	2163	3006	3006	2163	3001	3001	2163	2812	
Gara Porch	420	420	420	420	420	420	420	420	420	420	420	420	5040
Cali Room	130	0	130	130	0	130	130	0	130	130	0	130	1040
1st Floor	1330	965	1500	1500	965	1400	1400	965	1500	1500	965	1330	15320
2nd Floor	1482	1198	1501	1501	1198	1606	1606	1198	1501	1501	1198	1482	16975
Total Living SF	2812	2163	3001	3001	2163	3006	3006	2163	3001	3001	2163	2812	32295
	Total Site Coverage: Garage + Porch + Cali Room + 1st floor Living:												21400
	Max Site Coverage Allowed 30% of (246 width X 290 Length):												21402
	PLAN A			PLAN B			PLAN C			PLAN D			

SITE PLAN

COASTAL COMMUNITY CHURCH
10460 SLATER AVE.
FOUNTAIN VALLEY, CA 92708

REV. DESCRIPTION DATE

SCALE	AS NOTED
DRAWN BY	
CHECKED BY	
PLAN DATE	
PRINT DATE	
PROJECT No.	
SHEET No.	

A - 1

LEGEND

B-1

NUMBER AND APPROXIMATE LOCATION OF BORING

GEOBODEN INC.
Geotechnical Consultants

BORING LOCATION PLAN
Proposed Residential Buildings
10460 Slater Avenue
Fountain Valley, California

Figure By S.R.	Project No. Slater
Map No. XX	Figure No. 2
Date 07-28-18	

APPENDIX A
BORING LOGS

APPENDIX A
SUBSURFACE EXPLORATION PROGRAM

PROPOSED RESIDENTIAL BUILDINGS
10460 SLATER AVENUE
FOUNTAIN VALLEY, CALIFORNIA

Prior to drilling, the proposed boring was located in the field by measuring from existing site features.

A total of one exploratory boring (B-1) was drilled using a hollow-stem auger drill rig equipped with 8-inch outside diameter (O.D.) augers. GeoBoden of Irvine, California performed the drilling. The boring location is shown on Figure 2.

Depth-discrete soil samples were collected at selected intervals from the exploratory boring using a 2 ½ -inch inside diameter (I.D.) modified California Split-barrel sampler fitted with 12 brass ring of 2 ½ inches in O.D. and 1-inch in height and one brass liner (2 ½ -inch O.D. by 6 inches long) above the brass rings. The sampler was lowered to the bottom of the borehole and driven 18 inches into the soil with a 140-pound hammer falling 30 inches. The number of blows required to drive the sampler the lower 12 inches is shown on the blow count column of the boring logs.

After removing the sampler from the borehole, the sampler was opened and the brass rings and liner containing the soil were removed and observed for soil classification. Brass rings containing the soil were sealed in plastic canisters to preserve the natural moisture content of the soil. Soil samples collected from exploratory boring were labeled, and submitted to the laboratory for physical testing.

Standard Penetration Tests (SPTs) were also performed at alternative depths in Boring. The SPT consists of driving a standard sampler, as described in the ASTM 1586 Standard Method, using a 140-pound hammer falling 30 inches. The number of blows required to drive the SPT sampler the lower 12 inches of the sampling interval is recorded on the blow count column of the boring logs.

The soil classifications and descriptions on field logs were performed using the Unified Soil Classification System as described by the American Society for Testing and Materials (ASTM) D 2488-90, “Standard Practice for Description and Identification of Soils (Visual-Manual Procedure).” The final boring logs were prepared from the field logs and are presented in this Appendix.

At the completion of the sampling and logging, the exploratory boring was backfilled with the drilled cuttings.

GEOBODEN, INC.

BORING NUMBER B-1

CLIENT Mr. David Nguyen
PROJECT NAME Proposed Residential Development
PROJECT NUMBER Slater-1-01
PROJECT LOCATION 10460 Slater Avenue, Fountain Valley, CA
DATE STARTED 7/13/18 **COMPLETED** 7/13/18
GROUND ELEVATION **HOLE SIZE** 8 inches
DRILLING CONTRACTOR Geoboden, Inc.
GROUND WATER LEVELS:
DRILLING METHOD HSA
∇ **AT TIME OF DRILLING** 20.00 ft
LOGGED BY C.R. **CHECKED BY**
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
0		SAND (SP): light olive, moist, fine sand										
5			MC R-1		12		105	6				6
		CLAY (CL): dark grayish brown, moist, ~10% fine sand										
10			SS S-2		4			20	46	19	27	69
		SILTY SAND (SM): light olive brown, moist										
15			SS S-3		6			8				22
		FAT CLAY (CH): dark gray, moist										
20		∇	SS S-4		6			33	53	15	38	97
		CLAY (CL): light olive, moist										
25			SS S-5		4			29	44	22	22	91
30		light greenish gray	SS S-6		9							
		SANDY CLAY (CL): greenish gray, moist										
35												

(Continued Next Page)

GEOBODEN, INC.

BORING NUMBER B-1

CLIENT Mr. David Nguyen

PROJECT NAME Proposed Residential Development

PROJECT NUMBER Slater-1-01

PROJECT LOCATION 10460 Slater Avenue, Fountain Valley, CA

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	ATTERBERG LIMITS: LIQUID LIMIT	ATTERBERG LIMITS: PLASTIC LIMIT	ATTERBERG LIMITS: PLASTICITY INDEX	FINES CONTENT (%)
35		SANDY CLAY (CL): greenish gray, moist *(continued)*	SS S-7		10			31	47	18	29	69
		CLAY (CL): light greenish gray, moist										
40			SS S-8		7			32				93
45		olive gray	SS S-9		4							
50			SS S-10		12							

Bottom of borehole at 51.5 feet below ground surface. Ground water was encountered at 20 feet below the existing grade. Boring was backfilled with cuttings.

Bottom of borehole at 51.5 feet.

APPENDIX B
LABORATORY TESTING

APPENDIX B
LABORATORY TESTING

PROPOSED RESIDENTIAL BUILDINGS
10460 SLATER AVENUE
MIDWAY, CALIFORNIA

Laboratory tests were performed on selected samples to assess the engineering properties and physical characteristics of soils at the site. The following tests were performed:

- moisture content and dry density
- direct shear
- No. 200 Wash
- Atterberg limits
- corrosion potential

Test results are summarized on laboratory data sheets or presented in tabular form in this appendix.

Moisture Density Tests

The field moisture contents, as a percentage of the dry weight of the soils, were determined by weighing samples before and after oven drying. The dry density, in pounds per cubic foot, was also determined fir all relatively undisturbed ring samples collected. These analyses were performed in accordance with ASTM D 2937. The results of these determinations are shown on the boring log in Appendix A.

Direct Shear

Direct shear tests were performed on undisturbed samples of on-site soils. A different normal stress was applied vertically to each soil sample ring which was then sheared in a horizontal direction. The resulting shear strength for the corresponding normal stress was measured at a maximum constant rate of strain of 0.005 inches per minute. The direct shear results are shown graphically on laboratory data sheets included in this appendix.

No. 200 Wash Sieve

A quantitative determination of the percentage of soil finer than 0.075 mm was performed on selected soil sample by washing the soil through the No. 200 sieve. Test procedures were performed in accordance with ASTM Method D1140. The results of the test is shown on the boring log.

Atterberg Limits

Liquid limit, plastic limit, and plasticity index were determined for selected soil samples in accordance with ASTM D 4318. The soil sample was air-dried and passed through a No. 40 sieve and moisturized. The liquid and plastic limit tests were performed on the fraction passing the No. 40 sieve. Results of the Atterberg limits tests are shown in this Appendix.

Corrosion Potential

Corrosion was tested on the selected soil sample in the near surface to determine the corrosivity of the site soil to steel and concrete. The soil samples were tested for soluble sulfate (Caltrans 417), soluble chloride (Caltrans 422), and pH and minimum resistivity (Caltrans 643). The results of corrosion tests are summarized in Table B-1.

TABLE B-1 (Corrosion Test Results)

Boring No.	Depth (ft)	Chloride Content (Calif. 422) ppm	Sulfate Content (Calif. 417) % by Weight	pH (Calif. 643)	Resistivity (Calif. 643) Ohm*cm
B-1	0-5	47	0.0123	7.1	1,725

GEOBODEN, INC.

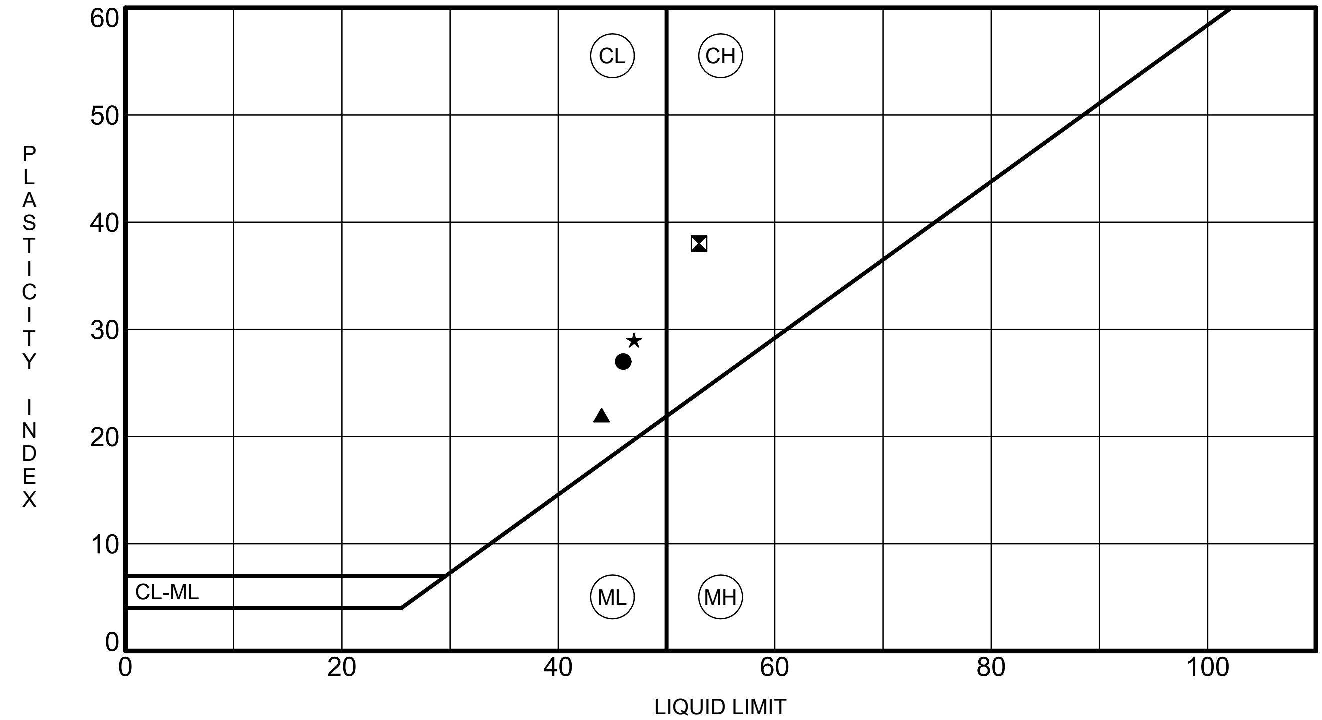
ATTERBERG LIMITS' RESULTS

CLIENT Mr. David Nguyen

PROJECT NAME Proposed Residential Development

PROJECT NUMBER Slater-1-01

PROJECT LOCATION 10460 Slater Avenue, Fountain Valley, CA



	Specimen Identification		LL	PL	PI	Fines	Classification
●	**B-1**	**10.0**	**46**	**19**	**27**	**69**	**SANDY LEAN CLAY(CL)**
⊠	**B-1**	**20.0**	**53**	**15**	**38**	**97**	**FAT CLAY(CH)**
▲	**B-1**	**25.0**	**44**	**22**	**22**	**91**	**LEAN CLAY(CL)**
★	**B-1**	**35.0**	**47**	**18**	**29**	**69**	**SANDY LEAN CLAY(CL)**

GEOBODEN, INC.

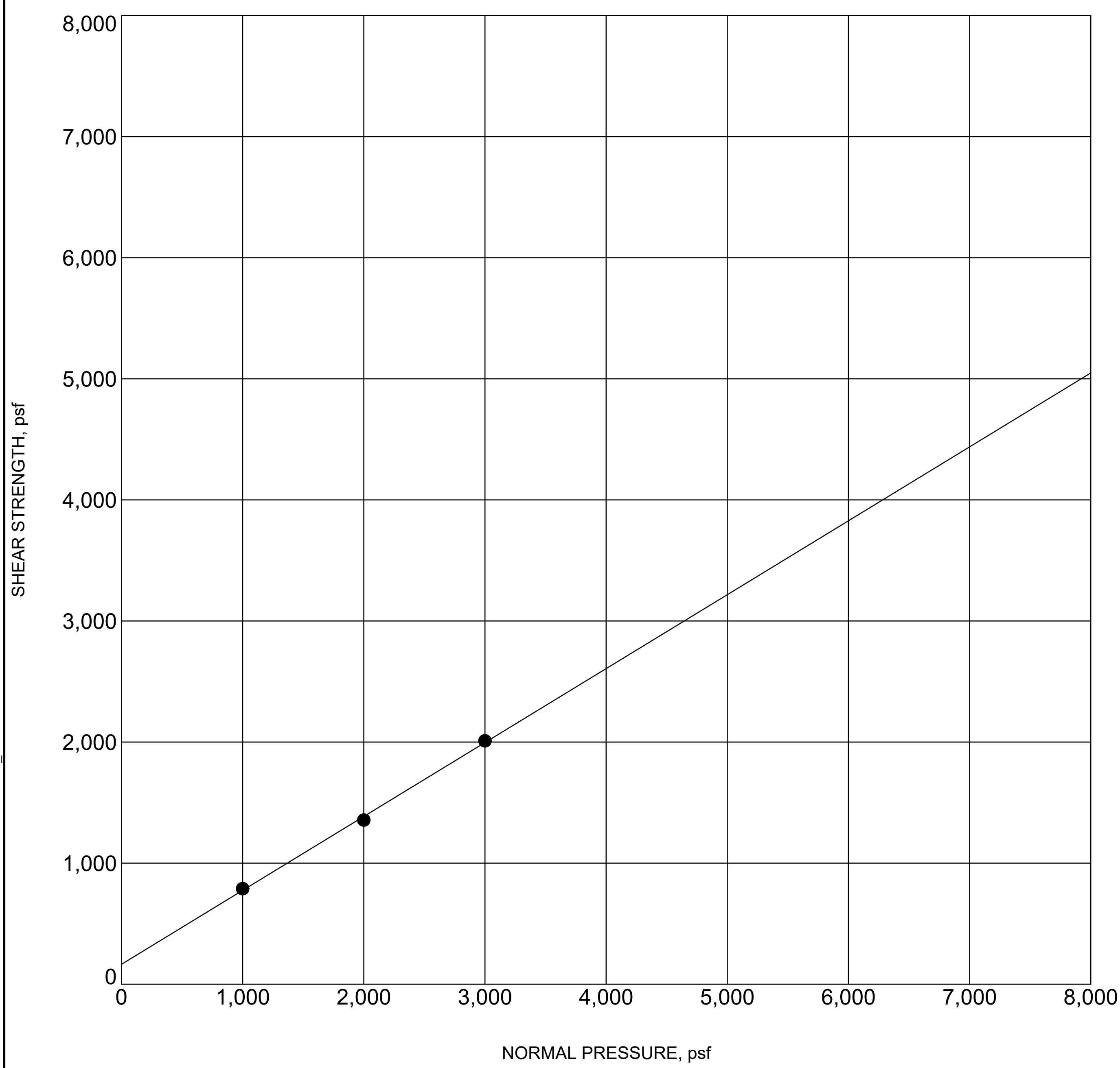
DIRECT SHEAR TEST

CLIENT Mr. David Nguyen

PROJECT NAME Proposed Residential Development

PROJECT NUMBER Slater-1-01

PROJECT LOCATION 10460 Slater Avenue, Fountain Valley, CA



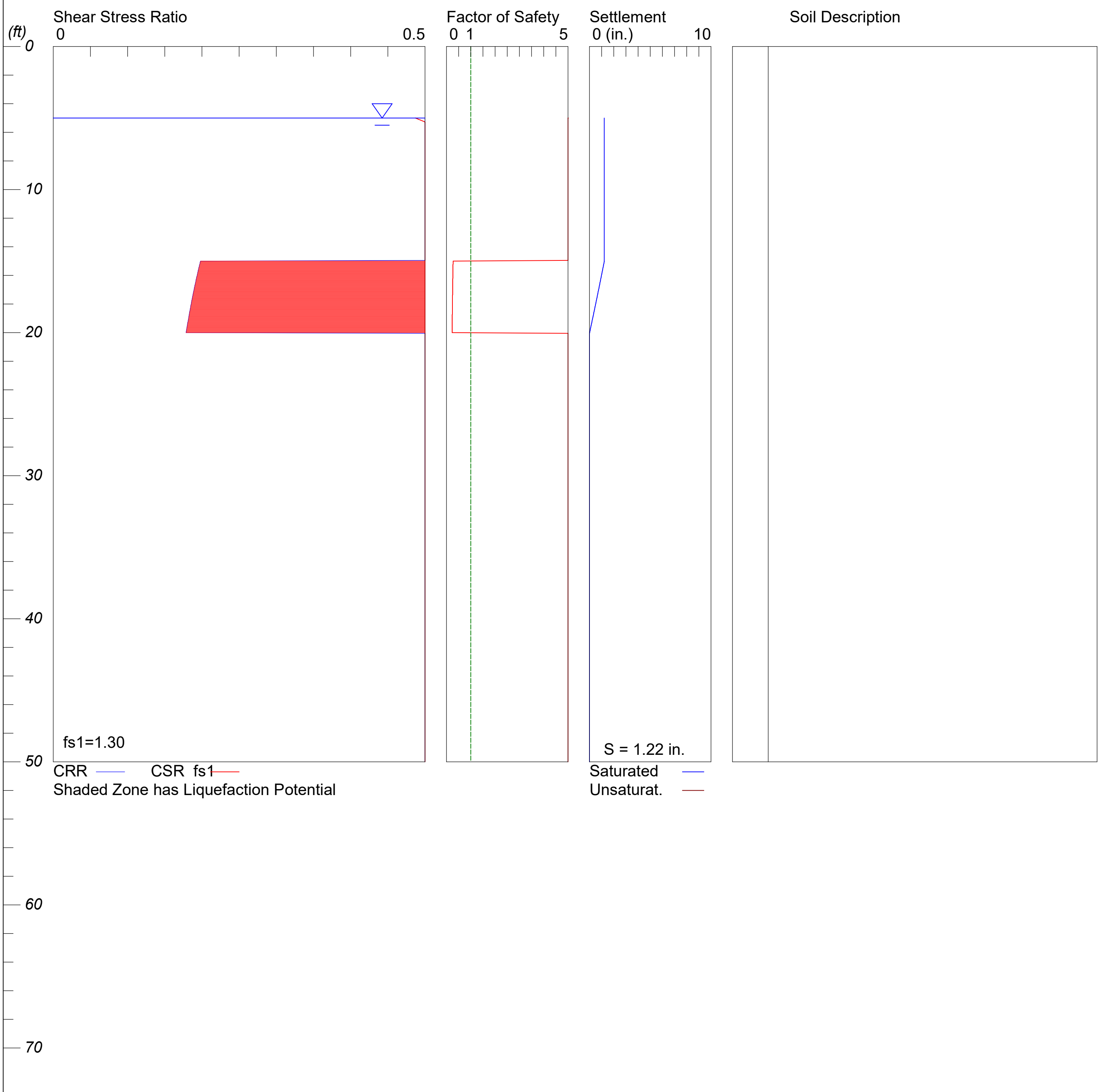
	Specimen Identification		Classification	γ_d	MC%	c	ϕ
●	**B-1**	**5.0**	**SAND (SP): light olive, moist**	**105**	**6**	**164.0**	**31**

APPENDIX C
LIQUEFACTION ANALYSIS

LIQUEFACTION ANALYSIS
Proposed Residential Development

Hole No.=B-1 Water Depth=5 ft

Magnitude=6.89
Acceleration=0.583g



10460 Slater Avenue, Fountain Valley **Plate C-1**

****

LIQUEFACTION ANALYSIS CALCULATION DETAILS

****

Font: Courier New, Regular, Size 8 is recommended for this report.
Licensed to , 7/27/2018 9:17:31 AM

Input File Name: C:\Passport\GBI\10460 Slatter Avenue_Fountain Valley-David Nguyen\B-1.liq
Title: Proposed Residential Development
Subtitle: 10460 Slater Avenue, Fountain Valley

Input Data:
Surface Elev.=
Hole No.=B-1
Depth of Hole=50.00 ft
Water Table during Earthquake= 5.00 ft
Water Table during In-Situ Testing= 20.00 ft
Max. Acceleration=0.58 g
Earthquake Magnitude=6.89
No-Liquefiable Soils: CL, OL are Non-Liq. Soil
1. SPT or BPT Calculation.
2. Settlement Analysis Method: Tokimatsu/Seed
3. Fines Correction for Liquefaction: Idriss/Seed
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce = 1.27
7. Borehole Diameter, Cb= 1.05
8. Sampling Method, Cs= 1.2
9. User request factor of safety (apply to CSR) , User= 1.3
Plot one CSR curve (fs1=User)
10. Average two input data between two Depths: No
* Recommended Options

In-Situ Test Data:

Depth ft	SPT	Gamma pcf	Fines %
5.00	8.00	120.00	NoLiq
10.00	4.00	120.00	NoLiq
15.00	6.00	120.00	22.00
20.00	6.00	120.00	NoLiq
25.00	4.00	120.00	NoLiq
30.00	9.00	120.00	NoLiq
35.00	10.00	120.00	NoLiq
40.00	7.00	120.00	NoLiq
45.00	4.00	120.00	NoLiq
50.00	12.00	120.00	NoLiq

Output Results:
Calculation segment, dz=0.050 ft
User defined Print Interval, dp=5.00 ft

Peak Ground Acceleration (PGA), a_max = 0.58g

CSR Calculation:

Depth ft	gamma pcf	sigma atm	gamma' pcf	sigma' atm	rd	mZ g	a(z) g	CSR	x fs1	=CSRfs
5.00	57.60	0.284	57.60	0.284	0.99	0.000	0.583	0.37	1.30	0.49
10.00	120.00	0.567	57.60	0.420	0.98	0.000	0.583	0.50	1.30	0.65
15.00	120.00	0.851	57.60	0.556	0.97	0.000	0.583	0.56	1.30	0.73
20.00	120.00	1.134	57.60	0.692	0.95	0.000	0.583	0.59	1.30	0.77
25.00	120.00	1.418	57.60	0.828	0.94	0.000	0.583	0.61	1.30	0.79
30.00	120.00	1.701	57.60	0.964	0.93	0.000	0.583	0.62	1.30	0.81

35.00	120.00	1.985	57.60	1.100	0.89	0.000	0.583	0.61	1.30	0.79
40.00	120.00	2.268	57.60	1.236	0.85	0.000	0.583	0.59	1.30	0.77
45.00	120.00	2.552	57.60	1.372	0.81	0.000	0.583	0.57	1.30	0.74
50.00	120.00	2.835	57.60	1.508	0.77	0.000	0.583	0.55	1.30	0.71

CSR is based on water table at 5.00 during earthquake

CRR Calculation from SPT or BPT data:

Depth ft	SPT	Cebs	Cr	sigma' atm	Cn	(N1)60	Fines %	d(N1)60	(N1)60f	CRR7.5
5.00	8.00	1.60	0.75	0.284	1.70	16.32	NoLiq	8.26	24.59	0.28
10.00	4.00	1.60	0.85	0.567	1.33	7.23	NoLiq	6.45	13.67	0.15
15.00	6.00	1.60	0.95	0.851	1.08	9.89	22.00	4.85	14.74	0.16
20.00	6.00	1.60	0.95	1.134	0.94	8.56	22.00	4.72	13.29	0.14
25.00	6.00	1.60	0.95	1.272	0.89	8.09	NoLiq	6.62	14.71	0.16
30.00	4.00	1.60	1.00	1.408	0.84	5.39	NoLiq	6.08	11.47	0.12
35.00	9.00	1.60	1.00	1.544	0.80	11.59	NoLiq	7.32	18.91	0.20
40.00	10.00	1.60	1.00	1.680	0.77	12.35	NoLiq	7.47	19.82	0.21
45.00	7.00	1.60	1.00	1.816	0.74	8.31	NoLiq	6.66	14.97	0.16
50.00	4.00	1.60	1.00	1.952	0.72	4.58	NoLiq	5.92	10.50	0.11

CRR is based on water table at 20.00 during In-Situ Testing

Factor of Safety, - Earthquake Magnitude= 6.89:

Depth ft	sigC' atm	CRR7.5	x Ksig	=CRRv	x MSF	=CRRm	CSRfs	F.S.=CRRm/CSRfs
5.00	0.18	0.28	1.00	0.28	1.24	2.00	0.49	5.00 ^
10.00	0.37	0.15	1.00	0.15	1.24	2.00	0.65	5.00 ^
15.00	0.55	0.16	1.00	0.16	1.24	0.20	0.73	0.27 *
20.00	0.74	0.14	1.00	0.14	1.24	0.18	0.77	0.23 *
25.00	0.83	0.16	1.00	0.16	1.24	2.00	0.79	5.00 ^
30.00	0.92	0.12	1.00	0.12	1.24	2.00	0.81	5.00 ^
35.00	1.00	0.20	1.01	0.21	1.24	2.00	0.79	5.00 ^
40.00	1.09	0.21	0.99	0.21	1.24	2.00	0.77	5.00 ^
45.00	1.18	0.16	0.98	0.16	1.24	2.00	0.74	5.00 ^
50.00	1.27	0.11	0.97	0.11	1.24	2.00	0.71	5.00 ^

* F.S.<1: Liquefaction Potential Zone. (If above water table: F.S.=5)
^ No-liquefiable Soils or above Water Table.
(F.S. is limited to 5, CRR is limited to 2, CSR is limited to 2)

CPT convert to SPT for Settlement Analysis:
Fines Correction for Settlement Analysis:

Depth ft	Ic	qc/N60	qc1 atm	(N1)60	Fines %	d(N1)60	(N1)60s
5.00	-	-	-	24.59	NoLiq	0.00	24.59
10.00	-	-	-	13.67	NoLiq	0.00	13.67
15.00	-	-	-	14.74	22.00	0.00	14.74
20.00	-	-	-	13.29	22.00	0.00	13.29
25.00	-	-	-	14.71	NoLiq	0.00	14.71
30.00	-	-	-	11.47	NoLiq	0.00	11.47
35.00	-	-	-	18.91	NoLiq	0.00	18.91
40.00	-	-	-	19.82	NoLiq	0.00	19.82
45.00	-	-	-	14.97	NoLiq	0.00	14.97
50.00	-	-	-	10.50	NoLiq	0.00	10.50

(N1)60s has been fines corrected in liquefaction analysis, therefore d(N1)60=0.
Fines=NoLiq means the soils are not liquefiable.

Settlement of Saturated Sands:
Settlement Analysis Method: Tokimatsu/Seed

Depth	CSRsf	/ MSF*	=CSRm	F.S.	Fines	(N1)60s	Dr	ec	dsz	dsp	S

ft					%		%	%	in.	in.	in.
49.95	0.71	1.00	0.71	5.00	NoLiq	10.50	51.72	2.468	0.0E0	0.000	0.000
45.00	0.74	1.00	0.74	5.00	NoLiq	14.97	61.23	1.923	0.0E0	0.000	0.000
40.00	0.77	1.00	0.77	5.00	NoLiq	19.82	70.21	1.518	0.0E0	0.000	0.000
35.00	0.79	1.00	0.79	5.00	NoLiq	18.91	68.57	1.594	0.0E0	0.000	0.000
30.00	0.81	1.00	0.81	5.00	NoLiq	11.47	53.95	2.340	0.0E0	0.000	0.000
25.00	0.79	1.00	0.79	5.00	NoLiq	14.71	60.70	1.946	0.0E0	0.000	0.000
20.00	0.77	1.00	0.77	0.23	22.00	13.29	57.84	2.101	1.3E-2	0.013	0.013
15.00	0.73	1.00	0.73	0.27	22.00	14.74	60.76	1.943	1.2E-2	1.211	1.224
10.00	0.65	1.00	0.65	5.00	NoLiq	13.67	58.63	2.051	0.0E0	0.000	1.224
5.00	0.49	1.00	0.49	5.00	NoLiq	24.59	78.93	1.211	0.0E0	0.000	1.224

Settlement of Saturated Sands=1.224 in.
qc1 and (N1)60 is after fines correction in liquefaction analysis
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=5.00 ft
S is cumulated settlement at this depth

Settlement of Unsaturated Sands:

Depth	sigma'	sigC'	(N1)60s	CSRsf	Gmax	g*Ge/Gm	g_eff	ec7.5	Cec	ec	dsz	dsp	S
ft	atm	atm			atm			%		%	in.	in.	in.
5.00	0.28	1.27	0.00	0.49	0.00	0.0E0	0.0000	0.0000	0.00	1.2106	0.00E0	0.000	0.000

Settlement of Unsaturated Sands

Settlement of Unsaturated Sands=0.000 in.
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=5.00 ft
S is cumulated settlement at this depth

Total Settlement of Saturated and Unsaturated Sands=1.224 in.
Differential Settlement=0.612 to 0.808 in.

Units: Unit: qc, fs, Stress or Pressure = atm (1.0581tsf); Unit Weight = pcf; Depth = ft; Settlement = in.

1 atm (atmosphere) = 1.0581 tsf(1 tsf = 1 ton/ft2 = 2 kip/ft2)
1 atm (atmosphere) = 101.325 kPa(1 kPa = 1 kN/m2 = 0.001 Mpa)

SPT	Field data from Standard Penetration Test (SPT)
BPT	Field data from Becker Penetration Test (BPT)
qc	Field data from Cone Penetration Test (CPT) [atm (tsf)]
fs	Friction from CPT testing [atm (tsf)]
Rf	Ratio of fs/qc (%)
gamma	Total unit weight of soil
gamma'	Effective unit weight of soil
Fines	Fines content [%]
D50	Mean grain size
Dr	Relative Density
sigma	Total vertical stress [atm]
sigma'	Effective vertical stress [atm]
sigC'	Effective confining pressure [atm]
rd	Acceleration reduction coefficient by Seed
a_max.	Peak Ground Acceleration (PGA) in ground surface
mZ	Linear acceleration reduction coefficient X depth
a_min.	Minimum acceleration under linear reduction, mZ
CRRv	CRR after overburden stress correction, CRRv=CRR7.5 * Ksig
CRR7.5	Cyclic resistance ratio (M=7.5)
Ksig	Overburden stress correction factor for CRR7.5
CRRm	After magnitude scaling correction CRRm=CRRv * MSF
MSF	Magnitude scaling factor from M=7.5 to user input M
CSR	Cyclic stress ratio induced by earthquake
CSRfs	CSRfs=CSR*fs1 (Default fs1=1)
fs1	First CSR curve in graphic defined in #9 of Advanced page

fs2	2nd CSR curve in graphic defined in #9 of Advanced page
F.S.	Calculated factor of safety against liquefaction F.S.=CRRm/CSRsf
Cebs	Energy Ratio, Borehole Dia., and Sampling Method Corrections
Cr	Rod Length Corrections
Cn	Overburden Pressure Correction
(N1)60	SPT after corrections, (N1)60=SPT * Cr * Cn * Cebs
d(N1)60	Fines correction of SPT
(N1)60f	(N1)60 after fines corrections, (N1)60f=(N1)60 + d(N1)60
Cq	Overburden stress correction factor
qc1	CPT after Overburden stress correction
dqc1	Fines correction of CPT
qc1f	CPT after Fines and Overburden correction, qc1f=qc1 + dqc1
qc1n	CPT after normalization in Robertson's method
Kc	Fine correction factor in Robertson's Method
qc1f	CPT after Fines correction in Robertson's Method
Ic	Soil type index in Suzuki's and Robertson's Methods
(N1)60s	(N1)60 after settlement fines corrections
CSRm	After magnitude scaling correction for Settlement calculation CSRm=CSRsf / MSF*
CSRfs	Cyclic stress ratio induced by earthquake with user inputed fs
MSF*	Scaling factor from CSR, MSF*=1, based on Item 2 of Page C.
ec	Volumetric strain for saturated sands
dz	Calculation segment, dz=0.050 ft
dsz	Settlement in each segment, dz
dp	User defined print interval
dsp	Settlement in each print interval, dp
Gmax	Shear Modulus at low strain
g_eff	gamma_eff, Effective shear Strain
g*Ge/Gm	gamma_eff * G_eff/G_max, Strain-modulus ratio
ec7.5	Volumetric Strain for magnitude=7.5
Cec	Magnitude correction factor for any magnitude
ec	Volumetric strain for unsaturated sands, ec=Cec * ec7.5
NoLiq	No-Liquefy Soils

References:

1. NCEER Workshop on Evaluation of Liquefaction Resistance of Soils. Youd, T.L., and Idriss, I.M., eds., Technical Report NCEER 97-0022.
SP117. Southern California Earthquake Center. Recommended Procedures for Implementation of DMG Special Publication 117, Guidelines for Analyzing and Mitigating Liquefaction in California. University of Southern California. March 1999.
2. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING AND SEISMIC SITE RESPONSE EVALUATION, Paper No. SPL-2, PROCEEDINGS: Fourth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, San Diego, CA, March 2001.
3. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING: A UNIFIED AND CONSISTENT FRAMEWORK, Earthquake Engineering Research Center, Report No. EERC 2003-06 by R.B Seed and etc. April 2003.

Note: Print Interval you selected does not show complete results. To get complete results, you should select 'Segment' in Print Interval (Item 12, Page C).