GEO ENVIRON
GEOTECHNICAL AND ENVIRONMENTAL ENGINEERING CONSULTANTS, INC.
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Job No. 19-990P
July 20, 2020

Mr. Joseph Karaki
Western State Eng. & Construction, Inc.
4887 E. La Palma Street, Ste # 707
Anaheim, Ca 92807

Subject: **Preliminary Geotechnical Investigation Report for Foundation Design, Proposed Commercial Retail, N.W.C Central Ave (74 Hwy) & Allan St, Lake Elsinore California**

Reference:

1) WS Eng., Inc , 6/9/20, "Site Plan, Proposed Commercial Retail, N.W.C Central Ave (74 Hwy) & Allan St, Lake Elsinore California

Gentlemen:

In accordance with your request and authorization, we have performed a preliminary geotechnical engineering investigation for the subject project. The accompanying report presents the preliminary results of our field exploration work, laboratory tests, our geotechnical experience previously performed in the vicinity of the project site, as well as engineering analysis. The subsurface and foundation conditions are discussed and preliminary recommendations for the geotechnical engineering aspects of the project are presented.

This opportunity to be of service is appreciated. If you have any questions concerning our findings, please call at your convenience.

Respectfully submitted,

Geo Environ Eng. Consultants, Inc.

Jabed Masud, MSCE
Principal

Esmail Rastegari, P.E.
Principal

JM/ER/gm

Attachments: Appendix 'A' - Drawings, Boring Location
Appendix 'B' - Boring Logs
Appendix 'C' - Laboratory Test Results
Appendix 'D' -Liquefaction Analysis

SCOPE

The scope of this study was designed to determine and evaluate the surface and subsurface conditions of the subject site and to present preliminary recommendations for the foundation systems and grading requirements as they relate to the planned development

The scope included the following geotechnical functions:

- Review of available literature pertaining to the site and vicinity.
- Evaluation of natural and manmade surface features at the site and contiguous areas.
- Drilling and logging of exploratory borings.
- Securing of bulk and undisturbed samples of earth materials from the trenches for laboratory testing.
- Laboratory testing of selected samples.
- Geotechnical engineering analysis of data obtained during the study.
- Preparation of this report and the accompanying illustrations to present the findings, conclusions, and recommendations pertaining to the planned construction.

The scope of work did not include any environmental assessment of the property or opinions relating to possible soil or subsurface contamination by hazardous or toxic substances.

SITE DESCRIPTION

Location

The subject property upon which the soil exploration has been performed is located at N.W.C Central Ave (74 Hwy) & Allan St, Lake Elsinore, California. Surrounding the site is vacant land, commercial building, and multi-family buildings.

Site Conditions

The subject site is a vacant plot of land, 2.57 acres in size. The site is currently vacant and the north-west side of the property rests on a descending slope.

PROPOSED CONSTRUCTION & GRADING

Preliminary details of the proposed construction and the reference drawing were provided by the project Architect. As shown on the site plan, attached herein (Appendix 'B').

Three (3) commercial retail stores approximately 4425 sq. ft, 4050 sq. ft, and 3525 sft in size with associated parking pavement are planned within the subject site. Loads on the foundations are unknown but are expected to be between 2 and 4 kips per linear foot. Column loads are expected to be between 50 and 100 kips. Other structures will remain in place. No grading plans were to provided to this firm for our use during this study. However, cut and fill grading are anticipated within the proposed construction areas. Should details involved in final design vary from those outlined above, this firm should be notified for review and possible revision of our recommendations.

FIELD STUDY

A field study consisting of site observations and subsurface exploration was conducted on July 13, 2020. Three exploratory borings were drilled to a maximum depth of 50 feet below existing surface The soils encountered in the exploratory drilling were logged by our field personnel. The boring log are included in Appendix 'B'. The approximate location of the borings are shown on the plot plan in Appendix 'A'. Disturbed and undisturbed samples of the soils encountered were obtained at frequent intervals in the borings. Undisturbed samples were obtained by driving a thin walled steel sampler with successive drops of a 140-pound weight having a free fall of 30 inches. The blow count for each one foot of penetration is shown on the boring logs. Undisturbed soils were retained in brass rings with a 1-inch height and 2.413-inch inside diameter. The ring samples were retained in close fitting moisture proof containers and transported to our laboratory for testing.

LABORATORY TEST

The results of laboratory tests performed on disturbed, undisturbed, and remolded soil samples are presented in appendix 'C'. Following is a listing and brief explanation of the laboratory tests which were performed as part of this study. The remaining soil samples are stored in our laboratory for future reference. Unless notified to the contrary, all samples will be disposed of 30 days after the date of this report.

Classification

The field classification of the soils were verified in the laboratory in general accordance with the Unified Soil Classification System. The final classification is shown on the boring logs.

Field Moistures and Densities

The field moisture content was determined for each of the disturbed and undisturbed soil samples. The dry density was also determined for each of the undisturbed samples. The dry density was determined in pounds per cubic foot and the field moisture content was determined as a percentage of the dry weight of the soil. Both results are shown on boring logs.

Consolidation Tests

Settlement predictions of the soil's behavior under load were made on the basis of the consolidation tests which are performed in general accordance with ASTM D-2435 procedures. The Consolidation apparatus is designed to receive a one inch high ring.

Expansion Characteristics

Laboratory expansion tests were performed on a near surface soil sample in general accordance with ASTM D-4829 procedures.

Direct Shear Test

Direct Shear test was performed in the Direct Shear Test Machine which is of the strain control type in general with ASTM D-3080 procedure. Each sample was sheared under varying pressures normal to the face of the specimen to determine the shear strength (cohesion and angle of internal friction). Samples were tested in a submerged condition. The result is plotted on the "Direct Shear Test Graph."

GEOTECHNICAL CONDITIONS

Earth Materials

The site consists of top native soils (alluvium) classified as brown-orange, sandy silt with coarse sand & gravel, dense to very dense in consistency, moist to very moist all the way to a depth of 50 feet.

A more detailed description of the earth materials encountered is presented on the log borings in Appendix 'B'. The boring locations are presented on the log borings in Appendix 'A'. The soil strata as the boring logs represents the soil conditions in the actual boring locations other variations may occur between the borings. Lines of demarcation represent the approximate boundary between the soil types, but the transition may be gradual. Please note that we have performed a limited soil investigation. Based on the soil borings, it appeared that the site is underlain with native alluvial soils. During grading and construction, the soil engineer or his representatives will definitely verify unsuitable fill depths if any exist. Revised recommendations will be provided, if needed.

Groundwater

Groundwater was encountered at approximately 37 feet below the existing grade in the exploratory boring during our subsurface exploration. The historic groundwater may have been existed at 15 feet below the existing surface obtained from near by soil boring data.

Seismicity

The frequency of earthquake and intensity of seismic ground shaking to be expected at the site depends upon which fault produces the earthquake, the earthquake magnitude and the distance to the epicenter.

Nearby active fault lines are the Elsinore-Glen Ivy and Elsinore-Temecula, these have associated postulated, maximum probable earthquake magnitudes of 6.8. In turn, the probabilistic ground motion acceleration range upwards to $\pm$ 0.859g. The related California Building Code factors include the type B, Elsinore-Glen Ivy Fault the near source zone at 5.0 kilometers towards the south-west and a soil profile type of alluvium or Sd.

Based on the California Building Code acceptance of some structural damage without collapse, the subject development may be designed in accordance with the seismic formulas and requirements presented in the latest version of the California Building Code (CBC 2016). It is the responsibility of the project structural engineer to utilize the critical seismic factors to be used for building design and to implement the applicable sections of the code. Please refer to the referenced boring log B-1A. B-1A.There was not enough information regarding the liquefaction potential of the site. A liquefaction analysis will be performed to determine the liquefaction potential at the site .

Liquefaction

A liquefaction analysis was performed to verify the susceptibility to liquefaction at the subject location due to no current data in existence to verify liquefaction potential in the area. **Our analysis has determined that subject property is NOT susceptible to liquefaction.** Please reference the liquefaction analysis report attached in Appendix 'D' to this report.

CONCLUSIONS

- The plan construction and development of the site is considered feasible from a geotechnical engineering point of view provided the engineering recommendations of this report are followed.
- The surface and the subsurface soil on the site will be adequate for the support of the structure and any fill soils proposed for the site.
- The proposed structure, grading, and development of the site will not cause adverse safety hazards or instability to the adjacent properties or their structures.
- Conversely, the adjacent properties or their structures will not cause adverse safety hazards or instability to the planned development.
- Laboratory expansion test indicate that the soils on the site have low expansion potential.
- The site is not susceptible to liquefaction.

RECOMMENDATIONS

Rough Grading Recommendation

The following recommendations may need to be modified and/ or supplemented during rough grading as field conditions necessitate. All earthwork and grading shall be performed in accordance with the recommendations presented herein, and in accordance with all applicable requirements of the Grading Code of the City of Lake Elsinore, California.

The proposed building areas should be overexacavated to a depth of 3.0 feet below the existing soil grade, or 2.0 feet below the proposed footing bottoms, whichever is greater. Where possible, the limits of overexcavation for building areas shall extend at least 5.0 feet beyond the proposed building limits or to the property line whichever is less. The excavated soils free from debris or other organic may be replaced as a certified compacted fill. The depth of the excavations will be determined by the soil engineer during grading and construction, based on the previously placed un-certified fill.

The competency of the exposed overexcavation bottoms must be determined by the soil engineer or his representative at the time they are exposed and prior to scarification or placement of fill. All overexcavation bottoms and any areas to receive fill shall be scarified a minimum of 6 inches, watered or aerated as necessary to achieve optimum moisture content, and properly compacted to at least 90% of maximum dry density prior to filling.

For the purpose of estimating earthwork quantities, a shrinkage factor of 15% (0.15) may be assumed for the existing near surface on-site soil to be used as fill and compacted to 90% of maximum dry density. Subsidence due to grading is estimated to be .1 feet.

Any soil to be placed as fill, whether natural or import, shall be approved by the soil engineer or his representative prior to their placement. The fill material shall be free from vegetation, organic material or debris. Import soil shall be no more expansive than the existing near surface soils on the site.

Suitable fill soil shall be placed in horizontal lifts not exceeding 6 inches in thickness after compaction and uniformly watered or aerated to obtain optimum moisture content. Each layer shall be spread evenly and shall be thoroughly mixed during the spreading to ensure uniformity of the soil and optimum moisture in each layer. After each lift has been placed, it shall be thoroughly compacted to not less than 90% of maximum dry density.

The soil engineer or his representative shall observe the placement of fill and should take sufficient tests to verify the moisture content and the uniformity and degree of compaction obtained. In-place density testing should be performed in accordance with ASTM acceptable to the local building authority. The optimum moisture content and the maximum dry density for compacted soils shall be determined in accordance with ASTM D-1557 procedures.

Due to the possibility of imported fill soil in the building areas and / or variable soil strata that may be exposed in the building pad, typical soil samples should be obtained at completion of rough grading for laboratory testing to confirm the expansion characteristics of the graded site.

FOUNDATION DESIGN RECOMMENDATIONS

Conventional Footing Recommendations

- The footings for the proposed building should be founded into certified compacted fills and minimum 24 inches below lowest adjacent finished grade.

- Continuous footing should be reinforced with at least two (2) #4 rebars at the top and at the bottom of the footing in order to minimize the effects of any minor variations in the engineering characteristics in the supporting soils.
- All pad footings should be a minimum of 24 inches square by 24 inches in depth.

The project Architect / or Structural Engineer shall determine actual footing widths, depths and reinforcements necessary to resist design vertical, horizontal and uplift forces.

Allowable Soil Bearing Capacities

Based on the field and laboratory test data, an allowable soil bearing value of 2000 psf is recommended for the design of the continuous and spread footings for the proposed building. A 1/3 increase in the above bearing value may be used when considering short term loading from wind or seismic sources. A factor of safety of 3 was used in calculation of the soil allowable bearing capacity.

Settlement

Using the recommended bearing value and the maximum assumed wall and column loads, the total settlement is estimated to be 0.5 inches . The differential settlement is estimated to be on the order of 0.25 inches, between similarly loading footing of the same size, over a minimum horizontal distance of 30 feet.

Lateral Bearing Pressure

Additional soil design parameters that may be pertinent to the design and development based on undisturbed natural soil or properly compacted fill are as follows:

- Allowable lateral soil pressures using a factor of safety of 3 (Equivalent Fluid Pressure) Passive case: 300 psf/ft.
- Allowable Coefficient of Friction between concrete and soil: 0.30

Seismic Design

In accordance with the ASCE 7-16 the structural design should consider the following design parameters:

Site Latitude: 33.699782 degree

Site Longitude:117.329646 degree

Site Class: Default

Short Period Site Coefficient- **Fa: 1.2**

Long Period Site Coefficient- **Fv: Null**

Mapped Spectral Response Acceleration-Short Period: (0.2 sec)-**Ss: 2.021**

Mapped Spectral Response Acceleration-Short Period: (1 sec)-**S1: 0.724**

Adjusted Spectral Response Acceleration-Short Period: (0.2 sec)-**Sms: 2.425**

Adjusted Spectral Response Acceleration-Short Period: (1 sec)-**Sm1**: **Null**

Design Spectral Response Acceleration-Short Period: (0.2 sec)-**Sds**: **1.617**

Design Spectral Response Acceleration-Short Period: (1 sec)-**Sd1**: **Null**

FLOOR SLAB RECOMMENDATIONS

Concrete slabs should be constructed in accordance with the following section.

Floor slabs should be a minimum of 4 inches thick. Floor slabs should be reinforced with # 3 rebars at 18- inches on centers.

Concrete slabs should be underlain with a minimum 6 mil polyvinyl chloride membrane vapor retarder with a minimum overlap of 12 inches in all directions. This membrane should be sandwiched between two, two-inch layers of sand.

The slab subgrade should be moisture conditioned to near optimum moisture content condition to a depth of 12 inches immediately prior to placement of the moisture barrier or pouring concrete.

CEMENT TYPE

A very low exposure to sulfate can be expected for concrete placed in contact with on site soil and native material. Therefore, based on the CBC no special cement will be required for concrete in contact with these materials.

Sample	PH	Soluble Sulfates per CA 417 (PPM)	Soluble Sulfates per CA 422 (PPM)	Minimum Resistivity per CA 643 (ohm-cm)
#1	6.0	20	45	2600

RETAINING WALL RECOMMENDATIONS

Retaining walls if planned should be designed to resist the active pressures summarized in the following table. The active pressure is no rmally calculated from the lowermost portion of the footing to the highest ground surface at the back of the wall, including necessary factors for sloping ground. The active and passive pressures indicated in the table are equivalent fluid densities. Walls that are not free to rotate or that are braced at the top should use active pressures that are 50% greater than those indicated in the table. Retaining wall design for passive resistance should neglect the top foot of earth in front of the wall.

Retaining Wall Design Parameters

Equivalent Fluid Pressures

Slope of adjacent ground	Active Pressure backfill with low expansive soil.
Level	30 pcf
2:1	45 pcf

The pressures shown on above table are for retaining walls backfilled with non-cohesive granular materials available on the site, and provided with drainage devices such as weep holes or subdrains to prevent the build-up of hydrostatic pressures beyond the design values. Also, it is strongly recommended that all backfill material be compacted to a minimum of 90 percent relative compaction, as this is the density from which the pressure are calculated. This recommendation cannot be overemphasized.

TEMPORARY CONSTRUCTION CUTS

Temporary construction cuts for retaining walls, foundations, utility trenches, etc., in excess of 5 feet in depth should cut back into an inclination not steeper than 1:1 (horizontal to vertical). Where more restrictive, the safety requirements for excavations contained in the State Construction Safety Orders enforced by the State Division of Industrial Safety (CAL-OSHA) and or the safety codes of the local agency having jurisdiction over the project shall apply. All excavations shall be initially observed by the geotechnical engineer or his representative to verify the recommendations presented or to make any additional recommendations necessary to maintain stability.

TRENCH BACKFILL

Trench excavations for utility lines which extend under building and paved areas are within the zone of influence of adjacent foundations shall be properly backfilled and compacted in accordance with the following recommendations.

The pipe should be bedded and backfilled with clean sand or approved granular soil (minimum Sand Equivalent Value of 30) to a depth of at least 1 foot over the pipe. This backfill should be uniformly watered and compacted to a firm condition.

The remainder of the backfill should be on-site soil or very low to low expansive import soil, which should be placed in loose lifts not exceeding 12 inches in thickness, watered or aerated to optimum moisture content, and mechanically compacted to at least 90% of maximum dry density as determined by ASTM D-1557 procedures. Water jetting of the backfill is not allowed.

PAVEMENT RECOMMENDATIONS

For preliminary design purposes, the typical soil anticipated in the subgrade will consist of fine silty sand. Based on this soil type (silty sand), a conservative R-Value of 50 can be estimated for preliminary design of the pavement section. The actual R-Value of the subgrade soil should be tested and verified at the time of construction. The following are our preliminary recommendations for the structural pavement section calculated in general accordance with Caltrans procedures and based on the estimated R-Value and the Traffic Indexes (TI).

Site Area	Traffic Index	R-value	Pavement Section
Parking	4.5	50	3" A.C. over 4.5" Class II Base
Vehicle Drive Area	5.5	50	4" A.C. over 6" Class II Base
Heavy Truck Area	6.5	50	4" A.C. over 6.0" Class II Base

As an alternative to asphaltic concrete pavement, Portland Cement Concrete (PCC) pavement may be utilized. Concrete driveway and parking slabs shall be at least 5 inches thick and provided with saw cuts or expansion joints every 10 feet or less. The reinforcing shall consist with No. 3 bars spaced 24 inches on centers, both ways. Concrete pavement should be underlain by a minimum 4 inches of base course. The concrete should have a 28-day concrete strength of at least 2500 psi.

To reduce the potential of unsightly cracking concrete pavement for sidewalk and hardscape should be at least 4 inches thick and provided with saw cuts or expansion joints every 6 feet or less.

Subgrade soils should e overexcavated, scarified and compacted to at least 90% + of laboratory maximum dry density as recommended in the previous section of rough grading. Base course should be compacted to at least 95% + of laboratory maximum dry.

PLAN REVIEW

Subsequent to formulation of final development plans and specifications but prior to construction, grading and foundation plans should be reviewed by Geo Environ to verify compatibility with site geotechnical conditions and conformance with recommendations contained herein.

CONSTRUCTION OBSERVATIONS

All rough grading of the property shall be performed under engineering observation of Geo Environ.

Geo Environ shall observe all rough grading, foundation excavations, utility trenches backfill subgrade, and base compaction. Observations should be made prior to installation of concrete forms and reinforcing steel in order to verify or modify, if necessary, conclusions and recommendations in this report.

CLOSURE& LIMITATIONS

The findings, conclusions, and recommendations presented reflect our best estimate of subsurface conditions based on the data obtained from a limited subsurface exploration performed during the field study. The conclusions and recommendations are based on generally accepted geotechnical engineering principles and practices. No further warranties are implied nor made.

Due to the possible variability of soil and subsurface conditions within the site, conditions may be encountered during grading and development that may differ from those presented herein. Should any variation or unusual condition become apparent during grading and development, this office should be contacted to evaluate these conditions prior to continuation of work and necessary_revisions to the recommendations.

This office should be notified if changes of ownership occur or if the final plans for the site development indicate structures areas, type of structures, or structural loading conditions differing from those presented in this report.

If the site is not developed or grading does not begin within 12 months following the date of this report, further studies may be required to ensure that the surface or subsurface conditions have not changed.

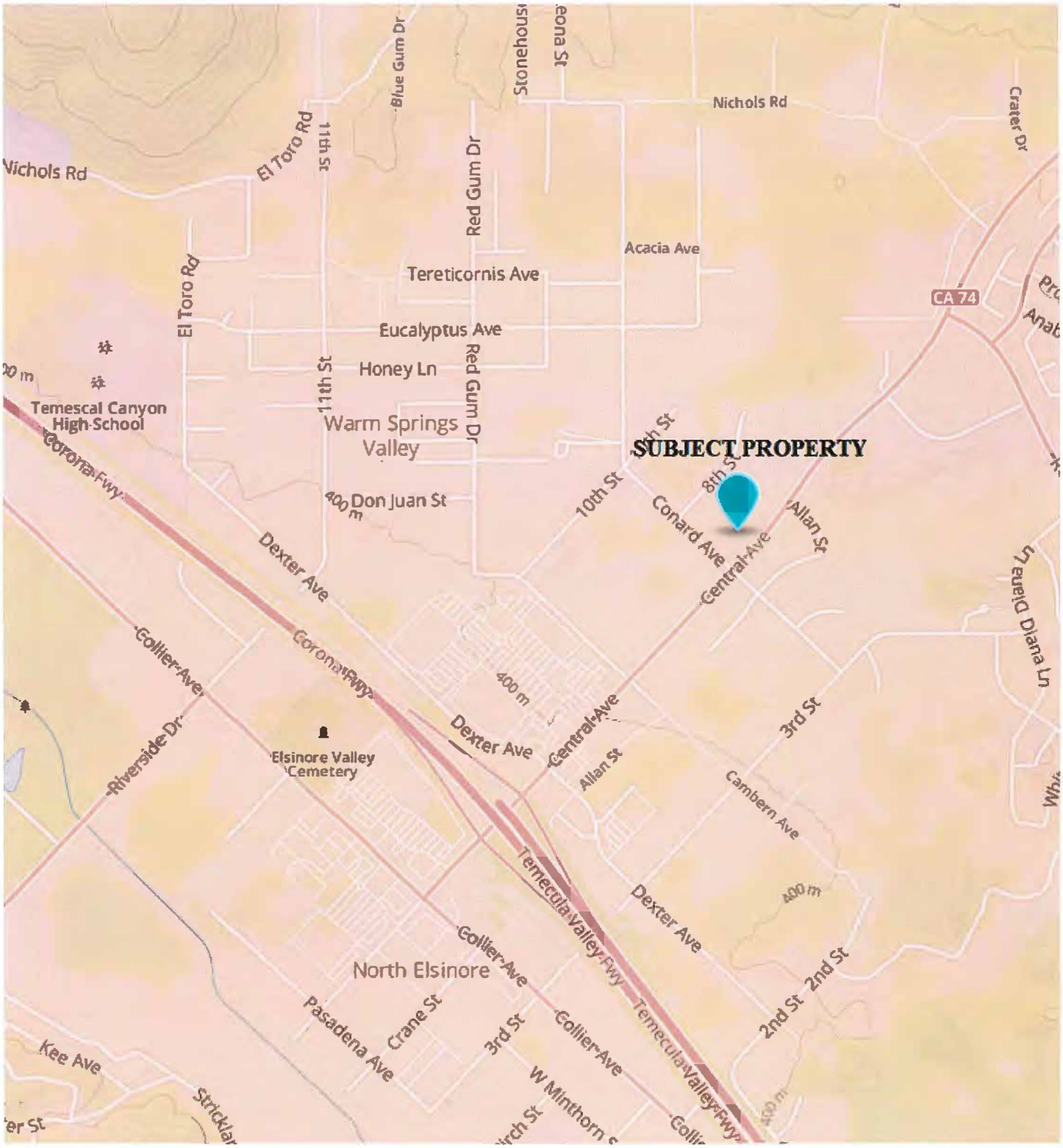
Any charges for necessary review or updates will be at the prevailing rate at the time the review work is performed.

TECHNICAL REFERENCES

1. California Building Code 2019 foundation design parameters
2. County of Riverside Building and Grading Code
3. USGS, Ground Acceleration from Earthquakes
4. USGS, Seismic Design Values for Buildings
5. California Division of Mines and Geology (CDMG), Seismic Hazard & Liquefaction Evaluation
6. Geotracker- Historic Groundwater Data

APPENDIX A

DRAWINGS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Red Gum Dr
Stonehouse
Crater Dr
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Honey Ln
CA 74
Temescal Canyon High School
Warm Springs Valley
Corona Fwy
Don Juan St
400 m
10th St
8th St
Conard Ave
Central Ave
Allan St
Dexter Ave
Collier Ave
Riverside Dr
Elsinore Valley Cemetery
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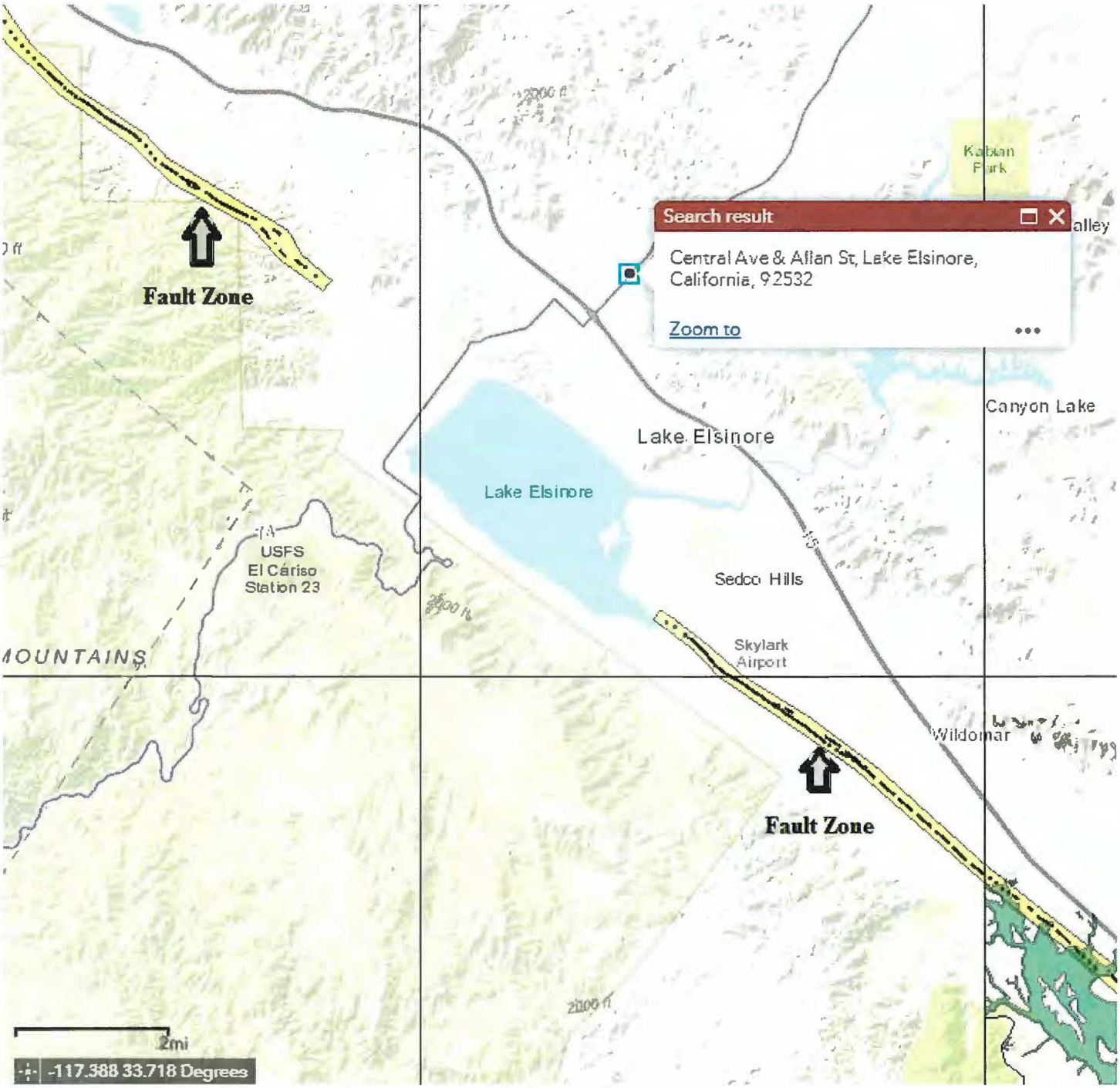
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Canyon Lake
Lake Elsinore
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Sedco Hills
Skylark Airport
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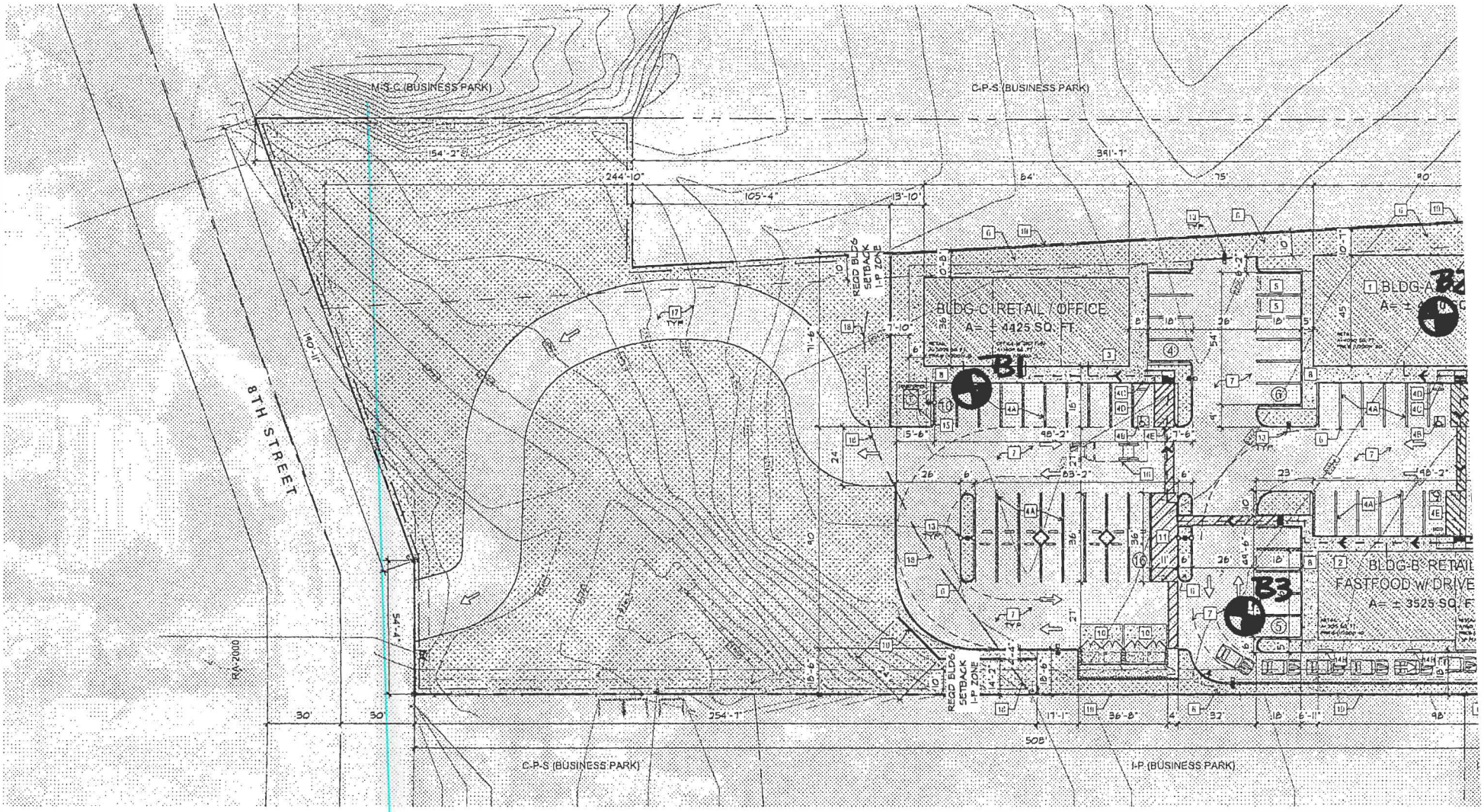

B1 SOIL BORING

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APPENDIX B

BORING LOGS

Project: **Commercial Retail**	Project Number: **20-1041P**	Client: Westerm States	Boring No. **B-1**
Address, City, State: **NWC Central & Allan St Lake Elsinore California**		Drilling Contractor: Geo Environ	Drill Rig Type: **HSA**
Logged By: **Fahad Masud**	Started: **12:00 PM** (4/26/2020)	Bit Type: **N/A**	Diameter: **2 feet**
Drill Crew: **N/A**	Completed: **4:00 PM**	Hammer Type: **Slide Hammer**	
USA Ticket Number: **N/A**	Backfilled:	Hammer Weight: **34 lb**	Hammer Drop: **24 inches**
	Groundwater Depth: **37 feet**	Elevation:	Total Depth of Boring: **50**

Depth (feet)	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log		Dry Density (pcf)	Moisture Content (%)
2			21		Brownish orange, **sandy silt**, slightly moist, w/ fine-med sand, dense	90.1	5.8
5			30		Brownish orange, **sandy silt**, moist, w/ fine-coarse sand and gravel, dense	110.1	7.4
10			45		Brownish red, **sandy silt**, moist, w/ fine-coarse sand and gravel, medium dense	118.2	8.2
15			28			109.5	9.1
20			41			115.2	10.1
25			48			119.2	9.8
30			33		Greenish-brown, **sandy silt**, moist, w/ fine-and coarse gravel, medium dense	113.3	14.1
35			26		Greenish-brown, **sandy silt**, wet, w/ coarse sand and gravel, very dense	107.6	16.2
40			50/6"			120.1	21.2
45			50/6"			122.4	20.1
50			50/6"			121.3	20.8

Standard Penetration Slit Spoon Sampler (SPT)
California Sampler
Shelby Tube
CPP Sampler
Bulk/ Bag Sample
Stabllized Ground water
Groundwater At time of Drilling

Project: **Commercial Retail**	Project Number: **20-1041P**	Client: Westerm States	Boring No. **B-2**
Address, City, State: **NWC Central & Allan St Lake Elsinore California**		Drilling Contractor: Geo Environ	Drill Rig Type: **HSA**
Logged By: **Fahad Masud**	Started: **12:00 PM**	Bit Type: **N/A**	Diameter: **2 feet**
Drill Crew: **N/A**	Completed: **4:00 PM**	Hammer Type: **Slide Hammer**	
USA Ticket Number: **N/A**	Backfilled:	Hammer Weight: **34 lb**	Hammer Drop: **24 inches**
	Groundwater Depth:	Elevation:	Total Depth of Boring: **15**

4/26/2020

Depth (feet)	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log		Dry Density (pcf)	Moisture Content (%)
2	⊠		18		Brownish orange, **sandy silt**, slightly moist, w/ fine-med sand, dense	92.1	6.1
5	⊠		27		Reddish-brown, **sandy silt**, moist, w/ fine-coarse sand and gravel, dense	109.9	7.9
10	⊠		43		Reddish Brown, sandy silt, moist, w/ fine-coarse sand and gravel, medium dense	117.6	8.4
15	⊠		26			110.1	9.7
20							
25							
30							
35							
40							
45							
50							

- Standard Penetration Slit Spoon Sampler (SPT)
- California Sampler
- Shelby Tube
- CPP Sampler
- ⊠ Bulk/ Bag Sample
- Stabllized Ground water
- ▽ Groundwater At time of Drilling

Project: **Commercial Retail**	Project Number: **20-1041P**	Client: Westerm States	Boring No. **B-3**
Address, City, State: **NWC Central & Allan St Lake Elsinore California**		Drilling Contractor: Geo Environ	Drill Rig Type: **HSA**
Logged By: **Fahad Masud**	Started: **12:00 PM** (4/26/2020)	Bit Type: **N/A**	Diameter: **2 feet**
Drill Crew: **N/A**	Completed: **4:00 PM**	Hammer Type: **Slide Hammer**	
USA Ticket Number: **N/A**	Backfilled:	Hammer Weight: **34 lb**	Hammer Drop: **24 inches**
	Groundwater Depth:	Elevation:	Total Depth of Boring: **15**

Depth (feet)	Sample Type	Sample Number	Blow Counts (blows/foot)	Graphic Log		Dry Density (pcf)	Moisture Content (%)
2	⊠		18		Brownish orange, **sandy silt**, slightly moist, w/ fine-med sand, dense	94.1	6.4
5	⊠		27		Reddish-brown, **sandy silt**, moist, w/ fine-coarse sand and gravel, dense	111.9	7.3
10	⊠		43		Reddish Brown, sandy silt, moist, w/ fine-coarse sand and gravel, medium dense	119.1	8.6
15	⊠		26			111.4	8.9
20							
25							
30							
35							
40							
45							
50							

- Standard Penetration Slit Spoon Sampler (SPT)
- California Sampler
- Shelby Tube
- CPP Sampler
- Bulk/ Bag Sample
- Stabllized Ground water
- Groundwater At time of Drilling

APPENDIX C

LABORATORY TEST RESULTS

EXPANSION CHARACTERISTICS
(ASTM D-2435)

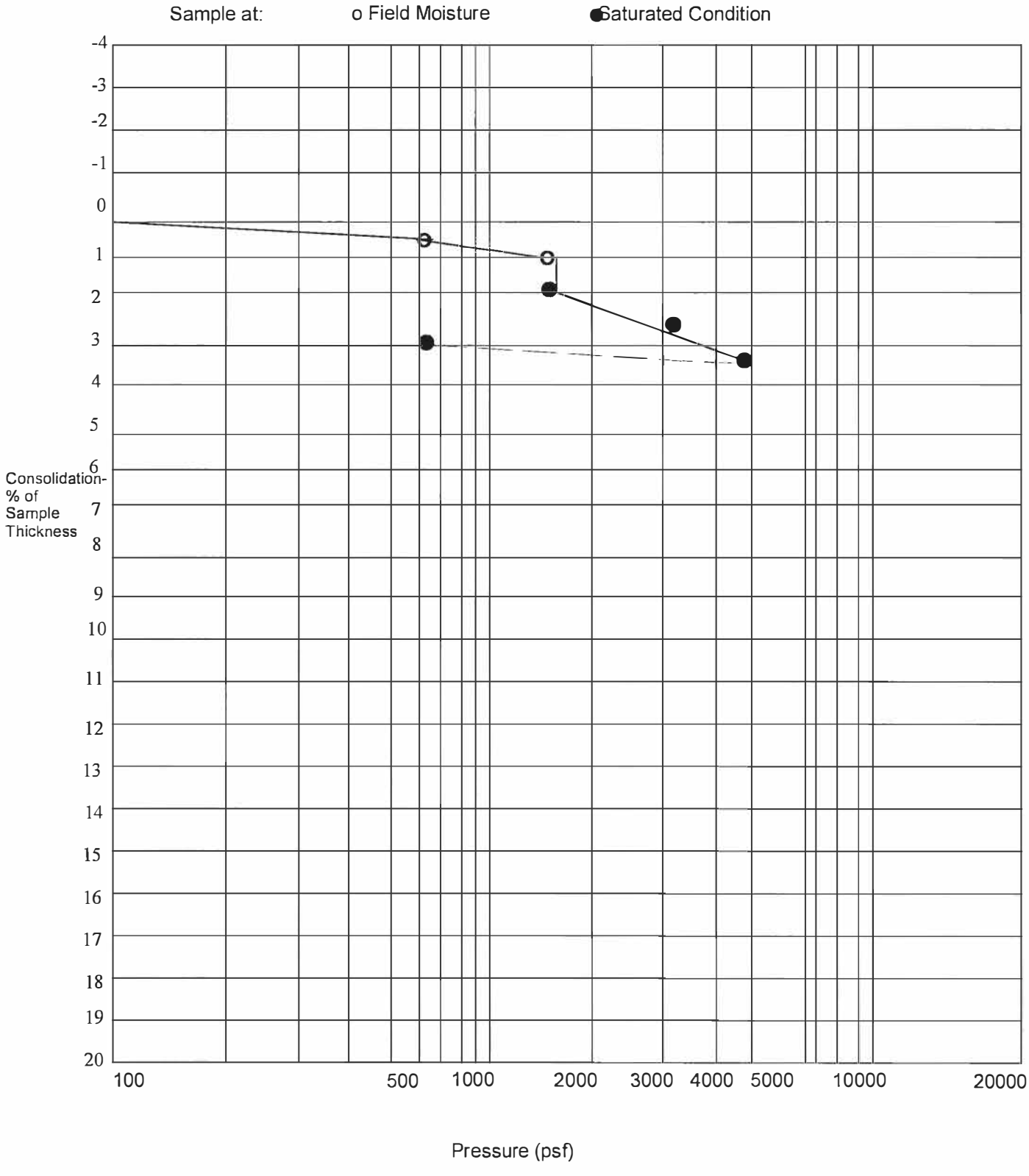
0-21	Very Low
21-50	Low
51-90	Medium
91-130	High
131+	Very High

Sample	Soil Type	Expansion Index	Expansion Classification
B1 @ 0-5'	Fine Sandy Silt	12	Low

MAXIMUM DRY DENSITY
(ASTM D1557)

Sample		Max. Density (pcf)	Opt. Mois.(%)
B1 @ 0-5'	Fine Sandy Silt	126.0	12.5

CONSOLIDATION CURVE: ASTM D-2435
PROJECT NO: 20-1041P
CLIENT: WS Eng.
JOB ADDRESS: N.W.C Central Ave & Allan St, Lake Elsinore
SAMPLE ID: B-1 @ 5.0 ft.
SOIL CLASS: SM-ML
TECH: F.M.
DATE: 6/15/20



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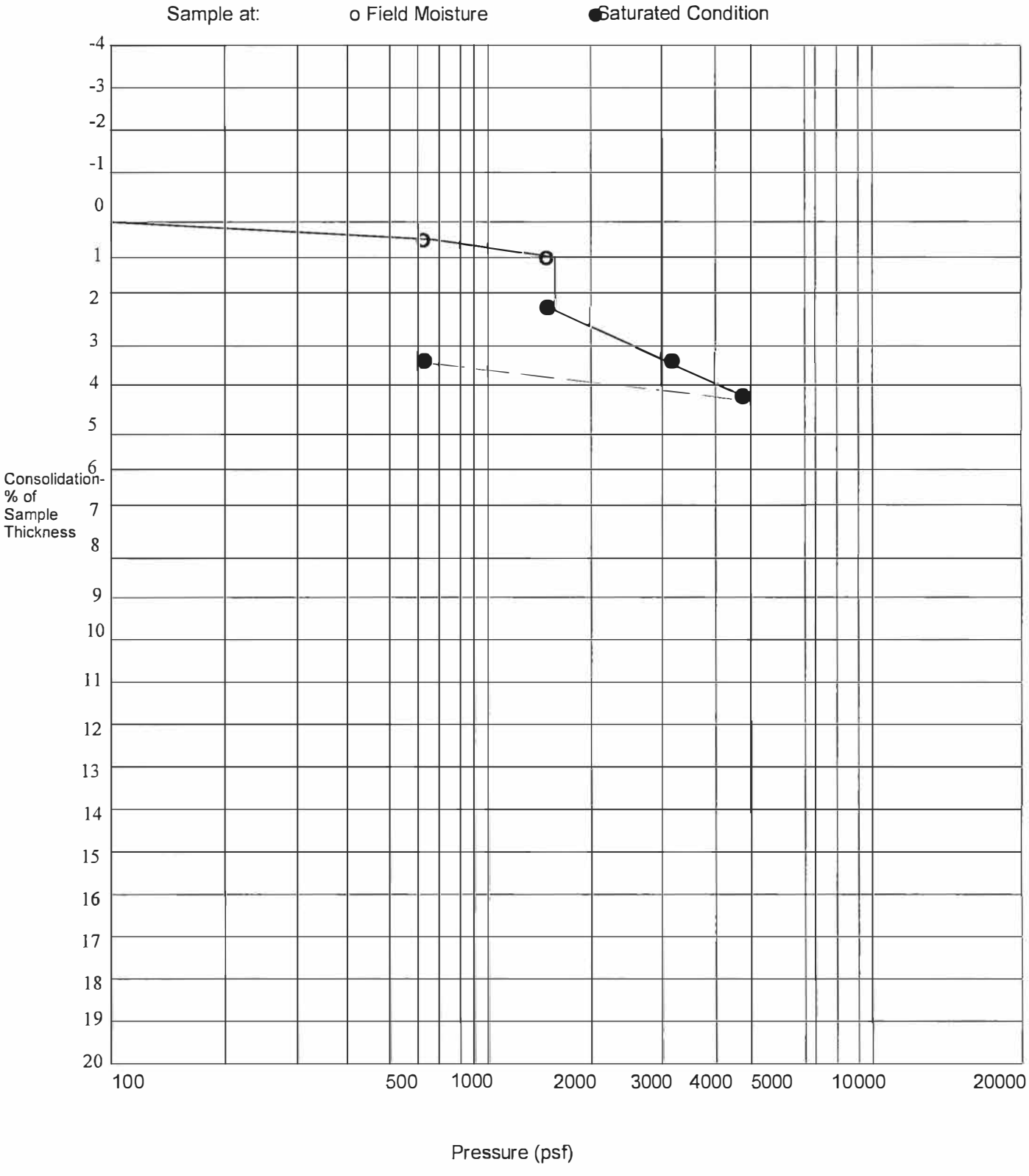
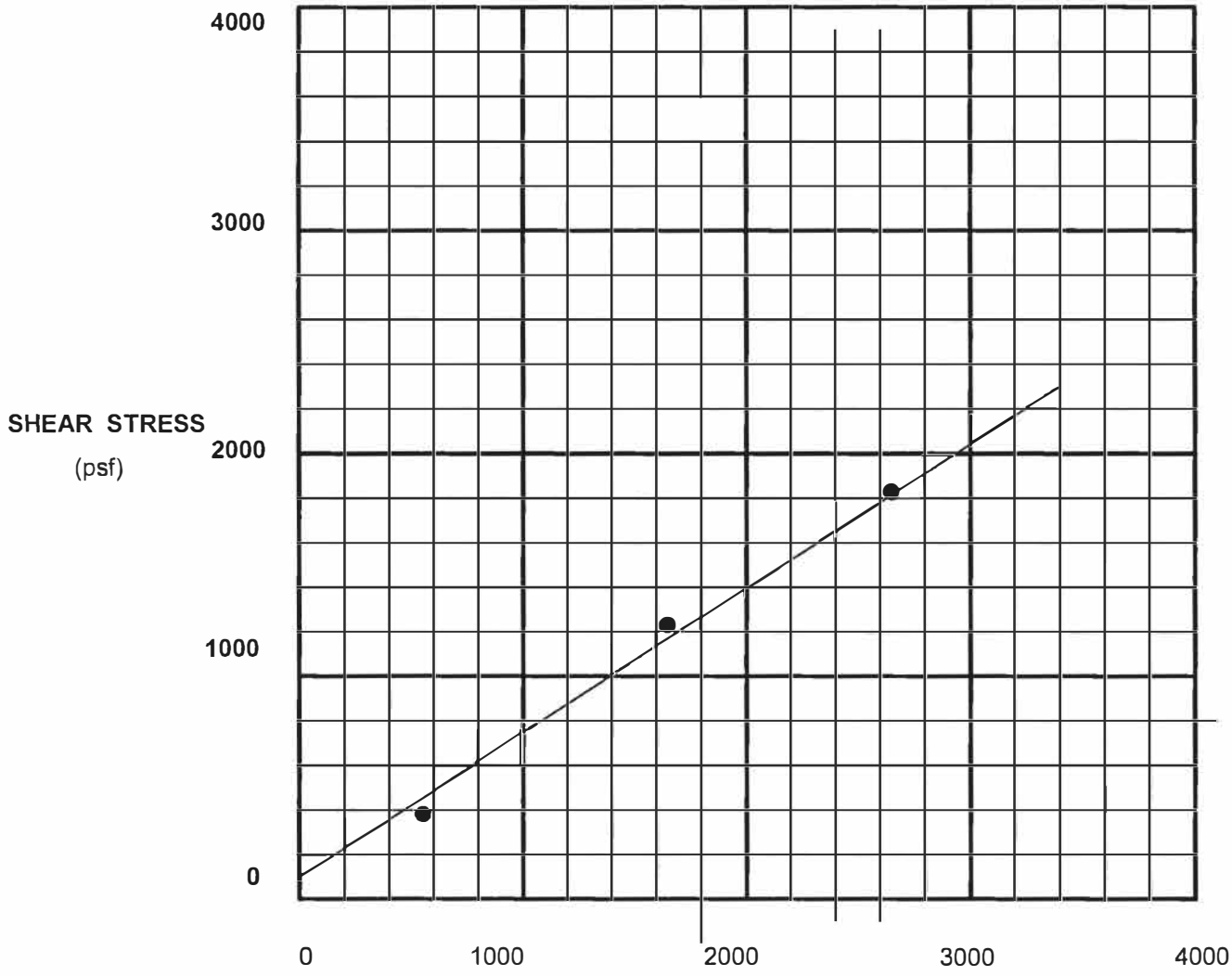
CONSOLIDATION CURVE: ASTM D-2435
PROJECT NO: 20-1041P
CLIENT: WS Eng.
JOB ADDRESS: N.W.C Central Ave & Allan St, Lake Elsinore
SAMPLE ID: B-3 @ 5.0 ft.
SOIL CLASS: SM-ML
TECH: F.M.
DATE: 6/16/20



GEO ENVIRON

DIRECT SHEAR TEST

DATE: 7/16/20
JOB NO: 20-1041P
CLIENT: WS Eng.
JOB ADDRESS: NWC Central Ave & Allan St, Lake Elsinore
SAMPLE ID: B-1 @ 5 '
SOIL CLASS: Fine Sandy Silt
MOIS. CONTENT: 10.5%
DRY DENSITY: 110.0
Sample Type: Un-disturbed
STRAIN RATE: 0.002 in/min
SHEAR STRENGTH: Ultimate
ANGLE OF FRICTION : 31 deg.
COHESION (psf): 100



GEO ENVIRON

APPENDIX D

LIQUEFACTION ANALYSIS

GEOLOGISMIKI Geotechnical Software

SPT BASED LIQUEFACTION ANALYSIS REPORT

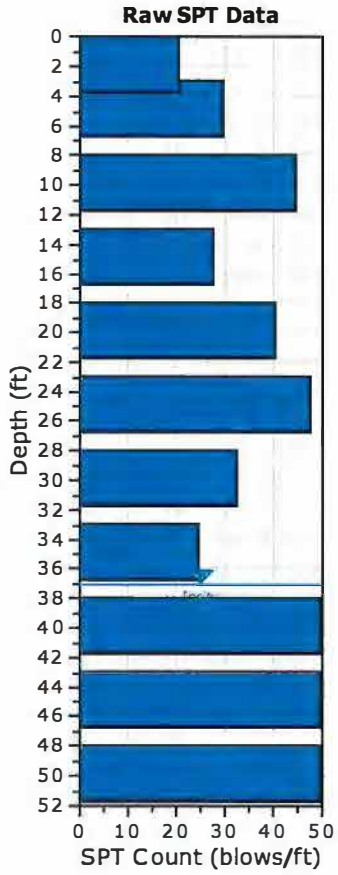
Project title : Edwin Bray (Job No. 20-1027)

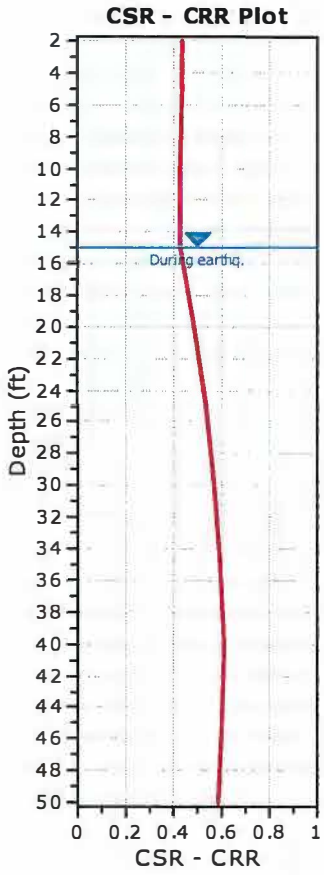
SPT Name: SPT #1

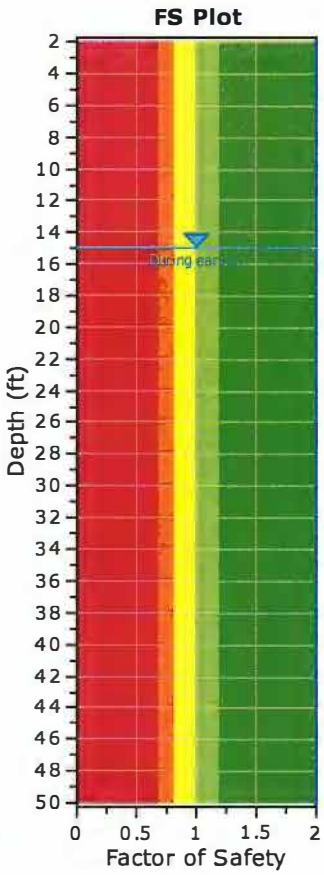
Location : 10626 Felson St, Bellflower

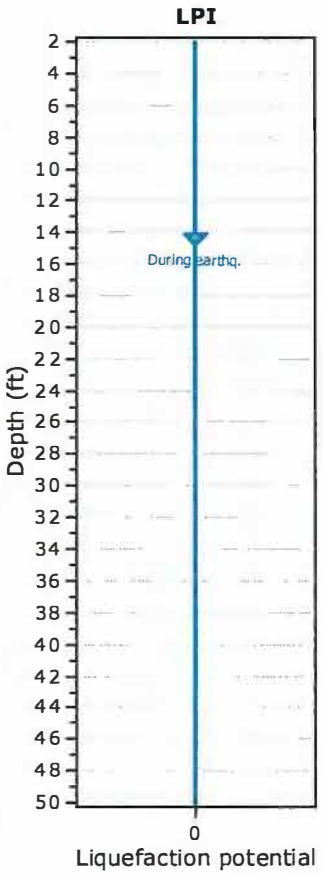
:: Input parameters and analysis properties ::

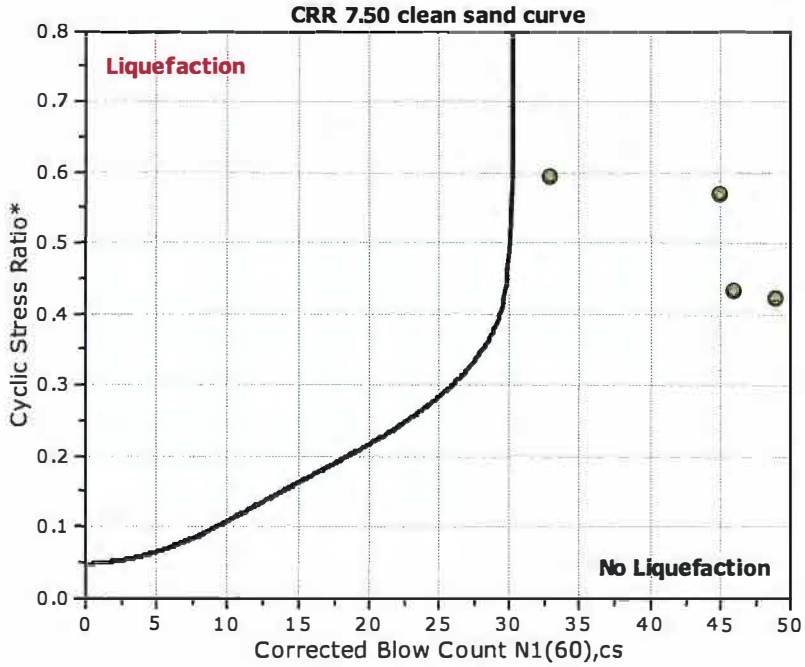
Analysis method:	NCEER 1998	G.W.T. (in-situ):	37.00 ft
Fines correction method:	NCEER 1998	G.W.T. (earthq.):	15.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	6.80
Borehole diameter:	150mm	Peak ground acceleration:	0.86 g
Rod length:	5.00 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.20		











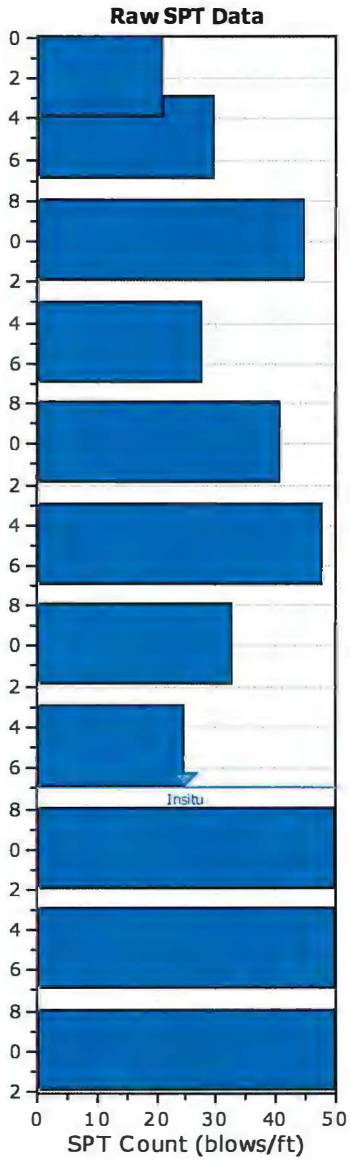
F.S. color scheme

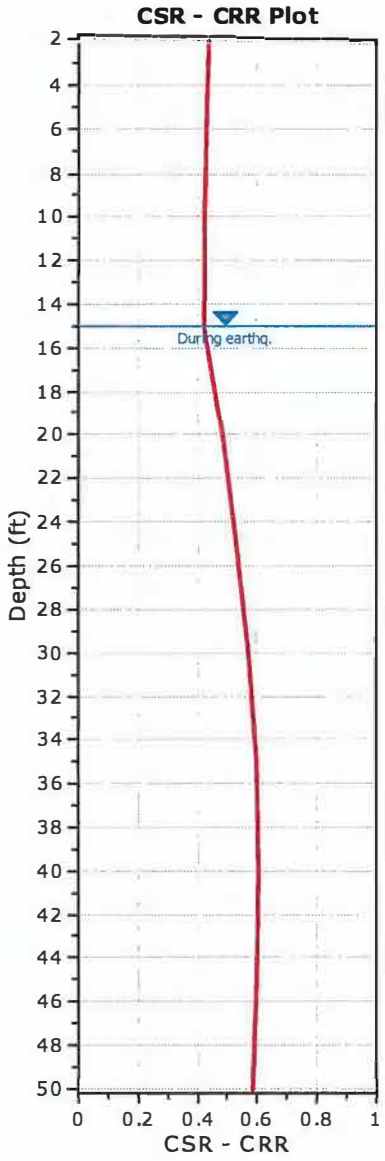
- Almost certain it will liquefy
- Very likely to liquefy
- Liquefaction and no liq. are equally likely
- Unlike to liquefy
- Almost certain it will not liquefy

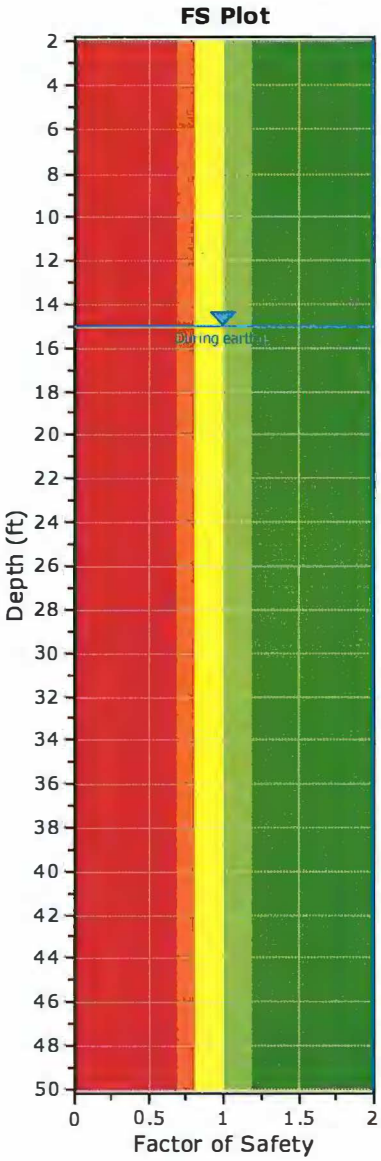
LPI color scheme

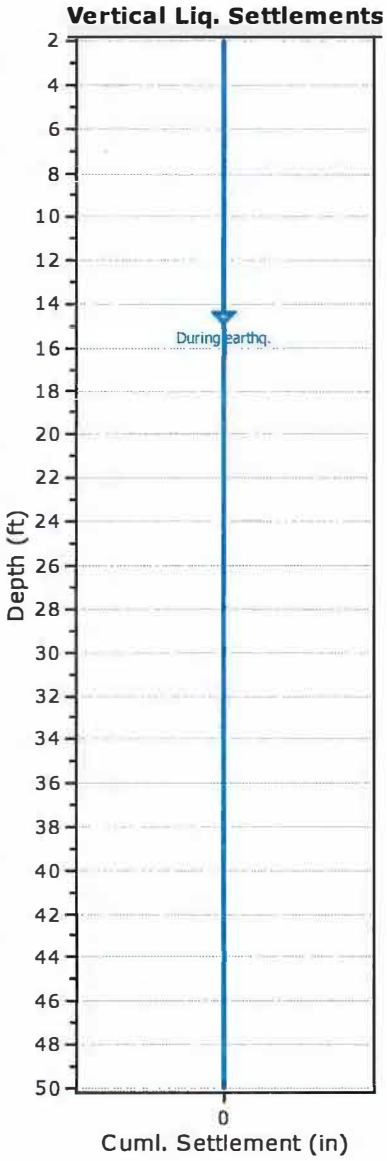
- Very high risk
- High risk
- Low risk

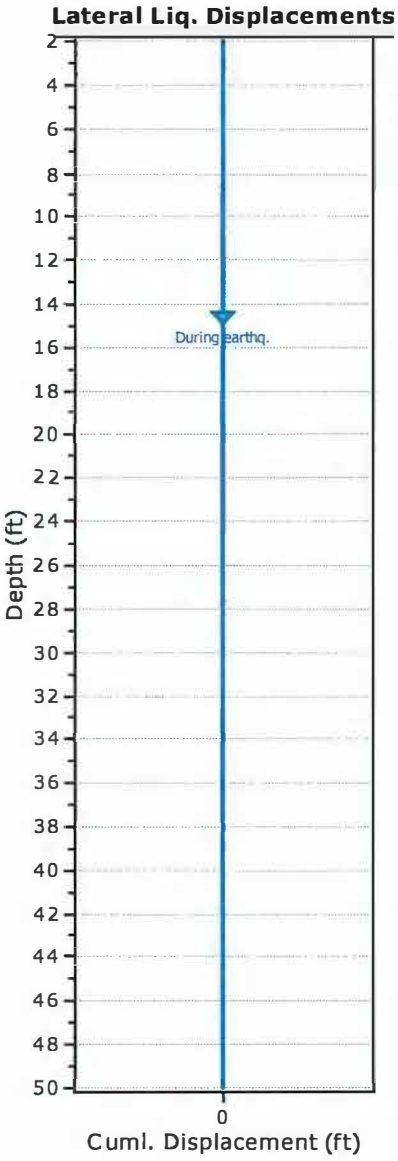
erall Liquefaction Assessment Analysis Plots ::











:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
2.00	21	52.00	90.10	5.00	Yes
5.00	30	52.00	110.10	5.00	Yes
10.00	45	52.00	118.20	5.00	Yes
15.00	28	52.00	109.50	5.00	Yes
20.00	41	52.00	115.20	5.00	Yes
25.00	48	52.00	119.20	5.00	Yes
30.00	33	52.00	113.30	5.00	Yes
35.00	25	51.00	107.50	5.00	Yes
40.00	50	55.00	120.10	5.00	Yes
45.00	50	55.00	122.40	5.00	Yes
50.00	50	55.00	121.30	5.00	Yes

Abbreviations

Depth: Depth at which test was performed (ft)
SPT Field Value: Number of blows per foot
Fines Content: Fines content at test depth (%)
Unit Weight: Unit weight at test depth (pcf)
Infl. Thickness: Thickness of the soil layer to be considered in settlements analysis (ft)
Can Liquefy: User defined switch for excluding/including test depth from the analysis procedure

:: Cyclic Resistance Ratio (CRR) calculation data ::

Depth (ft)	SPT Field Value	Unit Weight (pcf)	σ_v (tsf)	u_o (tsf)	σ'_{vo} (tsf)	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	Fines Content (%)	α	β	$(N_1)_{60cs}$	$CRR_{7.5}$
2.00	21	90.10	0.09	0.00	0.09	1.70	1.20	1.05	0.75	1.00	34	52.00	5.00	1.20	46	4.000
5.00	30	110.10	0.26	0.00	0.26	1.53	1.20	1.05	0.75	1.00	43	52.00	5.00	1.20	57	4.000
10.00	45	118.20	0.55	0.00	0.55	1.28	1.20	1.05	0.85	1.00	62	52.00	5.00	1.20	79	4.000
15.00	28	109.50	0.82	0.00	0.82	1.11	1.20	1.05	0.95	1.00	37	52.00	5.00	1.20	49	4.000
20.00	41	115.20	1.11	0.00	1.11	0.98	1.20	1.05	0.95	1.00	48	52.00	5.00	1.20	63	4.000
25.00	48	119.20	1.41	0.00	1.41	0.87	1.20	1.05	0.95	1.00	50	52.00	5.00	1.20	65	4.000
30.00	33	113.30	1.69	0.00	1.69	0.79	1.20	1.05	1.00	1.00	33	52.00	5.00	1.20	45	4.000
35.00	25	107.50	1.96	0.00	1.96	0.72	1.20	1.05	1.00	1.00	23	51.00	5.00	1.20	33	4.000
40.00	50	120.10	2.26	0.09	2.17	0.68	1.20	1.05	1.00	1.00	43	55.00	5.00	1.20	57	4.000
45.00	50	122.40	2.57	0.25	2.32	0.65	1.20	1.05	1.00	1.00	41	55.00	5.00	1.20	54	4.000
50.00	50	121.30	2.87	0.41	2.47	0.62	1.20	1.05	1.00	1.00	39	55.00	5.00	1.20	52	4.000

Abbreviations

σ_v: Total stress during SPT test (tsf)
u_o: Water pore pressure during SPT test (tsf)
σ'_{vo}: Effective overburden pressure during SPT test (tsf)
C_N: Overburden corretion factor
C_E: Energy correction factor
C_B: Borehole diameter correction factor
C_R: Rod length correction factor
C_S: Liner correction factor
$N_{1(60)}$: Corrected N_{SPT} to a 60% energy ratio
α, β: Clean sand equivalent clean sand formula coefficients
$N_{1(60)cs}$: Corected $N_{1(60)}$ value for fines content
$CRR_{7.5}$: Cyclic resistance ratio for M=7.5

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS

:: Cyclic Stress Ratio calculation (CSR fully adjusted and normalized) ::

Depth (ft)	Unit Weight (pcf)	$\sigma_{v,eq}$ (tsf)	$u_{o,eq}$ (tsf)	$\sigma'_{vo,eq}$ (tsf)	r_d	α	CSR	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR*	FS	
2.00	90.10	0.09	0.00	0.09	1.00	1.00	0.558	1.28	0.434	1.00	0.434	2.000	●
5.00	110.10	0.26	0.00	0.26	0.99	1.00	0.554	1.28	0.431	1.00	0.431	2.000	●
10.00	118.20	0.55	0.00	0.55	0.98	1.00	0.547	1.28	0.426	1.00	0.426	2.000	●
15.00	109.50	0.82	0.00	0.82	0.97	1.00	0.541	1.28	0.421	1.00	0.421	2.000	●
20.00	115.20	1.11	0.16	0.96	0.96	1.00	0.622	1.28	0.484	1.00	0.484	2.000	●
25.00	119.20	1.41	0.31	1.10	0.94	1.00	0.676	1.28	0.526	0.99	0.530	2.000	●
30.00	113.30	1.69	0.47	1.23	0.92	1.00	0.711	1.28	0.554	0.97	0.570	2.000	●
35.00	107.50	1.96	0.62	1.34	0.89	1.00	0.730	1.28	0.568	0.95	0.596	2.000	●
40.00	120.10	2.26	0.78	1.48	0.85	1.00	0.726	1.28	0.565	0.93	0.605	2.000	○
45.00	122.40	2.57	0.94	1.63	0.80	1.00	0.707	1.28	0.550	0.92	0.600	2.000	○
50.00	121.30	2.87	1.09	1.78	0.75	1.00	0.679	1.28	0.529	0.90	0.587	2.000	●

Abbreviations

$\sigma_{v,eq}$: Total overburden pressure at test point, during earthquake (tsf)
$u_{o,eq}$: Water pressure at test point, during earthquake (tsf)
$\sigma'_{vo,eq}$: Effective overburden pressure, during earthquake (tsf)
r_d : Nonlinear shear mass factor
α: Improvement factor due to stone columns
CSR : Cyclic Stress Ratio (adjusted for improvement)
MSF : Magnitude Scaling Factor
$CSR_{eq,M=7.5}$: CSR adjusted for M=7.5
K_{sigma}: Effective overburden stress factor
CSR*: CSR fully adjusted (user FS applied)***
FS: Calculated factor of safety against soil liquefaction

*** User FS: 1.00

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
2.00	2.000	0.00	9.70	3.00	0.00
5.00	2.000	0.00	9.24	3.00	0.00
10.00	2.000	0.00	8.48	5.00	0.00
15.00	2.000	0.00	7.71	5.00	0.00
20.00	2.000	0.00	6.95	5.00	0.00
25.00	2.000	0.00	6.19	5.00	0.00
30.00	2.000	0.00	5.43	5.00	0.00
35.00	2.000	0.00	4.67	5.00	0.00
40.00	2.000	0.00	3.90	5.00	0.00
45.00	2.000	0.00	3.14	5.00	0.00
50.00	2.000	0.00	2.38	5.00	0.00

Overall potential I_L : 0.00

I_L = 0.00 - No liquefaction
I_L between 0.00 and 5 - Liquefaction not probable
I_L between 5 and 15 - Liquefaction probable
I_L > 15 - Liquefaction certain

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	α	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)

:: Vertical settlements estimation for dry sands ::

Depth (ft)	$(N_1)_{60}$	τ_{av}	p	G_{max} (tsf)	a	b	γ	ε_{15}	N_c	ε_{Nc} (%)	Δh (ft)	ΔS (in)
5.00	43	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	5.00	0.000
10.00	62	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00	5.00	0.000

Cumulative settlemetns: 0.000

Abbreviations

τ_{av}: Average cyclic shear stress
p: Average stress
G_{max}: Maximum shear modulus (tsf)
a, b: Shear strain formula variables
γ: Average shear strain
ε_{15}: Volumetric strain after 15 cycles
N_c: Number of cycles
ε_{Nc}: Volumetric strain for number of cycles N_c (%)
Δh: Thickness of soil layer (in)
ΔS: Settlement of soil layer (in)

:: Vertical settlements estimation for saturated sands ::

Depth (ft)	D_{50} (in)	q_c/N	e_v (%)	Δh (ft)	s (in)
15.00	1.00	7.99	0.00	5.00	0.000
20.00	1.00	7.99	0.00	5.00	0.000
25.00	1.00	7.99	0.00	5.00	0.000
30.00	1.00	7.99	0.00	5.00	0.000
35.00	1.00	7.99	0.00	5.00	0.000
40.00	1.00	7.99	0.00	5.00	0.000
45.00	1.00	7.99	0.00	5.00	0.000
50.00	1.00	7.99	0.00	5.00	0.000

Cumulative settlements: 0.000

Abbreviations

D_{50}: Median grain size (in)
q_c/N: Ratio of cone resistance to SPT
e_v: Post liquefaction volumetric strain (%)
Δh: Thickness of soil layer to be considered (ft)
s: Estimated settlement (in)

:: Lateral displacements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60}$	D_r (%)	γ_{max} (%)	d_z (ft)	LDI	LD (ft)
2.00	34	81.63	0.00	5.00	0.000	0.00
5.00	43	100.00	0.00	5.00	0.000	0.00
10.00	62	100.00	0.00	5.00	0.000	0.00
15.00	37	85.16	0.00	5.00	0.000	0.00
20.00	48	100.00	0.00	5.00	0.000	0.00
25.00	50	100.00	0.00	5.00	0.000	0.00
30.00	33	80.42	0.00	5.00	0.000	0.00
35.00	23	67.14	0.00	5.00	0.000	0.00
40.00	43	100.00	0.00	5.00	0.000	0.00
45.00	41	89.64	0.00	5.00	0.000	0.00
50.00	39	87.43	0.00	5.00	0.000	0.00

:: Lateral displacements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60}$	D_r (%)	γ_{max} (%)	d_z (ft)	LDI	LD (ft)

Cumulative lateral displacements: 0.00

Abbreviations

D_r: Relative density (%)
γ_{max}: Maximum amplitude of cyclic shear strain (%)
d_z: Soil layer thickness (ft)
LDI: Lateral displacement index (ft)
LD: Actual estimated displacement (ft)

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