Cal Land Engineering & Associates, Inc.
Land Surveying, Geotechnical, Environmental, and Civil Engineering

August 29, 2022

Mr. Binh Tran
401 Marion Boulevard
Fullerton, CA 92835

Subject: Report of Geotechnical Engineering Investigation, Proposed Industrial Facility Addition, 12881 166th Street, APN 7010-016-054, Cerritos, California
CLE Project No.: 22-009-050GE

Dear Mr. Tran:

In accordance with your request, Cal Land Engineering (CLE) 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 for the proposed construction.

Based on the findings of our field exploration, laboratory testing, and engineering analysis, the proposed construction of the subject site for the intended use is feasible from the geotechnical engineering viewpoint, 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 & Associates, Inc. (CLE)

Jack C. Lee, GE 2153

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REPORT OF GEOTECHNICAL ENGINEERING INVESTIGATION

Proposed Industrial Facility Addition

At

12881 166th Street

APN: 7010-016-054

Cerritos, California

Prepared by

CALLAND ENGINEERING (CLE)

Project No.: 22-009-050GE

August 29, 2022

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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 for the proposed development.

1.2 Scope of Services

Our scope of services included:

- Review of available soil engineering data of the area.
- Subsurface exploration consisting of logging and sampling of one 8-inch diameter hollow stem auger boring and one 6-inch hand auger boring to a maximum depth of 51.5 feet below the existing grade at the subject site. The exploration was logged by a CLE engineer. Boring log is 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 for the proposed construction.

1.3 Proposed Construction

The subject site would be used for commercial development, consisting of wood or steel framed one and/or two story addition on both the eastern and western sides of the existing building, The proposed building is anticipated to be concrete slab-on-grade structure. Column loads are unknown at this time, but are expected to be light to medium. Minor cut and fill grading operation is anticipated to reach the desired grades.

1.4 Site Conditions

The project site is located on the north side of 166th Street, between Shoemaker Avenue and Bloomfield Avenue, in the City of Cerritos, California. The approximate regional location is shown on the attached Site Location Plan (Figure 1). The lot size is approximately (2.61 acres). The site is relatively flat. No major surface erosions were observed during our subsurface excavations.

2.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

2.1 Subsurface Exploration

Our subsurface exploration consisted of one 8-inch diameter hollow stem auger boring and one 6-inch hand auger boring to a maximum depth of 51.5 feet at the locations shown on the attached Site Plan, Figure 2. The drilling of the boring was supervised and logged by a CLE's engineer. Relatively undisturbed and bulk samples were collected for laboratory testing. In addition, Standard Penetration Tests (SPT) was also conducted during drilling of the boring. Boring logs are presented in Appendix A.

2.2 Laboratory Testing

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

3.0 SUMMARY OF GEOTECHNICAL CONDITIONS

3.1 Soil Conditions

The onsite near surface soils consist of dark gray clayey sand (SC). In general, these soils exist in loose to medium dense and slightly moist to moist condition. Underlying the surface soils, medium brown sandy silt and clayey silt (ML), medium grained sand and silty sand mixtures (SP-SM) and fine grained silty sand (SM) were disclosed in the borings to the depths explored (51.5 feet below the existing ground surface). These soils exist in stiff and medium dense to dense and moist to wet conditions. In general, the soils become denser as depth increases.

3.2 Groundwater

Groundwater level was encountered in the boring at the depth of 25 feet during our subsurface investigation. Based on our review of the "Historically Highest Ground Water Contours and Borehole Log Data Locations, Whittier Quadrangle", by CGS (previously CDMG) Open File Report 98-28, it is estimated that the highest historical ground water level is approximately 5 to 8 feet below the existing grade.

3.3 Faulting

Based on our study, there are no known active faults crossing the property. The nearest known active regional fault is the Puente Hills (Coyote Hills) Fault zones located at 1.4 miles from the site.

3.4 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 Puente Hills (Coyote Hills) fault zones are considered to have the most significant effect to the site from a design standpoint.

TABLE 1

Characteristics and Estimated Earthquakes for Regional Faults

Fault Name	**Approximate Distance to Site (mile)**	Maximum Magnitude Earthquake (Mw)
Puente Hills (Coyote Hills)	1.4	6.9
Puente Hills (Santa Fe Springs)	2.7	6.7
Puente Hills (LA)	7.2	7.0
Elsinore;W+GI+T+J+CM	7.3	7.8
Newport Inglewood Conn. alt 2	8.2	7.5
Newport-Inglewood, alt 1	8.3	7.2
Newport Inglewood Conn. alt 1	8.3	7.5
Elysian Park (Upper)	13.1	6.7
Palos Verdes	14.6	7.3
Palos Verdes Connected	14.6	7.7
San Joaquin Hills	14.7	7.1
San Jose	15.0	6.7
Raymond	16.9	6.8
Verdugo	18.1	6.9
Hollywood	19.2	6.7

Reference: 2008 National Seismic Hazard Maps-Source Parameters

3.5 Estimated Earthquake Ground Motions

In order to estimate the seismic ground motions at the subject site, CLE 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 0.662g and 0.402g, respectively (2008 USGS Interactive Deaggregation). Site modified peak ground acceleration (PGA_M), corresponding to USGS Design Map Summary Report, ASCE 7-16 Standard, is 0.803g.

4.0 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 7–16 Standard (CBC 2019), the following seismic related values may be used.

Seismic Parameters (Latitude: 33.88106350, Longitude: -118.05821870)	Site Class "D"
Mapped 0.2 Sec Period Spectral Acceleration **S_s**	1.567 g
Mapped 1.0 Sec Period Spectral Acceleration **S_1**	0.559g
Site Coefficient for Site Class "D", **F_a**	1.2
Site Coefficient for Site Class "D", **F_v**	1.741
Maximum Considered Earthquake Spectral Response Acceleration Parameter at 0.2 Second, **S_{MS}**	1.881
Maximum Considered Earthquake Spectral Response Acceleration Parameter at 1.0 Second, **S_{M1}**	0.974g
Design Spectral Response Acceleration Parameters for 0.2 sec, **S_{DS}**	1.254g
Design Spectral Response Acceleration Parameters for 1.0 Sec, **S_{D1}**	0.649g

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

5.0 SEISMIC HAZARDS

5.1 Liquefaction Potential

Liquefaction is the transformation of a granular material from a solid to a liquid state as a result of increasing pore-water pressure. The material will then lose 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 soils was evaluated using the computer program "LiqSVs 2.2.18-SPT & Vs Liquefaction Assessment Software", by GEOLOGISMIKI Geotechnical Software, the subsurface data from Boring B-1, the design earthquake (M =6.9), and ground acceleration of 0.803g (PGA_M, site modified peak ground acceleration, ASCE 7-16 Standard). The total unit weight used for the onsite soils is 120 pcf. The calculated ground water level is raised to the depth of 5 feet below the existing ground surface. Conversion from California modified split spoon to field SPT blow counts is 0.67 (County of L.A. GS045.0 October 1, 2014). Based on the analyses presented on the enclosed Appendix C, the factor of safety is less than 1.30 for the onsite soils layer at the depth of 20, 30 and 45 feet.

Based on the laboratory test results on fine grained soils, for B-1 @ 5, 10, 15, 40 and 50 feet, the saturated moisture content of the encountered clayey soils is less than 80 percent of liquid limit when PI is less than 18 **(**Bray and Sancio 2006 and County of L.A. GS 045.0 October 1, 2014, if PI is less than 18 and wc/LL is less than 0.80, the clayey soil is not susceptible to liquefaction).

Based on the saturation formula : $S(\gamma_W/\gamma_D - 1/G)$, and assumed G value of 2.70, The saturated moisture content and calculated MC/LL are utilized in the following table:

Sample No.	USCS Class. ASTM D2488	Dry Density (pcf)	Calculated Saturated Moisture Content (%)	Liquid Limit ASTM D4318 LL	Plastic Limit ASTM D4318 PL	Plasticity Index ASTM D4318 PI	Saturated M.C. Divided by Liquid Limit
B-1 @ 5'	ML	101.7	24.3	33	25	8	0.74<0.80
B-1 @ 10'	ML	104.6	22.6	31	23	8	0.73<0.80
B-1 @ 15'	ML	98.3	26.4	34	25	9	0.78<0.80
B-1 @ 40'	ML	99.9	25.4	33	26	7	0.77<0.80
B-1 @ 50'	ML	100.8	24.9	35	26	9	0.72<0.80

According to procedures referenced in SP117A, (Guideline for Evaluating and Mitigating Seismic Hazards in California), and based on our laboratory Atterberg Limits, saturated moisture content and liquid and plasticity index of clayey and silty soils material, it is our opinion that the encountered clayey soil is 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 GEOLOGISMIKI Geotechnical Software, total seismic vertical settlement for saturated sand is **2.47 inches** and differential settlement is **1.65 inches.**

5.3 Surface Manifestation of Liquefaction

One of the most dramatic causes of damage to structures during earthquakes has been the development of liquefaction in saturated sandy soils, manifested either by the formation of boils and mud-spouts at the ground surface, by seepage of water through ground cracks. Based on the 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.

5.4 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 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.

5.7 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.

6.0 CONCLUSIONS

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

6.2 Faulting and Seismicity

The subject site, like the rest of Southern California, is located within a seismically active region as a result of being located near the active margin between the North American and Pacific tectonic plates. The principal source of seismic activity is movement along the northwest-trending regional faults such as the San Andreas, San Jacinto and Elsinore fault zones. These fault systems produce approximately 5 to 35 millimeters per year of slip between the plates.

The most significant geologic hazard to be the potential for moderate to strong seismic shaking that is likely to occur at the subject site. The subject site is located in the highly seismic Southern California region within the influence of several faults that are considered to be Holocene-active or pre-Holocene faults. A Holocene-active fault is defined by the State of California as a fault that has exhibited surface displacement within the Holocene time (about the last 11,700 years). A pre-Holocene fault is defined by the State as a fault whose history of past movement is older than 11,700 years ago and does not meet the criteria for a Holocene-active fault.

These Holocene-active and pre-Holocene faults are capable of producing potentially damaging seismic shaking at the site. It is anticipated that the subject site will periodically experience ground acceleration as the result of small to moderate magnitude earthquakes. Other active faults without surface expression (blind faults) or other potentially active seismic sources that are not currently zoned and may be capable of generating an earthquake are known to be present under in the region.

The subject site is not included within any Earthquake Fault Zones as created by the Alquist-Priolo Earthquake Fault Zoning Act (CGS, 2018). Our review of geologic literature pertaining to the site area indicates that there are no known active or potentially active faults located within or immediately adjacent to the subject property. As indicated in Table 1, the Puente Hills (Coyote Hills) fault zone is considered to have the most significant effect to the site from a design standpoint.

6.3 Seismic Induced Hazard

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 layers at the depth of 20, 30 and 45 feet.

6.4 Other Secondary Hazard

<u>6.4.1 Fault Rupture</u>

There are no known active faults crossing the property and the property is not located within the Alquist-Priolo Earthquake Fault Zone. The nearest fault is Puente Hills (Coyote Hills) fault zone which is located approximately 1.4 miles from the property. Therefore, the likelihood of a fault surface rupture is considered low.

<u>6.4.2 Seiches and Tsunamis</u>

Seiches are large waves generated in enclosed bodies of water as the results of major ground shaking. The site is located within the commercial area in the incorporated area of the County of Los Angeles. No water retaining structures are located immediately adjacent to site. Earthquake induced flooding, tsunamis and seiches is considered unlikely at the site.

<u>6.4.3 Expansive Soils</u>

Soil expansion is the tendency of the soil to expand when the soil contacts with water. Our laboratory test of the onsite soils indicated that onsite soils is low in expansion potential. Foundation design and construction should be designed in accordance with the current building code report to reduce the potential of any adverse effect as the results of the expansive soils.

6.4.4 Oil Field and Historical Oil Wells

Based on our review of the Munger Map Book of the California Oil and Gas Field, no oil wells are located on the subject property or any adjacent properties. It is our opinion that the potential of the presence of the methane zone at the site is considered low. However, should it be determined that the methane study is required, a qualified consultant should be retained for the study.

6.4.5 Subsidence

Land subsidence occurs when large amounts of groundwater, oil or gas have been withdrawn from fine-grained soils. It is our understanding that large scale extraction of groundwater, oil or gas is not planned at the site. The land subsidence at site appears to be unlikely.

6.5 Excavatability

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

6.6 Groundwater

Groundwater level was encountered in the boring at the depth of 25 feet during our subsurface investigation. In our opinion, groundwater will not be a problem during the near surface construction.

7.0 RECOMMENDATIONS

7.1 Grading

7.1.1 Site Preparation

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

7.1.2 Surficial Soil Removals

Based on our field exploration, liquefaction analysis and laboratory data obtained to date, it is recommended that the surficial soils be removed to a depth of 7 feet below existing grade or 4 feet below the bottom of the footing, whichever is deeper. The existing near surface soils should also be removed at least 18 inches within the proposed driveway areas. The recommended removal should extend at least 5 feet beyond the proposed building footprint or to the limits of the existing building.

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.

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 ASTM D-1557-12.

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.

7.1.5 Seepage

Our subsurface investigation encountered the groundwater at the depth of 25 feet below the existing grade. Perched water or seepage water might be encountered during the onsite grading and construction. Should the water be encountered during grading/construction, the water should be drained and/or pumped away from the construction area. Any loose/soft soils within the construction area should be removed under the direction of the project geotechnical consultants. It is also recommended that geotexile be placed at the bottom of the excavation. Approximate two feet of 3/4 inches crushed rock may be placed on the top of the geotexile and an additional layer of geotexile shall be placed on the top of the gravel follow by the compacted fill to the design grade.

7.2 Temporary Vertical Excavation

Shoring, sloping or ABC slot cut method will be required for temporary excavation made vertically or near vertically within the proposed building pad area.

7.2.1 Temporary Sloping Excavation

Temporary excavation may be cut vertically up to 4 feet and slope back at the slope ratio of 1 to 1 (horizontal to vertical) above 4 feet. If sloping excavation is not feasible near the existing building areas, slot cut and/or shoring system may be used.

7.2.2 Slot Cut

Should the slot cut method be used for the onsite vertical excavation of no more than 7 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. 6 feet)
3. All excavations should be made under the inspection and testing of the project geotechnical consultant.
4. 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.
5. Provisions for drainage should be implemented to prevent saturation of un-shored excavations.
6. Excavate in alternative slots with each slot no wider than 6 feet in width.
7. Excavate 7 feet below existing grade at each slot, exposed competent soil then backfill per project standard.
8. Excavate the adjacent slots and repeat the above procedures to complete grading
9. All excavations should be made under the inspection and testing of the project geotechnical consultant.
10. Provisions for drainage should be implemented to prevent saturation of un-shored excavations.

7.2.3 Shoring System

Temporary vertical excavation of more than 4 feet may be supported by the shoring system. The active earth pressure to be utilized for cantilever shoring system designs may be computed as an equivalent fluid having a density of 33 pounds per cubic foot when the slope of the backfill behind the wall is level.

The resistance of the lateral loads may be provided by the passive earth pressure for the portion of the soldier piles embedded below the bottom of the excavation and the friction between the onsite soils and solider piles. Passive earth pressure for the underlain soils may be computed as an equivalent fluid pressure of 350 psf, with a maximum earth pressure of 3000 psf. An allowable coefficient of friction between soil and concrete of 0.3 may be used with the dead load forces. For the caissons spacing at 3- piles diameter apart, the passive earth pressure may be increased by 100 percent. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one third (1/3).

7.3 Foundation Design

7.3.1 Conventional Shallow Foundation (Miscellaneous Structures)

An allowable bearing value of 2000 pounds per square foot (psf) may be used for design of continuous or pad footings with a minimum of 12 or 24 inches in width, respectively. All footings should be a minimum of 30 inches deep and founded on soils approved by the project geotechnical engineer. This value may be increased by one-third when considering short duration seismic or wind loads. Resistance to the lateral loads can be assumed to be provided by the passive earth pressure and the friction between the concrete and competent soils. Passive earth pressure may be computed as an equivalent fluid pressure of 300 pcf, with a maximum earth pressure of 2000 psf. An allowable coefficient of friction between soil and concrete of 0.30 may be used with the dead load forces. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one third (1/3).

7.2.2 Foundation (Building)

Due to the liquefaction potential of the underlain soils, the proposed commercial foundation may be supported on a mat or structurally stiffened foundation system with deepen exterior footings.

For mat foundation placed at the a depth of 24 inches below the lowest adjacent grade and founded on the encountered onsite material may be designed for an allowable bearing value of 2000 pounds per square foot (psf). Mat foundations should be a minimum of 12-inches thick. Should the elastic method be used for the mat foundation design, the allowable subgrade modulus of 80 pounds per cubic inch may be used. A minimum exterior footing depth of 30 inches into approved soils is recommended for perimeter wall footings. All foundations should be reinforced in accordance with the structural design and specifications.

Resistance to the lateral loads can be assumed to be provided by the passive earth pressure and the friction between the concrete and competent soils. Passive earth pressure may be computed as an equivalent fluid pressure of 300 psf, with a maximum earth pressure of 2000 psf. An allowable coefficient of friction between soil and concrete of 0.30 may be used with the dead load forces. When combining passive pressure and frictional resistance, the passive pressure component should be reduced by one third (1/3).

The mat foundation should have sufficient stiffness and thickness to minimize foundation settlement due to the liquefaction potential of induced seismic settlement. Foundation design of widths, depths, and reinforcing are the responsibility of the project structural engineer considering the structural loads and the geotechnical conditions and occurrence of seismic induced settlement due to liquefaction as presented in this report. The reinforcement of the proposed mat foundation should be designed by the project structural engineer.

7.2.3 Post Tension Foundation System

As an alternative to the foundation as recommended above, post tension footing and slab may be used. Post tension footing and slab should be designed by the project structural engineer. CLE can provide the weighted and effective plasticity index upon request.

7.2.4 Lateral Pressures

Passive earth pressure may be computed as an equivalent fluid pressure of 300 pounds per cubic foot, with a maximum earth pressure of 2000 pounds per square foot. 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.

7.2.5 Foundation 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 **2.47 inches** and **1.67 inches** respectively.

7.2.6 Foundation Construction (Commercial Building)

It is estimated that the combined static and liquefaction induced total and differential settlement will be on the order of 2.97 inches and 1.92 inches, respectively. It is recommended that the proposed commercial building may be supported on the mat or structurally stiffened foundation system. The slab thickness, foundation width, depth and reinforcement should be designed by the project structural engineer. It is recommended that mat foundation should be a minimum of 12 inches in thickness from the geotechnical engineering viewpoint.

It is anticipated that the entire structure will be underlain by onsite soils of medium expansion potential (EI=21). In accordance with Section 1808.6.4 of the 2016 California Building Code the soil should be stabilized by presaturation and all footings for miscellaneous structures and slabs should be constructed as follows:

All footings for miscellaneous structures should be founded at a minimum depth of 30 inches below the lowest adjacent ground surface. All continuous footings should have at least two No. 4 reinforcing bars placed within four inches of the top of the footing and two No. 4 bars shall be placed between 3 inches and 4 inches of the bottom of the footing. Foundations for exterior walls and interior bearing walls shall be tied to the floor slabs by reinforcing bars (dowels) having a diameter of not less than ½ inch (No. 4 bar) reinforcing bars and spaced at intervals not exceeding 16 inches on center. The reinforcing bars extend at least 40 bar diameters into the footings and the slabs.

Presaturation of soils is recommended for concrete slab areas. The moisture condition of each slab area should be 120 percent or greater of optimum moisture content to a depth of 24 inches below slab grade prior to pouring of slabs. Presaturation may be facilitated by maintaining the water content prior to foundation construction by periodic spraying and by slowly adding additional water after foundations are in.

7.2.7 Concrete Flatwork

Concrete flatwork should be a minimum of 5 inches thick and reinforced with a minimum of No. 3 reinforcing bar spaced 16-inch each way or its equivalent. All slab reinforcement should be supported to ensure proper positioning during placement of concrete.

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.

7.3 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 CORROSION POTENTIAL

Chemical laboratory tests were conducted on the existing onsite near surface materials sampled during CLE'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 2019 CBC and ACI 318-19, 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.

9.0 CONCRETE PAVEMENT SECTION

Concrete Pavement section for Truck Apron is as follows:

Slab Thickness for Truck Loading Areas; …….. **…………...7"/8" (Concrete/Class 2)**

Concrete should have a minimum modulus of rupture of **550 psi. or f'c of about 3000 psi.** Concrete slab should be reinforced with a minimum of No. 3 reinforcing bar spaced 16-inch each way or its equivalent. The subgrade materials should be brought to near optimum moisture content, then compacted to at least 90 percent of ASTM D-1557-12. The Class 2 aggregate base materials should be brought to near optimum moisture content, then compacted to at least 95 percent of ASTM D-1557-12. Adjacent landscaping should be graded to drain into local area drain and away from the pavement subgrade material.

10.0 INSPECTION

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

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

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 the grading process to discuss specific procedures and scheduling.

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**.**

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 Wittier 7.5-minutes Quadrangle, Los Angeles County, California Seismic Hazard Zone report 037.

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

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

Los Angeles County, Department of Public Works, "Review of Geotechnical Reports Addressing Liquefaction", File NO. GME-3 dated February 24, 2009.

"Department of Building and Safety, City of Los Angeles, California", 2014 Los Angeles Building Code Requirements, Geology and Soils Engineering, dated July 16, 2014.

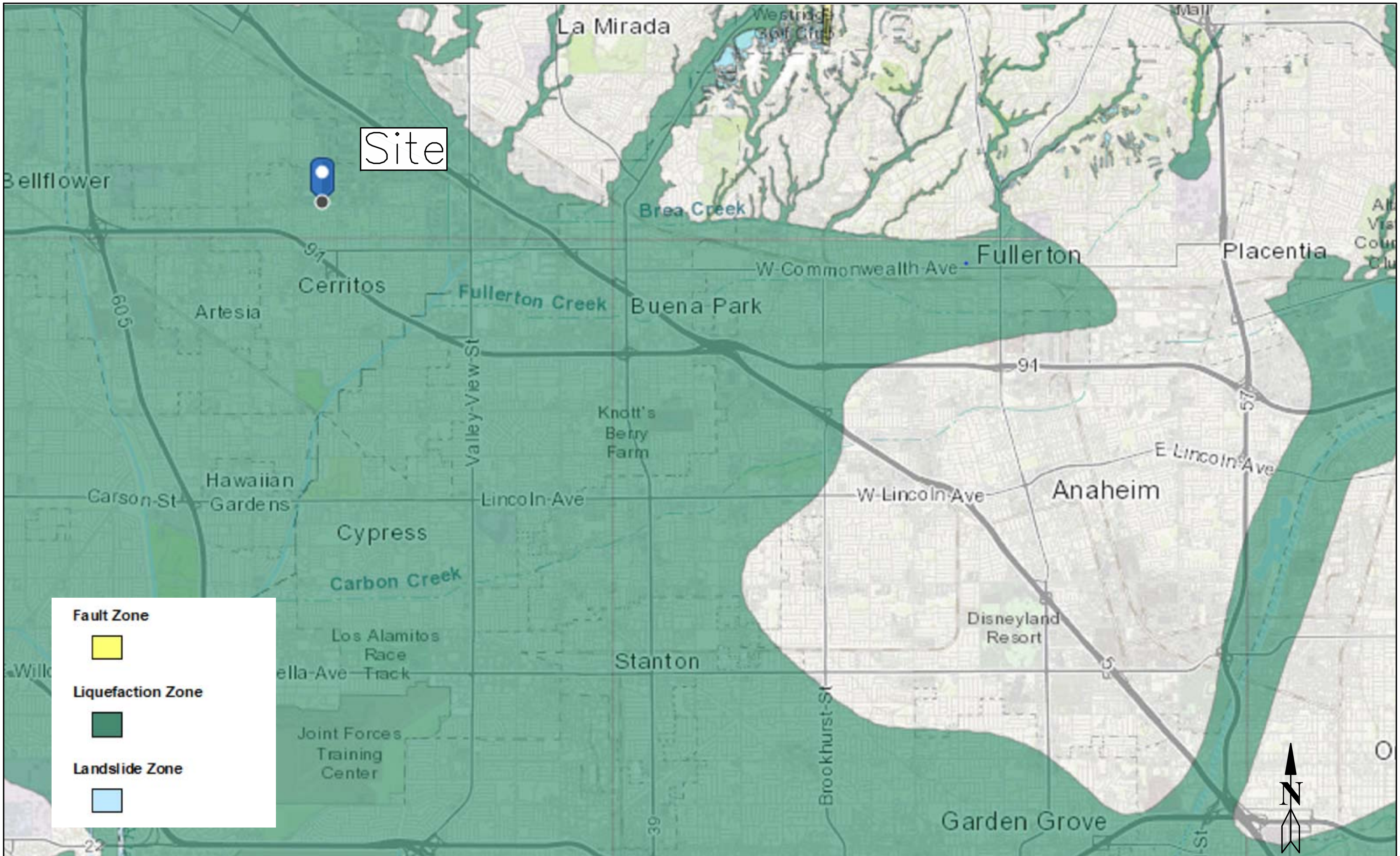
Los Angeles County, Department of Public Works, "Geotechnical and Material Engineering Division, Liquefaction /Lateral Spread ",GS45.0, Revised dated October 1, 2014

"LiqSVs 2.2.18-SPT & Vs Liquefaction Assessment Software", by: GEOLOGISMIKI Geotechnical Software.

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

"Earthquake Hazards Program, Seismic Design Maps and tools", ASCE 7-16 Standard

https://geohazards.usgs.gov/deaggint/2008/
http://eqint.cr.usgs.gov/deaggint/2008/index.php
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/



Maps modified from "Seismic Hazard Zone" by California Geological Survey

SCALE AS SHOWN

CAL LAND
ENGINEERING & ASSOCIATES, Inc.

Site Location Map
12881 166th Street, Cerritos, California

Project No.: 22-009-050

FIGURE 1

Maps modified from "Historically Highest Ground Water Contours and Borehole Log Data Locations, Whittier Quadrangle" by CGS

SCALE AS SHOWN

CAL LAND ENGINEERING & ASSOCIATES, Inc.	Site Locational Map Historical Highest Groundwater Level 12881 166th Street, Cerritos, California	Project No.: 22-009-050 FIGURE 1a

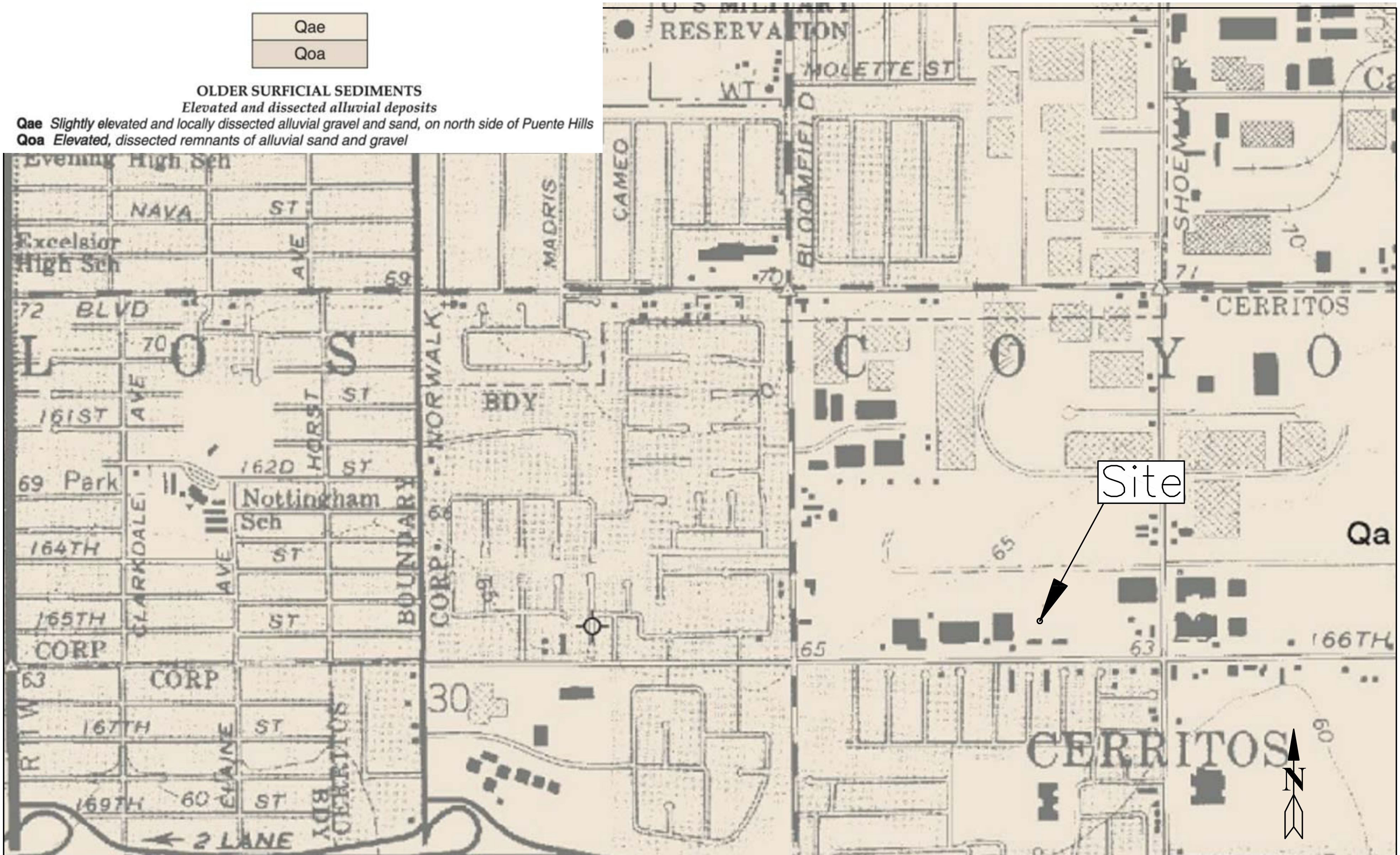
Qae	
Qoa	

OLDER SURFICIAL SEDIMENTS

Elevated and dissected alluvial deposits

Qae *Slightly elevated and locally dissected alluvial gravel and sand, on north side of Puente Hills*

Qoa *Elevated, dissected remnants of alluvial sand and gravel*



SCALE AS SHOWN

Maps modified from "Geologic map of the Whittier and La Habra quadrangles (western Puente Hills) Los Angeles and Orange Counties, California by Dibblee, T.W., and Ehrenspeck, H.E. 2001

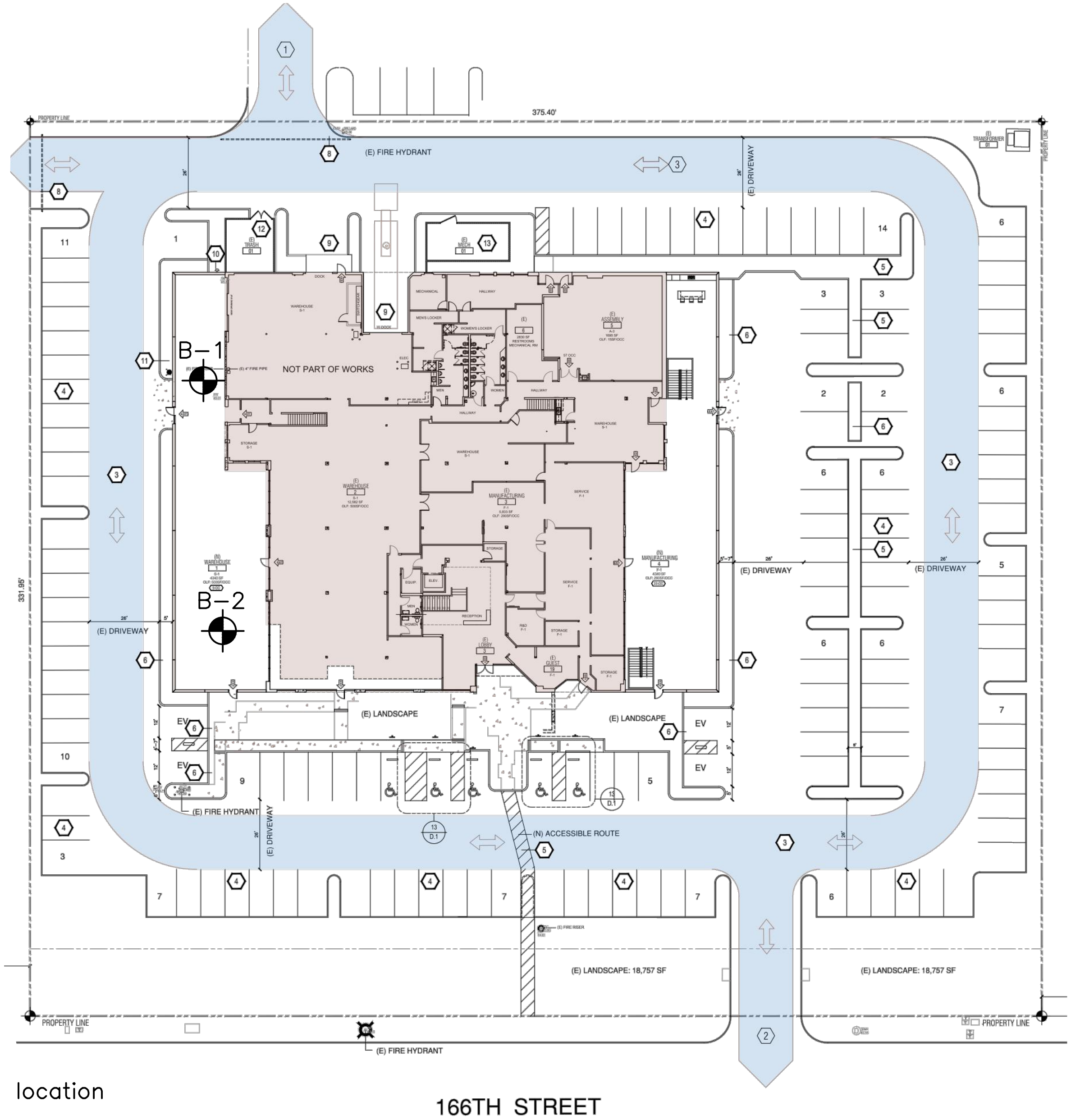
CAL LAND
ENGINEERING & ASSOCIATES, Inc.

Regional Geology Map

12881 166th Street, Cerritos, California

Project No.: 22-009-050

FIGURE 1b



LEGEND

B–1 Approximate boring location

SCALE: 1" = 64'

CAL LAND ENGINEERING & ASSOCIATES, Inc.

SITE PLAN

12881 166th Street, Cerritos, California

Project No.: 22-009-050

FIGURE 2

APPENDIX A
FIELD INVESTIGATION

Subsurface conditions were explored by drilling one 8-inch diameter hollow stem auger boring and one 6-inch hand auger boring to a maximum depth of 51.5 feet at approximate locations shown on the enclosed site plan, Figure 2. Upon completion of drilling, the holes were backfilled with onsite soils that were removed from the excavations.

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

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

CalLand Engineering and Associates, Inc.

BORING LOG B-1

PROJECT LOCATION: 12881 166th Street, Cerritos, California
PROJECT NO.: 22-009-050

DATE DRILLED: 8/8/2022
SAMPLE METHOD: Hollow Stem
ELEVATION: N/A
LOGGED BY: HF

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
	B			SC		8.8	Fill: Clayey sand, fine grained, dark gray, slightly moist, loose Percent of Fines: 36.5
		R	8 7 6	SC	101.2	10.9	Clayey sand, fine grained, dark gray, moist, loose to medium dense
5		R	3 5 10	ML	101.7	17.6	Natural: Sandy silt, medium brown, moist, firm Percent of Fines: 87.1, LL=33, PL=25, PI=8
10		R	5 8 11	ML	104.6	12.0	Sandy silt, medium brown, moist, stiff Percent of Fines: 68.2, LL= 31, PL=23, PI=8
15		R	5 6 8	ML	98.3	23.2	Clayey silt, medium brown, moist, stiff Percent of Fines: 70.3, LL= 34, PL=25, PI= 9
20		R	11 17 22	SP/ SM	109.1	19.6	Sand and silty sand, medium grained, grayish brown, very moist, medium dense Percent of Fines: 11.9
25	▼	R	15 26 28	SM	106.4	24.3	Groundwater @ 25' Silty sand, fine grained, dark gray, wet, dense. Percent of Fines: 23.9,
30		S	6 12 14	SM		25.7	Silty sand, fine grained medium brown, wet, medium dense Percent of Fines: 27.6
35		S	1 10 36	SM		19.8	Silty sand, fine grained, medium brown, wet, dense Percent of Fines: 48.3

CalLand Engineering and Associates, Inc.

BORING LOG B-1

PROJECT LOCATION: 12881 166th Street, Cerritos, California

PROJECT NO.: 22-009-050

DATE DRILLED: 8/8/2022

SAMPLE METHOD: Hollow Stem

ELEVATION: N/A

LOGGED BY: HF

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		R	4 5 12	ML	99.9	26.1	Clayey silt, gray brown, wet, stiff Percent of Fines: 66.9, LL= 33, PL= 26, PI=7
45		R	12 18 23	SM	108.2	20.6	Silty sand, fine grained, dark grayish brown, wet, medium dense Percent of Fines: 36.1
50		R	3 8 12	ML	100.8	25.6	Clayey silt, dark gray, wet, stiff Percent of Fines: 71.2, LL=35, PL= 26, PI= 9
55							Total Depth: 51.5 feet Groundwater @ 25 feet Hole Backfilled Hammer Driving Weight: 140 lbs Hammer Driving Height: 30 inches
60							
65							
70							
75							

CalLand Engineering and Associates, Inc.

BORING LOG B-2

PROJECT LOCATION: 12881 166th Street, Cerritos, California
PROJECT NO.: 22-009-050

DATE DRILLED: 8/8/2022
SAMPLE METHOD: Hand Auger
ELEVATION: N/A
LOGGED BY: HF

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

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/12"	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Description of Material
							Fill:
		R	12	ML	103.1	12.2	Sandy silt, grayish brown to dark brown, slightly moist, firm
							Natural:
5		R	21	ML	104.3	14.6	Sandy silt, medium brown, moist, firm to medium stiff, trace of clay
10		R	44	SC	107.6	11.3	Clayey sand, fine grained, olive brown, medium dense to dence, trace of silt
15							Total Depth: 11.0 feet No Groundwater Hole Backfilled Hammer Driving Weight: 32 lbs Hammer Driving Height: 48 inches
20							
25							
30							
35							

APPENDIX B

LABORATORY TESTING

During the subsurface exploration, CLE 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 CLEI'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'	21	Low

Corrosion Potential

Chemical laboratory tests were conducted on the existing onsite near surface materials sampled during CLE'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'-4'	8.90	170	0.0120	2,200

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'	36.5
B-1 @ 5'	87.1
B-1 @ 10'	68.2
B-1 @ 15'	70.3
B-1 @ 20'	11.9
B-1 @ 25'	23.9
B-1 @ 30'	27.6
B-1 @ 35'	48.3
B-1 @ 40'	66.9
B-1 @ 45'	36.1
B-1 @ 50'	71.2

Atterberg Limits

Laboratory Atterberg Limits tests were conducted on the existing onsite materials sampled during CLE'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 @ 5'	ML	33	25	8
B-1 @ 10'	ML	31	23	8
B-1 @ 15'	ML	34	25	9
B-1 @ 40'	ML	33	26	7
B-1 @ 50'	ML	35	26	9

APPENDIX C

RESULTS OF LIQUEFACTION ANALYSES

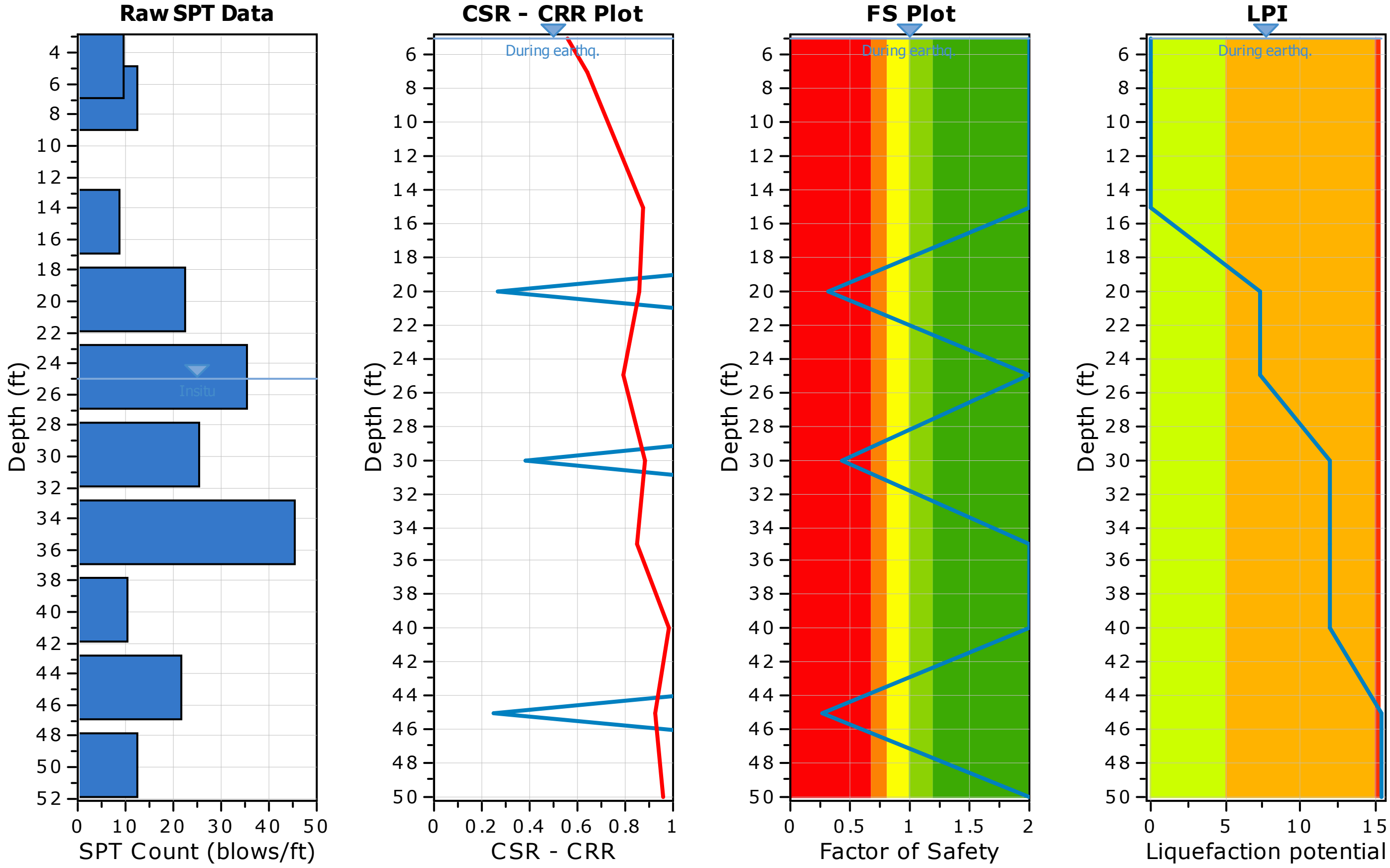
SPT BASED LIQUEFACTION ANALYSIS REPORT

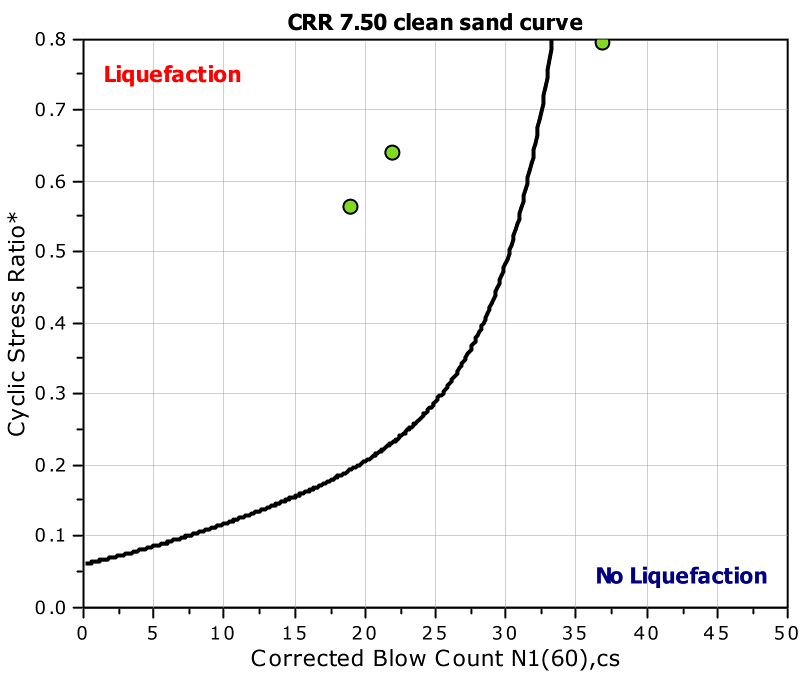
Project title : 22-009-050 **SPT Name: SPT #1**

Location : 12281 166th Street, Cerritos, CA

:: Input parameters and analysis properties ::

Analysis method:	Boulanger & Idriss, 2014	G.W.T. (in-situ):	25.00 ft
Fines correction method:	Boulanger & Idriss, 2014	G.W.T. (earthq.):	5.00 ft
Sampling method:	Standard Sampler	Earthquake magnitude M_w:	6.90
Borehole diameter:	150mm	Peak ground acceleration:	0.81 g
Rod length:	3.30 ft	Eq. external load:	0.00 tsf
Hammer energy ratio:	1.00		





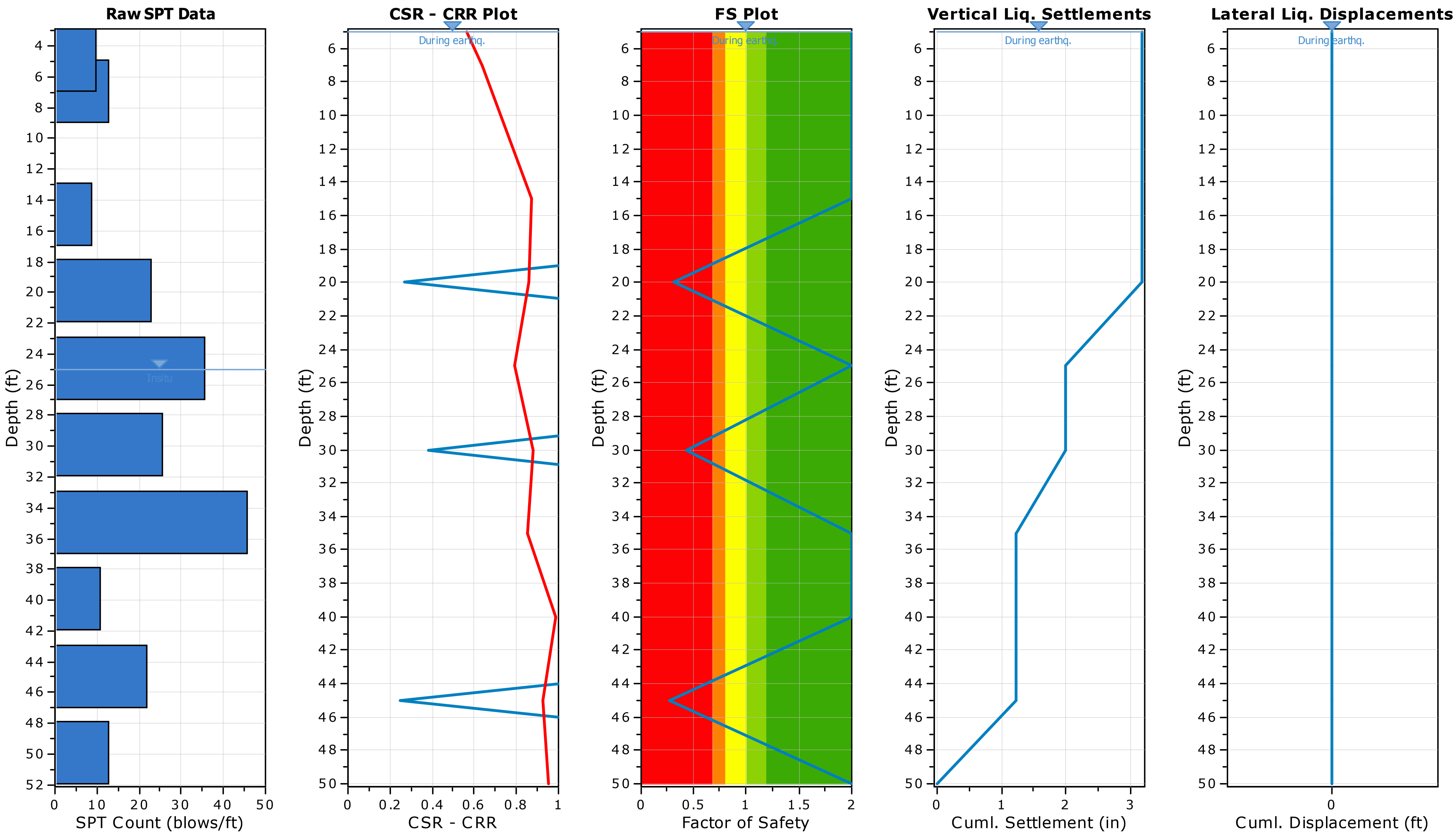
F.S. color scheme

- 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

:: Overall Liquefaction Assessment Analysis Plots ::



:: Field input data ::

Test Depth (ft)	SPT Field Value (blows)	Fines Content (%)	Unit Weight (pcf)	Infl. Thickness (ft)	Can Liquefy
5.00	10	87.10	120.00	5.00	No
7.00	13	68.20	120.00	2.00	No
15.00	9	70.30	120.00	8.00	No
20.00	23	11.90	120.00	5.00	Yes
25.00	36	23.90	120.00	5.00	Yes
30.00	26	27.60	120.00	5.00	Yes
35.00	46	48.30	120.00	5.00	Yes
40.00	11	66.90	120.00	5.00	No
45.00	22	36.10	120.00	5.00	Yes
50.00	13	71.20	120.00	5.00	No

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)	m	C_N	C_E	C_B	C_R	C_S	$(N_1)_{60}$	FC (%)	$\Delta(N_1)_{60}$	$(N_1)_{60cs}$	$CRR_{7.5}$
5.00	10	120.00	0.30	0.00	0.30	0.42	1.69	1.00	1.05	0.75	1.00	13	87.10	5.52	19	4.000
7.00	13	120.00	0.42	0.00	0.42	0.41	1.46	1.00	1.05	0.80	1.00	16	68.20	5.58	22	4.000
15.00	9	120.00	0.90	0.00	0.90	0.49	1.08	1.00	1.05	0.85	1.00	9	70.30	5.57	15	4.000
20.00	23	120.00	1.20	0.00	1.20	0.41	0.95	1.00	1.05	0.95	1.00	22	11.90	2.03	24	0.268
25.00	36	120.00	1.50	0.00	1.50	0.32	0.89	1.00	1.05	0.95	1.00	32	23.90	4.98	37	4.000
30.00	26	120.00	1.80	0.16	1.64	0.38	0.85	1.00	1.05	1.00	1.00	23	27.60	5.25	28	0.384
35.00	46	120.00	2.10	0.31	1.79	0.26	0.87	1.00	1.05	1.00	1.00	42	48.30	5.61	48	4.000
40.00	11	120.00	2.40	0.47	1.93	0.51	0.73	1.00	1.05	1.00	1.00	8	66.90	5.58	14	4.000
45.00	22	120.00	2.70	0.62	2.08	0.43	0.75	1.00	1.05	1.00	1.00	17	36.10	5.53	23	0.249
50.00	13	120.00	3.00	0.78	2.22	0.50	0.69	1.00	1.05	1.00	1.00	9	71.20	5.57	15	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)
m:	Stress exponent normalization factor
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
$\Delta(N_1)_{60}$	Equivalent clean sand adjustment
$N_{1(60)cs}$:	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_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
5.00	120.00	0.30	0.00	0.30	0.99	1.00	0.522	1.45	19	1.10	0.476	1.10	0.731	2.000

:: 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_{max}	$(N_1)_{60cs}$	MSF	$CSR_{eq,M=7.5}$	K_{sigma}	CSR^*	FS
7.00	120.00	0.42	0.06	0.36	0.98	1.00	0.608	1.58	22	1.12	0.541	1.10	0.832	2.000
15.00	120.00	0.90	0.31	0.59	0.95	1.00	0.766	1.32	15	1.07	0.717	1.07	1.138	2.000
20.00	120.00	1.20	0.47	0.73	0.93	1.00	0.799	1.67	24	1.14	0.699	1.06	1.117	0.312
25.00	120.00	1.50	0.62	0.88	0.90	1.00	0.811	2.20	37	1.26	0.645	1.06	1.033	2.000
30.00	120.00	1.80	0.78	1.02	0.87	1.00	0.810	1.88	28	1.19	0.682	1.01	1.144	0.436
35.00	120.00	2.10	0.94	1.16	0.84	1.00	0.801	2.20	48	1.26	0.637	0.97	1.108	2.000
40.00	120.00	2.40	1.09	1.31	0.81	1.00	0.787	1.29	14	1.06	0.741	0.98	1.282	2.000
45.00	120.00	2.70	1.25	1.45	0.79	1.00	0.769	1.62	23	1.13	0.678	0.95	1.204	0.269
50.00	120.00	3.00	1.40	1.60	0.76	1.00	0.749	1.32	15	1.07	0.702	0.95	1.242	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
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: 1.30

:: Liquefaction potential according to Iwasaki ::

Depth (ft)	FS	F	wz	Thickness (ft)	I_L
5.00	2.000	0.00	9.24	2.00	0.00
7.00	2.000	0.00	8.93	2.00	0.00
15.00	2.000	0.00	7.71	8.00	0.00
20.00	0.312	0.69	6.95	5.00	7.29
25.00	2.000	0.00	6.19	5.00	0.00
30.00	0.436	0.56	5.43	5.00	4.67
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	0.269	0.73	3.14	5.00	3.50
50.00	2.000	0.00	2.38	5.00	0.00

Overall potential I_L : 15.45

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 & Lateral displ.acements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60cs}$	γ_{lim} (%)	F_α	FS_{liq}	γ_{max} (%)	e_v weight factor	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)
5.00	19	0.00	0.00	2.000	0.00	0.00	0.00	5.00	0.000	0.00
7.00	22	0.00	0.00	2.000	0.00	0.00	0.00	2.00	0.000	0.00
15.00	15	0.00	0.00	2.000	0.00	0.00	0.00	8.00	0.000	0.00
20.00	24	10.02	0.29	0.312	10.02	1.00	1.97	5.00	1.181	0.00

:: Vertical & Lateral displ.acements estimation for saturated sands ::

Depth (ft)	$(N_1)_{60cs}$	γ_{lim} (%)	F_α	FS_{liq}	γ_{max} (%)	e_v weight factor	e_v (%)	dz (ft)	S_{v-1D} (in)	LDI (ft)
25.00	37	1.56	-0.58	2.000	0.00	1.00	0.00	5.00	0.000	0.00
30.00	28	6.08	0.04	0.436	6.08	1.00	1.29	5.00	0.777	0.00
35.00	48	0.09	-1.43	2.000	0.00	1.00	0.00	5.00	0.000	0.00
40.00	14	0.00	0.00	2.000	0.00	0.00	0.00	5.00	0.000	0.00
45.00	23	11.27	0.35	0.269	11.27	1.00	2.04	5.00	1.227	0.00
50.00	15	0.00	0.00	2.000	0.00	0.00	0.00	5.00	0.000	0.00

Cumulative settlements: 3.184 0.00

Abbreviations

γ_{lim}: Limiting shear strain (%)
F_α/N: Maximun shear strain factor
γ_{max}: Maximum shear strain (%)
e_v:: Post liquefaction volumetric strain (%)
S_{v-1D}: Estimated vertical settlement (in)
LDI: Estimated lateral displacement (ft)

References

- Ronald D. Andrus, Hossein Hayati, Nisha P. Mohanan, 2009. Correcting Liquefaction Resistance for Aged Sands Using Measured to Estimated Velocity Ratio, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 135, No. 6, June 1

- Boulanger, R.W. and Idriss, I. M., 2014. CPT AND SPT BASED LIQUEFACTION TRIGGERING PROCEDURES. DEPARTMENT OF CIVIL & ENVIRONMENTAL ENGINEERING COLLEGE OF ENGINEERING UNIVERSITY OF CALIFORNIA AT DAVIS

- Dipl.-Ing. Heinz J. Priebe, Vibro Replacement to Prevent Earthquake Induced Liquefaction, Proceedings of the Geotechnique-Colloquium at Darmstadt, Germany, on March 19th, 1998 (also published in Ground Engineering, September 1998), Technical paper 12-57E

- Robertson, P.K. and Cabal, K.L., 2007, Guide to Cone Penetration Testing for Geotechnical Engineering. Available at no cost at http://www.geologismiki.gr/

- Youd, T.L., Idriss, I.M., Andrus, R.D., Arango, I., Castro, G., Christian, J.T., Dobry, R., Finn, W.D.L., Harder, L.F., Hynes, M.E., Ishihara, K., Koester, J., Liao, S., Marcuson III, W.F., Martin, G.R., Mitchell, J.K., Moriwaki, Y., Power, M.S., Robertson, P.K., Seed, R., and Stokoe, K.H., Liquefaction Resistance of Soils: Summary Report from the 1996 NCEER and 1998 NCEER/NSF Workshop on Evaluation of Liquefaction Resistance of Soils, ASCE, Journal of Geotechnical & Geoenvironmental Engineering, Vol. 127, October, pp 817-833

- Zhang, G., Robertson. P.K., Brachman, R., 2002, Estimating Liquefaction Induced Ground Settlements from the CPT, Canadian Geotechnical Journal, 39: pp 1168-1180

- Zhang, G., Robertson. P.K., Brachman, R., 2004, Estimating Liquefaction Induced Lateral Displacements using the SPT and CPT, ASCE, Journal of Geotechnical & Geoenvironmental Engineering, Vol. 130, No. 8, 861-871

- Pradel, D., 1998, Procedure to Evaluate Earthquake-Induced Settlements in Dry Sandy Soils, ASCE, Journal of Geotechnical & Geoenvironmental Engineering, Vol. 124, No. 4, 364-368

- R. Kayen, R. E. S. Moss, E. M. Thompson, R. B. Seed, K. O. Cetin, A. Der Kiureghian, Y. Tanaka, K. Tokimatsu, 2013. Shear-Wave Velocity–Based Probabilistic and Deterministic Assessment of Seismic Soil Liquefaction Potential, Journal of Geotechnical and Geoenvironmental Engineering, Vol. 139, No. 3, March 1

SLOT CUT CALCULATIONS
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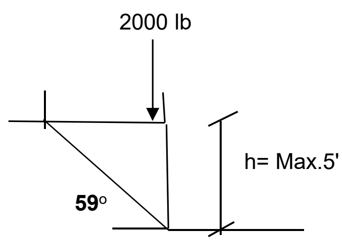

Surcharge =	2000	lb	
α (Failure Surface inclination) =	60	deg	
γ m =	120.0	pcf	
ϕ =	28	deg	
C =	150	psf	
Ko =	1-SIN(ϕ)	0.53	
H (Height) =	7	ft	
d (Slot Width) =	6	ft	
b=	Height/TAN(α)	4.0	ft
A (Side Area) =	1/2(H)(b)	14.1	ft^2
Δ F = Side Shear =	A(1/2*γ_{m*H*} K_o*TAN(ϕ)+C) =	3797.6	lb
W (weight of soil + surcharge) =	A*γ_m + Surcharge =	3697.4	lb

F.S. = $\dfrac{d*[W*COS^2\alpha TAN(\phi) + Cb] + 2\,\Delta f}{d*(WSIN\alpha COS\alpha)}$ = **1.5**