Ninyo & Moore
Geotechnical & Environmental Sciences Consultants

35+ YEARS

December 14, 2022
Project No. 208829001

Ms. Shelby Gonzales
HR Green
1260 Corona Pointe Court, Suite 305
Corona, California 92879

Subject: Geotechnical Review for GP22-036
Proposed Industrial Warehouse Building
2240 Fleetwood Drive
City of Jurupa Valley, California

References: Cannon, 2022, Rough Grading Plans for 2240 Fleetwood Drive, HYB, LLC, - Fleetwood Industrial, Sheets 1-5, dated November 28.

Environmental Geotechnology Laboratory, Inc., (EGL) 2022, Report of Geotechnical Engineering Investigation, Proposed Industrial Warehouse Building and Associated Structures, APN: 178-350-024, 029 & 030, 2240 Fleetwood Drive, City of Jurupa Valley, County of Riverside, California, EGL Project No.: 22-257-004L, dated October 31.

Dear Ms. Gonzales:

In accordance with your request, we have performed a geotechnical review of the referenced geotechnical report prepared by Environmental Geotechnology Laboratory, Inc. (EGL, 2022) and grading plans prepared by Cannon (2022), pertaining to the proposed industrial warehouse development project in the city of Jurupa Valley, California. Our review is based generally on the standards presented in the 2019 California Building Code, the County of Riverside Technical Guidelines for Review of Geotechnical and Geologic Reports, the Riverside County Flood Control Water Conservation District Design Handbook for Low Impact Development Best Management Practices, and current standards of practice.

We have noted items that should be addressed by the geotechnical consultant for the project. Our comments are presented below:

1. The consultant described the soils encountered in the borings, but did not indicate if any fill was observed either in the borings or elsewhere on the site. If fill materials were encountered in the borings, the contacts between the fill and native soils should be shown on the boring logs. If fill was observed in areas that did not have exploratory borings, the consultant should describe the fill and show where it was observed on the Site Plan.

2. The consultant prepared geologic Cross Section A-A', performed a slope stability analysis of the cross section, and the analyses indicated adequate factors of safety under static and pseudo-static conditions. The cross section was drawn in a northwest-southeast direction and through the corner of the proposed warehouse building that will be in close proximity to the adjacent slope that descends to the Santa Ana River. The consultant should describe the planned grading and construction in the area to the south of the proposed warehouse building near the area where

boring B-3 was drilled. The consultant should address the stability of the easterly descending slope that appears to be much steeper than the slope that was evaluated and if the stability of the slope could have an impact to the proposed improvements in this area, if any.

3. In Section 8.1.2 the consultant stated that a detailed grading plan was not available at the time they prepared their report. The consultant should review the referenced grading plans prepared by Cannon and provide an updated description of the planned grading and construction, including a summary of the proposed cut and fill depths, and the heights and slope ratios of new slopes cut and fill slopes. Updated conclusions and recommendations should be provided, as appropriate.

4. The grading plans indicate that 2:1 and 3:1 (horizontal to vertical) slopes are proposed in the southeast area of the site, portions of which are in close proximity to the existing slope that descends to the Santa Ana River. The proposed slope grading should be incorporated into the slope stability analyses and the consultant should provide recommendations for slope grading, including construction of fill keys and slope face compaction.

5. In Sections 8.1.2 and 8.1.4, the consultant recommends that if expansive materials are encountered in slab area that the expansive material be replaced with sandy import material. The consultant should provide a recommendation for the depth below and the lateral distance beyond the slabs that the expansive material should be removed and replaced with granular soils.

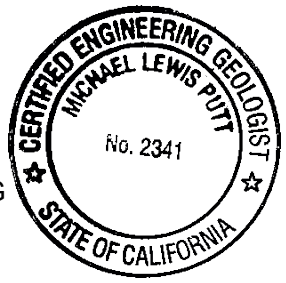
6. In Section 13.0 the consultant states that an infiltration basin within the natural soil is not feasible due to poor infiltration rates in the clayey soil. The consultant suggested that the granular materials encountered in borings B-1 and B-2 may be more suitable for infiltration, but that additional percolation testing would be needed. The area where boring B-1 was drilled is relatively close to the approximately 60 feet high slope that descends to the Santa Ana River. In addition to the additional percolation testing, the consultant should address what impact stormwater infiltration may have on the adjacent slope that descends to the river and provide a recommendation for how far away the basin should be setback from the slope. Updates to the slope stability analyses should be provided, as appropriate.

7. Some of the USCS soil types shown on the direct shear and consolidation test result pages do not match the USCS soil types shown on the boring logs. The boring logs and/or lab test result should be updated accordingly.

8. The consultant should review future development plans including the grading plans that were submitted to the City and any updates to the plans, and provide updated recommendations if needed. The consultant should also review the foundation plans and the landscape structure plans and provide updated geotechnical recommendations, as appropriate.

Sincerely,
NINYO & MOORE

Michael L. Putt, PG, CEG
Principal Geologist



Madan Chirumalla, PE, GE
Principal Engineer



MLP/MAC/mlc

Environmental Geotechnology Laboratory, Inc.

October 31, 2022

HYB Group, Inc.
Mr. Hovel Babikian
1601 Hasting Heights Lane
Pasadena, California 91107

Subject: **Report of Geotechnical Engineering Investigation, Proposed Industrial Warehouse Building and Associated Structures, APN: 178-350-024, 029 & 030, 2240 Fleetwood Drive, City of Jurupa Valley, County of Riverside, California, EGL Project No.: 22-257-004L**

Ladies and Gentlemen:

In accordance with your request, Environmental Geotechnology Laboratory, Inc. (EGL) is pleased to submit this Geotechnical Engineering Report for the subject site. The purpose of this report was to evaluate the subsurface conditions and provide recommendations for foundation designs and other relevant parameters of the proposed construction.

Based on the findings and observations during our investigation, the proposed construction of the subject site for the intended use is considered feasible from the geotechnical engineering viewpoints, provided that specific recommendations set forth herein are followed.

This opportunity to be of service is sincerely appreciated. If you have any questions pertaining to this report, please call the undersigned.

Respectfully submitted,

Environmental Geotechnology Laboratory, Inc.

REGISTERED PROFESSIONAL ENGINEER
RYAN KEITH JONES
No. 2852
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REPORT OF GEOTECHNICAL ENGINEERING INVESTIGATION

Proposed Industrial Warehouse Building
and Associated Structures

APN: 178-350-024, 029 & 030

at

2240 Fleetwood Drive
City of Jurupa Valley, County of Riverside, California

Prepared by
ENVIRONMENTAL GEOTECHNOLOGY LABORATORY, INC.
Project No.: 22-257-004L
October 31, 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 development at 2240 Fleetwood Drive, City of Jurupa Valley, County of Riverside, California. The approximate regional location is shown on the Site (Location) Map, Figure 1. The purposes of this investigation were to evaluate the subsurface conditions at the area of proposed construction and to provide recommendations pertinent to grading, foundation design and other relevant parameters of the proposed development.

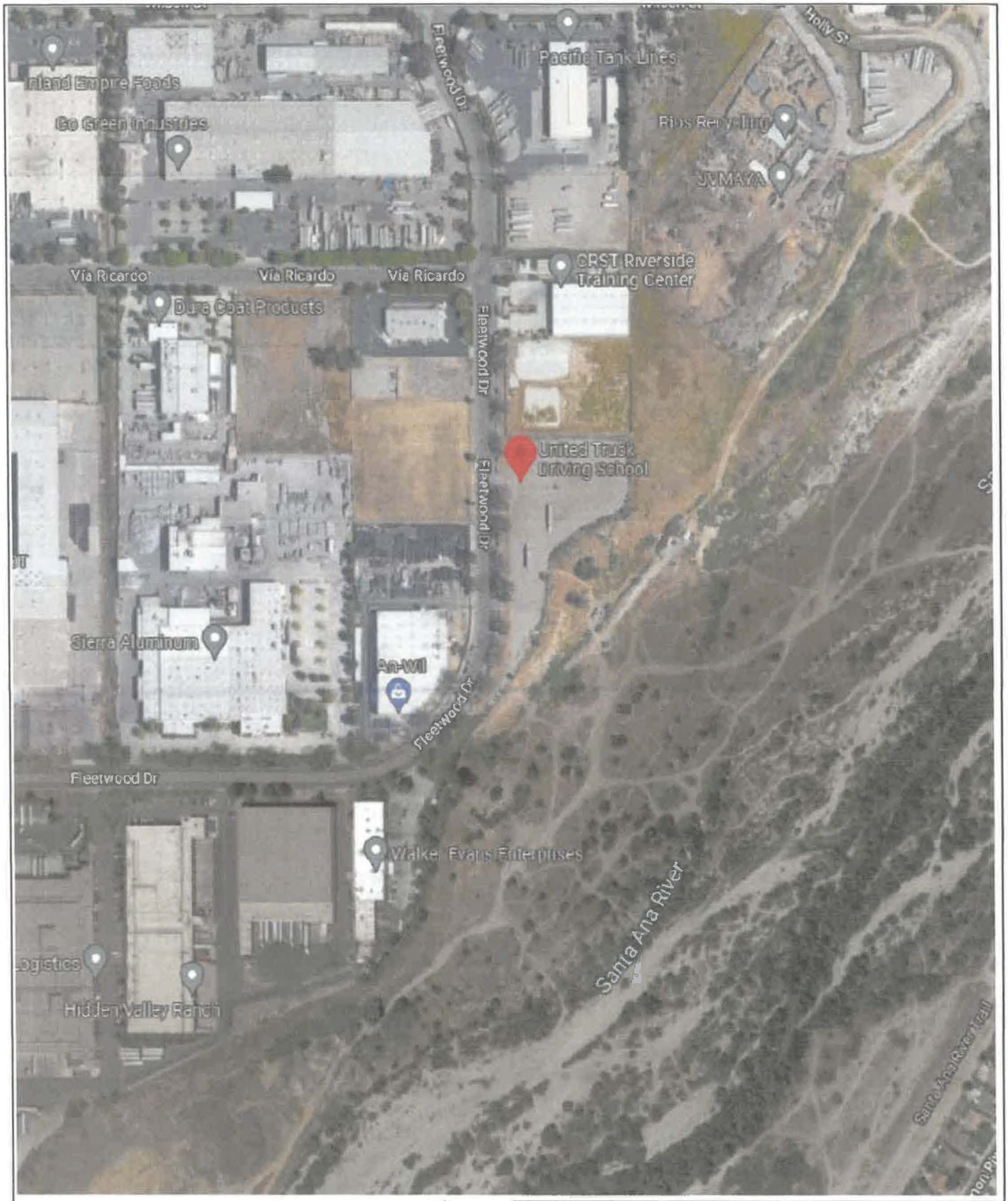
1.2 Scope of Services

Our scope of services included:

- Review of available soil engineering data of the area.
- Subsurface exploration consisting of logging and sampling of seven (7) 8-inch diameter hollow-stem auger borings. Borings were extended to a maximum depth of 51.0 feet below the existing ground surface. The boring logs are presented in Appendix A.
- Perform laboratory testing on representative onsite samples to establish soil-engineering characteristics. Field moisture and density are presented on boring logs in Appendix A. Laboratory test results are presented in Appendix B.
- Engineering analyses of the geotechnical data obtained from our background studies, field investigation, and laboratory testing.
- Perform percolation tests on boring B-3, B-5 and B-6 at the depths of 6', 6' and 10', respectively to determine the design infiltration rate of the soil at the site. Infiltration test results are presented in Appendix E.
- Preparation of this report presenting our findings, conclusions, and recommendations for the proposed construction.

1.3 Site Location

The subject site is located on the east side of Fleetwood Drive and at a relatively short distance southeast of the intersection of the Fleetwood Drive and Via Ricardo, and bounded easterly-southeasterly by Santa Ana River in the City of Jurupa Valley, County of Riverside, California. The approximate location of the site is presented in the Site Location Map (Figure1). The project site consists of three vacant lots (APN: 178-350-024, 029 & 030) and currently used for truck driving school. Topographically, the subject property is comprised of two relatively flat pads separated by an approximate 5 feet high retaining wall, and flanked on the southeast by



Scale: 1" = 1,000'

Base map adopted from County of San Bernardino and Riverside, USGS , USDA/ FPAC/ GEO, Map data @ 2022,, Google Earth.

ENVIRONMENTAL GEOTECHNOLOGY LABORATORY

EGL Proj. No.: 22-257-004
2240 Fleetwood Drive
Jurupa Valley, California

SITE (LOCATION) MAP

10/22

Figure 1

11819 Goldring Road, Unit A, Arcadia, California 91006; Phone (626) 263-3588; Fax (626) 263-3599

descending slopes. This descending slope has an overall slope ratio approximately of 2.9: 1 (Horizontal: Vertical) and locally as steep as 1.5: 1 (H: V) with a total relief of approximately 60 feet. Maximum relief is measured from anticipated proposed building pad elevation to the southeasterly descending slope toe. Tall trees with moderately wide bushes occupied near the top of slope area and scattered down in the southeasterly descending slope faces. Detailed configuration of the site is shown on the Site Plan, Figure 2a. Cross-Section A-A', Figure 2b, depicts surficial gradients and approximate subsurface soil conditions and contacts within the proposed development area.

1.4 Proposed Construction

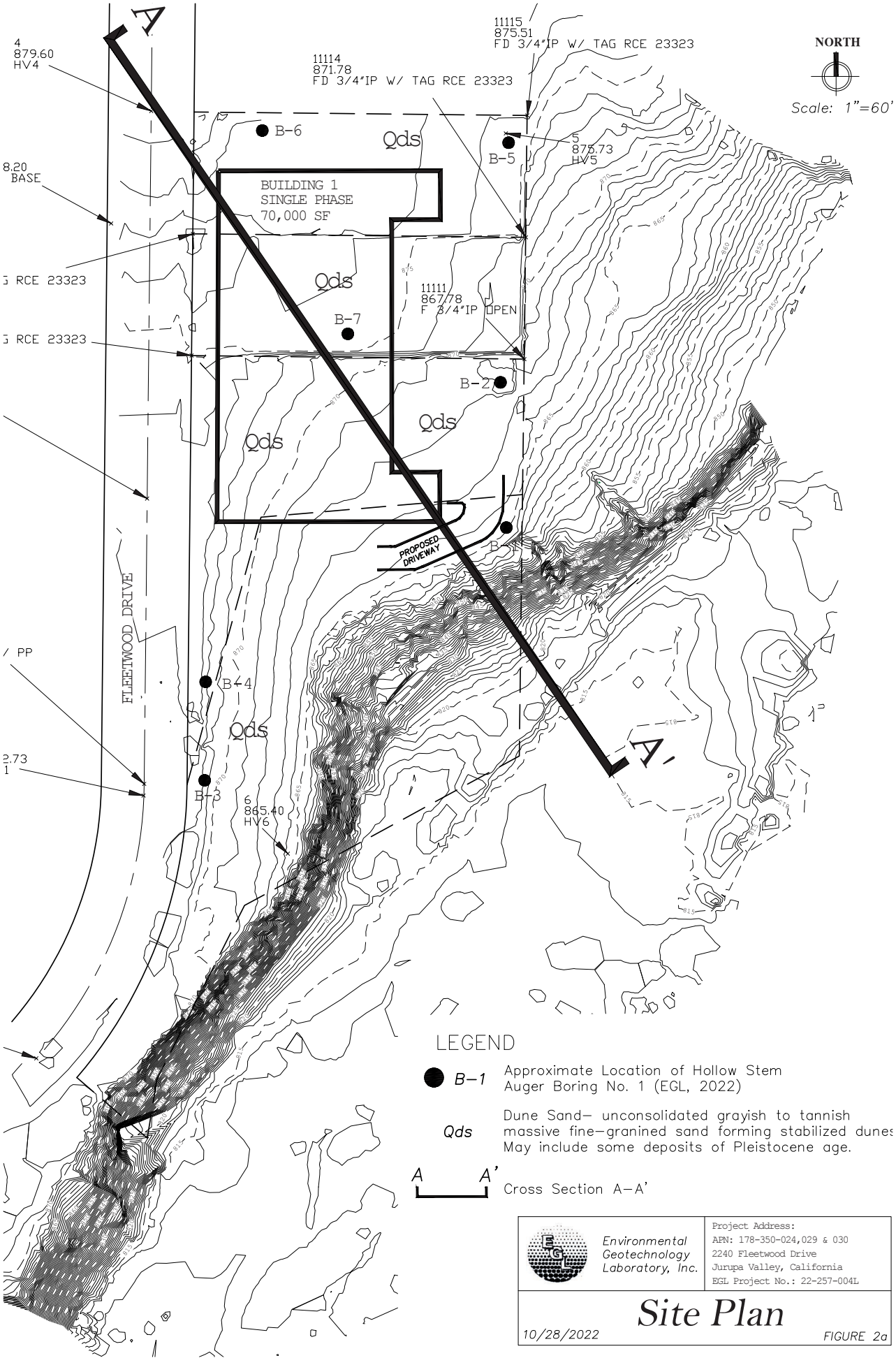
Based on the referenced *Conceptual Site Plan* provided by the project architect, it is our understanding that the proposed development at the site consists of a warehouse building and associated structures. The proposed building is anticipated to be one and/or two-story masonry and wood frame structures with a concrete slab-on-grade. Column loads are unknown at this time, but are expected to be light to medium. Cut and/or fill is anticipated to reach the desired grades.

2.0 SUBSURFACE EXPLORATION AND LABORATORY TESTING

2.1 Field Exploration

Subsurface conditions were explored on October 13, 2022 with the aid of a hollow-stem drill rig of 2R Drilling Services. A total of seven (7) 8-inch diameter hollow-stem auger borings were drilled to a maximum depth of 51.0 feet below the existing ground surface. Upon completion of drilling and percolation testing, all borings were backfilled with onsite soil removed from excavations and tamped. The purpose of the excavation was to investigate the engineering characteristics of the onsite soils with respect to the proposed development.

EGL's field engineer supervised and logged all borings. Relatively undisturbed ring samples and bulk samples were collected during drilling for laboratory testing. Approximate locations of these borings are shown on the Site Plan, Figure 2. Exploratory boring logs are presented in Appendix A. Ring and SPT samples were taken at frequent intervals. The samples taken by a hollow stem auger were obtained by driving a sampler with successive blows of a 140-pound hammer dropping from a height of 30 inches.



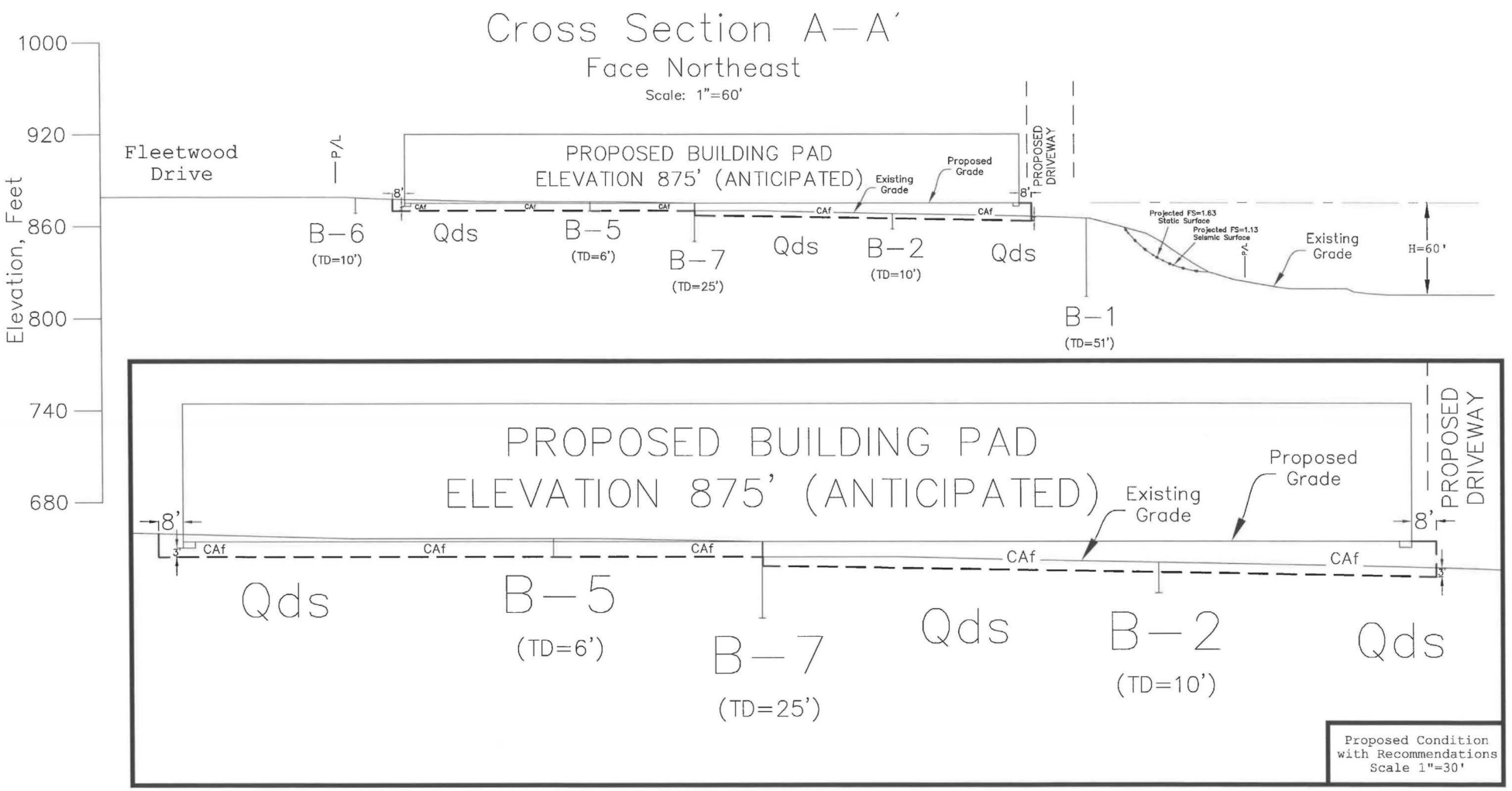
Environmental Geotechnology Laboratory, Inc.

Project Address:
APN: 178-350-024,029 & 030
2240 Fleetwood Drive
Jurupa Valley, California
EGL Project No.: 22-257-004L

Site Plan

10/28/2022

FIGURE 2a



LEGEND

Qds	Natural Soils
CAf	Future Compacted Fill

Cross Section A-A'
Site:2240 Fleetwood Drive
Jurupa Valley, California
Date: 11-1-2022

EGL Project No.:22-257-004L
Environmental Geotechnology Laboratory, Inc.

Figure 2b

2.2 Laboratory Testing

Representative samples were tested for the following parameters: in-situ moisture content and density, direct shear strength, consolidation, percent of fine, corrosion potential and expansion index. 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 presented on the boring logs (Appendix A).

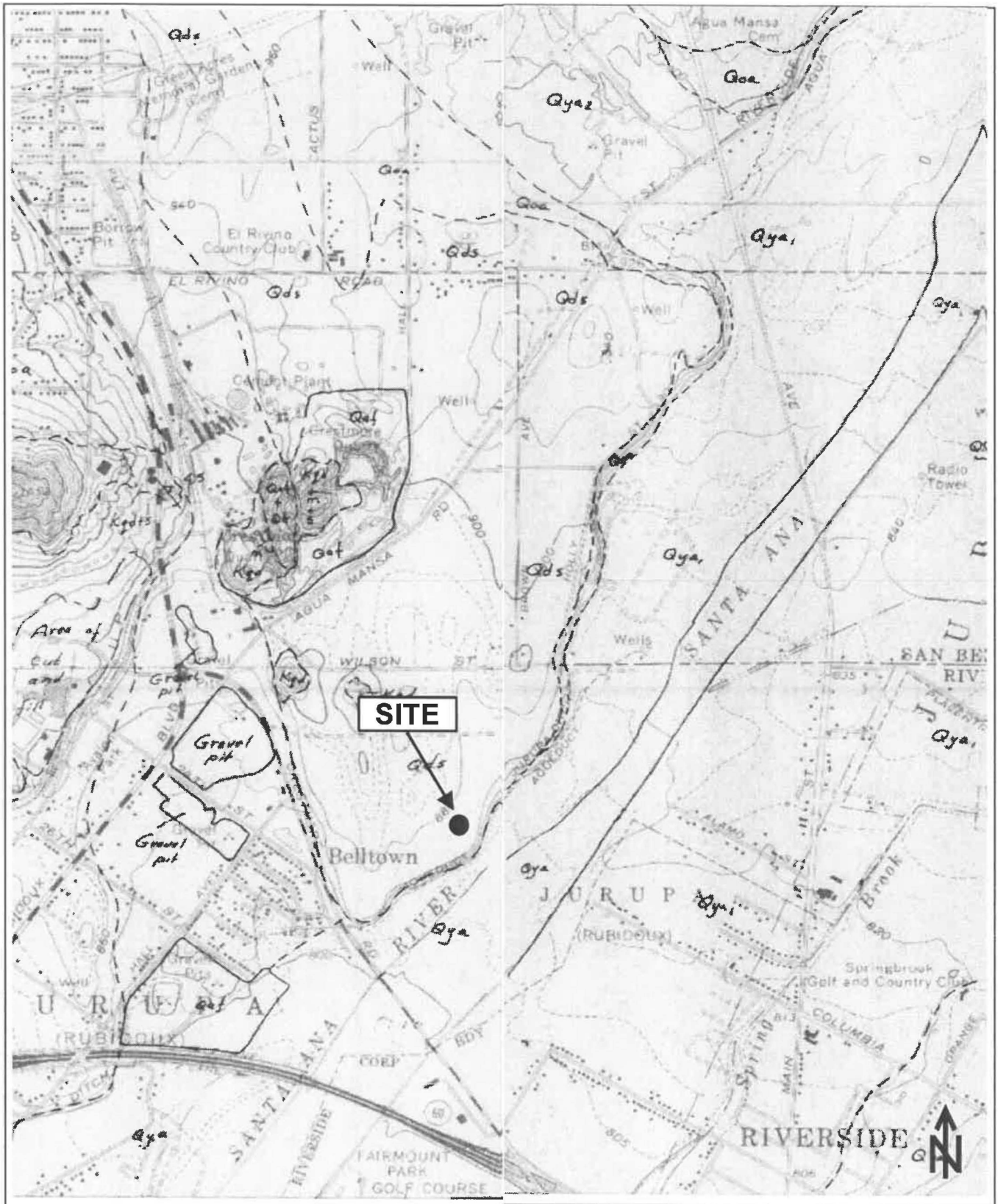
3.0 SUMMARY OF GEOTECHNICAL CONDITIONS

3.1 Soil Conditions

The onsite soils consist predominantly of silty sand (SM), poorly graded sand (SP), well-graded sand (SW) and sandy clay (CL) materials. The near surface soils at boring B-1 consist of dark olive brown to dark yellowish brown silty sand (SM) with trace of clay. These soils exist in a fine to medium grained, dry to slightly moist, and medium dense condition to a depth of approximately 4 feet. Below 4 feet, our boring B-1 encountered layers of dark yellowish brown to olive brown, fine and fine to medium grained, dry to moist, and medium dense silty sand (SM) and poorly graded sand (SP) to a depth of approximately 34 feet. Below this, a thin layer of grayish brown, slightly moist and very stiff sandy clayey silt (ML) and sandy clay (CL) was encountered to a depth of approximately 36 feet. This was followed by layers of grayish brown to olive brown, fine to coarse grained, dry to moist, and very dense well-graded sand (SW) to the maximum explored depth of 51 feet below the existing ground surface. Dark yellowish brown to brown, slightly moist, and stiff to hard sandy clay (CL) were encountered in the near surface at borings B-3, B-4 and B-5 to the depths of approximately 6 feet to 10 feet below the existing ground surface. Borings B-6 & B-7 also encountered sandy clay material at the depths of 9' & 14', respectively. Based on USGS by Douglas M. Morton (1978) the site is underlain by unconsolidated grayish to tannish massive fine grained sand forming stabilized dunes. May include some deposits of Pleistocene age (Qds; see Figure 3).

3.2 Groundwater

Static ground water levels were not encountered during our subsurface investigations to the maximum explored depth of 51 feet below the existing ground surface. Based on the historically high groundwater depth map prepared by Wildermuth Environmental Inc. the historic groundwater is approximately greater than 50 feet below ground surface at the subject site (Ref. # 16) Groundwater is therefore not expected to be a significant constraint during the construction. However, groundwater may be a significant constraint if grading is completed during the rainy season when perched water is more likely to occur.



Qds	DUNE SAND--- Unconsolidated grayish to tannish massive fine-grained sand forming stabilized dunes. May include some deposits of Pleistocene age.

Base map adopted from USGS "Geologic Maps of Fontana and San Bernardino South 7.5' Quads, San Bernardino and Riverside Countties, California"; dated 1978, Scale 1"=24,000'

ENVIRONMENTAL GEOTECHNOLOGY LABORATORY

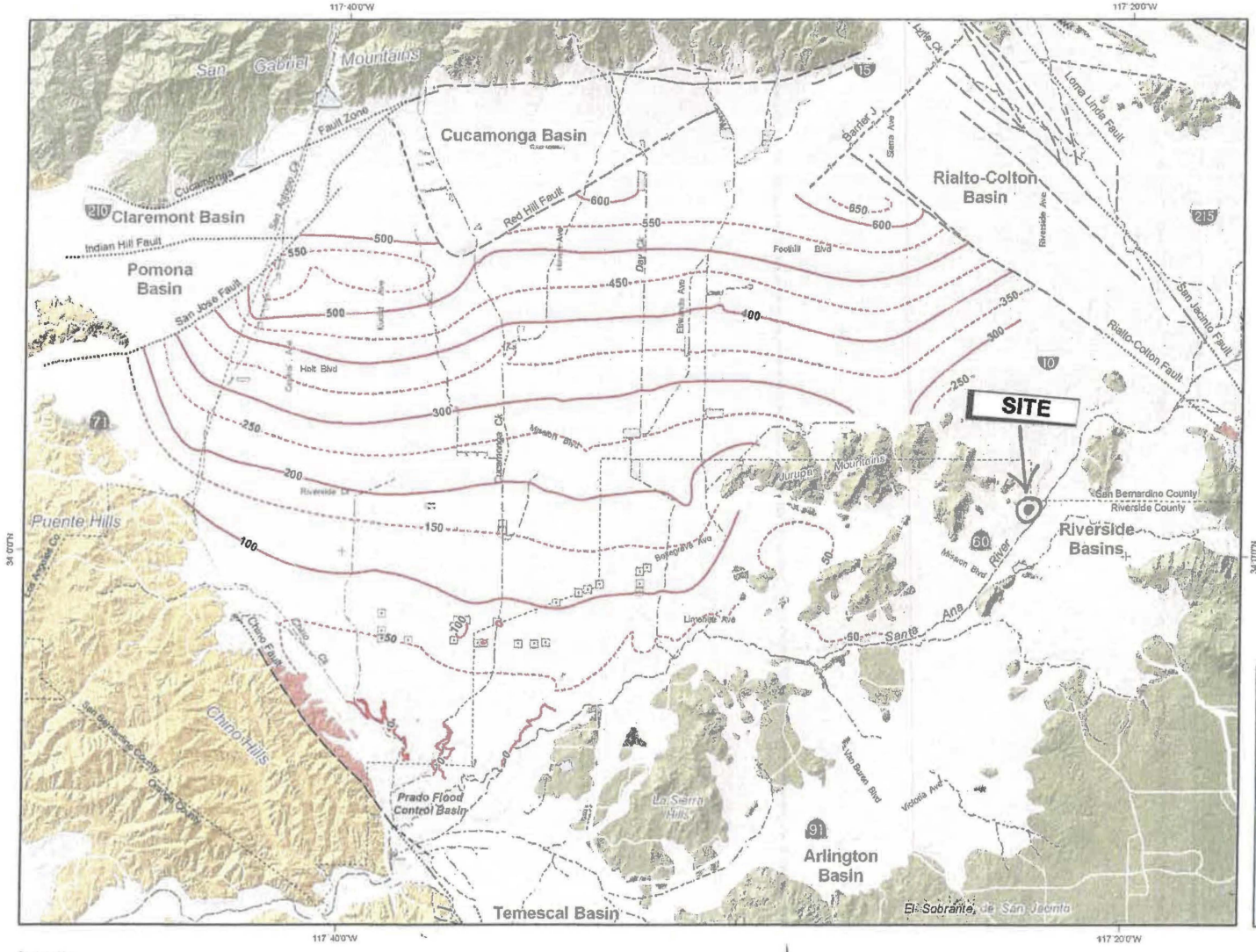
EGL Proj. No.: 22-257-004
2240 Fleetwood Drive
Jurupa Valley, California

REGIONAL (GEOLOGY) MAP

10/22

Figure 3

11819 Goldring Road, Unit A, Arcadia, California 91006; Phone (626) 263-3588; Fax (626) 263-3599



800 Depth to Groundwater Contours
775 (feet below ground surface)

Other Features

- Chino Desalter Well
- Flood Control and Conservation Basins

Geology

Water-Bearing Sediments

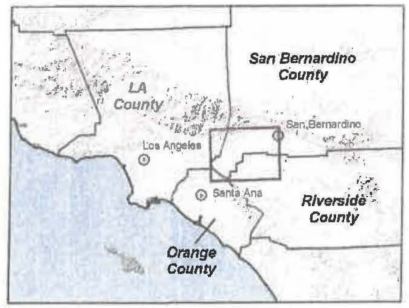
- Quaternary Alluvium

Consolidated Bedrock

- Plio-Pleistocene Sedimentary Rocks
- Cretaceous to Miocene Sedimentary Rocks
- Pre-Tertiary Igneous and Metamorphic Rocks

Faults

- Location Certain
- Location Approximate
- Location Concealed
- Location Uncertain



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Date: 20070720
File: DTW_CBWM.mxd

0 1 2 3 4 5 Miles
0 2 4 6 8 KM

Chino Basin

Depth to Groundwater Contours

Chino Basin -- Fall 2006

Figure 1

4.0 SEISMICITY

4.1 Faulting

By definition of the California Mining and Geology Board, an active fault is a fault that has had surface displacement within Holocene time (about the last 11,000 years). A potentially active fault is defined by the State Geologist as any fault that has been active during Quaternary time (the last 1,600,000 years).

EGL's studies of regional and local seismicity indicate that there are no known active faults crossing subject property. However, site locates 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. Based on our review of the "*Public Safety Element, Riverside County General Plan, Liquefaction Zones*" it is concluded that the site is located within a "Moderate" potential liquefaction area.

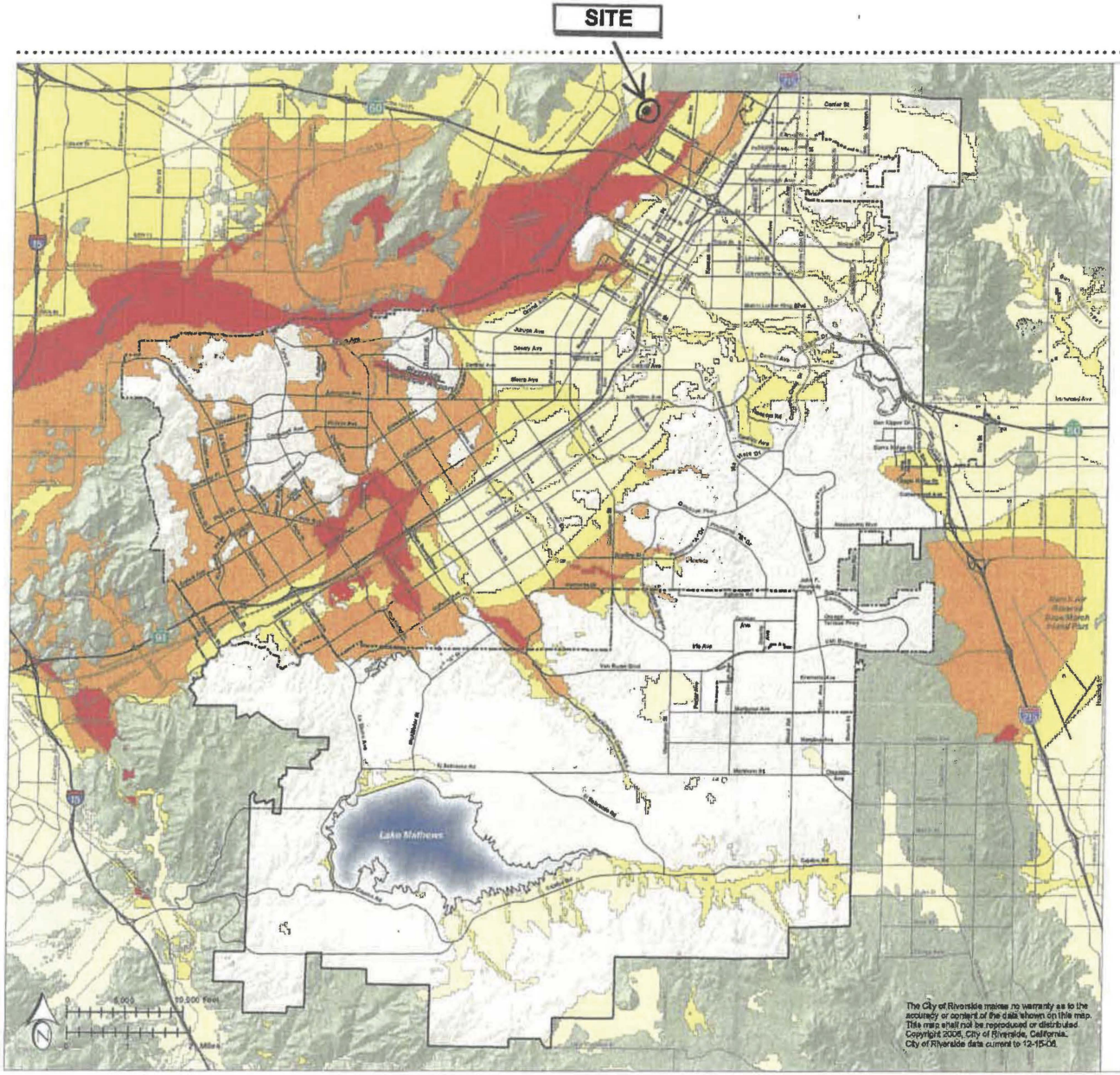
4.2 Estimated Earthquake Ground Motions

In order to estimate the seismic ground motions at the subject site, EGL has utilized the U.S. Seismic Design Maps website prepared by United States Geological Survey USGS and the Unified Hazard Tool website. According to U.S. Seismic Design Maps, the peak ground acceleration corresponding to the PGA_M was used in our analyses. A PGA_M of 0.668g was obtained from the USGS, U.S. Seismic Design Maps website, (https://earthquake.usgs.gov/ws/designmaps/asce7-16.html). The earthquake magnitude (M) of 6.98 was obtained from the USGS, Unified Hazard Tool website (https://earthquake.usgs.gov/hazards/interactive/). Based on a 2% probability of exceedance in 50 years (2475-year return period) a magnitude of 6.98 was used with PGA_M and a factor of safety of 1.3 when calculating the seismic settlement. Seismic design map detail reports and the liquefaction analyses are presented in Appendix C.

5.0 SEISMIC HAZARDS

5.1 Liquefaction

Liquefaction is the transformation of a granular material from a solid to a liquid state as a result of increasing pore-water pressure. The material then loses strength and can flow as 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.



CITY OF RIVERSIDE
GENERAL PLAN 2025

LEGEND

- VERY LOW
- LOW
- MODERATE
- HIGH
- VERY HIGH
- RIVERSIDE CITY BOUNDARY
- RIVERSIDE PROPOSED SPHERE OF INFLUENCE

SOURCE: TRANSPORTATION AND LAND MANAGEMENT AGENCY (TLMA) GEOGRAPHIC INFORMATION SERVICES COUNTY OF RIVERSIDE, JANUARY 1, 2005
http://www.tlma.co.riverside.ca.us/index.html

Figure PS-2
LIQUEFACTION ZONES

The ground water table is located at least 50 feet below the existing ground surface at the subject site. The potential for liquefaction at the subject site was evaluated using the procedures suggested by Seed and others (1983 and 1985), based on data obtained from Boring B-1. The design earthquake magnitudes (M) of 6.98 and peak ground accelerations (PGA_M) of 0.668g are discussed in the previous Section. Based on the historically high groundwater depth map prepared by Wildermuth Environmental Inc. the historic groundwater is approximately greater than 50 feet below ground surface at the subject site. A ground water level was assumed to be at the depth of 50 feet below the existing ground surface, for the calculation purposes. The analyses presented in the enclosed Appendix C indicated that the existing onsite sandy soils for boring B-1 will not liquefy under the design earthquake events, based on calculation procedures of Seed, et. al. 1985. Due to the historic groundwater table and presence of high blow counts of the earth materials present in boring B-1, liquefaction is not expected to occur within the soil at the subject site. The summary of the total and differential settlements are as follows:

Return Period	Acceleration (PGA_M)(g)	Magnitude	Unsaturated Settlement (in)	Saturated Settlement (in)	Total Settlement (in)	Differential Settlement (in)
2475 years	0.668	6.98	0.63	0	0.63	0.315

5.2 Earthquake Induced Settlement

The sandy soils tend to settle and densify when they are subjected to earthquake shaking. Should the sand be saturated and there is no possibility for drainage so that constant volume conditions are maintained, the primary effect of the shaking is the generation of excess pore water pressures. Settlement then occurs as the excess pore pressures dissipate. The primary factors controlling seismic induced settlement are the cyclic stress ratio, maximum shear strain induced by earthquake, the strength and density of the sand, and the magnitude of the earthquake.

Based on the procedures developed by Tokimatsu and Seed (1987), it is estimated that liquefaction induced settlement of dense soil or clayey soil is negligible. Due to the high groundwater and the presence of dense sandy soil it is EGL's opinion that settlement of saturated sandy soil under the design earthquake conditions can be negligible. The unsaturated soil between 0 feet and 50 feet below the ground surface may settle during seismic activity

approximately 0.63 inches. It is estimated that the maximum total settlement of saturated and unsaturated sandy soil due to seismic activity that may occur at the subject site is approximately 0.63 inches.

5.3 Landsliding

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

5.4 Lurching

Soil lurching refers to the rolling motion on the surface due to the passage of seismic surface waves. Effects of this nature are not considered significant on the subject site where the thickness of alluvium does not vary appreciably under structures.

5.5 Surface Rupture

Surface rupture is a break in the ground surface during or as a consequence of seismic activity. The potential for surface rupture on the subject site is considered low due to the absence of known active faults at the site.

5.6 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 Lateral Spread

Lateral spread is a type of liquefaction-induced ground surface failure. Generally, lateral spread occurs in surficial soils on a gentle slope or near a free face. A shear zone is formed within liquefied soils and causes the upper soil to become displaced or spread. Deformations due to lateral spread can range from inches to several feet and can cause significant damage to structures and lifelines. Based on the paper published in ASCE Journal of Geotechnical Engineering, "Empirical Prediction of Liquefaction-Induced Lateral Spread," by Bartlett and Youd

(1995) lateral spreading will not occur at the site. According to the study presented by Bartlett & Youd, the site needs to be sloped or near a free face for lateral displacement to occur. Due to the lack of high groundwater at the site, it is our opinion that the potential for lateral spreading is remote.

6.0 SLOPE STABILITY

6.1 General

Stability of the existing southeasterly descending slope with an average slope ratio approximately of 2.9: 1 (Horizontal: Vertical) and locally as steep as 1.5: 1 (H: V) with a total relief of approximately 60 feet high, has been considered and calculations are presented in Appendix D. Shear strength parameters were derived using in-place ultimate shear strengths of soils obtained from this and past investigations and evaluated in light of past experience in similar soil condition. Based on EGL's field observation, the existing slopes and proposed development are considered geotechnically stable for the current and proposed conditions. Simplified Bishop's method was used to analyze the entire slope for the critical area depicted in Cross Section A-A' extending to the top and bottom of the existing and proposed slope conditions.

Lowest ultimate soil shear values were used for static and seismic conditions for existing and proposed slope conditions. Seismic coefficient of 0.2 g (g: gravity) was used in the seismic condition. No deep-seated failures were observed during our field investigation. Stability calculations and sections are presented on the plates following the report and the results of EGL's slope stability analyses of existing and proposed slope conditions are briefly described in the following sections. Analytical results are presented in Appendix D.

Earth Materials	Unit Weight (pcf)	Friction Angle (Degree)	Cohesion (psf)
Soil, Ultimate^{+} (Qds; B-1 @ 25.0')	120.0	29	168
Proposed Compacted Fill, Ultimate (CAf; Assumed)	125.0	35	200

Noted: $^{+}$Lowest ultimate soil shear values were used for static and seismic conditions for existing and proposed slope conditions.

6.2 Existing Slope

Existing southeasterly descending slope with an average slope ratio approximately of 2.9: 1 (Horizontal: Vertical) and locally as steep as 1.5: 1 (H: V) with a total relief of approximately 60 feet high was analyzed. Deep-seated failures were not encountered and/or observed during EGL's field investigation the existing slopes within the subject site. It is EGL's opinion that existing

slopes within the subject site considered satisfactory and possess adequate factors of safety against instabilities.

Analyzed Section	Calculated Condition	Factors of Safety (Static / Seismic)
A – A '	Existing southeasterly descending slope, approximately 60 feet high	1.63 / 1.13

Slope stability analyses utilizing Simplified Bishop's method was performed for the existing descending slope conditions. Existing southeasterly slope under static condition have a factor of safety, FS=1.63. Under seismic condition, FS=1.13. Analyzed slope is greater than the minimum required of FS=1.5 and FS=1.1, under static and seismic conditions, respectively. Therefore, the existing slope may be considered stable.

6.3 Proposed Condition

Under similar analytical condition used above, proposed slope with building loads, under static condition has a factor of safety, FS=1.63, and under seismic condition, FS=1.13. Both conditions are greater than the minimum required of FS=1.50 and FS=1.1, under static and seismic conditions, respectively. Based on the slope stability analyses results, the proposed development does not significantly impact the existing slope or significantly change the factors of safety of the existing slope conditions. Therefore, the proposed permanent slope condition may be considered stable. Results of slope stability analyses are presented in Slope Stability Analyses, Appendix D.

Analyzed Section	Calculated Condition	Factors of Safety (Static / Seismic)
A – A'	Proposed Slope with Building Loads	1.63 / 1.13

6.4 Surficial Slope Stability

Existing natural soil materials located on the existing descending slopes are subjected to sloughing under saturated conditions. Surficial slope stability of the existing descending slopes at slope gradient of 2.9: 1 to 1.5:1 (Horizontal: Vertical) possess factors of safety FS=1.20 and FS=0.69, respectively. The factor of safety is less than the minimum required factor of safety, FS=1.50, for an assumed saturated depth of four (4) feet (see Figure 4). However, it is EGL's opinion that the surficial soils at the existing descending slopes will not significantly adversely affect the subject property or neighbor's property due to medium to heavy vegetation growth in the descending slope area, and should remain in the current condition.

SURFICIAL SLOPE STABILITY
INFINITE SLOPE W/ PARALLEL SEEPAGE

$$F.S. = \frac{C + (\gamma t - \gamma w) \times H \times \cos^2\alpha \times \tan\phi}{\gamma t \times H \times \cos\alpha \times \sin\alpha}$$

Soil Properties and Slope Ratio:

2.9: 1 Slope, Horizontal to Vertical
Direct Shear Test Results, from B-2 @ 5'

C =	54	psf
α =	19	degrees
ϕ =	31	degrees
H =	4	ft
γt =	120.0	pcf
γw =	62.4	pcf

$\cos\alpha$ = 0.945519
$\cos^2\alpha$ = 0.894005
$\sin\alpha$ = 0.325568
$\tan\phi$ = 0.600861

F.S. = 1.20 (< 1.5)

Soil Properties and Slope Ratio:

1.5: 1 Slope, Horizontal to Vertical
Direct Shear Test Results, from B-2 @ 5'

C =	54	psf
α =	33	degrees
ϕ =	31	degrees
H =	4	ft
γt =	120.0	pcf
γw =	62.4	pcf

$\cos\alpha$ = 0.838671
$\cos^2\alpha$ = 0.703368
$\sin\alpha$ = 0.544639
$\tan\phi$ = 0.600861

F.S. = 0.69 (< 1.5)

Therefore, existing 2.9:1 and 1.5: 1 descending slopes located at the subject site do not have adequate factor of safety against surficial instability under saturated 4.0 feet thick soil conditions.

EGL Project No.: 22-257-004L
Site: 2240 Fleetwood Drive, Jurupa Valley, CA

Figure 4

All slopes will be subject to surficial erosion. Therefore, slopes should be protected from surface runoff by means of top-of-slope compacted earth berms or concrete interceptor drains. All slopes should be landscaped with suitable plants requiring minimal cultivation and irrigation water in order to thrive. Irrigation system should be installed and properly maintained. Over-watering and subsequent saturation of slope surface should be avoided.

7.0 CONCLUSIONS

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

7.1 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 characteristic of all Southern California.

7.2 Liquefaction Potential

Based on our liquefaction evaluation, it is concluded that the existing on-site sandy soils for boring B-1 are not potentially liquefiable soil under the design earthquake event. The soils are considered non-liquefiable due to the fact that they are either above ground water, contain high clay content or have high blow counts. The estimated potential settlement induced by the underlying potentially liquefiable soils is negligible.

7.3 Excavatability

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

7.4 Surficial Soils Removal

Based on our field investigation and laboratory testing program, it is concluded that the existing near surface soils are relatively dense/stiff but may be unsuitable for support of the new structures as they presently exist and will require remedial works as discussed herein.

7.5 Groundwater

Static ground water levels were not encountered during our subsurface investigations to the maximum explored depth of 51 feet below the existing ground surface. Based on the historically high groundwater depth map prepared by Wildermuth Environmental Inc. the historic groundwater is approximately greater than 50 feet below ground surface at the subject site (Ref. # 16) Groundwater is therefore not expected to be a significant constraint during the construction. However, groundwater may be a significant constraint if grading is completed during the rainy season when perched water is more likely to occur.

8.0 RECOMMENDATIONS

Based on the subsurface conditions exposed during field investigation and laboratory testing program, it is recommended that the following recommendations be incorporated in the design and construction phases of the project.

8.1 Grading

8.1.1 Site Preparation

Prior to initiating grading operations, any existing footings, vegetation, trash, debris, over-sized materials (greater than 6 inches), and other deleterious materials within fill areas should be removed from the site.

8.1.2 Surficial Soil Removals

No detailed grading plan was available at the time of preparing this report however, based on our field exploration and laboratory data obtained to date, it is recommended that the building pad for the new warehouse be removed and recompacted. The removal depth should be 3 feet below existing grade or 3 feet beneath proposed footing bottom whichever is deeper. The recommended removal should be extended at least 8 feet beyond proposed building lines or to a distance equal to the depth of fill below the proposed footings, whichever is greater. Existing near surface soils should also be removed at least one foot within proposed concrete slab and driveway areas. The construction areas should be excavated and then observed by a representative of this office to verify the soil conditions for any potential needs of removal of loose soils and replacement with compacted fill. This may also be necessary due to difference in expansion characteristics of foundation materials beneath a structure.

During the grading of the proposed slab area if expansive material is encountered it should be removed and replaced with sandy import material (EI < 20). If import is mixed with onsite material EGL should provide inspections to verify the soils are mixed uniformly and testing of the mixed fill material to determine the expansion potential. The expansion index of the mixed soil should be less than 20. Some preliminary testing of the import and onsite should be performed to determine the soil mixture ratio prior to backfilling the building pad.

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.

8.1.3 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, and then compacted to minimum project standards.

8.1.4 Structural Backfill

The onsite soils may be used as compacted fill provided they are free of organic materials and debris. During the grading of the proposed slab area if expansive material is encountered it should be removed and replaced with sandy import material (EI < 20). If import is mixed with onsite material EGL should provide inspections to verify the soils are mixed uniformly and testing of the mixed fill material to determine the expansion potential. The expansion index of the mixed soil should be less than 20. Some preliminary testing of the import and onsite should be performed to determine the soil mixture ratio prior to backfilling the building pad. Fills should be placed in relatively thin lifts; brought to near optimum moisture content, then compacted to obtain at least 90 percent relative compaction based on laboratory standard ASTM D-1557-12.

8.1.5 Sloped Excavation

Sloping excavation should be used provided the space is available and adjacent structures are located a horizontal distance away from the excavation equal or greater than the depth of excavation. Based on materials encountered at the subject site and the depth of the proposed excavation, it is EGL's opinion that sloped excavations may be considered to the depths and slope ratios tabulated below:

Height of Cut (feet)	Maximum Slope Ratio
0 – 5.0	Vertical
Higher than 5.0	1: 1

The existing adjacent structures should be protected during construction so no damage is done to the structures. No heavy construction vehicle loading or additional surcharge loading should be permitted at the top of the slope. A representative of this office should inspect the temporary excavation to make any necessary modification.

8.2 Foundation Design

8.2.1 Bearing Value

An allowable bearing value of 1800 pounds per square foot (psf) may be used for design of the footings placed at a depth of at least 18 inches below the lowest adjacent ground and founded on the new certified compacted fill. Single spread footings should be at least 24 inches square and continuous footings should be at least 12 inches wide. These bearing values may be increased by 200 psf for each additional foot of depth or width to a maximum value of 3500 psf. The above recommended value may be increased by one third (1/3) when considering short duration seismic or wind loads.

8.2.2 Settlement

Settlement of the footings placed as recommended and subject to no more than allowable loads is not anticipated to exceed 1/2 inch. Differential settlement between adjacent columns is not anticipated to exceed 1/4 inch. The potential total combined settlement (seismic plus static) and total combined differential settlement (seismic plus static) is approximately 1.13 inches and 0.565 inches under the design earthquake events, respectively.

8.2.3 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 2500 pounds per square foot. An allowable coefficient of friction between soil and concrete of 0.35 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).

Active earth pressure from horizontal backfill may be computed as an equivalent fluid weighting of 35 pounds per cubic foot. An at-rest fluid weighting of 55 pounds per cubic foot may be used for free-draining, level backfill. For basement (restrained) walls greater than 6' with level backfill, an

additional seismic lateral force of $16H^2$ plf should be applied at 0.37H from the base of the wall (H = retained height). For cantilever (unrestrained) walls greater than 6' with level backfill, an additional seismic lateral force of $10H^2$ plf should be applied at 0.37H from the base of the wall (H = retained height). The above value assumes free-draining conditions.

8.2.4 Foundation Construction

It is anticipated that the entire structure will be underlain by onsite soils of high expansion potential. The following presents our recommendations for the foundation construction. All footings should be founded at a minimum depth of 18" below the lowest adjacent ground surface and founded into new certified compacted fill. Proposed footings should include surcharge from adjacent neighboring structures, including structural footings and/or walls. All continuous footings should have at least two No. 4 reinforcing bar placed both at the top and two No. 4 reinforcing bar placed at the bottom of the footings. A grade beam of at least 12" square, reinforced as recommended above for footings, should be utilized across the garage entrance. Base of the reinforced beam should be at the same elevation as the bottom of the adjoining footings.

8.2.5 Concrete Slabs

Concrete slabs should be a minimum of 5" thick, underlain with 2" of sand and reinforced with a minimum of #3 rebar spaced at 18" on center each way, or its equivalent. If heavy machinery or equipment is to be used within the warehouse than the structural engineer should design a thicker slab with appropriate reinforcement. All slab reinforcement should be supported to ensure proper positioning during placement of concrete. A positive separation should be maintained with expansive joint material to permit relative movement. Concrete slabs in moisture sensitive areas should be underlain with a vapor barrier consisting of a minimum of six-mil polyethylene membrane with all laps sealed. A minimum of 2" of sand should be placed over the membrane to aid in uniform curing of concrete.

8.2.6 Retaining Wall

Wall should be provided with subdrains to reduce the potential for the buildup of hydrostatic pressure. Backdrains may consist of free drainage materials (SE of 30 or greater) or CalTrans Class 2 permeable materials immediately behind the wall and extending to within 18" of the ground surface. A 4"-diameter perforated pipe wrapped in gravel and geofabric should be installed at the base of the wall and sloped to discharge to a suitable collection facility or through weep holes. Alternatively, drainage fabric could be used. The fabric manufacturer's recommendations should be followed in the installation of the drainage fabric backdrain.

8.3 Temporary Excavation and Backfill of Utility Trenches

All trench excavations should conform to CAL-OSHA and local safety codes. All utilities trench backfill should be brought to near optimum moisture content and then compacted to obtain a minimum relative compaction of 90 percent of ASTM D-1557-12. All temporary excavations should be observed by a field engineer of this office so as to evaluate the suitability of the excavation to the exposed soil conditions.

9.0 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. The following CBC 2019 (Chapter 16) & ASCE 7-16 seismic related values may be used:

Site Classification: (ASCE, Table 20.3-1)	D
Spectral Response Accelerations (g):	
(CBC, Figure 1613.2.1 (1) 0.2-Second, S_S	1.500
(CBC, Figure 1613.2.1 (2)) 1-Second, S_1	0.600
Site Coefficient:	
(CBC, Table 1613.2.3 (1)) F_a	1.0
(CBC, Table 1613.2.3 (2)) F_v	1.7

Based on the U.S. Seismic Design Maps (USGS, updated January 2019), the proposed structures may be designed to accommodate up to a maximum site horizontal acceleration of 0.668g with 2% probability of being exceeded in 50 years. However, the Project Structural Engineer should be aware of the information provided to determine if any additional structural strengthening is warranted.

10.0 CORROSION POTENTIAL

Chemical laboratory tests were conducted on the existing onsite near surface materials sampled during EGL's field investigation to aid in evaluation of soil corrosion potential and the attack on concrete by sulfate in the soils. The test results are presented in the Appendix B.

According to ACI 318-14 Table 19.3.1.1, a sulfate content of 0.007 – 0.030 percent by weight in soils is assigned to Class "S0" and the severity of exposure to sulfate for concrete placed in contact with the onsite soil is considered "Not Applicable". Based on the testing results and ACI 318-14 Table 19.3.2.1, it is concluded that there is no restriction on the type of cement ("No Type Restriction") to be used at the site; however EGL recommends that Type II cement be used.

Based on the minimum resistivity test results, the subsurface soils are corrosive to moderately corrosive buried metal pipe. Any underground steel utilities should be blasted and given protective coating. Should additional protective measures be warranted, a corrosion specialist should be consulted.

11.0 INSPECTION

As a necessary requisite to the use of this report, it is recommended that the following construction activities be observed by a representative of this office:

- Removal of surficial and unsuitable soils,
- Backfill placement and compaction,
- Foundation Excavation and
- Utility trench excavation and backfill.

The geotechnical engineer should be notified at least 1 day in advance of the start of construction. A joint meeting between the client, the contractor, and the geotechnical engineer is recommended prior to the start of construction to discuss specific procedures and scheduling.

12.0 106 STATEMENT

Based on our field investigation and the laboratory testing results, it is our opinion that the grading and proposed structures will be safe against hazard from landslide, settlement, or slippage and the proposed construction will have no adverse effect on the geologic stability of the adjacent properties provided our recommendations are followed.

13.0 DRAINAGE

Building pads should be properly drained toward the street away from the slope and structure via swales or area drains. Positive pad drainage shall be incorporated into the final plans. In no cases should water be allowed to pond within the site, impound against structures or flow in a concentrated and/or uncontrolled manner down the descending slope areas.

In order to evaluate the feasibility of the infiltration system, EGL has evaluated the soil conditions at the subject site based on the *Low Impact Development BMP Design Handbook* (Riverside County, 2011). The tests were performed within borings B-3, B-5 and B-6 at the depths of 6', 6' and 10' below existing ground surface, respectively. Approximate locations of the test borings are shown on the Site Plan, Figure 2a. The test borings B-3 and B-5 were presoaked and tested on

October 13 and October 14, 2022, respectively. The test boring B-6 was presoaked and tested on October 14, 2022. The test procedures are described as following:

- 3"-diameter perforated pipe surrounded with gravel was placed in the test borings so that caving would not occur during the percolation testing. The bottoms of test borings were also covered with 2 inches of gravel.
- The test borings were filled with a minimum depth of 20 inches water multiple times for presoak and allowed to completely drain prior to refilling for the infiltration test.
- For the percolation test of borings B-3 and B-5, a minimum depth of 20 inches of water was placed within the test borings. The test time interval between readings used for boring B-3 was 10 minutes due to at least 6 inches of water drop two consecutive times in 25 minutes during the first two tests. The test time interval between readings used for boring B-5 was 30 minutes due to less than 6 inches of water drop two consecutive times in 25 minutes during the first two tests.
- The test boring B-6 was filled with a depth of 20 inches water for the presoak. However, only approximately 1.5 inches of water drained in 60 mins time interval. Therefore, no additional testing was able to be performed.
- Additional six (6) 10-minute increment percolation tests were performed for test borings B-3 and B-5. Field data of two (2) 25 minutes readings and six (6) 10 minutes and/or 30 minutes readings are presented in Appendix E. The last measured drop was used to calculate the design infiltration rate of the soil. Design Infiltration rate calculations are presented in Appendix E.

Based on the results of our preliminary percolation test of the material, the minimum design infiltration rate is 0.16 in/hr. Reduction factor of 3.0 has been applied to our design infiltration rate. Due to the high percentage of clay material encountered within test borings and percolation test results, it is EGL's opinion that an infiltration basin within the natural soil is not feasible due to the very stiff/hard clayey material. Borings B-1 & B-2 encountered sandier material and may provide a better percolation rate; however, additional testing would be required. An infiltration system using planter boxes or approved equivalent may be used. If planter boxes are used, they should be waterproofed and designed with an overflow to the street.

14.0 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.

REFERENCES

1. American Concrete Institute, (2014), "*Building Code Requirements for Structural Concrete (ACI 318-14) and Commentary*", Chapter 19: Durability Requirements, Sections 19.3.1: Exposure Categories and Classes & 19.3.2: Requirements for Concrete Mixtures; pages 317 to 323, Tables 19.3.1.1 and 19.3.2.1".
2. ASCE, (2017), "ASCE/SEI 7-16, Minimum Design Loads and Associated Criteria for Buildings and Other Structures; dated 05-31-2017, 889 pages; prepared and published by American Society of Civil Engineers.
3. Bartlett, Steven F. and Youd, T. Leslie, (1995), "Empirical Prediction of Liquefaction-Induced Lateral Spread," ASCE Journal of Geotechnical Engineering.
4. California Divisions of Mines and Geology, "Guidelines for Evaluating and Mitigating Seismic Hazards in California", Special Publication 117A, 2008.
5. CBC, (2019), "California Building Code: California Code of Regulations, Title 24, Part 2, Volume 2 of 2, California Building Standards Commission"; Section 1613 Earthquake Loads; pages 33-46.
6. Douglas M. Morton (1978), "Geologic Map of Fontana and San Bernardino South 7.5- minute Quadrangles, San Bernardino and Riverside Counties, California"; prepared by California Division of Mines and Geology; Official Map; scale 1" = 24,000
7. GAA Architects (2022) "Conceptual Site Plan, 2240 Fleetwood Drive, Jurupa Valley, California", Project No.: HYB001.01, dated October 25, 2022.
8. 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.
9. Riverside County, (2011), "Low Impact Development BMP Design Handbook, Appendix A - Infiltration testing"; revised September, 2011, Page 34.
10. Riverside County, (2018), "Public Safety Element, General Plan, Figure PS-2, Liquefaction Zones" https://riversideca.gov/cedd/sites/riversideca.gov.cedd/files/pdf/planning/generalplan/18_Public_Safety_Element_with%20maps.pdf
11. Seed, H.B. and Idriss, I.M. (1983), "Ground Motions and Soil Liquefaction During Eathquakes", Earthquake Engineering Research Institute.
12. 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.
13. 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.
14. USGS, (2019), "US Seismic Design Maps"; updated 01-2019; prepared by United States Geological Survey; https://earthquake.usgs.gov/ws/designmaps/asce7-16.html
15. USGS, "BETA – Unified Hazard Tool" prepared by United States Geological Survey, https://earthquake.usgs.gov/hazards/interactive/
16. Wildermuth Environmental Inc. (2016), "Depth to Groundwater Contours, Chino Basin- Fall 2006,Figure 1; http://www.wildermuthenvironmental.com/
17. Yeats, Robert S., (2004) "Tectonics of the San Gabriel Basin and Surroundings, Southern California"; GSA Bulletin; September/October 2004; v.116; no. 9/10; p. 1158-1182

APPENDIX A
FIELD INVESTIGATION

Subsurface conditions were explored on October 13, 2022 with the aid of a hollow-stem drill rig of 2R Drilling Services. A total of seven (7) 8-inch diameter hollow-stem auger borings were drilled to a maximum depth of 51.0 feet below the existing ground surface. Upon completion of drilling and percolation testing, all borings were backfilled with onsite soil removed from excavations and tamped. The purpose of the excavation was to investigate the engineering characteristics of the onsite soils with respect to the proposed development.

EGL's field engineer supervised and logged all borings. Relatively undisturbed ring samples and bulk samples were collected during drilling for laboratory testing. Approximate locations of these borings are shown on the Site Plan, Figure 2. Ring and SPT samples were taken at frequent intervals. The samples taken by a hollow stem auger were obtained by driving a sampler with successive blows of a 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.

EGL

BORING LOG: B-1

PROJECT LOCATION: 2240 Fleetwood Drive, Jurupa Valley

PROJECT NO: 22-257-004L

EXCAVATION SERVICE: 2R Drilling
DATE EXCAVATED: 10/13/2022
DATE LOGGED: 10/13/2022
EXCAVATION METHOD: Hollow-Stem
SAMPLE METHOD: Split-Tube
ELEVATION: N/A
LOGGED BY: KY

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

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/6"	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Description of Material
2	B	R	19/12"	SM	115.7	4.5	Silty sand, dark olive brown to dark yellowish brown, fine to medium grained, dry to slightly moist, medium dense, trace of clay
		S	11,12,13	SM	-	2.6	
5		R	12/12"	SP-SM	102.2	2.5	Poorly graded sand with silt, dark yellowish brown, fine to medium grained, dry, medium dense
		S	3,7,10	SP-SM	-	2.8	
10		R	17/12"	SM	106.4	2.6	Silty sand, fine to medium grained, dark yellowish brown, dry, medium dense, trace of clay
		S	7,14,14	SM	-	2.8	
15		R	29/12"	SP-SM	112.6	6.3	Poorly graded sand with silt, yellowish brown, fine to medium grained, dry to slightly moist, medium dense
		S	6,10,10	SP-SM	-	3.0	
20		R	12/12"	SP-SM	101.6	2.5	Poorly graded sand with silt, olive brown, fine to medium grained, dry to slightly moist, medium dense
		S	5,9,11	SP-SM	-	6.7	
25		R	40/12"	SM	104.4	8.1	Silty sand, olive brown, fine grained, slightly moist to moist, medium dense, trace of clay
		S	9,10,12	SM	-	4.9	
30		R	35/12"	SP-SM	106.0	1.4	Poorly graded sand with silt, olive brown, fine to coarse grained, dry, medium dense
		S	12,12,16	SP-SM	-	2.5	
35		R	32/12"	ML	103.4	9.6	Sandy clayey silt, grayish brown, slightly moist, very stiff
		S	5,12,14	CL/SM	-	23.6/16.5	Sandy clay to silty sand, fine grained, grayish brown to olive gray, very moist, very stiff to medium dense
40		R	50/6"	SW-SM	86.6	9.1	Well graded sand with silt, fine to coarse grained, grayish brown to olive brown, slightly moist to moist, very dense, little gravels were encountered
		S	25,35,47	SW- SM	-	2.5	

Plate A-1

11819 Goldring Road, Unit A, Arcadia, California, 91006; Phone: (626) 263-3588; Email: ryan@eglab.com

EGL

BORING LOG: B-1 (Cont.)

PROJECT LOCATION: 2240 Fleetwood Drive, Jurupa Valley

PROJECT NO: 22-257-004L

EXCAVATION SERVICE: 2R Drilling
DATE EXCAVATED: 10/13/2022
DATE LOGGED: 10/13/2022
EXCAVATION METHOD: Hollow-Stem
SAMPLE METHOD: Split-Tube
ELEVATION: N/A
LOGGED BY: KY

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

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/6"	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Description of Material
45		R	50/7"	SW-SM	115.6	2.0	Well graded sand with silt, fine to coarse grained, olive brown,
		S	50/5"	SW-SM		3.6	dry, very dense, little gravels were encountered
50		R	50/5"	SW	No Recovery		Well graded sand, fine to coarse grained, light olive brown, dry,
		S	50/6"	SW	-	1.6	very dense, little gravels were encountered
55							Total Depth = 51 feet No Caving No Groundwater Boring Backfilled and Tamped After Groundwater Observation
60							Hammer Driving Weight = 140 lbs. Hammer Driving Height = 30 inches
65							
70							
75							
80							

Plate A-2

EGL

BORING LOG: B-2

PROJECT LOCATION: 2240 Fleetwood Drive, Jurupa Valley

PROJECT NO: 22-257-004L

EXCAVATION SERVICE: 2R Drilling
DATE EXCAVATED: 10/13/2022
DATE LOGGED: 10/13/2022
EXCAVATION METHOD: Hollow-Stem
SAMPLE METHOD: Split-Tube
ELEVATION: N/A
LOGGED BY: KY

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

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/12"	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Description of Material
2		R	17	SM	108.5	3.1	Silty sand, dark yellowish brown, fine to medium grained, dry to slightly moist, medium dense, few fine gravel and trace of clay
5		R	25	SM	108.2	3.2	Silty sand, dark yellowish brown, fine to medium grained, dry to slightly moist, medium dense
10		R	15	SM	105.2	5.6	Silty sand, dark yellowish brown, fine to medium grained, slightly moist, medium dense
15							Total Depth = 10 feet No Caving No Groundwater Boring Backfilled and Tamped Hammer Driving Weight = 140 lbs. Hammer Driving Height = 30 inches
20							

Plate A-3

11819 Goldring Road, Unit A, Arcadia, California, 91006; Phone: (626) 263-3588; Email: ryan@eglab.com

EGL

BORING LOG: B-3

PROJECT LOCATION: 2240 Fleetwood Drive, Jurupa Valley

PROJECT NO: 22-257-004L

EXCAVATION SERVICE: 2R Drilling
DATE EXCAVATED: 10/13/2022
DATE LOGGED: 10/13/2022
EXCAVATION METHOD: Hollow-Stem
SAMPLE METHOD: Split-Tube
ELEVATION: N/A
LOGGED BY: KY

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

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/12"	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Description of Material
2		R	18	CL	94.8	7.0	Sandy clay, dark yellowish brown, slightly moist, stiff to very stiff
5		R	41	CL	95.4	18.2	Sandy clay, brown, very moist, hard
10							Total Depth = 6 feet No Caving No Groundwater Boring Backfilled and Tamped After Percolation Test Hammer Driving Weight = 140 lbs. Hammer Driving Height = 30 inches
15							
20							

Plate A-4

11819 Goldring Road, Unit A, Arcadia, California, 91006; Phone (626) 263-3588; Fax (626) 263-3599

EGL

BORING LOG: B-4

PROJECT LOCATION: 2240 Fleetwood Drive, Jurupa Valley

PROJECT NO: 22-257-004L

EXCAVATION SERVICE: 2R Drilling
DATE EXCAVATED: 10/13/2022
DATE LOGGED: 10/13/2022
EXCAVATION METHOD: Hollow-Stem
SAMPLE METHOD: Split-Tube
ELEVATION: N/A
LOGGED BY: KY

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

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/12"	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Description of Material
2		R	26	CL	104.3	6.0	Sandy clay, yellowish brown, slightly moist, very stiff
5		R	38	CL	95.1	15.1	Sandy clay, brown, slightly moist to moist, very stiff to hard
10		R	38	CL/ML	81.3	22.0	Sandy clay to sandy clayey silt, brown to light gray, very moist, very stiff to hard
15							Total Depth = 10 feet No Caving No Groundwater Boring Backfilled and Tamped Hammer Driving Weight = 140 lbs. Hammer Driving Height = 30 inches
20							

Plate A-5

11819 Goldring Road, Unit A, Arcadia, California, 91006; Phone (626) 263-3588; Fax (626) 263-3599

EGL

BORING LOG: B-5

PROJECT LOCATION: 2240 Fleetwood Drive, Jurupa Valley

PROJECT NO: 22-257-004L

EXCAVATION SERVICE: 2R Drilling
DATE EXCAVATED: 10/13/2022
DATE LOGGED: 10/13/2022
EXCAVATION METHOD: Hollow-Stem
SAMPLE METHOD: Split-Tube
ELEVATION: N/A
LOGGED BY: KY

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

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/12"	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Description of Material
2	B	R	36	CL	113.8	10.3	Sandy clay, dark yellowish brown, slightly moist, very stiff to hard
5		R	40	CL	102.8	7.0	Sandy clay, brown to light brown, slightly moist, very stiff to hard
10							Total Depth = 6 feet No Caving No Groundwater Boring Backfilled and Tamped After Percolation Test Hammer Driving Weight = 140 lbs. Hammer Driving Height = 30 inches
15							
20							

Plate A-6

11819 Goldring Road, Unit A, Arcadia, California, 91006; Phone (626) 263-3588; Fax (626) 263-3599

EGL

BORING LOG: B-6

PROJECT LOCATION: 2240 Fleetwood Drive, Jurupa Valley

PROJECT NO: 22-257-004L

EXCAVATION SERVICE: 2R Drilling
DATE EXCAVATED: 10/13/2022
DATE LOGGED: 10/13/2022
EXCAVATION METHOD: Hollow-Stem
SAMPLE METHOD: Split-Tube
ELEVATION: N/A
LOGGED BY: KY

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

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/12"	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Description of Material
2		R	50/9"	SM	97.0	13.1	Silty sand, gray to olive, fine to coarse grained, very moist, very dense, with miscellaneous base and gravels up to 1" in size
5		R	17	SM	107.1	5.4	Silty sand, dark yellowish brown, fine to coarse grained, slightly moist, medium dense
10		R	33	CL	98.9	21.3	Sandy clay, brown, very moist, very stiff to hard
15							Total Depth = 10 feet No Caving No Groundwater Boring Backfilled and Tamped After Percolation Test
20							Hammer Driving Weight = 140 lbs. Hammer Driving Height = 30 inches

Plate A-7

11819 Goldring Road, Unit A, Arcadia, California, 91006; Phone (626) 263-3588; Fax (626) 263-3599

EGL

BORING LOG: B-7

PROJECT LOCATION: 2240 Fleetwood Drive, Jurupa Valley

PROJECT NO: 22-257-004L

EXCAVATION SERVICE: 2R Drilling
DATE EXCAVATED: 10/13/2022
DATE LOGGED: 10/13/2022
EXCAVATION METHOD: Hollow-Stem
SAMPLE METHOD: Split-Tube
ELEVATION: N/A
LOGGED BY: KY

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

Depth (ft)	Sample: Bulk	Sample: Undisturbed	Sample: Blows/12"	USCS Symbol	Dry Unit Wt. (pcf)	Moisture (%)	Description of Material
5		R	30	SM	119.4	3.3	Silty sand, dark yellowish brown, fine to coarse grained, dry, medium dense, trace of clay
10		R	17	SM	108.4	6.3	Silty sand, dark yellowish brown, fine to coarse grained, slightly moist, medium dense, trace of clay
15		R	21	CL	110.8	9.8	Sandy clay, brown, slightly moist, very stiff
20		R	40	CL	89.7	22.1	Sandy clay, brown to gray, very moist, very stiff to hard
25		R	38	CL	103.9	21.9	Sandy clay, light gray, very moist, very stiff to hard
30							Total Depth = 25 feet No Caving No Groundwater Boring Backfilled and Tamped Hammer Driving Weight = 140 lbs. Hammer Driving Height = 30 inches
35							

Plate A-8

11819 Goldring Road, Unit A, Arcadia, California, 91006; Phone (626) 263-3588; Fax (626) 263-3599

APPENDIX B

LABORATORY TESTING

During the subsurface exploration, EGL 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 0.0125 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 one kilo-pound (kips) per square foot, and the test results are shown on the attached Figures.

Corrosion Potential

Chemical laboratory tests were conducted on the existing onsite near surface materials sampled during EGL's field investigation to aid in evaluation of soil corrosion potential and the attack on concrete by sulfate soils. These tests are performed in accordance with California Test Method 417, 422, 532, and 643. The testing results are presented below:

Sample Location	PH	Chloride (ppm)	Sulfate (% by weight)	Min. Resistivity (ohm-cm)
B-1 @ 0'-5'	8.29	160	0.007	4200
B-5 @ 0'-5'	9.38	120	0.030	1200

Expansion Index

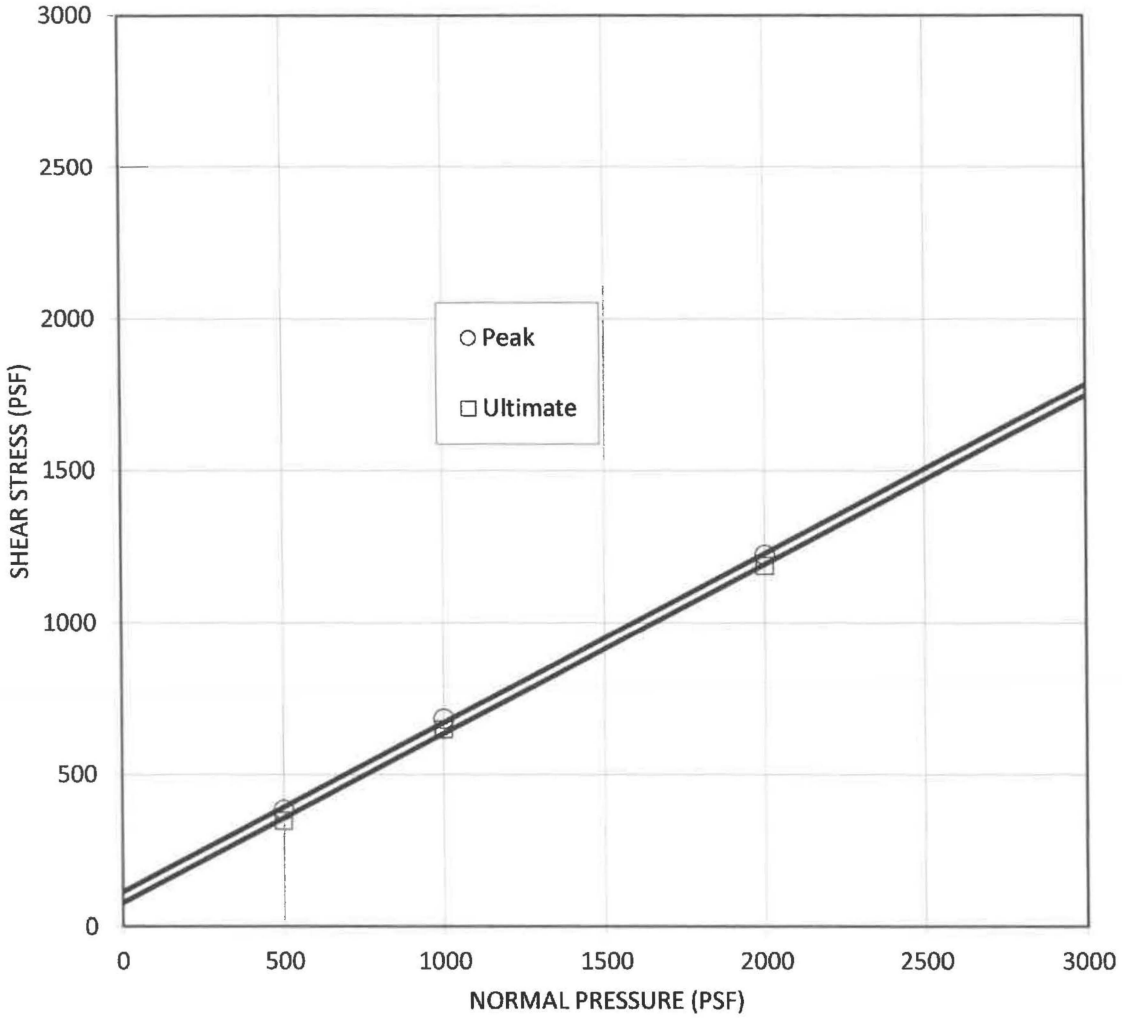
The Expansion Index was determined for the typical site material encountered in the borings. The laboratory standard used was ASTM D4829-95 and the test results are as follows:

Sample Location	Expansion Index	UBC Classification
B-5 @ 0-5'	94	High

Percent Passing #200 Sieve

Percent of soil passing #200 sieve were determined for selected soil samples in accordance with ASTM D422 and California Test Method CT-203 standard. The test results are presented in the following table.

Sample Location	% Passing #200
B-1 @ 3'	12.1
B-1 @ 6'	8.9
B-1 @ 11'	18.1
B-1 @ 19'	11.3
B-1 @ 21'	6.7
B-1 @ 26'	19.1
B-1 @ 31'	9.0
B-1 @ 36'	58.8 (top) 42.5 (bottom)
B-1 @ 41'	6.4
B-1 @ 46'	7.3
B-1 @ 51'	4.2



Boring No.:	Sample No.	Depth (ft)	Sample Type	Soil Type	Symbol	Cohesion (PSF)	Friction Angle
B-1	3	10.0	Ring	SM	○	114	29
					□	78	29

Normal Stress (psf)	Initial Moisture (%)	Final Moisture (%)	γ_d (pcf)	S (%)
500	2.6	20.9	106.2	96.4
1000	2.6	20.3	106.7	94.9
2000	2.6	20.1	107.0	94.4

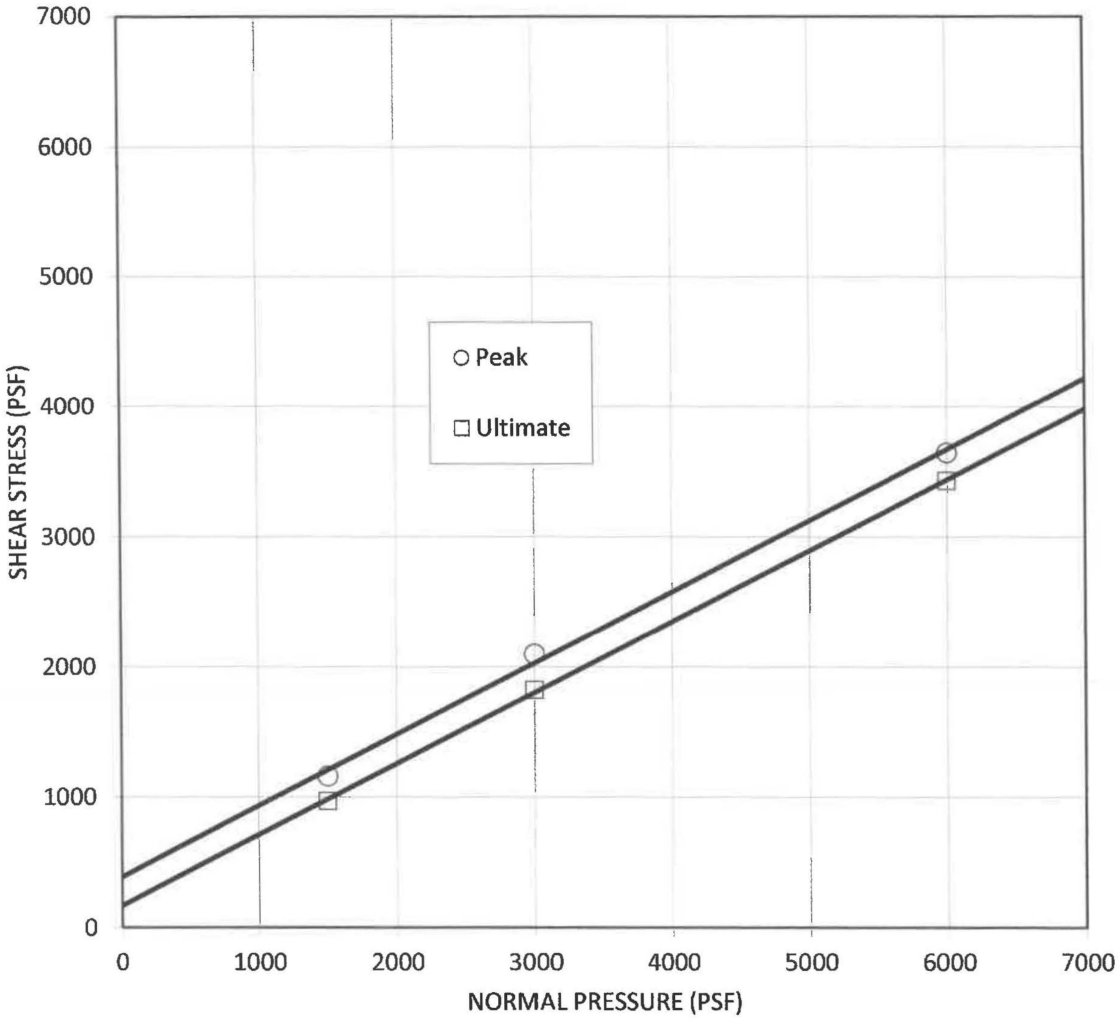
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Jurupa Valley, California

DIRECT SHEAR

10/22 (ASTM D3080) Figure

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Boring No.:	Sample No.	Depth (ft)	Sample Type	Soil Type	Symbol	Cohesion (PSF)	Friction Angle
B-1	6	25.0	Ring	SM	○	389	29
					□	168	29

Normal Stress (psf)	Initial Moisture (%)	Final Moisture (%)	γ_d (pcf)	S (%)
1500	8.1	22.6	104.7	100.0
3000	8.1	22.3	105.1	100.0
6000	8.1	21.8	106.0	100.0

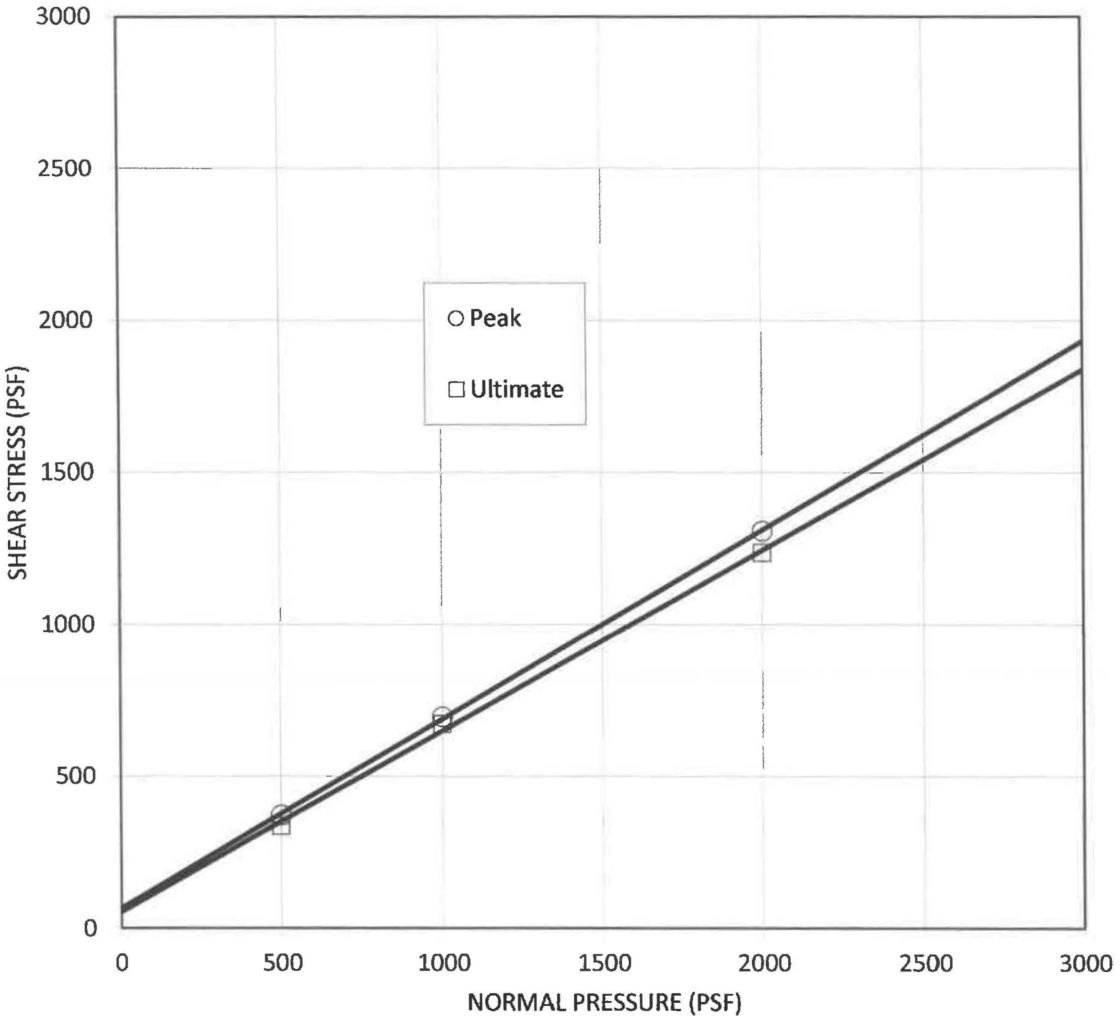
ENVIRONMENTAL GEOTECHNOLOGY LABORATORY

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Address: 2240 Fleetwood Drive
Jurupa Valley, California

DIRECT SHEAR

11/22 (ASTM D3080) Figure

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Boring No.:	Sample No.	Depth (ft)	Sample Type	Soil Type	Symbol	Cohesion (PSF)	Friction Angle
B-2	2	5.0	Ring	SM	○	66	32
					□	54	31

Normal Stress (psf)	Initial Moisture (%)	Final Moisture (%)	γ_d (pcf)	S (%)
500	3.2	19.6	107.9	94.4
1000	3.2	19.5	108.3	94.9
2000	3.2	19.3	108.7	94.7

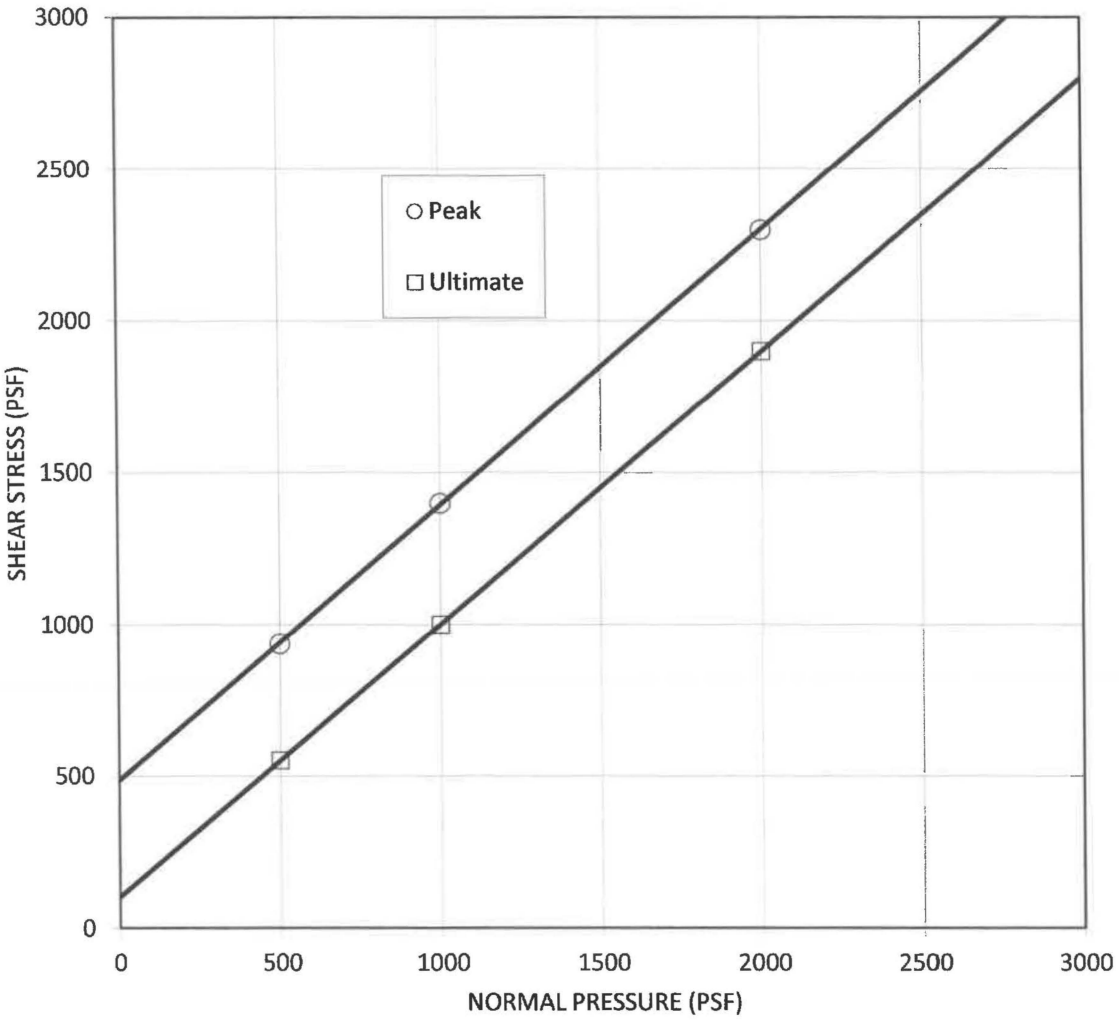
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Address: 2240 Fleetwood Drive
Jurupa Valley, California

DIRECT SHEAR

10/22 (ASTM D3080) Figure

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Boring No.:	Sample No.	Depth (ft)	Sample Type	Soil Type	Symbol	Cohesion (PSF)	Friction Angle
B-6	1	2.0	Ring	SM	○	486	42
					□	102	42

Normal Stress (psf)	Initial Moisture (%)	Final Moisture (%)	γ_d (pcf)	S (%)
500	13.1	35.4	94.2	100.0
1000	13.1	32.5	96.5	100.0
2000	13.1	27.7	97.6	100.0

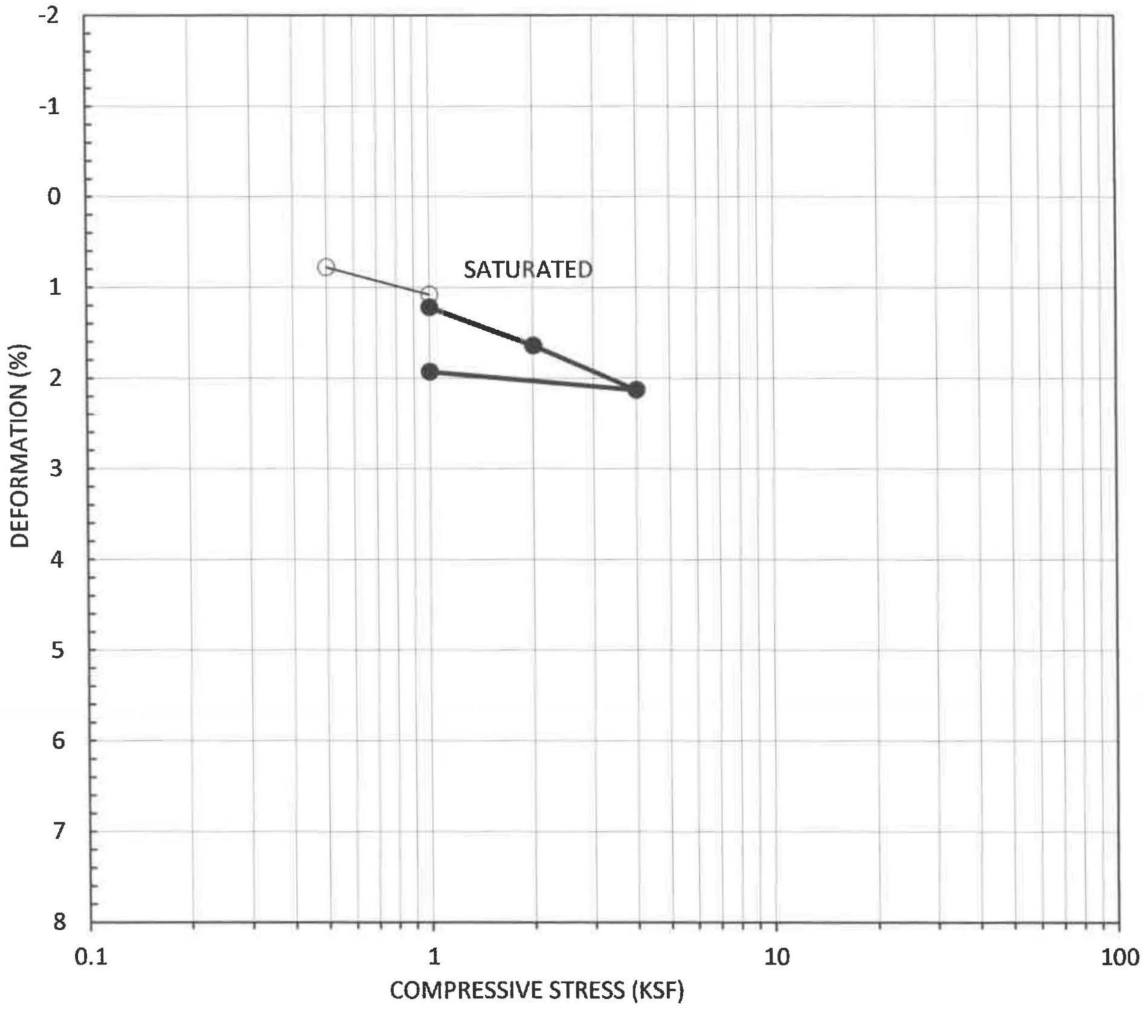
ENVIRONMENTAL GEOTECHNOLOGY LABORATORY

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DIRECT SHEAR

10/22 (ASTM D3080) Figure

11819 Goldring Road, Unit A, Arcadia, California 91006; Phone (626) 263-3588; Fax (626) 263-3599



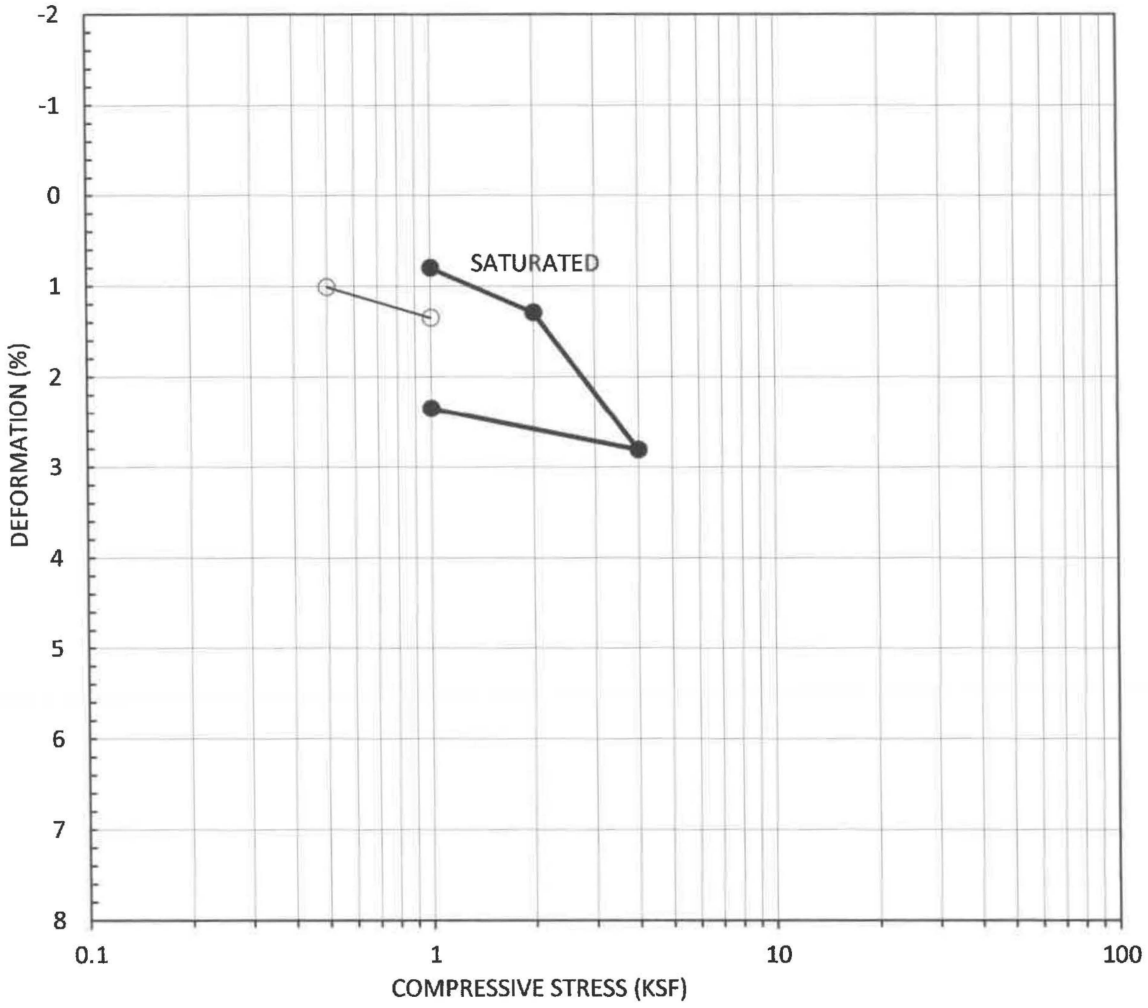
Symbol	Boring No.	Sample No.	Depth (Ft.)	Soil Type	Init. Moisture Content (%)	Init. Dry Density	Init. Void Ratio
○	B-1	2	5.0	SP-SM	2.5	102.6	0.642

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10/22 (ASTM D2435) Figure



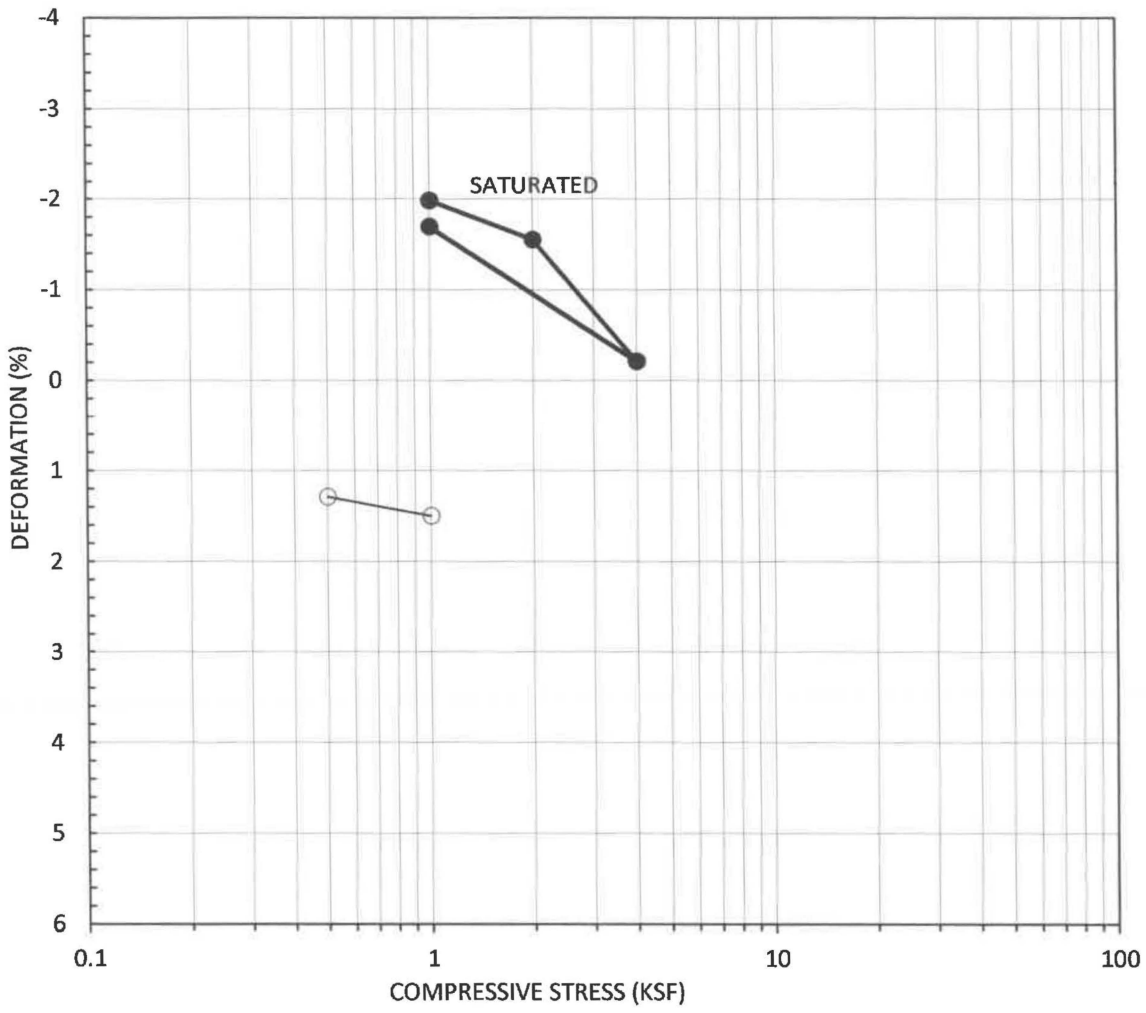
Symbol	Boring No.	Sample No.	Depth (Ft.)	Soil Type	Init. Moisture Content (%)	Init. Dry Density	Init. Void Ratio
○	B-4	1	2.0	CL	6.0	105.6	0.595

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10/22 (ASTM D2435) Figure



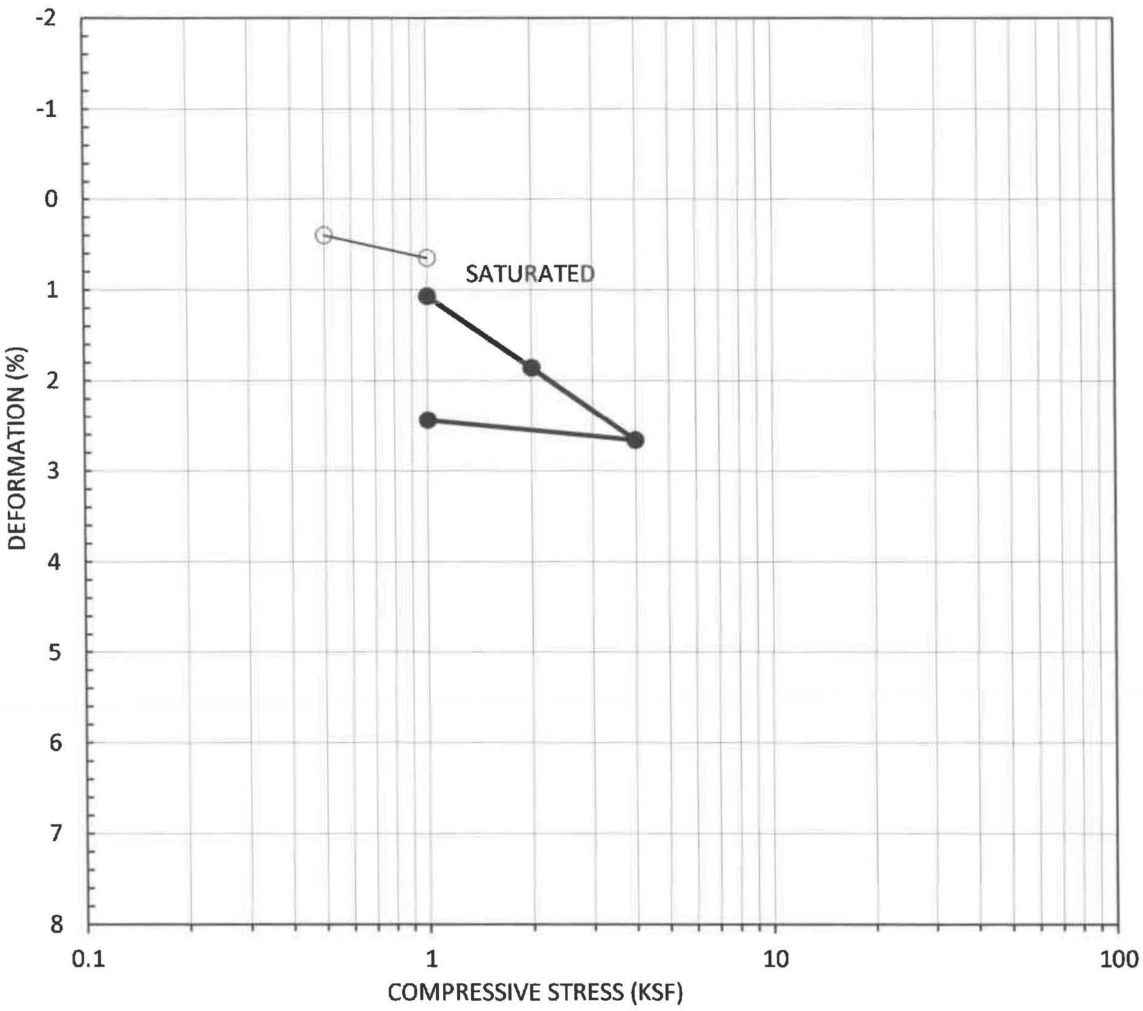
Symbol	Boring No.	Sample No.	Depth (Ft.)	Soil Type	Init. Moisture Content (%)	Init. Dry Density	Init. Void Ratio
○	B-6	3	10.0	CL	21.3	100.6	0.675

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10/22 (ASTM D2435) Figure



Symbol	Boring No.	Sample No.	Depth (Ft.)	Soil Type	Init. Moisture Content (%)	Init. Dry Density	Init. Void Ratio
○	B-7	1	5.0	SM	3.3	120.0	0.404

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CONSOLIDATION

10/22 (ASTM D2435) Figure

APPENDIX C

LIQUEFACTION INDUCED SETTLEMENT AND ANALYSIS

References:

1. "Evaluation of Settlement in Sands Due to Earthquake Shaking", Seed, et. al., ASCE Journal of Geotechnical Engineering, Page 861 – 878, Vol. 113, No. 8, August 1987
2. Civil Tech, (2012), "Liquefy Pro", Version 5.8n with USB key.

Settlement Due to Liquefaction

(Calculations shown on following sheets)

Boring B-1:

Saturated Settlement = 0 inches

Dry Sand Settlement: = 0.63 inches

Total Settlement: 0 +0.63 = 0.63 inches

Differential Settlement: 50% x total settlement: = 0.50 x 0.63 = 0.315 inches

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U.S. Geological Survey - Earthquake Hazards Program

Unified Hazard Tool

Please do not use this tool to obtain ground motion parameter values for the design code reference documents covered by the U.S. Seismic Design Maps web tools (e.g., the International Building Code and the ASCE 7 or 41 Standard). The values returned by the two applications are not identical.

Input

Edition

Dynamic: Conterminous U.S. 2014 (u…

Spectral Period

Peak Ground Acceleration

Latitude

Decimal degrees

34.016

Time Horizon

Return period in years

2475

Longitude

Decimal degrees, negative values for western longitudes

-117.375

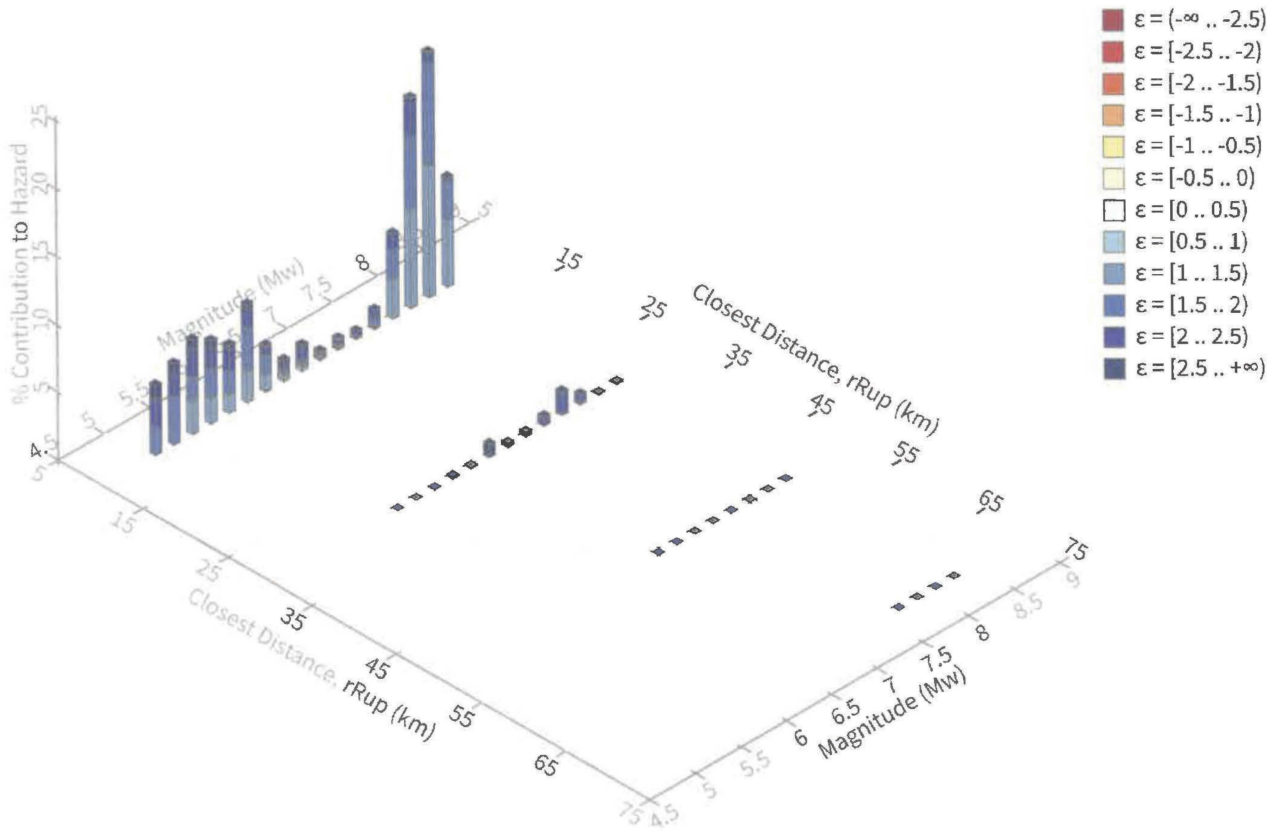
Site Class

259 m/s (Site class D)

Deaggregation

Component

Total



Summary statistics for, Deaggregation: Total

Deaggregation targets

Return period: 2475 yrs
Exceedance rate: 0.0004040404 yr^{-1}
PGA ground motion: 0.85942265 g

Recovered targets

Return period: 3219.5924 yrs
Exceedance rate: 0.00031059832 yr^{-1}

Totals

Binned: 100 %
Residual: 0 %
Trace: 0.05 %

Mean (over all sources)

m: 6.98
r: 11.45 km
ε₀: 1.75 σ

Mode (largest m-r bin)

m: 8.1
r: 12.55 km
ε₀: 1.51 σ
Contribution: 18.12 %

Mode (largest m-r-ε₀ bin)

m: 8.1
r: 9.75 km
ε₀: 1.27 σ
Contribution: 9.64 %

Discretization

r: min = 0.0, max = 1000.0, Δ = 20.0 km
m: min = 4.4, max = 9.4, Δ = 0.2
ε: min = -3.0, max = 3.0, Δ = 0.5 σ

Epsilon keys

ε0: [-∞ .. -2.5)
ε1: [-2.5 .. -2.0)
ε2: [-2.0 .. -1.5)
ε3: [-1.5 .. -1.0)
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ε11: [2.5 .. +∞]

Deaggregation Contributors

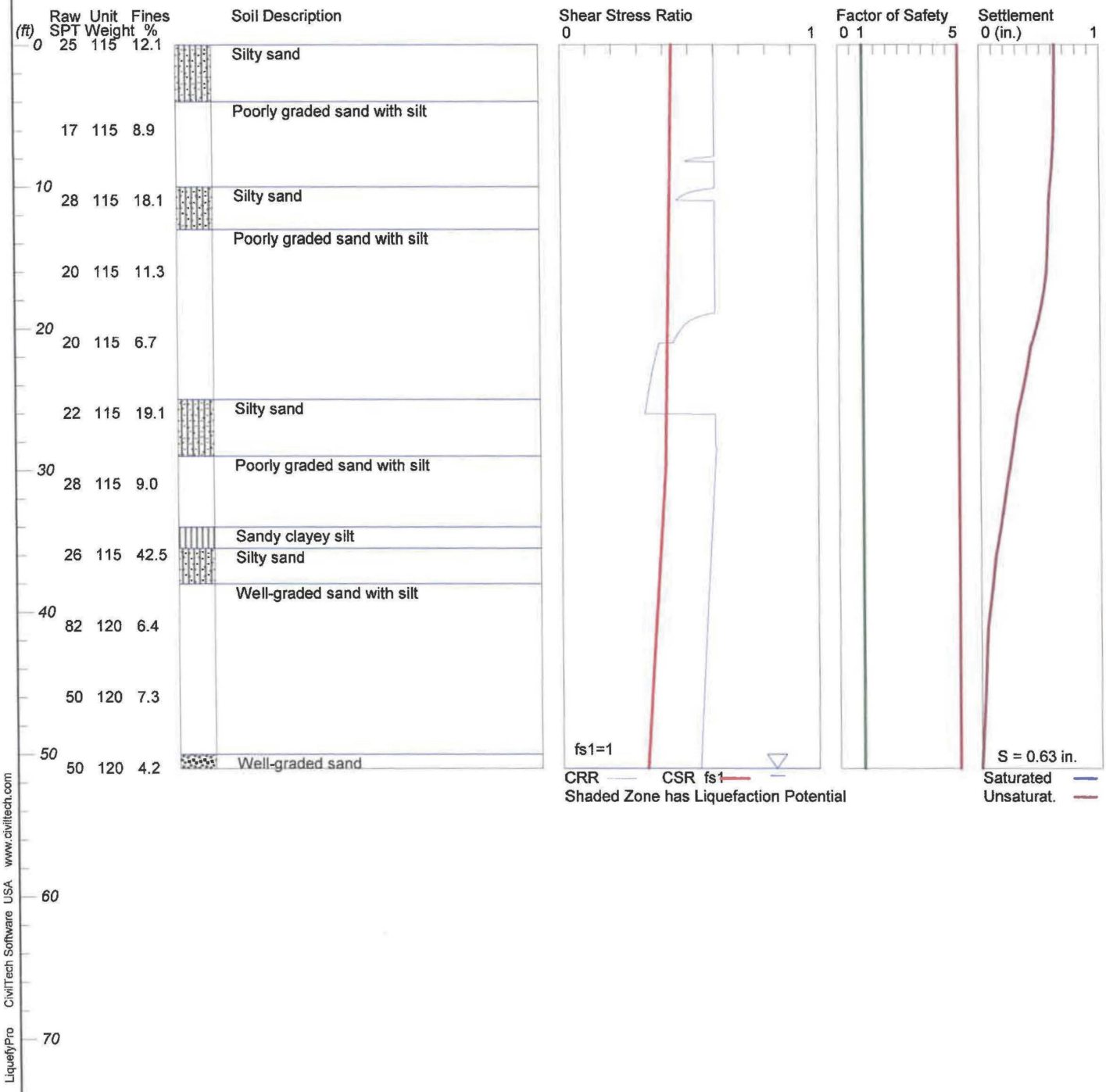
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UC33brAvg_FM31	System							28.64
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San Andreas (San Bernardino N) [5]		19.79	7.88	1.98	117.269°W	34.171°N	29.51	6.14
San Andreas (North Branch Mill Creek) [0]		19.24	8.01	1.77	117.270°W	34.171°N	29.12	1.49
UC33brAvg_FM32	System							28.57
San Jacinto (San Bernardino) [3]		9.73	8.02	1.40	117.296°W	34.074°N	48.59	16.43
San Andreas (San Bernardino N) [5]		19.79	7.88	1.98	117.269°W	34.171°N	29.51	6.19
San Andreas (North Branch Mill Creek) [0]		19.24	8.01	1.77	117.270°W	34.171°N	29.12	1.55
UC33brAvg_FM31 (opt)	Grid							21.40
PointSourceFinite: -117.375, 34.038		5.66	5.66	1.58	117.375°W	34.038°N	0.00	5.35
PointSourceFinite: -117.375, 34.038		5.66	5.66	1.58	117.375°W	34.038°N	0.00	5.35
PointSourceFinite: -117.375, 34.083		8.70	5.73	2.02	117.375°W	34.083°N	0.00	2.62
PointSourceFinite: -117.375, 34.083		8.70	5.73	2.02	117.375°W	34.083°N	0.00	2.62
PointSourceFinite: -117.375, 34.101		10.24	5.75	2.21	117.375°W	34.101°N	0.00	1.05
PointSourceFinite: -117.375, 34.101		10.24	5.75	2.21	117.375°W	34.101°N	0.00	1.05
UC33brAvg_FM32 (opt)	Grid							21.38
PointSourceFinite: -117.375, 34.038		5.66	5.66	1.58	117.375°W	34.038°N	0.00	5.35
PointSourceFinite: -117.375, 34.038		5.66	5.66	1.58	117.375°W	34.038°N	0.00	5.35
PointSourceFinite: -117.375, 34.083		8.70	5.73	2.02	117.375°W	34.083°N	0.00	2.62
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PointSourceFinite: -117.375, 34.101		10.24	5.75	2.21	117.375°W	34.101°N	0.00	1.05

LIQUEFACTION ANALYSIS

2240 Fleetwood Drive, Jurupa Valley

Hole No.=B-1 Water Depth=51 ft

Magnitude=6.98
Acceleration=0.668g



LiquefyPro CivilTech Software USA www.civiltech.com

****

LIQUEFACTION ANALYSIS CALCULATION DETAILS

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****

Font: Courier New, Regular, Size 8 is recommended for this report.

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Input File Name: Z:\Hank\Report\Soil\Report 2022\22257004L Fleetwood Jurupa Valley\data & figs\22257004L.liq

Title: 2240 Fleetwood Drive, Jurupa Valley

Subtitle:

Input Data:

Surface Elev.=

Hole No.=B-1

Depth of Hole=51.00 ft

Water Table during Earthquake= 51.00 ft

Water Table during In-Situ Testing= 51.00 ft

Max. Acceleration=0.67 g

Earthquake Magnitude=6.98

No-Liquefiable Soils: Based on Analysis

1. SPT or BPT Calculation.
2. Settlement Analysis Method: Ishihara / Yoshimine
3. Fines Correction for Liquefaction: Idriss/Seed
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: All zones*
6. Hammer Energy Ratio, Ce = 1.3
7. Borehole Diameter, Cb= 1.15
8. Sampling Method, Cs= 1
9. User request factor of safety (apply to CSR) , User= 1.3
   Plot one CSR curve (fs1=1)
10. Average two input data between two Depths: No

* Recommended Options

In-Situ Test Data:

Depth ft	SPT	Gamma pcf	Fines %
0.00	25.00	115.00	12.10
6.00	17.00	115.00	8.90
11.00	28.00	115.00	18.10
16.00	20.00	115.00	11.30
21.00	20.00	115.00	6.70
26.00	22.00	115.00	19.10
31.00	28.00	115.00	9.00
36.00	26.00	115.00	42.50
41.00	82.00	120.00	6.40
46.00	50.00	120.00	7.30
51.00	50.00	120.00	4.20

Output Results:

Calculation segment, dz=0.050 ft

User defined Print Interval, dp=1.00 ft

Peak Ground Acceleration (PGA), a_max = 0.67g

CSR Calculation:

Depth ft	gamma pcf	sigma atm	gamma' pcf	sigma' atm	rd	mZ g	a(z) g	CSR	x fs1	=CSRfs
0.00	115.00	0.000	115.00	0.000	1.00	0.000	0.668	0.43	1.00	0.43
1.00	115.00	0.054	115.00	0.054	1.00	0.000	0.668	0.43	1.00	0.43

2.00	115.00	0.109	115.00	0.109	1.00	0.000	0.668	0.43	1.00	0.43
3.00	115.00	0.163	115.00	0.163	0.99	0.000	0.668	0.43	1.00	0.43
4.00	115.00	0.217	115.00	0.217	0.99	0.000	0.668	0.43	1.00	0.43
5.00	115.00	0.272	115.00	0.272	0.99	0.000	0.668	0.43	1.00	0.43
6.00	115.00	0.326	115.00	0.326	0.99	0.000	0.668	0.43	1.00	0.43
7.00	115.00	0.380	115.00	0.380	0.98	0.000	0.668	0.43	1.00	0.43
8.00	115.00	0.435	115.00	0.435	0.98	0.000	0.668	0.43	1.00	0.43
9.00	115.00	0.489	115.00	0.489	0.98	0.000	0.668	0.43	1.00	0.43
10.00	115.00	0.543	115.00	0.543	0.98	0.000	0.668	0.42	1.00	0.42
11.00	115.00	0.598	115.00	0.598	0.97	0.000	0.668	0.42	1.00	0.42
12.00	115.00	0.652	115.00	0.652	0.97	0.000	0.668	0.42	1.00	0.42
13.00	115.00	0.706	115.00	0.706	0.97	0.000	0.668	0.42	1.00	0.42
14.00	115.00	0.761	115.00	0.761	0.97	0.000	0.668	0.42	1.00	0.42
15.00	115.00	0.815	115.00	0.815	0.97	0.000	0.668	0.42	1.00	0.42
16.00	115.00	0.869	115.00	0.869	0.96	0.000	0.668	0.42	1.00	0.42
17.00	115.00	0.924	115.00	0.924	0.96	0.000	0.668	0.42	1.00	0.42
18.00	115.00	0.978	115.00	0.978	0.96	0.000	0.668	0.42	1.00	0.42
19.00	115.00	1.033	115.00	1.033	0.96	0.000	0.668	0.41	1.00	0.41
20.00	115.00	1.087	115.00	1.087	0.95	0.000	0.668	0.41	1.00	0.41
21.00	115.00	1.141	115.00	1.141	0.95	0.000	0.668	0.41	1.00	0.41
22.00	115.00	1.196	115.00	1.196	0.95	0.000	0.668	0.41	1.00	0.41
23.00	115.00	1.250	115.00	1.250	0.95	0.000	0.668	0.41	1.00	0.41
24.00	115.00	1.304	115.00	1.304	0.94	0.000	0.668	0.41	1.00	0.41
25.00	115.00	1.359	115.00	1.359	0.94	0.000	0.668	0.41	1.00	0.41
26.00	115.00	1.413	115.00	1.413	0.94	0.000	0.668	0.41	1.00	0.41
27.00	115.00	1.467	115.00	1.467	0.94	0.000	0.668	0.41	1.00	0.41
28.00	115.00	1.522	115.00	1.522	0.93	0.000	0.668	0.41	1.00	0.41
29.00	115.00	1.576	115.00	1.576	0.93	0.000	0.668	0.40	1.00	0.40
30.00	115.00	1.630	115.00	1.630	0.93	0.000	0.668	0.40	1.00	0.40
31.00	115.00	1.685	115.00	1.685	0.92	0.000	0.668	0.40	1.00	0.40
32.00	115.00	1.739	115.00	1.739	0.91	0.000	0.668	0.40	1.00	0.40
33.00	115.00	1.793	115.00	1.793	0.91	0.000	0.668	0.39	1.00	0.39
34.00	115.00	1.848	115.00	1.848	0.90	0.000	0.668	0.39	1.00	0.39
35.00	115.00	1.902	115.00	1.902	0.89	0.000	0.668	0.39	1.00	0.39
36.00	115.00	1.956	115.00	1.956	0.88	0.000	0.668	0.38	1.00	0.38
37.00	115.00	2.011	115.00	2.011	0.87	0.000	0.668	0.38	1.00	0.38
38.00	115.00	2.065	115.00	2.065	0.86	0.000	0.668	0.38	1.00	0.38
39.00	115.00	2.119	115.00	2.119	0.86	0.000	0.668	0.37	1.00	0.37
40.00	115.00	2.174	115.00	2.174	0.85	0.000	0.668	0.37	1.00	0.37
41.00	115.00	2.228	115.00	2.228	0.84	0.000	0.668	0.36	1.00	0.36
42.00	120.00	2.285	120.00	2.285	0.83	0.000	0.668	0.36	1.00	0.36
43.00	120.00	2.341	120.00	2.341	0.82	0.000	0.668	0.36	1.00	0.36
44.00	120.00	2.398	120.00	2.398	0.82	0.000	0.668	0.35	1.00	0.35
45.00	120.00	2.455	120.00	2.455	0.81	0.000	0.668	0.35	1.00	0.35
46.00	120.00	2.511	120.00	2.511	0.80	0.000	0.668	0.35	1.00	0.35
47.00	120.00	2.568	120.00	2.568	0.79	0.000	0.668	0.34	1.00	0.34
48.00	120.00	2.625	120.00	2.625	0.78	0.000	0.668	0.34	1.00	0.34
49.00	120.00	2.682	120.00	2.682	0.78	0.000	0.668	0.34	1.00	0.34
50.00	120.00	2.738	120.00	2.738	0.77	0.000	0.668	0.33	1.00	0.33
51.00	120.00	2.795	120.00	2.795	0.76	0.000	0.668	0.33	1.00	0.33

CSR is based on water table at 51.00 during earthquake

CRR Calculation from SPT or BPT data:

Depth ft	SPT	Cebs	Cr	sigma' atm	Cn	(N1)60	Fines %	d(N1)60	(N1)60f	CRR7.5
0.00	25.00	1.49	0.75	0.000	1.70	47.65	12.10	3.12	50.77	0.50
1.00	25.00	1.49	0.75	0.054	1.70	47.65	12.10	3.12	50.77	0.50
2.00	25.00	1.49	0.75	0.109	1.70	47.65	12.10	3.12	50.77	0.50
3.00	25.00	1.49	0.75	0.163	1.70	47.65	12.10	3.12	50.77	0.50
4.00	25.00	1.49	0.75	0.217	1.70	47.65	12.10	3.12	50.77	0.50
5.00	25.00	1.49	0.75	0.272	1.70	47.65	12.10	3.12	50.77	0.50
6.00	17.00	1.49	0.75	0.326	1.70	32.40	8.90	1.06	33.47	0.50

7.00	17.00	1.49	0.75	0.380	1.62	30.90	8.90	1.04	31.94	0.50
8.00	17.00	1.49	0.75	0.435	1.52	28.91	8.90	1.01	29.92	0.44
9.00	17.00	1.49	0.85	0.489	1.43	30.89	8.90	1.04	31.93	0.50
10.00	17.00	1.49	0.85	0.543	1.36	29.30	8.90	1.01	30.32	0.50
11.00	28.00	1.49	0.85	0.598	1.29	46.02	18.10	6.34	52.36	0.50
12.00	28.00	1.49	0.85	0.652	1.24	44.06	18.10	6.21	50.27	0.50
13.00	28.00	1.49	0.85	0.706	1.19	42.33	18.10	6.09	48.42	0.50
14.00	28.00	1.49	0.85	0.761	1.15	40.79	18.10	5.99	46.78	0.50
15.00	28.00	1.49	0.95	0.815	1.11	44.05	18.10	6.21	50.25	0.50
16.00	20.00	1.49	0.95	0.869	1.07	30.46	11.30	2.17	32.63	0.50
17.00	20.00	1.49	0.95	0.924	1.04	29.55	11.30	2.14	31.69	0.50
18.00	20.00	1.49	0.95	0.978	1.01	28.72	11.30	2.12	30.84	0.50
19.00	20.00	1.49	0.95	1.033	0.98	27.95	11.30	2.09	30.05	0.47
20.00	20.00	1.49	0.95	1.087	0.96	27.25	11.30	2.08	29.32	0.39
21.00	20.00	1.49	0.95	1.141	0.94	26.59	11.30	2.06	28.65	0.36
22.00	20.00	1.49	0.95	1.196	0.91	25.98	6.70	0.28	26.25	0.30
23.00	20.00	1.49	0.95	1.250	0.89	25.41	6.70	0.27	25.68	0.29
24.00	20.00	1.49	0.95	1.304	0.88	24.87	6.70	0.27	25.14	0.28
25.00	20.00	1.49	0.95	1.359	0.86	24.37	6.70	0.26	24.63	0.28
26.00	20.00	1.49	0.95	1.413	0.84	23.90	6.70	0.26	24.16	0.27
27.00	22.00	1.49	0.95	1.467	0.83	25.79	19.10	5.35	31.14	0.50
28.00	22.00	1.49	1.00	1.522	0.81	26.66	19.10	5.41	32.08	0.50
29.00	22.00	1.49	1.00	1.576	0.80	26.20	19.10	5.38	31.58	0.50
30.00	22.00	1.49	1.00	1.630	0.78	25.76	19.10	5.35	31.10	0.50
31.00	22.00	1.49	1.00	1.685	0.77	25.34	19.10	5.31	30.65	0.50
32.00	28.00	1.49	1.00	1.739	0.76	31.74	9.00	1.10	32.84	0.50
33.00	28.00	1.49	1.00	1.793	0.75	31.26	9.00	1.09	32.35	0.50
34.00	28.00	1.49	1.00	1.848	0.74	30.80	9.00	1.08	31.88	0.50
35.00	28.00	1.49	1.00	1.902	0.73	30.35	9.00	1.07	31.43	0.50
36.00	28.00	1.49	1.00	1.956	0.71	29.93	9.00	1.07	30.99	0.50
37.00	26.00	1.49	1.00	2.011	0.71	27.41	42.50	10.48	37.89	0.50
38.00	26.00	1.49	1.00	2.065	0.70	27.05	42.50	10.41	37.46	0.50
39.00	26.00	1.49	1.00	2.119	0.69	26.70	42.50	10.34	37.04	0.50
40.00	26.00	1.49	1.00	2.174	0.68	26.36	42.50	10.27	36.64	0.50
41.00	26.00	1.49	1.00	2.228	0.67	26.04	42.50	10.21	36.25	0.50
42.00	82.00	1.49	1.00	2.285	0.66	81.10	6.40	0.56	81.66	0.50
43.00	82.00	1.49	1.00	2.341	0.65	80.12	6.40	0.55	80.67	0.50
44.00	82.00	1.49	1.00	2.398	0.65	79.16	6.40	0.55	79.71	0.50
45.00	82.00	1.49	1.00	2.455	0.64	78.24	6.40	0.54	78.78	0.50
46.00	82.00	1.49	1.00	2.511	0.63	77.36	6.40	0.54	77.89	0.50
47.00	50.00	1.49	1.00	2.568	0.62	46.64	7.30	0.62	47.26	0.50
48.00	50.00	1.49	1.00	2.625	0.62	46.14	7.30	0.61	46.75	0.50
49.00	50.00	1.49	1.00	2.682	0.61	45.65	7.30	0.61	46.26	0.50
50.00	50.00	1.49	1.00	2.738	0.60	45.17	7.30	0.60	45.78	0.50
51.00	50.00	1.49	1.00	2.795	0.60	44.71	7.30	0.60	45.31	0.50

CRR is based on water table at 51.00 during In-Situ Testing

Factor of Safety, - Earthquake Magnitude= 6.98:

Depth ft	sigC' atm	CRR7.5	x Ksig	=CRRv	x MSF	=CRRm	CSRfs	F.S.=CRRm/CSRfs
0.00	0.00	0.50	1.00	0.50	1.20	0.60	0.43	5.00
1.00	0.04	0.50	1.00	0.50	1.20	0.60	0.43	5.00
2.00	0.07	0.50	1.00	0.50	1.20	0.60	0.43	5.00
3.00	0.11	0.50	1.00	0.50	1.20	0.60	0.43	5.00
4.00	0.14	0.50	1.00	0.50	1.20	0.60	0.43	5.00
5.00	0.18	0.50	1.00	0.50	1.20	0.60	0.43	5.00
6.00	0.21	0.50	1.00	0.50	1.20	0.60	0.43	5.00
7.00	0.25	0.50	1.00	0.50	1.20	0.60	0.43	5.00
8.00	0.28	0.44	1.00	0.44	1.20	0.53	0.43	5.00
9.00	0.32	0.50	1.00	0.50	1.20	0.60	0.43	5.00
10.00	0.35	0.50	1.00	0.50	1.20	0.60	0.42	5.00
11.00	0.39	0.50	1.00	0.50	1.20	0.60	0.42	5.00

12.00	0.42	0.50	1.00	0.50	1.20	0.60	0.42	5.00
13.00	0.46	0.50	1.00	0.50	1.20	0.60	0.42	5.00
14.00	0.49	0.50	1.00	0.50	1.20	0.60	0.42	5.00
15.00	0.53	0.50	1.00	0.50	1.20	0.60	0.42	5.00
16.00	0.57	0.50	1.00	0.50	1.20	0.60	0.42	5.00
17.00	0.60	0.50	1.00	0.50	1.20	0.60	0.42	5.00
18.00	0.64	0.50	1.00	0.50	1.20	0.60	0.42	5.00
19.00	0.67	0.47	1.00	0.47	1.20	0.57	0.41	5.00
20.00	0.71	0.39	1.00	0.39	1.20	0.47	0.41	5.00
21.00	0.74	0.36	1.00	0.36	1.20	0.43	0.41	5.00
22.00	0.78	0.30	1.00	0.30	1.20	0.37	0.41	5.00
23.00	0.81	0.29	1.00	0.29	1.20	0.35	0.41	5.00
24.00	0.85	0.28	1.00	0.28	1.20	0.34	0.41	5.00
25.00	0.88	0.28	1.00	0.28	1.20	0.33	0.41	5.00
26.00	0.92	0.27	1.00	0.27	1.20	0.32	0.41	5.00
27.00	0.95	0.50	1.00	0.50	1.20	0.60	0.41	5.00
28.00	0.99	0.50	1.00	0.50	1.20	0.60	0.41	5.00
29.00	1.02	0.50	1.00	0.50	1.20	0.60	0.40	5.00
30.00	1.06	0.50	1.00	0.50	1.20	0.60	0.40	5.00
31.00	1.10	0.50	0.99	0.50	1.20	0.60	0.40	5.00
32.00	1.13	0.50	0.99	0.49	1.20	0.59	0.40	5.00
33.00	1.17	0.50	0.98	0.49	1.20	0.59	0.39	5.00
34.00	1.20	0.50	0.98	0.49	1.20	0.59	0.39	5.00
35.00	1.24	0.50	0.97	0.48	1.20	0.58	0.39	5.00
36.00	1.27	0.50	0.96	0.48	1.20	0.58	0.38	5.00
37.00	1.31	0.50	0.96	0.48	1.20	0.58	0.38	5.00
38.00	1.34	0.50	0.95	0.48	1.20	0.57	0.38	5.00
39.00	1.38	0.50	0.95	0.47	1.20	0.57	0.37	5.00
40.00	1.41	0.50	0.94	0.47	1.20	0.57	0.37	5.00
41.00	1.45	0.50	0.94	0.47	1.20	0.56	0.36	5.00
42.00	1.49	0.50	0.93	0.47	1.20	0.56	0.36	5.00
43.00	1.52	0.50	0.93	0.46	1.20	0.56	0.36	5.00
44.00	1.56	0.50	0.93	0.46	1.20	0.56	0.35	5.00
45.00	1.60	0.50	0.92	0.46	1.20	0.55	0.35	5.00
46.00	1.63	0.50	0.92	0.46	1.20	0.55	0.35	5.00
47.00	1.67	0.50	0.91	0.46	1.20	0.55	0.34	5.00
48.00	1.71	0.50	0.91	0.45	1.20	0.54	0.34	5.00
49.00	1.74	0.50	0.90	0.45	1.20	0.54	0.34	5.00
50.00	1.78	0.50	0.90	0.45	1.20	0.54	0.33	5.00
51.00	1.82	0.50	0.89	0.45	1.20	0.54	0.33	5.00

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

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

Depth ft	Ic	qc/N60	qc1 atm	(N1)60	Fines %	d(N1)60	(N1)60s
0.00	-	-	-	50.77	12.10	0.00	50.77
1.00	-	-	-	50.77	12.10	0.00	50.77
2.00	-	-	-	50.77	12.10	0.00	50.77
3.00	-	-	-	50.77	12.10	0.00	50.77
4.00	-	-	-	50.77	12.10	0.00	50.77
5.00	-	-	-	50.77	12.10	0.00	50.77
6.00	-	-	-	33.47	8.90	0.00	33.47
7.00	-	-	-	31.94	8.90	0.00	31.94
8.00	-	-	-	29.92	8.90	0.00	29.92
9.00	-	-	-	31.93	8.90	0.00	31.93
10.00	-	-	-	30.32	8.90	0.00	30.32
11.00	-	-	-	52.36	18.10	0.00	52.36
12.00	-	-	-	50.27	18.10	0.00	50.27

13.00	-	-	-	48.42	18.10	0.00	48.42
14.00	-	-	-	46.78	18.10	0.00	46.78
15.00	-	-	-	50.25	18.10	0.00	50.25
16.00	-	-	-	32.63	11.30	0.00	32.63
17.00	-	-	-	31.69	11.30	0.00	31.69
18.00	-	-	-	30.84	11.30	0.00	30.84
19.00	-	-	-	30.05	11.30	0.00	30.05
20.00	-	-	-	29.32	11.30	0.00	29.32
21.00	-	-	-	28.65	11.30	0.00	28.65
22.00	-	-	-	26.25	6.70	0.00	26.25
23.00	-	-	-	25.68	6.70	0.00	25.68
24.00	-	-	-	25.14	6.70	0.00	25.14
25.00	-	-	-	24.63	6.70	0.00	24.63
26.00	-	-	-	24.16	6.70	0.00	24.16
27.00	-	-	-	31.14	19.10	0.00	31.14
28.00	-	-	-	32.08	19.10	0.00	32.08
29.00	-	-	-	31.58	19.10	0.00	31.58
30.00	-	-	-	31.10	19.10	0.00	31.10
31.00	-	-	-	30.65	19.10	0.00	30.65
32.00	-	-	-	32.84	9.00	0.00	32.84
33.00	-	-	-	32.35	9.00	0.00	32.35
34.00	-	-	-	31.88	9.00	0.00	31.88
35.00	-	-	-	31.43	9.00	0.00	31.43
36.00	-	-	-	30.99	9.00	0.00	30.99
37.00	-	-	-	37.89	42.50	0.00	37.89
38.00	-	-	-	37.46	42.50	0.00	37.46
39.00	-	-	-	37.04	42.50	0.00	37.04
40.00	-	-	-	36.64	42.50	0.00	36.64
41.00	-	-	-	36.25	42.50	0.00	36.25
42.00	-	-	-	81.66	6.40	0.00	81.66
43.00	-	-	-	80.67	6.40	0.00	80.67
44.00	-	-	-	79.71	6.40	0.00	79.71
45.00	-	-	-	78.78	6.40	0.00	78.78
46.00	-	-	-	77.89	6.40	0.00	77.89
47.00	-	-	-	47.26	7.30	0.00	47.26
48.00	-	-	-	46.75	7.30	0.00	46.75
49.00	-	-	-	46.26	7.30	0.00	46.26
50.00	-	-	-	45.78	7.30	0.00	45.78
51.00	-	-	-	45.31	7.30	0.00	45.31

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

Settlement of Saturated Sands:
Settlement Analysis Method: Ishihara / Yoshimine

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

No Settlement of Saturated Sands

Settlement of Saturated Sands=0.000 in.
qc1 and (N1)60 is after fines correction in liquefaction analysis
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=1.00 ft
S is cumulated settlement at this depth

Settlement of Unsaturated Sands:

Depth ft	sigma' atm	sigC' atm	(N1)60s	CSRsf	Gmax atm	g*Ge/Gm	g_eff	ec7.5 %	Cec	ec %	dsz in.	dsp in.	S in.
50.95	2.79	1.81	45.33	0.33	2145.51	4.3E-4	0.0908	0.0287	0.92	0.0265	3.18E-4	0.000	0.000
50.00	2.74	1.78	45.78	0.33	2131.59	4.3E-4	0.0903	0.0285	0.92	0.0263	3.16E-4	0.006	0.006
49.00	2.68	1.74	46.26	0.34	2116.74	4.3E-4	0.0896	0.0283	0.92	0.0262	3.14E-4	0.006	0.013

48.00	2.62	1.71	46.75	0.34	2101.68	4.2E-4	0.0889	0.0281	0.92	0.0260	3.11E-4	0.006	0.019
47.00	2.57	1.67	47.26	0.34	2086.40	4.2E-4	0.0881	0.0279	0.92	0.0257	3.09E-4	0.006	0.025
46.00	2.51	1.63	77.89	0.35	2436.69	3.6E-4	0.0634	0.0200	0.92	0.0185	2.22E-4	0.006	0.031
45.00	2.45	1.60	78.78	0.35	2418.19	3.6E-4	0.0628	0.0199	0.92	0.0183	2.20E-4	0.004	0.036
44.00	2.40	1.56	79.71	0.35	2399.41	3.5E-4	0.0622	0.0197	0.92	0.0181	2.18E-4	0.004	0.040
43.00	2.34	1.52	80.67	0.36	2380.33	3.5E-4	0.0615	0.0194	0.92	0.0180	2.15E-4	0.004	0.044
42.00	2.28	1.49	81.66	0.36	2360.94	3.5E-4	0.0808	0.0256	0.92	0.0236	2.83E-4	0.005	0.049
41.00	2.23	1.45	36.25	0.36	1779.01	4.6E-4	0.1640	0.0664	0.92	0.0613	7.35E-4	0.006	0.055
40.00	2.17	1.41	36.64	0.37	1763.43	4.5E-4	0.1609	0.0636	0.92	0.0587	7.04E-4	0.014	0.070
39.00	2.12	1.38	37.04	0.37	1747.60	4.5E-4	0.1576	0.0608	0.92	0.0561	6.73E-4	0.014	0.083
38.00	2.07	1.34	37.46	0.38	1731.52	4.5E-4	0.1542	0.0579	0.92	0.0534	6.41E-4	0.013	0.097
37.00	2.01	1.31	37.89	0.38	1715.18	4.4E-4	0.1507	0.0550	0.92	0.0508	6.09E-4	0.012	0.109
36.00	1.96	1.27	30.99	0.38	1582.29	4.7E-4	0.1825	0.0987	0.92	0.0911	1.09E-3	0.012	0.121
35.00	1.90	1.24	31.43	0.39	1567.37	4.7E-4	0.1771	0.0937	0.92	0.0865	1.04E-3	0.021	0.143
34.00	1.85	1.20	31.88	0.39	1552.16	4.6E-4	0.1716	0.0887	0.92	0.0819	9.82E-4	0.020	0.163
33.00	1.79	1.17	32.35	0.39	1536.65	4.6E-4	0.1660	0.0837	0.92	0.0772	9.27E-4	0.019	0.182
32.00	1.74	1.13	32.84	0.40	1520.83	4.5E-4	0.1603	0.0787	0.92	0.0727	8.72E-4	0.018	0.200
31.00	1.68	1.10	30.65	0.40	1462.95	4.6E-4	0.1683	0.0926	0.92	0.0855	1.03E-3	0.017	0.217
30.00	1.63	1.06	31.10	0.40	1446.15	4.6E-4	0.1621	0.0872	0.92	0.0805	9.66E-4	0.020	0.237
29.00	1.58	1.02	31.58	0.40	1429.01	4.5E-4	0.1529	0.0802	0.92	0.0741	8.89E-4	0.018	0.255
28.00	1.52	0.99	32.08	0.41	1411.49	4.4E-4	0.1441	0.0737	0.92	0.0680	8.16E-4	0.017	0.272
27.00	1.47	0.95	31.14	0.41	1372.51	4.3E-4	0.1416	0.0760	0.92	0.0702	8.42E-4	0.017	0.290
26.00	1.41	0.92	24.16	0.41	1237.60	4.7E-4	0.1738	0.1338	0.92	0.1235	1.48E-3	0.017	0.306
25.00	1.36	0.88	24.63	0.41	1221.49	4.5E-4	0.1616	0.1212	0.92	0.1119	1.34E-3	0.028	0.335
24.00	1.30	0.85	25.14	0.41	1204.95	4.4E-4	0.1501	0.1096	0.92	0.1011	1.21E-3	0.025	0.360
23.00	1.25	0.81	25.68	0.41	1187.94	4.3E-4	0.1392	0.0988	0.92	0.0912	1.09E-3	0.023	0.383
22.00	1.20	0.78	26.25	0.41	1170.43	4.2E-4	0.1290	0.0888	0.92	0.0820	9.84E-4	0.021	0.404
21.00	1.14	0.74	28.65	0.41	1177.22	4.0E-4	0.2826	0.1723	0.92	0.1590	1.91E-3	0.026	0.429
20.00	1.09	0.71	29.32	0.41	1157.80	3.9E-4	0.2445	0.1440	0.92	0.1330	1.60E-3	0.035	0.464
19.00	1.03	0.67	30.05	0.41	1137.73	3.8E-4	0.2113	0.1199	0.92	0.1107	1.33E-3	0.029	0.493
18.00	0.98	0.64	30.84	0.42	1116.96	3.6E-4	0.1823	0.0994	0.92	0.0917	1.10E-3	0.024	0.517
17.00	0.92	0.60	31.69	0.42	1095.44	3.5E-4	0.1570	0.0819	0.92	0.0756	9.07E-4	0.020	0.537
16.00	0.87	0.57	32.63	0.42	1073.07	3.4E-4	0.1351	0.0671	0.92	0.0619	7.43E-4	0.016	0.554
15.00	0.82	0.53	50.25	0.42	1199.70	2.8E-4	0.0746	0.0236	0.92	0.0218	2.61E-4	0.006	0.559
14.00	0.76	0.49	46.78	0.42	1131.72	2.8E-4	0.0728	0.0230	0.92	0.0213	2.55E-4	0.005	0.564
13.00	0.71	0.46	48.42	0.42	1103.16	2.7E-4	0.0639	0.0202	0.92	0.0186	2.24E-4	0.005	0.569
12.00	0.65	0.42	50.27	0.42	1073.16	2.6E-4	0.0560	0.0177	0.92	0.0163	1.96E-4	0.004	0.573
11.00	0.60	0.39	52.36	0.42	1041.51	2.4E-4	0.0490	0.0155	0.92	0.0143	1.72E-4	0.004	0.577
10.00	0.54	0.35	30.32	0.42	827.85	2.8E-4	0.0699	0.0391	0.92	0.0361	4.33E-4	0.010	0.587
9.00	0.49	0.32	31.93	0.43	799.03	2.6E-4	0.0948	0.0489	0.92	0.0451	5.41E-4	0.014	0.601
8.00	0.43	0.28	29.92	0.43	737.17	2.5E-4	0.0772	0.0441	0.92	0.0407	4.89E-4	0.009	0.610
7.00	0.38	0.25	31.94	0.43	704.79	2.3E-4	0.0533	0.0274	0.92	0.0253	3.04E-4	0.008	0.618
6.00	0.33	0.21	33.47	0.43	662.72	2.1E-4	0.0423	0.0201	0.92	0.0185	2.22E-4	0.005	0.623
5.00	0.27	0.18	50.77	0.43	695.03	1.7E-4	0.0308	0.0097	0.92	0.0090	1.08E-4	0.002	0.625
4.00	0.22	0.14	50.77	0.43	621.66	1.5E-4	0.0295	0.0093	0.92	0.0086	1.03E-4	0.002	0.627
3.00	0.16	0.11	50.77	0.43	538.37	1.3E-4	0.0242	0.0076	0.92	0.0071	8.47E-5	0.002	0.629
2.00	0.11	0.07	50.77	0.43	439.59	1.1E-4	0.0221	0.0070	0.92	0.0064	7.74E-5	0.002	0.631
1.00	0.05	0.04	50.77	0.43	310.85	7.6E-5	0.0129	0.0041	0.92	0.0038	4.52E-5	0.001	0.632
0.00	0.00	0.00	50.77	0.43	4.22	1.0E-6	0.0010	0.0003	0.92	0.0003	3.56E-6	0.001	0.632

Settlement of Unsaturated Sands=0.632 in.
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=1.00 ft
S is cumulated settlement at this depth

Total Settlement of Saturated and Unsaturated Sands=0.632 in.
Differential Settlement=0.316 to 0.417 in.

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

1 atm (atmosphere) = 1.0581 tsf(1 tsf = 1 ton/ft2 = 2 kip/ft2)
1 atm (atmosphere) = 101.325 kPa(1 kPa = 1 kN/m2 = 0.001 Mpa)
SPT Field data from Standard Penetration Test (SPT)
BPT Field data from Becker Penetration Test (BPT)

qc	Field data from Cone Penetration Test (CPT) [atm (tsf)]
fs	Friction from CPT testing [atm (tsf)]
Rf	Ratio of fs/qc (%)
gamma	Total unit weight of soil
gamma'	Effective unit weight of soil
Fines	Fines content [%]
D50	Mean grain size
Dr	Relative Density
sigma	Total vertical stress [atm]
sigma'	Effective vertical stress [atm]
sigC'	Effective confining pressure [atm]
rd	Acceleration reduction coefficient by Seed
a_max.	Peak Ground Acceleration (PGA) in ground surface
mZ	Linear acceleration reduction coefficient X depth
a_min.	Minimum acceleration under linear reduction, mZ
CRRv	CRR after overburden stress correction, CRRv=CRR7.5 * Ksig
CRR7.5	Cyclic resistance ratio (M=7.5)
Ksig	Overburden stress correction factor for CRR7.5
CRRm	After magnitude scaling correction CRRm=CRRv * MSF
MSF	Magnitude scaling factor from M=7.5 to user input M
CSR	Cyclic stress ratio induced by earthquake
CSRfs	CSRfs=CSR*fs1 (Default fs1=1)
fs1	First CSR curve in graphic defined in #9 of Advanced page
fs2	2nd CSR curve in graphic defined in #9 of Advanced page
F.S.	Calculated factor of safety against liquefaction F.S.=CRRm/CSRsf
Cebs	Energy Ratio, Borehole Dia., and Sampling Method Corrections
Cr	Rod Length Corrections
Cn	Overburden Pressure Correction
(N1)60	SPT after corrections, (N1)60=SPT * Cr * Cn * Cebs
d(N1)60	Fines correction of SPT
(N1)60f	(N1)60 after fines corrections, (N1)60f=(N1)60 + d(N1)60
Cq	Overburden stress correction factor
qc1	CPT after Overburden stress correction
dqc1	Fines correction of CPT
qc1f	CPT after Fines and Overburden correction, qc1f=qc1 + dqc1
qc1n	CPT after normalization in Robertson's method
Kc	Fine correction factor in Robertson's Method
qc1f	CPT after Fines correction in Robertson's Method
Ic	Soil type index in Suzuki's and Robertson's Methods
(N1)60s	(N1)60 after settlement fines corrections
CSRm	After magnitude scaling correction for Settlement calculation CSRm=CSRsf / MSF*
CSRfs	Cyclic stress ratio induced by earthquake with user inputed fs
MSF*	Scaling factor from CSR, MSF*=1, based on Item 2 of Page C.
ec	Volumetric strain for saturated sands
dz	Calculation segment, dz=0.050 ft
dsz	Settlement in each segment, dz
dp	User defined print interval
dsp	Settlement in each print interval, dp
Gmax	Shear Modulus at low strain
g_eff	gamma_eff, Effective shear Strain
g*Ge/Gm	gamma_eff * G_eff/G_max, Strain-modulus ratio
ec7.5	Volumetric Strain for magnitude=7.5
Cec	Magnitude correction factor for any magnitude
ec	Volumetric strain for unsaturated sands, ec=Cec * ec7.5
NoLiq	No-Liquefy Soils

References:

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

Report No. EERC 2003-06 by R.B Seed and etc. April 2003.

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

APPENDIX D

SLOPE STABILITY ANALYSES

Simplified Bishop's method was used to analyze the existing and proposed slope conditions as depicted on the Cross Section A-A' (Figure 2b). Conditions analyzed for slope stability analyses were performed for search ranges. Seismic coefficient of 0.20 g (g: gravity) was used in the seismic condition. Stability calculations and sections are presented on the plates following the report and the results of EGL's slope stability analyses of existing and proposed slope conditions are briefly described with corresponding geotechnical recommendations.

Earth Materials	Unit Weight (pcf)	Friction Angle (Degree)	Cohesion (psf)
Soil, Ultimate[+] (Qds; B-1 @ 25.0')	120.0	29	168
Proposed Compacted Fill, Ultimate (CAf; Assumed)	125.0	35	200

Noted: [+]Lowest ultimate soil shear values were used for static and seismic conditions for existing and proposed slope conditions.

EXISTING SLOPE CONDITION

Calculated Conditions	Analyzed Method	Factor of Safety (Static/Seismic)	Preliminary Geotechnical Recommendation
existing descending slope at 60 feet high	Simplified Bishop's	1.63 / 1.13	Stable as Exists

PROPOSED SLOPE CONDITION

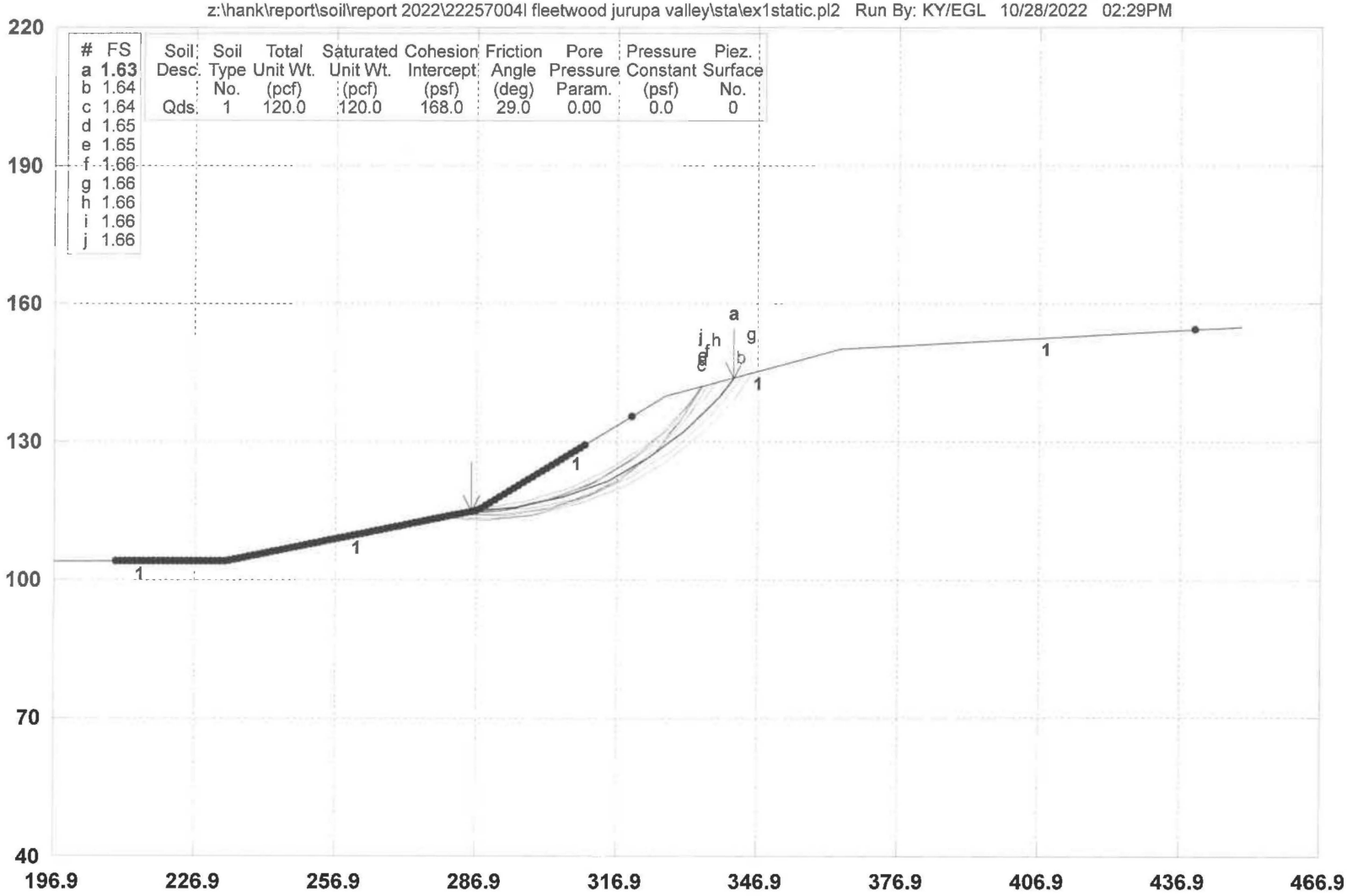
Calculated Conditions	Analyzed Method	Factor of Safety (Static/Seismic)	Preliminary Geotechnical Recommendation
Proposed slope with building loads	Simplified Bishop's	1.63 / 1.13	Stable as Proposed

SURFICIAL SLOPE STABILITY

Calculated Conditions	Factor of Safety (Static/Seismic)	Conclusions and Preliminary Geotechnical Recommendation
Natural Soils on Existing 1.5:1 – 2.9:1 (H: V) Slopes	1.20 / ----- 0.69 / -----	Unstable as Existed (see Section 6.4 for Recommendation)

2240 Fleetwood Drive, Jurupa Valley, Section A-A', Exisiting Slope_Static

z:\hank\report\soil\report 2022\22257004l fleetwood jurupa valley\sta\ex1static.pl2 Run By: KY/EGL 10/28/2022 02:29PM



#	FS
a	1.63
b	1.64
c	1.64
d	1.65
e	1.65
f	1.66
g	1.66
h	1.66
i	1.66
j	1.66

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Qds.	1	120.0	120.0	168.0	29.0	0.00	0.0	0

GSTABL7 v.2 FSmin=1.63

Safety Factors Are Calculated By The Modified Bishop Method

*** GSTABL7 ***

** GSTABL7 by Dr. Garry H. Gregory, Ph.D.,P.E.,D.GE **

** Original Version 1.0, January 1996; Current Ver. 2.005.3, Feb. 2013 **

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SLOPE STABILITY ANALYSIS SYSTEM Modified Bishop, Simplified Janbu, or GLE Method of Slices. (Includes Spencer & Morgenstern-Price Type Analysis) Including Pier/Pile, Reinforcement, Soil Nail, Tieback, Nonlinear Undrained Shear Strength, Curved Phi Envelope, Anisotropic Soil, Fiber-Reinforced Soil, Boundary Loads, Water Surfaces, Pseudo-Static & Newmark Earthquake, and Applied Forces.

****

Analysis Run Date: 10/28/2022
Time of Run: 02:29PM
Run By: KY/EGL
Input Data Filename: Z:\Hank\Report\Soil\Report 2022\22257004L Fleetwood Jurupa Valley\sta\ex1static.in
Output Filename: Z:\Hank\Report\Soil\Report 2022\22257004L Fleetwood Jurupa Valley\sta\ex1static.OUT
Unit System: English
Plotted Output Filename: Z:\Hank\Report\Soil\Report 2022\22257004L Fleetwood Jurupa Valley\sta\ex1static.PLT

PROBLEM DESCRIPTION: 2240 Fleetwood Drive, Jurupa Valley, Section A-A', Exisiting Slope_Static

BOUNDARY COORDINATES
5 Top Boundaries
5 Total Boundaries

Boundary No.	X-Left (ft)	Y-Left (ft)	X-Right (ft)	Y-Right (ft)	Soil Type Below Bnd
1	196.90	104.00	233.50	104.00	1
2	233.50	104.00	287.20	115.00	1
3	287.20	115.00	327.20	140.00	1
4	327.20	140.00	364.70	150.00	1
5	364.70	150.00	450.00	155.00	1

User Specified Y-Origin = 40.00(ft)
Default X-Plus Value = 0.00(ft)
Default Y-Plus Value = 0.00(ft)

ISOTROPIC SOIL PARAMETERS
1 Type(s) of Soil

Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
1	120.0	120.0	168.0	29.0	0.00		0.0

A Critical Failure Surface Searching Method, Using A Random Technique For Generating Circular Surfaces, Has Been Specified.

5000 Trial Surfaces Have Been Generated.

50 Surface(s) Initiate(s) From Each Of 100 Points Equally Spaced Along The Ground Surface Between X = 210.00(ft) and X = 310.00(ft)

Each Surface Terminates Between X = 320.00(ft) and X = 440.00(ft)

Unless Further Limitations Were Imposed, The Minimum Elevation At Which A Surface Extends Is Y = 0.00(ft)

10.00(ft) Line Segments Define Each Trial Failure Surface.

Following Are Displayed The Ten Most Critical Of The Trial Failure Surfaces Evaluated. They Are Ordered - Most Critical First.

* * Safety Factors Are Calculated By The Modified Bishop Method * *

Total Number of Trial Surfaces Attempted = 5000

Number of Trial Surfaces With Valid FS = 5000

Statistical Data On All Valid FS Values:
FS Max = 6.131 FS Min = 1.634 FS Ave = 2.823
Standard Deviation = 0.753 Coefficient of Variation = 26.66 %

Failure Surface Specified By 8 Coordinate Points

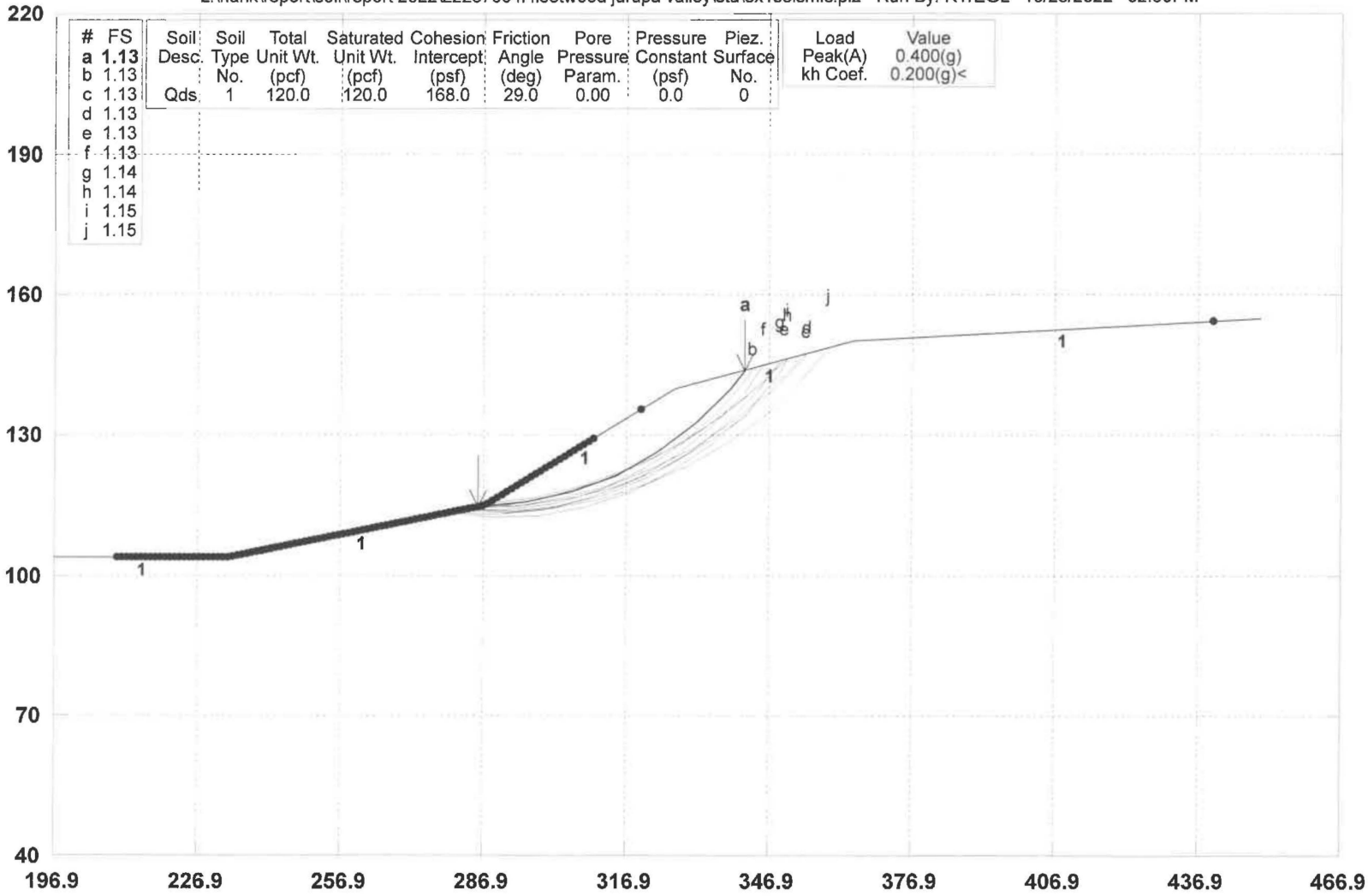
Point No.	X-Surf (ft)	Y-Surf (ft)
1	285.758	114.705
2	295.719	115.580
3	305.459	117.845
4	314.785	121.454
5	323.512	126.338
6	331.467	132.397
7	338.492	139.514
8	341.725	143.873

Circle Center At X = 284.714 ; Y = 184.985 ; and Radius = 70.288

Factor of Safety
*** 1.634 ***

2240 Fleetwood Drive, Jurupa Valley, Section A-A', Exisiting Slope_Seismic

z:\hank\report\soil\report 2022\22257004l fleetwood jurupa valley\sta\ex1seismic.pl2 Run By: KY/EGL 10/28/2022 02:30PM



#	FS
a	**1.13**
b	1.13
c	1.13
d	1.13
e	1.13
f	1.13
g	1.14
h	1.14
i	1.15
j	1.15

Soil Desc.	Soil Type No.	Total Unit Wt. (pcf)	Saturated Unit Wt. (pcf)	Cohesion Intercept (psf)	Friction Angle (deg)	Pore Pressure Param.	Pressure Constant (psf)	Piez. Surface No.
Qds	1	120.0	120.0	168.0	29.0	0.00	0.0	0

Load	Value
Peak(A)	0.400(g)
kh Coef.	0.200(g)<

GSTABL7 v.2 FSmin=1.13

Safety Factors Are Calculated By The Modified Bishop Method