Appendix E:
Geotechnical Supporting Information

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E.1 - Geotechnical Investigation Report

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GEOTECHNICAL INVESTIGATION REPORT

PREPARED FOR

OUTFRONT MEDIA

FOR

TRI-FACE L.E.D. BOARDS

AT

INTERSECTION OF ARTESIA BLVD. & FIRESTONE BLVD.
BUENA PARK, CA 90621

REGISTERED PROFESSIONAL ENGINEER
C. DENNIS LEE
GE 2152
EXP. 03/31/2024
GEOTECHNICAL
STATE OF CALIFORNIA

JOB NO.: 8309G **Date: November 15, 2022**

LEEDCO ENGINEERS, INC.

Index

LEEDCO ENGINEERS, INC.

STRUCTURAL AND CIVIL
ENGINEERING CONSULTANTS

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November 15, 2022

Outfront Media
1731 Workman Street
Los Angeles, CA 90031

Attn: **Mr. Dave Ryan**

Subject: **Geotechnical Investigation Report for New Tri-Face L.E.D. Boards**
Located at Intersection of Firestone Blvd. & Artesia Blvd.,
City of Buena Park, County of Orange, CA 90621
APNs # 066-020-36
Leedco File No.: 8309G

Dear Mr. Ryan:

We are pleased to submit herewith the results of our geotechnical investigation for the subject project.

The purpose of this investigation was to determine subsurface soil conditions and provide geotechnical recommendations with respect to design and construction feasibility of the proposed development. Implementation of the recommendations made in this report is intended to reduce certain risks associated with the construction project. The scope of this investigation does not include any work related to finding any environmental problems and identify hazardous waste materials.

This investigation consists of excavating one (1) exploratory boring holes, obtaining representative soil samples, laboratory testing, engineering evaluations, and the preparation of this report. The exploratory boring locations are shown on the attached site plan in the Appendix A (Figure 2). The boring logs and the results of our laboratory tests are also shown in the Appendix A of this report.

PROPOSED DEVELOPMENT

It is our understanding that you are planning to build a a new tri-face L.E.D. boards on the subject parking lot.

The subject new tri-face L.E.D. boards will be built on or near the boring location. The new tri-face L.E.D. boards will be constructed with concrete, steel, and light steel frames. Proposed structure is expected to be supported by a shallow concrete foundation and will have lateral loads from winds and seismicity.

It is understood that the site will require minimum grading for the development, and no permanent cut or fill slopes greater than 10 horizontal to 1 vertical are planned for the project.

Recommendations for site preparation and for designs and constructions of the foundation of the proposed development are provided with this report.

LOCATION AND SITE DESCRIPTION

The proposed site lies within a commercial vacant lot located at intersection of Firestone Blvd., and Artesia Blvd. in the City of Buena Park, CA 90621. The representative coordinates of the site are approximately 33°52'25.2294"N and 118°0'30.7218" W. The property exhibits a relatively level topography with no pronounced highs and lows.

The property investigated is bounded by Cate Driveto the north, Village Drive to the south, and Knott Avenue to the West, and Cambridge Avenue to the East (T.G. pg. 737, G6).

Geographically this site is situated North of the Santa Ana Freeway 5. The proposed tri-face L.E.D. boards will be located on east side of Firestone Boulevard.

Nearby businesses consist of commercial establishments including auto dealer, hotel, and office building.

Drainage appears to be good with sheet flow along the existing contours to the city and county roads.

No signs of ground water table or seepage were observed anywhere observed on the subject project location during our subsurface investigation.

FIELD OBSERVATION & RECONNAISSANCE

Field inspection and reconnaissance were performed on October 27, 2022 by drilling of one (1) test hole. The soil is continuously logged by a field geotechnical engineer and classified by visual examination in accordance with the Unified Soil Classification System and Symbols (See Figure 4). The results of the laboratory tests, along with a description of the borings and laboratory tests procedures used are presented in the Appendix B.

The geotechnical investigation for the new tri-face L.E.D. boards consisted of test holes and lab tests. Figures 2 in Appendix A shows location of the boring hole, and other field testing performed for the project, as well as the site layout. The site investigation and lab tests both were completed in November 15, 2022. Proposed site location is mapped in Figure 2 in Appendix A.

FIELD WORK

Test Hole

Total of one (1) test boring was performed in order to determine the soil stratigraphy and layer thicknesses more effectively for the new tri-face L.E.D. boards.

Figures 2 shows a plan view of the boring performed for the project. The boring was up to depths of 50 feet below the existing ground surface. Samples were collected using thin-walled tubes and bulk samples. The soil samples were retained by and tested by Leedco Engineers's in-house laboratory which is currently approved as a testing agency by Los Angeles City. Boring logs are included in Appendix A.

The test holes were backfilled with original material upon completion.

SOIL TESTING

The following tests were performed by Leedco Engineers, Inc.

- Atterberg Limit determinations in accordance with ASTM D4318, "Standard Test Methods for Liquid Limit, Plastic Limit, and Plasticity Index of Soils"
- Standard Proctor Density determinations in accordance with ASTM D698, "Standard Test Methods for Laboratory Compaction Characteristics of Soil Using Standard Effort (12,400 ft-lbf/ft3 (600 kN-m/m3))
- Moisture content tests were performed in accordance with ASTM D2216, "Standard Test Method for Laboratory Determination of Water (Moisture) Content of Soil and Rock by Mass"
- Hydrometer analysis in accordance with ASTM D422-63, "Standard Test Method for Particle Size Analysis of Soils"
- Expansion Test per UBC standard #18.2.
- Dry unit weight tests

All field and laboratory test reports are provided in Appendix B.

RESULTS

Groundwater Conditions

A review of well data from the United States Geological Survey indicates that historic groundwater levels in the project vicinity was about 20 feet below the ground surface (CDMG, 1997).

However, it must be noted that fluctuations in the level of the groundwater may occur due to variations in rainfall, temperature, and other factors not evident at the time measurements were made and reported herein.

Referring to Boring No. 1, the water table was not encountered up to depth of 50 feet below the grade level during the subsoil exploration.

Soil Plasticity and Natural Moisture Content

The overburden encountered across the site is characterized by varying amounts of silt and sandy clay resulting in classifications ranging from silt to silty clay with sand. Atterberg limits analyses were performed on samples at various locations. The results indicated that the surficial soil has a liquid limit ranging from 37.0 to 42.0 percent. The plasticity index was found to range from 18.0 to 24.0 with an average of 21.0, indicating a slightly coarse grained soils. It should be expected that the plasticity of the soil will increase with an increasing percentage of silt. The results of the soil plasticity testing are included in Appendix B.

The natural moisture content was determined for the near surface material for 5 samples. The natural moisture content of these samples ranged from 12.9 to 20.1 percent (with an average of 16.5. The results of the natural moisture content testing are included in Appendix B.

Soil Density, Soil Specific Gravity and Grain Size

The results of the tests are presented in Appendix B. The measured densities of the soil are in Appendix B.

Measured dry density values for the overlying in-situ soil vary between 108 and 116 pcf, with an average of 112 pcf. Assumed specific gravity values for the sands of 2.60. Grain size curves are in Appendix B.

Sulfate Contents

The results show that the sulfate content measured was 15 µg/g corresponds to a negligible (0 to 0.1 percent) sulfate exposure. Recommendations regarding the concrete type, as a result of these laboratory results, are indicated in Conclusion Section of this report.

SOIL STRENGTH

Compaction Testing

The laboratory compaction testing was performed on shallow soils. These results indicate the maximum dry density of the on site soils is 112.9 pcf, with optimum moisture contents of 17.9 percent.

LABORATORY TESTING

Moisture content, unit dry density and shear strength characteristics were determined for samples of soils considered representative of those encountered. A description of laboratory test criteria and a summary of all test data are presented in Appendix B. An evaluation of the data is reflected throughout the "Conclusions and Recommendations" section of this report.

Shear tests are performed with a strain controlled, direct shear machine manufactured by CONTROLS, Milano. The rate of deformation is approximately 0.025 inches per minute. Each sample is sheared under varying confining pressures in order to determine the Mohr-Coulomb shear strength parameters of the cohesion intercept and the angle of internal friction. Samples are generally tested in an artificially saturated condition. Depending upon the sample location and future site conditions, samples may be tested at field moisture content. The results are plotted on the "Shear Test Diagram" in Appendix B.

ANALYSIS AND RECOMMENDATIONS

Results of the field and laboratory investigation have been presented herein. Based on these results, herein, we provides analysis, conclusions and recommendations for the design of the improvements, private roads, and general construction considerations as follows.

FINDINGS

Subsurface Conditions

Based on the results of our sub-surface exploration and investigation, the site appears underlain by fill, alluvial deposit, and bedrock. Descriptions of these units shall be presented in Appendices.

The fill consists of mixture of organic silts and sands, moist, and grey at top few inches in thickness. The natural deposit soils underlying the loose fill consist of silty sand, and clayey sand and silt, which are tan to grey, moist, dense to medium dense, fine grained, and containing varying amounts of clay. The native soils consisted of predominantly silty sand with some silts. More detailed soil profiles may be obtained from the boring log in Appendix A.

Bedrock was not encountered in the bore holes. Ground water was not encountered within the test holes except in Boring Hole No. 1 indicated in the attached boring logs. Caving did not occur within the test hole.

Detailed descriptions of the soils encountered at the test holes and the soil tests conducted, with their results, are presented in the Appendices.

Our exploratory borings were advanced up to depth of 50 feet, and the surfcial subsurface soils encountered consisted of silty sand (SM), silt (ML), and clay to sandy silt (SC and SC-CL).

SEISMIC SETTING

Seismicity is a general term relating to the abrupt release of accumulated strain energy in the rock materials of the earth's crust in a given geographic area. The recurrence of accumulation and subsequent release of strain have resulted in faults and fault systems. The major fault systems in the Southern California area are shown on the Earthquake Epicenter and Fault Map. (See Figure 3).

The primary geologic hazards at the site are those associated with seismic events such as strong ground shaking and ground rupture. Secondary hazards associated with the strong ground shaking are liquefaction and seismic settlement when adverse conditions within the site earth materials are present.

The site falls within Regional 1 of the 2019 California Building Code. Consequently, a seismic hazards screening was performed for the site to evaluate potential seismic hazards. The seismic hazards screening consisted of reviewing available data published by the California Geological Survey (CGS).

The site is located in the United States Geological Survey Los Alamitos 7.5-Minute Quadrangle (CDMG, 1998). The reviewed data included: Fault-Rupture Hazard Zones in California (CDMG, 1997), the Earthquake Fault Zone Map (CDMG, 1998), the Fault Activity Map of California and Adjacent Areas (Jennings, 1994), the Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada (International Conference of Building Officials, 1999), and the Seismic Hazards Zone Map (CDMG, 1998).

There are not active or potentially active faults mapped as crossing the project site (Jennings, 1994). The nearest known active fault area is the Whittier Fault zone, which is 12 kilometers north of the project site. The CGS does not delineate any part of the site as being within an Alquist-Priolo Earthquake Fault Zone (CDMG, 1997 and 1991). Because there is an active or potentially active fault known to be present crossing the project site, the potential for surface fault rupture is considered likely. Moreover, with the active faults in the region, the site could be subjected to future strong ground shaking that may result from earthquakes on local to distant sources.

Table below presents seismic design factors in keeping with the criteria presented in the 2019 California Building Code (CBC) and ASCE 7-16. The following parameters should be used in computing seismic base shear forces:

TABLE 1- SEISMIC DESIGN PARAMETERS
BASED ON SITE COORDINATES
33°52'25.2294"N and 118°0'30.7218" W
RISK CATEGORY I/II/III

	S_S	S_1	S_{DS}	F_a	F_v
	1.546	0.548	1.237	1.2	null
Parameters	S_{Ms}	S_{M1}	S_{D1}	Site Class	
Value	1.855	null	null	D	

OTHER SEISMIC HAZARDS

1. Liquefaction Potential

Liquefaction is a mode of ground failure that results from the generation of high pore water pressures during earthquake ground shaking, causing loss of shear strength. Liquefaction is typically a hazard where loose sandy soils exist below groundwater. The CGS has designated certain areas within Southern California as potential liquefaction hazard zones. These are areas considered at risk for liquefaction-related ground failure during a seismic event, based upon mapped surficial deposits and the presence of a relatively shallow water table.

The subject site is mapped within an area identified as susceptible to liquefaction according to the California Division of Mines and Geology (CDMG). A review of well data from the United States Geological Survey indicates that the historic groundwater level of the subject site was about 20 feet below the ground surface (CDMG, 1997).

Liquefaction analysis was performed on soil layers below ground level, that is 20 feet, utilizing SPT blow count data. The blow count data indicate that SPT blow counts are relatively high for a clayey and sandy silt and would indicate little or no liquefaction potential.

Percent passing #200 sieve (D = 0.002 mm) was used to determine percent fines based on ASTM (unified) method.

We calculated Plasticity Indices (PI) based on the results of our Laboratory Atterberg Limits tests, that are Liquid Limits and Plastic Limits (See Boring B-1). They are indicated in the boring logs, which illustrate PI in the range between 19 and 22.

We believe that the site is not susceptible to liquefaction for the following reasons:

1. Factor of Safety against liquefaction is greater than one.
2. There is no groundwater in our subsurface boring to 50 feet depth.
3. Per Guidelines for Evaluating Seismic Hazards in California (SP 117A) and The Liquefaction Susceptibility Criteria by Bray and Sancio (2006), the average plasticity index, PI = 19 > 12, and the average moisture content, M.C. ≤ 85%.

We further conclude that the seismic total and differential settlement are 1-1/8" and 1/2", respectively.

According to our study, we believe that occurrence of Liquefaction and Seismic Hazards due to loss of shear strengths, and excessive structural settlements are less than significant.

The soil samples taken from our field subsurface boring show predominantly clayey sand, sandy silt, and silt. Our laboratory Atterburg Testing performed on those in-situ samples are shown in the attached Boring Logs. As the data in the Boring Logs show PI's (plasticity Indices) being greater than 18 and the average moisture content, M.C. ≤ 85%, we believed that the project building foundation is not susceptible to liquefaction per "Guideline for Evaluating Seismic

Hazards in California (SP1 17A)" and "Liquefaction Susceptibility Criteria" by Bray and Sancio (2006).

The soil samples were tested by Leedco Engineers' in-house soils testing laboratory, which is currently certified by the City of Los Angeles as an approved Testing Agency.

We also believe that any local and short-term saturation due to possible perching water are not susceptable to liquefaction.

2. Tsunamis, Seisches, and Flooding

Risks associated with tsunamis and seiches are considered negligible based on the distance from open water and site elevation. The risk of flooding from a seismically-induced siche is considered to be remote.

3. Landsliding

The probability of seismically-induced landslide occurring on the site is considered to be low due to the relatively flat topography of the site.

The site is not within a mapped earthquake-induced landslide hazard zone, as shown on the CDMG Seismic Hazard Map.

CONCLUSIONS & RECOMMENDATIONS

From a geotechnical engineering point of view, the subject property is considered suitable for the proposed development / construction provided our recommendations is incorporated into the design and project specifications.

Based on our final evaluation of the site conditions, the proposed tri-face L.E.D. boards may be supported on a directly embedded concrete footing according to design recommendations presented in this report..

SITE PREPARATION

General

Precautions should be taken during the performance of all work under the following sections, especially if construction is performed during the rainy season of approximately October 15 to April 15. Protection should be provided to the work site, particularly excavated areas, from flooding, ponding, and inundation due to poor or improper temporary provisions should be made to adequately direct surface drainage, from all sources, away from and off the work site and to provide adequate pumps and sumps to handle any flow into the excavations.

Trees and Surface Vegetation

Removal of designated trees and shrubs in areas of proposed construction should include rootballs. Resultant cavities should be cleaned of loose soils and roots and rolled to a firm unyielding surface prior to backfilling.

Grass and weed growth in areas of future construction should be stripped and disposed of off-site. Stripping should penetrate three to six inches into surface soils. Any soils sufficiently contaminated with organic matter (such as root systems or stripping mixed into the soils) so as to prevent proper compaction shall be disposed of off-site or set aside for future use in landscape areas.

Subgrade Preparation

Prior to receiving of new fill or in areas where slab-on-grade or pavement is proposed, it is recommended that all of the existing onsite fill material be removed to underlying competent foundation soil and replaced with properly compacted fill, if necessary, for slab and pavement support. The exposed bottom surface in each removal area should first be scarified to a depth of at least 8 inches, processed, watered or air dried as necessary to achieve near optimum moisture conditions, and then compacted in-place to at least 90 percent of the maximum laboratory density or relative compaction. Based on the results of the investigation, the depth of fill material encountered at the test holes locations in the proposed construction area is approximately one foot below the existing grade. Locally, some areas exposed soft or loose soils may require somewhat deeper removal than indicated above. Actual depth of over excavation will have to be determined in the field at the time of grading.

FOUNDATION DESIGN

Foundation Type

The proposed structural foundation may be a directly embeded concrete fill footing.

Investigation and testing of the proposed new tri-face L.E.D. boards location found the presence of generally medium dense sandy silt and medium stiff clayey-silt. We have determined that the most feasible foundation type is deep foundation.

Drilling excavation for the new footing construction would encounter groundwater and having a possibility of soil cave-in. Therefore, there would be needs of a serious shoring during construction including employment of casing pipe.

Termie method shall be used to pour concrete under water.

Soil Bearing Capacities

A shallow foundation constructed in accordance with recommendations provided herein may be designed to resist downward loads using an allowable bearing pressure of 1,800 pounds per square foot (psf).

The basic vertical bearing pressures may be increased by 10% per foot of the foundation penetration below the minimum recommended bearing level below the natural grade, that is 18 inches.

The allowable bearing pressure provide about is net value; therefore, the weight of the footing may be neglected when evaluating downward capacities. Total downward capacities derived from the parameters provided above may be increased by 1/3 for shoot-term loading due to wind or seismic forces.

Estimated Settlements

Total settlement of the proposed footing and differential settlements are estimated 1-1/8 inch and ½ inch, respectively. However, the seismic differential settlement shall be taken minimum 3/4 inch over the horizontal distance of 30 feet.

As indicated in the Summary of Laboratory test results of this geotechnical report (PLATE E), we believe that the presence of expansive soils is so little and insignificant in terms of soil structure interactions.

We also believe that the susceptibility for hydro-consolidation and any ground settlements due to soil saturation from infiltration is minimal owing to silty and clayey sand nature of the site soils that exhibit less potential for volume changes due to fluctuation of the soils moisture contents. Indeed, the in-situ soils show low tendency of creep-relaxation and higher seepage rates.

Lateral Capability

Lateral loads exerted on the structure may be resisted by passive resistance of soil against the piers based on an allowable equivalent fluid pressure of 120 pounds per cubic foot acting over two pier diameters. This allowable pressure should be limited to a maximum uniform pressure of 1,800 psf.

Sliding Friction

The friction coefficient between the foundation soil of the site and concrete should be taken as 0.35 in accordance with recommendations assuming plain concrete surface.

Retaining Wall Design & Construction

The project site is relatively flat with level difference no greater than three (3) foot within the lot. The project design drawings indicate there are no Retaining/Basement walls being proposed for the project.

The proposed building foundation is a continuous strip footing. Excavation for the new footing construction is so shallow that shoring and underpinning situation would not be encountered in the proposed project site. Further, there are no proposed excavations on or near the public R.O.W. in project development.

General Guideline:

Small unrestrained retaining walls with a level backfill should be designed to resist active soil pressures equivalent to a fluid pressure of 45 pounds per cubic foot, plus additional surcharge expected from the surface.

Basement walls or other retaining walls restrained both at the bottom and top should be designed for soil pressures of 70 pounds per cubic foot is recommended for the on-site soils.
Weep holes consisting of open joints in block walls or 2 inch diameter holes at 4 foot intervals should be placed at the base of the wall 8 to 12 inches above finished grade, or an adequate drainage system at the base of the wall should be provided to reduce hydrostatic pressures.

All walls should have a backfill compacted as fill soil. Jetting should not be permitted.

All footing excavations should be inspected and approved by the Soils Engineer or Geologist prior to placing forms or reinforcement, in order to verify minimum depths into the recommended supporting material.

It is recommended that walls retaining earth be designed for the following:

Surface Slope of Retained Material (Horizontal to Vertical)	Equivalent Fluid Weight For Natural Soils or Rock (Pounds per Cubic Foot)	Equivalent Fluid Weight For Existing or New Fill Soils (Pounds per Cubic Foot)
Level	40	45
5 to 1	42	47
4 to 1	45	50
3 to 1	48	53
2 to 1	50	55

Additional surcharge load due to traffic should be added to above values where applicable.

Construction Consideration

The excavation of all drilled shafts should be observed by Leedco Engineers, Inc. representative to confirm the soil profile and the piers are constructed in accordance with our recommendations and project requirements. The drilled shafts should be straight, dry and relatively free of loose material before steel is placed and concrete is poured. If ground water is encountered and cannot be removed from the excavations prior to concrete placement, drilling slurry or casing may be required to stabilize the shaft and the concrete should be placed using a tremie pipe, keeping the tremie pipe below the surface of the concrete to avoid entrapment of water or drilling slurry in the concrete.

Backfill Density

Based on results of laboratory investigations, the native material has a design minimum dry density of approximately 113.2 pcf corresponding to a minimum long term average moisture content of 17.6 percent. The native material can be used as backfill for the foundation if it meets the following conditions:

Material shall be placed in 1 foot lifts.

- The material is placed at a minimum dry unit weight required by foundation design specifications, that is 90% relative compaction based on ASTM D1556 & D1557.

- The backfill surface is graded such that water is directed away from the foundation to prevent moisture infiltration.

- Density is checked in the field periodically to ensure adequate compaction.

Groundwater Buoyancy Effects

Groundwater is neither present nor expected in the construction zone. As a result, it is not necessary to consider any buoyancy effects in the foundation design.

Cement Type

A number of samples were submitted for soluble sulfates testing. As a result, we believe that Type I or II structural cement is suitable for construction of the foundation for this project.

The in-situ soils were tested for chloride contents. As the results (Plate E) indicates, the contents is so insignificant to cause any deleterious effects on underground steel and rebars.

PLAN REVIEW

In order to prevent misinterpretation of this report by other consultants it is recommended that the Soils Engineer be provided the opportunity to review the final grading and foundation plans. The Soils Engineer will also determine whether any changes in concept may have had any affect on the validity of the Soils Engineer's recommendations, and whether those recommendations have, in fact, been implemented in the design and specifications.

If the Soils Engineer is not accorded the privilege of making this recommended review, he/she can assume no responsibility for misinterpretation or misapplication of his recommendations or for their validity in the event changes have been made in the original design concept without this prior review.

GEOTECHNICAL INSPECTION

All rough grading of the property must be performed under engineering supervision of the geotechnical consultants. Rough grading includes, but is not limited to, site preparation, cleaning, over-excavation, and fill placement.

The geotechnical consultant should inspect all foundation excavations. Inspections should be made prior to installation of concrete forms and reinforcing steel to verify or modify, if necessary, conclusions, and recommendations in this report.

Inspections of the finish grading, utility or other trench backfill, retaining wall backfill, or other earthwork completed for the subject project should also be performed by the geotechnical consultant.

If any of theses inspections to verify site geotechnical conditions are not performed by the geotechnical consultant, liability for the safety and stability of the project is limited only to the actual portions of the project approved by the geotechnical consultant.

Please advise this office at lease 48 hours prior to any required site inspection.

INVESTIGATION LIMITATIONS

This report is based on the project as described and the geotechnical data obtained from the field tests performed at the requested location. The materials encountered on the project site and utilized in our laboratory investigation are believed representative of the total. However, soils can vary in characteristics, both laterally and vertically.

The conclusions and opinions contained in this report are based on the results of the described geotechnical evaluations and represent our best professional judgment. Should subsurface conditions be discovered during the constructions processes that are different from the conditions described herein, a geotechnical consultant should be retained to review these conditions to provide additional recommendations if necessary. This report has not been prepared for use by parties or project other than those named or described above. It may not contain sufficient information for other parties or other purposes. Our professional services have been performed in accordance with generally accepted engineering procedures under similar circumstances. No other warranty, expressed or implied, is made as to the professional advice include in this report.

No responsibility of construction compliance with the design concepts, specifications, or recommendations is assumed unless an on-site review by a representative of this office is performed during the course of construction that pertains to the specific areas covered by the recommendations contained herein.

The findings of this report are valid as of the present date. However, changes in the conditions of the property can occur with the passage of time, whether they are due to natural processes or to the works of man, on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they report may be invalidated, wholly or partially, by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of one year without such a review.

The report is issued with the understanding that it is the responsibility of the owner, or the proper representative thereof, to insure that the information and recommendations contained herein are called to the attention of all parties interested in the project and the necessary steps are taken to see that the contractors and subcontractors carry out such recommendations in the field.

Final approval of plans and reports by all consultants, and issuance of any building and grading permits, rests with the controlling agencies. As the circumstances that control the decision process are clearly beyond the control of this facility, we cannot assume any responsibility for the success of obtaining proper authorizations, nor the costs involved.

An exploration holes used for subsurface exploration were backfilled with reasonable effort to restore the areas to their original condition. As with any backfill, some consolidation and subsidence of the backfill soils may result in time, causing some depression of the holes areas and possibly a potentially hazardous condition. The client and/or owner of the property are advised to periodically examine the holes areas, and if necessary, backfill any resulting depressions. Leedco Engineers, Inc.'s shall not be liable for any resulting injuries or damages.

The report is subject to review by controlling public agencies having jurisdiction.

This opportunity of service is sincerely appreciated. Please call if you have any questions pertaining to this report.

Respectfully Submitted,

REGISTERED PROFESSIONAL ENGINEER
C. DENNIS LEE
GE 2152
EXP. 03/31/2024
GEOTECHNICAL
STATE OF CALIFORNIA

C. Dennis Lee, P.E.
Principal

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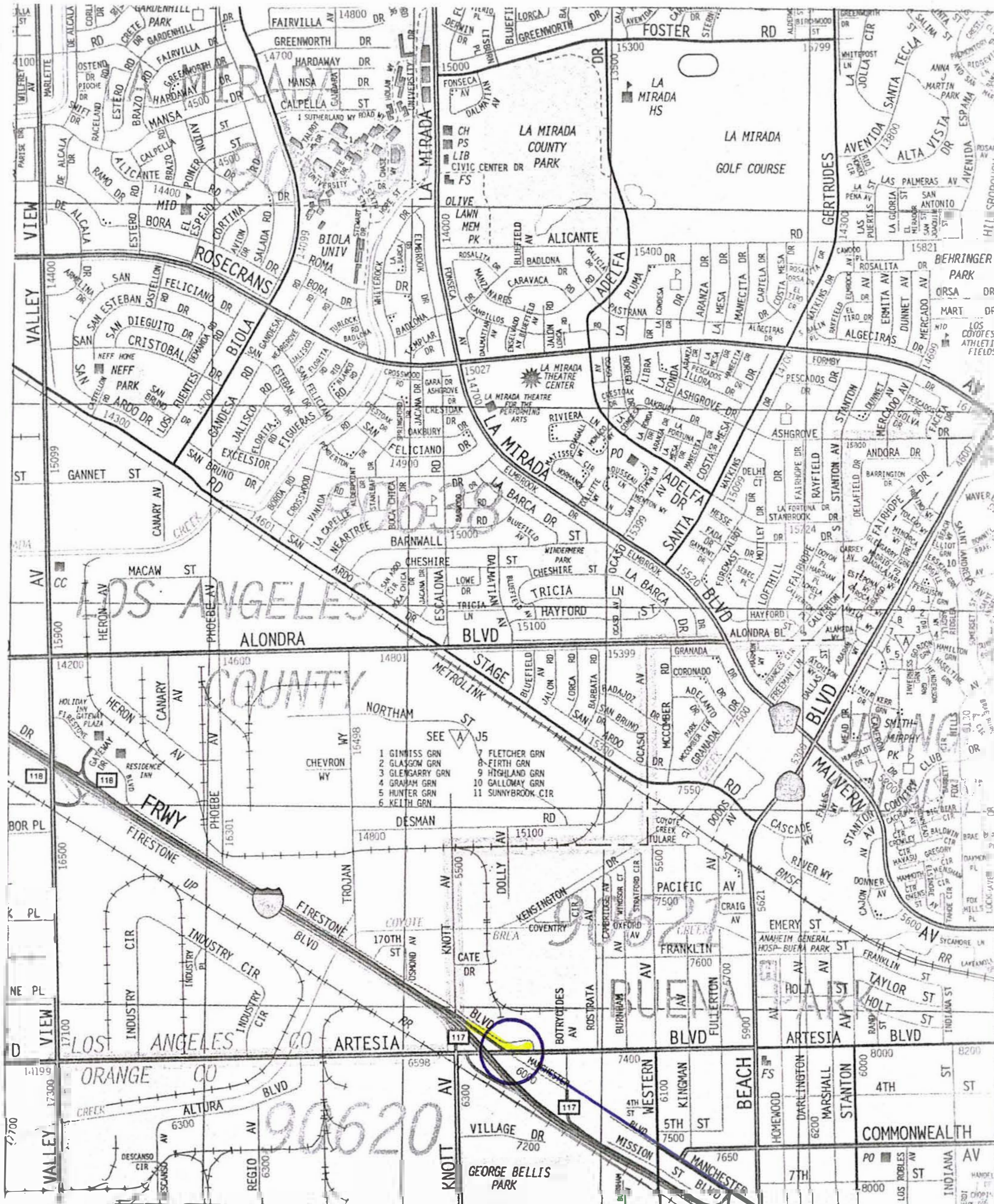
APPENDIX A

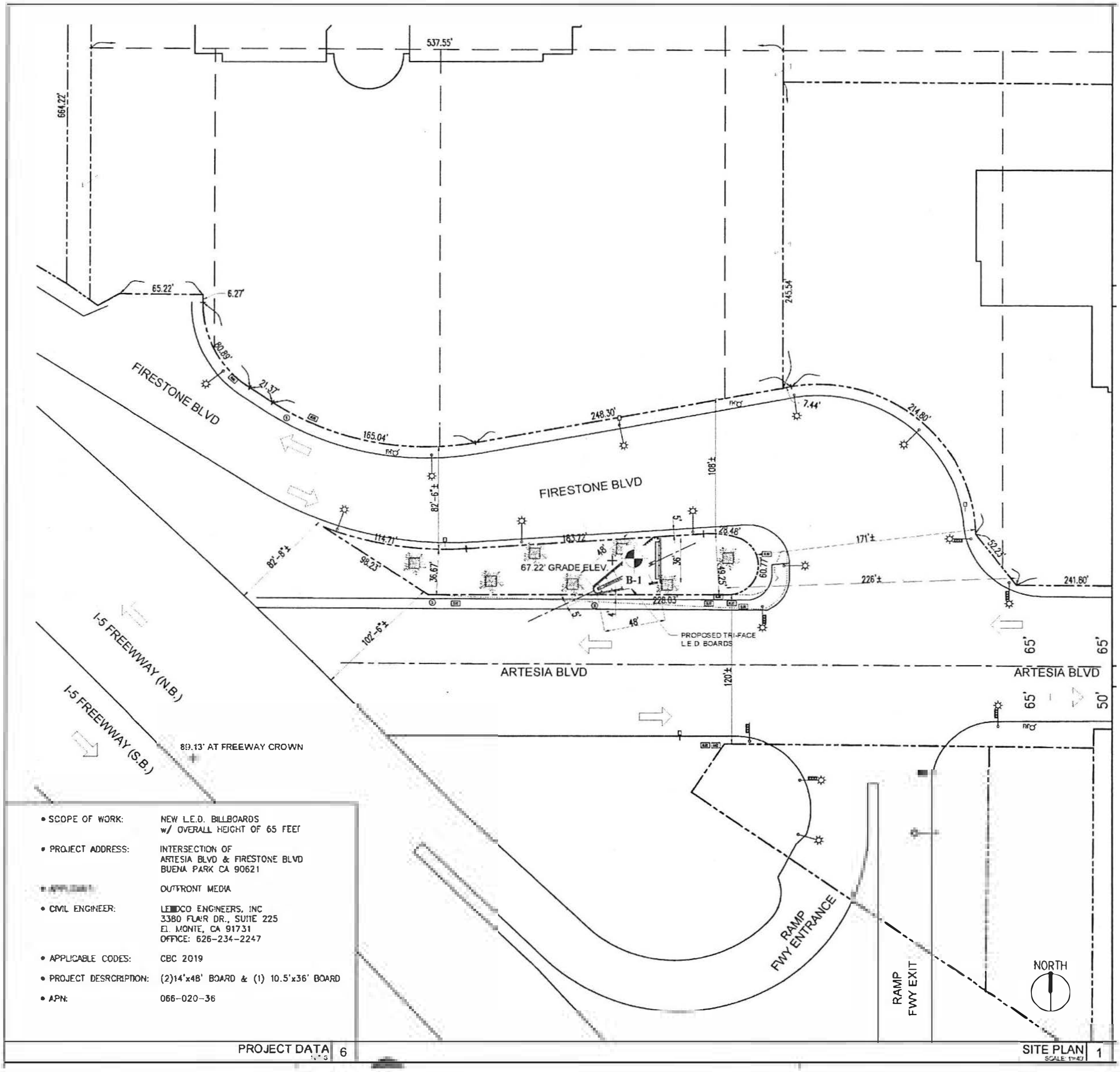


SITE

VICINITY MAP

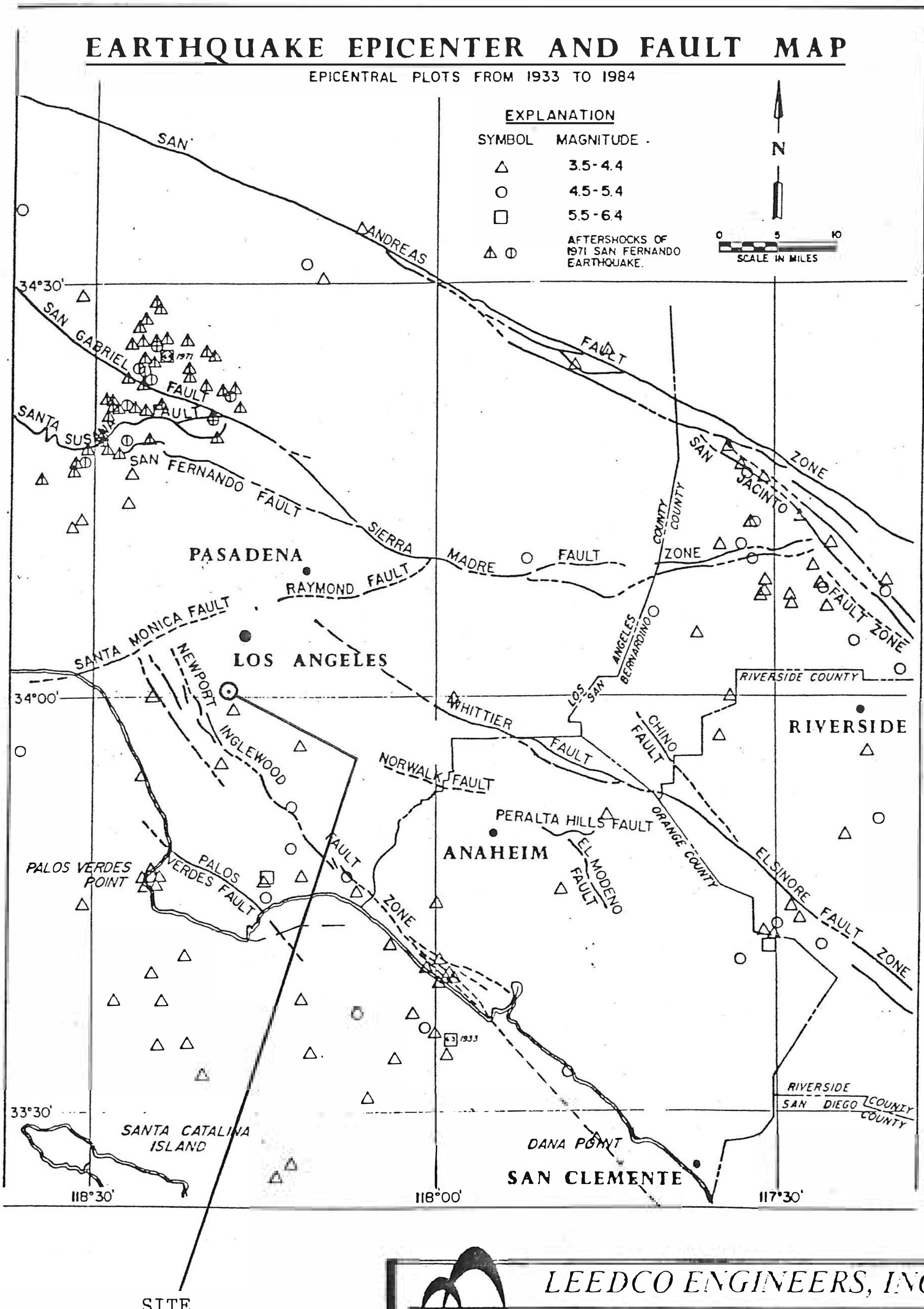
LEEDCO ENGINEERS, INC.	PROJECT No.	8309G
FIGURE OR PLATE No. 1	DATE	11-15-22

SITE PLAN
AND
BORING LOG LOCATION



Property Address: Vacant Lot at Intersection of Firestone Blvd., & Artesia Blvd ,
near the on-ramp of the Santa Ana Fwy 5, City of Buena Park, CA 90621

LEEDCO ENGINEERS, INC.	PROJECT NO.	8309G
FIGURE 2	DATE	11-15-22



LEEDCO ENGINEERS, INC.

EARTHQUAKE EPICENTER AND FAULT MAP

(Figure 3)

Job # 8309G	Date 11-15-22

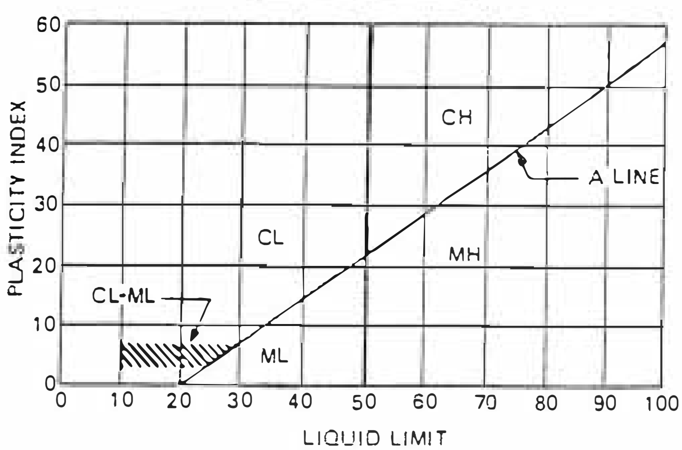
UNIFIED SOIL CLASSIFICATION SYSTEM

Soils are visually classified by the Unified Soil Classification system on the boring logs presented in this report. Grain-size analysis and Atterberg Limits Tests are often performed on selected samples to aid in classification. The classification system is briefly outlined on this chart. For a more detailed description of the system, see "The Unified Soil Classification System" Corp of Engineers, US Army Technical Memorandum No. 3-357 (Revised April 1960) or ASTM Designation: D2487-66T.

MAJOR DIVISIONS				GRAPHIC SYMBOL	GROUP SYMBOL	TYPICAL NAMES
COARSE-GRAINED SOILS (Less than 50% passes No. 200 sieve)	GRAVELS (50% or less of coarse fraction passes No. 4 sieve)	CLEAN GRAVELS (Less than 5% passes No. 200 sieve)			GW	Well graded gravels, gravel-sand mixtures, or sand-gravel-cobble mixtures.
					GP	Poorly graded gravels, gravel-sand mixtures, or sand-gravel-cobble mixtures.
		GRAVELS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart		GM	Silty gravels, gravel-sand-silt mixtures.
			Limits plot above "A" line & hatched zone on plasticity chart		GC	Clayey grav‹‹s, gravel-sand-clay mixtures.
	SANDS (More than 50% of coarse fraction passes No. 4 sieve)	CLEAN SANDS (Less than 5% passes No. 200 seive)			SW	Well graded sands, gravelly sands.
					SP	Poorly graded sands, gravelly sands.
		SANDS WITH FINES (More than 12% passes No. 200 sieve)	Limits plot below "A" line & hatched zone on plasticity chart		SM	Silty sands, sand-silt mixtures.
			Limits plot above "A" line & hatched zone on plasticity chart		SC	Clayey sands, sand-clay mixtures.
FINE-GRAINED SOILS (50% or more passes No. 200 sieve)	SILTS LIMITS PLOT BELOW "A" LINE & HATCHED ZONE ON PLASTICITY CHART	SILTS OF LOW PLASTICITY (Liquid Limit Less Than 50)			ML	Inorganic silts, clayey silts with slight plasticity.
		SILTS OF HIGH PLASTICITY (Liquid Limit More Than 50)			MH	Inorganic silts, micaceous or diatomaceous silty soils, elastic silts.
	CLAYS LIMITS PLOT ABOVE "A" LINE & HATCHED ZONE ON PLASTICITY CHART	CLAYS OF LOW PLASTICITY (Liquid Limit Less Than 50)			CL	Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays.
		CLAYS OF HIGH PLASTICITY (Liquid Limit More Than 50)			CH	Inorganic clays of high plasticity, fat clays, sandy clays of high plasticity.

NOTE: Coarse grained soils with between 5% & 12% passing the No. 200 sieve and fine grained soils with limits plotting in the hatched zone on the plasticity chart to have double symbol.

PLASTICITY CHART



DEFINITIONS OF SOIL FRACTIONS

SOIL COMPONENT	PARTICLE SIZE RANGE
Cobbles	Above 3 in.
Gravel	3 in. to No. 4 sieve
Coarse gravel	3 in. to ¾ in.
Fine gravel	¾ in. to No. 4 sieve
Sand	No. 4 to No. 200
Coarse	No. 4 to No. 10
Medium	No. 10 to No. 40
Fine	No. 40 to No. 200
Fines (silt or clay)	Below No. 200 sieve

FOGIRE 4

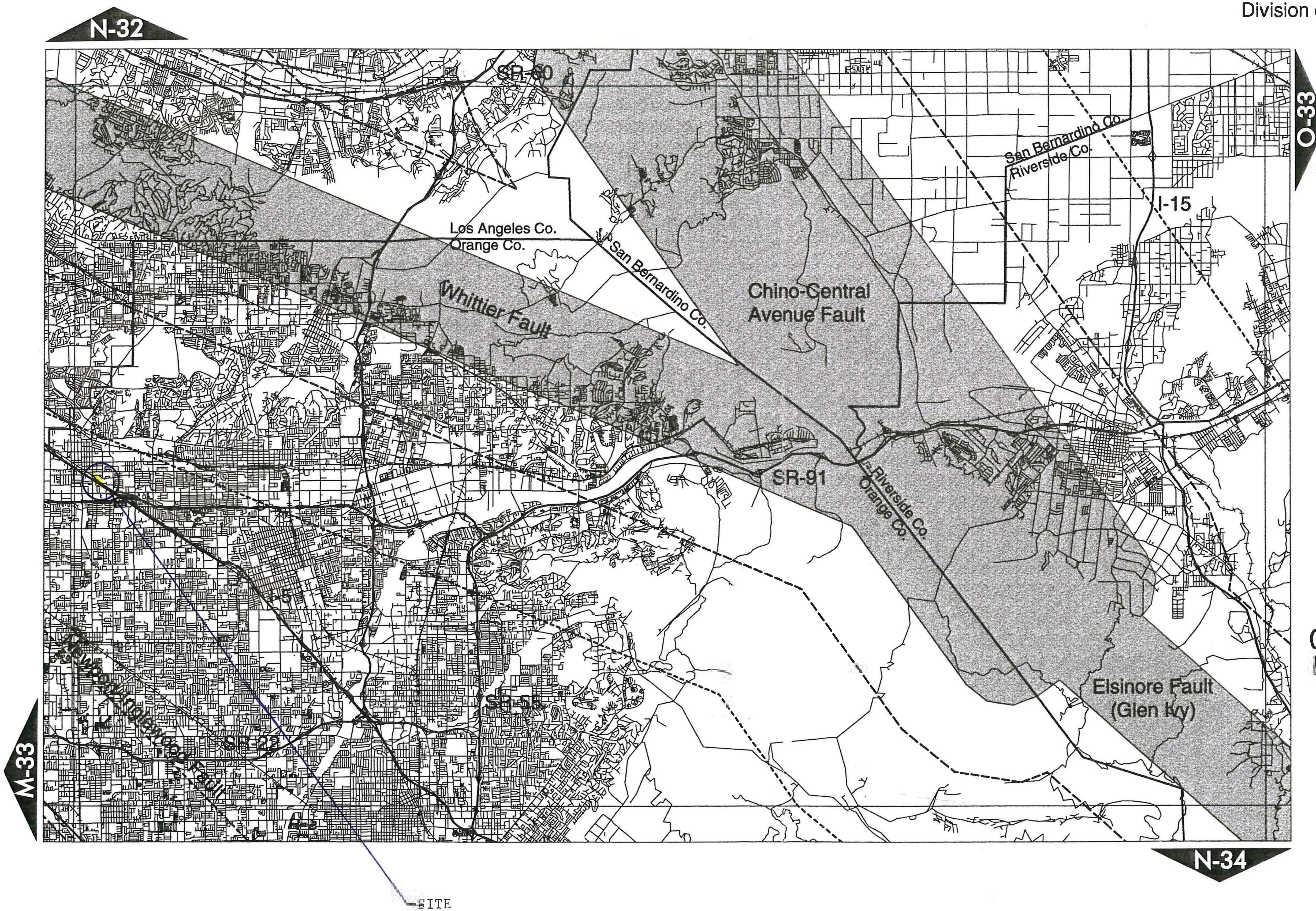
Active Fault Near-Source Zones

N-33

This map is intended to be used in conjunction with
the 1997 Uniform Building Code, Tables 16-S and 16-T

N-33

California Department of Conservation
Division of Mines and Geology



LEGEND

See expanded legend and index map

Shaded zones are within 2 km of known seismic sources.

A fault

B fault

Contours of closest horizontal distance to known seismic sources.

5 km

10 km

15 km

0 5 10

Kilometers

1/4" is approximately equal to 1 km

August, 1997

FIGURE 5

LEEDCO ENGINEERS, INC.
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BORING NUMBER B-1

CLIENT Outfront Media
PROJECT NUMBER 8309G
DATE STARTED 10-8-2022 COMPLETED 11-15-2022
DRILLING CONTRACTOR One Way Drilling
DRILLING METHOD 6" Hollow Stem Auger
LOGGED BY C. D. Lee CHECKED BY C. Dennis Lee
NOTES Boring Hole #B-1

PROJECT NAME New Tri-Face L.E.D. Boards
PROJECT LOCATION Firestone Blvd & Artesia Blvd., Buena Park, CA
GROUND ELEVATION - SAMPLE OUTER DIAMETER -
GROUND WATER DEPTHS:
AT TIME OF DRILLING NONE
AT END OF DRILLING NONE
AFTER DRILLING NONE

DEPTH (ft)		USCS	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OPTIMAL MOISTURE CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	ATTERBURY LIMT TESTS
0		SM	2.5" TOP SOIL & GRAVEL COVERS DARK GREY SANDY SILT, FINE, COMPACTED.								
5		SC	DARK BROWN SANDY SILTS & TRACE OF CLAY, MEDIUM DENSE, MOIST.	S-1			12.9	108	17.9	112.9	LL = 42 PL = 24 PI = 18
10		SC	DARK BROWN, SILTY SAND & SILTS, SOFT TO MEDIUM STIFF MOIST.	S-2		13	17.7	113			C = 380 PSI ϕ = 25°
15		ML	DARK GREY, CLAYEY SILT, FINE, MOIST, MEDIUM STIFF.	S-3			18.2	111			LL = 38 PL = 18 PI = 20
20		SC	CLAYEY SAND, TAN GREY, MOIST, AND STIFF.			23					
25		ML	DARK GREY SILT & SILTY CLAY, MOIST, MEDIUM STIFF.	S-4		21	19.0	115			C = 270 PSI ϕ= 29°
30		ML	DARK BROWN CLAYEY SILT & SILT MIXTURES, MEDIUM STIFF, MOIST.								LL = 37 PL = 20 PI = 17
35		SC-CL	CLAYEY SAND & SILT, DARK GREY, WET, AND MEDIUM STIFF.	S-5		18					

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BORING NUMBER B-1

CLIENT Outfront Media

PROJECT NAME New 14' x 48' Vee Digital / L.E.D. Message Sign

PROJECTRNUMBE 8309G

PROJECT LOCATION Firestone Blvd & Artesia Blvd, Buena Park, CA

DEPTH (ft)	GRAPHIC LOG	USCS	MATERIAL DESCRIPTION	SAMPLE TYPE NUMBER	RECOVERY % (RQD)	BLOW COUNTS	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	OPTIMAL MOISTURE CONTENT (%)	MAXIMUM DRY DENSITY (pcf)	ATTERBURG LIMIT TESTS
35											
40		SC-CL	DARK GREY SANDY CLAY-CLAY MIXTURES, MEDIUM STIFF, MOIST.	S-6	18		19.8	116			LL = 41 PL = 19 PI = 22
45		ML	DARK GREY SILT WITH TRACE OF FAT CLAY, MEDIUM STIFF TO SOFT, MOIST.				19.9	115			LL = 41 PL = 23 PI = 18
50		SC	GREY SILTY CLAY & SILT, MEDIUM STIFF, MOIST.	S-8	28		20.1	114			
			END OF BORING @ 50'								
55											
60											
65											
70											

NOTES: DATA ON THIS PAGE OBTAINED FROM SUBSURFACE INVESTIGATION PERFORMED ON 12-29-2020
FOR STUDY OF LIQUEFACTION POTENTIAL.

APPENDIX B

LABORATORY TEST CRITERIA

LABORATORY TEST DATA

LABORATORY TEST CRITERIA

Soil Classification

Soils encountered within the property were classified and described utilizing the visual-manual procedures of the United Soil Classification System, and in general accordance with Test Method ASTM D 2488-84. The assigned group symbols are presented in the "Exploration Log", Appendix A.

In Situ Moisture and Density

Moisture content and unit dry density of the in place soils were determined in representative strata. Test data are summarized in the "Exploration Log", Appendix A.

Direct Shear

The Coulomb shear strength parameters, angle of internal friction and cohesion, were determined for an undisturbed sample of onsite soil. This test were performed in general accordance with Test Method No. ASTM D-3080. Three test specimens were prepared for this test, artificially saturated, then shear under varying normal loads at a constant rate of strain 0.05 inches per minute. Results are shown in the "Direct Shear Tests, Plate A", Appendix B.

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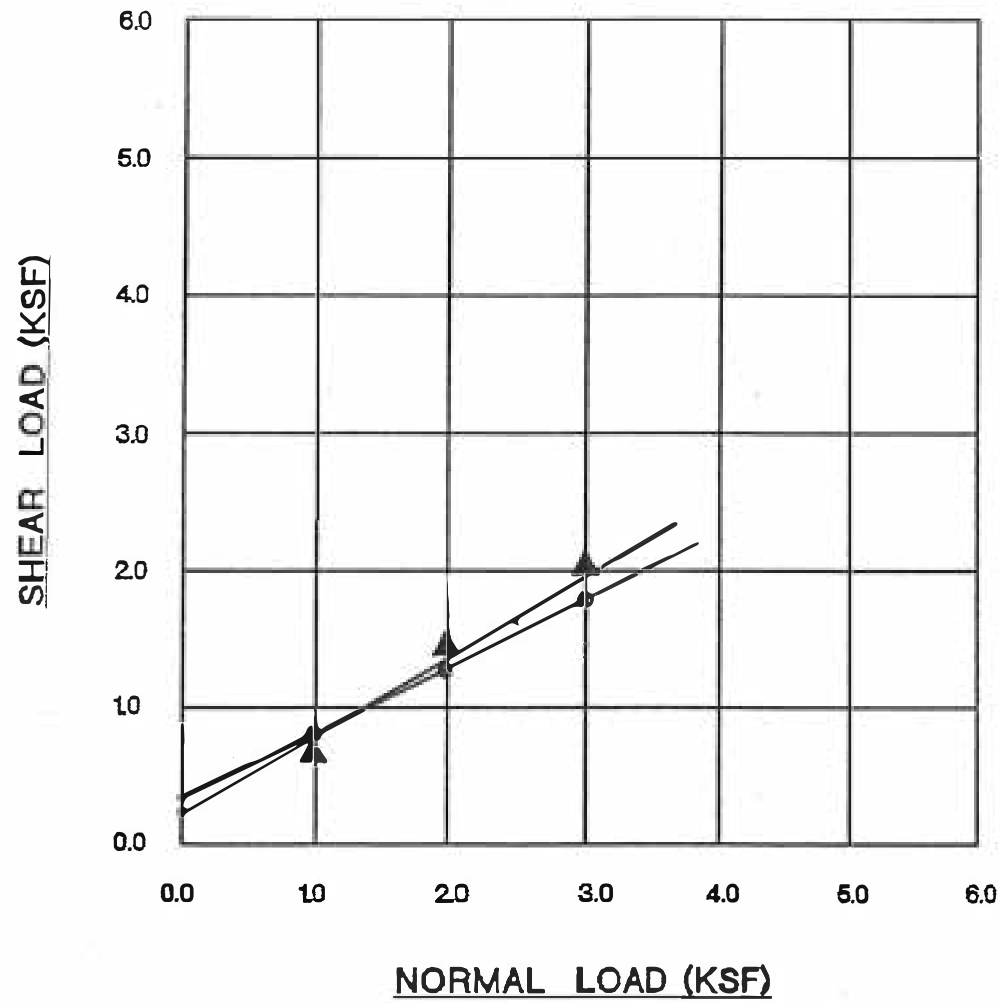
DIRECT SHEAR TESTS



SYMBOL	LOCATION	DEPTH (FT.)	DESCRIPTION	COHESION (P. S. F.)	FRICTION (DEG.)
●	Boring 1 (S-2)	10	Saturated -Drained	380	25°
Δ	Boring 1 (S-4)	25	Unsaturated -Drained	270	29°

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Plate B

GRAIN SIZE DISTRIBUTION

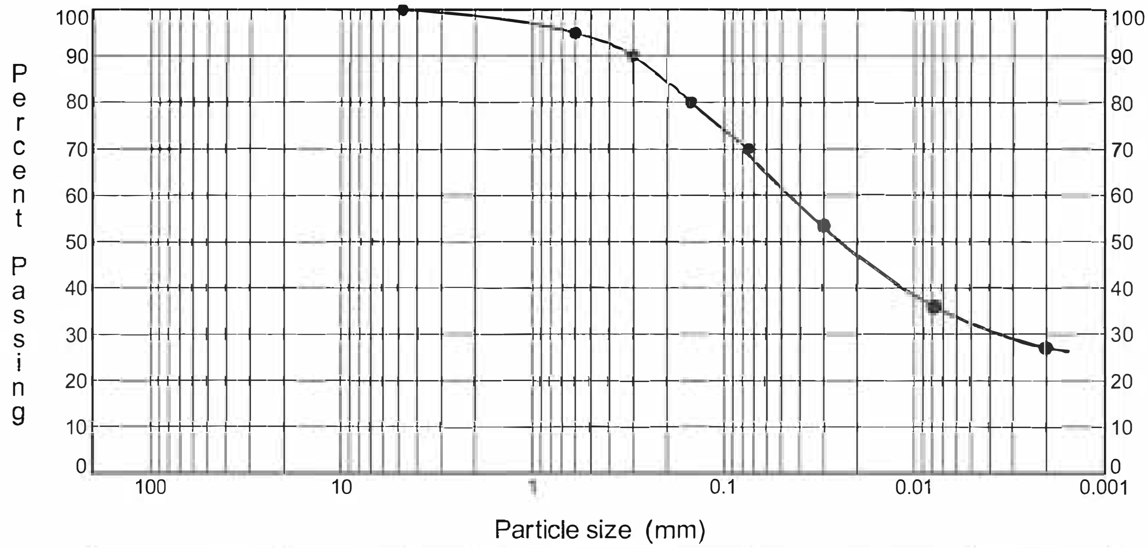
CLIENT OUTFRONT MEDIA

PROJECT NAME: NEW TRI-FACE BOARDS

PROJECT NUMBER 8309G

PROJECT LOCATION: Firestone Blvd. & Artesia Blvd., Buena Park, CA

BOREHOLE No. **B-1 (S-2)** DEPTH **10.0**



Cobbles	Gravel		Sand			Silt	Clay
	coarse	fine	coarse	medium	fine		

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MOISTURE-DENSITY RELATIONSHIP

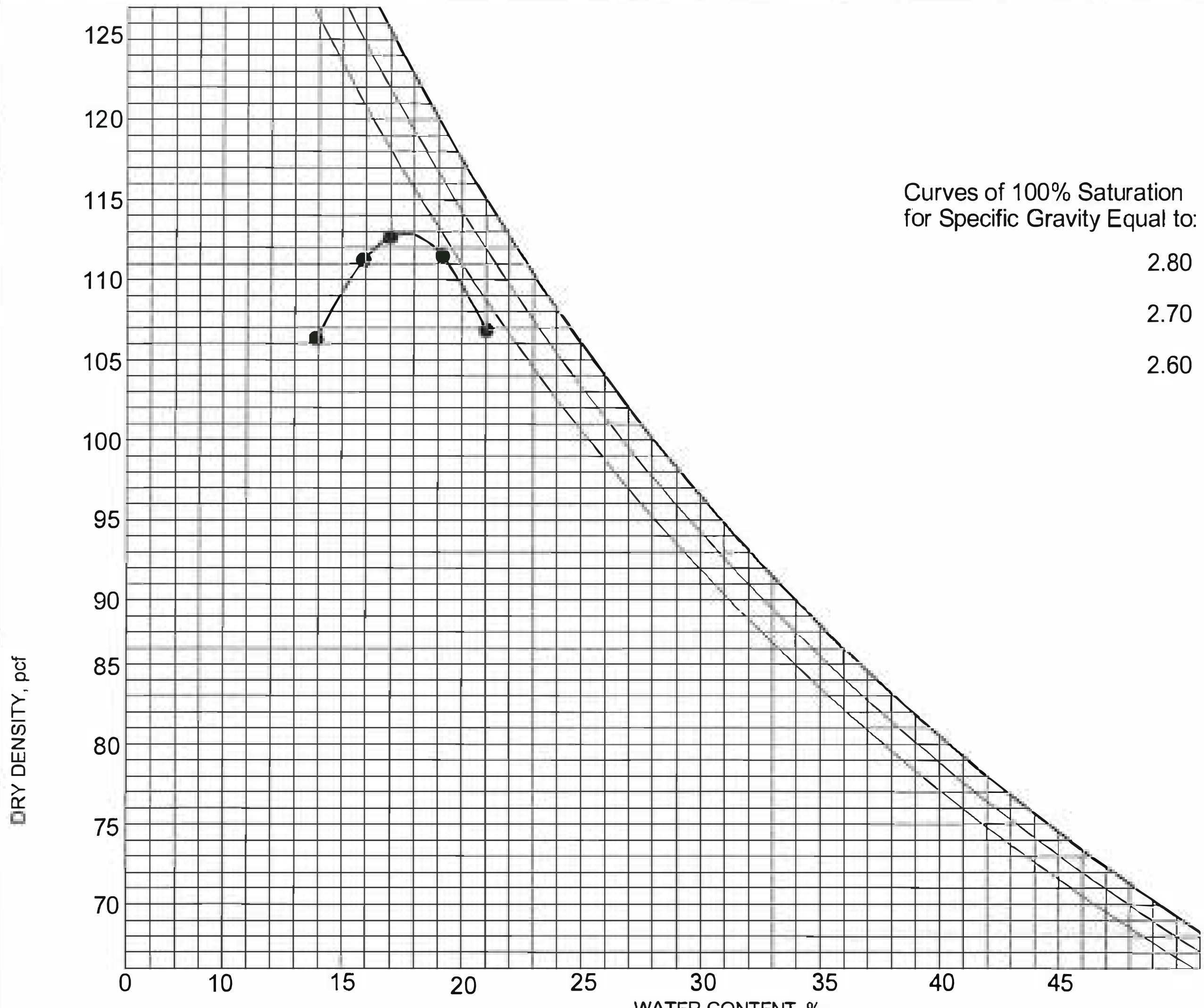
Plate C

CLIENT Outfront Media

PROJECT NAME NEW TRI-FACE BOARDS

PROJECT NUMBER 8309G

PROJECT LOCATION Firestone Blvd & Artesia Blvd., Buena Park, CA



	BOREHOLE	DEPTH	Description of Materials
●	LAB-1	3.0	**Borrow source near B-1**

	BOREHOLE	DEPTH	Test Method	LL	PL	PI	Max DD	Optimum WC
●	LAB-1	3.0	ASTM D1557 Method A	42	18 24	18	112.9 PCF	17.9 %

PLATE D

MAXIMUM DENSITY – OPTIMUM MOISTURE

Test Method: ASTM D1556 & D1557

Sample Number	Optimum Moisture (Percent)	Maximum Density (lbs/ft^3)
Lab-1 (B1-3.0)	17.9	112.9

EXPANSION TEST

Test Method: U.B.C. Standard No. 18-2

Sample Number	Molding Moisture Content (Percent)	Final Moisture Content (Percent)	Initial Dry Density (lbs/ft^3)	Expansion Index	Expansion Classification
B-1 (S-4)	6.3	13.4	114.2	5	Low

SOLUBLE SULFATES

Test Method: Hach DR3 (Calcium Phosphate Extractable)

Sample Number	Soluble Sulfate (ppm)
B1, S-2	14

CHLORIDE TESTS

Test Method: California Test 422 (Department of Transportation)

Sample Number	Soluble Chloride Content (Average) (ppm)
B1, S-2	15

APPENDIX C

Liquefaction Analysis

LIQUEFACTION EVALUATION REPORT

Historic ground water level is recorded at minus 20 feet although no groundwater observed during the field explorations. We evaluated Liquefaction Potential at 15 feet below the ground at which level the liquefaction is believe most critical for the proposed building foundation responses, utilizing SPT blow count data obtained from our field soil explorations. The SPT blow count data obtained from subsurface boring are relatively high for silty and clayey sand and would indicate generally low liquefaction potential.

We calculated Plasticity Indices (PI) based on the results of our laboratory Atterberg Limits tests that are Liquid Limit and Plastic Limits. They are indicated in the boring logs in this report, which show Plasticity Indices (PI) in the range between 17-22.

We believe that the project specific site data indicate that the building foundation is less likely susceptible to liquefaction of the following reasons:

1. Factor of safety against liquefaction is greater than one;
2. There were no groundwater observed in the foundation soil during our subsurface boring up to 50 feet in depth;
3. Per Guidelines for Evaluating Seismic Hazards in California (SP 117A) and the Liquefaction Susceptibility Criteria by Bray and Sancio (2006), the average plasticity index, PI =18 > 12, and the average moisture content, M.C. ≤ 85 %.

We further conclude that the seismic total and differential settlements are anticipated 1" and 1/2", respectively, according to the Simplified Method of Evaluating Earthquake-induced Differential Settlements of Buildings on Cohesive Soils by Xiaming Yuan and others (2004). The aforementioned differential settlement shall be considered to develop over the horizontal distance of 30 feet.

We believe, according to our study, that liquefaction Potential and Seismic Hazards due to loss of shear strength and structural settlements are less than significant.

The soil samples were tested by LEEDCO Engineers' soil laboratory, which is currently certified by the City of Los Angeles as an approved Testing Agency.

LIQUEFACTION POTENTIAL CALCULATIONS AT 30 FEET DEPTH

1. **Cyclic Stress Ratio (CSR) Evaluation:**

$$CSR = 0.65 \cdot a_{\max} \cdot \left(\frac{\sigma_0}{\sigma_0^{'}} \right) \cdot r = (0.65) \cdot (0.773) \cdot (1.09) \cdot (0.96) = 0.526$$

Where,

$$r = 1 - 0.00765 * Z = 1 - 0.00765 * (6.1m) = 0.96$$

$a_{\max}$ is the peak horizontal ground acceleration (PGA) in g.

σ_0 = total vertical overburden stress

$\sigma_0^{'}$ = effective vertical overburden stress

$\sigma_0^{'} = \sigma_0$ because water table is observed far below the foundation soil prism.

Therefore $\sigma_0 / \sigma' = 1.0$

Z is the depth of the soil in meters

2. **Cyclic Resistance Ratio $(CRR)_{7.5}$ Evaluation:**

Blow count numbers, N = 23
Corrected number, N_{60}:

$$N_{60} = N \cdot C_N \cdot C_E \cdot C_B \cdot C_R \cdot C_S = 23*(1.105) \cdot (1.13)*(1.0)*(0.95)*(1.2) = 32.8$$

Where,

$$C_N = \left(\frac{P_a}{\sigma_0^{'}} \right)^{0.5} = \left(\frac{2,088\,psf}{1,708\,psf} \right)^{0.5} = 1.105,$$

$P_a = 2,088 psf \qquad (atmospheric \quad pressure \quad 1atm = 2088\ psf)$

$\sigma'_0 = 1,708\ psf = 126.7\ pcf * 20' + 110\ psf(new\ building) - 62.4$ x 15' water $\qquad (overburden \quad stress)$

$C_E = 1.13 \qquad (ERi/60) \qquad (Hammer \quad Energy \quad Correction \quad Factor)$

$C_R = 0.95 \qquad (rod \quad length \quad correction \quad factor)$

$C_B = 1.0 \qquad (borehole \quad diameter \quad correction \quad factor)$

$C_S = 1.2$ (no liner used)

$(CRR)_{7.5}$: Cyclic Resistance Ratio

CSR : Cyclic Stress Ratio

$N_{60} = 32.8$

$$(CRR)_{7.5} = \frac{1}{34-32.8} + \frac{32.8}{135} + \frac{50}{(10N_{60}+45)^2} - \frac{1}{200} = 0.833 + 0.23 + 0.00036 - 0.005 = 1.071$$

$$F.S. = \frac{(CRR)_{7.5}}{CSR} = \frac{1.071}{0.526} = 2.04 > 1.00 \quad \text{(O.K.)}$$

CONCLUSION:

Liquefaction Analysis indicates that Liquefaction Potential at the Project site is less than significant by observing the higher F.S. greater than unity. Therefore, any special dynamic design to consider Liquefaction Potential is unnecessary.

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E.2 - Professional Opinion Letter

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LEEDCO ENGINEERS, INC.

STRUCTURAL AND CIVIL
ENGINEERING CONSULTANTS

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December 29, 2022

Outfront Media
1731 Workman Street
Los Angeles, CA 90031

Attn: **Mr. Dave Ryan**

Subject: **New Tri-Face L.E.D. Boards located at Intersection of Firestone Blvd. & Artesia Blvd., Buena Park, CA 90621**
APN # 066-020-36
Leedco File No.: 8309G

References:
1) **Geotechnical Report by Leedco Engineers, Inc., dated November 15, 2022.**
2) **E-Mail Memo from Cecilia So, Sr. Project Manager, City of Buena Park to Dave Ryan of Outfront Media, dated December 21, 2022.**

Dear Mr. Ryan:

In connection with the City's comments (Ref. 2), we wish to address a few notes as follows:

Our Geotechnical Investigation Report (Ref. 1) for the subject project contains several recommendations and the conclusions stating that the project site is suitable for the proposed project so long as the recommendations are incorporated.

We wish to address that the recommendations are to give design parameters and recommendations in structural aspects so that the sign designer can follow in his/her design work. We further state that there are no mitigations recommended in terms of any chemical, biologic, ground contamination or other environmental aspects including geotechnical remedies that need to be incorporated as mitigation measures.

In conclusion, we believe that our recommendations in our Geotechnical Investigation Report need not be incorporated as mitigation measures in the ISND, and that the recommendations in our geotechnical report are project structural design purposes only.

Please contact us if you have any questions.

Sincerely,

REGISTERED PROFESSIONAL ENGINEER
C. DENNIS LEE
GE 2152
EXP. 03/31/2024
GEOTECHNICAL
STATE OF CALIFORNIA

C. Dennis Lee, P.E.
Principal

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