Appendix C

Preliminary Geotechnical Investigation and Percolation/Infiltration Rate Study

P.A. & ASSOCIATES, Inc.

May 22, 2019
Proposal File No. 219118-101

Soil Engineering
Geology
Material Testing
Environmental

Mr. Richard De La Fuente, NCARB AIA
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Diamond Bar, CA 91765

Subject: Preliminary Geotechnical Investigation
& Percolation/Infiltration Rate Study
Proposed Retail Plaza Land Development
2530-2534 Westminster Ave.
City of Santa Ana, California

References: Appendix G

Dear Mr. De La Fuente:

We are pleased to submit the results of our preliminary geotechnical investigation and percolation/Infiltration rate study for the subject project. This report is based upon our review of the referenced reports, geologic documents, and engineering review and analysis.

In general, the site appears suitable for redevelopment from a geotechnical standpoint. It is our professional opinion that the proposed buildings and parking areas should not have an adverse geotechnical effect on adjacent properties provided our recommendations contained in this report are incorporated in the project design and development. This report documents our findings, conclusions, and recommendations.

We appreciate the opportunity to be of service. Should questions arise pertaining to any portion of this report, please contact this firm, in writing, for further clarification.

Respectfully,
P.A. & Associates, Inc.

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PRELIMINARY GEOTECHNICAL INVESTIGATION
& PERCOLATION/INFILTRATION RATE STUDY
PROPOSED RETAIL PLAZA LAND DEVELOPMENT
2530-2534 WESTMINSTER AVENUE
CITY OF SANTA ANA, CALIFORNIA

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May 22, 2019
PROJECT FILE NO. 219118-101

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& PERCOLATION/INFILTRATION RATE STUDY
PROPOSED RETAIL PLAZA LAND DEVELOPMENT
2530-2534 WESTMINSTER AVENUE
CITY OF SANTA ANA, CALIFORNIA**

1.0 INTRODUCTION

This report presents the results of our Preliminary Geotechnical Investigation and percolation/infiltration rate study conducted for the proposed two commercial buildings with associated parking space project located at 2530-2534 Westminster Avenue, in the City of Santa Ana, California. (see Site Vicinity Map, Figure 1).

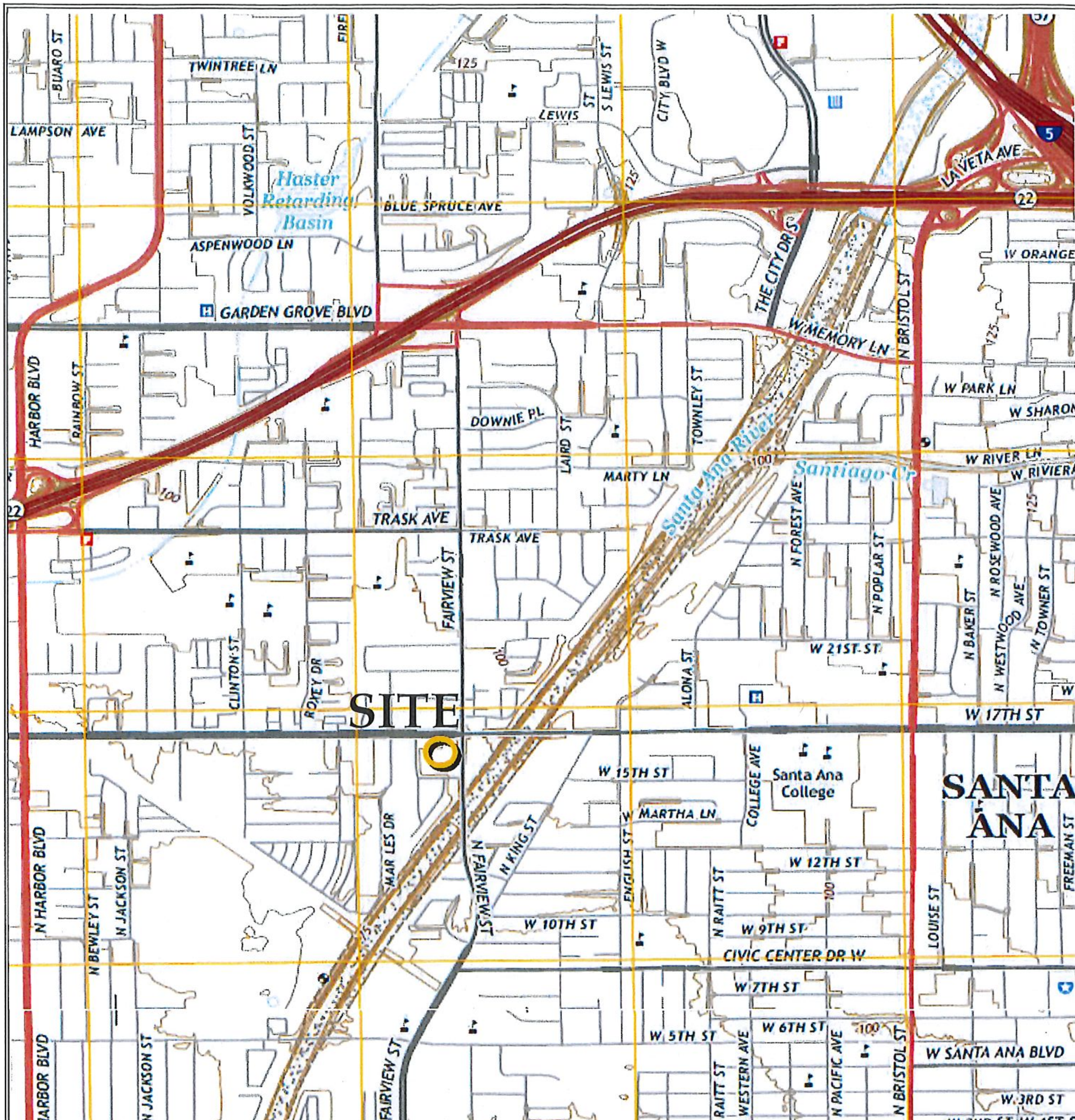
1.1 Scope of Work

This investigation was performed in order to obtain geotechnical data pertinent to redevelopment of the site. The procedures used for this investigation consisted of:

- Review of referenced report and available published and unpublished geotechnical documentation pertaining to the subject site,
- Sub-surface exploration, logging and soil sampling with hollow stem auger;
- Correlative laboratory testing;
- Geotechnical review and analysis;
- Percolation/Infiltration rate testing of the proposed BMP dry well/infiltration trench facility location; and;
- Preparation of this report, illustrations, and plans.

1.2 Objective of this Report

The purpose of this report is to summarize our findings, provide conclusions as to the overall suitability of the project site, and present recommendations for development from a geotechnical standpoint. The field exploration was undertaken on May 13 to 14, 2019. The geotechnical plan with approximate boring locations is shown in Appendix A, Figure A-1-1.



Reference: TOPO! USGS 7.5 Min. Quadrangle Maps, Los Angeles Region

Scale: 1 — 0.5 — 0 — 1 MILES

P. A. & ASSOCIATES, INC.

TITLE:	SITE VICINITY MAP		FIGURE NO.:
PROJECT:	2530-2534 Westminster Ave., Santa Ana, CA		1
PROJECT NO:	219118-101	DATE: 5/22/19	BY: JR

1.3 Site Description

At the time of our investigation, the previous development had been demolished, leaving remnant pavement and gravel on the project site. The property is relatively level, "L" shaped, composed of two rectangular shapes with approximate dimensions of 380 feet long x 195 feet wide and 140 feet long and 97 feet wide. The site is bordered by Westminster Ave. on the north, commercial/industrial property on the east, Harbor Drive and residential properties on the west, and more residential properties on the south.

1.4 Planned Land Use

Based upon our review of the project site plan, it is our understanding that new construction will include the construction of two commercial/industrial buildings with associated parking areas.

1.5 Use of Report

This report was prepared for the exclusive use of Primior and their design consultants for re development of the site. Use of this report by others, or for purposes other than those stated herein may be subject to misinterpretation, and is therefore not recommended. P.A. & Associates, Inc. should review the final project and foundation plans for applicability and compliance with the recommendations presented herein.

2.0 GENERAL GEOLOGY AND SUBSURFACE CONDITIONS

2.1 Regional Geology

The project is situated within the southern portion of the Central Block of the Los Angeles Basin which is bounded by the Whittier Fault to the east, the Santa Ana Mountains of the Peninsular Ranges Province to the southeast, the Newport-Inglewood Fault to the west, and the Santa Monica Fault to the north. The basement rocks comprise slightly metamorphosed sedimentary rocks intruded by late Cretaceous plutonic rocks of the southern California batholith. They are exposed in the Santa Ana Mountains with a thick sequence of younger marine and non-marine late Cretaceous sedimentary rocks superimposed. The basement is bowed downward from the edges of the Central Block to the deepest part of the Los Angeles basin more than 30,000 ft. below sea level near the juncture of the Los Angeles River and Rio Hondo. The project site lies approximately 0.1 miles to the west of the Santa Ana River Channel, and is mapped as underlain by Quaternary young alluvial fan deposits. No fault traces are mapped across the site in our geological references. See Geologic Site Vicinity Map, Figure 2 on page 5.

2.2 Quaternary Young Alluvial Fan Deposits (Qyf)

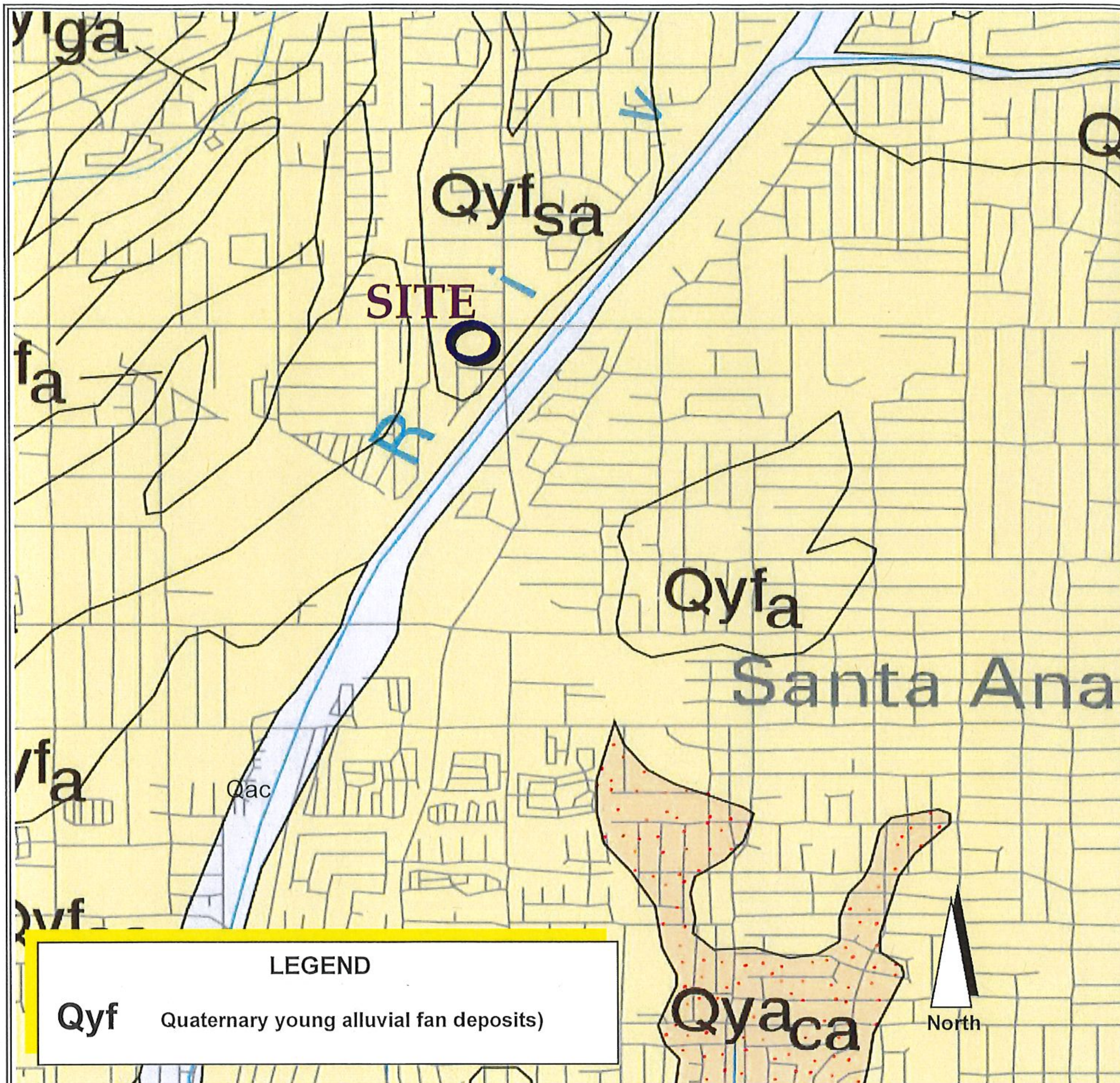
The site is underlain by Quaternary young alluvial fan deposits; which underlie the ground surface and existing pavement remaining from the previous development and locally approximately three feet of distressed pavement comprising concrete slab over layers of asphalt and aggregate base as well as anticipated utility trench fill. The native alluvium sampled generally consisted of light yellowish brown or gray quartzo-feldspathic poorly graded sand encountered at depths of 0-9.5 feet, which was fine to medium-grained, moist and medium dense, overlying clays and sands below 13 feet. The alluvial sands encountered in our boring were wet or saturated below approximately 22.5 feet.

2.3 Groundwater and Caving

Groundwater was encountered in our borings at depth of 22.5 feet during our subsurface investigation. Caving was not evident during advancement of the augers.

2.4 Fault Zones

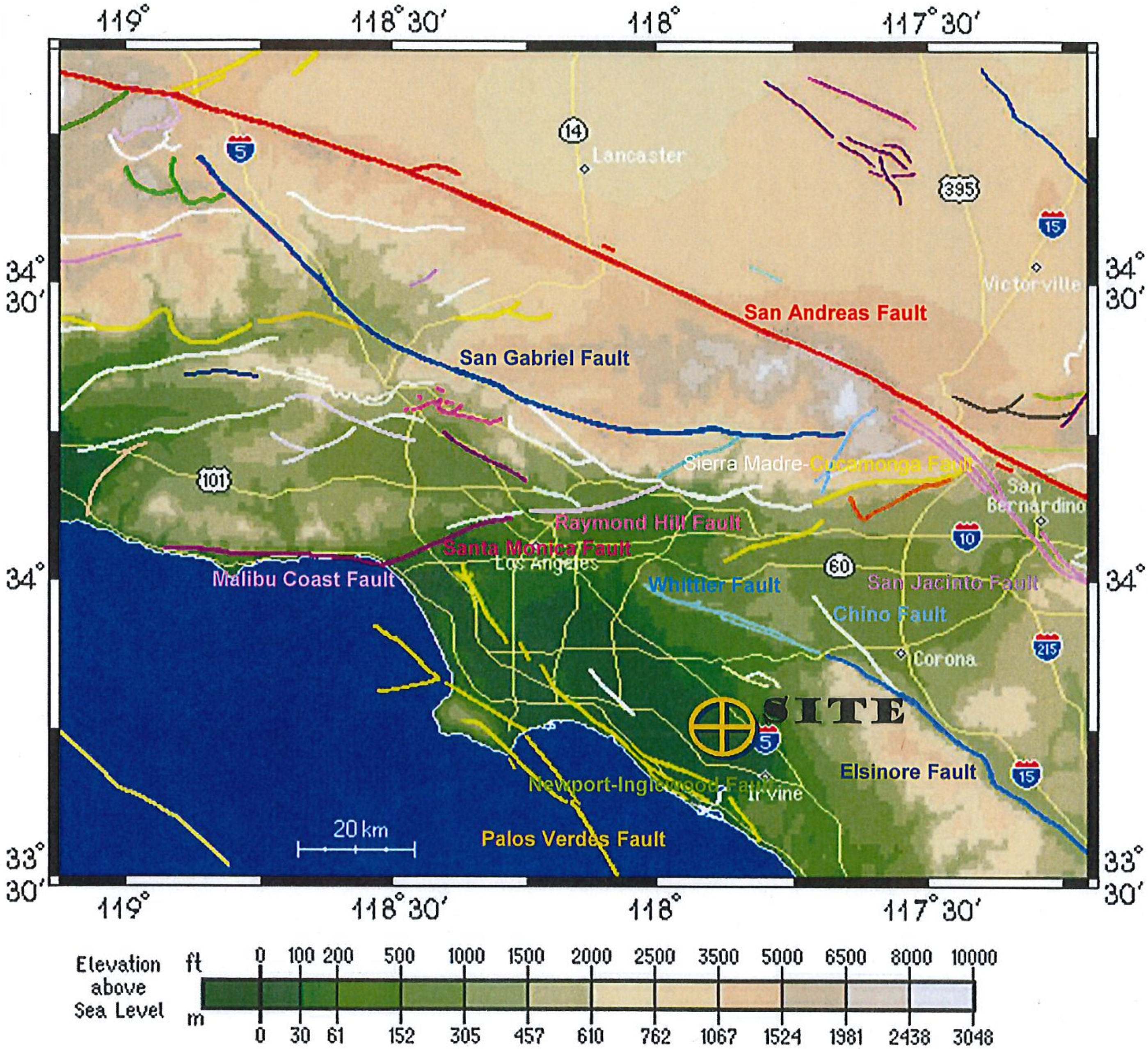
The site is not located within an Alquist-Priolo Earthquake Fault Zone. It may be expected to experience moderate to potentially severe ground shaking from earthquakes generated on the Newport-Inglewood Fault, Norwalk Fault, Santa Monica–Raymond Hill, Sierra Madre–Cucamonga, San Jacinto, and San Andreas Faults which are present within 100 kilometers of the site. In addition, site may also experience lesser ground shaking from earthquakes on other faults within the southern California region. See Active Fault Map for Southern California, Figure 2a.



Reference: Morton, D. M., 2004, USGS Open-File Report 99-172.

Scale: 1:100,000 (400%)

P. A. & ASSOCIATES, INC.	TITLE:	GEOLOGIC SITE VICINITY MAP				FIGURE NO.: 2
	PROJECT:	2530-2534 Westminster Ave., Santa Ana, CA				
	PROJECT NO:	219118-101	DATE:	5/22/19	BY: JR	



Reference:	Southern California Earthquake Center (SCEC)		N
Scale:	0 — 40 km		

P. A. & ASSOCIATES, INC.

TITLE:	**ACTIVE FAULTS OF S. CALIFORNIA MAP**		FIGURE NO.:
PROJECT:	**2530-2534 Westminster Ave., Santa Ana, CA**		**2a**
PROJECT NO:	**219118-101**	DATE: **5/22/19**	BY: **JR**

3.0 SOILS ENGINEERING

Four (4) exploratory borings, including three (3) percolation/infiltration rate test borings, were advanced on site to evaluate soil conditions to the maximum depth of 52 feet. The borings were logged and sampled by a qualified representative of this office. Approximate boring locations are presented on the attached geotechnical plan, Appendix A, Figure A-1-1. Engineering properties and physical parameters of soils sampled and tested during our investigation are summarized as follows:

3.1 Classification

Field classifications were verified in the laboratory by visual examination. The final soil classifications are described in accordance with the Unified Soil Classification System and are shown on the boring logs.

3.2 In-Place Moisture and Density

In-place moisture and density were determined in accordance with ASTM Test Designation D-2937. Samples were obtained in our exploration borings in steel cylinders (rings) of known volume, sealed on-site, and transported to our laboratory for testing. Results are shown on the exploration logs in Appendix A.

3.3 Maximum Density - Optimum Moisture

Maximum density and optimum moisture were determined on representative samples secured from our subsurface explorations in accordance with ASTM Test Designation D-1557. The results of these tests are presented in the Summary of Laboratory Testing in Appendix B.

3.4 Direct Shear

Soil strength was determined for soils samples in accordance with ASTM Test Designation D-3080. The direct shear machine employed was a conventional single shear, strain-controlled device. Strain rates of approximately 0.005 inches per minute were utilized for remold samples. Results of these tests are utilized in determining stability analyses of the site and allowable soil bearing values. The result of these tests indicated a maximum cohesion of 50 psf, with a corresponding internal friction angle of 28 degrees. This test is summarized in Appendix B as Plate B-3.

3.5 Expansion Potential

Expansion potential was tested in accordance with U.B.C. Standard 18-2. In general, soils sampled during our field exploration very low expansion potential. However, corroborative expansion tests should be performed after grading for the proposed building pad. The expansion test result is summarized in Appendix B.

3.6 Consolidation Test

Consolidation tests are used to determine the potential for soils to consolidate under increasing load. Results of these tests assist in determining remedial work requirements. The results of these tests, in accordance with ASTM Test Designation D-2435, are presented in the Summary of Laboratory Testing in Appendix B.

3.7 Plasticity Index

The Plasticity Index (PI) was determined in accordance with A.S.T.M. Test D 4318 and the test results are summarized in Appendix B.

4.0 CONCLUSIONS

4.1 Feasibility of Development

From a geotechnical standpoint, redevelopment of this site is considered feasible and the proposed development will not adversely affect adjacent structures provided our conclusions and recommendations herein are incorporated in the design and construction of the project under supervision from a representative of this firm.

4.2 Earth Materials

The earth materials encountered on the site generally consisted of sands to an approximate depth of 13 feet and clays below to more than 30 feet.

4.3 Possible Near-Source Earthquake Events

The subject site is located within a State of California earthquake seismic liquefaction hazard zone, determined in compliance with the Seismic Hazards Mapping Act (the Act) of 1990 (Public Resources Code, Chapter 7.8, Division 2); the site is located in a seismically active area, and the potential for strong ground motion in the project area is considered significant. The site is located approximately 13.6 kilometers northeast of the Newport-Inglewood fault and approximately 11.6 km southwest of the Chino fault and may be expected to experience potentially severe shaking.

The site is expected to experience Magnitude 6.9 Mw, Moment Magnitude (USGS Unified Hazard Tool) with Mercalli Intensity of VIII from a major earthquake and Peak Horizontal Acceleration of 0.529g with a 2% probability of exceedance in 50 years (USGS Seismic Design Maps). Seismic criteria for a near source event should be considered in the design of the structure. Based on available information from our field investigation and geologic publications, the likelihood of significant surface fault rupture or ground deformation at the site is considered to be relatively low.

4.4 Groundwater

Groundwater was encountered at depth of 22.5 ft. b.g.s. (below ground surface) in our deep boring. However, historically highest groundwater elevation contours (Open-File Report 97-08) indicate that the groundwater could rise to about 15 ft. b.g.s. In consideration of the more recent river channel paving engineered to mitigate flooding and infiltration from the Santa Ana River (0.1 miles to the east), we anticipate highest groundwater elevations to remain in approximately 20 feet below the surface.

4.5 Liquefaction

For liquefaction to occur, all of three key ingredients are required: liquefaction-susceptible soils, groundwater within a depth of 50 feet or less, and strong earthquake shaking. Soils susceptible to liquefaction are generally saturated loose to medium dense sands and non-plastic silt deposits below the water table. According to the State of California Seismic Hazard Zones Map (CDMG, 1998), the site is within of an area identified as having a potential for liquefaction. A historic high groundwater level of 15 feet, although groundwater was encountered at depth of 22.5 feet, was adopted in our liquefaction analysis based on documented historically high groundwater elevations plotted on the contour map prepared by California Division of Mines and Geology (Open-File-Report 97-08),.

To evaluate the site-specific liquefaction potential, we computed the geometric mean peak ground acceleration (PGA) for the ground motion with a 2% probability of being exceeded in 50 years (Peak Ground Acceleration = 0.529) from USGS Seismic Design Maps website for MCE_R. We used USGS 2008 Interactive Deaggregations website tool developed by USGS to calculate probabilistic response spectra with different hazard levels for spectral periods of up to 5 seconds at any location with a given average shear wave velocity in the upper 30 meters (Vs30). Based on the available documentation on the website, the attenuation relationships used in development of the response spectra in the Western United States are the three Next Generation Attenuation (NGA) relationships of Boore and Atkinson (2007), Campbell and Bozorgnia (2008), and Chiou and Youngs (2008). The results of the probabilistic seismic hazard analysis indicate the modal seismic event is Moment Magnitude (Mw) 6.9. The SPT consists of driving a standard sampler, as described in the ASTM 1586 Standard Method, using a 140 pound hammer falling 30 inches. An Automatic "Trip" Hammer was used to drive samplers 18 inches into the soil. For the measured 79 percent hammer efficiency of an automatic hammer, the energy ratio 1.32 was used in our liquefaction evaluation. We have used the borehole diameter correction factor (CB) 1.15 in our liquefaction evaluation.

The screening criteria of Bray and Sancio (2006) were used to determine if fine-grained soils within boring B-1 are susceptible to liquefaction. To determine if soils are susceptible to liquefaction, the Plasticity Index (PI) and in-situ moisture content were determined. For screening analysis purposes, all soil samples above and below the groundwater table were soaked and saturated, and then tested for moisture content. For PI greater than 12 and moisture content less than 80 percent of liquid limit, clayey soils are not susceptible to liquefaction. For liquefaction of sandy soils, we used methods proposed by Tokimatsu and Seed (1987). Based on our analysis and under the current site conditions, we estimate that the maximum total liquefaction-induced ground settlements at the site would of about 1.44 inches during the postulated earthquake. Differential settlements of approximately 1 inch or less could occur over a span of 40 feet. The computer outputs are attached for reference in Appendix C. It is our opinion that potential for liquefaction at the site is low to moderate and will not adversely impact the proposed construction. We recommend that the proposed structures be supported on a mat foundation system.

4.6 Percolation Rates

Three borings (PB-1, PB-2, & PB-3) were utilized for in-situ percolation testing for location of dry well/infiltration trenches on the site. Please see Figure A-1-1, Appendix A for the boring locations. The borings were excavated with an 8-inch diameter auger to depth of 5 feet below the surface, and standard testing was performed. Please see Appendix E attachments for the boring testing method and Appendix F for our percolation testing field logs utilized in accordance with Technical Guidance Document Appendices, California, dated March 22, 2011. Soils encountered in our borings generally consisted of well sorted sands with trace silt. Our percolation testing results for the three percolation test locations indicate an feasible tested infiltration rate of 4.53 inches per hour. A safety factor of 2 is suggested.

5.0 RECOMMENDATIONS

Proposed development for the site are considered geotechnically feasible. Re-grading and any foundation plans should take into account the soil conditions of the site. All earthworks, including excavation, backfill and preparation of subgrade, should be performed in accordance with the geotechnical recommendations presented in this report and applicable portions of the grading code of local regulatory agencies. All earthwork should be performed under the observation and testing of a qualified geotechnical engineer.

5.1 Clearing and Grubbing

Prior to the start of proposed grading operations all demolition and other deleterious materials should be stripped and removed from the site and legally disposed of off-site.

5.2 Sub-Surface Soil Preparation

We recommend that the upper 5 feet of existing soils be removed and recompacted. If loose, disturbed, or otherwise unsuitable materials are encountered at the bottom of excavation, removal of unsuitable soils will be required until firm soils are encountered. At least 3 feet of 93 percent compacted fill soils should be provided underneath the footings.

Pavement areas should be overexcavated 18 inches below existing or proposed grade, whichever is lower, and proposed hardscape areas should be overexcavated 2 feet below existing or proposed grade, whichever is lower. All excavation bottoms should be tested for a minimum of 90 percent relative compaction, if the exposed material possess less than 90 percent, further removal will be required by the project soils engineer.

5.3 Fill

The on-site soils are suitable for use as structural fill, provided the soil is free of any deleterious substance. Fill materials should be free of construction debris, roots, organic matter, rubble, contaminated soils, and any other unsuitable or deleterious material as determined by the Geotechnical Engineer. All fill soils should be placed in maximum 6-inch lifts moisture, moisture conditioned to near optimum moisture content, and properly compacted to at least 90 percent of their maximum density.

5.4 Imported Fill

Any import fill material should be approved by the Geotechnical Engineer prior to importing to the site for use as compacted fill. Any proposed import fill should possess an Expansion Index less than 20, and not to possess any oversize rock larger than four inches, have soil strength parameters equal or greater than the on-site soils, and should be tested and approved by a representative of this firm prior to its use.

5.5 Utility Trench

Bedding material should consist of sand with a Sand Equivalence (SE) of not less than 35, which may then be jetted. Jetting in trenches adjacent to slopes should be carried out only under the specific approval of the soil engineer. Backfill of all trenches should be compacted to achieve a relative compaction of at least 90 percent of their maximum density. Care should be taken not to damage utility lines. The site soils are considered suitable for use as trench backfill, provided they are at or near optimum moisture. The walls of temporary construction trenches are expected to be stable when excavated nearly vertical, with only minor shoring, provided the total depth does not exceed about 4 feet. Shoring of excavation walls or flattening of slopes may be required, if greater depth is necessary. All work associated with trench shoring must minimally confirm to Cal-OSHA and local safety codes.

5.6 Suggested Preliminary Pavement Section

Based on an assumed R-Value of 30, the State of California Highway Department design procedures, a Traffic Index = 5.0 for driveways and parking areas with light automobile traffic, and a traffic index of 7.0 for light traffic of heavy trucks and heavy traffic of light vehicles, the following pavement sections are suggested:

- Driveways with light auto traffic & parking areas
  Traffic Index (TI) = **5 .0**
  Asphalt Concrete (AC) = **3.5 inches**
  Aggregate Base (AB) Class II = **4.5 inches**

- Light traffic of heavy trucks & heavy traffic of light vehicles
  Traffic Index (TI) = **7.0**
  Asphalt Concrete (AC) = **5.5 inches**
  Aggregate Base (AB) Class II = **6 inches**

AB and AC should be compacted to a minimum of 95 percent relative compaction. AC should be compacted in layers not exceeding 3.5 inches. During grading operations, subgrade material sampled from the parking area should be tested for R-Value.

6.0 FOUNDATION RECOMMENDATIONS

6.1 Continuous and Square Pad Footings

Continuous footings should be minimum of 18 inches and 24 inches wide and 24 inches deep for one and two stories, respectively. Square pad footings should be founded at least 24 inches below lowest adjacent grade, and should have a minimum dimension of 24 inches square. All continuous footings should be reinforced with two No.4 bar on top and two at the bottom. Spread footings should be reinforced with No.4 bars spaced 12 inches on-center each-way, unless determined otherwise by the Structural Engineer. All square pad footings should be inter-connected through an 18-inch deep grade beam system.

6.2 Bearing Capacity (Continuous and Square Pad Footings)

Continuous footings may be designed with an allowable bearing capacity of 1600 pounds per square foot. Square pad footings may be designed with an allowable bearing capacity of 1800 pounds per square foot. These values may be increased by 150 psf for each additional width or depth of footing to a maximum of 2000 psf.

6.3 Short-term Seismic or Wind Loads

When designing for short duration wind or seismic loads, the above values can be increased by one-third.

6.4 Seismic Recommendations

The structural design at the site should conform to the most recent 2016 California Building Code requirements for Region 1 and the most recent design standards of the Structural Engineers Association of California. Based on the artificial fill materials encountered and site coordinates (Lat. 33.759152°N, Long. -117.903991°W) the following ASCE 7-10 Standard seismic recommendations are made:

- Site Class = D, CBC 2016, Table 1613.5.2;
- S_S = 1.455; CBC 2016, Section 1613.5.1;
- S_1 = 0.533; CBC 2016, Section 1613.5.1;
- F_a = 1.0, CBC 2016, Table 1613.5.3(1);
- F_v = 1.5, CBC 2016, Table 1613.5.3(2);
- S_{MS} = 1.455, CBC 2016, Section 1613.5.3;
- S_{M1} = 0. 800, CBC 2016, Section 1613.5.3;
- S_{DS} = 0.970, CBC 2016, Section 1613.5.4;
- S_{D1} = 0.533, CBC 2016, Section 1613.5.4;
- Seismic Design Category is D, CBC 2016, Table 1613.5.6(1&2).

6.5 Slabs-on-Grade

Slabs-on-grade should be at least 6 inches in thickness and should be reinforced with No.4 bars at 16 inches on center each way. Six (6) inches of crushed rock should be placed under all slab areas. Sawcuts (or cold joints) should be made on all exterior slabs, walkways and driveways, at maximum of 10 feet intervals each way, with a maximum length to width ratio of 2. Exterior slabs should have thickened edges of 18 inches deep and 6 inches wide.

6.6 Type of Cement

Cement to be used in the concrete which is in contact with on-site soils should be determined after the completion of grading operations. In lieu of water soluble sulfate testing, sulfate resistant cement consisting of Type V cement with minimum compressive strength of 4500 psi and a maximum water to cement ratio of 0.45 should be specified (Table 19-A-4 of Uniform Building Code).

6.7 Temporary Shoring/Bracing

For shoring/bracing of temporary cut surfaces steeper than 1H:1V higher than 4 feet, we recommend trapezoidal/triangular distribution of earth pressure. For design detail, please refer to Appendix D, Fig. D-1. As an alternative to temporary shoring, the excavations may be sloped back 1:1 H:V above the 4 feet vertical cut.

6.8 Settlement

Subject to implementation of recommendations contained herein, structures may be expected to settle a maximum of three quarter-inch and a differential settlement on the order of one quarter-inch over a 40 feet horizontal distance. Most settlement may be expected to occur during construction. The structural tolerance of the expected differential settlement and maximum settlement should be adequately evaluated by the project structural engineer. Our analysis of potential seismic induced settlement of saturated soils for the proposed development indicate the potential for approximately 1.44 inches of total settlement on the site due to earthquakes with historically high groundwater (15 ft. b.g.s.). The combined total and differential settlements are not expected to be greater than 2.2 inches and 1.3 inches, respectively.

6.9 Surface Drainage Provisions

Positive surface gradients should be maintained away from planned structures, berm areas, etc., such that water is not allowed to flow uncontained on site, and should be contained in approved drainage devices. The ground immediately adjacent to the foundation shall be sloped away from the building at a slope of not less than one unit vertical in 20 units horizontal (5 percent slope) for a minimum distance of 10 feet (3048 mm) measured perpendicular to the face of the wall. If physical obstructions or lot lines prohibit 10 feet (3048 mm) of horizontal distance, a 5 percent slope shall be provided to an approved alternative method of diverting water away from the foundation. Swales used for this purpose shall be sloped a minimum of 2 percent where located within 10 feet (3048 mm) of the building foundation. Impervious surfaces within 10 feet (3048 mm) of the building foundation shall be sloped a minimum of 2 percent away from the building. Exception: Where climatic or soil conditions warrant, the slope of the ground away from the building foundation shall be permitted to be reduced to not less than one unit vertical in 48 units horizontal (2-percent slope). The procedure used to establish the final ground level adjacent to the foundation shall account for additional settlement of the backfill. Construction plans shall indicate how the site grading or drainage system will manage all surface water flows to keep water from entering buildings in accordance with the California Green Building Standards Code (CALGreen), Chapter 4,

6.10 Geotechnical Observation and Testing

Based on the City of Santa Ana's quality control requirements and as an assurance for desired workmanship, observation and testing by our representative is recommended with the specified activities below:

- During clearing of the site;
- During rough and precise grading of the site;
- During footing excavation for building and/or retaining wall, prior to backfill;
- During placement of sand and moisture barrier for the proposed slabs;
- During trenching and placement of dry well or any on-site infiltration system;
- During backfill of utility trenches and retaining walls;
- During sub-base, aggregate base, and paving asphalt placement;
- When any unusual geotechnical conditions are encountered.

7.0 CLOSURE

7.1 Uniformity of Conditions

The recommendations and opinions expressed in this report reflect P.A. & Associates best estimate of the project requirements based on an evaluation of the subsurface soil conditions encountered at the subsurface exploration locations and the assumption that the soil conditions do not deviate appreciably from those encountered. It should be recognized that the performance of the pavement, slab, and foundation may be influenced by undisclosed and unforeseen variations in the soil conditions that may occur in intermediate and unexplored subsurface areas. Any unusual conditions not covered in this report that may be encountered during grading and construction should be brought to the immediate attention of the Soils Engineer and/or Engineering Geologist so that we may make modifications to our recommendations if necessary.

7.2 Time Limitations

The findings of this report are valid as of this date. Changes in the condition of a property can, however, occur with the passage of time, whether they be due to natural processes or the work of man on this or adjacent properties. In addition, changes in the State-of-the-Art and/or government codes may occur. Due to such changes, the findings of this report may be invalidated wholly or in part by changes beyond our control. Therefore, this report should not be relied upon after a period of one year without a review by us verifying the validity of the herein contained conclusions and recommendations.

7.3 Professional Standard

In the performance of our professional services, P.A. & Associates complies with the standard of care and skill normally exercised under similar circumstances by members of our profession currently practicing under similar conditions in the same or similar localities. The client recognizes that subsurface conditions may vary from those encountered at the locations where our borings, surveys and explorations are made, and that P.A. & Associates data, interpretations, and recommendations are based solely on the information obtained by us. We will be responsible for those data, interpretations and recommendations, but shall not be responsible for the interpretations by others of the information developed. Our services consist of professional consultation and observation only, and no warranty is expressed, implied, or intended in connection with the work performed by us, by the furnishing of oral or written reports or findings.

APPENDIX A

BORING LOGS

APPENDIX A-1

FIELD EXPLORATION

Subsurface Exploration

The site exploration program was performed on the dates shown on the accompanying record of field exploration log sheets. Subsurface exploration was conducted in order to visually identify subgrade conditions, sample in-situ moisture and density, and other salient properties. Excavation locations were selected based on the proposed development concept relevance and necessity of subsurface data and accessibility. The number and depth of these excavations are considered adequate for estimation of subsurface conditions and characteristics, based on the development concept discussed in the text of the accompanying report.

Excavation Log Sheets

Exploration borings are graphically shown on the following log sheets. Each log shows the total depth achieved, the materials encountered, their observed engineering properties, results of correlative laboratory tests, and the presence, if any, of groundwater, and the relative competency of excavation walls.

Classification

Earth materials encountered in the excavations were described in accordance with A.S.T.M. Test Designation D- 2487, visual classification of soils or aggregate rock mixtures, per the Unified Soil Classification System (U.S.C.S.). This classification system is explained on Page A-2 of Appendix A.

Sampling

Selected samples from these excavations were collected on site and transported to the laboratory in sealed containers in order to preserve field conditions. Relatively undisturbed samples for in-place soil moisture, density, shear strength, swell and/or consolidation potential were contained utilizing driven steel tubes or brass rings (N* = blow counts per foot. S* = sample type: R= Ring, B= Bulk, T= Drive Tube). Bulk samples were also obtained for determination of maximum density and optimum moisture, as well as for preparation of removed samples for selected laboratory tests. Laboratory testing procedures are described in Appendix B-1. Laboratory test results are shown on the accompanying log sheets, commencing with Figure A-3, and is shown in table format beginning with Figure B-2 in Appendix B.

Use of Logs

Subsurface data depicted on the accompanying logs represent subgrade conditions relevant to the specific location and date shown on Figure A-1 and the log sheets, respectively. Between exploratory excavations, subsurface conditions can and do change with respect to vertical and lateral extent, subsequent precipitation, land use, and other conditions. Interpretation of these logs by others is solely at the risk of the user. These logs document conditions which were used, in part, to form the basis of findings, conclusions and recommendations presented in the text of the accompanying report, and may have no relevance to other applications.

APPENDIX A-2

UNIFIED SOIL CLASSIFICATION SYSTEM

COARSE GRAINED SOILS

(More than 50 percent of material is LARGER than No. 200 sieve size.)

GW Well-graded gravel, gravel-sand mixtures, little or no fines

GP Poorly graded gravel or gravel-sand mixtures, little or no fines

GM Silty gravel, gravel-sand-silt mixtures

GC Clayey gravel, gravel-sand-clay mixtures

SW Well-graded sands, gravelly sands, little or no fines

SP Poorly graded sands or gravelly sands, little or no fines

SM Silty sands, sand-silt mixtures

SC Clayey sands, sand-clay mixtures

FINE GRAINED SOILS

(More than 50 percent of material is SMALLER than 200 sieve size)

ML Inorganic Silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity

CL Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays

OL Organic silts and organic silty clays of low plasticity

MH Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts

CH Inorganic clays of high plasticity, fat clays

OH Organic clays of high plasticity, organic silts

Pt Peat and other highly organic soils

Boundary Classifications: Soils possessing characteristics of two groups are designated by combinations of group symbols.

Particle Size Limits

< No. 200	Silt or Clay
No. 200 to No. 40	Fine Sand
No. 40 to No. 10	Medium Sand
No. 10 to No. 4	Coarse Sand
No. 4 to 3/4 inch	Fine Gravel
3/4 inch to 3 inch	Coarse Gravel
3 inch to 12 inch	Cobbles
> 12 inches	Boulders

Date: 5/13/2019	Logged By: JMR	Equipment: CME 75	Sheet: 1 of 3	Boring No: B-1
Location: Figure A-1-1	Engineer: PA	Hammer: 140lb/30"	Groundwater encountered @ 22.5 ft.	Total Depth: 52 ft.

Depth (ft.)	Blow Count per 12"	Sample Type	Moisture (%)	Dry Density	Visual Log	Soil Class	Material Description	Laboratory Test Results
						SM	**Silty Sand with some pebbles,** dark brown, moist, loose.fine-medium.	
5	11	R	11.7	116.0		SM	**Silty fine-medium Sand,** yellowish brown, moist, medium dense.	
							pea gravel	
10	10	R	9.6	80.8		SP	**Sand,** light gray, moist, medium dense, fine to medium-grained.	
15	15	SPT				CL	**Clay,** dark brown, moist, stiff to very stiff.	
						SP	**Sand,** light gray, moist, medium dense, fine to medium-grained.	
20	9	SPT				CL	**Clay,** brown, moist, stiff.	
						▼	Groundwater at 22.5 feet	
25	9	SPT				CL	**Clay,** brown, moist, stiff.	

NOTE: Please refer to Figure A-2 for Explanation of symbols.

P. A. & ASSOCIATES, INC.

TITLE:	**LOG OF EXPLORATORY BORING**		FIGURE NO.:
PROJECT:	2530-2534 Westminster Ave., Santa Ana, CA		**A-3**
PROJECT NO:	**219118-101**	DATE: **5/22/2019** BY: **JR**	

Date: 5/13/2019	Logged By: JMR	Equipment: CME 75	Sheet: 2 of 3	Boring No: B-1
Location: Figure A-1-1	Engineer: PA	Hammer: 140lb/30"	Groundwater encountered @ 22.5 ft.	Total Depth: 52 ft.

Depth (ft.)	Blow Count	Sample Type	Moisture (%)	Dry Density	Visual Log	Soil Class	Material Description	Laboratory Test Results
---	9					CL	**Clay,** brown, moist, stiff.	

---								
30 ----		SPT						
---	18					CL	**Clay,** brown, moist, very stiff.	

---								
35 ----		SPT						
---	16					SM	**Silty Sand,** grayish brown, saturated, medium dense, medium-grained.	

---								
40 ----		SPT						
---	15					SM	**Silty Sand,** grayish brown, saturated, medium dense, medium-grained.	

---								
45 ----		SPT						
---	15					CL	**Sand Clay,** grayish brown, saturated, medium dense, medium-grained.	

---								
50 ----		SPT						
---	14					SM	**Silty Sand,** yellow, saturated, medium dense, medium-grained.	

NOTE: Please refer to Figure A-2 for Explanation of symbols.

P. A. & ASSOCIATES, INC.	TITLE:	**LOG OF EXPLORATORY BORING**		FIGURE NO.:
	PROJECT:	2530-2534 Westminster Ave., Santa Ana, CA		**A-4**
	PROJECT NO:	**219118-101**	DATE: **5/22/2019** BY: **JR**	

Date: 5/13/2019	Logged By: JMR	Equipment: CME 75	Sheet: 3 of 3		Boring No: B-1
Location: Figure A-1-1	Engineer: PA	Hammer: 140lb/30"	Groundwater encountered @ 22.5 ft.		Total Depth: 52 ft.

Depth (ft.)	Blow Count	Sample Type	Moisture (%)	Dry Density	Visual Log	Soil Class	Material Description	Laboratory Test Results
	14					SM	**Silty Sand**, yellow, saturated, medium dense, medium-grained.	
						CL	**Clay**, gray, moist, stiff.	

T.D. 52 ft.

No Caving

Groundwater encountered at 22.5 ft.

Depth scale: 55, 35, 40, 45, 50

NOTE: Please refer to Figure A-2 for Explanation of symbols.

P. A. & ASSOCIATES, INC.	TITLE:	LOG OF EXPLORATORY BORING		FIGURE NO.: A-5
	PROJECT:	2530-2534 Westminster Ave., Santa Ana, CA		
	PROJECT NO:	219118-101	DATE: 5/22/2019	BY: JR

Date: 5/13/2019	Logged By: JMR	Equipment: CME 75	Sheet: 1 of 1	Boring No: PB-1
Location: Figure A-1-1	Engineer: PA	Hammer: 140lb/30"	Groundwater not encountered	Total Depth: 5 ft.

Depth (ft.)	Blow Count	Sample Type	Moisture (%)	Dry Density	Visual Log	Soil Class	Material Description	Laboratory Test Results
	per 12"					SP	**Sand with trace Silt,** yellowish brown, moist, medium dense, fine to medium-grained.	Expansion index = 12 Expansion Potential = Very Low Max. Dry Density = 111.0 pcf Optimum Moisture =9.5 % Internal Friction Angle = 28 ° Cohesion = 50 psf
5		R	14.3	101.6				
10							**T.D. 5 ft.** **No Caving** **Groundwater not encountered.**	
15								
20								
25								

NOTE: Please refer to Figure A-2 for Explanation of symbols.

P. A. & ASSOCIATES, INC.	TITLE:	**LOG OF EXPLORATORY BORING**			FIGURE NO.: **A-6**
	PROJECT:	2530-2534 Westminster Ave., Santa Ana, CA			
	PROJECT NO:	**219118-101**	DATE: **5/22/2019**	BY: **JR**	

Date: 5/13/2019	Logged By: JMR	Equipment: CME 75	Sheet: 1 of 1	Boring No: PB-2
Location: Figure A-1-1	Engineer: PA	Hammer: 140lb/30"	Groundwater not encountered	Total Depth: 6.5 ft.

Depth (ft.)	Blow Count	Sample Type	Moisture (%)	Dry Density	Visual Log	Soil Class	Material Description	Laboratory Test Results
	per 12"					SP	**Sand withsome Clay,** yellowish brown, moist, medium dense, medium-grained.	Expansion index = 16 Expansion Potential = Very Low Max. Dry Density = 127.0 pcf Optimum Moisture =10.0 %
5		R	14.6	89.0			Test hole filled to 5 feet	
	37						**T.D. 5 ft.** **No Caving** **Groundwater not encountered.**	
10								
15								
20								
25								

NOTE: Please refer to Figure A-2 for Explanation of symbols.

P. A. & ASSOCIATES, INC.				FIGURE NO.: A-7
TITLE:	**LOG OF EXPLORATORY BORING**			
PROJECT:	2530-2534 Westminster Ave., Santa Ana, CA			
PROJECT NO:	**219118-101**	DATE: **5/22/2019**	BY: **JR**	

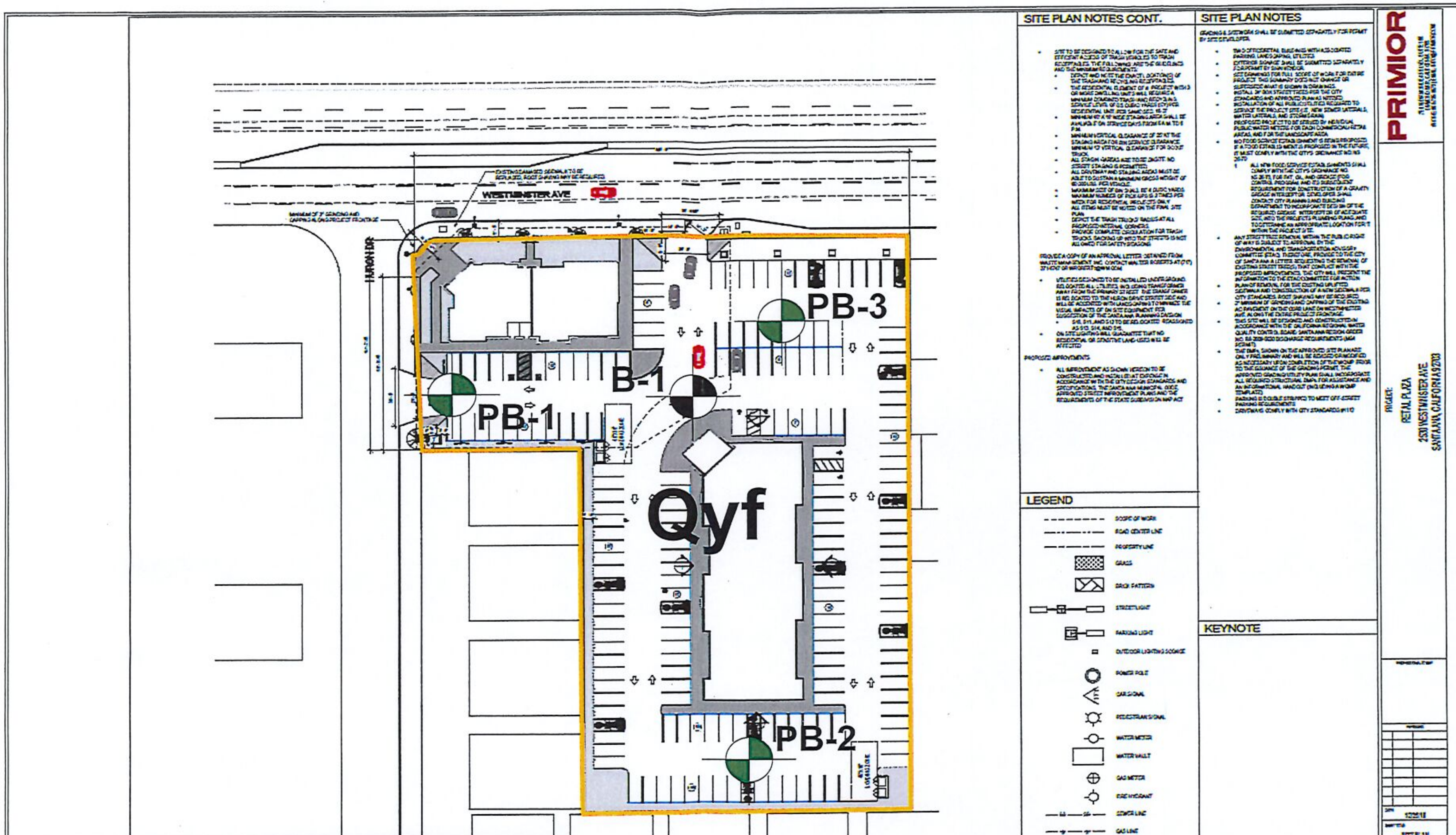
Date: 5/13/2019	Logged By: JMR	Equipment: CME 75	Sheet: 1 of 1	Boring No: PB-3
Location: Figure A-1-1	Engineer: PA	Hammer: 140lb/30"	Groundwater not encountered	Total Depth: 11.5 ft.

Depth (ft.)	Blow Count	Sample Type	Moisture (%)	Dry Density	Visual Log	Soil Class	Material Description	Laboratory Test Results
	per 12"					SP	**Sand with some Pebbles,** yellowish brown, moist, loose to medium dense, medium-grained.	
5		R	8.3	114.6				
	6					SP	**Sand with trace Silt,** yellowish brown, moist, loose to medium dense, medium-grained.	
						SP	**Sand with trace Silt,** yellowish brown, moist, medium dense, fine to medium-grained.	
10		R	15.0	106.1				
	31						**T.D. 11.5 ft.** **No Caving** **Groundwater not encountered.**	
15								
20								
25								

NOTE: Please refer to Figure A-2 for Explanation of symbols.

P. A. & ASSOCIATES, INC.	TITLE: LOG OF EXPLORATORY BORING			FIGURE NO.: A-8
	PROJECT: 2530-2534 Westminster Ave., Santa Ana, CA			
	PROJECT NO: 219118-101	DATE: 5/22/2019	BY: JR	

BORING LOCATION PLAN



LEGEND

Qyf	Quaternary young alluvial fan deposits
B-1	Approximate Boring Location
PB-3	Approximate Percolation Test Boring
	Approximate Limits of Investigation

Reference: Site Plan, 12-26-18, prepared by Primior

Scale: 0 — 80 ft. approx.

P. A. & ASSOCIATES, INC.

TITLE:	BORING LOCATION PLAN			
PROJECT:	2530-2534 Westminster Ave., Santa Ana, CA			
PROJECT NO:	25138-101	DATE: 5/22/19	BY:	JR

FIGURE NO.: A-1-1

APPENDIX B

LABORATORY TESTING

APPENDIX B-1

LABORATORY TESTING PROGRAM

Laboratory tests were performed on selected samples in accordance with one or more of the standard testing methods shown below. A summary of laboratory test results follows, beginning with Appendix B-2. Discussion, conclusions and recommendations pertaining to these results may be found in the text of the accompanying report.

Unified Building Code

No. 18-2 Expansion Potential

State of California
Department of Transportation
(CALTRANS)

No. 216 In-Place Soil Density
No. 301 Resistance (R-Value)

American Standards
for Testing and Materials
(A.S.T.M)

D-4829-11	Expansion Potential
D-421-85(2007)	Grain Size Analysis - Mechanical Method
D-422-63(2007)e2	Grain Site Analysis - Hydrometer Method
D-698-12e2	Moisture, Density
D-854-14	Specific Gravity
D-1556/- D-1556M-15e1	In-Place Soil Density - Sand Cone
D-1557-12e1	Maximum Density and Optimum Moisture
D-1883-16	California Bearing Ratio
D-4254-16	Relative Density
D-2166/D2166M-16	Unconfined Compression
D-2216-10	Water Content Determination
D-2419-14	Sand Equivalence Testing
D-2435/D2435M-11	Consolidation Testing
D-2487-11	Unified Soil Classification System
D-2937-10	In-Place Soil Density - Driven Tube
D-3080/D3080M-11	Direct Shear
D-4318-10e1	Liquid and Plastic Limits and Plasticity Index
D-4253-16	Maximum Index Density
D-4254-16	Maximum Index Density

APPENDIX B-2

LABORATORY TEST SUMMARY

Expansion Test Results

Location	Depth (Feet)	Soil Type	Expansion Index	Expansion Potential
PB-1	1-5	Sand with trace Silt, yellowish brown	12	V. Low
PB-2	1-5	Sand with some Clay yellowish brown	16	V. Low

Maximum Dry Density - Optimum Moisture

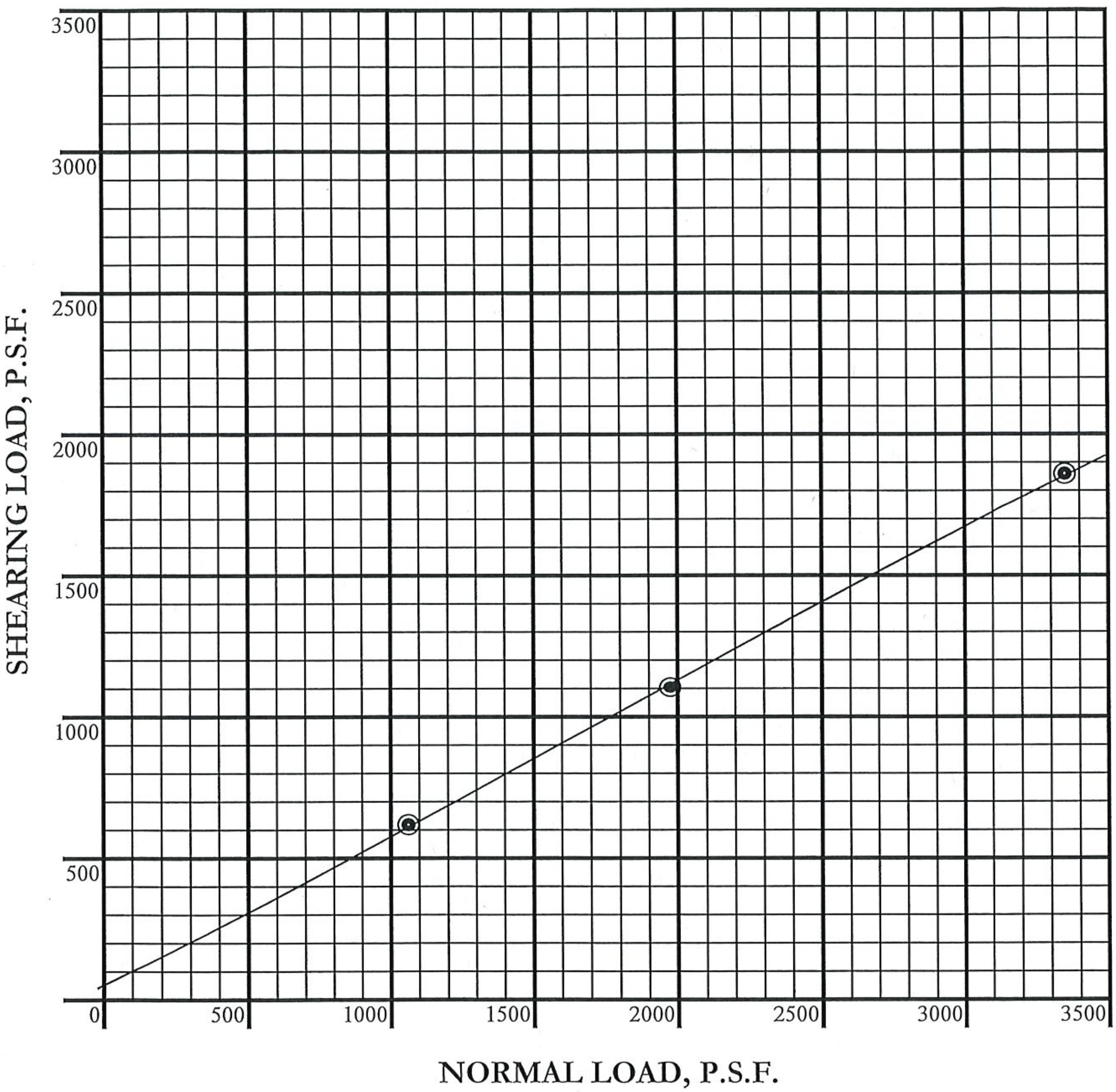
Location	Depth (Feet)	Soil Type	Maximum Index	Optimum Potential
PB-1	1-5	Sand with trace Silt, yellowish brown	111.0 pcf.	9.5 %
PB-2	1-5	Sand with some Clay	127.0 pcf.	10.0 %

Direct Shear Strength

Location	Depth (ft)	Soil Type	Internal Friction Angle	Cohesion (psf)
PB-1	1-5	Sand with trace Silt, yellowish brown	28 degrees	50

PLATE B-3

DIRECT SHEAR SUMMARY



PROJECT NO.	219118-101
CLIENT NAME	Primior
BORING NO.	PB-1
DEPTH	1-5 ft.
SATURATED	NO ___ YES x
UNDISTURBED	
REMOLDED TO	90 %

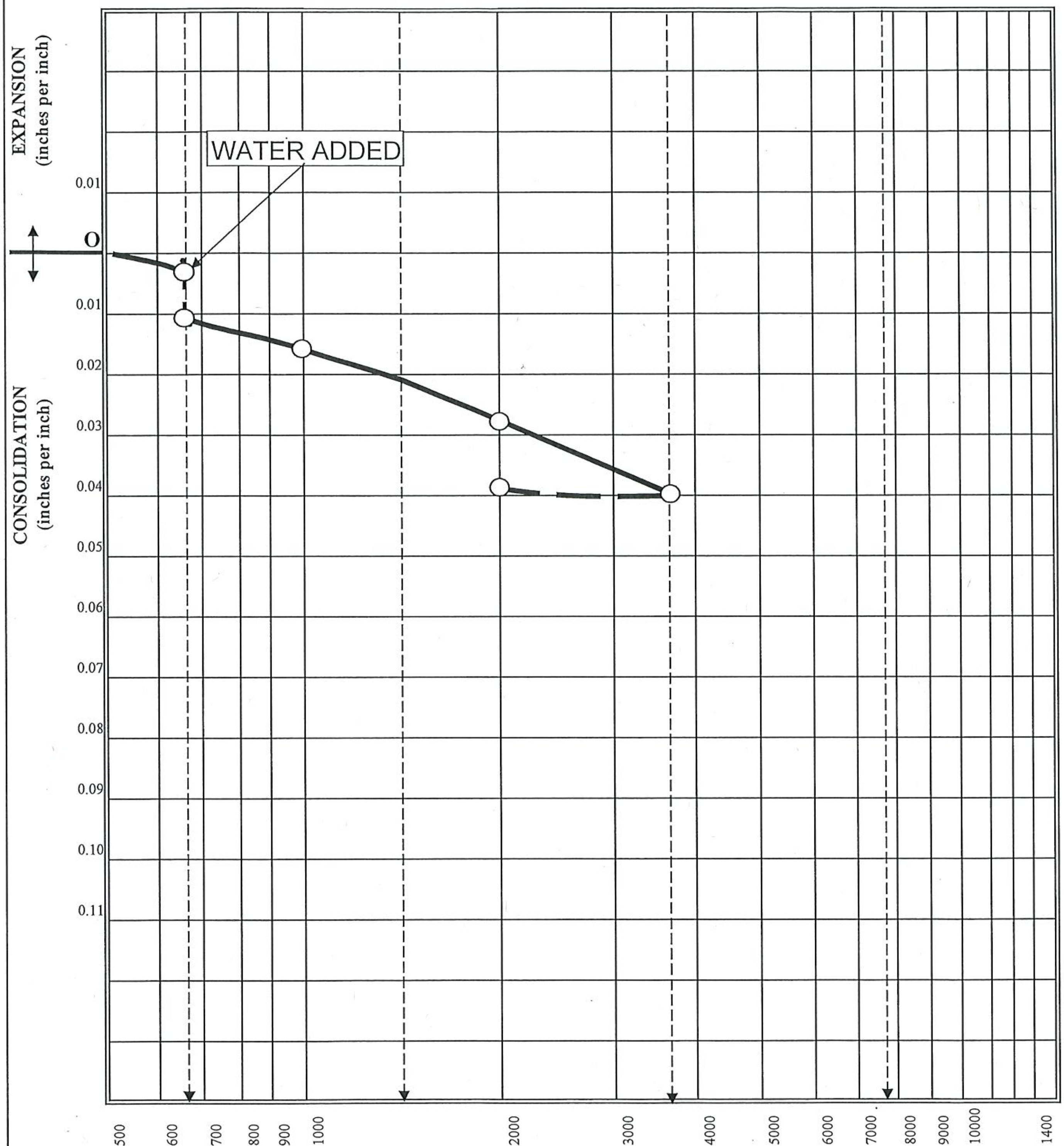
P. A. & ASSOCIATES, INC.

INTERNAL FRICTION ANGLE 28°

COHESION 50 **P.S.F.**

PROJECT NO.: **219118-101**
CLIENT: **Primior**
PROJECT NAME: **Westminster Ave.**
BORING NO.: **B-1** DEPTH: **5 ft.**

LEGEND
CONSOLIDATED
EFFECT OF ADDED WATER
REBOUND

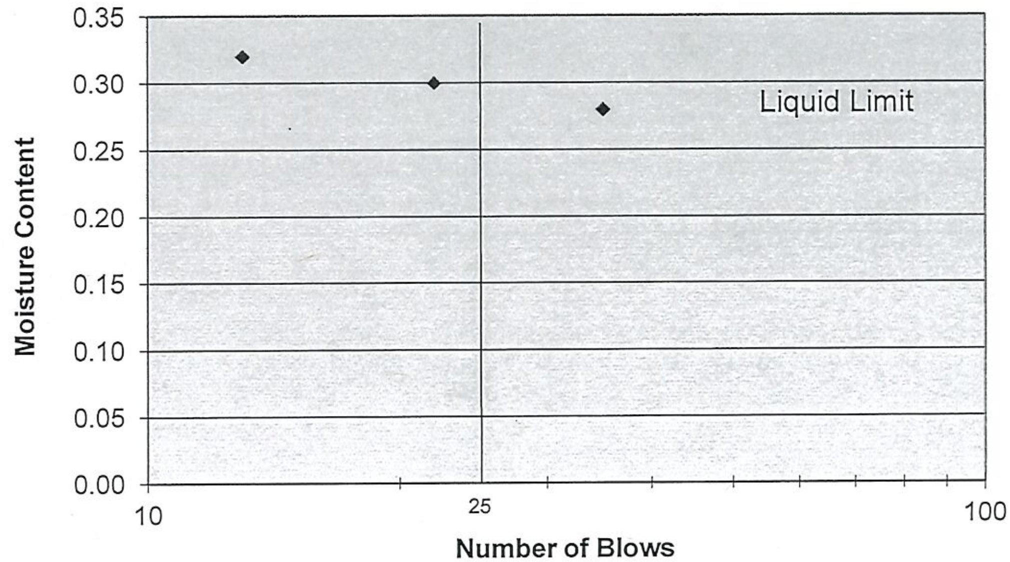


P. A. & ASSOCIATES, INC.

Plate B-4

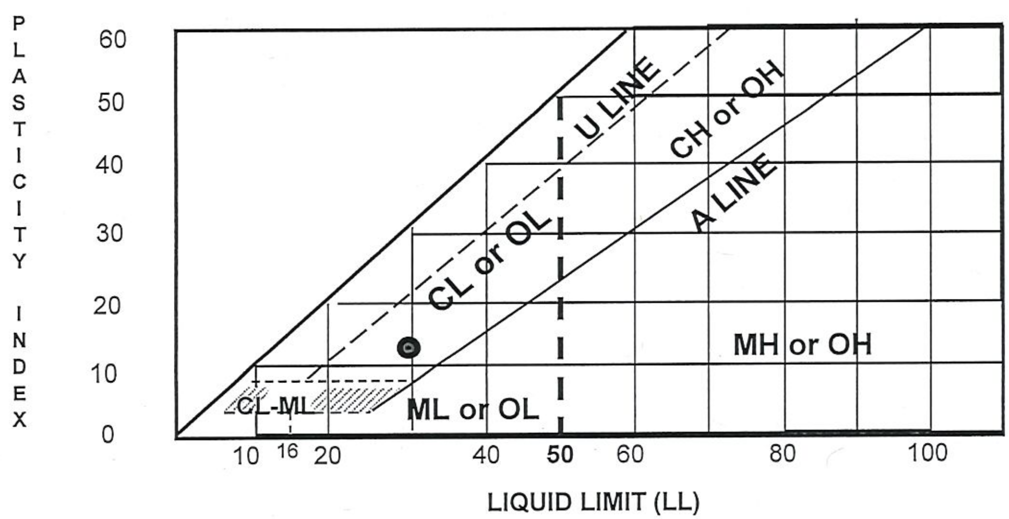
Plate B-5, Boring B1 @ 15

MOISTURE CONTENT - Liquid Limit				
Pan #	18	78	6	L.L.
Wt. Of Pan	29.5	33	32	at 25
Wt. Of Pan + Wet Soil	114	97	91.5	blows
Wt. Of Pan + Dry Soil	88.6	79.1	72.5	29.9
Wt. Of Water Loss	25.4	17.9	19	
Wt. Of Dry Soil	59.1	46.1	40.5	Sum
Moisture Content	0.30	0.28	0.32	0.90
Number of Blows	22	35	13	



MOISTURE CONTENT - Plastic Limit				
Wt. Of Pan	32	46	33	P.L.
Wt. Of Pan + Wet Soil	51	65	49.5	Ave w%
Wt. Of Pan + Dry Soil	48	61.5	47	16.5
Wt. Of Water Loss	3	3.5	2.5	
Wt. Of Dry Soil	16	15.5	16.5	Sum
Moisture Content	15.80	18.40	15.15	49.35

Plasticity Index: 13



Project #: 219118-101
Project Name: Westminster Ave.
Client: Primior
Date: 5/17/2019 Tech.: Z.A.
Boring / Test Pit: B-1
Sample / Depth: 15 ft.
USCS symbol: CL
Remarks/Description

P. A. & ASSOCIATES, INC.

APPENDIX C

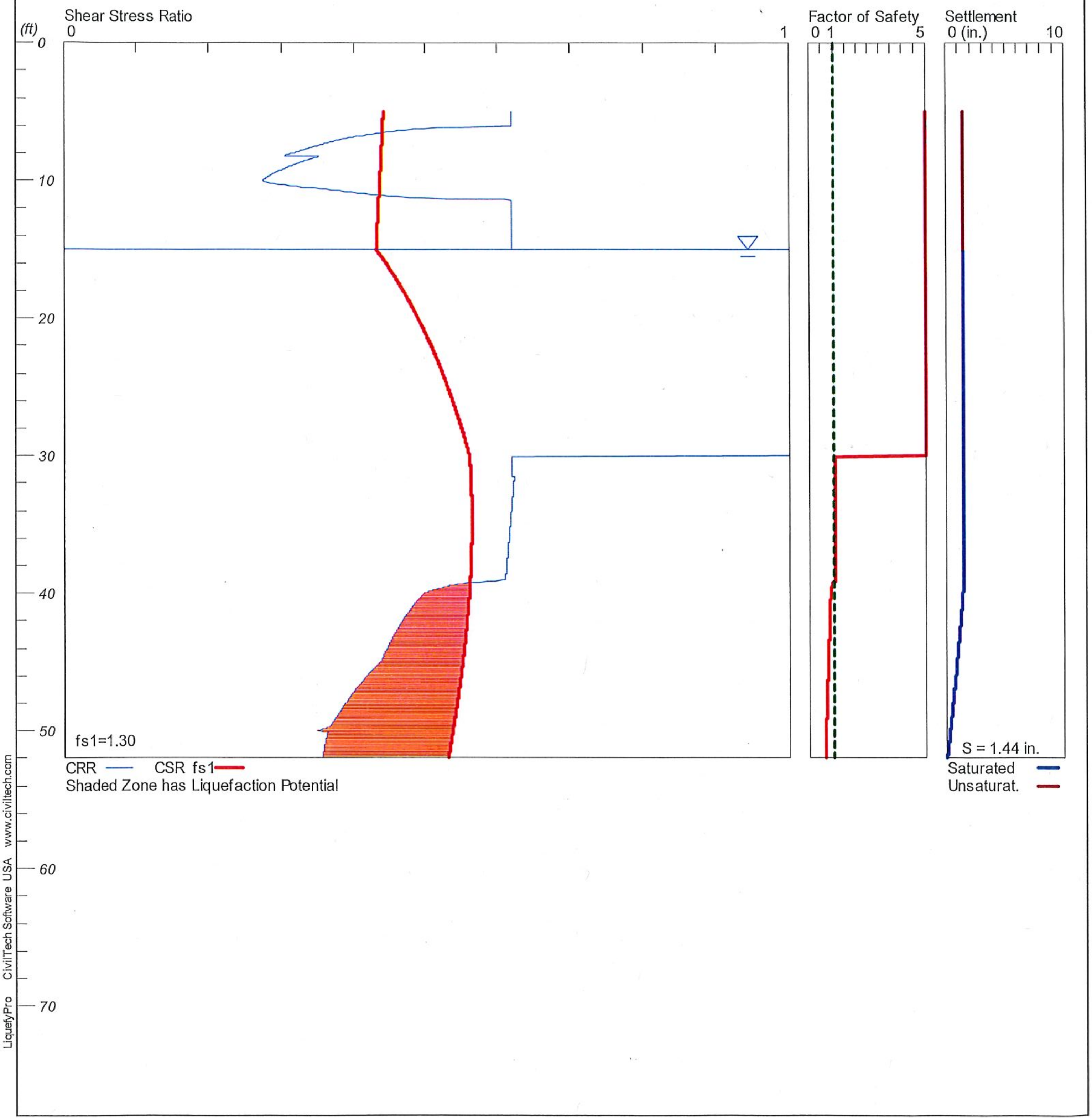
LIQUEFACTION ANALYSIS

LIQUEFACTION ANALYSIS

219118-101

Hole No.=B-1 Water Depth=15 ft

Magnitude=6.9
Acceleration=0.529g



CivilTech Corporation **2530-2534 Westminster Ave**

```
*****************************************************************************************************
                    LIQUEFACTION ANALYSIS CALCULATION DETAILS
                         Copyright by CivilTech Software
                              www.civiltech.com
*****************************************************************************************************
```

Font: Courier New, Regular, Size 8 is recommended for this report.
Licensed to , 5/21/2019 1:26:47 PM

Input File Name: C:\Liquefy5\219118-101 2530 Westminster.liq
Title: 219118-101
Subtitle: 2530-2534 Westminster Ave

Input Data:

```
Surface Elev.=
Hole No.=B-1
Depth of Hole=52.00 ft
Water Table during Earthquake= 15.00 ft
Water Table during In-Situ Testing= 22.50 ft
Max. Acceleration=0.53 g
Earthquake Magnitude=6.90
No-Liquefiable Soils:   CL, OL are Non-Liq. Soil
1. SPT or BPT Calculation.
2. Settlement Analysis Method: Tokimatsu/Seed
3. Fines Correction for Liquefaction: Idriss/Seed
4. Fine Correction for Settlement: During Liquefaction*
5. Settlement Calculation in: Liq. zone only
6. Hammer Energy Ratio,                                Ce = 1.32
7. Borehole Diameter,                                      Cb= 1.15
8. Sampling Method,                                       Cs= 1.2
9. User request factor of safety (apply to CSR) ,   User= 1.3
   Plot one CSR curve (fs1=User)
10. Average two input data between two Depths: Yes*
* Recommended Options
```

In-Situ Test Data:

Depth ft	SPT	Gamma pcf	Fines %
5.00	11.00	121.00	22.00
10.00	10.00	121.00	5.00
15.00	15.00	121.00	NoLiq
20.00	9.00	121.00	NoLiq
25.00	9.00	121.00	NoLiq
30.00	18.00	121.00	NoLiq
35.00	16.00	121.00	36.00
40.00	15.00	121.00	36.00
45.00	15.00	121.00	NoLiq
50.00	14.00	121.00	32.00
52.00	14.00	121.00	NoLiq

Output Results:

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

Peak Ground Acceleration (PGA), a_max = 0.53g

CSR Calculation:

Depth ft	gamma pcf	sigma atm	gamma' pcf	sigma' atm	rd	mZ g	a(z) g	CSR	x fs1	=CSRfs
5.00	121.00	0.286	121.00	0.286	0.99	0.000	0.529	0.34	1.30	0.44
10.00	121.00	0.572	121.00	0.572	0.98	0.000	0.529	0.34	1.30	0.44
15.00	121.00	0.858	58.60	0.858	0.97	0.000	0.529	0.33	1.30	0.43
20.00	121.00	1.144	58.60	0.996	0.95	0.000	0.529	0.38	1.30	0.49
25.00	121.00	1.429	58.60	1.135	0.94	0.000	0.529	0.41	1.30	0.53

30.00	121.00	1.715	58.60	1.273	0.93	0.000	0.529	0.43	1.30	0.56
35.00	121.00	2.001	58.60	1.412	0.89	0.000	0.529	0.43	1.30	0.56
40.00	121.00	2.287	58.60	1.550	0.85	0.000	0.529	0.43	1.30	0.56
45.00	121.00	2.573	58.60	1.688	0.81	0.000	0.529	0.42	1.30	0.55
50.00	121.00	2.859	58.60	1.827	0.77	0.000	0.529	0.41	1.30	0.54

CSR is based on water table at 15.00 during earthquake

CRR Calculation from SPT or BPT data:

Depth ft	SPT	Cebs	Cr	sigma' atm	Cn	(N1)60	Fines %	d(N1)60	(N1)60f	CRR7.5
5.00	11.00	1.82	0.75	0.286	1.70	25.55	22.00	6.31	31.85	0.50
10.00	10.00	1.82	0.85	0.572	1.32	20.48	5.00	0.03	20.50	0.22
15.00	15.00	1.82	0.95	0.858	1.08	28.03	NoLiq	10.61	38.63	0.50
20.00	9.00	1.82	0.95	1.144	0.94	14.56	NoLiq	7.91	22.48	0.25
25.00	9.00	1.82	0.95	1.357	0.86	13.37	NoLiq	7.67	21.04	0.23
30.00	18.00	1.82	1.00	1.496	0.82	26.81	NoLiq	10.36	37.17	0.50
35.00	16.00	1.82	1.00	1.634	0.78	22.80	36.00	9.56	32.36	0.50
40.00	15.00	1.82	1.00	1.773	0.75	20.52	36.00	9.10	29.63	0.41
45.00	15.00	1.82	1.00	1.911	0.72	19.77	NoLiq	8.95	28.72	0.36
50.00	14.00	1.82	1.00	2.050	0.70	17.81	32.01	7.88	25.69	0.29

CRR is based on water table at 22.50 during In-Situ Testing

Factor of Safety, - Earthquake Magnitude= 6.90:

Depth ft	sigC' atm	CRR7.5	x Ksig	=CRRv	x MSF	=CRRm	CSRfs	F.S.=CRRm/CSRfs
5.00	0.19	0.50	1.00	0.50	1.24	0.62	0.44	5.00
10.00	0.37	0.22	1.00	0.22	1.24	0.27	0.44	5.00
15.00	0.56	0.50	1.00	0.50	1.24	2.00	0.43	5.00 ^
20.00	0.74	0.25	1.00	0.25	1.24	2.00	0.49	5.00 ^
25.00	0.88	0.23	1.00	0.23	1.24	2.00	0.53	5.00 ^
30.00	0.97	0.50	1.00	0.50	1.24	2.00	0.56	5.00 ^
35.00	1.06	0.50	1.00	0.50	1.24	0.62	0.56	1.09
40.00	1.15	0.41	0.98	0.40	1.24	0.50	0.56	0.89 *
45.00	1.24	0.36	0.97	0.35	1.24	0.44	0.55	0.79 *
50.00	1.33	0.29	0.96	0.28	1.24	0.35	0.54	0.65 *

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

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

Depth ft	Ic	qc/N60	qc1 atm	(N1)60	Fines %	d(N1)60	(N1)60s
5.00	-	-	-	31.85	22.00	0.00	31.85
10.00	-	-	-	20.50	5.00	0.00	20.50
15.00	-	-	-	38.63	NoLiq	0.00	38.63
20.00	-	-	-	22.48	NoLiq	0.00	22.48
25.00	-	-	-	21.04	NoLiq	0.00	21.04
30.00	-	-	-	37.17	NoLiq	0.00	37.17
35.00	-	-	-	32.36	36.00	0.00	32.36
40.00	-	-	-	29.63	36.00	0.00	29.63
45.00	-	-	-	28.72	NoLiq	0.00	28.72
50.00	-	-	-	25.69	32.01	0.00	25.69

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

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

Depth ft	CSRsf	/ MSF*	=CSRm	F.S.	Fines %	(N1)60s	Dr %	ec %	dsz in.	dsp in.	S in.
51.95	0.53	1.00	0.53	0.67	99.26	26.10	81.85	1.121	6.7E-3	0.007	0.007
50.00	0.54	1.00	0.54	0.65	32.01	25.69	81.05	1.146	6.9E-3	0.261	0.267
45.00	0.55	1.00	0.55	0.79	NoLiq	28.72	87.23	0.913	0.0E0	0.604	0.871
40.00	0.56	1.00	0.56	0.89	36.00	29.63	89.22	0.797	4.8E-3	0.508	1.379
35.00	0.56	1.00	0.56	1.09	36.00	32.36	95.65	0.422	0.0E0	0.065	1.444
30.00	0.56	1.00	0.56	5.00	NoLiq	37.17	100.00	0.000	0.0E0	0.000	1.444
25.00	0.53	1.00	0.53	5.00	NoLiq	21.04	72.42	1.437	0.0E0	0.000	1.444
20.00	0.49	1.00	0.49	5.00	NoLiq	22.48	75.02	1.344	0.0E0	0.000	1.444
15.00	0.43	1.00	0.43	5.00	NoLiq	38.63	100.00	0.000	0.0E0	0.000	1.444

Settlement of Saturated Sands=1.444 in.
qc1 and (N1)60 is after fines correction in liquefaction analysis
dsz is per each segment, dz=0.05 ft
dsp is per each print interval, dp=5.00 ft
S is cumulated settlement at this depth

Settlement of Unsaturated Sands:
Settlement of Unsaturated Sands=0 due to Option 5, Calculation settlement only in liquefied zone.

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.
14.95	0.85	0.56	38.58	0.43	1125.02	3.3E-4	0.1193	0.0416	0.90	0.0374	0.00E0	0.000	0.000
10.00	0.57	0.37	20.50	0.44	745.46	3.3E-4	0.1293	0.1233	0.90	0.1109	0.00E0	0.000	0.000
5.00	0.29	0.19	31.85	0.44	610.41	2.1E-4	0.0409	0.0212	0.90	0.0190	0.00E0	0.000	0.000

Settlement of Unsaturated Sands

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

Total Settlement of Saturated and Unsaturated Sands=1.444 in.
Differential Settlement=0.722 to 0.953 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.

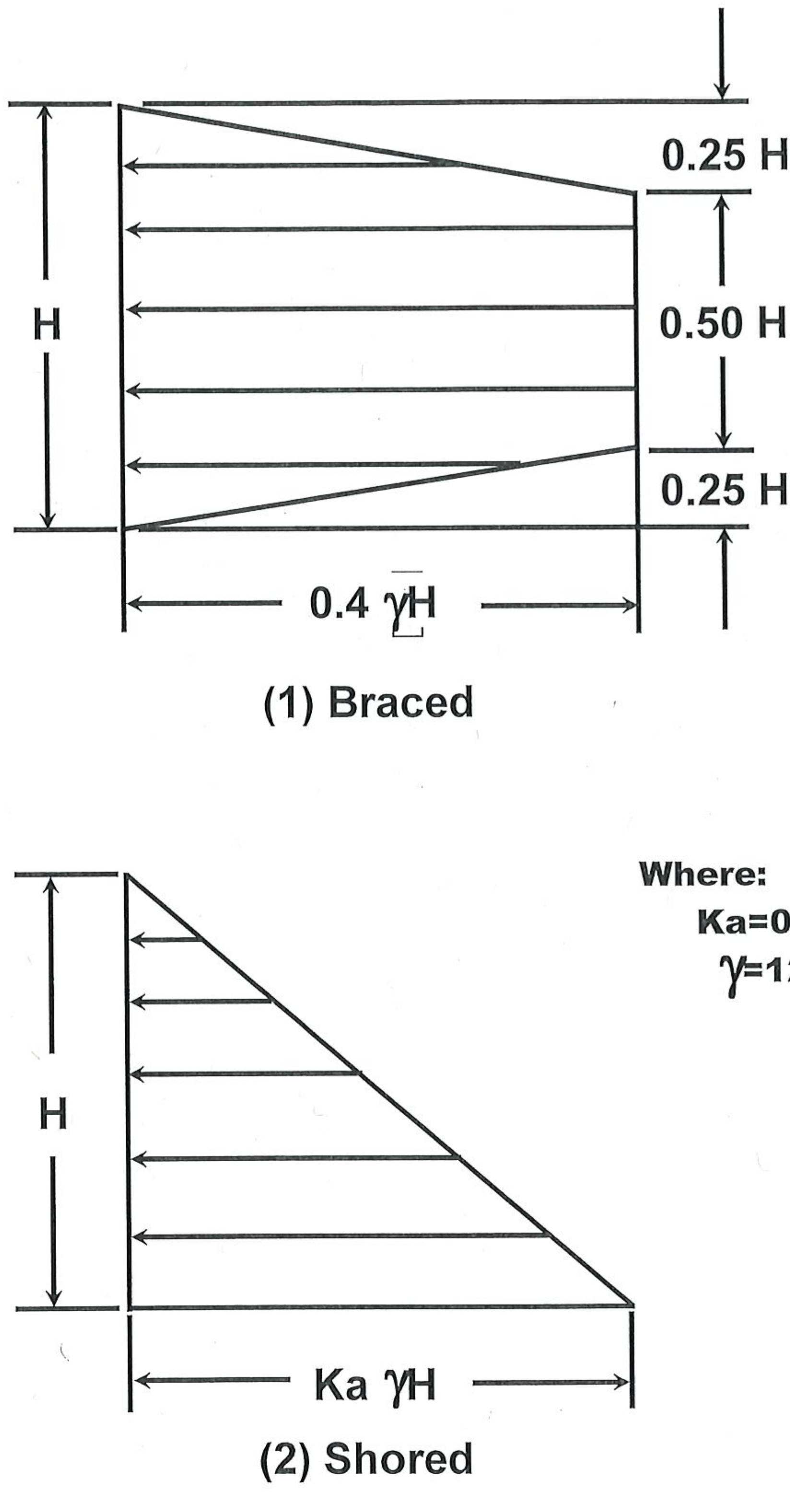
2. RECENT ADVANCES IN SOIL LIQUEFACTION ENGINEERING AND SEISMIC SITE RESPONSE EVALUATION, Paper No. SPL-2, PROCEEDINGS: Fourth International Conference on Recent Advances in Geotechnical Earthquake Engineering and Soil Dynamics, San Diego, CA, March 2001.

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

TEMPORARY SHORING
& BRACING DESIGN



Where:

Ka=0.26

γ=125 pcf

P. A. & ASSOCIATES, INC.

TITLE:	TEMPORARY SHORING & BRACING DESIGN				FIGURE NO.: D-1
PROJECT:	2530-2534 Westminster Ave., Santa Ana, CA				
PROJECT NO:	25138-101	DATE:	5/22/19	BY: JR	

APPENDIX E

PERCOLATION/INFILTRATION RATE STUDY

PROPOSED INFILTRATION DRY WELL/TRENCH

The project site is located at the existing relatively level coastal alluvial flood plain roughly 0.5 miles west of the Santa Ana River channel, in the Santa Ana. The site comprises 1.44 AC.

FIELD INVESTIGATION

In order to obtain pertinent geotechnical and percolation rate data concerning the feasibility of subsurface soils for proposed infiltration trenches, three (3) percolation test borings were excavated in the proposed dry well/infiltration trench facility areas. These were tested to determine representative percolation rates and to verify an acceptable depth to groundwater below the proposed infiltration inverts. The approximate locations of the borings and the boring logs are attached in Appendix A.

EARTH MATERIALS

The subsurface soils of the study area encountered in the borings generally comprised young alluvial fan soils that consisted generally of light yellowish brown and gray medium sand with trace silt and sand with some pebbles that were moist and medium dense in-place.

PERCOLATION TEST

Percolation Rate Test Preparation

For the percolation rate testing, three 8-inch diameter test holes were excavated, and rate of percolation efficiency was studied in accordance with pertinent the Technical Guidance Document Appendices (Reference 5). The test holes were tested to depth of 5 feet below existing ground surface at the bottom elevation of the proposed dry well/trench (infiltration surface), and two inches of gravel were placed in the bottom of the test hole. A 4-inch diameter perforated pipe backfilled with fine gravel was placed in the hole to preserve the drilled depth.

Groundwater

Groundwater was encountered at boring depth of 22.5 ft. b.g.s. (below ground surface) in our deep boring. However, historically highest groundwater elevation contours (Open-File Report 97-08) indicate that the groundwater could rise to about 15 ft. b.g.s. In consideration of the more recent river channel paving engineered to mitigate flooding and infiltration from Santa Ana River (0.1 miles to the east), we anticipate highest groundwater elevations to remain in approximately 20 feet below the surface.

Percolation Testing

The percolation test holes were filled with clean water, and from a fixed reference point, the drops in water level were measured at two consecutive intervals of 25 minutes during which more than six inches of water seeped away. The tests were run for an additional hour with measurements taken at 10 minute intervals. The critical rates are the final reading; the field percolation test data sheets summarizing the readings are attached in Appendix F.

Percolation Rate

The percolation rates were calculated in accordance with percolation test methods (tests less than 10 feet in depth) provided in the Riverside Technical Guidance Document Appendices (Reference 5). The final percolation test reading and infiltration rates are shown in Table I, below:

TABLE I

FINAL PERCOLATION TEST RESULTS

Test Hole No.	Depth of Test (ft)*	Average Head (inches)	Tested Infiltration Rate (inches/ hour)
PB- 1 (Infiltration Facility)	5	24.75	4.71
PB- 2 (Infiltration Facility)	5	25.25	4.18
PB- 3 (Infiltration Facility)	5	24.75	4.71

*Depth of holes at time of test

Infiltration rate was calculated using the conversion equation: $I_t = \lambda H(60r)/\ \lambda t(r+2H_{avg})$

Where: λt = time interval
r = test hole radius
λH = change in height over time interval
H_{avg} = average head height over time interval
I_t = tested infiltration rate

Infiltration Rate Calculations:

Test Hole PB-1

$I_t = \lambda H(60r)/\ \lambda t(r+2H_{avg})$ = (10.5 in) (60min/hr)(4in)/ (10min)((4)+(2(24.75in)) = 4.71 in/hr

Test Hole PB-2

$I_t = \lambda H(60r)/\ \lambda t(r+2H_{avg})$ = (9.5 in) (60min/hr)(4in)/ (10min)((4in)+(2(25.25in)) = 4.18 in/hr

Test Hole PB-3

$I_t = \lambda H(60r)/\ \lambda t(r+2H_{avg})$ = (10.5 in) (60min/hr)(4in)/ (10min)((4in)+(2(24.75in)) = 4.71 in/hr

INFILTRATION RATE WITH FACTOR OF SAFETY

Our percolation testing results indicated an average infiltration rate of 4.53 inches/hour. However, we recommend that a long term safety factor of 2 be applied giving a design infiltration rate for the proposed stormwater infiltration trench of 2.27 inches/hour.

INVESTIGATION LIMITATIONS

This report is based on the project as described and the geotechnical data obtained for the field tests performed at the locations indicated on the plan. The materials encountered on the project site are believed representative of the total study area. Our firm should be notified of any pertinent change in the project plans, or if subsurface conditions are encountered which differ from those described in this report, as this may require re-evaluation of our recommendations. This report has not been prepared for use by parties or projects other than those named or described herein. It may not contain sufficient information for other parties or other purposes. Since our investigation is based on the site materials observed and engineering analysis, the conclusions and recommendations presented herein are professional opinions. These opinions have been derived in accordance with current standards of practices, and no warranty is expressed or implied.

APPENDIX F

PERCOLATION TEST FIELD LOGS

Perc Test Data Sheet

Project: 2530-2534 Westminster Ave., Santa Ana			Project File No.: 219118-101
Test Hole No. PB-1	Tested by: Z.A.	USCS: SP	Date: 5/14/2019
Depth of Hole (8" Dia.) as Drilled: 5 ft.		Before Test: 5 ft.	After Test: 5 ft.

Trial No.	Start Time	Stop Time	Time Interval (min)	Initial Depth to Water (in.)	Final Depth to Water (in.)	△ In Water Level (in.)	Greater than or Equal to 6"? (y/n)
1	3:05 PM	3:30 PM	25	0.00	30.25	30.25	y
2	3:35 PM	4:00 PM	25	0.00	27.00	27.00	y

If two consequtive measurements show that six inches of water seep away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every ten minutes. Oherwise, presoak fill overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Time Interval (min)	Initial Depth to Water (in.)	Final Depth to Water (in.)	△ In Water Level (in.)	Percolation Rate (min./in.)
1	4:09 PM	4:19 PM	10	0.00	16.00	16.00	0.63
2	4:21 PM	4:31 PM	10	0.00	13.00	13.00	0.77
3	4:33 PM	4:43 PM	10	0.00	13.00	11.50	0.87
4	4:57 PM	5:07 PM	10	0.00	12.00	10.25	0.98
5	5:09 PM	5;19 PM	10	0.00	10.50	10.50	0.95
6	5:22 PM	5:32 PM	10	0.00	10.50	10.50	0.95
7							
8							
9							
10							
11							
12							
13							
14							
15							

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Plate No.1

Perc Test Data Sheet

Project: 2530-2534 Westminster Ave., Santa Ana			Project File No.: 219118-101	
Test Hole No. PB-2	Tested by: S.A.	USCS: SP	Date: 5/14/2019	
Depth of Hole (8" Dia.) as Drilled: 5 ft.		Before Test: 5 ft.		After Test: 5 ft.

Trial No.	Start Time	Stop Time	Time Interval (min)	Initial Depth to Water (in.)	Final Depth to Water (in.)	△ In Water Level (in.)	Greater than or Equal to 6"? (y/n)
1	3:05 PM	3:30 PM	25	120.00	135.25	15.25	y
2	3:30 PM	3:55 PM	25	120.00	134.75	14.75	y

If two consequtive measurements show that six inches of water seep away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every ten minutes. Oherwise, presoak fill overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Time Interval (min)	Initial Depth to Water (in.)	Final Depth to Water (in.)	△ In Water Level (in.)	Percolation Rate (min./in.)
1	4:00 PM	4:10 PM	10	0.00	22.00	22.00	0.45
2	4:13 PM	4:23 PM	10	0.00	13.00	13.00	0.77
3	4:25 PM	4:35 PM	10	0.00	11.00	11.00	0.91
4	4:50 PM	5:00 PM	10	0.00	9.00	9.00	1.11
5	5:01 PM	5:11 PM	10	0.00	9.00	9.00	1.11
6	5:13 PM	5:23 PM	10	0.00	9.50	9.50	1.05
7							
8							
9							
10							
11							
12							
13							
14							
15							

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Plate No.2

Perc Test Data Sheet

Project: 2530-2534 Westminster Ave., Santa Ana			Project File No.: 219118-101
Test Hole No. PB-3	Tested by: J.R	USCS: SW	Date: 5/14/2019
Depth of Hole (8" Dia.) as Drilled: 5 ft.		Before Test: 5 ft.	After Test: 5 ft.

Trial No.	Start Time	Stop Time	Time Interval (min)	Initial Depth to Water (in.)	Final Depth to Water (in.)	△ In Water Level (in.)	Greater than or Equal to 6"? (y/n)
1	3:10 PM	3:35 PM	25	0.00	26.75	26.75	y
2	2:35 PM	3:00 PM	25	0.00	20.50	20.50	y

If two consequtive measurements show that six inches of water seep away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every ten minutes. Oherwise, presoak fill overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

Trial No.	Start Time	Stop Time	Time Interval (min)	Initial Depth to Water (in.)	Final Depth to Water (in.)	△ In Water Level (in.)	Percolation Rate (min./in.)
1	3:05 PM	3:15 PM	10	0.00	16.00	16.00	0.63
2	3:16 PM	3:26 PM	10	0.00	13.00	13.00	0.77
3	3:28 PM	3:38 PM	10	0.00	13.00	13.00	0.77
4	3:53 PM	4:03 PM	10	0.00	12.00	12.00	0.83
5	4:04 PM	4:14 PM	10	0.00	11.50	11.50	0.87
6	4:15 PM	4:25 PM	10	0.00	10.50	10.50	0.95
7							
8							
9							
10							
11							
12							
13							
14							
15							

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Plate No.3

APPENDIX G

REFERENCES

REFERENCES

1. Preliminary Geotechnical Investigation, Proposed Industrial/Commercial Building, Ahern Rental, 2530 Westminster Ave., Santa Ana, County of Orange, CA, by P.A. & Associates, Inc, Project Number 25138-101, dated August 9, 2005.

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

3. Barrows, Allan G., 1974, A Review of the Geology and Earthquake History of the Newport-Inglewood Structural Zone, Southern California, California Division of Mines and Geology, Special Report 114

4. Preliminary Geologic Map of the Santa Ana 30'x60' Quadrangle, Southern California, compiled by D. M. Morton, USGS Open-File Report 99-172, 2004.

5. Technical Guidance Document Appendices, California, dated March 22, 2011.