Appendix F Geotechnical Study

Appendices

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GEOTECHNICAL STUDY REPORT

Wedgeworth Elementary School Development Project
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745

Converse Project No. 18-31-330-02

Prepared For:

Hacienda La Puente Unified School District
Facilities Services Division
15959 East Gale Avenue
City of Industry, California 91745

Prepared By:

Converse Consultants
717 South Myrtle Avenue
Monrovia, California 91016

May 03, 2019

May 03, 2019

Mr. Robert Wilcox
Facilities Services Division
Hacienda La Puente Unified School District
15959 East Gale Avenue
City of Industry, California 91745

Subject: **GEOTECHNICAL STUDY REPORT**
Proposed Wedgeworth Elementary School Development Project
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745
Hacienda La Puente Unified School District
Converse Project No. 18-31-330-02

Dear Mr. Wilcox:

Enclosed is the Geotechnical Study Report prepared by Converse Consultants (Converse) for the proposed Wedgeworth Elementary School Development Project within Wedgeworth Elementary School located in Hacienda Heights, Los Angeles County, California.

The purpose of the study was to investigate the geotechnical site conditions and provide recommendations for the proposed development project at Wedgeworth Elementary School. The planned development would be constructed within the existing Wedgeworth Elementary School, a Hacienda La Puente Unified School District (HLPUSD) property.

Based on our field exploration, laboratory testing, geologic evaluation, and geotechnical analysis, the site is suitable from a geotechnical standpoint for the proposed project located within Wedgeworth Elementary School, provided our conclusions and recommendations are implemented during design and construction.

We appreciate the opportunity to be of continued service to Hacienda La Puente Unified School District. If you should have any questions, please do not hesitate to contact us at (626) 930-1200.

Sincerely,

CONVERSE CONSULTANTS

Sivathasan

Siva K. Sivathasan, PhD, PE, GE, DGE, QSD, F. ASCE
Senior Vice President/Principal Engineer

Dist: 4/Addressee

PROFESSIONAL CERTIFICATION

This report for the Proposed Development Project located within Wedgeworth Elementary School in Hacienda Heights, Los Angeles County, California, has been prepared by the staff of Converse under the professional supervision of the individuals whose seals and signatures appear hereon.

The findings, recommendations, specifications or professional opinions contained in this report were prepared in accordance with generally accepted professional engineering and engineering geologic principles and practice in this area of Southern California. There is no warranty, either expressed or implied.

In the event that changes to the property occur, or additional, relevant information about the property is brought to our attention, the conclusions contained in this report may not be valid unless these changes and additional relevant information are reviewed, and the recommendations of this report are modified or verified in writing.

Parameswaran Ariram, EIT
Senior Staff Engineer

PROFESSIONAL GEOLOGIST
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Siva K. Sivathasan, PhD, PE, GE, DGE, QSD, F. ASCE
Senior Vice President/Principal Engineer

EXECUTIVE SUMMARY

The following is a summary of our geotechnical investigation, conclusions and recommendations as presented in the body of this report. Please refer to the appropriate sections of the report for complete conclusions and recommendations. In the event of a conflict between this summary and the report, or an omission in the summary, the report shall prevail.

- The proposed project entails five (5) new buildings including four (4) two story buildings and one (1) one story building, soccer fields, play fields, hardcourts, fire lane and parking lots and other adjacent improvements within the existing Wedgeworth Elementary School, a HLPUSD property.

- Nineteen (19) exploratory borings (BH-1 through BH-18 and PT-5) were drilled within the project site on April 2nd and April 3rd using a truck mounted drill rig with an 8-inch diameter hollow stem auger to depths ranging from 6.5 feet to 51.5 feet below the existing ground surface (bgs).

- There are no known active faults projecting toward or extending across the proposed site. The project site is not located within a currently designated State of California Earthquake Fault Zone (formerly Alquist-Priolo Special Studies Zones) for surface fault rupture.

- Groundwater was encountered during our subsurface exploration at a depth of 39 feet bgs in Boring BH-2 and at a depth of 37 feet bgs in Boring 15. Review of the Seismic Hazard Zone Report for the La Habra 7.5-Minute Quadrangle (CDMG 1997) indicated the historically highest groundwater levels at depths of approximately 25 feet below ground surface. More recent groundwater level monitoring in local groundwater wells has shown depths to groundwater varies between approximately 27 feet and 35 feet below ground surface. Groundwater is not anticipated during construction, however, may need to be considered in design if deeper foundations are used based on the historically highest groundwater levels and groundwater depth encountered in Boring BH-15.

- The site is located within a mapped Seismic Hazard Zone for liquefaction as shown on Drawing No. 7, *Seismic Hazard Zones Map*. Based on the results of our subsurface exploration and laboratory tests the site soils are comprised of silt, silty clay and clay, and the risk of liquefaction is considered low. The seismically-induced settlement is negligible.

- The observed fill soils consist primarily of silty clay and clays. The depth of the fill observed was up to depths of approximately five (5) feet to seven (7) below existing ground surfaces. The alluvial sediments consisted predominately of silty clays, clays,

- clayey sands, sandy clays and silty sands to the maximum drilled depth of approximately 51.5 feet below ground surface.

- The surficial site soils at the site exhibit a “Low” expansive potential. Mitigation for expansive soil is not considered necessary.

- In general, the pH value and concentrations of water soluble sulfates saturated resistivity of the site soils are in the non-corrosive range. The saturated resistivity and chloride content of the site soils are in corrosive range to ferrous metals.

- The earth materials at the site should be excavatable with conventional heavy-duty earth moving equipment. Earthwork should be performed with suitable equipment for gravelly materials.

- Shallow spread and continuous footings are considered suitable for building structures provided the recommendations in this report are incorporated into the project plans and specifications and are followed during site construction.

- For non-building structures (e.g. signs, fence walls, short retaining walls, etc.), conventional footings can be used.

- Percolation testing was performed utilizing exploratory boring PT-5 to evaluate soil infiltration rates of the native soils encountered between depths of 10 to 20 feet below the ground surface. The percolation results are provided in Appendix C: Percolation Testing

Results of our investigation indicate that the site is suitable from a geotechnical standpoint for the proposed development, provided that the recommendations contained in this report are incorporated into the design and construction of the project.

TABLE OF CONTENTS

TABLES

DRAWINGS

APPENDICES

1.0 INTRODUCTION

This report contains the findings and recommendations of our geotechnical study performed at the site of the Proposed Wedgeworth Elementary School Development Project located at the existing Wedgeworth Elementary School site in Hacienda Heights, Los Angeles County, California, as shown on Drawing No. 1, *Site Location Map*.

The purpose of the study was to evaluate the subsurface soil conditions and provide geotechnical recommendations and design recommendations for the design and construction of the proposed project, consistent with the current edition of California Building Code, Title 24, Chapter 16A; Earthquake Design, Chapter 18A, Foundation and Retaining Wall; Appendix Chapter 33, Excavation and Grading; and CGS Note 48- Checklist for the review of Geologic/Seismic Reports for California Public Schools, Hospitals and Essential Services Buildings.

This report is written for the project described herein and is intended for use solely by the Hacienda La Puente Unified School District, Wedgeworth Elementary School and its design team. It should not be used as a bidding document but may be made available to potential contractors for information on factual data only. For bidding purposes, the contractors should be responsible for making their own interpretation of the data contained in this report.

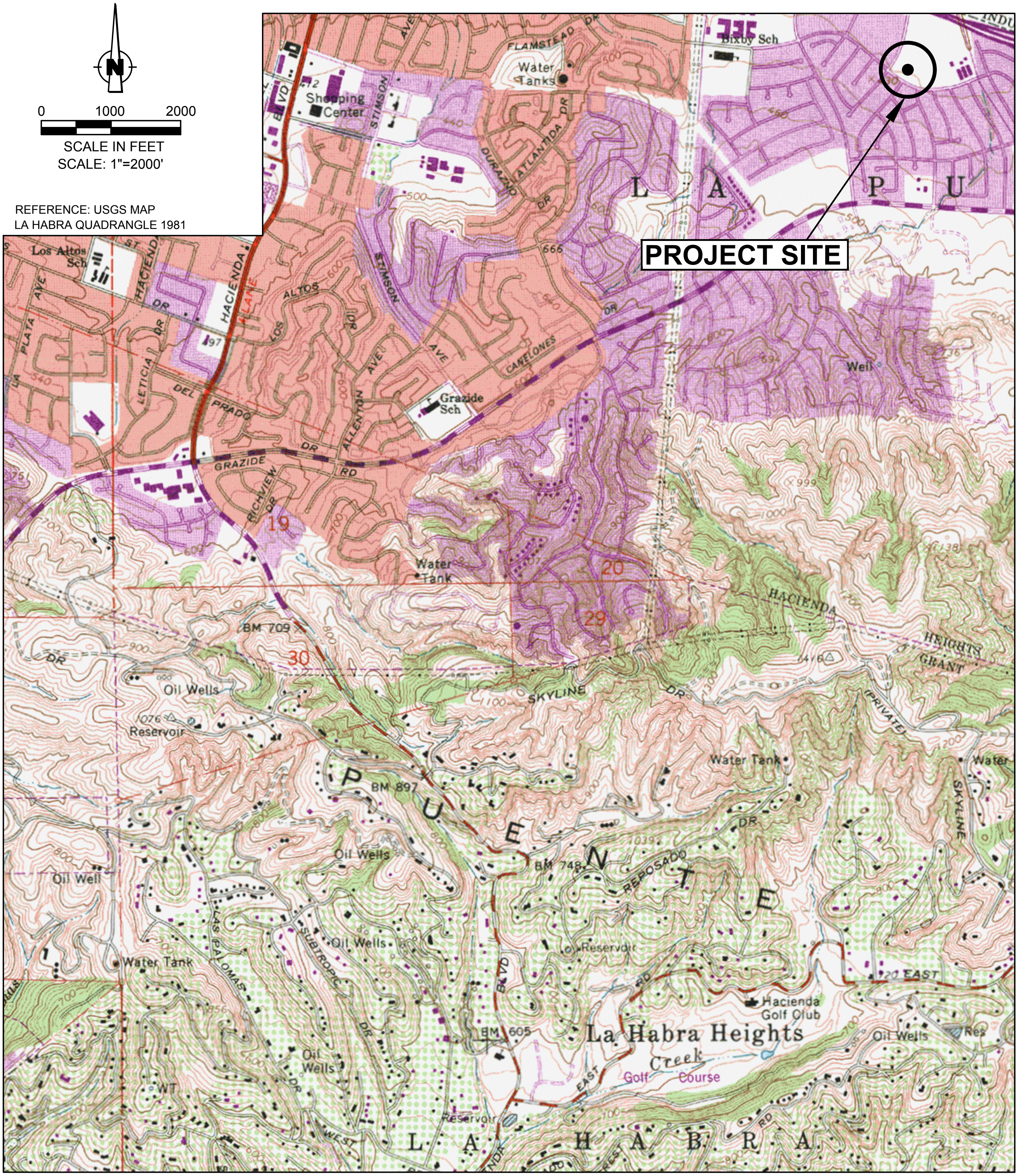
2.0 SITE AND PROJECT DESCRIPTION

The proposed project is located at the existing Wedgeworth Elementary School site located at 16949 Wedgeworth Drive, Hacienda Heights, California. The subject school site, a HLPUSD property, was previously graded relatively flat with surface elevations ranging from approximately 398 to 410 feet relative to mean-sea-level (MSL). The school site is bounded by Ridge Park Drive to the east, Eagle Park Road to the west, Wedgeworth Drive to the south, and by Pomona Freeway Route 60 to the north. The site coordinates are: 33.99644 degrees North Latitude, -117.93666 degrees West Longitude.

We understand that the proposed project entails five (5) new buildings including four (4) two-story buildings and one (1) one story building, soccer fields, play fields, hardcourts, fire lane and parking lots and other adjacent improvements within the Wedgeworth Elementary School site, a HLPUSD property as shown on Drawing No. 2, *Site Plan and Approximate Location of Borings.*

3.0 SCOPE OF WORK

The scope of our work included a site reconnaissance, subsurface exploration with soil sampling, laboratory testing, engineering analyses, and preparation of this report.



SITE LOCATION MAP

WEDGEWORTH ELEMENTARY SCHOOL
16949 WEDGEWORTH DRIVE, HACIENDA HEIGHTS, CA
FOR: HACIENDA LA PUENTE UNIFIED SCHOOL DISTRICT

Project No.
18-31-330-02

Converse Consultants

Drawing No.
1

3.1 Site Reconnaissance

During the site reconnaissance on March 27, 2019, the surface conditions were noted, and the locations of the borings were determined so that drill rig access to all the locations was available. The borings were located using existing boundary features as a guide and should be considered accurate only to the degree implied by the method used. Underground Service Alert (USA) of Southern California was notified of our proposed drilling locations at least 48 hours prior to initiation of the subsurface field work. Drilling Permit was obtained from LA County, Department of Health, Drinking Water Program.

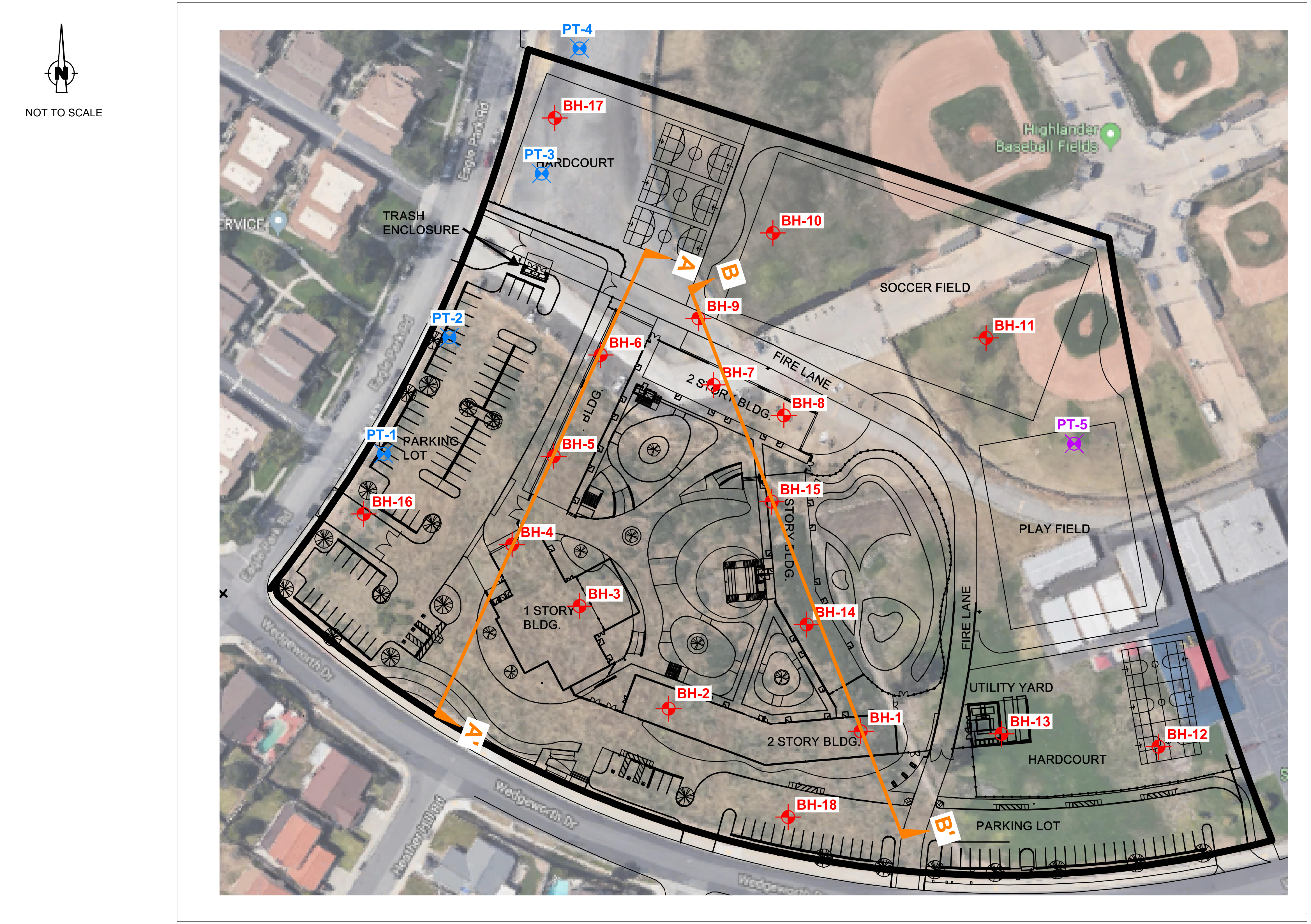
3.2 Subsurface Exploration and Percolation Testing

Nineteen (19) exploratory borings (BH-1 through BH-18 and PT-5) were drilled within the project site on April 2nd and 3rd, 2019 using a truck mounted drill rig with an 8-inch diameter hollow stem auger to depths ranging from 6.5 feet to 51.5 feet below the existing ground surface (bgs). Each boring was visually logged by a Converse engineer and sampled at regular intervals and at changes in subsurface soils. Detailed descriptions of the field exploration and sampling program are presented in Appendix A, *Field Exploration*.

California Modified Sampler ring samples, Standard Penetration Test samples, and bulk soil samples were obtained for laboratory testing. Standard Penetration Tests (SPTs) were performed in selected borings at selected intervals using a standard split-barrel sampler (1.4 inches inside diameter and 2.0 inches outside diameter). The boreholes were backfilled with cement grout following the completion of drilling to match existing conditions. The bore holes which were less than 10 feet depth, were backfilled and compacted with soil cuttings by reverse spinning of the augers and tamped following the completion of drilling to match existing conditions.

Percolation test was performed at PT-5 in an existing baseball field along the east side of the project development to the depth of 10 to 20 feet below ground level by using Boring Percolation Testing Procedure.

The approximate locations of the exploratory borings are shown in Drawing No. 2, *Site Plan and Approximate Location of Borings*. Detailed descriptions of the field exploration and sampling program are presented in Appendix A, *Field Exploration*.



LEGEND

APPROXIMATE LOCATION OF BORING (APRIL 2019)

APPROXIMATE LOCATION OF PERCOLATION TESTS (FEBRUARY 2019)

APPROXIMATE LOCATION OF PERCOLATION TEST (APRIL 2019)

A A' CROSS SECTION

SITE PLAN AND APPROXIMATE LOCATION OF BORINGS

Converse Consultants

WEDGEWORTH ELEMENTARY SCHOOL PROJECT
16949 WEDGEWORTH DRIVE, HACIENDA HEIGHTS, CA
FOR: HACIENDA LA PUENTE UNIFIED SCHOOL DISTRICT

Project No.	Figure No.
18-31-330-02	2

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3.3 Laboratory Testing

Representative samples of the site soils were tested in the laboratory to aid in classification and to evaluate relevant engineering properties. The tests performed included:

- In Situ Moisture Contents and Dry Densities (ASTM Standard D2216)
- Grain-Size Analysis (ASTM D422)
- Passing Sieve No. 200 (ASTM D1140)
- Direct Shear (ASTM Standard D3080)
- Maximum dry density and optimum-moisture content relationship (ASTM Standard D1557)
- Expansion Index (ASTM Standard D4829)
- R- Value (ASTM Standard D2844)
- Consolidation (ASTM D2435)
- Soil Corrosivity Tests (Caltrans 643, 422, 417, and 532)

For a description of the laboratory test methods and test results, see Appendix B, *Laboratory Testing Program*. For *in-situ* moisture and density data, see the Logs of Borings in Appendix A, *Field Exploration*.

3.4 Engineering Analyses and Report

Data obtained from the exploratory fieldwork and laboratory-testing program were analyzed and evaluated. This report was prepared to provide the findings, conclusions and recommendations developed during our investigation and evaluation.

4.0 GEOLOGIC CONDITIONS

4.1 Regional Geology

The site is located within the south eastern portion of the San Gabriel Valley Basin, a broad sediment-filled basin located at the convergence of the Transverse Ranges and Peninsular Ranges geomorphic provinces of California. Local stream channels and drainages have deposited stream and flood sediments across the northern flank of the Puente Hills during Holocene time (0 – 11,000 years) to form a gently sloping alluvial fan that descends into the lower valley basin. Soils underlying the project site consist of silty clays, clays, silty sands, and mixtures of silt sediments deposited over time by San Jose Creek and local streams and drainage tributaries which once drained across the valley basin to the Pacific Ocean. Most of these natural river and stream channels are now controlled by debris basins, flood control channels and flood control dams that collect surface runoff and convey storm water to the ocean. Drawing No. 3, *Regional Geologic Map*, has been prepared to show the project site with respect to regional geology of the Whittier and La Habra Quadrangles.

4.2 Subsurface Profile of Subject Site

Based on our soil borings drilled at the site, the subsurface conditions generally consist of existing fill soils placed during previous site grading operations overlying natural alluvial sediments as encountered in the borings drilled to the maximum depth explored of 51.5 feet below the ground surface (bgs). The observed fill soils consist primarily of silty clay and clays. The depth of the fill observed was up to depths of approximately five (5) feet to seven (7) below existing ground surfaces. The alluvial sediments consisted predominately of silty clays, clays, clayey sands, sandy clays and silty sands to the maximum drilled depth of approximately 51.5 feet below ground surface.

Drawing No. 4, *Geologic Cross Section A-A' and B-B'*, has been drawn across the project site to illustrate the subsurface conditions. For additional information on the subsurface conditions, see the Logs of Boring Data in Appendix A, *Field Exploration*.

4.3 Groundwater

Groundwater was encountered during our subsurface exploration at a depths of 39 feet below ground surface in Boring BH-2 and at a depth of 37 feet in Boring BH-15. Review of the Seismic Hazard Zone Report for the La Habra 7.5-Minute Quadrangle, Plate No. 1.2 (CDMG, 1997), indicates the historically highest groundwater levels in the vicinity of the site are shown to be approximately 25 feet below ground surface. Groundwater is not anticipated during construction however may need to be considered in design.

In general, groundwater levels fluctuate with the seasons and local zones of perched groundwater may be present within the near-surface deposits due to local conditions or during rainy seasons. Groundwater conditions below any given site vary depending on numerous factors including seasonal rainfall, local irrigation, storm water recharge, groundwater recharge and pumping, among other factors. The regional groundwater table is not expected to be encountered during the planned construction.

4.4 Subsurface Variations

Based on results of the subsurface exploration and our experience with the subject area, some variations in the continuity and nature of subsurface conditions within the project site are anticipated. Because of the uncertainties involved in the nature and depositional characteristics of the earth material at the site, care should be exercised in interpolating or extrapolating subsurface conditions between or beyond the boring locations. If, during construction, subsurface conditions are encountered that are different from those presented in this report, this office should be notified immediately so that recommendations can be modified, if necessary.

5.0 FAULTING AND SEISMIC HAZARDS

Geologic hazards are defined as geologically related conditions that may present a potential danger to life and property. Typical geologic hazards in Southern California include earthquake ground shaking, fault surface rupture, liquefaction and seismically induced settlement, lateral spreading, landslides, earthquake induced flooding, tsunamis and seiches, and volcanic eruption hazard.

Results of a site-specific evaluation for each type of possible seismic hazard are discussed in the following sections.

5.1 Seismic Characteristics of Nearby Faults

The subject site is situated within a seismically active region. As is the case for most areas of Southern California, ground-shaking resulting from earthquakes associated with nearby and more distant faults may occur at the project site. During the life of the project, seismic activity associated with active faults can be expected to generate moderate to strong ground shaking at the site.

5.2 Surface Fault Rupture

The project site is not located within a currently designated State of California Earthquake Fault Zone (formerly Alquist-Priolo Special Studies Zones) for surface fault rupture. No surface faults are known to project through or towards the site. The closest known fault with the potential for surface rupture is the Whittier Fault located approximately 2.3 miles to the south and southwest. As a result, the potential for surface rupture resulting from the movement of this fault or other nearby faults is considered to be low. The approximate locations of local and regional active faults with respect to the project site are shown on Drawing No. 5, *Southern California Regional Fault Map*. The mapped epicenters of earthquakes with magnitude 5.0 or greater in Southern California during the past 200 years are shown on Drawing No. 6, *Epicenters Map of Southern California Earthquakes (1800-1999)*.

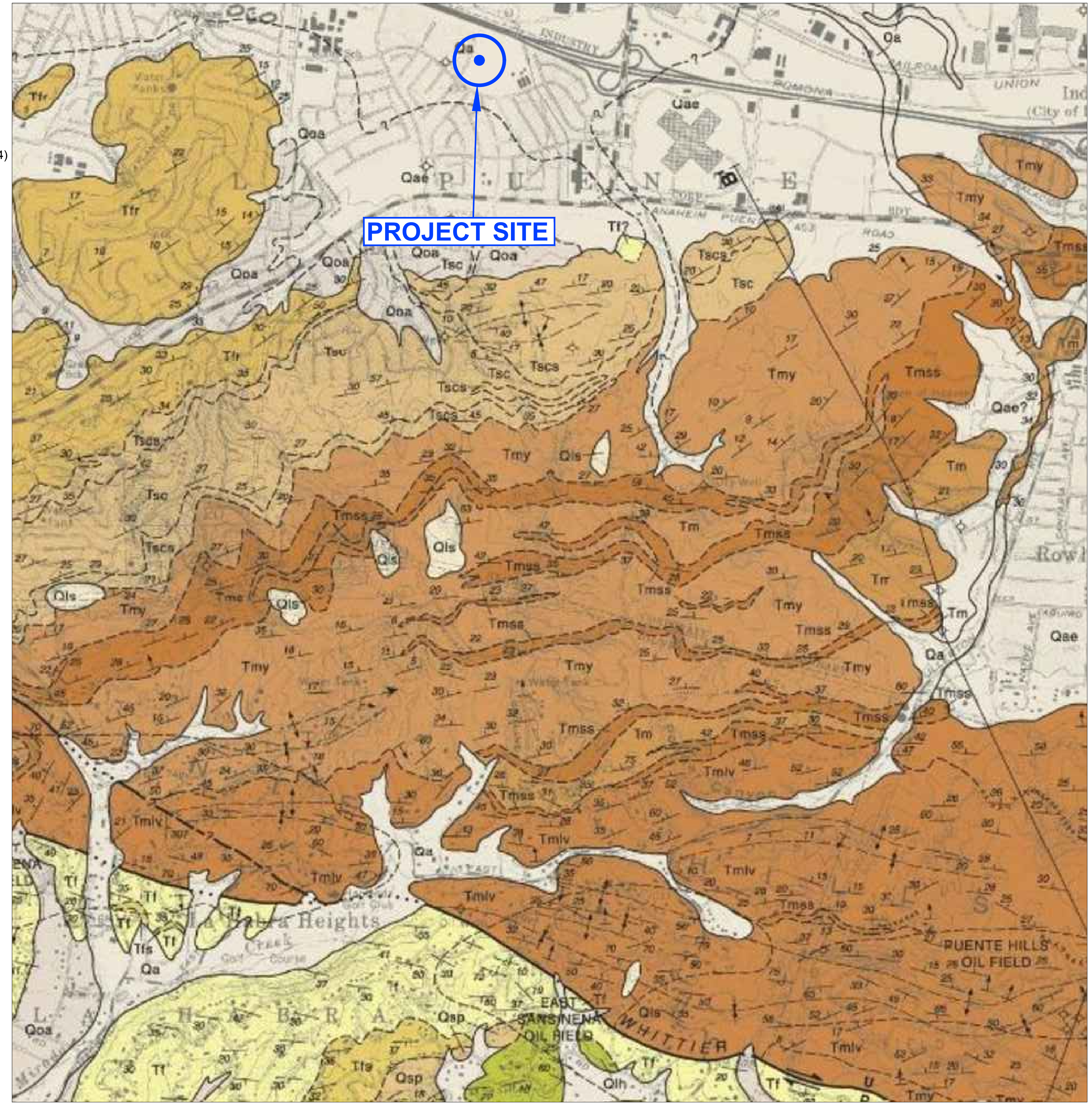
<u>Whittier Fault</u>

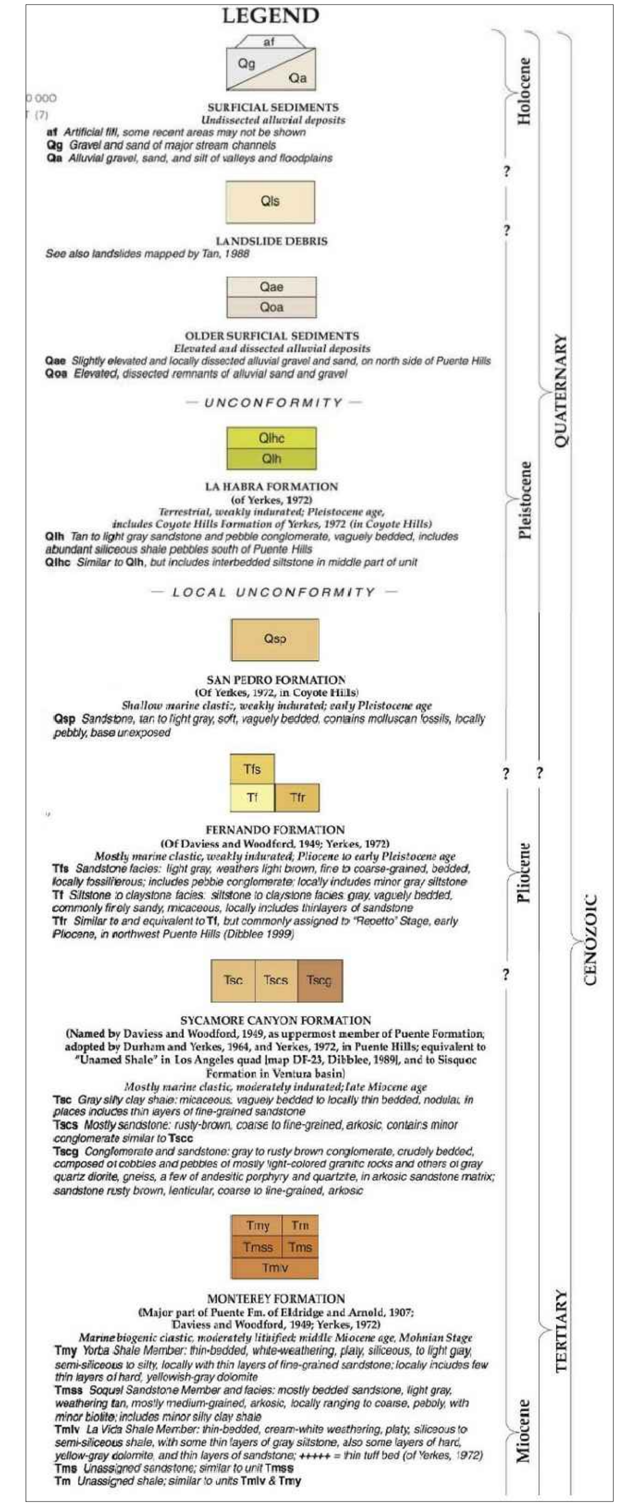
The mapped surface trace of the Whittier Fault is located approximately 2.3 miles southwest of the project site along the northern flank of the Puente Hills. Portions of this fault are included in the revised official map for the State of California Special Studies Zone, La Habra Quadrangle effective November 1, 1991.

The Whittier Fault is considered part of the Elsinore Fault system, which is one of the major right-lateral strike slip faults of the Peninsular Ranges geomorphic province. The Elsinore fault system splits northwestward into the Chino-Central Avenue fault and westward into the Whittier fault near the City of Corona.

NOT TO SCALE

REFERENCE: THOMAS W. DIBBLE, Jr. (DF-74)
WHITTIER QUADRANGLE
AND LA HABRA QUADRANGLE (2001)





REGIONAL GEOLOGIC MAP

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WEDGEWORTH ELEMENTARY SCHOOL
16949 WEDGEWORTH DRIVE, HACIENDA HEIGHTS, CA
FOR: HACIENDA LA PUENTE UNIFIED SCHOOL DISTRICT

Project No.	Drawing No.
18-31-330-02	3

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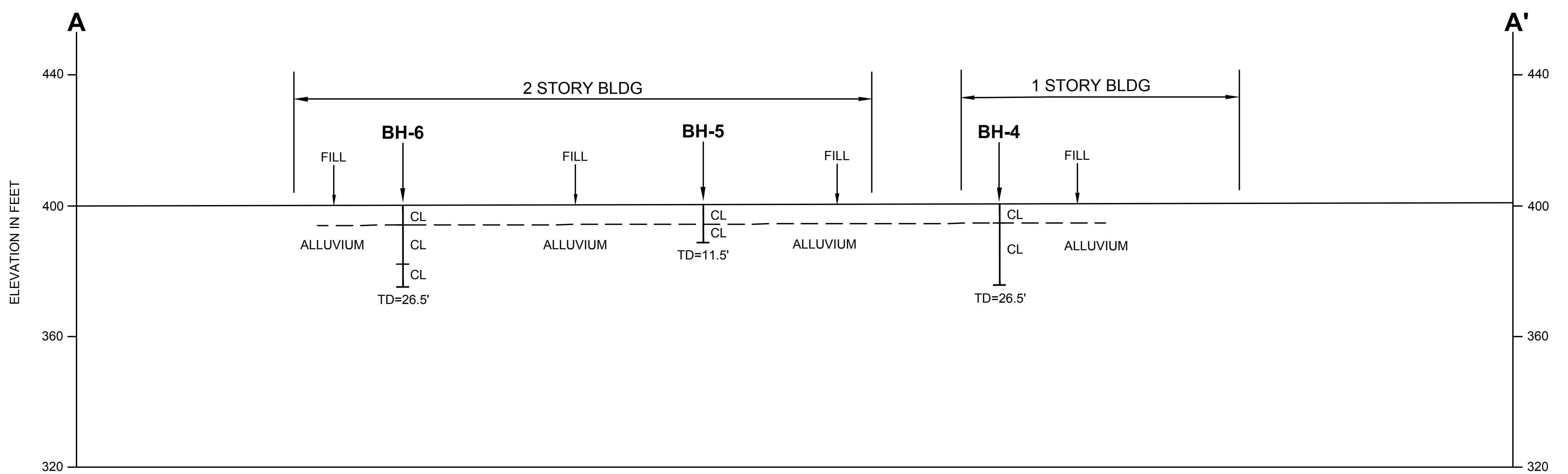

PROFILE:
HORIZONTAL: 1"=40'
VERTICAL: 1"=40'

GEOLOGIC CROSS SECTION A-A'

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WEDGEWORTH ELEMENTARY SCHOOL PROJECT
16949 WEDGEWORTH DRIVE, HACIENDA HEIGHTS, CA
FOR: HACIENDA LA PUENTE UNIFIED SCHOOL DISTRICT

Project No. 18-31-330-02

Drawing No. 4A

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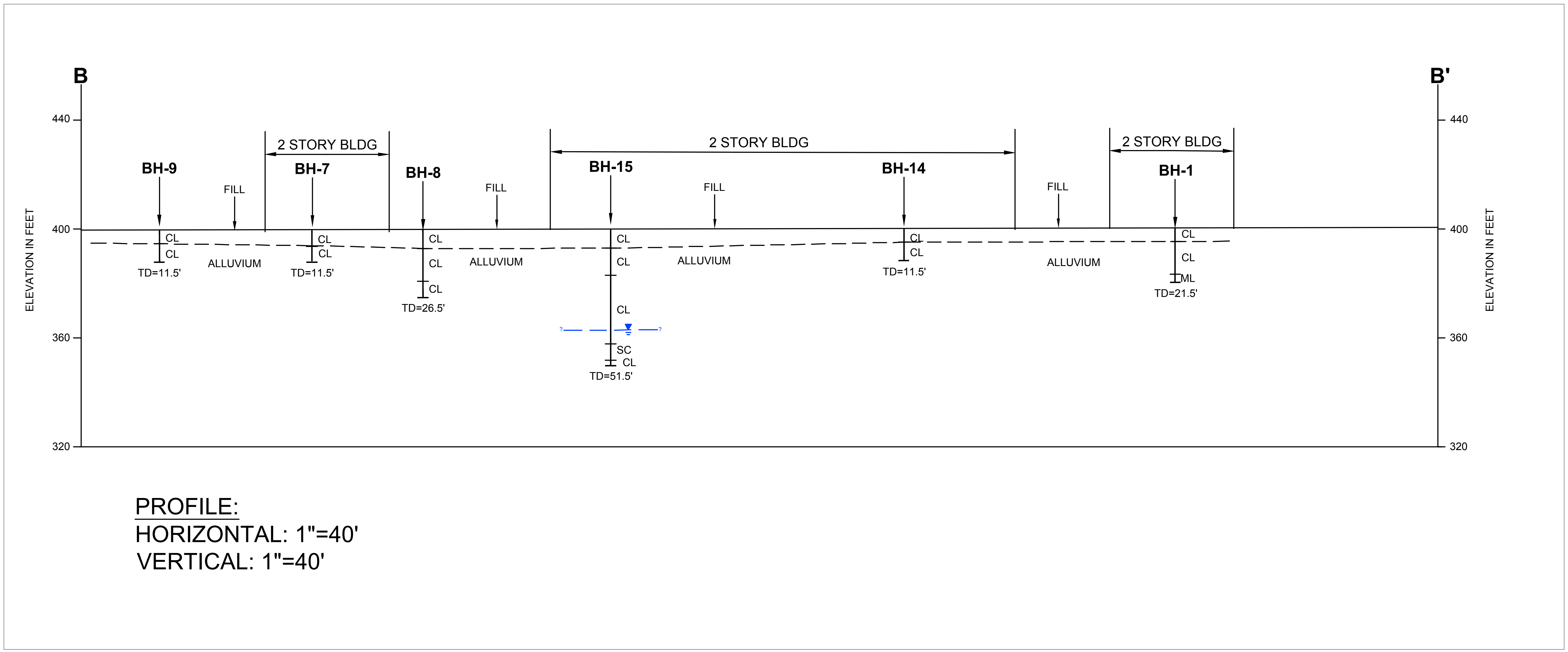

GEOLOGIC CROSS SECTION B-B'

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WEDGEWORTH ELEMENTARY SCHOOL PROJECT
16949 WEDGEWORTH DRIVE, HACIENDA HEIGHTS, CA
FOR: HACIENDA LA PUENTE UNIFIED SCHOOL DISTRICT

Project No. 18-31-330-02

Drawing No. 4B

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The Whittier fault dips steeply northward with some reverse separation along most of its length. However, the late Quaternary evidence is for nearly pure strike slip movement (Gath, 1997). The Whittier fault is considered to be capable of producing a maximum movement magnitude of 6.8 (Mw) earthquake.

5.3 Liquefaction and Seismically-Induced Settlement

Liquefaction is the sudden decrease in the strength of cohesionless soils due to dynamic or cyclic shaking. Saturated soils behave temporarily as a viscous fluid (liquefaction) and consequently lose their capacity to support the structures founded on them. The potential for liquefaction decreases with increasing clay and gravel content, but increases as the ground acceleration and duration of shaking increase. Liquefaction potential has been found to be the greatest where the groundwater level and loose sands occur within 50 feet of the ground surface.

The site is located within a mapped Seismic Hazard Zone for liquefaction as shown on Drawing No. 7, *Seismic Hazard Zones Map*. Based on the results of our subsurface exploration and laboratory tests the site is comprised of silt, silty clay and clay, the risk of liquefaction is considered low. The seismically-induced settlement is negligible.

Table No. 1, Summary of Regional Faults

Fault Name and Section	Approximate Distance to Site (miles)	Max. Moment Magnitude (Mmax)	Slip Rate (mm/yr)
Whittier / Elsinore	2.71	7.8	2.5
San Jose	4.4	6.7	0.5
Puente Hills (Santa Fe Springs)	6.52	6.7	0.7
Sierra Madre	10.56	7.2	2
Elysian Park (Upper)	10.64	6.7	1.3
Puente Hills (LA)	11.09	7.0	0.7
Chino, alt 2	11.21	6.8	1
Raymond	11.88	6.8	1.5
Clamshell-Sawpit	13.01	6.7	0.5
Cucamonga	14.83	6.7	5
Verdugo	15.57	6.9	0.5
Newport Inglewood Connected alt 2	18.81	7.5	1.3
Newport Inglewood Connected alt 1	18.88	7.5	1.3
Hollywood	18.89	6.7	1
San Joaquin Hills	20.77	6.5	0.5
Santa Monica Connected	21.44	7.4	2.4

***** Review of published geologic data and USGS faults_2008 website tool.

5.4 Lateral Spreading

Seismically induced lateral spreading involves primarily lateral movement of earth materials due to ground shaking. It differs from the slope failure in that complete ground failure involving large movement does not occur due to the relatively smaller gradient of the initial ground surface. Lateral spreading is demonstrated by near-vertical cracks with predominantly horizontal movement of the soil mass involved. The topography at the project site and in the immediate vicinity of the site is gently sloping. Under these circumstances, the potential for lateral spreading at the subject site is considered very low.

5.5 Seismically-Induced Slope Instability

Seismically induced landslides and other slope failures are common occurrences during or after earthquakes in areas of significant relief. The project site is not adjacent to any steep slopes and is gently sloping. In the absence of significant ground slopes, the potential for seismically induced landslides to affect the proposed site is considered to be low.

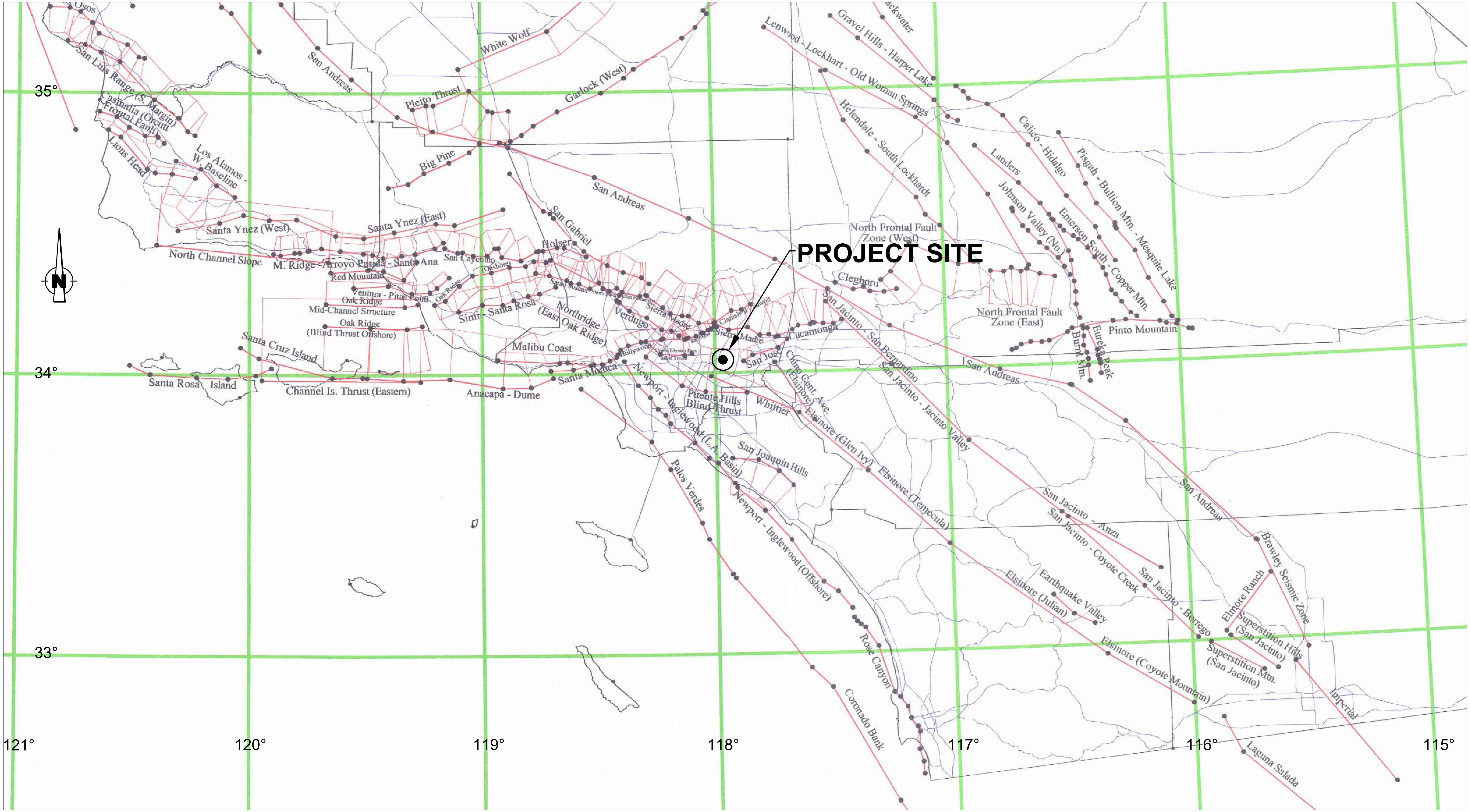
5.6 Earthquake-Induced Flooding

This is flooding caused by failure of dams or other water-retaining structures as a result of earthquakes. Review of the Flood Insurance Rate Map (FIRM), Los Angeles County 065043, Panel 06037C1875F, effective date September 26, 2008, from the Map Service Center (MSC) viewer, indicates that the site is designated as Zone “X”, “Area of Minimal Flood Hazard”.

The potential of earthquake induced flooding of the subject site is considered to be remote because of regional storm drain and flood control structures and the fact the site is listed in an area of minimal flood hazard by FEMA.

5.7 Tsunami and Seiches

Tsunamis are seismic sea waves generated by fault displacement or major ground movement. Based on the location of the site from the ocean (approximately 29 kilometers), tsunamis do not pose a hazard. Seiches are large waves generated in enclosed bodies of water in response to ground shaking. Based on site location away from lakes and reservoirs, seiches do not pose a significant hazard.



REFERENCE: PORTION OF CGS 2002 CALIFORNIA FAULT MODEL MODIFIED FOR USE WITH FRISKSP AND EQFAULT BY THOMAS F. BLAKE, AUGUST 2004

FAULT SOURCES

BLIND THRUST FAULT, POLYGONS INDICATE RUPTURE PLANES AND DIP DIRECTION

SOUTHERN CALIFORNIA REGIONAL FAULT MAP

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WEDGEWORTH ELEMENTARY SCHOOL
16949 WEDGEWORTH DRIVE, HACIENDA HEIGHTS, CA
FOR: HACIENDA LA PUENTE UNIFIED SCHOOL DISTRICT

Project No.	Drawing No.
18-31-330-02	5

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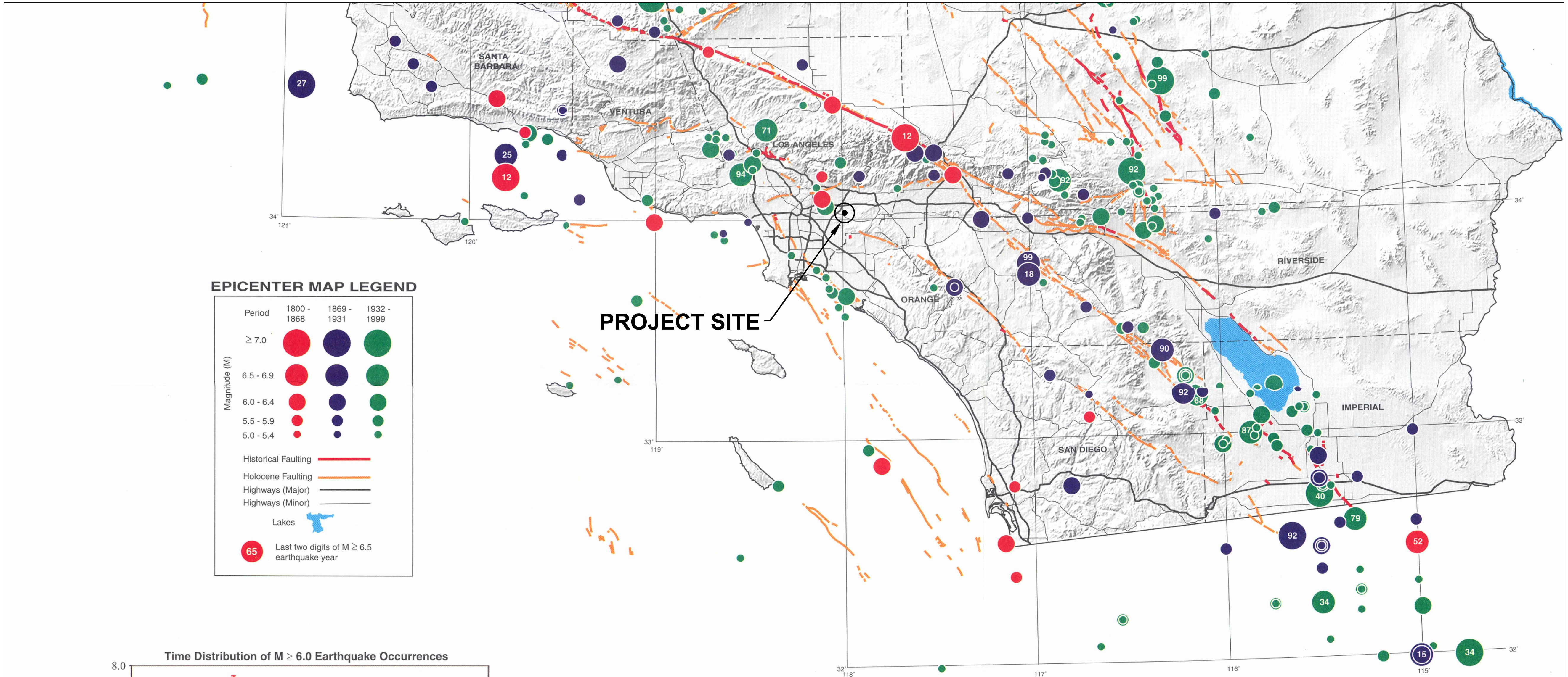

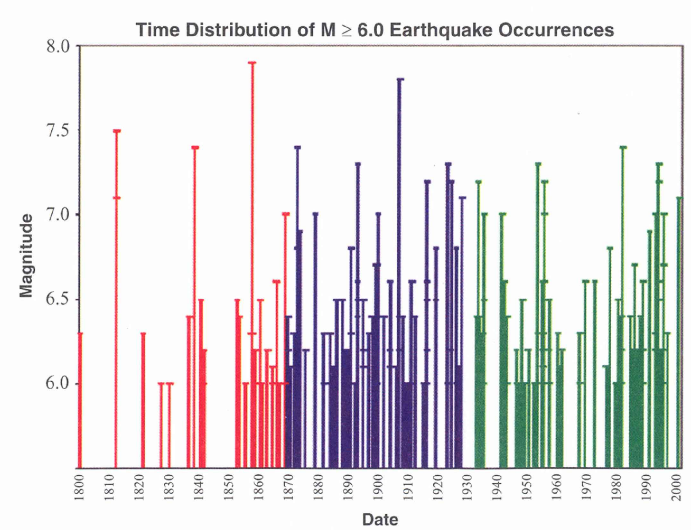

Main Sources of Information

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Toppozada, T.R. and Branum, D.M. (in press). California M ≥ 5.5 earthquakes, history and areas damaged, *in* Lee, W.H., Kanamori, H. and Jennings, P., International Handbook of Earthquake and Engineering Seismology, International Association of Seismology and Physics of the Earth's Interior.

Toppozada, T.R. and Parke, D.L. (1982). Areas damaged by California earthquakes, 1900-1949, California Division of Mines and Geology Open-File Report 82-17, 65 p.

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Toppozada, T.R., Real, C.R. and Parke, D.L. (1981). Preparation of isoseismal maps and summaries of reported effects for pre-1900 California earthquakes, California Division of Mines and Geology Open-File Report 81-11 SAC, 182 p.

UC Berkeley/USGS earthquake catalog, http://quake.geo.berkeley.edu/ncedc/catalog-search.html. Northern California earthquake catalog and phase data, Northern California Seismic Network, U.S. Geological Survey, Menlo Park; Berkeley Seismological Laboratory, University of California, Berkeley, California.

MINES AND GEOLOGY
CALIFORNIA CONSERVATION

REFERENCE: PORTION OF EPICENTERS AND AREAS DAMAGED BY M≥5 CALIFORNIA EARTHQUAKES, 1800-1999 CALIFORNIA DEPARTMENT OF CONSERVATION, MAP SHEET 49 DATED 2000.

EPICENTER MAP OF SOUTHERN CALIFORNIA EARTHQUAKES (1800-1999)

Converse Consultants	WEDGEWORTH ELEMENTARY SCHOOL 16949 WEDGEWORTH DRIVE, HACIENDA HEIGHTS, CA FOR: HACIENDA LA PUENTE UNIFIED SCHOOL DISTRICT	Project No. 18-31-330-02	Drawing No. 6

6.0 SEISMIC ANALYSIS

6.1 CBC Seismic Design Parameters

Seismic parameters based on the 2016 California Building Code are calculated using the ATC Hazard by location *Seismic Design Maps* website application and the site coordinates (33.99644 degrees North Latitude, -117.93666 degrees West Longitude). The seismic parameters are presented below.

Table No. 2, CBC Seismic Design Parameters

Seismic Parameters	2016 CBC
Site Class	D
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_S	2.182 g
Mapped 1-second Spectral Response Acceleration, S_1	0.776 g
Site Coefficient (from Table 1613.5.3(1)), F_a	1.0
Site Coefficient (from Table 1613.5.3(2)), F_v	1.5
MCE 0.2-sec period Spectral Response Acceleration, S_{MS}	2.182 g
MCE 1-second period Spectral Response Acceleration, S_{M1}	1.165 g
Design Spectral Response Acceleration for short period, S_{DS}	1.455 g
Design Spectral Response Acceleration for 1-second period, S_{D1}	0.776 g
Seismic Design Category	E

6.2 Site-Specific Response Spectra

A site-specific response spectrum was developed for the project for a Maximum Considered Earthquake (MCE), defined as a horizontal peak ground acceleration that has a 2 percent probability of being exceeded in 50 years (return period of approximately 2,475 years). The controlling source was determined to be the USGS 2008 California Gridded Source.

In accordance with ASCE 7-10, Section 21.2 the site-specific response spectra can be taken as the lesser of the probabilistic maximum rotated component of MCE ground motion and the 84th percentile of deterministic maximum rotated component of MCE ground motion response spectra. The design response spectra can be taken as 2/3 of site-specific MCE response spectra but should not be lower than 80 percent of CBC general response spectra. The risk coefficient C_R has been incorporated at each spectral response period for which the acceleration was computed in accordance with ASCE 7-10, Section 21.2.1.1.

The 2016 CBC mapped acceleration parameters are provided in the following table. These parameters were determined using the ATC Hazard by location *Seismic Design Maps* website application, and in accordance with ASCE 7-10 Sections 11.4, 11.6, 11.8 and 21.2.

Table No. 3, 2016 CBC Mapped Acceleration Parameters

Site Class	D	Seismic Design Category	E
S_s	2.182	C_{RS}	0.954
S_1	0.776	C_{R1}	0.975
F_a	1	$0.08\ F_v/F_a$	0.120
F_v	1.5	$0.4\ F_v/F_a$	0.600
S_{MS}	2.182	T_0	0.107
S_{M1}	1.164	T_S	0.533
S_{DS}	1.455	T_L	8
S_{D1}	0.776		

A Site-Specific response analysis, using faults within 200 kilometers of the sites, was developed using the computer program EZ-FRISK by Risk Engineering (v. 7.65) and the 2008 USGS Fault Model database. Attenuation relationships proposed by Boore and Atkinson (2008), Campbell and Bozorgnia (2008), Chiou and Youngs (2007) were used in the analysis. These attenuation relationships are based on Next Generation Attenuation (NGA) project model. An average shear wave velocity at upper 30 meters of soil profile (V_{s30}) of 260 meters per second, depth to bedrock of with a shear wave velocity 1,000 meters per second at 150 meters below grade, and depth of bedrock where the shear wave velocity is 2,500 meters per second at 2,500 meters below grade were selected for EZ-Frisk Analysis.

The probabilistic response spectrum results and peak ground acceleration for each attenuation relationship are presented in the following table.

Table No. 4, Probabilistic Response Spectrum Data

Attenuation Relationship	Probabilistic Mean	Boore-Atkinson (2008)	Campbell-Bozorgnia (2008)	Chiou-Youngs (2007)
Peak Ground Acceleration (g)	0.964	0.959	0.910	1.016

Spectral Period (sec)	2% in 50yr Probabilistic Spectral Acceleration (g)			
0.05	1.127	1.131	1.050	1.196
0.10	1.544	1.604	1.420	1.605
0.20	2.008	2.107	1.834	2.051
0.30	2.026	2.172	1.835	2.036
0.40	1.978	2.180	1.801	1.911
0.50	1.907	2.146	1.771	1.762
0.75	1.622	1.831	1.533	1.489

1.00	1.382	1.460	1.357	1.326
2.00	0.785	0.804	0.823	0.724
3.00	0.519	0.541	0.533	0.474
4.00	0.387	0.402	0.405	0.346

Applicable response spectra data are presented in the table below and on Drawing No. 8, *Site-Specific Design Response Spectrum*. These curves correspond to response values obtained from above attenuation relations for horizontal elastic single-degree-of-freedom systems with equivalent viscous damping of 5 percent of critical damping.

Table No. 5, Site-Specific Response Spectrum Data

Period (sec)	2% in 50yr Probabilistic Spectral Acceleration (g)	Risk Coefficient C_R	Probabilistic MCE_R Spectral Acceleration (g)	84th Percentile Deterministic MCE Response Spectra (g)	Deterministic CBC Lower Level, (g)	Site Specific MCE_R Spectral Acceleration (g)	80% CBC Design Response Spectrum	Site Specific Design Spectral Acceleration (g)
0.05	1.127	0.954	1.075	1.212	0.375	*1.075*	*0.793*	***0.79***
0.10	1.544	0.954	1.473	1.597	0.750	*1.473*	*1.120*	***1.12***
0.20	2.008	0.954	1.916	2.065	1.500	*1.916*	*1.164*	***1.28***
0.30	2.026	0.957	1.938	2.172	1.500	*1.938*	*1.164*	***1.29***
0.40	1.978	0.959	1.897	2.215	1.500	*1.897*	*1.164*	***1.26***
0.50	1.907	0.962	1.834	2.197	1.500	*1.834*	*1.164*	***1.22***
0.75	1.622	0.968	1.571	1.986	1.200	*1.571*	*0.828*	***1.05***
1.00	1.382	0.975	1.347	1.919	0.900	*1.347*	*0.621*	***0.90***
2.00	0.785	0.975	0.766	1.538	0.450	*0.766*	*0.310*	***0.51***
3.00	0.519	0.975	0.506	1.246	0.300	*0.506*	*0.207*	***0.34***
4.00	0.387	0.975	0.377	1.013	0.225	*0.377*	*0.155*	***0.25***

The site-specific design response parameters are provided in the following table. These parameters were determined from Design Response Spectra presented in table above and following guidelines of ASCE Section 21.4.

Table No. 6, Site-Specific Seismic Design Parameters

Parameter	Value (5% Damping)	Lower Limit, 80% of CBC Design Spectra
Site-Specific 0.2-Second Period Spectral Response Acceleration, S_{MS}	*1.916*	*1.746*
Site-Specific1-Second Period Spectral Response Acceleration, S_{M1}	*1.531*	*0.931*
Site-Specific Design Spectral Response Acceleration for Short Period S_{DS}	*1.277*	*1.164*
Site-Specific Design Spectral Response Acceleration for 1-Second Period, S_{D1}	*1.021*	*0.621*

7.0 GEOTECHNICAL EVALUATIONS AND CONCLUSIONS

Based on the results of our background review, subsurface exploration, laboratory testing, geotechnical analyses, and understanding of the planned site re-development, it is our opinion that the proposed project is feasible from a geotechnical standpoint, provided the following conclusions and recommendations are incorporated into the project plans and specifications, and are followed during site construction. The following geotechnical findings should be considered for the planned project:

- Groundwater was encountered during our subsurface exploration at a depth of 39 feet bgs in Boring BH-2 and at a depth of 37 feet bgs in Boring 15. Review of the Seismic Hazard Zone Report for the La Habra Quadrangle (CDMG 1997) indicated the historically highest groundwater levels are at depths of approximately 25 feet below ground surface. More recent groundwater level monitoring in local groundwater wells has shown depths to groundwater varies between approximately 27 feet and 35 feet below ground surface. Groundwater is not anticipated during construction, however may need to be considered in design if deeper foundations are used based on the historically highest groundwater levels and groundwater depth encountered in Boring BH-15.

- The site is located within a mapped Seismic Hazard Zone for liquefaction as shown on Drawing No. 7, *Seismic Hazard Zones Map*. Based on the results of our subsurface exploration and laboratory tests the site is comprised of silt, silty clay and clay, the risk of liquefaction is considered low. The seismically-induced settlement is negligible.

- Undocumented fills consisting of silty clay and clays were encountered ranging in depth from approximately five (5) feet to seven (7) in depth below ground surface at the exploratory boring sites. Undocumented fill should be excavated and recompacted. The alluvial sediments consist predominately of silty clays, sandy silts, clayey sands, silty sands, sandy clays and clays to depths of approximately 5 feet below ground surface to the maximum explored depth of 51.5 feet below ground surface.

- The surficial site soils at the site exhibit a “Low” expansive potential. Mitigation for expansive soil is not considered necessary.

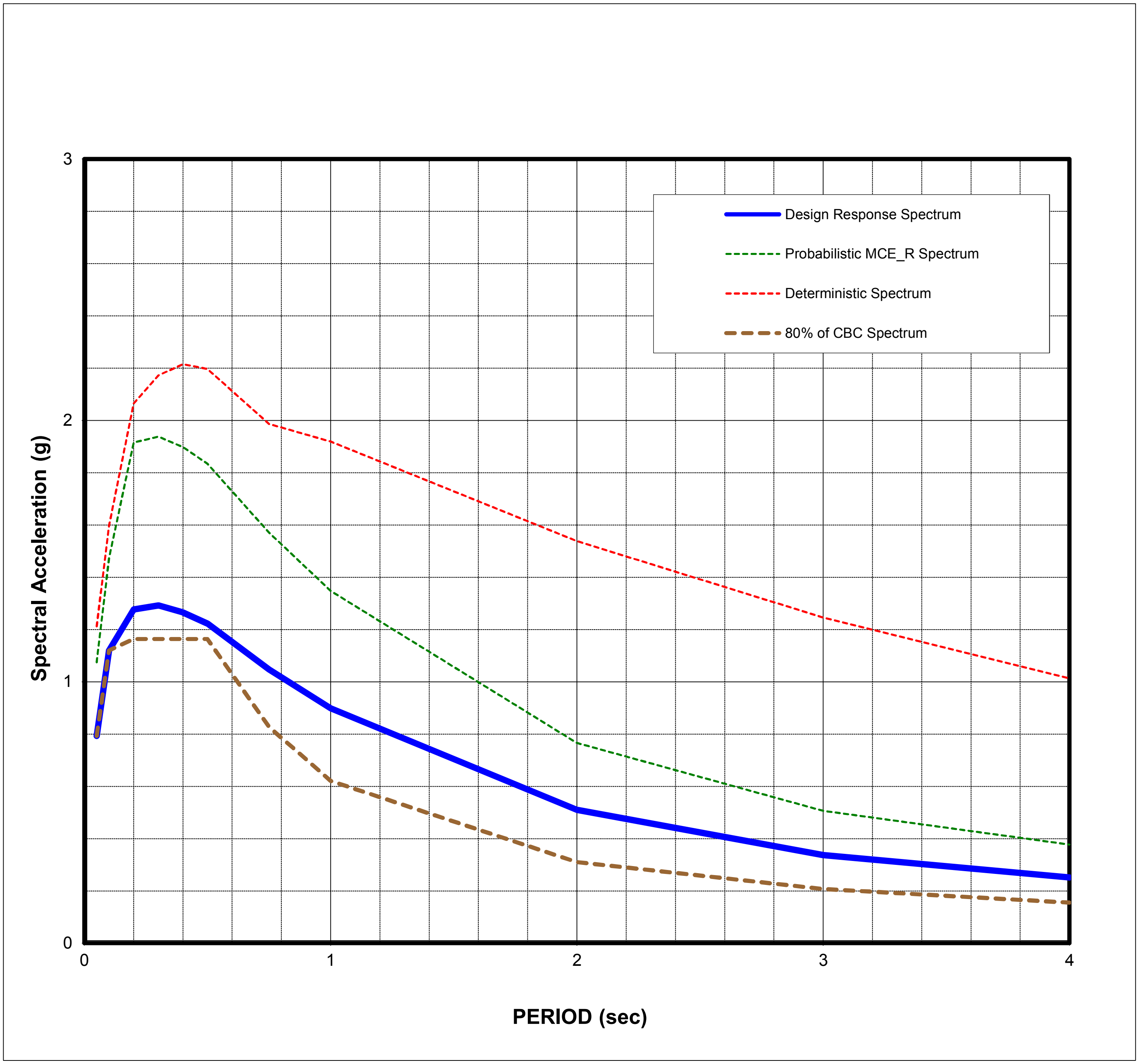
- In general, the pH value and concentrations of water soluble sulfates saturated resistivity of the site soils are in the non-corrosive range. The saturated resistivity and chloride content of the site soils are in corrosive range to ferrous metals.

- The earth materials at the site should be excavatable with conventional heavy-duty earth moving equipment. Earthwork should be performed with suitable equipment for gravelly materials.

- Shallow spread and continuous footings are considered suitable for retaining wall support provided the recommendations in this report are incorporated into the project plans and specifications and are followed during site construction.

- For non-building structures (e.g. signs, fence walls, short retaining walls, etc.), conventional footings can be used.

- Percolation testing was performed utilizing exploratory boring PT-5 to evaluate soil infiltration rates of the native soils encountered between depths of 10 to 20 feet below the ground surface. The percolation results are provided in appendix C: Percolation Testing.



Note: Calculated using EZFRISK program Risk Engineering, version 7.62
and USGS 2008 fault model database.

SITE SPECIFIC DESIGN RESPONSE SPECTRUM

Wedgeworth ES

Hacienda Heights

For : HLPUSD

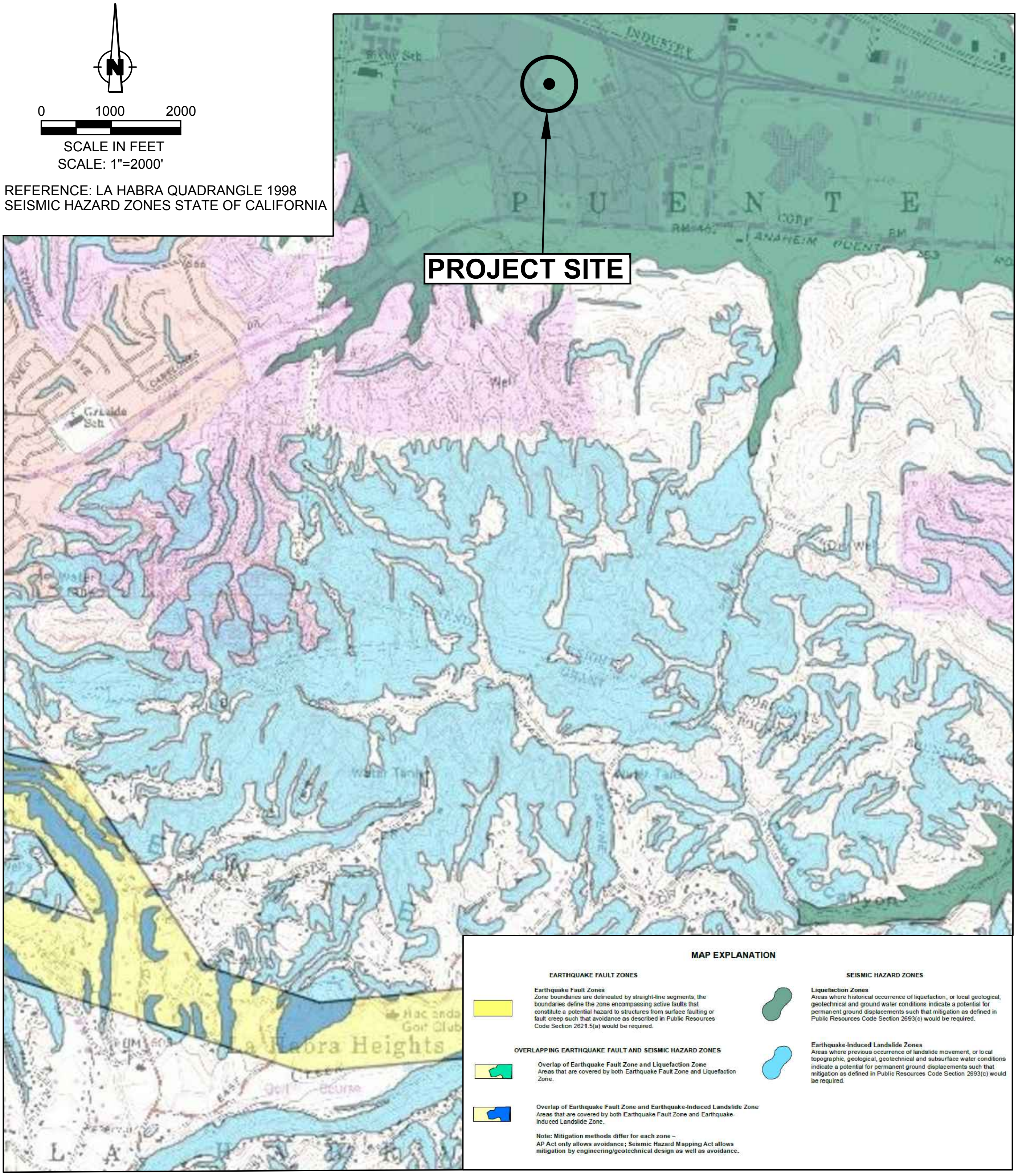
Project Number:

18-31-330-02

Converse Consultants

Drawing No.

8



SEISMIC HAZARD ZONES MAP

WEDGEWORTH ELEMENTARY SCHOOL PERCOLATION
16949 WEDGEWORTH DRIVE, HACIENDA HEIGHTS, CA
FOR: HACIENDA LA PUENTE UNIFIED SCHOOL DISTRICT

Project No.
18-31-330-02

Drawing No.
7

Converse Consultants

8.0 EARTHWORK RECOMMENDATIONS

8.1 General Evaluation

Site earthwork recommendations provided in this section are based on our experience with similar projects and our evaluation of this study. Based on our understanding of the proposed project and the results of our field exploration, laboratory testing, and analysis of subsurface conditions at the site, we anticipate that the main earthwork activities associated with construction will be remedial grading (over-excavation and re-compaction), foundation excavations and trench excavation/backfill for utilities.

Excavated site soils, free of deleterious materials and rock particles larger than three (3) inches in the largest dimension, should be suitable for placement as compacted trench fill. Any import fill should be tested and approved by geotechnical engineer or their representative. Any import fill should have an expansion potential less than 20. All compacted fill soils should be observed and tested by a Converse representative in accordance with the specifications presented in this section.

8.2 Over-Excavation

Prior to the start of construction, all loose soil, undocumented fill and soils disturbed during demolition should be removed to firm and unyielding native material or compacted fill.

Due to the undocumented fill encountered at the site, we recommend over-excavation for structures to be at least five (5) feet below the existing grade or at least three (3) feet below the bottom of footings, whichever is deeper. Deeper removal will be needed if firm soil conditions are not exposed on the excavation bottom. Over-excavation should extend at least five (5) feet laterally beyond the limits of perimeter footings where feasible. The on-site soil is considered suitable for re-use as regular compacted fill.

The upper 24 inches of site soils should be removed in areas of sidewalks and surface parking. The over-excavation and re-compaction should extend at least two (2) feet laterally beyond the sidewalk and surface parking areas. If loose, disturbed, or otherwise unsuitable materials are encountered at the bottom of excavation, deeper removals will be required until firm native soils are encountered.

Excavation activities should not disturb adjacent utilities or undermine any adjacent buildings and structures to remain. Existing utilities should be removed and adequately capped at the project boundary line, or salvaged/rerouted as designed.

The actual depth of removal should be based on recommendations and observation made during grading. Therefore, some variations in the depth and lateral extent of over-excavation recommended in this report should be anticipated.

8.3 Structural Preparation

All exposed subgrade soil surface should be observed by a geotechnical engineer or their representative prior to placement of fill, base materials or slabs. The exposed subgrade should be scarified at least 6 inches, moisture conditioned as needed to near-optimum moisture content, mixed and compacted to at least 90 percent relative compaction. The upper 12 inches of subgrade below new pavement should be compacted to 95 percent relative compaction.

If loose, yielding soil conditions are encountered at the excavation bottom, the following options can be considered:

a. Over-excavate until a firm bottom is reached.
b. Over-excavate an additional 18 inches deep, and then place at least 18-inch-thick compacted base material (CAB or equivalent) to bridge the soft bottom. Base materials should be compacted to 95% relative compaction.
c. Over-excavate an additional 18 inches deep, and then place a layer of geotextile reinforcement fabric (i.e. Mirafi HP570, or equivalent), place 18-inch-thick compacted base material (CAB or equivalent) to bridge the soft bottom. Base materials should be compacted to 95% relative compaction. An additional layer of geotextile reinforcement may be needed on top of base depending on the actual site conditions.

8.4 Engineered Fill

All engineered fill should be placed on competent, scarified and compacted bottom as evaluated by the geotechnical engineer and in accordance with the specifications presented in this section. Excavated site soils, free of deleterious materials and rock particles larger than three (3) inches in the largest dimension, should be suitable for placement as compacted fill. Any proposed import fill should be evaluated and approved by geotechnical engineer or their representatives prior to import to the site. Import fill material should have an expansion index less than 20.

Prior to compaction, fill materials should be thoroughly mixed and moisture conditioned to within three (3) percent of the optimum moisture content for granular soils and to approximate three (3) percent above the optimum moisture for fine-grained soils. Fill soils shall be evenly spread in maximum 8-inch lifts, watered or dried as necessary, mixed and compacted to at least the density specified below. The fill shall be placed and compacted on a horizontal plane, unless otherwise approved by the geotechnical engineer. All fill, if not specified otherwise elsewhere in this report, should be compacted to at least 90 percent of the laboratory dry density in accordance with the ASTM Standard D2922 test method.

8.5 Excavatability

Based on our field exploration, the earth materials at the site may be excavated with conventional heavy-duty earth moving and trenching equipment. The onsite materials may contain demolition debris and gravel and/or cobbles. Earthwork should be performed with suitable equipment and methods for removal of debris from the engineered fill.

8.6 Expansive Soil

The on-site shallow soils at the subject site have a “low” expansion potential. Mitigation for expansive soil is not considered necessary. If encountered at the excavation depth, on-site soils with an expansion index exceeding 20 should not be re-used for compaction within 5 feet below the planned finish grade or for retaining wall backfill. Soils containing organic materials should not be used as structural fill. The extent of removal should be determined by the geotechnical representative based on soil observation during grading.

The recommendations contained in this report are based upon the anticipated expansion soil conditions. Any proposed import fill should have an expansion index less than 20, and should be evaluated and approved by Converse prior to import to the site.

8.7 Trench Zone Backfill

The following specifications are recommended to provide a basis for quality control during the placement of trench backfill.

Trench excavations to receive backfill shall be free of trash, debris or other unsatisfactory materials at the time of backfill placement. Excavated on-site soils free of oversize particles, defined as larger than one (1) inch in maximum dimension in the upper 12 inches of subgrade soils and larger than three (3) inches in the largest dimension in the trench backfill below, and deleterious matter after proper processing may be used to backfill the trench zone. Imported trench backfill, if used, should be approved by the project soils consultant prior to delivery at the site. No more than 30 percent of the backfill volume should be larger than ¾ inch in the largest dimension.

Trench backfill shall be compacted to 90 percent of the laboratory maximum dry density as per ASTM Standard D2922 test method. At least the upper twelve (12) inches of trench underlying pavements should be compacted to at least 95 percent of the laboratory maximum dry density.

Trench backfill shall be compacted by mechanical methods, such as sheepsfoot, vibrating or pneumatic rollers, or mechanical tampers, to achieve the density specified herein. The backfill materials shall be brought to within three (3) percent of optimum moisture content and then placed in horizontal layers if the expansion index is less than or equal to 30. Should the expansion index be greater than 30, backfill materials shall be brought to approximately three (3) percent above optimum moisture content. The thickness of uncompacted layers should not exceed eight (8) inches. Each layer shall be evenly spread, moistened or dried as necessary, and then tamped or rolled until the specified density has been achieved.

The contractor shall select the equipment and processes to be used to achieve the specified density without damage to adjacent ground and completed work. The field density of the compacted soil shall be measured by the ASTM Standard D1556 or ASTM Standard D2922 test methods or equivalent. Observation and field tests should be performed by geotechnical engineer or their representatives during construction to confirm that the required degree of compaction has been obtained. Where compaction is less than that specified, additional compactive effort shall be made with adjustment of the moisture content as necessary, until the specified compaction is obtained. It should be the responsibility of the contractor to maintain safe conditions during cut and/or fill operations. Trench backfill shall not be placed, spread or rolled during unfavorable weather conditions. When the work is interrupted by heavy rain, fill operations shall not be resumed until field tests by the project's geotechnical consultant indicate that the moisture content and density of the fill are as previously specified.

Imported soils, if any, used as compacted trench backfill should be predominantly granular and meet the following criteria:

- Expansion Index less than 20
- Free of all deleterious materials
- Contain no particles larger than 3 inches in the largest dimension
- Contain less than 30 percent by weight retained on ¾-inch sieve
- Contain at least 15 percent fines (passing #200 sieve)
- Have a Plasticity Index of 10 or less

Any import fill should be tested and approved by the geotechnical representative prior to delivery to the site.

8.8 Shrinkage and Subsidence

Soil shrinkage and/or bulking as a result of remedial grading depends on several factors including the depth of over-excavation, and the grading method and equipment utilized, and average relative compaction. For preliminary estimation, bulking and shrinkage factors for various units of earth material at the site may be taken as presented below:

- The approximate shrinkage factor for the undocumented fill soils is estimated to range from ten (10) to fifteen (15) percent.
- The approximate shrinkage factor for the native alluvial soils is estimated to range from five (5) to ten (10) percent.
- For estimation purposes, ground subsidence may be taken as 0.1 feet as a result of remedial grading.

Although these values are only approximate, they represent our best estimates of the factors to be used to calculate lost volume that may occur during grading. If more accurate shrinkage and subsidence factors are needed, it is recommended that field-testing using the actual equipment and grading techniques be conducted.

The various design recommendations provided in this section are based on the assumptions that in preparing the site, the earthwork and site grading recommendations provided in this report will be followed. The proposed structures may be supported by shallow continuous and isolated square footings.

9.0 DESIGN RECOMMENDATIONS

9.1 Shallow Foundations

9.1.1 Vertical Capacity

Continuous and square footings should be founded at least 24 inches below lowest adjacent final grade on the recommended earth materials. A minimum footing width of 24 inches is recommended for continuous and square footings. The net allowable dead plus live load bearing value for isolated square and continuous footings is 2,000 psf. The net allowable bearing pressure can be increased by 250 psf for each additional foot of excavation depth and width up to a maximum value of 4,000 psf.

The net allowable bearing values indicated above are for the dead loads and frequently applied live loads and are obtained by applying a factor of safety of 3.0 to the net ultimate bearing capacity. Surcharge load form new structures should be considered for the adjacent existing buildings or retaining walls.

9.1.2 Lateral Capacity

Resistance to lateral loads can be provided by friction acting at the base of the foundation and by passive earth pressure. A coefficient of friction of 0.3 may be assumed with normal dead load forces. An allowable passive earth pressure of 180 psf per foot of depth up to a maximum of 1,800 psf may be used for footings poured against properly compacted fill or undisturbed stiff natural soils. The values of coefficient of friction and allowable passive earth pressure include a factor of safety of 1.5.

9.1.3 Settlement

The static settlement of structures supported on continuous and/or spread footings founded on compacted fill will depend on the actual footing dimensions and the imposed vertical loads. Most of the footing settlement at the project site is expected to occur immediately after the application of the load. Based on the maximum allowable net bearing pressures presented above, static settlement is anticipated to be less than 1.0 inch. Differential settlement is expected to be up to one-half of the total settlement over a 30-foot span.

9.1.4 Dynamic Increases

Bearing values indicated above are for total dead load and frequently applied live loads. The above vertical bearing may be increased by 33% for short durations of loading which will include the effect of wind or seismic forces. The allowable passive pressure may be increased by 33% for lateral loading due to wind or seismic forces.

9.2 Modulus of Subgrade Reaction

For the subject project, design of the structures supported on compacted fill subgrade prepared in accordance with the recommendations provided in this report may be based on a soil modulus of subgrade reaction of (k_s) of 100 pounds per square inch per inch.

9.3 Lateral Earth Pressure

The proposed retaining walls are anticipated to be up to 15 feet in height. The earth pressure behind any buried wall depends primarily on the allowable wall movement, type of backfill materials, backfill slopes, wall inclination, surcharges, and any hydrostatic pressure. The following fluid pressures are recommended for vertical walls with no hydrostatic pressure, no surcharge, and level backfill.

Table No. 7, Lateral Earth Pressures for Retaining Wall Design

Wall Type	Equivalent Fluid Pressure (pcf)
	Level Backfill
Cantilever Wall (Active pressure)	45 (Triangular Distribution)
Restrained Wall (At-rest pressure)	65 (Triangular Distribution)

The recommended lateral pressures assume that the walls are fully back-drained with granular, free-draining, non-expansive soil materials to prevent build-up of hydrostatic pressure. Adequate drainage could be provided by means of permeable drainage materials wrapped in filter fabric installed behind the walls. The drainage system should consist of perforated pipe surrounded by free draining, uniformly graded, ¾ -inch washed, permeable aggregate material, and wrapped in filter fabric (Mirafi 140N or equivalent) and should extend to about 2 feet below the finished grade. The filter fabric should overlap approximately 12 inches or more at the joints. The subdrain pipe should consist of perforated, four-inch diameter, Schedule 40 PVC or rigid ABS (SDR-35), or equivalent, with perforations placed down. Alternatively, a prefabricated drainage composite system such as the Miradrain G100N or equivalent can be used. The subdrain should be connected to a suitable outlet point, surface drain or sump pump. Subterranean walls should be waterproofed to prevent moisture migration and moisture problems.

In addition, walls with inclined backfill should be designed for an additional equivalent fluid pressure of one (1) pound per cubic foot for every two (2) degrees of slope inclination. Walls subjected to surcharge loads located within a distance equal to the height of the wall should be designed for an additional uniform lateral pressure equal to one-third or one-half the anticipated surcharge load for unrestrained or restrained walls, respectively. These values are applicable for backfill placed between the wall stem and an imaginary plane rising 45 degrees from below the edge (heel) of the wall footings.

Cantilever retaining walls greater than 12 feet, as measured from the surface, should be designed to resist additional earth pressure caused by seismic ground shaking. A dynamic earth pressure of 26H (psf), based on an inverted triangular distribution, can be used for design of wall.

9.4 Slabs-on-Grade

Slabs-on-grade should have a minimum thickness of five (5) inches for support of nominal ground-floor live loads without hydrostatic uplift pressures. Minimum reinforcement for slabs-on-grade should be No. 3 reinforcing bars, spaced at 18 inches on-center each way. The thickness and reinforcement of more heavily-loaded slabs will be dependent upon the anticipated loads and should be designed by a structural engineer.

Slabs should be designed and constructed as promulgated by the American Concrete Institute (ACI) and the Portland Cement Association (PCA). Prior to the slab pour, all utility trenches should be properly backfilled and compacted. Care should be taken during concrete placement to avoid slab curling.

In areas where a moisture-sensitive floor covering (such as vinyl tile or carpet) is used, slabs should be protected by at least a 15-mil-thick moisture barrier between the slab and compacted subgrade that meets the performance criteria of ASTM E 1745 Class A material. Polyethylene sheets should be overlapped a minimum of six inches and should be taped or otherwise sealed.

9.5 Soil Corrosivity Evaluation

Converse retained the Environmental Geotechnology Laboratory, Inc., located in Arcadia, California, to test two (2) selected soil samples taken in the general area of the proposed structures. The tests included minimum resistivity, pH, soluble sulfates, and chloride content, with the results summarized on the following table:

Table No. 8, Soil Corrosivity Test Results

Boring No.	Sample Depth (feet)	pH (Caltrans 643)	Soluble Chlorides (Caltrans 422) ppm	Soluble Sulfate (Caltrans 417) (%)	Saturated Resistivity (Caltrans 532) Ohm-cm
BH-5	0-5	7.10	255	0.048	980
BH-12	0-5	7.62	555	0.080	770

Based on our review of soil corrosivity test results (see Appendix B), the soluble sulfate concentration and pH are not in the corrosive range to concrete in accordance with the Caltrans Corrosive Guidelines (2015). However, the minimum saturated resistivity is in the corrosive range to ferrous metal. Protections of underground metal pipe should be considered. The soluble sulfate concentrations tested for this project are less than 2,000 ppm in the soil and the soluble chloride concentration is higher than 500 ppm. Chloride ions can lead to corrosion of steel reinforcement in concrete and steel structures by breaking down the normally present protective layer of oxides (Passive layer) present on the steel surface. Type I or II Portland Cement may be used for the construction of the foundations and slabs.

The test results presented herein are considered preliminary. Additional testing and evaluation of the as-graded soils is recommended. A corrosion engineer may be consulted for appropriate mitigation procedures and construction design, if needed. Conventional corrosion mitigation measures may include the following:

- Steel and wire concrete reinforcement should have at least three inches of concrete cover where cast against soil, unformed. Below-grade ferrous metals should be given a high-quality protective coating, such as 18-mil plastic tape, extruded polyethylene, coal-tar enamel, or Portland cement mortar.
- Below-grade metals should be electrically insulated (isolated) from above-grade metals by means of dielectric fittings in ferrous utilities and/or exposed metal structures breaking grade.

9.6 Flexible Pavement

The flexible pavement structural section design recommendations were performed in accordance with the method contained in the *CALTRANS Highway Design Manual*, Chapter 630 without the factor of safety. No specific traffic study was performed to determine the Traffic Index (TI) for the proposed project, therefore a wide range of TI values were evaluated.

Due to various earth materials encountered at the site, flexible pavement structural section recommendations are prepared for subgrade soils with the design R-value of 20. We recommend that the project structural engineer consider the traffic loading conditions at various locations and select the appropriate pavement sections from the following table:

Table No. 9, Flexible Pavement Structural Sections

Design R-value	Design TI	Asphalt Concrete (AC) Over Aggregate Base (AB) Structural Sections		Full AC Structural Section
		AC (inches)	AB (inches)	AC (inches)
20	4	3.0	*3.5*	*4.5*
	5	4.0	*5.0*	*6.5*
	6	5.0	*6.5*	*7.5*
	7	6.0	*8.0*	*9.0*
	8	7.0	*9.5*	*10.5*
	9	8.0	*10.5*	*12.0*

Base material shall conform to requirements for Crushed Miscellaneous Base (CMB) or equivalent and should be placed in accordance with the requirements of the Standard Specifications for Public Works Construction (SSPWC, latest Edition).

Asphaltic materials should conform to Section 203-1, "Paving Asphalt," of the Standard Specifications for Public Works Construction (SSPWC, latest Edition) and should be placed in accordance with Section 302-5, "Asphalt Concrete Pavement," of the SSPWC, 2012 edition.

Positive drainage should be provided away from all pavement areas to prevent seepage of surface and/or subsurface water into the pavement base and/or subgrade.

9.7 Rigid Pavement

Rigid pavement design recommendations were provided in accordance with the Portland Cement Association's (PCA) Southwest Region Publication P-14, *Portland Cement Concrete Pavement (PCCP) for Light, Medium, and Heavy Traffic*. We recommend that the project structural engineer consider the loading conditions at various locations and select the appropriate pavement sections from the following table:

Table No. 10, Rigid Pavement Structural Sections

Design R-Value	Design Traffic Index (TI)	PCCP Pavement Section (inches)
19	4.0	7.0
	5.0	7.0
	6.0	7.5
	7.0	8.0
	8.0	8.0
	9.0	8.5

The pavement sections presented in the table are based on a minimum 28-day Modulus of Rupture (M-R) of 550 psi and a compressive strength of 3,000 psi. The third point method of testing beams should be used to evaluate modulus of rupture. The concrete mix design should contain a minimum cement content of 5.5 sacks per cubic yard. Recommended maximum and minimum values of slump for pavement concrete are three (3) inches and one (1) inch, respectively.

Transverse contraction joints should not be spaced more than 15 feet and should be cut to a depth of ¼ the thickness of the slab. Longitudinal joints should not be spaced more than 12 feet apart. A longitudinal joint is not necessary in the pavement adjacent to the curb and gutter section.

All outside edges should conform to Section 201 of the Standard Specifications for Public Works Construction (SSPWC, latest edition), and should be constructed in accordance with Section 302-6 of the SSPWC. Pavement subgrade should be prepared in accordance with Section 9.7 of this report.

The PCCP materials should conform to Section 201 of the Specifications for Public Works Construction and should be constructed in accordance with Section 302-6 of the SSPWC.

Positive drainage should be provided away from all pavement areas to prevent seepage of surface and/or subsurface water into the pavement base and/or subgrade.

9.8 Site Drainage

Adequate positive drainage should be provided away from the structures to prevent ponding and to reduce percolation of water into structural backfill. We recommend that the landscape area immediately adjacent to the foundation shall be designed sloped away from the building with a minimum 5% slope gradient for at least 10 feet measured perpendicular to the face of the wall. Impervious surfaces within 10 feet of the building foundation shall be sloped a minimum of 2 percent away from the building per 2016 CBC.

Planters and landscaped areas adjacent to the building perimeter should be designed to minimize water infiltration into the subgrade soils. Gutters and downspouts should be installed on the roofs, and runoff should be directed to storm drains through non-erosive drainage devices. Lower level walkways, open patio areas and pool decks may require special drainage provisions and sump pumps to provide suitable drainage.

10.0 CONSTRUCTION RECOMMENDATIONS

10.1 General

Site soils should be excavatable using conventional heavy-duty excavating equipment. Temporary sloped excavation is feasible if performed in accordance with the slope ratios provided in Section 10.2, *Temporary Excavations*. Existing utilities should be accurately located and either protected or removed as required. For steeper temporary construction slopes or deeper excavations, shoring should be provided by the contractor as necessary, to protect the workers in the excavation.

10.2 Temporary Excavations

Based on the materials encountered in the exploratory borings, sloped temporary excavations may be constructed according to the slope ratios presented in Table No. 11, *Slope Ratios for Temporary Excavation*. Any loose utility trench backfill or other fill encountered in excavations will be less stable than the native soils. Temporary cuts encountering loose fill or loose dry sand may have to be constructed at a flatter gradient than presented in the following table:

Table No. 11, Slope Ratios for Temporary Excavation

Maximum Depth of Cut (feet)	Maximum Slope Ratio* (horizontal: vertical)
0 – 5	vertical
4 – 8	1:1
8 +	1.5:1

*Slope ratio assumed to be uniform from top to toe of slope.

Surfaces exposed in slope excavations should be kept moist but not saturated to retard raveling and sloughing during construction. Adequate provisions should be made to protect the slopes from erosion during periods of rainfall. Surcharge loads, including construction, should not be placed within five (5) feet of the unsupported trench edge. The above maximum slopes are based on a maximum height of six (6) feet of stockpiled soils placed at least five (5) feet from the trench edge.

For steeper temporary construction slopes or deeper excavations, shoring should be provided by the contractor as necessary, to protect the workers in the excavation.

All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1987 and current amendments, and the Construction

Safety Act should be met. The soils exposed in cuts should be observed during excavation by the project's geotechnical consultant. If potentially unstable soil conditions are encountered, modifications of slope ratios for temporary cuts may be required.

If the excavation occurs near existing structures, special construction considerations would be required during excavation to protect these existing structures in place during construction. The proposed excavations should not cause loss of bearing and/or lateral supports of the existing structures, streets, sidewalks and storm drains.

10.3 Shoring Design

Temporary shoring may be required for the recommended excavation due to space limitations and property line boundaries and because of nearby existing structures or facilities and traffic loading. Temporary shoring may consist of the use of a trench box (where feasible), or conventional soldier piles and lagging. Shoring should ultimately be designed by a qualified structural engineer considering the recommendations below in their final design and others which are applicable.

Drilled excavations for soldier piles may require the use of drilling fluids to prevent caving and to maintain an opened hole for pile installation. Casing may be needed if granular earth material is located behind the existing retaining wall.

10.3.1 Cantilevered Shoring

Cantilevered shoring systems may include soldier piles with lagging to maintain temporary support of vertical wall excavations. Shoring design must consider the support of adjacent underground utilities and/or structures and should consider the effects of shoring deflection on supported improvements. Due to sandy nature of on-site soils, some caving during the drilling of soldier-pile borings should be anticipated. A soldier pile system will require continuous lagging to control caving and sloughing in the excavation between soldier piles.

Temporary cantilevered shoring should be designed to resist a lateral earth pressure equivalent to a fluid density of 45 pounds per cubic foot (pcf) for non-surcharged condition. This pressure is valid only for shoring retaining level ground.

In addition to the lateral earth pressure, surcharge pressures due to miscellaneous loads, such as soil stockpiles, vehicular traffic or construction equipment located adjacent to the shoring, should be included in the design of the shoring. A uniform lateral pressure of 100 psf should be included in the upper 10 feet of the shoring to account for normal vehicular and construction traffic within 10 feet of the trench excavation. Surcharge pressures from the existing structures should be added to the above earth pressures for surcharges within a horizontal distance less than or equal to the wall height. Surcharge coefficients of 50% of any uniform vertical surcharge should be added as a horizontal earth pressure for shoring design. All shoring should be designed and installed in accordance with state and federal safety regulations.

The minimum embedment depth for piles is ten (10) feet from the lowest adjacent grade into firm alluvium, below the bottom of the excavation. Vertical skin friction against soldier piles for may be taken as 150 psf. Fixity may be assumed at two (2) feet below the excavation into firm native alluvium or bedrock. For the design of soldier piles spaced at least 3.0 diameters on-center, the passive resistance of the soils adjacent to the piles may be assumed to be 180 psf per foot of embedment depth. Soldier pile members placed in drilled holes should be properly backfilled with a sand/cement slurry or lean concrete in order to develop the required passive resistance.

Caving soils should be anticipated between the piles. To limit local sloughing, caving soils can be supported by continuous lagging or guniting. The lagging between the soldier piles may consist of pressure-treated wood members or solid steel sheets. In our opinion, steel sheeting is expected to be more expedient than wood lagging to install. Although soldier piles and any bracing used should be designed for the full-anticipated earth pressures and surcharge pressures, the pressures on the lagging are less because of the effect of arching between the soldier piles. Accordingly, the lagging between the piles may be designed for a nominal pressure of up to a maximum of 350 psf. All lumber to be left in the ground should be treated in accordance with Section 204-2 of the "Standard Specifications for Public Works Construction" (Latest Edition).

10.3.2 Tie-Back Shoring

A tie-back soldier-pile shoring system may be used to maintain temporary support of deep vertical walled excavations. Braced or tied-back shoring, retaining a level ground surface, should be designed for a uniform pressure of 25H psf, where H is the height of the retained cut in feet.

Surcharge pressures should be added to this earth pressure for surcharges within a distance from the top of the shoring less than or equal to the shoring height. A surcharge coefficient of 50 percent of any uniform vertical surcharge should be added as a horizontal shoring pressure for braced shoring. A uniform lateral pressure of 100 psf should be included in the upper 10 feet of the shoring to account for normal vehicular and construction traffic within 10 feet of the trench excavation.

Tie-Backs: For design of tie-back shoring, it should be assumed that the potential wedge of failure is determined by a plane at 30 degrees from the vertical, through the bottom of the excavation. Tie-back anchors may be installed at angles of 15 to 40 degrees below a horizontal plane. Soil friction values, for estimating the allowable capacity of drilled friction anchors, may be computed using the following equation:

$q = 40\text{H}$; $q \leq 500$ *pounds-per-square-foot (psf)*

where:

H = average depth of anchor below ground surface, shown on
q = anchor surface area resistance, in psf (excluding tip),

Only the frictional resistance developed beyond the assumed failure plane should be included in the tie-back design for resisting lateral loads. After shoring/tie-back is no longer needed to support the excavation, stress should be carefully released and shoring system including tieback may be able to be left in place.

All shoring and tie-back should be designed by experienced California licensed Civil Engineer and installed by experienced contractors. Shoring/tie-back design should also be reviewed by a geotechnical consultant to verify the soil parameters used in the design are in conformance with geotechnical report. All applicable requirements of the California Construction and General Industry Safety Orders, the Occupational Safety and Health Act of 1987 and current amendments, and the Construction Safety Act should be met. The soils exposed in cuts should be observed during excavation by a competent person employed by the contractor. If potentially unstable soil conditions are encountered, modifications of slope ratios for temporary cuts may be required.

It is recommended that Converse review plans and specifications for proposed shoring and that a Converse representative observes the installation of shoring. A licensed surveyor should be retained to establish monuments on shoring and the surrounding ground prior to excavation. Such monuments should be monitored for horizontal and vertical movement during construction. Results of the monitoring program should be provided immediately to the project Structural (shoring) Engineer and Converse for review and evaluation. Adjacent building elements should be photo-documented prior to construction.

10.4 Slot Cut Recommendations

Temporary excavations during possible improvements should not extend below a 1:1 horizontal:vertical (H:V) plane extending beyond and down from the bottom of the existing foundations, utility lines or structures. The remedial grading excavations should not cause loss of bearing and/or lateral support for adjacent foundations, utilities or structures.

If remedial grading excavations extend below a 1:1 horizontal:vertical (H:V) plane extending beyond and down from the bottom of adjacent off-site utility lines or structure foundations, shoring or slot cutting shall be employed. The ABC slot cutting method for over-excavation could be a possible option as an alternative to shoring for excavation less than 8 feet in width and depth or with cohesive soils. In general, for structures it is not recommended for slot cutting if the height of excavation exceeds more than 8 feet or into sandy soils and with surcharging load. Backfill should be accomplished in the shortest period of time possible and in alternating sections.

11.0 CLOSURE

The findings and recommendations of this report were prepared in accordance with generally accepted professional engineering and engineering geologic principles and practice. We make no other warranty, either expressed or implied. Our conclusions and recommendations are based on the results of the field and laboratory investigations, combined with an interpolation and extrapolation of soil conditions between and beyond boring locations. If conditions encountered during construction appear to be different from those shown by the borings, this office should be notified.

Design recommendations given in this report are based on the assumption that the earthwork and site grading recommendations contained in this report are implemented. Additional consultation may be prudent to interpret Converse's findings for contractors, or to possibly refine these recommendations based upon the review of the final site grading and actual site conditions encountered during construction. If the scope of the project changes, if project completion is to be delayed, or if the report is to be used for another purpose, this office should be consulted.

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Appendix A

Field Exploration

APPENDIX A: FIELD EXPLORATION

Field exploration included a site reconnaissance and subsurface exploration program. During the site reconnaissance, the surface conditions were noted, and the approximate locations of the borings were determined. The exploratory borings were approximately located using existing boundary and other features as a guide and should be considered accurate only to the degree implied by the method used. The various field study methods performed are discussed below.

Exploratory Borings

Nineteen (19) exploratory borings (BH-1 through BH-18 and PT-5) were drilled within the project site on April 2nd and 3rd, 2019 using a truck mounted drill rig with an 8-inch diameter hollow stem auger to depths ranging from 6.5 feet to 51.5 feet below the existing ground surface (bgs). Each boring was visually logged by a Converse engineer and sampled at regular intervals and at changes in subsurface soils. Detailed descriptions of the field exploration and sampling program are presented in Appendix A, *Field Exploration*.

Ring samples of the subsurface materials were obtained at frequent intervals in the exploratory borings using a drive sampler (2.4-inches inside diameter and 3.0-inches outside diameter) lined with sample rings. The steel ring sampler was driven into the bottom of the borehole with successive drops of a 140-pound driving weight falling 30 inches, using an automatic hammer. Samples are retained in brass rings (2.4-inches inside diameter and 1.0-inch in height). The central portion of the samples were retained and carefully sealed in waterproof plastic containers for shipment to the Converse laboratory. Blow counts for each sample interval are presented on the logs of borings. Bulk samples of typical soil types were also obtained.

Standard Penetration Test (SPT) was also performed using a standard split-barrel sampler (1.4-inches inside diameter and 2.0-inches outside diameter). The mechanically driven hammer for the SPT sampler was 140 pounds, failing 30 inches for each blow. The recorded blow counts for every six inches for a total of 1.5 feet of sampler penetration are shown on the Logs of Borings in the “BLOWS" column. The standard penetration test was performed in accordance with the ASTM Standard D1586 test method.

Percolation test was performed at PT-5 to the depth of 10 to 20 feet below ground level by using Boring Percolation Testing Procedure.

It should be noted that the exact depths at which material changes occur cannot always be established accurately. Changes in material conditions that occur between driven samples are indicated in the logs at the top of the next drive sample. A key to soil symbols and terms is presented as Drawing No. A-1, *Soil Classification Chart*. The log of the exploratory boring is presented in Drawing Nos. A-2 through A-20, *Log of Borings*.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	GRAVEL AND GRAVELLY SOILS MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	CLEAN GRAVELS (LITTLE OR NO FINES)		GW	WELL-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
				GP	POORLY-GRADED GRAVELS, GRAVEL - SAND MIXTURES, LITTLE OR NO FINES
		GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
	SAND AND SANDY SOILS MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	CLEAN SANDS (LITTLE OR NO FINES)		SW	WELL-GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
				SP	POORLY-GRADED SANDS, GRAVELLY SAND, LITTLE OR NO FINES
		SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	SILTS AND CLAYS	LIQUID LIMIT LESS THAN 50		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
	SILTS AND CLAYS	LIQUID LIMIT GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SAND OR SILTY SOILS
				CH	INORGANIC CLAYS OF HIGH PLASTICITY
				OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS				PT	PEAT, HUMUS, SWAMP SOILS WITH HIGH ORGANIC CONTENTS

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

BORING LOG SYMBOLS

SAMPLE TYPE

STANDARD PENETRATION TEST
Split barrel sampler in accordance with ASTM D-1586-84 Standard Test Method

DRIVE SAMPLE 2.42" I.D. sampler.

DRIVE SAMPLE No recovery

BULK SAMPLE

GRAB SAMPLE

GROUNDWATER WHILE DRILLING

GROUNDWATER AFTER DRILLING

LABORATORY TESTING ABBREVIATIONS

TEST TYPE
(Results shown in Appendix B)

CLASSIFICATION

Plasticity	pi
Grain Size Analysis	ma
Passing No. 200 Sieve	wa
Sand Equivalent	se
Expansion Index	ei
Compaction Curve	max
Hydrometer	h

STRENGTH

Pocket Penetrometer	p
Direct Shear	ds
Direct Shear (single point)	ds*
Unconfined Compression	uc
Triaxial Compression	tx
Vane Shear	vs
Consolidation	c
Collapse Test	col
Resistance (R) Value	r
Chemical Analysis	ca
Electrical Resistivity	er

UNIFIED SOIL CLASSIFICATION AND KEY TO BORING LOG SYMBOLS

Converse Consultants

Project Name
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745

Project No. **18-31-330-02**

Figure No. **A-1**

Log of Boring No. BH1

Dates Drilled: 4/2/2019 Logged by: RAM Checked By: MBS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 408 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
0		**FILL (Af):** **SILTY CLAY (CL):** dry, light brown.		X				
5		**ALLUVIUM (Qal):** **SILTY CLAY (CL):** with gravels and weathered lithic fragments, dry, light brown.	X		8/11/13	17	106	ds
10			X		3/4/4			
15			X		11/33/30	15	115	
17		**SANDY SILT (ML):** with gravels and weathered lithic fragments, dry, brown and gray.						
20			X		10/16/21			

End of boring at 21.5 feet. Groundwater was not encountered. Borehole was backfilled with cement grout on 4/2/19.

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Project Name
Wedgeworth Elementary School
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Project No. 18-31-330-02

Figure No. A-2

Log of Boring No. BH2

Dates Drilled: 4/2/2019 Logged by: RAM Checked By: MBS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 408 Depth to Water (ft): 39

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
0		**FILL (Af):** **SILTY CLAY (CL):** dark brown.		X				ma (fc=81%)
5		**ALLUVIUM (Qal):** **SILTY CLAY (CL):** moist, light brown.	X		3/5/8	20	101	
10			X		8/19/32	19	114	
14		**CLAYEY SAND (SC):** fine to coarse-grained, with gravels and weathered lithic fragments, dry, light brown.						
15			X		6/12/10			wa (fc=38%)
20			X		10/43/28	8	128	
24		**SILTY CLAY (CL):** with gravels, moist, light brown to brown.						
25			X		5/9/15			wa (fc=77%)
30			X		5/13/19	19	111	

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Project Name
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745

Project No. **18-31-330-02**

Figure No. **A-3a**

Log of Boring No. BH2

Dates Drilled: 4/2/2019 Logged by: RAM Checked By: MBS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 408 Depth to Water (ft): 39

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		SILTY CLAY (CL): soft, light brown.			2/3/6			wa (fc=79%)
		-groundwater encountered.						
40		**SILTY CLAY (CL):** soft, wet, brown.			3/6/7	24	103	
45					2/4/6			wa (fc=82%)
		SILTY SAND (SM): fine to coarse-grained, wet, light brown to gray.						
50					3/7/9			
		End of boring at 51.5 feet. Groundwater was encountered at 39 feet bgs. Borehole was backfilled with cement grout on 4/2/19.						

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Project Name
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745

Project No. **18-31-330-02**

Figure No. **A-3b**

Log of Boring No. BH3

Dates Drilled: 4/3/2019 Logged by: RAM Checked By: MBS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 407 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		FILL (Af): **SILTY CLAY (CL):** brown.		X				max
5		**ALLUVIUM (Qal):** **SILTY CLAY (CL):** moist, light brown.	X		3/7/9	23	98	
10			X		2/4/6			
		End of boring at 11.5 feet. Groundwater was not encountered. Borehole was backfilled with soil cuttings and loosely compacted on 4/3/19.						

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Project Name
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Hacienda Heights, California 91745

Project No. **18-31-330-02**

Figure No. **A-4**

Log of Boring No. BH4

Dates Drilled: 4/3/2019 Logged by: RAM Checked By: MBS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 407 Depth to Water (ft): NOT ENCOUNTERED

SUMMARY OF SUBSURFACE CONDITIONS

This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.

Depth (ft)	Graphic Log	Summary of Subsurface Conditions	Samples: Drive	Samples: Bulk	Blows/6"	Moisture (%)	Dry Unit Wt. (pcf)	Other
0		**FILL (Af):** **SILTY CLAY (CL):** with some gravel, brown.		X				ei
5			X		3/6/7	23	98	
		ALLUVIUM (Qal): **SILTY CLAY (CL):** moist, light brown.						
10			X		7/22/32	16	114	
15			X		5/8/11			
20		-light brown to brown	X		5/15/24	17	107	
25		-soft	X		2/5/8			

End of boring at 26.5 feet. Groundwater was not encountered. Borehole was backfilled with cement grout on 4/3/19.

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Project Name
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745

Project No. 18-31-330-02

Figure No. A-5

Log of Boring No. BH5

Dates Drilled: 4/3/2019 Logged by: RAM Checked By: MBS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 406 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		FILL (Af): **SILTY CLAY (CL):** with some gravel, dark brown to brown.						
5					4/8/11	20	109	
		ALLUVIUM (Qal): **SILTY CLAY (CL):** dry, light brown.						
10					8/11/11			
		End of boring at 11.5 feet. Groundwater was not encountered. Borehole was backfilled with soil cuttings and loosely compacted on 4/3/19.						

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Project Name
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Hacienda Heights, California 91745

Project No. **18-31-330-02**

Figure No. **A-6**

Log of Boring No. BH6

Dates Drilled: 4/3/2019 Logged by: RAM Checked By: MBS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 406 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
0		**FILL (Af):** **SILTY CLAY (CL):** trace gravels, dark brown.		X				
5			X		4/7/11	21	105	
6		**ALLUVIUM (Qal):** **SILTY CLAY (CL):** with gravels, moist, dark brown to light brown.						
10			X		7/31/46	11	113	
15			X		6/11/12			
18		**SANDY CLAY (CL):** with gravels and lithic fragments, -light brown to brown.						
20			X		7/22/22	5	109	
25		-hard	X		14/34/29			

End of boring at 26.5 feet. Groundwater was not encountered. Borehole was backfilled with cement grout on 4/3/19.

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Project Name
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745

Project No. **18-31-330-02**

Figure No. **A-7**

Project ID: 18-31-330-02.GPJ; Template: LOG

Log of Boring No. BH7

Dates Drilled: 4/3/2019 | Logged by: RAM | Checked By: MBS

Equipment: 8" HOLLOW STEM AUGER | Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 404 | Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		FILL (Af): **SILTY CLAY (CL):** with gravel, topped with 3" of CAB, brown.						
5					5/8/11	20	104	ds
		ALLUVIUM (Qal): **SILTY CLAY (CL):** with gravel, moist, brown.						
10					4/7/13			
		End of boring at 11.5 feet. Groundwater was not encountered. Borehole was backfilled with soil cuttings and loosely compacted on 4/3/19.						

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Project Name
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745

Project No. 18-31-330-02

Figure No. A-8

Log of Boring No. BH8

Dates Drilled: 4/3/2019 Logged by: RAM Checked By: MBS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 404 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		FILL (Af): **SILTY CLAY (CL):** trace gravels, topped with 3" CAB and silty sand, brown.						
5					9/10/11	16	113	c
		ALLUVIUM (Qal): **CLAY (CL):** with trace silt, moist, brown.						
10		-light brown			7/16/29	14	115	
15					5/7/10			
		SILTY CLAY (CL): moist, light brown.						
20					4/11/15	19	100	
25		-hard			3/5/10			
		End of boring at 26.5 feet. Groundwater was not encountered. Borehole was backfilled with cement grout on 4/3/19.						

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Project Name
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745

Project No. 18-31-330-02

Figure No. A-9

Log of Boring No. BH9

Dates Drilled: 4/2/2019 Logged by: RAM Checked By: MBS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 404 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		FILL (Af): **SILTY CLAY (CL):** moist, Topped with 3" CAB and SiltySand, dark brown.						
					5/9/13	17	108	
5		**ALLUVIUM (Qal):** **SILTY CLAY (CL):** dry, light brown.			4/5/8	18	109	
10					5/13/22	16	112	
		End of boring at 11.5 feet. Groundwater was not encountered. Borehole was backfilled with soil cuttings and loosely compacted on 4/2/19.						

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Project Name
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745

Project No. **18-31-330-02**

Figure No. **A-10**

Log of Boring No. BH10

Dates Drilled: 4/2/2019 Logged by: RAM Checked By: MBS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 402 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		FILL (Af): **SILTY CLAY (CL):** moist, dark brown.						
					3/5/7	19	105	
5					3/7/8	18	111	
		ALLUVIUM (Qal): **SILTY CLAY (CL):** light brown.						
10					7/13/24	14	118	
		End of boring at 11.5 feet. Groundwater was not encountered. Borehole was backfilled with soil cuttings and loosely compacted on 4/2/19.						

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Project Name
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745

Project No. **18-31-330-02**

Figure No. **A-11**

Log of Boring No. BH11

Dates Drilled: 4/2/2019 | Logged by: RAM | Checked By: MBS

Equipment: 8" HOLLOW STEM AUGER | Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 402 | Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		FILL (Af): **SILTY CLAY (CL):** wet, dark brown.						
					4/7/11	21	99	
5					3/6/8	17	102	
		ALLUVIUM (Qal): **SILTY CLAY (CL):** dry, light brown.						
10					13/23/31	12	116	
		End of boring at 11.5 feet. Groundwater was not encountered. Borehole was backfilled with soil cuttings and loosely compacted on 4/2/19.						

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Project Name
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745

Project No. **18-31-330-02**

Figure No. **A-12**

Log of Boring No. BH12

Dates Drilled: 4/3/2019 | Logged by: RAM | Checked By: MBS

Equipment: 8" HOLLOW STEM AUGER | Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 405 | Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		<u>FILL (Af):</u> **SILTY CLAY (CL):** moist, dark brown.						
					4/11/12	19	106	
5					4/6/7	19	105	
		End of boring at 6.5 feet. Groundwater was not encountered. Borehole was backfilled with soil cuttings and loosely compacted on 4/3/19.						

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Project Name
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745

Project No. **18-31-330-02**

Figure No. **A-13**

Log of Boring No. BH13

Dates Drilled: 4/3/2019 Logged by: RAM Checked By: MBS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 405 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		FILL (Af): **SILTY CLAY (CL):** moist, dark brown to brown.						
					5/9/12	19	105	
5					4/8/10	19	100	
		ALLUVIUM (Qal): **SANDY CLAY (CL):** with gravel, light brown.						
10					7/17/17	16	109	
		End of boring at 11.5 feet. Groundwater was not encountered. Borehole was backfilled with soil cuttings and loosely compacted on 4/3/19.						

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Project Name
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745

Project No. 18-31-330-02

Figure No. A-14

Log of Boring No. BH14

Dates Drilled: 4/3/2019 Logged by: RAM Checked By: MBS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 408 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
0		**FILL (Af):** **SILTY CLAY (CL):** moist, dark brown.		X				
5		**ALLUVIUM (Qal):** **SILTY CLAY (CL):** brown.	X		4/7/12	17	111	c
10			X		7/17/17	15	113	
		End of boring at 11.5 feet. Groundwater was not encountered. Borehole was backfilled with soil cuttings and loosely compacted on 4/3/19.						

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Project Name
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745

Project No. **18-31-330-02**

Figure No. **A-15**

Log of Boring No. BH15

Dates Drilled: 4/2/2019 | Logged by: RAM | Checked By: MBS

Equipment: 8" HOLLOW STEM AUGER | Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 408 | Depth to Water (ft): 37

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		FILL (Af): **SILTY CLAY (CL):** with gravels, brown.		X				
5			X		9/12/15	8	117	pi
		ALLUVIUM (Qal): **SANDY CLAY (CL):** with silt, dry, light brown.						
10			X		8/14/14	6	120	
15		-moist	X		6/16/24	11	119	
		CLAY (CL): some silt, dark brown.						
20			X		5/10/15			
25		-moist	X		6/13/21	14	114	
30			X		3/7/11			

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Project Name
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745

Project No. **18-31-330-02**

Figure No. **A-16a**

Log of Boring No. BH15

Dates Drilled: 4/2/2019 | Logged by: RAM | Checked By: MBS

Equipment: 8" HOLLOW STEM AUGER | Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 408 | Depth to Water (ft): 37

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		CLAY (CL): some silt, soft, moist, dark brown.			5/9/11	20	106	
		-groundwater encountered.						
40		-wet, brown			1/4/5			
		CLAYEY SAND (SC): fine to coarse-grained, wet, brown.						
45					10/23/28	16	114	
		CLAY (CL): some silt, brown.						
50					3/3/7			
		End of boring at 51.5 feet. Groundwater was encountered at 39 feet bgs. Borehole was backfilled with cement grout on 4/2/19.						

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Project Name
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745

Project No. **18-31-330-02**

Figure No. **A-16b**

Log of Boring No. BH16

Dates Drilled: 4/3/2019 Logged by: RAM Checked By: MBS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 406 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		FILL (Af): **SILTY CLAY (CL):** moist, brown.						r
					9/8/9	19	104	
5					4/9/11	16	107	
		ALLUVIUM (Qal): **SILTY CLAY (CL):** brown to light brown.						
10					8/19/13			
		End of boring at 11.5 feet. Groundwater was not encountered. Borehole was backfilled with soil cuttings and loosely compacted on 4/3/19.						

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Project Name
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745

Project No. **18-31-330-02**

Figure No. **A-17**

Log of Boring No. BH17

Dates Drilled: 4/2/2019 Logged by: RAM Checked By: MBS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 404 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		FILL (Af): **SILTY CLAY (CL):** moist, topped with 3" CAB, dark brown to brown.						ma (fc=80%)
					6/14/20	18	111	
5		-light brown			8/8/7	13	104	
		ALLUVIUM (Qal): **SILTY CLAY (CL):** dark brown to light brown.						
10					5/10/20	10	118	
15		-light brown			9/14/21	14	116	
20		**SILTY SAND (SM):** fine to coarse-grained, with gravel and lithic fragments,.			12/22/10			ma (fc=25%)
		End of boring at 21.5 feet. Groundwater was not encountered. Borehole was backfilled with cement grout on 4/2/19.						

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Project Name
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745

Project No. **18-31-330-02**

Figure No. **A-18**

Log of Boring No. BH18

Dates Drilled: 4/3/2019 Logged by: RAM Checked By: MBS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 408 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES: DRIVE	SAMPLES: BULK	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
0		**FILL (Af):** **SILTY CLAY (CL):** with gravel, moist, dark brown.		X				
5		**ALLUVIUM (Qal):** **SANDY CLAY (CL):** moist, dark brown to brown.	X		3/6/8	17	106	
10			X		4/12/13			
		End of boring at 11.5 feet. Groundwater was not encountered. Borehole was backfilled with soil cuttings and loosely compacted on 4/3/19.						

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Project Name
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745

Project No. **18-31-330-02**

Figure No. **A-19**

Log of Boring No. PT5

Dates Drilled: 4/2/2019 Logged by: RAM Checked By: MBS

Equipment: 8" HOLLOW STEM AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 402 Depth to Water (ft): NOT ENCOUNTERED

Depth (ft)	Graphic Log	SUMMARY OF SUBSURFACE CONDITIONS This log is part of the report prepared by Converse for this project and should be read together with the report. This summary applies only at the location of the boring and at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with the passage of time. The data presented is a simplification of actual conditions encountered.	SAMPLES DRIVE	SAMPLES BULK	BLOWS/6"	MOISTURE (%)	DRY UNIT WT. (pcf)	OTHER
		FILL (Af): **SILTY CLAY (CL):** moist, brown.						
					5/9/10	21	105	
5		-light brown			1/3/4	18	100	
		ALLUVIUM (Qal): **SILTY CLAY (CL):** moist, soft, light brown.						
					5/9/10	15	109	wa (fc=83%)
10					3/8/15	14	112	
		SILTY SAND (SM): fine to coarse-grained, with gravels, cobbles and weathered lithic fragments, dry, light brown.						
15					11/22/31			wa (fc=12%)
20					24/31/24	6	118	
		End of boring at 21.5 feet. Groundwater was not encountered. Percolation test performed for bottom 10 feet. Borehole was backfilled with cement grout on 4/2/19.						

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Project Name
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745

Project No. **18-31-330-02**

Figure No. **A-20**

Appendix B

Laboratory Testing Program

APPENDIX B: LABORATORY TESTING PROGRAM

Tests were conducted in our laboratory on representative soil samples for the purpose of classification and evaluation of their relevant physical characteristics and engineering properties. The amount and selection of tests were based on the geotechnical requirements of the project. Test results are presented herein and on the Logs of Borings in Appendix A, *Field Exploration*. The following is a summary of the laboratory tests conducted for this project.

Moisture Content and Dry Density

Results of moisture content and dry density tests performed on relatively undisturbed ring samples were used to aid in the classification of the soils and to provide quantitative measure of the *in-situ* dry density. Data obtained from this test provides qualitative information on strength and compressibility characteristics of site soils. For test results, see the Logs of Borings in Appendix A, *Field Exploration*.

Grain-Size Analysis

To assist in classification of soils, mechanical grain-size analysis was performed on two (2) selected samples. Testing was performed in general accordance with the ASTM Standard C136 test method. Grain-size curve is shown in Drawing No. B-1, *Grain Size Distribution Results*.

Percent Finer Than Sieve No. 200

The percent finer than sieve No. 200 tests were performed on seven (7) selected soil samples to aid in the classification of the on-site soils and to estimate other engineering parameters. Testing was performed in general accordance with the ASTM Standard D1140 test method. The test results are presented in the boring logs.

Table No. B-1, Summary of Percent Passing Sieve #200 Test Results

Boring No.	Depth (feet)	Soil Classification	Percent Passing Sieve No. 200
BH-2	15	Clayey Sand (SC)	38%
BH-2	25	Silty Clay (CL)	77%
BH-2	35	Silty Clay (CL)	79%
BH-2	45	Silty Clay (CL)	82%
BH-17	20	Silty Sand (SM)	25%
PT-5	7.5	Silty Clay (CL)	83%
PT-5	15	Silty Sand (SM)	12%

Maximum Dry Density Test

One (1) laboratory maximum dry density-moisture content relationship test was performed on a representative bulk sample of the upper 5 feet of soil material. The testing was conducted in accordance with ASTM Standard D1557 laboratory procedure. The test result is presented on Drawing No. B-2, *Moisture-Density Relationship Results*.

Consolidation Test

Consolidation tests were performed on two (2) relatively undisturbed samples. Data obtained from this test was used to evaluate the settlement characteristics of the foundation soils under load. Preparation for this test involved trimming the sample and placing the 1-inch high brass ring into the test apparatus, which contained porous stones, both top and bottom, to accommodate drainage during testing. Normal axial loads were applied to one end of the sample through the porous stones, and the resulting deflections were recorded at various time periods. The load was increased after the sample reached a reasonable state equilibrium. Normal loads were applied at a constant load-increment ratio, successive loads being generally twice the preceding load. The samples were tested at field and submerged conditions. The test results, including sample density and moisture content, are presented in Drawing No. B-3a and B-3b, *Consolidation Test Results*.

Direct Shear

Direct shear tests were performed on two (2) relatively undisturbed samples at soaked moisture conditions. For each test, three samples contained in brass sampler rings were placed, one at a time, directly into the test apparatus and subjected to a range of normal loads appropriate for the anticipated conditions. The samples were then sheared at a constant strain rate of 0.04 inch/minute. Shear deformation was recorded until a maximum of about 0.50-inch shear displacement was achieved. Ultimate strength was selected from the shear-stress deformation data and plotted to determine the shear strength parameters. For test data, including sample density and moisture content, see Drawing Nos. B-4a and B-4b, *Direct Shear Test Results,* and the following table:

Table No. B-2, Direct Shear Test Results

Boring No.	Depth (feet)	Soil Classification	Peak Strength Parameters	
			Friction Angle (degrees)	Cohesion (psf)
BH-1	5	Silty Clay (CL)	25	370
BH-7	5	Silty Clay (CL)	26	510

Atterberg Limits

Atterberg limits test was performed on one (1) sample to assist the classification of the soil and fill materials according to ASTM Standard D4318 test method. The test results are presented in the following table and on Drawing No. B-5, *Atterberg Limit Test Results*.

Table No. B-3, Atterberg Limit Test Results

Boring No.	Depth (feet)	Soil Classification	Liquid Limit (%)	Plastic Limit (%)	Plastic Index (%)
BH-15	5	Silty Clay (CL)	46	23	23

Expansion Index Test

One (1) representative bulk sample was tested to evaluate the expansion potential of material encountered at the site. The test was conducted in accordance with ASTM D4829 Standard. Test results are presented in the following table:

Table No. B-4, Expansion Index Test Result

Boring No.	Depth (feet)	Soil Description	Expansion Index	Expansion Potential
BH-4	0-5	Silty Clay (CL)	13	Low

R- Value

One (1) representative bulk soil sample was tested for resistance value (R-value) in accordance with ASTM D2844 Standard. This test is designed to provide a relative measure of soil strength for use in pavement design. The test results are shown in the following table:

Table No. B-5, R-value Test Result

Boring No.	Depth (feet)	Soil Classification	Measured R-value
BH-16	1-5	Silty Clay (CL)	35

Soil Corrosivity

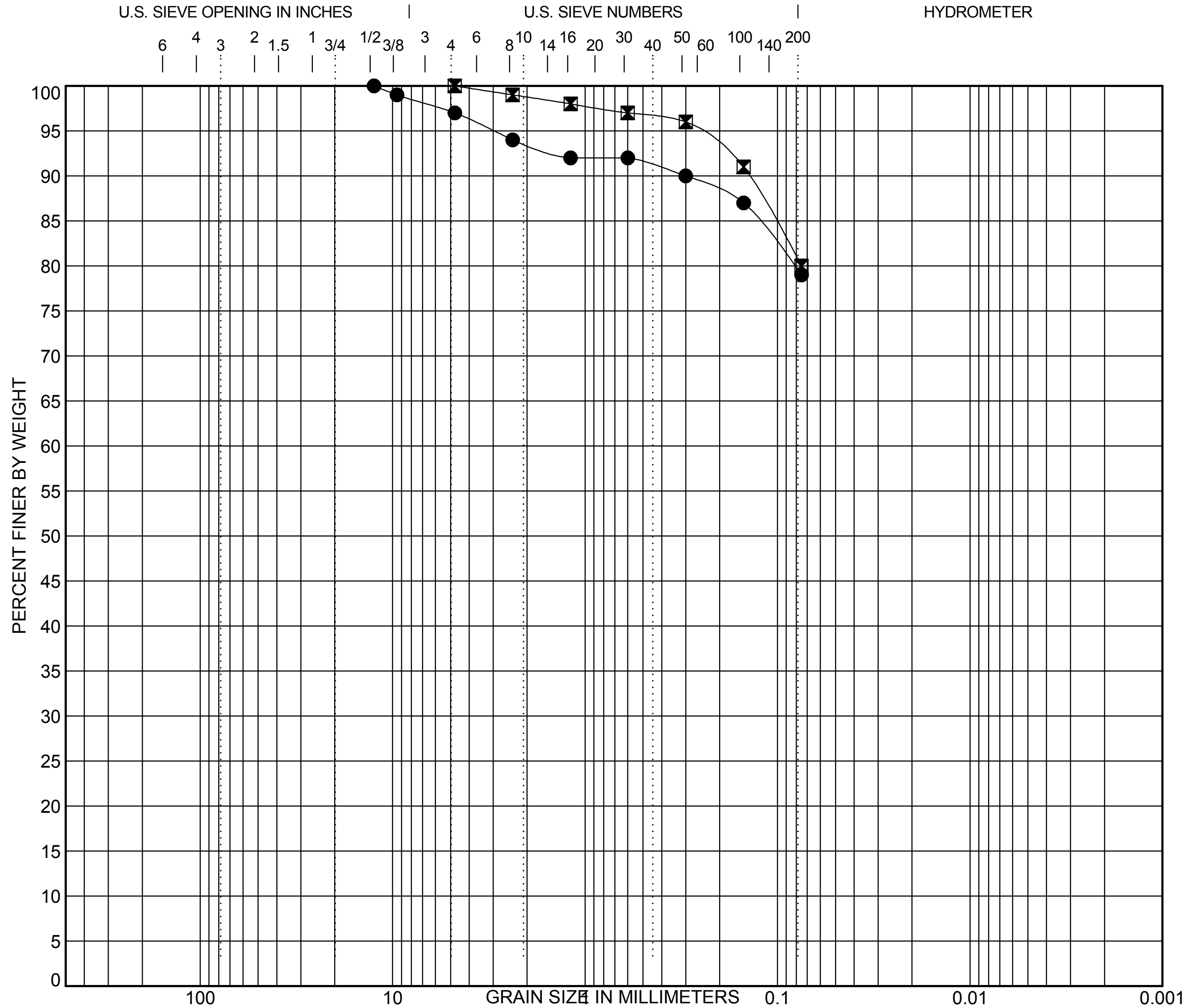
Two (2) representative soil samples were tested to determine minimum electrical resistivity, pH, and chemical content, including chloride concentrations, and soluble sulfate. The purpose of these tests is to determine the corrosion potential of site soils when placed in contact with common construction materials. These tests were performed by EGL in Arcadia, California. The test results received from EGL are included in the following table:

Table No. B-6, Corrosivity Test Results

Boring No.	Sample Depth (feet)	pH (Caltrans 643)	Soluble Chlorides (Caltrans 422) ppm	Soluble Sulfate (Caltrans 417) (%)	Saturated Resistivity (Caltrans 532) Ohm-cm
BH-5	0-5	7.10	255	0.048	980
BH-12	0-5	7.62	555	0.080	770

Sample Storage

Soil samples presently stored in our laboratory will be discarded 30 days after the date of this report, unless this office receives a specific request to retain the samples for a longer period of time.



	Boring No.	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	**BH17**	**5**	**Silty Clay (CL)**					
⊠	**BH2**	**5**	**Silty Clay (CL)**					

	Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH17**	**5**	**12.5**				**3.0**	**18.0**	**79.0**	
⊠	**BH2**	**5**	**4.75**				**0.0**	**20.0**	**80.0**	

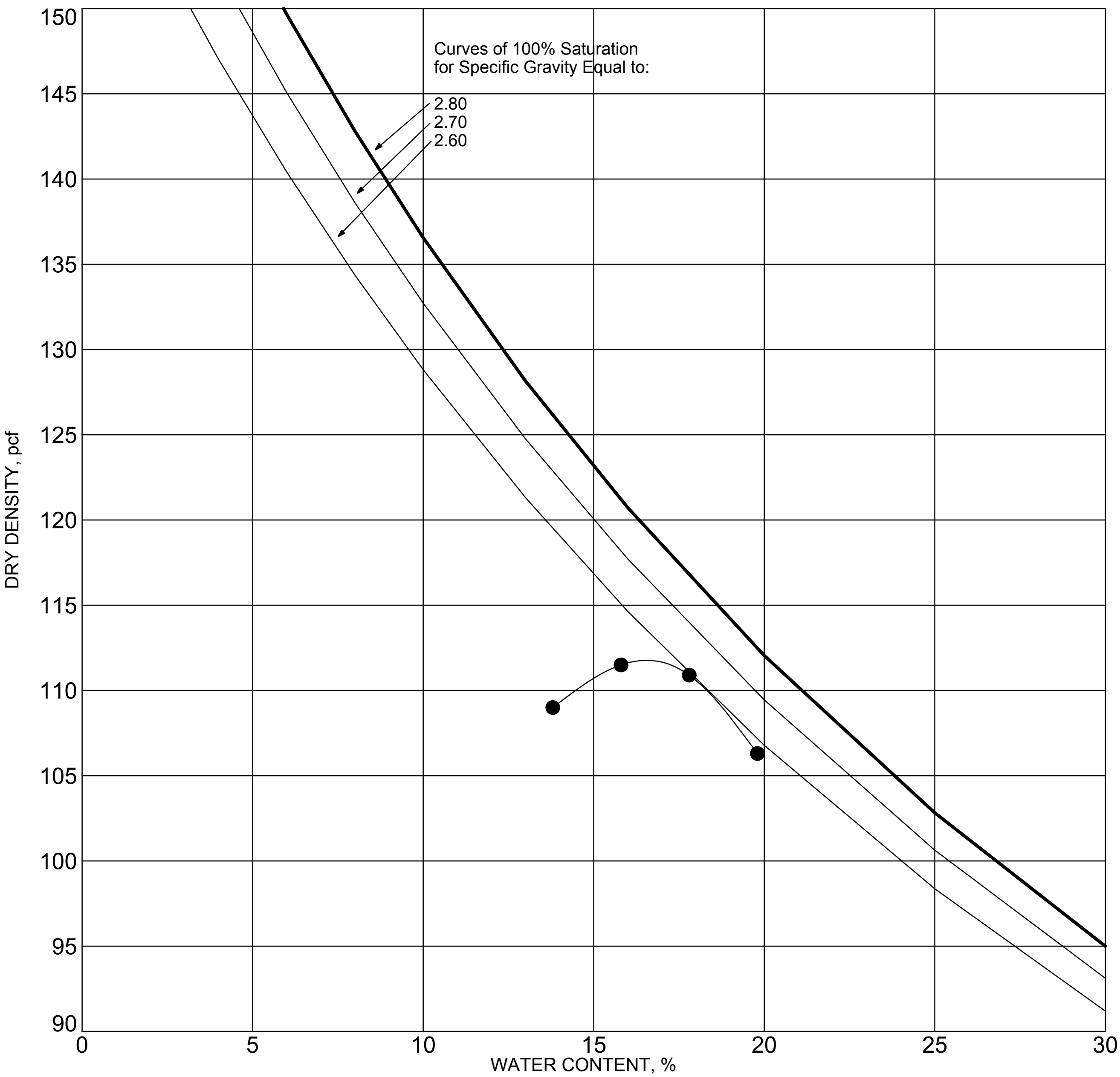
GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

Project Name
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745

Project No.
18-31-330-02

Figure No.
B-1



SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	BH3	5	Silty Clay (CL)	D1557 Method B	16.7	112.9

NOTE:

MOISTURE-DENSITY RELATIONSHIP RESULTS

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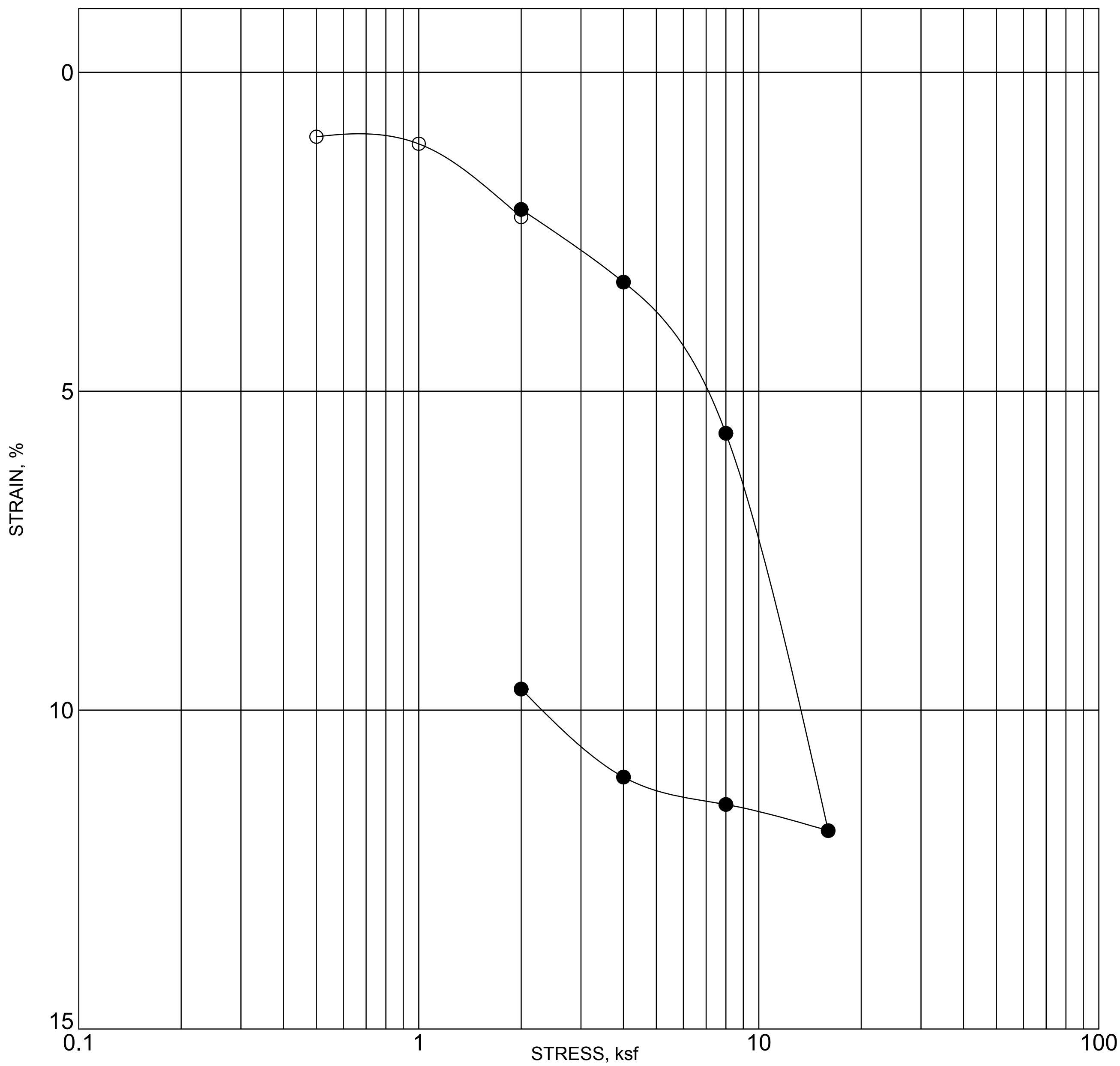
Project Name
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745

Project No.
18-31-330-02

Figure No.
B-2



BORING NO. :	BH8	DEPTH (ft) :	5
DESCRIPTION :	**Silty Clay (CL)**		

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	**10**	**120**	**71**	**0.512**
FINAL	**14**	**121**	**100**	**0.499**

NOTE: SOLID CIRCLES INDICATE READINGS AFTER ADDITION OF WATER

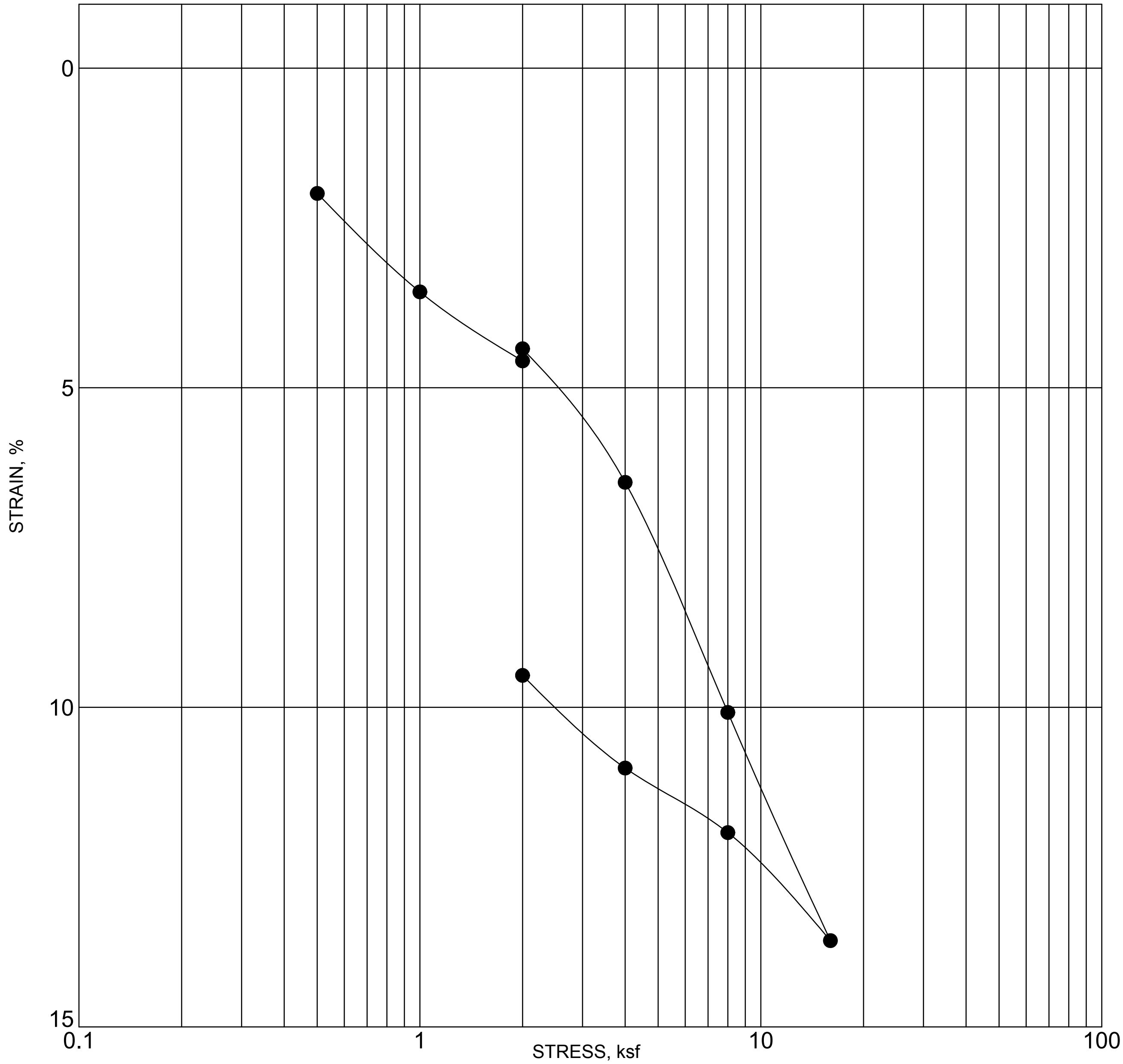
CONSOLIDATION TEST RESULTS

Converse Consultants

Project Name
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745

Project No.
18-31-330-02

Figure No.
B-3a



BORING NO. :	BH14	DEPTH (ft) :	5
DESCRIPTION :	Silty Clay (CL)		

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	**8**	**120**	**57**	**0.512**
FINAL	**14**	**120**	**100**	**0.512**

NOTE: SOLID CIRCLES INDICATE READINGS AFTER ADDITION OF WATER

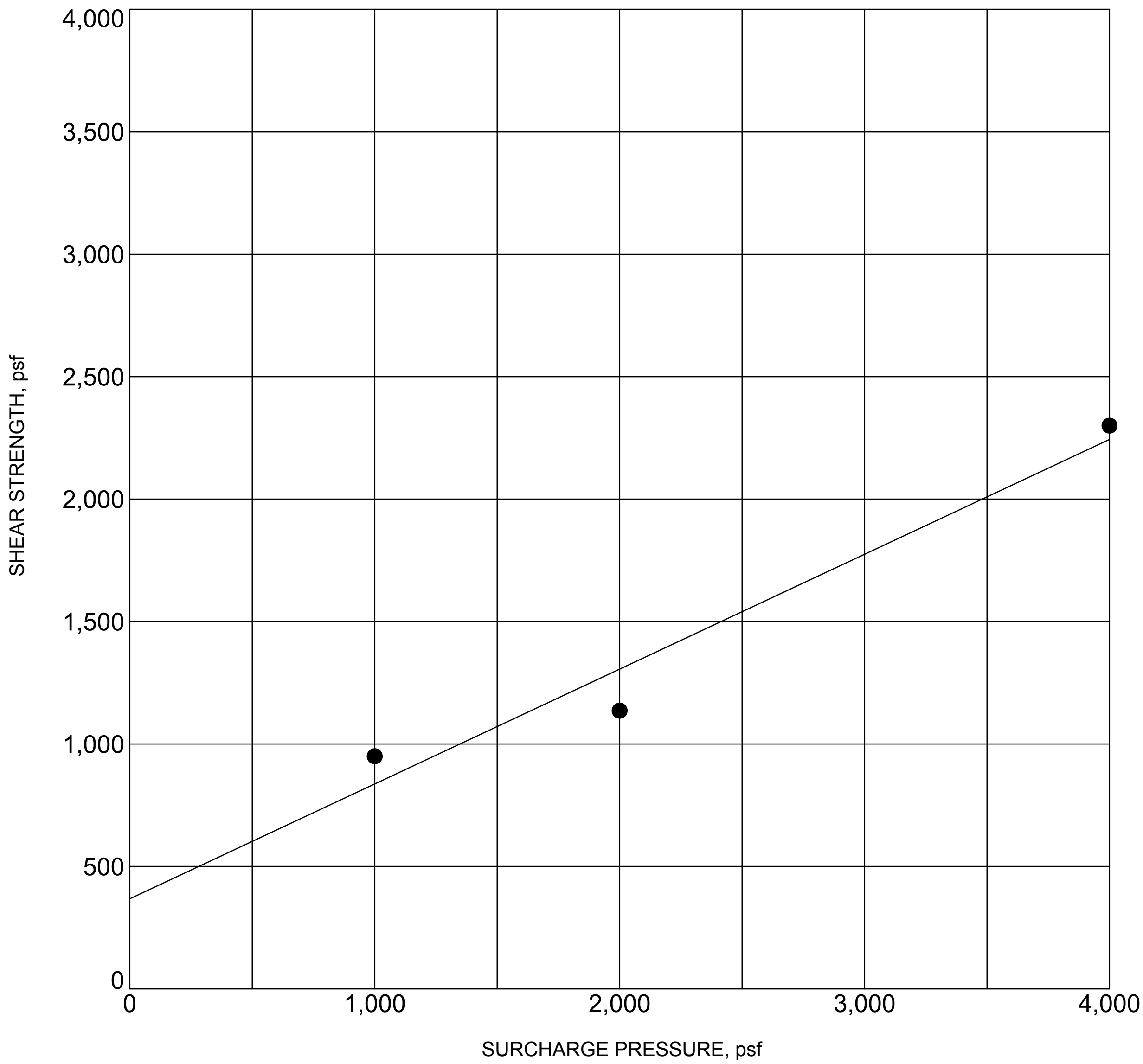
CONSOLIDATION TEST RESULTS

Converse Consultants

Project Name
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745

Project No. **18-31-330-02**

Figure No. **B-3b**



BORING NO. :	**BH1**	DEPTH (ft) :	**5**
DESCRIPTION :	**Silty Clay (CL)**		
COHESION (psf) :	**370**	FRICTION ANGLE (degrees):	**25**
MOISTURE CONTENT (%) :	**17.0**	DRY DENSITY (pcf) :	**106.9**

NOTE: Ultimate Strength.

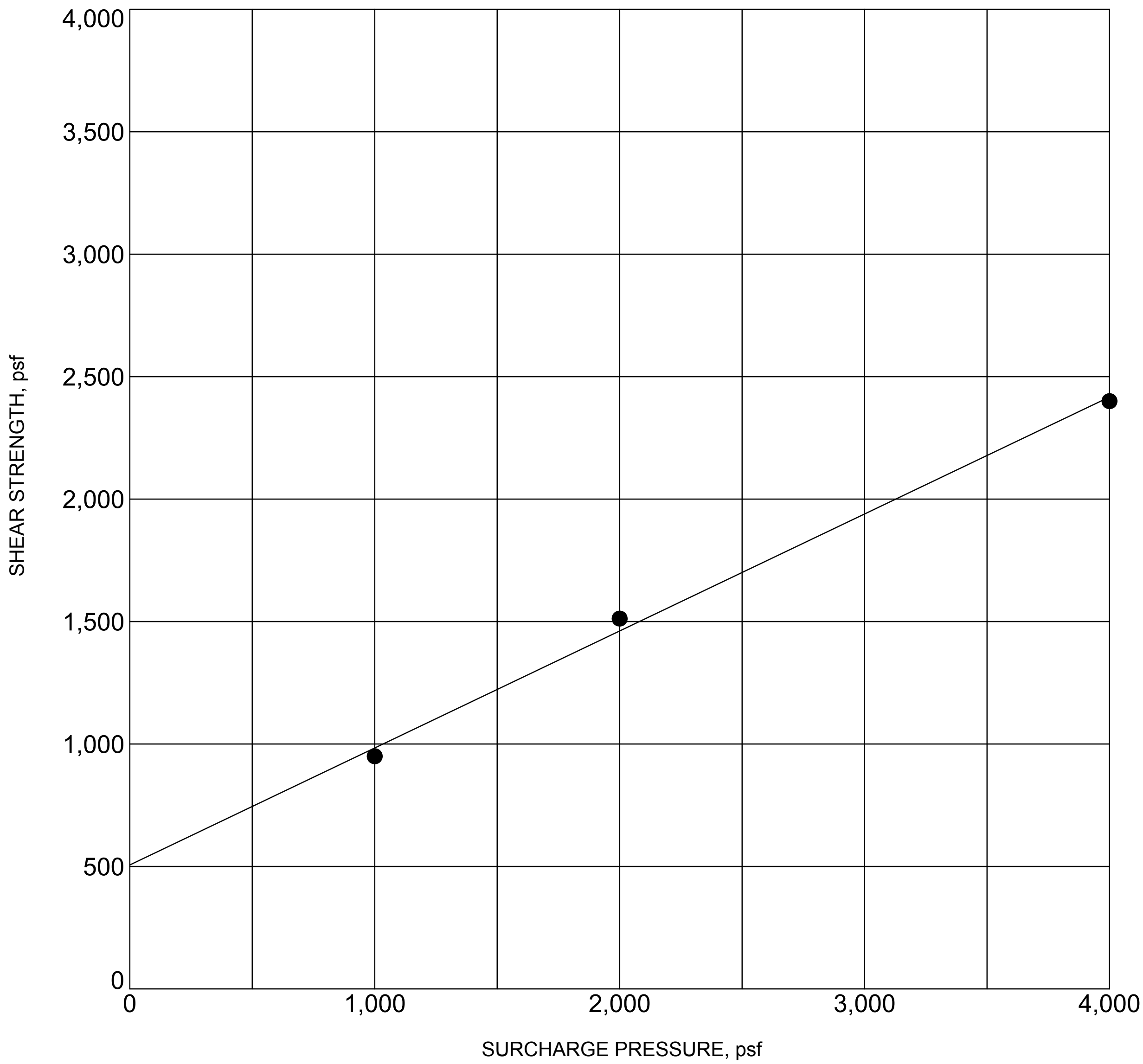
DIRECT SHEAR TEST RESULTS

Converse Consultants

Project Name
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745

Project No.
18-31-330-02

Figure No.
B-4a



BORING NO. :	**BH7**	DEPTH (ft) :	**5**
DESCRIPTION :	**Silty Clay(CL)**		
COHESION (psf) :	**510**	FRICTION ANGLE (degrees):	**26**
MOISTURE CONTENT (%) :	**19.8**	DRY DENSITY (pcf) :	**104.4**

NOTE: Ultimate Strength.

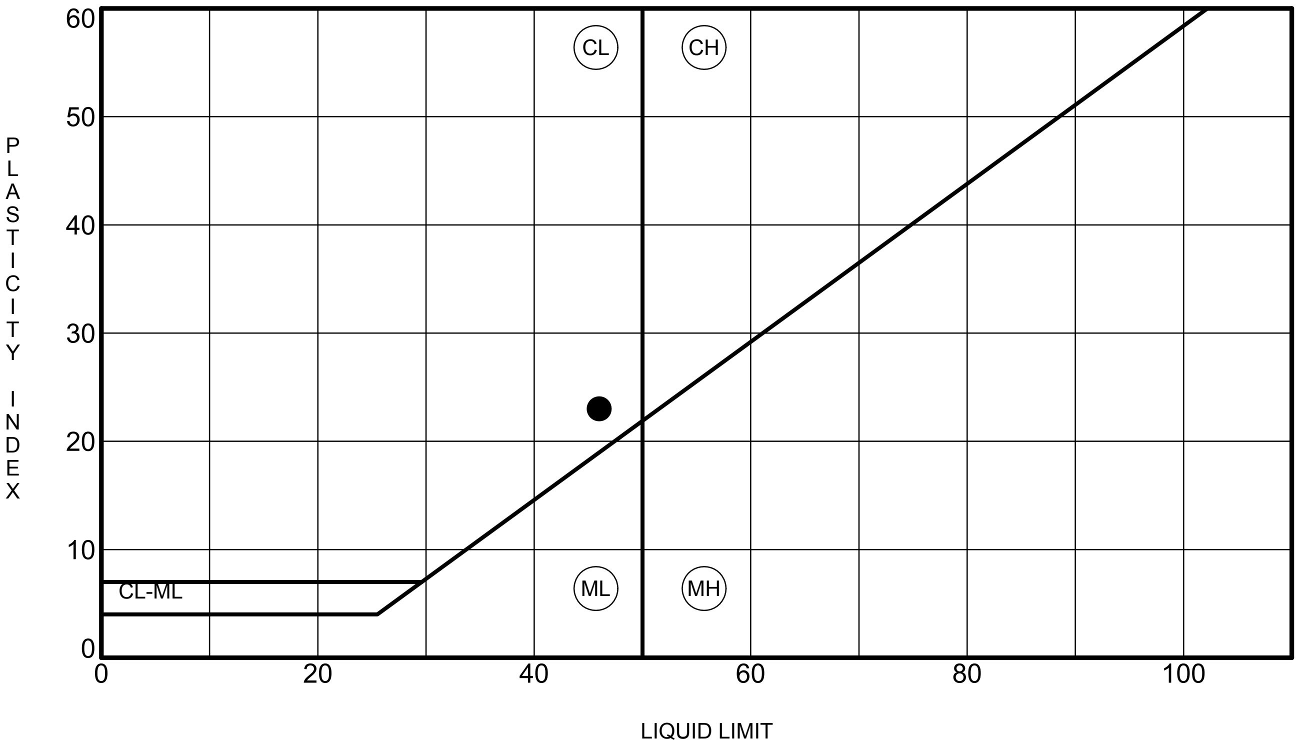
DIRECT SHEAR TEST RESULTS

Converse Consultants

Project Name
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745

Project No.
18-31-330-02

Figure No.
B-4b



Symbol	Boring No.	Depth (ft)	LL (%)	PL (%)	PI (%)	Description
●	**BH15**	**5**	**46**	**23**	**23**	Silty Clay(CL)

ATTERBERG LIMITS RESULTS

Converse Consultants

Project Name
Wedgeworth Elementary School
16949 Wedgeworth Drive
Hacienda Heights, California 91745

Project No.
18-31-330-02

Figure No.
B-5

Appendix C

Percolation Testing

APPENDIX C: PERCOLATION TESTING

Percolation testing was performed utilizing exploratory borings PT-5 on April 2nd, 2019. The continuous pre-soak falling-head test method for water percolation testing was utilized to evaluate soil infiltration rates of the native soils encountered between depths of 10 to 20 feet below the ground surface at the respective boring locations in accordance with Los Angeles County (2017), Administrative Manual--Guidelines for Design, Investigation, and Reporting Low Impact Development Storm Water Infiltration. The test location was prepared by placing a perforated 2-inch diameter PVC pipe surrounded by pea gravel after drilling and sampling. Water was filled to the ground surface to pre-soak prior to testing.

The borings were cased using a two-inch diameter perforated casing. Water was added to the bore hole until the water level was as near the ground surface as could be achieved and allowed to pre-soak for at least 4 hours if the water did not drain entirely within 30 minutes after filling the boring two (2) consecutive times. After pre-soak, water was added to the bore hole until the water level was as near ten (10) feet below the ground surface as could be achieved. The water level was measured to the nearest 1/8-inch. There were at least three (3) sets of measurements taken for each test and each set consisted of at least three (3) measurements. The results of the percolation tests are tabulated in the tables below:

Table No. C-1, Soil Boring Percolation Test Results

Boring No.	Depth of Test (feet)	Top Soil Types (USCS)	Average Percolation Rate (inches/hour)	Lowest Percolation Rate (inches/hour)
PT-5*	10–20 bgl	Silty Sand (SM)	0.85	0.62

*Percolation rate was obtained from an 8-inch diameter bore hole to a depth which shows in the next column (Depth of Test). The percolation rate may change with different well dimensions. The adjustment to the provided percolation rate to a well with different dimensions should be determined by the well designer.

In accordance with County of Los Angeles requirements, the minimum percolation rate for design of infiltration systems for storm water management is 0.3 inches per hour. It should be noted that per Los Angeles County Low Impact Development, Best Management Practices Guidelines, any planned infiltration systems should be at least 10 feet above historically highest groundwater levels. Review of the Seismic Hazard Zone Report for the La Habra Quadrangle (CDMG 1997) indicated the historically highest groundwater levels at depths of approximately 25 feet below ground surface. More recent groundwater level monitoring in local groundwater wells has shown depths to groundwater varies between approximately 27 and 35 feet below ground surface. The project Civil Engineer shall review the percolation rates presented for design of the proposed infiltration system. Additional details about drywell design and requirements can be found in the Low Impact Development Manual, County of Los Angeles Department of

Public Works, latest edition. The infiltration system should be properly maintained periodically to minimize sedimentation in the infiltration system.

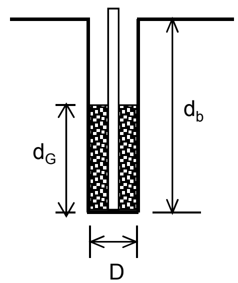
Table No. C-2, Infiltration Facility Setback Requirements per Los Angeles County

Setback from	Distance
Property lines and public right of way	5 feet
Any foundation	15 feet or within 1:1 plane drawn up from the bottom of foundation, whichever greater
Face of any slope	H/2, 5 feet minimum (H is height of slope)
Water wells used for drinking water	100 feet
Historically highest groundwater levels	10 feet above

Percolation Testing

Job Name: Wedgeworth ES, HLPUSD
Job No.: 18-31-330-02
Location: Inside the baseball field
Test Date: April 2, 2019

Test Boring No **PT-5**
Depth of Boring (d_b): 10.0 feet
Diameter of Boring (D): 0.67 feet
Test Performer: PA



Percolation Test was performed from 10 feet to 20 feet below ground level

Time of Testing			Water Level Measurement		Water Level Calculations				Percolation Rate Calculations		
Initial Time	Final Time	Time Interval	Initial depth to water	Final depth to water	Initial Height of water column	Final Height of water column	Drop in Height	Average height of water column	Pre-adjusted Percolation Rate	Reduction Factor	Adjusted Percolation Rate
T_i	T_f	ΔT	d_1	d_2	d_i	d_f	$\Delta d = d_i - d_f$	L_{ave}	$k_i = \Delta d / \Delta T$	R_f	$k = k_i / R_f$
		(hr)	(feet)	(feet)	(feet)	(feet)	(feet)	(feet)	(inch/hr)		(inch/hr)
Percolation Test											
11:00:00 AM	11:30:00 AM	0.50	0.00	1.20	10.00	8.80	1.20	9.40	28.80	29.1	0.99
11:30:00 AM	12:00:00 PM	0.50	1.20	2.30	8.80	7.70	1.10	8.25	26.40	25.6	1.03
12:00:00 PM	12:30:00 PM	0.50	2.30	3.30	7.70	6.70	1.00	7.20	24.00	22.5	1.07
12:30:00 PM	1:00:00 PM	0.50	0.00	1.10	10.00	8.90	1.10	9.45	26.40	29.2	0.90
1:00:00 PM	1:30:00 PM	0.50	1.10	2.00	8.90	8.00	0.90	8.45	21.60	26.2	0.82
1:30:00 PM	2:00:00 PM	0.50	2.00	2.80	8.00	7.20	0.80	7.60	19.20	23.7	0.81
2:00:00 PM	2:30:00 PM	0.50	0.00	0.90	10.00	9.10	0.90	9.55	21.60	29.5	0.73
2:30:00 PM	3:00:00 PM	0.50	0.90	1.60	9.10	8.40	0.70	8.75	16.80	27.1	0.62
3:00:00 PM	3:30:00 PM	0.50	1.60	2.30	8.40	7.70	0.70	8.05	16.80	25.0	0.67

Note: Reduction Factor, $R_f = (2 * d_i - \Delta d)/D + 1$

Lowest Perlcolaton Rate = 0.62 inch/hr
Average Percolation Rate = 0.85 inch/hr

Reference: Los Angeles County (2017). Administrative Manual - Guidelines for Design, Investigation, and Reporting Low Impact Development Storm Water Infiltration , 6/30/17.

Appendix D

Earthwork Specifications

APPENDIX D: EARTHWORK SPECIFICATIONS

Scope of Work

The work includes all labor, supplies and construction equipment required to construct the retaining wall in a good, workman-like manner, as shown on the drawings and herein specified. The major items of work covered in this section include the following:

- Site Inspection
- Authority of Geotechnical Engineer
- Site Clearing
- Excavations
- Preparation of Fill Areas
- Placement and Compaction of Fill
- Observation and Testing

Site Inspection

- The Contractor shall carefully examine the site and make all inspections necessary, in order to determine the full extent of the work required to make the completed work conform to the drawings and specifications. The Contractor shall satisfy himself as to the nature and location of the work, ground surface and the characteristics of equipment and facilities needed prior to and during prosecution of the work. The Contractor shall satisfy himself as to the character, quality, and quantity of surface and subsurface materials or obstacles to be encountered. Any inaccuracies or discrepancies between the actual field conditions and the drawings, or between the drawings and specifications must be brought to the Owner's attention in order to clarify the exact nature of the work to be performed.

- This G*eotechnical Study Report* by Converse Consultants may be used as a reference to the surface and subsurface conditions on this project. The information presented in this report is intended for use in design and is subject to confirmation of the conditions encountered during construction. The exploration logs and related information depict subsurface conditions only at the particular time and location designated on the boring logs. Subsurface conditions at other locations may differ from conditions encountered at the exploration locations. In addition, the passage of time may result in a change in subsurface conditions at the exploration locations. Any review of this information shall not relieve the Contractor from performing such independent investigation and evaluation to satisfy himself as to the nature of the surface and subsurface conditions to be encountered and the procedures to be used in performing his work.

Authority of the Geotechnical Engineer

- The Geotechnical Engineer will observe the placement of compacted fill and will take sufficient tests to evaluate the uniformity and degree of compaction of filled ground.

- As the Owner's representative, the Geotechnical Engineer will (a) have the authority to cause the removal and replacement of loose, soft, disturbed and other unsatisfactory soils and uncontrolled fill; (b) have the authority to approve the preparation of native ground to receive fill material; and (c) have the authority to approve or reject soils proposed for use in building areas.

- The Civil Engineer and/or Owner will decide all questions regarding (a) the interpretation of the drawings and specifications, (b) the acceptable fulfillment of the contract on the part of the Contractor and (c) the matters of compensation.

Site Clearing

- Clearing and grubbing shall consist of the removal from building areas to be graded of all existing structures, pavement, utilities, and vegetation.

- Organic and inorganic materials resulting from the clearing and grubbing operations shall be hauled away from the areas to be graded.

Excavations

- Based on observations made during our field explorations, the surficial soils can be excavated with conventional earthwork equipment.

Preparation of Fill Areas

- All organic material, organic soils, incompetent alluvium, undocumented fill soils and debris should be removed from the proposed building areas.

- In order to provide uniform support for the new structures, the minimum depth of over-excavation should be five (5) feet below the existing grade, or three (3) foot below proposed shallow foundations whichever is deeper. Deeper over-excavation will be needed if soft, yielding soils are exposed on the excavation bottom. The actual depth of removal should be determined based on observations made during grading. Over-excavation should extend a least two (2) feet beyond the limits of footings, or equal distance of over-excavation depth, whichever is greater, or as limited by the existing structures. Excavation activities should not disturb existing utilities, buildings, and remaining structures. Existing utilities should be removed and adequately capped at the project boundary line, or salvaged/rerouted as designed for sidewalks and flatwork area, at least the upper 24 inches of existing soils should be scarified and recompacted to at least 90 percent of compaction. Deeper over-excavation will be needed if soft, yielding soils are exposed on the excavation bottom. The excavation should be extended to at least 12 inches beyond the driveway and flatwork limit where space is permitted.

- The subgrade in all areas to receive fill shall be scarified to a minimum depth of six inches, the soil moisture adjusted within three (3) percent above optimum, and then compacted to at least 90 percent of the laboratory maximum dry density as determined by ASTM Standard D1557 test method.

- Compacted fill may be placed on native soils that have been properly scarified and re-compacted as discussed above.

- All areas to receive compacted fill will be observed and approved by the Geotechnical Engineer before the placement of fill.

Placement and Compaction of Fill

- Compacted fill placed for the support of footings, slabs-on-grade, exterior concrete flatwork, and driveways will be considered structural fill. Structural fill may consist of approved on-site soils or imported fill that meets the criteria indicated below.

- Fill consisting of selected on-site earth materials or imported soils approved by the Geotechnical Engineer shall be placed in layers on approved earth materials. Soils used as compacted structural fill shall have the following characteristics:

- All fill soil particles shall not exceed three (3) inches in nominal size and shall be free of organic matter and miscellaneous inorganic debris and inert rubble.

- Imported fill materials shall have an Expansion Index (EI) less than 20. All imported fill should be compacted to at least 90 percent of the laboratory maximum dry density (ASTM Standard D1557) at about to three percent above optimum moisture.

- Fill soils shall be evenly spread in maximum 8-inch lifts, watered or dried as necessary, mixed and compacted to at least the density specified below. The fill shall be placed and compacted on a horizontal plane, unless otherwise approved by the Geotechnical Engineer.

- All fill placed at the site shall be compacted to at least 90 percent of the laboratory maximum dry density as determined by ASTM Standard D1557 test method. The on-site soils shall be moisture conditioned at approximate three (3) percent above the optimum moisture content.

- Representative samples of materials being used, as compacted fill will be analyzed in the laboratory by the Geotechnical Engineer to obtain information on their physical properties. Maximum laboratory density of each soil type used in the compacted fill will be determined by the ASTM Standard D1557 compaction method.

- Fill materials shall not be placed, spread or compacted during unfavorable weather conditions. When site grading is interrupted by heavy rain, filling operations shall not resume until the Geotechnical Engineer approves the moisture and density conditions of the previously placed fill.

- It shall be the Grading Contractor's obligation to take all measures deemed necessary during grading to provide erosion control devices in order to protect slope areas and adjacent properties from storm damage and flood hazard originating on this project. It shall be the contractor's responsibility to maintain slopes in their as-graded form until all slopes are in satisfactory compliance with job specifications, all berms have been properly constructed, and all associated drainage devices meet the requirements of the Civil Engineer.

Trench Backfill

The following specifications are recommended to provide a basis for quality control during the placement of trench backfill.

- Trench excavations to receive backfill shall be free of trash, debris or other unsatisfactory materials at the time of backfill placement.

- Trench backfill shall be compacted to a minimum relative compaction of 90 percent as per ASTM Standard D1557 test method.

- Rocks larger than one inch should not be placed within 12 inches of the top of the pipeline or within the upper 12 inches of pavement or structure subgrade. No more than 30 percent of the backfill volume shall be larger than 3/4-inch in largest dimension. Rocks shall be well mixed with finer soil.

- The pipe design engineer should select bedding material for the pipe. Bedding materials generally should have a Sand Equivalent (SE) greater than or equal to 30, as determined by the ASTM Standard D2419 test method.

- Trench backfill shall be compacted by mechanical methods, such as sheepsfoot, vibrating or pneumatic rollers, or mechanical tampers, to achieve the density specified herein. The backfill materials shall be brought to between optimum and three percent above optimum, then placed in horizontal layers. The thickness of uncompacted layers should not exceed eight inches. Each layer shall be evenly spread, moistened or dried as necessary, and then tamped or rolled until the specified density has been achieved.

- The contractor shall select the equipment and processes to be used to achieve the specified density without damage to adjacent ground and completed work.

- The field density of the compacted soil shall be measured by the ASTM Standard D1556 or ASTM Standard D2922 test methods or equivalent.

- Observation and field tests should be performed by geotechnical representative during construction to confirm that the required degree of compaction has been obtained. Where compaction is less than that specified, additional compactive effort shall be made with adjustment of the moisture content as necessary, until the specified compaction is obtained.

- It should be the responsibility of the Contractor to maintain safe conditions during cut and/or fill operations.

- Trench backfill shall not be placed, spread or rolled during unfavorable weather conditions. When the work is interrupted by heavy rain, fill operations shall not be resumed until field tests by the project's geotechnical consultant indicate that the moisture content and density of the fill are as previously specified.

Observation and Testing

- During the progress of grading, the Geotechnical Engineer will provide observation of the fill placement operations.

- Field density tests will be made during grading to provide an opinion on the degree of compaction being obtained by the contractor. Where compaction of less than specified herein is indicated, additional compactive effort with adjustment of the moisture content shall be made as necessary, until the required degree of compaction is obtained

- A sufficient number of field density tests will be performed to provide an opinion to the degree of compaction achieved. In general, density tests will be performed on each one-foot lift of fill, but not less than one for each 500 cubic yards of fill placed.