Converse Consultants
Geotechnical Engineering
Environmental & Groundwater Science
Inspection & Testing Services

GEOTECHNICAL STUDY REPORT

PROPOSED NEW FOOTBALL STADIUM PROJECT

LA MIRADA HIGH SCHOOL
13520 ADELFA DRIVE
LA MIRADA, CALIFORNIA

CONVERSE PROJECT NO. 19-31-285-01

Prepared For:

NORWALK-LA MIRADA UNIFIED SCHOOL DISTRICT
Ms. Bomee Yoon, Facilities Coordinator
Facilities Planning & Construction
15711 Pioneer Boulevard, Building G
Norwalk, California 90650

Presented By:

CONVERSE CONSULTANTS
717 South Myrtle Avenue
Monrovia, California 91016
626-930-1200

October 24, 2019

October 24, 2019

Ms. Bomee Yoon
Facilities Coordinator
Facilities Planning & Construction
Norwalk-La Mirada Unified School District
15711 Pioneer Boulevard, Building G
Norwalk, California 90650

Subject: **GEOTECHNICAL STUDY REPORT**
Proposed New Football Stadium Project
La Mirada High School
13520 Adelfa Drive
La Mirada, California 90638
Norwalk-La Mirada Unified School District
Converse Project No. 19-31-285-01

Dear Ms. Yoon:

Enclosed is the Geotechnical Study Report prepared by Converse Consultants (Converse) for the proposed New Football Stadium Project within La Mirada High School in La Mirada, California.

The purpose of the study was to investigate the geotechnical site conditions and provide recommendations for the Proposed New Football Stadium Project which entails proposed improvements to the existing football field, baseball field and softball field located within the existing La Mirada High School site.

Based on our field exploration, laboratory testing, geologic evaluation, and geotechnical analysis, the site is suitable from a geotechnical standpoint for the proposed New Football Stadium Project located within La Mirada High School, provided our conclusions and recommendations are implemented during design and construction.

We appreciate the opportunity to be of continued service to Norwalk-La Mirada 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 New Football Stadium Project located within La Mirada High School in La Mirada, 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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STATE OF CALIFORNIA

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 sites are located at 13520 Adelfa Drive in La Mirada, Los Angeles County, California. The subject site is relatively flat to gently sloping with surface elevations ranging from approximately 185 to 215 feet relative to mean-sea-level (MSL). We understand that the proposed project entails improvements to the existing football field, baseball field and softball field which includes new scoreboards, synthetic turfs, field lighting, pavements and other related improvements within La Mirada High School. The structural loads are anticipated to be low to moderate.

- Eight (8) exploratory borings (BH-1 through BH-8) were drilled within the project site on October 10, 2019. Borings were advanced using a truck mounted drill rig with an 8-inch diameter hollow stem auger to depths ranging from 10 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.

- The site is not located within a mapped Seismic Hazard Zone for liquefaction potential. Based on the results of our subsurface exploration, the site is comprised of dense granular materials and stiff fine-grained soil. As a result, we anticipate liquefaction potential to be very low. Based on the generally high blow count and the fine-grained soils in our borings, we anticipate that the total seismically-induced settlement is negligible.

- Groundwater was encountered during our subsurface exploration at the depth of 48 feet in Boring BH-7. In accordance with the Seismic Hazard Zone Report for the Whittier 7.5-Minute Quadrangle (CDMG, 1998), Plate 1.2, the historically highest groundwater contour levels in the vicinity of the site are reportedly approximately 50 feet below ground surface. Groundwater is not anticipated during construction; however, it may need to be considered in design based on the historically highest groundwater contours and depth encountered in Boring BH-7.

- Undocumented fill consisting of sandy silt, clay and clayey sand were encountered from four (4) to five (5) feet in depth at the sites. Undocumented fill should be excavated and recompacted. The older surficial sediments were encountered beneath the fills consisting predominately of claystone, siltstone and sandstone to a depth of approximately 51.5 feet below ground surface. Sedimentary bedrock of the La Habra Formation is mapped in the low-lying hills approximately 3,000 feet east of the project site.

- The surficial site soils at the site exhibit a “medium” expansive potential. Mitigation for expansive soil is considered necessary for slabs, foundations and pavement.
- In general, the pH value, chloride content, and concentrations of water soluble sulfates saturated resistivity of the site soils are in the non-corrosive range. The saturated resistivity of the site soils is in the 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 structure 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.
- Musco lightings and other non-buildings structures can be supported on Cast-In-Drilled-Hole (CIDH) pile foundations.
- Percolation test was performed utilizing exploratory boring BH-2, on October 10, 2019 and the percolation rate is 0.09 inches/hour.

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.

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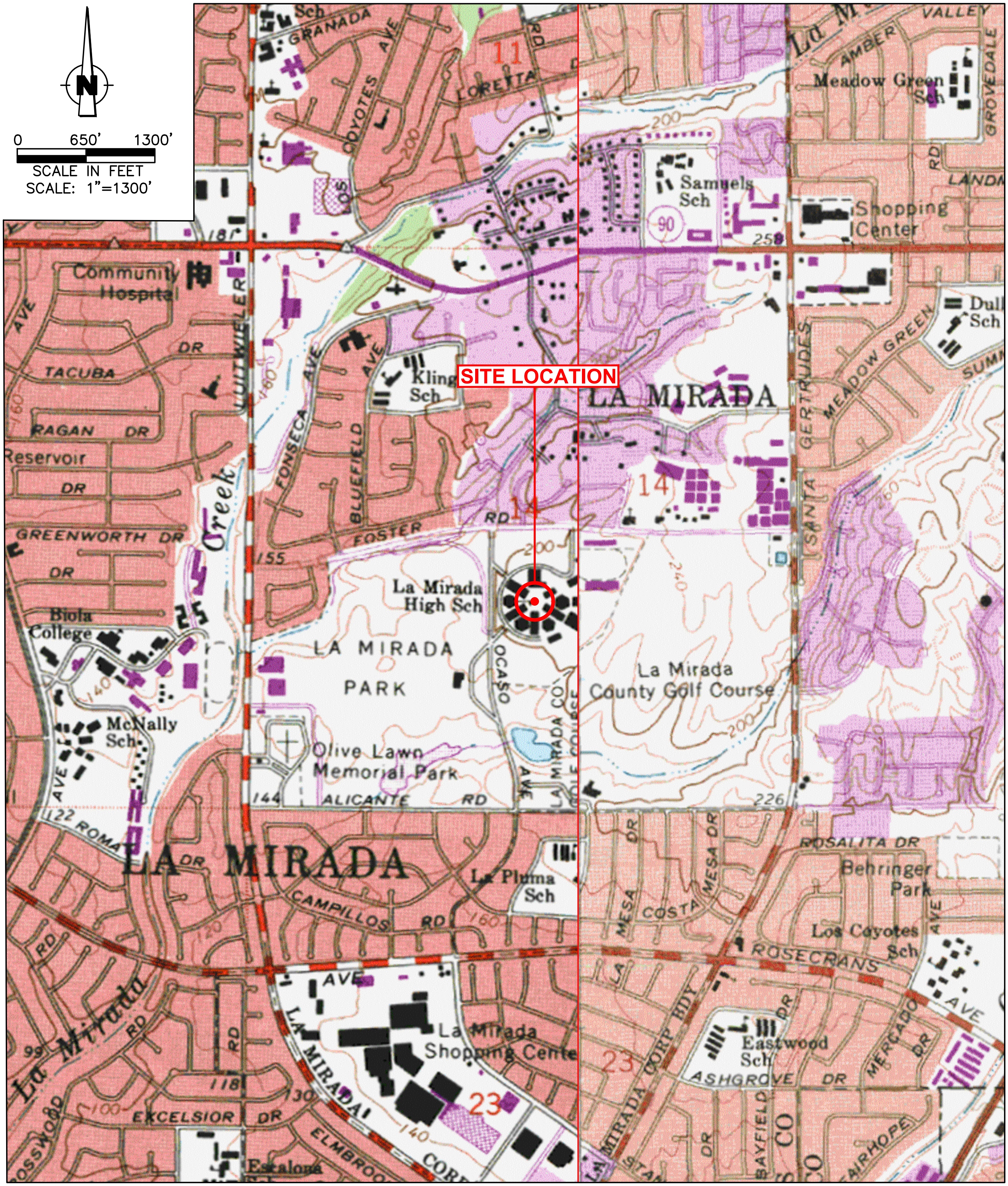
1.0 INTRODUCTION

This report contains the findings and recommendations of our geotechnical study performed at the site of the proposed New Football Stadium Project located at La Mirada High School, La Mirada, California, as shown on Drawing No. 1, *Site Location Map*.

The purpose of the study will be 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 Norwalk-La Mirada Unified School District, La Mirada High 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.



SITE LOCATION MAP

La Mirada High School
13520 Adelfa Dr,
La Mirada, CA 90638

Project No.
19–31–285–01

Converse Consultants

Drawing No.
1

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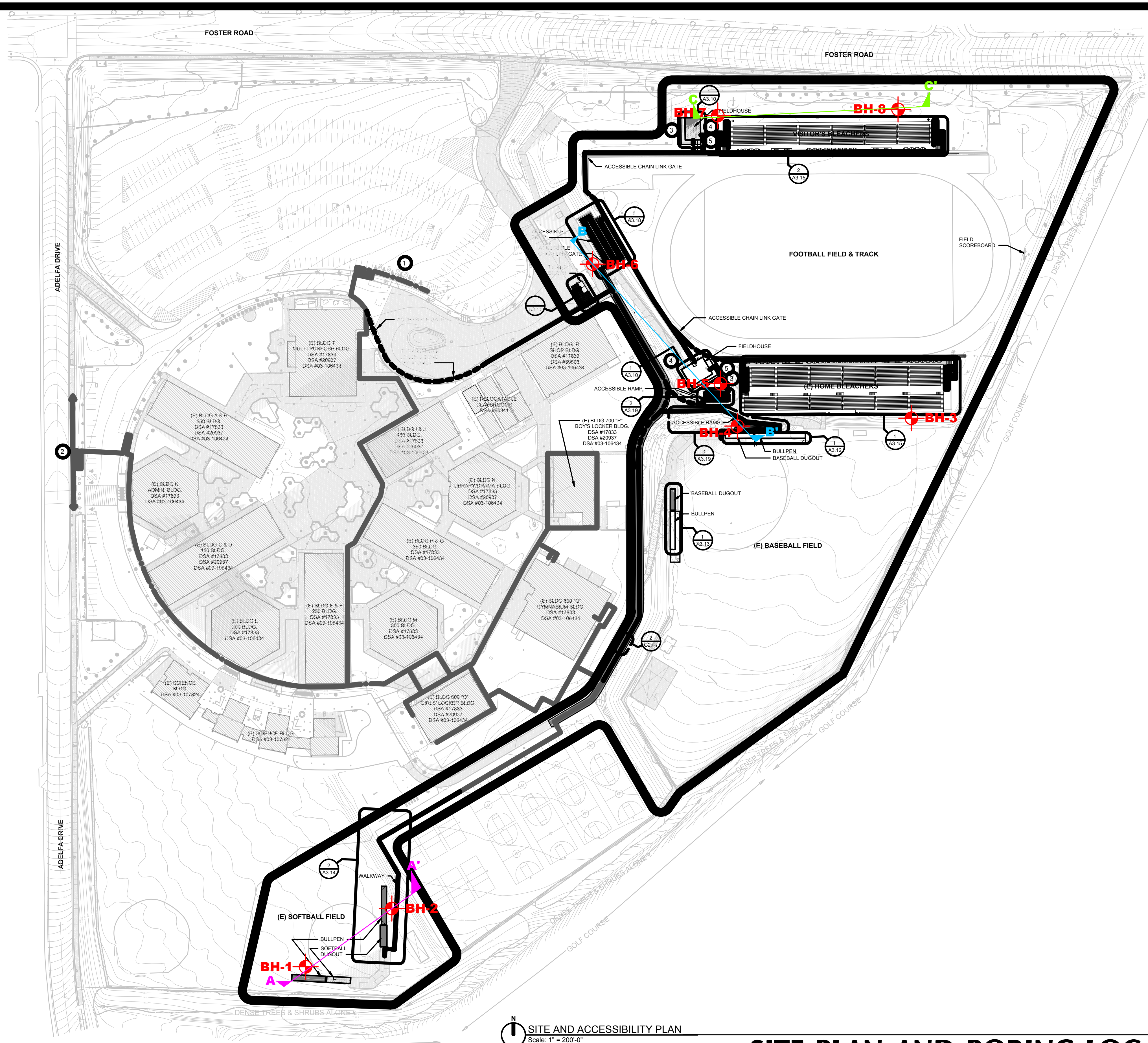
2.0 SITE AND PROJECT DESCRIPTION

The proposed project is located at 13520 Adelfa Drive in La Mirada, Los Angeles County, California. The subject site is relatively flat to gently sloping with surface elevation of approximately 185 to 215 feet relative to mean-sea-level (MSL). The site is bounded by Foster Road to the north, Adelfa Drive to the west, and by La Mirada Golf Course to south and east. The site coordinates are: North latitude: 33.90820 degrees, West longitude: -118.00309 degrees.

We understand that the proposed project entails improvements to the existing football field, baseball field and softball field which includes new scoreboards, synthetic turfs, field lighting, pavements and other related improvements within La Mirada High School as shown on Drawing No. 2, *Site Plan and Boring Location Map.* The structural loads are anticipated to be low to moderate.



SITE PLAN AND BORING LOCATION MAP

BORING LOCATION

Converse Consultants

La Mirada High School
13520 Adelfa Dr.
La Mirada, CA 90638

Project No.	Drawing No.
19-31-285-01	2

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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.

3.1 Site Reconnaissance

During the site reconnaissance on October 7, 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.

3.2 Subsurface Exploration

Eight (8) exploratory borings (BH-1 through BH-8) were drilled within the project sites on October 10, 2019. Borings were advanced using a truck mounted drill rig with an 8-inch diameter hollow stem auger to depths ranging from 10 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). Borings terminated at a depth less than 10 feet below existing ground surface were backfilled with soil cuttings and capped to match surface conditions. Borings extending into groundwater or deeper than 10 feet were backfilled with cement grout and capped to match surface conditions.

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

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)
- Consolidation (ASTM Standard 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 north central portion of the Los Angeles Basin, a broad sediment-filled basin located at the convergence of the Transverse Ranges and Peninsular Ranges geomorphic provinces of California. The San Gabriel and Rio Hondo drainages have deposited stream and flood sediments across the coastal flood plain during Holocene time (0 – 11,000 years) to form a relatively flat and broad flood plain geomorphic feature known as the Downey Plain. The project site is located on an elevated remnant of older surficial sediments located along the western end of the Coyote Hills. The elevated hillside remnant and Coyote Hills separate the smaller Whitter and La Habra drainage basins to the north from the much larger Los Angeles Basin to the south. The La Mirada Creek and Coyote Creek stream drainages still provide southward drainage for the Whittier and La Habra basins through the hillside areas. Soils underlying the project site consist of sands, silty sands, silts and clay sediments deposited over time by local stream tributaries which once drained across the coastal plain to the Pacific Ocean. Most of these natural river, creek and stream channels are now controlled by dams, debris basins and flood control channels 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.

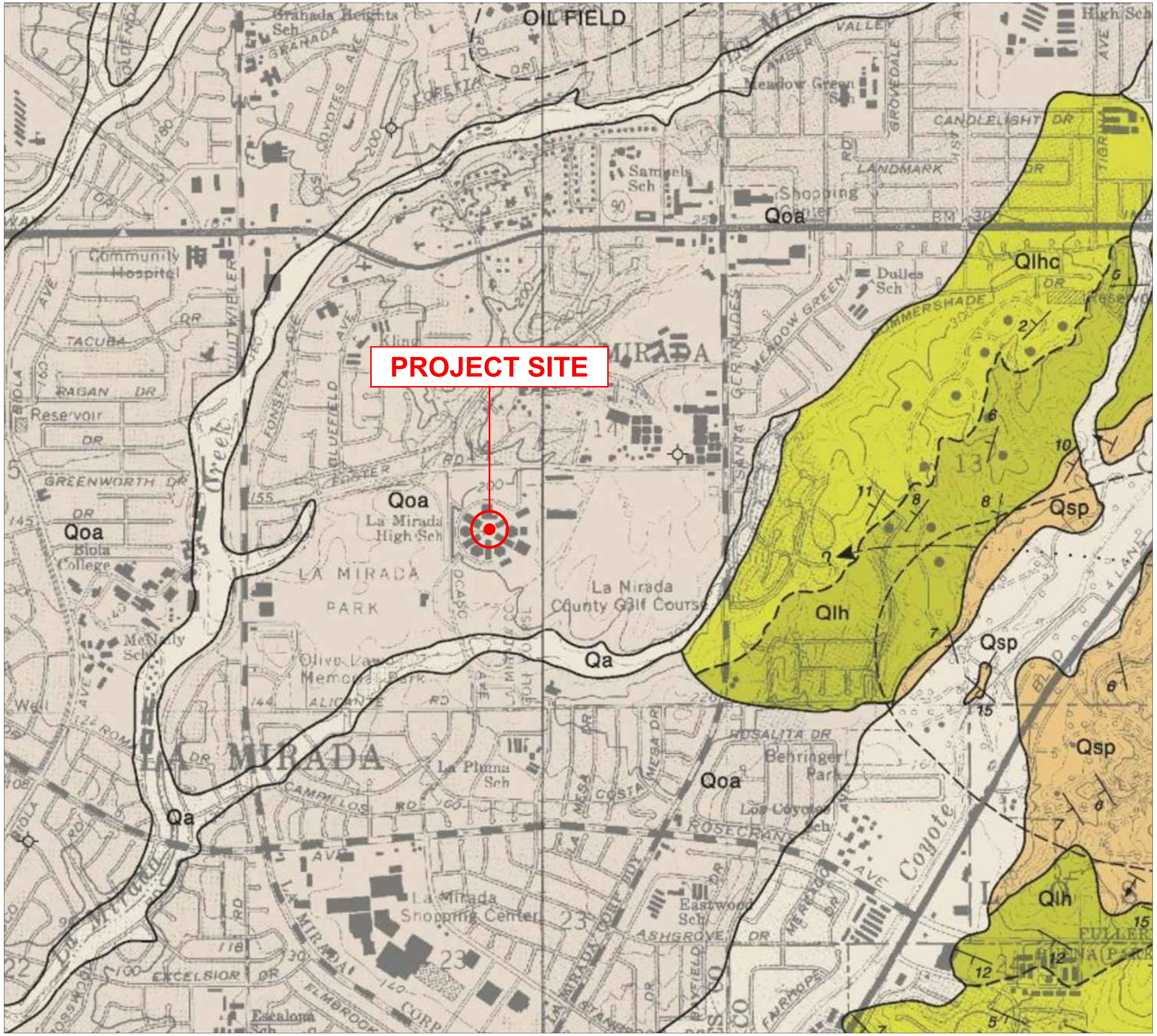
The western end of the Coyote Hills is underlain by several oil fields including the West Coyote, East Coyote, Leffingwell, New Gate and Santa Fe Springs oil fields. The closest oil field to the project site is the West Coyote field located beneath the hillside areas approximately 3,500 feet east of the school site.

4.2 Subsurface Profile of Subject Site

Based on our soil borings drilled at the site on October 10, 2019, the subsurface conditions generally consist of existing fill soils placed during previous site grading operations and natural older 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 sandy silt, silty clay, sandy clays, clays and silty sands. The depth of the fill observed ranged from approximately four (4) to five (5) feet in the borings. The older alluvial sediments consist predominately of weathered siltstone, claystone and sandstone sediments to the maximum drilled depth of approximately 51.5 feet below ground surface. Drawing Nos. 4a, 4b and 4c, *Geologic Cross Sections A-A', B-B' and C-C',* have been drawn across the subject 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 in Boring BH-7 at a depth of 48 feet during drilling. In accordance with the Seismic Hazard Zone Report for



WHITTIER AND LA HABRA MAP (DF-74)

LEGEND

af
Qg
Qa

SURFICIAL SEDIMENTS
Undissected alluvial deposits
af *Artificial fill, some recent areas may not be shown*
Qg *Gravel and sand of major stream channels*
Qa *Alluvial gravel, sand, and silt of valleys and floodplains*

Qls

LANDSLIDE DEBRIS
See also landslides mapped by Tan, 1988

Qae
Qoa

OLDER SURFICIAL SEDIMENTS
Elevated and dissected alluvial deposits
Qae *Slightly elevated and locally dissected alluvial gravel and sand, on north side of Puente Hills*
Qoa *Elevated, dissected remnants of alluvial sand and gravel*

— UNCONFORMITY —

Qlhc
Qlh

LA HABRA FORMATION
(of Yerkes, 1972)
Terrestrial, weakly indurated; Pleistocene age, includes Coyote Hills Formation of Yerkes, 1972 (in Coyote Hills)
Qlh *Tan to light gray sandstone and pebble conglomerate, vaguely bedded, includes abundant siliceous shale pebbles south of Puente Hills*
Qlhc *Similar to Qlh, but includes interbedded siltstone in middle part of unit*

— LOCAL UNCONFORMITY —

Qsp

SAN PEDRO FORMATION
(Of Yerkes, 1972, in Coyote Hills)
Shallow marine clastic, weakly indurated; early Pleistocene age
Qsp *Sandstone, tan to light gray, soft, vaguely bedded, contains molluscan fossils, locally pebbly, base unexposed*

Holocene
?
?
Pleistocene
QUATERNARY

N
NOT TO SCALE

REGIONAL GEOLOGIC MAP

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La Mirada High School
13520 Adelfa Dr.
La Mirada, CA 90638

Project No.	Drawing No.
19-31-285-01	3

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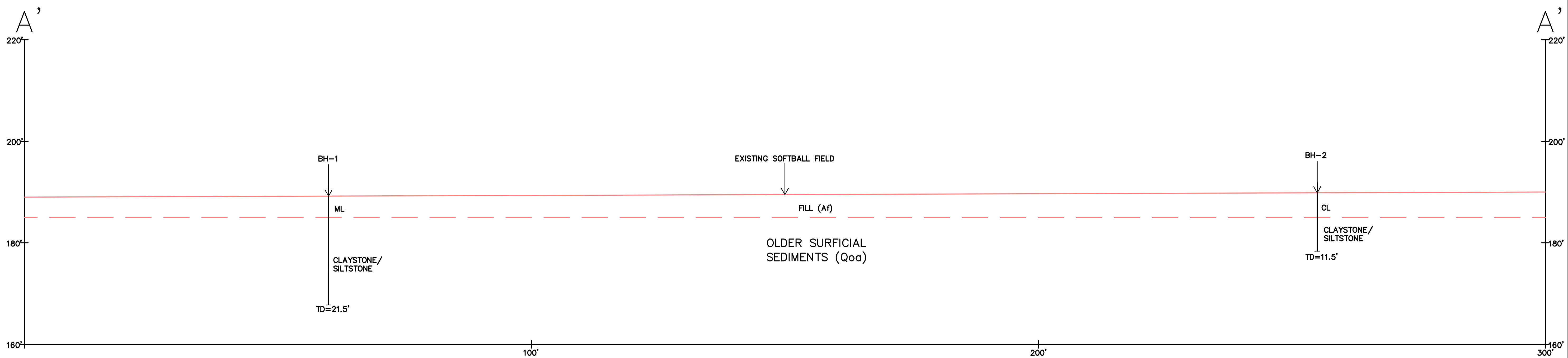

0 10' 20'
SCALE: 1"=20'

GEOLOGIC SECTION A-A'

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La Mirada High School
13520 Adelfa Dr.
La Mirada, CA 90638

Project No.	Drawing No.
19-31-285-01	4a

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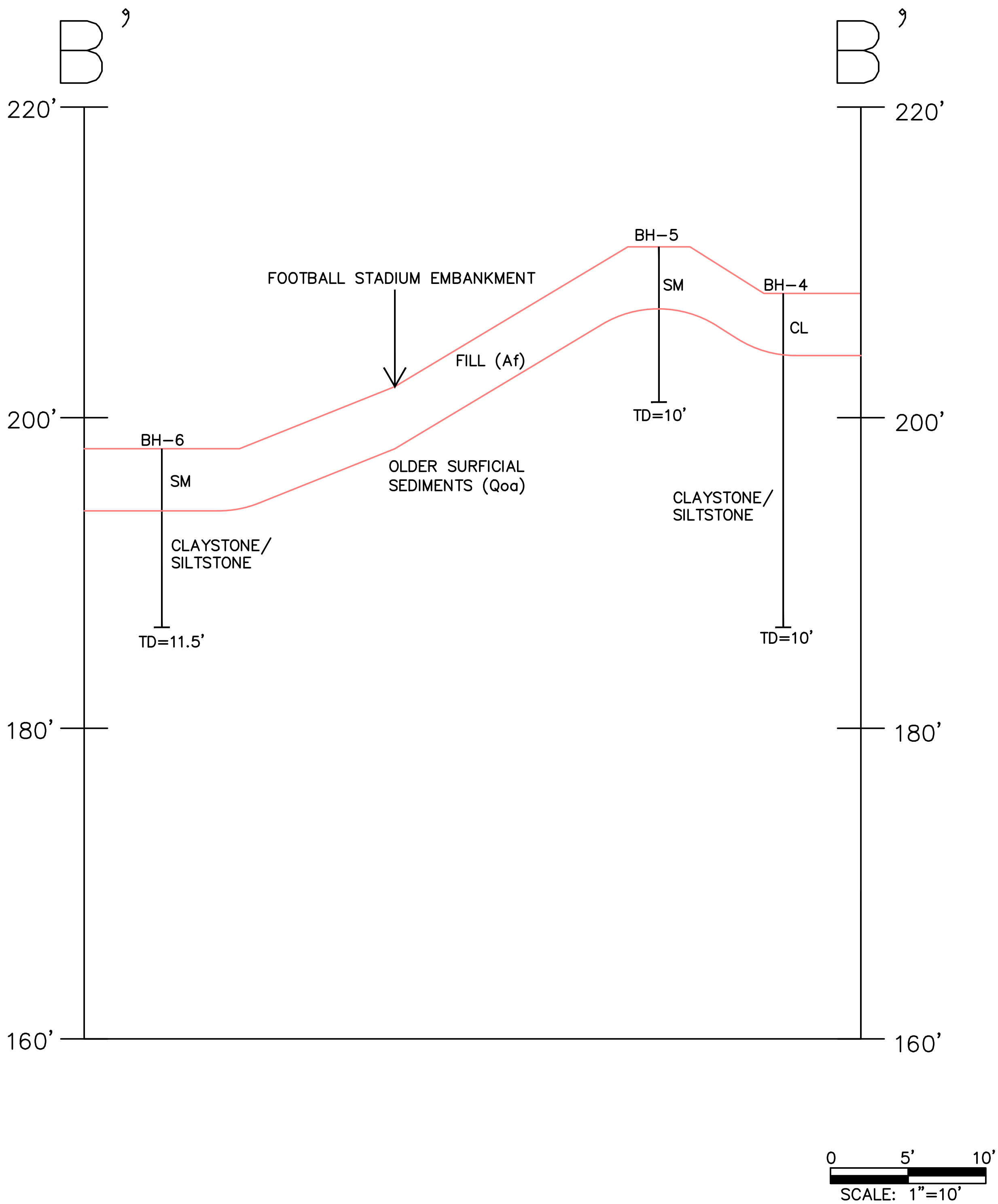

GEOLOGIC SECTION B-B'

La Mirada High School
13520 Adelfa Dr,
La Mirada, CA 90638

Project No.
19-31-285-01

Converse Consultants

Drawing No.
4b

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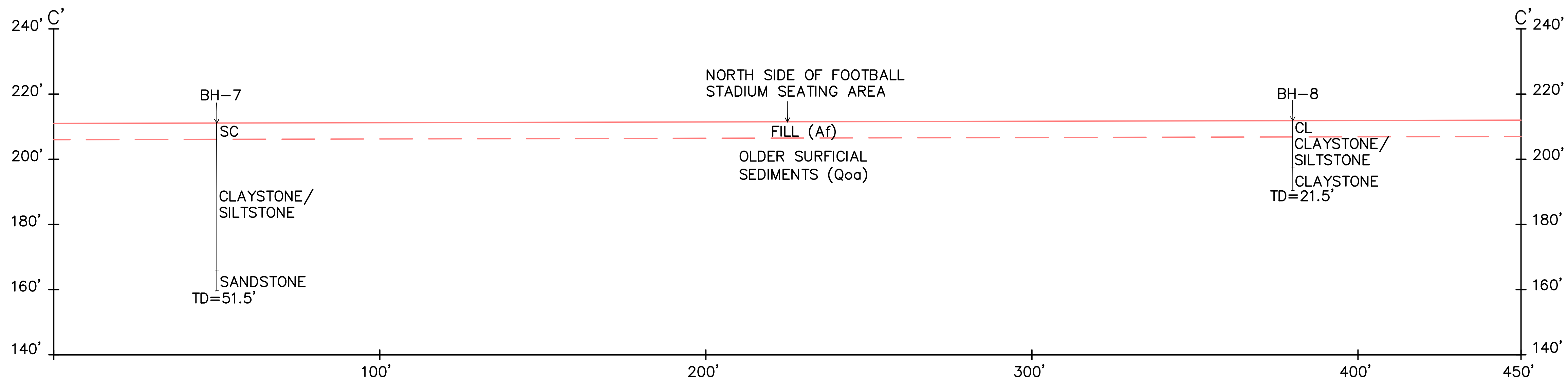


0 20' 40'
SCALE: 1"=40'

GEOLOGIC SECTION C-C'

Converse Consultants

La Mirada High School
13520 Adelfa Dr.
La Mirada, CA 90638

Project No.	Drawing No.
19-31-285-01	4c

I:\ACADDRAWINGS\19\31\285\19-31-285-01-XSECTIONS.DWG the Whittier 7.5-Minute Quadrangle (CDMG, 1998), the historically highest groundwater contour levels in the vicinity of the site are reportedly approximately 50 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

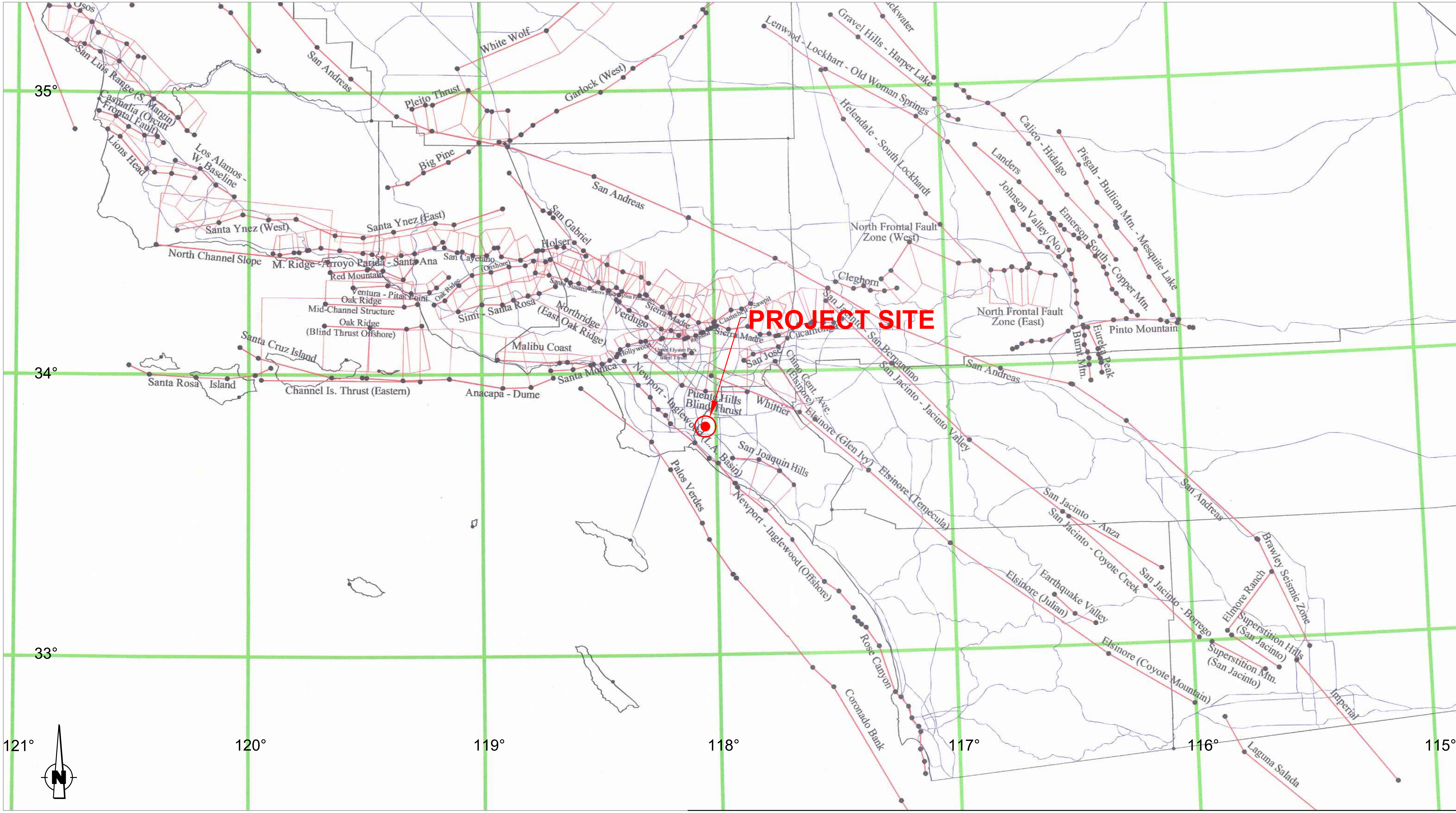
The project site is not located within a currently designated State of California Earthquake Fault Zone (formerly Alquist-Priolo Special Studies Zones) for surface fault rupture. No surface faults are known to project through or towards the site. The closest known fault with the potential for surface rupture is the Whittier Fault located approximately 4.2 miles to the northeast. 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>5.2.1 Whittier Fault</u>

The mapped trace of the Whittier Fault is located approximately 4.2 miles northeast of the project site along the southern 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.

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



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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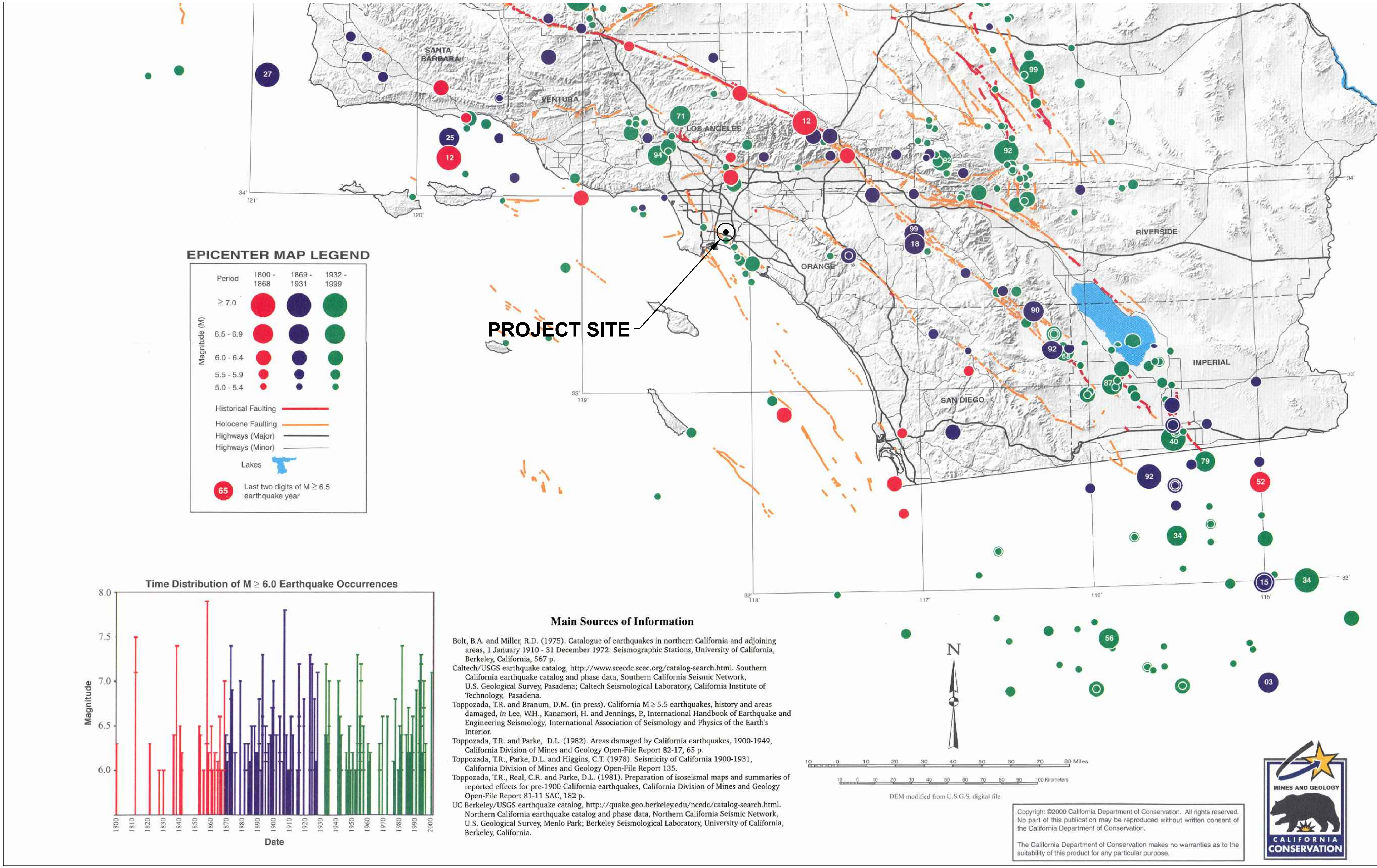
La Mirada High School
13520 Adelfa Dr.
La Mirada, CA 90638

Project No. 19-31-285-01
Drawing NO. 5
Date October 2019



Main Sources of Information

Bolt, B.A. and Miller, R.D. (1975). Catalogue of earthquakes in northern California and adjoining areas, 1 January 1910 - 31 December 1972: Seismographic Stations, University of California, Berkeley, California, 567 p.

Caltech/USGS earthquake catalog, http://www.scecdc.scec.org/catalog-search.html. Southern California earthquake catalog and phase data, Southern California Seismic Network, U.S. Geological Survey, Pasadena; Caltech Seismological Laboratory, California Institute of Technology, Pasadena.

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.

Toppozada, T.R., Parke, D.L. and Higgins, C.T. (1978). Seismicity of California 1900-1931, California Division of Mines and Geology Open-File Report 135.

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.

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)

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La Mirada High School
13520 Adelfa Dr,
La Mirada, CA 90638

Project No.	Drawing No.
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(Gath, 1997). The Whittier fault is considered to be capable of producing a maximum movement magnitude of 6.8 (Mw) earthquake.

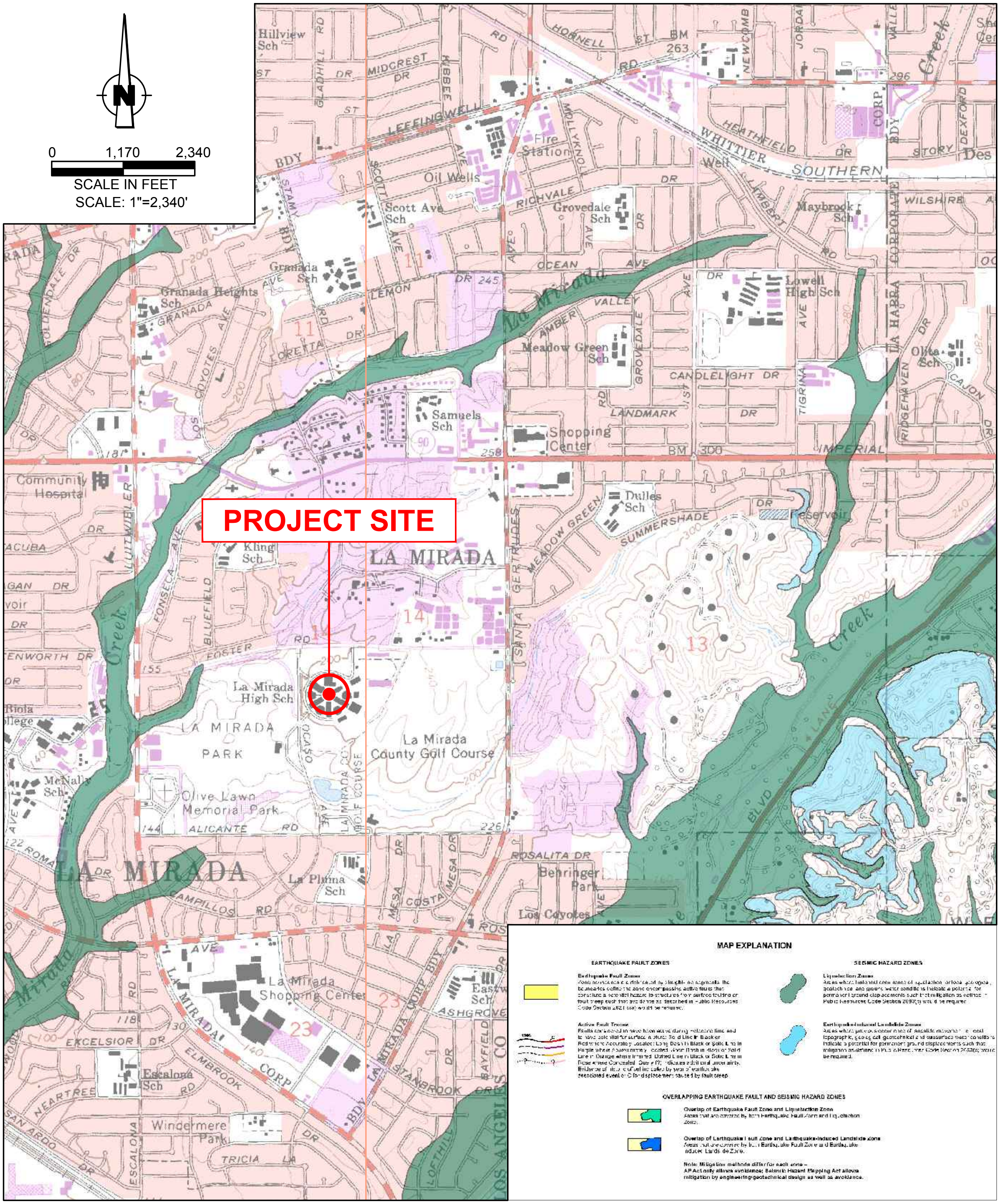
5.3 Liquefaction and Seismically-Induced Settlement

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 not 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, the site is comprised of dense granular materials and stiff fine-grained soil. As a result, we anticipate liquefaction potential to be very low. Based on the generally high blow count and the fine-grained soils in our borings, we anticipate that the total seismically-induced settlement is negligible.

Table No. 1, Summary of Regional Faults

Fault Name and Section	Approximate Distance to Site (kilometers)	Max. Moment Magnitude (Mmax)	Slip Rate (mm/yr)
WHITTIER	8.0	6.8	2.5
ELYSIAN PARK THRUST	10.1	6.7	1.5
COMPTON THRUST	13.4	6.8	1.4
NEWPORT-INGLEWOOD (L.A. Basin)	18.6	6.9	1
SAN JOSE	19.1	6.5	0.5
CHINO-CENTRAL AVE. (Elsinore)	25.0	6.7	1.0
RAYMOND	27.6	6.5	1.5
VERDUGO	30.1	6.7	0.5
PALOS VERDES	30.5	7.1	3
SIERRA MADRE	30.7	7	2
HOLLYWOOD	32.5	6.4	1
CLAMSHELL-SAWPIT	32.6	6.5	0.5
ELSINORE-GLEN IVY	34.4	6.8	1.5
CUCAMONGA	36.2	7	5.0
NEWPORT-INGLEWOOD (Offshore)	36.3	6.9	1.5
SANTA MONICA	42.9	6.6	1
MALIBU COAST	51.0	6.7	0.3
SIERRA MADRE (San Fernando)	51.8	6.7	2
SAN GABRIEL	52.3	7	1
NORTHRIDGE (E. Oak Ridge)	54.7	6.9	1.5



SEISMIC HAZARD ZONES MAP

Converse Consultants

La Mirada High School
13520 Adelfa Dr.
La Mirada, CA 90638

Project No.	Figure No.
19-31-285-01	7

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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 relatively flat. 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. In the absence of significant ground slopes, the potential for seismically induced landslides to affect the proposed site is considered to be very 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 FEMA Flood Map , Panel 06037C1842F, 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 due the project site elevation and regional flood control structures.

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 (over 20 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 hazard.

6.0 SEISMIC ANALYSIS

6.1 CBC Seismic Design Parameters

Seismic parameters based on the 2019 California Building Code are calculated using the Applied Technology Council (ATC) *Seismic Design Maps* website application and the site coordinates (33.90820 degrees North Latitude, -118.00309 degrees West Longitude). The seismic parameters are presented below.

Table No. 2, CBC Seismic Design Parameters

Seismic Parameters	2019 CBC
Site Class	D
Mapped Short period (0.2-sec) Spectral Response Acceleration, S_S	1.640 g
Mapped 1-second Spectral Response Acceleration, S_1	0.583 g
Site Coefficient (from Table 1613.5.3(1)), F_a	1.0
Site Coefficient (from Table 1613.5.3(2)), F_v	1.7
MCE 0.2-sec period Spectral Response Acceleration, S_{MS}	1.640 g
MCE 1-second period Spectral Response Acceleration, S_{M1}	0.991 g
Design Spectral Response Acceleration for short period, S_{DS}	1.094 g
Design Spectral Response Acceleration for 1-second period, S_{D1}	0.661 g
Seismic Design Category	D

7.0 GEOTECHNICAL EVALUATIONS AND CONCLUSIONS

Based on the results of our literature 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:

- The proposed project is located at 13520 Adelfa Drive in La Mirada, Los Angeles County, California. The subject site is relatively flat to gently sloping with surface elevations ranging from approximately 185 to 215 feet relative to mean-sea-level (MSL). We understand that the proposed project entails improvements to the existing football field, baseball field and softball field which includes new scoreboards, synthetic turfs, field lighting, pavements and other related improvements within La Mirada High School. The structural loads are anticipated to be low to moderate.

- There are no active faults projecting toward or extending across the proposed site. The site is not located within a currently designated State of California Earthquake Fault Zone. Moderate to strong ground shaking from earthquakes associated with the Whittier Fault and other nearby and distant faults may occur during the lifetime of the project.

- The site is not located within a mapped Seismic Hazard Zone for liquefaction potential. Based on the results of our subsurface exploration, the site is comprised of dense granular materials and stiff fine-grained soil. As a result, we anticipate liquefaction potential to be very low. Based on the generally high blow count and the fine-grained soils in our borings, we anticipate that the total seismically-induced settlement is negligible.

- Groundwater was encountered during our subsurface exploration at the depth of 48 feet in Boring BH-7. In accordance with the Seismic Hazard Zone Report for the Whittier 7.5-Minute Quadrangle (CDMG, 1998), the historically highest groundwater contour levels in the vicinity of the site are reportedly approximately 50 feet below ground surface. Groundwater is not anticipated during construction; however, it may need to be considered in design based on the historically highest groundwater levels.

- The surficial site soils at the site exhibit a “medium” expansive potential. Mitigation for expansive soil is considered necessary for slabs, foundations and pavement.

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

- Shallow spread and continuous footings are considered suitable for structure 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.

- Musco lightings and other non-buildings structures can be supported on Cast-In-Drilled-Hole (CIDH) pile foundations.

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), 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 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, fill and soil disturbed during demolition should be removed to firm and unyielding native material or compacted fill.

Due to the undocumented fills encountered at the site, we recommend over-excavation for structure footings to be at least five(5) feet below the existing grade or 3 feet below bottom of footing, whichever is deeper. Deeper removal and re-compaction 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.

Over-excavation and re-compaction for retaining walls, if any, should be three (3) feet below bottom of footings and should extend three (3) feet laterally beyond the retaining wall area. The upper 24-inches of site soils should be removed and re-compacted in areas of sidewalks and surface parking. The upper 18 inches of soil should be processed and compacted in field areas. The over-excavation should extend 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 removal 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 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 at least 95% relative compaction.
c. Over-excavate an additional 18-inches deep, and then place a layer of geotextile (i.e. Mirafi HP570, or equivalent), then place 18-inch-thick compacted base material (CAB or equivalent) to bridge the soft bottom. Base should be compacted to at least 95% relative compaction. An additional layer of geotextile may be needed on top of the compacted base materials 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 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

One (1) expansion index test was performed in accordance with ASTM Standard D4829 for the upper five (5) feet of soil. Results of the test indicated an expansion index of 56 in BH-8 which corresponds to medium expansion potential. Mitigation for expansive soil is necessary. The on-site soil materials will be mixed during the grading and the expansion potential might change. Therefore, the potential expansion index of site soils should be tested and verified after the grading of new slabs, foundations and pavements placed directly on on-site or native expansive subgrade soils, otherwise they may likely crack over time. If the expansion potential of mixed soil is found to be above 20, Converse recommends mixing on-site soil used for support of slabs, foundations, walkways, and pavements with 5 percent cement to reduce expansion potential.

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

The proposed structures can be supported by conventional shallow footings. We recommend continuous and square footings be founded at least 18 inches below lowest adjacent final grade entirely into compacted fill or into native soil. A minimum footing width of 24 inches is recommended for square footings and 18 inches for continuous footings. The allowable bearing value for footings with above minimum sizes founded on compacted fill and competent native soils may be designed for a net bearing pressure of 2,000 pounds per square foot (psf) for dead-plus-live-loads. The net allowable bearing pressure can be increased by 250 psf for each additional foot of excavation depth and by 250 psf for each additional foot of excavation width up to a maximum value of 3,000 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.

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 150 psf per foot of depth up to a maximum of 1,500 psf may be used for footings poured against properly compacted fill. 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 and native soil will depend on the actual footing dimensions and the imposed vertical loads. 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 Cast-In-Drilled-Hole Pile Foundations for Musco Lighting

The planned Musco Lighting poles may be supported on a Cast-In-Drilled-Hole (CIDH) pile foundation provided the following recommendations are incorporated into design and construction.

9.2.1 Vertical Capacity

CIDH piles should be at least 24-inches in diameter extending at least 10 feet below adjacent final grade on compacted fill or native alluvial soils or bedrock and can be designed for an allowable skin friction of 200 psf against the perimeter of pile. The diameter and length of CIDH pile shall be determined by the structural engineer based on design loads. The uplift capacities can be taken as one-half of compressive capacities for pile design.

9.2.2 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 0.30 may be assumed with normal dead load forces. An allowable passive earth pressure of 150 psf per foot of depth up to a maximum of 1,500 psf may be used for foundations poured against compacted fill. The values of coefficient of friction and allowable passive earth pressure include a factor of safety of 1.5.

For ground surface restrained by concrete slab, the passive resistance may be calculated from the ground surface. For unrestrained ground condition, the passive resistance of the upper one (1) foot of earth material should be neglected in design.

9.2.3 Settlement

Based on the maximum allowable net vertical capacity presented above, static settlement is anticipated to be less than 0.5 inch.

9.3 Cast-In-Drilled-Hole Pile Foundations for Non-building Structures

The planned non-building structures (e.g. lighting for parking lot, walkway, and court, fence walls, signs, etc.) may be supported on a Cast-In-Drilled-Hole (CIDH) pile foundation provided the following recommendations are incorporated into design and construction.

9.3.1 Vertical Capacity

CIDH piles should be at least 18-inches in diameter and can be designed for an allowable skin friction of 150 psf against the perimeter of pile. The diameter and length of CIDH pile shall be determined by the structural engineer based on design loads. The uplift capacities can be taken as one-half of compressive capacities for pile design.

9.3.2 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 0.30 may be assumed with normal dead load forces. An allowable passive earth pressure of 150 psf per foot of depth up to a maximum of 1,500 psf may be used for foundations poured against compacted fill. The values of coefficient of friction and allowable passive earth pressure include a factor of safety of 1.5.

For ground surface restrained by concrete slab, the passive resistance may be calculated from the ground surface. For unrestrained ground condition, the passive resistance of the upper one (1) foot of earth material should be neglected in design.

9.3.3 Settlement

Based on the maximum allowable net vertical capacity presented above, static settlement is anticipated to be less than 0.5 inch.

9.4 Slabs-on-Grade

Slabs-on-grade should have a minimum thickness of five (5) inches nominal for support of normal ground-floor live loads. Minimum reinforcement for slabs-on-grade should be No. 4 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. A static modulus of subgrade reaction equal to 100 pounds per square inch per inch may be used in structural design of concrete slabs-on-grade.

It is critical that the exposed subgrade soils should not be allowed to desiccate prior to the slab pour. Care should be taken during concrete placement to avoid slab curling. Slabs should be designed and constructed as promulgated by the ACI and Portland Cement Association (PCA). Prior to the slab pour, all utility trenches should be properly backfilled and compacted.

In areas where a moisture-sensitive floor covering (such as vinyl tile or carpet) is used, a 10-mil-thick moisture retarder/barrier between the bottom of slab and subgrade that meets the performance criteria of ASTM E1745 Class A material. Retarder/barrier sheets should be overlapped a minimum of six inches and should be taped or otherwise sealed per the product specifications.

9.5 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.

9.6 Lateral Earth Pressure

Although not anticipated, the following provisional design values may be used for any utility vaults and/or walls below grade that are less than 8 feet high.

The earth pressure behind any buried wall depends primarily on the allowable wall movement, type of soil behind the wall, backfill slopes, wall inclination, surcharges, and any hydrostatic pressure. The following earth pressures are recommended for vertical walls with no hydrostatic pressure.

Table No. 3, Lateral Earth Pressures for Retaining Wall Design

Backfill Slope (H:V)	Cantilever Wall Equivalent Fluid Pressure (pcf)	Restrained Wall (psf)
Level	50 (triangular pressure distribution)	65 (triangular pressure distribution)

The recommended lateral pressures assume that the walls are fully back-drained 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 a minimum one (1) square feet per lineal feet of free draining, uniformly graded, 3/4-inch washed, crushed aggregate, and wrapped in filter fabric such as Mirafi 140N or equivalent. The filter fabric should overlap approximately 12 inches or more at the joints. The subdrain pipe should consist of perforated, four-inch diameter, rigid ABS (SDR-35) or PVC A-2000, 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 solid pipe outlets, with a maximum outlet spacing of 100 feet. Waterproofing membranes should be added to the subterranean wall levels for moisture sensitive areas to mitigate moisture migration through the walls.

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.

Retaining walls taller than 6 feet should be designed to resist additional earth pressure caused by seismic ground shaking based on CBC latest edition. A seismic earth pressure of 25H (psf), based on an inverted triangular distribution, can be used for design of wall.

9.7 Soil Corrosivity Evaluation

Based on our review of soil corrosivity test results (see Appendix B), the soluble sulfate concentration, pH, and chloride content are not in the corrosive range to concrete In accordance with the Caltrans Corrosive Guidelines (2012). The minimum saturated resistivity is in the corrosive range to ferrous metal. Mitigation measures to protect concrete in contact with the soils are not anticipated.

A corrosion engineer may be consulted for appropriate mitigation procedures and construction design, if needed. General considerations for 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.

9.8 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 13. 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. 4, 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)
13	4	3.0	4.5	5.0
	5	4.0	6.0	6.5
	6	5.0	7.7	8.0
	7	6.0	9.5	9.5

Design R-value	Design TI	Asphalt Concrete (AC) Over Aggregate Base (AB) Structural Sections		Full AC Structural Section
		AC (inches)	AB (inches)	AC (inches)
	8	7.0	11.0	11.0
	9	8.0	12.5	12.5

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.

9.9 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 Rigid Pavement. 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. 5, Rigid Pavement Structural Sections

Design R-Value	Design Traffic Index (TI)	PCCP Pavement Section (inches)
13	4.0	7.0
	5.0	7.5
	6.0	7.5
	7.0	8.0
	8.0	8.0
	9.0	8.5

The above pavement section is 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 inches to one inch, respectively.

Transverse contraction joints should not be spaced more than 10 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.

Prior to placement of concrete, at least the upper 12 inches of subgrade soils below rigid pavement sections should be compacted to at least 95 percent relative compaction as defined by the ASTM D 1557 standard test method.

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

9.10 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 2019 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 roof, and runoff should be directed to the storm drain through non-erosive devices. Lower level walkways and open patio areas 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. 6, *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 should be constructed at a flatter gradient than presented in the following table:

Table No. 6, Slope Ratios for Temporary Excavation

Maximum Depth of Cut (feet)	Maximum Slope Ratio* (horizontal: vertical)
0 – 4	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 minimize 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 excavation 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 excavation edge.

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 during construction. The proposed excavation should not cause loss of bearing and/or lateral supports of the existing structures.

10.3 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 utility lines or structures. The remedial grading excavations should not cause loss of bearing and/or lateral support for adjacent 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. “A-B-C” slot cuts exposing native sandy soils may be excavated with maximum 8 feet wide and 8 feet depth sections to prevent the existing utility lines or off-site structures from becoming unstable. Backfill should be accomplished in the shortest period of time possible and in alternating sections.

The ABC slot cutting method for retaining walls could be a possible option as an alternative to shoring for excavation less than 8 feet 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.

10.4 Geotechnical Services During Construction

This report has been prepared to aid in the foundation plans and specifications, and to assist the architect, civil and structural engineers in the design of the proposed structures. It is recommended that this office be provided an opportunity to review final design drawings and specifications to verify that the recommendations of this report have been properly implemented.

Footing excavations should be observed by geotechnical engineer or their representative prior to placement of steel and concrete so that footings are founded on satisfactory materials and excavations are free of loose and disturbed materials. Trench backfill should be placed and compacted with observation and field density testing provided by this office.

During construction, the geotechnical engineer and/or their authorized representatives should be present at the site to provide a source of advice to the client regarding the geotechnical aspects of the project and to observe and test the earthwork performed. Their presence should not be construed as an acceptance of responsibility for the performance of the completed work, since it is the sole responsibility of the contractor performing the work to ensure that it complies with all applicable plans, specifications, ordinances, etc.

This firm does not practice or consult in the field of safety engineering. We do not direct the contractor’s operations and cannot be responsible for other than our own personnel on the site; therefore, the safety of others is the responsibility of the contractor. The contractor should notify the owner if he considers any recommended actions presented herein to be unsafe.

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.

12.0 REFERENCES

AMERICAN SOCIETY OF CIVIL ENGINEERS, *ASCE/SEI 7-10, Minimum Design Loads for Structures and Other Structures*, copyright 2013.

ASTM INTERNATIONAL, Annual Book of ASTM Standards, Current.

CALIFORNIA BUILDING STANDARDS COMMISSION, Latest edition, *California Building Code* (CBC), California Code of Regulations Title 24, Part 2, Volumes 1 and 2.

CALIFORNIA DEPARTMENT OF CONSERVATION, DIVISION OF MINES AND GEOLOGY, 1994, *Fault-Rupture Hazard Zones in California, Alquist-Priolo Earthquake Fault Zoning Act with Index to Earthquake Fault Zones Maps*: Special Publication 42, by Hart, E.W., & Bryant, W.A., dated 1994, revised 1997 with Supplements 1 and 2 added in 1999, and Supplement 3 added in 2003.

CALIFORNIA DEPARTMENT OF CONSERVATION, DIVISION OF MINES AND GEOLOGY, 1999, *Official Seismic Hazard Zone Map, Whittier quadrangle: California Geological Survey, Official Map of Seismic Hazard Zones*, dated March 25, 1999.

CALIFORNIA GEOLOGICAL SURVEY, 1999, *Earthquake Zones of Required Investigation*, Whittier Quadrangle, dated March 25, 1999.

CALIFORNIA GEOLOGICAL SURVEY, 2008, *Guidelines for Evaluating and Mitigating Seismic Hazards in California:* Special Publication 117A.

DEPARTMENT OF THE NAVY, Naval Facilities Engineering Command, Alexandria, VA, *SOIL MECHANICS DESIGN MANUAL 7.1 (NAVFAC DM-7.1)*, 1982.

PORTLAND CEMENT ACCOCIATION, Southwest Region Publication P-14, Portland Cement Concrete Pavement Design Nomograph for City and County Roads.

RUBIN, C. M., et. al, 1998, Evidence for Large Earthquakes in Metropolitan Los Angeles, AAAS Science, vol. 281, p. 398-402.

SOUTHERN CALIFORNIA EARTHQUAKE CENTER, *Recommended Procedures for Implementation of DMG Special Publication 117* Guidelines *for Analyzing and Mitigating Liquefaction in California*, March 1999.

STANDARD SPECIFICATIONS FOR PUBLIC WORKS CONSTRUCTION, 2015, Public Works Standards, Inc.

STUDIES IN GEOPHYSICS, 1986, Active Tectonics, Geophysics Study Committee, National Academy Press.

TOKIMATSU, K. AND SEED, H. B., 1987; *Evaluation of Settlement in Sands Due to Earthquake Shaking,* ASCE Journal of Geotechnical Engineering, Vol. 118.

UNITED STATES DEPARTMENT OF THE INTERIOR GEOLOGIC SURVEY (USGS), Whittier Quadrangle, California-Los Angeles Co., 7.5 Minute Series (Topographic) map, revised 1995.

UNITED STATES GEOLOGICAL SURVEY, 2015, *U.S. Seismic Design Maps Application* by the United States Geological Survey dated October 19, 2015.

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

Eight (8) exploratory borings (BH-1 through BH-8) were drilled within the project site on October 10, 2019. Borings were advanced using a truck mounted drill rig with an 8-inch diameter hollow stem auger to depths ranging from 10 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. Where appropriate, field descriptions and classifications have been modified to reflect laboratory test results.

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.

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-9, *Log of Borings*.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS			SYMBOLS GRAPH	SYMBOLS LETTER	TYPICAL DESCRIPTIONS
COARSE GRAINED SOILS	GRAVEL AND GRAVELLY SOILS	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
	MORE THAN 50% OF COARSE FRACTION RETAINED ON NO. 4 SIEVE	GRAVELS WITH FINES (APPRECIABLE AMOUNT OF FINES)		GM	SILTY GRAVELS, GRAVEL - SAND - SILT MIXTURES
				GC	CLAYEY GRAVELS, GRAVEL - SAND - CLAY MIXTURES
MORE THAN 50% OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	SAND AND SANDY SOILS	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
	MORE THAN 50% OF COARSE FRACTION PASSING ON NO. 4 SIEVE	SANDS WITH FINES (APPRECIABLE AMOUNT OF FINES)		SM	SILTY SANDS, SAND - SILT MIXTURES
				SC	CLAYEY SANDS, SAND - CLAY MIXTURES
FINE GRAINED SOILS	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
MORE THAN 50% OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	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
- 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	Project No.	Figure No.
Norwalk-La Mirada Unified School District (NLMUSD) **La Mirada High School** **13520 Adelfa Drive** **La Mirada, California**	**19-31-285-01**	**A-1**

Log of Boring No. BH-1

Dates Drilled: 10/10/2019 Logged by: RAM Checked By: MBS

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

Ground Surface Elevation (ft): 189 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
0–5		**FILL (Af):** **SANDY SILT (ML):** with gravel, moist, light brown, white mottling.		X				
5		**OLDER SURFICIAL SEDIMENTS: (Qoa)** **SILTSTONE/CLAYSTONE: weathered, granite rock, hard** with gravel, moist, stiff, gray	X		8/20/23	14	114	ds
10		few sand and gravel, gray to brown	X		9/15/21	12	121	
15		stiff, moist, brown	X		5/15/17	15	110	
20			X		12/31/50/6"	24	102	

End of Boring at 21.5 feet below ground level.
No ground water encountered.
Borehole backfilled with cement grout on 10/10/2019

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Project Name
Norwalk-La Mirada Unified School District (NLMUSD)
La Mirada High School
13520 Adelfa Drive
La Mirada, California

Project No. **19-31-285-01**

Figure No. **A-2**

Log of Boring No. BH-2

Dates Drilled: 10/10/2019 Logged by: RAM Checked By: MBS

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

Ground Surface Elevation (ft): 190 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):** with gravel, dark brown.						max
5		**<u>OLDER SURFICIAL SEDIMENTS:</u> (Qoa)** **SILTSTONE/CLAYSTONE: weathered, thin bedded** with gravel, dark brown	■		4/6/9	20	102	c
10			■		4/8/11	22	104	
		End of Boring at 11.5 feet below ground level. No water encountered. Percolation test was performed. Borehole backfilled with soil cuttings on 10/10/2019 and compacted.						

Log of Boring No. BH-3

Dates Drilled: 10/10/2019 Logged by: RAM Checked By: MBS

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

Ground Surface Elevation (ft): 212 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, white motling, gray-brown.						ca, er
		OLDER SURFICIAL SEDIMENTS: (Qoa) **SILTSTONE/CLAYSTONE: weathered, thin bedded,** stiff, moist,						
5			■		5/15/22	19	110	
10		light brown to brown	■		8/19/28	21	103	
15			■		6/21/38	21	103	
20		claystone/siltstone, gray white mottled, weathered, moist, brown	■		9/19/26	32	98	
		End of Boring at 21.5 feet below ground level. No groundwater encountered. Borehole backfilled with cement grout on 10/10/2019						

Log of Boring No. BH-4

Dates Drilled: 10/10/2019 Logged by: RAM Checked By: MBS

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

Ground Surface Elevation (ft): 208 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
		6.5" CONCRETE OVER NO BASE MATERIAL						r
		FILL (Af): **CLAY (CL):** with gravel, brown.						
5		**OLDER SURFICIAL SEDIMENTS:(Qoa)** **CLAYSTONE/SILTSTONE, weathered, thunly bedded** moist, yellow brown			6/14/25	24	98	ds
10		moist, yellow brown			6/18/29	20	107	
15		slightly moist, gray and brown			4/11/22	18	106	
20		reddish-brown			3/6/13			
		End of Boring at 21.5 feet below ground level. No groundwater encountered. Borehole backfilled with cement grout and patched with quick set concrete on 10/10/2019						

Log of Boring No. BH-5

Dates Drilled: 10/10/2019 Logged by: RAM Checked By: MBS

Equipment: HAND AUGER Driving Weight and Drop: 140 lbs / 30 in

Ground Surface Elevation (ft): 211 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 SAND (SM):** with clay, yellowish brown.						
5		**OLDER SURFICIAL SEDIMENTS: (Qoa)** yellow brown **SILTSTONE/CLAYSTONE: weathered, thin bedded** few gravel, moist, yellow brown			N/A			
10		moist,			N/A			
		End of Boring at 10 feet below ground level. Borehole backfilled with soil cuttings and compacted on 10/10/2019.						

Converse Consultants

Project Name
Norwalk-La Mirada Unified School District (NLMUSD)
La Mirada High School
13520 Adelfa Drive
La Mirada, California

Project No. **19-31-285-01**

Figure No. **A-6**

Log of Boring No. BH-6

Dates Drilled: 10/10/2019 Logged by: RAM Checked By: MBS

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

Ground Surface Elevation (ft): 198 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
		4" ASPHALT CONCRETE OVER 2" AGGEREGATE BASE						max
		FILL (Af): **SILTY CLAY (CL):** with few sand, yellowish brown.		X				
		OLDER SURFICIAL SEDIMENTS: (Qoa)						
5		**SILTSTONE/CLAYSTONE: weathered, thin bedded** moist, yellowish brown	X		5/10/17	21	107	
10		stiff, moist, brown	X		7/16/23	24	104	

End of Boring at 11.5 feet below ground level.
No groundwater encountered.
Borehole backfilled with soil cuttings and patched with quick set concrete 10/10/2019.

Converse Consultants

Project Name	Project No.	Figure No.
Norwalk-La Mirada Unified School District (NLMUSD) **La Mirada High School** **13520 Adelfa Drive** **La Mirada, California**	**19-31-285-01**	**A-7**

Log of Boring No. BH-7

Dates Drilled: 10/10/2019 | Logged by: RAM | Checked By: MBS

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

Ground Surface Elevation (ft): 211 | Depth to Water (ft): 48 feet below ground level

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		**4" ASPHALT CONCRETE OVER 2" AGGEREGATE BASE**						
		FILL (Af): **CLAYEY SAND (SC):** with gravel, moist, light brown.		X				ma (fc=16%)
		OLDER SURFICIAL SEDIMENTS: (Qoa)						
5		**CLAYSTONE/SILTSTONE: weathered, thinly bedded** moist, light brown to yellowish brown	X		3/6/12	18	110	ds
10		weathered, brown to dark brown	X		4/12/20	21	103	
15		**CLAYSTONE:** white color motling, few rocks and gravel,	X		6/23/41	17	111	
20			X		4/13/18			wa (fc=53%)
25		**CLAYSTONE/SILTSTONE** weathered, white color motling, stiff,	X		16/26/43	19	108	
30			X		4/8/11			wa (fc=55%)

Log of Boring No. BH-7

Dates Drilled: 10/10/2019 Logged by: RAM Checked By: MBS

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

Ground Surface Elevation (ft): 211 Depth to Water (ft): 48 feet below ground level

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
		OLDER SURFICIAL SEDIMENTS: (Qoa) **CLAYSTONE/SILTSTONE: weathered, thinly bedded** white color motling, moist, light brown			6/11/19	23	98	
40		few sand, yellowish brown			7/10/16			
45		**SANDSTONE: weathered, thinly bedded** fine to coarse-grained, yellowish brown			22/50/4 inches	6	104	
50					7/12/21			wa (fc=28%)

End of Boring at 51.5 feet below ground level.
Groundwater encountered at 48 feet below ground level.
Borehole backfilled with cement grout and patched with quick set concrete on 10/10/2019.

Converse Consultants

Project Name
Norwalk-La Mirada Unified School District (NLMUSD)
La Mirada High School
13520 Adelfa Drive
La Mirada, California

Project No. **19-31-285-01**

Figure No. **A-8b**

Log of Boring No. BH-8

Dates Drilled: 10/10/2019 Logged by: RAM Checked By: MBS

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

Ground Surface Elevation (ft): 212 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		**4" ASPHALT CONCRETE OVER 2" AGGEREGATE BASE MATERIAL**		X				ei
		FILL (Af): **SANDY CLAY (CL):** with gravel, light brown.		X				
5		**OLDER SURFICIAL SEDIMENTS: (Qoa)** **CLAYSTONE/SILTSTONE: weathered, thinly bedded** claystone with chalk white rock, moist, light brown to yellowish brown	X		13/21/31	18	106	c
10			X		10/29/41	15	113	
15		**CLAYSTONE: weathered, thinly bedded** stiff, chunks of chalk white rock, brown	X		5/13/20			
20		brown with splots of white, orange and rusty brown red	X		8/13/33	8	111	

End of Boring at 21.5 feet below ground level.
Borehole backfilled with cement grout and backfilled with quick set concrete on 10/10/2019.

Converse Consultants

Project Name
Norwalk-La Mirada Unified School District (NLMUSD)
La Mirada High School
13520 Adelfa Drive
La Mirada, California

Project No. **19-31-285-01**

Figure No. **A-9**

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 one (1) selected sample. 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 test was performed on three (3) 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-1	20	Claystone	53%
BH-1	30	Claystone/ Siltstone	55%
BH-1	50	Sandstone	28%

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*.

Direct Shear

A Direct shear test was performed on three (3) relatively undisturbed sample 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 No. B-3, *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	Siltstone/Claystone	23	210
BH-4	5	Siltstone/Claystone	24	970
BH-7	5	Siltstone/Claystone	18	760

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 sample was tested at field and submerged conditions. The test results, including sample density and moisture content, are presented in Drawing Nos. B-4a and B-4b, *Consolidation Test Results*.

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-3, Expansion Index Test Result

Boring No.	Depth (feet)	Soil Description	Expansion Index	Expansion Potential
BH-8	1-5	Sandy Clay (CL)	56.0	Medium

Soil Corrosivity

One (1) representative soil sample was 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-4, Corrosivity Test Results

Boring No.	Sample Depth (feet)	pH (Caltrans 643)	Soluble Chlorides (Caltrans 422) ppm	Soluble Sulfate (Caltrans 417) (%)	Saturated Resistivity (Caltrans 643) Ohm-cm
BH-3	0-5	8.09	95	0.012	550

R-value

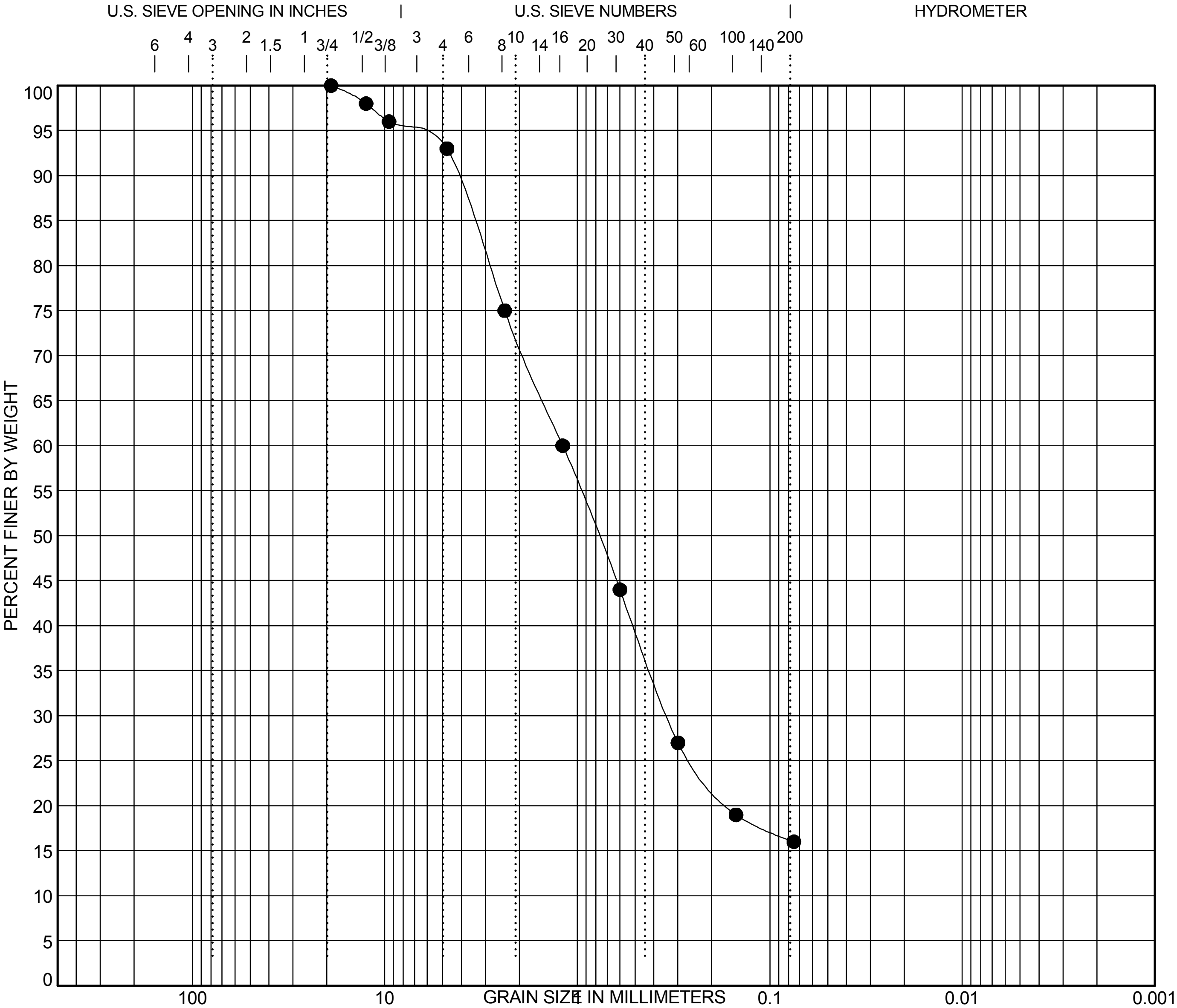
One (1) representative bulk soil sample was tested for resistance value (R-value) in accordance with ASTM D2844 Standard. This test is designed to provide a relative measure of soil strength for use in pavement design. The test results are shown in the following table:

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

Boring No.	Depth (feet)	Soil Classification	Measured R-value
BH-4	1-5	Clay (CL)	13

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.



COBBLES	GRAVEL		SAND			SILT OR CLAY
	coarse	fine	coarse	medium	fine	

	Boring No.	Depth (ft)	Description	LL	PL	PI	Cc	Cu
●	**BH-7**	**0-5**	**Clayey Sand (SC)**					

	Boring No.	Depth (ft)	D100	D60	D30	D10	%Gravel	%Sand	%Silt	%Clay
●	**BH-7**	**5**	**19**	**1.19**	**0.339**		**7.0**	**77.0**	**16.0**	

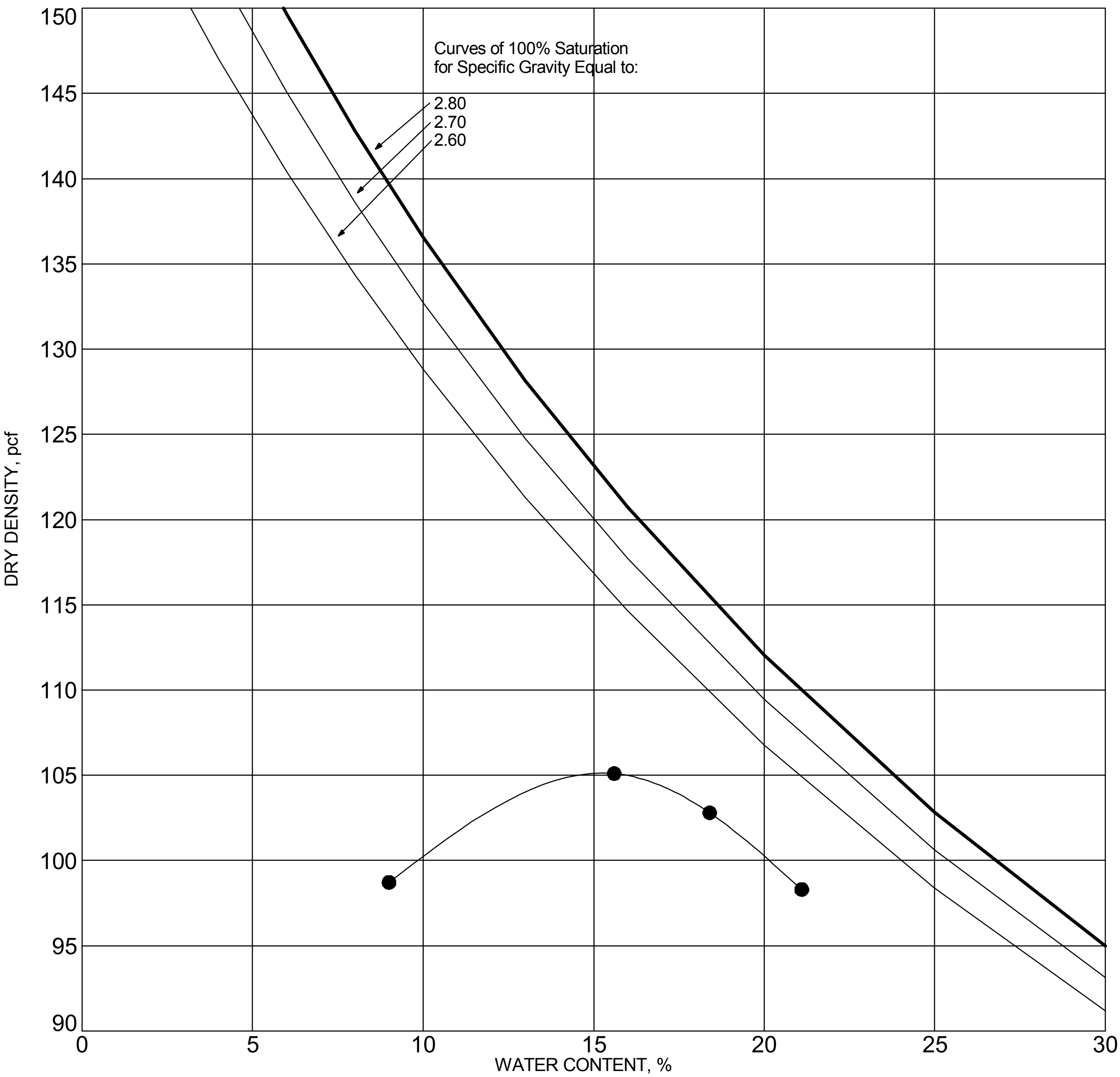
GRAIN SIZE DISTRIBUTION RESULTS

Converse Consultants

Project Name
Norwalk-La Mirada Unified School District (NLMUSD)
15711 Pioneer Blvd., Bldg. G
Norwalk, CA 90650-2894

Project No.
19-31-285-01

Figure No.
B-1



SYMBOL	BORING NO.	DEPTH (ft)	DESCRIPTION	ASTM TEST METHOD	OPTIMUM WATER, %	MAXIMUM DRY DENSITY, pcf
●	BH-2	0-5	Silty Clay (CL)	D1557 Method B	15.1	105.5

NOTE:

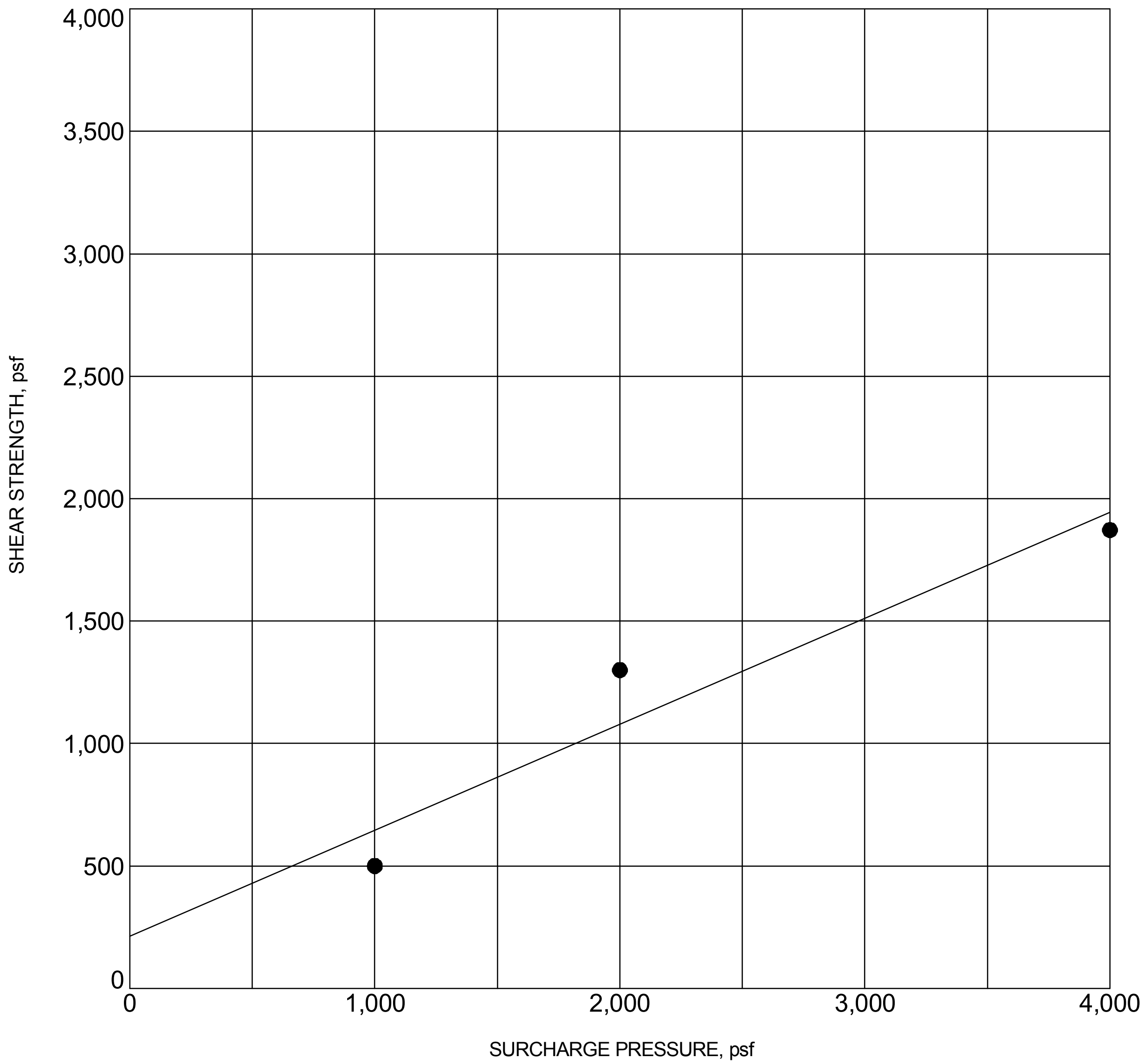
MOISTURE-DENSITY RELATIONSHIP RESULTS

Converse Consultants

Project Name
Norwalk-La Mirada Unified School District (NLMUSD)
15711 Pioneer Blvd., Bldg. G
Norwalk, CA 90650-2894

Project No.
19-31-285-01

Figure No.
B-2



BORING NO. :	**BH-1**	DEPTH (ft) :	**5**
DESCRIPTION :	**Siltstone / Claystone**		
COHESION (psf) :	**210**	FRICTION ANGLE (degrees) :	**23**
MOISTURE CONTENT (%) :	**14.0**	DRY DENSITY (pcf) :	**115.0**

NOTE: Ultimate Strength.

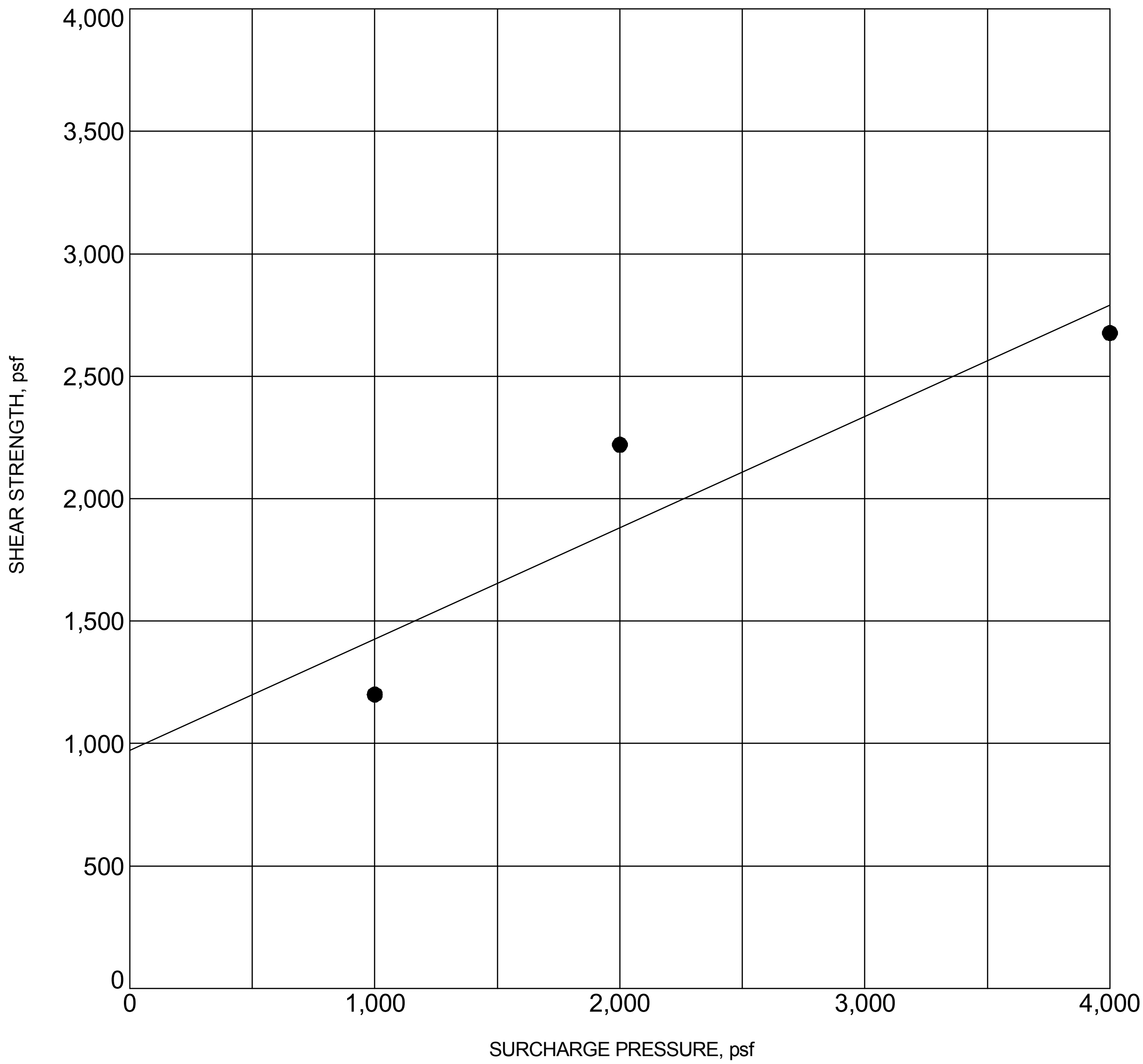
DIRECT SHEAR TEST RESULTS

Converse Consultants

Project Name
Norwalk-La Mirada Unified School District (NLMUSD)
15711 Pioneer Blvd., Bldg. G
Norwalk, CA 90650-2894

Project No.
19-31-285-01

Figure No.
B-3a



BORING NO.	:	**BH-4**	DEPTH (ft)	:	**5**
DESCRIPTION	:	**Claystone / Siltstone**			
COHESION (psf)	:	**970**	FRICTION ANGLE (degrees)	:	**24**
MOISTURE CONTENT (%)	:	**24.0**	DRY DENSITY (pcf)	:	**98.0**

NOTE: Ultimate Strength.

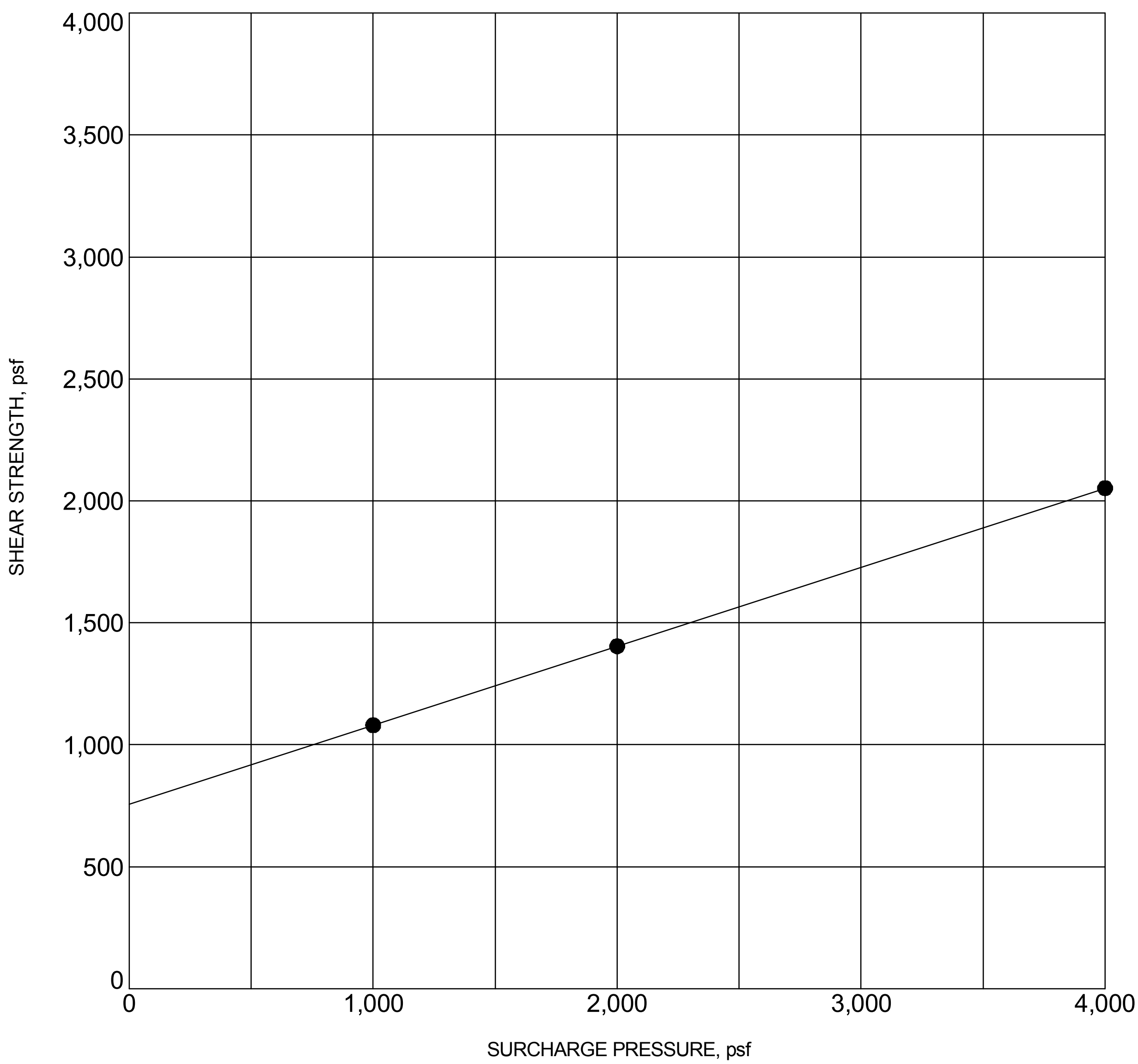
DIRECT SHEAR TEST RESULTS

Converse Consultants

Project Name
Norwalk-La Mirada Unified School District (NLMUSD)
15711 Pioneer Blvd., Bldg. G
Norwalk, CA 90650-2894

Project No.
19-31-285-01

Figure No.
B-3b



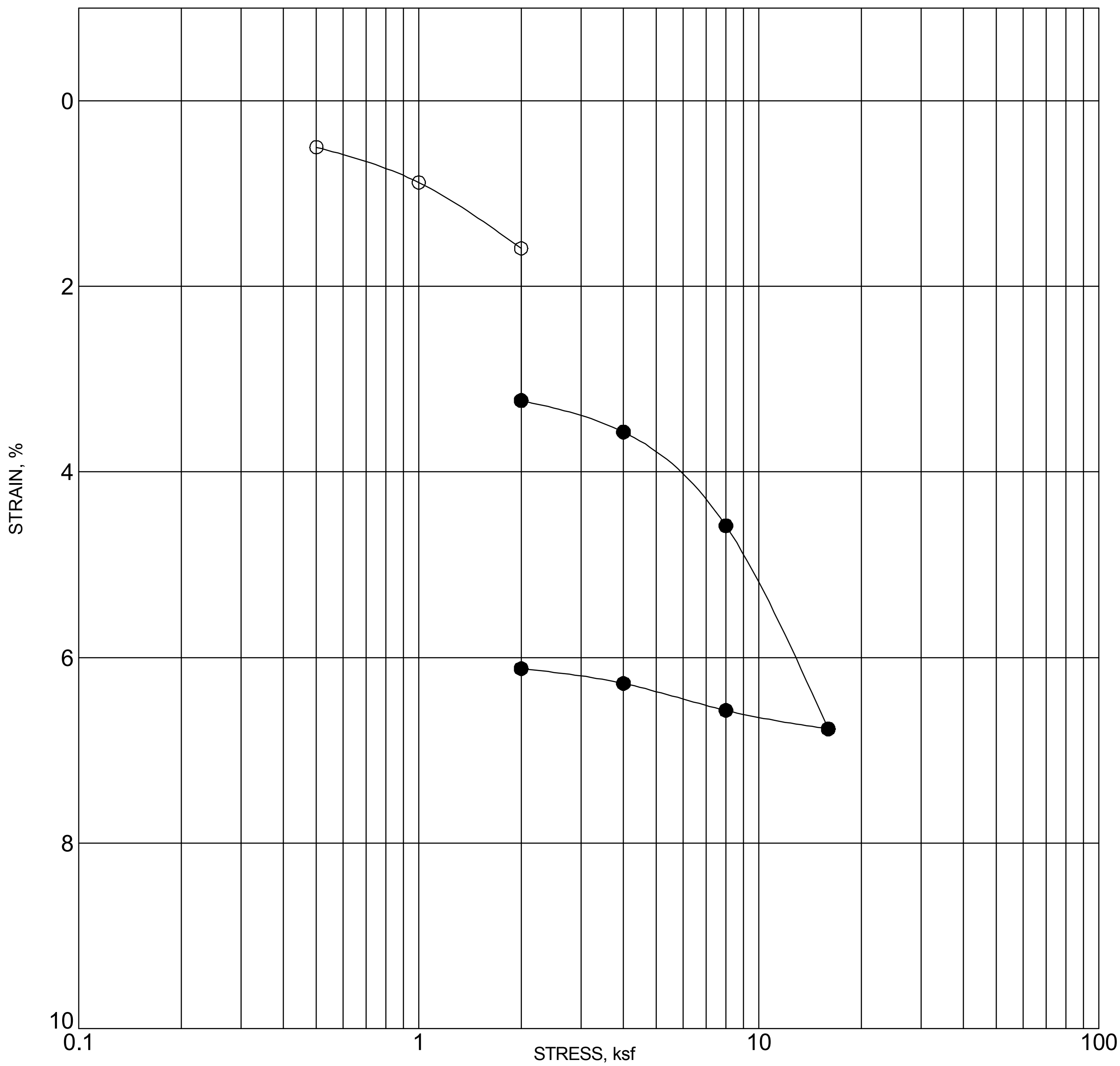
BORING NO.	:	**BH-7**	DEPTH (ft)	:	**5**
DESCRIPTION	:	**Claystone / Siltstone**			
COHESION (psf)	:	**760**	FRICTION ANGLE (degrees)	:	**18**
MOISTURE CONTENT (%)	:	**18.0**	DRY DENSITY (pcf)	:	**110.0**

NOTE: Ultimate Strength.

DIRECT SHEAR TEST RESULTS

Converse Consultants

Project Name	Project No.	Figure No.
Norwalk-La Mirada Unified School District (NLMUSD) **15711 Pioneer Blvd., Bldg. G** **Norwalk, CA 90650-2894**	**19-31-285-01**	**B-3c**



BORING NO. :	**BH-2**	DEPTH (ft) :	**5**
DESCRIPTION :	**Siltstone/ Claystone**		

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	**20**	**101**	**85**	**0.785**
FINAL				

NOTE: SOLID CIRCLES INDICATE READINGS AFTER ADDITION OF WATER

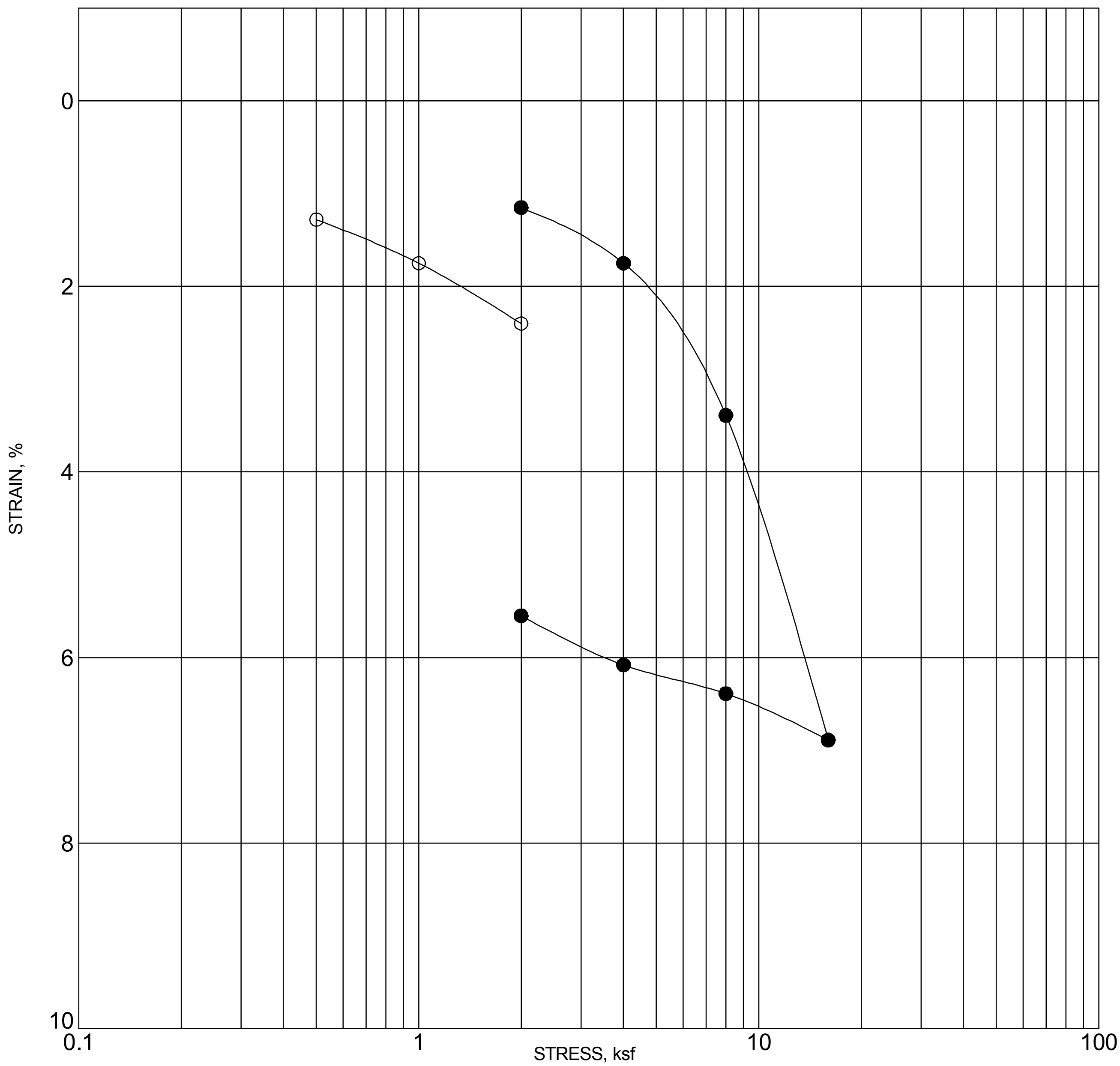
CONSOLIDATION TEST RESULTS

Converse Consultants

Project Name
Norwalk-La Mirada Unified School District (NLMUSD)
15711 Pioneer Blvd., Bldg. G
Norwalk, CA 90650-2894

Project No.
19-31-285-01

Figure No.
B-4a



BORING NO. :	BH-8	DEPTH (ft) :	5
DESCRIPTION :	**Claystone / Siltstone**		

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	**18**	**105.0**	**84**	**0.727**
FINAL				

NOTE: SOLID CIRCLES INDICATE READINGS AFTER ADDITION OF WATER

CONSOLIDATION TEST RESULTS

Converse Consultants

Project Name
Norwalk-La Mirada Unified School District (NLMUSD)
15711 Pioneer Blvd., Bldg. G
Norwalk, CA 90650-2894

Project No.
19-31-285-01

Figure No.
B-4b

Appendix C

Earthwork Specifications

APPENDIX C: EARTHWORK SPECIFICATIONS

Scope of Work

The work includes all labor, supplies and construction equipment required to construct the building pads 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 3 feet 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 five (5) 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.

Appendix D

Percolation Testing

APPENDIX D: PERCOLATION TESTING

Percolation test was performed utilizing exploratory boring BH-2, on October 10, 2019. The boring were pre-soaked prior to performing a falling-head percolation test to determine infiltration rates of the fill and native soils encountered between depths of 0 to 10 feet below the ground surface at the Borings in accordance with LA County Low Impact Development, Best Management Practices Guidelines. The test borings were 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 boring was 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 2 hours. After pre-soak, water was added to the bore hole until the water level was as near ground surface as could be achieved. The water level was measured to the nearest 1/10-foot and recorded every 30 minutes for 90 minutes for BH-2 and the water level was measured to the nearest 1/10-foot. There were three (3) sets of measurements taken and each set consisted of at least three (3) measurements. The results of the percolation tests are tabulated below.

Table No. D-1, Percolation Test Results

Boring No.	Depth of Boring* (feet)	Predominant Soil Types (USCS)	Average Percolation Rate (inches/hour)	Lowest Percolation Rate (inches/hour)
BH-2	0-10	Silty Clay, Siltstone/Claystone	0.19	0.08

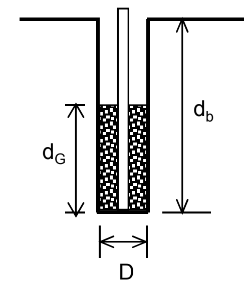
*Approximate

In accordance with County of Los Angeles requirements, the minimum percolation rate for design of infiltration system for storm water management is 0.30 inch per hour. Percolation rates of BH-2 do not meet the minimum percolation rate. The project Civil Engineer should review the raw data of percolation test presented herein to determine specific soil layers and percolation rates for design of the proposed infiltration system. Such systems should be constructed a minimum distance of 10 feet laterally from any existing or future planned building or subsurface structure as not to disturb or undermine foundations. The proposed infiltration system should be constructed above historical ground water table. The percolation rates were determined in general accordance with Los Angeles County guidelines. The detailed percolation test results are shown on the following data sheet.

Percolation Testing

Job Name: La Mirada HS
Job No.: 19-31-285-01
Location: Base Ball Field
Test Date: October 10, 2019

Test Boring No **BH-2**
Depth of Boring (d_b): 10.0 feet
Diameter of Boring (D): 0.67 feet
Test Performer: PA



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											
9:00:00 AM	9:30:00 AM	0.50	0.00	0.40	10.00	9.60	0.40	9.80	9.60	30.3	0.32
9:30:00 AM	10:00:00 AM	0.50	0.40	0.70	9.60	9.30	0.30	9.45	7.20	29.2	0.25
10:00:00 AM	10:30:00 AM	0.50	0.70	0.90	9.30	9.10	0.20	9.20	4.80	28.5	0.17
10:30:00 AM	11:00:00 AM	0.50	0.00	0.30	10.00	9.70	0.30	9.85	7.20	30.4	0.24
11:00:00 AM	11:30:00 AM	0.50	0.30	0.50	9.70	9.50	0.20	9.60	4.80	29.7	0.16
11:30:00 AM	12:00:00 PM	0.50	0.50	0.70	9.50	9.30	0.20	9.40	4.80	29.1	0.17
12:00:00 PM	12:30:00 PM	0.50	0.00	0.30	10.00	9.70	0.30	9.85	7.20	30.4	0.24
12:30:00 PM	1:00:00 PM	0.50	0.30	0.40	9.70	9.60	0.10	9.65	2.40	29.8	0.08
1:00:00 PM	1:30:00 PM	0.50	0.40	0.50	9.60	9.50	0.10	9.55	2.40	29.5	0.08

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

Lowest Perlcolaton Rate = 0.08 inch/hr
Average Percolation Rate = 0.19 inch/hr

Reference: Los Angeles County (2014). Administrative Manual - Guidelines for Design, Investigation, and Reporting Low Impact Development Storm Water Infiltration