REPORT OF GEOTECHNICAL INVESTIGATION
PROPOSED CTE BUILDING ADDITION
CLARK MAGNET HIGH SCHOOL
4747 NEW YORK AVENUE
GLENDALE, CALIFORNIA

Prepared for

GLENDALE UNIFIED SCHOOL DISTRICT
349 W. Magnolia Avenue
Glendale, California 91204

Submitted by

GROUP DELTA CONSULTANTS, INC.
370 Amapola Avenue, Suite 212
Torrance, California 90501

GDC Project No. LA-1420

March 16, 2020

Glendale Unified School District
349 W. Magnolia Avenue
Glendale, California 91204

March 16, 2020
Group Delta Project No. LA 1420

Attention: Mr. Jeffrey Bohn
Project Manger

Subject: Proposal for Report of Geotechnical Investigation
Proposed CTE Building Addition
Clark Magnet High School
4747 New York Avenue
Glendale, California

Dear Bohn:

Group Delta Consultants (GDC) is pleased to submit the results of our geotechnical investigation for the proposed building addition to be constructed at the campus of the Clark Magnet High School in Glendale, California. Our scope of work was conducted in general accordance with our proposal dated September 13, 2019, and the Independent Consultant Agreement between Glendale Unified School District and Group Delta Consultants, Inc. dated October 2, 2019 (Professional Service No. 557), and Task Order dated November 7, 2019 (PO No. 0020103480.

The results of our investigation and design recommendations are presented in this report. Please note that you or your representative should submit copies of this report to the appropriate governmental agencies for their review and approval prior to obtaining a building permit.

We appreciate the opportunity to provide geotechnical services for this significant project. Should you have any questions regarding this report, or if we can be of further service, please do not hesitate to contact us.

Sincerely,

GROUP DELTA CONSULTANTS, INC.

Ethan Tsai, G.E.
Associate Engineer

REGISTERED PROFESSIONAL ENGINEER
ETHAN TSAI
GE 3004
Exp 12/31/21
GEOTECHNICAL
STATE OF CALIFORNIA

Michelle A. Sutherland, C.E.G.
Senior Engineering Geologist.

PROFESSIONAL GEOLOGIST
MICHELLE ALLISON SUTHERLAND
No. EG2577
EXP. 2/29/20
CERTIFIED ENGINEERING GEOLOGIST
STATE OF CALIFORNIA

Distribution: Addressee (electronic copy)

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1.0 INTRODUCTION

This report presents our recommendations for the foundation design of the proposed CTE building addition. The vicinity map and site location are shown on Figure 1. The exploration locations are shown on Figures 2.

1.1 Scope of Work

This investigation was authorized to determine the static physical characteristics of the soils at the site of the proposed CTE building addition, and to provide recommendations for foundation and retaining wall design, floor slab support, and grading for the development. We were to evaluate the existing soil and groundwater conditions at the site, including the corrosion potential of the soils, and develop recommendations for the following:

- Review available published geotechnical and geologic reports, maps, and subsurface data for the surrounding area;
- Perform a geotechnical field investigation to evaluate subsurface conditions;
- Evaluate geologic and seismic hazards including local seismicity, surface fault rupture, ground shaking, liquefaction, and other considered geologic hazards;
- Provide geotechnical recommendations for site grading;
- Evaluate geotechnical data and perform geotechnical analyses to develop foundation recommendations for the proposed new construction;
- Provide pavement design recommendations;
- Evaluate expansion potential and corrosivity of on-site soils; and
- Prepare this geotechnical design report.

1.2 Project Description

The project site is located at northwest side within the campus of the existing Clark Magnet High School in Glendale, California, as shown on Figure 1. The proposed building will be a 1 story structure and will be supported near the existing grade. The finished floor elevation will be at about Elevation 1,825 feet.

The site is currently occupied by an asphalt concrete paved parking lot to the south and an existing slope up to 17 feet in height ascending to the north. The inclination of the existing slope is slope generally 5:1 (horizontal:vertical) with the steepest portion of about 3:1.

The construction of the new building will require a cut into the existing slope. The cut will be supported by planned retaining walls, extending up to 6 feet in height along the north and west walls of the new building and up to 11 feet in height, around the perimeter of the north and west walls of the new building. The locations of the proposed structure and retaining wall are shown on Figure 2.

The maximum dead-plus-live column load is not available at this time. However, we understand that the proposed structure is anticipated to be lightly loaded.

The proposed ground profiles are shown on Figure 3, Cross Sections.

2.0 GEOTECHNICAL FIELD INVESTIGATION

The soil conditions beneath the site were explored by drilling two borings to depths between of 20 to 25 feet below the existing grade at the locations shown on Figure 2. Details of the explorations and the logs of the borings are presented in Appendix A.

In addition, we performed a geophysical study using ReMi technique (Refraction Microtremor) to develop shear-wave velocity profile of the site. Details of the ReMi profile are presented in Appendix A.

3.0 LABORATORY TESTING PROGRAM

Laboratory tests were performed on selected samples obtained from the borings to aid in the classification of the soils and to determine the pertinent engineering properties of the foundation soils. The following tests were performed:

- Soil Classification: USCS (ASTM D 2487) and Visual Manual (ASTM D 2488);
- Moisture content (ASTM D 2216) and Dry Unit Weight (ASTM D 2937);
- Grain Size Distribution and Percent Passing No. 200 Sieve (ASTM D 1140);
- R-value (ASTM D2844, CTM 301));
- Soil Corrosivity:
    - pH (CTM 643);
    - Water-Soluble Sulfate (ASTM D 516, CTM 417);
    - Water-Soluble Chloride (Ion-Specific Probe, CTM 422);
    - Minimum Electrical Resistivity (CTM 643);

A detailed description of the prior laboratory testing program and test results are presented in Appendix B.

4.0 SITE AND SUBSURFACE CONDITIONS

4.1 Site Conditions

The site is currently occupied by an asphalt concrete paved parking lot to the south and an existing slope ascending up to 17 feet in height to the north. The inclination of the existing slope is sloped generally in 5:1 (horizontal:vertical) with the steepest portion of about 3:1. The top of the slope is at an elevation of 1,842 feet above Mean Sea Level (MSL). The existing parking lot is at an elevation of about 1,825 feet. The topography map is shown on Figure 2.

4.2 Subsurface Materials

Existing fill soils, about 1 foot thick, were found in Boring B-1 located at the top of the slope. The existing fill soils consists of silty sand covered by mulch that is used for landscape purpose. Existing fill soil was not encountered in Boring B-2 which is located at the existing parking lot area. However, deeper fill could occur elsewhere between borings.

The natural soils consist of very dense sand, gravels and cobbles, and possible decomposed granite at depth within borings. Basement rock of grantic assemblage outcrop within the mountains to the northeast and southwest of the site as well as within the valley floor.

4.3 Groundwater

Groundwater was not encountered within the 25-foot depth explored. The historic high ground water contour is not developed near at the project site from the Seismic Hazard Zone Report for Burbank Quadrangle (CGS, 1998). Based on the nearby borehole data about 0.9 miles southwest of the site from the CGS Borehole Database (https://maps.conservation.ca.gov/cgs/informationwarehouse/bhdb/), no groundwater was encountered within total depth of 61 feet explored from the Well I11823B1 (Official Name 000012_00001_34118B2). Weathered granitic basement rock was encountered at 21 feet depth and less weathered rock at 35 feet depth.

5.0 GEOLOGIC AND SEISMIC HAZARD EVALUATION

5.1 Geologic Setting

The project site is situated within a shallow valley floor of the Transverse Ranges Geomorphic Province of southern California. The San Gabriel Mountains are to the northeast and Verdugo Mountains to the southwest. The Sierra Madre fault zone trends northwest along the southern flank of the San Gabriels, approximately 0.6 miles northeast of the site. Most of the valley floor is thinly blanketed with course grained granitic sediment (Qyf); cobbles and boulders are common. Granitic rock is exposed within the local mountains and in isolated areas within the valley.

5.2 Seismic Setting

The site is located within the seismically active area of southern California and there is a high potential for the site to experience strong ground shaking from local and regional faults. These hazards and their potential impact can be mitigated with proper seismic design. The intensity of ground shaking is highly dependent upon the distance of the site to the earthquake source, the magnitude of the earthquake, and the underlying soil conditions. Data evaluated for the regional fault and seismic hazard at the site was obtained from U.S. Geological Survey (USGS) and California Geological Survey (CGS) online earthquake catalog and Quaternary Fault Database resources unless otherwise noted. The site in relation to regional seismic faults and significant historical earthquake epicenters is presented in Figure 5, Regional Seismicity and Fault Map.

5.3 Seismic History

The school transitioned from a junior high to the current high school in 1998 to address overcrowding. At that time the campus underwent significant remodeling. There is no known history of earthquake damage at the site, however there is a significant history of earthquake events in the local area. Local historical earthquakes recorded within a 100 km radius of the site from 1955 to present include 371 recorded events with M4.0 or greater (USGS, 12/30/2019). Of the 371 events, 6 were M6.0 and greater and include the 1971 M6.6 San Fernando Earthquake and the 1994 M6.7 Northridge Earthquake. Forty-five recorded events were M5.0 to less than M6.0 earthquakes. The closest recorded seismic event is epicentered about 7.5 kilometers northwest of the site. The Event is a M4.0 earthquake in 1971 shortly following the surface rupture event during the San Fernando 1971 earthquake. While not within the search radius, earthquakes of M7.0 and greater have been recorded in southern California. As recently as 2019, a M7.1 earthquake ruptured about 185 kilometers north, northeast of the site. A M7.5 earthquake occurred in 1952 located about 100 kilometers northwest of the site and a M7.3 earthquake in 1992 was located about 167 kilometers east of the site. Construction in this area should be designed with accepted engineering practices and in compliance with current building codes that accommodate strong seismic ground motion.

A list of nearby active faults considered capable of producing significant shaking at the site is provided in Table 1 below:

Table 1: List of Known Earthquake Faults Closest to the Subject Site

Abbreviated Fault Name	Fault Type	Max. Magnitude (Mw)	Slip Rate (mm/yr)	Approximate Closest Distance* (Km)
Sierra Madre	Reverse	7.2	2	0.83
Verdugo	Reverse	6.9	0.5	5.69
San Gabriel	Strike Slip	7.3	1	9.16
Raymond	Strike Slip	6.8	1.5	13.23
Hollywood	Strike Slip	6.7	1	13.35
Northridge	Thrust	6.9	1.5	13.92
S. San Andreas	Strike Slip	8+	N/A	36.9

Notes: USGS, Accessed December 31, 2019. *- Input from 2008 USGS Seismic Hazard Source Maps online data base

While the site is located within a seismically active area of southern California, proper engineering design can mitigate the potential hazard to life and property. Significant geologic hazards include ground surface rupture, landslide, and liquefaction. A geologic hazard evaluation was performed through review of published reports and maps made available by USGS and CGS as well as the City of Glendale General Plan Safety Element (2003). The project site is not located within an Earthquake Zone of Required Investigation (EZRI) or a Fault Hazard Management Zone (FHMZ). A discussion of these hazards and other significant geologic hazards is presented below.

5.1 Surface Rupture Potential

The closest known Earthquake Fault is the Sierra Madre fault, which trends northwest about 0.83 km northeast of the site. The Sierra Madre fault is a reverse fault. It dips about 53 degrees to the northeast below the San Gabriel Mountains locally. Segments of the fault are mapped within EZRI and FMHZ. The project site is not located within an EZRI for or FHMZ for known Earthquake Faults. There are no known Earthquake Faults trending toward the project site nearby. The potential hazard for surface fault rupture is considered low.

5.2 Liquefaction Potential

Liquefaction involves sudden loss in strength of a saturated, cohesionless soil caused by the build-up of pore water pressure during cyclic loading, such as that produced by an earthquake. This increase in pore water pressure can temporarily transform the soil into a fluid mass, resulting in differential settlements and ground deformations. Typically, liquefaction occurs in areas where there are loose soils and the depth to groundwater is less than 50 feet from the surface. Seismic shaking can also cause soil compaction and ground settlement without liquefaction occurring, including settlement of dry sands above the water table.

According to the State of California Seismic Hazards Zone Map for the Burbank Quadrangle (CGS, 1998), the campus is not located within a zone of required investigation for liquefaction, shown in Figure 6. The surface soils consist of very dense sand, gravel, and cobbles and are underlain by granitic rock which is not susceptible to liquefaction. Therefore, the potential of liquefaction during strong earthquake is considered low.

5.3 Landslides

The project site is situated centrally within a shallow valley, nested between the San Gabriel Mountains in the northeast and Verdugo Mountains in the southwest. Landslides are common within the steep mountain canyon slopes and foothills. The closest significant natural slopes are over 0.5 miles away from the site and not considered a potential hazard at the site. There is a 17-foot high slope at the project site which is planned to be graded and retained with benched walls, 6 feet and 11 feet in maximum height. The existing slope ascends to the northeast no steeper than 3:1 and is generally 5:1. The potential hazard for landslide at the site is considered low as long as construction of new walls is performed according to governing building codes and OSHA regulations.

5.4 Flooding, Seiche, Tsunami, and Inundation

The site is situated centrally within a natural flood plain for the San Gabriel Mountains. Seasonally debris flows and mudslides can occur and be hazardous. Flood control basins have been constructed across the base of the mountains to mitigate debris flow and flood hazard in the developed areas of the valley flood plain. Main drainage pathways extending from canyons have been lined with concrete and/or culverted to control and direct flooding from the mountain shed. The flood control channel for the Dunsmore Canyon Channel is located west of the site, flowing south. These flood control methods are maintained and monitored by the Los Angeles County Public Works and City of Glendale. The site is not located within a flood hazard area as identified in the City of Glendale Safety Element. The potential hazard for flooding as the result of heavy rain fall at the site is low as long as drainage for the project is engineered and maintained properly. The site is not located within an inundation hazard area as identified by the City of Glendale Safety Element. Therefore, the potential hazard for seiche and inundation is considered low. Lastly, the potential hazard of tsunami at the site is considered nil due to the height in elevation at the site (El. 1825 feet) and distance from the nearest coastline (>20 miles).

5.5 Naturally Occurring Hazardous Elements

Naturally occurring hazardous elements considered for the local southern California region include, radon, asbestos, and methane. According to available online maps and data provided by the CGS, the site has a low potential hazard for these elements.

6.0 DISCUSSION AND RECOMMENDATIONS

6.1 General

The site soils consist of very dense sand, gravels and cobbles. The structure may be supported on spread footings established in the dense undisturbed natural soils or properly compacted fill soils. If the recommendations on grading are followed, the floor slab may be supported on grade.

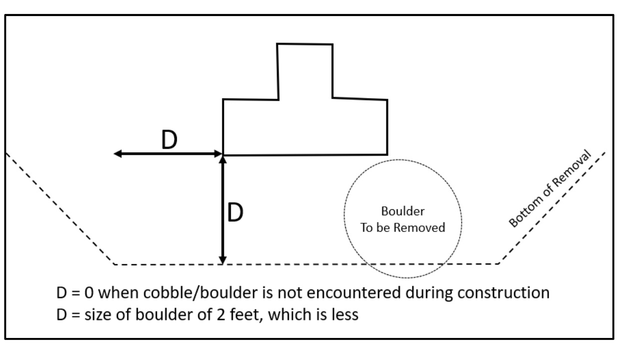
The proposed building footprint will extend into an existing 17-foot high slope. Stepped retaining walls are proposed to support a cut into the slope. We recommend that permanent soldier pile retaining wall may be used. Alternatively, conventional cantilevered retaining wall may also be used. Temporary shoring or laid back slope may be required during construction of the retaining walls.

6.2 Demolition

Prior to the start of earthwork, the existing structural elements on the site will require demolition and removal, including the existing foundations, slabs, pavements, walls and utilities. It should be anticipated that the remnants of previous construction could be encountered anywhere on the site. The civil engineer should identify the presence and location of all existing utilities on and adjacent to the site. Precautions will be required to remove, relocate or protect existing utilities, as appropriate.

6.3 Removal

Any suspected fill or any loose soils should be removed and recompacted. In addition, demolition activities may create disturbance of near surface soils, which will also require removal and recompaction. It should also be noted that cobbles and boulders may be encountered, if removal of a boulder or cobble causes a void in the subgrade below foundations, the subgrade should be overexcavated to provide a uniform subgrade, i.e. uniform fill thickness. The uniform subgrade should extend below foundations to a distance of equal to size of cobble and/or boulder, or 2 feet, which is lesser. The uniform subgrade should extend laterally to a distance of depth of fill below foundation. Requirement of removal may be illustrated below:



The actual limits for removals or recompaction should be determined by the project geotechnical engineer during grading, based on the actual conditions encountered.

The civil engineer should identify the presence and location of all existing utilities and underground storage tanks in and near the work area. Precautions should be taken to remove, relocate or protect existing utilities and underground storage tank, as appropriate.

6.4 Earthwork

All grading should conform to the requirements of the 2019 California Building Code, and the general grading recommendations outlined below.

1. The grading contractor is responsible for notifying the project geotechnical engineer of a pre-grading meeting prior to the start of excavation/grading operations and anytime that the operations are resumed after an interruption.

2. Prior to the start of earthwork existing improvements will require demolition. The project civil engineer should locate any existing utilities in the area. Existing utilities should be removed, relocated or protected, as appropriate.

3. Where excavations are deeper than about 4 feet, the sides of the excavations should be sloped back at 1½:1 (horizontal to vertical) or shored for safety. Unshored excavations should not extend below a plane drawn at 1½:1 (horizontal to vertical) extending downward from adjacent existing footings.

4. The bottoms of excavations should be scarified to a depth of 6 inches, brought to near-optimum moisture content, and rolled with heavy compaction equipment. At least the upper 6 inches of the exposed soils should be compacted to at least 95% of the maximum dry density obtainable by the ASTM Designation D1557 method of compaction.

5. Structural fill or backfill should be compacted to at least 95 percent of the maximum dry density. Fill placed in non-structural areas should be compacted to at least 90 percent of the maximum dry density. The moisture content of the on-site soils at the time of compaction should vary no more than 2% below or above optimum moisture content. The moisture content of the on-site clayey soils at the time of compaction should be between 2% and 4% above optimum moisture content.

6. The on-site soils, less any debris or organic matter, may be used in required fills. All structural fill soils should be sandy soils, free of highly expansive clay, organics, debris, rocks greater than 3 inches in any dimension, and other deleterious material. All fill soils shall be approved by the project geotechnical engineer.

7. Any required import material should consist of relatively non-expansive soils with an expansion index of less than 35. The imported materials should contain sufficient fines (binder material) so as to be relatively impermeable and result in a stable subgrade when compacted. Import soils should be approved before being brought to the site. If expansive soils are found, they should be removed and replaced with non-expansive compacted fill.

8. All earthwork and grading should be performed under the observation of GDC. Compaction testing of the fill soils shall be performed at the discretion of GDC. Testing should be performed for approximately every 2 feet in fill thickness or 500 cubic yards of fill placed, whichever occur first. If specified compaction is not achieved, additional compactive effort, moisture conditioning of the fill soils, and/or removal and recompaction of the below-minimum-compaction soils will be required.

9. All materials used for asphalt concrete and aggregate base shall conform to the most recent version of the “Green Book” specifications or the equivalent, and shall be compacted to at least 95 percent relative compaction.

If, in the opinion of the geotechnical engineer, contractor, or owner, an unsafe condition is created or encountered during grading, all work in the area shall be stopped until measure can be taken to mitigate the unsafe condition. An unsafe condition shall be considered any condition that creates a danger to workers, on-site structures, on-site construction, or any off-site properties or persons.

6.5 Temporary Excavation and Shoring

Excavations up to about 17 feet to the existing slope are planned. If conventional cantilevered retaining wall is being used for the support of the existing slope, the existing slope behind the proposed retaining wall may be laid back at 1½:1. Alternatively, temporary shoring will be required to support the excavation. Conventional soldier beams with lagging may be used for shoring. This method of shoring would consist of steel soldier piles placed in drilled holes, backfilled with concrete, and either tied back with earth anchors or braced internally. The tie-back anchors will have to be planned to avoid utilities in the street.

Some difficulty may be encountered in the drilling because of caving in the sandy alluvial fan deposits. Special techniques and measures may be necessary in some areas to permit the proper installation of the soldier piles.

Excavations can be readily accomplished with light to heavy effort using conventional heavy duty grading equipment such as scrapers, loaders, dozers, and excavators.

6.5.1 Lateral Pressure

For design of cantilevered shoring, we recommend using a triangular pressure distribution for calculating earth pressures. An active earth pressure equal to that of a fluid with a density of 30 pcf may be used for *level* retained ground. However, where the required soils are sloped back at up to 4:1 above the shoring, it may be assumed that the soils will exert an active lateral earth pressure equal to that of a fluid with a density of 34 pcf.

In addition to the recommended earth pressure, the upper 10 feet of shoring adjacent to normal vehicular traffic should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the shoring due to normal traffic. If the traffic is kept back at least 10 feet from the shoring, the traffic surcharge may be neglected. Furthermore, the shoring should be designed to resist any lateral surcharge pressure imposed resulting from loads placed above the excavation and within a 1:1 plane extending upward from the base of the excavation.

6.5.2 Design of Soldier Pile

For design of soldier piles embedded in compacted fill or formational materials, and spaced at least 2 pile diameters on centers, an allowable passive pressure of 600 psf per foot of embedment (over twice the pile width) up to a maximum of 6,000 psf may be used. To develop the full passive pressure, provisions should be taken to assure firm contact between the soldier piles and the undisturbed soils.

The concrete placed in the solider pile excavations may be a lean-mix concrete. However, the concrete used in that portion of the soldier pile which is below the planned excavated level should be of sufficient strength to adequately transfer the imposed loads to the surrounding soils. If lean-mix concrete is used around the soldier pile below the planned excavation level, only the passive resistance developed by the steel soldier pile itself may be used, not the entire diameter of the drilled hole.

Caving may be anticipated during drilling. Special technique, such as casing or drilling mud may be used to prevent caving. In addition, either lean-mix concrete or structural concrete should be pumped from the bottom up through a rigid pipe extending to the bottom of the drilled excavation, with the pipe being slowly withdrawn as the concrete level rises. The discharge end of the pipe should be at least 5 feet below the surface of the concrete at all times during placement. The discharge pipe should be kept full of concrete during the entire placing operation and should not be removed from the concrete until all of the concrete is placed and fresh concrete appears at the top of the pile. The volume of concrete pumped into the hole should be recorded and compared to design volume.

6.5.3 Lagging

Continuous lagging will be required throughout. The soldier piles and anchors should be designed for the full-anticipated lateral pressure. However, the pressure on the lagging will be less due to arching in the soils. We recommend that the lagging be designed for the recommended earth pressure but may be limited to a maximum value of 400 psf.

6.5.4 Deflection

It is difficult to accurately predict the amount of deflection of a shored excavation. It should be realized, however, that some deflection will occur. We estimate that this deflection could be on the order of about ½ inch at the top of a up to 17-foot deep shored excavation. If greater deflection occurs during construction, additional bracing may be necessary to minimize damage to utilities in the adjacent streets. A greater lateral pressure could also be used in the shoring design to reduce deflection.

6.5.5 Monitoring

Some means of monitoring the performance of the shoring system and permanent retaining wall is recommended. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all the soldier piles and wall. We will be pleased to discuss this further with the design consultants and the contractor when the design of the shoring system and retaining wall has been finalized.

6.6 Foundations – Spread Footing

6.6.1 Bearing Value

Spread footings extending at least one foot into the undisturbed natural soils and at least 2 feet below the floor slab or lowest adjacent grade, may be designed to impose a net dead-plus-live load pressure of 4,000 pounds per square foot. The allowable bearing pressure may be increased by one-third when considering temporary loads associated with wind and seismic loading. The recommended bearing value is a net value, and the weight of concrete in the footings can be taken as 50 pounds per cubic foot; the weight of soil backfill can be neglected when determining the downward loads.

Footing excavations should be observed by the project geotechnical engineer before placement of concrete to verify that the foundation conditions meet the requirements of the geotechnical report. The project geotechnical engineer may perform compaction tests, probing, or use other methods, to verify that the foundations will be supported in competent soils. If disturbed, wet, or otherwise unsuitable soils are encountered, or if water saturates the soils, the soils shall be excavated or stabilized as recommended by the project geotechnical engineer

6.6.2 Settlement

We estimate the settlement of the structure supported on spread footings in the manner recommended is expected to be about ½ inch or less. The differential settlement between adjacent columns is expected to be ¼ inch or less.

6.6.3 Lateral Capacity

Resistance to lateral loads can be provided by friction developed between the bottom of footings and the supporting soil, and by the passive soil pressure developed on the face of the footing. For preliminary design purposes, an allowable passive fluid pressure of 300 pcf and a coefficient of friction of 0.45 may be used for lateral sliding resistance of footings.

The recommended bearing and lateral load design values stated above are for use with loadings determined by a conventional working stress design. When considering an ultimate design approach, the recommended design values shall be multiplied by the following factors:

Design Item	Ultimate Design Factor
Bearing Value	3.0
Passive Pressure	1.5
Coefficient of Friction	1.5

If strength design is being used, resistance factors for foundation designed provided in the table below may be used.

Design Item	Resistance Factors, ϕ
Bearing Value	0.45
Passive Pressure	0.5
Coefficient of Friction	0.85

6.7 Floor Slab

If recommendations for earthwork provided in this report are followed, the floor slab may be supported on grade.

Construction activities and exposure to the environment can cause deterioration of the prepared subgrade. Therefore, we recommend our that our field representative observe the condition of the final subgrade soils immediately prior to slab-on-grade construction, and, if necessary, perform further density and moisture content tests to determine the suitability of the final prepared subgrade.

If vinyl or other moisture-sensitive floor covering is planned for portions of the development, we recommend that the floor slab in those areas be underlain by a capillary break consisting of a vapor-retarding membrane over a 4-inch-thick layer of gravel. A 2-inch-thick layer of sand should be placed between the gravel and the membrane to decrease the possibility of damage to the membrane unless thicker membrane (greater than 15 mil) will be used. We suggest the following gradation for the gravel:

Sieve Size	Percent Passing
¾”	90 – 100
No. 4	0 – 10
No. 100	0 - 3

6.8 Seismic Design Parameters

Design ground motion parameters were also developed in accordance with CBC 2019 / ASCE7-16 for the proposed project. The site coordinates used in our seismic hazard analysis are: -118.2546 (Longitude) and 34.2384 (Latitude). The site is classified as Site Class C, corresponding to a “Very Dense soil” profile based on shear wave velocity V_{s30} of 1,734 feet per second measured from the ReMi profile study. Mapped design acceleration parameters determined in accordance with ASCE 7-16 Section 11.4 for Site Class C are presented in **Table 3**. Based on Section 11.4.8 of ASCE 7-16, site-specific ground motion procedure is not required for a Site Class C site.

Table 3. Mapped Seismic Design Acceleration Parameters

Design Parameters	**General Seismic Design Parameter (ASCE 7-16 Section 11.4)**
S_s (g)	1.965
S_1 (g)	0.735
Site Class	C
F_a	1.2
F_v	1.4
S_{MS} (g)	2.358
S_{M1} (g)	1.028
S_{DS} (g)	1.572
S_{D1} (g)	0.686

6.9 Retaining Walls and Walls Below Grade

6.9.1 Permanent Soldier Pile Wall

6.9.1.1 Lateral Earth Pressure

For design of cantilevered permanent soldier pile wall, we recommend using a triangular pressure distribution for calculating earth pressures. An active earth pressure equal to that of a fluid with a density of 30 pcf may be used for *level* retained ground. However, where the required soils are sloped back at up to 4:1 above the shoring, it may be assumed that the soils will exert an active lateral earth pressure equal to that of a fluid with a density of 34 pcf.

In addition to the recommended earth pressure, the upper 10 feet of wall adjacent to normal vehicular traffic should be designed to resist a uniform lateral pressure of 100 pounds per square foot, acting as a result of an assumed 300 pounds per square foot surcharge behind the shoring due to normal traffic. If the traffic is kept back at least 10 feet from the shoring, the traffic surcharge may be neglected.

6.9.1.2 Seismic Earth Pressure

Retaining wall where wall height is greater than 6 feet should be designed to resist an active pressure combined with a seismic increment of lateral active earth pressure. The combined active static and seismic lateral earth pressure were computed based on an k_{eq} of 0.5g (one-half of PGA_M). The combined active static and seismic lateral earth pressure is equivalent to a fluid with a density of 74 pounds per cubic foot. The active static lateral earth pressure is equivalent to a fluid with a density of 34 pounds per cubic foot. Therefore, a seismic increment of 40 pounds per cubic foot may be used for design of seismic earth pressure.

6.9.1.3 Design of Permanent Soldier Pile

The recommendations for design of soldier pile provided in Section 6.5.2 may be used for design of permanent soldier pile walls.

6.9.1.4 Permanent Lagging

Cast-in-place or precast concrete fascia panels shall be used for permanent lagging. The recommendations for design of lagging provided in Section 6.5.3 may be used for design of permanent lagging.

6.9.1.5 Retaining Wall Drainage

We recommend that a drainage system be placed behind retaining walls and walls below grade of the project building and other walls below grade and retaining walls to help dissipate the hydrostatic forces that may develop behind the walls. The drainage system may consist of Miradrain 6000 or equivalent strips, collected at the base of the basement wall, and disposed-of through an outlet. If the outlet is planned to be connected to the storm drain or sanitary sewer system, then environmental testing of the collected groundwater will need to be provided.

6.9.2 Cantilevered Retaining Wall

For design of cantilevered retaining walls (unrestrained along the height of the wall), the recommendations for earth pressures provided in Section 6.9.1.1, for seismic earth pressure provided in Section 6.9.2.2, and for drainage provided in Section 6.9.1.5 may be used for design of cantilevered retaining wall.

6.9.3 Wall Below Grade

6.9.3.1 Lateral Earth Pressure

Braded walls below grade should be designed to resist at-rest earth pressures. Accordingly, for the case where the grade is level behind the walls, a triangular distribution of lateral earth pressure equivalent to that developed by a fluid with a density of 50 pounds per cubic foot plus any surcharge loadings occurring as a result of traffic and adjacent foundations should be used.

6.9.3.2 Seismic Earth Pressure

Retaining wall where wall height is greater than 6 feet should be designed to resist an active pressure combined with a seismic increment of lateral active earth pressure. We understand that the proposed building wall below grade will be less than 6 feet. Therefore, seismic earth pressure needs not be used for design of the portion of the building wall below grade.

6.9.3.3 Drainage

Recommendations for drainage provided in Section 6.9.1.5 may be used for design of drainage system of the building wall below grade.

6.10 Site Drainage

The site should be graded to maintain positive drainage, so all runoff is properly collected and conveyed to proper disposal in approved storm drains or drainage devices. The area around foundations should be sloped at 2 percent to drain runoff away and prevent ponding of water.

6.11 Utility Installations

Excavations for utility trenches should be readily accomplished with conventional excavating equipment. All shoring and excavation should comply with current OSHA and CALOSHA regulations and should be observed by the designated competent person on site. The contractor should be responsible for the structural design and safety of any temporary shoring system.

The bedding for any new sewer and water service pipeline should be a minimum of 4 inches thick and should consist of clean sand, No. 4 concrete aggregate, or gravel and should have a sand equivalent of not less than 30. The pipe zone material, which extends to a level 12 inches above the pipe, should consist of sand and should have a sand equivalent of no less than 30, and a maximum rock size of 1 inch. All imported materials should be approved by the project geotechnical engineer before being brought on site.

Fill placement within the trench zone above the pipe backfill should be performed in accordance with the recommendations of this report. Jetting or flooding of backfill should not be permitted. In backfill areas where mechanical compaction of soil backfill is impractical due to space constraints, CLSM may be substituted for compacted backfill.

6.12 Soil Corrosivity

Representative samples of the near surface soils encountered were tested to evaluate corrosion characteristics. The results indicate the test samples had a pH of 7.02; a water-soluble sulfate content of less than 0.01%, and a soluble chloride content of less than 0.01%, respectively. The sulfate results indicate that sulfate exposure is classified as non corrosive.

The tested samples were also found to have a minimum measured electrical resistivity of 12,862 Ohm-cm. The following correlation can generally be used between electrical resistivity and corrosion potential:

Elect. Resistivity (Ohm-cm)	Corrosion Potential
less than 1,000	Severe
1,000-2,000	Corrosive
2,000-10,000	Moderate
Greater than 10,000	Mild

On the basis of the laboratory testing, the test samples are classified as mild corrosive to buried metals. Further evaluation and testing and alternatives for corrosion protection should be provided by a corrosion consultant.

6.13 Paving Design

To provide support for paving, the subgrade soils should be prepared and graded. Any suspected fill or any loose soils should be removed and recompacted. In addition, demolition activities may create disturbance of near surface soils, which will also require removal and recompaction. The actual limits for removals or recompaction should be determined by the project geotechnical engineer during grading, based on the actual conditions encountered.

Compaction of the subgrade, including trench backfills, to at least 90%, and achieving a firm, hard, and unyielding surface will be important for paving support. The preparation of the paving area subgrade should be done immediately prior to placement of the base course. Proper drainage of the paved areas should be provided since this will reduce moisture infiltration into the subgrade and increase the life of the paving.

To provide data for design of asphalt paving for others TI, a maximum R-value of 40 was assumed for design based on the lab results.

The required paving and base thicknesses will depend on the expected wheel loads and volume of traffic (Traffic Index or TI). Assuming that the paving subgrade will consist of the on-site or comparable soils compacted to at least 90% as recommended, the minimum recommended paving thicknesses are presented in the following table.

Paving Thickness

Traffic Index	Asphaltic Concrete (inches)	Base Course (inches)
4	3	4
5	3	4
6	3½	5½
7	4½	6

The asphalt paving sections were determined using the Caltrans Asphalt Institute design method. We can determine the recommended paving and base course thicknesses for other Traffic Indices if required. Careful inspection is recommended to verify that the recommended thicknesses or greater are achieved, and that proper construction procedures are followed.

The base course should conform to requirements of Section 26 of State of California Department of Transportation Standard Specifications (Caltrans), latest edition, or meet the specifications for untreated base as defined in Section 200-2 of the latest edition of the Standard Specifications for Public Works Construction (Green Book). The base course should be compacted to at least 95%.

7.0 LIMITATIONS

This investigation was performed in accordance with generally accepted Geotechnical Engineering principles and practice. The professional engineering work and judgments presented in this report meet the standard of care of our profession at this time. No other warranty, expressed or implied, is made. This report has been prepared for the Glendale Unified School

District, and their design consultants. It may not contain sufficient information for other parties or other purposes, and should not be used for other projects or other purposes without review and approval by GDC.

The recommendations for this project, to a high degree, are dependent upon proper quality control of site grading, fill and backfill placement, and pile foundation installation. The recommendations are made contingent on the opportunity for GDC to observe the earthwork operations. This firm should be notified of any pertinent changes in the project, or if conditions are encountered in the field, which differ from those described herein. If parties other than GDC are engaged to provide such services, they must be notified that they will be required to assume complete responsibility for the geotechnical phase of the project, and must either concur with the recommendations in this report or provide alternate recommendations.

8.0 REFERENCES

ASCE/SEI 7-16 (2018), "Minimum Design Loads for Buildings and Other Structures", American Society of Civil Engineers.

California Building Code (CBC), 2019, California Building Code, "California Code of Regulations Title 24, Part 2, Volumes 1 and 2." Sacramento, CA.

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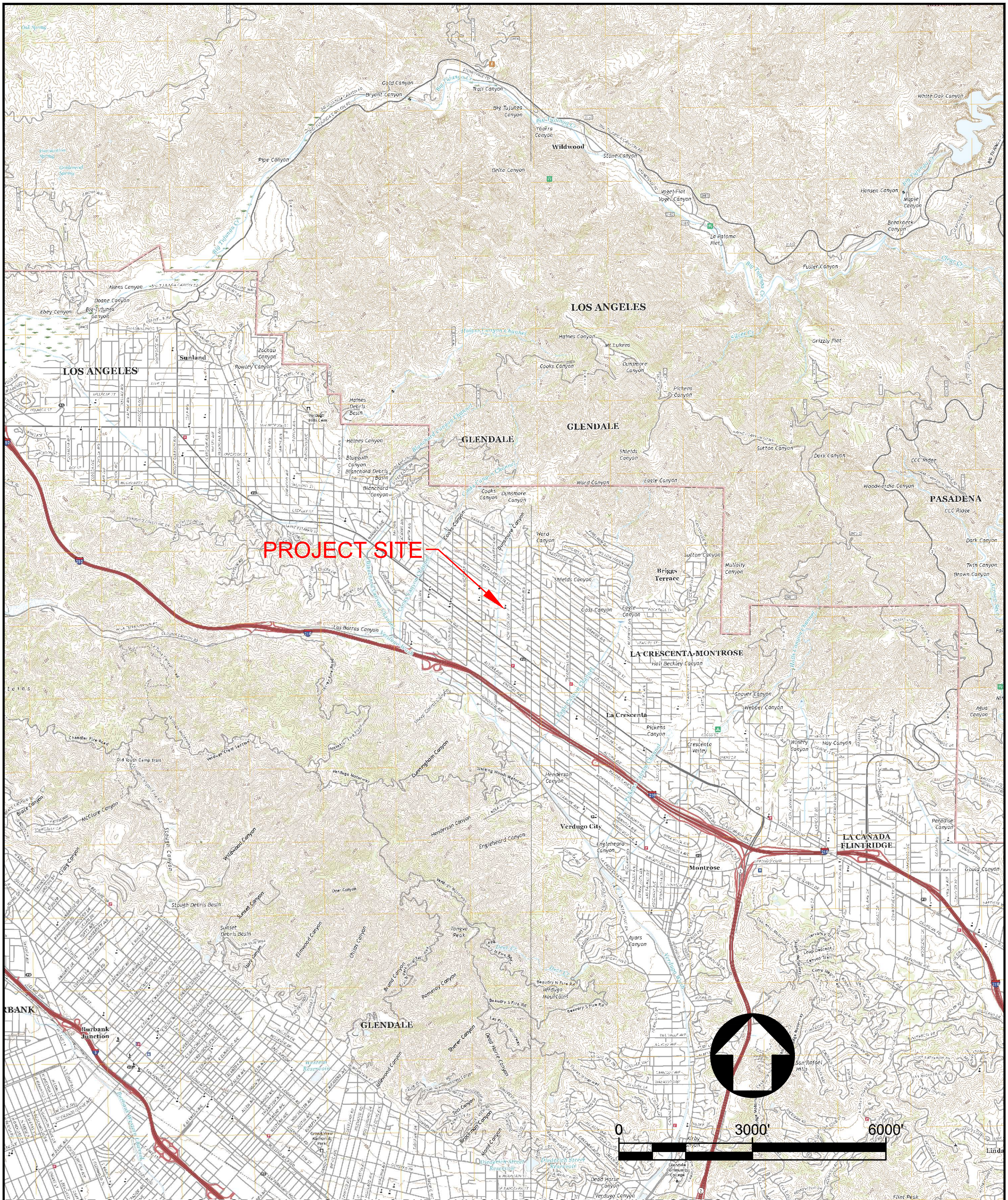
FIGURES



REFERENCE: SUNLAND, CONDOR PEAK, PASADENA & BURBANK QUADS USGS 7.5 MINUTE SERIES

DATE: 12/17/2019	DRAWN BY: JMT	GROUP DELTA CONSULTANTS, INC 370 Amapola Ave. Suite 212 Torrance, CA. 90501	SITE VICINITY MAP	PROJECT NUMBER: LA-1420
REVIEWED BY: -	APPROVED BY: -		4747 NEW YORK AVENUE	SCALE: AS SHOWN
PREPARED BY: -			GLENDALE, CALIFORNIA	FIGURE NUMBER: 1

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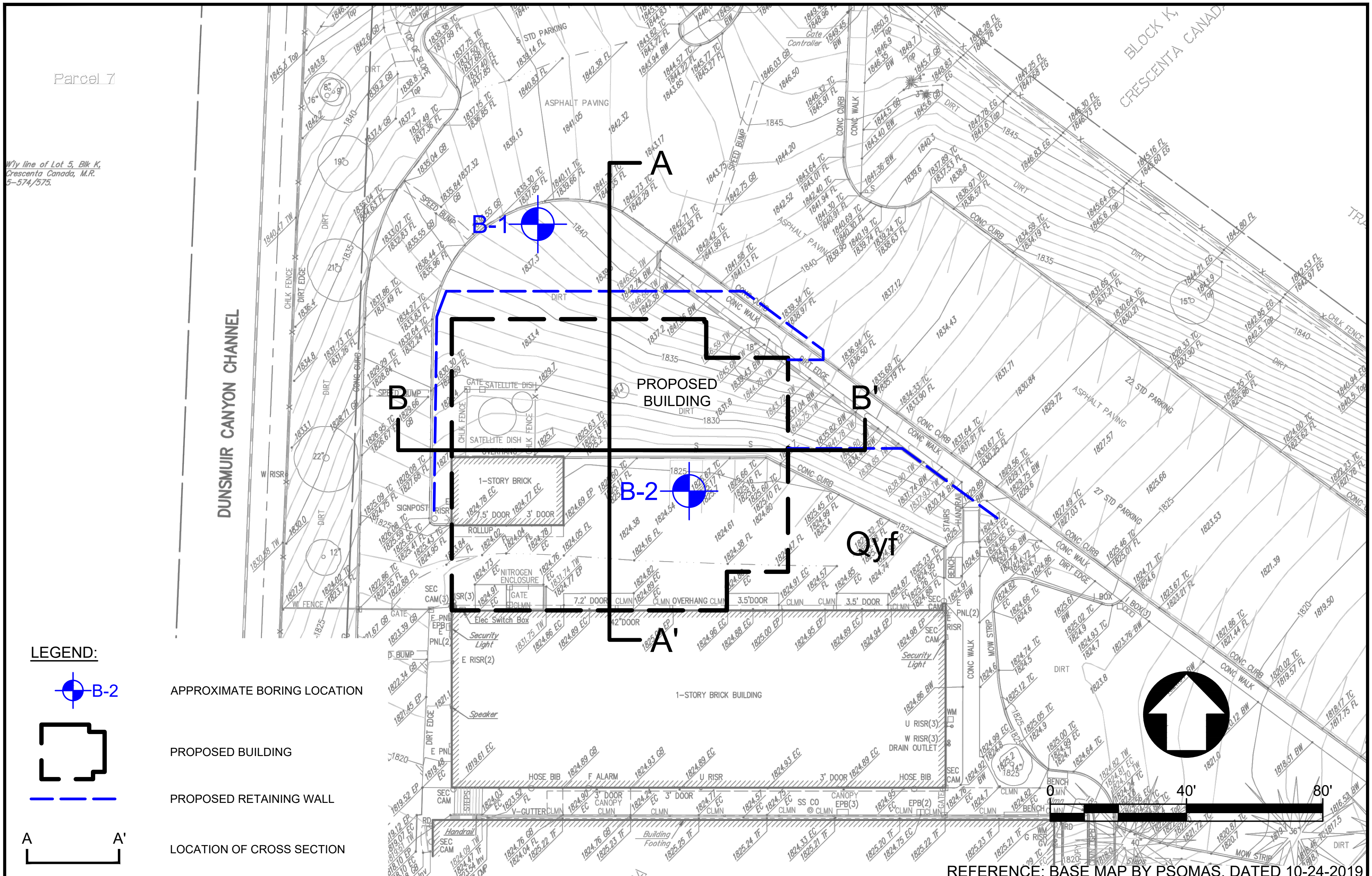


LEGEND:

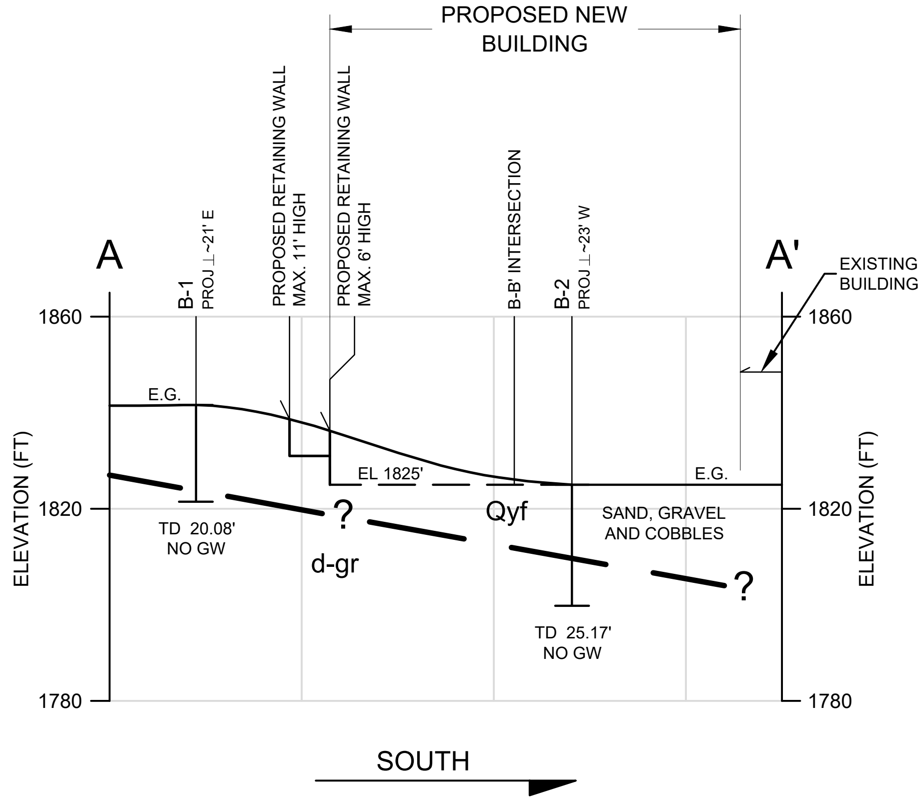
B-2 APPROXIMATE BORING LOCATION

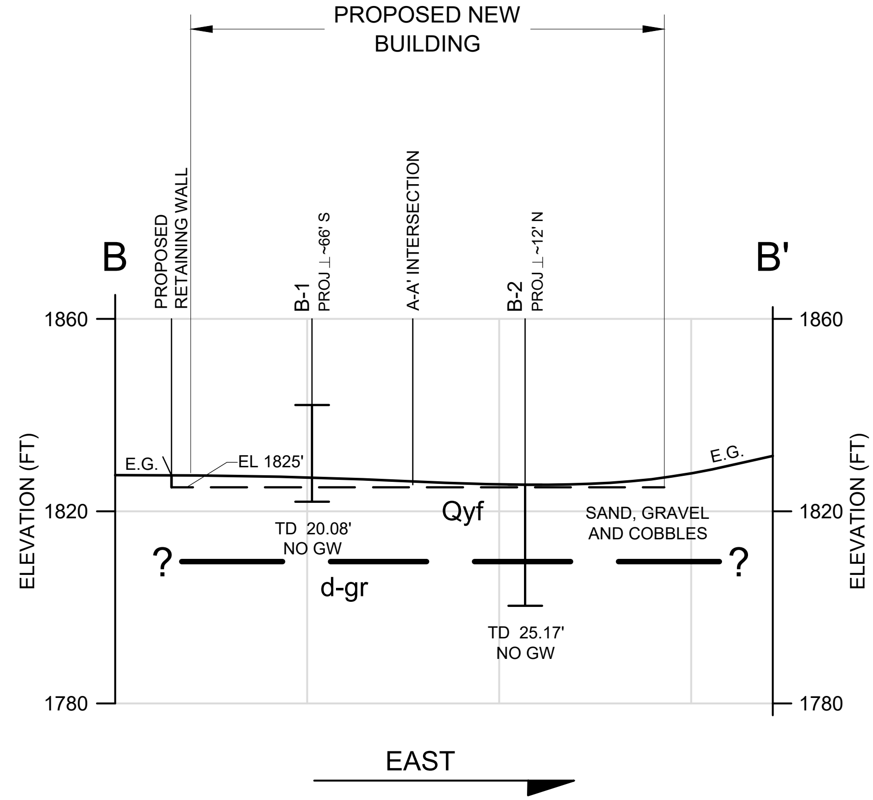
PROPOSED BUILDING

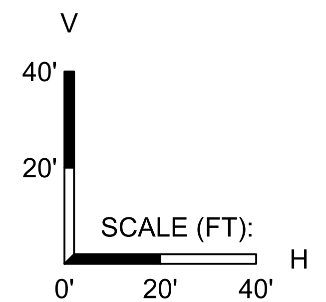
PROPOSED RETAINING WALL

A A' LOCATION OF CROSS SECTION

DATE: 12/30/2019	DRAWN BY: JMT	GROUP DELTA	GROUP DELTA CONSULTANTS, INC 370 Amapola Ave. Suite 212 Torrance, CA. 90501	EXPLORATION PLAN	PROJECT NUMBER: LA-1420
REVIEWED BY: MS	APPROVED BY: -			4747 NEW YORK AVENUE GLENDALE, CALIFORNIA	SCALE: AS SHOWN
PREPARED BY: -		DELTA			FIGURE NUMBER: 2







KEY:

Qyf YOUNG FAN DEPOSITS, NEAR SOURCE GRANITIC SEDIMENT

d-gr POSSIBLE DECOMPOSED GRANITE OR COBBLES AND BOULDERS

DATE: 12/30/2019	DRAWN BY: JMT	GROUP DELTA	GROUP DELTA CONSULTANTS, INC 370 Amapola Ave. Suite 212 Torrance, CA. 90501	CROSS SECTION A-A' CROSS SECTION B-B'	PROJECT NUMBER: LA-1420
REVIEWED BY: -	APPROVED BY: -			4747 NEW YORK AVENUE GLENDALE, CALIFORNIA	SCALE: AS SHOWN
PREPARED BY: -					FIGURE NUMBER: 3

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MAP UNITS AND SYMBOL EXPLORATION

Late Holocene (Surficial Deposits)

Unit	Description
af	**Artificial Fill** - deposits of fill resulting from human construction, mining, or quarrying activities; includes engineered fill for buildings, roads, dams, airport runways, harbor facilities, and waste landfills
Qsu	**Undifferentiated Surficial Deposits** - includes colluvium, slope wash, talus deposits, and other surface deposits of all ages; generally unconsolidated but locally may contain consolidated layers
Qls	**Landslide Deposits** - may include debris flows and older landslides of various earth material and movement types; unconsolidated to moderately well-consolidated
Qb	**Beach Deposits** - unconsolidated marine beach sediments consisting mostly of fine- and medium-grained, well-sorted sand
Qw	**Alluvial Wash Deposits** - unconsolidated sandy and gravelly sediment deposited in recently active channels of streams and rivers; may contain loose to moderately loose sand and silty sand
Qf	**Alluvial Fan Deposits** - unconsolidated boulders, cobbles, gravel, sand, and silt recently deposited where a river or stream issues from a confined valley or canyon; sediment typically deposited in a fan-shaped cone; gravelly sediment generally more dominant than sandy sediment
Qa	**Alluvial Valley Deposits** - unconsolidated clay, silt, sand, and gravel recently deposited parallel to localized stream valleys and/or spread more regionally onto alluvial flats of larger river valleys; sandy sediment generally more dominant than gravelly sediment
Qt	**Terrace Deposits** - includes marine and stream terrace deposits; marine deposits include slightly to moderately consolidated and bedded gravel and conglomerate, sand and sandstone, and silt and siltstone; river terrace deposits consist of unconsolidated thin- to thick-bedded gravel
Ql	**Lacustrine, Playa, and Estuarine (Paralic) Deposits** - mostly unconsolidated fine-grained sand, silt, mud, and clay from fresh water (lacustrine) lakes, saline (playa) dry lakes that are periodically flooded, and estuaries; deposits may contain salt and other evaporites
Qe	**Eolian and Dune Deposits** - unconsolidated, generally well-sorted wind-blown sand; may occur as dune forms or sheet sand

Holocene to Late Pleistocene (Surficial Deposits)

Unit	Description
Qyf	**Young Alluvial Fan Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon
Qya	**Young Alluvial Valley Deposits** - unconsolidated to slightly consolidated, undissected to slightly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers

Late to Middle Pleistocene (Surficial Deposits)

Unit	Description
Qof	**Old Alluvial Fan Deposits** - slightly to moderately consolidated, moderately dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon
Qoa	**Old Alluvial Valley Deposits** - slightly to moderately consolidated, moderately dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers
Qot	**Old Terrace Deposits** - slightly to moderately consolidated, moderately dissected marine and stream terrace deposits
Qol	**Old Lacustrine, Playa, and Estuarine (Paralic) Deposits** - slightly to moderately consolidated, moderately dissected fine-grained sand, silt, mud, and clay from lake, playa, and estuarine deposits of various types

Middle to Early Pleistocene (Surficial Deposits)

Unit	Description
Qvof	**Very Old Alluvial Fan Deposits** - moderately to well-consolidated, highly dissected boulder, cobble, gravel, sand, and silt deposits issued from a confined valley or canyon
Qvoa	**Very Old Alluvial Valley Deposits** - moderately to well-consolidated, highly dissected clay, silt, sand, and gravel along stream valleys and alluvial flats of larger rivers; generally uplifted and deformed

Quaternary (Bedrock)

Unit	Description
Qss	**Coarse-grained formations of Pleistocene age and younger** - primarily sandstone and conglomerate
Qsh	**Fine-grained formations of Pleistocene age and younger** - includes fine-grained sandstone, siltstone, mudstone, shale, siliceous and calcareous sediments

Tertiary (Bedrock)

Unit	Description
Tss	**Coarse-grained Tertiary age formations** - primarily sandstone and conglomerate
Tsh	**Fine-grained Tertiary age formations** - includes fine-grained sandstone, siltstone, mudstone, shale, siliceous and calcareous sediments
Tv	**Tertiary age formations of volcanic origin**

Mesozoic and Older (Bedrock)

Unit	Description
Kss	**Coarse-grained Cretaceous age formations of sedimentary origin**
Ksh	**Fine-grained Cretaceous age formations of sedimentary origin**
pKm	**Cretaceous and pre-Cretaceous metamorphic formations of sedimentary and volcanic origin**
sp	**Serpentinite of all ages**
gr	**Granitic and other intrusive crystalline rocks of all ages**

Contact

Gradational contact

Reference contact -- Used to delineate geologic units that were mapped as separate units on the original source map, but are consolidated on this map.

Fault -- Includes strike-slip, normal, reverse, oblique, and unspecified slip

Lineament

Folds -- Showing direction of plunge where appropriate

Anticline

Overturned anticline

Syncline

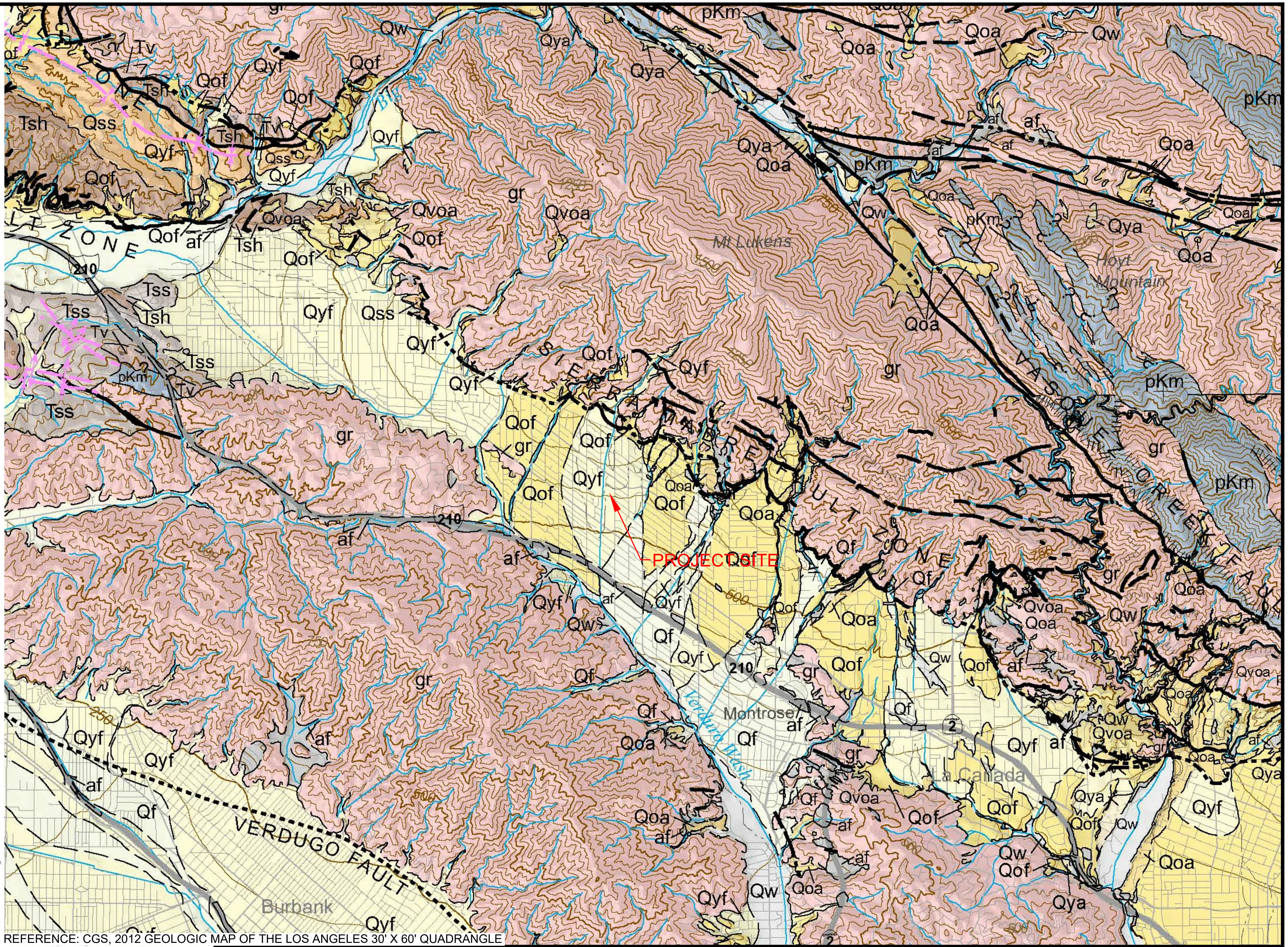
Dike

Stream

Spring

Road

County boundary



REFERENCE: CGS, 2012 GEOLOGIC MAP OF THE LOS ANGELES 30' X 60' QUADRANGLE

0 5000' 10000'

DATE: 12/17/2019	DRAWN BY: JMT	GROUP DELTA	GROUP DELTA CONSULTANTS, INC 370 Amapola Ave. Suite 212 Torrance, CA. 90501	REGIONAL GEOLOGY MAP	PROJECT NUMBER: LA-1420
REVIEWED BY: -	APPROVED BY: -			4747 NEW YORK AVENUE	SCALE: AS SHOWN
REVISED: -				GLENDALE, CALIFORNIA	FIGURE NUMBER: 4

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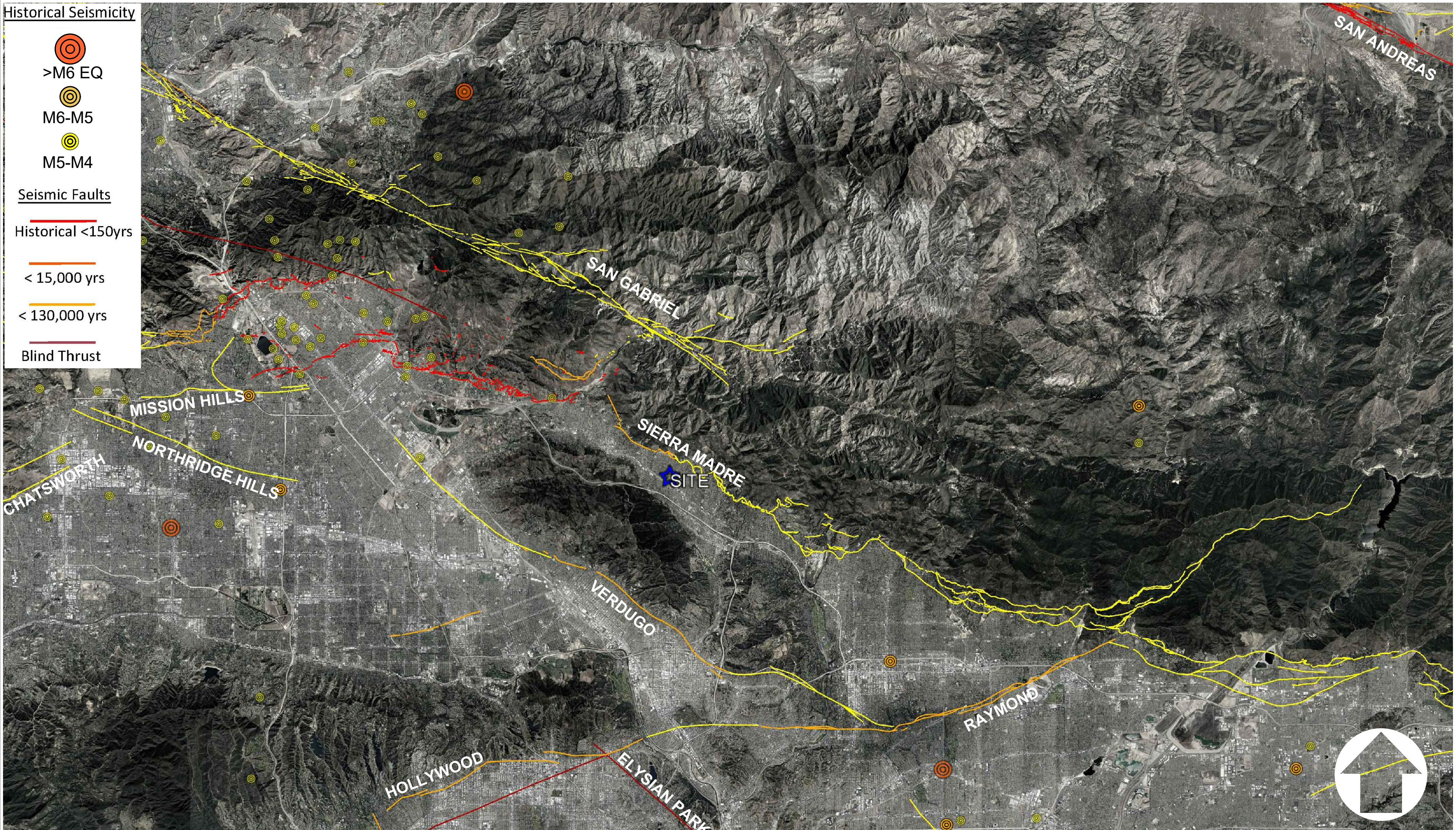

REFERENCE: U.S. Geological Survey and California Geological Survey, 2006, Quaternary fault and fold database for the United States, accessed 7/27/2018, from USGS web site: http//earthquake.usgs.gov/hazards/qfaults/. M4.0 and greater on record with USGS Earthquake Catalog accessed 08292018. Community Fault Model V8

DATE: 12/17/2019	DRAWN BY: JMT	GROUP DELTA	GROUP DELTA CONSULTANTS, INC 370 Amapola Ave. Suite 212 Torrance, CA. 90501	REGIONAL SIESMICITY AND FAULT MAP	PROJECT NUMBER: LA-1420
REVIEWED BY: -	APPROVED BY: -			4747 NEW YORK AVENUE GLENDALE, CALIFORNIA	SCALE: AS SHOWN
PREPARED BY: -					FIGURE NUMBER: 5

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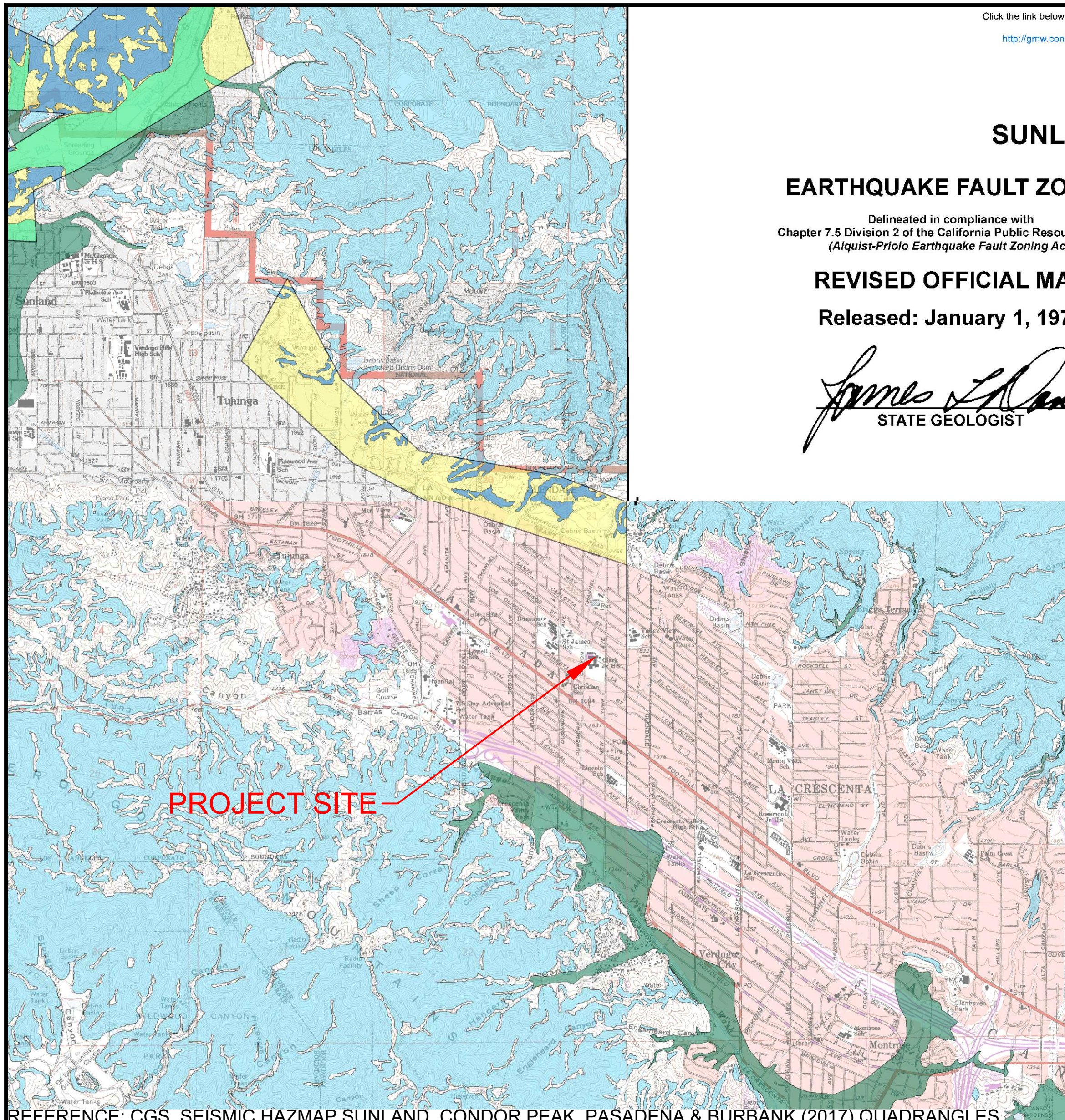

REFERENCE: CGS, SEISMIC HAZMAP SUNLAND, CONDOR PEAK, PASADENA & BURBANK (2017) QUADRANGLES

MAP EXPLANATION

EARTHQUAKE FAULT ZONES

Earthquake Fault Zones
Zone boundaries are delineated by straight-line segments; the boundaries define the zone encompassing active faults that constitute a potential hazard to structures from surface faulting or fault creep such that avoidance as described in Public Resources Code Section 2621.5(a) would be required.

1906 C

Active Fault Traces
Faults considered to have been active during Holocene time and to have potential for surface rupture: Solid Line in Black or Red where Accurately Located; Long Dash in Black or Solid Line in Purple where Approximately Located; Short Dash in Black or Solid Line in Orange where Inferred; Dotted Line in Black or Solid Line in Rose where Concealed; Query (?) indicates additional uncertainty. Evidence of historic offset indicated by year of earthquake-associated event or C for displacement caused by fault creep.

SEISMIC HAZARD ZONES

Liquefaction Zones
Areas where historical occurrence of liquefaction, or local geological, geotechnical and ground water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslide Zones
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

OVERLAPPING EARTHQUAKE FAULT AND SEISMIC HAZARD ZONES

Overlap of Earthquake Fault Zone and Liquefaction Zone
Areas that are covered by both Earthquake Fault Zone and Liquefaction Zone.

Overlap of Earthquake Fault Zone and Earthquake-Induced Landslide Zone
Areas that are covered by both Earthquake Fault Zone and Earthquake-Induced Landslide Zone.

0 5000' 10000'

DATE: 12/17/2019	DRAWN BY: JMT	GROUP DELTA	GROUP DELTA CONSULTANTS, INC 370 Amapola Ave. Suite 212 Torrance, CA. 90501	EARTHQUAKE ZONES OF REQUIRED INVESTIGATION MAP 4747 NEW YORK AVENUE GLENDALE, CALIFORNIA	PROJECT NUMBER: LA-1420
REVIEWED BY: -	APPROVED BY: -				SCALE: AS SHOWN
PREPARED BY: -					FIGURE NUMBER: 6

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APPENDIX A – FIELD INVESTIGATION

APPENDIX A
FIELD EXPLORATION

A.1 Introduction

GDC conducted a geotechnical subsurface investigation for the site on November 5 and November 7, 2019. The investigation consisted of the performance of a refraction microtremor profile and two (2) hollow-stem auger soil borings (B-1 and B-2).

Field subsurface investigation borings within the project site are included in this appendix. The exploration locations are shown in Figure 2 of the report. A summary of the field investigations within the project site is provided in Table A-1. The refraction microtremor profile is included in Attachment of this appendix.

A.2 Soil Borings

Two (2) hollow- stem auger borings (B-1, B-2) were advanced from the ground surface to the depths of 20.1 feet and 25.2 feet. Subsurface materials were visually classified and recorded by a GDC field engineer in accordance with the Unified Soil Classification System (USCS).

Drive samples and bulk samples of the encountered materials were obtained from the borings and recorded on the boring logs. Drive samples were obtained with a Modified California Sampler lined with 1-inch high metal sample rings and a Standard Penetration Test (SPT) sampler. The Modified California Sampler has an outside diameter of 3-inches, and the inside diameter of the rings is 2.42-inches. The samples were retained in brass rings and placed in sealed plastic canisters to prevent moisture loss. Standard penetration tests (SPT) were conducted using a standard 2-inch outside diameter, 1.375-inch inside diameter, split-spoon sampler in accordance with ASTM D 1586. SPT samples were placed in sealable plastic bags to protect the natural moisture. The SPT and Modified California samplers were driven into the soil at the bottom of the borehole using a 140-pound hammer free falling 30 inches. The penetration resistance (or "blowcount") in blows per six inches of driving was recorded on the logs. Bulk samples were obtained by a shovel and placed into polyethylene bags.

A.3 Refraction Microtremor (ReMi) Profile

The geophysical study was performed to the site to develop a Shear-wave velocity profile to be used for design and construction at the site with the ReMi technique (Refraction Microtremor). The ReMi technique uses recorded surface waves (specifically Rayleigh waves) that are contained in background noise to develop a Shear-wave velocity profile of the study area down to a depth, in this case, of approximately 100 feet. The depth of exploration is dependent on the length of the line and the frequency content of the background noise. The results of the ReMi method are displayed as a one-dimensional sounding which represents the average condition across the length of the line. The ReMi method does not require an increase of material velocity with depth; therefore, low velocity zones (velocity inversions) are detectable with ReMi.

The ReMi Reports are presented in the attachment portion of this appendix.

A.4 List of Attached Tables and Figures

The following table and figures are attached and complete this appendix:

Table A-1	Summary of GDC Field Explorations
Figure A-1	Key for Soil Classification
Figure A-2	Boring Log Legend
Figures A-3 to A-4	Field Investigation Boring Logs
Attachments	Refraction Microtremor Report

TABLES

Table A-1
Summary of Recent GDC Field Explorations

Exploration No.	Date Performed	Ground Surface Elevation (feet, MSL)	Total Depth	Groundwater Depth (ft)	Exploration Type
B-1	11/7/2019	1842	20.08	Not Encountered	Hollow Stem Auger
B-2	11/7/2019	1828	25.17	Not Encountered	Hollow Stem Auger

* Elevations based off County of LA 2017 Aerial and 2ft Contours

FIGURES

BORING RECORD

PROJECT NAME Glendale USD - Clark Magnet School	**PROJECT NUMBER** LA-1420	**HOLE ID** B-1	
SITE LOCATION Glendale, CA	**START** 11/7/2019	**FINISH** 11/7/2019	**SHEET NO.** 1 of 1
DRILLING COMPANY ABC Drilling	**DRILL RIG** LAR	**DRILLING METHOD** HSA	**LOGGED BY** Y. Gao / **CHECKED BY** Y. Gao
HAMMER TYPE (WEIGHT/DROP) SPT: 140 lb. 30"; Ring: 300 lb. 18"	**HAMMER EFFICIENCY (ERi)** 79.9%	**BORING DIA. (in)** 8	**TOTAL DEPTH (ft)** 20.083
GROUND ELEV (ft) 1842	**DEPTH/*ELEV.* GW (ft)** ⊻ NE / *na* DURING DRILLING	▼ NE / *na* AFTER DRILLING	
DRIVE SAMPLER TYPE(S) & SIZE (ID) Bulk, ModCAL, SPT	**NOTES**		

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	MOISTURE (%)	DRY DENSITY (PCF)	PASSING #200 (%)	ATTERBERG LIMITS (LL:PL:PI)	POCKET PEN (tsf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
0														TOP SOIL - ORGANIC SOIL with GRAVEL (OL/OH); dark brown.
	1840													Well Graded SAND (SW); dense; yellowish brown; moist; few fine to coarse GRAVEL; mostly fine to coarse SAND; few fines; low plasticity. (NATIVE)
		Bulk	B-1											
5		ModCAL	R-2	19 19 50/3	69/9	2.74	123.92							Well-graded SAND with SILT and GRAVEL (SW-SM); very dense; yellowish brown; moist; some fine to coarse GRAVEL; mostly fine to coarse SAND; little fines; low plasticity.
	1835													Well-graded SAND with GRAVEL (SW); very dense; light yellowish brown; moist; some fine GRAVEL; some fine to coarse SAND; trace fines; no plasticity.
		ModCAL	R-3	25 33 38	71	1.59	131.23	5			PA			49% GRAVEL; 46% SAND; 5% Fines. More fines.
10														Well-graded GRAVEL with SILT and SAND (GW-GM); very dense; light yellowish brown; moist; mostly fine to coarse GRAVEL and COBBLEs; little fine to coarse SAND; few fines; no plasticity.
	1830	ModCAL	R-4	32 50/4	50/4	4.58	115.73							More fines
15		ModCAL	R-5	17 50/4	50/4	1.12	116.5	8			PA			67% GRAVEL; 25% SAND; 8% Fines.
	1825													SILTY SAND with GRAVEL and COBBLES (SM); very dense; dark brown; moist; fine to medium SAND; little fine to coarse GRAVEL; little fines; 40% SANDSTONE COBBLES, 8-12", hard.
20		SPT	S-6	50/1	REF									Boring terminated at 20.08 feet. No groundwater encountered. Backfilled with soil cuttings.
	1820													

GROUP DELTA

GROUP DELTA CONSULTANTS, INC.

370 Amapola Ave., Suite 212

Torrance, CA 90501

FIGURE

A-3

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Glendale USD - Clark Magnet School	LA-1420	**B-2**

SITE LOCATION	START	FINISH	SHEET NO.
Glendale, CA	11/7/2019	11/7/2019	1 of 2

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
ABC Drilling	LAR	HSA	Y. Gao	Y. Gao

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
SPT: 140 lb. 30"; Ring: 300 lb. 18"	79.9%	8	25.2	1828	∇ NE / *na* DURING DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES	
Bulk, ModCAL, SPT		AFTER DRILLING ▼ NE / *na*

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	MOISTURE (%)	DRY DENSITY (PCF)	PASSING #200 (%)	ATTERBERG LIMITS (LL:PL:PI)	POCKET PEN (tsf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
														ASPHALT CONCRETE (4")
			B-1								CR			Well-graded SAND with Silt and GRAVEL (SW-SM); very dense; dark brown; moist; mostly fine to medium SAND with trace coarse SAND; few fine to coarse GRAVEL; little fines; low plasticity. (NATIVE)
	1825													
			B-2											Yellowish brown; little fine to coarse GRAVEL; little fines; low plasticity.
5														
			R-3	15 30 50/5	80/11	2.37	130.64	9			PA			Little fine GRAVEL; few fines; low to no plasticity. 18% GRAVEL; 73% SAND; 9% Fines
	1820		R-4	50/6	REF	3.02	103.63							Little fine to coarse GRAVEL and COBBLES.
10														
	1815		R-5	26 33 38	71	2.37	125.14	7			PA			Dark yellowish brown; little fine GRAVEL; few fines; low plasticity. 20% GRAVEL; 73% SAND; 7% Fines.
15			R-6	50/1	REF									SILTY SAND with GRAVEL and COBBLES (SM); very dense; dark brown; moist; fine to medium SAND; few fine to coarse GRAVEL; little fines; 40% SANDSTONE COBBLES, 8-12", hard.
	1810													
20			S-7	36 50/3	50/3									
	1805													
				50/2										

BORING RECORD

PROJECT NAME	PROJECT NUMBER	HOLE ID
Glendale USD - Clark Magnet School	LA-1420	**B-2**

SITE LOCATION	START	FINISH	SHEET NO.
Glendale, CA	11/7/2019	11/7/2019	2 of 2

DRILLING COMPANY	DRILL RIG	DRILLING METHOD	LOGGED BY	CHECKED BY
ABC Drilling	LAR	HSA	Y. Gao	Y. Gao

HAMMER TYPE (WEIGHT/DROP)	HAMMER EFFICIENCY (ERi)	BORING DIA. (in)	TOTAL DEPTH (ft)	GROUND ELEV (ft)	DEPTH/*ELEV.* GW (ft)
SPT: 140 lb. 30"; Ring: 300 lb. 18"	79.9%	8	25.2	1828	▽ NE / *na* DURING DRILLING; ▼ NE / *na* AFTER DRILLING

DRIVE SAMPLER TYPE(S) & SIZE (ID)	NOTES
Bulk, ModCAL, SPT	

DEPTH (feet)	ELEVATION (feet)	SAMPLE TYPE	SAMPLE NO.	PENETRATION RESISTANCE (BLOWS / 6 IN)	BLOW/FT "N"	MOISTURE (%)	DRY DENSITY (PCF)	PASSING #200 (%)	ATTERBERG LIMITS (LL:PL:PI)	POCKET PEN (tsf)	OTHER TESTS	DRILLING METHOD	GRAPHIC LOG	DESCRIPTION AND CLASSIFICATION
			S-8		REF			18			PA			Few fine GRAVEL; mostly fine to coarse SAND; little fines. 12% GRAVEL; 70% SAND; 18% Fines. Boring terminated at 25.17 feet. No groundwater encountered. Backfilled with bentonite grout.
	1800													
30														
	1795													
35														
	1790													
40														
	1785													
45														
	1780													

GDC_LOG_BORING_2011 LA1420.GPJ GDCLOG.GDT 12/17/19

GROUP DELTA

GROUP DELTA CONSULTANTS, INC.

370 Amapola Ave., Suite 212

Torrance, CA 90501

FIGURE

A-4 b

ATTACHMENTS

November 26, 2019

Project No. 119576
Report No. 1

Mr. Ethan Tsai, G.E.
Group Delta
370 Amapola Avenue, Suite 212
Torrence, California 90501

Subject: GEOPHYSICAL EVALUATION
ANDERSON W. CLARK MAGNET HIGH SCHOOL
GLENDALE, CALIFORNIA

Dear Mr. Tsai:

In accordance with your authorization, we have performed geophysical study services pertaining to the Anderson W. Clark Magnet High School project located in Glendale, California (Figure 1). The purpose of our study was to develop a Shear-wave velocity profile to be used for design and construction at the site. Our services were performed on November 5, 2019. This report presents the study methodology, equipment used, analysis, and findings from our study.

Our scope of services included the performance of a refraction microtremor (ReMi) profile (RL-1) at a preselected area of the project site (see Figures 2 and 3). The ReMi technique uses recorded surface waves (specifically Rayleigh waves) that are contained in background noise to develop a Shear-wave velocity profile of the study area down to a depth, in this case, of approximately 100 feet. The depth of exploration is dependent on the length of the line and the frequency content of the background noise. The results of the ReMi method are displayed as a one-dimensional sounding which represents the average condition across the length of the line. The ReMi method does not require an increase of material velocity with depth; therefore, low velocity zones (velocity inversions) are detectable with ReMi.

Our ReMi study included the use of a 24-channel Geometrics Geode seismograph and 24 4.5-Hz vertical component geophones. The geophones were spaced 10 feet apart for a total line length of 230 feet. Fifteen records, each 32 seconds long, were recorded and then downloaded to a computer. The data were later processed using SeisOpt® ReMi™ software (© Optim LLC, 2005), which uses the refraction microtremor method (Louie, 2001). The program generates phase-velocity dispersion curves for each record and provides an interactive dispersion modeling tool where the users determine the best fitting model. The result is a one-dimensional Shear-wave velocity model of the site with roughly 85 to 95 percent accuracy. Figure 3 depicts the general site conditions in the study area.

Figure 3 presents the results from our study. Based on our analysis of the collected data, the average characteristic site Shear-wave velocity down to a depth of 100 feet for RL-1 is 1,734 feet per second (CBC, 2016). This value corresponds to site classification of C. It should be noted the ReMi results represent the average condition across the length of the line.

The field evaluation and geophysical analyses presented in this report have been conducted in general accordance with current practice and the standard of care exercised by consultants performing similar tasks in the project area. No warranty, express or implied, is made regarding the conclusions and opinions presented in this report. There is no evaluation detailed enough to reveal every subsurface condition. Variations may exist and conditions not observed or described in this report may be present. Uncertainties relative to subsurface conditions can be reduced through additional subsurface exploration. Additional subsurface studying will be performed upon request.

This document is intended to be used only in its entirety. No portion of the document, by itself, is designed to completely represent any aspect of the project described herein. Southwest Geophysics should be contacted if the reader requires additional information or has questions regarding the content, interpretations presented, or completeness of this document. This report is intended exclusively for use by the client. Any use or reuse of the findings, conclusions, and/or recommendations of this report by parties other than the client is undertaken at said parties' sole risk.

We appreciate the opportunity to be of service on this project. Should you have any questions related to this report, please contact the undersigned at your convenience.

Respectfully submitted,
SOUTHWEST GEOPHYSICS, LLC

Caleb de Silveira

Caleb D. de Silveira
Staff Geophysicist

Patrick Lehrmann

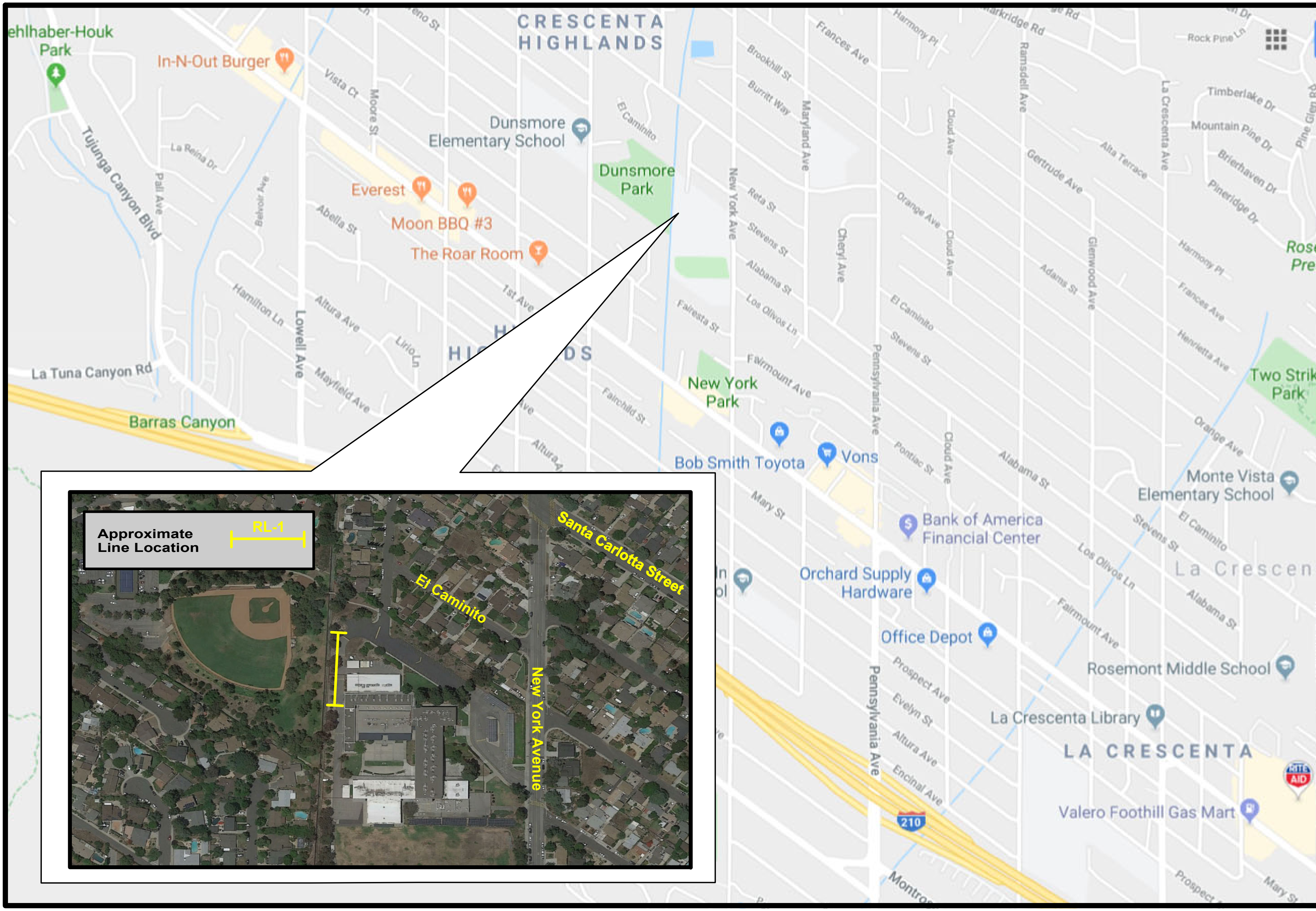
Patrick F. Lehrmann, P.G., Pg.
Principal Geologist/Geophysicist

REGISTERED GEOPHYSICIST
PATRICK F. LEHRMANN
No. 1043
Exp. 1/31/20
STATE OF CALIFORNIA

CDD:PFL:pfl:ds

(1) Addressee via e-mail: Ethan Tsai, G.E, Ethant@groupdelta.com

Attachments: Figure 1 – Site Location Map
Figure 2 – Seismic Line Location Map
Figure 3 – ReMi Results, RL-1



SITE LOCATION MAP

N

Anderson W. Clark Magnet High School
Glendale, California

Project No.: 119576 | Date: 11/19

SouthWest Geophysics LLC

Figure 1

LINE LOCATION MAP
(RL-1)

N

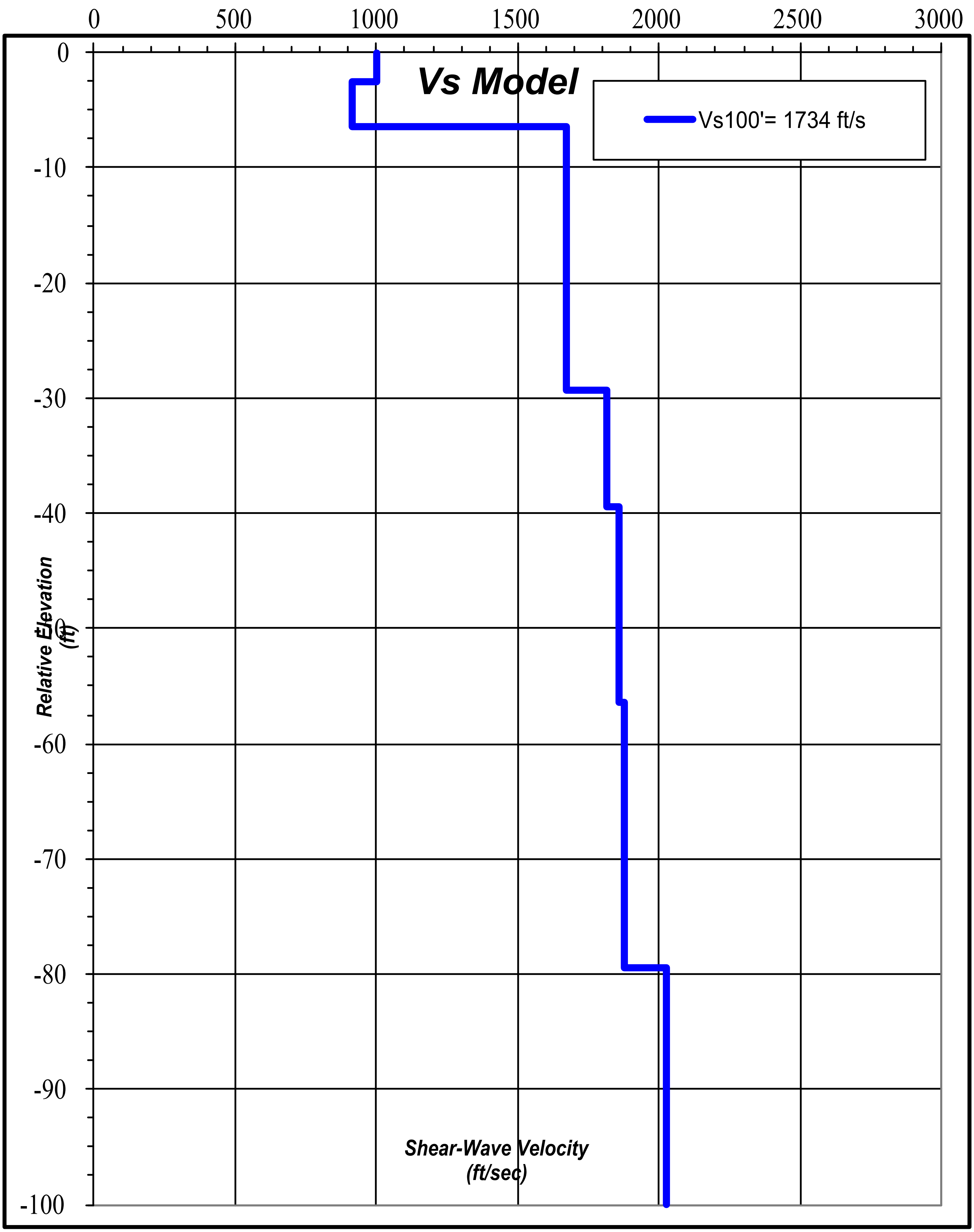
Anderson W. Clark Magnet High School
Glendale, California

Project No.: 119576 | Date: 11/19

SOUTHWEST GEOPHYSICS LLC

Figure 2

0 100 200
approximate scale in feet



ReMi RESULTS RL-1	Anderson W. Clark Magnet High School Glendale, California Project No.: 119576 — Date: 11/19	SOUTHWEST GEOPHYSICS LLC Figure 3

APPENDIX B – LABORATORY TEST DATA

APPENDIX B
LABORATORY TESTING

B.1 Introduction

The laboratory testing was performed using appropriate American Society for Testing and Materials (ASTM) and Caltrans Test Methods (CTM).

Modified California drive samples and or Standard Penetration Test (SPT) drive samples, and bulk samples collected during the field investigation were carefully sealed in the field to prevent moisture loss. The samples of earth materials were then transported to the laboratory for further examination and testing. Tests were performed on selected samples as an aid in classifying the earth materials and to evaluate their physical properties and engineering characteristics. Laboratory testing for this investigation included:

- Soil Classification: USCS (ASTM D 2487) and Visual Manual (ASTM D 2488);
- Moisture content (ASTM D 2216) and Dry Unit Weight (ASTM D 2937);
- Grain Size Distribution and Percent Passing No. 200 Sieve (ASTM D 1140);
- R-value (ASTM D2844, CTM 301));
- Soil Corrosivity:
    - pH (CTM 643);
    - Water-Soluble Sulfate (ASTM D 516, CTM 417);
    - Water-Soluble Chloride(Ion-Specific Probe, CTM 422);
    - Minimum Electrical Resistivity (CTM 643);

A brief description of the laboratory testing program and test results are presented below.

B.2 Moisture Content and Dry Unit Weight

The natural moisture content of selected SPT and or California ring samples and dry unit weight of California ring samples were determined in general accordance with ASTM D 2216 and ASTM D2937. Results of these tests are presented on the boring logs in Appendix A.

B.3 Grain Size Distribution and Percent Passing No. 200 Sieve

Determination of fines verses coarser soil particles was performed by the percent #200 Sieve test. Representative samples were dried, weighed, soaked in water until individual soil particles were separated, and then washed on the No. 200 sieve. The percentage of fines (soil passing No. 200 sieve) was determined in accordance with ASTM D1140. The washed fraction retained on the No. 200 sieve was then screened on a No. 4 sieve, and the fraction retained on No. 4 was weighed to determine the percentage of gravel. The results of percent passing No. 200 sieve is presented in the boring logs in Appendix A.

Grain size distribution results are shown in Figure B-1.

B.7 R-Value

An R-Value test was performed to measure the potential strength of the upper soils on site to use as potential subgrade. The results of this test are shown in Figure B-2.

B.8 Soil Corrosivity

Tests were performed in order to determine corrosion potential of site soils on concrete and ferrous metals. Corrosivity testing included minimum electrical resistivity and soil pH (Caltrans method 643), water soluble chlorides (Orion 170A+ Ion Probe or Caltrans Test Method 422), and water-soluble sulfates (ASTM D 516). The test results are summarized in Table B-1.

B.9 List of Attached Tables and Figures

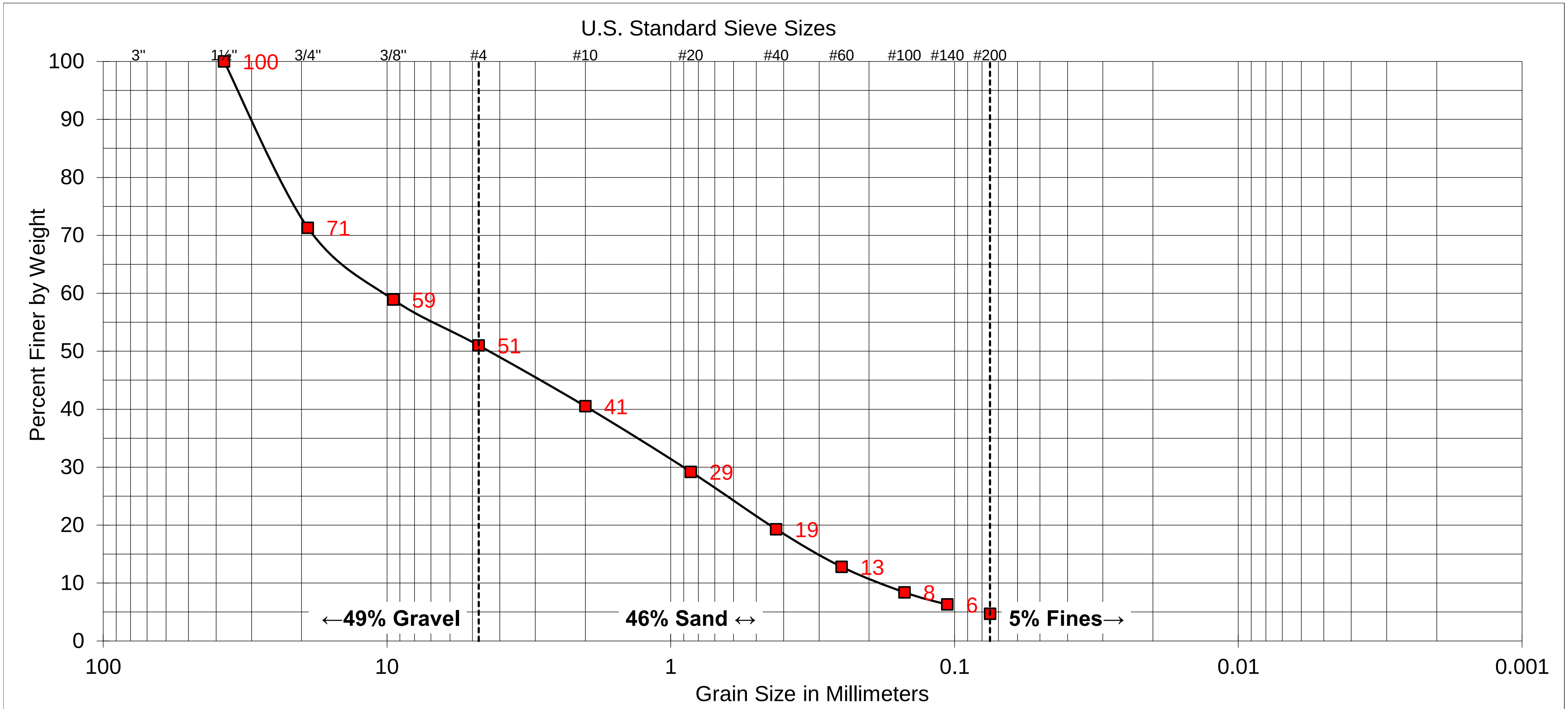
The following tables and figures are attached and complete this appendix:

TABLES

Table B-1
Summary of Soil Corrosivity

Boring No.	Depth (ft)	pH	Sulfate Content (%)	Chloride Content (%)	Minimum Resistivity (ohm-cm)
B-2	0-4	7.02	<0.01	<0.01	12,862

FIGURES



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY
GRAVEL		SAND			

SAMPLE B-1
SAMPLE NUMBER: R-3
SAMPLE DEPTH: 9' - 9.5'

UNIFIED SOIL CLASSIFICATION: SW

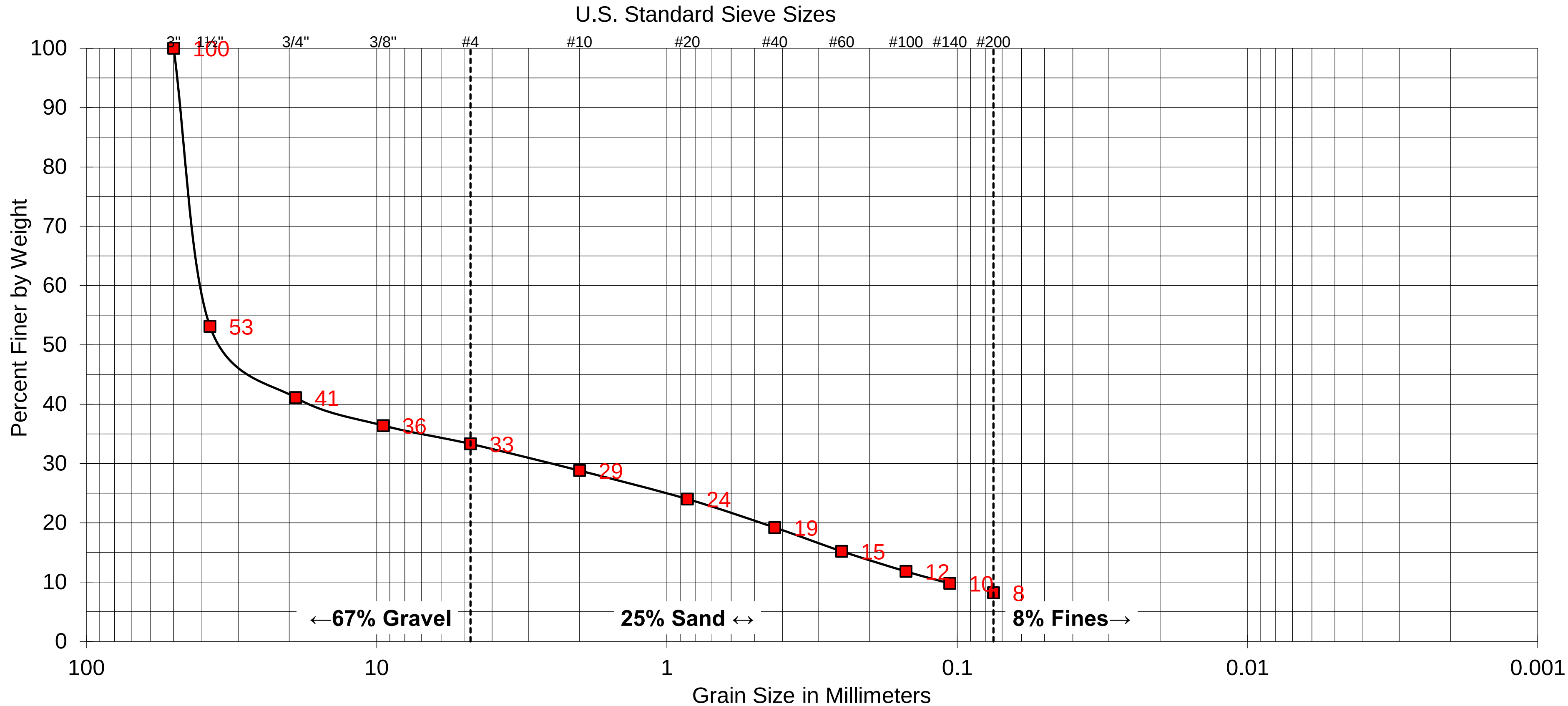
DESCRIPTION: Well Graded Sand with Gravel

ATTERBERG LIMITS
LIQUID LIMIT:
PLASTIC LIMIT:
PLASTICITY INDEX:

GROUP DELTA

SOIL CLASSIFICATION

Lab. No. SO5573
Project No. LA1420
FIGURE B-1.1



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	B-1
SAMPLE NUMBER:	R-5
SAMPLE DEPTH:	15.5' - 16'

UNIFIED SOIL CLASSIFICATION: GW

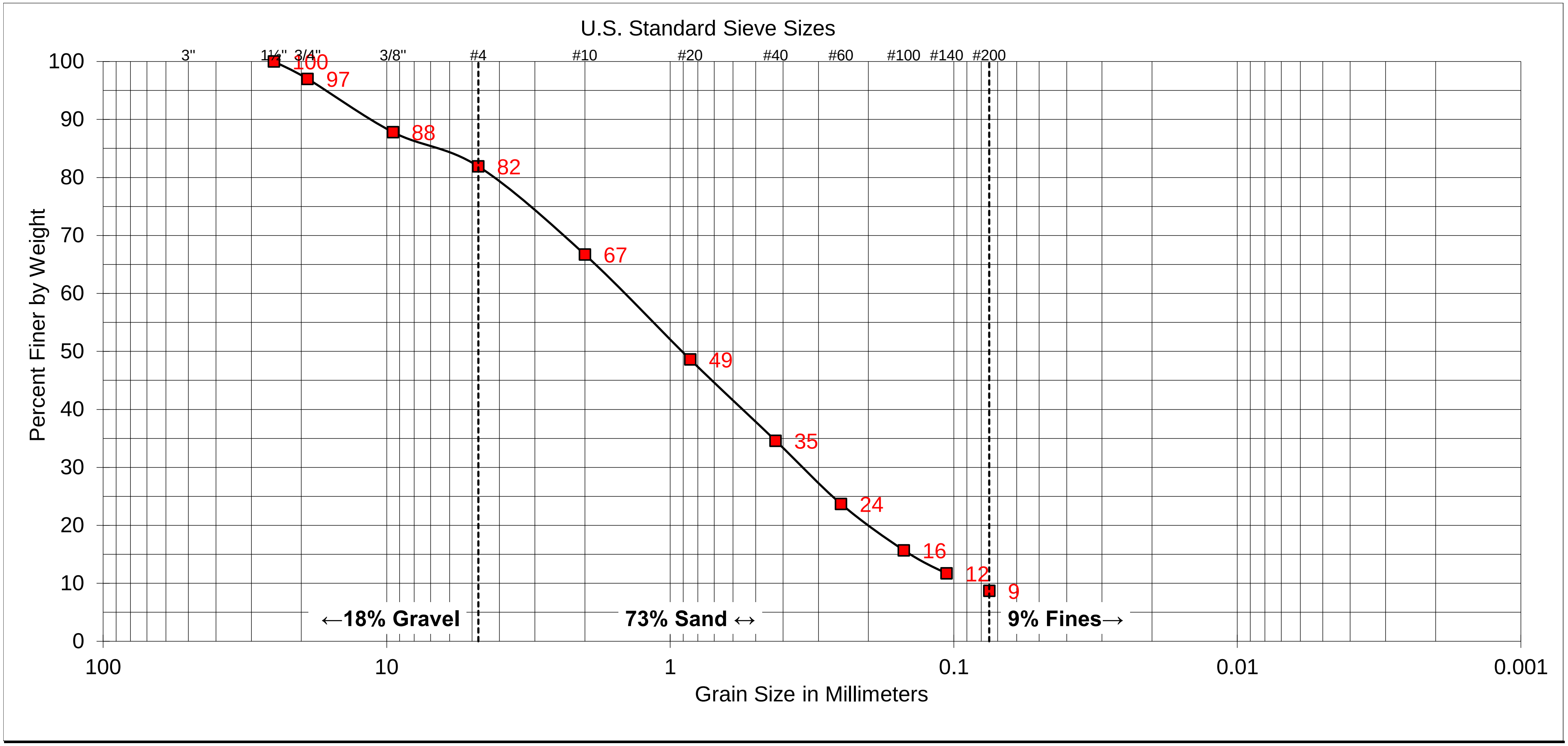
DESCRIPTION: Well Graded Gravel with Silt and Sand

ATTERBERG LIMITS	
LIQUID LIMIT:	
PLASTIC LIMIT:	
PLASTICITY INDEX:	

GROUP DELTA

SOIL CLASSIFICATION

Lab. No. SO5573
Project No. LA1420
FIGURE B-1.2



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND
GRAVEL		SAND			CLAY

SAMPLE	B-2
SAMPLE NUMBER:	R-3
SAMPLE DEPTH:	6' - 6.5'

UNIFIED SOIL CLASSIFICATION: SW

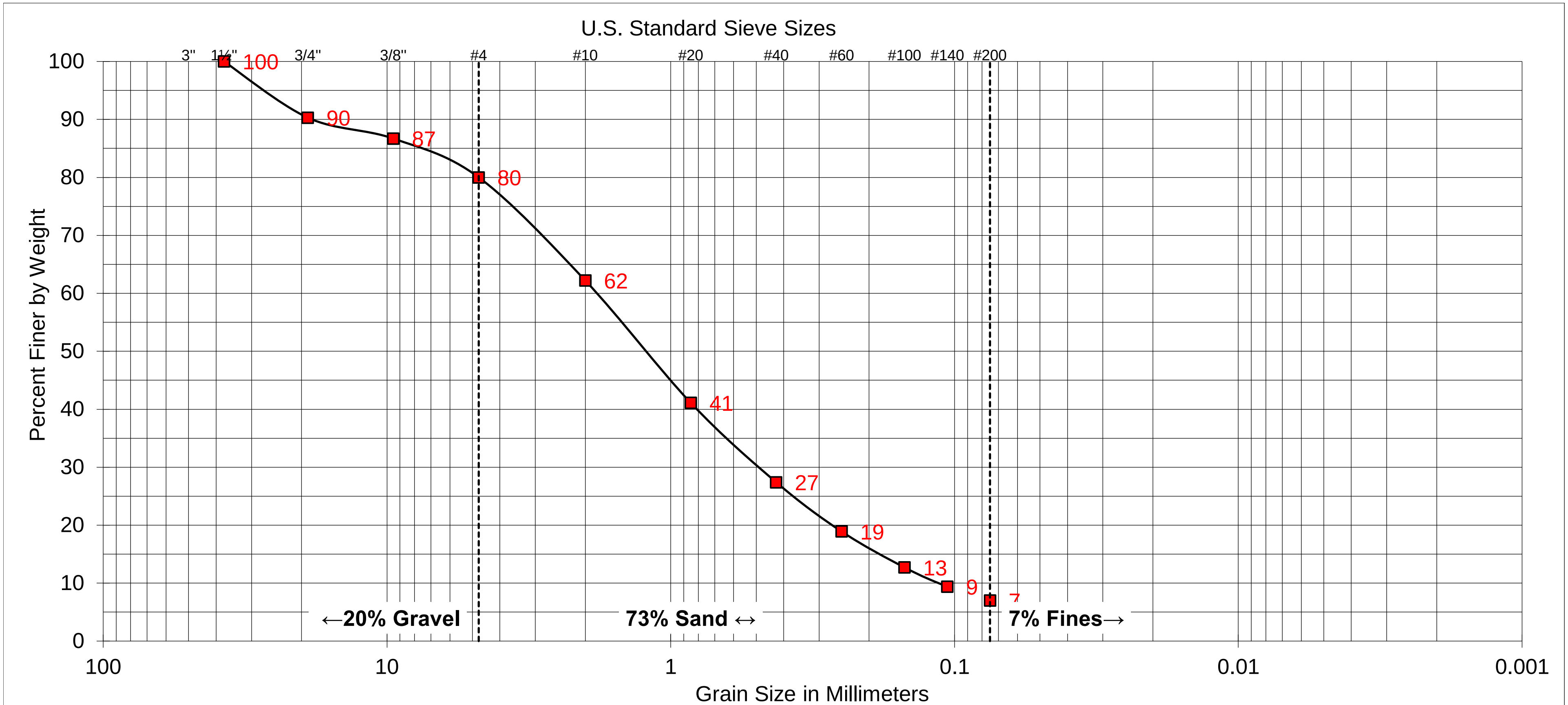
DESCRIPTION: Well Graded Sand with Silt and Gravel

ATTERBERG LIMITS
LIQUID LIMIT:
PLASTIC LIMIT:
PLASTICITY INDEX:

GROUP DELTA

SOIL CLASSIFICATION

Lab. No. SO5573
Project No. LA1420
FIGURE B-1.3



COARSE	FINE	COARSE	MEDIUM	FINE	SILT AND CLAY
GRAVEL		SAND			

SAMPLE	B-2
SAMPLE NUMBER:	R-5
SAMPLE DEPTH:	13' - 13.5'

UNIFIED SOIL CLASSIFICATION: SW

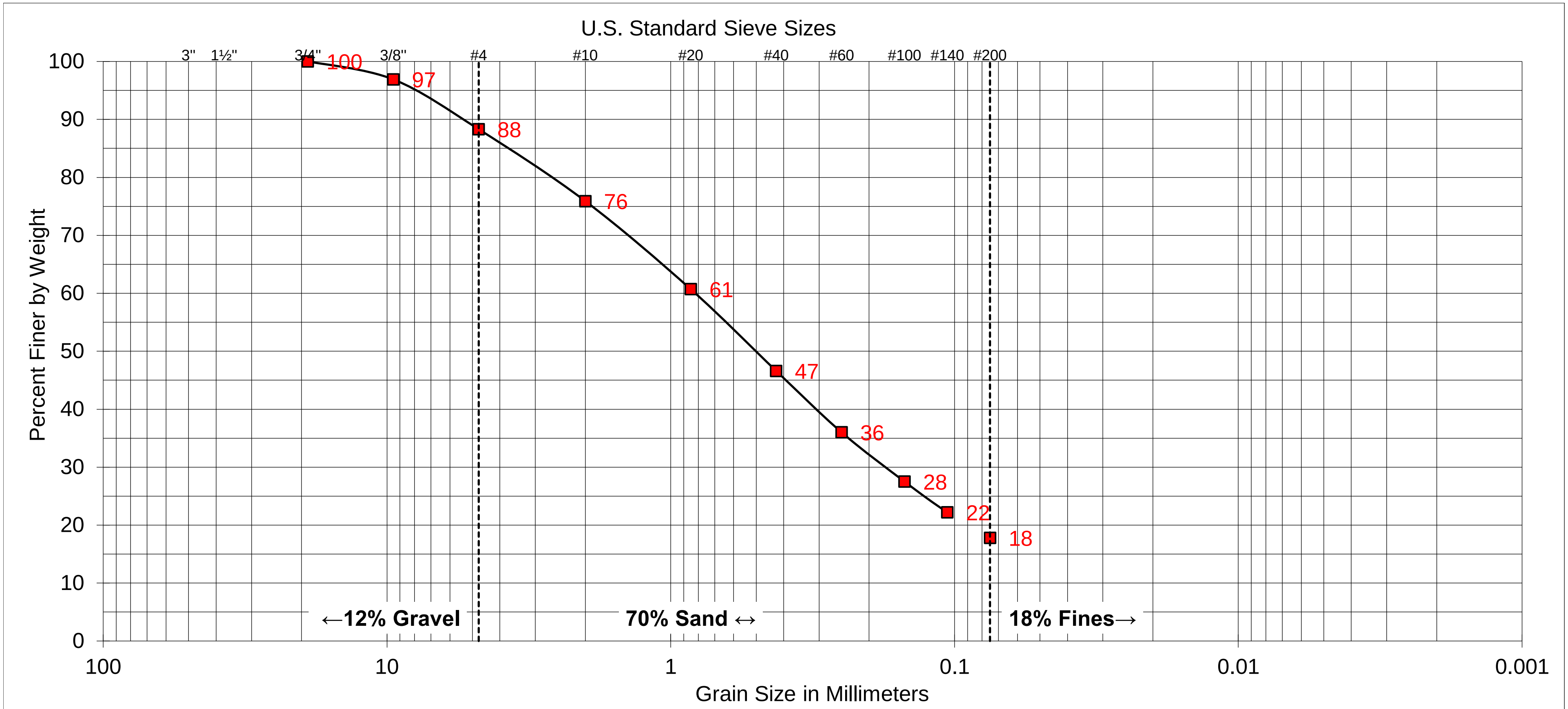
DESCRIPTION: Well Graded Sand with Silt and Gravel

ATTERBERG LIMITS
LIQUID LIMIT:
PLASTIC LIMIT:
PLASTICITY INDEX:

GROUP DELTA

SOIL CLASSIFICATION

Lab. No. SO5573
Project No. LA1420
FIGURE B-1.4



GRAVEL		SAND			SILT AND CLAY
COARSE	FINE	COARSE	MEDIUM	FINE	

SAMPLE	B-2
SAMPLE NUMBER:	R-8
SAMPLE DEPTH:	25' - 25.5'

UNIFIED SOIL CLASSIFICATION: SM

DESCRIPTION: Silty Sand with Gravel

ATTERBERG LIMITS
LIQUID LIMIT:
PLASTIC LIMIT:
PLASTICITY INDEX:

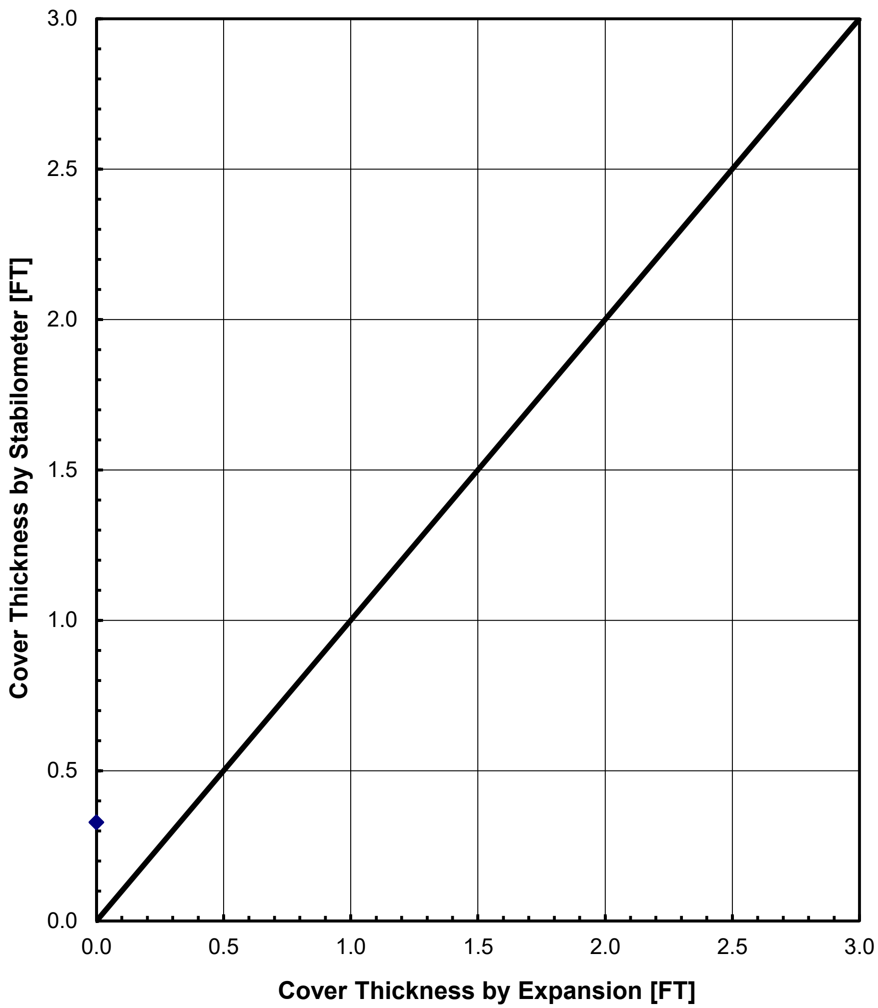
GROUP DELTA

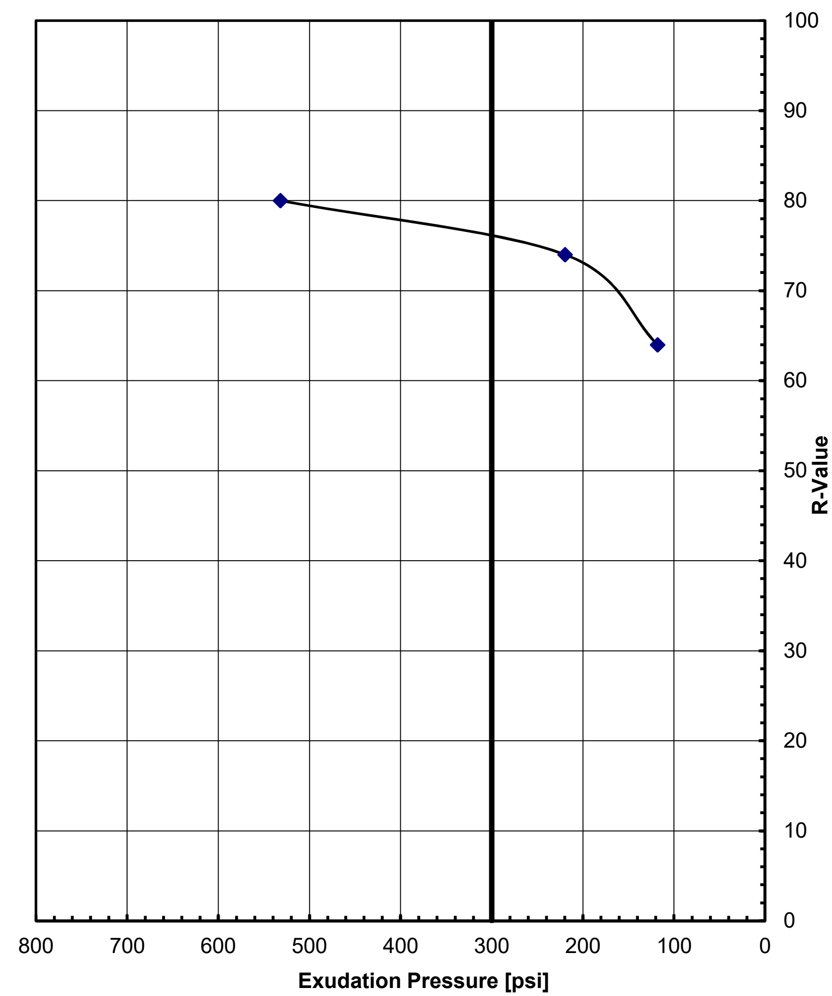
SOIL CLASSIFICATION

Lab. No. SO5573
Project No. LA1420
FIGURE B-1.5

Sample: B-2, 0-4'

R-Value at Equilibrium: 76





GROUP DELTA CONSULTANTS, INC.
ENGINEERS AND GEOLOGISTS
9245 ACTIVITY ROAD, SUITE 103
SAN DIEGO, CALIFORNIA 92126

COVER AND EXUDATION CHARTS

Document No.
Project No. LA1420
FIGURE B-2