Appendix D Geotechnical Investigation

Appendix

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GEOTECHNICAL INVESTIGATION
CORNERSTONE BIBLE CHURCH
400 N GLENDORA AVE
GLENDORA, CA 91741

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TABLE OF CONTENTS

1.0 INTRODUCTION AND SCOPE OF SERVICES

1.1 Introduction

This report presents the results of the preliminary geotechnical investigation, performed by CTE South, Inc. (CTE), and provides conclusions and recommendations for the proposed development at the existing Cornerstone Bible Church in Glendora, California (Figure 1). The investigation has been performed in general accordance with the terms of our proposal no. R-19125, dated December, 2019. We understand that proposed improvements are to consist of the demolition of selected structures present on the site, remodeling of one historical structure to remain, construction of a two-story storage building and a single story main with a basement classroom area. The remaining portion of the site will be developed with paved parking areas, driveways utilities, landscaping and flatwork as indicated on figure 2. Recommendations for excavations, fill placement, and foundation design for the proposed structure are presented in this report. References reviewed for this report are provided in Appendix A.

1.2 Scope of Services

The scope of services provided included:

- Review of readily available geologic and soils reports.
- Coordination of utility mark-outs.
- Excavation of truck-mounted exploratory borings and percolation test pits, and soil sampling.
- The performance of percolation testing in six percolation test pits.
- Laboratory testing of selected soil samples.
- Description of the geology and evaluation of potential geologic hazards.
- Engineering and geologic analysis.
- Preparation of this report.

2.0 SITE DESCRIPTION

The subject site is located northeast of the intersection of Glendora Ave. and Whitcomb Ave, in the city of Glendora, California (Figure 1). This site is currently developed with a one story church and 3 one-story and two-story residences. The subject site is bounded on the north, by an alleyway, beyond which one-story and two-story single family residences. The site is bounded to the west of Glendora Ave, beyond which are one-story single family residences. The subject site is bounded on the south by Whitcomb Ave, beyond which there is a relatively small one-story commercial office, and one-story and two-story single family residences. The subject site is bounded on the east by Vista Bonita Ave, beyond which are one-story single family residences.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

3.1 Field Investigation

The field investigation, conducted from December 27 and 28, 2019, included a visual reconnaissance, the excavation of three exploratory borings, as well as the excavation of six percolation test pits, and the performance of percolation testing. Borings were excavated with a truck-mounted drill rig equipped with eight-inch-diameter, hollow-stem augers that were advanced to depths ranging from 25 feet to 41.5 feet below the ground surface (bgs). Disturbed bulk samples representing a mixture of soils at relatively shallow depths were recovered from boring cuttings. Relatively undisturbed samples were collected by driving Standard Penetration Test and Modified California samplers within the borings at various depths as indicated on the Boring Logs (Appendix B).

The soils were logged in the field and visually classified in general accordance with the Unified Soil Classification System. The field descriptions have been modified, where appropriate, to reflect laboratory test results. Boring logs, including descriptions of the soils encountered, are included in Appendix B. The approximate locations of the borings and percolation test pits are presented on Figure 2.

3.2 Laboratory Testing

Laboratory tests were conducted on selected soil samples for classification purposes, and to evaluate physical properties and engineering characteristics. Laboratory tests included: Particle-Size distribution, in place moisture and density, shear, Soil Corrosion, Test descriptions and laboratory results are presented in Appendix C.

4.0 GEOLOGY

4.1 General Setting

The site is located on an alluvial fan/apron near the base of the southwestern side of the San Gabriel Mountains, within the Transverse Ranges geomorphic province. The east-west trending Transverse Ranges, a group of mountain ranges in Southern California that span between Point Conception to the west and the San Bernardino Mountains to the east, also include the San Gabriel Mountains, the Santa Ynez Mountains, and several of the northerly Channel Islands. The Geology found in the San Gabriel Mountains is composed of mostly Mesozoic granitic rocks the remaining geology is composed of Precambrian igneous and metamorphic rock complexes. Sediments in the alluvial fan are derived from the aforementioned rock types that comprise the San Gabriel Mountains.

4.2 Geologic and Soil Conditions

Based on the regional geologic map prepared by Dibblee, et. al. (2002). the site is underline Holocene alluvial gravel and sand deposits from the San Gabriel Mountains (Figure 3).

The United States Department of Agriculture's (USDA) Web Soil Survey identifies two soils in the project area (Figure 5). One soil is identified as "Urban land-Palmview-Tujunga, complex, 0 to 5 percent slopes", described as typically containing fine sandy loam to a profile depth of 79 inches. The USDA website indicates that Urban land-Palmview-Tujunga complex comprises the majority of the site (90%). The website indicates that the north/northeast quarter of the site is underlain with "Urban land-Palmview-Tujunga gravely complex 0 to 5 percent slopes", which are described as being comprised of fine sandy loam and sandy loam to a profile depth of 79 inches.

Alluvial Deposits were encountered in the exploratory borings to the maximum explored depth of 41.5 feet bgs. These soils were generally found to consist of medium dense to dense silty sands interbedded with clayey and gravely sands with lenses of clay below approximately 33ft. Soils encountered during subsurface investigation were generally found to be consistent with Dibblee (2000) and USDA findings.

4.3 Groundwater Conditions

At the time of drilling, no groundwater was observed in Boring B-1 through B-3.

The Los Angeles County Department of Public Works maintains monitoring wells at various locations. One of the wells (State Well ID #4356) is located approximately 6561.7 feet to the southeast near the intersection of S Loraine Ave and Steffen St. elevation of 829 feet above msl. Groundwater measurements were taken periodically between March 1975 and June 2009. The County of Los Angeles indicates that historical groundwater varied between 83.6 and 101.2 feet below current ground levels. In addition, a “Seismic Hazards Report” for the Glendora Quadrangle by the California Geological Survey shows historical groundwater may have reached between 100 feet below ground levels at the project location.

4.4 Percolation Rate Calculations

The percolation tests were performed per the “Conventional and Non-Conventional Onsite Wastewater Treatment Systems - Requirements and Procedures,” by the Los Angeles County Department of Public Health, Environmental Health Land Use Program, dated November 28, 2018, within native materials encountered in the excavated percolation test pits. The soil percolation rate is defined by the average time in minutes for a 1-inch column of water to “seep” into the soil, or how many inches that the water column will seep into the soil. Percolation rates were calculated (in minutes per inch) by dividing the time (in minutes) by the change (drop) in water level (in inches), and by dividing the change in water (in inches) per hour. No correction factor was used in the calculation for test pit diameter. The following is a summary of the percolation test results.

Table 1: Percolation Test Results			
Percolation Test Designation	Total Depth (in inches)	Percolation Rate (minutes per inch)	Average Percolation Rate Per Location (minutes per inch)
P-1	36	72	54
P-2	36	51	
P-3	36	40	
P-4	36	51	48
P-5	36	60	
P-6	36	33	

The percolation rates observed in the six percolation test pits were generally moderate. As such, the infiltration system should be designed moderately conservatively per the recommendations in Section 5.10 of this report.

It has been our experience in the subject area that entrained minerals often precipitate within drain systems over time in the form of calcium carbonate, sodium bicarbonate and other "evaporative" minerals. We recommend that any subsurface dispersion system designed account for the likelihood of mineral deposits that may clog or disrupt the system over the long term.

4.5 Geologic Hazards

Geologic hazards that were considered to have potential impacts to site development were evaluated based on field observations, literature review, and laboratory test results. It appears that the geologic hazards at the site are primarily limited to those caused by shaking from earthquake-generated ground motions. The following paragraphs discuss the geologic hazards considered and their potential risk to the site.

4.5.1 Surface Fault Rupture

Based on site reconnaissance and review of the referenced literature, the site is not within a State of California-designated Alquist-Priolo Earthquake Fault Studies Zone (Figure 5), and no known active fault traces underlie or project directly toward the site. According to the California Geological Survey, a fault is active if it displays evidence of activity in the last 11,000 years (CGS Special Publication 42, 2007). Therefore, the potential for surface rupture from displacement or fault movement beneath the proposed improvements is considered to be low.

4.5.2 Local and Regional Faulting

The California Geological Survey (CGS) and the United States Geological Survey (USGS) broadly group faults as “Class A” or “Class B”. Class A faults are identified based upon relatively well-defined paleoseismic activity, and a fault-slip rate of more than 5 millimeters per year (mm/yr). In contrast, Class B faults have comparatively less defined paleoseismic activity and are considered to have a fault-slip rate less than 5 mm/yr. The nearest known Class B fault is a segment of the Raymond fault located approximately 12.2 kilometers northwest of the site.

TABLE 2 NEAR-SITE FAULT PARAMETERS			
FAULT NAME	APPROXIMATE DISTANCE FROM SITE (KM)	MAXIMUM EARTHQUAKE MAGNITUDE	CLASSIFICATION
Raymond Fault	12.2	6.5	B
San Andreas Fault	29.4		A

Seismic ground motion values listed on the following Table 3 were derived in accordance with the 2018 International Building Code (IBC) and the 2016 California Building Code (CBC) that became effective January 1, 2014. The ground motion parameters were established based on Site Class and coordinates using the Applied Technology Council website (on line at https://hazards.atcouncil.org). Results for each set of seismic ground motion values, shown on Table 3, are based on the site coordinates of 34.141817 north latitude and -117.964831 longitude. These values are intended for the design of structures to resist the effects of earthquake ground motion.

TABLE 3 2018 IBC and 2020 CBC SEISMIC GROUND MOTION VALUES		
PARAMETER	VALUE	REFERENCE
Site Class	D	2018 IBC
Peak Ground Acceleration, PGA	0.865	2018 IBC
Mapped Spectral Response Acceleration Parameter, S_S	2.407	CBC Figure 22.1 (1)
Mapped Spectral Response Acceleration Parameter, S_1	.901	CBC Figure 22.2(2)
Seismic Coefficient, F_a	1	CBC Table 14.4-1(1)
Seismic Coefficient, F_v	1.5	CBC Table 14.2-2(2)
MCE Spectral Response Acceleration Parameter, S_{MS}	2.407	CBC Equation14.4-1
MCE Spectral Response Acceleration Parameter, S_{M1}	0.901	CBC Equation14.4-2
Design Spectral Response Acceleration, Parameter S_{DS}	1.605	CBC Equation14.4-3
Design Spectral Response Acceleration, Parameter S_{D1}	0.901	CBC Equation14.4-4

As such, the site could be subjected to significant shaking in the event of a major earthquake on any of the faults listed above or other faults in the Southern California.

4.5.3 Historic Seismicity

The recent seismic history (last 50 years) of the site area is moderate compared to other areas of Southern California. [No earthquakes above a 6.0 have been reported within a 50 km radius of the project site during the period of instrumental recordings, which began in the early 1900s.] The largest of the recorded earthquakes was the 1987 Whittier Narrows (moment magnitude of 5.9).

Review of the CGS historical California earthquake epicenters (http://redirect.conservation.ca.gov/cgs/rghm/quakes/historical/index.htm) for earthquakes with magnitude greater than M5.5 within 50 kilometers of the project site are provided on the following Table 4.

TABLE 4
Relatively Nearby Earthquake History

EARTHQUAKE DATE (yr-mo-day)	EARTHQUAKE TIME (GMT)	MAGNITUDE	DISTANCE FROM SITE (km)	GENERAL LOCATION
1990-02-28	1544	5.4	15.2	Upland
1987-10-01	NA	5.9	21.5	Whittier Narrows
2008-07-29	1142	5.4	23.0	Chino Hills
1926-02-18	0338	5.0	24.7	Pasadena
1991-06-28	0743	5.8	25.7	Sierra Madre

4.5.4 Liquefaction and Seismic Settlement Evaluation

Liquefaction occurs when saturated fine-grained sands or silts lose their physical strengths during earthquake-induced shaking and behave like a liquid. This is due to loss of point-to-point grain contact and transfer of normal stress to the pore water. Liquefaction potential varies with water level, soil type, material gradation, relative density, and probable intensity and duration of ground shaking. Seismic settlement results from densification of loose soils and can occur with or without liquefaction.

State of California Seismic Hazard Zones mapping for the Glendora Quadrangle (CGS, 1999) indicates that the site is located within a zone of required investigation for liquefaction. The mapping is based on historical occurrence of liquefaction or local, geological, geotechnical, or groundwater conditions.

Based on the relatively deep groundwater conditions, the relatively dense soils encountered during the field exploration, and the high quantity of fine-grained soils (silts and clays), it is our opinion that the potential for liquefaction during a major seismic event is relatively low.

4.4.5 Flooding, Tsunamis and Seiche Evaluation

According to the 2010 Federal Emergency Management Agency (FEMA) mapping, the project site lies within a “0.2% Annual Chance Flood Hazard Zone.” As such, we consider the chance of flooding to be relatively minimal, but should be considered during the design of the proposed building.

Due to the site’s location, elevation, and distance to bodies of water, the risk of flooding or damage to the site due to tsunamis or oscillatory waves (seiches) is considered negligible.

4.5.6 Landsliding and Debris Flows

Based on our review of regional maps prepared in the site vicinity, no landslides were mapped in the site area or in the adjacent hills. In addition, landslides were not encountered during our field exploration. The adjacent hills, canyons and an intermittent creek to the north of subject site are relatively close enough to the site such that the threat from debris flows should be considered low to moderate.

4.5.7 Compressible and Expansive Soils

Based on geologic observation and laboratory testing, the near-surface materials at the site generally have low to moderate expansion potential (EI lower than 90). The presence of expansive materials is not anticipated but maybe present due to the interbedded nature of the soils observed in the borings at the site. An expansion index test should be performed on the soils at final grade to confirm the findings in Section 4.2 of this report.

4.5.8 Corrosive Soils

Laboratory test results indicate that near-surface soils at the site generally present a negligible sulfate exposure to Portland cement concrete (2016 CBC; ACI 318, Table 4.3.1). The subject site would generally be considered to have a low corrosion potential. However, the soil resistivity test results for have been interpreted by others to represent a moderate potential for corrosivity for buried metallic conduits (Roberge, 1999). Additionally, there is a high percentage of clay materials present at the site, which are considered to have a lower resistivity, and thus a higher potential for corrosivity, than coarser grained materials such as sands or gravels. As such, it would seem prudent to utilize plastic conduits where proposed below grade and feasible. However, CTE does not practice corrosion engineering.

Therefore, if corrosion of metallic improvements is of more significant concern, a qualified corrosion engineer could be consulted.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 General

We conclude that the proposed development of the site is feasible from a geotechnical standpoint, provided the preliminary recommendations in this report are incorporated into the design and construction of the project. Recommendations for the proposed earthwork and improvements are included in the following sections and Appendix D. However, recommendations in the text of this report supersede those presented in Appendix D. The recommendations may require modifications as project plans evolve or based on the conditions encountered during earthwork.

5.2 Site Preparation

Following demolition of any existing structures in the area of the proposed addition and associated improvements, the proposed improvement areas should be cleared of existing debris and deleterious materials. Objectionable materials, such as existing fill material, construction debris, vegetation, and other deleterious materials not suitable for structural backfill should be disposed of off-site at a regulated disposal facility. Based on our explorations and review of available information, we anticipate approximately 2 or 3 feet of the existing soils at the site may be disturbed during previous construction or grading for the current improvements at the site. As such, the depth to competent native material may vary across the site, and should be identified by a CTE representative prior to placement of compacted fill.

In the area of the proposed buildings and distress-sensitive improvements, existing fill material and any eroded, desiccated, burrowed, or otherwise loose or disturbed soils should be excavated to the depth of competent native materials, at a minimum of three feet below existing grades, whichever depth is greatest, as described herein.

Total depths of overecavations will vary, depending on the portion of the project to be constructed. Overexcavation and removal recommendations for three types of improvements have been given below:

1. Overexcavations in the area of the proposed main building with a proposed basement should not be required, so long as foundations are placed in undisturbed, natural materials at a minimum depth of 5 feet. Further recommendations may be required if the basement does not cover or extend to the entire footprint of the proposed main building.
2. Overexcavations in the area of the proposed 2-story building should extend a minimum of five feet, or three feet below proposed footings, whichever is deepest. Overexcavations should extend 5 feet laterally.
3. Removals in proposed pavement, flatwork, or similar surface improvement areas should extend to competent materials, as directed by CTE during grading, and to a minimum depth of 12-inches below proposed or existing grades, whichever is deeper.

Exposed subgrades should be scarified, moisture conditioned, and properly compacted prior to receiving compacted fill.

If overexcavations extend such that they are adjacent to existing buildings, the overexcavations generally should not extend within a 1:1 plane extended down from the bottom of the existing footing that are to remain. However, depending on the condition of soils that will remain in place due to these limits, additional review and recommendations for additional removals could be required during grading. Depending on the depth of the existing building footings to remain, alternating slot excavations could be required during earthwork in order to properly prepare the underlying soils within the proposed improvements' bearing zone of influence.

Existing below-ground utilities should be redirected around the structure or, alternatively, the conflicting utility backfill material should be overexcavated to the depth of suitable material with the resultant void filled with a minimum one-sack cement/sand slurry or compacted fill. Existing utilities at an elevation to extend through the proposed footings should be sleeved and caulked to minimize the potential for moisture migration below the structure slab. Any existing utility backfill present within the prism created by a 1:1 plane extending from the outer edges of the footings to suitable material up to a minimum five feet beyond the building perimeter should be overexcavated and one-sack cement/sand slurry or compacted fill soil should be placed in the resulting area, as feasible. Abandoned pipes exposed by grading should be removed or securely capped at the limits of the removal excavations to prevent moisture from migrating beneath foundation and slab soils.

An engineer or geologist from CTE should observe all exposed ground surfaces prior to scarification and placement of compacted fill. As indicated, excavation should continue until suitable native materials are encountered. Organic and other deleterious materials not suitable for structural backfill should be properly disposed of off site.

5.3 Site Excavation

Based on CTE's observations, shallow excavations at the site should be feasible using standard, well-maintained heavy-duty construction equipment run by experienced operators. Excessively dense soils or large boulders requiring larger equipment or non-standard methods of excavation are not anticipated at the subject site, but cannot be precluded, especially in undocumented fill areas (if encountered).

5.4 Fill Placement and Compaction

Following removal of all existing fill material and any loose, disturbed soils, the areas to receive fills or improvements should be scarified approximately six inches, moisture conditioned, and properly compacted or wetted and proof-rolled, as appropriate. Fill and backfill should be compacted to a minimum relative compaction of 95 percent, at a moisture content at or near optimum, as evaluated by ASTM D 1557. The optimum lift thickness for fill soil will depend on the type of compaction equipment used. Generally, backfill should be placed in uniform, horizontal lifts not exceeding eight inches in loose thickness. Fill placement and compaction should be conducted in conformance with local ordinances.

5.5 Fill Materials

The on-site existing fill material and native materials are anticipated to be suitable for use as fill and backfill material. However, localized unsuitable materials, especially within the undocumented fill areas, could be encountered. Fill materials should be screened of organic materials and materials generally greater than three inches in maximum dimension. Irreducible materials greater than three inches in maximum dimension were not identified in the preliminary investigation; however, if such materials are encountered, they generally should not be used in shallow fills (within two feet of proposed foundations and utilities, or within three feet of proposed grades). In utility trenches, adequate bedding should surround pipes.

Imported fill beneath structures, pavement, and walks should have an expansion index of 20 or less (per ASTM D 4829). Imported fill soils for use in structural or slope areas should be evaluated by the soils engineer before importation to the site.

Retaining wall backfill, including proposed elevator pits or similar, located within a 45-degree wedge extending up from the heel of the wall should consist of soil having an Expansion Index of 20 or less (ASTM D 4829) with less than 30 percent passing the No. 200 sieve. The upper 12 to 18 inches of wall backfill could consist of lower permeability soils, in order to reduce surface water infiltration behind walls. Wall back drain details should be as per the project structural engineer and/or architect.

5.6 Temporary Construction Excavations

Temporary, un-surcharged excavations up to four feet deep may be cut vertically. Deeper excavations should be sloped back or shored. Temporary sloped excavations should be cut at a slope of 1:1 (horizontal:vertical) or flatter. Permanent slopes should be no steeper than 2:1. Vehicles and storage loads should not be placed within 10 feet of the top of the excavation. Deeper temporary slopes may be sloped or retained with shoring.

If temporary slopes are to be maintained during the rainy season, berms are recommended along the tops of slopes to divert runoff water from entering the excavation and eroding the slope faces.

5.6.1 General Shoring Recommendations

Shoring methods such as cantilevered shoring, tied-back walls or bracing with struts may be considered. The actual method of support should be evaluated by a shoring specialist.

The following recommendations are preliminary. Additional and more detailed recommendations can be provided on request.

5.6.2 Lateral Pressures

For design of cantilevered shoring, a triangular distribution of lateral earth pressure may be used. It may be assumed that the retained soils with a level surface behind the cantilevered shoring will exert a lateral pressure equal to that developed by a fluid with a density of 35 pounds per cubic foot. The use of cantilevered shoring is usually limited to a retained height of approximately 15 to 20 feet and shoring adjacent to any existing structures should be carefully evaluated since such shoring will be subject to deflection greater than braced or tied-back shoring.

5.6.3 Design of Soldier Piles

For the design of soldier piles spaced at least two diameters on-center, the allowable lateral bearing value (passive value) of the soils below the level of excavation may be assumed to be 250 pounds per square foot per foot of depth. To develop the full lateral value, provisions should be taken to assure firm contact between the soldier piles and the undistributed soils. The concrete placed in the soldier pile excavations may be a lean-mix concrete. However, the concrete used in that portion of the pile which is below the planned excavated level should be of sufficient strength to adequately transfer the imposed loads to the surrounding materials.

5.6.4 Lagging

Continuous lagging will be required between the soldier piles. The lagging should be installed as the excavation proceeds. The soldier piles 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 limited to a maximum value of 400 pounds per square foot.

5.6.5 Monitoring

Some means of monitoring the performance of the shoring system is recommended. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of soldier piles and the lateral movement along the lengths of selected soldier piles.

5.7 Foundations and Slab Recommendations

The following recommendations are for preliminary design purposes only. These foundation recommendations should be reviewed after completion of rough grading of the building pad areas. We anticipate that the expansion potential of the site will be relatively low to moderate. The recommendations herein have been developed for soils with an EI of less than 90. Therefore, after finish grades are reached, testing for EI should be conducted and these recommendations updated, as/if necessary.

5.7.1 Foundations

Continuous and isolated spread footings are suitable for use at this site. Foundation dimensions and reinforcement should be based on allowable bearing values of 2,000 pounds per square foot (psf) for minimum 15-inch wide footings embedded at least 24 inches below the lowest exterior rough grade elevation. The allowable bearing value may be increased by 250 psf for each additional 12 inches of width or depth of embedment, up to a maximum value of 3,000 psf.

Footings for the proposed new main building with a basement can be founded on natural soils. If the basement for the proposed main building is partial and not covering the entire footprint of the proposed building, then further overexcavation and recompaction recommendations, and footing recommendations may be necessary once final plans are available.

We anticipate that all building footings for the proposed 2-story structure will be founded on a minimum of three feet of properly compacted fill derived from suitable materials as recommended herein with a moderate expansion potential (EI less than 90). Since footing depths should be a minimum of 2 feet below grades, overexavations for the proposed 2-story building will require a minimum of 5-foot overexcavation, both vertically and laterally.

The allowable bearing value may also be increased by one-third for short-duration loading which includes the effects of wind or seismic forces. If elastic design is utilized, an uncorrected subgrade modulus of reaction (k) of 135 pounds per square inch per inch is considered appropriate for the proposed foundations.

Minimum footing reinforcement for continuous footings should be as per the structural engineer, but consist of a minimum four #5 bars, two near the top and two near the bottom, in order to provide additional rigidity against the anticipated moderately expansive site soils. The structural engineer should provide recommendations for reinforcement of any spread footings and footings with pipe penetrations.

Footing excavations should generally be maintained above optimum moisture content until concrete placement. If footing excavations are allowed to dry out, presoaking of the excavation to a minimum 120% of the optimum moisture content would likely be recommended.

5.7.2 Foundation Settlement

The static settlement is expected to be on the order of one inch and the differential settlement is expected to be on the order of 0.5 inch.

5.7.3 Foundation Setback

Footings for structures should be designed such that the horizontal distance from the face of adjacent slopes to the outer edge of the footing is at least 10 feet. In addition, footings should bear beneath a 1:1 plane extended up from the nearest bottom edge of adjacent trenches and/or excavations. Deepening of affected footings may be a suitable means of attaining the prescribed setbacks.

5.7.4 Interior Concrete Slabs

Due to the anticipated moderately expansive site soils, lightly loaded concrete slabs should be designed for the anticipated loadings but measure at least 5 inches in thickness with minimum slab reinforcement of #4 reinforcing bars spaced no more than 18 inches, on center, each way, at above mid-slab height, but with proper concrete cover; unless more stringent recommendations by the project architect or structural engineer are provided. If elastic design is utilized, a subgrade modulus of reaction of 135 pounds per square inch per inch is considered appropriate.

In moisture-sensitive floor areas, a suitable vapor retarder of at least ten-mil thickness (with all laps or penetrations sealed or taped) overlying a two-inch layer of consolidated aggregate base or clean sand (with SE of 30 or greater) should be installed. A maximum two-inch layer of similar material may be placed above the vapor retarder to protect the membrane during steel and concrete placement; however, best protection from moisture intrusion or emission through the slab is anticipated to result from placement of slab concrete directly upon the vapor retarder. This recommended protection is generally considered typical in the industry. If proposed floor areas or coverings are considered especially sensitive to moisture emissions, additional recommendations from a specialty consultant could be obtained. CTE is not an expert at preventing moisture penetration through slabs. A qualified architect or other experienced professional could be contacted if moisture penetration is a more significant concern.

Subgrade materials should be maintained near or above optimum moisture content until slab underlayment or concrete are placed.

5.8 Lateral Resistance and Earth Pressures

Lateral loads acting against structures may be resisted by friction between the footings and the supporting soil or passive pressure acting against structures. If frictional resistance is used, we recommend allowable coefficients of friction of 0.30 (total frictional resistance equals the coefficient of friction multiplied by the dead load) for concrete cast directly against compacted fill. A design passive resistance value of 200 pounds per square foot per foot of depth (with a maximum value of 1,500 pounds per square foot) may be used. The allowable lateral resistance can be taken as the sum of the frictional resistance and the passive resistance, provided the passive resistance does not exceed two-thirds of the total allowable resistance.

Retaining walls up to approximately eight feet high and backfilled using granular soils (most likely imported or select onsite soils) may be designed using the equivalent fluid weights given in Table 6.

TABLE 6
EQUIVALENT FLUID UNIT WEIGHTS
(pounds per cubic foot)

WALL TYPE	LEVEL BACKFILL	SLOPE BACKFILL 2:1 (HORIZONTAL: VERTICAL)
CANTILEVER WALL (YIELDING)	40	50
RESTRAINED WALL	65	80

Lateral pressures on cantilever retaining walls (yielding walls) due to earthquake motions may be calculated based on work by Seed and Whitman (1970). The total lateral thrust against a properly drained and backfilled cantilever retaining wall above the groundwater level can be expressed as:

$P_{AE} = P_A + \Delta P_{AE}$

For non-yielding (or "restrained") walls, the total lateral thrust may be similarly calculated based on work by Wood (1973):

$$P_{KE} = P_K + \Delta P_{KE}$$

Where:

P_A = Static Active Thrust (given previously Table 8)
P_K = Static Restrained Wall Thrust (given previously Table 8)
ΔP_{AE} = Dynamic Active Thrust Increment = $(3/8)\ k_h\ \gamma H^2$
ΔP_{KE} = Dynamic Restrained Thrust Increment = $k_h\ \gamma H^2$
k_h = ½ Peak Ground Acceleration = $½(S_{DS}/2.5)$
H = Total Height of the Wall
γ = Total Unit Weight of Soil ≈ 135 pounds per cubic foot

The increment of dynamic thrust in both cases should be distributed trapezoidally, with a line of action located at 0.6H above the bottom of the wall.

These values assume non-expansive backfill and free-draining conditions. Measures should be taken to prevent moisture buildup behind all retaining walls. Drainage measures should include free-draining backfill materials and sloped, perforated drains. These drains should discharge to an appropriate off-site location. Wall backdrains and waterproofing should be adequately detailed and/or specified as per the project structural engineer or architect.

5.9 Underground Structure

The finished surface of the basement for the proposed building is expected to be constructed to a depth of approximately 15 feet below the existing ground surface (including the footing depths). Based on the anticipated grading, the structure may be founded on spread footings designed in accordance with the foundation recommendations previously presented.

For basement walls below grade, we recommend that they be designed for a triangular pressure distribution of 40 pcf. The seismic pressure should be added as outlined in Section 5.8. The recommended earth pressure assumes that a drainage system will be installed the behind the base of the walls, so that external water pressure will not develop against the basement walls.

In addition to the recommended earth pressures, the walls adjacent to vehicular traffic areas 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 walls due to normal traffic.

5.9.1 Drainage System

Walls below grade should be properly drained. Drainage behind the parking garage walls may be provided by a geosynthetic drainage composite. In our opinion, Miradrain 6000 (or the equivalent), attached to the back of the wall before backfilling, would provide satisfactory drainage. The drain should be placed continuously along the back of the wall and connected to a 6-inch-diameter perforated discharge pipe. The pipe should be sloped at least 2% and surrounded by one cubic foot per foot of filter gravel wrapped in geo-fabric. The drain should discharge through a solid pipe to a sump or other appropriate outlet. The wall should be appropriately waterproofed.

The filter gravel should meet the requirements of Class II Permeable Material as defined in the current State of California, Department of Transportation, Standard Specifications. If Class II Permeable Material is not available, 3/4-inch crushed rock or gravel can be used. The crushed rock or gravel should have less than 5% passing a No. 200 sieve.

5.10 Exterior Flatwork

To reduce the potential for cracking in exterior flatwork caused by minor movement of subgrade soils and concrete shrinkage, we recommend that such flatwork be installed with crack-control joints at appropriate spacing as designed by the project architect. Additionally, we recommend that flatwork be installed with at least #3 reinforcing bars at maximum 24-inch centers, each way, at mid-height of slab or as per the other project consultants. Flatwork, which should be installed with crack control joints, includes driveways, sidewalks, and architectural features. All subgrades should be prepared according to the earthwork recommendations previously given before placing concrete. Positive drainage should be established and maintained next to all flatwork. Subgrade materials shall be maintained or elevated to above optimum moisture content until just before concrete placement.

5.11 Infiltration/Percolation Recommendations

The infiltration rates across the site vary from 48 to 54 minutes per inch We recommend designing a system that assumes approximately **60 minutes per inch** or **½ inch per hour** with the assumption that portions of the infiltration system will overlay **extremely slow or non-percolating soils**.

5.12 Drainage

Surface runoff should be collected and directed away from improvements by means of appropriate erosion-reducing devices and positive drainage should be established around the proposed improvements. Positive drainage should be directed away from improvements at a gradient of at least two percent for a distance of at least five feet. However, the project civil engineers should evaluate the on-site drainage and make necessary provisions to keep surface water from affecting the site.

Generally, CTE recommends against allowing water to infiltrate building pads or adjacent to slopes. We understand that some agencies are encouraging the use of storm-water infiltration devices. Use of such devices tends to increase the possibility of high groundwater and soil, slope, and/or wall instability. If infiltration devices must be used, then we recommend that they be underlain by an impervious barrier and that the infiltrate be collected via subsurface piping and discharged off site.

5.13 Pavement Design

We understand that part of the proposed new building will include new parking areas and driveways, roads, as indicated on Figure 2.

Presented in Table 7 are preliminary pavement sections based on an assumed R-Value and estimated traffic indices (TI). The upper 12 inches of subgrade and all base materials should be properly compacted to 90 percent relative compaction at above optimum moisture content. All base materials should be properly compacted to 95 percent relative compaction at or near optimum moisture content.

TABLE 7 RECOMMENDED PAVEMENT THICKNESS				
Traffic Area	Assumed Traffic Index	Preliminary Subgrade "R"-Value	AC Thickness (in)	Class 2 * Aggregate Base Thickness (in)
Pedestrian or Emergency Vehicle Roads	4.0	5	2.5	7.5
Auto Parking Areas Only	5.0	5	3.0	10.0

* Caltrans Class 2 aggregate base

After grading, CTE recommends laboratory testing of at-grade soils for an as-graded R-Value. Pavement sections can be reduced if the subgrade is lime-treated. Further recommendations can be made should this option be pursued.

Asphalt paved areas should be designed, constructed, and maintained in accordance with, for example, the recommendations of the Asphalt Institute, or other widely recognized authority. The Standard Specifications for Public Works Construction ("Greenbook") may be referenced for pavement materials specifications.

5.14 Construction Observation

The recommendations provided in this report are based on preliminary design information for the proposed construction and the subsurface conditions observed in the exploratory borings. The interpolated subsurface conditions should be checked in the field during construction to verify that conditions are as anticipated. When applicable, soil samples should be collected prior to grading and tested for laboratory-defined optimum moisture contents with respect to maximum soil densities of compacted fill material. Upon completion of precise grading, soil samples should be collected to evaluate as-built expansion index and soluble-sulfate content of at-grade soils. Foundation recommendations may be revised upon completion of grading, and as-built laboratory tests results.

Recommendations provided in this report are based on the understanding and assumption that CTE will provide the observation and testing services for the project. All earthwork should be observed and tested to verify that grading activity has been performed according to the recommendations contained within this report. The project engineer should evaluate all footing trenches before reinforcing steel placement.

5.15 Plan Review

CTE should be authorized to review the project foundation plans and grading plans before commencement of earthwork to identify potential conflicts with the intent of CTE's recommendations.

6.0 LIMITATIONS OF INVESTIGATION

The field evaluation, laboratory testing and geotechnical analysis presented in this report have been conducted according to current engineering practice and the standard of care exercised by reputable geotechnical consultants performing similar tasks in this area. No other warranty, expressed or implied, is made regarding the conclusions, recommendations and opinions expressed in this report. Variations may exist and conditions not observed or described in this report may be encountered during construction.

The findings of this report are valid as of the present date. However, changes in the conditions of a property can occur with the passage of time, whether they be due to natural processes or the works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur, whether they result from legislation or the broadening of knowledge. Accordingly, the findings of this report may be invalidated wholly or partially by changes outside our control. Therefore, this report is subject to review and should not be relied upon after a period of three years.

The recommendations presented herein have been developed to reduce the adverse affects of expansive onsite soils. However, even with the design and construction precautions provided herein, some post-construction movement could occur.

CTE's conclusions and recommendations are based on an analysis of the observed conditions. If conditions different from those described in this report are encountered, our office should be notified and additional recommendations, if required, will be provided.

We appreciate this opportunity to be of service on this project. If you have any questions regarding this report, please do not hesitate to contact the undersigned.

Respectfully submitted,
CTE South, Inc.

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Project Geologist

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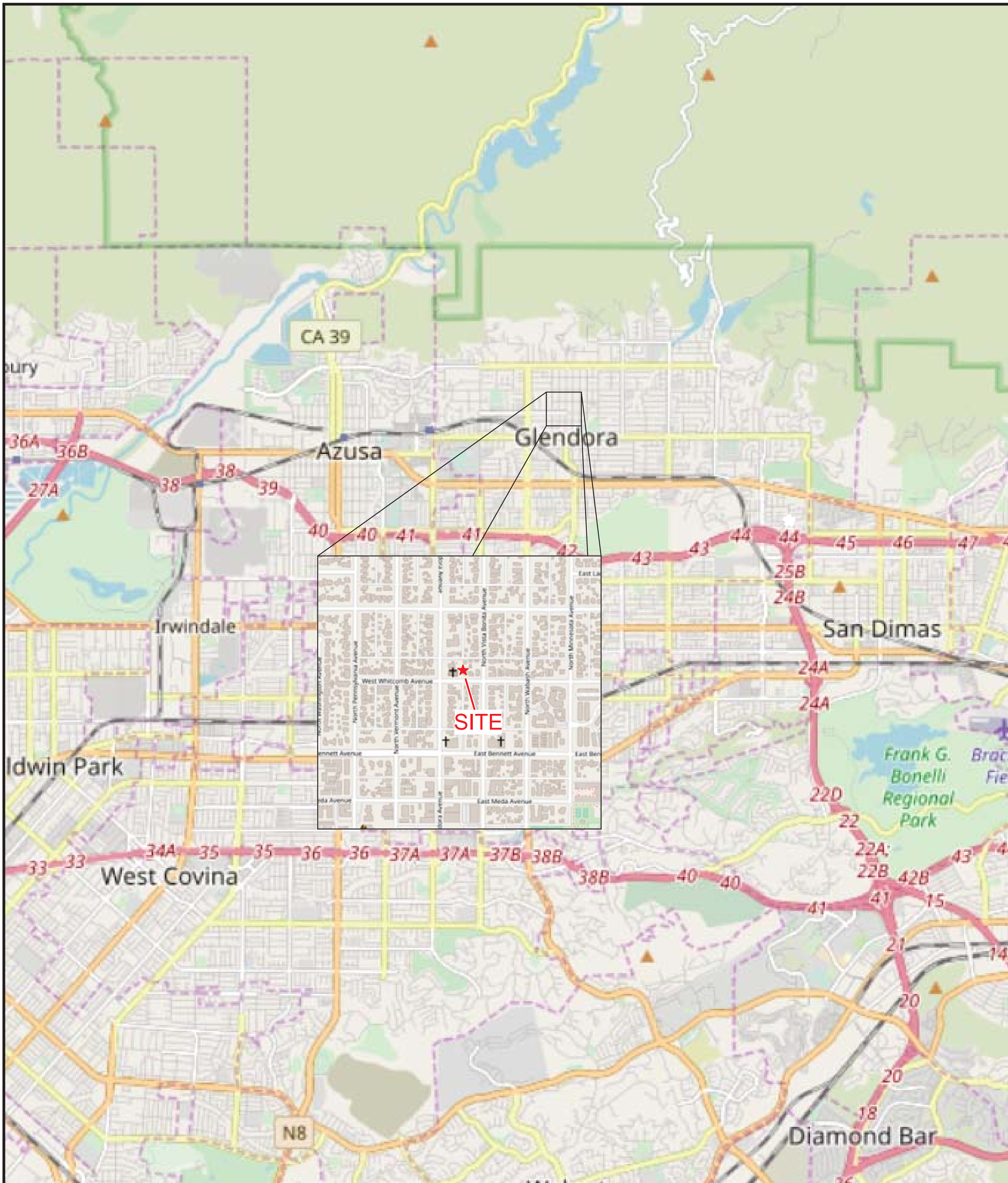
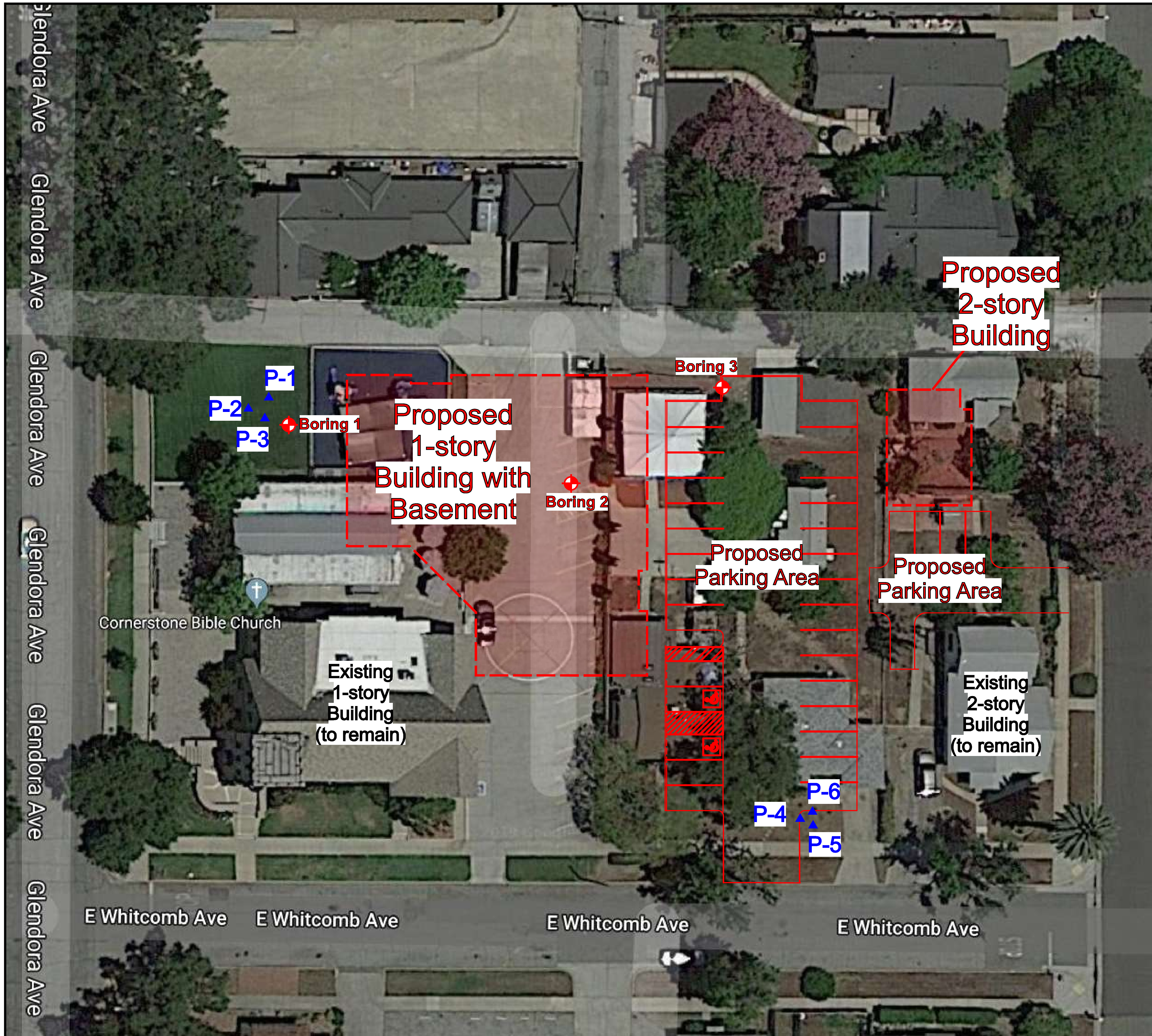


CTE SOUTH

Site Location			
Cornerstone Bible Church, 400 N. Glendora Ave., Glendora CA			
Not to Scale	January 2020	Pr. No. 40-3817G	Figure 1



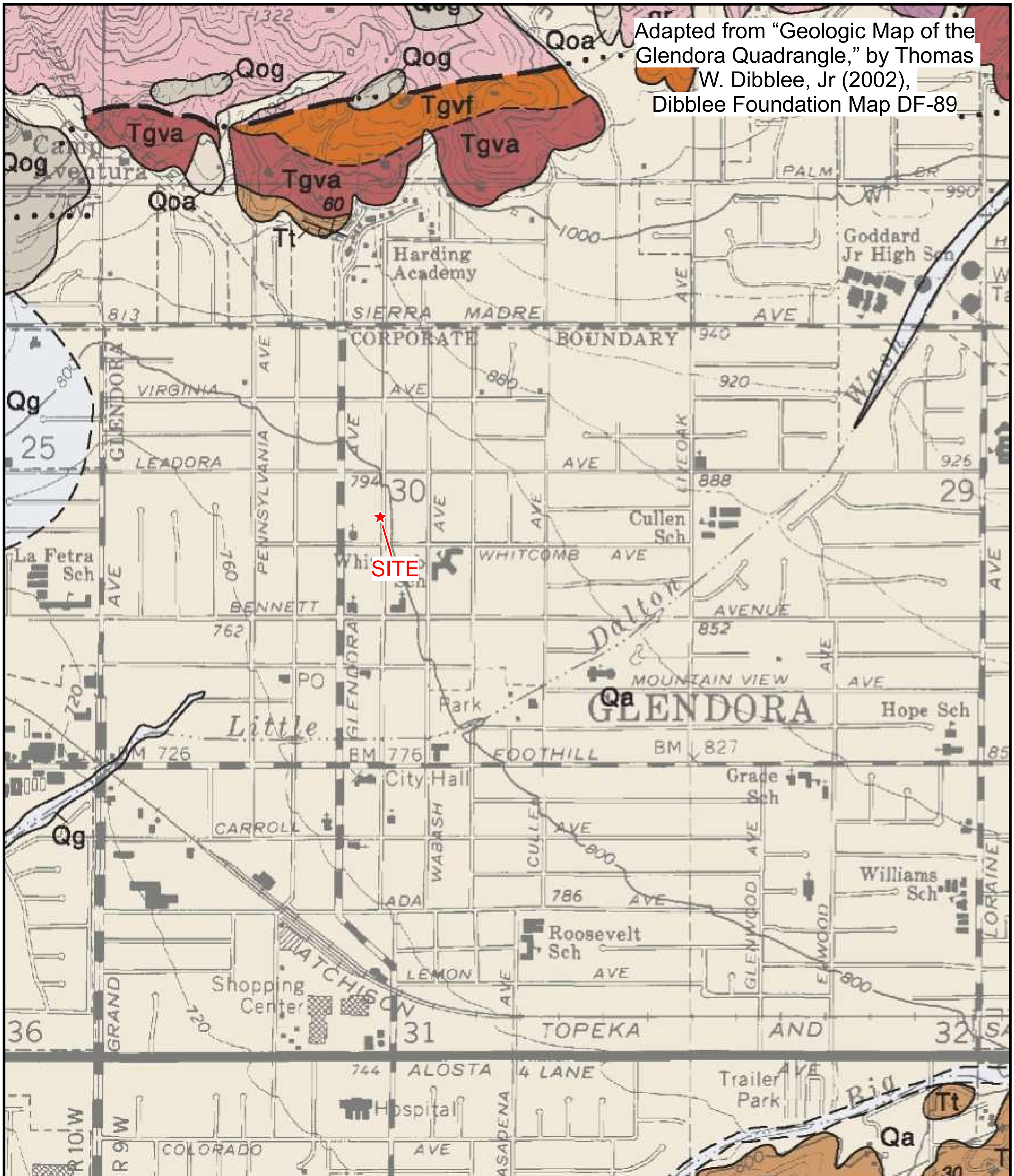
LEGEND

Boring 3 — Approximate Location of Boring

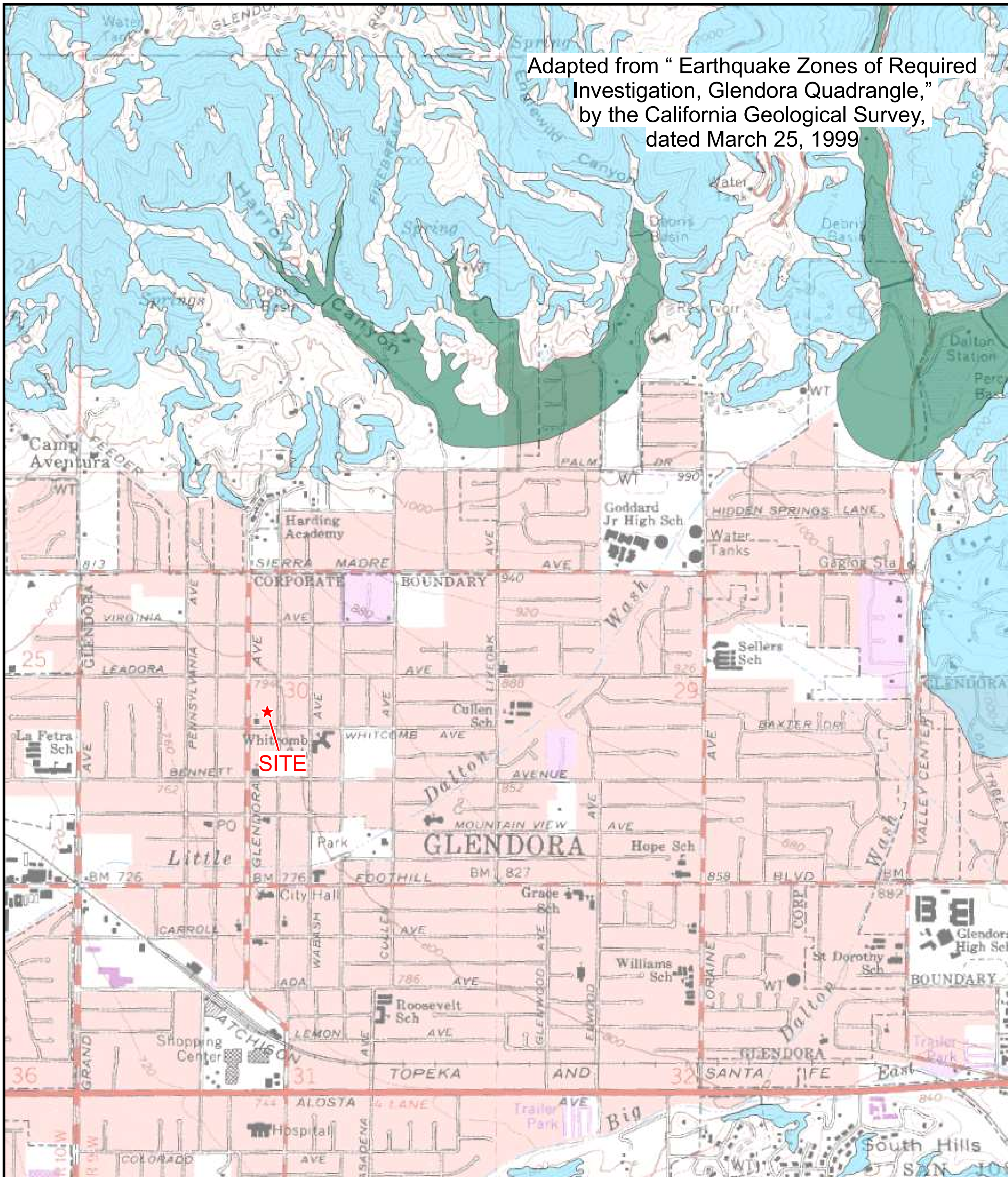
P-6 — Approximate Location of Percolation Test

Site Plan			
Cornerstone Bible Church, 400 N. Glendora Ave., Glendora CA			
Approximate Scale 1" = 40'	January 2020	Project No. 40-3817G	Figure 2

CTE SOUTH



<table>
<tr><td rowspan="3">CTE SOUTH</td><td colspan="4">Geologic Map</td></tr>
<tr><td colspan="4">Cornerstone Bible Church, 400 N. Glendora Ave., Glendora, CA</td></tr>
<tr><td>Not to Scale</td><td>January 2020</td><td>Pr. No. 40-3817G</td><td>Figure 3</td></tr>
</table>



Geohazards Map			
Cornerstone Bible Church, 400 N. Glendora Ave., Glendora, CA			
Not to Scale	January 2020	Pr. No. 40-3817G	Figure 4

CTE SOUTH

APPENDIX A

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REFERENCES

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<u>APPENDIX B</u>

EXPLORATION LOGS

EXPLORATION METHODS

The soil conditions within the site were explored by drilling 3 hollow-stem auger borings and at the location shown on Figure 2. The borings were drilled using 8-inch-diameter hollow-stem auger drilling equipment. The soils encountered were classified in the accordance with the Unified Soil Classification System. Results of the borings are presented in this Appendix.

Our field representatives obtained relatively undisturbed and bulk samples for laboratory observation and testing. The number of blows of the hammer needed to drive the sampler 12 inches was recorded as an indication of the density or consistency of the earth materials.

In addition to obtaining undisturbed samples, Standard Penetration Tests (SPT) were performed in hollow stem borings. The results of the tests are indicated on the boring logs. The standard penetration tests were performed in accordance with the ASTM D1586 Test Method.

The hammer weights for various depths and drilling equipment are summarized in the following tables.

HAMMER WEIGHTS

Sampling Type	Weight in pounds
Undisturbed (30-inch drop)	140
SPT (30-inch drop)	140

CTE SOUTH

DEFINITION OF TERMS

PRIMARY DIVISIONS			SYMBOLS	SECONDARY DIVISIONS
COARSE GRAINED SOILS MORE THAN HALF OF MATERIAL IS LARGER THAN NO. 200 SIEVE SIZE	**GRAVELS** MORE THAN HALF OF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS < 5% FINES	GW	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES LITTLE OR NO FINES
			GP	POORLY GRADED GRAVELS OR GRAVEL SAND MIXTURES, LITTLE OF NO FINES
		GRAVELS WITH FINES	GM	SILTY GRAVELS, GRAVEL-SAND-SILT MIXTURES, NON-PLASTIC FINES
			GC	CLAYEY GRAVELS, GRAVEL-SAND-CLAY MIXTURES, PLASTIC FINES
	SANDS MORE THAN HALF OF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS < 5% FINES	SW	WELL GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
			SP	POORLY GRADED SANDS, GRAVELLY SANDS, LITTLE OR NO FINES
		SANDS WITH FINES	SM	SILTY SANDS, SAND-SILT MIXTURES, NON-PLASTIC FINES
			SC	CLAYEY SANDS, SAND-CLAY MIXTURES, PLASTIC FINES
FINE GRAINED SOILS MORE THAN HALF OF MATERIAL IS SMALLER THAN NO. 200 SIEVE SIZE	**SILTS AND CLAYS** LIQUID LIMIT IS LESS THAN 50		ML	INORGANIC SILTS, VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, SLIGHTLY PLASTIC CLAYEY SILTS
			CL	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY, SANDY, SILTS OR LEAN CLAYS
			OL	ORGANIC SILTS AND ORGANIC CLAYS OF LOW PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT IS GREATER THAN 50		MH	INORGANIC SILTS, MICACEOUS OR DIATOMACEOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTY CLAYS
HIGHLY ORGANIC SOILS			PT	PEAT AND OTHER HIGHLY ORGANIC SOILS

GRAIN SIZES

BOULDERS	COBBLES	GRAVEL		SAND			SILTS AND CLAYS
		COARSE	FINE	COARSE	MEDIUM	FINE	

12" 3" 3/4" 4 10 40 200

CLEAR SQUARE SIEVE OPENING U.S. STANDARD SIEVE SIZE

ADDITIONAL TESTS

(OTHER THAN TEST PIT AND BORING LOG COLUMN HEADINGS)

MAX- Maximum Dry Density
GS- Grain Size Distribution
SE- Sand Equivalent
EI- Expansion Index
CHM- Sulfate and Chloride
Content , pH, Resistivity
COR - Corrosivity
SD- Sample Disturbed

PM- Permeability
SG- Specific Gravity
HA- Hydrometer Analysis
AL- Atterberg Limits
RV- R-Value
CN- Consolidation
CP- Collapse Potential
HC- Hydrocollapse
REM- Remolded

PP- Pocket Penetrometer
WA- Wash Analysis
DS- Direct Shear
UC- Unconfined Compression
MD- Moisture/Density
M- Moisture
SC- Swell Compression
OI- Organic Impurities

FIGURE: BL1

CTE SOUTH

PROJECT:	Cornerstone Bible Church	DRILLER:	Choice Drilling
CTE JOB NO:	40-3817G	DRILL METHOD:	8" Hollow Stem
LOGGED BY:	Kris Hernandez and Rob Lomino	SAMPLE METHOD:	SPT, Cal. Barrel and Bulk

Depth (Feet)	Bulk Sample	Driven Type	Blows/Foot	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	BORING LEGEND DESCRIPTION	Laboratory Tests
0								Block or Chunk Sample	
								Bulk Sample	
5									
								Standard Penetration Test	
10								Modified Split-Barrel Drive Sampler (Cal Sampler)	
								Thin Walled Army Corp. of Engineers Sample	
15								Groundwater Table	
								Soil Type or Classification Change	
20								? —— ? —— ? —— ? —— ? —— ? —— ? –	
								Formation Change [(Approximate boundaries queried (?)]	
25						"SM"		Quotes are placed around classifications where the soils exist in situ as bedrock	

FIGURE: BL2

CTE SOUTH

PROJECT:	Cornerstone Bible Church	DRILLER:	Choice Drilling	SHEET:	1 of 2
CTE JOB NO:	40-3817G	DRILL METHOD:	8" Diam. Hollow Stem	DRILLING DATE:	12/27/2019
LOGGED BY:	Robert Lomino, Kris Hernandez	SAMPLE METHOD:	Bulk, Cal. Barrel, and SPT	ELEVATION:	

Boring B-1

Depth (Feet)	Blows/6 Inches	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	DESCRIPTION	Laboratory Tests/Notes
0				CL (Fill)	**Fill**: Silty clay (CL), medium stiff, moist, dark brown	
	3 12 18	104.7	15.5	CL	**Natural soil:** Silty clay (CL), stiff, moist to slightly moist, stiff, moist, dark brown	M/D
5	4 7 7				Increase in sand content, stiff	
10	19 24 27	108.4	15.9		Increase in density	M/D; #200 Wash (54.9% passing)
15	9 12 18			SM	Silty sand (SM), some gravel, medium dense, slightly moist, medium to dark brown	
20	15 4 6	119.1	9.3			M/D; #200 Wash (40.2% passing)
25						

B-1a

CTE SOUTH

PROJECT:	Cornerstone Bible Church	DRILLER:	Choice Drilling	SHEET:	2 of 2
CTE JOB NO:	40-3817G	DRILL METHOD:	8" Diam. Hollow Stem	DRILLING DATE:	12/27/2019
LOGGED BY:	Robert Lomino, Kris Hernandez	SAMPLE METHOD:	Bulk, Cal. Barrel, and SPT	ELEVATION:	

Boring B-1

Depth (Feet)	Bulk Sample	Driven Type	Blows/6 Inches	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests/Notes
25			12 19 47			SM		Slightly clayey silty sand (SM), dense to very dense, slightly moist, dark brown	Minimal sample recovery
						SC		Clayey sand (SC), dense, slightly moist, dark brown	
30			12 18 22	114.9	10.0				M/D
								Total depth of boring: 31.5' No groundwater observed Boring caved to a depth of 26.5' Boring was backfilled with bentonite chips.	
35									
4									
45									
50									

B-1b

CTE SOUTH

PROJECT:	Cornerstone Bible Church	DRILLER:	Choice Drilling	SHEET:	1 of 2
CTE JOB NO:	40-3817G	DRILL METHOD:	8" Diam. Hollow Stem	DRILLING DATE:	12/27/2019
LOGGED BY:	Robert Lomino, Kris Hernandez	SAMPLE METHOD:	Bulk, Cal. Barrel, and SPT	ELEVATION:	

Boring B-2

Depth (Feet)	Bulk Sample	Driven Sample Type	Blows/6 Inches	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests/Notes
0								3" of Concrete Slab	
						SW		**Fill**: Well graded sand (SW), slightly gravely, light brown-grey	
						SM-SC		**Natural soil**: Slightly silty clayey sand (SM-SC), dense, slightly moist, dark brown	
			18 22 27	112.4	12.8				M/D
5			9 12 18			SC		Slightly clayey sand (SC), dense, slightly moist, dark brown	Minimal Sample Recovery
10			18 27 30			SM		Very silty sand (SM), dense, slightly moist, light to medium brown, some gravel content	
15			18 22 27						
20			27 30 32	118.1	8.7			Increase in clay content	M/D
25									

B-2a

PROJECT:	Cornerstone Bible Church	DRILLER:	Choice Drilling	SHEET:	2 of 2
CTE JOB NO:	40-3817G	DRILL METHOD:	8" Diam. Hollow Stem	DRILLING DATE:	12/27/2019
LOGGED BY:	Robert Lomino, Kris Hernandez	SAMPLE METHOD:	Bulk, Cal. Barrel and SPT	ELEVATION:	

Boring B-2

Depth (Feet)	Sample: Bulk	Sample Type: Driven	Blows/12 Inches	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests/Notes
25			10 12 18			SM		Silty sand, as described on previous page Increase in gravel content	
30			27 15 56	119.8	6.8	SP-SM		Poorly graded silty sand (SP-SM), very dense, slightly moist, medium brown	M/D
35			12 18 22			ML		Clayey, sandy silt (ML), light brown, very stiff, slightly moist	
4			27 32 40	109.2	14.0			Increase in clay content	M/D; #200 Wash (60.0% passing)
45								Total depth of boring: 41.5' No groundwater observed Boring caved to a depth of 36.0' Boring was backfilled with bentonite chips.	
50									

B-2b

PROJECT:	Cornerstone Bible Church	DRILLER:	Choice Drilling	SHEET:	1 of 2
CTE JOB NO:	40-3817G	DRILL METHOD:	8" Diam. Hollow Stem	DRILLING DATE:	12/27/2019
LOGGED BY:	Robert Lomino, Kris Hernandez	SAMPLE METHOD:	Bulk, Cal. Barrel, and SPT	ELEVATION:	

Boring B-3

Depth (Feet)	Bulk Sample	Driven Type	Blows/6 Inches	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests/Notes
0						CL (Fill)		**Fill:** Slightly silty clay (CL), medium dense, moist, dark brown	
			9 9 26			CL		**Natural Soil:** Slightly silty clay (CL), medium dense, moist, dark brown	
5			7 9 12			SM		Silty sand (SM), medium dense, slightly moist, medium to dark brown	
10			37 15 32	121.3	5.1	SM		Slightly silty sand (SM), very dense, slightly moist, medium to dark brown	M/D
15			18 19 22					Very silty sand (SM), very dense, slightly moist, medium to light brown	
20			19 22 27	115.9	10.5	SM			M/D
25			15 19 21					Light brown	

B-3a

PROJECT:	Cornerstone Bible Church	DRILLER:	Choice Drilling	SHEET:	2 of 2
CTE JOB NO:	40-3817G	DRILL METHOD:	8" Diam. Hollow Stem	DRILLING DATE:	12/27/2019
LOGGED BY:	Robert Lomino, Kris Hernandez	SAMPLE METHOD:	Bulk, Cal. Barrel and SPT	ELEVATION:	

Boring B-3

Depth (Feet)	Bulk Sample	Driven Sample Type	Blows/6 Inches	Dry Density (pcf)	Moisture (%)	U.S.C.S. Symbol	Graphic Log	DESCRIPTION	Laboratory Tests/Notes
25						SM		Silty sand (SM), medium dense, slightly moist, medium to dark brown	
30			12 19 32	109.4	8.7			Decrease in silt content	M/D; #200 Wash (28.0% passing)
35									
4									
45									
50									

Total depth of boring: 30.5'
No groundwater observed
Boring caved to a depth of 25.0'
Boring was backfilled with bentonite chips.

B-3b

APPENDIX C

LABORATORY METHODS AND RESULTS

APPENDIX C
LABORATORY METHODS AND RESULTS

Laboratory Testing Program

Laboratory tests were performed on representative soil samples to detect their relative engineering properties. Tests were performed following test methods of the American Society for Testing Materials or other accepted standards. The following presents a brief description of the various test methods used.

Classification

Soils were classified visually according to the Unified Soil Classification System. Visual classifications were supplemented by laboratory testing of selected samples according to ASTM D2487. The soil classifications are shown on the Exploration Logs in Appendix B.

Moisture and Density Tests

Moisture content and unit dry density tests were performed on samples of undisturbed soil obtained in the borings. Dry density and field moisture information is useful in correlating field and laboratory data, and in providing a gross picture of the variations of soil characteristics. The results of the tests are presented on the boring logs in Appendix A.

Particle-Size Analysis

Particle-size analyses were performed on selected representative samples according to ASTM D 422. To estimate the particle size distribution of the soils and to aid in classifying the soils, grain size analyses were performed on samples obtained from the borings. The percentage of “fines” (percent passing the No. 200 sieve) of various samples is presented on the boring logs in Appendix A.

Direct Shear

Direct shear tests were performed on either samples direct from the field or on samples recompacted to a specific density. Direct shear testing was performed in accordance with ASTM D 3080. The samples were inundated during shearing to represent adverse field conditions.

Soil Corrosion Analysis

Soil materials were sent to HDR Laboratories Inc. in Claremont, CA. Results from HDR are enclosed with this appendix.

Boring No.	B-1	B-1	B-2	B-3				
Sample No.	1	2	3	4				
Depth (ft.)	10-11.5	20-21.5	40-41.5	29-30.5				
Sample Type	CAL	CAL	CAL	CAL				
Visual Soil Classification	Dark Brown (CL)	Dark Brown (SM)	Light Brown (CL)	Dark Brown (SM)				
Total Sample Weight (Coarse Fraction)								
Total Sample Weight (g):								
Plus #4 Weight (g):								
Percent Retained Coarse Fraction (+ #4):								
Sample Weight (Finer Fraction)								
Weight of Moist Sample + Container (g):	358.3	289.8	366.8	359.3				
Weight of Dry Sample + Container (g):	322.2	270.3	332.3	338.3				
Weight of Container (g):	86.7	94.3	87.1	86.3				
Moisture Content (%):	13.3	10.0	12.3	7.7				
Container No.:	1.0	2.0	3.0	4.0				
Weight After Wash								
Dry Weight of Sample + Container (g):	192.8	199.5	185.3	267.8				
Weight of Container (g):	86.7	94.3	87.1	86.3				
Dry Weight of Sample (gm):	106.1	105.2	98.2	181.5				
% Retained No. 200 Sieve	45.1	59.8	40.0	72.0				
% Passing No. 200 Sieve	**54.9**	**40.2**	**60.0**	**28.0**				

CTE SOUTH

PERCENT PASSING No. 200 SIEVE
ASTM D 1140

Project Name: Cornerstone Bible Church
Project No.: 40-3817G
Lab No.: 1285
Tested By: GN Date: 01/03/20

MOISTURE & DENSITY TEST

In Accordance with ASTM D2216, D2937

Project Name: Cornerstone Bible Church
Job Number: 40-3817G **Date Sampled:** December 27, 2019
Lab Number: 1285 **Date Tested:** December 30, 2019

Boring No.	B-1	B-1	B-1	B-1		
Depth (ft.)	3-4.5	10 - 11.5	20- 21.5	30-31.5		
Sample Ht. (in.)	3.0	3.0	3.0	3.0		
Soil+Rings+Cont (g)	691.3	717.6	727.7	712.5		
Container (g)	86.5	94.6	87.1	86.5		
Soil+Rings (g)	604.8	623.0	640.6	626.0		
Wt. of Rings (g)	138.7	138.7	138.7	138.7		
Wt. of Soil (g)	466.1	484.3	501.9	487.3		
Dry Soil+Cont (g)	490.1	512.5	546.4	529.6		
Wt. of Soil (lb)	1.0276	1.0677	1.1065	1.0743		
Vol. of Rings ($ft.^3$)	0.00850	0.00850	0.00850	0.00850		
Wet Density (pcf)	120.9	125.6	130.2	126.4		
Wet Wt. (g)	466.1	484.3	501.9	487.3		
Dry Wt. (g)	403.6	417.9	459.3	443.1		
% Moisture	**15.5**	**15.9**	**9.3**	**10.0**		
Dry Density (pcf)	**104.7**	**108.4**	**119.1**	**114.9**		

Tested By: Miguel Z.T. Reviewed By:
Josh Myers, Laboratory Manager

1645 Pacific Avenue, Suite 107 | Oxnard, CA 93033 | Ph (805) 486-6475 | Fax (805) 486-9016
Inspection | Testing | Geotechnical | Construction Engineering | Civil Engineering | Surveying

TRANSMITTAL LETTER

DATE: January 10, 2020

ATTENTION: **Josh Myers**

TO: CTE South - Oxnard
1645 Pacific Ave., #107
Oxnard, CA 93033

SUBJECT: Laboratory Test Data
Cornerstone Church
Your #40-3817G, HDR Lab #20-0002LAB

COMMENTS: Enclosed are the results for the subject project.

James T. Keegan, MD
Corrosion and Lab Services Section Manager

431 West Baseline Road · Claremont, CA 91711
Phone: 909.962.5485 · Fax: 909.626.3316

Table 1 - Laboratory Tests on Soil Samples

CTE South - Oxnard
Cornerstone Church
Your #40-3817G, HDR Lab #20-0002LAB
10-Jan-20

Sample ID			Boring 2 @ 0-5'
Resistivity		**Units**	
as-received		ohm-cm	13,200
saturated		ohm-cm	3,440
pH			7.5
Electrical Conductivity		mS/cm	0.09
Chemical Analyses			
Cations			
calcium	Ca^{2+}	mg/kg	47
magnesium	Mg^{2+}	mg/kg	8.0
sodium	Na^{1+}	mg/kg	39
potassium	K^{1+}	mg/kg	67
Anions			
carbonate	CO_3^{2-}	mg/kg	ND
bicarbonate	HCO_3^{1-}	mg/kg	171
fluoride	F^{1-}	mg/kg	46
chloride	Cl^{1-}	mg/kg	3.0
sulfate	SO_4^{2-}	mg/kg	57
phosphate	PO_4^{3-}	mg/kg	52
Other Tests			
ammonium	NH_4^{1+}	mg/kg	1.8
nitrate	NO_3^{1-}	mg/kg	9.3
sulfide	S^{2-}	qual	na
Redox		mV	na

Resistivity per ASTM G187, Cations per ASTM D6919, Anions per ASTM D4327, and Alkalinity per APHA 2320-B.
Electrical conductivity in millisiemens/cm and chemical analyses were made on a 1:5 soil-to-water extract.
mg/kg = milligrams per kilogram (parts per million) of dry soil.
Redox = oxidation-reduction potential in millivolts
ND = not detected
na = not analyzed

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APPENDIX D

STANDARD SPECIFICATIONS FOR GRADING AND TRENCHING

RECOMMENDED EARTHWORK SPECIFICATIONS

The following specifications are recommended to provide a basis for quality control during the placement of compacted fill or backfill as applicable.

1. Areas that are to receive compacted fill shall be observed by Soil/Geotechnical Engineer (GE) or his/her representative prior to the placement of fill.

2. All drainage devices shall be properly installed and observed by GE and/or owner's representative(s) prior to placement of backfill.

3. Fill soils shall consist of imported soils or on-site soils free of organics, cobbles, and deleterious material provided each material is approved by GE. GE shall evaluate and/or test the import material for its conformance with the report recommendations prior to its delivery to the site. The contractor shall notify GE 72 hours prior to importing material to the site

4. Fill shall be placed in controlled layers (lifts), the thickness of which is compatible with the type of compaction equipment used. The fill materials shall be brought to optimum moisture content or above, thoroughly mixed during spreading to obtain a near uniform moisture condition and uniform blend of materials, and then placed in layers with a thickness (loose) not exceeding 8 inches. Each layer shall be compacted to a minimum compaction of 90% relative to the maximum dry density determined per the latest ASTM D1557 test. Density testing shall be performed by GE to verify relative compaction. The contractor shall provide proper access and level areas for testing.

5. Rocks or rock fragments less than eight (8) inches in the largest dimension may be utilized in the fill, provided they are not placed in concentrated pockets, except rocks larger than four (4) inches shall not be placed within three (3) feet of finish grade.

6. Rocks greater than eight (8) inches in largest dimension shall be taken offsite, or placed in accordance with the recommendation of the Soils Engineer in areas designated as suitable for rock disposal.

7. Where space limitations do not allow for conventional fill compaction operations, special backfill materials and procedures may be required. Pea gravel or other select fill can be used in areas of limited space. A sand and Portland cement slurry (2 sacks per cubic-yard mix) shall be used in limited space areas for shallow backfill near final pad grade, and pea gravel shall be placed in deeper backfill near drainage systems.

8. GE shall observe the placement of fill and conduct in-place field density tests on the compacted fill to check for adequate moisture content and the required relative compaction. Where less than specified relative compaction is indicated, additional compacting effort shall be applied and the soil moisture conditioned as necessary until adequate relative compaction is attained.

9. The Contractor shall comply with the minimum relative compaction out to the finish slope face of fill slopes, buttresses, and stabilization fills as set forth in the specifications for compacted fill. This may be achieved by either overbuilding the slope and cutting back as necessary, or by direct compaction of the slope face with suitable equipment, or by any other procedure that produces the required result.

10. Any abandoned underground structures such as cesspools, cisterns, mining shafts, tunnels, septic tanks, wells, pipelines or others not discovered prior to grading are to be removed or treated to the satisfaction of the Soils Engineer and/or the controlling agency for the project.

11. The Contractor shall have suitable and sufficient equipment during a particular operation to handle the volume of fill being placed. When necessary, fill placement equipment shall be shut down temporarily in order to permit proper compaction of fills, correction of deficient areas, or to facilitate required field-testing.

12. The Contractor shall be responsible for the satisfactory completion of all earthwork in accordance with the project plans and specifications.

13. Final reports shall be submitted after completion of earthwork and after the Soils Engineer and Engineering Geologist have finished their observations of the work. No additional excavation or filling shall be performed without prior notification to the Soils Engineer and/or Engineering Geologist.

14. Whenever the words “supervision”, “inspection” or “control” are used, they shall mean <u>observation</u> of the work and/or testing of the compacted fill by GE to assess whether substantial compliance with plans, specifications and design concepts has been achieved, and does not include direction of the actual work of the contractor or the contractor’s workmen.

RECOMMENDED SPECIFICATIONS FOR PLACEMENT OF TRENCH BACKFILL

1. Trench excavations to receive backfill shall be free of trash, debris or other unsatisfactory materials prior to backfill placement, and shall be observed by project soil/geotechnical engineer (GE) representative.

2. Except as stipulated herein, soils obtained from the excavation may be used as backfill if they are essentially free of organics and deleterious materials.

3. Rocks generated from the trench excavation not exceeding three (3) inches in largest dimension may be used as backfill material. However, such material may not be placed within 12 inches of the top of the pipeline. No more than 30 percent of the backfill volume shall contain particles larger than 1-½ inches in diameter, and rocks shall be well mixed with finer soil.

4. Soils (other than aggregates) with a Sand Equivalent (SE) greater than or equal to 30, as determined by ASTM D 2419 Standard Test Method or at the discretion of the engineer or representative in the field, may be used for bedding and shading material in the pipe zone areas.

5. No jetting will be permitted. Trench backfill other than bedding and shading shall be compacted by mechanical methods as tamping sheepsfoot, vibrating or pneumatic rollers or other mechanical tampers to achieve the density specified herein. The backfill materials shall be brought to optimum moisture content or above, thoroughly mixed during spreading to obtain a near uniform moisture condition and uniform blend of materials, and then placed in horizontal layers with a thickness (loose) not exceeding 8 inches. Trench backfills shall be compacted to a minimum compaction of 90 percent relative to the maximum dry density determined per the latest ASTM D1557 test.

6. The contractor shall select the equipment and process to be used to achieve the specified density without damage to the pipeline, the adjacent ground, existing improvements or completed work.

7. Observations and field tests shall be carried on during construction by GE to confirm that the required degree of compaction has been obtained. Where compaction is less than that specified, additional compaction effort shall be made with adjustment of the moisture content as necessary until the specified compaction is obtained. Field density tests may be omitted at the discretion of the engineer or his representative in the field.

8. Whenever, in the opinion of GE or the Owner's Representative(s), an unstable condition is being created, either by cutting or filling, the work shall not proceed until an investigation has been made and the excavation plan revised, if deemed necessary.

9. Fill material 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 GE indicate the moisture content and density of the fill are as specified.

10. Whenever the words "supervision", "inspection", or "control" are used, they shall mean <u>observation</u> of the work and/or testing of the compacted fill by GE to assess whether substantial compliance with plans, specifications and design concepts has been achieved.