GEOTECHNICAL ENGINEERING REPORT

FOR

CONEJO VALLEY CHURCH OF CHRIST

2525 EAST HILLCREST DRIVE

THOUSAND OAKS, CALIFORNIA

PROJECT NO.: 301911-001

AUGUST 20, 2018

PREPARED FOR

RJR ENGINEERING

ATTENTION: ROBERT W. ANDERSON

BY

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August 20, 2018

Project No.: 301911-001
Report No.: 18-8-32

RJR Engineering
Attention: Robert W. Anderson
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Ventura, CA 93003

Project: Conejo Valley Church of Christ
2525 East Hillcrest Drive
Thousand Oaks, California
Subject: Geotechnical Engineering Report

As authorized, Earth Systems Pacific (Earth Systems) has performed a geotechnical study for proposed construction at the existing "Conejo Valley Church of Christ" facility at 2525 East Hillcrest Drive in Thousand Oaks, California. The accompanying Geotechnical Engineering Report presents the results of our subsurface exploration and laboratory testing programs, and our conclusions and recommendations pertaining to geotechnical aspects of project design. This report completes Phase 1 of the scope of services described within our Proposal VEN-18-01-004 dated January 11, 2018, revised February 8, 2018, and authorized by Robert W. Anderson on March 19, 2018.

We have appreciated the opportunity to be of service to you on this project. Please call if you have any questions, or if we can be of further service.

Respectfully submitted,
EARTH SYSTEMS

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Copies: 4 - Client (3 hardcopies, 1 email)
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INTRODUCTION

Project Description

This report presents results of a Geotechnical Engineering study performed for proposed construction at the existing “Conejo Valley Church of Christ” facility at 2525 East Hillcrest Drive in Thousand Oaks, California (see Vicinity Map in Appendix A).

It is our understanding that the proposed construction will include: 1) renovation of the existing Church Building; 2) a new one-story office building and a two-story classroom building; 3) a new access driveway, parking lot, and revisions to an existing driveway; 4) new conventional retaining walls and new geogrid-reinforced retaining walls along the new/revised driveways; 5) new areas for bioretention/permeable pavers; and 6) a new debris basin with a new storm drain system.

Structural considerations for building column loads of up to 25 kips with maximum wall loads of 2 kips per lineal foot were used as a basis for the recommendations of this report. If actual loads vary significantly from these assumed loads, Earth Systems should be notified since reevaluation of the recommendations contained in this report may be required.

Purpose and Scope of Work

The purpose of the geotechnical study that led to this report was to analyze the soil/bedrock conditions of the project site and to provide geotechnical recommendations for construction. The soil conditions include surface and subsurface soil types, expansion potential, soil strength, settlement potential, bearing capacity, and the presence or absence of subsurface water. The scope of work included:

- Performing a reconnaissance of the project site.
- Drilling/excavating, sampling, and logging 9 hollow-stem-auger borings and 9 test pits to study bedrock, soil, and groundwater conditions. Four of the borings drilled (B-1 to B-4) were used for infiltration testing.
- Laboratory testing soil samples obtained from the subsurface exploration to determine their physical and engineering properties.
- Performing infiltration tests.
- Consulting with owner representatives and design professionals.

- Analyzing the geotechnical data obtained.
- Preparing this report.

Contained in this report are:

- Descriptions and results of field and laboratory tests that were performed.
- Conclusions and recommendations pertaining to site grading and structural design.

Site Setting

The project site is a relatively flat pad with a slight drainage to the southwest. An existing church building currently occupies the project site. The project site is bounded by East Hillcrest Drive to the southwest, residential properties to the northwest and southeast, and Hillcrest (ascending slopes) to the northeast. These ascending slopes are up to about 130 feet high, and the project site appears to be located on an alluvium filled canyon/valley. The geographic coordinates of the project site are 34.1762˚ North Latitude and 118.8378° West Longitude. The area surrounding the existing church building is covered by landscaping (planters, lawns, and trees) and hardscaping (walkways, driveways, and parking spaces). The site slopes gently upward from Hillcrest.

REGIONAL GEOLOGY

The project site is within the Conejo Valley area of the Santa Monica Mountains, which in turn lie within the western Transverse Ranges geomorphic province. The Santa Monica Mountains and the Transverse Ranges are characterized by ongoing tectonic activity. In the vicinity of the subject site, Tertiary and Quaternary sediments and volcanic rocks have been folded and faulted along predominant east-west structural trends.

The ongoing regional compression has locally resulted in the southwest-northeast trending Simi-Santa Rosa fault, which is located approximately 6 miles north of the project site (see Appendix A for Regional Geologic Map by T.W. Dibblee, Jr., Geologic Map of the Thousand Oaks Quadrangle, 1992). The project site does not lie within any study zones for liquefaction or earthquake induced landslides (See Seismic Hazard Zones Map in Appendix A). No faults were encountered during field studies

The project site is mapped by T.W. Dibblee, Jr. as underlain by alluvium, Detrital Sedmients of Lindero Canyon, and Monterey Formation bedrock, which were encountered during our field exploration.

SEISMICITY AND SEISMIC DESIGN

Although the project site is not within a State-designated "fault rupture hazard zone", it is located in an active seismic region where large numbers of earthquakes are recorded each year. Historically, major earthquakes felt in the vicinity of the project site have originated from faults near the area. These include the 1857 Fort Tejon earthquake, the 1872 Owens Valley earthquake, and the 1952 Arvin-Tehachapi earthquake. An exception is the December 21, 1812 "Santa Barbara Region" earthquake, that was presumably centered in the Santa Barbara Channel.

It is assumed that the 2016 CBC and ASCE 7-10 guidelines will apply for the seismic design parameters. The 2016 CBC includes several seismic design parameters that are influenced by the geographic site location with respect to active and potentially active faults, and with respect to subsurface soil or rock conditions. The seismic design parameters presented herein were determined by the United States Seismic Design Maps "risk-targeted" calculator on the USGS website for the project site coordinates (34.1762˚ North Latitude and 118.8378˚ West Longitude). The calculator adjusts for Soil Site Class C, and for Occupancy (Risk) Category I/II/III. The calculated 2016 California Building Code (CBC) and ASCE 7-10 seismic parameters typically used for structural design are included in Appendix D and summarized in the following table.

Summary of Seismic Parameters (2016 CBC)

Seismic Design Category	D
Site Class (Table 20.3-1 of ASCE 7-10 with 2013 update)	C
Occupancy (Risk) Category	I/II/III
Maximum Considered Earthquake (MCE) Ground Motion	
Peak Modified Ground Acceleration – PGA_m	0.500 g
Spectral Response Acceleration, Short Period – S_s	1.500 g
Spectral Response Acceleration at 1 sec. – S_1	0.600 g
Site Coefficient – F_a	1.00
Site Coefficient – F_v	1.30

Site-Modified Spectral Response Acceleration, Short Period – S_{MS}	1.500 g
Site-Modified Spectral Response Acceleration at 1 sec. – S_{M1}	0.780 g
Design Earthquake Ground Motion	
Short Period Spectral Response – S_{DS}	1.000 g
One Second Spectral Response – S_{D1}	0.520 g

The values presented in the table above are appropriate for a 2 percent probability of exceedance in 50 years. A listing of the calculated 2016 CBC and ASCE 7-10 seismic parameters is included in Appendix D.

The Fault Parameters table in Appendix D lists the significant "active" and "potentially active" faults within a 29-mile (46-kilometer) radius of the project site. The distance between the project site and the nearest portion of each fault is shown, as well as the respective estimated maximum earthquake magnitudes, and the deterministic mean site peak ground accelerations.

SOIL/BEDROCK CONDITIONS

Based on our field exploration, the proposed office building area (Borings B-4 and B-5) is underlain directly by Detrital Sedmients of Lindero Canyon (gravel conglomerate); the proposed classroom area (Boring B-2) is blanketed by a layer of artificial fill (sandy clay with gravels, thickness of about 4 feet), which is underlain by alluvium (clayey and sandy silt with gravels, thickness of about 7.5 feet), which is underlain by Detrital Sedmients of Lindero Canyon. For the northeastern portion of the project site, artificial fill, alluvium/colluvium, Monterey Formation bedrock, and Detrital Sedmients of Lindero Canyon were encountered.

Testing indicates that anticipated bearing soils lie in the "Low to Medium" expansion range based on the measured expansion indices of 35 from Boring B-2, and 80 from Test Pit TP-5. It should be noted that the geotechnical recommendations provided in this report is based on the “Medium” expansion range. A locally adopted version of this classification of soil expansion, Table 1809.7(1), is included in Appendix C of this report.

Groundwater was not encountered within the maximum depth of exploration of 25.5 feet below ground surface. According to the Seismic Hazard Zones Report for the Thousand Oaks 7.5-Minute Quadrangle, Ventura and Los Angeles Counties, California (CGS, 2000), the depth of historical high groundwater is estimated to be deeper than 10 feet. It should be noted that fluctuations in groundwater levels may occur because of variations in rainfall, regional climate, and other factors.

Samples of near-surface soils were tested for pH, resistivity, soluble sulfates, and soluble chlorides. The test results provided in Appendix B should be distributed to the design team for their interpretations pertaining to the corrosivity or reactivity of various construction materials (such as concrete and piping) with the soils. It should be noted that sulfate contents (520 mg/Kg from Boring B-2, and 7.7 mg/Kg from Test Pit TP-5) are in the "S0" exposure class (i.e. "Negligible" severity range) of Table 19.3.1.1 of ACI 318-14. Therefore, special concrete designs will not be necessary for the measured sulfate contents according to Table 19.3.2.1 of ACI 318-14.

Based on criteria established by the County of Los Angeles, the measured resistivity values of a near-surface soil sample (2,000 ohms-cm from Boring B-2, and 11,000 ohms-cm from Test Pit TP-5) indicate that near-surface soils are "Corrosive to Moderately Corrosive" to ferrous metal (i.e. cast iron, etc.) pipes. It should be noted that Earth Systems does not practice soil corrosion engineering.

HYDROCOLLAPSE POTENTIAL

Hydrocollapse is a phenomenon in which naturally occurring soil deposits, or non-engineered fill soils, collapse when wetted. Natural soils that are susceptible to this phenomenon are typically aeolian, debris flow, alluvial, or colluvial deposits with high apparent strength when dry. Loosely compacted fills can also be susceptible to this phenomenon. The dry strength is attributed to salts, clays, silts, and in some cases capillary tension, "bonding" larger soil grains together. So long as these soils remain dry, their strength and resistance to compression are retained. However, when wetted, the salt, clay, or silt bonding agent is weakened or dissolved, or capillary tension reduced, eventually leading to collapse. Soils susceptible to this phenomenon are found throughout the southwestern United States.

Regarding the proposed classroom building, Earth Systems considers the consolidation test from Boring B-2 at 5 feet deep to be representative. A hydrocollapse of 0.5%, when applied to the artificial fill and alluvium overlying the Detrital Sedmients of Lindero Canyon, yielded an hydrocollapse potential estimation of three-quarters of an inch (0.75”).

Hydrocollapse is not anticipated for the proposed office building because that area is underlain directly by firm Detrital Sedmients of Lindero Canyon.

Tests on samples form the proposed new/revised driveways area yielded a hydrocollapse of 2.5% (TP-8@4'), and a hydro expansion of 1.5% (TP-1@7').

LIQUEFACTION POTENTIAL

Earthquake-induced cyclic loading can be the cause of several significant phenomena, including liquefaction in fine sands and silty sands. Liquefaction results in a loss of soil strength and can cause structures to settle and, in extreme cases, to experience bearing failure.

The potential hazard posed by liquefaction is considered to be low at the project site because the project site does not lie within a potentially liquefiable zone (see Seismic Hazard Zones Map in Appendix A).

SEISMIC-INDUCED SETTLEMENT OF DRY SANDS

Dry (unsaturated) soils tend to settle and densify when subjected to earthquake shaking. The amount of settlement is a function of relative density, cyclic shear strain magnitude, and the number of strain cycles. A procedure to evaluate this type of settlement was developed by Seed and Silver (1972) and later modified by Pyke, et al. (1975). Tokimatsu and Seed (1987) presented a simplified procedure that has been reduced to a series of equations by Pradel (1998). Research on this subject is continuing (Stewart, et al., 2004).

The potential of this phenomenon is considered to be low at the project site because the calculated dry sand settlement values are negligible using blow counts from both Borings B-1 and B-2. See Appendix E for Dry Sand Settlement Calculation Results.

STATIC SETTLEMENT

Earth Systems analyzed the potential static settlement because of structural loads based on the consolidation data. The methodology/parameters used in our analyses were summarized in the table on next page:

Methodology/Parameters Used in the Static Settlement Analyses

Influence Factor Methodology	Westergaard – footing center
Soil Total Unit Weight	85 pcf
Column Load	25 kips
Footing Size	4 feet by 4 feet
Footing Shape	Square
Footing Embedment	1.75 feet below ground surface

A static settlement of about 2.5 inches is estimated. However, the static settlement can be reduced to about 1.5 inches if overexcavation and recompaction are performed to 5 feet below the bottom of footings; or reduced to about 1 inch if overexcavation and recompaction are performed to 10 feet below the bottom of footings; or reduced to a negligible level if all the artificial and alluvium are overexcavated and recompacted.

FAULT RUPTURE HAZARD

A fault is a break in the earth's crust upon which movement has occurred in the recent geologic past and future movement is expected. A summary of nearby active faults is presented in Appendix D under Table 1 Fault Parameters.

The project site does not lie within a State of California designated active fault hazard zone. The activity of faults is classified by the State of California based on the Alquist-Priolo Earthquake Fault Zoning Act (1972). An active fault has had surface rupture with Holocene time (the past 11,000 years). A potentially active fault shows evidence of surface displacement during Quaternary time (last 1.6 million years). An inactive fault has no evidence of movement within the Quaternary time.

As previously discussed, the Simi-Santa Rosa fault is located approximately 6 miles north of the project site (see Appendix A for Regional Geologic Map by T.W. Dibblee, Jr., Geologic Map of the Thousand Oaks Quadrangle, 1992), the potential for fault rupture at the project site is considered low.

LANDSLIDES

Landsliding is a process where a distinct mass of rock or soil moves downslope because of gravity. No landslides are mapped on the project site by Dibblee (see Regional Geologic Map in Appendix A). Because there are no identified landslides either on or trending into the project site, hazards associated with this phenomenon are considered low.

ROCKFALL

Loose boulder-sized rocks and/or weathering bedrock outcrops located upslope from construction can lead to a rockfall hazard. Because no loose boulder-sized rock or weathering bedrock outcrops were present near the uphill side of the project site, the potential for rockfall onto the project site appears to be low.

EARTHQUAKE-INDUCED FLOODING

Earthquake-induced flooding types include tsunamis, seiches, and reservoir failure. Because of the inland location of the project site, hazards from tsunamis and seiches are considered unlikely. Additionally, there are no reservoirs upstream of the project site. Therefore, earthquake-induced flooding is not considered a potential hazard at the project site.

OTHER FLOODING

The project site is not within any of the flood hazard areas mapped by Federal Emergency Management Agency (FEMA), FEMA Flood Map for City of Thousand Oaks, effective January 20, 2010, Map No. 06111C0990E.

INFILTRATION TESTING

On May 8, 2018, four of the nine borings (B-4/IT-1, B-5/IT-2, B-8/IT-3, and B-9/IT-4) were drilled to depths of about 13, 2, 13, and 2 feet, respectively, below the ground surface to determine the soil profile and allow installation of plastic casing for infiltration testing (see Site Plan in Appendix A for infiltration boring locations). All infiltration borings were bottomed into native Detrital Sediments of Lindero Canyon (see Logs of Borings in Appendix A).

After drilling was completed, 2-inch diameter slotted PVC casings were lowered into the boreholes. The annuli between the casings and boring walls were then filled with pea gravel. On June 14, 2018, the infiltration tests were run according to a procedure consistent with the Ventura County Technical Guidance Manual for Stormwater Quality Control Measures. The falling-head borehole infiltration test procedure was used for infiltration testing. About 2 feet of water was added to the bottom of each of the holes to start the tests and the drop in the water surface monitored by taking periodic measurements. Multiple readings were taken at certain frequency within the test interval depending on the infiltration rate. After each of these intervals, water was added to return the original refill depth above the hole bottom for the next test interval. The tests were run until the infiltration rate became reasonably stable, see Infiltration Testing Results in Appendix F. The results in Appendix F include conversion of the data from percolation data measured in the field, to infiltration data, using the reduction equation provided in the Los Angeles County stormwater manual. The reduction equation is a modification of the Porchet equation. The equation is as follows:

Infiltration Rate = Percolation Rate/Reduction Factor

Reduction Factor = $[(2d_1 - \Delta d) / \text{Diameter}] + 1$, where:
d_1 = Initial Water Height above Bottom (in inches),
Δd = Water Drop of Final Reading (in inches), and
Diameter = Diameter of Boring (in inches).

Based on the testing, the test infiltration rates for the depths tested and boring locations are:

Boring	Boring Depth (feet)	Infiltration Rate (inch/hour)	Infiltration Rate (cm/s)
B-4/IT-1	13	0.8	0.0006
B-5/IT-2	2	2.5	0.0018
B-8/IT-3	13	0.0	0.0000
B-9/IT-4	2	1.9	0.0013

There are many factors that influence the infiltration rate. Clear water was used in our tests, whereas deleterious material will likely be contained in the storm water. Variations in soil conditions within the limits of the proposed infiltration system will likely affect infiltration characteristics. The designer who utilizes the infiltration results should consider these factors, as well as apply a factor-of-safety to the infiltration rate to account for future disposal bed siltation.

CONCLUSIONS AND RECOMMENDATIONS

Based on the data provided in this report, it appears that the project site is suitable for the proposed construction from a geotechnical engineering standpoint provided that the recommendations provided herein are properly implemented into the project.

Earth Systems recommends a conventional footing system with slab-on-grade floors be used to support the proposed office and classroom buildings. Given the site conditions encountered, we conclude that remedial grading [removing all artificial and alluvium, until firm native bedrock (Monterey Formation or Detrital Sedmients of Lindero Canyon) is encountered] will be needed to mitigate the negative effects of a potential settlement (static and seismic combined) of up to about 3.5 inches. When the recommended grading is successfully completed, the total potential settlement (static and seismic combined) should be reduced to about 0.5 inch and potential differential settlement to about 0.25 inch. The Project Structural Engineer will need to design the foundation system to accommodate the potential settlement values.

However, if the proposed overexcavation is not preferred, drilled piers could also be considered. Although drilled pier analysis is not included in the scope of work of this project, Earth Systems could provide further service if needed.

Specific conclusions and recommendations addressing these geotechnical considerations, as well as general recommendations regarding the geotechnical aspects of design and construction, are presented in the following sections.

A. Grading

1. Pre-Grading Considerations

   a. Roof draining systems, if required by the appropriate jurisdictional agency, should be designed so that water is not discharged into bearing soils or near structures.

   b. Final site grade should be designed so that all water is diverted away from the structures over paved surfaces, or over landscaped surfaces in accordance with current codes. Water should not be allowed to pond anywhere on the pad.

   c. Shrinkage of soils (uncertified fills) affected by compaction is estimated to be about 15 percent based on an anticipated average compaction of 92 percent. This does not include losses from removing oversized rocks.

d. Earth Systems should be retained to provide geotechnical engineering services during site development and grading, and foundation construction phases of the work to observe compliance with the design concepts, specifications and recommendations. This will allow for timely design changes in the event that subsurface conditions differ from those anticipated prior to the start of construction.
e. Plans and specifications should be provided to Earth Systems prior to grading. Plans should include the grading plans, foundation plans, and foundation details. Earth Systems will review these plans only for conformity with geotechnical parameters not including drainage. It is the responsibility of the Client and other Engineers to review and approve designs and plans for conformity with all engineering and design requirements necessary to the proper function and performance of the structure.
f. Compaction tests should be made to determine the relative compaction of the fills in accordance with the following minimum guidelines: two tests for each 1.5-foot vertical lift in every isolated area graded; one test for each 1,000 cubic yards of material placed; and two tests in each building pad; and two tests at finished subgrade elevation in the areas of remedial grading.

2. Rough Grading/Areas of Development
   a. Grading at a minimum should conform to the 2016 California Building Code.
   b. The existing ground surface should be initially prepared for grading by removing all unwanted existing features, vegetation, trees, large roots, debris, other organic material and non-complying fill. Organics and debris should be stockpiled away from areas to be graded, and ultimately removed from the project site to prevent their inclusion in fills. Voids created by removal of such material should be properly backfilled and compacted. No compacted fill should be placed unless the underlying soil has been observed by the Geotechnical Engineer.
   c. Soils should be overexcavated to the greatest depth of the following: 1) 2 feet below the bottom of footings; 2) 4 feet below the finished pad grade throughout the entire construction areas; or 3) removing any loose soil (artificial fill and/or alluvium) until either firm native bedrock (Monterey Formation or Detrital Sedmients of Lindero Canyon) is encountered. Overexcavation should be extended to a distance of at least 5 feet laterally,

but not less than a distance equal to the depth of removal, beyond the outside edge of the foundation elements. Lateral overexcavation may not be possible where the proposed renovation abut the existing church building.

d. Where the proposed renovation will abut the existing church building, the remedial excavations should be completed by the A-B-C slot cut method because of limited space. The width of any slot cut should not exceed 8 feet. The complete construction and backfill of the 'A' slot must be performed prior to excavation of the 'B' slot, and likewise the 'C' slot.

e. The bottoms of all excavations should be observed by a representative of Earth Systems prior to processing or placing fill.

f. The resulting surface(s) should then be scarified an additional 6 inches, uniformly moisture conditioned to about 3 percent over the optimum moisture content, and compacted to achieve a minimum relative compaction of 90 percent of the ASTM D 1557 maximum dry density. Compaction of the prepared subgrade should be verified by testing prior to the placement of engineered fill.

g. Areas outside of the proposed office and classroom buildings to receive fill, exterior slabs-on-grade, sidewalks, or paving should be overexcavated either throughout the existing loose soil (artificial fill and alluvium) or to a minimum of 1 foot below finished pad grade, whichever is deeper. The resulting surfaces should then be scarified an additional 6 inches, moisture conditioned, and recompacted. If the owner decides to leave any loose soil (artificial fill and alluvium) in place under and/or within the influence of proposed exterior improvements, then the owner should aware that there is a risk of settlement that may cause displacement and cracking of exterior improvements.

h. Voids created by dislodging cobbles during scarification should be backfilled and recompacted and the dislodged cobbles larger than 6 inches in diameter should be removed from the subgrade.

i. On-site soils may be used for fill once they are cleaned of all organic material, rocks, debris, and irreducible material larger than 6 inches.

j. Fill and backfill placed above optimum moisture in layers with a loose thickness not greater than 8 inches should be compacted to a minimum of 90 percent of the maximum dry density obtainable by the ASTM D 1557 test method unless otherwise recommended or specified. Random compaction tests by Earth Systems can assist the Grading Contractor in evaluating whether the Grading Contractor is meeting compaction requirements. However, compaction tests pertain only to a specific location and do not guaranty that all fill has been compacted to the prescribed percentage of maximum density. It is the ultimate responsibility of the Grading Contractor to achieve uniform compaction in accordance with the requirements of this report and the grading ordinance.

k. It should be noted that a overexcavation may be somewhat difficult because of the cobbles and bedrock materials. Appropriate measures should be taken prior to grading to prepare for mitigation of this problem.

l. Import soils used (if any) to raise site grade should be equal to, or better than, on-site soils in strength, expansion, and compressibility characteristics. Import soil can be evaluated, but will not be prequalified by the Geotechnical Engineer. Final comments on the characteristics of the import will be given after the material is at the project site.

m. In landscape areas adjacent to the building, the 2016 CBC (Section 1803.3) requires a minimum gradient of 5% away from the edge of the building foundation for a minimum distance of 10 feet.

n. Periodic wetting of the soils after grading would be beneficial in regard to presaturation.

3. <u>Utility Trenches</u>

a. Utility trench backfill should be governed by the provisions of this report relating to minimum compaction standards. In general, on-site service lines may be backfilled with native soils compacted to 90 percent of maximum density. Backfill of offsite service lines will be subject to the specifications of the jurisdictional agency or this report, whichever are greater.

b. Compacted on-site native soils should be utilized for backfill below structures. Clean sand backfill should be avoided under structures because it provides a conduit for water to migrate under foundations.

c. Backfill operations should be observed and tested by the Geotechnical Engineer to monitor compliance with these recommendations.

d. Some difficulties may be encountered when excavating trenches because of cobbles and bedrock.

e. Rocks greater than 6 inches in diameter should not be placed in trench zones (from 12 inches below pavement subgrade or ground surface to 12 inches above top of pipe or box); rocks greater than 2.5 inches in diameter should not be placed in pipe zones (from 12 inches above top of pipe or box to 6 inches below bottom of pipe or box exterior).

f. Jetting should not be utilized for compaction in utility trenches.

B. Structural Design

1. Footings

a. Conventional continuous footings and/or interior pad footings can be used to support structures. It should be noted that if pad footings are to be used, they must be tied together by grade beams (each way) or by slabs. Based on the tested expansion range of “Medium”, perimeter continuous and/or pad footings should have a minimum depth of 21 inches; and interior pad footings should have a minimum embedment depth of 18 inches for the porposed two-story classroom building, and 12 inches for the porposed one-story office building. The expansion index should be re-evaluated at the completion of rough grading.

b. Footings should bear into firm recompacted fill as recommended elsewhere in this report. Foundation excavations should be observed by a representative of this firm after excavation, but prior to placing of reinforcing steel or concrete, to verify bearing conditions.

c. Conventional continuous and isolated pad footings may be designed based on an allowable bearing value of 1,700 psf. This value includes a safety factor of 3. This allowable bearing value is net (weight of footing and soil surcharge may be neglected) and is applicable for dead plus reasonable live loads.

d. Bearing values may be increased by one-third when transient loads such as wind and/or seismicity are included.

e. Lateral loads may be resisted by soil friction on floor slabs and foundations and by passive resistance of the soils acting on foundation stem walls. Lateral capacity is based on the assumption that any required backfill adjacent to foundations and grade beams is properly compacted.

f. The information that follows regarding reinforcement and premoistening for footings is the same as that given in the Table 1809.7(1) for the "Medium" expansion range. Actual footing designs should be provided by the project Structural Engineer, but the dimensions and reinforcement he recommends should not be less than the criteria set forth in the Table 1809.7(1) for the appropriate expansion range.

g. Continuous footings bottomed in soils in the "Medium" expansion range should be reinforced, at a minimum, with one No. 4 bar along the bottom and one No. 4 bar along the top. In addition, bent No. 3 bars on 24-inch centers should extend from within the footings to a minimum of 3 feet into adjacent slabs.

h. Bearing soils in the "Medium" expansion range should be premoistened to 3 percent over optimum moisture content to a depth of 18 inches below lowest adjacent grade. Premoistening should be confirmed by testing.

2. Slabs-on-Grade

a. Concrete slabs on grade should be supported by firm recompacted fills as recommended elsewhere in this report. Because the soils of the project site are in the “Medium” expansion range, it should be anticipated that exterior concrete supported on grade will be susceptible to movement with seasonal change in soil moisture content. The following recommendations for concrete slabs on grade can help mitigate, but not eliminate, such movement.

b. It is recommended that perimeter slabs (walkways, patios, etc.) be designed relatively independent of footing stems (i.e. free floating) so foundation adjustment will be less likely to cause cracking. Because the on-site soils are expansive, the exterior concrete slabs on grade should have turned-down edges of at least 8 inches into the soil.

c. The information that follows regarding design criteria for slabs is generally the same as that given in the Table 1809.7(1) for the "Medium" expansion range. Actual slab designs should be provided by the project Structural Engineer, but the reinforcement and slab thicknesses he recommends should not be less than the criteria set forth in the Table 1809.7(1) for the appropriate expansion range, or as recommended below, whichever is more stringent.

d. Slabs bottomed on soils in the "Medium" expansion range should be underlaid with a minimum of 4 inches of sand. Areas where floor wetness would be undesirable should be underlaid with a vapor retarder (as specified by the Project Architect or Civil Engineer) to reduce moisture transmission from the subgrade soils to the slab. The retarder should be placed as specified by the project Structural Engineer or Architect.

e. Slabs bottomed on soils in the "Medium" expansion range should at a minimum be reinforced at mid-slab with No. 3 bars on 24-inch centers, each way. No. 3 bars acting as dowels should also extend out of the perimeter footings, and should be bent so that they extend a minimum of 3 feet into adjacent slabs.

f. Soils underlying slabs that are in the "Medium" expansion range should be premoistened to 3 percent over optimum moisture content to a depth of 18 inches below lowest adjacent grade.

g. Premoistening of slab areas should be observed and tested by this firm for compliance with these recommendations prior to placing of sand, reinforcing steel, or concrete.

3. Frictional and Lateral Coefficients

a. Resistance to lateral loading may be provided by soil friction acting on the base of foundations. A coefficient of friction of 0.53 may be applied to dead load forces. This value does not include a safety factor.

b. Passive resistance acting on the sides of foundation stems equal to 275 pcf of equivalent fluid weight may be included for resistance to lateral load. This value does not include a safety factor.

c. A minimum safety factor of 1.5 should be used when designing for sliding or overturning.

d. Passive resistance may be combined with frictional resistance provided that a one-third reduction in the coefficient of friction is used.

4. Retaining Walls

a. Conventional cantilever retaining walls should not be backfilled with on-site soils because of the expansion potential of those soils. Walls that are backfilled at a 1:1 projection upward from the heels of the wall footings with crushed rock or non-expansive sand, may be designed for active pressures of 33 pcf of equivalent fluid weight for well-drained, level backfill. Active pressures developed from 41 pcf of equivalent fluid weight may be used for well-drained backfill sloping at 2 horizontal to 1 vertical. An 18-inch thick cap of compacted native soils should be placed above the rock or sand. Filter fabric should be placed between the rock or sand and native soils and/or backfill over the top.

b. The pressures listed above were based on the assumption that backfill soils will be compacted to 90 percent of maximum dry density as determined by the ASTM D 1557 Test Method.

c. Retaining walls may need to be designed for a seismic loading force that is applied in addition to the static forces when seismic shaking occurs. A seismic increment of earth pressure determined using 20 pcf of additional equivalent fluid weight needs to be considered for cantilever retaining walls that retain more than 6 feet of soil. This pressure has been determined by a procedure presented by Al Atik and Sitar (2010). The seismic increment of pressure can be assumed to be distributed so that the centroid of pressure acts at 0.33H above the base of a retaining wall, where H is the wall height in feet. Because this seismic force is transient, and in accordance with CBC Section 1807.2.3, a minimum safety factor of 1.1 may be used for sliding and overturning when seismic loads are included.

d. The lateral earth pressure to be resisted by the retaining walls or similar structures should also be increased to allow for any other applicable surcharge loads. The surcharges considered should include forces generated by any structures or temporary loads that would influence the wall design.

e. A system of backfill drainage should be incorporated into retaining wall designs. Backfill comprising the drainage system immediately behind retaining structures should be free-draining granular material with a filter fabric between it and the rest of the backfill soils. As an alternative, the backs of walls could be lined with geodrain systems. The backdrains should extend from the bottoms of the walls to about 18 inches from finished backfill grade. Waterproofing may aid in reducing the potential for efflorescence on the faces of retaining walls.

f. Compaction on the uphill sides of walls within a horizontal distance equal to one wall height should be performed by hand-operated or other lightweight compaction equipment. This is intended to reduce potential "locked-in" lateral pressures caused by compaction with heavy grading equipment.

g. Water should not be allowed to pond near the tops of retaining walls. To accomplish this, final backfill site grades should be such that all water is diverted away from retaining walls.

5. Settlement Considerations
    a. A maximum settlement (static and seismic combined) of about half of an inch (0.5”) is anticipated for foundations and floor slabs designed as recommended.
    b. Differential settlement between adjacent load bearing members could be about one-half the maximum settlement.
    c. The Project Structural Engineer will need to design the foundation system to accommodate the potential settlement values.

6. Preliminary Asphalt Paving Sections
    a. Earth Systems assumed an R-value of 10 based on the explored surficial soil types. This R-Value was used to determine a final paving section for the project site.
    b. Assuming a Traffic Index of 4.0 and using an R Value of 10, paving sections should have a total minimum gravel equivalent of 1.13 feet. This can be achieved by using 3 inches of asphaltic concrete on 5.5 inches of Class II Base compacted to a minimum of 95 percent of maximum dry density on subgrade soils compacted to a minimum of 90 percent of maximum dry density.
    c. Assuming a Traffic Index of 5.0 and using an R Value of 10, paving sections should have a total minimum gravel equivalent of 1.45 feet. This can be achieved by using 3 inches of asphaltic concrete on 9 inches of Class II Base compacted to a minimum of 95 percent of maximum dry density on subgrade soils compacted to a minimum of 90 percent of maximum dry density.
    d. Assuming a Traffic Index of 6.0 and using an R Value of 10, paving sections should have a total minimum gravel equivalent of 1.72 feet. This can be achieved by using 3 inches of asphaltic concrete on 12.5 inches of Class II Base compacted to a minimum of 95 percent of maximum dry density on subgrade soils compacted to a minimum of 90 percent of maximum dry density.
    e. The preliminary paving sections provided above have been designed for the type of traffic indicated. If the pavement is placed before construction on the project is complete, construction loads, which could increase the Traffic Indices above those assumed above, should be taken into account.
    f. Subgrade R-Value(s) should be reevaluated at or near the end of rough grading so that final pavement designs can be made.

ADDITIONAL SERVICES

This report is based on the assumption that an adequate program of monitoring and testing will be performed by Earth Systems during construction to check compliance with the recommendations given in this report. The recommended tests and observations include, but are not necessarily limited to the following:

- Review of the building and grading plans during the design phase of the project.
- Observation and testing during site preparation, grading, placing of engineered fill, and foundation construction.
- Consultation as required during construction.

LIMITATIONS AND UNIFORMITY OF CONDITIONS

The analyses and recommendations submitted in this report are based in part upon the data obtained from the on-site borings and test pits. The nature and extent of variations beyond the points of exploration may not become evident until construction. If variations then appear evident, it will be necessary to reevaluate the recommendations of this report.

The scope of services did not include any environmental assessment or investigation for the presence or absence of wetlands, hazardous or toxic materials in the soil, surface water, groundwater or air, on, below, or around this site. Any statements in this report or on the soil boring logs regarding odors noted, unusual or suspicious items or conditions observed, are strictly for the information of the client.

Findings of this report are valid as of this date; however, changes in conditions of a property can occur with passage of time whether they are because of natural processes or works of man on this or adjacent properties. In addition, changes in applicable or appropriate standards may occur whether they result from legislation or broadening of knowledge. Accordingly, 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 1 year.

In the event that any changes in the nature, design, or location of the structures and other improvements are planned, the conclusions and recommendations contained in this report should not be considered valid unless the changes are reviewed and conclusions of this report modified or verified in writing.

This report is issued with the understanding that it is the responsibility of the Owner, or of his representative to ensure that the information and recommendations contained herein are called to the attention of the Architect and Engineers for the project and incorporated into the plan and that the necessary steps are taken to see that the Contractor and Subcontractors carry out such recommendations in the field.

As the Geotechnical Engineers for this project, Earth Systems has striven to provide services in accordance with generally accepted geotechnical engineering practices in this community at this time. No warranty or guarantee is expressed or implied. This report was prepared for the exclusive use of the Client for the purposes stated in this document for the referenced project only. No third party may use or rely on this report without express written authorization from Earth Systems for such use or reliance.

It is recommended that Earth Systems be provided the opportunity for a general review of final design and specifications in order that earthwork and foundation recommendations may be properly interpreted and implemented in the design and specifications. If Earth Systems is not accorded the privilege of making this recommended review, it can assume no responsibility for misinterpretation of the recommendations contained herein.

BIBLIOGRAPHY

Al Atik, and N. Sitar, 2010, Seismic Earth Pressures on Cantilever Retaining Structures, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, October.

California Building Standards Commission, 2016, California Building Code, California Code of Regulations Title 24.

California Division of Mines and Geology (CDMG.), 1972 (Revised 1999), Fault Rupture Hazard Zones in California, Special Publication 42.

CDMG, 1998, Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada to Be Used with the 1997 Uniform Building Code.

California Geological Survey (CGS.), 2000a, Seismic Hazard Evaluation of the Thousand Oaks 7.5-Minute Quadrangle, Ventura and Los Angeles Counties, California, Open-File Report 2000-008.

CGS, 2000b, State of California Seismic Hazard Zones, Thousand Oaks Quadrangle, Official Map, November 17.

CGS, 2008, Guidelines for Evaluating and Mitigating Seismic Hazards in California, Special Publication 117A.

County of Los Angeles Department of Public Works, 2013, Manual for Preparation of Geotechnical Reports, July 1.

Dibblee, Jr., Thomas W., and Helmut E. Ehrenspeck, 1992, Geologic Map of the Thousand Oaks Quadrangle, Ventura and Los Angeles Counties, California, Dibblee Foundation Map No. DF-42.

El Ehwany, M. and Houston, S.L., 1990, Settlement and Moisture Movement in Collapsible Soils, Journal of Geotechnical Engineering, Vol. 116, October.

Jennings, C.W. and W.A. Bryant, 2010, Fault Activity Map of California, CGS Geologic Data Map No. 6.

Petersen, M.D., W.A. Bryant, C.H. Cramer, T. Cao, M.S. Reichle, A.D. Frankel, J.J. Lienkaemper, P.A. McCrory and D.P. Schwartz, 1996, Probabilistic Seismic Hazard Assessment for the State of California, CDMG. Open-File Report 96-08, USGS Open-File Report 96-706.

Petersen, M., D. Beeby, W. Bryant, C. Cao, C. Cramer, J. Davis, M. Reichle, G. Saucedo, S. Tan, G. Taylor, T. Toppozada, J. Treiman, C. Wills, 1999, Seismic Shaking Hazard Maps of California, CDMG. Map Sheet 48.

Pradel, D., 1998 Procedure to Evaluate Earthquake-Induced Settlements in Dry Sandy Soils, Journal of Geotechnical and Geoenvironmental Engineering, ASCE, Vol. 124, No. 4, April.

Pyke, R., Seed, H. B. And Chan, C. K., 1975, Settlement of Sands Under Multidirectional Shaking, ASCE, Journal of Geotechnical Engineering, Vol. 101, No. 4, April, 1975.

Seed, H. B., and Silver, M. L., 1972, Settlement of Dry Sands During Earthquakes, ASCE, Journal of Geotechnical Engineering, Vol. 98, No. 4.

Tokimatsu, Kohji and H. Bolton Seed, 1987, Evaluation of Settlements in Sands Due to Earthquake Shaking, Journal of Geotechnical Engineering, ASCE, August 1987, New York, New York.

APPENDIX A

Vicinity Map

Regional Geologic Map (Dibblee)

Seismic Hazard Zones Map

Historical High Groundwater Map

Site Plan

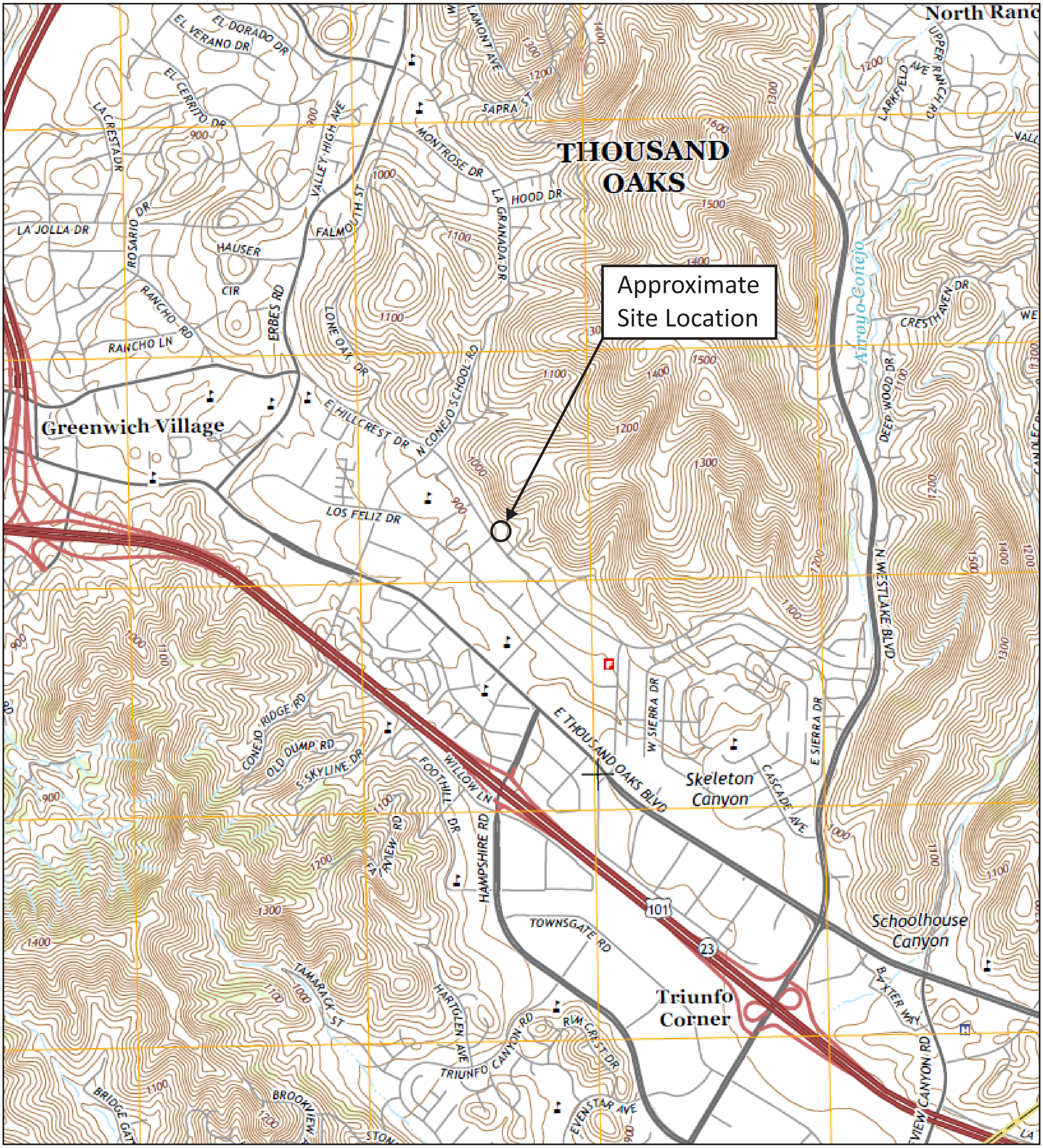
Field Study

Logs of Borings

Logs of Test Pits

Boring Log Symbols

Unified Soil Classification System



*Taken from USGS Topo Map, Thousand Oaks Quadrangle, California, 2015.

Approximate Scale: 1" = 2,000’

0 2,000’ 4,000’

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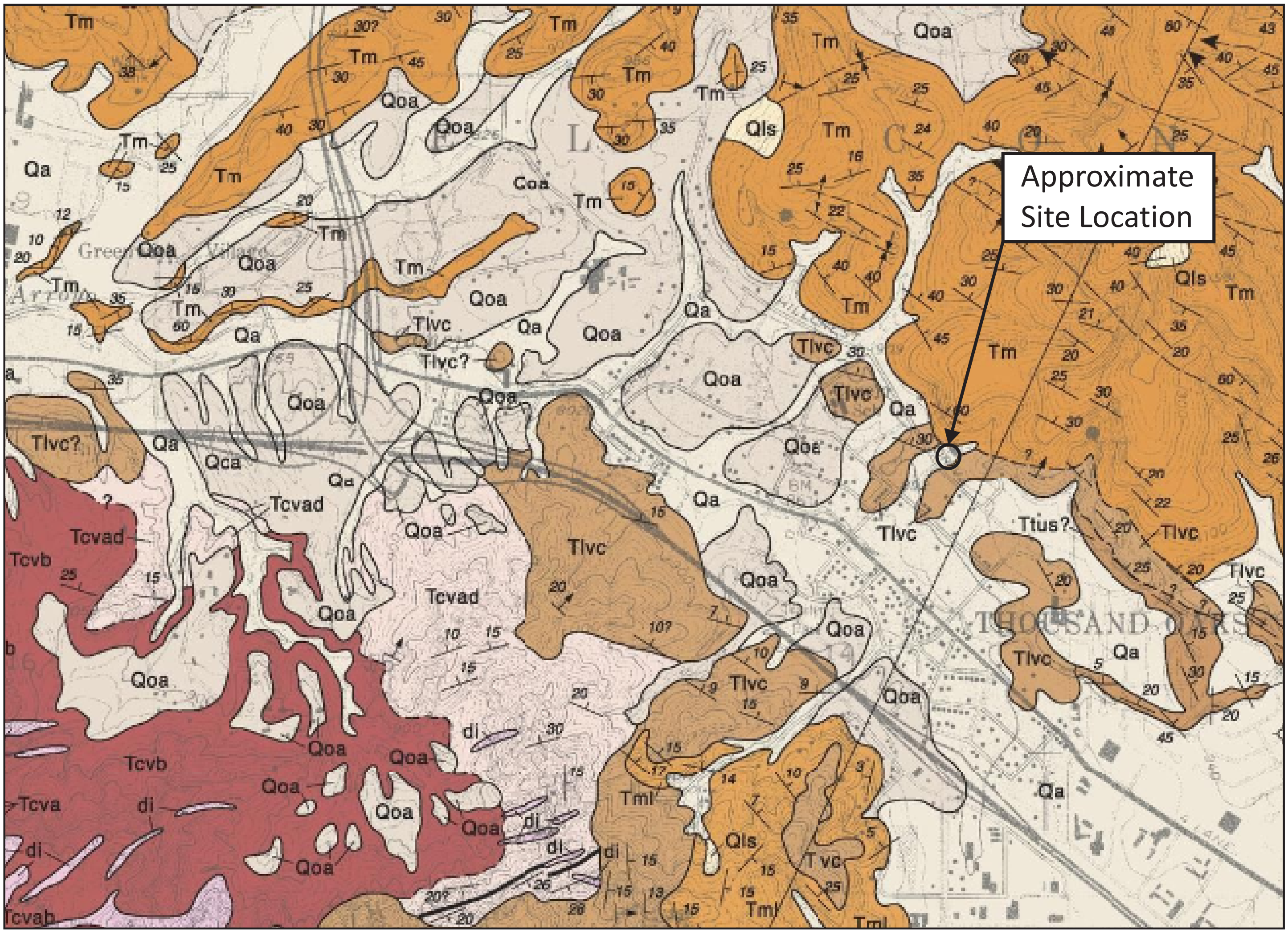
VICINITY MAP

Conejo Valley Church of Christ
2525 East Hillcrest Drive
Thousand Oaks, California

Earth Systems

August 2018	301913-001



*Taken from Dibblee, Jr., Geologic Map of The Thousand Oaks Quadrangle, Ventura and Los Angeles Counties, California, 1992, DF-42.

Qg / Qa

SURFICIAL SEDIMENTS

Unconsolidated alluvial deposits, generally undissected

Qg *Gravel, sand and silt of major stream channels*

Qa *Alluvium: silt, sand and gravel of valley and floodplain areas*

Tls / Tlvc / cv / Tlsc

DETRITAL SEDIMENTS OF LINDERO CANYON

***Typically exposed in upper Lindero and Skeleton Canyons (Dibblee 1989); assigned to upper Topanga Formation by Weber 1984, Calabasas Formation by Yerkes and Showalter 1991; in Simi Hills in part equivalent to upper Topanga Formation; cut by Calcareous veins* cv**

Marine and non-marine (?), clastic; late middle Miocene age

Tls *Sandstone: light gray to tan, soft, friable to semi-coherent, massive to vaguely bedded, locally calcareous, hard, rarely fossiliferous; east of upper Medea Creek locally includes conglomerate similar to* **Tlgc**; *in northwest area may be in part intertongued with unit* **Tlvc**; *in a few places contains shallow marine molluscan fossils*

Tlsc *Conglomerate of granitic detritus: light gray, composed of rounded pebbles, cobbles and small boulders of granitic and lesser metavolcanic rocks in incoherent sandstone matrix; massive to poorly bedded; in Simi Hills comprises facies of unit* **Tls**

Tlvc *Basal epiclastic (reworked) conglomerate of detritus derived from Conejo Volcanics: gray to rusty brown, massive to crudely layered; in southwestern area and Simi Hills composed of poorly sorted, subrounded clasts as large as small boulders, of mostly andesitic rocks in incoherent detrital matrix; southwest of Thousand Oaks partly intertongued with shale of unit* **Tml**; *in Simi Hills contains smooth rounded cobbles reworked from Simi Conglomerate* (**Tsug**) *as well as volcanic (andesitic) detritus; in northwestern area massive to crudely bedded, light brown pebble-small cobble conglomerate and breccia of volcanic detritus in semi-coherent matix; probably deposited subaerially on platform of deeply eroded pre-exitsing rocks; Lindero Canyon and older rocks cut by calcareous veins* **cv**

Tm / Tml

MONTEREY FORMATION

(Modelo Formation and in part upper Topanga Formation of Weber 1984; Yerkes and Showalter 1991, Monterey Formation of Truex and Hall 1969; same lithologic unit as Monterey Shale of Ventura basin of Dibblee 1989)

Marine biogenic; middle and late Miocene age, Luisian and Mohnian Stages, (Yerkes and Showalter 1991)

Tm *White weathering, thin bedded, platy, locally brittle siliceous shale to soft, punky shale; devoid of sandstone in this quadrangle; mostly late Miocene age (Mohnian Stage)*

Tml *Lower part, similar to* **Tm**, *but soft, fissile to punky; includes scattered thin, hard calcareous layers and concretions; middle Miocene age (Luisian Stage)*

Approximate Scale: 1" = 2,000'

GEOLOGIC SYMBOLS

not all symbols shown on each map

FORMATION CONTACT *dashed where inferred or indefinite dotted where concealed*

MEMBER CONTACT *between units of a formation* **Prominent bed**

CONTACT BETWEEN SURFICIAL SEDIMENTS *located only approximately in places*

FAULT: *Dashed where indefinite or inferred, dotted where concealed, queried where existence is doubtful. Parallel arrows indicate inferred relative lateral movement. Relative vertical movement is shown by U/D (U=upthrown side, D=downthrown side). Short arrow indicates dip of fault plane. Sawteeth are on upper plate of low angle thrust fault.*

U D 25 ?

FOLDS: *overturned* **ANTICLINE** **SYNCLINE**

arrow on axial trace of fold indicates direction of plunge; dotted where concealed by surficial sediments

Strike and dip of sedimentary rocks: 18 *inclined*; 20 *inclined (approximate)*; 80 *overturned*; *horizontal*; *vertical*

Strike and dip of metamorphic or igneous rock foliation or flow banding or compositional layers: 75 *inclined*; *inclined (approximate)*; *vertical*; 80 *overturned*

OTHER SYMBOLS: *Direction of landslide movement*; *outline of water bodies shown on map*; *water well*; *oil well*; *springs*

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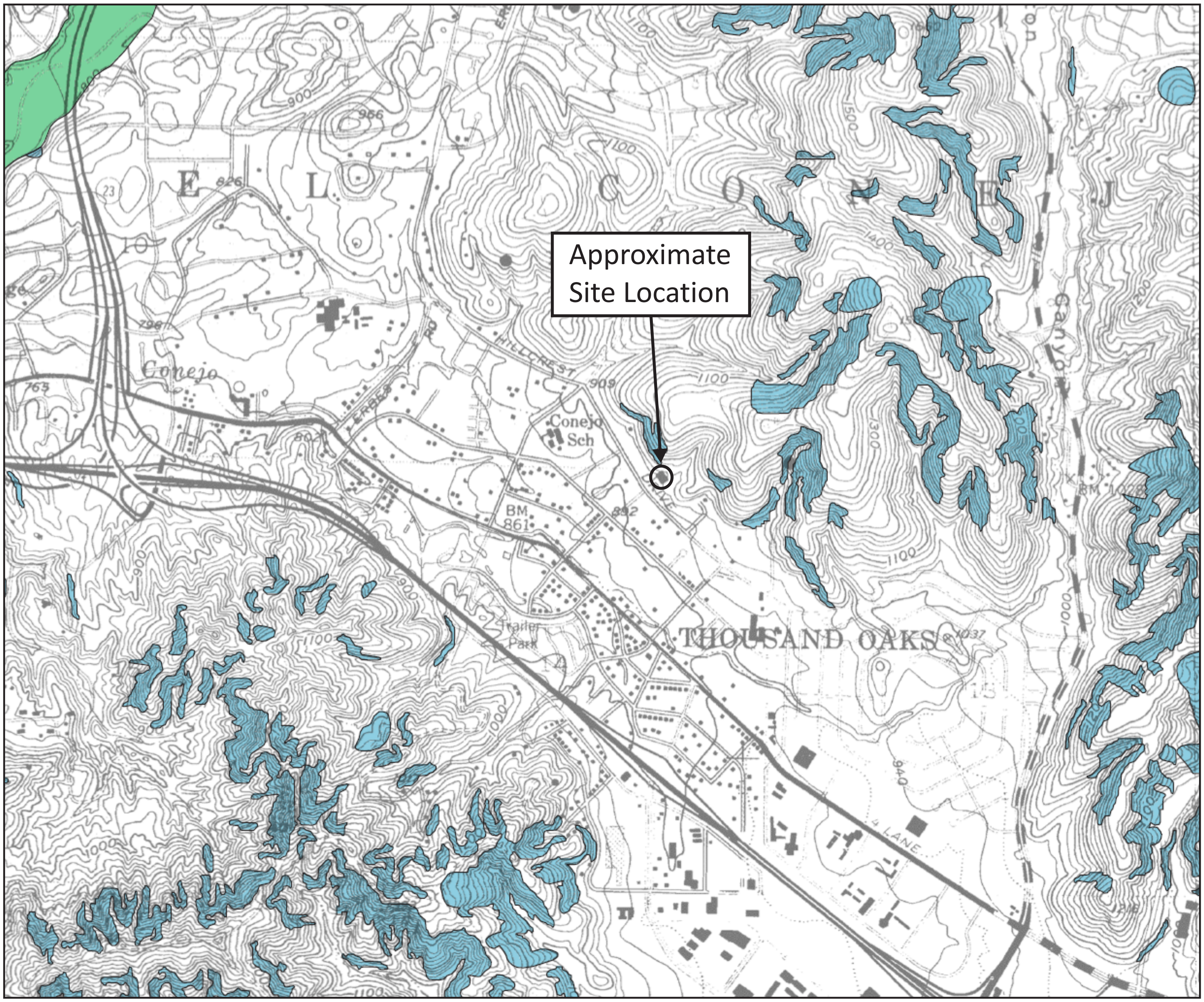
REGIONAL GEOLOGIC MAP

Conejo Valley Church of Christ
2525 East Hillcrest Drive
Thousand Oaks, California

Earth Systems

August 2018	301911-001



MAP EXPLANATION

Zones of Required Investigation:

Liquefaction

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 Landslides

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.

NOTE: Seismic Hazard Zones identified on this map may include developed land where delineated hazards have already been mitigated to city or county standards. Check with your local building/planning department for information regarding the location of such mitigated areas.

Approximate Scale: 1" = 2,000'

0 2,000' 4,000'

N

STATE OF CALIFORNIA
SEISMIC HAZARD ZONES
Delineated in compliance
Chapter 7.8, Division 2 of the California Public Resources
(Seismic Hazards Mapping

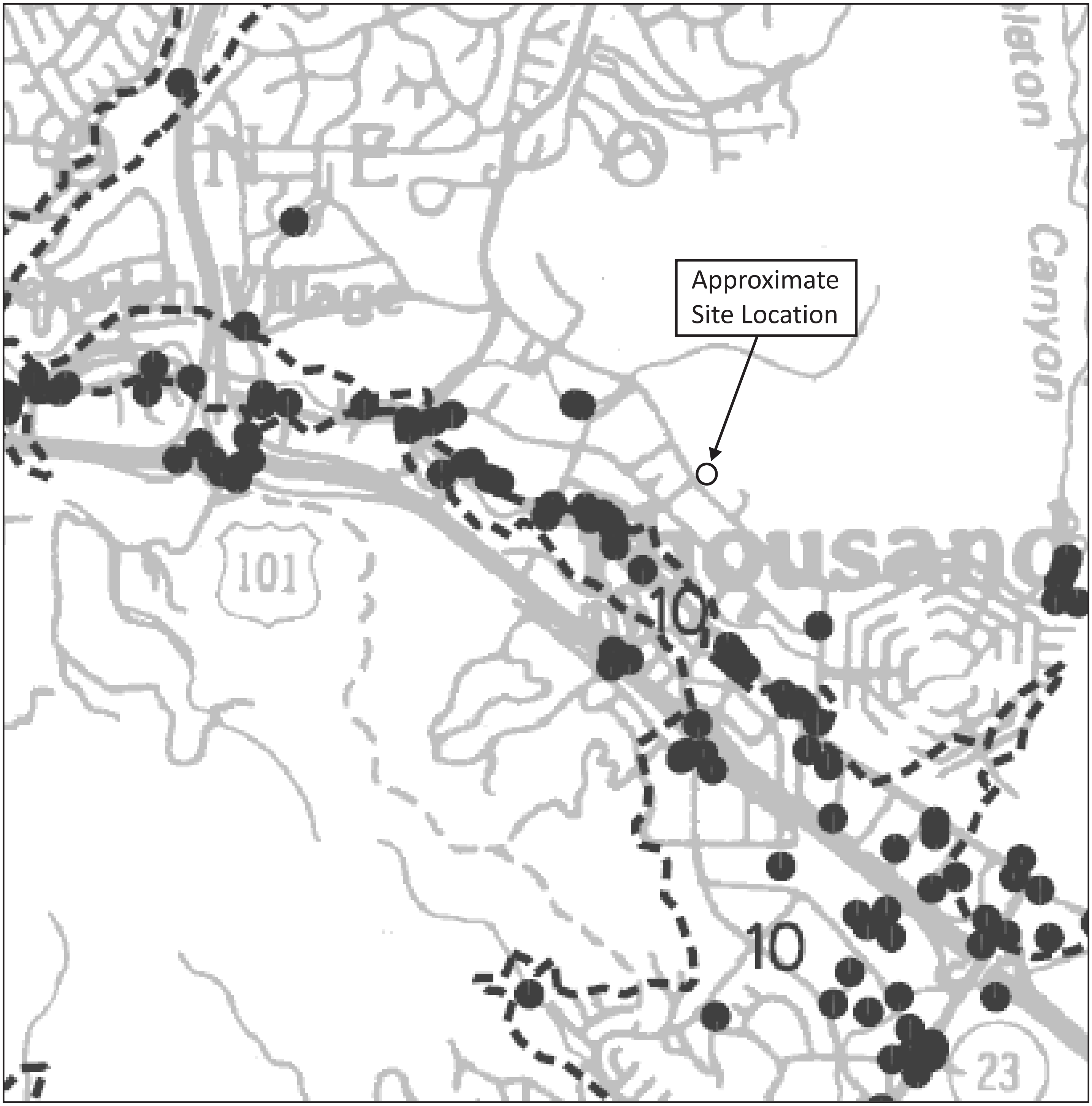
THOUSAND OAKS QUADRANGLE
OFFICIAL MAP
Released: November 17, 2000

SEISMIC HAZARD ZONES MAP

Conejo Valley Church of Christ
2525 East Hillcrest Drive
Thousand Oaks, California

Earth Systems

August 2018	301911-001



*Taken from CGS, Seismic Hazard Zone Report For The Thousand Oaks 7.5-Minute Quadrangle, Ventura and Los Angeles Counties, California, 2000.

20 Historically shallowest ground–water depth contours (in feet)

>40 Historically shallowest ground–water depth greater than 40 feet over a broad area

Alluviated Valley Boundary

Borehole Site

Approximate Scale: 1" = 2,000'

0 2,000' 4,000'

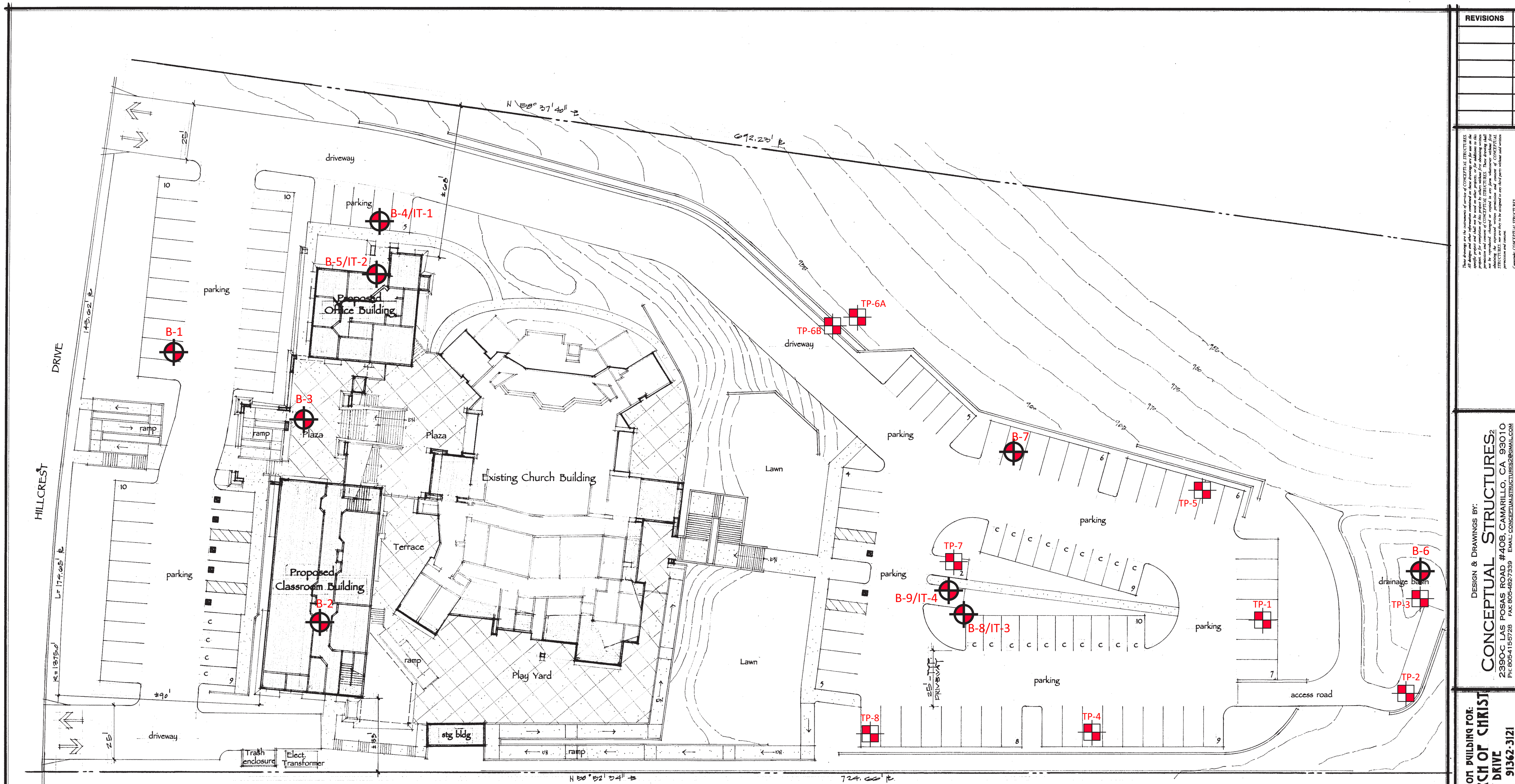
N

HISTORICAL HIGH GROUNDWATER MAP

Conejo Valley Church of Christ
2525 East Hillcrest Drive
Thousand Oaks, California

Earth Systems

August 2018	301911-001



SITE AREA TABULATION:

PROPOSED SITE DATA –

Gross Site Area -	199,486 s.f.	(4.58 ac.)	(100.0 %)
Total Proposed Building Area – (ground level area only)	17,664 s.f.	(0.40 ac.)	(08.85 %)
Total Proposed Paved Area – (including existing paving – new is 20,787 s.f.)	46,801 s.f.	(1.07 ac.)	(23.46 %)
Total Proposed Concrete/Hardscape Area - (including existing conc./hrdscp – new is 5,707 s.f.)	7,759 s.f.	(0.18 ac.)	(0.39 %)
Total Proposed Concrete Sidewalks/Ramps -	4,040 s.f.	(0.09 ac.)	(0.02%)
Total Proposed Landscaped Area -	63,633 s.f.	(1.46 ac.)	(31.89 %)
Total Proposed Catch Basin Area -	8,000 s.f.	(0.18 ac.)	(0.40 %)
Total Undisturbed Area -	51,589 s.f.	(1.18 ac.)	(25.8 %)
Total Impervious Area -	76,264 s.f.	(1.75 ac.)	(0.38 %)

EXISTING SITE DATA -

Gross Site Area -	199,486 s.f.	(4.58 ac.)	(100.0 %)
Existing Building Area -	9,225 s.f.	(0.21 ac.)	(04.62 %)
Total Paved Area -	26,014 s.f.	(0.60 ac.)	(13.04 %)
Total Concrete/Hardscape Area -	2,052 s.f.	(0.05 ac.)	(01.02 %)
Total Landscaped or Undisturbed Area	162,195 s.f.	(3.72 ac.)	(81.32 %)
Total Impervious Area -	37,291 s.f.	(0.85 ac.)	(18.69 %)

PARKING TABULATION:

REQUIRED:

Worship Center – 358 seats @ 1 space per 4 seats
358 seats – 4 = 89.5 spaces = 90 spaces required
Classrooms – 18 classrooms @ 1 space each = 18 spaces required

Total parking spaces required - 108 spaces

PROVIDED:

Standard 9'x20' spaces provided - 85
Compact 8'x18' spaces provided - 23 (20%)
(25% of total spaces is maximum allowed)
Handicapped 9'x20' spaces provided - 7
(5 required)

Total parking spaces provided - 115 spaces

Master Site Plan

B-9/IT-4 : Approximate boring/Infiltration testing locations.

TP-8 : Approximate test pit locations.

Approximate Scale: 1" = 50'

0 50' 100'

N

SITE PLAN

Conejo Valley Church of Christ
2525 East Hillcrest Drive
Thousand Oaks, California

Earth Systems

August 2018	301911-001

FIELD STUDY

A. Nine borings (B-1 through B-9) were drilled to a maximum depth of about 25.5 feet below the existing ground surface to observe the soil profile and to obtain samples for laboratory analyses. The borings were drilled on May 8, 2018, using 8-inch diameter hollow-stem continuous flight auger powered by a CME-85 truck mounted drilling rig. The approximate locations of the borings were determined in the field by pacing and sighting, and are shown on the Site Plan in this Appendix.

B. Nine test pits (TP-1 through TP-8) were hand excavated to a maximum depth of about 9 feet below the existing ground surface to observe the soil profile and to obtain samples for laboratory analyses. The test pits were hand excavated on May 11, 2018. The approximate locations of the test pits were determined in the field by pacing and sighting, see Site Plan in this Appendix.

C. Samples were obtained within the borings with a Modified California (M.C.) ring sampler (ASTM D 3550 with shoe similar to ASTM D 1586). The M.C. sampler has a 3-inch outside diameter, and a 2.42-inch inside diameter when used with brass ring liners (as it was during this study). In Borings B-1 through B-9, the samples were obtained by driving the sampler with a 140-pound hammer dropping 30 inches in accordance with ASTM D 1586. The hammer was operated with an automatic trip mechanism. In Test Pits TP-1 through TP-8, the samples were obtained by driving the sampler with the bucket of a backhoe. Due to the sampling method employed, blow counts were not recorded for samples obtained from all test pits.

D. Three bulk samples were collected from the cuttings of the soils encountered between the depths of 0 and 5 feet in Boring B-2, between 0 and 3.5 feet in Boring B-7, and between 5 and 6 feet in Test Pit TP-5.

E. The final logs of the borings and test pits represent interpretations of the contents of the field logs and the results of laboratory testing performed on the samples obtained during the subsurface study. The final logs are included in this Appendix.

Logs of Borings

Earth Systems
1731-A Walter Street, Ventura, California 93003
PHONE: (805) 642-6727 FAX: (805) 642-1325

BORING NO: B-1
PROJECT NAME: Conejo Valley Church of Christ
PROJECT NUMBER: 301911-001
BORING LOCATION: Per Plan

DRILLING DATE: May 8, 2018
DRILL RIG: CME-85
DRILLING METHOD: Eight-Inch Hollow Stem Auger
LOGGED BY: LG

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0									3" of asphalt overlying 2" of aggregate base.
			■	7,8,11		ML	68.4	22.5	**ARTIFICIAL FILL:** Brown silt with silty fine sand and gravels, firm, moist.
5			■	8,10,11		ML	71.8	23.2	**ALLUVIUM:** Brown sandy silt with gravels and caliche stringers, firm, moist.
10			■	35,50-3"		Tlvc	92.6	13.1	**DETRITAL SEDIMENTS OF LINDERO CANYON:** Orangish brown and brown silty sand with gravels; very dense; damp.
15			■	28,50-3"		Tlvc	100.4	11.0	**DETRITAL SEDIMENTS OF LINDERO CANYON:** Organish brown and brown silty sand with gravels; oxidized fine to coarse sand; very dense; damp.
20			■	16,50-5"		Tlvc	86.6	27.9	**DETRITAL SEDIMENTS OF LINDERO CANYON:** Organish brown medium to coarse sandstone with gravels and cobbles; very dense; moist.
25			■	50-5"		Tlvc	85.8	21.1	**DETRITAL SEDIMENTS OF LINDERO CANYON:** Organish brown sandstone with gravels and cobbles; very dense; moist.
									Total Depth: 25.4 feet. **No Groundwater Encountered.**
30									
35									

Note: The stratification lines shown represent the approximate boundaries between soil and/or rock types and the transitions may be gradual.

Earth Systems

1731-A Walter Street, Ventura, California 93003
PHONE: (805) 642-6727 FAX: (805) 642-1325

BORING NO: B-2
PROJECT NAME: Conejo Valley Church of Christ
PROJECT NUMBER: 301911-001
BORING LOCATION: Per Plan

DRILLING DATE: May 8, 2018
DRILL RIG: CME-85
DRILLING METHOD: Eight-Inch Hollow Stem Auger
LOGGED BY: LG

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0	X								3" of asphalt overlying 4" of aggregate base.
	X		■	6,6,7		CL			**ARTIFICIAL FILL:** Brown sandy clay with gravels, firm, moist.
5			■	6,5,7		ML	68.3	24.2	**ALLUVIUM:** Brown clayey silt with gravels and caliche stringers, firm, moist.
10			■	8,9,10		ML	71.7	20.5	**ALLUVIUM:** Brown sandy silt with clays and gravels, firm, moist.
									DETRITAL SEDIMENTS OF LINDERO CANYON: Sandy Gravel and gravel conglomerate.
15			■	50-4"		Tlvc			No Recovery.
20			■	50-5"		Tlvc	79.0	22.6	**DETRITAL SEDIMENTS OF LINDERO CANYON:** Organish brown and olive brown silty fine sandstone and silt stone; dense; moist.
25			■	50-5"		Tlvc			**DETRITAL SEDIMENTS OF LINDERO CANYON:** Organish brown, olive brown, and gray fine sandstone with gravel conglomerate; dense; moist.
									Total Depth: 25.4 feet.
									No Groundwater Encountered.
30									
35									

Note: The stratification lines shown represent the approximate boundaries between soil and/or rock types and the transitions may be gradual.

Earth Systems
1731-A Walter Street, Ventura, California 93003
PHONE: (805) 642-6727 FAX: (805) 642-1325

BORING NO: B-3
PROJECT NAME: Conejo Valley Church of Christ
PROJECT NUMBER: 301911-001
BORING LOCATION: Per Plan

DRILLING DATE: May 8, 2018
DRILL RIG: CME-85
DRILLING METHOD: Eight-Inch Hollow Stem Auger
LOGGED BY: LG

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0									3" asphalt over 9" concrete base
				7,5,6		ML	66.9	21.2	**ARTIFICAL FILL:** Gravelly silt to silt with little gravel; firm; damp; brown; pockets of gravel and caliche pods.
5				10,13,19		ML	83.8	20.7	**ALLUVIUM:** Gravelly silt to silt with little gravel; firm; damp; brown; pockets of gravel and caliche pods.
						Tlvc			**DETRITAL SEDIMENTS OF LINDERO CANYON:** Silty fine to medium sandstone; orangish brown and pale brown weak competency; slightly weathered.
10				50-6"		Tlvc	92.0	13.8	**Same As Above**
15				20,50-2"		Tlvc	94.0	15.8	**DETRITAL SEDIMENTS OF LINDERO CANYON:** Gravel conglomerate; silty sandy gravel; pale brown and orangish brown; moderate competency.
									DETRITAL SEDIMENTS OF LINDERO CANYON: Blue gray gravel conglomerate with fine cobbles.
20				23,50-4"		Tlvc	95.3	19.6	**DETRITAL SEDIMENTS OF LINDERO CANYON:** Silty medium to coarse sandstone; orangish brown and olive; moderate competency.
25				50-6"		Tlvc	70.0	30.4	**DETRITAL SEDIMENTS OF LINDERO CANYON:** Silty fine sandstone with few to little gravel; olive and orangish brown; moderate competency.
									Total Depth: 25.5 feet.
									No Groundwater Encountered.
30									
35									

Note: The stratification lines shown represent the approximate boundaries between soil and/or rock types and the transitions may be gradual.

Earth Systems
1731-A Walter Street, Ventura, California 93003
PHONE: (805) 642-6727 FAX: (805) 642-1325

BORING NO: B-4/IT-1
PROJECT NAME: Conejo Valley Church of Christ
PROJECT NUMBER: 301911-001
BORING LOCATION: Per Plan

DRILLING DATE: May 8, 2018
DRILL RIG: CME-85
DRILLING METHOD: Eight-Inch Hollow Stem Auger
LOGGED BY: LG

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0									4" asphalt over 10" aggregate base
	X					Tlvc			
	X		■	12,24,50-3"		Tlvc			**DETRITAL SEDIMENTS OF LINDERO CANYON:** Brown gravel conglomerate; weaterhed to coarse sandy gravel; very dense; damp weak competency.
5			■	50-6"		Tlvc			**DETRITAL SEDIMENTS OF LINDERO CANYON:** Gravel conglomerate; orangish brown; very dense; damp; moderate competency; slightly weathered.
10			■	50-6"		Tlvc			**DETRITAL SEDIMENTS OF LINDERO CANYON:** Gravel conglomerate; silty; coarse to medium sandstone; moderate copetency.
									Total Depth: 13.0 feet.
									No Groundwater Encountered.
15									
20									
25									
30									
35									

Note: The stratification lines shown represent the approximate boundaries between soil and/or rock types and the transitions may be gradual.

Earth Systems
1731-A Walter Street, Ventura, California 93003
PHONE: (805) 642-6727 FAX: (805) 642-1325

BORING NO: B-5/IT-2	DRILLING DATE: May 8, 2018
PROJECT NAME: Conejo Valley Church of Christ	DRILL RIG: CME-85
PROJECT NUMBER: 301911-001	DRILLING METHOD: Eight-Inch Hollow Stem Auger
BORING LOCATION: Per Plan	LOGGED BY: LG

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0									4" asphalt over 10" aggregate base
						SM			**DETRITAL SEDIMENTS OF LINDERO CANYON:** Orangish brown sandstone weathered; weak; competency; weathers to silty sand; dense; damp.
									Total Depth: 2.0 feet.
5									**No Groundwater Encountered.**
10									
15									
20									
25									
30									
35									

Note: The stratification lines shown represent the approximate boundaries between soil and/or rock types and the transitions may be gradual.

Earth Systems

1731-A Walter Street, Ventura, California 93003
PHONE: (805) 642-6727 FAX: (805) 642-1325

BORING NO: B-6	DRILLING DATE: May 8, 2018
PROJECT NAME: Conejo Valley Church of Christ	DRILL RIG: CME-85
PROJECT NUMBER: 301911-001	DRILLING METHOD: Eight-Inch Hollow Stem Auger
BORING LOCATION: Per Plan	LOGGED BY: LG

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0–5	X		X	2,3,3		SM	57.7	20.1	**ARTIFICAL FILL**: Very fine sandy silt to silty very fine sand; few fine gravel; light brown; very loose; dry to damp.
5			X	4,5,6		ML	68.6	20.1	**ALLUVIUM**: Slightly sandy silt; few -trace gravel; light brown; caliche; loosel dry to damp.
10			X	8,10,15		ML	77.4	25.0	**ALLUVIUM**: very coarse sandy silt; few-little fine gravel; light brown.
15			X	9,11,10		ML	80.2	24.3	**ALLUVIUM**: Slightly sandy silt with few-little gravel trace cobbles; light brown minor fine caliche nodules.
20			X	4,8,11		GM	74.3	24.7	**ALLUVIUM**: Slightly gravelly to gravelly silt; few-little sand; few clay; light brown; moist; firm to stiff.
25									**Total Depth: 21.5 feet.** **No Groundwater Encountered.**
30									
35									

Note: The stratification lines shown represent the approximate boundaries between soil and/or rock types and the transitions may be gradual.

Earth Systems
1731-A Walter Street, Ventura, California 93003
PHONE: (805) 642-6727 FAX: (805) 642-1325

BORING NO: B-7
PROJECT NAME: Conejo Valley Church of Christ
PROJECT NUMBER: 301911-001
BORING LOCATION: Per Plan

DRILLING DATE: May 8, 2018
DRILL RIG: CME-85
DRILLING METHOD: Eight-Inch Hollow Stem Auger
LOGGED BY: LG

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0	X					CL			**SOIL:** Silty clay; few sand and gravel dark brown; grades to clayey silt.
	X		■	4,5,12		Tm	78.5	30.9	**MONTEREY FORMATION:** Orangish brown; olive and green calcareous shale and bedded siltstone; weak to moderate competency; slightly- moderately weathered.
5			■	39,50-5"		Tm	76.2	25.5	**Same As Above**
10			■	50-4"		Tm	82.8	20.8	**MONTEREY FORMATION:** Orangish brown; olive and green calcareous shale and bedded siltstone; moderate competency; slightly- moderately weathered.
15			■	32,50-2"		Tm	78.0	24.2	**MONTEREY FORMATION:** Shale; fissile; brown blueish gray and orangish brown; moderate competency.
20									
25									
30									
35									

Total Depth: 15.7 feet.

No Groundwater Encountered.

Note: The stratification lines shown represent the approximate boundaries between soil and/or rock types and the transitions may be gradual.

Earth Systems
1731-A Walter Street, Ventura, California 93003
PHONE: (805) 642-6727 FAX: (805) 642-1325

BORING NO: B-8/IT-3
PROJECT NAME: Conejo Valley Church of Christ
PROJECT NUMBER: 301911-001
BORING LOCATION: Per Plan

DRILLING DATE: May 8, 2018
DRILL RIG: CME-85
DRILLING METHOD: Eight-Inch Hollow Stem Auger
LOGGED BY: LG

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0						ML			**ARTIFICAL FILL:** Slightly clayey silt to silty with little clay; few to little fine gravel and cobble; dark brown damp.
5						ML			**ALLUVIUM:** Very fine sandy silt with few to little gravel; damp; firm; trace cobbles.
10						ML			**ALLUVIUM:** Very fine sandy silt with few to little gravel; damp; firm to stiff; trace cobbles.
15									**Total Depth: 13.0 feet.** **No Groundwater Encountered.**
20									
25									
30									
35									

Note: The stratification lines shown represent the approximate boundaries between soil and/or rock types and the transitions may be gradual.

Earth Systems

1731-A Walter Street, Ventura, California 93003
PHONE: (805) 642-6727 FAX: (805) 642-1325

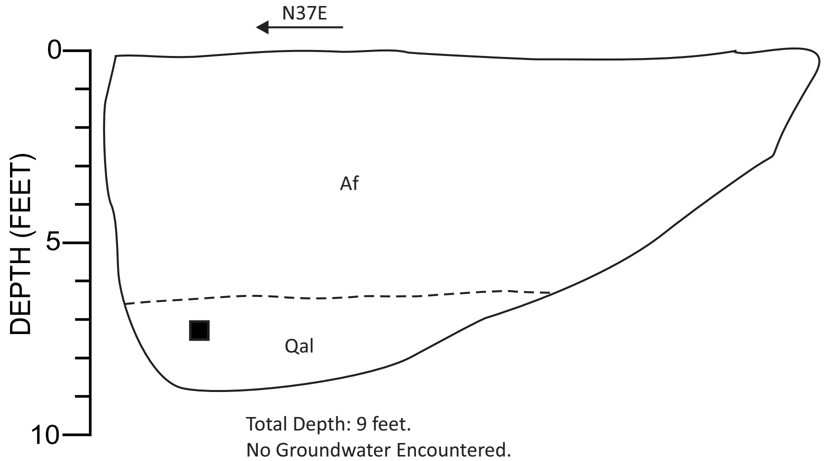
BORING NO: B-9/IT-4	DRILLING DATE: May 8, 2018
PROJECT NAME: Conejo Valley Church of Christ	DRILL RIG: CME-85
PROJECT NUMBER: 301911-001	DRILLING METHOD: Eight-Inch Hollow Stem Auger
BORING LOCATION: Per Plan	LOGGED BY: LG

Vertical Depth	Sample Type: Bulk	Sample Type: SPT	Sample Type: Mod. Calif.	PENETRATION RESISTANCE (BLOWS/6")	SYMBOL	USCS CLASS	UNIT DRY WT. (pcf)	MOISTURE CONTENT (%)	DESCRIPTION OF UNITS
0						ML			**ARTIFICAL FILL:** Slightly clayey silt to silt with little clay; few gravel and cobbles; dark brown damp; soft to firm.
									Total Depth: 2.0 feet. **No Groundwater Encountered.**
5									
10									
15									
20									
25									
30									
35									

Note: The stratification lines shown represent the approximate boundaries between soil and/or rock types and the transitions may be gradual.

Logs of Test Pits

TP-1



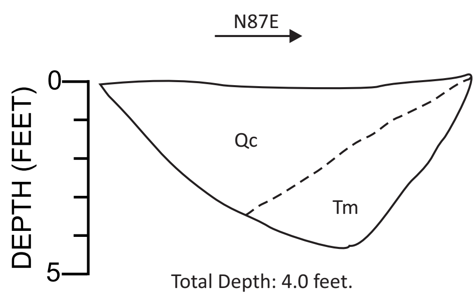
Total Depth: 9 feet.
No Groundwater Encountered.

Descriptions

Artificial Fill: Af
Brown slightly silty sand to sandy silt with some gravels, cobbles, and clay; loose/soft; damp.

Alluvium: Qal
Brown sandy silt with some fine gravels; medium stiff to firm; trace cobbles; damp.

TP-2



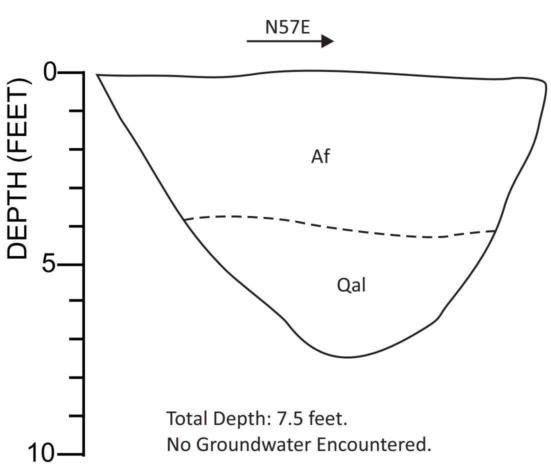
Total Depth: 4.0 feet.
No Groundwater Encountered.

Descriptions

Colluvium: Qc
Brown silty sand to sandy silt with some gravels and fine roots; trace cobbles; loose/soft; damp.

Monterey Formation: Tm
Pale yellowish brown, organish brown, and olive brown shale; fissile; moderate competency; slightly weathered; bedded to laminated; damp.

TP-3



Total Depth: 7.5 feet.
No Groundwater Encountered.

Descriptions

Artificial Fill: Af
Brown slightly silty sand to sandy silt with some gravels and cobbles; loose/soft; damp.

Alluvium: Qal
Brown silty sand with some gravels; medium dense; trace cobbles; damp.

■ Approximate Sample Location
TP-1 @ 7’ Dry Density 83.5 lbs/ft^3, Moisture 25.3 %

Logged by LG on 5-11-18 1 in = 5 ft

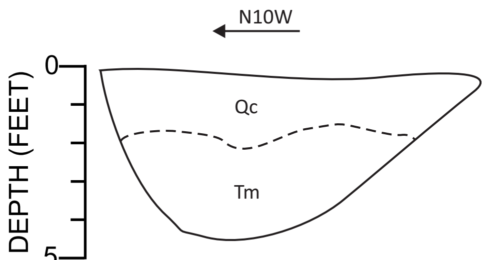
TEST PIT LOGS TP-1 THROUGH TP-3

Conejo Valley Church of Christ
2525 East Hillcrest Drive
Thousand Oaks, California

Earth Systems

August 2018 301911-001

TP-4



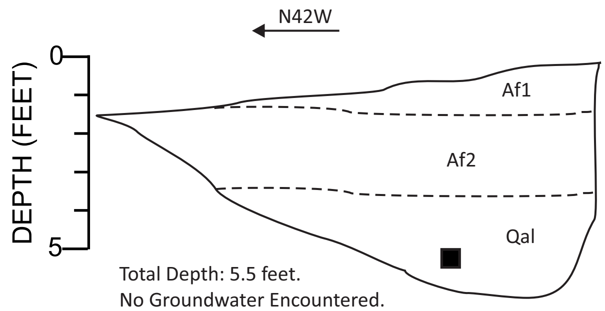
Total Depth: 4.5 feet.
No Groundwater Encountered.

Descriptions

Colluvium: Qc
Brown slightly clayey silty sand to sandy clayey silt with some fine roots; loose/soft; damp.

Monterey Formation: Tm
White and organish brown siltstone and shale; massive; slightly weathered; moderate competency; moderately weathered in the upper 0.5 foot; weak competency in the upper 0.5 foot.

TP-5



Total Depth: 5.5 feet.
No Groundwater Encountered.

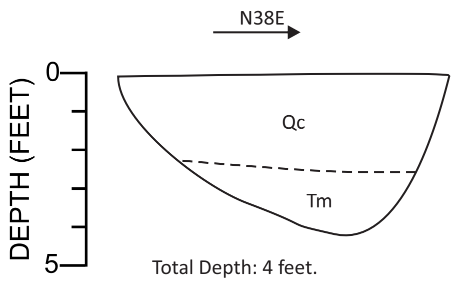
Descriptions

Artificial Fill: Af1
Dark brown slightly sandy silty clay with some gravels, and fine roots; soft; moist.

Artificial Fill: Af2
Dark brown very fine sandy silt with some gravels, cobbles, and pockets of very loose silty sand; soft; damp.

Alluvium: Qal
Dark brown silty clay with some fine caliche stringers and fine gravles; firm to medium stiff; damp.

TP-6A



Total Depth: 4 feet.
No Groundwater Encountered.

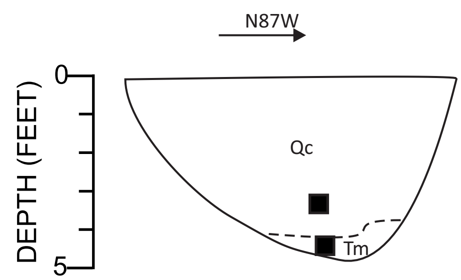
Descriptions

Colluvium: Qc
Dark brown sandy silty clay with some gravels; medium stiff to stiff; damp.

Monterey Formation: Tm
Olive and orange brown shale; fissile; moderate competency; slightly weathered; massive; damp.

*Backhoe along slope contour, unable to push sampler, reset facing up slope in TP-6B.

TP-6B



Total Depth: 5 feet.
No Groundwater Encountered.

Descriptions

Colluvium: Qc
Dark brown sandy silty clay with some fine gravels; medium stiff; damp.

Monterey Formation: Tm
Olive, pale blue gray, and orange brown shale; fissile; moderate competency; slightly weathered; massive; damp.

■ Approximate Sample Location

TP-5 @ 5' Dry Density 77.1 lbs/ft^3, Moisture 22.2 %
TP-6B @ 3' Dry Density 73.6 lbs/ft^3, Moisture 23.8 %
TP-6B @ 4' Dry Density 83.0 lbs/ft^3, Moisture 22.0 %

Logged by LG on 5-11-18

1 in = 5 ft

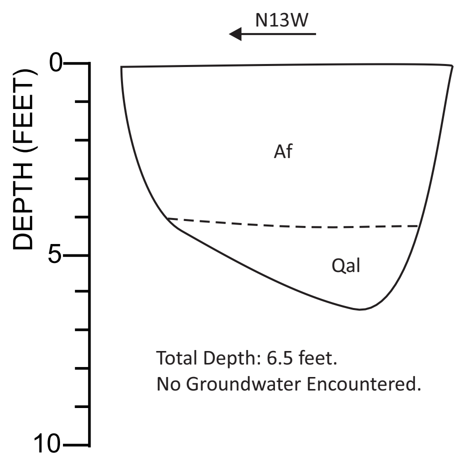
TEST PIT LOGS TP-4 THROUGH TP-6B

Conejo Valley Church of Christ
2525 East Hillcrest Drive
Thousand Oaks, California

Earth Systems

August 2018 | 301911-001

TP-7



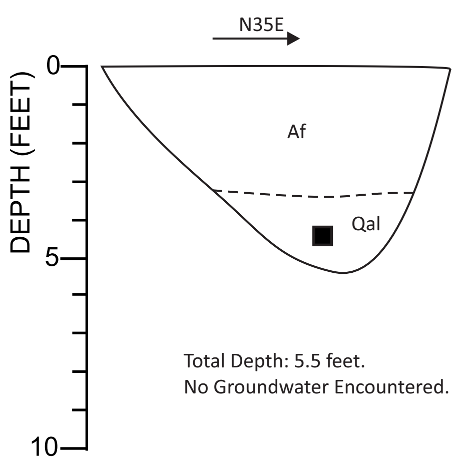
Total Depth: 6.5 feet.
No Groundwater Encountered.

Descriptions

Artificial Fill: Af
Dark brown clayey silt with some cobbles; moist; trace cobbles.

Alluvium: Qal
Pale to medium brown very fine sandy silt; firm; few fine caliche stringers; damp.

TP-8



Total Depth: 5.5 feet.
No Groundwater Encountered.

Descriptions

Artificial Fill: Af
Organish brown sandy silt with some gravels; damp; platy texture; trace cobbles.

Alluvium: Qal
Brown sandy silt with some fine gravels; medium stiff; few fine caliche stringers; damp.

■ Approximate Sample Location
TP-8 @ 4’ Dry Density 66.1 lbs/ft^3, Moisture 23.9 %

Logged by LG on 5-11-18

1 in = 5 ft

TEST PIT LOGS TP-7 AND TP-8

Conejo Valley Church of Christ
2525 East Hillcrest Drive
Thousand Oaks, California

Earth Systems

August 2018	301911-001

BORING LOG SYMBOLS

Modified California Split Barrel Sampler

Modified California Split Barrel Sampler - No Recovery

Standard Penetration Test (SPT) Sampler

Standard Penetration Test (SPT) Sampler - No Recovery

Perched Water Level

Water Level First Encountered

Water Level After Drilling

Pocket Penetrometer (tsf)

Vane Shear (ksf)

1. The location of borings were approximately determined by pacing and/or siting from visible features. Elevations of borings are approximately determined by interpolating between plan contours. The location and elevation of the borings should be considered.

2. The stratification lines represent the approximate boundary between soil types and the transition may be gradual.

3. Water level readings have been made in the drill holes at times and under conditions stated on the boring logs. This data has been reviewed and interpretations made in the text of this report. However, it must be noted that fluctuations in the level of the groundwater may occur due to variations in rainfall, tides, temperature, and other factors at the time measurements were made.

BORING LOG SYMBOLS

Earth Systems

UNIFIED SOIL CLASSIFICATION SYSTEM

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

NOTE: DUAL SYMBOLS ARE USED TO INDICATE BORDERLINE SOIL CLASSIFICATIONS

UNIFIED SOIL CLASSIFICATION SYSTEM

Earth Systems

APPENDIX B

Laboratory Testing

Tabulated Laboratory Test Results

Individual Laboratory Test Results

LABORATORY TESTING

A. Samples were reviewed along with field logs to determine which would be analyzed further. Those chosen for laboratory analyses were considered representative of soils that would be exposed and/or used during grading, and those deemed to be within the influence of proposed structures. Test results are presented in graphic and tabular form in this Appendix.

B. In-situ moisture content and dry unit weight for the ring samples were determined in general accordance with ASTM D 2937.

C. Maximum density tests were performed to estimate the moisture-density relationship of typical soil materials. The tests were performed in accordance with ASTM D 1557.

D. The relative strength characteristics of soils were determined from the results of direct shear tests on 2 remolded samples and 2 relatively undisturbed ring samples. The specimens were placed in contact with water at least 24 hours before testing, and were then sheared under normal loads ranging from 0.5 to 3 ksf in general accordance with ASTM D 3080.

E. Settlement characteristics were developed from the results of one-dimensional consolidation tests performed in general accordance with ASTM D 2435. The samples were incrementally loaded to 0.5 ksf, flooded with water, and then incrementally loaded to 1.0, 2.0, and 4.0 ksf. The samples were allowed to consolidate under each load increment. Rebound was measured under reverse alternate loading. Compression was measured by dial gauges accurate to 0.0001 inch. Results of the consolidation tests are presented in this Appendix in the form of percent consolidation versus log of pressure curves.

F. Expansion index tests were performed on bulk soil samples in accordance with ASTM D 4829. The samples were surcharged under 144 pounds per square foot at moisture content of near 50 percent saturation. Samples were then submerged in water for 24 hours and the amount of expansion was recorded with a dial indicator.

G. The gradation characteristics of certain samples were evaluated by hydrometer (in accordance with ASTM D 7928) and sieve analysis procedures. The samples were soaked in water until individual soil particles were separated, then washed on the No. 200 mesh sieve, oven dried, weighed to calculate the percent passing the No. 200 sieve, and mechanically sieved. Additionally, hydrometer analyses were performed to assess the distribution of the particles that passed the No. 200 screen. The hydrometer portions of the tests were run using sodium hexametaphosphate as a dispersing agent.

H. Portions of the bulk samples were sent to another laboratory for analyses of soil pH, resistivity, chloride contents, and sulfate contents. Soluble chloride and sulfate contents were determined on a dry weight basis. Resistivity testing was performed in accordance with California Test Method 424, wherein the ratio of soil to water was 1:3.

TABULATED LABORATORY TEST RESULTS

REMOLDED SAMPLES

BORING AND DEPTH	B-2@0'-5'	TP-5@5'-6'
USCS	CL	CL
MAXIMUM DRY DENSITY (pcf)	90.5 94.5*	89 92.5*
OPTIMUM MOISTURE (%)	19 17*	21 19.5*
PEAK COHESION (psf)	270	60
PEAK FRICTION ANGLE	26°	30°
ULTIMATE COHESION (psf)	140	60
ULTIMATE FRICTION ANGLE	28°	30°
EXPANSION INDEX	35	80
pH	8.1	8.5
RESISTIVITY (ohms-cm)	2,000	11,000
SOLUBLE CHLORIDES (mg/Kg)	35	1.8
SOLUBLE SULFATES (mg/Kg)	520	7.7
GRAIN SIZE DISTRIBUTION (%)		
GRAVEL	14.5	9.4
SAND	30.5	21.1
SILT	21.8	28.6
CLAY (2ųm to 5ųm)	8.5	9.5
CLAY (≤2ųm)	24.7	31.4

*Corrected for Oversize (ASTM D4718)

RELATIVELY UNDISTURBED SAMPLE

BORING AND DEPTH	B-2@20'	B-7@5'
USCS	SM	ML
DISCRIPTION	Sandstone	Shale and Silt Stone
IN-PLACE DRY DENSITY (pcf)	79.1	76.2
IN-PLACE MOISTURE (%)	22.6	25.5
PEAK COHESION (psf)	900	970
PEAK FRICTION ANGLE	37°	47°
ULTIMATE COHESION (psf)	440	270
ULTIMATE FRICTION ANGLE	41°	54°

Individual Laboratory Test Results

File Number: 301911-001 Lab Number: 097799

MAXIMUM DENSITY / OPTIMUM MOISTURE

ASTM D 1557-12 (Modified)

Job Name: Conejo Valley Church of Christ
Sample ID: B 2 @ 0-5'

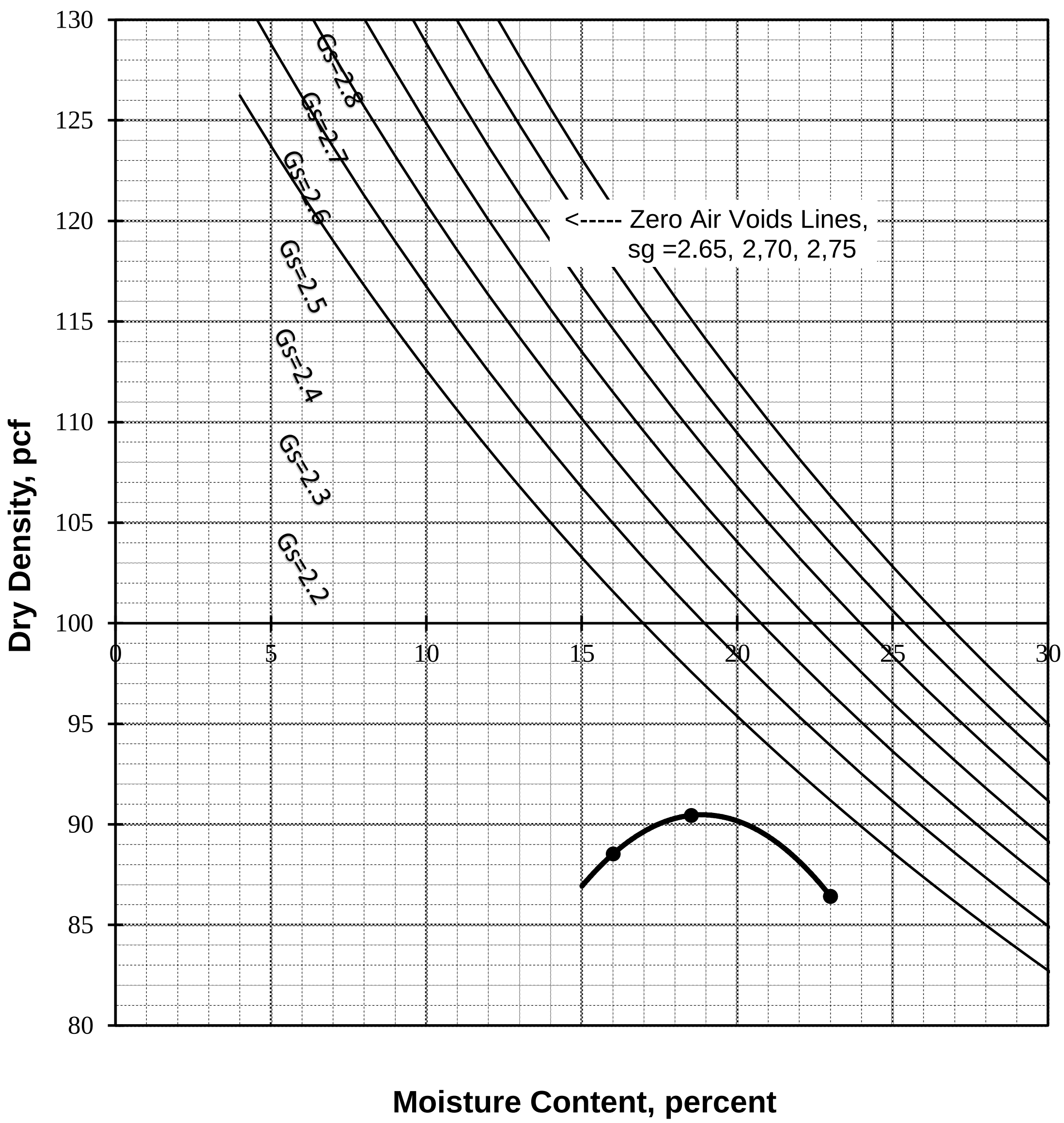
Procedure Used: A
Prep. Method: Moist

Date: 5/28/2018
Description: Brown Sandy Clay
SG: 1.99

Rammer Type: Automatic

Maximum Density: **90.5 pcf**
Optimum Moisture: **19%**

Sieve Size	% Retained
3/4"	0.0
3/8"	0.0
#4	14.5



File Number: 301911-001 Lab Number: 097799

MAXIMUM DENSITY / OPTIMUM MOISTURE

ASTM D 1557-12 (Modified)

Job Name: Conejo Valley Church of Christ

Sample ID: B 2 @ 0-5'

Procedure Used: A

Prep. Method: Moist

Date: 5/28/2018

Rammer Type: Automatic

Description: Brown Sandy Clay

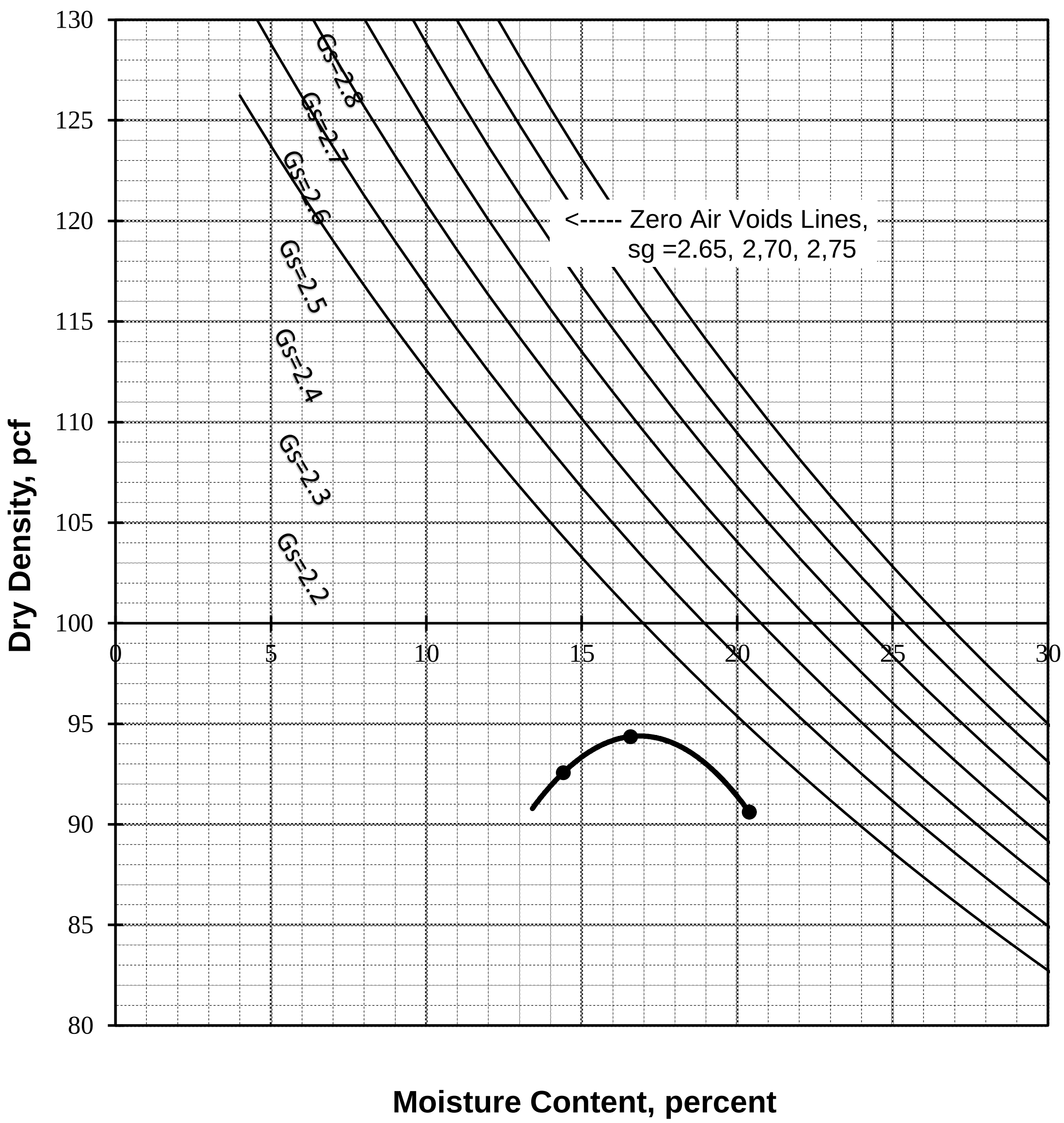
SG: 2.03

Maximum Density: **94.5 pcf**

Optimum Moisture: **17%**

Corrected for Oversize (ASTM D4718)

Sieve Size	% Retained
3/4"	0.0
3/8"	0.0
#4	14.5



File Number: 301911-001 Lab Number: 097794

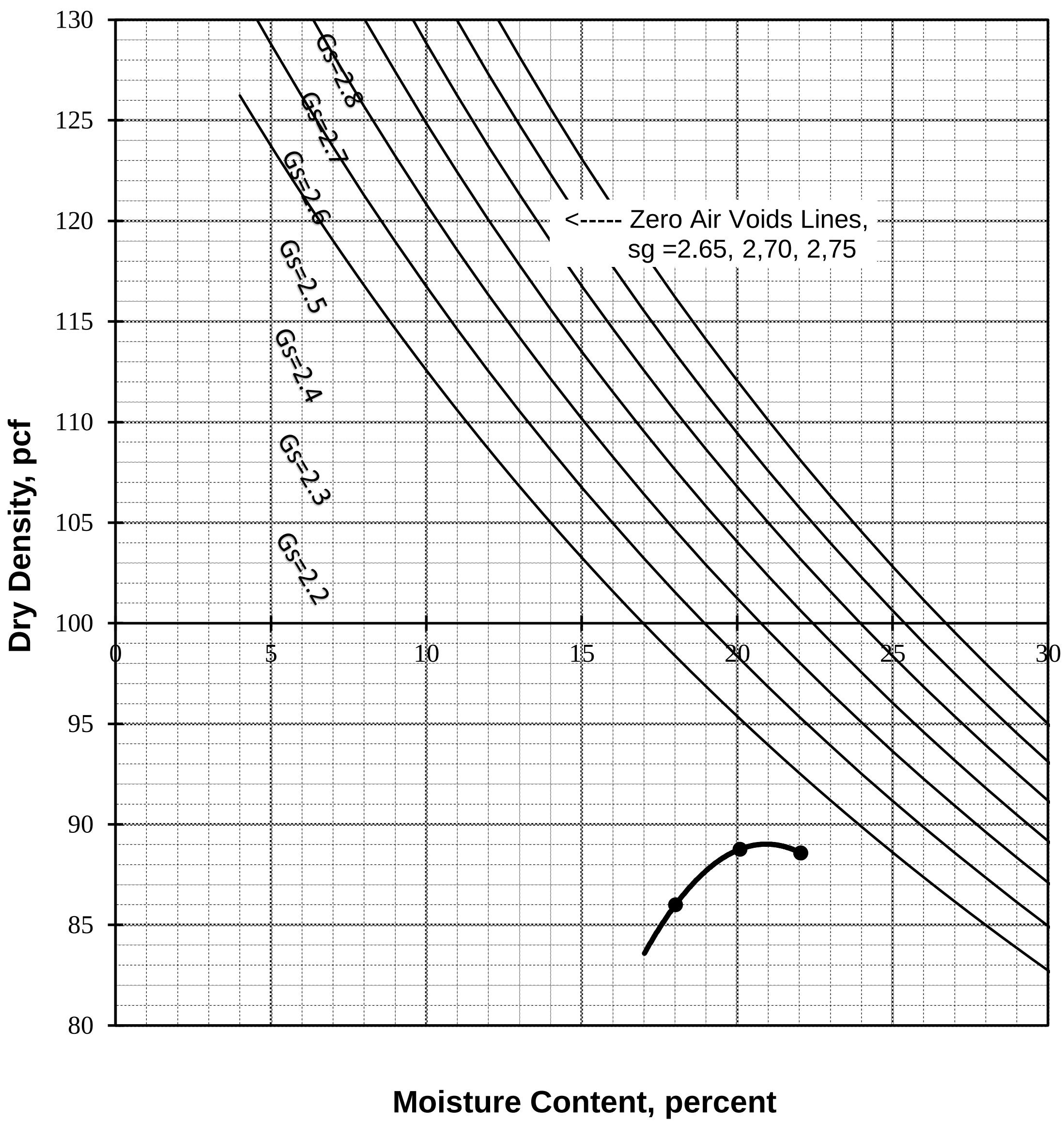
MAXIMUM DENSITY / OPTIMUM MOISTURE

ASTM D 1557-12 (Modified)

Job Name:	Conejo Valley Church of Christ		Procedure Used: A
Sample ID:	T P 5 @ 5'-6'		Prep. Method: Moist
Date:	6/26/2018		Rammer Type: Automatic
Description:	Dark Brown Silty Clay		
SG:	2.03		

Maximum Density: **89 pcf**
Optimum Moisture: **21%**

Sieve Size	% Retained
3/4"	0.0
3/8"	0.0
#4	10.4



File Number: 301911-001 Lab Number: 097794

MAXIMUM DENSITY / OPTIMUM MOISTURE

ASTM D 1557-12 (Modified)

Job Name: Conejo Valley Church of Christ
Sample ID: T P 5 @ 5'-6'

Procedure Used: A
Prep. Method: Moist

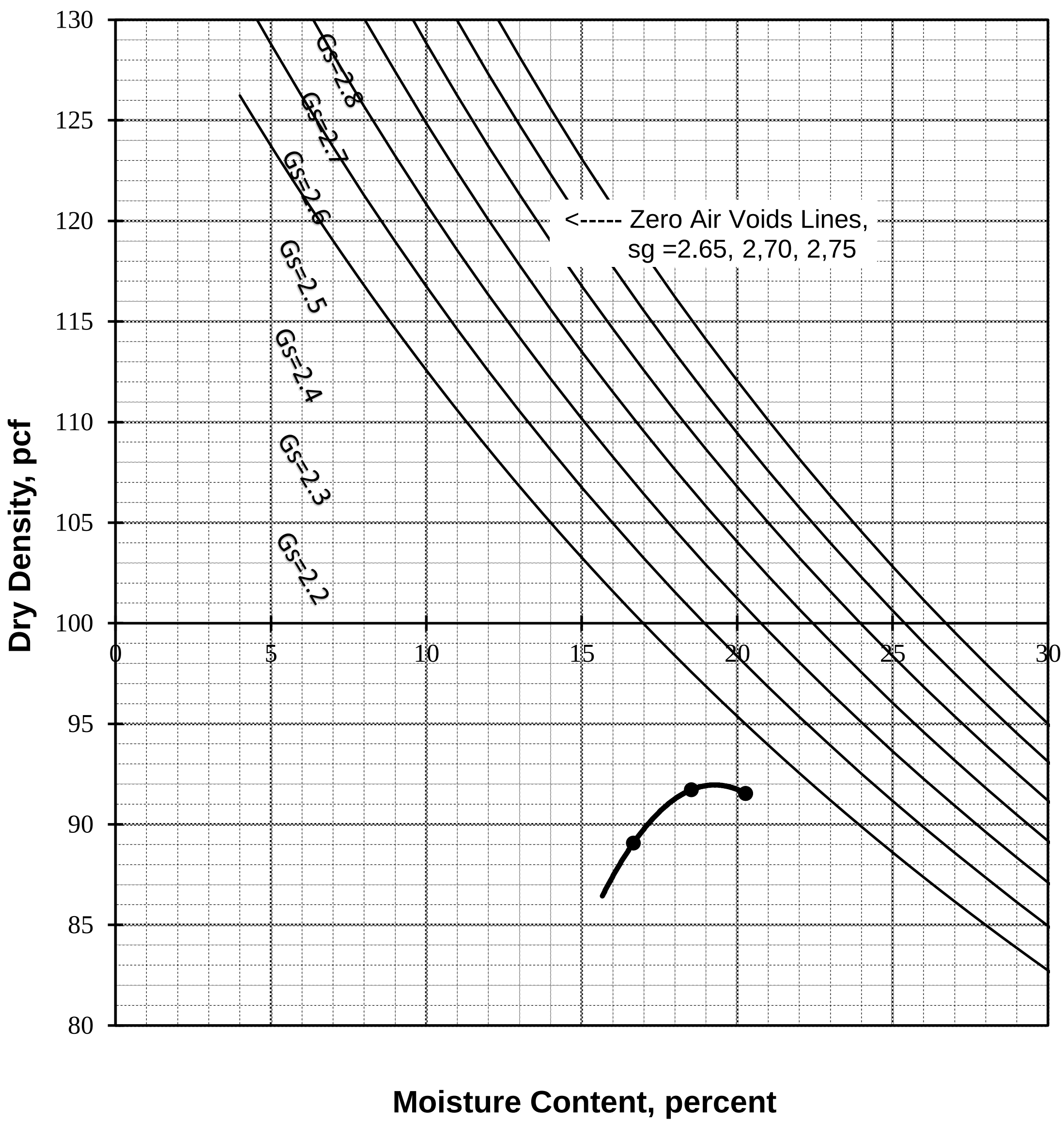
Date: 6/26/2018
Description: Dark Brown Silty Clay
SG: 2.06

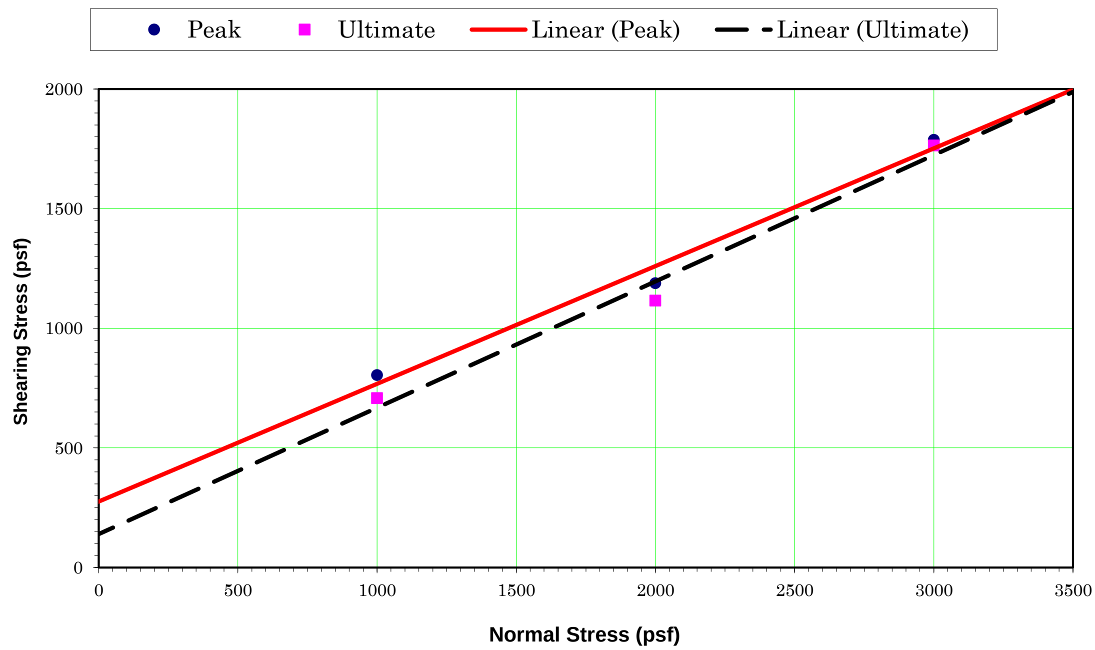
Rammer Type: Automatic

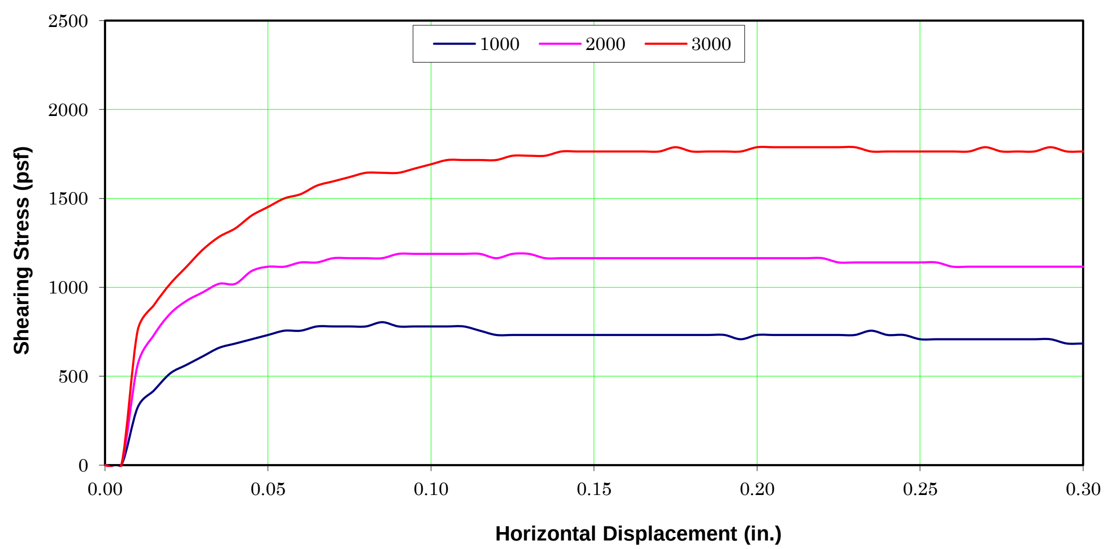
Maximum Density: **92 pcf**
Optimum Moisture: **19.5%**

Corrected for Oversize (ASTM D4718)

Sieve Size	% Retained
3/4"	0.0
3/8"	0.0
#4	10.4







DIRECT SHEAR DATA*

Sample Location: B 2 @ 0-5'
Sample Description: Sandy Clay
Dry Density (pcf): 82.4
Intial % Moisture: 18.9
Average Degree of Saturation: 100.0
Shear Rate (in/min): 0.005 in/min

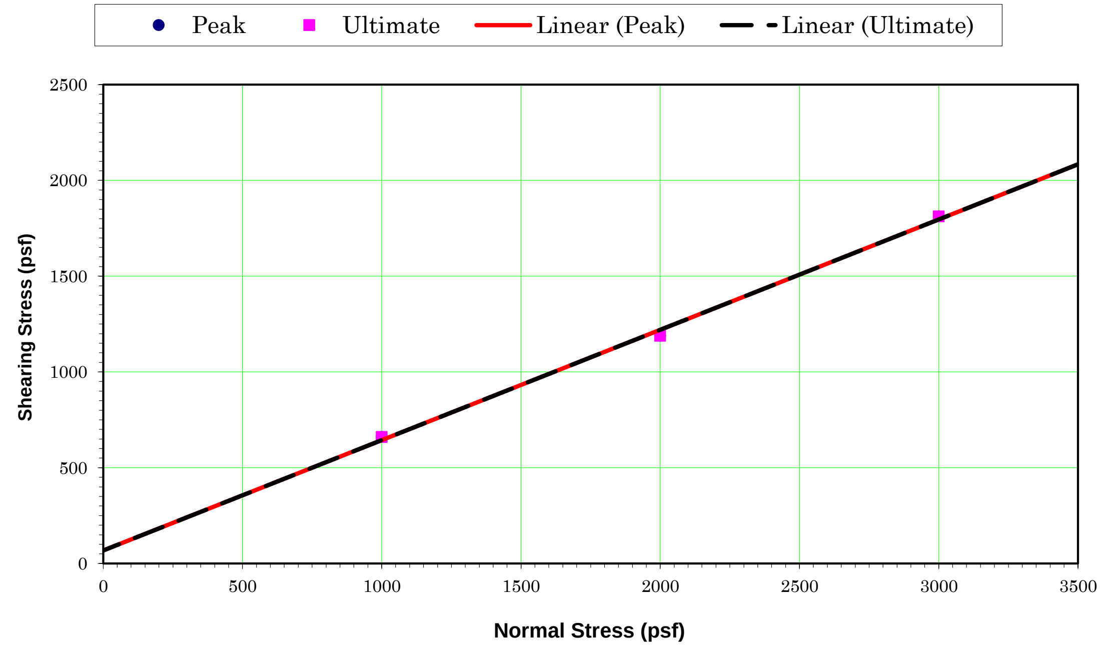
Normal stress (psf)	1000	2000	3000
Peak stress (psf)	804	1188	1788
Ultimate stress (psf)	708	1116	1764

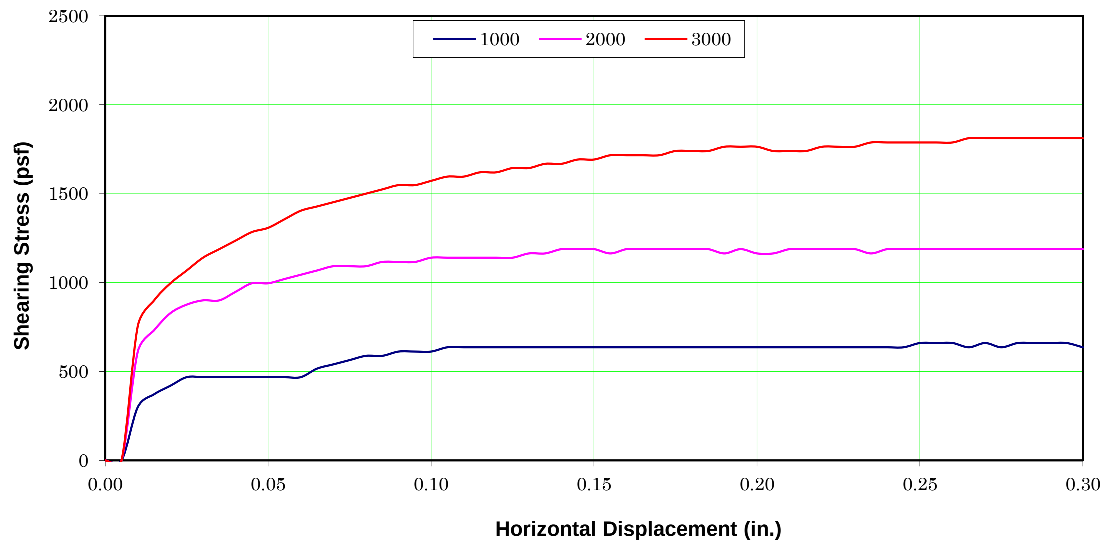
	Peak	Ultimate
ϕ Angle of Friction (degrees):	26	28
c Cohesive Strength (psf):	270	140

Test Type: Peak & Ultimate

* Test Method: ASTM D-3080

DIRECT SHEAR TEST	
Conejo Valley Church of Christ	
Earth Systems	
7/30/2018	301911-001





DIRECT SHEAR DATA*

Sample Location: T P 5 @ 5'-6'
Sample Description: Sandy Silty Clay
Dry Density (pcf): 78.9
Intial % Moisture: 20.4
Average Degree of Saturation: 100.0
Shear Rate (in/min): 0.005 in/min

Normal stress (psf)	1000	2000	3000
Peak stress (psf)	660	1188	1812
Ultimate stress (psf)	660	1188	1812

	Peak	Ultimate
ϕ Angle of Friction (degrees):	30	30
c Cohesive Strength (psf):	60	60

Test Type: Peak & Ultimate

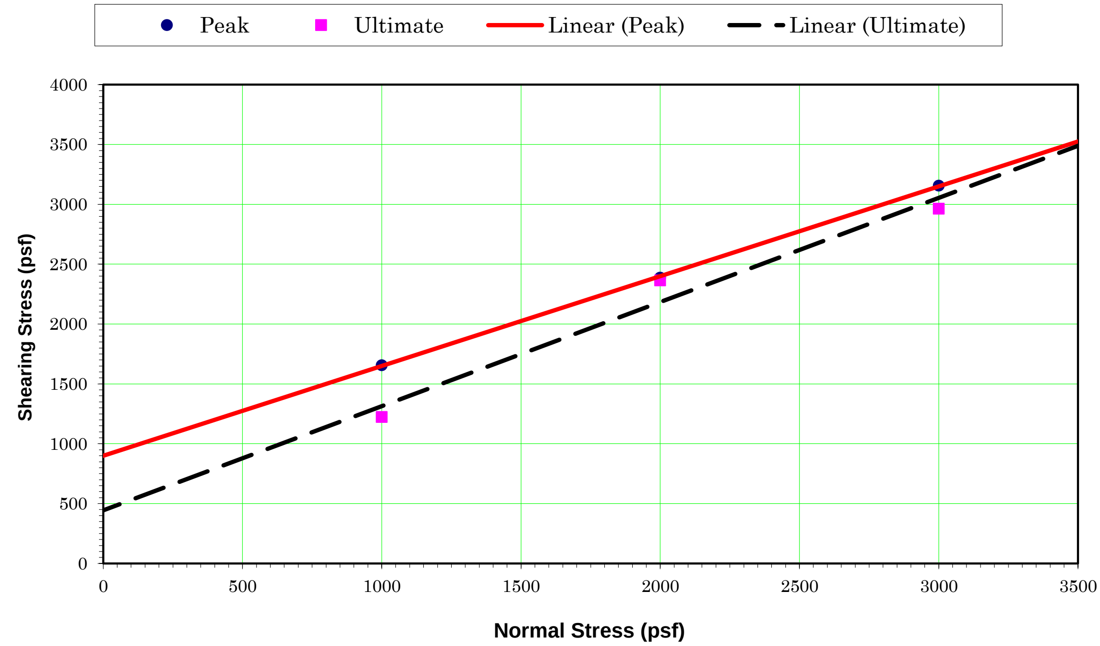
* Test Method: ASTM D-3080

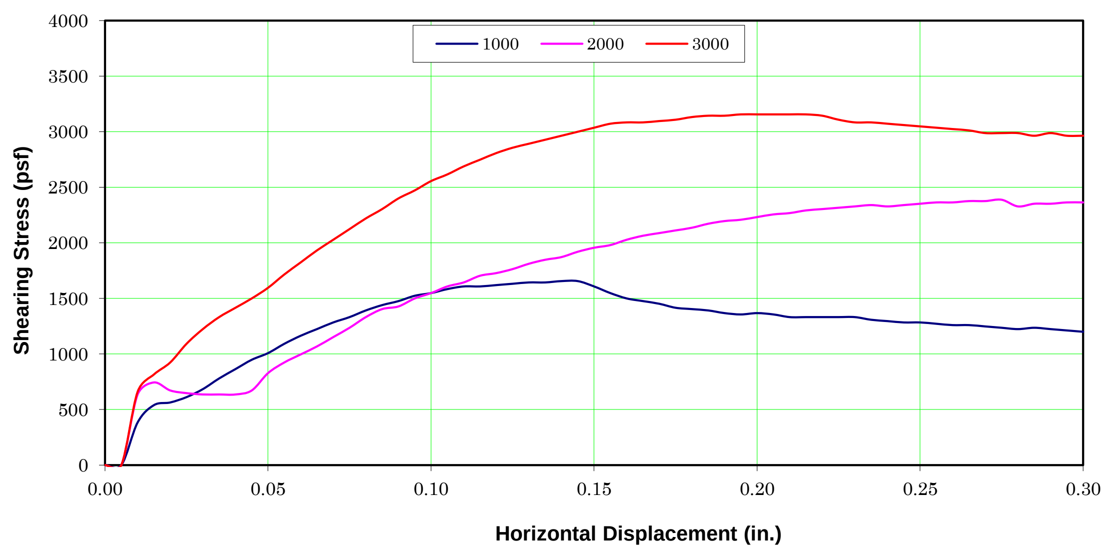
DIRECT SHEAR TEST

Conejo Valley Church of Christ

Earth Systems

8/3/2018 | 301911-001





DIRECT SHEAR DATA*

Sample Location: B 2 @ 20'
Sample Description: Sandstone
Dry Density (pcf): 79.1
Intial % Moisture: 22.6
Average Degree of Saturation: 89.1
Shear Rate (in/min): 0.005 in/min

Normal stress (psf)	1000	2000	3000
Peak stress (psf)	1656	2388	3156
Ultimate stress (psf)	1224	2364	2964

	Peak	Ultimate
ϕ Angle of Friction (degrees):	37	41
c Cohesive Strength (psf):	900	440

Test Type: Peak & Ultimate

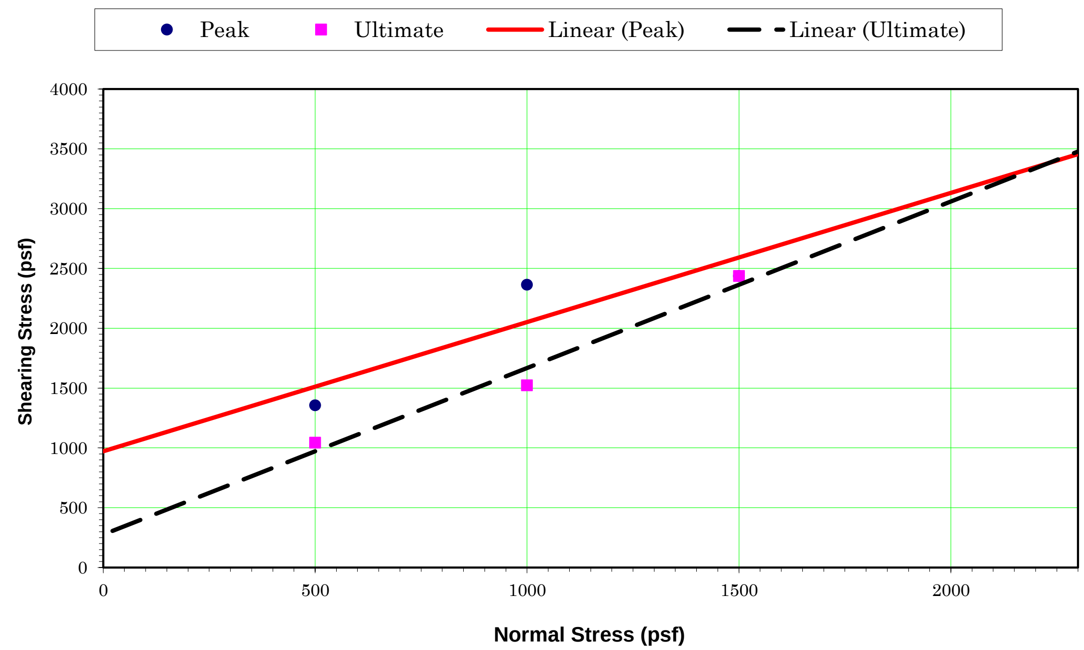
* Test Method: ASTM D-3080

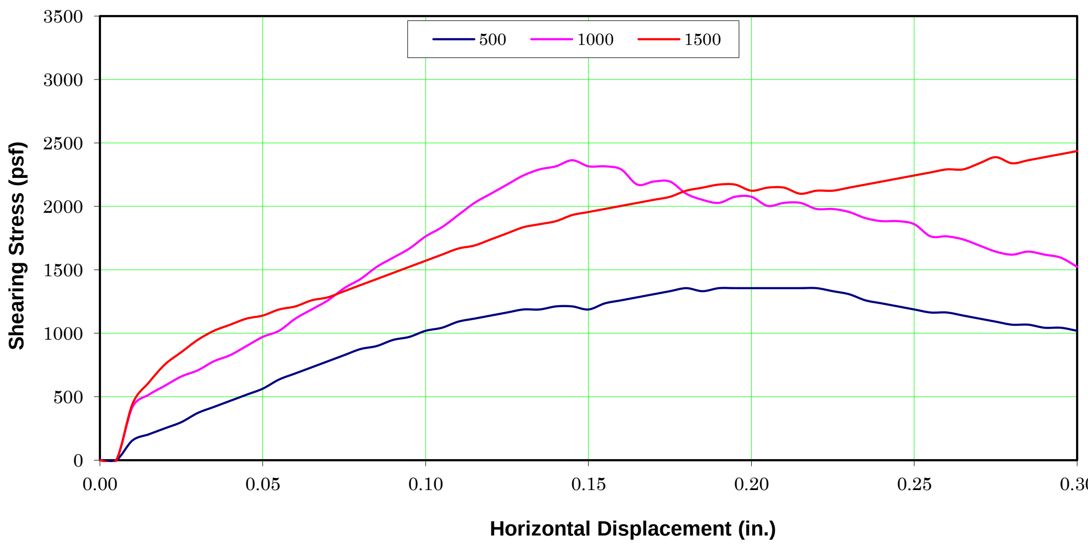
DIRECT SHEAR TEST

Conejo Valley Church of Christ

Earth Systems

7/30/2018 | 301911-001





DIRECT SHEAR DATA*

Sample Location: B 7 @ 5'
Sample Description: Shale and Siltstone
Dry Density (pcf): 76.2
Intial % Moisture: 25.5
Average Degree of Saturation: 88.7
Shear Rate (in/min): 0.005 in/min

Normal stress (psf)	500	1000	1500
Peak stress (psf)	1356	2364	2436
Ultimate stress (psf)	1044	1524	2436

	Peak	Ultimate
ϕ Angle of Friction (degrees):	47	54
c Cohesive Strength (psf):	970	270

Test Type: Peak & Ultimate

* Test Method: ASTM D-3080

DIRECT SHEAR TEST

Conejo Valley Church of Christ

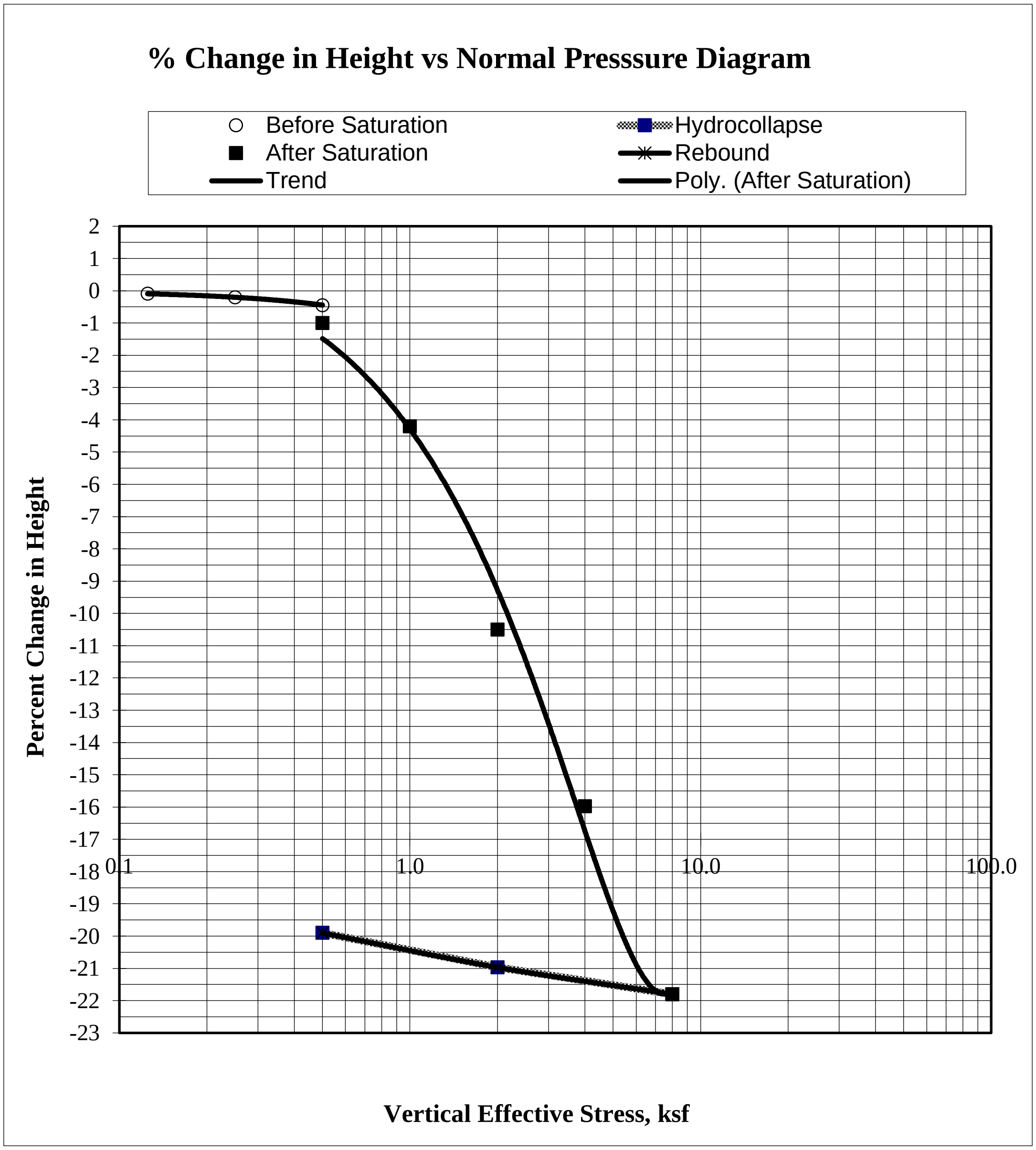
Earth Systems

7/30/2018 | 301911-001

CONSOLIDATION TEST

ASTM D 2435-90 & D5333

Conejo Valley Church of Christ	Initial Dry Density: 68.3 pcf
B 2 @ 5'	Initial Moisture, %: 24.2%
ML	Specific Gravity: 2.67 (assumed
Ring Sample	Initial Void Ratio: 1.441

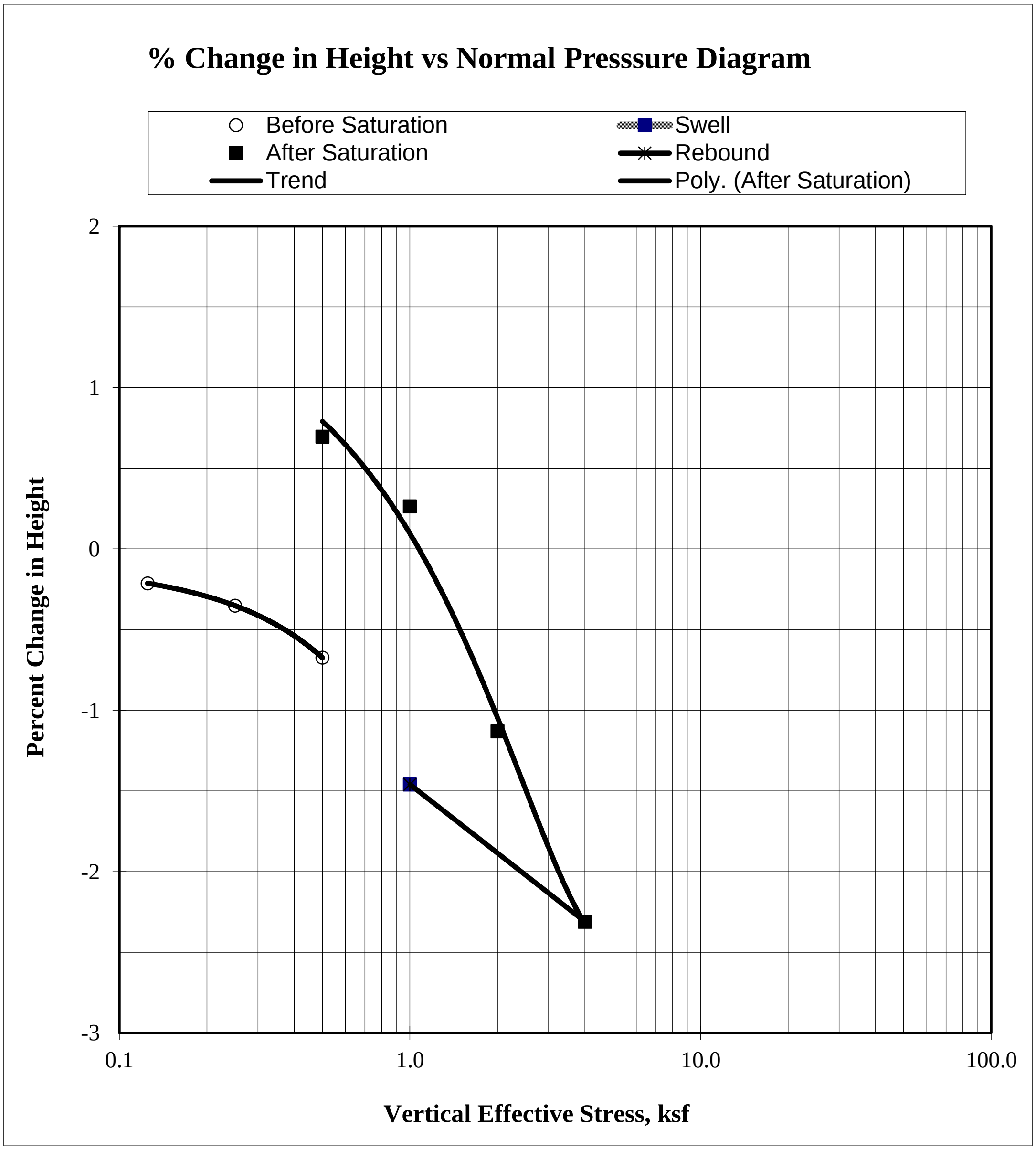


CONSOLIDATION TEST

ASTM D 2435-90

Conejo Valley Church of Christ
TP 1 @ 7'
ML
Ring Sample

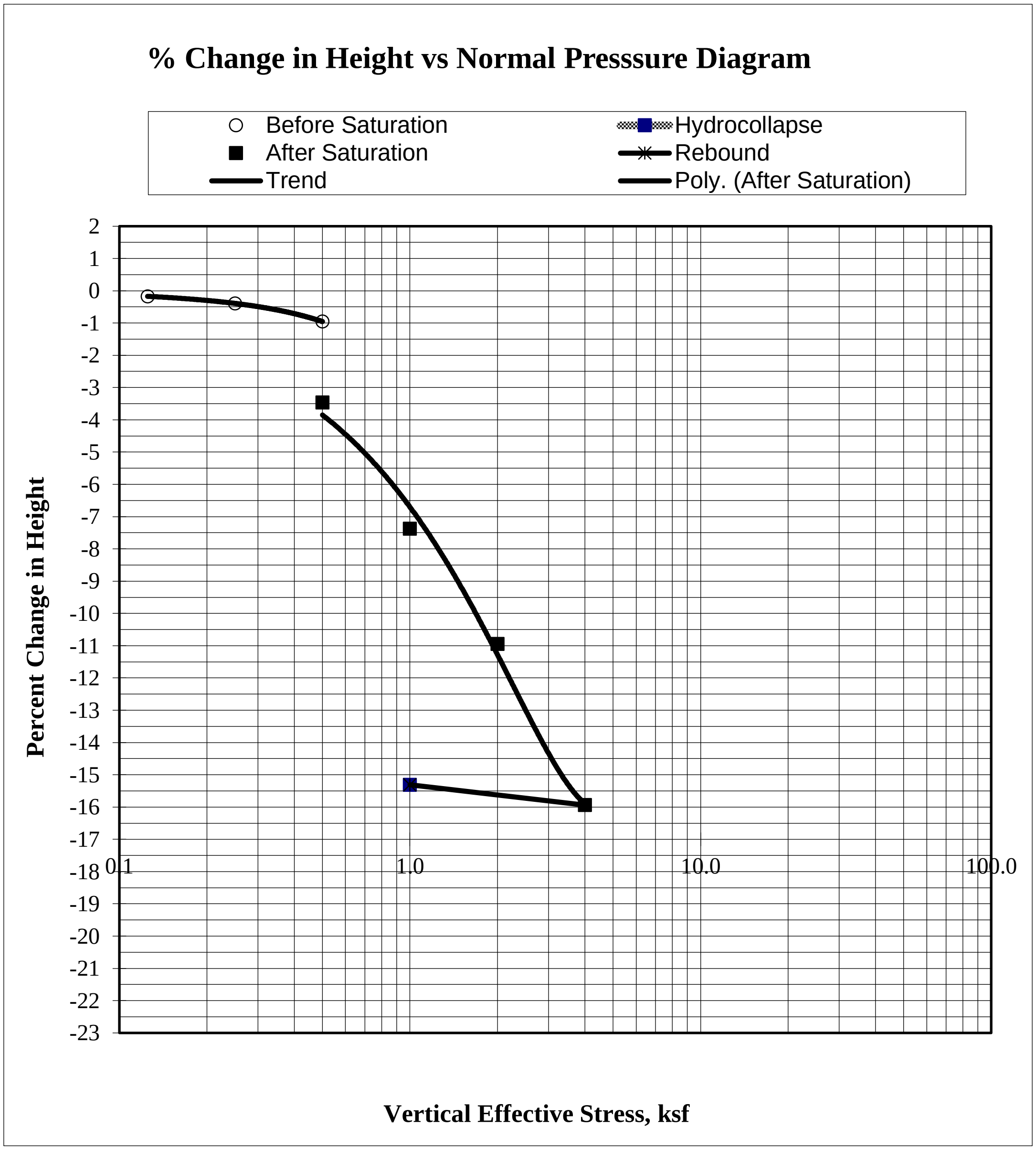
Initial Dry Density: 83.5 pcf
Initial Moisture, %: 25.3%
Specific Gravity: 2.67 (assumed
Initial Void Ratio: 0.997



CONSOLIDATION TEST

ASTM D 2435-90 & D5333

Conejo Valley Church of Christ		Initial Dry Density:	66.0 pcf
TP 8 @ 4'		Initial Moisture, %:	23.9%
ML		Specific Gravity:	2.67 (assumed
Ring Sample		Initial Void Ratio:	1.524



File No.: 301911-001

EXPANSION INDEX

ASTM D-4829, UBC 18-2

Job Name: Conejo Valley Church of Christ
Sample ID: B 2 @ 0-5'
Soil Description: CL

Initial Moisture, %:	18.9
Initial Compacted Dry Density, pcf:	83.7
Initial Saturation, %:	51
Final Moisture, %:	40.9
Volumetric Swell, %:	3.5

Expansion Index: **35** **Low**

EI	UBC Classification
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
130+	Very High

File No.: 301911-001

EXPANSION INDEX

ASTM D-4829, UBC 18-2

Job Name: Conejo Valley Church of Christ
Sample ID: T P 5 @ 5'-6'
Soil Description: CL

Initial Moisture, %:	18.5
Initial Compacted Dry Density, pcf:	83.4
Initial Saturation, %:	49
Final Moisture, %:	38.2
Volumetric Swell, %:	8.0

Expansion Index: **80** **Medium**

EI	UBC Classification
0-20	Very Low
21-50	Low
51-90	Medium
91-130	High
130+	Very High

MECHANICAL ANALYSIS

Job Name:	Conejo Valley Church of Christ
Job No.:	301911-001
Sample ID:	**B2 @ 0-5'**
Soil Description:	**CL**
Hydrometer ID:	504229

Hydroscopic Moisture

Air Dry Wt, g:	100.0
Oven Dry Wt, g	100.0
% Moisture:	0.0

Air Dry Sample Wt., g:	685
Corrected Wt., g:	685.0

Sieve Analysis for +#10 Material

Sieve Size	Wt Ret	% Ret	% Passing
1/2 inch	0.0	0.00	100.00
3/8 inch	0.0	0.00	100.00
#4	99.4	14.51	85.49
#8	140.2	20.47	79.53
#10	145.6	21.26	78.74

Air Dry Hydro Sample Wt., g:	65
Corrected Wt., g:	65.0
Calculation Factor	0.8255

Hydrometer Analysis for <#10 Material

Start time: 1:03:00 AM

Short Hydro	Time of Reading	Hydro Reading	Temp. at Reading, °C	Correction Factor	Corrected Hydro Reading
20 sec	1:03:20 AM	50	23	4.6	45.4
1 hour	2:03:00 AM	32	23	4.6	27.4
6 hour	7:03:00 AM	25	23	4.6	20.4

% Gravel:	14.5
% Sand(2mm - 74μm):	30.5
% Silt(74μm- 5μm):	21.8
% Clay(5μm - 2μm):	8.5
% Clay(≤2μm):	24.7

MECHANICAL ANALYSIS

CTM 203-08

Job Name:	Conejo Valley Church of Christ
Job No.:	301911-001
Sample ID:	**T P 5 @ 5'-6'**
Soil Description:	**CL**
Hydrometer ID:	504229

Hydroscopic Moisture

Air Dry Wt, g:	100.0
Oven Dry Wt, g	100.0
% Moisture:	0.0

Air Dry Sample Wt., g:	770.3
Corrected Wt., g:	770.3

Sieve Analysis for +#10 Material

Sieve Size	Wt Ret	% Ret	% Passing
1/2 inch	0.0	0.00	100.00
3/8 inch	0.0	0.00	100.00
#4	72.6	9.42	90.58
#8	74.5	9.67	90.33
#10	86.4	11.22	88.78

Air Dry Hydro Sample Wt., g:	74.6
Corrected Wt., g:	74.6
Calculation Factor	0.8403

Hydrometer Analysis for <#10 Material

Start time: 1:18:00 AM

Short Hydro	Time of Reading	Hydro Reading	Temp. at Reading, °C	Correction Factor	Corrected Hydro Reading
20 sec	1:18:20 AM	63	23	4.6	58.4
1 hour	2:18:00 AM	39	23	4.6	34.4
6 hour	7:18:00 AM	31	23	4.6	26.4

% Gravel:	9.4
% Sand(2mm - 74µm):	21.1
% Silt(74µm- 5µm):	28.6
% Clay(5µm - 2µm):	9.5
% Clay(≤2µm):	31.4

CAPCO Analytical Services, Inc.

Environmental and Analytical Services-Since 1994
California State Accredited Laboratory in Accordance with ELAP Certificate # 2332

Prepared for: Earth Systems Pacific
1731 A Walter Street
Ventura, CA 93003
Attn:

Report Date: June 7, 2018
Laboratory Number: 181003
Project Name: Conejo Valley Church of Christ
Project No: 301911-001
Sampled by: Client

Enclosed are the analysis results for samples received May 30, 2018 with the Chain of Custody document. The samples were received in good condition, at 24.5°C, and they were identified and assigned the laboratory ID numbers listed below:

SAMPLE DESCRIPTION	CAS LAB NUMBER ID
B2@0-5'	181003-01
TP5@5'-6'	181003-02

By my signature below, I certify that the results contained in this laboratory report comply with applicable standards for certification by the California Department of Public Health's Environmental Laboratories Accreditation Program (ELAP), both technically and for completeness, and that, based on my inquiry of the person or persons directly responsible for performing the analyses, the information submitted is, to the best of my knowledge and belief, true, accurate, and complete.

Lance Lewy-Laboratory Director

If you have any further questions or concerns, please contact me at your convenience.
This report consists of 3 pages excluding the cover letter and the Chain of Custody.

CERTIFICATE OF ANALYSIS

Client: Earth Systems Pacific
CAS LAB NO: 181003-01
Sample ID: B2@0-5'
Analyst: GP

Date Sampled: 05/29/18
Date Received: 05/30/18
Sample Matrix: Soil

WET CHEMISTRY ANALYSIS SUMMARY

COMPOUND	RESULTS	UNITS	DF	PQL	METHOD	ANALYZED
pH (Corrosivity)	8.1	S.U.	1	---	9045	06/01/18
Resistivity*	2000	Ohms-cm	1	---	SM 120.1M	06/01/18
Chloride	35	mg/Kg	1	0.6	300.0M	06/01/18
Sulfate	520	mg/Kg	1	0.6	300.0M	06/01/18

*Sample was extracted using a 1:3 ratio of soil and DI water.

DF: Dilution Factor
PQL: Practical Quantitation Limit
BQL: Below Quantitation Limit
mg/Kg: Milligrams/Kilograms(ppm)

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California State Accredited Laboratory in Accordance with ELAP Certificate # 2332

CERTIFICATE OF ANALYSIS

Client: Earth Systems Pacific
CAS LAB NO: 181003-03
Sample ID: TP5@5'-6'
Analyst: GP

Date Sampled: 05/29/18
Date Received: 05/30/18
Sample Matrix: Soil

WET CHEMISTRY ANALYSIS SUMMARY

COMPOUND	RESULTS	UNITS	DF	PQL	METHOD	ANALYZED
pH (Corrosivity)	8.5	S.U.	1	---	9045	06/01/18
Resistivity*	11000	Ohms-cm	1	---	SM 120.1M	06/01/18
Chloride	1.8	mg/Kg	1	0.6	300.0M	06/01/18
Sulfate	7.7	mg/Kg	1	0.6	300.0M	06/01/18

*Sample was extracted using a 1:3 ratio of soil and DI water.

DF: Dilution Factor
PQL: Practical Quantitation Limit
BQL: Below Quantitation Limit
mg/Kg: Milligrams/Kilograms(ppm)

APPENDIX C

Table 1809.7(1) - Minimum Footing and Slab Requirements

CITY OF THOUSAND OAKS

Table 1809.7(1)

MINIMUM FOOTING AND SLAB REQUIREMENTS [a]

Expansion Index	Foundations for slab and Raised Floor Systems							Concrete Slabs 3 1/2' Minimum Thickness 4” over 51 E1		Premoistening of Soils under footings, piers and slabs	Restrictions on piers under raised floors
	Number of Stories	Stem Thickness	Footing Width	Footing Thickness	All perimeter footings	Interior footings for slab & raised floors [b]	Reinforcement for continuous foundations	Reinforcement [c]	Total thickness of Sand		
					Depth Below Natural Surface of Ground and Finish Grade						
0-20 Very low (non-expansive)	1 2 3	6” 8” 10”	12” 15” 18”	6” 7” 8”	12” 18” 24”	12” 18” 24”	1-#4 top and bottom	#4 @ 48” or #3 @36”	2”	Moistening ground recommended prior to placing concrete	Piers allowed for single floor loads only
21-50 Low	1 2 3	6” 8” 10”	12” 15” 18”	6” 7” 8”	15” 18” 24”	12” 18” 24”	1-#4 top and bottom		2”	3% over optimum moisture required to a depth of 18” below lowest adjacent grade. Testing required	Piers allowed for single floor loads only
51-90 Medium	1 2 3	6” 8” 10”	12” 12” 15”	6” 8” 8”	21” 21” 24”	12” 18” 24”	1-#4 top and bottom Footnote [e]	#3 @24” each way	4”	3% over optimum moisture required to a depth of 18” below lowest adjacent grade. Testing required.	Piers not allowed
91-130 High	1 2 3	6” 8” 10”	12” 12” 15”	8” 8” 8”	27” 27” 27”	12” 18” 24”	2-#4 top and bottom Footnote [e]	#3 @24” each way	6”	3% over optimum moisture required to a depth of 24” below lowest adjacent grade. Testing required.	Piers not allowed
Above 130 Very high	Special Design by registered design professional [d]										

Table 1809.7(1) Continued

a. Slabs for masonry fireplaces shall be reinforced with No. 4 deformed bars at twenty-four (24") inches on center both ways.

b. Interior footings on soils in the zero (0) index to twenty (20) index range of expansiveness need not be continuous.

c. All slab reinforcement shall be positioned above the center of the slab.

d. All foundations on soils that fall into an expansive index in excess of one hundred-thirty (130) shall have a special design by a foundation engineer registered as a soil or civil engineer in California, and such design shall not be less than the minimum standards specified in Table 1809.7(1) for soils with an expansion index between ninety-one (91) and one hundred thirty (130).

e. Exterior footing to slab L shaped dowel bars #3 bars @ 24" o.c., 12" into footings, 36" into slab required.

APPENDIX D

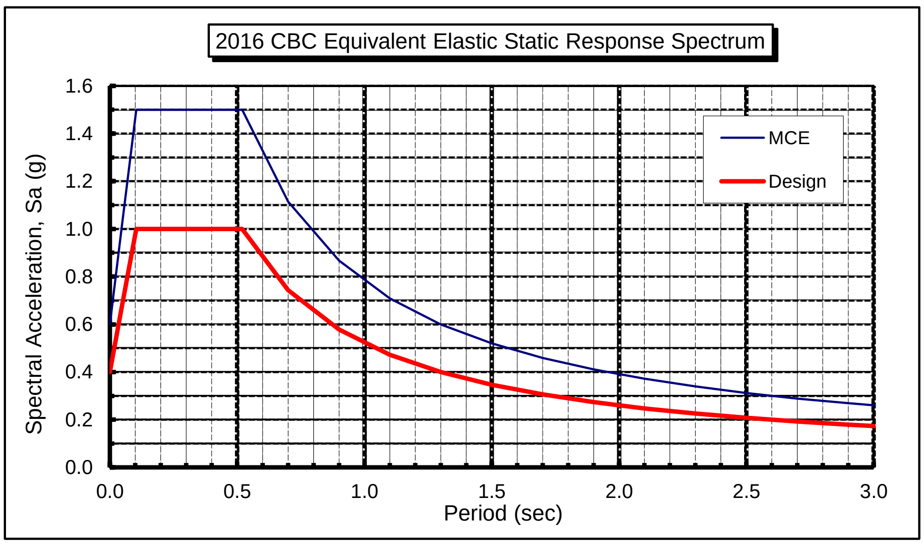
2016 CBC & ASCE 7-10 Seismic Parameters

USGS Design Maps Reports

Fault Parameters

2016 California Building Code (CBC) (ASCE 7-10) Seismic Design Parameters

				CBC Reference	ASCE 7-10 Reference
Seismic Design Category		**D**		Table 1613.5.6	Table 11.6-1
Site Class		**C**		Table 1613.5.2	Table 20.3-1
Latitude:		34.176	N		
Longitude:		-118.838	W		
Maximum Considered Earthquake (MCE) Ground Motion					
Short Period Spectral Reponse	S_S	**1.500**	**g**	Figure 1613.5	Figure 22-3
1 second Spectral Response	S_1	**0.600**	**g**	Figure 1613.5	Figure 22.4
Site Coefficient	F_a	1.00		Table 1613.5.3(1)	Table 11.4-1
Site Coefficient	F_v	1.30		Table 1613.5.3(2)	Table 11-4.2
	S_{MS}	1.500	g	$= F_a{*}S_S$	
	S_{M1}	0.780	g	$= F_v{*}S_1$	
Design Earthquake Ground Motion					
Short Period Spectral Reponse	S_{DS}	**1.000**	**g**	$= 2/3{*}S_{MS}$	
1 second Spectral Response	S_{D1}	**0.520**	**g**	$= 2/3{*}S_{M1}$	
	To	0.10	sec	$= 0.2{*}S_{D1}/S_{DS}$	
	Ts	0.52	sec	$= S_{D1}/S_{DS}$	
Seismic Importance Factor	I	1.00		Table 1604.5	Table 11.5-1
	F_{PGA}	1.00			

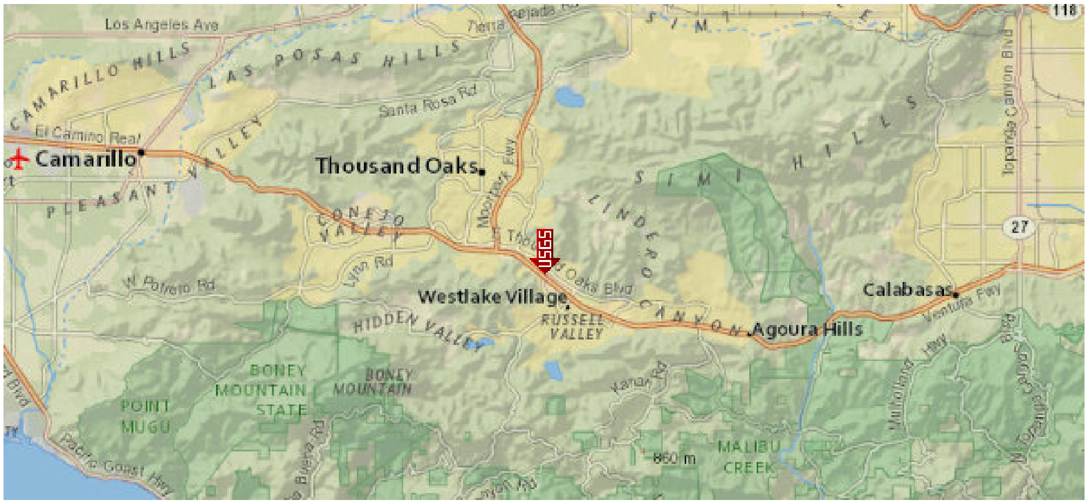


Period T (sec)	Design Sa (g)
0.00	0.400
0.05	0.688
0.10	1.000
0.52	1.000
0.70	0.743
0.90	0.578
1.10	0.473
1.30	0.400
1.50	0.347
1.70	0.306
1.90	0.274
2.10	0.248
2.30	0.226
2.50	0.208
2.70	0.193
2.90	0.179

USGS Design Maps Summary Report

User–Specified Input

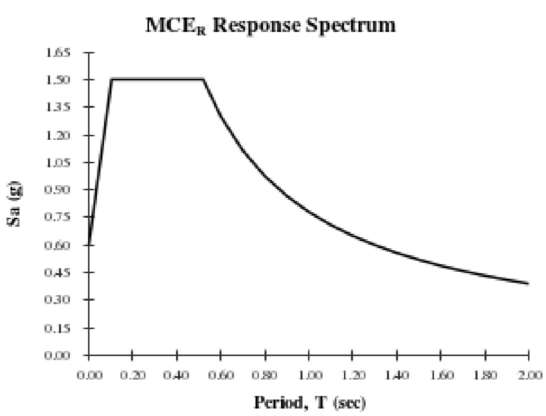
Report Title	Conejo Valley Church of Christ Mon August 13, 2018 16:47:01 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	34.1762°N, 118.8378°W
Site Soil Classification	Site Class C – "Very Dense Soil and Soft Rock"
Risk Category	I/II/III

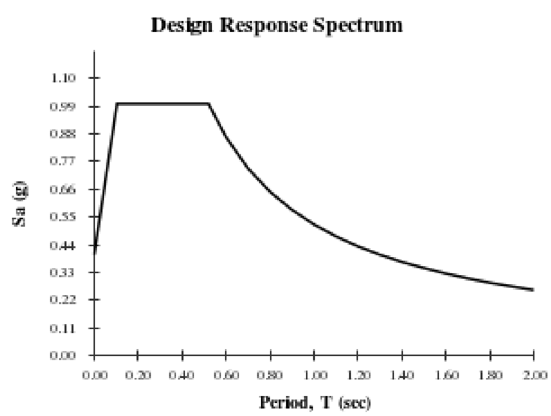


USGS–Provided Output

$S_S =$ 1.500 g	$S_{MS} =$ 1.500 g	$S_{DS} =$ 1.000 g
$S_1 =$ 0.600 g	$S_{M1} =$ 0.780 g	$S_{D1} =$ 0.520 g

For information on how the SS and S1 values above have been calculated from probabilistic (risk-targeted) and deterministic ground motions in the direction of maximum horizontal response, please return to the application and select the "2009 NEHRP" building code reference document.





For PGA_M, T_L, C_{RS}, and C_{R1} values, please view the detailed report.

USGS Design Maps Detailed Report

ASCE 7-10 Standard (34.1762°N, 118.8378°W)

Site Class C – "Very Dense Soil and Soft Rock", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

Note: Ground motion values provided below are for the direction of maximum horizontal spectral response acceleration. They have been converted from corresponding geometric mean ground motions computed by the USGS by applying factors of 1.1 (to obtain S_S) and 1.3 (to obtain S_1). Maps in the 2010 ASCE-7 Standard are provided for Site Class B. Adjustments for other Site Classes are made, as needed, in Section 11.4.3.

From Figure 22-1 [1]	S_S = 1.500 g
From Figure 22-2 [2]	S_1 = 0.600 g

Section 11.4.2 — Site Class

The authority having jurisdiction (not the USGS), site-specific geotechnical data, and/or the default has classified the site as Site Class C, based on the site soil properties in accordance with Chapter 20.

Table 20.3–1 Site Classification

Site Class	$\bar{v}_s$	$\bar{N}$ or $\bar{N}_{ch}$	$\bar{s}_u$
A. Hard Rock	>5,000 ft/s	N/A	N/A
B. Rock	2,500 to 5,000 ft/s	N/A	N/A
C. Very dense soil and soft rock	1,200 to 2,500 ft/s	>50	>2,000 psf
D. Stiff Soil	600 to 1,200 ft/s	15 to 50	1,000 to 2,000 psf
E. Soft clay soil	<600 ft/s	<15	<1,000 psf
	Any profile with more than 10 ft of soil having the characteristics: • Plasticity index *PI* > 20, • Moisture content *w* ≥ 40%, and • Undrained shear strength $\bar{s}_u$ < 500 psf		
F. Soils requiring site response analysis in accordance with Section 21.1	See Section 20.3.1		

For SI: 1ft/s = 0.3048 m/s 1lb/ft² = 0.0479 kN/m²

Section 11.4.3 — Site Coefficients and Risk–Targeted Maximum Considered Earthquake (MCE_R) Spectral Response Acceleration Parameters

Table 11.4–1: Site Coefficient F_a

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at Short Period				
	$S_S \le 0.25$	$S_S = 0.50$	$S_S = 0.75$	$S_S = 1.00$	$S_S \ge 1.25$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	$\boxed{1.0}$
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_S

For Site Class = C and S_S = 1.500 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

Site Class	Mapped MCE $_R$ Spectral Response Acceleration Parameter at 1–s Period				
	$S_1 \le 0.10$	$S_1 = 0.20$	$S_1 = 0.30$	$S_1 = 0.40$	$S_1 \ge 0.50$
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.7	1.6	1.5	1.4	$\boxed{1.3}$
D	2.4	2.0	1.8	1.6	1.5
E	3.5	3.2	2.8	2.4	2.4
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of S_1

For Site Class = C and S_1 = 0.600 g, F_v = 1.300

Equation (11.4–1):	$S_{MS} = F_aS_S = 1.000 \times 1.500 = 1.500$ g
Equation (11.4–2):	$S_{M1} = F_vS_1 = 1.300 \times 0.600 = 0.780$ g

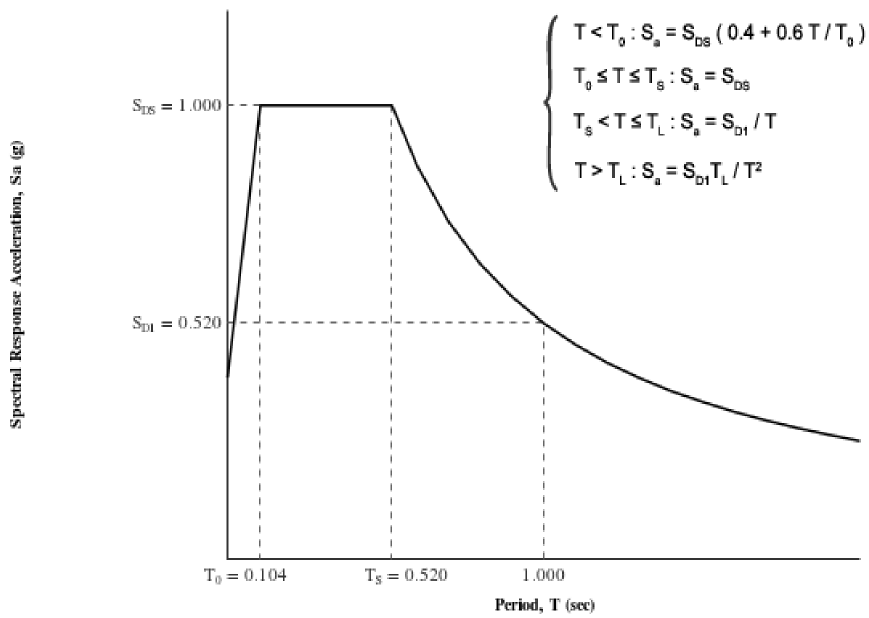
Section 11.4.4 — Design Spectral Acceleration Parameters

Equation (11.4–3):	$S_{DS} = ⅔ S_{MS} = ⅔ \times 1.500 = 1.000$ g
Equation (11.4–4):	$S_{D1} = ⅔ S_{M1} = ⅔ \times 0.780 = 0.520$ g

Section 11.4.5 — Design Response Spectrum

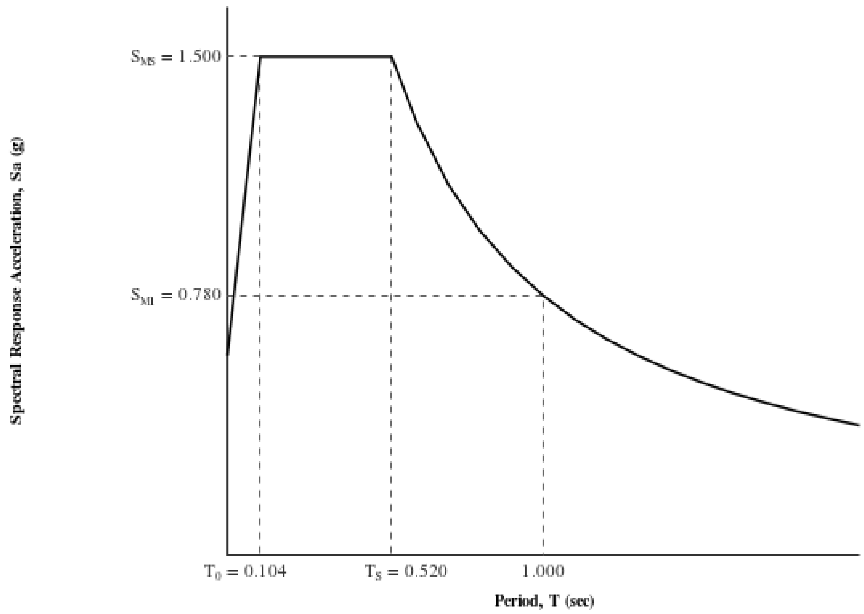
From Figure 22-12 [3]	T_L = 8 seconds

Figure 11.4–1: Design Response Spectrum



Section 11.4.6 — Risk-Targeted Maximum Considered Earthquake (MCE_R) Response Spectrum

The MCE_R Response Spectrum is determined by multiplying the design response spectrum above by 1.5.



Section 11.8.3 — Additional Geotechnical Investigation Report Requirements for Seismic Design Categories D through F

From Figure 22-7 [4] PGA = 0.500

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.500 = 0.5$ g

Table 11.8–1: Site Coefficient F_{PGA}

Site Class	Mapped MCE Geometric Mean Peak Ground Acceleration, PGA				
	PGA ≤ 0.10	PGA = 0.20	PGA = 0.30	PGA = 0.40	PGA ≥ 0.50
A	0.8	0.8	0.8	0.8	0.8
B	1.0	1.0	1.0	1.0	1.0
C	1.2	1.2	1.1	1.0	$\boxed{1.0}$
D	1.6	1.4	1.2	1.1	1.0
E	2.5	1.7	1.2	0.9	0.9
F	See Section 11.4.7 of ASCE 7				

Note: Use straight–line interpolation for intermediate values of PGA

For Site Class = C and PGA = 0.500 g, F_{PGA} = 1.000

Section 21.2.1.1 — Method 1 (from Chapter 21 – Site-Specific Ground Motion Procedures for Seismic Design)

From Figure 22-17 [5] C_{RS} = 1.033

From Figure 22-18 [6] C_{R1} = 1.042

Section 11.6 — Seismic Design Category

Table 11.6-1 Seismic Design Category Based on Short Period Response Acceleration Parameter

VALUE OF S_{DS}	RISK CATEGORY		
	I or II	III	IV
$S_{DS} < 0.167g$	A	A	A
$0.167g \le S_{DS} < 0.33g$	B	B	C
$0.33g \le S_{DS} < 0.50g$	C	C	D
$0.50g \le S_{DS}$	D	D	D

For Risk Category = I and S_{DS} = 1.000 g, Seismic Design Category = D

Table 11.6-2 Seismic Design Category Based on 1-S Period Response Acceleration Parameter

VALUE OF S_{D1}	RISK CATEGORY		
	I or II	III	IV
$S_{D1} < 0.067g$	A	A	A
$0.067g \le S_{D1} < 0.133g$	B	B	C
$0.133g \le S_{D1} < 0.20g$	C	C	D
$0.20g \le S_{D1}$	D	D	D

For Risk Category = I and S_{D1} = 0.520 g, Seismic Design Category = D

Note: When S_1 is greater than or equal to 0.75g, the Seismic Design Category is **E** for buildings in Risk Categories I, II, and III, and **F** for those in Risk Category IV, irrespective of the above.

Seismic Design Category ≡ “the more severe design category in accordance with Table 11.6-1 or 11.6-2” = D

Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

1. *Figure 22-1*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf
4. *Figure 22-7*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-7.pdf
5. *Figure 22-17*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-17.pdf
6. *Figure 22-18*: https://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-18.pdf

Table 1
Fault Parameters

Fault Section Name	**Distance** (miles)	(km)	Avg Dip Angle (deg.)	Avg Dip Direction (deg.)	Avg Rake (deg.)	Trace Length (km)	Fault Type	**Mean Mag**	Mean Return Interval (years)	Slip Rate (mm/yr)
Simi-Santa Rosa	5.9	9.5	60	346	30	39	B	**6.8**		1
Malibu Coast, alt 1	9.6	15.4	75	3	30	38	B	**6.6**		0.3
Malibu Coast, alt 2	9.6	15.4	74	3	30	38	B	**6.9**		0.3
Malibu Coast (Extension), alt 1	10.4	16.7	74	4	30	35	B'	**6.5**		
Malibu Coast (Extension), alt 2	10.4	16.7	74	4	30	35	B'	**6.9**		
Northridge Hills	11.4	18.4	31	19	90	25	B'	**7.0**		
Santa Susana, alt 2	13.0	20.9	53	10	90	43	B'	**6.8**		
Santa Susana, alt 1	13.3	21.4	55	9	90	27	B	**6.8**		5
Oak Ridge (Onshore)	14.3	23.0	65	159	90	49	B	**7.2**		4
Anacapa-Dume, alt 1	14.9	24.0	45	354	60	51	B	**7.2**		3
Anacapa-Dume, alt 2	14.9	24.0	41	352	60	65	B	**7.2**		3
San Cayetano	15.7	25.2	42	3	90	42	B	**7.2**		6
San Pedro Basin	16.4	26.4	88	51	na	69	B'	**7.0**		
Compton	17.2	27.6	20	34	90	65	B'	**7.5**		
Northridge	18.1	29.2	35	201	90	33	B	**6.8**		1.5
Del Valle	18.2	29.3	73	195	90	9	B'	**6.3**		
Holser, alt 1	18.7	30.2	58	187	90	20	B	**6.7**		0.4
Holser, alt 2	18.7	30.2	58	182	90	17	B'	**6.7**		
Santa Monica, alt 1	19.0	30.6	75	343	30	14	B	**6.5**		1
Santa Monica, alt 2	19.6	31.5	50	338	30	28	B	**6.7**		1
Ventura-Pitas Point	20.8	33.5	64	353	60	44	B	**6.9**		1
Santa Monica Bay	21.4	34.4	20	44	na	17	B'	**7.0**		
Palos Verdes	21.5	34.5	90	53	180	99	B	**7.3**		3
San Pedro Escarpment	22.1	35.6	17	38	na	27	B'	**7.3**		
Sierra Madre (San Fernando)	22.3	35.9	45	9	90	18	B	**6.6**		2
Sisar	22.5	36.2	29	168	na	20	B'	**7.0**		
Shelf (Projection)	23.0	37.1	17	21	na	70	B'	**7.8**		
San Gabriel	23.9	38.5	61	39	180	71	B	**7.3**		1
Verdugo	24.5	39.5	55	31	90	29	B	**6.8**		0.5
Oak Ridge (Offshore)	25.4	40.9	32	180	90	38	B	**6.9**		3
Hollywood	25.5	41.0	70	346	30	17	B	**6.6**		1
San Vicente	25.9	41.8	66	7	na	9	B'	**6.3**		
Channel Islands Thrust	26.4	42.5	20	354	90	59	B	**7.3**		1.5
Redondo Canyon, alt 2	26.6	42.8	80	187	na	25	B'	**6.6**		
Newport-Inglewood, alt 1	27.2	43.8	88	49	180	65	B	**7.2**		1
Newport-Inglewood, alt 2	27.2	43.8	90	49	180	66	B	**7.2**		1
Santa Cruz Island	27.6	44.4	90	188	30	69	B	**7.1**		1
Mission Ridge-Arroyo Parida-Santa Ana	27.6	44.4	70	176	90	69	B	**6.8**		0.4
Santa Ynez (East)	28.0	45.0	70	172	0	68	B	**7.2**		2
North Salt Lake	28.5	45.9	54	343	na	3	B'	**5.9**		

Reference: USGS OFR 2007-1437 (CGS SP 203)

Based on Site Coordinates of 34.1762 Latitude, -118.8378 Longitude

Mean Magnitude for Type A Faults based on 0.1 weight for unsegmented section, 0.9 weight for segmented model (weighted by probability of each scenario with section listed as given on Table 3 of Appendix G in OFR 2007-1437). Mean magntude is average of Ellworths-B and Hanks & Bakun moment area relationship.

APPENDIX E

Dry Sand Settlement Calculation Results

LIQUEFY-v 2.3.XLS - A SPREADSHEET FOR EMPIRICAL ANALYSIS OF LIQUEFACTION POTENTIAL AND INDUCED GROUND SUBSIDENCE

Developed 2006 by Shelton L. Stringer, PE, GE, PG - Earth Systems Southwest

Project: Conejo Valley Church of Christ
Job No: 301911-001
Date: 8/9/2018
Boring: B-1 Data Set: 1

Methods: **Liquefaction Analysis using 1996 & 1998 NCEER workshop method (Youd & Idriss, editors)**
Journal of Geotechnical and Enviromental Engineering (JGEE), October 2001, Vol 127, No. 10, ASCE
Settlement Analysis from Tokimatsu and Seed (1987), JGEE,Vol 113, No.8, ASCE
Modified by Pradel, JGEE, Vol 124, No. 4, ASCE

EARTHQUAKE INFORMATION:

Magnitude: 7.2 7.5
PGA, g: 0.50 0.45
MSF: 1.11
GWT: 25.5 feet
Calc GWT: 25.5 feet
Remediate to: 0.0 **feet**

SPT N VALUE CORRECTIONS:

Energy Correction to N60 (C_E): 1.33 Automatic Hammer
Drive Rod Corr. (C_R): 1 Default
Rod Length above ground (feet): 3.0
Borehole Dia. Corr. (C_B): 1.00
Sampler Liner Correction for SPT?: 1 Yes
Cal Mod/ SPT Ratio: 0.63

Threshold Acceler., g: 1.80

Total (ft) Liquefied Thickness: 0

Total (in.) Induced Subsidence: 0.0

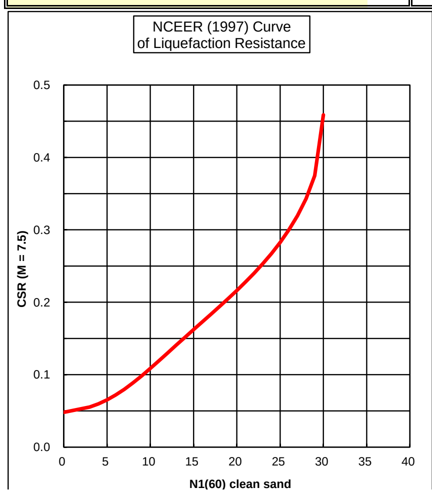
Required SF: 1.25
Minimum Calculated SF: 3.60

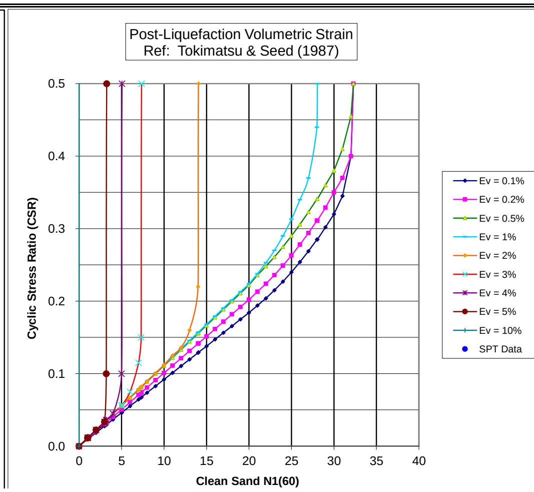
SETTLEMENT (SUBSIDENCE) OF DRY SANDS

Nc = 12.5

Base Depth (feet)	Cal Mod N	SPT N	Liquef. Suscept. (0 or 1)	Total Unit Wt. (pcf)	Fines Content (%)	Depth of SPT (feet)	Rod Length (feet)	Tot.Stress at SPT po (tsf)	Eff.Stress at SPT p'o (tsf)	rd	C_N	C_R	C_S	$N_{1(60)}$	Rel. Dens. Dr (%)	Trigger FC Adj. $\Delta N_{1(60)}$	Equiv. Sand $N_{1(60)CS}$	Kσ	M = 7.5 Available CRR	M =7.5 Induced CSR*	Liquefac. Safety Factor	Post FC Adj. $\Delta N_{1(60)}$	$N_{1(60)CS}$	Volumetric Strain (%)	Induced Subsidence (in.)
0.0				0				0.000																	
3.5	19	**12**	1	84	50	2.5	5.5	0.105	0.105	1.00	1.70	0.75	1.00	20.3	54	9.1	29.4	1.00	0.395	0.292	**Non-Liq.**	9.1	29.4	0.01	**0.01**
6.5	21	**13**	1	88	50	5.5	8.5	0.235	0.235	0.99	1.70	0.75	1.00	22.5	57	9.5	32.0	1.00	1.400	0.290	**Non-Liq.**	9.5	32.0	0.01	**0.01**
11.5	100	63	1	105	35	10.5	13.5	0.489	0.489	0.98	1.47	0.77	1.00	95.3	100	10.0	105.3	1.00	1.400	0.286	**Non-Liq.**	10.0	105.3	0.00	**0.00**
16.5	100	63	1	111	35	15.5	18.5	0.764	0.764	0.97	1.18	0.87	1.00	86.2	100	10.0	96.2	1.00	1.400	0.283	**Non-Liq.**	10.0	96.2	0.00	**0.00**
21.5	60	38	1	111	35	20.5	23.5	1.041	1.041	0.96	1.01	0.94	1.00	47.7	83	10.0	57.7	1.01	1.400	0.278	**Non-Liq.**	10.0	57.7	0.01	**0.01**
25.5	60	38	1	107	35	24.5	27.5	1.257	1.257	0.94	0.92	0.98	1.00	45.3	80	10.0	55.3	0.93	1.400	0.296	**Non-Liq.**	10.0	55.3	0.01	**0.01**
50.0	60	38	1	104	35	49.0	52.0	2.532	1.798	0.76	0.77	1.00	1.00	38.7	74	10.0	48.7	0.81	1.400	0.389	**3.60**	10.0	48.7	0.00	**0.00**
0.0	0	0	0	0	0	0.0																			
0.0	0	0	0	0	0	0.0																			
0.0	0	0	0	0	0	0.0																			
0.0	0	0	0	0	0	0.0																			
0.0	0	0	0	0	0	0.0																			
0.0	0	0	0	0	0	0.0																			
0.0	0	0	0	0	0	0.0																			
0.0	0	0	0	0	0	0.0																			
0.0	0	0	0	0	0	0.0																			
0.0	0	0	0	0	0	0.0																			

p (tsf)	G_{max} (tsf)	τ_{av} (tsf)	tav/Gmax	a	b	Shear Strain γ	Strain E_{15}	Strain Enc	Dry Sand Subsidence (in.)
0.070	366	0.023	0.0001	0.127	31,512	1.0E-04	6.5E-05	6.0E-05	**0.01**
0.158	563	0.050	0.0001	0.130	19,400	1.4E-04	7.8E-05	7.2E-05	**0.01**
0.327	1,208	0.104	0.0001	0.137	12,505	1.1E-04	1.4E-05	1.3E-05	**0.00**
0.512	1,466	0.160	0.0001	0.144	9,565	1.3E-04	2.0E-05	1.9E-05	**0.00**
0.698	1,443	0.216	0.0001	0.151	7,943	1.9E-04	5.4E-05	5.0E-05	**0.01**
0.842	1,563	0.257	0.0002	0.157	7,094	2.1E-04	6.3E-05	5.8E-05	**0.01**
1.696	2,125	0.418	0.0002	0.190	4,661	2.4E-04			





$N_{1(60)} = C_N{*}C_E{*}C_B{*}C_R{*}C_S{*}N$

C_R = 0.75 for Rod lengths < 3m, 1.0 for > 10m
= min(1,max(0.75,1.4666-2.556/(z(ft))$^{0.5}$))

C_N = (1 atm/p'o)$^{0.5}$, max 1.7

C_S = max(1.1,min(1.3,1+$N_{1(60)}$/100)) for SPT without liners

MSF = $10^{2.24}/M^{2.56}$

z = Depth (m)

pa = 1 atm = 101 KPa = 1.058 tsf

p = 0.67*po

τ_{av} = 0.65*PGA*po*rd

G_{max} = 447*$N_{1(60)CS}^{(1/3)}$*$p^{0.5}$

a = 0.0389*(p/1)+0.124

b = 6400*(p/1)$^{(-0.6)}$

γ = [1+a*EXP(b*τ_{av}/G_{max})]/[(1+a)*τ_{av}/G_{max}]

E_{15} = γ*($N_{1(60)CS}$/20)$^{-1.2}$

E_{nc} = (Nc/15)$^{0.45}$*E15

Nc = (MAG-4)$^{2.17}$

S = 2*H*E_{nc}

rd = (1-0.4113*z^0.5+0.04052*z+0.001753*z^1.5)/(1-0.4177*z^0.5+0.05729*z-0.006205*z^1.5+0.00121*z^2))

$\Delta N_{1(60)}$ = min(10,IF(FC<35,exp(1.76-(190/FC^2)),5)+IF(FC<=5,1,IF(FC<35,0.99+(FC^1.5/1000),1.2))*N1(60) - N1(60)

$N_{1(60)CS} = N_{1(60)CS} + \Delta N_{1(60)}$

Kσ = min of 1.0 or (p'o/1.058)$^{\text{(IF(Dr>0.7,0.6,IF(Dr<0.5,0.8,0.7))-1)}}$

Dr = ($N_{1(60)}$/70)$^{0.5}$

CSReq = 0.65*PGA*(po/p'o)*rd

CSR* = CSReq/MSF/Kσ

$CRR_{7.5}$ = (0.048-0.004721*N+0.0006136*N^2-0.00001673*N^3)/(1-0.1248*N+0.009578*N^2-0.0003285*N^3+0.000003714*N^4))

N = $N_{1(60)CS}$

SF = $CRR_{7.5,1atm}$/CSR*

LIQUEFY-v 2.3.XLS - A SPREADSHEET FOR EMPIRICAL ANALYSIS OF LIQUEFACTION POTENTIAL AND INDUCED GROUND SUBSIDENCE

Developed 2006 by Shelton L. Stringer, PE, GE, PG - Earth Systems Southwest

Project: Conejo Valley Church of Christ
Job No: 301911-001
Date: 8/9/2018
Boring: B-2 Data Set: 2

Methods: **Liquefaction Analysis using 1996 & 1998 NCEER workshop method (Youd & Idriss, editors)**
Journal of Geotechnical and Enviromental Engineering (JGEE), October 2001, Vol 127, No. 10, ASCE
Settlement Analysis from Tokimatsu and Seed (1987), JGEE,Vol 113, No.8, ASCE
Modified by Pradel, JGEE, Vol 124, No. 4, ASCE

EARTHQUAKE INFORMATION:
Magnitude: 7.2 7.5
PGA, g: 0.50 0.45
MSF: 1.11
GWT: 25.5 feet
Calc GWT: 25.5 feet
Remediate to: 0.0 **feet**

SPT N VALUE CORRECTIONS:
Energy Correction to N60 (C_E): 1.33 Automatic Hammer
Drive Rod Corr. (C_R): 1 Default
Rod Length above ground (feet): 3.0
Borehole Dia. Corr. (C_B): 1.00
Sampler Liner Correction for SPT?: 1 Yes
Cal Mod/ SPT Ratio: 0.63

Threshold Acceler., g: **1.82**
Required SF: **1.25**
Minimum Calculated SF: **3.64**

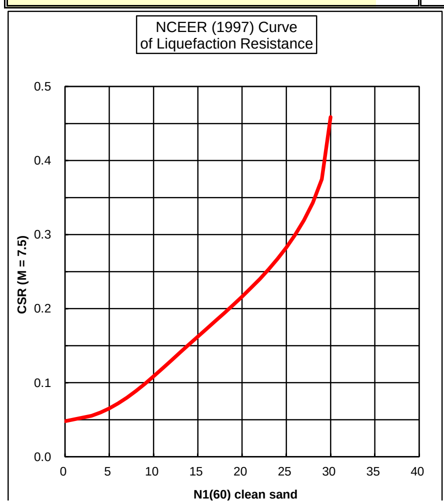
Total (ft) Liquefied Thickness	**Total (in.) Induced Subsidence**
0	**0.0**

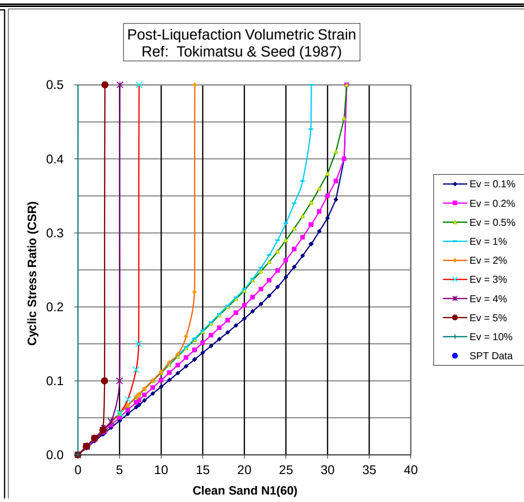
SETTLEMENT (SUBSIDENCE) OF DRY SANDS

Nc = 12.5

Base Depth (feet)	Cal Mod N	SPT N	Liquef. Suscept. (0 or 1)	Total Unit Wt. (pcf)	Fines Content (%)	Depth of SPT (feet)	Rod Length (feet)	Tot.Stress at SPT po (tsf)	Eff.Stress at SPT p'o (tsf)	rd	C_N	C_R	C_S	$N_{1(60)}$	Rel. Dens. Dr (%)	Trigger FC Adj. $\Delta N_{1(60)}$	Equiv. Sand $N_{1(60)CS}$	Kσ	M = 7.5 Available CRR	M = 7.5 Induced CSR*	Liquefac. Safety Factor
0.0				0				0.000													
3.5	13	**8**	1	85	60	2.5	5.5	0.106	0.106	1.00	1.70	0.75	1.00	13.9	45	7.8	21.7	1.00	0.236	0.292	**Non-Liq.**
6.5	12	**8**	1	85	50	5.5	8.5	0.234	0.234	0.99	1.70	0.75	1.00	12.9	43	7.6	20.4	1.00	0.221	0.290	**Non-Liq.**
11.5	19	12	1	86	35	10.5	13.5	0.449	0.449	0.98	1.54	0.77	1.00	18.9	52	8.8	27.7	1.00	0.335	0.286	**Non-Liq.**
16.5	75	47	1	90	35	15.5	18.5	0.672	0.672	0.97	1.25	0.87	1.00	69.0	99	10.0	79.0	1.00	1.400	0.283	**Non-Liq.**
21.5	60	38	1	97	35	20.5	23.5	0.911	0.911	0.96	1.08	0.94	1.00	51.0	85	10.0	61.0	1.00	1.400	0.280	**Non-Liq.**
25.5	60	38	1	100	35	24.5	27.5	1.109	1.109	0.94	0.98	0.98	1.00	48.2	83	10.0	58.2	0.98	1.400	0.282	**Non-Liq.**
50.0	60	38	1	100	35	49.0	52.0	2.334	1.601	0.76	0.81	1.00	1.00	41.0	77	10.0	51.0	0.85	1.400	0.384	**3.64**
0.0	0	0	0	0	0	0.0															
0.0	0	0	0	0	0	0.0															
0.0	0	0	0	0	0	0.0															
0.0	0	0	0	0	0	0.0															
0.0	0	0	0	0	0	0.0															
0.0	0	0	0	0	0	0.0															
0.0	0	0	0	0	0	0.0															

Base Depth (feet)	Post FC Adj. $\Delta N_{1(60)}$	$N_{1(60)CS}$	Volumetric Strain (%)	Induced Subsidence (in.)	p (tsf)	G_{max} (tsf)	τ_{av} (tsf)	tav/Gmax	a	b	Shear Strain γ	Strain E_{15}	Strain Enc	Dry Sand Subsidence (in.)
3.5	7.8	21.7	0.02	**0.01**	0.071	333	0.023	0.0001	0.127	31,242	1.3E-04	1.2E-04	1.1E-04	**0.01**
6.5	7.6	20.4	0.03	**0.01**	0.156	483	0.050	0.0001	0.130	19,475	1.8E-04	1.8E-04	1.6E-04	**0.01**
11.5	8.8	27.7	0.02	**0.01**	0.301	741	0.095	0.0001	0.136	13,162	2.0E-04	1.3E-04	1.2E-04	**0.01**
16.5	10.0	79.0	0.00	**0.00**	0.450	1,287	0.141	0.0001	0.142	10,330	1.4E-04	2.7E-05	2.4E-05	**0.00**
21.5	10.0	61.0	0.01	**0.01**	0.610	1,375	0.189	0.0001	0.148	8,608	1.8E-04	4.6E-05	4.3E-05	**0.01**
25.5	10.0	58.2	0.01	**0.00**	0.743	1,493	0.227	0.0002	0.153	7,648	2.0E-04	5.4E-05	5.0E-05	**0.00**
50.0	10.0	51.0	0.00	**0.00**	1.564	2,073	0.386	0.0002	0.185	4,894	2.3E-04			





$N_{1(60)}$ = C_N*C_E*C_B*C_R*C_S*N

C_R = 0.75 for Rod lengths < 3m, 1.0 for > 10m
= min(1,max(0.75,1.4666-2.556/(z(ft))$^{0.5}$))

C_N = (1 atm/p'o)$^{0.5}$, max 1.7

C_S = max(1.1,min(1.3,1+$N_{1(60)}$/100)) for SPT without liners

MSF = $10^{2.24}/M^{2.56}$

z = Depth (m)

pa = 1 atm = 101 KPa = 1.058 tsf

p = 0.67*po

Nc = (MAG-4)$^{2.17}$

τ_{av} = 0.65*PGA*po*rd

G_{max} = 447*$N_{1(60)CS}^{(1/3)}$*$p^{0.5}$

a = 0.0389*(p/1)+0.124

b = 6400*(p/1)$^{(-0.6)}$

γ = [1+a*EXP(b*τ_{av}/G_{max})]/[(1+a)*τ_{av}/G_{max}]

E_{15} = γ*($N_{1(60)CS}$/20)$^{-1.2}$

E_{nc} = (Nc/15)$^{0.45}$*E15

S = 2*H*E_{nc}

rd = (1-0.4113*z^0.5+0.04052*z+0.001753*z^1.5)/(1-0.4177*z^0.5+0.05729*z-0.006205*z^1.5+0.00121*z^2))

$\Delta N_{1(60)}$ = min(10,IF(FC<35,exp(1.76-(190/FC^2)),5)+IF(FC<=5,1,IF(FC<35,0.99+(FC^1.5/1000),1.2))*N1(60) - N1(60)

$N_{1(60)CS}$ = $N_{1(60)CS}$ + $\Delta N_{1(60)}$

Kσ = min of 1.0 or (p'o/1.058)$^{(IF(Dr>0.7,0.6,IF(Dr<0.5,0.8,0.7))-1)}$

Dr = ($N_{1(60)}$/70)$^{0.5}$

CSReq = 0.65*PGA*(po/p'o)*rd

CSR* = CSReq/MSF/Kσ

$CRR_{7.5}$ = (0.048-0.004721*N+0.0006136*N^2-0.00001673*N^3)/(1-0.1248*N+0.009578*N^2-0.0003285*N^3+0.000003714*N^4))

N = $N_{1(60)CS}$

SF = $CRR_{7.5,1atm}$/CSR*

APPENDIX F

Infiltration Testing Results

Infiltration Testing

Test Through Zone Near Bottom of Infiltration Device

Infiltration Testing Field Data

Date: 6/13/2018

Project Location:	Conejo Valley Church of Christ	Job Number:	301911-001
Earth Description:	Clayey Silt with some fine grain sand	Tested By:	JW
Field Test in Boring	B-4/IT-1	Start Time:	11:45 AM
Boring Diameter (inches):	8	Total Pipe Length (feet):	13.5
Boring Depth (feet):	13	Pipe Stick-Up (Inches)	6

Time of Day, t (hh:mm)	Delta Time, Δt (min.)	Delta Time, Δt (hr.)	Top of Pipe to Water, (ft.)	Water Depth, d (in.)	Water Depth, d (ft.)	Drop in Water Height, Δd (in.)	Drop in Water Height, Δd (ft.)	Perc Rate, (in/hr)	Corr. Factor, RF	Infilt. Rate (in/hr)
11:45		-	12.30	14.4	1.20					
12:15	30.0	0.50	12.45	12.6	1.05	1.80	0.15	3.60	4.38	0.82
12:15			12.45	12.6	1.05					
12:30	15.0	0.25	12.55	11.4	0.95	1.20	0.10	4.80	4.00	1.20
12:30			11.80	20.4	1.70					
12:45	15.0	0.25	11.90	19.2	1.60	1.20	0.10	4.80	5.95	0.81
12:45			11.80	20.4	1.70					
13:00	15.0	0.25	11.90	19.2	1.60	1.20	0.10	4.80	5.95	0.81
13:00			11.65	22.2	1.85					
13:15	15.0	0.25	11.85	19.8	1.65	2.40	0.20	9.60	6.25	1.54
13:15			11.50	24.0	2.00					
13:30	15.0	0.25	11.65	22.2	1.85	1.80	0.15	7.20	6.78	1.06
13:30			11.65	22.2	1.85					
13:45	15.0	0.25	11.83	20.0	1.67	2.16	0.18	8.64	6.28	1.38
13:45			11.65	22.2	1.85					
14:00	15.0	0.25	11.82	20.2	1.68	2.04	0.17	8.16	6.30	1.30

Infiltration Testing

Test Through Zone Near Bottom of Infiltration Device

Infiltration Testing Field Data

Date: 6/13/2018

Project Location:	Conejo Valley Church of Christ	Job Number:	301911-001
Earth Description:	Clayey Silt with some fine grain sand	Tested By:	JW
Field Test in Boring	B-5/IT-2	Start Time:	11:45 AM
Boring Diameter (inches):	8	Total Pipe Length (feet):	2
Boring Depth (feet):	2	Pipe Stick-Up (Inches)	0

Time of Day, t (hh:mm)	Delta Time, Δt (min.)	Delta Time, Δt (hr.)	Top of Pipe to Water, (ft.)	Water Depth, d (in.)	Water Depth, d (ft.)	Drop in Water Height, Δd (in.)	Drop in Water Height, Δd (ft.)	Perc Rate, (in/hr)	Corr. Factor, RF	Infilt. Rate (in/hr)
11:45		-	1.00	12.0	1.00					
12:15	30.0	0.50	2.00	0.0	0.00	12.00	1.00	24.00	2.50	9.60
12:15			1.00	12.0	1.00					
12:30	15.0	0.25	1.30	8.4	0.70	3.60	0.30	14.40	3.55	4.06
12:30			1.00	12.0	1.00					
12:45	15.0	0.25	1.22	9.4	0.78	2.64	0.22	10.56	3.67	2.88
12:45			1.00	12.0	1.00					
13:00	15.0	0.25	1.20	9.6	0.80	2.40	0.20	9.60	3.70	2.59
13:00			1.00	12.0	1.00					
13:15	15.0	0.25	1.28	8.6	0.72	3.36	0.28	13.44	3.58	3.75
13:15			1.00	12.0	1.00					
13:30	15.0	0.25	1.18	9.8	0.82	2.16	0.18	8.64	3.73	2.32
13:30			1.00	12.0	1.00					
13:45	15.0	0.25	1.22	9.4	0.78	2.64	0.22	10.56	3.67	2.88
13:45			1.00	12.0	1.00					
14:00	15.0	0.25	1.24	9.1	0.76	2.88	0.24	11.52	3.64	3.16

Infiltration Testing

Test Through Zone Near Bottom of Infiltration Device

Infiltration Testing Field Data

Date: 6/14/2018

Project Location:	Conejo Valley Church of Christ	Job Number:	301911-001
Earth Description:	Clayey Silt with some sand	Tested By:	JW
Field Test in Boring	B-8/IT-3	Start Time:	1:20 PM
Boring Diameter (inches):	8	Total Pipe Length (feet):	14
Boring Depth (feet):	13	Pipe Stick-Up (Inches)	12

Time of Day, t (hh:mm)	Delta Time, Δt (min.)	Delta Time, Δt (hr.)	Top of Pipe to Water, (ft.)	Water Depth, d (in.)	Water Depth, d (ft.)	Drop in Water Height, Δd (in.)	Drop in Water Height, Δd (ft.)	Perc Rate, (in/hr)	Corr. Factor, RF	Infilt. Rate (in/hr)
13:20		-	11.65	28.2	2.35					
13:50	30.0	0.50	11.75	27.0	2.25	1.20	0.10	2.40	7.90	0.30
13:50			11.75	27.0	2.25					
14:20	30.0	0.50	11.75	27.0	2.25	0.00	0.00	0.00	7.75	0.00
14:20			11.75	27.0	2.25					
14:50	30.0	0.50	11.75	27.0	2.25	0.00	0.00	0.00	7.75	0.00

Infiltration Testing

Test Through Zone Near Bottom of Infiltration Device

Infiltration Testing Field Data

Date: 6/14/2018

Project Location:	Conejo Valley Church of Christ	Job Number:	301911-001
Earth Description:	Clayey Silt with some fine grain sand	Tested By:	JW
Field Test in Boring	B-9/IT-4	Start Time:	1:20 PM
Boring Diameter (inches):	8	Total Pipe Length (feet):	3
Boring Depth (feet):	2	Pipe Stick-Up (Inches)	12

Time of Day, t (hh:mm)	Delta Time, Δt (min.)	Delta Time, Δt (hr.)	Top of Pipe to Water, (ft.)	Water Depth, d (in.)	Water Depth, d (ft.)	Drop in Water Height, Δd (in.)	Drop in Water Height, Δd (ft.)	Perc Rate, (in/hr)	Corr. Factor, RF	Infilt. Rate (in/hr)
13:20		-	1.70	15.6	1.30					
13:50	30.0	0.50	2.60	4.8	0.40	10.80	0.90	21.60	3.55	6.08
13:50			1.70	15.6	1.30					
14:05	15.0	0.25	1.95	12.6	1.05	3.00	0.25	12.00	4.53	2.65
14:05			1.65	16.2	1.35					
14:20	15.0	0.25	1.85	13.8	1.15	2.40	0.20	9.60	4.75	2.02
14:20			1.65	16.2	1.35					
14:35	15.0	0.25	1.84	13.9	1.16	2.28	0.19	9.12	4.77	1.91
14:35			1.60	16.8	1.40					
14:50	15.0	0.25	1.83	14.0	1.17	2.76	0.23	11.04	4.86	2.27
14:50			1.60	16.8	1.40					
15:05	15.0	0.25	1.80	14.4	1.20	2.40	0.20	9.60	4.90	1.96