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Soils & Geologic Consultants
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August 8, 2014
SS-202-S

Faith Community Church
c/o Jorge Escamilla
stitchstudio3d@gmail.com

Subject: **SOILS ENGINEERING REPORT**
Proposed Classroom Building & Lobby Addition, Faith Community Church, 355 D Street, Fillmore, California

Dear Mr. Escamilla:

Introduction

The following report summarizes the findings of our Soils Engineering Investigation with Liquefaction Analysis performed on the subject property. Our purpose was to evaluate the distribution and engineering characteristics of the earth materials present on the site so that we might assess their impact upon the proposed classroom building and lobby addition to the existing church building.

It is the intent of this report to aid in the design and completion of the proposed work and to reduce certain risks associated with construction projects. This report is prepared for the use of the client and authorized agents and should not be considered transferable. Prior to use by others, the site and this report should be reviewed by *Solid Soils & Geologic Consultants*. Following review, additional work may be required to update this report.

The scope of work for this project included: 1) a reconnaissance of the site and its immediate vicinity, 2) logging and sampling of 2 borings, one excavated with a 8-inch diameter, hollow stem drill rig and one excavated with a 3-inch diameter hand auger, 3) select laboratory testing of the retrieved samples, 4) soils engineering analysis of the assembled data, and 5) preparation of this report. Field data and the approximate locations of the exploratory excavations are shown on the enclosed Plot Plan. Descriptions of the materials encountered in the exploratory excavations are provided on the enclosed logs (Plates B-1 and B-2). Pertinent laboratory test results are provided in this report.

Site Location & Description

The subject property is located at 355 D Street in Fillmore, California. The property currently consists of a relatively level, partially developed lot with an existing church building located near the north-central portion of the property. The approximate site location is shown on the enclosed Vicinity Maps. Drainage on the property flows gently to the southwest, however the lot is essentially level.

Proposed Development
Mr. Jorge Escamilla provided the information regarding the proposed development. It is proposed to construct a classroom building to the southwest of the existing church building and construct a lobby addition to the west side of the existing church building, as shown on the enclosed Plot Plan. It is anticipated that the proposed structures will consist of conventional construction with normal bearing loads. Unusual design characteristics such as a basement are not currently anticipated. Grading will be limited to site preparation. This information was the basis for the field exploration.

Exploration Observations
The scope of our exploration was based on our understanding of the project, as described above. The site was explored on July 18, 2014 with the aid of an 8-inch diameter, hollow stem drill rig, a 3-inch diameter hand auger, and field mapping. A total of 2 borings were excavated to depths of between 6 and 40½ feet below existing grade, where they met refusal on hard cobbles. The excavations were backfilled and tamped.

The earth material observed consisted mainly of gravelly sand and was light to dark brown, dry to moist, and medium dense to very dense. The upper approximate 4 feet contained abundant voids and was compressible.

Gravel and cobbles (and possibly boulders) were logged by drilling. Refusal on cobbles was encountered in the hand auger boring at a depth of 6 feet below existing grade and in the drill rig boring at a depth of 40½ feet below existing grade. Gravel, cobbles, and boulders are common in this portion of the Sespe River drainage.

Groundwater was not encountered in either of the excavations. Fluctuations in the level of groundwater may occur due to variations in rainfall, irrigation, temperature, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site.

Seismicity
The subject property has no known active or potentially active faults crossing it. An "active fault" is one which has had movement in the last 11,000 years. The site is not located within an Alquist-Priolo "Earthquake Fault Zone". The "Earthquake Fault Zone" is the area designated by the State of California as being the zone where primary ground rupture is considered most likely to occur during a seismic event on a fault.

Earthquake epicenters may happen anywhere in Southern California along thrust faults or buried faults, as has been evidenced by several recent historic earthquakes, including the San Fernando Earthquake, the Whittier Narrows Earthquake, and the Northridge Earthquake. The proximity of a site to the surface trace of a fault may have little relationship to the potential of being near an earthquake epicenter.

The property is situated within the seismically active Southern California region and therefore will be subjected to moderate to strong ground shaking should one of the many active Southern California faults produce an earthquake. It is likely that at least one significant seismic event will affect the site during the lifetime of the structure. Secondary effects, such as earthquake-induced landsliding or ground rupture are not considered likely to occur. However, severe ground shaking may cause minor liquefaction, consolidation, and settlement of the underlying soils.

The nearest recognized active fault is the San Cayetano Fault, located approximately 7,000 feet to the northeast. A seismic evaluation for the site is enclosed. The anticipated seismic affects on the site are based on estimated horizontal acceleration calculations provided by *Tom Blake's* EQFAULT computer program for a 6.8 magnitude earthquake on the San Cayetano Fault, modeled to be approximately 1.3 miles away. Such an event would provide an estimated horizontal acceleration of approximately 0.7 g. The duration of ground shaking is estimated to be on the order of 15 to 45 seconds. It should be noted that modeling or predicting seismically induced ground accelerations is an inexact science, as indicated by the recent higher-than-expected ground accelerations from the Northridge Earthquake.

Soil parameters for current seismic design are provided in the enclosures. The calculation was performed on the USGS website on August 5, 2014.

Laboratory Testing

Bulk and relatively undisturbed samples of earth materials encountered at the site were collected during the course of our fieldwork. Select samples were transported to the laboratory for further testing and analysis. Laboratory tests completed on the retrieved samples are described below.

Moisture-Density

The field moisture content and dry unit weight were determined for each undisturbed sample. The dry unit weight is expressed in pounds per cubic foot and the moisture content represents a percentage of the dry unit weight. This test data is presented in Table I.

Expansion Tests

An expansion index test was performed in accordance with the UBC Standard 29-2 or equivalent. The results of these tests are included in Table I.

Shear Test

Shear tests were performed in a Direct Shear Machine of the strain control type. The rate of deformation was approximately 0.05 inches per minute. Shearing occurred under a variety of confining loads in order to determine the Coulomb shear strength parameters. The tests were performed on relatively undisturbed samples in an artificially flooded condition. The test results are presented graphically on Plate S-1.

TABLE I

Summary of
Laboratory Test Data

Sample	Depth (ft)	Field Dry Density (PCF)	Field Moisture Content (%)	Expansion Index
B-1	0-5			0
B-1	5	109.9	5.5	
B-1	8	115.0	4.0	

Liquefaction

The site is located within a potential liquefaction zone. In order to verify the subject properties liquefaction potential, the site was explored with a truck mounted, hollow-stemmed drill rig. One boring (B-1) was drilled to 40½ feet. Hard gravel, cobbles, and boulders prevented drilling past 40½ feet and created difficult drilling conditions the entire depth of the boring.

In order for liquefaction to occur, three conditions must simultaneously be met during a significant seismic event: 1) Soils must be saturated by the presence of ground water; 2) grain size must fall within certain limits; and 3) density of the earth materials must be relatively low, as indicated by blow counts. In general, soils with corrected blow counts in excess of about 30 blows per foot are of sufficient density and are not considered susceptible to liquefaction under normal seismic conditions. Sand falls within the grain size that may be prone to liquefaction when water and low blow counts (low densities) are present.

The results of our exploration reveal that the site is not considered susceptible to liquefaction, as groundwater was not encountered to the total depth explored (40½ feet below existing grade). A significant seismic event will result in considerable ground shaking and may cause settlement of the alluvial materials and some liquefaction may occur below the water table at some depth. However, shallow liquefaction is not likely to occur and the probability of cobbles and boulders at depth reduce the risk of liquefaction at depth.

The maximum depth at which soils are subject to liquefaction is still under debate. Liquefaction is not considered likely below 25 feet by some scientists and others suggest that soils may be subject to liquefaction to a depth of 40 feet or more. Liquefaction reportedly occurred at depths of 60 feet in Kobe, Japan. The relationship shown on the enclosed figure labeled "Affects of Liquefaction with Overburden" (*Liquefaction of Soils During Earthquakes, National Academy Press, 1985*), based on Ishirhara (1985), indicates that ground rupture does not occur with overburden greater than 3 meters (10 feet). The study performed on the subject property shows that the upper 40 feet has no groundwater and is therefore not subject to liquefaction. This qualifies for a greater than 3-meter thick layer of dense, liquefaction resistant overburden. Based on this, should liquefaction occur in a zone of potential liquefaction below 40 feet, ground rupture will likely not occur. Other surface manifestations are not anticipated to occur on the site. Lateral spreading will not occur, as daylighted slopes do not exist in the area.

Discussion & Recommendations

The following discussion and recommendations are based on the data presented in this report and our understanding of the project. Recommendations, derived from the data available at this time, are presented for your consideration.

Based upon the exploration performed for this investigation, it is our finding that construction of the proposed classroom building and lobby addition, as described, is feasible from a soils engineering standpoint, provided our advice and recommendations are made a part of the plans and are implemented during construction.

The surficial soils on the site are relatively soft and may be subject to consolidation and settlement upon loading. Therefore, it is recommended that the surficial soils be removed and recompacted to a minimum depth of 3 feet below the bottom of the proposed foundations and 5 feet beyond the perimeter, where possible. The fill should be compacted to a minimum of 90% of the laboratory maximum dry density or 95% in areas where fill cannot extend 5 feet beyond the perimeter of the footings. This will provide a dense, uniform bearing material and will distribute the loads more evenly upon the natural soils. Rocks larger than 8 inches in diameter should not be included in the fill.

Following proper removal and recompaction of the soils, conventional continuous foundations may be used to support the proposed structures. All foundations should be reinforced with four #4 rebar, two placed near the top and two placed near the bottom. All floor slabs shall be a minimum of 4 inches thick and reinforced with #4 rebar, spaced 16 inches on center, both ways. The rebar in the floor slabs should be bent in an "L" fashion and extend a minimum of 12 inches into the adjacent footings.

Spread Footings
Continuous spread footings may be used to support the proposed classroom building and lobby addition provided that they are founded entirely in future compacted fill. Two dissimilar materials such as compacted fill and natural material may not support footings. Continuous footings should be a minimum of 15 inches in width. Isolated pad footings should not be used. Foundation design parameters are outlined on the following chart.

Bearing Material	Minimum Depth into Bearing Material (Inches)	Vertical Bearing (psf)	Coefficient of Friction	Passive Earth Pressure (pcf)	Maximum Earth Pressure (psf)
Future Compacted Fill	18	1,500	0.35	200	1,500

The allowable soil strength parameters indicated above are for the total of dead and frequently applied live loads and may be increased by one third for short duration loading, which includes the effects of wind or seismic forces. When combining passive and friction for lateral resistance, the passive component should be reduced by one third. For the purpose of bearing calculations, the weight of the concrete in the footing may be neglected.

All continuous footings should be reinforced with a minimum of four #4 steel bars, two placed near the top and two placed near the bottom of the footings. Footing excavations should be cleaned of all debris, loose soil, moistened as required by the local government agency, and free of shrinkage cracks prior to placing concrete. Observation of the footing excavations should be performed by *Solid Soils & Geologic Consultants* prior to placing forms, steel or concrete to verify the proper depths. All work and materials should comply with the specifications of the building official.

Floor Slabs
The recommended material to support the proposed floor slabs is future compacted fill. Following proper removal and recompaction slabs may be supported on the compacted fill. All footing excavation spoils and debris should be removed from the area. Floor slabs should be a minimum of 4 inches thick and should be cast over a clean, firm subgrade and reinforced with a minimum of #4 steel bars spaced 16 inches on center, both ways. The rebar should be bent in an "L" fashion and

extend a minimum of 12 inches into the adjacent foundations. Care should be taken to cast the reinforcement near the center of the slab. Slabs should be provided with a bed of 4 inches of clean sand beneath the concrete.

All slabs should be protected with a polyethylene or visqueen plastic vapor barrier at least 10 mil thick, beneath the slab. The vapor barrier should be covered with about one inch of clean sand to help prevent punctures and to aid in the cure of the concrete.

Foundation Settlement
Settlement of the foundation system is expected to occur on initial application of loading. Anticipated differential settlement for properly supported foundations should be on the order of ½ to 1 inch over the length of 40 feet. Total settlement is not expected to exceed approximately 1 inch.

Additions attached to an existing structure may be subject to differential movement with respect to the existing structure. This may be due to many different variables, including different building materials, different building techniques, different grades of lumber, different foundation design or bearing material, and differential settlement of the soils beneath the foundations. Therefore, it is recommended that a flexible seam be provided between the existing building and the addition, where possible. The use of a facade of some type may help to cover any cracking, which may result from differential movement.

Grading - Compacted Fills
The following recommendations are for the preparation and placement of compacted fills. The contractor should be aware that if grading is done during or following periods of rain, or if the ground moisture is over optimum from any source of water, such as excessive irrigation watering, etc., then a considerable amount of time and/or effort may be needed to achieve proper moisture for compaction purposes.

1. The on-site soils are suitable for use as structural fill following removal of oversized rocks. Any imported materials that are to be used as fill should be approved by this office prior to placement.
2. All vegetation, trash, debris or other deleterious materials should be removed from the area to be graded and exported from the site. Rocks larger than 8 inches in diameter should not be included in the fill.
3. All existing fill and incompetent surface soils within the area to be filled should be removed to dense, natural material and replaced as properly compacted fill.
4. The foundations for the proposed structures should be provided with at least 3 feet of compacted fill <u>beneath</u> the base of proposed foundations. Final foundation plans should be given to the grader prior to starting work in order to determine the minimum depth of the excavation. The difference in the depth of the fill beneath the proposed structures should not exceed 5 feet. The fill should extend at least 5 feet beyond the edge of the footings or for a

distance equal to the depth of the fill below the footings, whichever is deeper. If the fill cannot extend 5 feet beyond the edge of the footings, due to property lines or structures, then 95% compaction should be obtained in that area. A licensed surveyor should verify the required vertical and lateral extent of the fill with respect to the location of the proposed structures.

5. The excavated fill bottoms should expose dense alluvium, per our recommendations. All bottom excavations should be observed by a representative of our office prior to placement of fill. The bottom excavation should be scarified, watered or dried to near optimum moisture content, and compacted to 90% using the most recent version of ASTM D 1557 as the standard.
6. Fill should be placed in thin lifts, watered to near optimum moisture content, and compacted to at least 90 percent (or 95%, see above) of the material's maximum dry density, using the latest version of ASTM D 1557 as the standard, prior to placement of the next lift. All fill should be placed under the observation and testing of *Solid Soils & Geologic Consultants* to assist the contractor in achieving proper compaction.
7. Approved fill material which is expansive should be placed slightly above optimum moisture. This will help reduce the detrimental affects of expansion and swelling.
8. Areas that are to receive paving should be processed to at least 24 inches below the existing grade or the finished subgrade, whichever is deeper.
9. All grading should comply with the grading specifications and the requirements of the County of Ventura.
10. It is anticipated that shrinkage of the material during the compaction process will be on the order of 15 to 20 percent.

Temporary Excavations

The excavations for the proposed fill bottoms will expose soil and alluvium. A vertical cut of up to 5 feet may be made in either of these materials. Excavations greater then 5 feet should be trimmed back to 1:1 (horizontal to vertical). Where the presence of a property line or existing structures prevents laying back the cut, temporary shoring or slot cutting may be used.

Slot Cutting

The slot cutting method uses the earth as a buttress and allows the excavation to proceed in phases. The initial excavation is made at a slope of 1:1. Alternate slots of 8 feet in width (maximum) are excavated and fill placed and properly compacted to rough grade before the remaining earth buttresses are excavated. The remaining earth buttresses should be 8 feet wide.

A representative of *Solid Soils & Geologic Consultants* should observe all slot cutting and compaction procedures. All excavations should be properly fenced off (or other appropriate method) for safety and should be stabilized within 30 days of the initial excavation. Water should not be allowed to pond near the top of the excavations nor flow towards them. No vehicles should be allowed within 7 feet of the cut.

Drainage

Positive control of surface water should be established. Irrigation water should not enter the development area. Roof gutters and downspouts should be provided to collect all roof water. Downspouts should deposit the water into a buried drain or paved swale. Downspouts should not direct water onto the soil next to the foundations. Pad and roof drainage should be collected and transferred to the street or approved drainage system in non-erosive drainage devices. Water should be directed away from foundations. Drainage should not be allowed to pond on the pad, under the building, against any foundations, or behind walls. A minimum of 2% (2 vertical per 100 horizontal) drainage should be provided in all areas. A 5% slope should be considered for non-paved areas in the vicinity of the structures. The 5% zone should be at least seven feet wide, where possible. Fine-grade fills placed to create pad drainage should be compacted in order to retard infiltration of surface water.

Preserving proper surface drainage is also important. Planters, decorative walls, plants, trees or accumulations of organic matter should not be allowed to retard surface drainage or clog drains. Area drains and roof gutters should be kept free of obstructions. Roof gutters and condensation lines from air conditioners should outlet to area drains or paved areas which conduct the water to the street. Positive drainage along the backs of walls should be maintained. Any other measures that will facilitate positive surface drainage should be employed. Long-term saturation of the soils or subsurface may adversely affect structure foundations, slabs, patios, sidewalks and other rigid surfaces. The property owner and gardener should be reminded of the need to preserve proper drainage.

Vegetation and Irrigation

The landscaping process should aid in abating erosion. Care should be taken not to over-irrigate the property. Watering patterns should be modified to reflect rainy periods. The irrigation system should be checked on a regular basis for leakage. All leaks should be repaired immediately. Irrigation water should be applied only to the minimum extent needed to support plant life. A good source of information is your local city or county agency, the "Sunset New Western Garden" book, or similar publications.

Planter boxes adjacent to building foundations should either be avoided or appropriately sealed so that the irrigation water does not impact the foundations. Sealing may be accomplished by constructing the planters with a solid base and sidewall weep holes (exiting on side away from the building), or by providing a cutoff wall adjacent to the foundations. Cutoff walls should be at least 6 inches thick and extend at least 30 inches below the grade.

Control of irrigation water is a necessary part of site maintenance. Soggy ground, perched water, seeps and/or water damage may result if irrigation water is excessively or improperly applied. All irrigation systems should be adjusted to provide the minimum water needed to sustain landscaping. Adjustments should be made for changes of the seasons. Irrigation should stop when sufficient water is provided by precipitation. Broken, leaking, or plugged sprinklers or irrigation lines should be repaired immediately. Frequent inspections of the irrigation systems should be performed. The property owner and gardener should be reminded of the need to properly irrigate the property and the potential damage which may occur from irresponsible watering.

Utility Trench Backfill

Backfill for utility trench excavations should be compacted to at least 90% relative compaction. The designer and contractor should be aware of the potential of backfill sand in utility trenches to act as a subdrain. Water can be collected in the utility trenches and transported considerable distances, often across property lines. Flooding of junction boxes or service laterals may result. Flooding of service laterals may cause water damage to the structures, including the interior of the structures. Appropriate measures should be taken in the design and construction phase to prevent such flooding.

Plan Review

Finalized plans should be submitted to *Solid Soils & Geologic Consultants* for comment and review. Additional recommendations may be provided at that time, if such are considered warranted. **A minimum of 48 hours should be allowed for the review of the plans.**

Construction Monitoring

A pre-construction meeting should be held at the site between the owner, contractor, grader, and *Solid Soils & Geologic Consultants*. The meeting should be held at least two days prior to starting any fieldwork. Compliance with *Solid Soils & Geologic Consultants* design concepts, specifications and recommendations during construction requires our review during the course of construction.

All temporary excavations should be observed by a representative of *Solid Soils & Geologic Consultants* to verify that the anticipated conditions are present and that our recommendations have been implemented at the construction site.

All fill bottom excavations should be observed prior to placement of fill. A representative of this office should monitor placement of all fill. Supplemental recommendations may prove warranted based upon the materials exposed in the actual excavations.

Foundation excavations should be observed by a representative of *Solid Soils & Geologic Consultants* to determine if the recommended depth into the proper bearing material has been achieved and that the site conditions are the same as those anticipated. Such observations should be made prior to placing concrete, steel or forms. Please notify our office at least 24 hours prior to a site visit. The approved plans and permits should be on the job site and available for our review.

General Conditions

In the event of any changes in the design or location of any structure, as outlined in this report, the conclusions and recommendations contained herein may not be considered valid. Any changes should be reviewed by *Solid Soils & Geologic Consultants* and our conclusions and recommendations modified or reaffirmed after such a review.

The subsurface conditions described herein have been projected from excavations on the site. They should in no way be construed to reflect any variations which may occur between these excavations or which may result from changes in subsurface conditions. If conditions encountered during construction appear to differ from those disclosed herein, notify *Solid Soils & Geologic Consultants* immediately so we may consider the need for modifications.

Exploration was performed on only a portion of the site. The findings for the study area cannot be considered as indicative of areas not explored.

This report is made and issued for the sole use and benefit of the client and is not transferable. This report states conditions as of the date of the exploration. Any liability in connection herewith shall not exceed our fee for the exploration. No warranty, expressed or implied, is made or intended in connection with the above exploration, by the furnishing of this report, or by any other oral or written statement.

Thank you for this opportunity to be of service. If you have any questions regarding this report, please feel free to contact the undersigned at (805) 202-6533.

Respectfully submitted,
SOLID SOILS & GEOLOGIC CONSULTANTS

Jeff Sivas
President

Don Villafana
R.C.E. 37354 expires 6/16

Enclosures:
- Vicinity Map
- Vicinity Geologic Map
- Vicinity Fault Zone Map
- Seismic Hazard Zones Map
- Plot Plan
- Boring Logs .... Plates B-1 & B-2
- Shear Test Results .... Plate S-1
- Liquefaction Figures
- Seismic Information

XC: (4) Addressee

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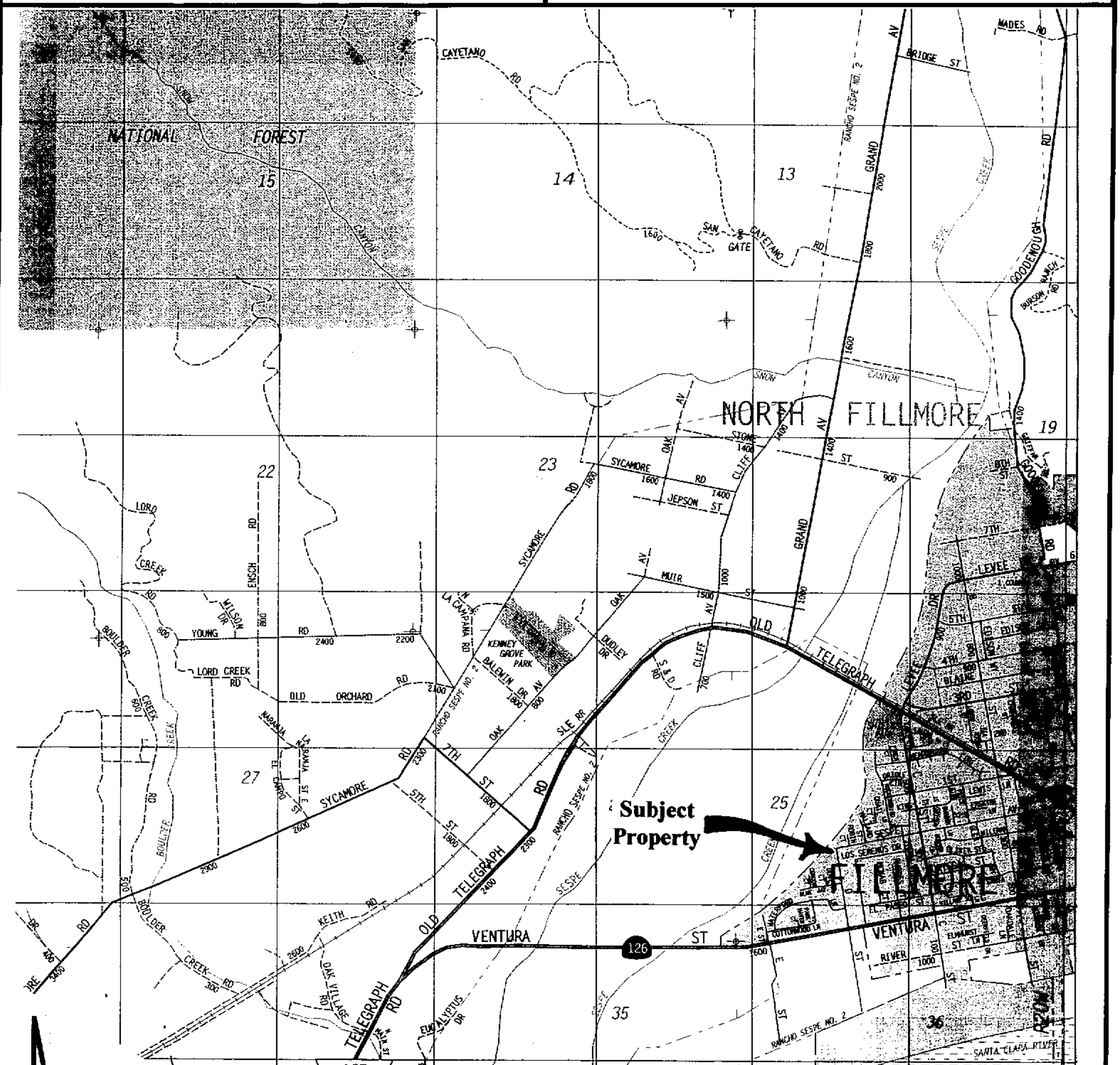
Subject: Vicinity Map Reference: Thomas Guide, Pg. 455 Scale: 1" = 2,400'	Client: Faith Community Church Job #: SS-202-S Date: 8/14



Vicinity Map

SCALE: 1" = 2,400'

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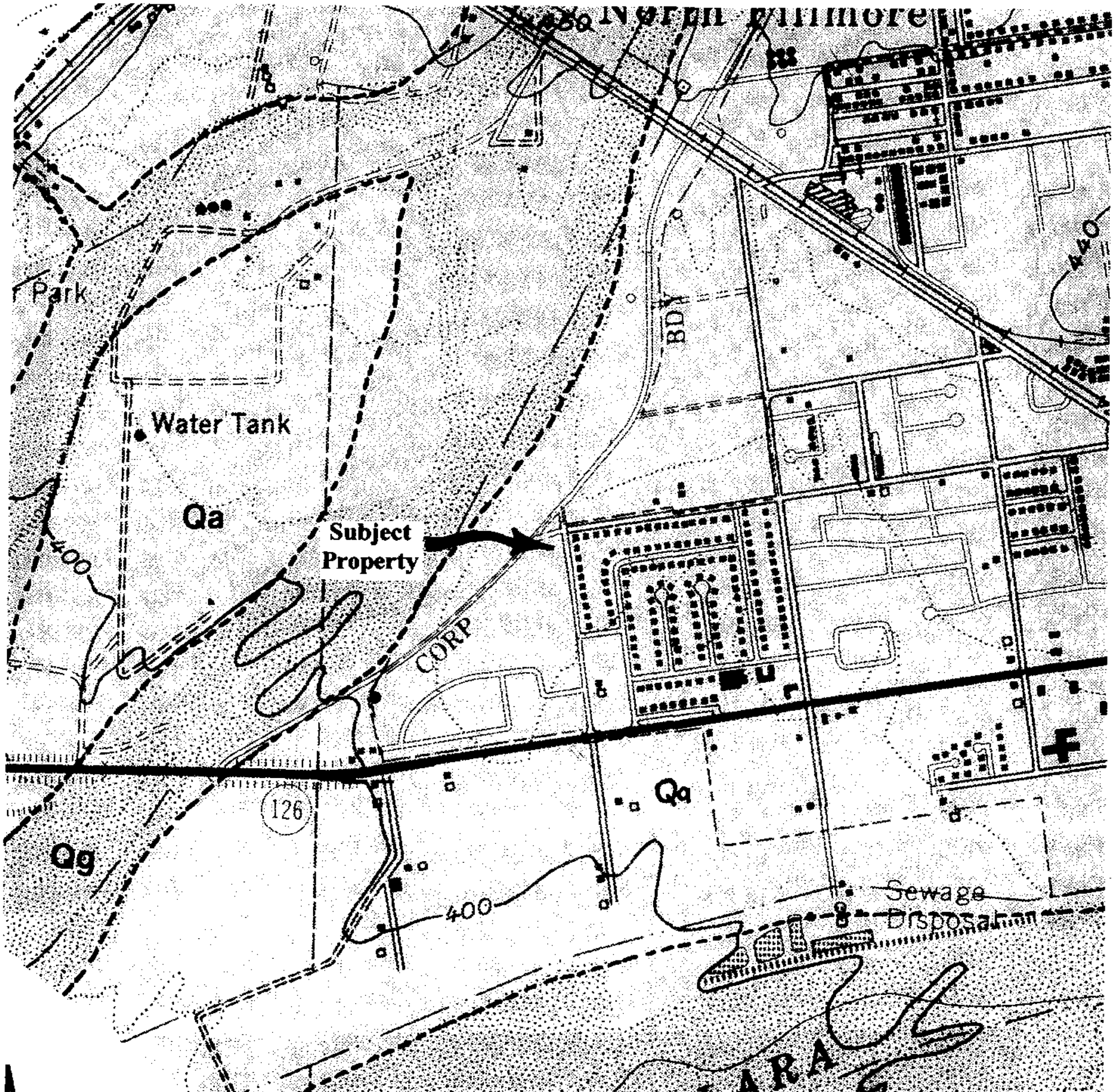
Subject: Vicinity Geologic Map Reference: Dibblee, Fillmore Quad. Scale: 1" = 1,000'	Client: Faith Community Church Job #: SS-202-S Date: 8/14

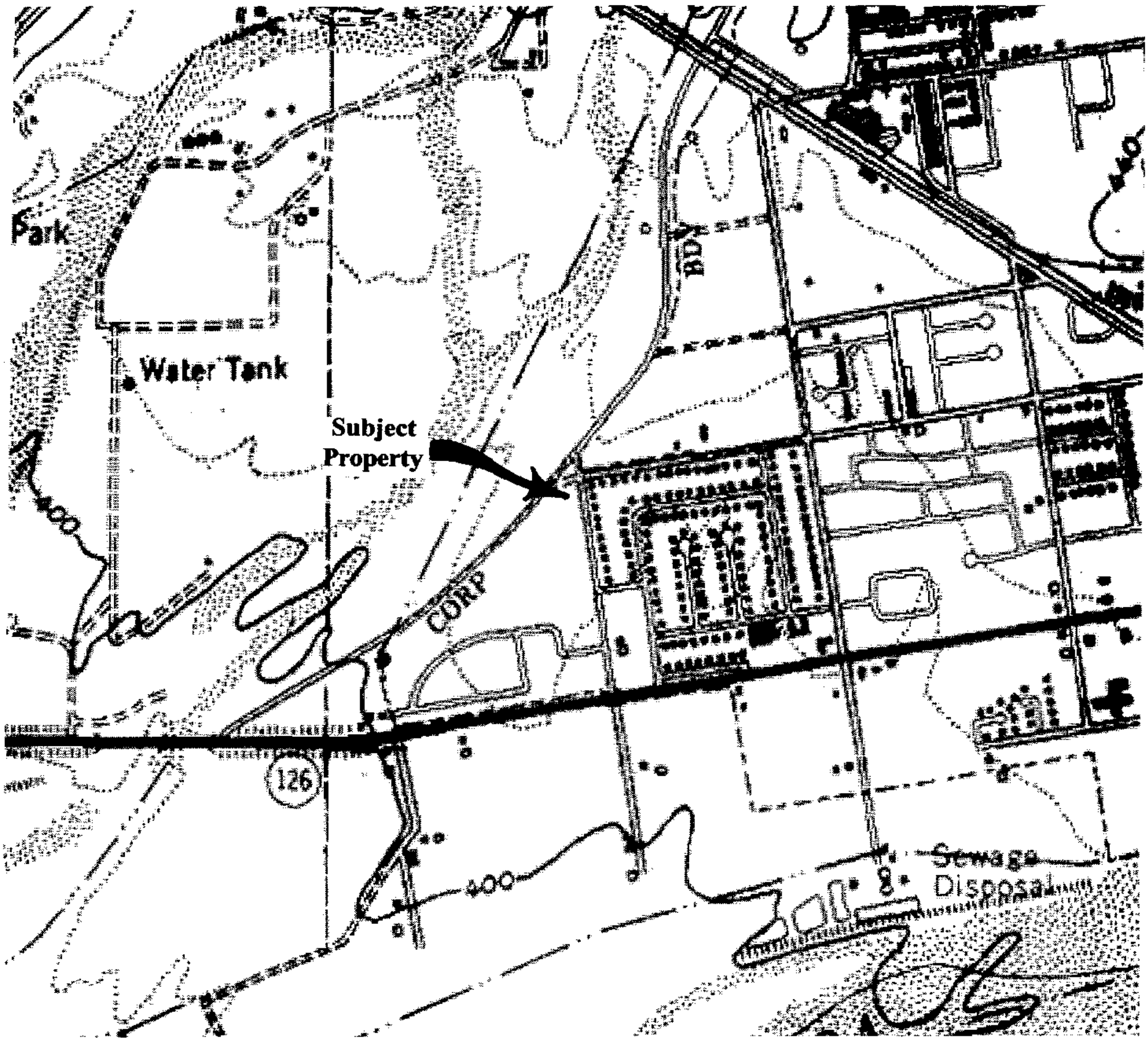


Vicinity Geologic Map

SCALE: 1" = 1,000'

Subject: Vicinity Fault Zone Map
Reference: Alquist-Priolo Maps
Scale: 1" = 1,000'

Client: Faith Community Church
Job #: SS-202-S
Date: 8/14



Vicinity Fault Zone Map

SCALE: 1" = 1,000'

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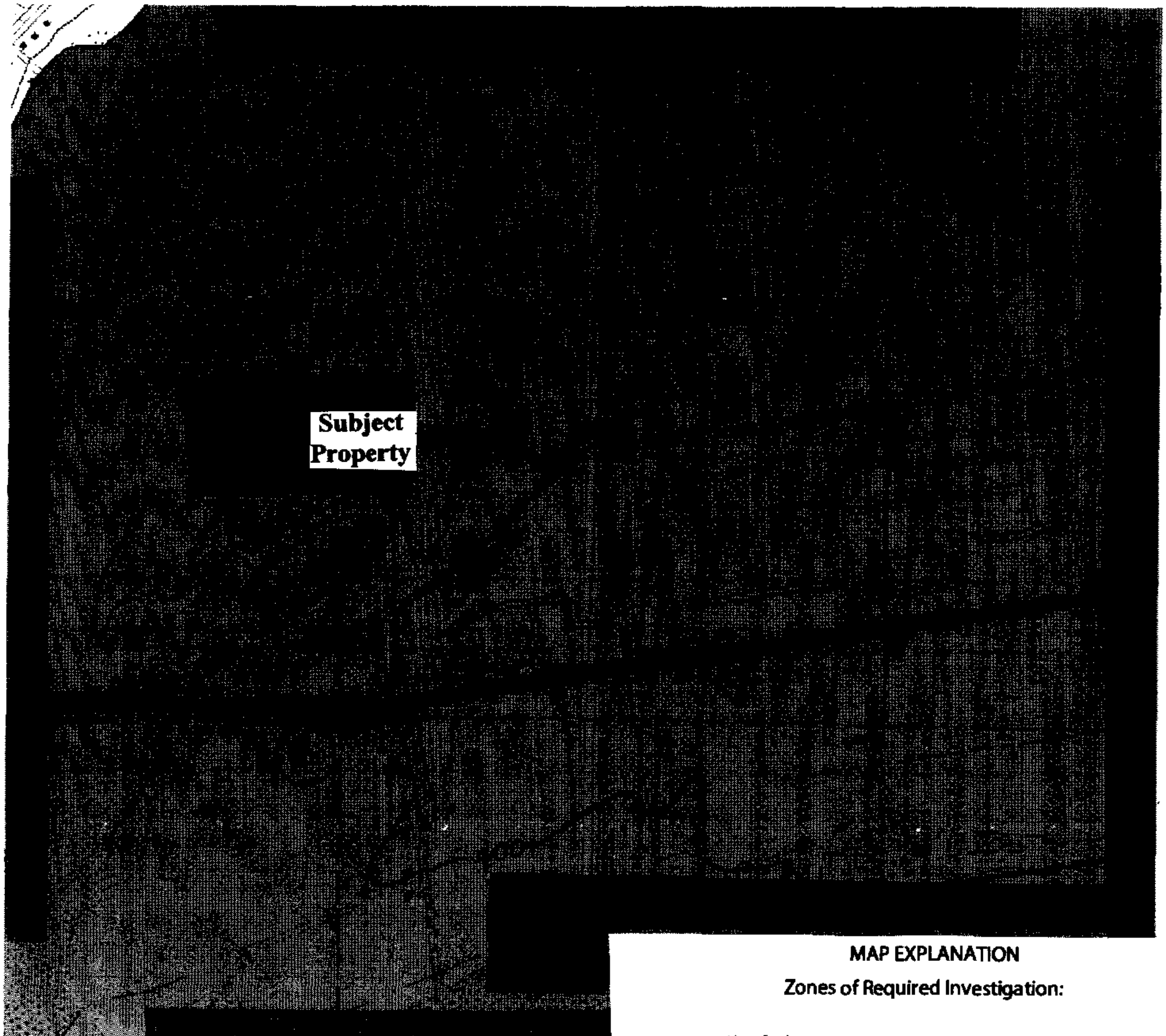
Subject: Seismic Hazard Zones Map Reference: CDMG, Fillmore Quad. Scale: 1" = 1,000'	Client: Faith Community Church Job #: SS-202-S Date: 8/14



MAP EXPLANATION

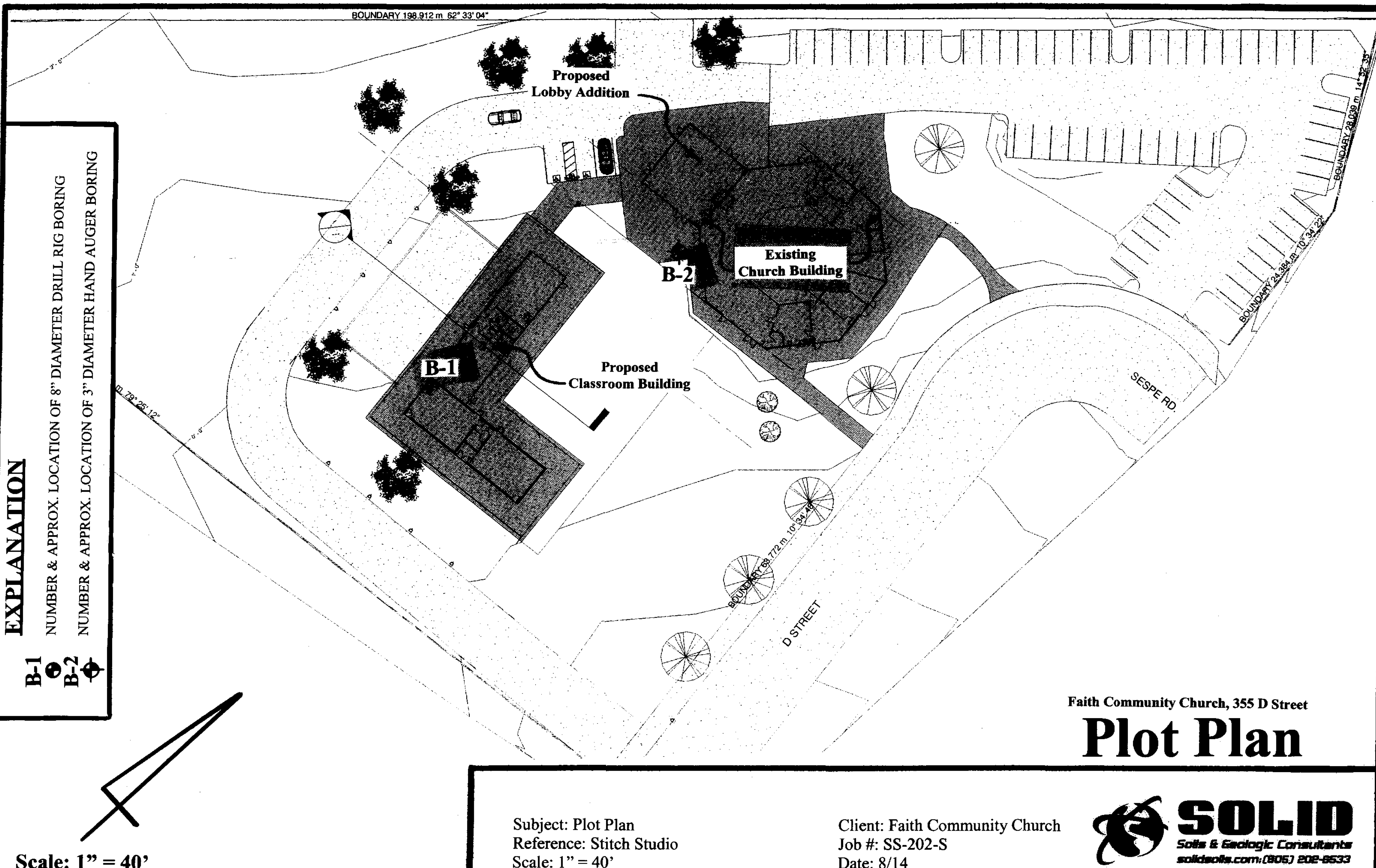
Zones of Required Investigation:

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.

Seismic Hazard Zones Map

SCALE: 1" = 1,000'



Faith Community Church, 355 D Street

Plot Plan

Subject: Plot Plan
Reference: Stitch Studio
Scale: 1" = 40'

Client: Faith Community Church
Job #: SS-202-S
Date: 8/14

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BORING LOG

Job #: SS-202-S
Client: Faith Community Church
Address: 355 D Street, Fillmore

Boring #: B-1a
Date Excavated: 7/18/14
Logged by: JS

SAMPLE	BLOWS/FOOT	DEPTH	DESCRIPTION
			Note: Drilled with an 8" diameter hollow stem rig.
		0-5'	SAND with Silt; very fine to fine grained, medium brown, dry to slightly moist, some medium to coarse grained sand and fine grained gravel.
R	19	5'	SAND with Gravel; very fine to very coarse grained, medium to dark brown, moist, medium dense, clasts sub-rounded to sub-angular and up to 1½" across.
R	77	8'	Gravelly SAND; very fine to medium grained, tannish medium brown, moist, dense, clasts sub-rounded to sub-angular and up to ¾" across, some coarse to very coarse grained sand.
SPT	50 (6")	10'	Gravelly SAND; very fine to very coarse grained, light brown, dry to slightly moist, very dense, clasts sub-angular and up to 2" across.
SPT	60	15'	Gravelly SAND; very fine to very coarse grained, light to dark brown, slightly moist, very dense, clasts sub-rounded to sub-angular and up to 1½" across.
SPT	50 (6")	20'	Gravelly SAND; very fine to very coarse grained, light brown, dry to slightly moist, very dense, clasts sub-rounded to sub-angular and up to 1½" across.
SPT	50 (6")	25'	Gravelly SAND; very fine to very coarse grained, dark brown, moist, very dense, clasts sub-rounded to sub-angular and up to 1½" across, trace clay.

PLATE B-1a

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BORING LOG

Job #: SS-202-S Client: Faith Community Church Address: 355 D Street, Fillmore	Boring #: B-1b Date Excavated: 7/18/14 Logged by: JS

SAMPLE	BLOWS/FOOT	DEPTH	DESCRIPTION
SPT	50 (5")	30'	Gravelly SAND; very fine to very coarse grained, medium brown, slightly moist, very dense, clasts sub-rounded to sub-angular and up to 1" across.
SPT	50 (6")	35'	Gravelly SAND; very fine to very coarse grained, medium brown, slightly moist, very dense, clasts sub-rounded to sub-angular and up to 1" across.
SPT	50 (4")	40'	Gravelly, Cobbley SAND; very fine to very coarse grained, medium brown, moist, very dense, clasts sub-rounded to sub-angular and up to 3" across.
			Total Depth: 40½' Due to Refusal on Cobbles and Possibly Boulders. No Water.

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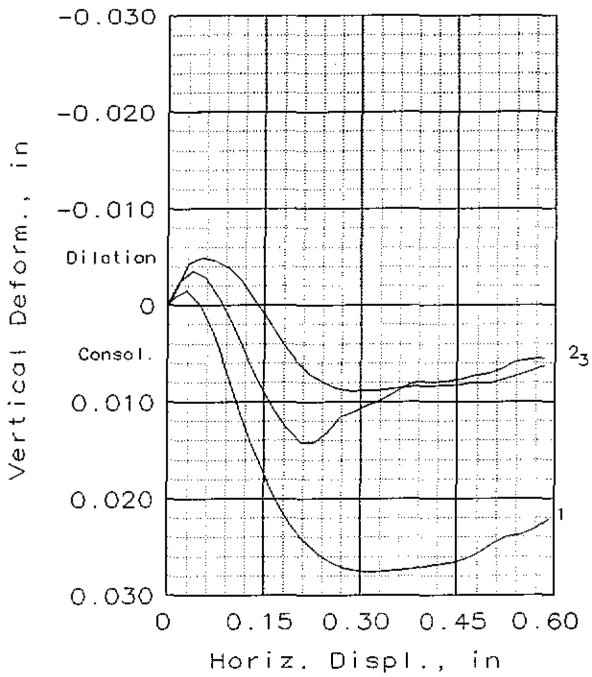
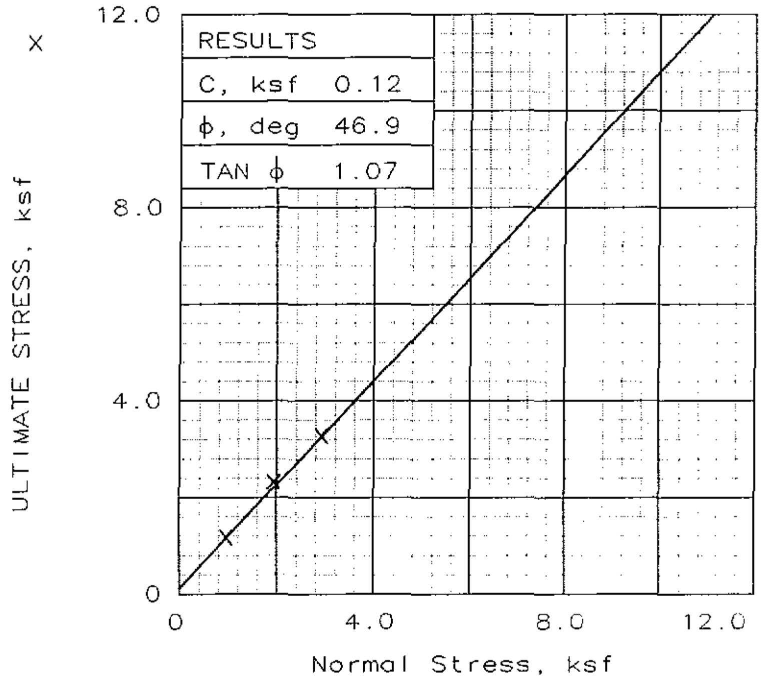
BORING LOG

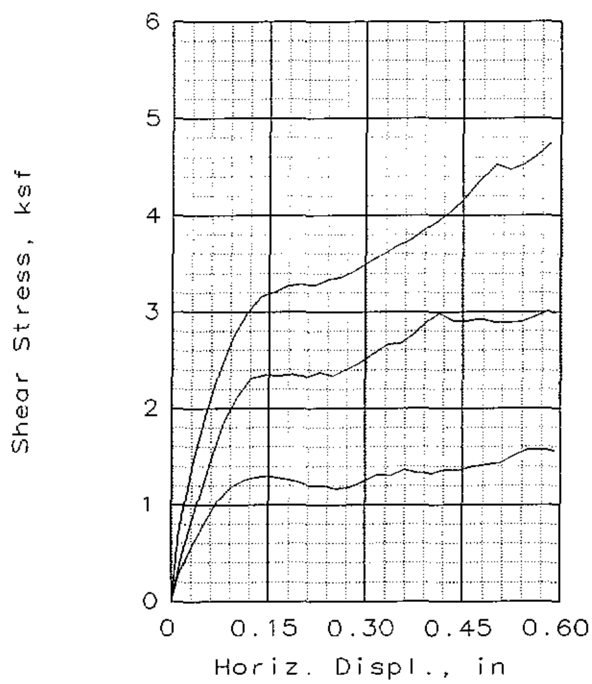
Job #: SS-202-S Client: Faith Community Church Address: 355 D Street, Fillmore	Boring #: B-2 Date Excavated: 7/18/14 Logged by: JS

SAMPLE	BLOWS/FOOT	DEPTH	DESCRIPTION
			Note: Drilled with a 3" diameter hand auger.
		0-2'	SAND; very fine to medium grained, light to medium brown, dry, loose, abundant roots and pinhole voids, some coarse to very coarse grained sand and fine grained gravel.
		2-3'	SAND with Silt; very fine to fine grained, medium brown, dry to slightly moist, loose to medium dense, some medium to coarse grained sand and fine grained gravel.
		3-5¾'	SAND; very fine to medium grained, dark brown, moist to very moist at approx. 5½', medium dense to dense at approx. 4', some silt, trace clay.
		5¾-6'	CLAY; tannish medium brown, moist, stiff.
		6'	Refusal on Gravel & Cobbles.
			Total Depth: 6' Due to Refusal on Gravel & Cobbles. No Water.

PLATE B-2







NORMAL STRESS, ksf	1.03	2.00	2.99
FAILURE STRESS, ksf	1.57	3.01	4.74
DISPLACEMENT, in	0.55	0.58	0.59
ULTIMATE STRESS, ksf	1.18	2.32	3.28
DISPLACEMENT, in	0.28	0.21	0.22
Strain rate, in/min	0.0471	0.0468	0.0464

SAMPLE TYPE: ALLUVIUM	CLIENT: FAITH COMMUNITY CHURCH
DESCRIPTION: SAND	PROJECT:
SPECIFIC GRAVITY= 2.65	SAMPLE LOCATION: B-1 @ 5'
REMARKS:	PROJ. NO.: SS-202-S DATE: 8/4/14
	DIRECT SHEAR TEST REPORT
Fig. No.: S-1	**SOLID SOILS & GEOLOGIC CONSULTANTS**

Soils & Geologic Consultants
solidsoils.com:(805) 202-6533

Subject: Figure 4-5 Reference: Seed et al. (1984) Scale: ——	Client: Faith Community Church Job #: SS-202-S Date: 8/14

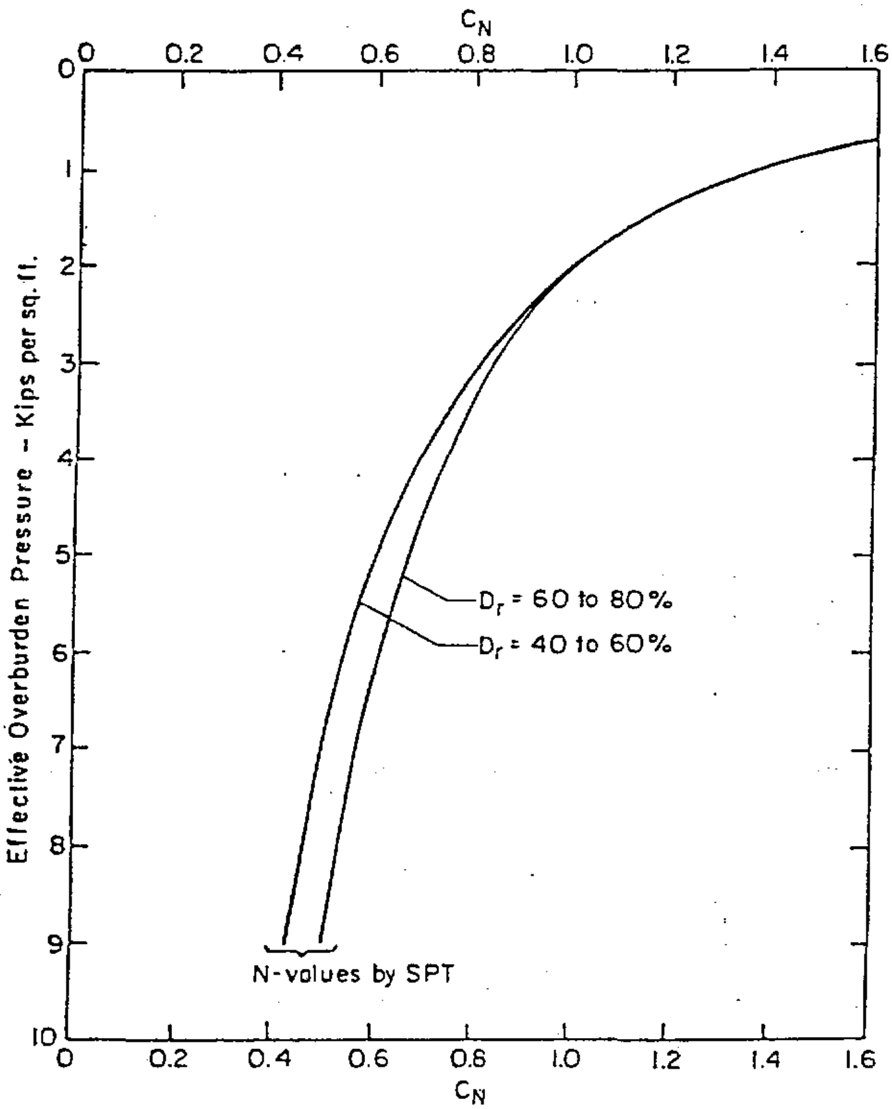


FIGURE 4-5 Charts for values of overburden correcting factor C_N. Source: Seed et al. (1984) based on data and analyses from Marcuson and Bieganousky (1977).

Figure 4-5

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Subject: Figure 4-7 Reference: Seed et al. (1984) Scale: ——	Client: Faith Community Church Job #: SS-202-S Date: 8/14

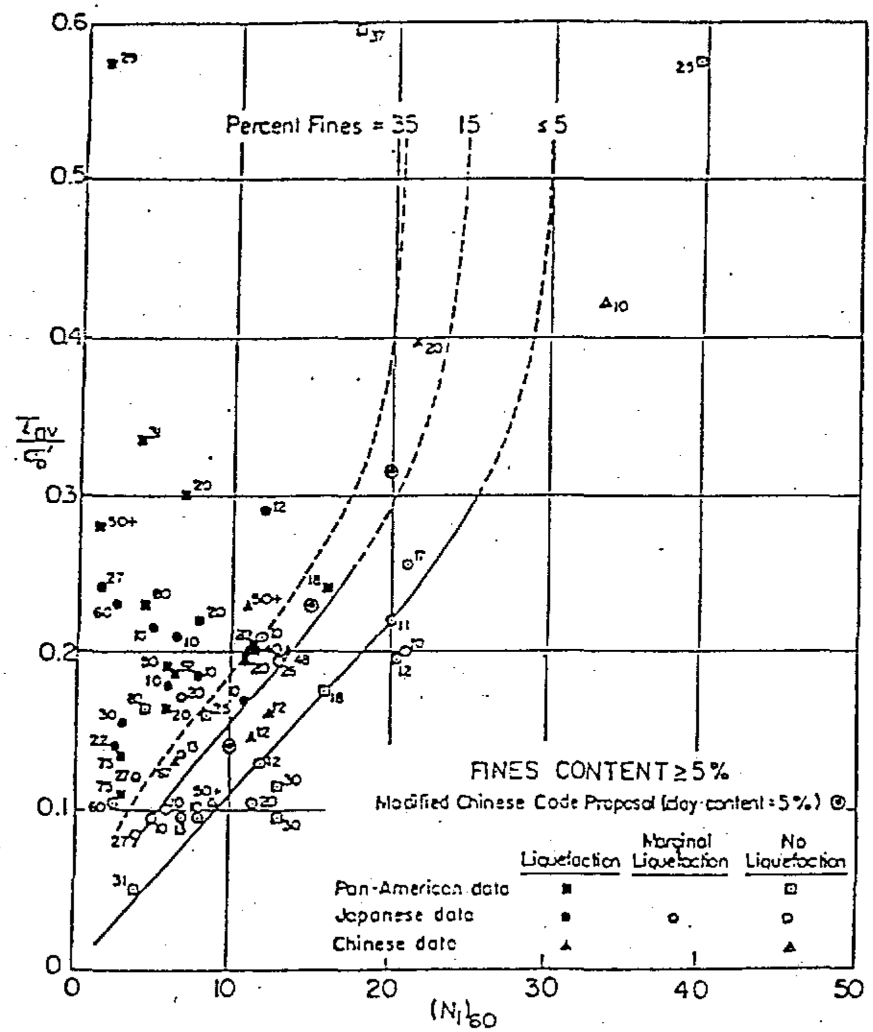

FIGURE 4-7 Relationships between stress ratio causing liquefaction and $(N_1)_{60}$ values for silty sands for magnitude 7.5 earthquakes. Boundary points specified by the Chinese Building Code are shown for comparison. Source: Seed et al. (1984).

Figure 4-7

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Subject: Figure 4-15 Reference: Ishihara (1985) Scale: ——	Client: Faith Community Church Job #: SS-202-S Date: 8/14

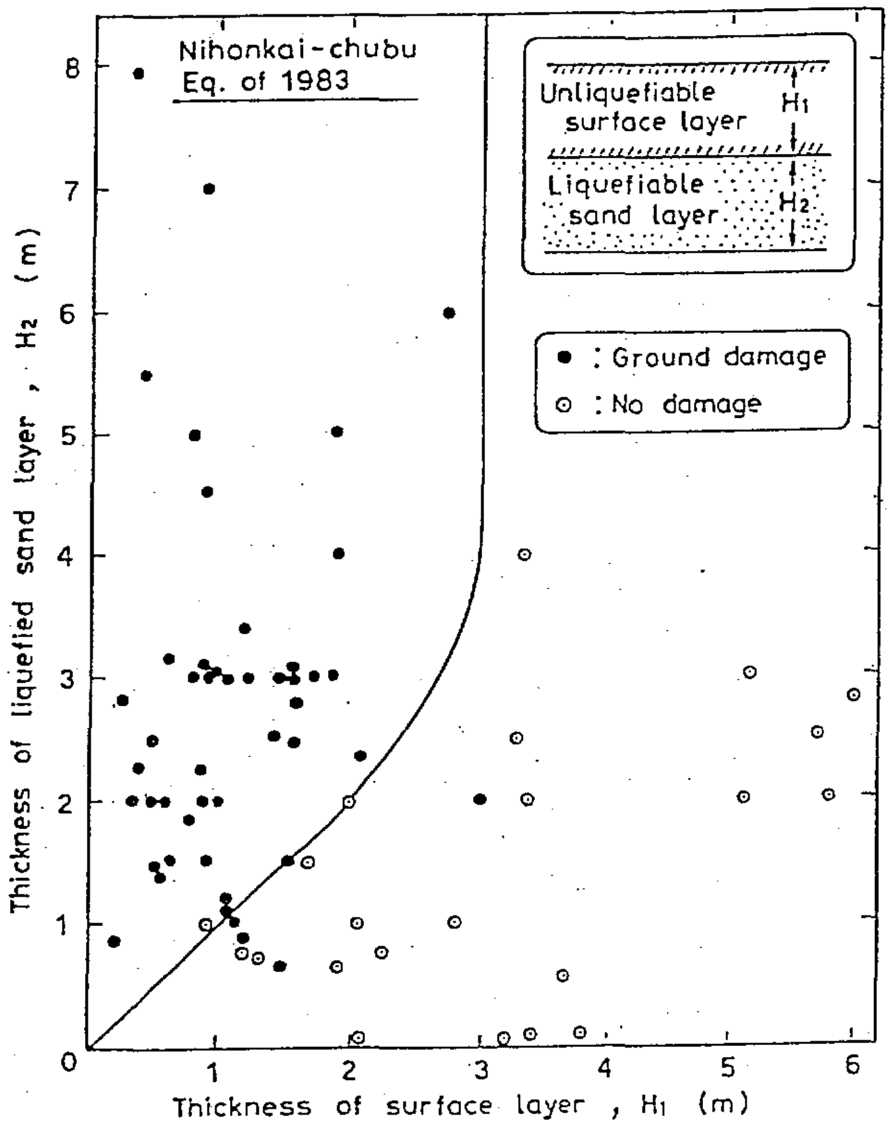


FIGURE 4-15 Conditions of subsurface soil stratification discriminating between occurrence and nonoccurrence of ground rupturing from liquefaction. Source: Ishihara (1985).

AFFECTS OF LIQUEFACTION WITH OVERBURDEN

Figure 4-15

USGS Design Maps Summary Report

User–Specified Input

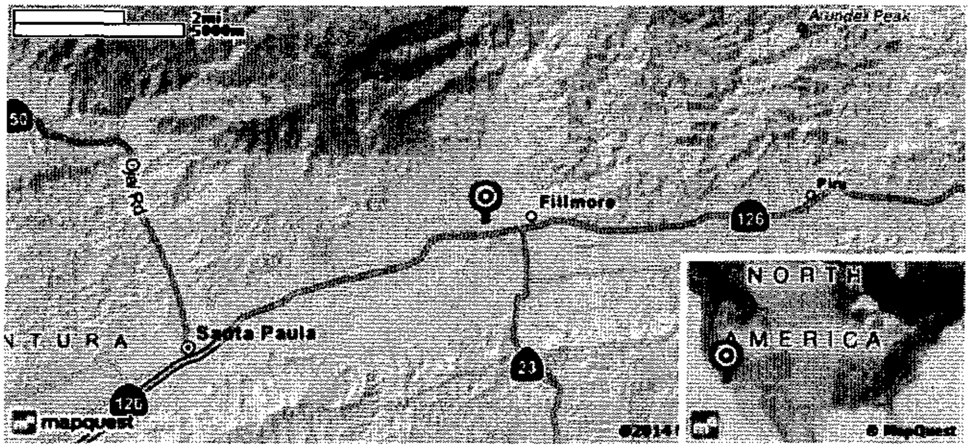
Report Title Faith Community Church
Tue August 5, 2014 18:13:16 UTC

Building Code Reference Document ASCE 7-10 Standard
(which utilizes USGS hazard data available in 2008)

Site Coordinates 34.3976°N, 118.9332°W

Site Soil Classification Site Class D – "Stiff Soil"

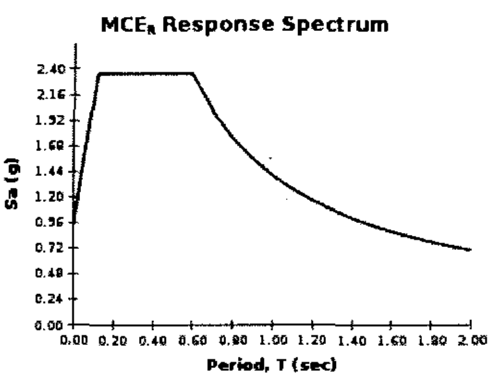
Risk Category I/II/III

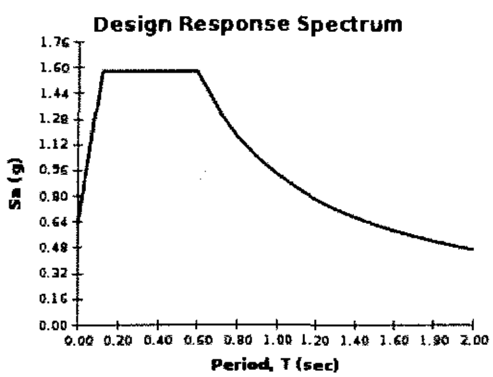


USGS–Provided Output

S_S =	2.360 g	S_{MS} =	2.360 g	S_{DS} =	1.574 g
S_1 =	0.935 g	S_{M1} =	1.402 g	S_{D1} =	0.935 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.

Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.

USGS Design Maps Detailed Report

ASCE 7-10 Standard (34.3976°N, 118.9332°W)

Site Class D – "Stiff Soil", 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] $S_S = 2.360$ g

From Figure 22-2 [2] $S_1 = 0.935$ 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 D, 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 $w \geq 40\%$, 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	1.0
D	1.6	1.4	1.2	1.1	$\boxed{1.0}$
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 = D and S_S = 2.360 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	1.3
D	2.4	2.0	1.8	1.6	$\boxed{1.5}$
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 = D and S_1 = 0.935 g, F_v = 1.500

Equation (11.4–1): $S_{MS} = F_aS_S = 1.000 \times 2.360 = 2.360$ g

Equation (11.4–2): $S_{M1} = F_vS_1 = 1.500 \times 0.935 = 1.402$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

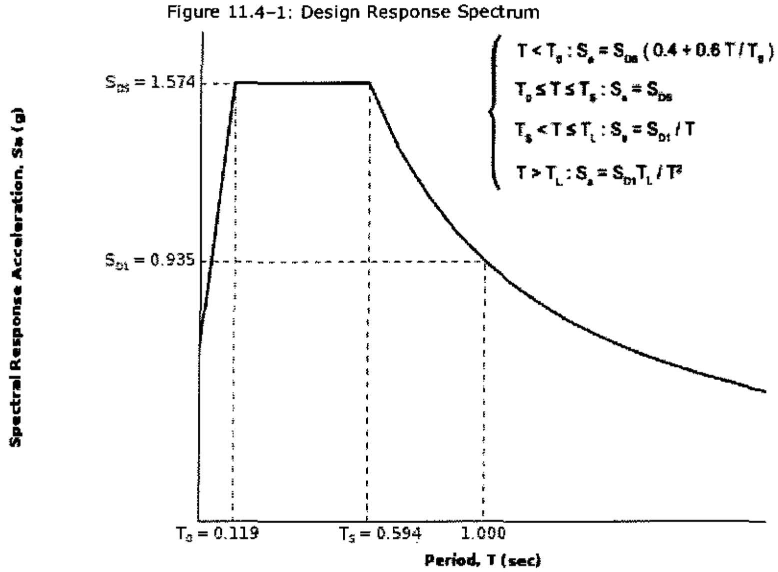
Equation (11.4–3): $S_{DS} = ⅔\, S_{MS} = ⅔ \times 2.360 = 1.574$ g

Equation (11.4–4): $S_{D1} = ⅔\, S_{M1} = ⅔ \times 1.402 = 0.935$ g

Section 11.4.5 — Design Response Spectrum

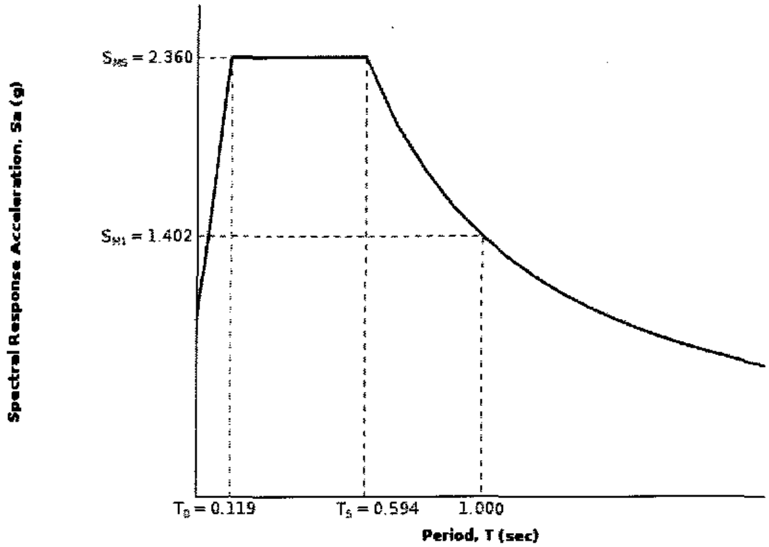
From Figure 22-12 [3] T_L = 8 seconds

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 = 0.899

Equation (11.8-1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.899 = 0.899$ 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	1.0
D	1.6	1.4	1.2	1.1	$\boxed{1.0}$
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 = D and PGA = 0.899 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} = 0.958

From Figure 22-18 [6] C_{R1} = 0.935

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} = 1.574 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} = 0.935 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" = E

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

References

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

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*    E Q F A U L T    *
*                     *
*    Version 3.00     *
*                     *
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DETERMINISTIC ESTIMATION OF
PEAK ACCELERATION FROM DIGITIZED FAULTS

JOB NUMBER: SS-202-S

JOB NAME: Faith Community Church

FAULT-DATA-FILE NAME: CDMGFLTE.DAT

SITE COORDINATES:
SITE LATITUDE: 34.3976
SITE LONGITUDE: 118.9332

SEARCH RADIUS: 100 mi

ATTENUATION RELATION: 14) Campbell & Bozorgnia (1997 Rev.) - Alluvium
UNCERTAINTY (M=Median, S=Sigma): M Number of Sigmas: 0.0
DISTANCE MEASURE: cdist
SCOND: 0
Basement Depth: 5.00 km Campbell SSR: 0 Campbell SHR: 0
COMPUTE PEAK HORIZONTAL ACCELERATION

FAULT-DATA FILE USED: CDMGFLTE.DAT

MINIMUM DEPTH VALUE (km): 3.0

Summary

THE SAN CAYETANO FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 1.3 MILES (2.1 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.7356 g

DETERMINISTIC SITE PARAMETERS

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
SAN CAYETANO	1.3(2.1)	6.8	0.736	XI
OAK RIDGE (Onshore)	3.1(5.0)	6.9	0.617	X
SIMI-SANTA ROSA	8.5(13.6)	6.7	0.339	IX
SANTA SUSANA	9.9(16.0)	6.6	0.276	IX
HOLSER	10.6(17.1)	6.5	0.243	IX
SANTA YNEZ (East)	11.8(19.0)	7.0	0.253	IX
VENTURA - PITAS POINT	12.7(20.4)	6.8	0.238	IX
NORTHRIDGE (E. Oak Ridge)	13.2(21.3)	6.9	0.241	IX
M.RIDGE-ARROYO PARIDA-SANTA ANA	15.0(24.2)	6.7	0.184	VIII
SAN GABRIEL	17.1(27.6)	7.0	0.176	VIII
RED MOUNTAIN	20.8(33.4)	6.8	0.132	VIII
ANACAPA-DUME	21.7(35.0)	7.3	0.175	VIII
MONTALVO-OAK RIDGE TREND	21.9(35.2)	6.6	0.107	VII
CHANNEL IS. THRUST (Eastern)	22.8(36.7)	7.4	0.176	VIII
MALIBU COAST	23.1(37.2)	6.7	0.107	VII
OAK RIDGE(Blind Thrust Offshore)	24.4(39.2)	6.9	0.115	VII
SIERRA MADRE (San Fernando)	26.3(42.4)	6.7	0.090	VII
BIG PINE	27.3(43.9)	6.7	0.080	VII
SAN ANDREAS - 1857 Rupture	28.0(45.0)	7.8	0.188	VIII
SAN ANDREAS - Carrizo	28.0(45.0)	7.2	0.118	VII
GARLOCK (West)	29.9(48.1)	7.1	0.101	VII
SANTA MONICA	30.5(49.1)	6.6	0.068	VI
PLEITO THRUST	31.1(50.0)	7.2	0.104	VII
VERDUGO	31.4(50.5)	6.7	0.071	VI
SAN ANDREAS - Mojave	32.2(51.9)	7.1	0.092	VII
HOLLYWOOD	35.4(56.9)	6.4	0.047	VI
PALOS VERDES	36.5(58.7)	7.1	0.079	VII
SIERRA MADRE	36.8(59.3)	7.0	0.071	VI
SANTA YNEZ (West)	40.4(65.0)	6.9	0.058	VI
NEWPORT-INGLEWOOD (L.A.Basin)	40.7(65.5)	6.9	0.058	VI
NORTH CHANNEL SLOPE	42.0(67.6)	7.1	0.064	VI
WHITE WOLF	43.5(70.0)	7.2	0.066	VI
SANTA CRUZ ISLAND	43.6(70.1)	6.8	0.048	VI
RAYMOND	44.4(71.4)	6.5	0.037	V
COMPTON THRUST	45.4(73.0)	6.8	0.046	VI
ELYSIAN PARK THRUST	48.5(78.0)	6.7	0.038	V
CLAMSHELL-SAWPIT	50.1(80.7)	6.5	0.031	V
WHITTIER	59.5(95.7)	6.8	0.032	V
SANTA ROSA ISLAND	61.3(98.6)	6.9	0.032	V
SAN JOSE	63.8(102.7)	6.5	0.022	IV

DETERMINISTIC SITE PARAMETERS

ABBREVIATED FAULT NAME	APPROXIMATE DISTANCE mi (km)	ESTIMATED MAX. EARTHQUAKE EVENT: MAXIMUM EARTHQUAKE MAG.(Mw)	PEAK SITE ACCEL. g	EST. SITE INTENSITY MOD.MERC.
CUCAMONGA	66.8(107.5)	7.0	0.031	V
LOS ALAMOS-W. BASELINE	67.5(108.7)	6.8	0.026	V
CHINO-CENTRAL AVE. (Elsinore)	70.5(113.4)	6.7	0.023	IV
LIONS HEAD	77.1(124.1)	6.6	0.018	IV
SAN JUAN	78.2(125.9)	7.0	0.027	V
GARLOCK (East)	80.4(129.4)	7.3	0.034	V
SAN LUIS RANGE (S. Margin)	80.4(129.4)	7.0	0.024	IV
SAN ANDREAS - Southern	80.5(129.5)	7.4	0.037	V
SAN ANDREAS - San Bernardino	80.5(129.5)	7.3	0.034	V
NEWPORT-INGLEWOOD (Offshore)	80.5(129.6)	6.9	0.024	IV
SAN JACINTO-SAN BERNARDINO	82.0(131.9)	6.7	0.019	IV
SAN ANDREAS - Cholame	82.5(132.7)	6.9	0.023	IV
ELSINORE-GLEN IVY	82.9(133.4)	6.8	0.021	IV
CLEGHORN	83.9(135.1)	6.5	0.016	IV
CASMALIA (Orcutt Frontal Fault)	87.2(140.4)	6.5	0.014	IV
LENWOOD-LOCKHART-OLD WOMAN SPRGS	87.7(141.2)	7.3	0.030	V
HELENDALE - S. LOCKHARDT	93.1(149.8)	7.1	0.023	IV
So. SIERRA NEVADA	95.5(153.7)	7.1	0.020	IV
NORTH FRONTAL FAULT ZONE (West)	96.0(154.5)	7.0	0.018	IV
CORONADO BANK	96.8(155.8)	7.4	0.029	V
LOS OSOS	98.5(158.6)	6.8	0.015	IV

-END OF SEARCH- 61 FAULTS FOUND WITHIN THE SPECIFIED SEARCH RADIUS.

THE SAN CAYETANO FAULT IS CLOSEST TO THE SITE.
IT IS ABOUT 1.3 MILES (2.1 km) AWAY.

LARGEST MAXIMUM-EARTHQUAKE SITE ACCELERATION: 0.7356 g