# Appendix 7.0

# Geotechnical Report New Church at St. Frances of Rome

Geotechnical Report

# New Church @ St. Frances of Rome

## Wildomar, California

Prepared for:

**Diocese of San Bernardino**
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**April 2016**

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April 25, 2016

Mr. David E. Meir
Diocese of San Bernardino
1201 E. Highland Avenue
San Bernardino, CA 92404

**Geotechnical Report**
**New Church @ St Frances of Rome**
**Wildomar, California**
***LCI Report No. LP16027***

Dear Mr. Meir:

The attached geotechnical report is provided for design and construction of the proposed new church at St Frances of Rome, 21591 Lemon Street, Wildomar, California. Our geotechnical investigation was conducted in response to your request for our services. The enclosed report describes our soil engineering investigation and presents our professional opinions regarding geotechnical conditions at the site.

The findings of this study indicate the site is underlain by interbedded silty sands with traces of gravels and silty sands, with near surface silty sands with traces of gravels. The near surface, silty sands are expected to be low to non-expansive. The subsurface soils are loose to medium dense in nature. Groundwater was not encountered in the borings (51.5 feet) during the time of exploration.

Elevated sulfate and chloride levels were not encountered in the soil samples tested for this study. However, the soil is severely corrosive to metal. We recommend a minimum of 2,500 psi concrete of Type II Portland Cement with a maximum water/cement ratio of 0.60 (by weight) should be used for concrete placed in contact with native soils of this project.

Evaluation of liquefaction potential at the site indicates that it is unlikely that the subsurface soil will liquefy under seismically induced ground shaking since groundwater is believed to be deeper than 50 feet. No mitigation is required for liquefaction effects at this site.

Seismic settlements of the dry sands have been calculated to be approximately ½ to 1 inch based on the field exploration data. Total seismic settlements are not expected to exceed an inch with differential settlements approximately ¼ to ½ inch.

We did not encounter soil conditions that would preclude developing the new church at the site provided the professional opinions contained in this report are implemented in the design and construction of this project. Our findings, professional opinions, and application options are related ***only through reading the full report***, and are best evaluated with the active participation of the engineer of record who developed them.

We appreciate the opportunity to provide our findings and professional opinions regarding geotechnical conditions at the site. If you have any questions or comments regarding our findings, please call our office at (760) 360-0665.

Respectfully Submitted,
***LandMark Consultants, Inc.***

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Section 1
**INTRODUCTION**

## 1.1 Project Description

This report presents the findings of our geotechnical exploration and laboratory evaluation of recovered soils for the proposed new church building located in northern portion of St Frances of Rome, 21591 Lemon Street, Wildomar, California (See Vicinity Map, Plate A-1). The proposed development will consist of 1,200 seats new church building, additional car parking areas and other on-site improvements on the existing complex. A site plan for the proposed development was provided by W.J. Mckeever Inc.

The structure is planned to consist of wood and metal frame construction founded on shallow concrete footings and concrete slabs-on-grade. Footing loads at exterior bearing walls are estimated at 2 to 10 kips per lineal foot. Column loads are estimated to range from 5 to 60 kips. If structural loads exceed those stated above, we should be notified so we may evaluate their impact on foundation settlement and bearing capacity. Site development will include mass grading, building pad preparation, underground utility installation, parking lots construction, sidewalk placement, landscape areas and retention basins.

## 1.2 Purpose and Scope of Work

The purpose of this geotechnical study was to investigate the upper 11.5 to 51.5 feet of subsurface soil at selected locations within the site for evaluation of in-situ soil strength and physical/engineering properties. Professional opinions report regarding geotechnical conditions at this site and the effect on design and construction were developed from field exploration and laboratory evaluation of recovered soils. The scope of our services consisted of the following:

- Field exploration and in-situ testing of the site soils at selected locations and depths.
- Laboratory testing for physical and/or chemical properties of selected recovered soil samples.
- Review of literature and publications pertaining to local geology, faulting, and seismicity.
- Engineering analysis and evaluation of the data collected.

< Preparation of this report presenting our findings and professional opinion regarding the geotechnical aspects of project design and construction.

This report addresses the following geotechnical parameters:

< Subsurface soil and groundwater conditions
< Site geology, regional faulting and seismicity, near-source seismic factors, and site seismic accelerations
< Liquefaction potential
< Hydro-Collapse potential
< Expansive soil and methods of mitigation
< Aggressive soil conditions to metals and concrete
< Soil percolation rates of the native soil for retention basin areas

Professional opinions with regard to the above parameters are presented for the following:

< Mass grading and earthwork
< Building pad and foundation subgrade preparation
< Allowable soil bearing pressures and expected settlements
< Deep Foundations (drilled piers)
< Concrete slabs-on-grade
< Mitigation of the potential effects of salt concentrations in native soil to concrete mixes and steel reinforcement
< Excavation conditions and buried utility installations
< Lateral earth pressures
< Seismic design parameters
< Preliminary Pavement structural sections

Our scope of work for this report did not include an evaluation of the site for the presence of environmentally hazardous materials or conditions.

## 1.3 Authorization

Mr. David E. Meir of the Diocese of San Bernardino provided authorization by written agreement to proceed with our work on February 19, 2016. We conducted our work according to our written proposal dated January 27, 2016.

Section 2
# METHODS OF INVESTIGATION

## 2.1 Field Exploration

Subsurface exploration was performed on March 15, 2016 using 2R Drilling of Ontario California to advance five (5) borings to depths of 11.5 to 51.5 feet below existing ground surface. The borings were advanced with a truck-mounted, CME 75 drill rig using 8-inch diameter, hollow-stem, continuous-flight augers. The approximate boring locations were established in the field and plotted on the site map by sighting to discernable site features. The boring locations are shown on the Site and Exploration Plan (Plate A-2).

A staff engineer observed the drilling operations and maintained a log of the soil encountered and sampling depths, visually classified the soil encountered during drilling in accordance with the Unified Soil Classification System, and obtained drive tube and bulk samples of the subsurface materials at selected intervals. Relatively undisturbed soil samples were retrieved using a 2-inch outside diameter (OD) split-spoon sampler or a 3-inch OD Modified California Split-Barrel (ring) sampler. The samples were obtained by driving the sampler ahead of the auger tip at selected depths.

The drill rig was equipped with a 140-pound CME automatic hammer with a 30-inch drop for conducting Standard Penetration Tests (SPT) in accordance with ASTM D1586. The number of blows required to drive the samplers the last 12 inches of an 18 inch drive length into the soil is recorded on the boring logs as "blows per foot". Blow count reported on the boring logs represent the field blow counts. No corrections have been applied for effects of overburden pressure, automatic hammer drive energy, drill rod lengths, liners, and sampler diameter.

After logging and sampling the soil, the exploratory borings were backfilled with the excavated material. The backfill was loosely placed and was not compacted to the requirements specified for engineered fill.

The subsurface logs are presented on Plates B-1 through B-6 in Appendix B. A key to the log symbols is presented on Plate B-7. The stratification lines shown on the subsurface logs represent the approximate boundaries between the various strata. However, the transition from one stratum to another may be gradual over some range of depth.

## 2.2 Laboratory Testing

Laboratory tests were conducted on selected bulk and relatively undisturbed soil samples to aid in classification and evaluation of selected engineering properties of the site soils. The tests were conducted in general conformance to the procedures of the American Society for Testing and Materials (ASTM) or other standardized methods as referenced below. The laboratory testing program consisted of the following tests:

< Particle Size Analyses (ASTM D422) – used for soil classification and liquefaction evaluation.

< Unit Dry Densities (ASTM D2937) and Moisture Contents (ASTM D2216) – used for insitu soil parameters.

< Moisture-Density Relationship (ASTM D1557) – used for soil compaction determinations.

< Direct Shear (ASTM D3080) – used for soil strength determination.

< Chemical Analyses (soluble sulfates & chlorides, pH, and resistivity) (Caltrans Methods) – used for concrete mix evaluations and corrosion protection requirements.

The laboratory test results are presented on the subsurface logs and on Plates C-1 through C-4 in Appendix C.

Engineering parameters of soil strength, compressibility and relative density utilized for developing design criteria provided within this report were either extrapolated from data obtained from the field and laboratory testing program.

Section 3
## DISCUSSION

### 3.1 Site Conditions

The project site is rectangular-shaped in plan view, is relatively flat-lying slopes gently to the north, and consists of approximately 9.5 acres of existing St Frances of Rome wordship complex. The site is bounded by Lemon Street to the north and Orchird Street to the west. Residential homes are surrounding the complex and these properties are flat-lying and are approximately at the same elevation with this site.

The project site lies at an elevation between approximately 1,330 and 1,345 feet above mean sea level (MSL) in the French Valley of Southern California. Annual average rainfall in this region is approximately 11 inches with average summertime temperature highs above 90ºF and lows in the mid 50's to low 60's. Average winter temperature highs are in the high 60's with lows in mid 30's to low 40's.

### 3.2 Geologic Setting

The project site is located within the French Valley, which is located to the east/northeast of the Elsinore-Temecula Trough and to the south of the Perris Plain within the Peninsular Ranges geomorphic province. The Peninsular Ranges are one of the largest geologic units in western North America. They extend 200 kilometers (125 miles) from the Transverse Ranges and the Los Angeles Basin south to the Mexican border and beyond another 1,250 kilometers (775 miles) to the tip of Baja California. The total province varies in width from 48 to 160 Kilometers (30-100 miles) (Norris & Webb, 1976).

The Peninsular Ranges are a northwest-southeast oriented complex of blocks separated by similarly trending faults (Norris & Webb, 1976). Major faults of the Peninsular Ranges are the San Jacinto and related branches within the San Jacinto zone and the Elsinore and associated faults within the Elsinore zone.

The Elsinore-Temecula trough, located to the west/southwest of the project site, is a linear, low-lying block northeast of the Santa Ana Mountains and southwest of the Perris Plain. It extends from

Corona on the northwest about 30 miles (48 km) southeast and has a maximum width of 3 miles (4.8 km). The Perris Plain, located to the north of the project site, is a major topographic feature between the San Jacinto (northeast) and Elsinore (southwest) fault zones. The plain is a broad, nearly flat surface dotted with bedrock hills extending from near Corona southeasterly to Hemet. The average elevation of the Perris Plain is 520 meters (1,700 feet) (Norris & Webb, 1976). The nearby hills to the project site are composed of Mesozoic granitic rocks, Mesozoic intrusive rocks, and upper Jurassic marine rocks. Figure 1 shows the location of the site in relation to regional faults and physiographic features.

The surrounding regional geology includes the San Jacinto and Santa Rosa Mountains to the east/southeast, the Santa Ana Mountains to the west/northwest, the Elsinore Fault zone to the southwest, and the San Jacinto Fault zone to the northeast. Lake Elsinore is located to the west of the project site.

### 3.3 Subsurface Soil

Subsurface soils encountered during the field exploration conducted on March 15, 2016 consist of dominantly medium dense to dense, silty sands (SM) to a depth of 51.5 feet, the maximum depth of exploration. The near surface soils are granular and non-expansive in nature. The subsurface logs (Plates B-1 through B-6) depict the stratigraphic relationships of the various soil types.

### 3.4 Groundwater

Groundwater was not encountered in the borings during the time of exploration. Groundwater levels may fluctuate with precipitation, irrigation of adjacent properties, drainage, and site grading. The groundwater level noted should not be interpreted to represent an accurate or permanent condition. Based on the regional topography, groundwater flow is assumed to be generally towards the east to southeast within the site area. Flow directions may vary locally in the vicinity of the site.

Historic groundwater records in the vicinity of the project site indicate that groundwater has fluctuated between 10 to 31 feet below the ground surface within the past 40 years according to The California Department of Water Resources, Division of Planning and Local Assistance web site.

### 3.5 Faulting

The project site is located in the seismically active French Valley of southern California with numerous mapped faults of the Elsinore Fault Zone traversing the region. We have performed a computer-aided search of known faults or seismic zones that lie within a 62 mile (100 kilometer) radius of the project site (Table 1).

A fault map illustrating known active faults relative to the site is presented on Figure 1, *Regional Fault Map*. Figure 2 shows the project site in relation to local faults. The criterion for fault classification adopted by the California Geological Survey defines Earthquake Fault Zones along active or potentially active faults. An active fault is one that has ruptured during Holocene time (roughly within the last 11,000 years). A fault that has ruptured during the last 1.8 million years (Quaternary time), but has not been proven by direct evidence to have not moved within Holocene time is considered to be potentially active. A fault that has not moved during Quaternary time is considered to be inactive.

Review of the current Alquist-Priolo Earthquake Fault Zone maps (CGS, 2000a) indicates that the nearest mapped Earthquake Fault Zone is the Elsinore-Temecula fault located approximately 1.5 miles southwest of the project site. Riverside County fault maps indicate that the nearest Riverside County mapped fault is the Glen Ivy segment of the Elsinore Fault Zone located approximately 0.2 miles southwest of the project site. A portion of the project site lies within the County Fault Zone boundary and may require additional evaluation.

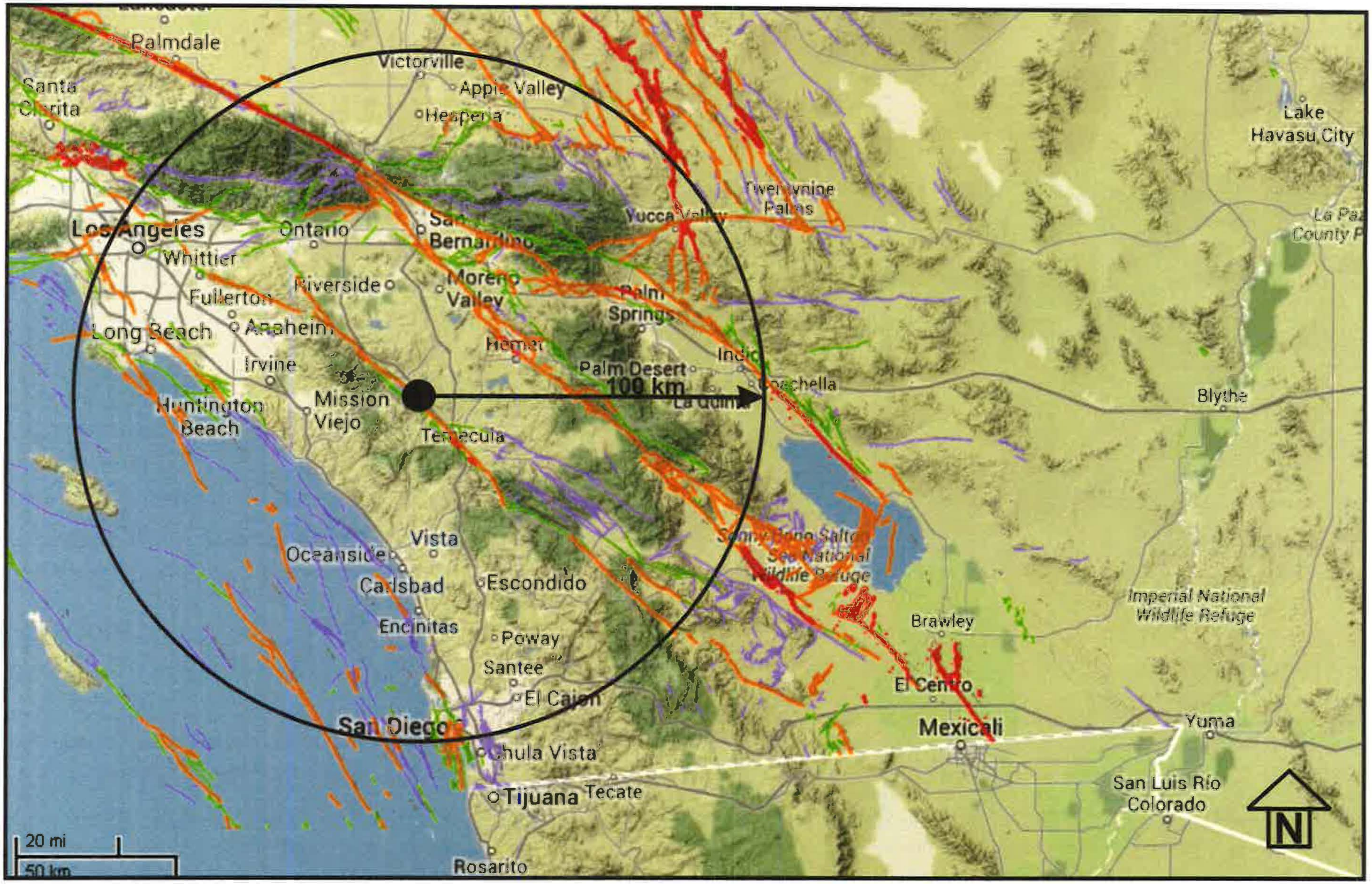
### 3.6 General Ground Motion Analysis

The project site is considered likely to be subjected to moderate to strong ground motion from earthquakes in the region. Ground motions are dependent primarily on the earthquake magnitude and distance to the seismogenic (rupture) zone. Acceleration magnitudes also are dependent upon attenuation by rock and soil deposits, direction of rupture and type of fault; therefore, ground motions may vary considerably in the same general area.

**Table 1**
**Summary of Characteristics of Closest Known Active Faults**

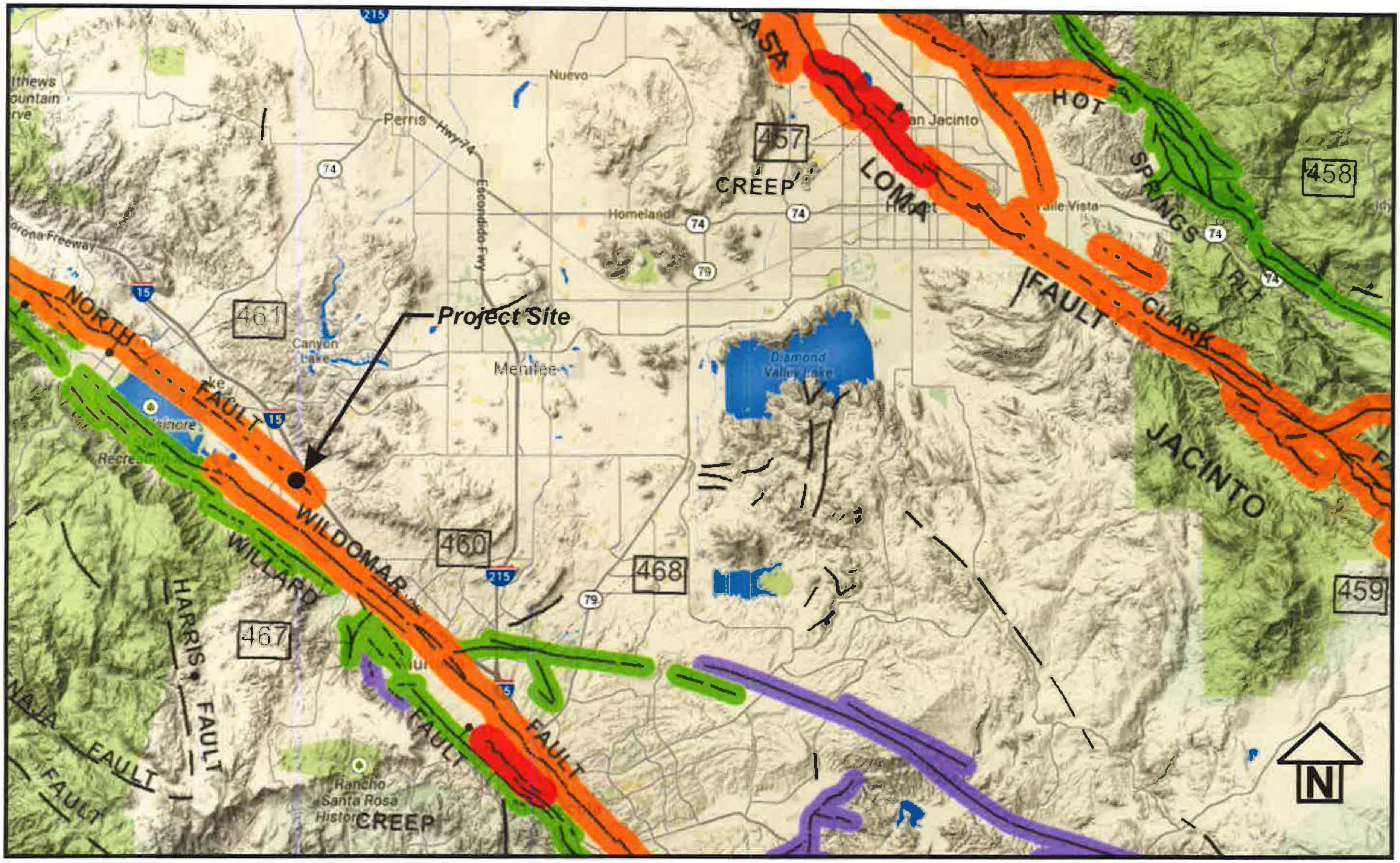
| Fault Name | Approximate Distance (miles) | Approximate Distance (km) | Maximum Moment Magnitude (Mw) | Fault Length (km) | Slip Rate (mm/yr) |
|---|---|---|---|---|---|
| Elsinore - Glen Ivy | 0.2 | 0.3 | 6.8 | 36 ± 4 | 5 ± 2 |
| Elsinore - Temecula | 1.5 | 2.4 | 6.8 | 43 ± 4 | 5 ± 2 |
| Whittier | 16.8 | 26.9 | 6.8 | 38 ± 4 | 2.5 ± 1 |
| Chino Avenue | 18.5 | 29.6 | 6.7 | 28 ± 3 | 1 ± 1 |
| San Jacinto - San Jacinto Valley | 22.0 | 35.1 | 6.9 | 43 ± 4 | 12 ± 6 |
| San Jacinto - Anza | 22.1 | 35.4 | 7.2 | 91 ± 9 | 12 ± 6 |
| San Joaquin Hills | 22.4 | 35.8 | 6.6 | 28 ± 3 | 0.5 ± 0.2 |
| Elsinore - Julian | 24.0 | 38.5 | 7.1 | 76 ± 8 | 5 ± 2 |
| San Jacinto - San Bernardino | 26.8 | 42.9 | 6.7 | 36 ± 4 | 12 ± 6 |
| Newport-Inglewood (offshore) | 28.8 | 46.0 | 7.1 | 66 ± 7 | 1.5 ± 0.5 |
| San Andreas - San Bernardino (South) | 34.1 | 54.6 | 7.4 | 103 ± 10 | 30 ± 7 |
| Rose Canyon | 35.1 | 56.1 | 7.2 | 70 ± 7 | 1.5 ± 0.5 |
| Newport-Inglewood | 36.7 | 58.7 | 7.1 | 66 ± 7 | 1 ± 0.5 |
| Cucamonga | 39.0 | 62.4 | 6.9 | 28 ± 3 | 5 ± 2 |
| Puente Hills Blind Thrust | 39.4 | 63.1 | 7.1 | 44 ± 4 | 0.7 ± 0.4 |
| Garnet Hill * | 40.0 | 64.1 | | | |
| San Jose | 40.4 | 64.6 | 6.4 | 20 ± 2 | 0.5 ± 0.5 |
| Sierra Madre | 42.7 | 68.4 | 7.2 | 57 ± 6 | 2 ± 1 |
| Pinto Mtn. | 43.7 | 69.9 | 7.2 | 74 ± 7 | 2.5 ± 2 |
| Cleghorn | 44.4 | 71.0 | 6.5 | 25 ± 3 | 3 ± 2 |
| Coronado Bank | 44.9 | 71.9 | 7.6 | 185 ± 19 | 3 ± 1 |
| Palos Verdes | 45.0 | 72.1 | 7.3 | 96 ± 10 | 3 ± 1 |

* Note: Faults not included in CGS database.




Source: California Geological Survey 2010 Fault Activity Map of California
http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html#

| | | |
|---|---|---|
| LANDMARK Geo-Engineers and Geologists<br>Project No.: LP16027 | Regional Fault Map | Figure 1 |




Source: California Geological Survey 2010 Fault Activity Map of California
http://www.quake.ca.gov/gmaps/FAM/faultactivitymap.html#

| | | |
|---|---|---|
| LANDMARK Geo-Engineers and Geologists<br>Project No.: LP16027 | Map of Local Faults | Figure 2 |

# EXPLANATION

Fault traces on land are indicated by solid lines where well located, by dashed lines where approximately located or inferred, and by dotted lines where concealed by younger rocks or by lakes or bays. Fault traces are queried where continuation or existence is uncertain. Concealed faults in the Great Valley are based on maps of selected subsurface horizons, so locations shown are approximate and may indicate structural trend only. All offshore faults based on seismic reflection profile records are shown as solid lines where well defined, dashed where inferred, queried where uncertain.

## FAULT CLASSIFICATION COLOR CODE
(Indicating Recency of Movement)

Fault along which historic (last 200 years) displacement has occurred and is associated with one or more of the following:

(a) a recorded earthquake with surface rupture. (Also included are some well-defined surface breaks caused by ground shaking during earthquakes, e.g. extensive ground breakage, not on the White Wolf fault, caused by the Arvin-Tehachapi earthquake of 1952). The date of the associated earthquake is indicated. Where repeated surface ruptures on the same fault have occurred, only the date of the latest movement may be indicated, especially if earlier reports are not well documented as to location of ground breaks.

(b) fault creep slippage - slow ground displacement usually without accompanying earthquakes.

(c) displaced survey lines.

A triangle to the right or left of the date indicates termination point of observed surface displacement. Solid red triangle indicates known location of rupture termination point. Open black triangle indicates uncertain or estimated location of rupture termination point.

Date bracketed by triangles indicates local fault break.

No triangle by date indicates an intermediate point along fault break.

Fault that exhibits fault creep slippage. Hachures indicate linear extent of fault creep. Annotation (creep with leader) indicates representative locations where fault creep has been observed and recorded.

Square on fault indicates where fault creep slippage has occured that has been triggered by an earthquake on some other fault. Date of causative earthquake indicated. Squares to right and left of date indicate terminal points between which triggered creep slippage has occurred (creep either continuous or intermittent between these end points).

Holocene fault displacement (during past 11,700 years) without historic record. Geomorphic evidence for Holocene faulting includes sag ponds, scarps showing little erosion, or the following features in Holocene age deposits: offset stream courses, linear scarps, shutter ridges, and triangular faceted spurs. Recency of faulting offshore is based on the interpreted age of the youngest strata displaced by faulting.

Late Quaternary fault displacement (during past 700,000 years). Geomorphic evidence similar to that described for Holocene faults except features are less distinct. Faulting may be younger, but lack of younger overlying deposits precludes more accurate age classification.

Quaternary fault (age undifferentiated). Most faults of this category show evidence of displacement sometime during the past 1.6 million years; possible exceptions are faults which displace rocks of undifferentiated Plio-Pleistocene age. Unnumbered Quaternary faults were based on Fault Map of California, 1975. See Bulletin 201, Appendix D for source data.

Pre-Quaternary fault (older that 1.6 million years) or fault without recognized Quaternary displacement. Some faults are shown in this category because the source of mapping used was of reconnaissnce nature, or was not done with the object of dating fault displacements. Faults in this category are not necessarily inactive.

## ADDITIONAL FAULT SYMBOLS

Bar and ball on downthrown side (relative or apparent).

Arrows along fault indicate relative or apparent direction of lateral movement.

Arrow on fault indicates direction of dip.

Low angle fault (barbs on upper plate). Fault surface generally dips less than 45° but locally may have been subsequently steepened. On offshore faults, barbs simply indicate a reverse fault regardless of steepness of dip.

## OTHER SYMBOLS

Numbers refer to annotations listed in the appendices of the accompanying report. Annotations include fault name, age of fault displacement, and pertinent references including Earthquake Fault Zone maps where a fault has been zoned by the Alquist-Priolo Earthquake Fault Zoning Act. This Act requires the State Geologist to delineate zones to encompass faults with Holocene displacement.

Structural discontinuity (offshore) separating differing Neogene structural domains. May indicate discontinuities between basement rocks.

Brawley Seismic Zone, a linear zone of seismicity locally up to 10 km wide associated with the releasing step between the Imperial and San Andreas faults.

| Geologic Time Scale | | | Years Before Present (Approx.) | Fault Symbol | Recency of Movement | DESCRIPTION: ON LAND | DESCRIPTION: OFFSHORE |
|---|---|---|---|---|---|---|---|
| Quaternary | Late Quaternary | Historic | | | | Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep. | |
| | | | 200 | | | | |
| | | Holocene | | | | Displacement during Holocene time | Fault offsets seafloor sediments or strata of Holocene age |
| | | | 11,700 | | | | |
| | | Pleistocene | | | | Faults showing evidence of displacement during late Quaternary time | Fault cuts strata of Late Pleistocene age |
| | | | 700,000 | | | | |
| | Early Quaternary | | | | | Undivided Quaternary faults - most faults in this category show evidence of displacement during the last 1,600,000 years, possible exceptions are faults which displace rocks of undifferentiated Plio Pleistocene age | Fault cuts strata of Quaternary age |
| | | | 1,600,000 | | | | |
| Pre-Quaternary | | | | | | Faults without recognized Quaternary displacement or showing evidence of no displacement during Quaternary time. Not necessarily inactive | Fault cuts strata of Pliocene or older age |
| | | | 4.5 billion (Age of Earth) | | | | |

* Quaternary now recognized as extending to 2.6 Ma (Walker and Geissman, 2009). Quaternary faults in this map were established using the previous 1.6 Ma criterion.

CBC General Ground Motion Parameters: The 2013 CBC general ground motion parameters are based on the Risk-Targeted Maximum Considered Earthquake ($MCE_R$). The U.S. Geological Survey "U.S. Seismic Design Maps Web Application" (USGS, 2014) was used to obtain the site coefficients and adjusted maximum considered earthquake spectral response acceleration parameters. The site soils have been classified as Site Class D (stiff soil profile). Design spectral response acceleration parameters are defined as the earthquake ground motions that are two-thirds (2/3) of the corresponding $MCE_R$ ground motions. Design earthquake ground motion parameters are provided in Table 2. ***A Risk Category II was determined using Table 1604.5 and the Seismic Design Category is E since $S_1$ is greater than 0.75.***

The Maximum Considered Earthquake Geometric Mean ($MCE_G$) peak ground acceleration ($PGA_M$) value was determined from the "U.S. Seismic Design Maps Web Application" (USGS, 2013) for liquefaction and seismic settlement analysis in accordance with 2013 CBC Section 1803.5.12 and CGS Note 48 ($PGA_M = F_{PGA}*PGA$). ***A $PGA_M$ value of 0.94g has been determined for the project site.***

### 3.7 Seismic and Other Hazards

► **Groundshaking.** The primary seismic hazard at the project site is the potential for strong groundshaking during earthquakes along the Temecula Segment of the Elsinore Fault Zone. A further discussion of groundshaking follows in Section 3.4.

► **Surface Rupture.** The project site does not lie within a State of California, Alquist-Priolo Earthquake Fault Zone. The project site lies within the Riverside County designated fault zone for the Glen Ivy fault segment of the Elsinore Fault. Surface fault rupture is considered to be unlikely at the project site because of the well-delineated fault lines through the French Valley as shown on USGS, CDMG, and Riverside County maps. However, because of the high tectonic activity and deep alluvium of the region, we cannot preclude the potential for surface rupture on undiscovered or new faults that may underlie the site.

► **Liquefaction.** Liquefaction is unlikely to be a potential hazard at the site, due to groundwater deeper than 50 feet (the maximum depth that liquefaction is known to occur).

# Table 2
## 2013 California Building Code (CBC) and ASCE 7-10 Seismic Parameters

| | | | CBC Reference |
|---|---|---|---|
| Soil Site Class: | **D** | | Table 20.3-1 |
| Latitude: | 33.6333 | N | |
| Longitude: | -117.2828 | W | |
| Risk Category: | II | | |
| Seismic Design Category: | E | | |

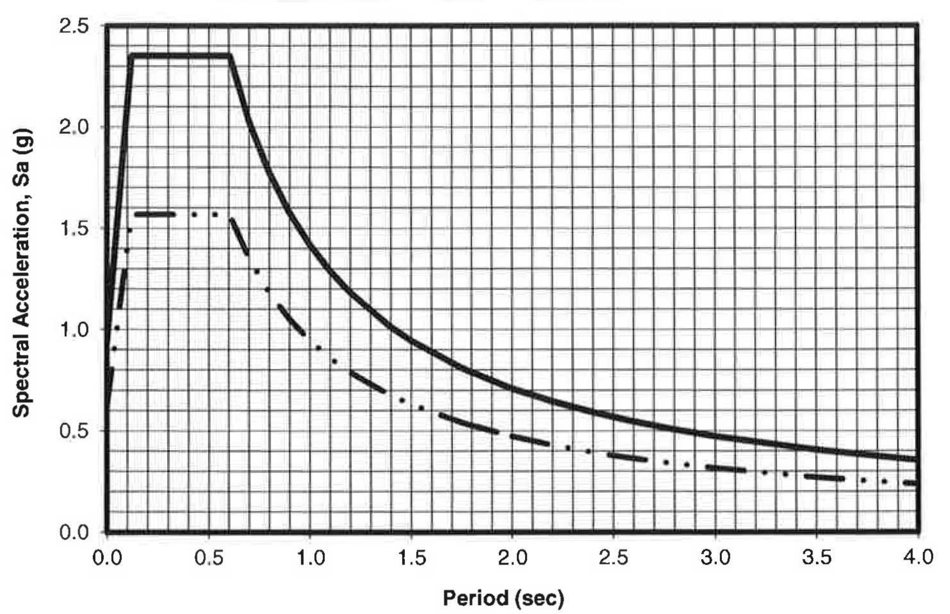
### Maximum Considered Earthquake (MCE) Ground Motion

| | | | | |
|---|---|---|---|---|
| Mapped $MCE_R$ Short Period Spectral Response | $S_s$ | 2.351 g | Figure 1613.3.1(1) | |
| Mapped $MCE_R$ 1 second Spectral Response | $S_1$ | 0.946 g | Figure 1613.3.1(2) | |
| Short Period (0.2 s) Site Coefficient | $F_a$ | 1.00 | Table 1613.3.3(1) | |
| Long Period (1.0 s) Site Coefficient | $F_v$ | 1.50 | Table 1613.3.3(2) | |
| $MCE_R$ Spectral Response Acceleration Parameter (0.2 s) | $S_{MS}$ | 2.351 g | $= F_a * S_s$ | Equation 16-37 |
| $MCE_R$ Spectral Response Acceleration Parameter (1.0 s) | $S_{M1}$ | 1.419 g | $= F_v * S_1$ | Equation 16-38 |

### Design Earthquake Ground Motion

| | | | | |
|---|---|---|---|---|
| Design Spectral Response Acceleration Parameter (0.2 s) | $S_{DS}$ | 1.567 g | $= 2/3*S_{MS}$ | Equation 16-39 |
| Design Spectral Response Acceleration Parameter (1.0 s) | $S_{D1}$ | 0.946 g | $= 2/3*S_{M1}$ | Equation 16-40 |
| | $T_L$ | 8.00 sec | | ASCE Figure 22-12 |
| | $T_O$ | 0.12 sec | $=0.2*S_{D1}/S_{DS}$ | |
| | $T_S$ | 0.60 sec | $=S_{D1}/S_{DS}$ | |
| Peak Ground Acceleration | $PGA_M$ | 0.94 g | | ASCE Equation 11.8-1 |

**Generalized Design Response Spectrum (ASCE 7-10 Section 11.4.5)**




| Period T (sec) | Sa (g) | $MCE_R$ Sa (g) |
|---|---|---|
| 0.00 | 0.63 | 0.94 |
| 0.12 | 1.57 | 2.35 |
| 0.60 | 1.57 | 2.35 |
| 0.70 | 1.35 | 2.03 |
| 0.80 | 1.18 | 1.77 |
| 0.90 | 1.05 | 1.58 |
| 1.00 | 0.95 | 1.42 |
| 1.10 | 0.86 | 1.29 |
| 1.20 | 0.79 | 1.18 |
| 1.20 | 0.79 | 1.18 |
| 1.40 | 0.68 | 1.01 |
| 1.50 | 0.63 | 0.95 |
| 1.75 | 0.54 | 0.81 |
| 2.00 | 0.47 | 0.71 |
| 2.20 | 0.43 | 0.65 |
| 2.40 | 0.39 | 0.59 |
| 2.60 | 0.36 | 0.55 |
| 2.80 | 0.34 | 0.51 |
| 3.00 | 0.32 | 0.47 |
| 3.50 | 0.27 | 0.41 |
| 4.00 | 0.24 | 0.35 |

Design Response Spectra

$MCE_R$ Response Spectra

Other Potential Geologic Hazards.

► **Landsliding.** The hazard of landsliding is unlikely due to the regional planar topography. No ancient landslides are shown on geologic maps of the region and no indications of landslides were observed during our site investigation.

► **Volcanic hazards.** The site is not located in proximity to any known volcanically active area and the risk of volcanic hazards is considered very low.

► **Tsunamis, sieches, and flooding.** The site does not lie near any large bodies of water, so the threat of tsunami, sieches, or other seismically-induced flooding is unlikely.

► **Expansive soil.** The near surface soils at the project site consist of silty sands which are non-expansive.

### 3.8 Seismic Settlement

An evaluation of the non-liquefaction seismic settlement potential was performed using the relationships developed by Tokimatsu and Seed (1984, 1987) for dry sands. This method is an empirical approach to quantify seismic settlement using SPT blow counts and PGA estimates from the probabilistic seismic hazard analysis.

The soils beneath the site consist primarily of loose to medium dense silty sands to maximum penetrated. Based on the empirical relationships, total induced settlements are estimated to be on the order or ½ to1 inch in the event of a $MCE_G$ earthquake (0.94g peak ground acceleration). Should settlement occur, buried utility lines and the buildings may not settle equally. Therefore we recommend that utilities, especially at the points of entry to the buildings, be designed to accommodate differential movement.

The computer printouts for the estimates of induced settlement are included in Appendix D.

### 3.9 Hydroconsolidation

In arid climatic regions, granular soils have a potential to collapse upon wetting. This collapse (hydroconsolidation) phenomena is the result of the lubrication of soluble cements (carbonates) in the soil matrix causing the soil to densify from its loose configuration during deposition.

Based on our experience in the vicinity of the project site, there is a slight risk of collapse upon inundation from at the site. Therefore, development of building foundation is not required to include provisions for mitigating the hydroconsolidation caused by soil saturation from landscape irrigation or broken utility lines.

### 3.10 Soil Infiltration Rate

A total of four (4) infiltration tests were conducted on March 18, 2016 at the proposed location for the on-site storm-water retention basins as shown on the Site and Exploration Plan (Plate A-2). The infiltration tests were performed to the guideline from Design Handbook for Low Impact Development Best Management Practices, prepared by Riverside County Flood Control and Water Conservation District, Appendix A, Section 2.3, dated September 2011.

The tests were performed using perforated pipes inside an 8-inch diameter flight auger borehole made to depths of approximately 5.0 feet below the existing ground surface, corresponding to the anticipated bottom depth of the stormwater retention basin. The pipes were filled with water and successive readings of drop in water levels were made every 10 minutes for a total elapsed time of 60 minutes, until a stabilization drop was recorded.

The test results indicate that the stabilized soil infiltration rate for the soil ranges from 1.61 to 1.98 inches per hour. A maximum soil infiltration rate of 1.61 inches per hour may be used for the on-site storm-water retention basin design. An oil/water separator should be installed at inlets to the stormwater retention basin to prevent sealing of the basin bottom with silt and oil residues. The field and conversion calculation worksheets are included in Appendix E.

We recommend additional testing should be performed after the completion of rough grading operations, to verify the soil infiltration rate.

Section 4
**DESIGN CRITERIA**

### 4.1 Site Preparation

Pre-grade Meeting: Prior to site preparation, a meeting should be held at the site with as a minimum, the owner's representative, grading contractor and geotechnical engineer in attendance.

Clearing and Grubbing: All surface improvements, debris and/or vegetation including grass, trees, and weeds on the site at the time of construction should be removed from the construction area. Root balls should be completely excavated. Organic stripping should be hauled from the site and not used as fill. Any trash, construction debris, concrete slabs, old pavement, landfill, and buried obstructions such as old foundations and utility lines exposed during rough grading should be traced to the limits of the foreign materials and removed. Any excavations resulting from site clearing and grubbing should be dish-shaped to the lowest depth of disturbance and backfilled with engineered fill.

Mass Grading: Prior to placing any fills, the surface 12 inches of soil should be removed, the exposed surface uniformly moisture conditioned to a depth of 8 inches by discing and wetting to ±2% of optimum moisture, and re-compacted to at least 90% of ASTM D1557 maximum density. Native soils may be used for mass grading, placed in 6 inch maximum lifts, uniformly moisture conditioned to a depth of 8 inches by discing and wetting to ±2% of optimum moisture, and re-compacted to at least 90% of ASTM D1557 maximum density.

Building Pad Preparation: The exposed surface soil within the proposed building pad areas should be removed to 30 inches below the lowest foundation grades, or 60 inches below the original grade (whichever is deeper), extending five feet beyond all exterior wall/column lines (including adjacent concrete areas). The exposed sub-grade shall be saturated to a minimum depth of 5 feet and compacted with a vibratory steel drum roller to achieve a minimum compaction of 95% of the maximum dry density. Moisture penetration and compaction should be verified prior to construction of the engineered fill pad.

After achieving the recommended compaction, the engineered building pad may be constructed by placing the removed soils in uniformly moisture conditioned to ±2% of optimum moisture, and re-compacted to at least 90% of ASTM D1557 maximum density.

The on-site soils are suitable for use as compacted fill and utility trench backfill. Imported fill soil (if required) should similar to onsite soil or non-expansive, granular soil meeting the USCS classifications of SM, SP-SM, or SW-SM with a maximum rock size of 3 inches. ***The geotechnical engineer should approve imported fill soil sources before hauling material to the site***. Native and imported materials should be placed in lifts no greater than 8 inches in loose thickness, uniformly moisture conditioned to ±2% of optimum moisture, and re-compacted to at least 90% of ASTM D1557 maximum density.

In areas other than the building pad which are to receive concrete slabs and asphalt concrete pavement, the ground surface should be over-excavated to a depth of 12 inches, uniformly moisture conditioned to ±2% of optimum moisture, and re-compacted to at least 90% of ASTM D1557 maximum density.

Trench Backfill: On-site soil free of debris, vegetation, and other deleterious matter may be suitable for use as utility trench backfill. Backfill within roadways should be placed in layers not more that 6 inches in thickness, uniformly moisture conditioned to ±2% of optimum moisture and mechanically compacted to a minimum of 90% of the ASTM D1557 maximum dry density except for the top 12 inches of the trench which shall be compacted to at least 95%. Native backfill should only be placed and compacted after encapsulating buried pipes with suitable bedding and pipe envelope material.

Pipe envelope/bedding should either be clean sand (Sand Equivalent SE>30) or crushed rock when encountering groundwater. A geotextile filter fabric (Mirafi 140N or equivalent) should be used to encapsulate the crushed rock to reduce the potential for in-washing of fines into the gravel void space. Precautions should be taken in the compaction of the backfill to avoid damage to the pipes and structures.

Adequate site drainage is essential to future performance of the project. Infiltration of excess irrigation water and stormwaters can adversely affect the performance of the subsurface soil at the site. Positive drainage should be maintained away from all structures (5% for 5 feet minimum across unpaved areas) to prevent ponding and subsequent saturation of the native soil. Gutters and downspouts may be considered as a means to convey water away from foundations. If landscape irrigation is allowed next to the building, drip irrigation systems or lined planter boxes should be used. The subgrade soil should be maintained in a moist, but not saturated state, and not allowed to dry out. Drainage should be maintained without ponding.

Observation and Density Testing: All site preparation and fill placement should be continuously observed and tested by a representative of a qualified geotechnical engineering firm. Full-time observation services during the excavation and scarification process is necessary to detect undesirable materials or conditions and soft areas that may be encountered in the construction area. The geotechnical firm that provides observation and testing during construction shall assume the responsibility of "***geotechnical engineer of record***" and, as such, shall perform additional tests and investigation as necessary to satisfy themselves as to the site conditions and the recommendations for site development.

Auxiliary Structures Foundation Preparation: Auxiliary structures such as free standing or retaining walls should have the existing soil beneath the structure foundation prepared in the manner recommended for the building pad except the preparation needed only to extend 30 inches below and beyond the footing.

### 4.2 Foundations and Settlements

Shallow column footings and continuous wall footings are suitable to support the structures provided they are founded on a layer of properly prepared and compacted soil as described in Section 4.1. The foundations may be designed using an allowable soil bearing pressure of 2,000 psf. The allowable soil pressure may be increased by 20% for each foot of embedment depth in excess of 18 inches and by one-third for short term loads induced by winds or seismic events. The maximum allowable soil pressure at increased embedment depths shall not exceed 2,800 psf.

All exterior and interior] foundations should be embedded a minimum of 18 inches below the building support pad or lowest adjacent final grade, whichever is deeper. Continuous wall footings should have a minimum width of 12 inches. Isolated column footings should have a minimum width of 24 inches. ***Recommended concrete reinforcement and sizing for all footings should be provided by the structural engineer.***

Resistance to horizontal loads will be developed by passive earth pressure on the sides of footings and frictional resistance developed along the bases of footings and concrete slabs. Passive resistance to lateral earth pressure may be calculated using an equivalent fluid pressure of 300 pcf to resist lateral loadings. The top one foot of embedment should not be considered in computing passive resistance unless the adjacent area is confined by a slab or pavement. An allowable friction coefficient of 0.35 may also be used at the base of the footings to resist lateral loading.

Foundation movement under the estimated static loadings and seismic site conditions are estimated to not exceed ¾ inch with differential movement of about two-thirds of total movement for the loading assumptions stated above when the subgrade preparation guidelines given above are followed. Foundation movements under the seismic loading due to dry settlement are provided in Section 3.8 of this report.

### 4.3 Slabs-On-Grade

Concrete slabs and flatwork should be a minimum of 5 inches thick. Concrete floor slabs may either be monolithically placed with the foundation or dowelled after footing placement. The concrete slabs may be placed on granular subgrade that has been compacted at least 90% relative compaction (ASTM D1557).

American Concrete Institute (ACI) guidelines (ACI 302.1R-04 Chapter 3, Section 3.2.3) provide recommendations regarding the use of moisture barriers beneath concrete slabs. The concrete floor slabs should be underlain by a 10-mil polyethylene vapor retarder that works as a capillary break to reduce moisture migration into the slab section. All laps and seams should be overlapped 6-inches or as recommended by the manufacturer. The vapor retarder should be protected from puncture. The joints and penetrations should be sealed with the manufacturer's recommended adhesive, pressure-sensitive tape, or both. The vapor retarder should extend a minimum of 12 inches into the footing excavations. The vapor retarder should be covered by 4 inches of clean sand (Sand Equivalent SE>30) unless placed on 2.5 feet of granular fill, in which case, the vapor retarder may lie directly on the granular fill with 2 inches of clean sand cover.

Placing sand over the vapor retarder may increase moisture transmission through the slab, because it provides a reservoir for bleed water from the concrete to collect. The sand placed over the vapor retarder may also move and mound prior to concrete placement, resulting in an irregular slab thickness. For areas with moisture sensitive flooring materials, ACI recommends that concrete slabs be placed without a sand cover directly over the vapor retarder, provided that the concrete mix uses a low-water cement ratio and concrete curing methods are employed to compensate for release of bleed water through the top of the slab. The vapor retarder should have a minimum thickness of 15-mil (Stego-Wrap or equivalent).

Concrete slab and flatwork reinforcement should consist of chaired rebar slab reinforcement (minimum of No. 4 bars at 18-inch centers, both horizontal directions) placed at slab mid-height to resist potential swell forces and cracking. ***Slab thickness and steel reinforcement are minimums only and should be verified by the structural engineer/designer knowing the actual project loadings.*** The construction joint between the foundation and any mowstrips/sidewalks placed adjacent to foundations should be sealed with a polyurethane based non-hardening sealant to prevent moisture migration between the joint.

Control joints should be provided in all concrete slabs-on-grade at a maximum spacing (in feet) of 2 to 3 times the slab thickness (in inches) as recommended by American Concrete Institute (ACI) guidelines. All joints should form approximately square patterns to reduce randomly oriented contraction cracks. Contraction joints in the slabs should be tooled at the time of the pour or sawcut (¼ of slab depth) within 6 to 8 hours of concrete placement. Construction (cold) joints in foundations and area flatwork should either be thickened butt-joints with dowels or a thickened keyed-joint designed to resist vertical deflection at the joint. All joints in flatwork should be sealed to prevent moisture, vermin, or foreign material intrusion. Precautions should be taken to prevent curling of slabs in this arid desert region (refer to ACI guidelines).

All independent concrete flatworks should be underlain by 12 inches of moisture conditioned and compacted soils. All flatwork should be jointed in square patterns and at irregularities in shape at a maximum spacing of 10 feet or the least width of the sidewalk.

## 4.4 Concrete Mixes and Corrosivity

Selected chemical analyses for corrosivity were conducted on bulk samples of the near surface soil from the project site (Plate C-4). The native soils tested were shown to have low levels of sulfate and chloride ion concentrations. Resistivity determinations on the soil indicate severely potential for metal loss because of electrochemical corrosion processes.

A minimum of 2,500 psi concrete of Type II Portland Cement with a maximum water/cement ratio of 0.60 (by weight) should be used for concrete placed in contact with native soil on this project (sitework including streets, sidewalks, driveways, patios, and foundations).

A minimum concrete cover of three (3) inches is recommended around steel reinforcing or embedded components (anchor bolts, hold-downs, etc.) exposed to native soil or landscape water (to 18 inches above grade). The concrete should also be thoroughly vibrated during placement.

***Landmark does not practice corrosion engineering. We recommend that a qualified corrosion engineer evaluate the corrosion potential on metal construction materials and concrete at the site.***

### 4.5 Excavations

All trench excavations should conform to CalOSHA requirements for Type C soil. The contractor is solely responsible for the safety of workers entering trenches. Temporary excavations with depths of 4 feet or less may be cut nearly vertical for short duration. Temporary slopes should be no steeper than 1.5:1 (horizontal:vertical). Sandy soil slopes should be kept moist, but not saturated, to reduce the potential of raveling or sloughing.

Trench excavations deeper than 4 feet will require shoring or slope inclinations in conformance to CAL/OSHA regulations for Type C soil. Surcharge loads of stockpiled soil or construction materials should be set back from the top of the slope a minimum distance equal to the height of the slope. All permanent slopes should not be steeper than 3:1 to reduce wind and rain erosion. Protected slopes with ground cover may be as steep as 2:1. However, maintenance with motorized equipment may not be possible at this inclination.

### 4.6 Lateral Earth Pressures

Earth retaining structures, such as retaining walls, should be designed to resist the soil pressure imposed by the retained soil mass. Walls with granular drained backfill may be designed for an assumed static earth pressure equivalent to that exerted by a fluid weighing 38 pcf for unrestrained (active) conditions (able to rotate 0.1% of wall height), and 52 pcf for restrained (at-rest) conditions. These values should be verified at the actual wall locations during construction.

### 4.7 Seismic Design

This site is located in the seismically active southern California area and the site structures are subject to strong ground shaking due to potential fault movements along the San Andreas Fault. Engineered design and earthquake-resistant construction are the common solutions to increase safety and development of seismic areas. Designs should comply with the latest edition of the CBC for Site Class D using the seismic coefficients given in Section 3.6 of this report.

### 4.8 Pavements

Pavements should be designed according to CALTRANS or other acceptable methods. Traffic indices were not provided by the project engineer or owner; therefore, we have provided structural sections for several traffic indices for comparative evaluation. The public agency or design engineer should determine the appropriate traffic index for the site. Maintenance of proper drainage is necessary to prolong the service life of the pavements. Based on the current State of California CALTRANS method, an estimated R-value of 30 for the subgrade soil and assumed traffic indices, the following table provides structure thicknesses for asphaltic concrete (AC) pavement sections.

## PAVEMENT STUCTURAL SECTIONS

R-Value of Subgrade Soil - 30 (estimated) Design Method - CALTRANS 2006

| Traffic Index (assumed) | Flexible Pavements | |
|---|---|---|
| | Asphaltic Concrete Thickness (in.) | Aggregate Base Thickness (in.) |
| 5.0 | 3.0 | 6.0 |
| 6.0 | 3.5 | 8.5 |
| 7.0 | 4.5 | 9.5 |
| 8.0 | 5.0 | 11.5 |

Notes:

1) Asphaltic concrete shall be Caltrans, Type B, ¾ inch maximum medium grading, (½ inch for parking areas) compacted to a minimum of 95% of the 50-blow Marshall density (ASTM D1559).
2) Aggregate base shall conform to Caltrans Class 2 (¾ in. maximum), compacted to a minimum of 95% of ASTM D1557 maximum dry density.
3) Place pavements on 12 inches of moisture conditioned (at least 2% of over optimum) native soil compacted to a minimum of 95% of the maximum dry density determined by ASTM D1557, or the governing agency requirements.

Final pavement sections may need to be determined by sampling and R-Value testing during grading operations when actual subgrade soils are exposed.

Section 5
# LIMITATIONS AND ADDITIONAL SERVICES

## 5.1 Limitations

The findings and professional opinions within this report are based on current information regarding the proposed new church at St Frances of Rome, 21591 Lemon Street, Wildomar, California. The conclusions and professional opinions of this report are invalid if:

< Proposed building(s) location and size are changed from those shown in this report
< Structural loads change from those stated or the structures are relocated.
< The Additional Services section of this report is not followed.
< This report is used for adjacent or other property.
< Changes of grade or groundwater occur between the issuance of this report and construction other than those anticipated in this report.
< Any other change that materially alters the project from that proposed at the time this report was prepared.

Findings and professional opinions in this report are based on selected points of field exploration, geologic literature, laboratory testing, and our understanding of the proposed project. Our analysis of data and professional opinions presented herein are based on the assumption that soil conditions do not vary significantly from those found at specific exploratory locations. Variations in soil conditions can exist between and beyond the exploration points or groundwater elevations may change. If detected, these conditions may require additional studies, consultation, and possible design revisions.

***This report contains information that may be useful in the preparation of contract specifications. However, the report is not worded is such a manner that we recommend its use as a construction specification document without proper modification. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk.***

This report was prepared according to the generally accepted *geotechnical engineering standards of practice* that existed in Riverside County at the time the report was prepared. No express or implied warranties are made in connection with our services. This report should be considered invalid for periods after two years from the report date without a review of the validity of the findings and professional opinions by our firm, because of potential changes in the Geotechnical Engineering Standards of Practice.

The client has responsibility to see that all parties to the project including, designer, contractor, and subcontractor are made aware of this entire report. The use of information contained in this report for bidding purposes should be done at the contractor's option and risk.

### 5.2 Additional Services

We recommend that a qualified geotechnical consultant be retained to provide the tests and observations services during construction. ***The geotechnical engineering firm providing such tests and observations shall become the geotechnical engineer of record and assume responsibility for the project.***

The professional opinions presented in this report are based on the assumption that:

< Consultation during development of design and construction documents to check that the geotechnical professional opinions are appropriate for the proposed project and that the geotechnical professional opinions are properly interpreted and incorporated into the documents.

< ***LandMark Consultants, Inc.*** will have the opportunity to review and comment on the plans and specifications for the project prior to the issuance of such for bidding.

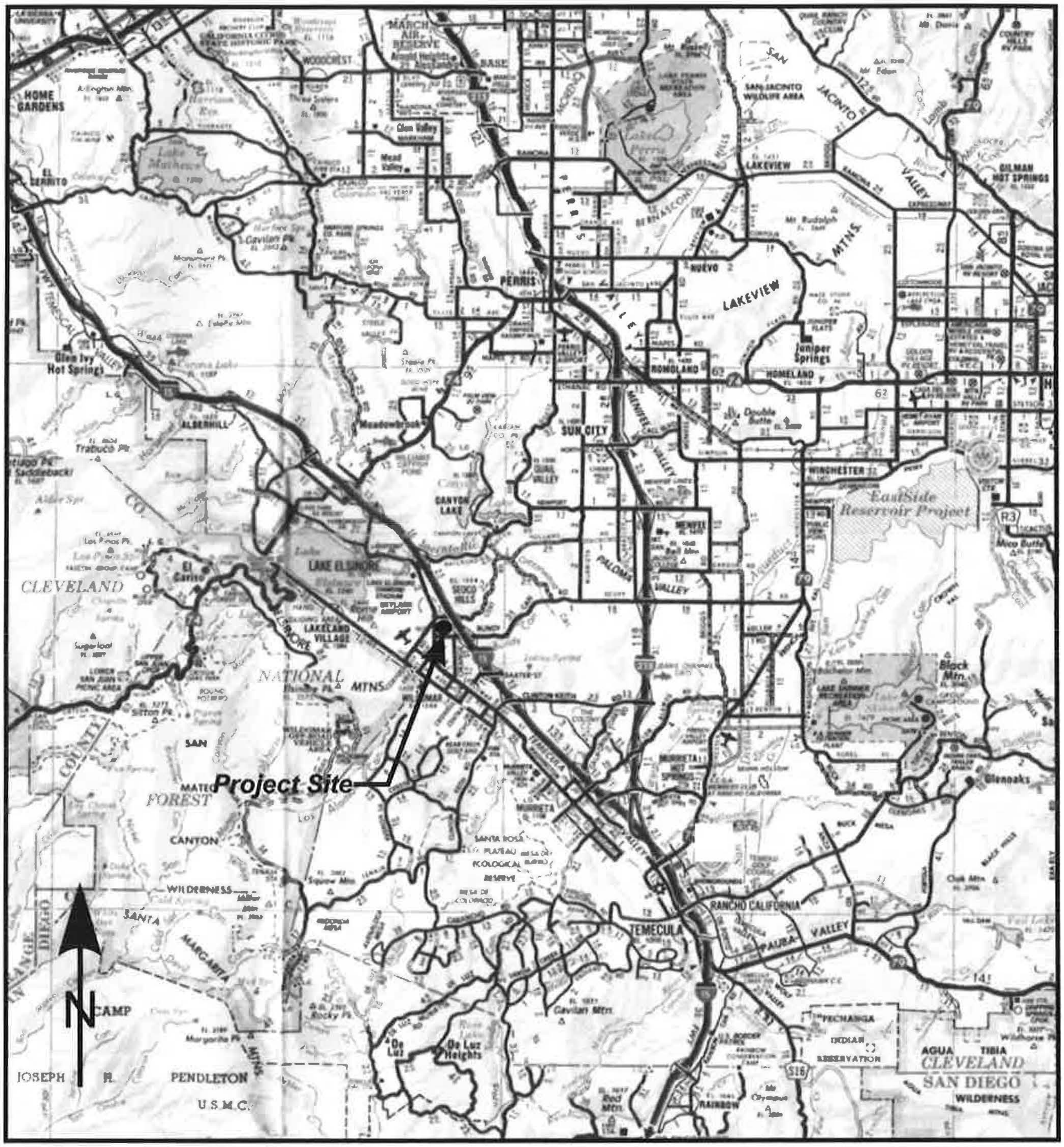
< Continuous observation, inspection, and testing by the geotechnical consultant of record during site clearing, grading, excavation, placement of fills, building pad and subgrade preparation, and backfilling of utility trenches.

< Observation of foundation excavations and reinforcing steel before concrete placement.

< Other consultation as necessary during design and construction.

We emphasize our review of the project plans and specifications to check for compatibility with our professional opinions and conclusions. Additional information concerning the scope and cost of these services can be obtained from our office.

# APPENDIX A

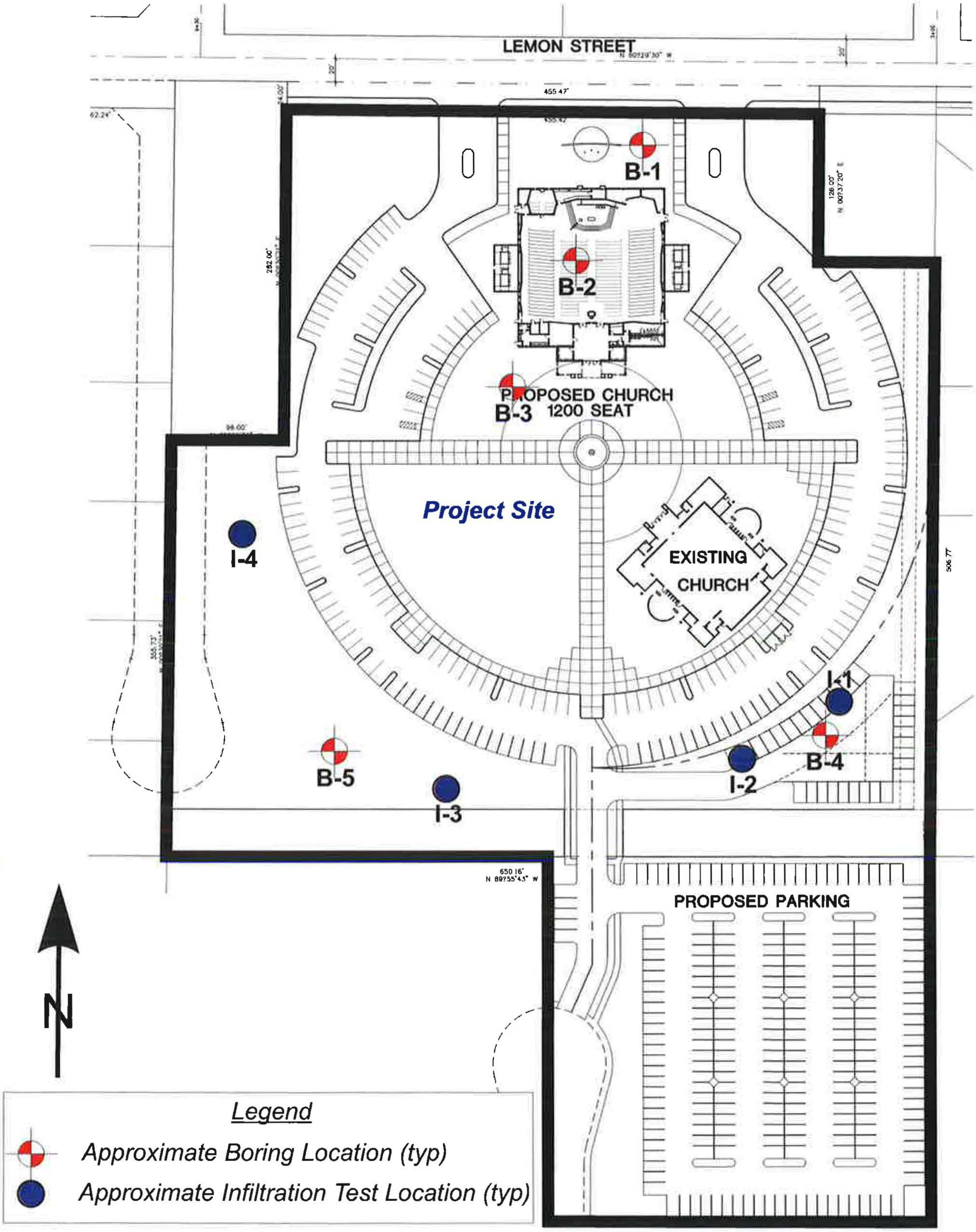



**LandMark**
Geo-Engineers and Geologists

**Project No.: LP16027**

**Vicinity Map**

**Plate A-1**




LANDMARK
Geo-Engineers and Geologists

Project No.: LP16027

Site Map

Plate A-2


Map Scale: 1:4,350 if printed on A portrait (8.5" x 11") sheet.

N

Meters: 0 50 100 200 300

Feet: 0 200 400 800 1200

Map projection: Web Mercator Corner coordinates: WGS84

| | | |
|---|---|---|
| LANDMARK Geo-Engineers and Geologists<br>Project No.: LP16027 | USDA Soil Conservation<br>Soil Service Map | Plate<br>A-3 |

## MAP LEGEND

**Area of Interest (AOI)**

- Area of Interest (AOI)

**Soils**

- Soil Map Unit Polygons
- Soil Map Unit Lines
- Soil Map Unit Points

**Special Point Features**

- Blowout
- Borrow Pit
- Clay Spot
- Closed Depression
- Gravel Pit
- Gravelly Spot
- Landfill
- Lava Flow
- Marsh or swamp
- Mine or Quarry
- Miscellaneous Water
- Perennial Water
- Rock Outcrop
- Saline Spot
- Sandy Spot
- Severely Eroded Spot
- Sinkhole
- Slide or Slip
- Sodic Spot
- Spoil Area
- Stony Spot
- Very Stony Spot
- Wet Spot
- Other
- Special Line Features

**Water Features**

- Streams and Canals

**Transportation**

- Rails
- Interstate Highways
- US Routes
- Major Roads
- Local Roads

**Background**

- Aerial Photography

## MAP INFORMATION

The soil surveys that comprise your AOI were mapped at 1:15,800.

Warning: Soil Map may not be valid at this scale.

Enlargement of maps beyond the scale of mapping can cause misunderstanding of the detail of mapping and accuracy of soil line placement. The maps do not show the small areas of contrasting soils that could have been shown at a more detailed scale.

Please rely on the bar scale on each map sheet for map measurements.

Source of Map: Natural Resources Conservation Service
Web Soil Survey URL: http://websoilsurvey.nrcs.usda.gov
Coordinate System: Web Mercator (EPSG:3857)

Maps from the Web Soil Survey are based on the Web Mercator projection, which preserves direction and shape but distorts distance and area. A projection that preserves area, such as the Albers equal-area conic projection, should be used if more accurate calculations of distance or area are required.

This product is generated from the USDA-NRCS certified data as of the version date(s) listed below.

Soil Survey Area: Western Riverside Area, California
Survey Area Data: Version 8, Sep 22, 2015

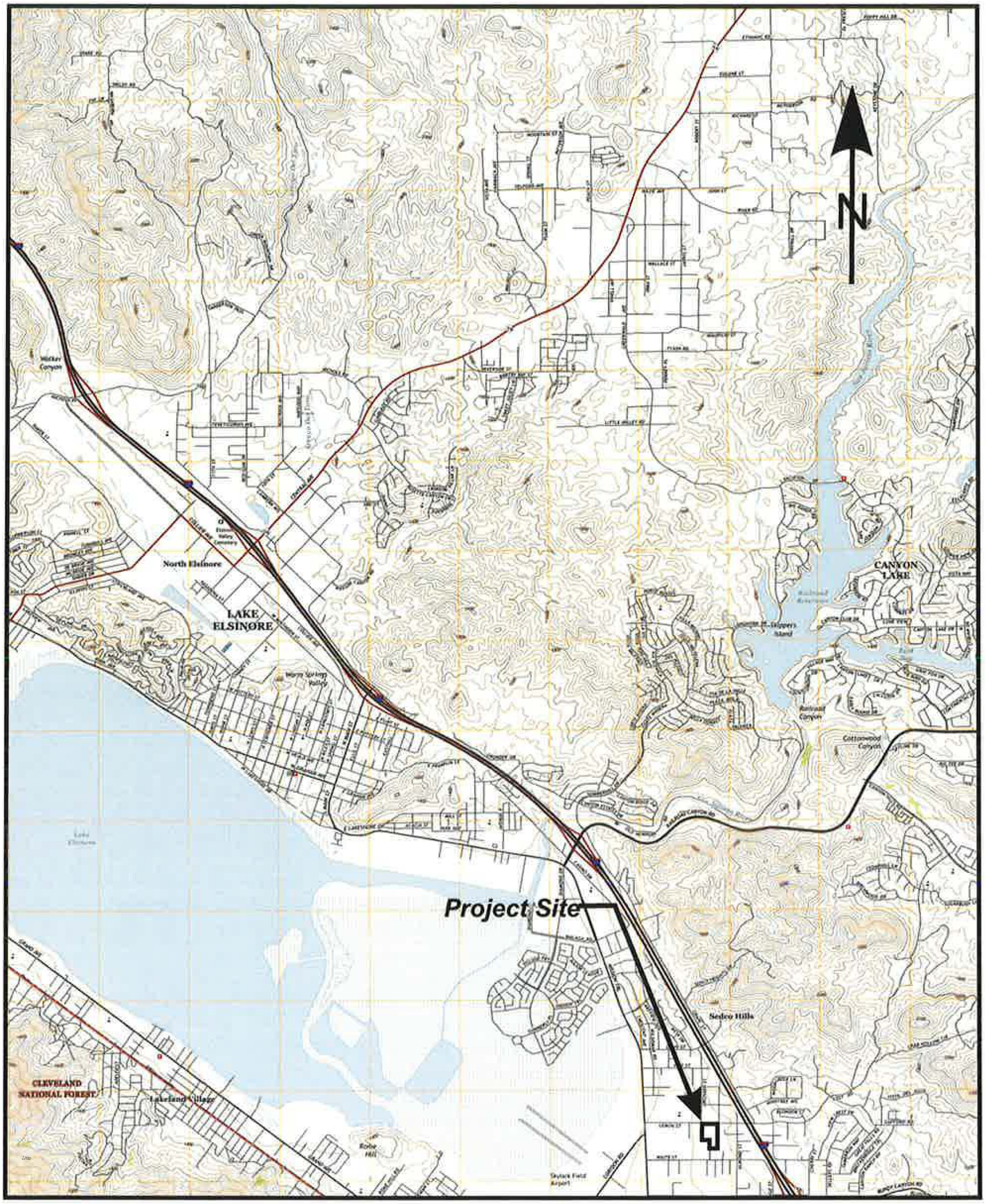
Soil map units are labeled (as space allows) for map scales 1:50,000 or larger.

Date(s) aerial images were photographed: Feb 24, 2015—Feb 26, 2015

The orthophoto or other base map on which the soil lines were compiled and digitized probably differs from the background imagery displayed on these maps. As a result, some minor shifting of map unit boundaries may be evident.

# Map Unit Legend

| Western Riverside Area, California (CA679) | | | |
|---|---|---|---|
| Map Unit Symbol | Map Unit Name | Acres in AOI | Percent of AOI |
| GyC2 | Greenfield sandy loam, 2 to 8 percent slopes, eroded | 77.5 | 79.8% |
| GyD2 | Greenfield sandy loam, 8 to 15 percent slopes, eroded | 0.3 | 0.3% |
| HcC | Hanford coarse sandy loam, 2 to 8 percent slopes | 16.8 | 17.2% |
| ReC2 | Ramona very fine sandy loam, 0 to 8 percent slopes, ero ded | 0.9 | 1.0% |
| TeG | Terrace escarpments | 1.7 | 1.7% |
| **Totals for Area of Interest** | | **97.2** | **100.0%** |




Lake Elsinore Quadrangle
California - Riverside Co.
7.5 Minute Series

Site Coordinates
Lat: 33.6333 N
Long: 117.2828 W

| | | |
|---|---|---|
| LANDMARK Geo-Engineers and Geologists<br>**Project No.: LP16027** | USGS<br>U.S. Department of the Interior<br>U.S. Gelogical Survey<br>**Topographic Map** | **Plate A-4** |


Reference: *Federal Emergency Management Agency (FEMA)*
*Wildomar, California - Riverside County*
*Community-Panel Numbers 06065C 2043G*

| | | |
|---|---|---|
| LANDMARK Geo-Engineers and Geologists<br>**Project No.: LP16027** | **Flood Insurance Rate Map (FIRM)** | **Plate**<br>**A-5** |

# LEGEND

SPECIAL FLOOD HAZARD AREAS SUBJECT TO INUNDATION BY THE 1% ANNUAL CHANCE FLOOD

The 1% annual flood (100-year flood), also known as the base flood, is the flood that has a 1% chance of being equaled or exceeded in any given year. The Special Flood Hazard Area is the area subject to flooding by the 1% annual chance flood. Areas of Special Flood Hazard include Zones A, AE, AH, AO, AR, A99, V, and VE. The Base Flood Elevation is the water-surface elevation of the 1% annual chance flood.

| | |
|---|---|
| **ZONE A** | No Base Flood Elevations determined. |
| **ZONE AE** | Base Flood Elevations determined. |
| **ZONE AH** | Flood depths of 1 to 3 feet (usually areas of ponding); Base Flood Elevations determined. |
| **ZONE AO** | Flood depths of 1 to 3 feet (usually sheet flow on sloping terrain); average depths determined. For areas of alluvial fan flooding, velocities also determined. |
| **ZONE AR** | Special Flood Hazard Area formerly protected from the 1% annual chance flood by a flood control system that was subsequently decertified. Zone AR indicates that the former flood control system is being restored to provide protection from the 1% annual chance or greater flood. |
| **ZONE A99** | Area to be protected from 1% annual chance flood by a Federal flood protection system under construction; no Base Flood Elevations determined. |
| **ZONE V** | Coastal flood zone with velocity hazard (wave action); no Base Flood Elevations determined. |
| **ZONE VE** | Coastal flood zone with velocity hazard (wave action); Base Flood Elevations determined. |

FLOODWAY AREAS IN ZONE AE

The floodway is the channel of a stream plus any adjacent floodplain areas that must be kept free of encroachment so that the 1% annual chance flood can be carried without substantial increases in flood heights.

OTHER FLOOD AREAS

**ZONE X** Areas of 0.2% annual chance flood; areas of 1% annual chance flood with average depths of less than 1 foot or with drainage areas less than 1 square mile; and areas protected by levees from 1% annual chance flood.

OTHER AREAS

| | |
|---|---|
| **ZONE X** | Areas determined to be outside the 0.2% annual chance floodplain. |
| **ZONE D** | Areas in which flood hazards are undetermined, but possible. |

COASTAL BARRIER RESOURCES SYSTEM (CBRS) AREAS

OTHERWISE PROTECTED AREAS (OPAs)

CBRS areas and OPAs are normally located within or adjacent to Special Flood Hazard Areas.

1% annual chance floodplain boundary

0.2% annual chance floodplain boundary

Floodway boundary

Zone D boundary

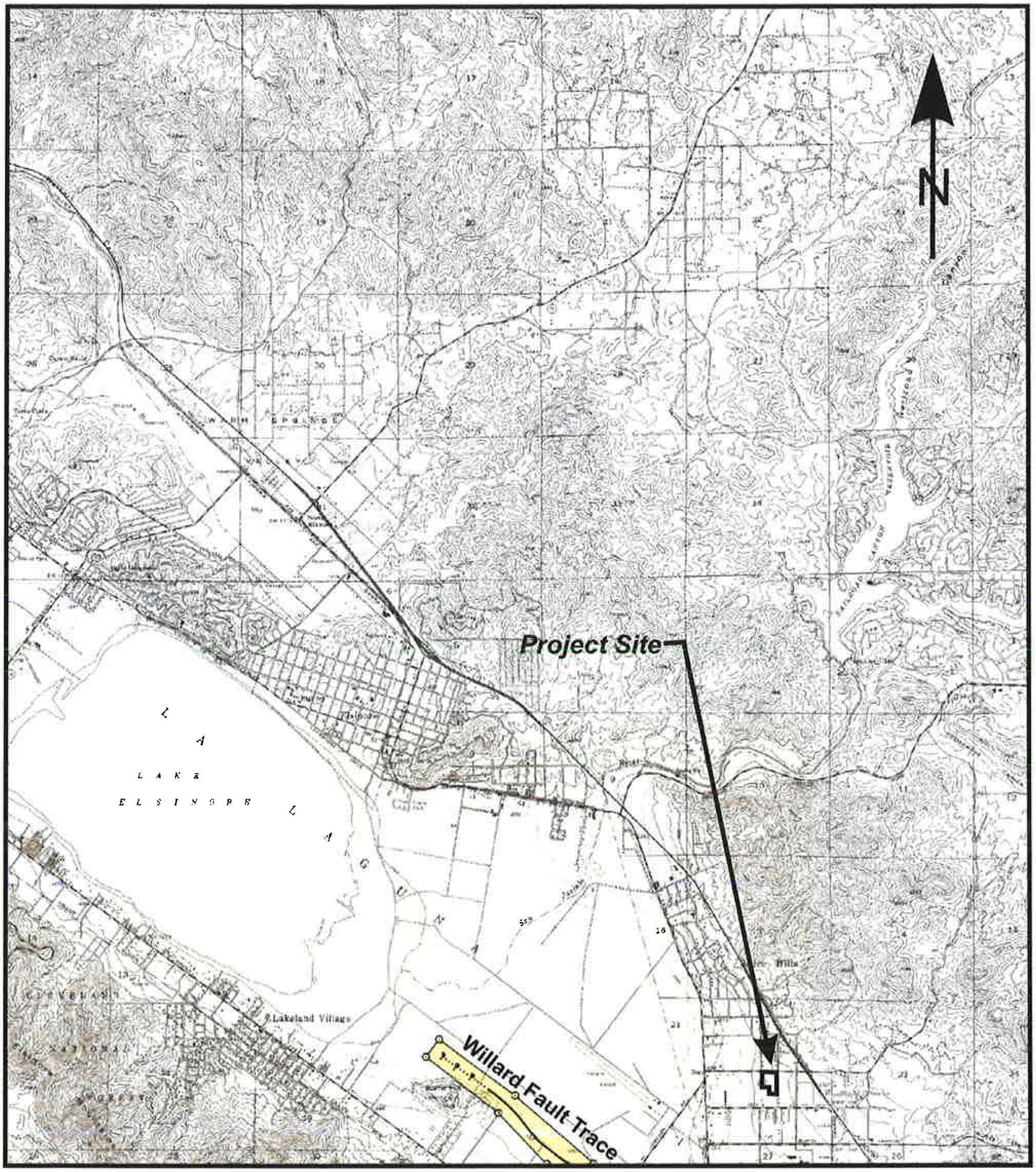
CBRS and OPA boundary

Boundary dividing Special Flood Hazard Area Zones and boundary dividing Special Flood Hazard Areas of different Base Flood Elevations, flood depths or flood velocities.

| | |
|---|---|
| 513 | Base Flood Elevation line and value; elevation in feet* |
| (EL 987) | Base Flood Elevation value where uniform within zone; elevation in feet* |

* Referenced to the North American Vertical Datum of 1988

| | |
|---|---|
| (A)——(A) | Cross section line |
| (23)- - - - - -(23) | Transect line |
| 87°07'45", 32°22'30" | Geographic coordinates referenced to the North American Datum of 1983 (NAD 83), Western Hemisphere |
| 2476000mN | 1000-meter Universal Transverse Mercator grid values, zone 11N |
| 600000 FT | 5000-foot grid ticks: California State Plane coordinate system, zone VI (FIPSZONE 0406), Lambert Conformal Conic projection |
| DX5510 × | Bench mark (see explanation in Notes to Users section of this FIRM panel) |
| ● M1.5 | River Mile |




Lake Elsinore Quadrangle
California - Riverside Co.
7.5 Minute Series

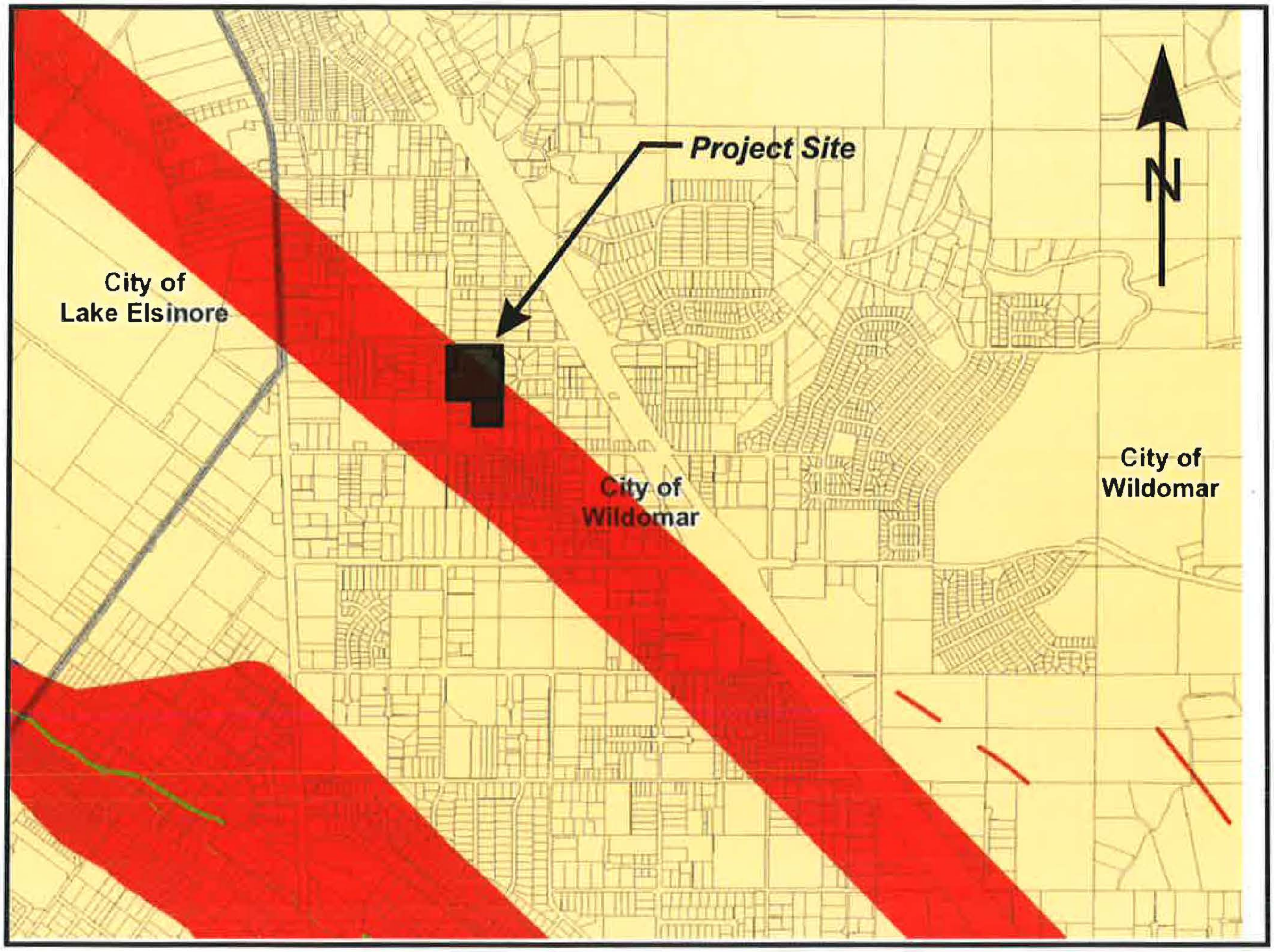
0 ½ 1
Scale in Miles

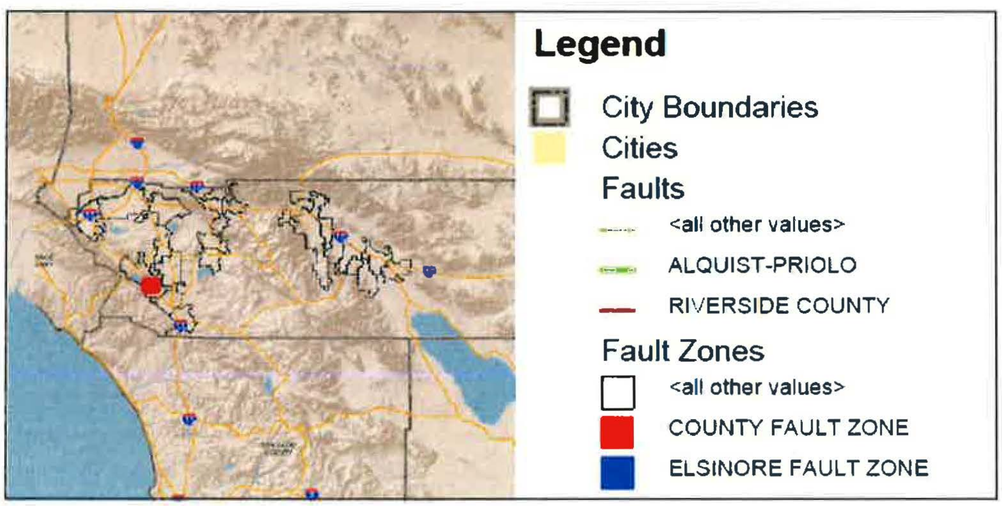
Site Coordinates
Lat: 33.6333 N
Long:117.2828 W

LANDMARK
Geo-Engineers and Geologists
**Project No.: LP16027**

State of California
The Resources Agency
Department of Conservation
**A-P Earthquake Fault Zone Map**

**Plate**
**A-6**




| | | |
|---|---|---|
| LANDMARK<br>Geo-Engineers and Geologists<br>**Project No.: LP16027** | Riverside County Information Technology (RCIT)<br>Geographic Information Services<br>**Fault Map** | **Plate**<br>**A-7** |

# APPENDIX B

**CLIENT:** Diosis of San Bernardino
**PROJECT:** St. Frances of Rome Catholic Church
**LOCATION:** 21591 Lemon Street, Wildomar, CA

**METHOD OF DRILLING:** CME 75 w/autohammer
**DATE OBSERVED:** 3/15/2016
**LOGGED BY:** G. Chandra

# LOG OF BORING: B-1

| DEPTH (FT) | FIELD: CLASSIFICATION | FIELD: SAMPLE TYPE | FIELD: BLOW COUNT | FIELD: POCKET PEN. (TSF) | DESCRIPTION OF MATERIAL | LABORATORY: MOISTURE CONTENT | LABORATORY: DRY UNIT WT. (PCF) | LABORATORY: UNCONFINED COMPRESSION (TSF) | LABORATORY: LIQUID LIMIT | LABORATORY: PLASTICITY INDEX | LABORATORY: PASSING #200 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0–14 | SM | | | | SILTYSAND (SM): Brown, with traces of gravel moist and dense with depth | | | | | | |
| ~2 | | Bulk | | | | | | | | | 40 |
| 5 | | Ring | 23 | | | 3.5 | 118.9 | | | | |
| 10 | | Ring | 55 | | | 8.5 | 131.3 | | | | |
| 14–24 | SM | | | | SILTY SAND (SM): Dark brown. moist and medium dense | | | | | | |
| 15 | | Ring | 47 | | | 10.3 | 134.5 | | | | |
| ~17 | | Bulk | | | | | | | | | 40 |
| 20 | | SPT | 24 | | | 9.2 | | | | | |
| 24–31.5 | SM | | | | SILTY SAND (SM): Brown. moist and medium dense | | | | | | |
| 25 | | SPT | 20 | | | 11.0 | | | | | 34 |
| 30 | | SPT | 22 | | | 8.1 | | | | | 24 |

SURFACE ELEVATION: 1334 ft TOTAL DEPTH: 31.5 ft DEPTH TO WATER: N/A

**PROJECT NO.:** LP16027

LANDMARK Geo-Engineers and Geologists

**PLATE** B-1

**CLIENT:** Diocese of San Bernardino
**PROJECT:** St. Francis of Rome Catholic Church
**LOCATION:** 21591 Lemon Street, Wildomar, CA

**METHOD OF DRILLING:** CME 75 w/autohammer
**DATE OBSERVED:** 3/15/2016
**LOGGED BY:** G. Chandra

# LOG OF BORING: B-2



| DEPTH (FT) | FIELD: CLASSIFICATION | FIELD: SAMPLE TYPE | FIELD: BLOW COUNT | FIELD: POCKET PEN. (TSF) | DESCRIPTION OF MATERIAL | LABORATORY: MOISTURE CONTENT | LABORATORY: DRY UNIT WT. (PCF) | LABORATORY: UNCONFINED COMPRESSION (TSF) | LABORATORY: LIQUID LIMIT | LABORATORY: PLASTICITY INDEX | LABORATORY: PASSING #200 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 5 | | | 15 | | SILTY SAND (SM): Brown, with traces of gravel. | 7.3 | 123.5 | | | | 30 |
| 10 | | | 21 | | | 4.9 | 113.0 | | | | |
| 15 | | | 29 | | | 3.0 | 110.9 | | | | |
| 20 | | | 21 | | SILTY SAND (SM): Brown. | 9.3 | | | | | 24 |
| 25 | | | 25 | | SILTY SAND (SM): Dark brown. | 10.2 | | | | | 31 |
| 30 | | | 25 | | | 8.1 | | | | | 21 |
| 35 | | | 22 | | | | | | | | 41 |
| 40 | | | 34 | | | | | | | | 27 |

SURFACE ELEVATION: 1331 ft TOTAL DEPTH: 51.5 ft DEPTH TO WATER: N/A

**PROJECT NO.:** LP16027

LANDMARK Geo-Engineers and Geologists

**PLATE** B-2

CLIENT: Diosis of San Bernardino | METHOD OF DRILLING: CME 75 w/autohammer
PROJECT: St. Frances of Rome Catholic Church | DATE OBSERVED: 3/15/2016
LOCATION: 21591 Lemon Street, Wildomar, CA | LOGGED BY: G. Chandra

# LOG OF BORING: B-2



| DEPTH (FT) | FIELD: CLASSIFICATION | FIELD: SAMPLE TYPE | FIELD: BLOW COUNT | FIELD: POCKET PEN. (TSF) PP | DESCRIPTION OF MATERIAL | LABORATORY: MOISTURE CONTENT | LABORATORY: DRY UNIT WT. (PCF) | LABORATORY: UNCONFINED COMPRESSION (TSF) | LABORATORY: LIQUID LIMIT | LABORATORY: PLASTICITY INDEX | LABORATORY: PASSING #200 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | SILTY SAND (SM): Brown. | | | | | | |
| 45 | | | 47 | | dense with depth | | | | | | 15 |
| 50 | | | 41 | | | | | | | | 13 |
| 55 | | | | | | | | | | | |
| 60 | | | | | | | | | | | |
| 65 | | | | | | | | | | | |
| 70 | | | | | | | | | | | |
| 75 | | | | | | | | | | | |
| 80 | | | | | | | | | | | |

SURFACE ELEVATION: 1331 ft TOTAL DEPTH: 51.5 ft DEPTH TO WATER: N/A

PROJECT NO.: LP16027

LANDMARK Geo-Engineers and Geologists

PLATE B-3

| | | | |
|---|---|---|---|
| **CLIENT:** | Diosis of San Bernardino | **METHOD OF DRILLING:** | CME 75 w/autohammer |
| **PROJECT:** | St. Frances of Rome Catholic Church | **DATE OBSERVED:** | 3/15/2016 |
| **LOCATION:** | 21591 Lemon Street, Wildomar, CA | **LOGGED BY:** | G. Chandra |

## LOG OF BORING: B-3

| DEPTH (FT) | FIELD: CLASSIFICATION | FIELD: SAMPLE TYPE | FIELD: BLOW COUNT | FIELD: POCKET PEN. (TSF) PP | DESCRIPTION OF MATERIAL | LABORATORY: MOISTURE CONTENT | LABORATORY: DRY UNIT WT. (PCF) | LABORATORY: UNCONFINED COMPRESSION (TSF) | LABORATORY: LIQUID LIMIT | LABORATORY: PLASTICITY INDEX | LABORATORY: PASSING #200 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | SILTY SAND (SM): Brown, with traces of gravel. | | | | | | |
| 5 | | | 23 | | moist and medium dense | 5.5 | 123.0 | | | | 32 |
| 10 | | | 15 | | | 5.2 | 119.5 | | | | |
| 15 | | | 49 | | SILTY SAND (SM): Brown. moist and dense | 7.0 | 135.5 | | | | 20 |
| 20 | | | 23 | | SILTY SAND (SM): Dark brown. moist and medium dense | 11.0 | | | | | |
| 25 | | | 22 | | | 9.0 | | | | | 23 |
| 30 | | | 21 | | | 11.1 | | | | | |
| 35 | | | | | | | | | | | |
| 40 | | | | | | | | | | | |

SURFACE ELEVATION: 1307 ft TOTAL DEPTH: 31.5 ft DEPTH TO WATER: N/A

**PROJECT NO.:** **LP16027**

LANDMARK
Geo-Engineers and Geologists

**PLATE** **B-4**

| | |
|---|---|
| **CLIENT:** Diosis of San Bernardino | **METHOD OF DRILLING:** CME 75 w/autohammer |
| **PROJECT:** St. Francis of Rome Catholic Church | **DATE OBSERVED:** 3/15/2016 |
| **LOCATION:** 21591 Lemon Street, Wildomar, CA | **LOGGED BY:** G. Chandra |

# LOG OF BORING: B-4

| DEPTH (FT) | FIELD: CLASSIFICATION | SAMPLE TYPE | BLOW COUNT | POCKET PEN. (TSF) PP | DESCRIPTION OF MATERIAL | LABORATORY: MOISTURE CONTENT | DRY UNIT WT. (PCF) | UNCONFINED COMPRESSION (TSF) | LIQUID LIMIT | PLASTICITY INDEX | PASSING #200 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | SILTY SAND (SM): Brown, with traces of gravel. | | | | | | |
| 5 | | | 6 | | moist and loose | 10.1 | | | | | 39 |
| 10 | | | 10 | | | 10.1 | | | | | |
| 15 | | | | | | | | | | | |
| 20 | | | | | | | | | | | |
| 25 | | | | | | | | | | | |
| 30 | | | | | | | | | | | |
| 35 | | | | | | | | | | | |
| 40 | | | | | | | | | | | |

SURFACE ELEVATION: 1329 ft TOTAL DEPTH: 11.5 ft DEPTH TO WATER: N/A

**PROJECT NO.:** **LP16027**

LANDMARK
Geo-Engineers and Geologists

**PLATE** **B-5**

**CLIENT:** Diosis of San Bernardino
**PROJECT:** St. Frances of Rome Catholic Church
**LOCATION:** 21591 Lemon Street, Wildomar, CA

**METHOD OF DRILLING:** CME 75 w/autohammer
**DATE OBSERVED:** 3/15/2016
**LOGGED BY:** G. Chandra

# LOG OF BORING: B-5

| DEPTH (FT) | FIELD: CLASSIFICATION | FIELD: SAMPLE TYPE | FIELD: BLOW COUNT | FIELD: POCKET PEN. (TSF) | DESCRIPTION OF MATERIAL | LABORATORY: MOISTURE CONTENT | LABORATORY: DRY UNIT WT. (PCF) | LABORATORY: UNCONFINED COMPRESSION (TSF) | LABORATORY: LIQUID LIMIT | LABORATORY: PLASTICITY INDEX | LABORATORY: PASSING #200 |
|---|---|---|---|---|---|---|---|---|---|---|---|
| 0 | | | | | SILTY SAND (SM): Brown, with traces of gravel. | | | | | | |
| 5 | | | 14 | | moist and medium dense | 9.8 | | | | | |
| 10 | | | 15 | | | 11.4 | | | | | 32 |

SURFACE ELEVATION: 1328 ft TOTAL DEPTH: 11.5 ft DEPTH TO WATER: N/A

**PROJECT NO.:** LP16027

LANDMARK
Geo-Engineers and Geologists

**PLATE** B-6

# DEFINITION OF TERMS

| PRIMARY DIVISIONS | | | SYMBOLS | SECONDARY DIVISIONS |
|---|---|---|---|---|
| Coarse grained soils More than half of material is larger that No. 200 sieve | **Gravels** — More than half of coarse fraction is larger than No. 4 sieve | Clean gravels (less than 5% fines) | **GW** | Well graded gravels, gravel-sand mixtures, little or no fines |
| | | | **GP** | Poorly graded gravels, or gravel-sand mixtures, little or no fines |
| | | Gravel with fines | **GM** | Silty gravels, gravel-sand-silt mixtures, non-plastic fines |
| | | | **GC** | Clayey gravels, gravel-sand-clay mixtures, plastic fines |
| | **Sands** — More than half of coarse fraction is smaller than No. 4 sieve | Clean sands (less than 5% fines) | **SW** | Well graded sands, gravelly sands, little or no fines |
| | | | **SP** | Poorly graded sands or gravelly sands, little or no fines |
| | | Sands with fines | **SM** | Silty sands, sand-silt mixtures, non-plastic fines |
| | | | **SC** | Clayey sands, sand-clay mixtures, plastic fines |
| Fine grained soils More than half of material is smaller than No. 200 sieve | **Silts and clays** | | **ML** | Inorganic silts, clayey silts with slight plasticity |
| | Liquid limit is less than 50% | | **CL** | Inorganic clays of low to medium plasticity, gravely, sandy, or lean clays |
| | | | **OL** | Organic silts and organic clays of low plasticity |
| | **Silts and clays** | | **MH** | Inorganic silts, micaceous or diatomaceous silty soils, elastic silts |
| | Liquid limit is more than 50% | | **CH** | Inorganic clays of high plasticity, fat clays |
| | | | **OH** | Organic clays of medium to high plasticity, organic silts |
| Highly organic soils | | | **PT** | Peat and other highly organic soils |

## GRAIN SIZES

| Silts and Clays | Sand | | | Gravel | | Cobbles | Boulders |
|---|---|---|---|---|---|---|---|
| | Fine | Medium | Coarse | Fine | Coarse | | |

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

US Standard Series Sieve Clear Square Openings

| Sands, Gravels, etc. | Blows/ft. * |
|---|---|
| Very Loose | 0-4 |
| Loose | 4-10 |
| Medium Dense | 10-30 |
| Dense | 30-50 |
| Very Dense | Over 50 |

| Clays & Plastic Silts | Strength ** | Blows/ft. * |
|---|---|---|
| Very Soft | 0-0.25 | 0-2 |
| Soft | 0.25-0.5 | 2-4 |
| Firm | 0.5-1.0 | 4-8 |
| Stiff | 1.0-2.0 | 8-16 |
| Very Stiff | 2.0-4.0 | 16-32 |
| Hard | Over 4.0 | Over 32 |

* Number of blows of 140 lb. hammer falling 30 inches to drive a 2 inch O.D. (1 3/8 in. I.D.) split spoon (ASTM D1586).

** Unconfined compressive strength in tons/s.f. as determined by laboratory testing or approximated by the Standard Penetration Test (ASTM D1586), Pocket Penetrometer, Torvane, or visual observation.

**Type of Samples:**

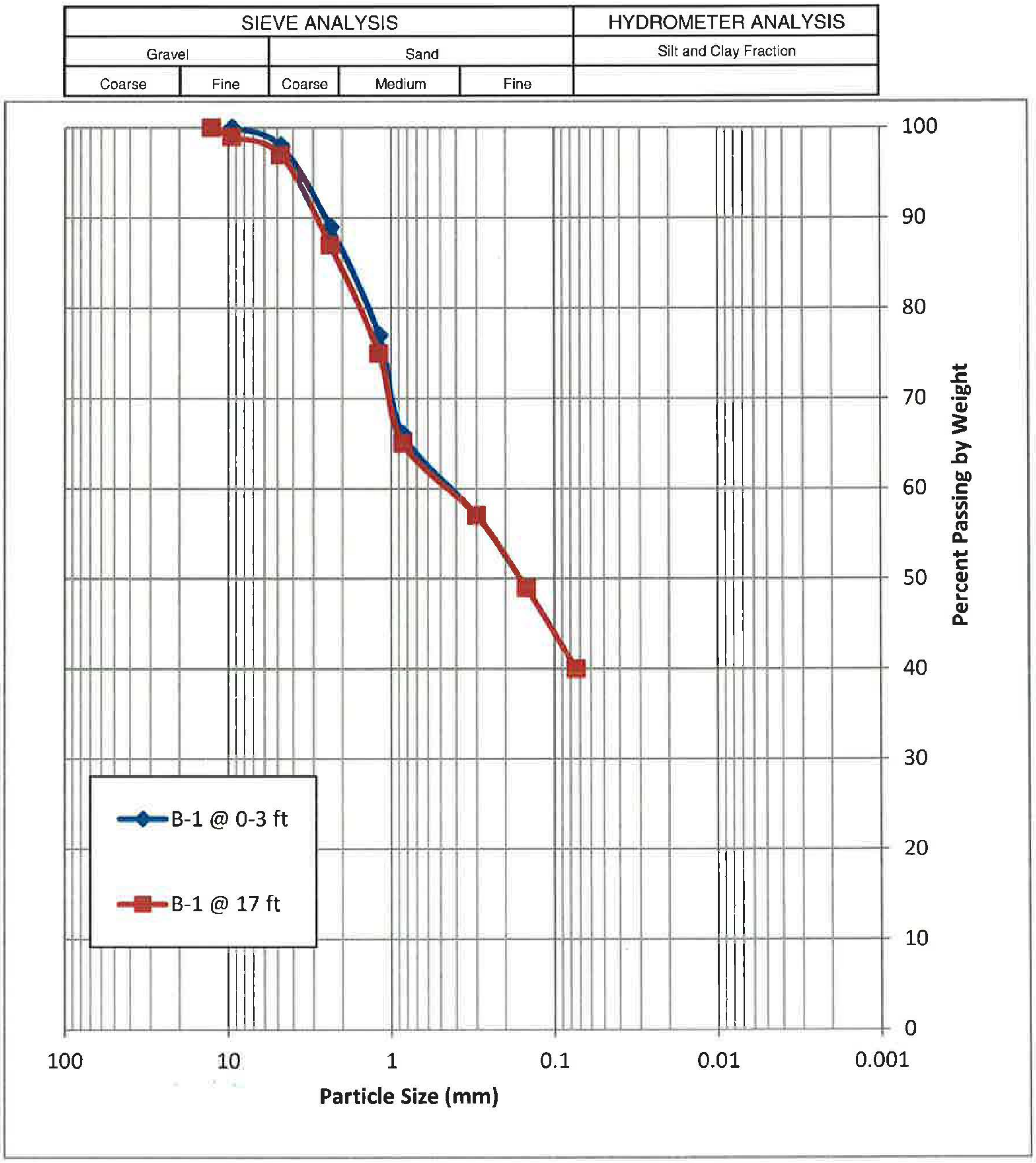
Ring Sample   Standard Penetration Test   Shelby Tube   Bulk (Bag) Sample

**Drilling Notes:**

1. Sampling and Blow Counts
   - Ring Sampler - Number of blows per foot of a 140 lb. hammer falling 30 inches.
   - Standard Penetration Test - Number of blows per foot.
   - Shelby Tube - Three (3) inch nominal diameter tube hydraulically pushed.
2. P. P. = Pocket Penetrometer (tons/s.f.).
3. NR = No recovery.
4. GWT = Ground Water Table observed @ specified time.

| LANDMARK Geo-Engineers and Geologists — Project No.: LP16027 | Key to Logs | Plate B-7 |
|---|---|---|

# APPENDIX C




LANDMARK
Geo-Engineers and Geologists

**Project No.: LP16027**

**Grain Size Analysis**

**Plate C-1**

**Client:** Diocese of San Bernadino

**Project:** St. Francis of Rome Catholic Church

**Project No.:** LP16027

**Date:** 3/23/2016

**Lab. No.:** N/A

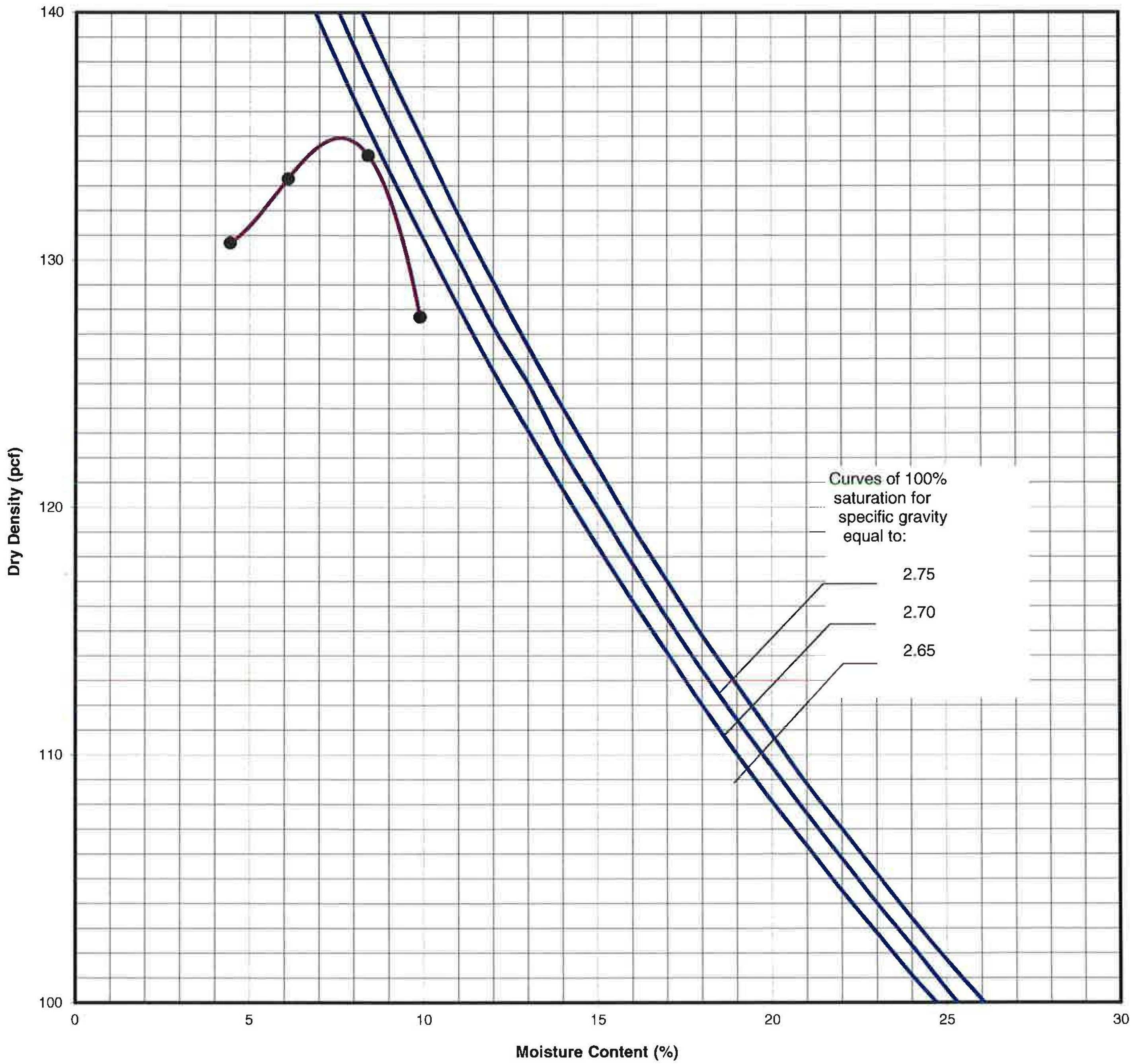
**Soil Description:** Brown Silty Sand (SM)

**Sample Location:** B1 @ 0-3'

**Test Method:** ASTM D-1557 A

**Maximum Dry Density (pcf):** **135.0**

**Optimum Moisture Content (%):** **7.5**




LANDMARK
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**Project No.: LP16027**

**Moisture Density Relationship**

**Plate C-2**

# LANDMARK CONSULTANTS, INC.

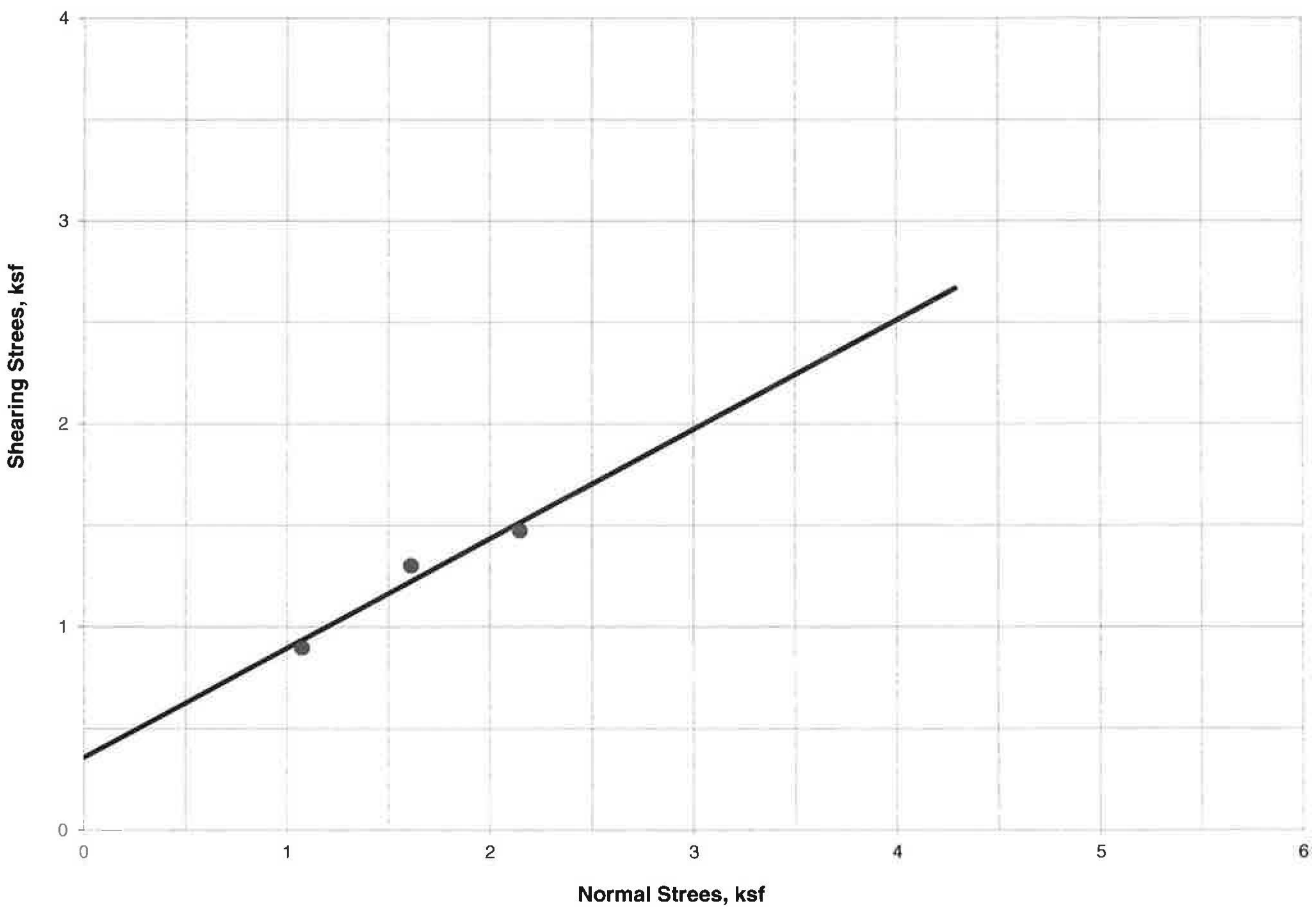
**CLIENT:** Diocese of San Bernardino
**PROJECT:** St. Frances of Rome Catholic Church
**PROJECT No:** LP16027 **DATE:** 3/28/2016

## DIRECT SHEAR TEST - REMOLDED (ASTM D3080)

**SAMPLE LOCATION:** B-1 @ 0 to 3 ft
**SAMPLE DESCRIPTION:** Brown Silty Sand (SM)

**Angle of Internal Friction:** 28°
**Cohesion:** 0.36 ksf
**Initial Dry Density:** 121.2 pcf
**Initial Moisture Content:** 7.6%

## DIRECT SHEAR TEST RESULTS




**PROJECT No: LP16027**

**Direct Shear Test Results**

**Plate C-3**

# LANDMARK CONSULTANTS, INC.

**CLIENT:** Diocese Of San Bernardino
**PROJECT:** St. Frances of Rome Catholic Church
**JOB No.:** LP16027
**DATE:** 04/16/16

## CHEMICAL ANALYSIS

| | | Caltrans Method |
|---|---|---|
| **Boring:** | B-1 | |
| **Sample Depth, ft:** | 0-3 | |
| **pH:** | 7.25 | 643 |
| **Electrical Conductivity (mmhos):** | --- | 424 |
| **Resistivity (ohm-cm):** | 1500 | 643 |
| **Chloride (Cl), ppm:** | 130 | 422 |
| **Sulfate (SO4), ppm:** | 126 | 417 |

General Guidelines for Soil Corrosivity

| Material Affected | Chemical Agent | Amount in Soil (ppm) | Degree of Corrosivity |
|---|---|---|---|
| Concrete | Soluble Sulfates | 0 - 1,000 | Low |
| | | 1,000 - 2,000 | Moderate |
| | | 2,000 - 20,000 | Severe |
| | | > 20,000 | Very Severe |
| Normal Grade Steel | Soluble Chlorides | 0 - 200 | Low |
| | | 200 - 700 | Moderate |
| | | 700 - 1,500 | Severe |
| | | > 1,500 | Very Severe |
| Normal Grade Steel | Resistivity | 1 - 1,000 | Very Severe |
| | | 1,000 - 2,000 | Severe |
| | | 2,000 - 10,000 | Moderate |
| | | > 10,000 | Low |

LANDMARK
Geo-Engineers and Geologists

**Project No.: LP16027**

**Selected Chemical Test Results**

**Plate C-4**

# APPENDIX D

## Seismic Settlement Calculation

**Project Name:** St Frances of Rome
**Project No.:** LP16027
**Location:** B-1

| | | |
|---|---|---|
| Maximum Credible Earthquake | 6.8 | |
| Design Ground Motion | 0.94 g | |
| Total Unit Weight, | 120 pcf | Nc |
| Water Unit Weight, | 62.4 pcf | 9.3 |
| Depth to Groundwater | 60 ft | |
| Hammer Effenciency | 90 | |
| Rod Length | 3 | |

| Mod. Cal | SPT | DEPTH (ft.) | THICKNESS (ft.) | Susceptible | O-PRESS | N1(60) | Fine Content | $N_{1(60)CS}$ | p | Gmax | Shear Strain Gam-eff | E15 | Enc | Settlement (in.) | TOTAL (in.) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 23 | | 5 | 5 | 0 | 0.30 | 27.7 | 40 | 38 | 0.201 | 675 | | | | | |
| 55 | | 10 | 5 | 0 | 0.60 | 52.3 | 40 | 68 | 0.402 | 1155 | | | | | |
| 47 | | 15 | 5 | 1 | 0.90 | 41.6 | 40 | 55 | 0.603 | 1320 | 2.07E-03 | 6.15E-04 | 4.97E-04 | 0.06 | |
| | 24 | 20 | 5 | 1 | 1.20 | 34.7 | 40 | 47 | 0.804 | 1443 | 2.69E-03 | 9.73E-04 | 7.86E-04 | 0.09 | |
| | 20 | 25 | 5 | 1 | 1.50 | 27.3 | 34 | 37 | 1.005 | 1498 | 3.69E-03 | 1.75E-03 | 1.41E-03 | 0.17 | |
| | 22 | 30 | 5 | 1 | 1.80 | 27.8 | 24 | 35 | 1.206 | 1606 | 3.93E-03 | 2.01E-03 | 1.62E-03 | 0.19 | |
| | | | | | 0.00 | #DIV/0! | | #DIV/0! | 0.000 | #DIV/0! | | | | | |
| | | | | | 0.00 | #DIV/0! | | #DIV/0! | 0.000 | #DIV/0! | | | | | |
| | | | | | 0.00 | #DIV/0! | | #DIV/0! | 0.000 | #DIV/0! | | | | | |
| | | | | | 0.00 | #DIV/0! | | #DIV/0! | 0.000 | #DIV/0! | | | | | |
| | | | | | | | | | | | | | | | **0.52** |


REFERENCES

(1) Tokimatsu and Seed, 1984. Simplified Procedures for the Evaluation of Settlements in Clean Sands.

(2) Seed and Idriss, 1982. Ground Motion and Soil Liquefaction During Earthquakes, EERI Monograph.

(3) Youd, Leslie, 1997. Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils

(4) Pradel, Daniel, 1998. JGEE, Vol. 124, No. 4, ASCE

(5) Seed, et.al., 2003, Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework. University of California, Earthquake Engineering Research Center Report 2003-06, 71 p.

## Seismic Settlement Calculation

**Project Name:** St Frances of Rome
**Project No.:** LP16027
**Location:** B-2

| | | | |
|---|---|---|---|
| Maximum Credible Earthquake | 6.8 | | |
| Design Ground Motion | 0.94 g | | |
| Total Unit Weight, | 120 pcf | Nc | |
| Water Unit Weight, | 62.4 pcf | 9.3 | |
| Depth to Groundwater | 60 ft | | |
| Hammer Effenciency | 90 | | |
| Rod Length | 3 | | |

| Mod. Cal | SPT | DEPTH (ft.) | THICKNESS (ft.) | Susceptible | O-PRESS | N1(60) | Fine Content | $N_{1(60)CS}$ | p | Gmax | Shear Strain Gam-eff | E15 | Enc | Settlement (in.) | TOTAL (in.) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 15 | | 5 | 5 | 0 | 0.30 | 18.1 | 30 | 26 | 0.201 | 590 | | | | | |
| 21 | | 10 | 5 | 0 | 0.60 | 20.0 | 30 | 28 | 0.402 | 858 | | | | | |
| 29 | | 15 | 5 | 1 | 0.90 | 25.7 | 30 | 34 | 0.603 | 1128 | 4.04E-03 | 2.11E-03 | 1.71E-03 | 0.20 | |
| | 21 | 20 | 5 | 1 | 1.20 | 30.4 | 24 | 38 | 0.804 | 1345 | 3.60E-03 | 1.67E-03 | 1.35E-03 | 0.16 | |
| | 25 | 25 | 5 | 1 | 1.50 | 34.1 | 31 | 44 | 1.005 | 1587 | 2.92E-03 | 1.12E-03 | 9.07E-04 | 0.11 | |
| | 25 | 30 | 5 | 1 | 1.80 | 31.6 | 21 | 38 | 1.206 | 1652 | 3.50E-03 | 1.61E-03 | 1.30E-03 | 0.16 | |
| | 22 | 35 | 5 | 1 | 2.10 | 25.8 | 41 | 36 | 1.407 | 1749 | 3.54E-03 | 1.75E-03 | 1.42E-03 | 0.17 | |
| | 34 | 40 | 5 | 1 | 2.40 | 37.2 | 27 | 47 | 1.608 | 2039 | 2.34E-03 | 8.50E-04 | 6.87E-04 | 0.08 | |
| | 47 | 45 | 5 | 1 | 2.70 | 48.5 | 15 | 53 | 1.809 | 2264 | 1.81E-03 | 5.57E-04 | 4.50E-04 | 0.05 | |
| | 41 | 50 | 5 | 1 | 3.00 | 40.2 | 13 | 44 | 2.010 | 2230 | 1.92E-03 | 7.56E-04 | 6.11E-04 | 0.07 | |
| | | | | | | | | | | | | | | | **1.01** |


REFERENCES

(1) Tokimatsu and Seed, 1984. Simplified Procedures for the Evaluation of Settlements in Clean Sands.

(2) Seed and Idriss, 1982. Ground Motion and Soil Liquefaction During Earthquakes, EERI Monograph.

(3) Youd, Leslie, 1997. Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils

(4) Pradel, Daniel, 1998. JGEE, Vol. 124, No. 4, ASCE

(5) Seed, et.al., 2003, Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework. University of California, Earthquake Engineering Research Center Report 2003-06, 71 p.

## Seismic Settlement Calculation

**Project Name:** St Frances of Rome
**Project No.:** LP16027
**Location:** B-3

| | | |
|---|---|---|
| Maximum Credible Earthquake | 6.8 | |
| Design Ground Motion | 0.94 g | |
| Total Unit Weight, | 120 pcf | Nc |
| Water Unit Weight, | 62.4 pcf | 9.3 |
| Depth to Groundwater | 60 ft | |
| Hammer Effenciency | 90 | |
| Rod Length | 3 | |

| Mod. Cal | SPT | DEPTH (ft.) | THICKNESS (ft.) | Susceptible | O-PRESS | N1(60) | Fine Content | $N_{1(60)CS}$ | p | Gmax | Shear Strain Gam-eff | E15 | Enc | Settlement (in.) | TOTAL (in.) |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| 23 | | 5 | 5 | 0 | 0.30 | 27.7 | 32 | 37 | 0.201 | 669 | | | | | |
| 15 | | 10 | 5 | 0 | 0.60 | 14.3 | 32 | 22 | 0.402 | 788 | | | | | |
| 49 | | 15 | 5 | 1 | 0.90 | 43.4 | 20 | 50 | 0.603 | 1283 | 2.32E-03 | 7.63E-04 | 6.16E-04 | 0.07 | |
| | 23 | 20 | 5 | 1 | 1.20 | 33.3 | 23 | 41 | 0.804 | 1378 | 3.25E-03 | 1.38E-03 | 1.12E-03 | 0.13 | |
| | 22 | 25 | 5 | 1 | 1.50 | 30.0 | 23 | 37 | 1.005 | 1494 | 3.73E-03 | 1.78E-03 | 1.44E-03 | 0.17 | |
| | 21 | 30 | 5 | 1 | 1.80 | 26.6 | 23 | 33 | 1.206 | 1579 | 4.21E-03 | 2.28E-03 | 1.85E-03 | 0.22 | |
| | | | | | 0.00 | #DIV/0! | | #DIV/0! | 0.000 | #DIV/0! | | | | | |
| | | | | | 0.00 | #DIV/0! | | #DIV/0! | 0.000 | #DIV/0! | | | | | |
| | | | | | 0.00 | #DIV/0! | | #DIV/0! | 0.000 | #DIV/0! | | | | | |
| | | | | | 0.00 | #DIV/0! | | #DIV/0! | 0.000 | #DIV/0! | | | | | |
| | | | | | | | | | | | | | | | **0.60** |


REFERENCES

(1) Tokimatsu and Seed, 1984. Simplified Procedures for the Evaluation of Settlements in Clean Sands.
(2) Seed and Idriss, 1982. Ground Motion and Soil Liquefaction During Earthquakes, EERI Monograph.
(3) Youd, Leslie, 1997. Proceeding of the NCEER Workshop on Evaluation of Liquefaction Resistance of Soils
(4) Pradel, Daniel, 1998. JGEE, Vol. 124, No. 4, ASCE
(5) Seed, et.al., 2003, Recent Advances in Soil Liquefaction Engineering: A Unified and Consistent Framework. University of California, Earthquake Engineering Research Center Report 2003-06, 71 p.

# APPENDIX E

## LANDMARK CONSULTANTS, INC

| | | | | | | |
|---|---|---|---|---|---|---|
| Project: | St Francis of Rome | | Project No: | LP16027 | Date: | 3/18/16 |
| Test Hole No: | | I-1 | Tested By: | Alex A | | |
| Depth of Test Hole, $D_T$: | | 5' | USCS Soil Classification: | | | |
| Test Hole Dimensions (inches) | | | | Length | Width | |
| Diameter (if round)= | | 6" | Sides (if rectangular)= | | | |

Sandy Soil Criteria Test*

| Trial No. | Start Time | Stop Time | Time Interval, (min.) | Initial Depth to Water (in.) | Final Depth to Water (in.) | Change in Water Level (in.) | Greater than or Equal to 6"? (y/n) |
|---|---|---|---|---|---|---|---|
| 1 | 8:50 | 9:15 | 25.00 | 29.00 | 55.00 | 26.00 | y |
| 2 | 9:15 | 9:40 | 25.00 | 30.00 | 50.00 | 20.00 | y |

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Other wise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

| Trial No. | Start Time | Stop Time | $\Delta t$ Time Interval (min.) | $D_o$ Initial Depth to Water (in.) | $D_f$ Final Depth to Water (in.) | $\Delta D$ Change in Water Level (in.) | Percolation Rate (min./in.) |
|---|---|---|---|---|---|---|---|
| 1 | 9:42 | 9:52 | 10.00 | 18.00 | 27.00 | 9.00 | 1.11 |
| 2 | 9:52 | 10:02 | 10.00 | 27.00 | 35.00 | 8.00 | 1.25 |
| 3 | 10:02 | 10:12 | 10.00 | 35.00 | 43.00 | 8.00 | 1.25 |
| 4 | 10:12 | 10:22 | 10.00 | 19.00 | 27.00 | 8.00 | 1.25 |
| 5 | 10:22 | 10:32 | 10.00 | 27.00 | 35.00 | 8.00 | 1.25 |
| 6 | 10:32 | 10:42 | 10.00 | 20.00 | 27.00 | 7.00 | 1.43 |
| 7 | | | | | | | |
| 8 | | | | | | | |
| 9 | | | | | | | |
| 10 | | | | | | | |
| 11 | | | | | | | |
| 12 | | | | | | | |

COMMENTS:

LANDMARK
Geo-Engineers and Geologists

Project No.: LP16027

**Percolation Test Results**

**Plate**
E-1

# PERCOLATION RATE CONVERSION

**CLIENT:** Diocese of San Bernardino
**PROJECT:** St Frances of Rome
**PROJECT NO.:** LP16027
**DATE:** 3/18/2016

**TEST HOLE NO:** I-1

Time interval, **Δt** = 10 minutes
Initial Depth to Water, $D_0$ = 20 inches
Final Depth to Water, $D_f$ = 27 inches
Total Depth of Test Hole, $D_T$ = 60 inches
[2]Test Hole Radius, **r** = 3 inches

The conversion equation is used:

$$I_t = \frac{\Delta H \, 60 \, r}{\Delta t(r + 2H_{avg}}$$

"$H_o$" is the initial height of water at the selected time interval

$$H_o = D_T - D_0 \qquad = 60 - 20 = 40 \text{ inches}$$

"$H_f$" is the final height of water at the selected time interval

$$H_f = D_T - D_f \qquad = 60 - 27 = 33 \text{ inches}$$

"$\Delta H$" is the change in height over the time interval

$$\Delta H = \Delta D = H_o - H_f \qquad = 40 - 33 = 7 \text{ inches}$$

"$H_{avg}$" is the average head height over the time interval

$$H_{avg} = (H_o + H_f)/2 \qquad = (40 + 33)/2 = 36.5 \text{ inches}$$

"$I_t$" is the tested infiltration rate

$$I_t = \frac{\Delta H \, 60 \, r}{\Delta t \left(r + 2H_{avg}\right)} = \frac{(7 \text{ in})(60\text{min/hr}) > 3\text{in})}{(10 \text{ min})((3 \text{ in}) + 2 \, (36.5 \text{ in}))} = 1.66 \text{ in/hr}$$

LANDMARK Geo-Engineers and Geologists

**Project No.: LP16027**

**Percolation Rate Conversion**

**Plate E-1A**

## LANDMARK CONSULTANTS, INC

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Project: | St Francis of Rome | | Project No: | LP16027 | | Date: | 3/18/16 |
| Test Hole No: | | I-2 | Tested By: | Alex A | | | |
| Depth of Test Hole, $D_T$: | | 5' | USCS Soil Classification: | | | | |
| Test Hole Dimensions (inches) | | | | | Length | Width | |
| Diameter (if round)= | | 6" | Sides (if rectangular)= | | | | |

Sandy Soil Criteria Test*

| Trial No. | Start Time | Stop Time | Time Interval, (min.) | Initial Depth to Water (in.) | Final Depth to Water (in.) | Change in Water Level (in.) | Greater than or Equal to 6"? (y/n) |
|---|---|---|---|---|---|---|---|
| 1 | 8:51 | 9:16 | 25.00 | 20.00 | 45.00 | 25.00 | y |
| 2 | 9:16 | 9:41 | 25.00 | 20.00 | 44.00 | 24.00 | y |

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Other wise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

| Trial No. | Start Time | Stop Time | $\Delta t$ Time Interval (min.) | $D_o$ Initial Depth to Water (in.) | $D_f$ Final Depth to Water (in.) | $\Delta D$ Change in Water Level (in.) | Percolation Rate (min./in.) |
|---|---|---|---|---|---|---|---|
| 1 | 9:43 | 9:53 | 10.00 | 18.00 | 25.00 | 7.00 | 1.43 |
| 2 | 9:53 | 10:03 | 10.00 | 25.00 | 33.00 | 8.00 | 1.25 |
| 3 | 10:03 | 10:13 | 10.00 | 16.00 | 23.00 | 7.00 | 1.43 |
| 4 | 10:13 | 10:23 | 10.00 | 19.00 | 25.00 | 6.00 | 1.67 |
| 5 | 10:23 | 10:33 | 10.00 | 25.00 | 32.00 | 7.00 | 1.43 |
| 6 | 10:33 | 10:43 | 10.00 | 32.00 | 38.00 | 6.00 | 1.67 |
| 7 | | | | | | | |
| 8 | | | | | | | |
| 9 | | | | | | | |
| 10 | | | | | | | |
| 11 | | | | | | | |
| 12 | | | | | | | |

COMMENTS:

LANDMARK
Geo-Engineers and Geologists

**Project No.:** LP16027

**Percolation Test Results**

**Plate**
E-2

# PERCOLATION RATE CONVERSION

**CLIENT:** **Diocese of San Bernardino**
**PROJECT:** **St Frances of Rome**
**PROJECT NO.:** **LP16027**
**DATE:** **3/18/2016**

**TEST HOLE NO:** I-2

Time interval, $\mathbf{\Delta t}$ = 10 minutes

Final Depth to Water, $D_f$ = 38 inches

[2]Test Hole Radius, **r** = 3 inches

Initial Depth to Water, $D_0$ = 32 inches

Total Depth of Test Hole, $D_T$ = 60 inches

The conversion equation is used:

$$I_t = \frac{\Delta H\ 60\ r}{\Delta t(r+2H_{avg}}$$

"$H_o$" is the initial height of water at the selected time interval

$$H_o = D_T - D_0 \qquad = 60 - 32 = 28\ \underline{\text{inches}}$$

"$H_f$" is the final height of water at the selected time interval

$$H_f = D_T - D_f \qquad = 60 - 38 = 22\ \underline{\text{inches}}$$

"$\Delta H$" is the change in height over the time interval

$$\Delta H = \Delta D = H_o - H_f \qquad = 28 - 22 = 6\ \underline{\text{inches}}$$

"$H_{avg}$" is the average head height over the time interval

$$H_{avg} = (H_o + H_f)/2 \qquad = (28 + 22)/2 = \underline{25\ \text{inches}}$$

"$I_t$" is the tested infiltration rate

$$I_t = \frac{\Delta H\ 60\ r}{\Delta t\left(r+2H_{avg}\right)} = \frac{(6\ \text{in})(60\text{min/hr})>3\text{in})}{(10\ \text{min})((3\ \text{in}) + 2\ (25\ \text{in}))} = \underline{1.98\ \text{in/hr}}$$

LANDMARK
Geo-Engineers and Geologists

**Project No.: LP16027**

**Percolation Rate Conversion**

**Plate**
**E-2A**

## LANDMARK CONSULTANTS, INC

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Project: | St Francis of Rome | | Project No: | LP16027 | | Date: | 3/18/16 |
| Test Hole No: | | I-3 | Tested By: | Alex A | | | |
| Depth of Test Hole, $D_T$: | | 5' | USCS Soil Classification: | | | | |
| Test Hole Dimensions (inches) | | | | | Length | Width | |
| Diameter (if round)= | | 6" | Sides (if rectangular)= | | | | |

Sandy Soil Criteria Test*

| Trial No. | Start Time | Stop Time | Time Interval, (min.) | Initial Depth to Water (in.) | Final Depth to Water (in.) | Change in Water Level (in.) | Greater than or Equal to 6"? (y/n) |
|---|---|---|---|---|---|---|---|
| 1 | 10:51 | 11:16 | 25.00 | 25.00 | 40.00 | 15.00 | y |
| 2 | 11:16 | 11:41 | 25.00 | 22.00 | 34.00 | 12.00 | y |

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Other wise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

| Trial No. | Start Time | Stop Time | $\Delta t$ Time Interval (min.) | $D_o$ Initial Depth to Water (in.) | $D_f$ Final Depth to Water (in.) | $\Delta D$ Change in Water Level (in.) | Percolation Rate (min./in.) |
|---|---|---|---|---|---|---|---|
| 1 | 11:43 | 11:53 | 10.00 | 7.00 | 13.00 | 6.00 | 1.67 |
| 2 | 11:53 | 12:03 | 10.00 | 13.00 | 18.00 | 6.00 | 1.67 |
| 3 | 12:03 | 12:13 | 10.00 | 18.00 | 23.00 | 5.00 | 2.00 |
| 4 | 12:13 | 12:23 | 10.00 | 19.00 | 25.00 | 6.00 | 1.67 |
| 5 | 12:23 | 12:33 | 10.00 | 25.00 | 31.00 | 6.00 | 1.67 |
| 6 | 12:33 | 12:43 | 10.00 | 31.00 | 36.00 | 5.00 | 2.00 |
| 7 | | | | | | | |
| 8 | | | | | | | |
| 9 | | | | | | | |
| 10 | | | | | | | |
| 11 | | | | | | | |
| 12 | | | | | | | |

COMMENTS:

# PERCOLATION RATE CONVERSION

CLIENT: Diocese of San Bernardino
PROJECT: St Frances of Rome
PROJECT NO.: LP16027
DATE: 3/18/2016

**TEST HOLE NO:** I-3

Time interval, $\Delta t$ = 10 minutes

Initial Depth to Water, $D_0$ = 31 inches

Final Depth to Water, $D_f$ = 36 inches

Total Depth of Test Hole, $D_T$ = 60 inches

[2]Test Hole Radius, **r** = 3 inches

The conversion equation is used:

$$I_t = \frac{\Delta H\ 60\ r}{\Delta t(r+2H_{avg}}$$

"$H_o$" is the initial height of water at the selected time interval

$$H_o = D_T - D_0 \qquad = 60 - 31 = 29 \text{ inches}$$

"$H_f$" is the final height of water at the selected time interval

$$H_f = D_T - D_f \qquad = 60 - 36 = 24 \text{ inches}$$

"$\Delta H$" is the change in height over the time interval

$$\Delta H = \Delta D = H_o - H_f \qquad = 29 - 24 = 5 \text{ inches}$$

"$H_{avg}$" is the average head height over the time interval

$$H_{avg} = (H_o + H_f)/2 \qquad = (29 + 24)/2 = 26.5 \text{ inches}$$

"$I_t$" is the tested infiltration rate

$$I_t = \frac{\Delta H\ 60\ r}{\Delta t\left(r+2H_{avg}\right)} = \frac{(5\text{ in})(60\text{min/hr})>3\text{in})}{(10\text{ min})((3\text{ in}) + 2\ (26.5\text{ in}))} = 1.61 \text{ in/hr}$$

LANDMARK
Geo-Engineers and Geologists

Project No.: LP16027

Percolation Rate Conversion

Plate
E-3A

## LANDMARK CONSULTANTS, INC

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **Project:** | St Francis of Rome | | **Project No:** | LP16027 | | **Date:** | 3/18/16 |
| **Test Hole No:** | | I-4 | **Tested By:** | Alex A | | | |
| **Depth of Test Hole, $D_T$:** | | 5' | **USCS Soil Classification:** | | | | |
| Test Hole Dimensions (inches) | | | | | Length | Width | |
| Diameter (if round)= | | 6" | Sides (if rectangular)= | | | | |

**Sandy Soil Criteria Test***

| Trial No. | Start Time | Stop Time | Time Interval, (min.) | Initial Depth to Water (in.) | Final Depth to Water (in.) | Change in Water Level (in.) | Greater than or Equal to 6"? (y/n) |
|---|---|---|---|---|---|---|---|
| 1 | 10:54 | 11:19 | 25.00 | 28.00 | 38.00 | 10.00 | y |
| 2 | 11:19 | 11:44 | 25.00 | 25.00 | 37.00 | 12.00 | y |

*If two consecutive measurements show that six inches of water seeps away in less than 25 minutes, the test shall be run for an additional hour with measurements taken every 10 minutes. Other wise, pre-soak (fill) overnight. Obtain at least twelve measurements per hole over at least six hours (approximately 30 minute intervals) with a precision of at least 0.25".

| Trial No. | Start Time | Stop Time | $\Delta t$ Time Interval (min.) | $D_0$ Initial Depth to Water (in.) | $D_f$ Final Depth to Water (in.) | $\Delta D$ Change in Water Level (in.) | Percolation Rate (min./in.) |
|---|---|---|---|---|---|---|---|
| 1 | 11:45 | 11:55 | 10.00 | 23.00 | 30.00 | 7.00 | 1.43 |
| 2 | 11:55 | 12:05 | 10.00 | 30.00 | 38.00 | 8.00 | 1.25 |
| 3 | 12:05 | 12:15 | 10.00 | 38.00 | 45.00 | 7.00 | 1.43 |
| 4 | 12:15 | 12:25 | 10.00 | 19.00 | 25.00 | 6.00 | 1.67 |
| 5 | 12:25 | 12:35 | 10.00 | 25.00 | 31.00 | 6.00 | 1.67 |
| 6 | 12:35 | 12:45 | 10.00 | 31.00 | 37.00 | 6.00 | 1.67 |
| 7 | | | | | | | |
| 8 | | | | | | | |
| 9 | | | | | | | |
| 10 | | | | | | | |
| 11 | | | | | | | |
| 12 | | | | | | | |

COMMENTS:

LANDMARK
Geo-Engineers and Geologists

**Project No.:** LP16027

**Percolation Test Results**

**Plate**
E-4

# PERCOLATION RATE CONVERSION

**CLIENT:** **Diocese of San Bernardino**
**PROJECT:** **St Frances of Rome**
**PROJECT NO.:** **LP16027**
**DATE:** **3/18/2016**

**TEST HOLE NO:** I-4

Time interval, $\Delta t$ = 10 minutes

Final Depth to Water, $D_f$ = 37 inches

[2]Test Hole Radius, **r** = 3 inches

Initial Depth to Water, $D_0$ = 31 inches

Total Depth of Test Hole, $D_T$ = 60 inches

The conversion equation is used:

$$I_t = \frac{\Delta H \, 60 \, r}{\Delta t(r + 2H_{avg}}$$

"$H_o$" is the initial height of water at the selected time interval

$$H_o = D_T - D_0 \qquad = 60 - 31 = 29 \text{ inches}$$

"$H_f$" is the final height of water at the selected time interval

$$H_f = D_T - D_f \qquad = 60 - 37 = 23 \text{ inches}$$

"$\Delta H$" is the change in height over the time interval

$$\Delta H = \Delta D = H_o - H_f \qquad = 29 - 23 = 6 \text{ inches}$$

"$H_{avg}$" is the average head height over the time interval

$$H_{avg} = (H_o + H_f)/2 \qquad = (29 + 23)/2 = 26 \text{ inches}$$

"$I_t$" is the tested infiltration rate

$$I_t = \frac{\Delta H \, 60 \, r}{\Delta t \, (r + 2H_{avg})} = \frac{(6 \text{ in})(60\text{min/hr})(3\text{in})}{(10 \text{ min})((3 \text{ in}) + 2 \ (26 \text{ in}))} = 1.96 \text{ in/hr}$$

# APPENDIX F

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