CORNERSTONE
EARTH GROUP

TYPE OF SERVICES	Geotechnical Investigation
PROJECT NAME	Clayton Community Church
LOCATION	1027 Pine Hollow Court Clayton, California
CLIENT	Clayton Community Church
PROJECT NUMBER	352-2-2
DATE	March 9, 2017 (revised February 27, 2019)

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Location	**1027 Pine Hollow Court** **Clayton, California**
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Client Address	**6055 Main Street** **Clayton, California**
Project Number	**352-2-2**
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Prepared by

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SECTION 1: INTRODUCTION

This geotechnical report was prepared for the sole use of Clayton Community Church for the property located at 1027 Pine Hollow Court in Clayton, California. The location of the site is shown on the Vicinity Map, Figure 1. For our use, we were provided with an updated plan titled "Clayton Community Church, 1027 Pine Hollow Court, Clayton, Page 01" prepared by EPIC Inc., dated February 28, 2019. We previously received a preliminary set of grading and improvement plans dated September 14, 2016 prepared by Cunha Engineering Inc.; however, we understand these plans are currently being updated to reflect the updated site plan prepared by EPIC Inc.

1.1 PROJECT DESCRIPTION

Clayton Community Church plans to construct a new church on the approximately 4.4-acre parcel located at 1027 Pine Hollow Court. Based on our review of the revised site plan, the main structure will include a 14,505 square foot building with sanctuary, classrooms, offices, and meeting areas. The project will also include a parking lot, utilities, paving, concrete flat work, and landscaping. We assume that grading will consist of minor cuts and fills less than 3 feet to create the new building pad. Building loads are assumed to be typical for this type of structure. A low retaining wall will be constructed near the top of the existing slope at the east side of the new church building. We understand that the planned building will be of wood-frame or light steel-frame construction with a concrete slab-on-grade floor.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposal dated April 21, 2016 and consisted of field and laboratory programs to evaluate physical and engineering properties of the subsurface soils, engineering analysis to prepare recommendations for site work and grading, building foundations, flatwork, retaining walls, and pavements, and preparation of this report. Brief descriptions of our exploration and laboratory programs are presented below.

1.3 EXPLORATION PROGRAM

Field exploration consisted of two (2) borings drilled using truck-mounted hollow-stem auger drilling equipment on February 16, 2017, and two (2) hand auger borings drilled February 23, 2017. The borings were drilled to depths ranging from 5 to 10 feet. The borings were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions.

During our previous feasibility study, we excavated six (6) test pits using rubber-tire backhoe equipment to depths of 4 to 6 feet. The pits were loosely backfilled and the loose fills will need to be re-worked during site grading unless the pits are outside the limits of sensitive improvements.

The approximate locations of our recent exploratory borings and previous test pits are shown on the Site Plan, Figure 2. Details regarding our field program are included in Appendix A.

1.4 LABORATORY TESTING PROGRAM

In addition to visual classification of samples, the laboratory program focused on obtaining data for foundation design and seismic ground deformation estimates. Testing included moisture contents, dry densities and Plasticity Index tests. Details regarding our laboratory program are included in Appendix B.

SECTION 2: REGIONAL SETTING

2.1 GEOLOGICAL SETTING

The site is located in the western Diablo Range of the Coast Ranges structural and geomorphic province of California, which is one mountain range in a series of northwesterly-aligned mountains forming the Coast Ranges geomorphic province of California that stretches from the Oregon border nearly to Point Conception. In the San Francisco Bay area, most of the Coast Ranges have developed on a basement of tectonically mixed Cretaceous- and Jurassic-age (70- to 200-million years old) rocks of the Franciscan Complex. Locally younger sedimentary and volcanic rocks cap these basement rocks. Still younger surficial deposits that reflect geologic conditions for the last million years or so cover most of the Coast Ranges.

Rocks of two main basement complexes juxtaposed by major regional faults are exposed in the East Bay hills and Diablo Range. These are the Franciscan Assemblage and the Coast Range Ophiolite. The basement complexes are unconformably overlain by Mesozoic and Cenozoic sedimentary and volcanic rocks. The two Mesozoic basement complexes are the Franciscan Complex and the Great Valley Complex (Graymer, 2000). The ophiolite rocks are the remnants of arc-related oceanic crust. The Great Valley Sequence is composed of turbidutes (sandstone, conglomerate and shale) of Jurassic and Cretaceous age that were deposited on top of the crustal rocks.

Movement on the many splays of the San Andreas Fault system has produced the dominant northwest-oriented structural and topographic trend seen throughout the Coast Ranges today. This trend reflects the boundary between two of the Earth's major tectonic plates: the North American plate to the east and the Pacific plate to the west. The San Andreas Fault system and its major branching faults is about 40 miles wide in the Bay area and extends from the San Gregorio Fault near the coastline to the Coast Ranges-Central Valley blind thrust at the western edge of the central valley as shown on the Regional Fault Map, Figure 3. The San Andreas Fault is the dominant structure in the system, nearly spanning the length of California, and capable of producing the highest magnitude earthquakes. Many other subparallel or branch faults within the San Andreas system are equally active and nearly as capable of generating large earthquakes. Right-lateral movement dominates on these faults but an increasingly large amount of thrust faulting resulting from compression across the system is now being identified also.

The geologic structure of the site area is severely complicated by folding and faulting associated with the Mt. Diablo anticline and related structures, which are part of an extensive belt of active deformation stretching from the northern Diablo Range to western Sacramento. Mt. Diablo is flanked by the seismically active faults, including the Greenville fault along its southeast side and by the Concord fault along its northwest side (Unruh, 2001; Jennings and Bryant, 2010).

2.2 REGIONAL SEISMICITY

The San Francisco Bay area region is one of the most seismically active areas in the Country. While seismologists cannot predict earthquake events, the U.S. Geological Survey’s Working Group on California Earthquake Probabilities 2015 revises earlier estimates from their 2008 (2008, UCERF2) publication. Compared to the previous assessment issued in 2008, the estimated rate of earthquakes around magnitude 6.7 (the size of the destructive 1994 Northridge earthquake) has gone down by about 30 percent. The expected frequency of such events statewide has dropped from an average of one per 4.8 years to about one per 6.3 years. However, in the new study, the estimate for the likelihood that California will experience a magnitude 8 or larger earthquake in the next 30 years has increased from about 4.7 percent for UCERF2 to about 7.0 percent for UCERF3.

UCERF3 estimates that each region of California will experience a magnitude 6.7 or larger earthquake in the next 30 years. Additionally, there is a 63 percent chance of at least one magnitude 6.7 or greater earthquake occurring in the Bay Area region between 2007 and 2036.

The faults considered capable of generating significant earthquakes are generally associated with the well-defined areas of crustal movement, which trend northwesterly. The table below presents the State-considered active faults within 25 kilometers of the site.

Table 1: Approximate Fault Distances

Fault Name	Distance	
	(miles)	(kilometers)
Greenville	1.1	1.8
Concord-Green Valley	3.6	5.9
Calaveras - North	7.7	12.5
Hayward (Total Length)	16.3	26.2
West Napa	22.8	36.8
Rodgers Creek	24.2	39.0

A regional fault map is presented as Figure 3, illustrating the relative distances of the site to significant fault zones.

SECTION 3: SITE CONDITIONS

3.1 SITE BACKGROUND

Based on our review of historical aerials and topographic maps, areas to the east of the site, including downtown Clayton, appears to have been developed around the turn of the 19th century. The site and surrounding land was developed as orchards prior to the 1940’s. The existing residence on the property was constructed by 1958 and residential development appears to have begun in the remaining area prior to the mid-1960s. With the exception of minor changes, the site appears to have remained in its current state since the early 1960’s.

3.2 SURFACE DESCRIPTION

The roughly 4½-acre parcel is located on the southeast side of Pine Hollow Court and to the west of the intersection of High Street and Oak Street. The parcel is bounded by residential development to the west, south and east, and Clayton Park and Mt. Diablo Elementary School to the north. A barbed-wire or metal fence borders the property.

The parcel is currently occupied by one single-family residence, a barn and small shed. The remainder of the parcel is undeveloped with the exception of several remnant walnut orchard trees. At the time of our site visit, the parcel was covered with 1 to 2 foot high grasses and weeds, and several remnant orchard trees and other mature trees. A gravel driveway from Pine Hollow Court extends onto the site near the residence.

Detailed topographic information was not available at this time. Based on our review of available topographic information, site grades currently range from about Elevation 460 feet along the west side of the site, to about Elevation 410 feet on the lower east end of the site (USGS, 7.5-Minute Clayton Quadrangle, 1982). The eastern approximately one-third of the site slopes downward to the east at an approximate inclination of 3:1 (horizontal:vertical). A seasonal stream was observed just east of the eastern property line. The stream channel is bounded by mature trees and bushes and appears to be about 5 to 8 feet deep along the parcel frontage.

3.3 SUBSURFACE CONDITIONS

Our recent explorations on the relatively level, western half of the site is blanketed by 4 to 6 feet of soft to very stiff, moist to wet lean clay and sandy lean clay. The upper 6 to 12 inches of the near-surface clays contained significant organics. The upper clay was underlain by loose to medium dense clayey sand to the maximum depth explored at 10 feet. Perched ground water was observed flowing through sandy clay/clayey sand soil at a depth of about 3 feet below the surface in Boring EB-4.

Our previous feasibility-level investigation dated March 22, 2016, was limited to the eastern-facing slope to aid in evaluating the subsurface conditions that may impact future development and grading on the slope. Our exploratory test pits (TP-1 through TP-6) encountered an approximately 6 to 12 inch thick layer of clayey topsoil mantling the slope that was soft to medium stiff and contained abundant organics. Below the surficial topsoil, a layer of stiff to very stiff sandy lean clay with varying percentages of gravel was observed to depths ranging from 4 to 5 feet. In test pits TP-1, 4 and 5, the stiff clay layer was underlain by weathered bedrock consisting of claystone with varying percentages of sand. The claystone was generally friable and intensely fractured. Bedrock was not encountered to the maximum depths explored in TP-2, 3 and 6.

3.3.1 Plasticity/Expansion Potential

We performed two Plasticity Index (PI) tests on representative samples. Test results were used to evaluate expansion potential of surficial soils. The results of the surficial PI tests indicated PIs ranging from 9 to 10, indicating low plasticity and expansion potential to wetting and drying cycles.

3.3.2 In-Situ Moisture Contents

Laboratory testing indicated that the in-situ moisture contents within the upper 10 feet range from approximately 2 to 10 percent over the estimated laboratory optimum moisture.

3.4 GROUND WATER

As noted above, perched ground water was encountered at a depth of approximately 3 feet during drilling for Boring EB-4. As the site is at the top of the hill and had received a significant amount of rain, any ground water is estimated to be seasonal, and we anticipate that static high ground water level will be on the order of 30 to 40 feet below current grades. Localized water seepage could be encountered at the soil/bedrock transition following periods of heavy rainfall. Fluctuations in ground water levels occur due to many factors including seasonal fluctuation, underground drainage patterns, regional fluctuations, and other factors.

SECTION 4: GEOLOGIC HAZARDS

4.1 FAULT RUPTURE

As discussed above several significant faults are located within 25 kilometers of the site. The site is not located within a State-designated Alquist-Priolo Earthquake Fault Zone. No known surface expression of fault traces is thought to cross the site; therefore, fault rupture hazard is not a significant geologic hazard at the site.

4.2 ESTIMATED GROUND SHAKING

Moderate to severe (design-level) earthquakes can cause strong ground shaking, which is the case for most sites within the Bay Area. A peak ground acceleration (PGA) was estimated for analysis using a value equal to F_{PGA} x PGA, as allowed in the 2016 edition of the California Building Code. For our liquefaction analysis we used a PGA of 0.75g.

4.3 LIQUEFACTION POTENTIAL

The site vicinity is not currently mapped by the State of California, but is within a zone mapped as having a very low liquefaction potential by the Association of Bay Area Governments (ABAG, 2007).

During strong seismic shaking, cyclically induced stresses can cause increased pore pressures within the soil matrix that can result in liquefaction triggering, soil softening due to shear stress loss, potentially significant ground deformation due to settlement within sandy liquefiable layers as pore pressures dissipate, and/or flow failures in sloping ground or where open faces are present (lateral spreading) (NCEER 1998). Limited field and laboratory data is available regarding ground deformation due to settlement; however, in clean sand layers settlement on the order of 2 to 3 percent of the liquefied layer thickness can occur. Soils most susceptible to liquefaction are loose, non-cohesive soils that are saturated and are bedded with poor drainage, such as sand and silt layers bedded with a cohesive cap.

As discussed in the “Subsurface” section above, we primarily encountered medium stiff to stiff cohesive soils underlain by bedrock. A localized layer of loose clayey sand was encountered in Boring EB-1; however, this layer appears to be localized and relatively shallow. In addition, the static design ground water level is anticipated to be greater than 30 feet below site grades. Based on the above, our screening of the site for liquefaction indicates a low potential for liquefaction and is in general agreement with local mapping for the site by ABAG.

4.4 LATERAL SPREADING

Lateral spreading is horizontal/lateral ground movement of relatively flat-lying soil deposits towards a free face such as an excavation, channel, or open body of water; typically lateral spreading is associated with liquefaction of one or more subsurface layers near the bottom of the exposed slope. As failure tends to propagate as block failures, it is difficult to analyze and estimate where the first tension crack will form.

The eastern property boundary is approximately 40 to 80 feet from a seasonal stream that is approximately 5 to 8 feet deep. Although the stream is likely underlain by Holocene-aged alluvial soils, the potential for liquefaction at the site appears to be low. Therefore, the potential for lateral spreading to impact the proposed development is considered low.

4.5 SEISMIC SETTLEMENT/UNSATURATED SAND SHAKING

Loose unsaturated sandy soils can settle during strong seismic shaking. As the soils encountered at the site were predominantly medium stiff to very stiff clays overlying medium dense sands or weathered bedrock, in our opinion, the potential for significant differential seismic settlement affecting the proposed improvements is low.

4.6 LANDSLIDING

We did not observe recent indications of slope movement or landslides on the east-facing slope. Our review of available historic aerial photographs also did not indicate past slope movement. Our experience with similar sites in the area indicates that natural or cut slopes steeper than 3:1 may be susceptible to shallow sloughing or minor debris flow movement within the upper clay soils mantling the hillside. Since the existing slope is inclined at about 3:1, the potential impact to the proposed development is considered low to moderate, unless the existing slope is steepened.

4.7 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone X, an area determined to be outside the 0.2% annual chance floodplain. The adjacent Mitchell Creek channel is designated as a special flood hazard area subject to inundation by the 1% annual chance flood (Zone AO), with flood depths of 1 to 3 feet (usually sheet flow on sloping terrain. The flood elevations are shown to range from about 401 to 402 feet at the High Street bridge crossing. We recommend the project civil engineer be retained to confirm this information and verify the base flood elevation, if appropriate.

SECTION 5: CONCLUSIONS

5.1 SUMMARY

From a geotechnical viewpoint, the project is feasible provided the concerns listed below are addressed in the project design. Descriptions of each concern with brief outlines of our recommendations follow the listed concerns.

- Presence of weak surficial soils
- Possible shallow bedrock

5.1.1 Weak Surficial Soils

As discussed in the “Subsurface” section, the surficial clayey soils encountered within the proposed building area were wet to moist and soft to medium stiff. These surficial soils will be moderately compressible and will not provide uniform support for the proposed structure, which could cause differential settlement for new foundations. To reduce the potential for differential settlement, we recommend that the shallow surficial soils be over-excavated and re-compacted prior to placing new fill in the building area. A more detailed discussion of the soft soil mitigation is discussed in the “Earthwork” section.

5.1.2 Shallow Bedrock

Although not encountered in our recent borings to a depth of 10 feet, relatively shallow weathered bedrock was previously encountered as shallow as 6 feet below the eastern slope area. If encountered, shallow bedrock may be impact excavation of deeper utility trenches. These impacts typically consist of additional time and heavier equipment requirements for excavation of trenches. If any excavations are planned for the eastern slope area, contractors should be made aware of the potential for shallow bedrock.

5.2 PLANS AND SPECIFICATIONS REVIEW

We recommend that we be retained to review the geotechnical aspects of the project structural, civil, and landscape plans and specifications, allowing sufficient time to provide the design team with any comments prior to issuing the plans for construction.

5.3 CONSTRUCTION OBSERVATION AND TESTING

As site conditions may vary significantly between the small-diameter borings performed during this investigation, we also recommend that a Cornerstone representative be present to provide geotechnical observation and testing during earthwork and foundation construction. This will allow us to form an opinion and prepare a letter at the end of construction regarding contractor compliance with project plans and specifications, and with the recommendations in our report. We will also be allowed to evaluate any conditions differing from those encountered during our investigation, and provide supplemental recommendations as necessary. For these reasons, the recommendations in this report are contingent of Cornerstone providing observation and testing during construction. Contractors should provide at least a 48-hour notice when scheduling our field personnel.

SECTION 6: EARTHWORK

6.1 SITE DEMOLITION, CLEARING AND PREPARATION

6.1.1 Site Stripping

The site should be stripped of all surface vegetation, and surface and subsurface improvements within the proposed development area. A detailed discussion of removal of soft surficial soils is provided later in this report. Surface vegetation and topsoil should be stripped to a sufficient depth to remove all material greater than 3 percent organic content by weight. Based on our site observations, surficial stripping should extend about 3 to 6 inches below existing grade in vegetated areas.

6.1.2 Tree and Shrub Removal

Trees and shrubs designated for removal should have the root balls and any roots greater than ½-inch diameter removed completely. Mature trees are estimated to have root balls extending to depths of 2 to 4 feet, depending on the tree size. Significant root zones are anticipated to extend to the diameter of the tree canopy. Grade depressions resulting from root ball removal should be cleaned of loose material and backfilled in accordance with the recommendations in the “Compaction” section of this report.

6.1.3 Demolition of Existing Slabs, Foundations and Pavements

Any existing slabs, foundations, and pavements should be completely removed from within planned building areas. Slabs, foundations, and pavements that extend into planned flatwork, pavement, or landscape areas may be left in place provided there is at least 3 feet of engineered fill overlying the remaining materials, they are shown not to conflict with new utilities, and that asphalt and concrete more than 10 feet square is broken up to provide subsurface drainage. A discussion of recycling existing improvements is provided later in this report.

6.1.4 Abandonment of Existing Utilities

All utilities should be completely removed from within planned building areas. For any utility line to be considered acceptable to remain within building areas, the utility line must be completely backfilled with grout or sand-cement slurry (sand slurry is not acceptable), the ends outside the building area capped with concrete, and the trench fills either removed and replaced as engineered fill with the trench side slopes flattened to at least 1:1, or the trench fills are determined not to be a risk to the structure. The assessment of the level of risk posed by the particular utility line will determine whether the utility may be abandoned in place or needs to be completely removed. The contractor should assume that all utilities will be removed from within building areas unless provided written confirmation from both the owner and the geotechnical engineer.

Utilities extending beyond the building area may be abandoned in place provided the ends are plugged with concrete, they do not conflict with planned improvements, and that the trench fills do not pose significant risk to the planned surface improvements.

The risks associated with abandoning utilities in place include the potential for future differential settlement of existing trench fills, and/or partial collapse and potential ground loss into utility lines that are not completely filled with grout. In general, the risk is relatively low for single utility lines less than 4 inches in diameter, and increases with increasing pipe diameter.

6.2 REMOVAL OF WEAK SURFICIAL SOILS

As discussed in the “Conclusions” section, weak surficial soils blanket the site. To reduce the potential for differential building settlement, the upper 3 feet of soil within the building pad area should be over-excavated and re-compacted prior to placing any new fill. The over-excavation should extend to a lateral distance of at least 3 feet beyond the building footprint. Provided the surficial soils and fills meet the “Material for Fill” requirements below, the soil may be reused when backfilling the excavation. Based on review of the samples collected from our borings, it appears that the fill may be reused. If materials are encountered that do not meet the requirements, such as debris, wood, trash, those materials should screened out of the remaining material and be removed from the site. Backfill of excavations should be placed in lifts and compacted in accordance with the “Compaction” section below.

6.3 TEMPORARY CUT AND FILL SLOPES

The contractor is responsible for maintaining all temporary slopes and providing temporary shoring where required. Temporary shoring, bracing, and cuts/fills should be performed in accordance with the strictest government safety standards. On a preliminary basis, the upper 8 feet at the site may be classified as OSHA Site C materials. A Cornerstone representative should be retained to confirm the preliminary site classification.

Excavations performed during site demolition and fill removal should be sloped at 2:1 (horizontal:vertical) within the upper 5 feet below building subgrade. Excavations extending more than 5 feet below building subgrade and excavations in pavement and flatwork areas should be slope at a 1:1 inclination unless the OSHA soil classification indicates that slope should not exceed 1.5:1.

6.4 SUBGRADE PREPARATION

After site clearing and demolition is complete, and prior to backfilling any excavations resulting from surficial soil removal or demolition, the excavation subgrade and subgrade within areas to receive additional site fills, slabs-on-grade and/or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted in accordance with the “Compaction” section below.

6.5 SUBGRADE STABILIZATION MEASURES

Soil subgrade and fill materials, especially soils with high fines contents such as clays and silty soils, can become unstable due to high moisture content, whether from high in-situ moisture contents or from winter rains. As the moisture content increases over the laboratory optimum, it becomes more likely the materials will be subject to softening and yielding (pumping) from construction loading or become unworkable during placement and compaction.

There are several methods to address potential unstable soil conditions and facilitate fill placement and trench backfill. Some of the methods are briefly discussed below. Implementation of the appropriate stabilization measures should be evaluated on a case-by-case basis according to the project construction goals and the particular site conditions.

6.5.1 Scarification and Drying

The subgrade may be scarified to a depth of approximately 12 inches and allowed to dry to near optimum conditions, if sufficient dry weather is anticipated to allow sufficient drying. More than one round of scarification may be needed to break up the soil clods.

6.5.2 Removal and Replacement

As an alternative to scarification, the contractor may choose to over-excavate the unstable soils and replace them with dry on-site or import materials. A Cornerstone representative should be present to provide recommendations regarding the appropriate depth of over-excavation, whether a geosynthethic (stabilization fabric or geogrid) is recommended, and what materials are recommended for backfill.

6.5.3 Chemical Treatment

Where the unstable area exceeds about 5,000 to 10,000 square feet and/or site winterization is desired, chemical treatment with quicklime (CaO), kiln-dust, or cement may be more cost-effective than removal and replacement. Recommended chemical treatment depths will typically range from 12 to 18 inches depending on the magnitude of the instability.

6.6 MATERIAL FOR FILL

6.6.1 Re-Use of On-site Soils

On-site soils with an organic content less than 3 percent by weight may be reused as general fill. General fill should not have lumps, clods or cobble pieces larger than 6 inches in diameter; 85 percent of the fill should be smaller than 2½ inches in diameter. Minor amounts of oversize material (smaller than 12 inches in diameter) may be allowed provided the oversized pieces are not allowed to nest together and the compaction method will allow for loosely placed lifts not exceeding 12 inches.

6.6.2 Potential Import Sources

Imported and non-expansive material should be inorganic with a Plasticity Index (PI) of 15 or less, and not contain recycled asphalt concrete where it will be used within the building areas. To prevent significant caving during trenching or foundation construction, imported material should have sufficient fines. Samples of potential import sources should be delivered to our office at least 10 days prior to the desired import start date. Information regarding the import source should be provided, such as any site geotechnical reports. If the material will be derived from an excavation rather than a stockpile, potholes will likely be required to collect samples from throughout the depth of the planned cut that will be imported. At a minimum, laboratory testing will include PI tests. Material data sheets for select fill materials (Class 2 aggregate base, ¾-inch crushed rock, quarry fines, etc.) listing current laboratory testing data (not older than 6 months from the import date) may be provided for our review without providing a sample. If current data is not available, specification testing will need to be completed prior to approval.

Environmental and soil corrosion characterization should also be considered by the project team prior to acceptance. Suitable environmental laboratory data to the planned import quantity should be provided to the project environmental consultant; additional laboratory testing may be required based on the project environmental consultant’s review. The potential import source should also not be more corrosive than the on-site soils, based on pH, saturated resistivity, and soluble sulfate and chloride testing.

6.7 COMPACTION REQUIREMENTS

All fills, and subgrade areas where fill, slabs-on-grade, and pavements are planned, should be placed in loose lifts 8 inches thick or less and compacted in accordance with ASTM D1557 (latest version) requirements as shown in the table below. In general, clayey soils should be compacted with sheepsfoot equipment and sandy/gravelly soils with vibratory equipment; open-graded materials such as crushed rock should be placed in lifts no thicker than 18 inches consolidated in place with vibratory equipment. Each lift of fill and all subgrade should be firm and unyielding under construction equipment loading in addition to meeting the compaction requirements to be approved. The contractor (with input from a Cornerstone representative) should evaluate the in-situ moisture conditions, as the use of vibratory equipment on soils with high moistures can cause unstable conditions. General recommendations for soil stabilization are provided in the “Subgrade Stabilization Measures” section of this report.

Table 2: Compaction Requirements

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
General Fill (within upper 5 feet)	On-Site Soils	90	>1
Trench Backfill	On-Site Soils	90	>1
Trench Backfill (upper 6 inches of pavement subgrade)	On-Site Soils	95	>1
Crushed Rock Fill	¾-inch Clean Crushed Rock	Consolidate In-Place	NA
Flatwork Subgrade	On-Site Soils	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base[3]	90	Optimum
Pavement Subgrade	On-Site Soils	95	>1
Pavement Aggregate Base	Class 2 Aggregate Base[3]	95	Optimum
Asphalt Concrete	Asphalt Concrete	95	NA

1 – Relative compaction based on maximum density determined by ASTM D1557 (latest version)
2 – Moisture content based on optimum moisture content determined by ASTM D1557 (latest version)
3 – Class 2 aggregate base shall conform to Caltrans Standard Specifications, latest edition, except that the relative compaction should be determined by ASTM D1557 (latest version)
4 – Using light-weight compaction or walls should be braced

6.8 TRENCH BACKFILL

Utility lines constructed within public right-of-way should be trenched, bedded and shaded, and backfilled in accordance with the local or governing jurisdictional requirements. Utility lines in private improvement areas should be constructed in accordance with the following requirements unless superseded by other governing requirements.

All utility lines should be bedded and shaded to at least 6 inches over the top of the lines with crushed rock (⅜-inch-diameter or greater) or well-graded sand and gravel materials conforming to the pipe manufacturer's requirements. Open-graded shading materials should be consolidated in place with vibratory equipment and well-graded materials should be compacted to at least 90 percent relative compaction with vibratory equipment prior to placing subsequent backfill materials.

General backfill over shading materials may consist of on-site native materials provided they meet the requirements in the "Material for Fill" section, and are moisture conditioned and compacted in accordance with the requirements in the "Compaction" section.

Where utility lines will cross perpendicular to strip footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated foundation settlement, or the utility lines should be backfilled to the bottom of footing with sand-cement slurry or lean concrete. Where utility lines will parallel footings and will extend below the "foundation plane of influence," an imaginary 1:1 plane projected down from the bottom edge of the footing, either the footing will need to be deepened so that the pipe is above the foundation plane of influence or the utility trench will need to be backfilled with sand-cement slurry or lean concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 psi.

6.9 SITE DRAINAGE

Surface runoff should not be allowed to flow over the top of or pond at the top or toe of engineered slopes or retaining walls. Ponding should also not be allowed on or adjacent to building foundations, slabs-on-grade, or pavements. Hardscape surfaces should slope at least 2 percent towards suitable discharge facilities; landscape areas should slope at least 3 percent towards suitable discharge facilities. Roof runoff should be directed away from building areas in closed conduits, to approved infiltration facilities, or on to hardscaped surfaces that drain to suitable facilities. Retention, detention or infiltration facilities should be spaced at least 10 feet from buildings, and preferably at least 5 feet from slabs-on-grade or pavements. However, if retention, detention or infiltration facilities are located within these zones, we recommend that these treatment facilities meet the requirements in the Storm Water Treatment Design Considerations section of this report.

6.10 LOW-IMPACT DEVELOPMENT (LID) IMPROVEMENTS

The Municipal Regional Permit (MRP) requires regulated projects to treat 100 percent of the amount of runoff identified in Provision C.3.d from a regulated project's drainage area with low impact development (LID) treatment measures onsite or at a joint stormwater treatment facility. LID treatment measures are defined as rainwater harvesting and use, infiltration, evapotranspiration, or biotreatment. A biotreatment system may only be used if it is infeasible to implement harvesting and use, infiltration, or evapotranspiration at a project site.

Technical infeasibility of infiltration may result from site conditions that restrict the operability of infiltration measures and devices. Various factors affecting the feasibility of infiltration treatment may create an environmental risk, structural stability risk, or physically restrict infiltration. The presence of any of these limiting factors may render infiltration technically infeasible for a proposed project. To aid in determining if infiltration may be feasible at the site, we provide the following site information regarding factors that may aid in determining the feasibility of infiltration facilities at the site.

- The near-surface soils at the site are clayey, and categorized as Hydrologic Soil Group D, and is expected to have infiltration rates of less than 0.2 inches per hour. In our opinion, these clayey soils will significantly limit the infiltration of stormwater.

- Seasonal high ground water is not mapped in the area, and therefore is expected to be greater than 10 feet below the base of the infiltration measures.

- Infiltration devices should be located at least 50 feet away from the existing hillside to prevent surficial instability or erosion of the hillside; otherwise, devices will need to be lined to prevent water infiltration and water will need to discharge to storm drain or to toe of existing slope.

- Infiltration measures, devices, or facilities may conflict with the location of existing or proposed underground utilities or easements. Infiltration measures, devices, or facilities should not be placed on top of or very near to underground utilities such that they discharge to the utility trench, restrict access, or cause stability concerns.

6.10.1 Storm Water Treatment Design Considerations

If storm water treatment improvements, such as shallow bio-retention swales, basins or pervious pavements, are required as part of the site improvements to satisfy Storm Water Quality (C.3) requirements, we recommend the following items be considered for design and construction.

6.10.1.1 General Bioswale Design Guidelines

- If possible, avoid placing bioswales or basins within 10 feet of the building perimeter or within 5 feet of exterior flatwork or pavements. If bioswales must be constructed within these setbacks, the side(s) and bottom of the trench excavation should be lined with 10-mil visqueen to reduce water infiltration into the surrounding clay.

- Bioswales constructed within 3 feet of proposed buildings may be within the foundation zone of influence for perimeter wall loads. Therefore, where bioswales will parallel foundations and will extend below the “foundation plane of influence,” an imaginary 1:1 plane projected down from the bottom edge of the foundation, the foundation will need to be deepened so that the bottom edge of the bioswale filter material is above the foundation plane of influence.

- The bottom of bioswale or detention areas should include a perforated drain placed at a low point, such as a shallow trench or sloped bottom, to reduce water infiltration into the surrounding soils near structural improvements, and to address the low infiltration capacity of the on-site clay soils.

6.10.1.2 Bioswale Infiltration Material

- Gradation specifications for bioswale filter material, if required, should be specified on the grading and improvement plans.

- Compaction requirements for bioswale filter material in non-landscaped areas or in pervious pavement areas, if any, should be indicated on the plans and specifications to satisfy the anticipated use of the infiltration area.

- If required, infiltration (percolation) testing should be performed on representative samples of potential bioswale materials prior to construction to check for general conformance with the specified infiltration rates.

- It should be noted that multiple laboratory tests may be required to evaluate the properties of the bioswale materials, including percolation, landscape suitability and possibly environmental analytical testing depending on the source of the material. We recommend that the landscape architect provide input on the required landscape suitability tests if bioswales are to be planted.

- If bioswales are to be vegetated, the landscape architect should select planting materials that do not reduce or inhibit the water infiltration rate, such as covering the bioswale with grass sod containing a clayey soil base.

- If required by governing agencies, field infiltration testing should be specified on the grading and improvement plans. The appropriate infiltration test method, duration and frequency of testing should be specified in accordance with local requirements.

- Due to the relatively loose consistency and/or high organic content of many bioswale filter materials, long-term settlement of the bioswale medium should be anticipated. To reduce initial volume loss, bioswale filter material should be wetted in 12 inch lifts during placement to pre-consolidate the material. Mechanical compaction should not be allowed, unless specified on the grading and improvement plans, since this could significantly decrease the infiltration rate of the bioswale materials.

- It should be noted that the volume of bioswale filter material may decrease over time depending on the organic content of the material. Additional filter material may need to be added to bioswales after the initial exposure to winter rains and periodically over the life of the bioswale areas, as needed.

6.10.1.3 Bioswale Construction Adjacent to Pavements

If bio-infiltration swales or basins are considered adjacent to proposed parking lots or exterior flatwork, we recommend that mitigative measures be considered in the design and construction of these facilities to reduce potential impacts to flatwork or pavements. Exterior flatwork, concrete curbs, and pavements located directly adjacent to bio-swales may be susceptible to settlement or lateral movement, depending on the configuration of the bioswale and the setback between the improvements and edge of the swale. To reduce the potential for distress to these improvements due to vertical or lateral movement, the following options should be considered by the project civil engineer:

- Improvements should be setback from the vertical edge of a bioswale such that there is at least 1 foot of horizontal distance between the edge of improvements and the top edge of the bioswale excavation for every 1 foot of vertical bioswale depth.

- Concrete curbs for pavements, or lateral restraint for exterior flatwork, located directly adjacent to a vertical bioswale cut should be designed to resist lateral earth pressures in accordance with the recommendations in the “Retaining Walls” section of this report, or concrete curbs or edge restraint should be adequately keyed into the native soil or engineered to reduce the potential for rotation or lateral movement of the curbs.

6.11 PERMANENT EROSION CONTROL MEASURES

Should any grading occur on the eastern hillside of the property, periodic maintenance after construction to reduce the potential for erosion and sloughing may be necessary. At a minimum all slopes should be vegetated by hydroseeding or other landscape ground cover. The establishment of vegetation will help reduce runoff velocities, allow some infiltration and transpiration, trap sediment within runoff, and protect the soil from raindrop impact. Depending on the exposed material type and the slope inclination, more aggressive erosion control measures may be needed to protect slopes for one or more winter seasons while vegetation is establishing. For slopes with inclinations of 2:1 (horizontal:vertical) or greater, erosion control may consist of jute netting, straw matting, or erosion control blankets used in combination with hydroseeding.

Both construction and post-construction Storm Water Pollution Prevention Plans (SWPPPs) should be prepared for the project-specific requirements. We recommend that final grading plans be provided for our review.

6.12 LANDSCAPE CONSIDERATIONS

We recommend greatly reducing the amount of surface water infiltrating these soils near foundations, exterior slabs-on-grade and the existing east-facing slope. This can typically be achieved by:

- Using drip irrigation
- Avoiding open planting within 3 feet of the building perimeter or near the top of existing slopes
- Regulating the amount of water distributed to lawns or planter areas by using irrigation timers
- Selecting landscaping that requires little or no watering, especially near foundations.

We recommend that the landscape architect consider these items when developing landscaping plans.

SECTION 7: FOUNDATIONS

7.1 SUMMARY OF RECOMMENDATIONS

In our opinion, the proposed structure may be supported on shallow foundations provided the recommendations in the “Earthwork” section and the sections below are followed.

7.2 SEISMIC DESIGN CRITERIA

We understand that the project structural design will be based on the 2016 California Building Code (CBC), which provides criteria for the seismic design of buildings in Chapter 16. The "Seismic Coefficients" used to design buildings are established based on a series of tables and figures addressing different site factors, including the soil profile in the upper 100 feet below grade and mapped spectral acceleration parameters based on distance to the controlling seismic source/fault system. Based on our borings and review of local geology, the site is underlain by alluvial soils with typical SPT "N" values between 15 and 50 blows per foot and very dense soil or soft bedrock. Therefore, we have classified the site as Soil Classification C. The mapped spectral acceleration parameters S_S and S_1 were calculated using the USGS web-based program *U.S. Seismic Design Maps* (http://geohazards.usgs.gov/designmaps/us/application.php), Version 3.1.0, revision date July 11, 2013, based on the site coordinates presented below and the site classification. The table below lists the various factors used to determine the seismic coefficients and other parameters.

Table 3: CBC Site Categorization and Site Coefficients

Classification/Coefficient	**Design Value**
Site Class	C
Site Latitude	37.940205°
Site Longitude	-121.937917°
0.2-second Period Mapped Spectral Acceleration[1], S_S	2.011g
1-second Period Mapped Spectral Acceleration[1], S_1	0.617g
Short-Period Site Coefficient – Fa	1.0
Long-Period Site Coefficient – Fv	1.3
0.2-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects - S_{MS}	2.011g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	0.802g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.341g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	0.535g

[1] For Site Class B, 5 percent damped.

7.3 SHALLOW FOUNDATIONS

7.3.1 Spread Footings

Spread footings should bear on natural, undisturbed soil or engineered fill, be at least 15 inches wide, and extend at least 15 inches below the lowest adjacent grade. Lowest adjacent grade is defined as the deeper of the following: 1) bottom of the adjacent interior slab-on-grade, or 2) finished exterior grade, excluding landscaping topsoil.

Footings constructed to the above dimensions and in accordance with the "Earthwork" recommendations of this report are capable of supporting maximum allowable bearing pressures of 2,000 psf for dead loads, 3,000 psf for combined dead plus live loads, and 4,000 psf for all loads including wind and seismic. These pressures are based on factors of safety of 3.0, 2.0, and 1.5 applied to the ultimate bearing pressure for dead, dead plus live, and all loads, respectively. These pressures are net values; the weight of the footing may be neglected for the portion of the footing extending below grade (typically, the full footing depth). Top and bottom mats of reinforcing steel should be included in continuous footings to help span irregularities and differential settlement.

7.3.2 Footing Settlement

Structural loads were not provided to us at the time this report was prepared. Based on the general range of anticipated building dead plus live loads of roughly 2 to 4 kips per lineal foot for exterior walls and 25 to 50 kips for interior column loads and the allowable bearing pressures presented above, we estimate that the total static footing settlement will be on the order of ½ inch or less, with up to approximately ¼ inch of post-construction differential settlement between adjacent foundation elements, assumed to be on the order of 30 feet. As our footing loads were assumed, we recommend we be retained to review the final footing layout and loading, and verify the settlement estimates above.

7.3.3 Lateral Loading

Lateral loads may be resisted by friction between the bottom of footing and the supporting subgrade, and also by passive pressures generated against footing sidewalls. An ultimate frictional resistance of 0.45 applied to the footing dead load, and an ultimate passive pressure based on an equivalent fluid pressure of 450 pcf may be used in design. The structural engineer should apply an appropriate factor of safety (such as 1.5) to the ultimate values above. Where footings are adjacent to landscape areas without hardscape, the upper 12 inches of soil should be neglected when determining passive pressure capacity.

7.3.4 Spread Footing Construction Considerations

Where utility lines will cross perpendicular to strip footings, the footing should be deepened to encase the utility line, providing sleeves or flexible cushions to protect the pipes from anticipated foundation settlement, or the utility lines should be backfilled to the bottom of footing with sand-cement slurry or lean concrete. Where utility lines will parallel footings and will extend below the "foundation plane of influence," an imaginary 1:1 plane projected down from the bottom edge of the footing, either the footing will need to be deepened so that the pipe is above the foundation plane of influence or the utility trench will need to be backfilled with sand-cement slurry or lean concrete within the influence zone. Sand-cement slurry used within foundation influence zones should have a minimum compressive strength of 75 psi.

Footing excavations should be filled as soon as possible or be kept moist until concrete placement by regular sprinkling to prevent desiccation. A Cornerstone representative should observe all footing excavations prior to placing reinforcing steel and concrete. If there is a significant schedule delay between our initial observation and concrete placement, we may need to re-observe the excavations.

SECTION 8: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

8.1 INTERIOR SLABS-ON-GRADE

The proposed slab-on-grade may be constructed over subgrade prepared in accordance with the recommendations in the "Earthwork" section of this report. If moisture-sensitive floor coverings are planned, the recommendations in the "Interior Slabs Moisture Protection Considerations" section below may be incorporated in the project design if desired. If significant time elapses between initial subgrade preparation and slab-on-grade construction, the subgrade should be proof-rolled to confirm subgrade stability, and if the soil has been allowed to dry out, the subgrade should be re-moisture conditioned to at least 1 to 2 percent over the optimum moisture content.

The structural engineer should determine the appropriate slab reinforcement for the loading requirements and considering the expansion potential of the underlying soils. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness.

8.2 INTERIOR SLABS MOISTURE PROTECTION CONSIDERATIONS

The following general guidelines for concrete slab-on-grade construction where floor coverings are planned are presented for the consideration by the developer, design team, and contractor. These guidelines are based on information obtained from a variety of sources, including the American Concrete Institute (ACI) and are intended to reduce the potential for moisture-related problems causing floor covering failures, and may be supplemented as necessary based on project-specific requirements. The application of these guidelines or not will not affect the geotechnical aspects of the slab-on-grade performance.

- Place a minimum 10-mil-thick vapor retarder conforming to ASTM E 1745, Class C requirements or better directly below the concrete slab; the vapor retarder should extend to the slab edges and be sealed at all seams and penetrations in accordance with manufacturer's recommendations and ASTM E 1643 requirements. A 4-inch-thick capillary break, consisting of ½- to ¾-inch crushed rock with less than 5 percent passing the No. 200 sieve, should be placed below the vapor retarder and consolidated in place with vibratory equipment.

- The concrete water:cement ratio should be 0.45 or less. Mid-range plasticizers may be used to increase concrete workability and facilitate pumping and placement.

- Water should not be added after initial batching unless the slump is less than specified and/or the resulting water:cement ratio will not exceed 0.45.

- Polishing the concrete surface with metal trowels is not recommended.
- Where floor coverings are planned, all concrete surfaces should be properly cured.
- Water vapor emission levels and concrete pH should be determined in accordance with ASTM F1869-98 and F710-98 requirements and evaluated against the floor covering manufacturer's requirements prior to installation.

8.3 EXTERIOR FLATWORK

Exterior concrete flatwork subject to pedestrian and/or occasional light pick up loading should be at least 4 inches thick and supported on at least 4 inches of Class 2 aggregate base overlying subgrade prepared in accordance with the "Earthwork" recommendations of this report. Flatwork that will be subject to heavier or frequent vehicular loading should be designed in accordance with the recommendations in the "Vehicular Pavements" section below. To help reduce the potential for uncontrolled shrinkage cracking, adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness. Flatwork should be isolated from adjacent foundations or retaining walls except where limited sections of structural slabs are included to help span irregularities in retaining wall backfill at the transitions between at-grade and on-structure flatwork.

SECTION 9: VEHICULAR PAVEMENTS

9.1 ASPHALT CONCRETE

The following asphalt concrete pavement recommendations tabulated below are based on the Procedure 608 of the Caltrans Highway Design Manual, estimated traffic indices for various pavement-loading conditions, and on a design R-value of 5. The design R-value was chosen based on the results of the laboratory testing performed on a surficial soil sample collected on the site and engineering judgment considering the observed surface conditions.

Table 4: Asphalt Concrete Pavement Recommendations, Design R-value = 5

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base* (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	8.0	10.5
4.5	2.5	10.0	12.5
5.0	3.0	10.0	13.0
5.5	3.0	12.0	15.0
6.0	3.5	13.0	16.5
6.5	4.0	14.0	18.0

*Caltrans Class 2 aggregate base; minimum R-value of 78

Frequently, the full asphalt concrete section is not constructed prior to construction traffic loading. This can result in significant loss of asphalt concrete layer life, rutting, or other pavement failures. To improve the pavement life and reduce the potential for pavement distress through construction, we recommend the full design asphalt concrete section be constructed prior to construction traffic loading. Alternatively, a higher traffic index may be chosen for the areas where construction traffic will be use the pavements.

9.2 PORTLAND CEMENT CONCRETE

The exterior Portland Cement Concrete (PCC) pavement recommendations tabulated below are based on methods presented in the Portland Cement Association (PCA) design manual (PCA, 1984). We have provided a few pavement alternatives as an anticipated Average Daily Truck Traffic (ADTT) was not provided. An allowable ADTT should be chosen that is greater than what is expected for the development.

Table 5: PCC Pavement Recommendations, Design R-value = 5

Allowable ADTT	Minimum PCC Thickness (inches)
0.8	5
13	5½
130	6

The PCC thicknesses above are based on a concrete compressive strength of at least 3,500 psi, supporting the PCC on at least 4 inches of Class 2 aggregate base compacted as recommended in the “Earthwork” section, and laterally restraining the PCC with curbs or concrete shoulders. Adequate expansion and control joints should be included. Consideration should be given to limiting the control joint spacing to a maximum of about 2 feet in each direction for each inch of concrete thickness.

9.2.1 Stress Pads for Trash Enclosures

Pads where trash containers will be stored, and where garbage trucks will park while emptying trash containers, should be constructed on Portland Cement Concrete. We recommend that the trash enclosure pads and stress (landing) pads where garbage trucks will store, pick up, and empty trash should consist of a minimum PCC thickness of 7 inches overlying at least 9 inches of Class 2 aggregate base. The compressive strength, underlayment, and construction details should be consistent with the above recommendations for PCC pavements.

9.3 PAVEMENT CUTOFF

Surface water penetration into the pavement section can significantly reduce the pavement life. While quantifying the life reduction is difficult, a normal 20-year pavement design could be reduce to less than 10 years; therefore, increased long-term maintenance may be required.

It would be beneficial to include a pavement cut-off, such as deepened curbs, redwood-headers, or “Deep-Root Moisture Barriers” that are keyed at least 4 inches into the pavement subgrade. This will help limit the additional long-term maintenance.

SECTION 10: RETAINING WALLS

10.1 STATIC LATERAL EARTH PRESSURES

The structural design of any site retaining wall should include resistance to lateral earth pressures that develop from the soil behind the wall, any undrained water pressure, and surcharge loads acting behind the wall. Provided a drainage system is constructed behind the wall to prevent the build-up of hydrostatic pressures as discussed in the section below, we recommend that the walls be designed for the following pressures:

Table 6: Recommended Lateral Earth Pressures

Sloping Backfill Inclination (horizontal:vertical)	**Lateral Earth Pressure***	
	Unrestrained – Cantilever Wall	Restrained – Braced Wall
Level	45 pcf	45 pcf + 8H
3:1	55 pcf	55 pcf + 8H
2½:1	60 pcf	60 pcf + 8H
2:1	65 pcf	65 pcf + 8H
Additional Surcharge Loads	⅓ of vertical loads at top of wall	½ of vertical loads at top of wall

* Lateral earth pressures are based on an equivalent fluid pressure
** H is the distance in feet between the bottom of footing and top of retained soil

If adequate drainage cannot be provided behind the wall, an additional equivalent fluid pressure of 40 pcf should be added to the values above for both restrained and unrestrained walls for the portion of the wall that will not have drainage. Damp proofing or waterproofing of the walls may be considered where moisture penetration and/or efflorescence are not desired.

The 2016 CBC states that lateral pressures from earthquakes should be considered in the design of retaining walls. We understand minor landscaping walls (i.e. walls 6 feet or less in height) may be proposed. In our opinion, design of these walls for seismic lateral earth pressures in addition to static earth pressures is not warranted. Should retaining walls greater than 6 feet tall be proposed, further analysis may be necessary to determine the applicable seismic increment above the design earth pressure.

10.2 WALL DRAINAGE

Adequate drainage should be provided by a subdrain system behind all walls. This system should consist of a 4-inch minimum diameter perforated pipe placed near the base of the wall (perforations placed downward). The pipe should be bedded and backfilled with Class 2 Permeable Material per Caltrans Standard Specifications, latest edition. The permeable backfill should extend at least 12 inches out from the wall and to within 2 feet of outside finished grade. Alternatively, ½-inch to ¾-inch crushed rock may be used in place of the Class 2 Permeable Material provided the crushed rock and pipe are enclosed in filter fabric, such as Mirafi 140N or approved equivalent. The upper 2 feet of wall backfill should consist of compacted on-site soil. The subdrain outlet should be connected to a free-draining outlet or sump.

Miradrain, Geotech Drainage Panels, or equivalent drainage matting can be used for wall drainage as an alternative to the Class 2 Permeable Material or drain rock backfill. Horizontal strip drains connecting to the vertical drainage matting may be used in lieu of the perforated pipe and crushed rock section. The vertical drainage panel should be connected to the perforated pipe or horizontal drainage strip at the base of the wall, or to some other closed or through-wall system such as the TotalDrain system from AmerDrain. Sections of horizontal drainage strips should be connected with either the manufacturer’s connector pieces or by pulling back the filter fabric, overlapping the panel dimples, and replacing the filter fabric over the connection. At corners, a corner guard, corner connection insert, or a section of crushed rock covered with filter fabric must be used to maintain the drainage path.

Drainage panels should terminate 18 to 24 inches from final exterior grade. The Miradrain panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil.

10.3 BACKFILL

Where surface improvements will be located over the retaining wall backfill, backfill placed behind the walls should be compacted to at least 95 percent relative compaction using light compaction equipment. Where no surface improvements are planned, backfill should be compacted to at least 90 percent. If heavy compaction equipment is used, the walls should be temporarily braced.

10.4 FOUNDATIONS

Retaining walls may be supported on a continuous spread footing designed in accordance with the recommendations presented in the “Foundations” section of this report.

SECTION 11: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of Clayton Community Church specifically to support the design of the Clayton Community Church project in Clayton, California. The opinions, conclusions, and recommendations presented in this report have been formulated in accordance with accepted geotechnical engineering practices that exist in Northern California at the time this report was prepared. No warranty, expressed or implied, is made or should be inferred.

Recommendations in this report are based upon the soil and ground water conditions encountered during our subsurface exploration. If variations or unsuitable conditions are encountered during construction, Cornerstone must be contacted to provide supplemental recommendations, as needed.

Clayton Community Church may have provided Cornerstone with plans, reports and other documents prepared by others. Clayton Community Church understands that Cornerstone reviewed and relied on the information presented in these documents and cannot be responsible for their accuracy.

Cornerstone prepared this report with the understanding that it is the responsibility of the owner or his representatives to see that the recommendations contained in this report are presented to other members of the design team and incorporated into the project plans and specifications, and that appropriate actions are taken to implement the geotechnical recommendations during construction.

Conclusions and recommendations presented in this report are valid as of the present time for the development as currently planned. Changes in the condition of the property or adjacent properties may occur with the passage of time, whether by natural processes or the acts of other persons. In addition, changes in applicable or appropriate standards may occur through legislation or the broadening of knowledge. Therefore, the conclusions and recommendations presented in this report may be invalidated, wholly or in part, by changes beyond Cornerstone's control. This report should be reviewed by Cornerstone after a period of three (3) years has elapsed from the date of this report. In addition, if the current project design is changed, then Cornerstone must review the proposed changes and provide supplemental recommendations, as needed.

An electronic transmission of this report may also have been issued. While Cornerstone has taken precautions to produce a complete and secure electronic transmission, please check the electronic transmission against the hard copy version for conformity.

Recommendations provided in this report are based on the assumption that Cornerstone will be retained to provide observation and testing services during construction to confirm that conditions are similar to that assumed for design, and to form an opinion as to whether the work has been performed in accordance with the project plans and specifications. If we are not retained for these services, Cornerstone cannot assume any responsibility for any potential claims that may arise during or after construction as a result of misuse or misinterpretation of Cornerstone's report by others. Furthermore, Cornerstone will cease to be the Geotechnical-Engineer-of-Record if we are not retained for these services.

SECTION 12: REFERENCES

Association of Bay Area Governments (ABAG), 2011, Interactive Liquefaction Hazard Map: http://quake.abag.ca.gov/liquifaction/

California Building Code, 2016, Structural Engineering Design Provisions, Vol. 2.

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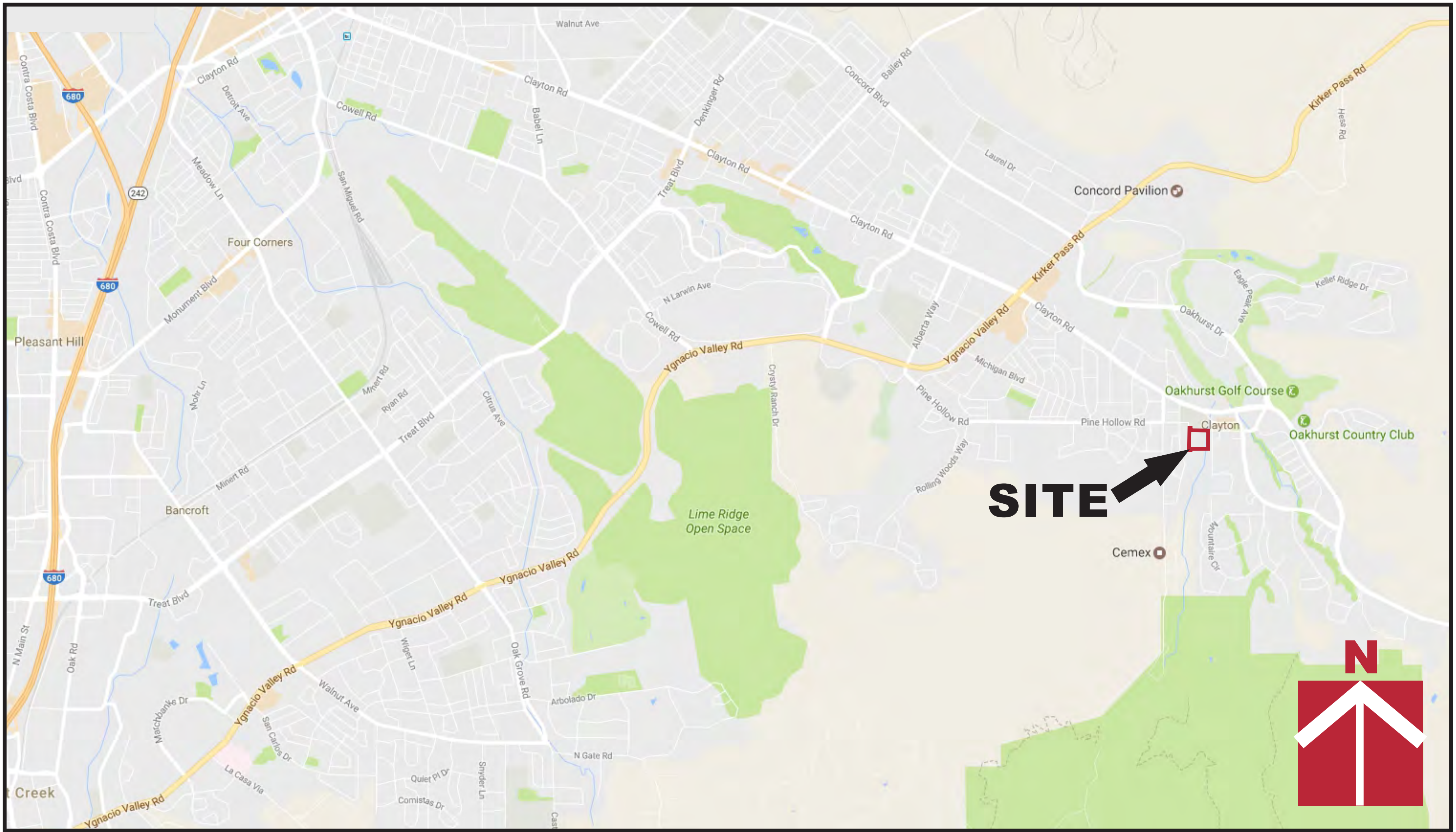
USGS, 2016, Earthquake Ground Motion Parameters - A Computer Program for determining mapped ground motion parameters available at http://Earthquake.usgs.gov/designmaps/us/application.php.

Working Group on California Earthquake Probabilities, 2013, The Uniform Earthquake Rupture Forecast, Version 3 (UCERF3) - The Time-Independent Model: U.S.Gelogical Survey Open File Report 2013-1165.

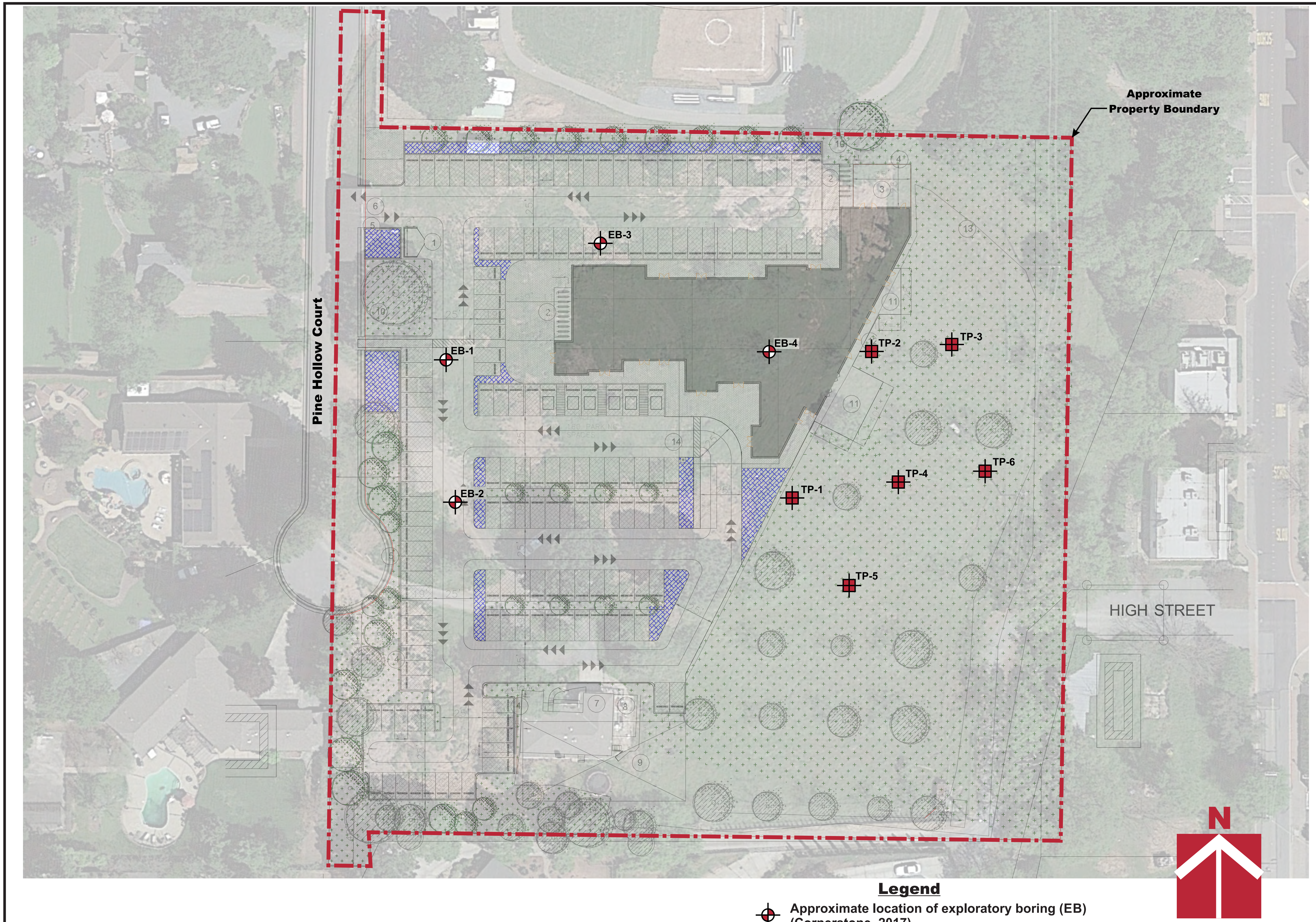


CORNERSTONE EARTH GROUP

Vicinity Map

Clayton Community Church
Clayton, CA

Project Number	352-2-2
Figure Number	Figure 1
Date	March 2017
Drawn By	RRN



Legend

⊕ Approximate location of exploratory boring (EB) (Cornerstone, 2017)

■ Approximate location of exploratory test pit (TP) (Cornerstone, 2013)

0 60 120
APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 4/2/2018
Overlay by EPIC, Site Plan - Page 1, dated 2/28/2019

Project Number 352-2-2

Figure Number Figure 2

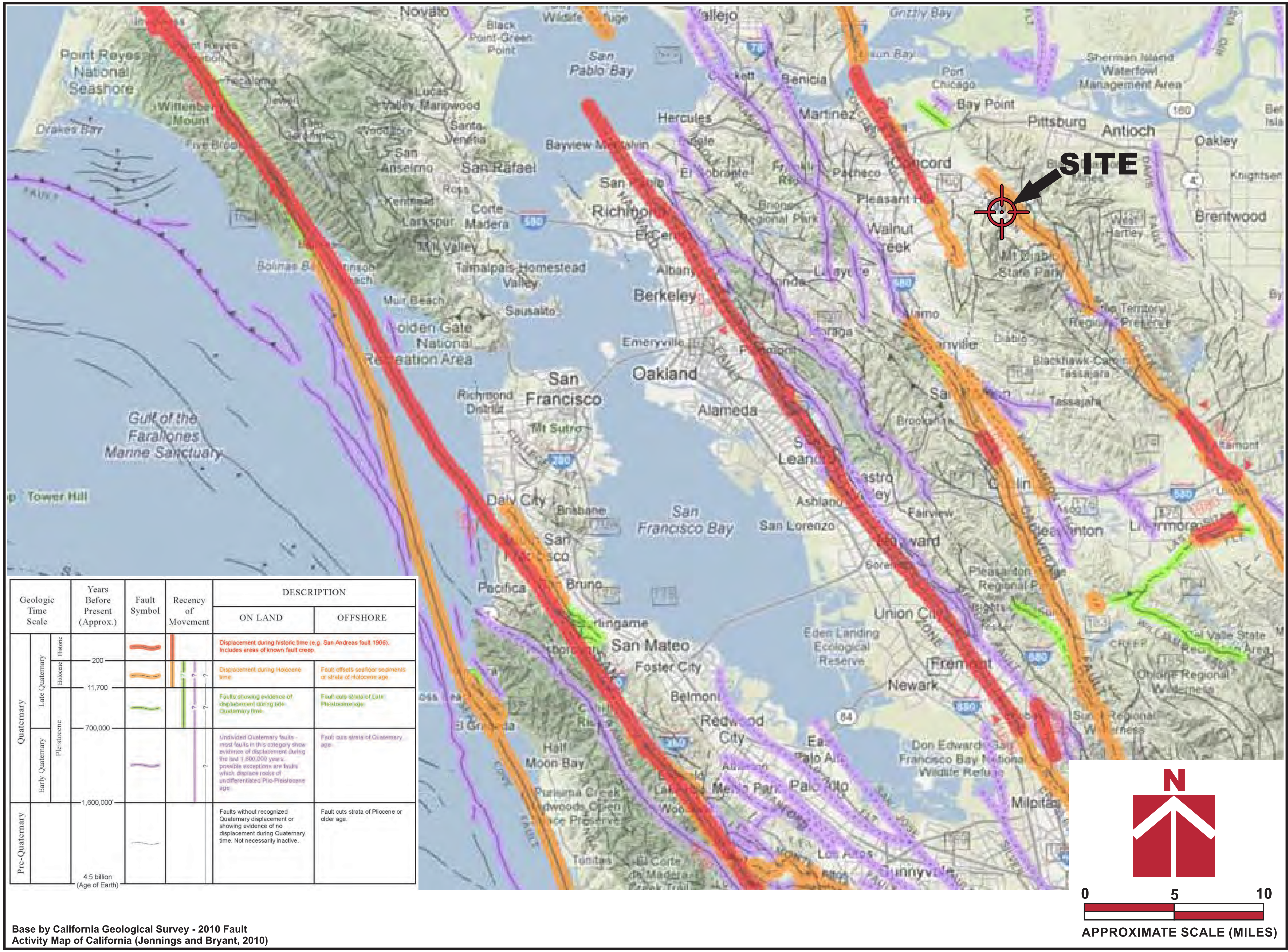
Date February 2019

Drawn By RRN

Site Exploration Plan

Clayton Community Church
Clayton, CA

CORNERSTONE
EARTH GROUP



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.	
		Holocene	200	🟧		Displacement during Holocene time	Fault offsets seafloor sediments or strata of Holocene age
		Pleistocene	11,700	🟩		Faults showing evidence of displacement during late Quaternary time	Fault cuts strata of Late Pleistocene age
	Early Quaternary		700,000	🟪		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
Pre-Quaternary			1,600,000 4.5 billion (Age of Earth)			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.

Base by California Geological Survey - 2010 Fault Activity Map of California (Jennings and Bryant, 2010)

Project Number
352-2-2

Figure Number
Figure 3

Drawn By
RRN

Date
March 2017

Regional Fault Map

Clayton Community Church
Clayton, CA

CORNERSTONE EARTH GROUP

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program. Two 8-inch-diameter exploratory borings were drilled on February 16, 2017, to depths of 5 to 10 feet using truck-mounted, hollow-stem auger drilling equipment. Two 4-inch diameter exploratory borings were drilled on February 23, 2017 to depths of 6 to 8 feet using hand auger drilling and sampling equipment. Previously, six test pits were excavated between 4 and 6 feet deep on February 6, 2013 using rubber tired backhoe equipment. The approximate locations of exploratory borings and test pits are shown on the Site Plan, Figure 2. The soils encountered were continuously logged in the field by our representative and described in accordance with the Unified Soil Classification System (ASTM D2488). Boring logs, as well as a key to the classification of the soil and bedrock, are included as part of this appendix.

Boring and test pit locations were approximated using existing site boundaries, a hand held GPS unit, and other site features as references. Boring and test pit elevations were not determined. The locations of the borings test pits should be considered accurate only to the degree implied by the method used.

Representative soil samples were obtained from the borings at selected depths. All samples were returned to our laboratory for evaluation and appropriate testing. The standard penetration resistance blow counts were obtained by dropping a 140-pound hammer through a 30-inch free fall. The 2-inch O.D. split-spoon sampler was driven 18 inches and the number of blows was recorded for each 6 inches of penetration (ASTM D1586). 2.5-inch I.D. samples were obtained using a Modified California Sampler driven into the soil with the 140-pound hammer previously described. Unless otherwise indicated, the blows per foot recorded on the boring log represent the accumulated number of blows required to drive the last 12 inches. The various samplers are denoted at the appropriate depth on the boring logs.

Field tests included an evaluation of the unconfined compressive strength of the soil samples using a pocket penetrometer device. The results of these tests are presented on the individual boring logs at the appropriate sample depths.

Attached boring and test pit logs and related information depict subsurface conditions at the locations indicated and on the date designated on the logs. Subsurface conditions at other locations may differ from conditions occurring at these boring [and CPT] locations. The passage of time may result in altered subsurface conditions due to environmental changes. In addition, any stratification lines on the logs represent the approximate boundary between soil types and the transition may be gradual.

UNIFIED SOIL CLASSIFICATION (ASTM D-2487-98)

MATERIAL TYPES	CRITERIA FOR ASSIGNING SOIL GROUP NAMES			GROUP SYMBOL	SOIL GROUP NAMES & LEGEND	
COARSE-GRAINED SOILS >50% RETAINED ON NO. 200 SIEVE	GRAVELS >50% OF COARSE FRACTION RETAINED ON NO 4. SIEVE	CLEAN GRAVELS <5% FINES	Cu>4 AND 1<Cc<3	GW	WELL-GRADED GRAVEL	●
			Cu>4 AND 1>Cc>3	GP	POORLY-GRADED GRAVEL	○
		GRAVELS WITH FINES >12% FINES	FINES CLASSIFY AS ML OR CL	GM	SILTY GRAVEL	○
			FINES CLASSIFY AS CL OR CH	GC	CLAYEY GRAVEL	▨
	SANDS >50% OF COARSE FRACTION PASSES ON NO 4. SIEVE	CLEAN SANDS <5% FINES	Cu>6 AND 1<Cc<3	SW	WELL-GRADED SAND	⁘
			Cu>6 AND 1>Cc>3	SP	POORLY-GRADED SAND	⁘
		SANDS AND FINES >12% FINES	FINES CLASSIFY AS ML OR CL	SM	SILTY SAND	▥
			FINES CLASSIFY AS CL OR CH	SC	CLAYEY SAND	▨
FINE-GRAINED SOILS >50% PASSES NO. 200 SIEVE	SILTS AND CLAYS LIQUID LIMIT<50	INORGANIC	PI>7 AND PLOTS>"A" LINE	CL	LEAN CLAY	▨
			PI>4 AND PLOTS<"A" LINE	ML	SILT	▥
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OL	ORGANIC CLAY OR SILT	☰
	SILTS AND CLAYS LIQUID LIMIT>50	INORGANIC	PI PLOTS >"A" LINE	CH	FAT CLAY	▨
			PI PLOTS <"A" LINE	MH	ELASTIC SILT	▥
		ORGANIC	LL (oven dried)/LL (not dried)<0.75	OH	ORGANIC CLAY OR SILT	〰
HIGHLY ORGANIC SOILS		PRIMARILY ORGANIC MATTER, DARK IN COLOR, AND ORGANIC ODOR		PT	PEAT	⩛

OTHER MATERIAL SYMBOLS

▨	Poorly-Graded Sand with Clay	⁘	Sand
▨	Clayey Sand	▥	Silt
▥	Sandy Silt	⁘	Well Graded Gravelly Sand
▩	Artificial/Undocumented Fill	○	Gravelly Silt
⁘	Poorly-Graded Gravelly Sand	■	Asphalt
⩛	Topsoil	●	Boulders and Cobble
●	Well-Graded Gravel with Clay		
●	Well-Graded Gravel with Silt		

SAMPLER TYPES

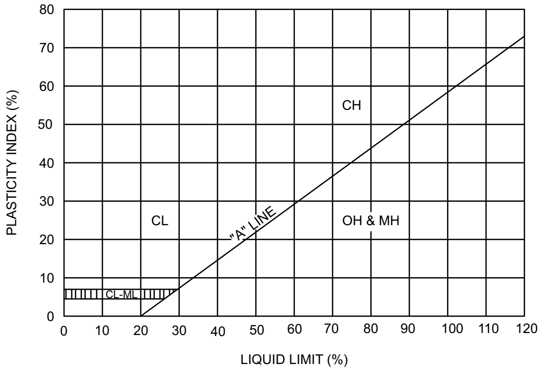
☒	SPT	■	Shelby Tube
⧗	Modified California (2.5" I.D.)	○	No Recovery
▯	Rock Core	✋	Grab Sample

ADDITIONAL TESTS

CA - CHEMICAL ANALYSIS (CORROSIVITY)
CD - CONSOLIDATED DRAINED TRIAXIAL
CN - CONSOLIDATION
CU - CONSOLIDATED UNDRAINED TRIAXIAL
DS - DIRECT SHEAR
PP - POCKET PENETROMETER (TSF)
(3.0) - (WITH SHEAR STRENGTH IN KSF)
RV - R-VALUE
SA - SIEVE ANALYSIS: % PASSING #200 SIEVE
▼ - WATER LEVEL

PI - PLASTICITY INDEX
SW SWELL TEST
TC - CYCLIC TRIAXIAL
TV - TORVANE SHEAR
UC - UNCONFINED COMPRESSION
(1.5) - (WITH SHEAR STRENGTH IN KSF)
UU - UNCONSOLIDATED UNDRAINED TRIAXIAL

PLASTICITY CHART



PENETRATION RESISTANCE
(RECORDED AS BLOWS / FOOT)

SAND & GRAVEL		SILT & CLAY		
RELATIVE DENSITY	BLOWS/FOOT*	CONSISTENCY	BLOWS/FOOT*	STRENGTH** (KSF)
VERY LOOSE	0 - 4	VERY SOFT	0 - 2	0 - 0.25
LOOSE	4 - 10	SOFT	2 - 4	0.25 - 0.5
MEDIUM DENSE	10 - 30	MEDIUM STIFF	4 - 8	0.5-1.0
DENSE	30 - 50	STIFF	8 - 15	1.0 - 2.0
VERY DENSE	OVER 50	VERY STIFF	15 - 30	2.0 - 4.0
		HARD	OVER 30	OVER 4.0

* NUMBER OF BLOWS OF 140 LB HAMMER FALLING 30 INCHES TO DRIVE A 2 INCH O.D. (1-3/8 INCH I.D.) SPLIT-BARREL SAMPLER THE LAST 12 INCHES OF AN 18-INCH DRIVE (ASTM-1586 STANDARD PENETRATION TEST).

** UNDRAINED SHEAR STRENGTH IN KIPS/SQ. FT. AS DETERMINED BY LABORATORY TESTING OR APPROXIMATED BY THE STANDARD PENETRATION TEST, POCKET PENETROMETER, TORVANE, OR VISUAL OBSERVATION.

CORNERSTONE
EARTH GROUP

LEGEND TO SOIL DESCRIPTIONS

Figure Number
A-1

BORING NUMBER EB-1

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME Clayton Community Church II
PROJECT NUMBER 352-2-2
PROJECT LOCATION Clayton, CA

DATE STARTED 2/15/17 **DATE COMPLETED** 2/15/17
DRILLING CONTRACTOR Exploration Geoservices, Inc.
DRILLING METHOD Mobile B-40, 8 inch Hollow-Stem Auger
LOGGED BY OL
NOTES

GROUND ELEVATION **BORING DEPTH** 10 ft.
LATITUDE 37.9402970° **LONGITUDE** -121.9386090°
GROUND WATER LEVELS:
▽ **AT TIME OF DRILLING** Not Encountered
▼ **AT END OF DRILLING** Not Encountered

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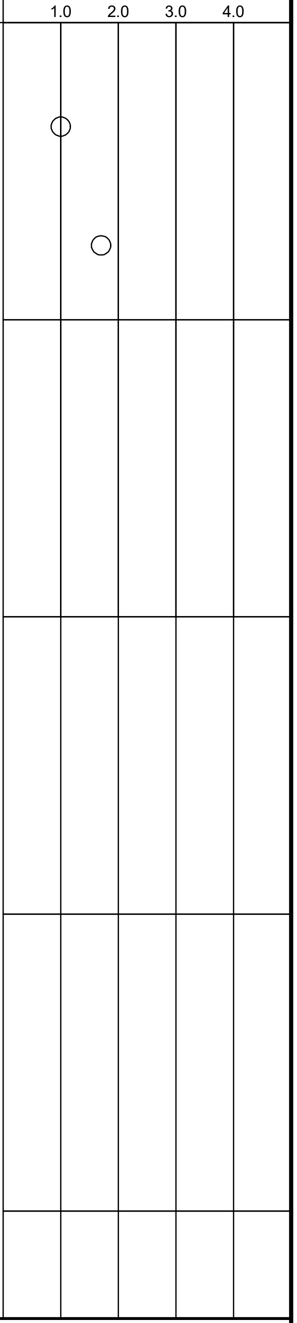
ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
	0	▨	**Lean Clay with Sand (CL)** soft to medium stiff, moist, reddish brown, fine to coarse sand, low plasticity Liquid Limit = 27, Plastic Limit = 17	6	MC-1B	106	15	10	
		▨		6	MC-2	108	14		
	5	▨	**Clayey Sand with Gravel (GC)** loose, moist, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel	13	MC-3B	116	12		
		▨	becomes medium dense	38	MC-4B	118	12		
	10		Bottom of Boring at 10.0 feet.						
	15								
	20								

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL



BORING NUMBER EB-2

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME Clayton Community Church II
PROJECT NUMBER 352-2-2
PROJECT LOCATION Clayton, CA
DATE STARTED 2/15/17 **DATE COMPLETED** 2/15/17
GROUND ELEVATION **BORING DEPTH** 5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.9400579° **LONGITUDE** -121.9385893°
DRILLING METHOD Mobile B-40, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY OL
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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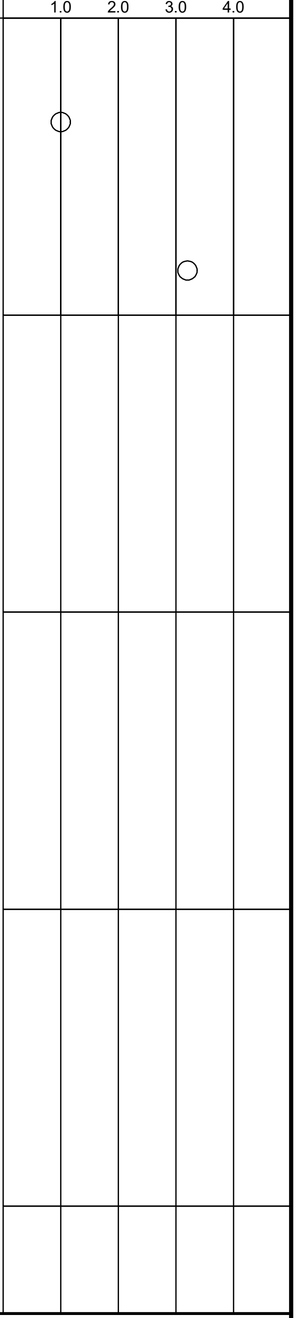
ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
	0	▨	**Lean Clay with Sand (CL)** medium stiff, moist, reddish brown, fine to coarse sand, low plasticity							
		▨		7	MC-1B		17			○
		▨	**Sandy Lean Clay (CL)** very stiff, moist, reddish brown, fine to coarse sand, some fine to coarse subangular to angular gravel, low plasticity							
		▨		21	MC-2B		15			○
	5		Bottom of Boring at 5.0 feet.							
	10									
	15									
	20									

UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL



BORING NUMBER EB-3

PAGE 1 OF 1

PROJECT NAME Clayton Community Church II

PROJECT NUMBER 352-2-2

PROJECT LOCATION Clayton, CA

DATE STARTED 2/15/17 **DATE COMPLETED** 2/15/17

GROUND ELEVATION ____ **BORING DEPTH** 8 ft.

DRILLING CONTRACTOR CK Construction, Inc.

LATITUDE 37.9404932° **LONGITUDE** -121.9382801°

DRILLING METHOD 4 inch Hand Auger

GROUND WATER LEVELS:

LOGGED BY OL

▽ **AT TIME OF DRILLING** Not Encountered

NOTES ____

▼ **AT END OF DRILLING** Not Encountered

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UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL

ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf 1.0 2.0 3.0 4.0
	0	▨	**Lean Clay with Sand (CL)** soft to medium stiff, moist, reddish brown, fine to coarse sand, low plasticity Liquid Limit = 26, Plastic Limit = 17		⊠ MC-1	113	13	9		○
		▨	**Sandy Lean Clay (CL)** hard, moist, reddish brown, fine to coarse sand, some fine to coarse subangular to angular gravel, low plasticity		⊠ MC-2	112	15			>4.5 ○
	5	▨	**Clayey Sand with Gravel (GC)** moist, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel		⊠ MC-3	123	14			
		▨	**Sandy Lean Clay (CL)** hard, moist, reddish brown, fine to coarse sand, some fine to coarse subangular to angular gravel, low plasticity		⊠ MC-4	103	18			>4.5 ○
			Bottom of Boring at 8.0 feet.							
	10									
	15									
	20									

BORING NUMBER EB-4

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME Clayton Community Church II
PROJECT NUMBER 352-2-2
PROJECT LOCATION Clayton, CA
DATE STARTED 2/23/17 **DATE COMPLETED** 2/23/17
GROUND ELEVATION **BORING DEPTH** 6 ft.
DRILLING CONTRACTOR CK Construction, Inc.
LATITUDE 37.9403130° **LONGITUDE** -121.9379720°
DRILLING METHOD 4 inch Hand Auger
GROUND WATER LEVELS:
LOGGED BY OL
▽ **AT TIME OF DRILLING** 3 ft.
NOTES Encountered perched water at 3'
▼ **AT END OF DRILLING** Not Encountered

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UNDRAINED SHEAR STRENGTH, ksf
○ HAND PENETROMETER
△ TORVANE
● UNCONFINED COMPRESSION
▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL
1.0 2.0 3.0 4.0

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION	N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE	UNDRAINED SHEAR STRENGTH, ksf
	0	**Lean Clay with Sand (CL)** stiff, moist, dark brown to reddish brown, fine to coarse sand, low plasticity		MC-1	106	18			○
▽		**Sandy Lean Clay (CL)** very stiff, moist, reddish brown, fine to coarse sand, some fine to coarse subangular to angular gravel, low plasticity		MC					
				MC-3	99	27			○
	5	**Lean Clay (CL)** very stiff, moist, reddish brown, some fine sand, moderate plasticity							
		Clayey Sand with Gravel (GC) moist, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel		MC					
		Bottom of Boring at 6.0 feet.							
	10								
	15								
	20								

APPENDIX B: LABORATORY TEST PROGRAM

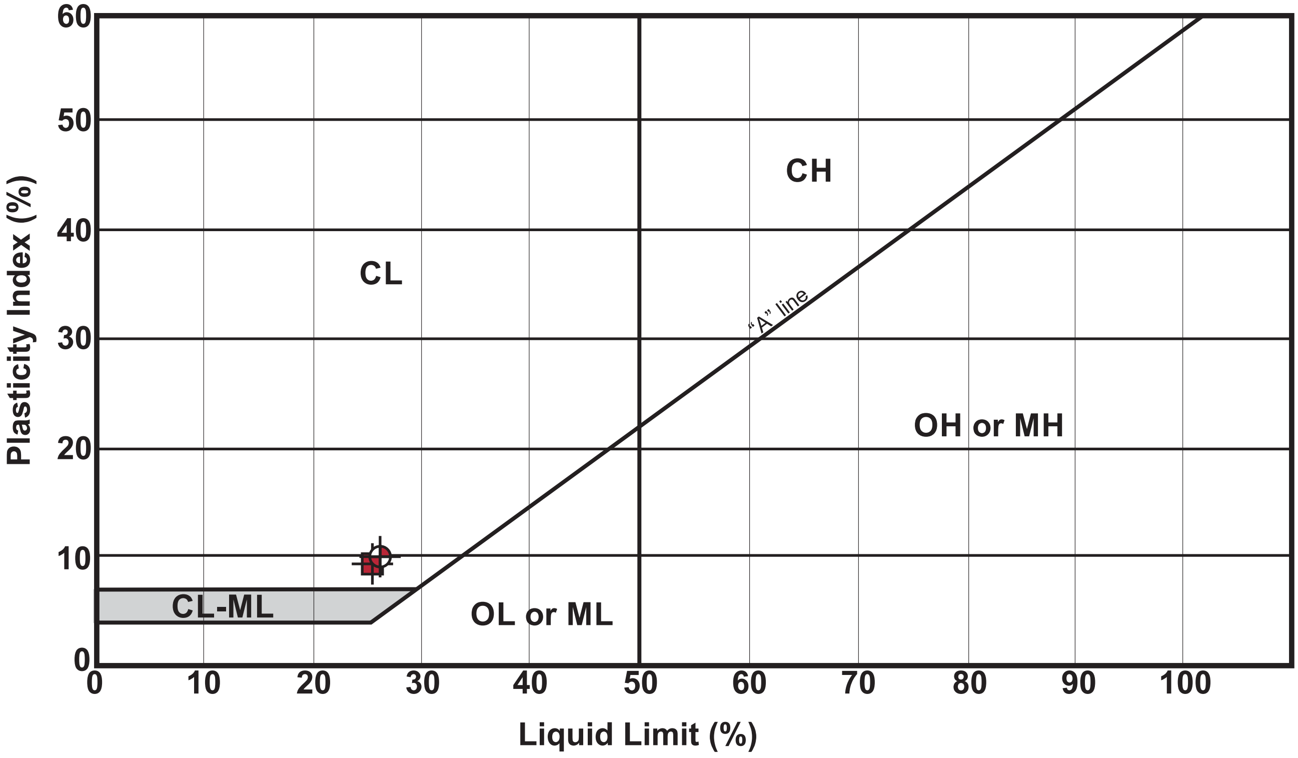
The laboratory testing program was performed to evaluate the physical and mechanical properties of the soils retrieved from the site to aid in verifying soil classification.

Moisture Content: The natural water content was determined (ASTM D2216) on 12 samples of the materials recovered from the borings. These water contents are recorded on the boring logs at the appropriate sample depths.

Dry Densities: In place dry density determinations (ASTM D2937) were performed on 10 samples to measure the unit weight of the subsurface soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Plasticity Index: Two Plasticity Index determinations (ASTM D4318) were performed on samples of the subsurface soils to measure the range of water contents over which this material exhibits plasticity. The Plasticity Index was used to classify the soil in accordance with the Unified Soil Classification System and to evaluate the soil expansion potential. Results of these tests are shown on the boring logs at the appropriate sample depths.

Plasticity Index (ASTM D4318) Testing Summary



Symbol	Boring No.	Depth (ft)	Natural Water Content (%)	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index	Passing No. 200 (%)	Group Name (USCS - ASTM D2487)
⊕	EB-1	1.5	15	27	17	10	—	Lean Clay with Sand (CL)
⊞	EB-3	1.5	13	26	17	9	—	Lean Clay with Sand (CL)

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EARTH GROUP

Plasticity Index Testing Summary

Clayton Community Church II
Clayton, CA

Project Number	352-2-2
Figure Number	Figure B1
Date	March 2017
Drawn By	FLL