APPENDIX E:
GEOTECHNICAL DATA

CORNERSTONE
EARTH GROUP

TYPE OF SERVICES Geotechnical and Geologic Hazard Investigation

PROJECT NAME The Forum Senior Community Update

LOCATION 23500 Cristo Rey Drive
Cupertino, California

CLIENT The Forum at Rancho San Antonio

PROJECT NUMBER 905-1-1

DATE April 14, 2017

CORNERSTONE
EARTH GROUP

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TABLE OF CONTENTS

Type of Services	**Geotechnical and Geologic Hazard Investigation**
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SECTION 1: INTRODUCTION

This geologic and geotechnical report was prepared for the sole use of Greenbrier Development, LLC and The Forum at Rancho San Antonio for the proposed Forum Senior Community Update project in Cupertino, California. The approximate location of the site is at Latitude 37.337595°, Longitude -122.087486° (WGS 84) and as shown on the Vicinity Map, Figure 1. This report is for the new memory care building, skilled nursing facility addition, multi-purpose building, villas and duplexes, and an addition to the existing fitness building. For our use, we were provided with the following documents:

- A topographic survey titled, "The Forum at Rancho San Antonio, Topographic Survey," prepared by BKF Engineers/Surveyors/Planners, dated June 7, 2016.
- A set of plans indicating the overall site and construction and renovation scope, prepared by Smith Group JJR.
- A seismic study titled, "Seismic Risk Study of the Apartment Buildings at The Forum at Rancho San Antonio, Cupertino, California," prepared by ABS Consulting, dated September, 2007.
- A set of existing utility plans titled, "Underground Utility Plan for The Forum at Rancho San Antonio," prepared by Brian Kangas Foulk, dated March 1, 1990.

1.1 PROJECT DESCRIPTION

Based on our understanding, the project will include construction of new residential units, memory care, skilled nursing facility and multi-purpose facilities as well as expansion and remodeling of other facilities, as summarized below:

Phase 1A:

- Construction of up to 25 villas, with single units approximately 2,250 square feet each, and duplex units approximately 4,490 to 5,060 square feet each. A new street will be constructed to accommodate circulation at the villas near the south end of the property.
- Renovations to the dining and kitchen spaces at the upper level of the community/commons building. Emergency services addition at the lower level of the commons/clubhouse building.
- Expansion of the fitness facilities at the swimming pool area. Finished floor to be at approximately Elevation 361.83 feet.
- Construction of a memory care building with ground-level parking. Finished floor for ground floor level to be at approximately Elevation 349 feet.

Phase 1B:

- New construction and renovation of The Forum's Skilled Nursing Facility and Rehabilitation Center. The proposed addition has a ground area footprint of approximately 21,101 square feet, plus the addition of a new fuel tank for the emergency generator. Finished floor for the new Skilled Nursing Addition to be at approximately Elevation 352.1 feet.
- Renovation of existing Assisted Living common areas.

Phase 2:

- Phase 2 includes new construction of a two-story multi-purpose building, finished floor to be at approximately Elevation 361 feet. Renovation work will be performed in the administrative space at the lower level of the commons/clubhouse building.

The one-story Skilled Nursing Facility, to be reviewed by the Office of Statewide Health Planning and Development (OSHPD), will likely be of wood-frame construction with a concrete slab-on-grade floor. Residential and one-story additions will be of wood-frame or steel-frame construction with concrete slab-on-grade floors. The Memory Care will likely be of wood-framed and/or concrete construction. Appurtenant parking and fire lane relocation, streets, new underground utilities, landscaping and other improvements necessary for new construction are also planned.

Preliminary building loads were provided by the structural engineer, Forrell/Elsesser Engineers, Inc. Based on review of the preliminary grading plans prepared by BKF, cuts and fills are anticipated to be as follows:

- ✓ Villas 61 & 62 – fills ranging from about 2 to 6 feet thick
- ✓ Villas 63 & 64 – cuts up to 6 feet and fills ranging from about 2 to 6 feet thick
- ✓ Villa 65 – cuts and fills ranging from about 2 to 3 feet
- ✓ Villas 66 through 85 – cuts ranging from about 2 to 8 feet and fills ranging from about 1 to 4 feet

- ✓ Memory Care Facility - cuts ranging up to 16 feet and fills ranging from about 1 to 7 feet
- ✓ Skilled Nursing Facility - fills of about 1 feet across most of the pad, except the western end of the pad, where up to 7 feet of fill is planned.

1.2 SCOPE OF SERVICES

Our scope of services was presented in our proposals dated May 25, 2016, and March 6, 2017, 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.

For the geologic hazard study, our scope of services included the following scope of work:

- Research and review of technical documents (including published maps, previous geologic and geotechnical reports, etc.).
- Review of historical aerial photography and online aerial imagery to help identify potential geologic hazards at the site.
- Geologic field reconnaissance to observe topographic and geologic conditions.
- Identification and evaluation of potential geologic hazards.
- Preparation of accompanying graphics to summarize the findings of our geologic hazards investigation.

1.3 PREVIOUS STUDIES

The Forum complex was constructed in 1991-1992 following geologic and geotechnical investigations by Earth Science Associates (ESA, 1979, 1983, 1985, 1986, 1988) and peer reviewed by the City’s geotechnical consultant William Cotton and Associates (WCA, 1985, 1986). In addition, a geologic and geotechnical investigation was performed for the adjacent St. Joseph Seminary property by Earth Systems Consultants (ESC, 1991), which was also peer reviewed by WCA (1993). The ESA and ESC investigations include geologic maps and logs of numerous borings, test pits and trenches on and in the vicinity of the Forum property, which form a robust geologic database used for our current evaluation of geologic hazards that potentially impact the new project. A list of the consultant reports reviewed for this expansion project include the following:

- A report titled, “Geologic and Geotechnical Investigation of the St. Joseph Water Tank, Cupertino, California,” prepared by Earth Sciences Associates, dated February, 1989.

- A letter titled, “Revised Flexible Roadway Pavement Section, Forum Life Continuing Care Center, Cupertino, California,” prepared by Earth Sciences Associates, dated April 6, 1989.

- A report titled, "Geotechnical Investigation for The Forum Life Continuing Care Center, Cupertino, California," prepared by Earth Sciences Associates, dated September, 1988.
- A supplemental report titled, "El Camino Hospital Continuing Care Center, Supplemental Report to Phase I: Geologic Hazards Site Investigation," prepared by Earth Sciences Associates, dated February, 1986.
- A report titled, "El Camino Hospital Continuing Care Center, Phase I: Geologic Hazards Site Investigation," prepared by Earth Sciences Associates, dated June, 1985.
- A report titled, "Geologic and Preliminary Geotechnical Study, St. Joseph Seminary Property, Cupertino, California," prepared by Earth Systems Consultants, dated November 1991.

1.4 EXPLORATION PROGRAM

Recent field exploration consisted of 23 borings drilled on July 11 through 14, 2016, and March 27, 2017, with truck-mounted and track-mounted hollow-stem auger drilling equipment. The borings were drilled to depths ranging from about 10 to 50 feet. The borings were backfilled with cement grout in accordance with local requirements; exploration permits were obtained as required by local jurisdictions.

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

1.5 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, grain size analyses, washed sieve analyses, Plasticity Index tests, triaxial compression tests. Details regarding our laboratory program are included in Appendix B.

1.6 CORROSION EVALUATION

Six samples from our borings from depths from 2 to 9 feet were tested for saturated resistivity, pH, and soluble sulfates and chlorides. JDH Corrosion Consultants prepared a brief corrosion evaluation based on the laboratory data, which is attached to this report in Appendix C. In general, the on-site soils can be characterized as corrosive to buried metal, and noncorrosive to buried concrete.

1.7 ENVIRONMENTAL SERVICES

Cornerstone Earth Group also provided environmental services for this project, including a Phase 1 site assessments; environmental findings and conclusions are provided under separate covers.

SECTION 2: REGIONAL SETTING

2.1 REGIONAL GEOLOGY

The project site is located in the Coast Ranges geomorphic province, which is characterized by generally northwest-trending, elongate mountain ranges from 600 to 1300 meters above sea level (2,000 to 4,000 feet) separated by narrow valleys. The Forum complex is located in the foothill terrain between the northeastern edge of the steep, rugged Santa Cruz Mountains and the gentle Santa Clara Valley alluvial plain. Permanente Creek, one the major drainages emanating from the eastern slopes of the Santa Cruz Mountains, flows generally northwestward along the base of the mountain front, approximately 2 miles southwest from the project area.

The site vicinity is underlain, at depth, by the Jurassic- and Cretaceous-age Franciscan Complex, consisting of greywacke sandstone, greenstone, chert, limestone and serpentinite. The Franciscan rocks are overlain by folded and faulted Tertiary-age sedimentary rocks which include the Monterey and Santa Clara Formations in the site vicinity (Figure 3). Locally, Quaternary-age stream terrace deposits overlie the bedrock formations. Landslides are present in the steep mountain area, but are not present on the property.

Stream alluvium that was deposited by older stream channels are present along the general trend of Permanente Creek. In general, the terrace deposits form a thin, discontinuous veneer over Santa Clara Formation materials. The terrace deposits consist of unconsolidated sand, varying from silty and fine-grained to locally coarse-grained, loose to medium dense, well sorted (poorly graded).

2.2 REGIONAL SEISMICITY

The San Andreas fault is the dominant structural feature within the Coast Ranges and is often observed as a long, narrow and linear valley associated with the active trace of the San Andreas fault zone. The San Andreas fault system is a fundamental geologic boundary between two of the earth’s tectonic plates. The fault system follows a northwest-trending path through most of California, arising in the south from a set of transform faults in the Gulf of California and joining, to the north, the Mendocino Fracture Zone offshore of the northern part of the state. In central California, the fault separates two major structural blocks: the Salinian block of granitic and metamorphic rocks on the southwest, and the Franciscan Complex and overlying strata of the Great Valley Sequence on the northeast.

The San Francisco Bay region is within a zone of distributed active deformation associated with the North America-Pacific plate boundary. The plate boundary zone has had a complex history that has involved over time plate subduction, and crustal extension and contraction in association with dextral (right-lateral) strike-slip movements along faults within the boundary zone. The present-day seismotectonic setting of the region is marked by high rates of earthquake occurrence, right-lateral shear deformation along the San Andreas fault system, and components of contractional strain both oblique and normal to the San Andreas.

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 (UCERF3), 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.

2.3 MAJOR FAULTS IN PROJECT VICINITY

The San Francisco Bay region is characterized by active, potentially active and inactive faults with a historical record of large and damaging earthquakes. Active faults of the San Andreas fault system, including the San Andreas, Hayward, Rodgers Creek, and Calaveras faults, could produce significant earthquakes during the life of the project. In addition, potentially active compressional features, including thrust faults along the foothills of the Santa Cruz Mountains may impact the project.

Figure 4 shows the location of significant faults in the vicinity of the Project. The following table summarizes source parameters for each of the significant faults. These parameters are based on the Working Group for California Earthquake Probabilities (WGCEP, 1999, 2003 and 2008), and CGS (2003).

Table 1: Source Parameters for Major Faults In Vicinity of the Site

Fault/Fault Segment	Sense of Movement[1]	Length (km)	± (km)	Slip Rate (mm/yr)	M_{max} (M)[2]	Fault Type	Closest Distance to Project Site (km)
San Andreas (1906)	(rl-ss)	470	47	24.0±3.0	8.0	A	5.6
San Andreas (North Coast)	(rl-ss)	322	32	24.0±3.0	7.6	A	52.1
San Andreas (Peninsula)	(rl-ss)	88	9	17.0±3.0	7.1	A	6.4
San Andreas (Santa Cruz Mountains)	(rl-ss)	59	6	14.0±3.0	7.0	A	5.6
Hayward (N)	(rl-ss)	43	4	9.0±1.0	6.9	A	30.0

Hayward (S)	(rl-ss)	43	11	9.0±1.0	6.9	A	10.1
Hayward (total)	(rl-ss)	86	9	9.0±1.0	7.1	A	10.1
Rodgers Creek	(rl-ss)	63	6	9.0±2.0	7.0	A	56.8
San Gregorio (N)	(rl-ss)	130	13	5.0±2.0	7.3	B	20.1
Calaveras (N)	(rl-ss)	52	5	6.0±2.0	6.8	B	14.2
Calaveras (S)	(rl-ss)	106	11	15.0±2.0	6.2	B	27.2
Monte Vista-Shannon	(reverse-oblique	41	4	0.4±0.3	6.8	B	0.4

Notes:

1. rl-ss = right-lateral strike-slip. r = reverse.
2. M_{max} is the maximum magnitude calculated for coseismic slip on the fault in Moment Magnitude (**M**). Sources: CGS, 2003; WGCEP, 2008.

The regional fault map is presented as Figure 4, illustrating the relative distances of the site to significant fault zones. The most significant active and potentially active (Late Quaternary and Holocene) faults in proximity of the project are described below:

2.3.1 San Andreas Fault

The 1,100-km-long (690 miles) San Andreas fault zone, extending from the Gulf of California, Mexico, to Point Delgada on the Mendocino Coast in northern California, is the principal element of the San Andreas fault system. It is a network of faults with predominantly dextral (right-lateral) strike-slip displacement that collectively accommodates the majority of relative motion between the North American and Pacific plates. The San Andreas fault is the largest active fault in California. Movement on the San Andreas fault is right-lateral strike-slip, with an estimated total offset of some 560 km (350 miles). Based on differences in geomorphic expression, fault geometry, paleoseismic chronology, slip rate, seismicity, and historic fault ruptures, the San Andreas fault can be divided into a number of fault segments (WGCEP, 2013; CGS, 2003). Each of those segments is capable of rupturing either independently or in conjunction with adjacent segments. In the San Francisco Bay area, the San Andreas fault is defined by the North Coast, Peninsula, and Santa Cruz Mountains segments, all of which ruptured in the 1906 San Francisco earthquake. Significant aspects of these three segments are described below:

2.3.1.1 North Coast Segment - The North Coast segment extends 178 km (111 miles) from Point Arena southeast to the Golden Gate (WGCEP, 2003). The southern boundary with the Peninsula segment is characterized by a 3-km-wide, right releasing bend, splaying off of the San Gregorio fault to the south, a reduced slip rate to the south, and a drop in geodetically modeled slip associated with the 1906 earthquake.

Correlation of several studies indicates that the most recent paleoevent may have occurred between 1600 and 1650, although wider earthquake windows have been suggested. Recurrence intervals for the past 2,000 years range from 200 to 400 years. Reported Holocene slip rates range from a minimum of 16-18 mm/year to 25-26 mm/year. CGS (2003) combines this segment with the Shelter Cove segment to the north, resulting in a total fault segment length of 322 km (201 miles), and assign an average slip rate of 24 ± 3 mm/year, and a calculated maximum Moment Magnitude of 7.6 (Table 1).

2.3.1.2 Peninsula Segment - The Peninsula segment extends approximately 88 km (55 miles) from offshore the Golden Gate southeast to the vicinity of Black Mountain near Los Altos Hills, and extends to within approximately 7 km (4.4 miles) of the Stone Complex project site. The Peninsula segment is delineated by well-defined geomorphic features characteristic of right-lateral strike-slip faulting, including deflected drainages, linear drainages, side hill benches, closed depressions (sag ponds), aligned benches, linear scarps, linear troughs and ridges, aligned saddles, and linear vegetation contrasts.

The most recent surface fault event in the Peninsula segment is the 1906 San Francisco earthquake, and the most recent paleoevent may be an earthquake that occurred in 1838, although direct evidence has not yet been determined. The reported dextral offset associated with the 1906 earthquake at the Filoli investigation site (Hall et al., 1999) site was 2.5 meters, with approximately 1.6 meters of offset representing a penultimate (possibly 1838) event. The timing of earlier events for the Peninsula segment has not been determined. The most recent paleoseismic studies of the Peninsula segment indicate that the Holocene slip rate is on the order of 17 to 19 mm/year (Hall et al., 1999). CGS (2003) assigns an average slip rate of 17 ± 3 mm/year along this segment, and a calculated maximum Moment Magnitude of 7.1 (Table 1).

2.3.1.3 Santa Cruz Mountains Segment - The Santa Cruz Mountains segment extends approximately 59 km (37 miles) from the vicinity of Black Mountain in the Santa Cruz Mountains southeast to just south of San Juan Bautista. The northern boundary of this segment is marked by an approximately 1-km-wide (3280 feet), left-contractional bend near Black Mountain. The southern boundary and transition to the adjoining Creeping segment to the south is taken as the approximate southern termination of surface fault rupture associated with the 1906 San Francisco earthquake (Lawson, 1908).

The most recent paleoearthquake along the Santa Cruz Mountains segment may have occurred in the mid-1600s, based on interpretation of trench excavations by Schwartz et al. (1998) who reported that 247 to 266 years elapsed between the 1906 earthquake and the most recent paleoevent in the mid-1600s. Fumal et al. (1999) identified as many as seven large earthquakes since 1100 A. D. CGS (2003) separates this segment into two segments (i.e., “Santa Cruz Mountains” and “Pajaro” segments), and assign an average slip rate of 14 ± 3 mm/year to both, with a calculated maximum Moment Magnitude of 7.0 (Table 1).

2.3.2 Hayward Fault

The Hayward fault is a right-lateral strike-slip fault, and one of the major structures of the San Andreas fault system in the San Francisco Bay region. The Hayward fault extends from San

Pablo Bay southeastward to at least Warm Springs in Fremont, for a distance of 69 km (43 miles). Recent studies indicate that the fault likely extends for another 17 km (11 miles), with a southern terminus near Milpitas and a total fault length of 86 km (54 miles). Further to the southeast, the fault appears to transfer slip, partly in a series of step-over structures, to the Calaveras fault.

The Hayward fault has been subdivided into two segments by the WGCEP (2003) based on the rupture associated with the most recent large earthquake (in 1868). Various estimates indicate that the 1868 event (M 6.9) ruptured the southern segment for a distance ranging from 30 to 54 km (19 to 34 miles). The boundary between the northern and southern segments is poorly constrained due to uncertainty in the northern extent of the 1868 rupture. The current segment boundary is a point between Montclair and Mills College (Oakland) with an uncertainty of ±6 miles (WGCEP, 2003).

The average recurrence interval for large earthquakes on the southern segment of the Hayward fault is between 141 to 199 years (mean + 2 sigma deviation). The timing of the penultimate event on the northern segment of the Hayward fault is not well constrained, but there have been at least 4 to 7 surface faulting events in the past 1627 radiocarbon years (WGCEP, 1999), indicating a recurrence interval of less than 270 to a maximum of 710 years over the past 2,000 years. CGS (2003) assigns an average slip rate of 9 ± 1 mm/year to both segments, with a calculated maximum Moment Magnitude of 7.1 for the entire fault (Table 1).

The Hayward fault is creeping, although the rate of creep varies locally along the fault. The average creep rate is 3.8 to 5.1 mm/year, with a maximum of about 9 mm/year near the southern end in Fremont (Lienkaemper and Galehouse, 1997). The hills to the east of the fault are rising at a rate of 1.5 mm/year, indicating a reverse slip component on the fault.

2.3.3 Monte Vista Fault

The Monte Vista fault is one of the primary range-front faults that mark the boundary between the southern Santa Cruz Mountains and the western margin of Santa Clara Valley (Sorg and McLaughlin, 1975). In the project vicinity, the Monte Vista fault is defined by two major traces located along the prominent northeast-facing escarpment between the towns of Los Altos Hills and Cupertino. The fault zone is approximately 1,200 feet southwest of the property.

Based on results of exploratory trenching, the Monte Vista fault has had late Quaternary and possibly Holocene displacement. Directly downstream of the range front, Permanente Creek parallels the Monte Vista fault and is bordered on the southwest by a series of prominent linear fronts and faceted ridge spurs. Exploratory trenching across the primary fault trace at the base of these facets exposed colluvial deposits thrust over fluvial gravel deposited by Permanente Creek (ESC, 1991 Evidence of recent faulting was observed in ESC's trenches T-1A, T-1B, T-2 and T-4, where steps in soil and colluvial units overlying sheared bedrock suggests that fault movement has occurred within Holocene time. The main shear zone in trenches T-2 and T-4 was observed within the Monterey Formation at the base of mountain front. Although the predominant sense of offset was reverse-slip, with the west block up relative to the east block, strike-slip displacement also was suspected based on the steep dips of the faults and associated linear geomorphology.

CGS (2003) assigns an average slip rate of 0.4 ± 0.3 mm/year to the Monte Vista-Shannon fault zone, with a calculated maximum Moment Magnitude of 6.8.

2.3.4 Foothills Thrust Belt (FTB)

A northwest-trending zone of uplifted sedimentary deposits and surfaces occurs along the southwestern margin of the Santa Clara Valley, along the foothills of the Santa Cruz Mountains and northeast of the San Andreas fault. The broad region of uplift is underlain by southwest-dipping reverse and oblique-slip faults collectively referred to as the Foothills thrust belt (Fenton and Hitchcock, 2001), or the Santa Clara Valley range front thrust system (McLaughlin el al., 2000). The fault system includes, from south to north, the Sargent, Sierra Azul, Berrocal, Monte Vista, Shannon, and Stanford faults. According to the most recent State fault map, the Sargent and Monte Vista faults are considered Holocene active, and the remainder of the thrust faults are considered to be Quaternary active (Jennings and Bryant, 2010).

Microseismicity indicates a continuous zone of fault-normal compression slip. However, triggered slip during large magnitude events on the San Andreas fault zone (such as occurred during the 1989 Loma Prieta earthquake), may account for some or all of the fault-normal contraction, and thus preclude the need for independent large magnitude events on the structures (Hitchcock and Kelson, 1999). Nonetheless, studies associated with the WGCEP (1999) estimated probable maximum earthquake magnitudes of M 6.25 to 6.75, and no larger than M 7.0 for the fault system. Estimated surface lengths of the faults in FTB are: 53 km (33 miles) for the Sargent-Berrocal, 30 to 54 km (19 to 34 miles) for the Monte Vista, 48 to 54 (30 to 40 miles) for the Shannon, and 18 km (11 miles) for the Stanford fault.

The 1989 Loma Prieta earthquake produced coseismic contractional deformation in several northwest-trending, elongate zones along the northeastern flank of the Santa Cruz Mountains in the southern San Francisco Bay region. The deformation occurred along, or subparallel to, the previously mapped Monte Vista, Shannon and Berrocal faults. Possible localized activation of these faults during the 1989 Loma Prieta earthquake suggests that they may be related at depth to the San Andreas fault. It is not known whether displacement on these faults occurs only as secondary coseismic deformation resulting from movement along the San Andreas fault during large-magnitude events, or if these faults are themselves potential seismic sources.

2.4 HISTORIC SEISMICITY

The project site is located in the seismically active region of the San Francisco Bay Area. The following table provides information of the dates and locations of earthquakes with reported magnitudes of 6 or greater within 100 km (63 miles) of the project site through August 26, 2016. The pre-1900 earthquakes are major regional events reported in historical records.

Table 2: Historical Earthquakes (M ≥ 6) Within 100 km of the Site

Year	Month	Day[1]	Latitude (°N)	Longitude (°W)	Magnitude[2]	Depth (km)	Radial Distance from site (km)
1808	06	21	37.80	122.50	6.0	-	58
1836	06	10	37.80	122.20	6.8	-	47
1838	06		37.60	122.40	7.0	-	36
1858	11	26	37.50	121.90	6.1	-	21
1865	10	08	37.30	121.90	6.5	-	19
1868	10	21	37.70	122.10	6.8	-	35
1889	05	19	38.00	121.90	6.0	-	70
1890	4	24	36.90	121.60	6.0	-	69
1897	06	20	37.00	121.50	6.2	-	68
1898	03	31	38.20	122.40	6.2	-	94
1906	04	18	37.70	122.50	8.25	-	50
1911	07	11	37.25	121.75	6.6	-	34
1926	10	22	36.61	122.35	6.1	10	90
1926	10	22	36.55	122.183	6.1	-	94
1984	04	24	37.320	121.698	6.2	8	35
1989	10	18	37.036	121.883	6.9	19	43
2014	08	24	38.215	122.312	6.0	11	94

1. Time is universal time.
2. Earthquake magnitudes are as reported in the CDMG, USHIS, or Berkeley Seismological Laboratory catalogs except for 1984 and 1989 where magnitudes are from the PDE catalog. Magnitudes are either local or Richter magnitudes (M_L) and moment magnitudes (**M**). Local magnitudes are determined from the amplitude (in mm) of the maximum wave recorded on a seismogram. Moment magnitude is the magnitude of an earthquake that is proportional to the slip on the fault times the area of the fault surface that slips. **M** is related to the total energy released in the earthquake and can be estimated from seismograms.

SECTION 3: SITE CONDITIONS

3.1 SITE BACKGROUND

Based on our understanding, the current structures at the approximately 51½-acre site are up to 26 years old and consist of 259 independent-living apartment units, 60 villas, and a separate healthcare building with 40 assisted-living suites, 18 memory care units, and skilled nursing facility with 30 rooms. Several geotechnical and geologic hazard investigations were performed at the site between 1979 and 1986 before a design-level geotechnical investigation was performed for the existing facility in 1988. Historic aerial photographs of the area, extending as far back as 1948, show that the area was used primarily for agricultural purposes up to the mid-1960s.

By 1968, Interstate 280 had been constructed, as well as single-family homes on the northeast side of the highway. The site was cleared of agricultural activity by the mid-1980s, at which time the previous investigations were being performed. A photograph from 1990 shows the site under construction, and another from 1993 shows the completed site.

The former Saint Joseph Seminary was located southwest of the site and had existed since 1924. However, the structure was greatly damaged during the Loma Prieta earthquake in 1989. It was demolished shortly after the earthquake, during the same time the Forum at Rancho San Antonio was being built, and by 2002 single-family homes were built at the former seminary location.

3.2 ORIGINAL SITE CONSTRUCTION

Based on our review of the “Final Grading Report” prepared by Smith-Emery Company dated July 17, 1990, and our understanding of the site conditions prior to original development, the site underwent significant grading to prepare the area for construction. The original drainage swale that traversed the site from south to north was cleared and subsequently backfilled with on-site soil materials generated from numerous cuts. A low ridge that bounded the western edge of the site was cut approximately 8 to 17 feet, and shallow to moderate cuts and fills east of the drainage swale were made to create terraced building pads and new streets. In general, engineered fill placed during construction was reportedly compacted to at least 92 percent relative compaction. Original buildings were reported to be supported on drilled, cast-in-place piers; the size and depths of the piers was not discussed in the 1990 summary report.

3.3 SITE RECONNAISSANCE AND SURFACE DESCRIPTION

Our Certified Engineering Geologist performed a reconnaissance of the site on August 10, 2016. The pre-development property was characterized by two smooth-sided, northwest-trending ridges and an intervening northwest-flowing ephemeral drainage swale. The northern ridge was higher and steeper than the more subdued ridge south of the swale. The natural hillside gradients varied between 5 to 20 degrees (9 to 33 percent). Most of the complex is situated over the southwest-facing slopes of the higher ridge between Interstate Highway 280 (to the northeast) and the central drainage swale, where slope gradients typically were between 10 to 20 degrees (approximately 15 to 35 percent). A smaller portion of the complex is situated over the northwestern part of the southern ridge, where natural slope gradients varied from less than 5 to 10 degrees (less than 8 to 15 percent).

The existing development includes one- to four-story buildings that appear to be generally of wood-frame and structural steel construction. However, the apartment structures appear to be of wood-frame construction over concrete parking garage podiums. The buildings are surrounded by landscaping and at-grade asphalt concrete drive aisles and parking lots.

The site elevations for this project vary from about Elevation 332 feet in the northwest to Elevation 440 feet in the southeast (NGVD 29). Several gentle to moderately slopes can be found throughout the site, including in the undeveloped parcel on the southwest end of the site. Further discussion of the proposed development areas are presented in the following sections.

3.3.1 Memory Care Facility

The new memory care building will be located on the north end of the site (Borings EB-1 through EB-4). Currently there is an at-grade parking lot and an undeveloped slope covered in tall grasses and weeds, as well as several mature trees. A dirt pedestrian path is located on the slope. The terrain slopes down from the east to west at an inclination of about 3:1 to 5:1 (horizontal:vertical).

Site grades in the proposed memory care building area appear to range from approximately Elevation 370 to 350 feet. Based on a comparison of original and recent topographic information, it appears little to no grading occurred within the proposed Memory Care building footprint during original site development. Fill was likely placed to the east of the proposed building in the former drainage ravine.

3.3.2 Multi-Purpose Building and Fitness Building Addition

The new multi-purpose building will be located near the center of the site at the intersection of Cristo Rey Drive and Stonehaven Drive, close to the main building (Borings EB-9, EB-10), and the fitness building addition is about 100 feet south of the intersection (Boring EB-22). Both of the areas are covered in landscaping with tall trees and are relatively level. However, a flag pole, and a small patio with walkways in the center of it are also in the area of the new multi-purpose building.

Site grades in the proposed multi-purpose building area appear to range from approximately Elevation 360 to 362 feet. Based on a comparison of original and recent topographic information, it appears up to roughly 12 feet of fill was placed within the proposed multi-purpose building footprint during original site development. Roughly 2 feet of fill was likely placed near the fitness center building.

3.3.3 New Villas and Duplexes

New single villas and villa duplexes are proposed in the areas of Borings EB-11, EB-12, and EB-13. The area of EB-11 is mostly covered in mulch for landscaping. There are also tall trees and a dirt path, and gently slopes down from east to west. The area of EB-12 is used for landscaping as well, is surrounded by asphalt concrete on the northeast, northwest, and southwest sides, and also gently slopes down from east to west. Surface pavements generally consisted of 2 inches of asphalt concrete over 6 inches of aggregate base. Based on visual observations, the existing pavements are in moderately good shape.

At EB-13, the relatively level area is landscaped and has a dirt path running through it.

The majority of the new duplex villas will be located in the undeveloped area on the south side of the site, at the locations of Borings EB-14 through EB-21. This undeveloped parcel was mostly covered in tall grasses and weeds at the time of our investigation. The terrain forms a narrow ridge running from southeast to northwest.

The northern end of the undeveloped parcel is at roughly Elevation 380 feet and the southern end near the site entrance is at roughly Elevation 432 feet. The western edge of the parcel adjacent to the existing residential development is inclined at approximately 2:1 to 4:1 (horizontal:vertical).

3.3.4 Skilled Nursing Facility Addition

The new skilled nursing facility addition will be located on the northwest end of the site (Borings EB-5 through EB-8 and EB-23). Currently there is an at-grade parking lot and surrounding landscaping and flatwork, as well as numerous mature trees. The proposed addition site is bounded by Via Esplendor to the north, east and west. The existing skilled nursing facility lies immediately to the south.

Site grades in the proposed skilled nursing facility addition area range from approximately Elevation 350 to 351 feet, except for the western 50 feet of the planned building area, where the existing landscaping area slopes down gradually from approximately Elevation 348 to 344 feet. Based on a comparison of original and recent topographic information, it appears up to 10 feet of cut occurred within the proposed skilled nursing facility addition footprint during original site development.

3.4 SITE GEOLOGY

3.4.1 Geomorphology

The project area is located with a broad zone of generally northwest-trending lineaments and topographic features. The strongest geomorphic lineaments in the vicinity are associated with the base of the steep mountain slopes about 1,200 feet southwest of the Forum property, where the mountain front forms an abrupt, linear contact with flat-lying terrace deposits and underlying Santa Clara Formation (Figures 5A and 5B, Vicinity Geology Maps). Permanente Creek follows a linear trend southwest of the property before bending abruptly to the north. The 1-mile-long, linear creek channel section is bordered on the southwest by a series of prominent linear fronts and faceted ridge spurs. Mapping and trenching by others (e.g., Sorg and McLaughlin, 1975; Bedrossian, 1980a, 1980b; ESC, 1991) indicate that the linear mountain front here marks the Monte Vista fault zone.

Topographic features on and around the property also have a northwest trend. The two northwest-trending drainage swales, one located on and another immediately south of, the property may be considered to be linear drainages, at least over a distance of about 1,800 feet.

However, trenching indicates that they are not fault controlled (ESA, 1985; ESC, 1991). No other lineaments, scarps or topographic features indicative of recent faulting are identified on the Forum property.

3.4.2 Geologic Units

The approximate distribution and thickness of various geologic units are depicted on The Site Plan & Geologic Maps for the specific development areas, as shown on Figures 6A through 6E. Geologic cross sections at the proposed Memory Care facility are presented on Figures 7A and 7B. A geologic cross at the proposed Skilled Nursing Facility is presented on Figure 7C. Descriptions of the geologic units on or adjacent to the subject property are provided below based on our subsurface exploration and review of previous site investigations (listed in order of youngest to oldest age):

<u>Artificial (Man-Made) Fill</u> – Portions of the existing Forum site are underlain by man-made fills constructed as part of the original site development. Based on our review of the “Final Grading Report” for the site from 1990, the existing fills are considered “documented” and were reported to have been placed in accordance with the original project plans and specifications for compaction and moisture content. The report also references keyways that were to be constructed in sloping ground areas.

In general, the fills were reported to be derived from Santa Clara formation (QTsc) soils or a mixture of Santa Clara formation soil and colluvium (Qc). Based on our review of the current and original site topography, fills were generally placed within the former drainage ravine that trended east-west across the site. Additional minor fills associated with original building pad construction were also made. Fills were reported to be compacted to at least 92 percent relative compaction in building and roadway areas (Smith-Emery, 1990).

<u>Soil and Colluvium (Qc)</u> – Soil and colluvium overlie Quaternary alluvial deposits and Santa Clara Formation in the project area, except where removed by previous grading activities. The soil and colluvium are generally a few to several feet in thickness on the steeper hillslopes, and locally deeper on lower, gentler slopes and intervening hillside swales. The soil is characteristically clay or sandy clay of moderate to high plasticity and high dry strength. Fine- to coarse-grained sand and fine gravel locally compose about 10 to 15 percent of the volume, and some areas contain up to 30 percent gravel.

<u>Santa Clara Formation (QTsc)</u> – The Pliocene-Pleistocene-age Santa Clara Formation is a sequence of continental sedimentary rock that underlies the entire property. Regional mapping (Rogers and Armstrong, 1974) indicates that the property is on the southwestern flank of a northwest-trending anticline developed within the oldest facies (Searsville member) of the Santa Clara Formation. The Searsville member is distinguished by deep weathering that is unrelated to modern topography, which occurred prior to deposition of younger members of the Santa Clara Formation. As encountered in subsurface exploration on the property, the Santa Clara Formation consists of interbedded sequences of non-marine clay, silt, sand and gravel, which are poorly to well stratified with laminae and beds varying from 1 inch to 10 or more feet in

thickness. Most recognizable beds observed in trenches and recent borings are from 5 to 10 feet thick and internally stratified with fine- or coarse-grained, discontinuous thin lenses.

The fine-grained materials vary from highly plastic, very stiff clay to sandy silt interbedded with fine- to coarse-grained sand layers. The coarse-grained beds are typically coarse-grained sand, with lesser amounts of silty sand and clayey gravelly sand. Cobbles and boulders are up to 1 to 2 feet in size, and include deeply weathered greywacke sandstone, mudstone, volcanic and/or metamorphic clasts which are weak and friable. Lesser amounts of hard, strong chert, metamorphic rock and greenstone clasts are also present. The poor to moderate sorting, bed lenticularity, abrupt changes in grain size, buttress unconformities, and channels filled with coarse sediment all indicate that the Searsville facies was deposited in an alluvial fan environment, probably similar to the alluvial plan currently being developed along the eastern edge of the Santa Cruz Mountains. The sands and sandy gravels deposits represent channel deposits and the silty sands and clayey silts are overbank materials deposited adjacent to stream channels.

The Santa Clara Formation materials are typically uncemented, but moderately well consolidated and dense. No clean, loose sand or gravel zones were encountered during site investigations.

3.4.3 Geologic Structure

Local geologic structure was determined from surface exposures and measurements from trenches provided in the ESA (1985, 1986) and ESC (1991) reports. Our interpretation of the general geologic conditions is shown on Figures 3A and 3B.

In general, the local structure matches the regional northwest trend shown by others (Dibblee, 1966; Rogers and Armstrong, 1974). Bedding in the Santa Clara Formation, as exposed in trenches, is typically gentle to moderate (up to 30 degrees), with dips to the southwest. Southwest of the property, bedding attitudes in trenches excavated relatively close to the property (ESC, 1991) are typically toward the northeast, indicating the presence of northwest-trending synclinal fold axis just southwest of the property boundary. As noted previously, the Santa Clara Formation in the vicinity is strongly folded.

Minor shears and fractures of diverse orientation were encountered in most trenches excavated on the property by ESA (1985, 1986). Shears were identified as thin laminae, generally less than 1 inch wide, along which dislocations were observed. Shears were often marked by a thin seam of gley clay. Seams of gleyed clay were interpreted as possible shears even if offset bedding was not observed. Fractures, or breaks in earth materials along which no differential slippage occurred, were typically marked by coatings of calcium carbonate or clay that had been translocated downward from overlying soil horizons.

Most of the shears and fractures terminate either within the Santa Clara Formation or against the base of overlying soil horizons. Small, nearly vertical fractures were exposed in the upper (northeasterly) ends of trenches T-1 and T-2, near the crest of the ridge that lies along the northeastern property boundary. Most of these are normal faults with apparent dip separations of 1 to 2 inches. The steeply dipping normal faults and shears appear to be secondary earthquake effects in response to strong seismic ground motion from past earthquakes. Other shears that are locally parallel to stratification were observed in trench T-1 and were interpreted to be the result of flexural slope between folded strata. In general, shears could not be traced between trenches and were not considered to be tectonic faults capable of experiencing significant ground rupture.

ESA's initial investigation (1985) revealed the presence of a localized area of shearing referred to as a "disturbed zone", an anomalous zone of steeply dipping to vertical stratification with the Santa Clara Formation cut by numerous fractures and/or shear surfaces (ESA, 1985). Following input from the City of Cupertino's geologic reviewer (WCA, 1985), ESA performed supplemental trenching to further evaluate their cause(s) and define the lateral extent of the shears. Based on that work, ESA concluded that the "disturbed zone" exposed in three of their trenches (T-7, T-2 and T-5) is a paleo-slope failure associated with an ancient stream channel within the Santa Clara Formation and not a seismically capable tectonic fault. The shears were not encountered in several trenches excavated across the projected trend of the sheared zone, indicating that the lateral extent of the "disturbed zone" is less than 300 feet in length along a trend of about N20-25^{O}W. The ESA investigations were reviewed by the City of Cupertino's geologic peer reviewer, who agreed with the results and findings of ESA's work (WCA, 1986).

3.4.4 Site Specific Soil Conditions

3.4.4.1 New Memory Care Building

Based on conditions encountered in Borings EB-1 through EB-4, the soils in the top 2½ to 6 feet above Santa Clara Formation bedrock generally consisted of hard lean clays with sand, sandy lean clays, and medium dense clayey sands. In Boring EB-4, fill material consisting of hard sandy lean clay and lean clay with sand was encountered until about 2½ feet. The Santa Clara Formation consisted mostly of hard lean clays with sand and sandy lean clays. Very dense clayey sands with gravel were also encountered in Boring EB-1 from about 17 feet until the terminal depth of the boring at 24½ feet; EB-3 from 11½ feet to 17½ feet and again from 42 feet until the end of the boring at 49½ feet; and EB-4 from 16 feet to 24 feet. Geologic cross sections depicting the subsurface conditions in the Memory Care facility area are presented on Figures 7A and 7B.

We performed two Plasticity Index (PI) tests on representative samples. Test results were used to evaluate expansion potential of surficial soils, and the plasticity of the soils at the basement level. The results of the surficial PI tests indicated a PI ranging from 22 to 24, indicating moderate expansion potential to wetting and drying cycles. Laboratory testing indicated that the in-situ moisture contents within the upper 20 feet range from about 2 percent under to 6 percent over the estimated laboratory optimum moisture.

3.4.4.2 New Multipurpose Building and Fitness Building Addition

Fill was encountered in all of the borings in this area. About 2½ feet of fill consisting of stiff lean clay with sand was encountered in Boring EB-10, and about 1½ feet of fill consisting of hard sandy lean clay was encountered in EB-22. Approximately 11½ feet of fill was encountered in EB-9. As previously discussed, geologic maps and historic topographic maps indicate that EB-9 was drilled within the former drainage ravine that previously ran southeast to northwest through the site.

Below the fill, EB-9 encountered about 4½ feet of native, very stiff lean clay with sand before Santa Clara Formation (QTsc) was encountered. The QTsc was encountered directly below the fill in EB-10 and EB-22. The Santa Clara Formation consisted of alternating layers of hard sandy lean clays and dense clayey sands with gravel.

We performed one Plasticity Index (PI) test on a representative sample. Test results were used to evaluate expansion potential of surficial fill soils. The results of the surficial PI test indicated a PI of 29, indicating moderate to high expansion potential to wetting and drying cycles. Laboratory testing indicated that the in-situ moisture contents within the upper 10 feet range from about 3 percent under to 2 percent over the estimated laboratory optimum moisture.

3.4.4.3 New Villas and Duplexes

In Boring EB-11 drilled within the proposed Villas V61 and V62 area, we encountered about 2½ feet of fill material, which was underlain by Santa Clara Formation material. The QTsc consisted of approximately 2½ feet of medium dense clayey sands with gravel underlain by very stiff to hard sandy lean clays followed the clayey sands to the maximum depth drilled at 20 feet.

Two of the proposed villas in the east-central portion of the property (V63, V64 and V65) are underlain by documented fill overlying Santa Clara Formation. In Boring EB-12, the fill consisted of very stiff sandy lean clay and lean clay to a depth of approximately 7½ feet. The fill was underlain by QTsc consisting of hard sandy lean clay to the terminal depth of the boring at 10 feet. In Boring EB-13, documented fill consisted of hard sandy lean clay and very stiff lean clay with sand to a depth of approximately 10 feet. The fill was underlain by very stiff lean clay (likely some remnant colluvium or residual soil) to a depth of approximately 17 feet, which was underlain by hard sandy lean clay (QTsc) that extended to the maximum depth explored at 20 feet.

In the undeveloped parcel where Borings EB-14 through EB-21 were drilled (V66 through V85), approximately 3½ to 7½ of undocumented fill and/or colluvial soil was encountered, mostly consisting of very stiff to hard sandy lean clays, very stiff to hard lean clays with sand, and medium dense clayey sands with gravel. Based on our review of the Final Grading Report (1990), density tests were not performed during fill placement in the open space parcel, therefore, the fill encountered in Borings EB-14 to EB-21 is considered “undocumented”.
A 5-foot-thick layer of colluvium (fat clay with sand) was encountered in EB-14. The fill and colluvial soils are underlain by Santa Clara Formation materials that consisted of hard sandy lean clays, hard lean clays with sand, and hard lean clays.

We performed three Plasticity Index (PI) tests on surficial samples in future villa areas from Borings EB-11, EB-12 and EB-14. The results of the surficial PI tests indicated PIs of 17 at EB-11, 27 at EB-12, and 25 at EB-14, indicating moderate expansion potential to wetting and drying cycles. Laboratory testing indicated that the in-situ moisture contents within the upper 10 feet range from about 2 percent under to 8 percent over the estimated laboratory optimum moisture.

3.4.4.4 Skilled Nursing Facility Addition

Based on conditions encountered in Borings EB-5 through EB-8 and EB-23, Santa Clara Formation materials were encountered immediately below the surface pavements or ground surface. Boring EB-23 encountered less than 12 inches of fill below the pavement consisting of hard sandy clay with gravel. Below the fill or in areas where no fill was encountered, the borings encountered Santa Clara Formation consisting primarily of medium dense to very dense clayey sand with gravel interbedded with very stiff to hard lean clays with sand and sandy lean clays. A general summary of the soil types encountered within the Skilled Nursing Facility footprint are summarized in the following table. Geologic cross sections depicting the subsurface conditions across the skilled nursing facility addition area are presented on Figures 7C and 7D.

Table 3: Summary of Subsurface Conditions at Skilled Nursing Facility Addition

Boring No.	Approximate Layer Depth* (feet)	USCS Soil Type**
5	0 - 7½	CL
	7½ - 22½	SC
	22½ - 25	CL
6	0 - 4	SC
	4 - 25	CL
7	0 - 2½	CL
	2½ - 22½	SC
	22½ - 32	CL
	32 - 37	SC
	37 - 50	CL
8	0 - 14½	SC
	14½ - 25	CL
23	0 - 11½	SC
	11½ - 28½	CL

*Layer depths are approximate as gradual layer transitions may occur; layer thickness at terminal depth of boring not known.

**All soils encountered are considered Santa Clara Formation (QTsc) materials

We performed one Plasticity Index (PI) tests on a representative sample of the fine-grained Santa Clara Formation within the upper 5 feet of the proposed building pad. Test results were used to evaluate expansion potential of surficial soils, and the plasticity of the soils. The results of the surficial PI tests indicated a PI of 20, indicating moderate expansion potential to wetting and drying cycles. Laboratory testing indicated that the in-situ moisture contents within the upper 10 feet range from about 2 percent under to 8 percent over the estimated laboratory optimum moisture.

3.5 GROUND WATER

Ground water was encountered in our Borings EB-5, EB-7 and EB-9 at depths ranging from about 15 to 26½ feet below current grades; however, due to the varying surface grades, unsaturated soils beneath the measured ground water, and depth to Santa Clara Formation materials, it is likely that the ground water encountered was perched within sand layers underlain by very stiff to hard clay. It should be anticipated that ground water may be perched in other areas of the site. California Geological Survey (CGS) historic high ground water maps indicate that free ground water may be at depths greater than 50 feet. All measurements were taken at the time of drilling and may not represent the stabilized levels that can be higher than the initial levels encountered.

During the previous investigations, Earth Sciences Associates indicated that ground water was encountered at depths ranging from 13 to 24 feet below existing grades during their investigation; however, they also indicate that ground water was encountered between 15 and 50 feet below existing grades during an investigation performed by Woodward-Clyde Consultants in August, 1981. Earth Sciences Associates explains that the ground water encountered may be perched on top of clayey soils, but also that in one area, “a consistent piezometric surface is present in this area without significant perching or artesian conditions” (unfortunately, no site plan was provided indicating where this area is). As they had described materials encountered as having variable permeability, migration of perched ground water and percolating surface runoff water should be expected to be inconsistent across the site. For this project, free ground water should be anticipated to be greater than 50 feet in depth; however, perched ground water may be encountered locally during construction and may require localized dewatering.

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

Movement along an active fault that intersects the ground surface can result in permanent ground displacements which may severely damage structures. The most common method of mitigating the hazard of surface fault rupture is to avoid active fault traces. However, in some circumstances, structures can be designed to accommodate or resist estimated fault displacements.

Faults are considered to be “active” if they display evidence of movement within Holocene time (the last 11,000 years), and “potentially active” if they display evidence of movement within Quaternary time (i.e., within the last 1.6 million years). The State of California regulates development near known active faults through the Alquist-Priolo Special Studies Zone Act. “Fault-Rupture Hazard Zones” (formerly “Special Study Zones”) have been established around known active faults by the California Division of Mines and Geology (Bryant and Hart, 2007). The property is not located within a Fault-Rupture Hazard Zone established around a designated Holocene-active fault by the California Geologic Survey (California Division of Mines and Geology); however, it is located within a Santa Clara County fault hazard zone for the Monte Vista-Shannon Fault (Figure 8). The County fault hazard zone was intended to encompass not only known fault traces, but also a wide variety of discontinuous geomorphic lineaments. The geomorphology of the project area is described in Section 3.4.1.

The Monte Vista fault zone has been mapped along the base of the mountain front, approximately 1,200 feet southwest of the property (Sorg and McLaughlin, 1975; Bedrossian, 1980a). The fault zone is defined by two primary traces that are evident from geomorphology and local faulted exposures. In general, the higher, southwestern trace juxtaposes Franciscan Complex over Monterey Formation, and the lower northeastern trace juxtaposes Monterey Formation and Santa Clara Formation. Trenching by ESC (1991) confirmed the locations of the main fault traces along the mountain front (Figures 5A and 5B). Specifically, their trenches T- 1, T-2 and T-4, excavated across the slope break at the base of the mountain front, exposed offset soil and colluvial units overlying sheared bedrock. Based on those trenches, ESC (1991) concluded that the Monte Vista fault at those locations demonstrates Holocene activity. Other trenches to investigate possible geomorphic indications of faulting further to the northeast (and closer to the Forum property), did not reveal any faulting (ESC, 1991).

In three of their trenches on the property, ESA encountered a localized area of shearing referred to as the “disturbed zone”. The shears were not encountered in several trenches excavated across the projected trend of the sheared zone, indicating that the lateral extent of the “disturbed zone” is less than 300 feet in length, and likely is a paleo-slope failure rather than tectonic fault rupture. The ESA investigations were reviewed by the City of Cupertino’s geologic peer reviewer, who agreed with the results and findings of ESA’s work (WCA, 1986). The “disturbed zone” investigated by ESA in two investigations (1985, 1986) is located in the southeastern portion of the property, and does not impact the proposed structures (Figures 5A and 5B).

Shears exposed in trenches excavated about a mile southeast of the property (ESA, 1979) appeared to extend into soils ranging in age between 3,000 and 5,000 years old. However, the shears could not be traced for more than 200 feet laterally, and individual shears showed discrepancies in apparent sense of movement. Subsequent trenching by ESC to evaluate the same shears encountered by ESA 1979 did not encounter continuous shears. ESC (1991) concluded that the general lack of shears and the dissimilar character and location of shears at that location suggested that they likely are attributed to seismic shaking and/or flexural slip associated with folding in the Santa Clara Formation, and are not the result of tectonic faulting.

Based on distance (1,200 feet) to the active Monte Vista fault zone and results of previous site investigations (ESA, 1985; ESC, 1991), no active fault traces or continuous tectonic shears are present across the proposed building sites on the property. Consequently, we judge the potential for primary tectonic surface fault rupture at the proposed sites to be low.

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) of 0.88g was estimated for analysis using a value equal to $PGA_M = F_{PGA} \times PGA_G$ (Equation 11.8-1) as allowed in the 2016 California Building Code (CBC).

We also developed site-specific seismic design parameters for the proposed Skilled Nursing Facility in accordance with Chapters 16A and 18A of the 2016 California Building Code (CBC) and Chapters 11 and 21 of ASCE 7-10. The results of this analysis are presented in Appendix E.

4.3 LIQUEFACTION POTENTIAL

As shown on Figure 9, the site is located from about 250 to 750 feet east of a State-designated Liquefaction Hazard Zone (CGS, Cupertino Quadrangle, 2002) and a Santa Clara County Liquefaction Hazard Zone (Santa Clara County, 2012). Additionally, the site is within a zone mapped as having a low liquefaction potential by the Association of Bay Area Governments (ABAG). However, we screened the site for liquefaction during our site exploration by retrieving samples from the site, performing visual classification on sampled materials, and performing various tests to further classify the soil properties.

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 stiff cohesive and dense granular soils. In addition, the design ground water level is anticipated to be greater than 50 feet. 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.

While the terrain is hilly, the soils encountered are relatively stiff or dense and are not susceptible to liquefaction; therefore, in our opinion, the potential for lateral spreading to impact new site improvements is 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 stiff to very stiff clays and medium dense to dense sands, in our opinion, the potential for significant differential seismic settlement affecting the proposed improvements is low.

4.6 LANDSLIDING

The site is located in a rolling hillside area, with gentle to moderate slopes underlain by bedrock materials at a shallow depth. According to the State Seismic Hazards Zone Report for this area (CGS, 2002), the site is not located in an area considered susceptible to earthquake-triggered landsliding (Figure 9).

Based on the preceding, we judge the potential for static and seismically-induced landsliding at the site to be low. Construction stability of any planned excavations should be analyzed and addressed as part of the shoring design for below-grade excavations for the project.

4.7 TSUNAMI/SEICHE

The terms tsunami or seiche are described as ocean waves or similar waves usually created by undersea fault movement or by a coastal or submerged landslide. Tsunamis may be generated at great distance from shore (far field events) or nearby (near field events). Waves are formed, as the displaced water moves to regain equilibrium, and radiates across the open water, similar to ripples from a rock being thrown into a pond. When the waveform reaches the coastline, it quickly raises the water level, with water velocities as high as 15 to 20 knots. The water mass, as well as vessels, vehicles, or other objects in its path create tremendous forces as they impact coastal structures.

Tsunamis have affected the coastline along the Pacific Northwest during historic times. The Fort Point tide gauge in San Francisco recorded approximately 21 tsunamis between 1854 and 1964. The 1964 Alaska earthquake generated a recorded wave height of 7.4 feet and drowned eleven people in Crescent City, California. For the case of a far-field event, the Bay area would have hours of warning; for a near field event, there may be only a few minutes of warning, if any.

A tsunami or seiche originating in the Pacific Ocean would lose much of its energy passing through San Francisco Bay. Based on the study of tsunami inundation potential for the San Francisco Bay Area (Ritter and Dupre, 1972), areas most likely to be inundated are marshlands, tidal flats, and former bay margin lands that are now artificially filled, but are still at or below sea level, and are generally within 1½ miles of the shoreline. The site is approximately 7½ miles inland from the San Francisco Bay shoreline, and is approximately 332 to 432 feet above mean sea level (NGVD 29). Therefore, the potential for inundation due to tsunami or seiche is considered low.

4.8 FLOODING

Based on our internet search of the Federal Emergency Management Agency (FEMA) flood map public database, the site is located within Zone D, an area of undetermined, but possible flood hazard. We recommend the project civil engineer be retained to confirm this information and verify the base flood elevation, if appropriate.

4.9 OTHER POTENTIAL GEOLOGIC HAZARDS

Other potential geologic hazards, including hazards posed by volcanic activity, naturally occurring asbestos, and radon gas were evaluated and found to be not significant to the project site. Naturally occurring asbestos is associated with ultramafic bedrock, which is not present in the near-surface at the project site.

Radon is a radioactive gas formed by decay of small amounts of uranium and thorium naturally present in rock and soil. Sometimes radon gas can move out from underlying soil and rock into houses and become concentrated in the indoor air, posing a health risk for occupants. According to Churchill (2014), four radon potential categories defined by the percentage of homes with indoor radon likely to equal or exceed 4.0 pCi/L are identified in State surveys: high (≥ 20 percent), moderate (≥ 5.0 to 19.9 percent), low (< 5 percent), and unknown (for geologic units with few or no data). The radon potential for the Santa Clara Formation is in "low" category.

Hazards associated with active volcanoes include inundation by ash, pyroclastic flows, and mudflows. The severity of volcanic hazards is associated with distance to the volcanic source, magnitude and type of volcanic activity, and direction of prevailing winds. No recent volcanic deposits or other indications of recent volcanic activity are present in the project area. Due to the lack of major volcanic activity in the vicinity, the potential for site to be impacted by volcanic hazards is judged to be negligible.

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 man-made fills
- Presence of expansive soils
- Potential for shallow, perched ground water
- Differential movement at on-grade to on-structure transitions

- Temporary cut slope or deep trench instability
- Soil corrosion potential

5.1.1 Presence of Man-Made Fill

As discussed, “documented” and “undocumented” fill was encountered in several of our recent borings. Documented fill consists of soil materials generated from on-site cuts that were reportedly placed and compacted during original site development in 1989 and 1990. We reviewed the Final Grading Report dated 1990 and observed that fills were generally compacted to at least 92 percent relative compaction based on ASTM D1557 and D2922 test methods. Where encountered, the thickness of the documented fills generally ranged from about 1½ to 4½ feet; however, the thickness of the fill in the area of the proposed multi-purpose building ranged from around 2 to 11½ feet. At the multi-purpose building, most of the documented fill can likely remain in place; however, the new structure should be supported on drilled pier foundations to reduce the potential for differential settlement. Since the area is heavily landscaped, the upper 2 feet of existing fill should be re-compacted prior to new fill placement or foundation construction.

In the undeveloped parcel area where new duplex villas are proposed (V66 through V85), undocumented fill was encountered up to 4½ feet thick. Based on our review of historic aerial photographs and the Final Grading Report (1990), this fill appears to have been placed during original site development for construction trailers and staging areas; however, the fill material was not tested for compaction. Therefore, we recommend that undocumented fills blanketing portions of the undeveloped parcel be over-excavated and re-compacted prior to placing new fill or foundation construction. Provided the undocumented fills are adequately mitigated during site grading, new duplex villas in the open space parcel can be supported on either shallow footings or drilled piers. Grading and foundation recommendations addressing this concern are presented in the following sections of this report.

5.1.2 Presence of Expansive Soils

As discussed, moderately to highly expansive surficial soils were encountered in the surficial soils that blanket the site. Expansive soils can undergo significant volume change with changes in moisture content. They shrink and harden when dried and expand and soften when wetted. To reduce the potential for damage to the planned structures, we recommend slabs-on-grade have sufficient reinforcement and be supported on a layer of non-expansive fill and that foundations extend below the zone of seasonal moisture fluctuation. In addition, it is important to limit moisture changes in the surficial soils by using positive drainage away from buildings as well as limiting landscaping watering. Grading and foundation recommendations addressing this concern are presented in the following sections of this report.

5.1.3 Potential for Shallow, Perched Ground Water

As previously mentioned, ground water was encountered in three of our borings at depths between 15 and 26½ feet below existing grades. Earth Sciences Associates also encountered ground water at depths ranging from 13 to 24 feet below original site grades, and Woodward-

Clyde Consultants reportedly encountered it between 15 and 50 feet below original site grades. CGS maps indicate that historically high ground water may be below 50 feet in depth; therefore, the ground water encountered is most likely perched and isolated within more permeable sand layers in the Santa Clara Formation or possibly near the fill/native soil contact. Earth Sciences Associates also indicated that the soils throughout the site have variable permeability; therefore, migration of perched ground water and percolating surface runoff water should be anticipated to be unpredictable. Shallower perched ground water could potentially impact deep excavations and utility installations. The contractor should anticipate localized dewatering if perched ground water is encountered.

5.1.4 Differential Movement at On-Grade to On-Structure Transitions

For the proposed Memory Care Facility, some improvements such as walkways, patios or stairways may transition from on-grade support to overlying the basement. Where the basement walls extend to within inches of finished grade, these transition areas typically experience increased differential movement due to a variety of causes, including difficulty in achieving compaction of retaining wall backfill closest to the wall. We recommend consideration be given to where engineered fill is placed behind retaining walls extending to near finished grade, and that subslabs be included beneath flatwork or pavers that can cantilever at least 3 feet beyond the wall. If surface improvements are included that are highly sensitive to differential movement, additional measures may be necessary. We also recommend that retaining wall backfill be compacted to 95 percent where surface improvements are planned (see “Retaining Wall” section).

5.1.5 Differential Movement Due to Material Transitions

Material transitions occur when two or more materials with differing geotechnical characteristics interface in a small area, such as within a single residential lot or pavement area. The materials that comprise these transitions can include bedrock, surficial soils, or engineered fill. Because the geotechnical characteristics of the materials are different, the long-term performance of the materials will also be different.

For instance, fills materials, even if well compacted, are typically more compressible than Santa Clara Formation (QTsc) materials and as a result will usually experience a greater amount of settlement under various loading conditions. The differences in the amount of settlement or expansion between fill materials and QTsc materials can cause distress to residential foundations and other site improvements. Such distress will often either add to the long-term maintenance costs or reduce the design life associated with the structure.

The preliminary grading plans indicate that the Memory Care Facility and several residential villas may expose cut or engineered fill materials. For the Memory Care Facility, we recommend that the building be supported on drilled piers to mitigate potential differential settlement concerns. Cut/fill and material transitions should be over-excavated and rebuilt with engineered fill to reduce the potential for differential movement beneath structures for V69/70, and V79 through V83. Recommendations addressing these concerns are presented in the “Earthwork” section of this report.

5.1.6 Potential for Localized Cut Slope Instability

The soils encountered during our recent exploration consisted primarily very stiff to hard lean clays with sand, sandy lean clays, and medium dense to very dense clayey sands. However, during exploratory trenching by Earth Sciences Associates, they noted a “disturbed zone” within an ancient stream channel that was exposed in exploratory trenches. Characteristics of this zone included “steeply dipping beds and numerous shear surfaces with various orientations” (Earth Sciences Associates, 1986). Based on our review of this information, in our opinion, deep excavations for the memory care basement or for deep utility trenches will need to be properly shored and supported to reduce the potential for localized cut slope failures. Deep cuts should also be observed by our Engineering Geologist to check for potential unstable shear surfaces. Further discussion on below-grade excavations may be found in the “Earthwork” section.

5.1.7 Soil Corrosion Potential

A preliminary soil corrosion screening was performed by JDH Corrosion Consultants based on the results of analytical tests on samples of the near-surface soil. In general, the JDH report concludes that the corrosion potential for buried concrete warrants the use of sulfate resistant concrete. In addition, the corrosion potential for buried metallic structures, such as metal pipes, is considered corrosive. JDH recommends that special requirements for corrosion control be made to protect metal pipes. A more detailed discussion of the site corrosion evaluation is presented in Appendix C. As the preliminary soil corrosion screening was based on the results of limited sampling, consideration may be given to collecting and testing additional samples from the upper 5 feet for sulfates and pH to confirm the classification of corrosive to mortar coated steel and concrete.

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 proposed development sites should be stripped of all surface vegetation, and surface and subsurface improvements. Demolition of existing improvements is discussed in detail below. A detailed discussion of removal of existing fills 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 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 EXISTING FILLS OR COLLUVIAL SOIL

New construction areas underlain by “documented” fill placed during original site development can mostly be left in-place if exposed during building pad and foundation construction. This will likely include the fill encountered in the proposed multi-purpose building (Borings EB-9 and EB-10). Once all landscaping and organic topsoil are removed from the multi-purpose building area, the upper 2 feet of existing “documented” fill should be re-compacted prior to placing any new fills or constructing the building foundation. In the Memory Care building area, the west-central edge of the building footprint will expose “documented” fill associated with the original channel fill. Compaction records indicate this fill was compacted to between 87 and 94 percent relative compaction. Once exposed, this fill should be observed and if necessary, over-excavated and re-compacted prior to foundation construction.

Our recent borings and our review of historic aerial photographs and as-built topographic plans indicate the undeveloped parcels for the new residential duplexes are blanketed by approximately 3 to 4 feet of undocumented fill and/or colluvial soils. These materials are more variable and potentially weak or compressible. Therefore, we recommend that undocumented fills and colluvial soils in the villas area (V66 through V85) be over-excavated and replaced as engineered fill within building areas prior to placing new fill or constructing new foundation. The over-excavation should extend to a lateral distance of at least 5 feet beyond the building footprint or to a lateral distance equal to fill depth below the perimeter footing, whichever is greater. Provided the fills meet the “Material for Fill” requirements below, the fills may be reused when backfilling the excavations.

Based on review of the samples collected from our borings, it appears that the existing 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.

Fills extending into planned pavement and flatwork areas may be left in place provided they are determined to be a low risk for future differential settlement and that the upper 12 to 18 inches of fill below pavement subgrade is re-worked and compacted as discussed in the “Compaction” section below. In our opinion, the fills encountered at this site are acceptable to be left in place.

6.3 CUT/FILL TRANSITION OVER-EXCAVATION

Residential villas on lots with cut/fill transitions should be over-excavated to provide a relatively uniform fill thickness beneath the building footprint. Based on our review of the preliminary grading plans, cut/fill transition over-excavation should be performed for the Memory Care Facility garage level pad and on residential lots for V69/70 and V79 through V83. For the Memory Care Facility, the depth of over-excavation should be at least 2 feet below pad grade. For the residential pads, the depth of over-excavation below pad grade should be equal to the maximum fill thickness on the pad but need not exceed 4 feet, as shown in Figure 10. If material transitions are observed within proposed building or street areas, it may be necessary to over-excavate exposed Santa Clara Formation materials to reduce the potential impact on improvements. The depth of the over-excavation will depend on the type of material exposed, and will be determined in the field during construction.

In general, over-excavation should extend to at least 5 feet beyond the building footprint or street improvements. Adjustments to the depth and lateral limits of the over-excavation may need to be made at the time of construction depending on the actual conditions encountered during grading.

6.4 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 10 feet at the site may be classified as OSHA Type C soils; cuts into competent Santa Clara Formation materials can likely be classified as Type B soils. A Cornerstone representative should be retained to confirm the preliminary site classifications. Recommended soil parameters for temporary shoring are provided in the following section of this report.

Excavations performed during site demolition and fill removal should be sloped at 3:1 (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.

6.5 BELOW-GRADE EXCAVATIONS

Below-grade excavations may be constructed with temporary slopes in accordance with the “Temporary Cut and Fill Slopes” section above if space allows. Alternatively, temporary shoring may support the planned cuts in the Memory Care facility area up to 20 feet. We have provided geotechnical parameters for shoring design in the section below. The choice of shoring method should be left to the contractor’s judgment based on experience, economic considerations and adjacent improvements such as utilities, pavements, and foundation loads. Temporary shoring should support adjacent improvements without distress and should be the contractor’s responsibility. A pre-condition survey including photographs and installation of monitoring points for existing site improvements should be included in the contractor’s scope. We should be provided the opportunity to review the geotechnical parameters of the shoring design prior to implementation; the project structural engineer should be consulted regarding support of adjacent structures.

6.5.1 Temporary Shoring

Based on the site conditions encountered during our investigation, the cuts may be supported by soldier beams and tie-backs, braced excavations, soil nailing, or potentially other methods. Where shoring will extend more than about 10 feet, restrained shoring will most likely be required to limit detrimental lateral deflections and settlement behind the shoring. In addition to soil earth pressures, the shoring system will need to support adjacent loads such as construction vehicles and incidental loading, existing structure foundation loads, and street loading. We recommend that heavy construction loads (cranes, etc.) and material stockpiles be kept at least 15 feet behind the shoring. Where this loading cannot be set back, the shoring will need to be designed to support the loading. The shoring designer should provide for timely and uniform mobilization of soil pressures that will not result in excessive lateral deflections. Minimum suggested geotechnical parameters for shoring design are provided in the table below.

Table 4: Suggested Temporary Shoring Design Parameters

Design Parameter	Design Value
Minimum Lateral Wall Surcharge (upper 5 feet)	120 psf
Cantilever Wall – Triangular Earth Pressure	45 pcf
Restrained Wall – Trapezoidal Earth Pressure	Increase from 0 to 25H* psf
Passive Pressure – Starting at 2 feet below the bottom of the excavation	500 pcf up to 3,000 psf maximum uniform pressure

* H equals the height of the excavation; passive pressures are assumed to act over twice the soldier pile diameter

The restrained earth pressure may also be distributed as described in Figure 23 of the *FHWA Circular No. 4 – Ground Anchors and Anchored Systems* (with the hinge points at ¼H and ¾H) provided the total pressure is established from the uniform pressure above.

If shotcrete lagging is used for the shoring facing, the permanent retaining wall drainage materials, as discussed in the “Wall Drainage” section of this report, will need to be installed during temporary shoring construction. At a minimum, 2-foot-wide vertical panels should be placed between soil nails or tiebacks that are spaced at 6-foot centers. For 8-foot centers, 4-foot-wide vertical panels should be provided. A horizontal strip drain connecting the vertical panels should be provided, or pass-through connections should be included for each vertical panel.

We performed our borings with both hollow-stem auger and solid-stem auger drilling equipment. The borings drilled with the solid-stem augers did not appear to cave in during drilling. Caving soils can be problematic during excavation and installations of soldier beams, lagging, tie-backs, and soil nails. The contractor is responsible for evaluating excavation difficulties prior to construction. Pilot holes performed by the contractor may be desired to further evaluate these conditions prior to the finalization of the shoring budget.

In addition to anticipated deflection of the shoring system, other factors such as voids created by soil sloughing, and erosion of granular layers due to perched water conditions can create adverse ground subsidence and deflections. The contractor should attempt to cut the excavation as close to neat lines as possible; where voids are created they should be backfilled as soon as possible with sand, gravel, or grout.

As previously mentioned, we recommend that a monitoring program be developed and implemented to evaluate the effects of the shoring on adjacent improvements. All sensitive improvements should be located and monitored for horizontal and vertical deflections and distress cracking based on a pre-construction survey. For multi-level excavations, the installation of inclinometers at critical areas may be desired for more detailed deflection monitoring. The monitoring frequency should be established and agree to by the project team prior to start of shoring construction.

The above recommendations are for the use of the design team; the contractor in conjunction with input from the shoring designer should perform additional subsurface exploration they deem necessary to design the chosen shoring system. A California-licensed civil or structural engineer must design and be in responsible charge of the temporary shoring design. The contractor is responsible for means and methods of construction, as well as site safety.

6.6 SUBGRADE PREPARATION

After site clearing and demolition is complete, and prior to backfilling any excavations resulting from fill 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.

6.7 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.

As discussed in the “Subsurface” section in this report, the in-situ moisture contents range from about 3 percent under to 8 percent over the estimated laboratory optimum in the upper 10 to 20 feet of the soil profile. The contractor should anticipate drying the soils above the optimum moistures prior to reusing them as fill. In addition, repetitive rubber-tire loading will likely de-stabilize the soils. As previously mentioned, perched ground water may be encountered during excavations; the soils under the water level should be anticipated to be above the optimum moistures.

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.

6.7.1 Scarification and Drying

The subgrade may be scarified to a depth of 6 to 9 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.

6.7.2 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.

6.7.3 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.

6.8 MATERIAL FOR FILL

6.8.1 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.

6.8.2 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 new 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.

6.8.3 Non-Expansive Fill Using Lime Treatment

As discussed above, non-expansive fill should have a Plasticity Index (PI) of 15 or less. Due to the high clay content and PI of the on-site soil and Santa Clara Formation materials, it is not likely that sufficient quantities of non-expansive fill would be generated from cut materials. As an alternative to importing non-expansive fill, chemical treatment can be considered to create non-expansive fill. It has been our experience that for high PI clayey soil and bedrock materials will likely need to be mixed with at least 3 to 4 percent quicklime (CaO) or approved equivalent to adequately reduce the PI of the on-site soils to 15 or less. If this option is considered, additional laboratory tests should be performed during initial site grading to further evaluate the optimum percentage of quicklime required.

6.9 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. Where the soil’s PI is 20 or greater, the expansive soil criteria should be used.

Table 5: Compaction Requirements

Description	Material Description	Minimum Relative[1] Compaction (percent)	Moisture[2] Content (percent)
General Fill (within upper 5 feet)	On-Site Expansive Soils	87 – 92	>3
	Low Expansion Soils	90	>1
General Fill (below a depth of 5 feet)	On-Site Expansive Soils	95	>3
	Low Expansion Soils	95	>1
Basement Wall Backfill	Without Surface Improvements	90	>2
	With Surface Improvements	95[4]	>2
Trench Backfill	On-Site Expansive Soils	87 – 92	>3
	Low Expansion Soils	90	>1
Trench Backfill (upper 6 inches of subgrade)	On-Site Low Expansion Soils	95	>1
Crushed Rock Fill	¾-inch Clean Crushed Rock	Consolidate In-Place	NA
Non-Expansive Fill	Imported Non-Expansive Fill	90	Optimum
Flatwork Subgrade	On-Site Expansive Soils	87 - 92	>3
	Low Expansion Soils	90	>1
Flatwork Aggregate Base	Class 2 Aggregate Base[3]	90	Optimum
Pavement Subgrade	On-Site Expansive Soils	87 - 92	>3
	Low Expansion Soils	95	>1
Pavement Aggregate Base	Class 2 Aggregate Base[3]	95	Optimum
Asphalt Concrete	Asphalt Concrete	95 (Marshall)	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

6.9.1 Construction Moisture Conditioning

Expansive soils can undergo significant volume change when dried then wetted. The contractor should keep all exposed expansive soil subgrade (and also trench excavation side walls) moist until protected by overlying improvements (or trenches are backfilled). If expansive soils are allowed to dry out significantly, re-moisture conditioning may require several days of re-wetting (flooding is not recommended), or deep scarification, moisture conditioning, and re-compaction.

6.10 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.

On expansive soils sites it is desirable to reduce the potential for water migration into building and pavement areas through the granular shading materials. We recommend that a plug of low-permeability clay soil, sand-cement slurry, or lean concrete be placed within trenches just outside where the trenches pass into building and pavement areas.

6.11 PERMANENT CUT AND FILL SLOPES

All permanent cut and fill slopes in soil should have a maximum inclination of 2:1 (horizontal:vertical) for slopes up to 10 feet high; slopes greater than 10 feet should be inclined at no greater than 2.5:1. All permanent cuts in competent bedrock may have a maximum inclination of 1:1. Fill slopes should be overbuilt and trimmed back, exposing engineered fill when complete. Refer to the "Erosion Control" section of this report for a discussion regarding protection of slope surfaces.

6.11.1 Keyways and Benches

Fill placed on existing ground inclined at 6:1 or greater should be benched into the existing slope and a keyway constructed at the toe of the fill. Benches should be angled slightly into the slope be spaced vertically at no greater than 4 feet between benches, and be at least 6 to 8 feet wide. Depending on the thickness of any colluvial/residual soil layer that blankets the Santa Clara Formation, the benches may need to be widened beyond the minimum width to extend into competent bedrock. The keyway should also be angled slightly into the slope (minimum 2 percent inclination), extend at least 3 feet into competent materials, and be at least 10 to 18 feet wide. A typical key and construction is depicted in Figure 11.

6.11.2 Fill Drainage

A permanent subsurface drainage system consisting of a series of perforated gravity pipes or drainage strips should be constructed between engineered fill placed against a bedrock slope and within all keyways. This system is intended to intercept perched water flowing through the bedrock and transmit it to suitable outlet structures and reduce the potential for hydrostatic pressures building up behind the fills, and causing slope instability. The drain lines should be placed at the back of the keyways and benches. Bench drains should be spaced vertically at no greater than 10 feet.

The drainage system should be constructed in small trenches or v-ditches as shown in Figure 12, and will consist of a minimum 4-inch-diameter perforated SDR 35 (perforations placed downward), bedded and shaded in Caltrans Class 2 Permeable Material (latest version) or ¾-inch crushed rock; if crushed rock is used, the rock should be encapsulated in filter fabric (Mirafi 140N or equivalent). The bedding should be at least 2 inches, and the trench should be at least 8 inches in width and depth. Alternatively, geocomposite strip drains may be used. All drainage lines should slope towards suitable outlet structures at an inclination of at least 0.5 percent. Suitable outlet structures may consist of connecting the drainage lines to a storm drain system, with a sump if required; if the drain lines will outlet overland at the toe of the slope, an appropriate rock spill pad should be provided; the drain lines should not outlet onto the slope.

Vertical cleanouts should be provided at all upslope ends of the drainage lines and at all 90-degree bends.

6.11.3 Plan Review and Construction Monitoring

We should be retained to review the conceptual grading and sub-drainage plans and we can provide more specific input regarding the location of keyways and fill drainage for the final plans. A Cornerstone representative should be on site during keyway and fill slope construction. Field modifications to the planned keyway and benching may be required based on encountered field conditions. In addition, it has been our experience that cut slopes in the Santa Clara Formation are prone to localized weak zones and sloughing along bedding planes. We recommend that a Cornerstone engineering geologist observe the condition of all cut slopes and evaluate the potential for localized adverse materials or bedding orientation.

We recommend that the project civil engineer or land surveyor be retained to survey in place all keyways, sub-drainage lines, solid pipes, and cleanouts, and create an as-built plan. This plan will be of use for any future maintenance or repair work.

6.12 SITE DRAINAGE

6.12.1 Surface Drainage for At-Grade Structures

Ponding should not be allowed 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 to at least 10 feet from the structure. 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.

Where minimal side yards are planned (10 feet or less), we recommend that area drains collect surface runoff and transmit the runoff to other suitable landscape drainage facilities to prevent ponding adjacent to building foundations. Landscape drainage such as drain inlets and storm water filtration and/or infiltration trenches should be provided to collect and transmit storm water runoff to project storm drains, and/or detention or retention facilities.

6.12.2 Surface Drainage for Slopes

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. These facilities are not recommended where stormwater infiltration may affect slopes at lower elevations on or adjacent to the site. However, if slopes are not present at lower elevations that could potentially be affected, and 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.

Lined v-ditches should be included at the top of slopes and intermediate benches, and at the toe of slopes or behind retaining walls adjacent to planned or existing development. All v-ditches and drain inlets should be sized to accommodate the design storm events for the upslope tributary area. Concrete-lined v-ditches should be reinforced as required and have adequate control and construction joints, and should be constructed neat in excavations; backfill around formed ditches should not be allowed.

Upslope sources of water should be evaluated. If upslope irrigation is present or planned, additional surface and subsurface drainage, or construction of drained buttress fills may be needed to protect site improvements. We should be consulted if this issue will affect the project.

6.12.3 Subsurface Drainage

As discussed in the “Permanent Cut and Fill Slopes” section, subsurface drainage improvements should be installed as part of earthwork for fill construction. These improvements should include positive surface gradients for keyways and benches and the installation of a subdrain system consisting of perforated pipe and permeable gravel or drain rock. If drain rock is used, the rock and pipe should be entirely wrapped with a permeable geotextile fabric. Subdrains should also be installed at the toe of any proposed cut slopes depending on the actual conditions observed during construction. A typical trench subdrain detail is shown on Figure 13. As previously discussed, a conceptual subdrain plan should be prepared once preliminary grading plans are finalized. The actual location of subdrains should be determined in the field at the time of construction.

6.13 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 consist of clayey soils with occasional interbedded sand layers, likely to be categorized as Hydrologic Soil Group C or D, and is expected to have infiltration rates of less than 0.2 to 0.5 inches per hour. In our opinion, these clayey soils will significantly limit the infiltration of stormwater.

- Locally, seasonal high ground water is mapped at a depth of greater than 50 feet, and therefore is expected to be at least 10 feet below the base of the infiltration measure.

- In our opinion, infiltration locations within 10 feet of the proposed buildings would create a geotechnical hazard.

6.13.1 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.

6.13.1.1 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 expansive 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.

6.13.1.2 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.

6.13.1.3 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, or

- 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.

6.14 PERMANENT EROSION CONTROL MEASURES

Hillside grading will require periodic maintenance after construction to reduce the potential for erosion and sloughing. 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.

6.15 LANDSCAPE CONSIDERATIONS

Since the near-surface soils are moderately to highly expansive, we recommend greatly reducing the amount of surface water infiltrating these soils near foundations and exterior slabs-on-grade. 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

Due to the potential for differential settlement associated with anticipated cut/fill or material transitions, new structures should be supported on drilled, cast-in-place, straight-shaft friction piers. As an alternative to drilled piers and provided cut/fill transitions are adequately mitigated during grading, the proposed memory care facility and residential villa structures on flat lots may also be supported on conventional shallow footings, as summarized in the following table. The recommendations in the "Earthwork" section and the sections below should be followed.

Table 6: Summary of Recommended Foundation Alternatives

Building Location	Foundation Type		Foundation Notes
	Drilled Piers	Shallow Footings	
Memory Care Facility	Yes	Yes	Drilled piers are required to mitigate differential settlement across cut/fill transition and due to presence of previously placed fill along the western edge of the building. If material transitions are over-excavated and footings are deepened, then conventional shallow footings may be considered
Multi-Purpose Building	Yes	No	Drilled piers only due to differential fill across footprint
Fitness Center/ Dining Room Addition	Yes	No	Drilled piers only due to prior fill and to match existing building foundation
Villas (V63/64)	Yes	No	Drilled piers only due to potential differential settlement from material transitions
Villas (V61/62)	Yes	Yes	Shallow footings may be considered
Villas (V66 through V85)	Yes	Yes	Shallow footings may be considered provided all undocumented fill, colluvial soil and cut/fill transitions are over-excavated prior to building pad construction
Skilled Nursing Facility	Yes	No	Drilled piers only due to potential differential fill settlement and to match existing building foundation

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. For the Skilled Nursing Facility, additional seismic design criteria is presented in Appendix E, as needed. 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 Santa Clara Formation, which is generally described as a very stiff soil or soft bedrock material, with an average shear wave velocity of about 450 to 500 meters per second (1,300 to 1,500 feet per second), and a typical SPT "N" values greater than 50 blows per foot. Therefore, we have classified the site as Site Class C – Very Dense Soil and Soft Rock. 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 7: CBC Site Categorization and Site Coefficients

Classification/Coefficient	**Design Value**
Site Class	C
Site Latitude	37.338947°
Site Longitude	-122. 088969°
0.2-second Period Mapped Spectral Acceleration[1], S_S	2.268g
1-second Period Mapped Spectral Acceleration[1], S_1	0.819g
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}	2.268g
1-second Period, Maximum Considered Earthquake Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	1.065g
0.2-second Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.512g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	0.710g

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

7.3 DRILLED PIER FOUNDATIONS

The proposed structures may be supported on drilled, cast-in-place, straight-shaft friction piers deriving support from the underlying Santa Clara Formation soils. Drilled pier design parameters are presented in the following table. Adjacent piers should be spaced at least three diameters apart, otherwise, a reduction for group effects may be required.

7.3.1 Vertical Capacity and Estimated Settlement

Since existing documented fills, such as in the memory care, multi-purpose and fitness center addition area, have been in place since 1990, we judge the potential for significant fill settlement to cause downdrag on new piers to be negligible; therefore, the vertical capacity of the existing documented fill has not been neglected, except to account for seasonal shrink and swell of expansive soils. The allowable skin friction may be increased by one-third for wind and seismic loads. Grade beams should extend at least 18 inches below the lowest adjacent grade.

Table 8: Design Criteria for Drilled Piers

Building Location	Minimum Pier Diameter (inches)	Minimum Pier Depth[1] (feet)	Allowable Skin Friction (psf)	Depth to Neglect for Vertical Capacity[2] (feet)
Multi-Purpose Building	18	15 feet or 5 feet into QTsc	400 (in fill) 750 (in QTsc)	1½
Fitness Building/Dining Room Addition	16	6 feet or 5 feet into QTsc	500	1½
Memory Care Facility*	24	15 feet or 5 feet into QTsc	400 (in fill) 750 (in QTsc)	1½
Residential Villas & Duplexes	16	6 feet or 5 feet into QTsc	400 (in fill) 750 (in QTsc)	1½

[1] QTsc = Santa Clara Formation materials. Minimum depth or 5 feet into QTsc, whichever is deeper.

[2] Depth of soil to neglect below bottom of grade beam or lowest adjacent ground surface.

Based on our review of the anticipated building loads, total settlement of individual piers or pier groups of four or less should not exceed ½-inch to mobilize static capacities and post-construction differential settlement between adjacent piers should not exceed ¼-inch due to static loads.

7.3.2 Lateral Capacity

Lateral loads exerted on the structure may be resisted by a passive resistance based on an ultimate equivalent fluid pressure of 450 pcf acting against twice the projected area of piers below the pier cap or grade beam within pier groups of two or more and over two pier diameters for single piers, up to a maximum uniform pressure of 4,000 psf at depth. The upper 18 inches of soil should be neglected when determining lateral capacity. The structural engineer should apply an appropriate factor of safety to the ultimate passive pressures.

If further analysis is required, we should be retained to provide a lateral load analysis using the computer program L-Pile once final building loads and grading plans have been finalized.

7.3.3 Construction Considerations

The excavation of all drilled shafts should be observed by a Cornerstone representative to confirm the soil profile and that the piers are constructed in accordance with our recommendations and project requirements. The drilled shafts should be straight, dry, and relatively free of loose material before reinforcing steel is installed and concrete is placed. If ground water cannot be removed from the excavations prior to concrete placement, drilling slurry or casing may be required to stabilize the shaft and the concrete should be placed using a tremie pipe, keeping the tremie pipe below the surface of the concrete to avoid entrapment of water or drilling slurry in the concrete.

As previously mentioned, the material encountered in our borings generally consisted of very stiff to hard clays and medium dense to very dense clayey sands. Caving soils were not observed in our borings using hollow stem augers. If localized caving conditions are encountered during the excavation of drilled piers, the used of drilling slurry and/or temporary casing may be required.

7.4 SHALLOW FOUNDATIONS – MEMORY CARE FACILITY & VILLAS V61/62, V66-V85

7.4.1 Conventional Shallow Foundations

As summarized above, as an alternative to drilled piers and provided the building pads are constructed in accordance with the “Earthwork” section of this report, the Memory Care Facility and residential villas V61/62 and V66 through V85 may be supported on conventional shallow foundations. Cantilevered site retaining walls on level ground conditions may also be supported on shallow footings. Footings should bear on natural, undisturbed soil or engineered fill and be constructed to the depths and widths presented in the following table. For the Memory Care Facility, where localized fills are encountered or proposed within the building pad, shallow footings will need to be deepened to extend at least 12 inches into Santa Clara Formation (QTsc) materials. 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. The deeper footing embedment is due to the presence of moderately to highly expansive soils, and is intended to embed the footing below the zone of significant seasonal moisture fluctuation, reducing the potential for differential movement. Footings constructed to the dimensions below and in accordance with the “Earthwork” recommendations of this report would be capable of supporting the maximum allowable bearing pressures presented in the following table..

Table 9: Allowable Bearing Capacity for Conventional Footings

Building Location	Minimum Footing Width (inches)	Minimum Footing Depth (inches)	Allowable Bearing Capacity (psf)
Residential Villas (V61/62 and V66 through V85)	15	24	2,000 (Dead Loads) 3,000 (Dead+Live Loads) 4,000 (All Loads)
Memory Care Facility*	18	24*	3,000 (Dead Loads) 4,500 (Dead+Live Loads) 6,000 (All Loads)

*Assumes all footings for the Memory Care Facility bear in undisturbed Santa Clara Formation (QTsc) materials. Where localized fills are encountered or proposed to construct the building pad, shallow footings will need to be deepened to extend at least 12 inches into QTsc materials.

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.

7.4.2 Footing Settlement

Structural loads were provided by the project structural engineer, as summarized in the following table.

Table 10: Anticipated Structural Loading

Building Location	Foundation Area	Range of Assumed Loads
Residential Villas	Isolated Columns	10 to 20 kips
	Perimeter Walls	1 to 2 kips per lineal foot
Memory Care Facility	Interior Columns	100 to 300 kips
	Perimeter Walls	4 to 8 kips per lineal foot

Based on the above loading and the allowable bearing pressures presented above and provided building pads are constructed in accordance with the “Earthwork” section of this report, we estimate that the total static footing settlement will be on the order of ½ to ¾ inch, with about ¼ to ½ inch of post-construction differential settlement between adjacent foundation elements, assumed to be on the order of 30 feet.

7.4.3 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 0.4 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 18 inches of soil should be neglected when determining passive pressure capacity.

7.4.4 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.

7.5 DRILLED PIER FOUNDATIONS – SKILLED NURSING FACILITY ONLY

The proposed Skilled Nursing Facility should be supported on drilled, cast-in-place, straight-shaft friction piers deriving support from the underlying Santa Clara Formation soils. Drilled pier design parameters are presented in the following table. Adjacent piers should be spaced at least three diameters apart, otherwise, a reduction for group effects may be required.

7.5.1 Vertical Capacity and Estimated Settlement

The ultimate skin friction values presented below should be adjusted using a factor of safety no less than the overstrength factor of the structure supported per Section 1605A of the 2016 California Building Code. Piers should have a minimum depth of 8 feet below lowest adjacent grade or extend at least 5 feet into Santa Clara Formation (QTsc) materials, whichever is deeper. Piers should have a minimum diameter of 18 inches; the upper 18 inches of soil should be neglected when determining vertical pier capacity as measured from finished pad grade. Grade beams should extend at least 18 inches below the lowest adjacent grade.

Table 12: Design Criteria for Drilled Piers – Skilled Nursing Facility Only

Drilled Pier Depth (feet)	Ultimate Skin Friction[1] (psf)
1½ to 5	450
5 to 10	1,200
>10	2,100

[1] The factor of safety applied to the ultimate skin friction should be no less than the overstrength factor of the structure supported.

Based on our review of the anticipated building loads for the Skilled Nursing Facility, total settlement of individual piers or pier groups of four or less should not exceed ½ inch to mobilize static capacities and post-construction differential settlement between adjacent piers should not exceed ¼ inch due to static loads.

7.5.2 Lateral Capacity

Lateral loads exerted on the structure may be resisted by a passive resistance based on an ultimate equivalent fluid pressure of 450 pcf acting against twice the projected area of piers below the pier cap or grade beam within pier groups of two or more and over two pier diameters for single piers, up to a maximum uniform pressure of 4,000 psf at depth. The upper 18 inches of soil should be neglected when determining lateral capacity. The structural engineer should apply an appropriate factor of safety to the ultimate passive pressures.

If further analysis is required, we should be retained to provide a lateral load analysis using the computer program L-Pile once final building loads and grading plans have been finalized.

7.5.3 Construction Considerations

The excavation of all drilled shafts should be observed by a Cornerstone representative to confirm the soil profile and that the piers are constructed in accordance with our recommendations and project requirements. The drilled shafts should be straight, dry, and relatively free of loose material before reinforcing steel is installed and concrete is placed. If ground water cannot be removed from the excavations prior to concrete placement, drilling slurry or casing may be required to stabilize the shaft and the concrete should be placed using a tremie pipe, keeping the tremie pipe below the surface of the concrete to avoid entrapment of water or drilling slurry in the concrete.

As previously mentioned, the material encountered in our borings generally consisted of very stiff to hard clays and medium dense to very dense clayey sands. Caving soils were not observed in our borings using hollow stem augers. If localized caving conditions are encountered during the excavation of drilled piers, the used of drilling slurry and/or temporary casing may be required.

SECTION 8: CONCRETE SLABS AND PEDESTRIAN PAVEMENTS

8.1 INTERIOR SLABS-ON-GRADE

As the Plasticity Index (PI) of the surficial soils ranges up to 29, any proposed interior slabs-on-grade, including the Skilled Nursing Facility, should be supported on at least 12 inches of non-expansive fill (NEF) to reduce the potential for slab damage due to soil heave. The NEF layer should 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 3 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. For unreinforced concrete slabs, ACI 302.1R recommends limiting control joint spacing to 24 to 36 times the slab thickness in each direction, or a maximum of 18 feet.

8.2 PODIUM GARAGE SLABS-ON-GRADE – MEMORY CARE FACILITY

Prior to slab-on-grade construction, the Memory Care building pad should be over-excavated to reduce the potential for differential movement across the cut/fill transition in accordance with the "Earthwork" section of this report. The Memory Care garage slab-on-grade should be at least 5 inches thick and if constructed with minimal reinforcement intended for shrinkage control only, should have a minimum compressive strength of 3,000 psi. If the slab will have heavier reinforcing because the slab will also serve as a structural diaphragm, the compressive strength may be reduced to 2,500 psi at the structural engineer's discretion.

The garage slab should also be supported on at least 12 inches of non-expansive fill (NEF), the upper 4 inches of which should consist of either Class 2 aggregate base or ¾-inch clean, crushed rock place and compacted in accordance with the "Compaction" section of this report. If there will be areas within the garage that are moisture sensitive, such as equipment and elevator rooms, the recommendations in the "Interior Slabs Moisture Protection Considerations" section below may be incorporated in the project design if desired. 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.

8.3 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 capillary break rock may be considered as the upper 4 inches of the non-expansive fill previously recommended.

- 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.

8.4 EXTERIOR FLATWORK

8.4.1 Pedestrian Concrete 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 6 inches of non-expansive fill 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.

8.4.2 Pedestrian Pavers

Concrete unit pavers subject to pedestrian and/or occasional light pick up loading should be at least 60 mm thick and supported on at least 6 inches of Class 2 aggregate base overlying subgrade prepared in accordance with the "Earthwork" recommendations of this report. A maximum 1-inch-thick layer of sand may be used as a leveling/setting bed over the aggregate base. Pavers that will be subject to heavier or frequent vehicular loading should be designed in accordance with the recommendations in the "Vehicular Pavements" section below.

Where pavers will span transitions from on-grade to on-structure, consideration should be given to including a concrete sub-slab supported on the basement wall capable of spanning over the first 2 to 3 feet of wall backfill.

SECTION 9: VEHICULAR PAVEMENTS

9.1 ASPHALT CONCRETE

The following asphalt concrete pavement recommendations tabulated below are based on the Caltrans Highway Design Manual (latest edition), estimated traffic indices for various pavement-loading conditions, and on a subgrade design R-value of 15. The design R-value was chosen based on previous laboratory testing during the 1988 ESA investigation and engineering judgment considering the variable surface conditions.

Table 13: Asphalt Concrete Pavement Recommendations, Design R-value = 15

Design Traffic Index (TI)	Asphalt Concrete (inches)	Class 2 Aggregate Base* (inches)	Total Pavement Section Thickness (inches)
4.0	2.5	6.0	8.5
4.5	2.5	8.0	10.5
5.0	2.5	9.0	11.5
5.5	3.0	10.0	13.0
6.0	3.0	11.0	14.0
6.5	4.0	12.0	16.0

*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.

Asphalt concrete pavements constructed on expansive subgrade where the adjacent areas will not be irrigated for several months after the pavements are constructed may experience longitudinal cracking parallel to the pavement edge. These cracks typically form within a few feet of the pavement edge and are due to seasonal wetting and drying of the adjacent soil. The cracking may also occur during construction where the adjacent grade is allowed to significantly dry during the summer, pulling moisture out of the pavement subgrade. Any cracks that form should be sealed with bituminous sealant prior to the start of winter rains. One alternative to reduce the potential for this type of cracking is to install a moisture barrier at least 24 inches deep behind the pavement curb.

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). Recommendations for garage slabs-on-grade were provided in the “Concrete Slabs and Pedestrian Pavements” section above. 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 14: PCC Pavement Recommendations

Allowable ADTT	Minimum PCC Thickness (inches)
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 6 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. Due to the expansive surficial soils present, we recommend that the construction and expansion joints be dowelled.

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 of Portland Cement Concrete. We recommend that the trash enclosure pads and stress (landing) pads where garbage trucks will store, pick up, and empty trash be increased to a minimum PCC thickness of 8 inches underlain by at least 6 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 VEHICULAR CONCRETE UNIT PAVERS

Where vehicular concrete unit pavers are desired in standard traffic areas, we recommend that the pavers be underlain by a 6-inch-thick concrete sub-slab designed as discussed above, including the aggregate base section. Pavers should be placed on a bituminous or mortar setting bed over the concrete sub-slab. Where the pavers will be used as an emergency vehicle access (EVA), the pavers should be placed over at least 15 inches of Class 2 aggregate base and prepared subgrade as recommended in the "Earthwork" section. A maximum 1 inch thick sand setting bed may be used to level the pavers on the aggregate base.

9.4 PAVEMENT CUTOFF

Surface water penetration into the pavement section can significantly reduce the pavement life, due to the native expansive clays. 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 15: 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

Basement walls should be designed as restrained walls. 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.

10.2 SEISMIC LATERAL EARTH PRESSURES

10.2.1 Basement Walls – Memory Care Facility Only

No retaining walls are planned for the proposed Skilled Nursing Facility; therefore, the following recommendations apply only to the basement walls for the Memory Care building. The 2016 CBC states that lateral pressures from earthquakes should be considered in the design of basements and site retaining walls. We reviewed the seismic earth pressures for the proposed basement of the Memory Care facility using procedures generally based on the Mononobe-Okabe method (Lew et al., SEAOC 2010). Because the basement walls are anticipated to be up to 16 feet in height, and peak ground accelerations are greater than 0.40g, we checked the result of the seismic increment when added to the recommended active earth pressure against the recommended fixed wall earth pressures. Because the wall is restrained, or will act as a restrained wall, and will be designed for 45 pcf (equivalent fluid pressure) plus a uniform earth pressure of 8H psf, based on current recommendations for seismic earth pressures, it appears that active earth pressures plus a seismic increment do not exceed the fixed wall earth pressures. Therefore, an additional seismic increment above the design earth pressures is not required as long as the walls are designed for the restrained wall earth pressures recommended above.

10.2.1 Site Walls

We also reviewed the anticipated cantilevered site walls (unrestrained) that will range from up to 8 feet high. The 2016 CBC states that lateral pressures from earthquakes should be considered in the design of basements and retaining walls. Because walls greater than about 6 feet are planned, and peak ground accelerations greater than 0.40g are expected, we recommend checking the walls for the seismic condition in accordance with the interim recommendations of the above referenced paper and the 2016 CBC. Wall less than 6 feet high will not require an additional seismic lateral force be applied to the wall design.

The CBC prescribes basic load combinations for structures, components and foundations with the intention that their design strength equals or exceeds the effects of the factored loads. With respect to the load from lateral earth pressure and ground water pressure, the CBC prescribes the basic combinations shown in CBC equations 16-2 and 16-7 below.

$1.2(D + F) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$ [Eq. 16-2]

In Eq. 16-2: H - should represent the total static lateral earth pressure, which for the site walls will be unrestrained (use 45 pcf)

$0.9(D + F) + 1.0E + 1.6H$ [Eq. 16-7]

In Eq. 16-7: H - should represent the static “active” earth pressure component under seismic loading conditions (use 45 pcf)

E - should represent the seismic increment component in Eq. 16-7, a triangular load with a resultant force of $10H^2$, which should be applied one third of the height up from the base of the wall.

The interim recommendations in the SEAOC paper more appropriately split out "active" earth pressure from the seismic earth pressure increment so that different load factors can be applied in accordance with different risk levels.

10.2 WALL DRAINAGE

10.2.1 At-Grade Site Walls

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.

10.2.2 Below-Grade Walls

Miradrain, AmerDrain or other equivalent drainage matting should be used for wall drainage where below-grade walls are temporarily shored and the shoring will be flush with the back of the permanent walls. The drainage panel should be connected at the base of the wall by a horizontal drainage strip and 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 unless capped by hardscape. The drainage panel filter fabric should be extended over the top of and behind the panel to protect it from intrusion of the adjacent soil. If the shoring system will be offset behind the back of permanent wall, the drainage systems discussed in the “At-Grade Site Walls” section may also be used.

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.

As discussed previously, consideration should be given to the transitions from on-grade to on-structure. Providing subslabs or other methods for reducing differential movement of flatwork or pavements across this transition should be included in the project design.

10.4 FOUNDATIONS

Basement retaining walls for the Memory Care facility may be supported on drilled piers or conventional footings designed in accordance with the recommendations presented in the “Foundations” section of this report. Unrestrained (cantilevered) site retaining walls may be supported on drilled piers or a continuous strip footing as presented in the “Foundations” section.

SECTION 11: LIMITATIONS

This report, an instrument of professional service, has been prepared for the sole use of The Forum at Rancho San Antonio specifically to support the design of new facilities at The Forum at Rancho San Antonio project in Cupertino, 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.

The Forum at Rancho San Antonio may have provided Cornerstone with plans, reports and other documents prepared by others. The Forum at Rancho San Antonio 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

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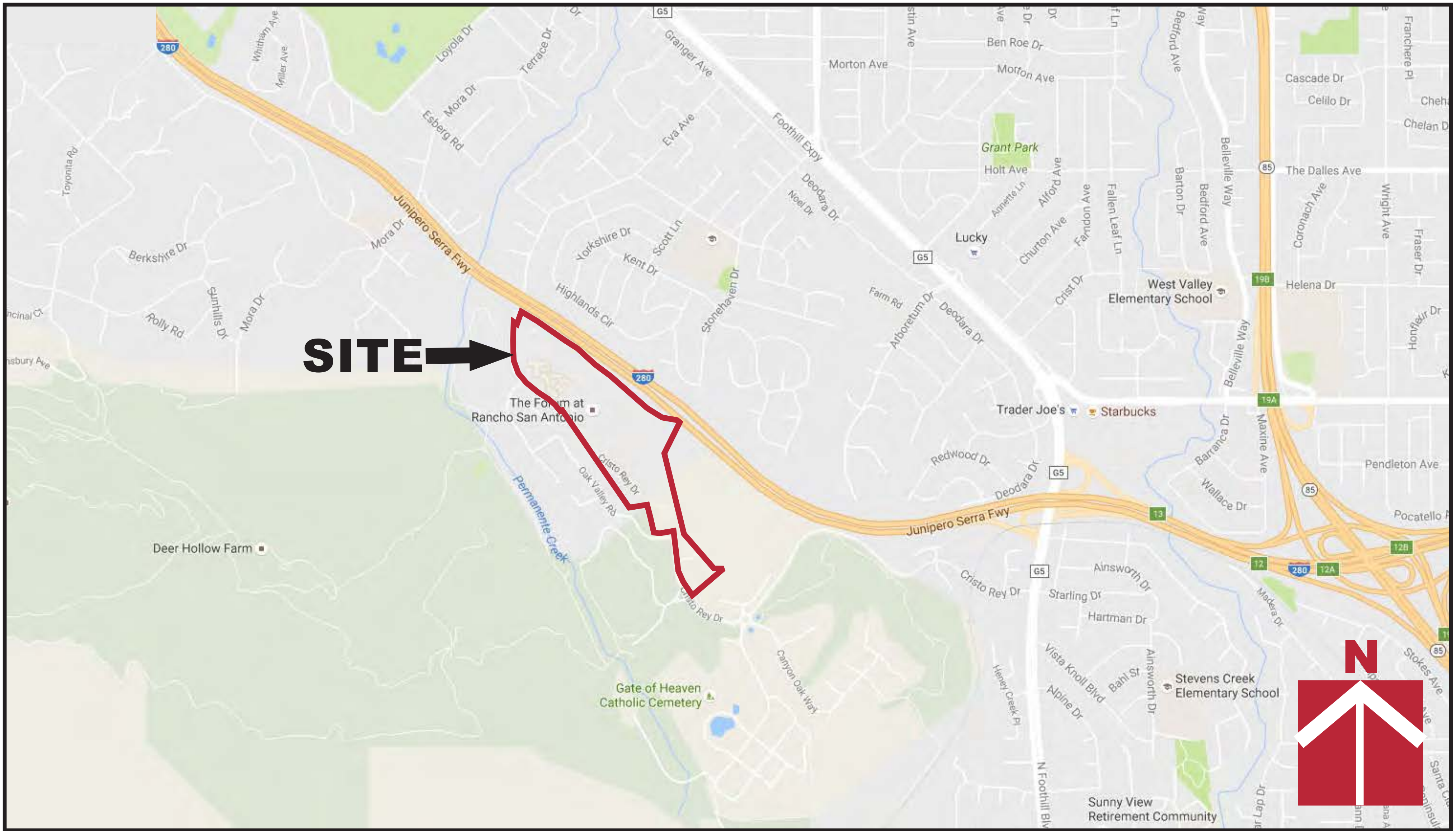
Aerial Photographs

Date	**Type**	**Approximate Scale**	**Identification**
8-23-1950	b&w	1:7,200	AV-43-03-6,7,8; 04-6,7,8
4-28-1974	b&w	1:12,000	AV-1138-03-12,13
4-28-1982	b&w	1:20,000	AV-2135-03-13,14
11-14-1994	b&w	1:20,000	AV-4625-0218-13, 14
7-17-2004	b&w	1:7,200	AV-8769-1-2,3,4
3-16-2005	b&w	1:10,000	KAV 9010-109-5,6,7



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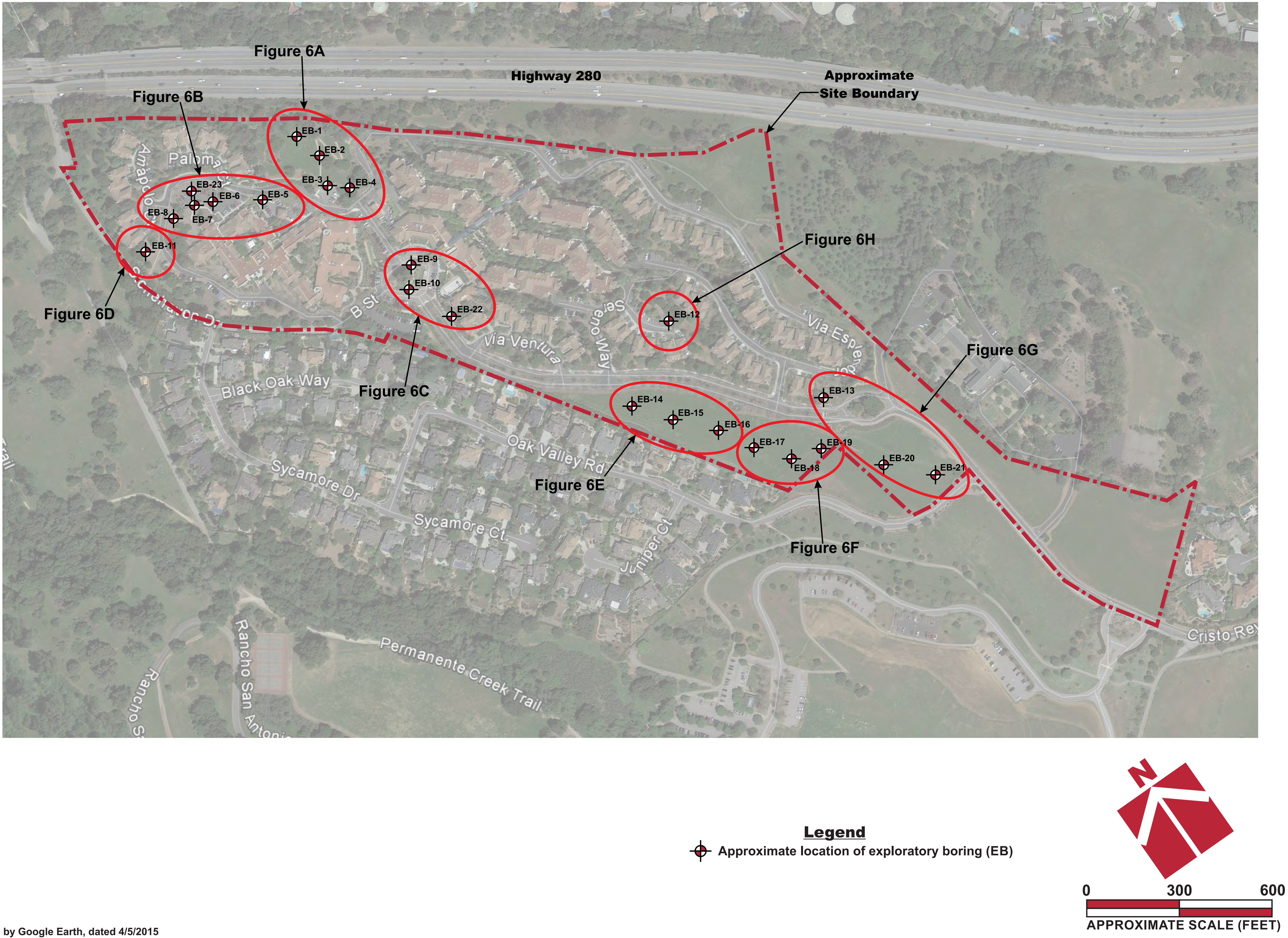
Vicinity Map

The Forum at Rancho San Antonio
Cupertino, CA

Project Number	905-1-1
Figure Number	Figure 1
Date	March 2017
Drawn By	RRN



Legend

● Approximate location of exploratory boring (EB)

Base by Google Earth, dated 4/5/2015

Project Number
905-1-1

Figure Number
Figure 2

Drawn By
RRN

Date
March 2017

Site Exploration Plan

The Forum at Rancho San Antonio
Cupertino, CA

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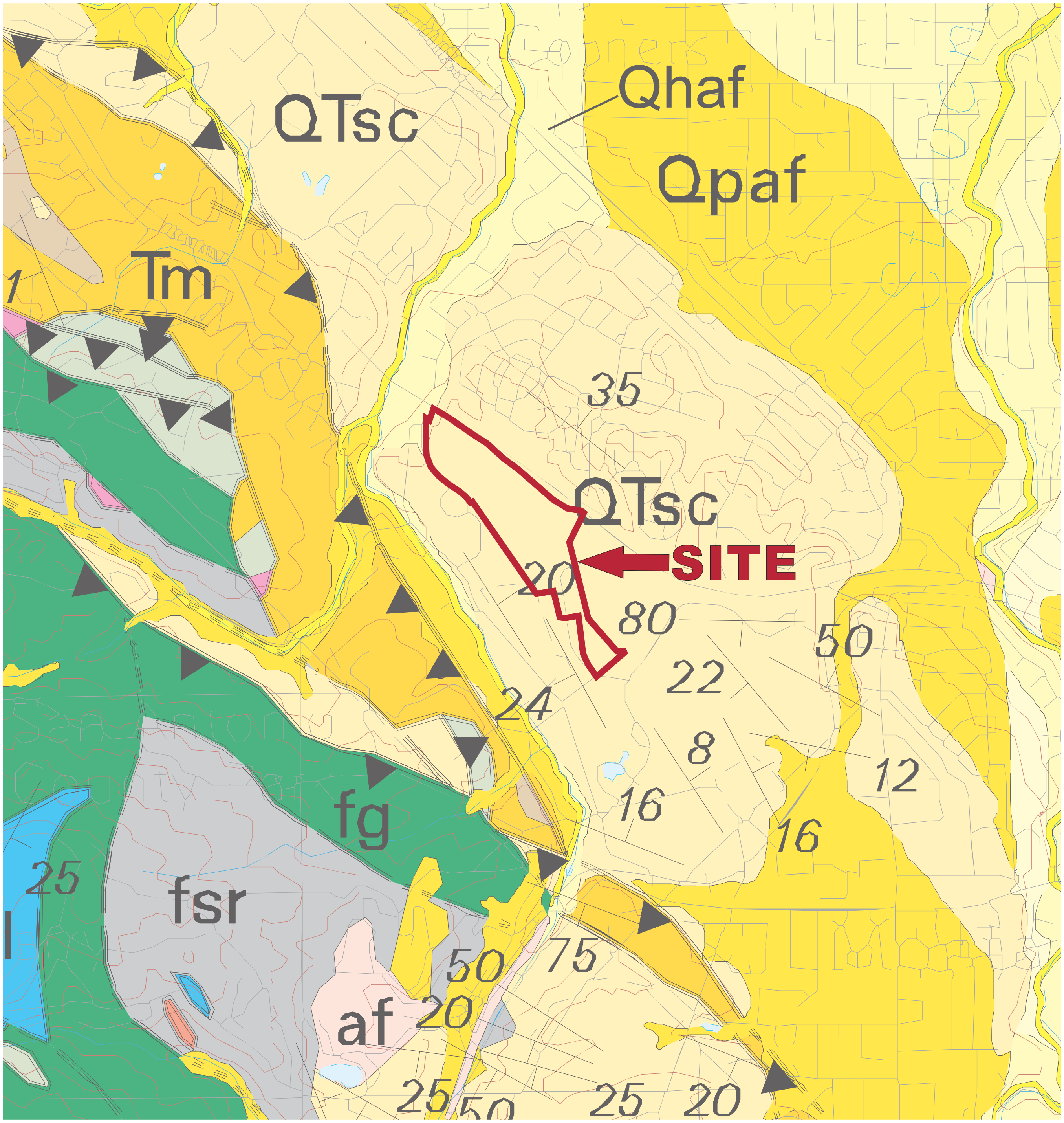

Geologic Units

QTsc	Santa Clara Formation (lower Pleistocene and upper Pliocene)
Qhaf	Alluvial fan deposits (Holocene)
Qpaf	Alluvial fan deposits (Pleistocene)

Explanation

Contact- dashed where approximate, dotted where concealed

N

0 2000 4000

APPROXIMATE SCALE (FEET)

Base: USGS, Geology of the Palo Alto 30x60 Minute Quadrangle, California, by Brabb, Graymer, and Jones, 1998

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Regional Geologic Map

The Forum at Rancho San Antonio
Cupertino, CA

Project Number
905-1-1

Figure Number
Figure 3

Date
March 2017

Drawn By
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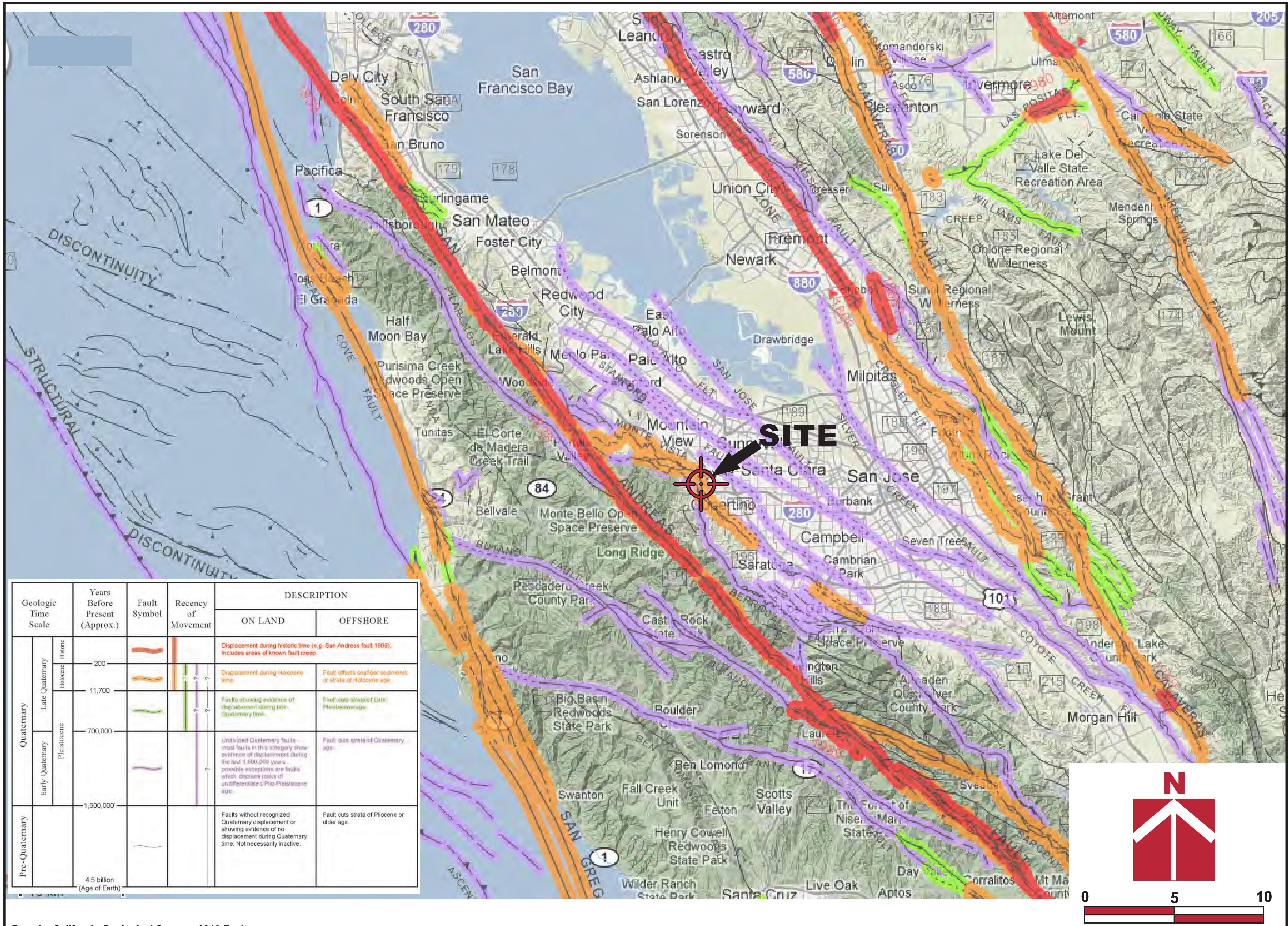

Geologic Time Scale			Years Before Present (Approx.)	Fault Symbol	Recency of Movement	DESCRIPTION	
						ON LAND	OFFSHORE
Quaternary	Late Quaternary	Historic		🟥	🟥	Displacement during historic time (e.g. San Andreas fault 1906). Includes areas of known fault creep.	
			200				
		Holocene		🟧	🟧 ? ? ?	Displacement 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			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
905-1-1

Figure Number
Figure 4

Drawn By
RRN

Date
March 2017

Regional Fault Map

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Cupertino, CA

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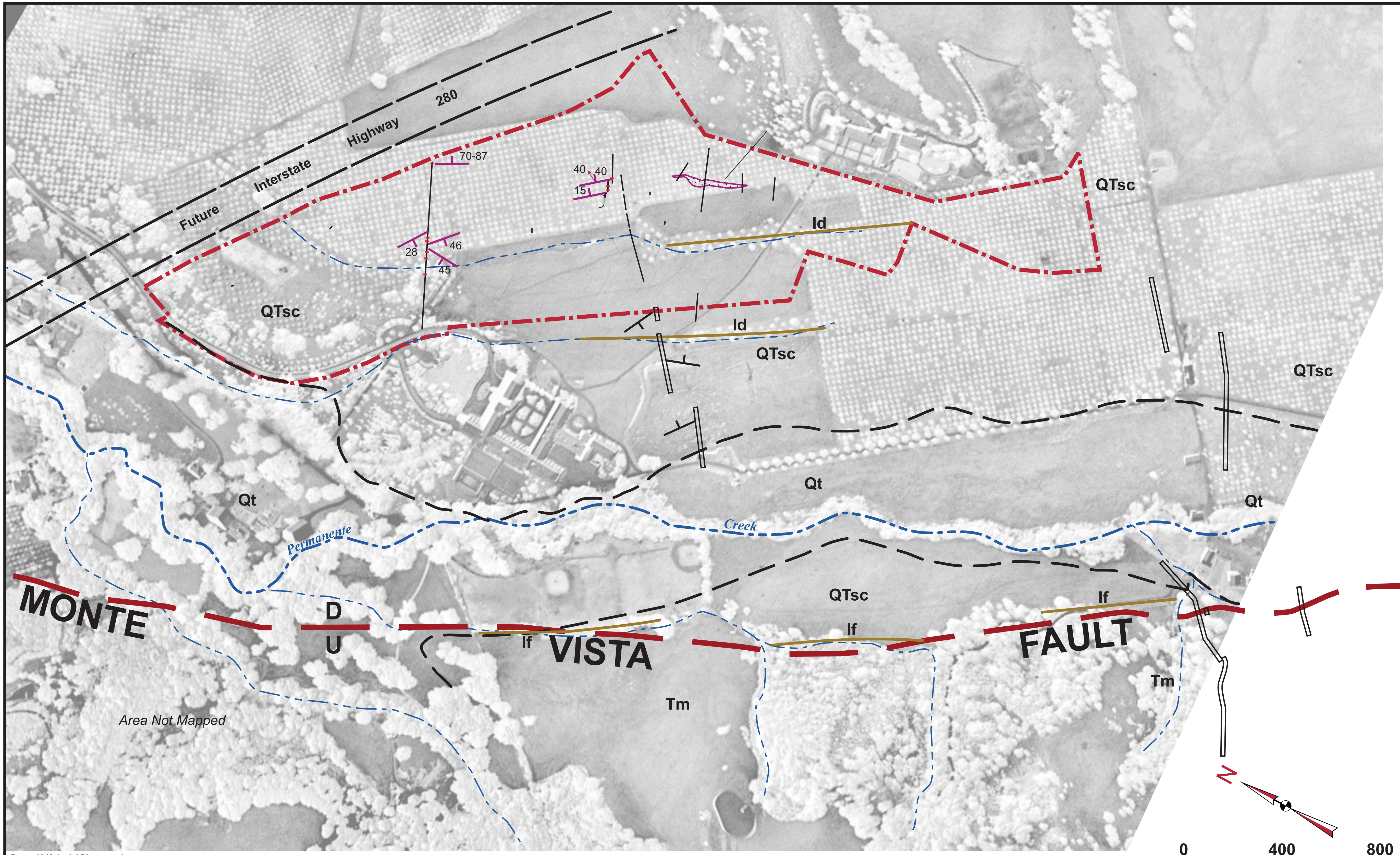

Base: 1948 Aerial Photograph

EXPLANATION

Qt Terrace deposits

QTsc Santa Clara Fm.

Tm Monterey Fm.

Contact, dashed where approximately located

D **U** Fault trace, dashed where approximately located. U on up-thrown block, D on down-thrown block.

Id, If Geomorphic lineaments; Id = linear drainage, If = linear front

12 Strike and dip of bedding

40 40 Strike and dip of shears, arrow indicates trend and plunge of striation

Exploratory trench (ESA, 1985)

Exploratory trench (ESC, 1991)

Property Boundary

Drainage axis

Project Number 905-1-1

Figure Number Figure 5A

Drawn By GDG, RRN

Date March 2017

Vicinity Geology Map (Pre-Development)

The Forum at Rancho San Antonio
Cupertino, CA

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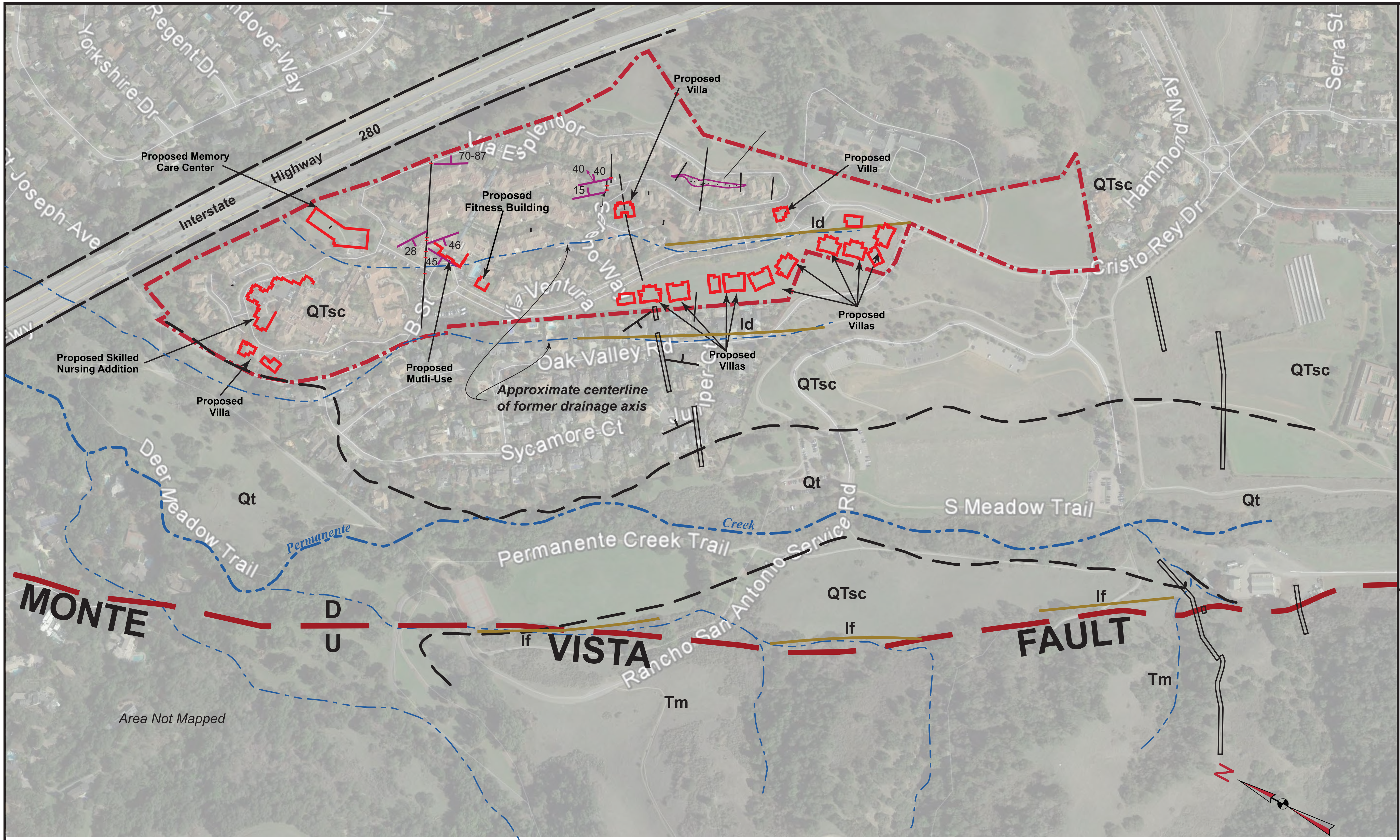

Base: 2016 Aerial Photograph

0 400 800
APPROXIMATE SCALE (FEET)

EXPLANATION

Qt	Terrace deposits
QTsc	Santa Clara Fm.
Tm	Monterey Fm.
⁀	Contact, dashed where approximately located
D / U	Fault trace, dashed where approximately located. U on up-thrown block, D on down-thrown block.
Id, If	Geomorphic linaments; Id = linear drainage, If = linear front
12	Strike and dip of bedding
40 40	Strike and dip of shears, arrow indicates trend and plunge of striation
├──┤	Exploratory trench (ESA, 1985)
▭	Exploratory trench (ESC, 1991)
▬ ▪ ▬	Property Boundary
- · -	Drainage axis
▭	Proposed structures

Project Number
905-1-1

Figure Number
Figure 5B

Drawn By GDG, RRN

Date March 2017

Vicinity Geology Map (Current Development)

The Forum at Rancho San Antonio
Cupertino, CA

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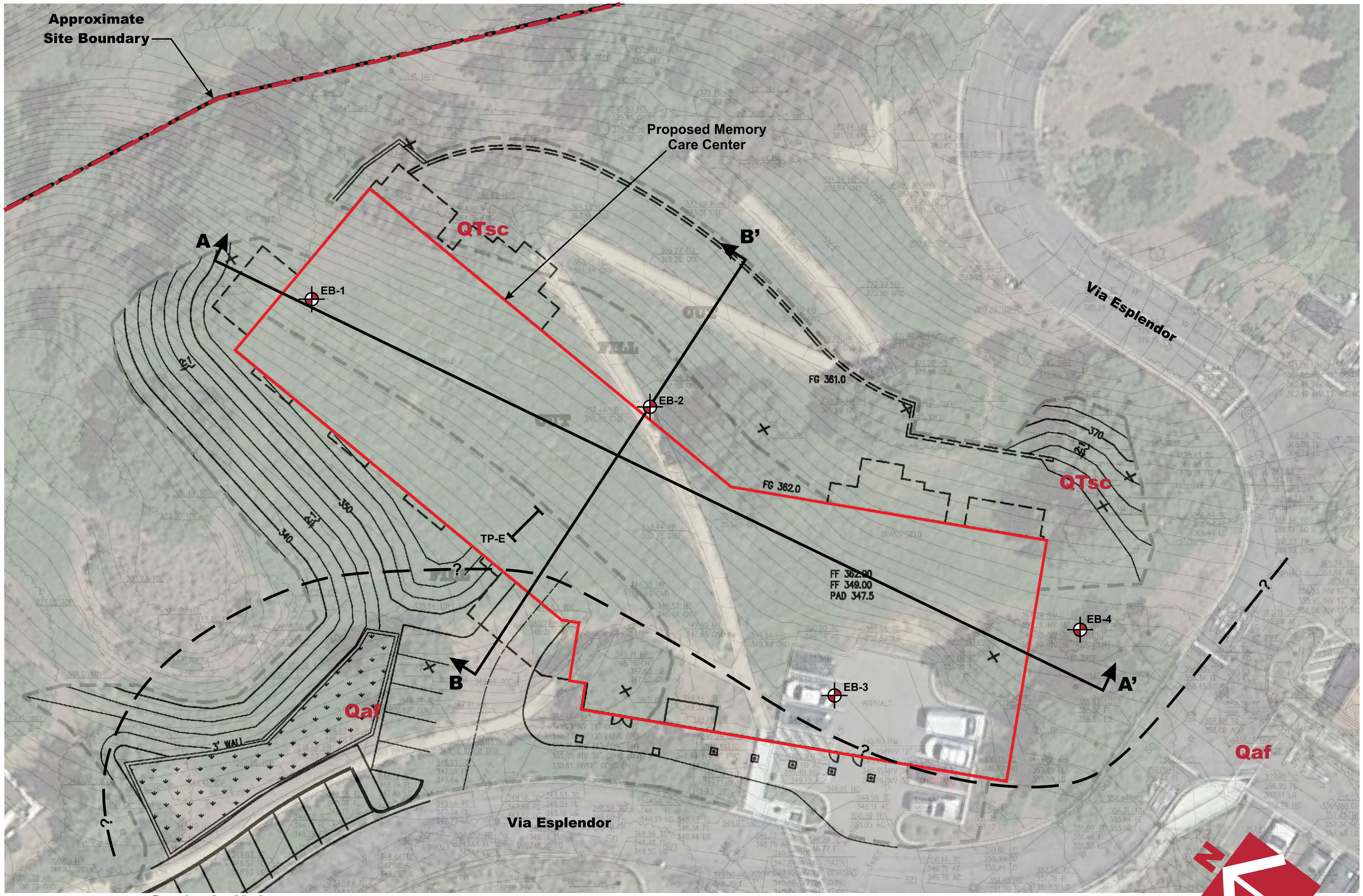

Geologic Units

QTsc	Santa Clara Formation (lower Pleistocene and upper Pliocene)
Qaf	Artificial fill (Man-made)

Legend

⊕ Approximate location of exploratory boring (EB)

TP-E ⊢ Approximate location of exploratory test pit (TP) (ESA, 1985)

A A' Approximate location of geologic cross section

—?— Approximate geologic contact (dashed where approximate; queried where uncertain)

0 30 60
APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 4/5/2015
Overlay by BKF Engineers, Memory Care - Grading Plan - C4.2, 1/31/2017

Project Number
905-1-1

Figure Number
Figure 6A

Drawn By
RRN

Date
March 2017

Site Plan and Geologic Map - Memory Care Center

The Forum at Rancho San Antonio
Cupertino, CA

CORNERSTONE
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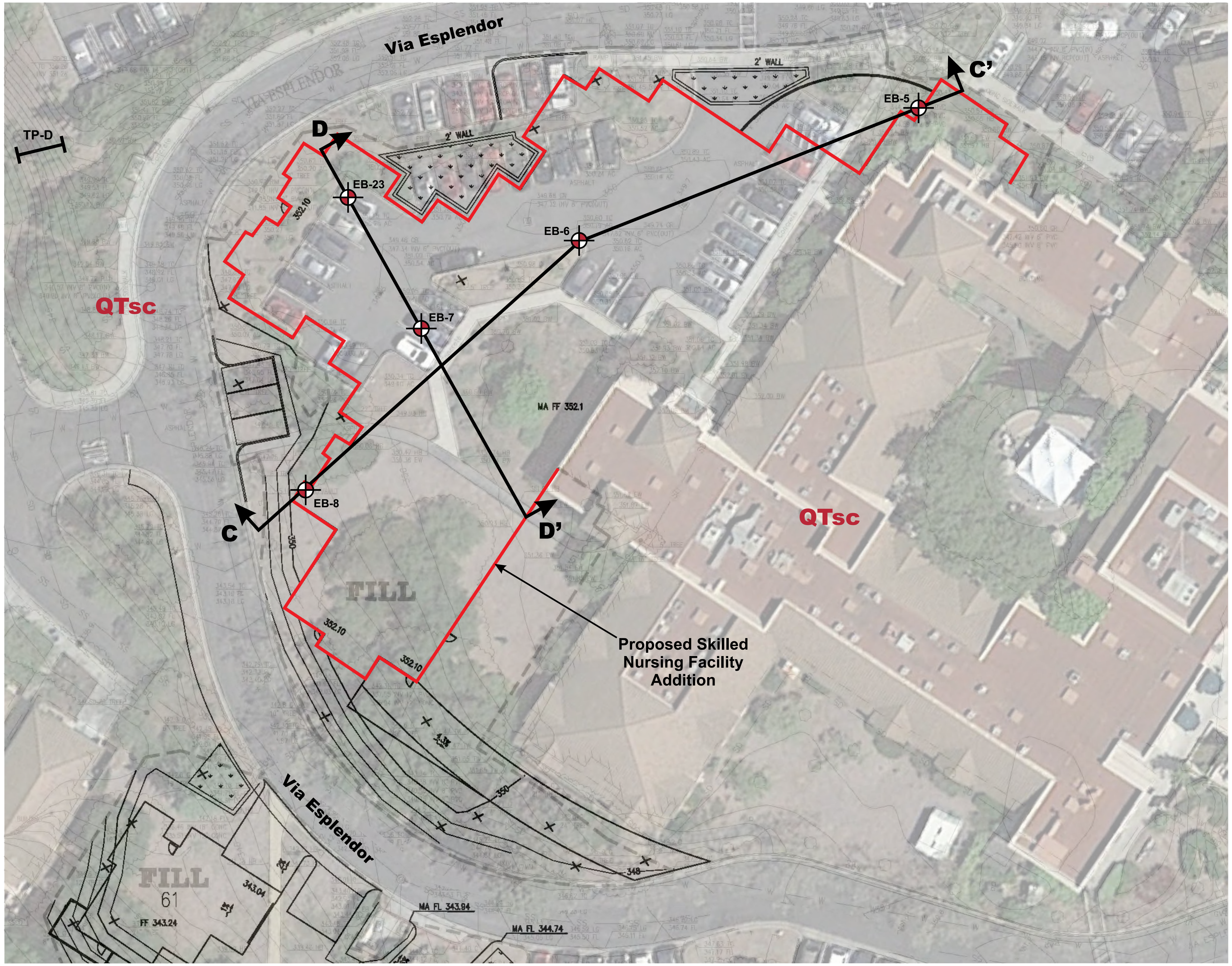

Geologic Units

QTsc Santa Clara Formation (lower Pleistocene and upper Pliocene)

Legend

⊕ Approximate location of exploratory boring (EB)

TP-D ⊢⊣ Approximate location of exploratory test pit (TP) (ESA, 1985)

C C' Approximate location of geologic cross section

0 40 80

APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 4/5/2015
Overlay by BKF Engineers, Skilled Nursing Addition - Grading Plan - C4.1, 1/31/2017

Project Number 905-1-1

Figure Number Figure 6B

Date March 2017

Drawn By RRN

Site Plan and Geologic Map - Skilled Nursing Facility Addition

The Forum at Rancho San Antonio
Cupertino, CA

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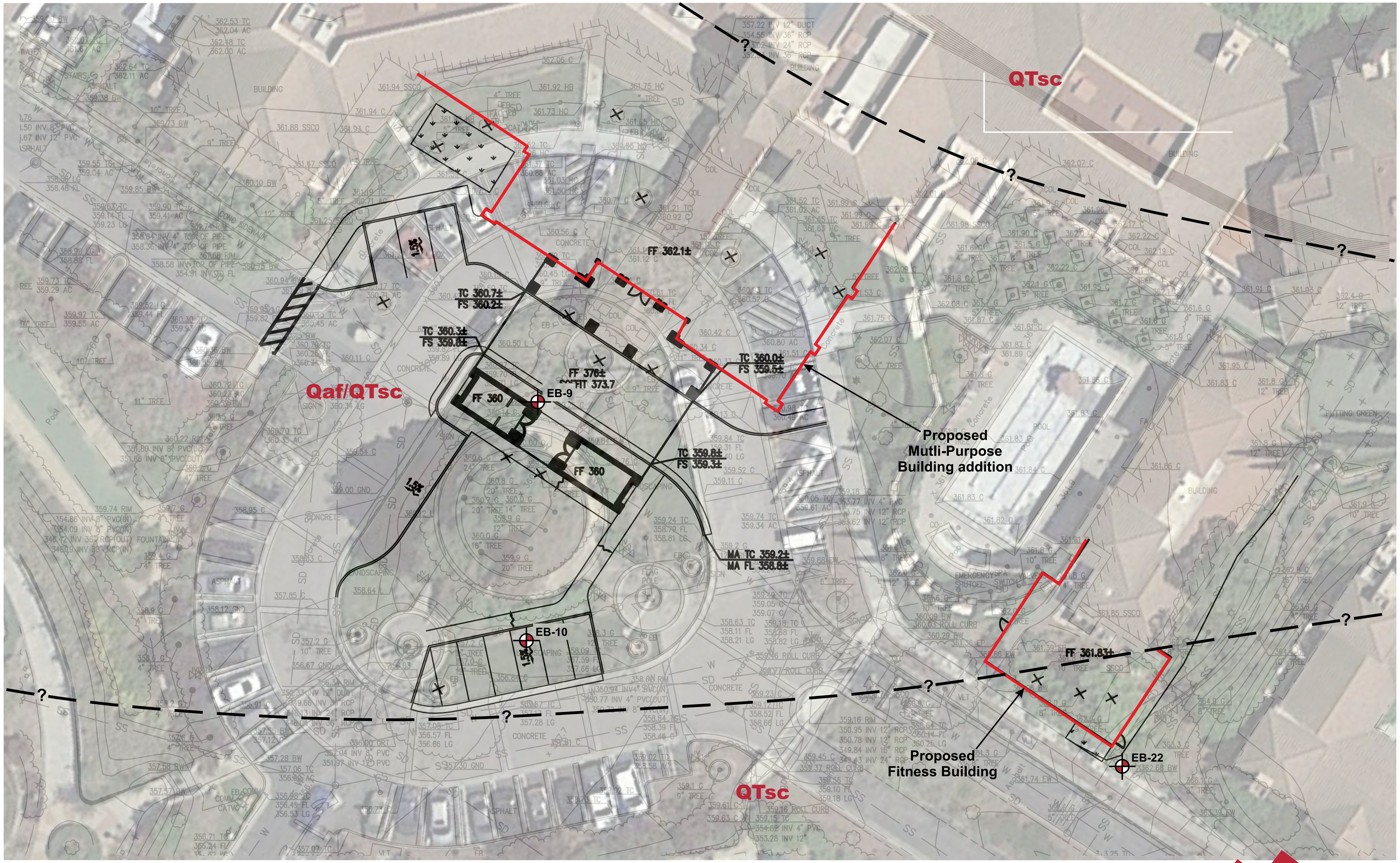

Geologic Units

QTsc Santa Clara Formation (lower Pleistocene and upper Pliocene)

Qaf Artificial fill (Man-made)

Legend

⊕ Approximate location of exploratory boring (EB)

—?— Approximate geologic contact (dashed where approximate; queried where uncertain)

0 30 60
APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 4/5/2015
Overlay by BKF Engineers, Dining Renovation and Multipurpose Building - Grading Plan - Grading Plan - C4.6, dated 12/16/2016

Project Number 905-1-1
Figure Number Figure 6C
Drawn By RRN
Date March 2017

Site Plan and Geologic Map - Mutli-purpose and Fitness Buildings

The Forum at Rancho San Antonio
Cupertino, CA

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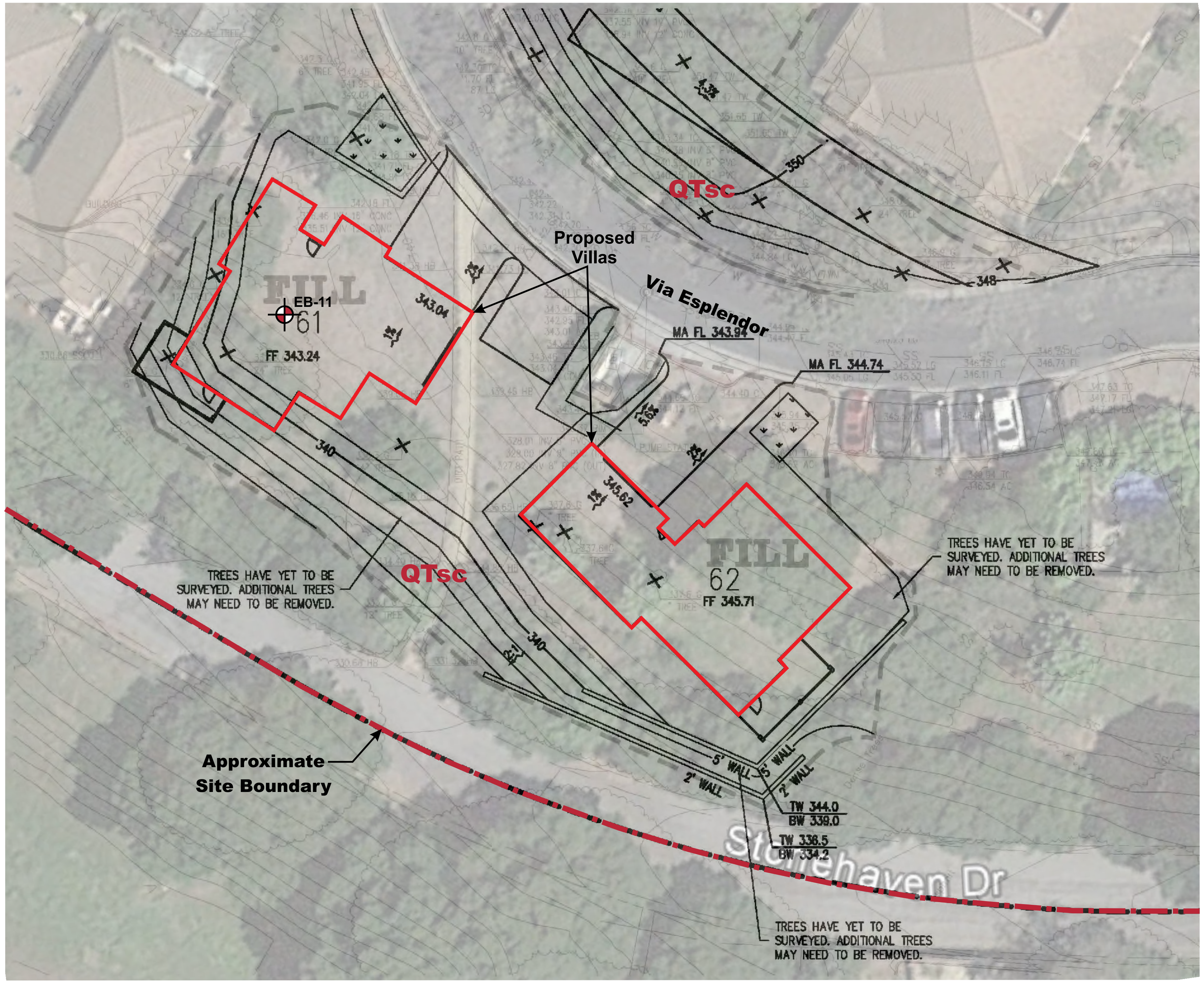

Geologic Units

QTsc Santa Clara Formation (lower Pleistocene and upper Pliocene)

Legend

⊕ Approximate location of exploratory boring (EB)

0 30 60

APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 4/5/2015
Overlay by BKF Engineers, Villas - Grading Plan - C4.7, dated 1/31/2017

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Site Plan and Geologic Map - Villas at Via Esplendor

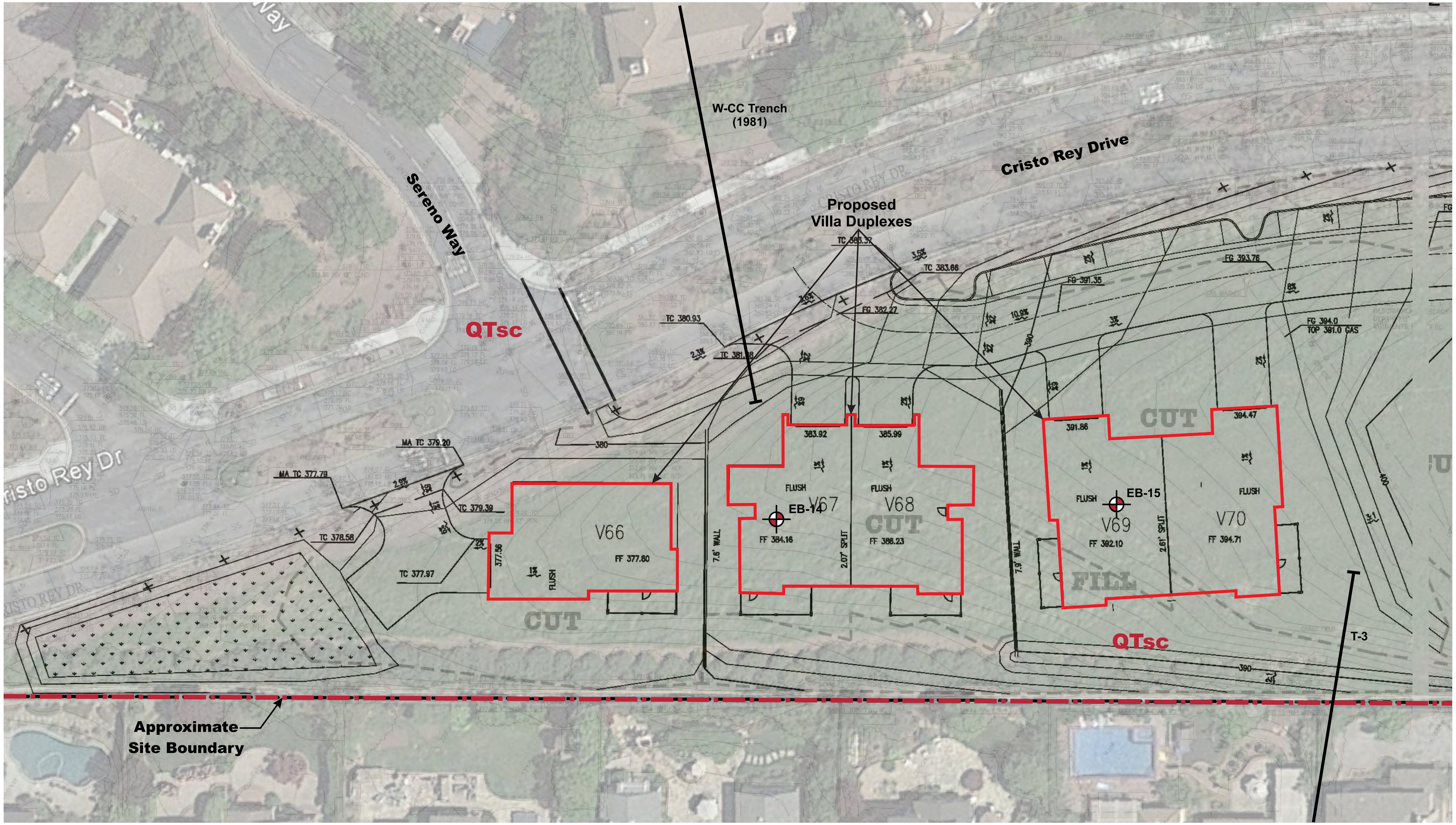
The Forum at Rancho San Antonio
Cupertino, CA

Project Number 905-1-1

Figure Number Figure 6D

Date March 2017

Drawn By RRN



Geologic Units

QTsc Santa Clara Formation (lower Pleistocene and upper Pliocene)

Legend

⊕ Approximate location of exploratory boring (EB)

T-1 Approximate location of exploratory trench (T) (ESA, 1985)

0 40 80
APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 4/5/2015
Overlay by BKF Engineers, Villas - Grading Plan - C-4.3, 1/31/2017

Project Number
905-1-1

Figure Number
Figure 6E

Drawn By
RRN

Date
March 2017

Site Plan and Geologic Map - Villa Duplexes at Cristo Rey Drive

The Forum at Rancho San Antonio
Cupertino, CA

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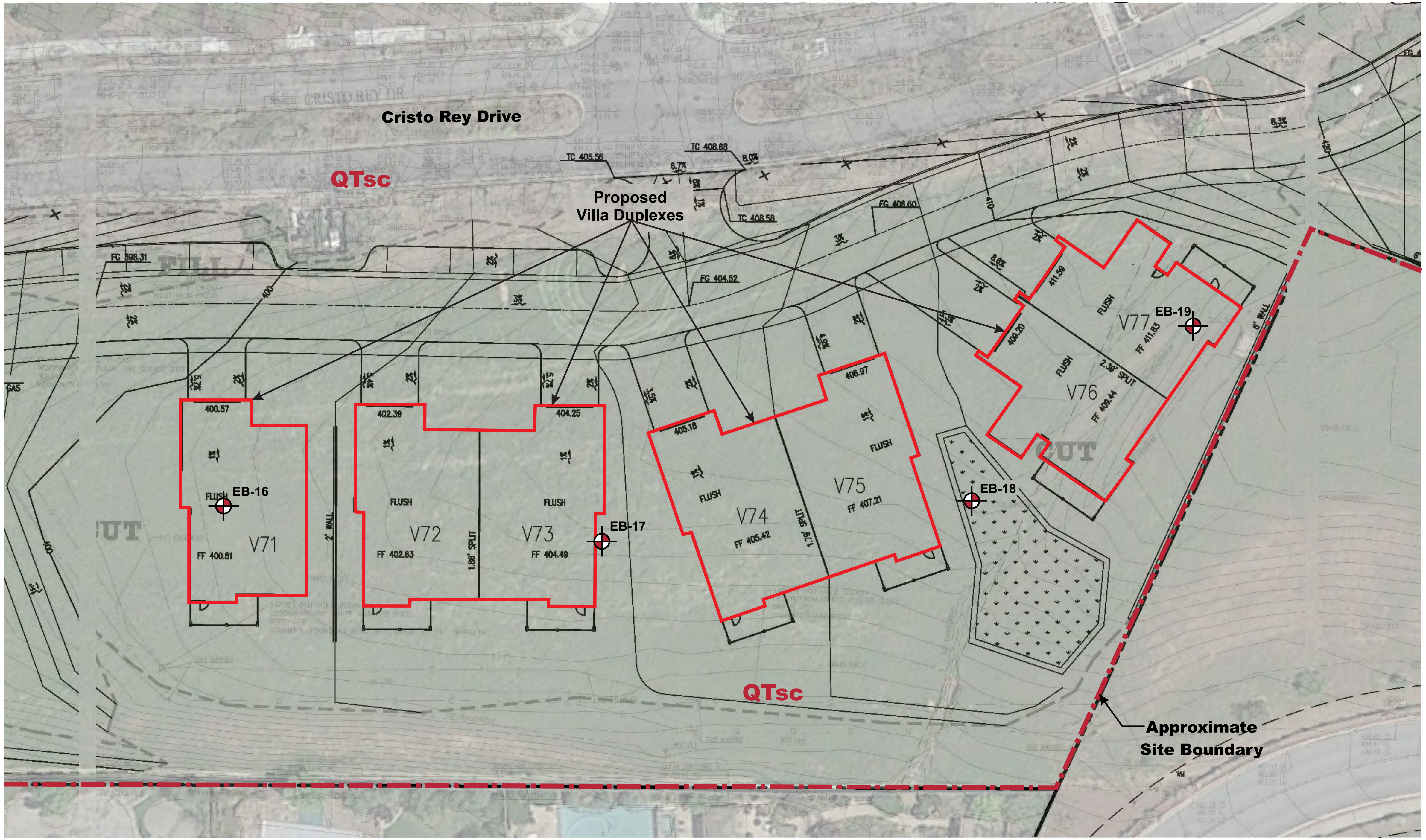


Geologic Units

QTsc	Santa Clara Formation (lower Pleistocene and upper Pliocene)

Legend

⊕ Approximate location of exploratory boring (EB)

0 40 80
APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 4/5/2015
Overlay by BKF Engineers, Villas - Grading Plan - C-4.4, 1/31/2017

Project Number
905-1-1

Figure Number
Figure 6F

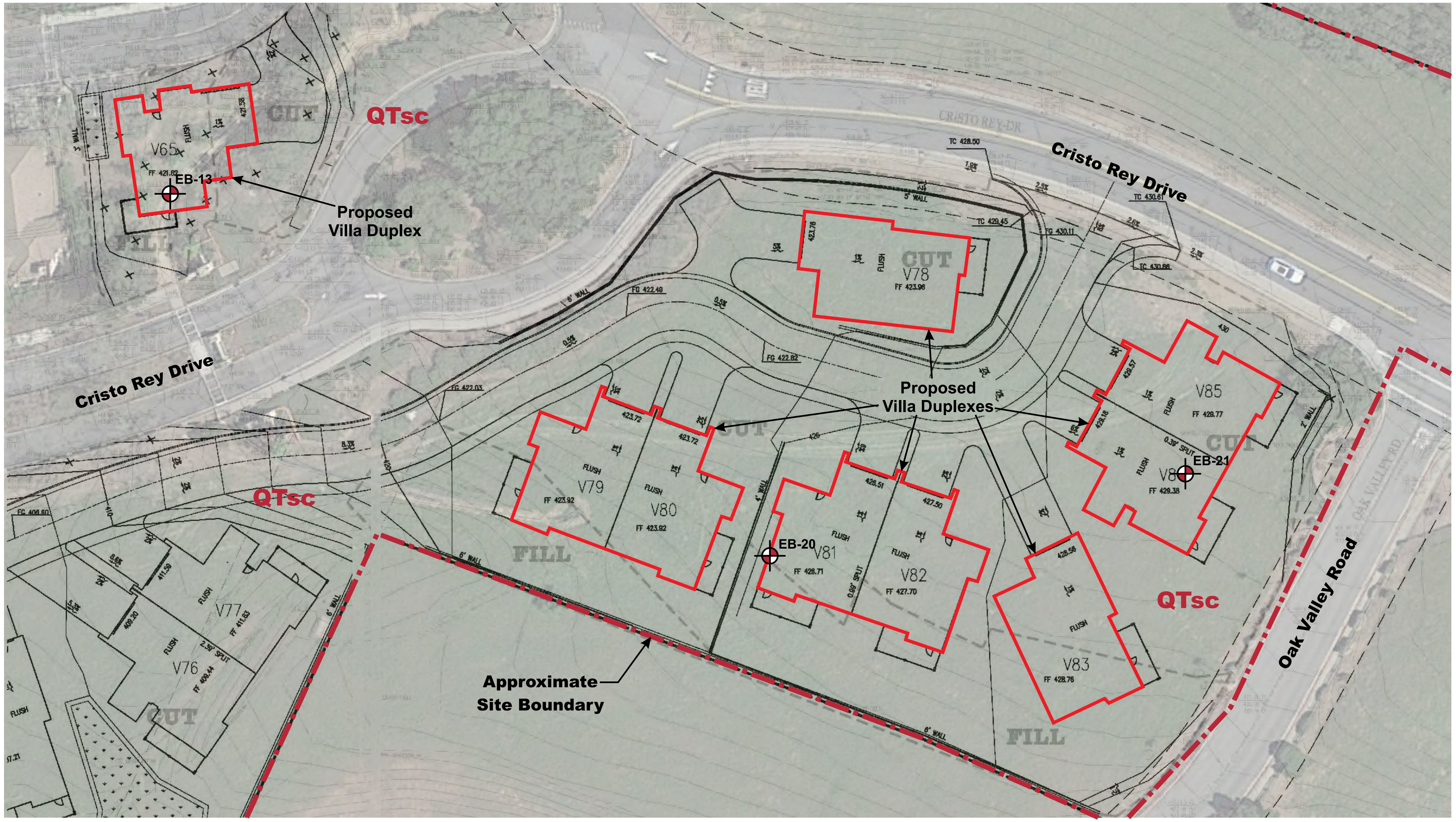
Drawn By
RRN

Date
March 2017

Site Plan and Geologic Map - Villa Duplexes at Cristo Rey Drive

The Forum at Rancho San Antonio
Cupertino, CA

CORNERSTONE
EARTH GROUP



Geologic Units

QTsc Santa Clara Formation (lower Pleistocene and upper Pliocene)

Legend

⌖ Approximate location of exploratory boring (EB)

0 50 100
APPROXIMATE SCALE (FEET)

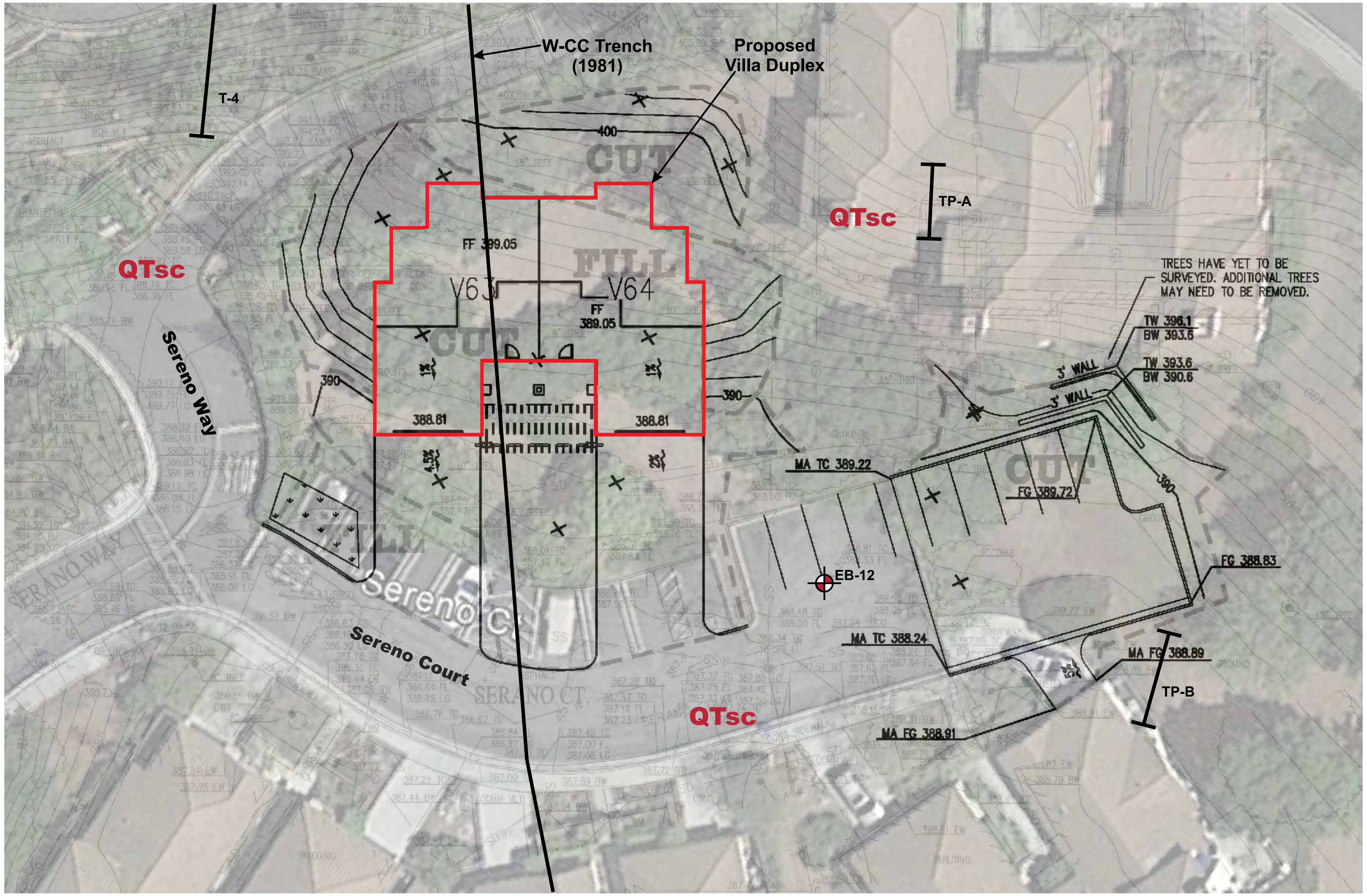
Base by Google Earth, dated 4/5/2015
Overlay by BKF Engineers, Villas - Grading Plan - C4.5, dated 1/31/2017

Project Number 905-1-1
Figure Number Figure 6G
Date March 2017
Drawn By RRN

Site Plan and Geologic Map - Villa Duplexes at Cristo Rey Drive

The Forum at Rancho San Antonio
Cupertino, CA

CORNERSTONE
EARTH GROUP



Geologic Units

QTsc Santa Clara Formation (lower Pleistocene and upper Pliocene)

Legend

⊕ Approximate location of exploratory boring (EB)

T-1 ├──┤ Approximate location of exploratory trench (T) (ESA, 1985)

TP-A ├─┤ Approximate location of exploratory test pit (TP) (ESA, 1985)

0 30 60
APPROXIMATE SCALE (FEET)

Base by Google Earth, dated 4/5/2015
Overlay by BKF Engineers, Villas - Grading Plan - C4.7, dated 1/31/2017

Project Number 905-1-1

Figure Number Figure 6H

Drawn By RRN

Date March 2017

Site Plan and Geologic Map - Villa Duplex at Sereno Court

The Forum at Rancho San Antonio
Cupertino, CA

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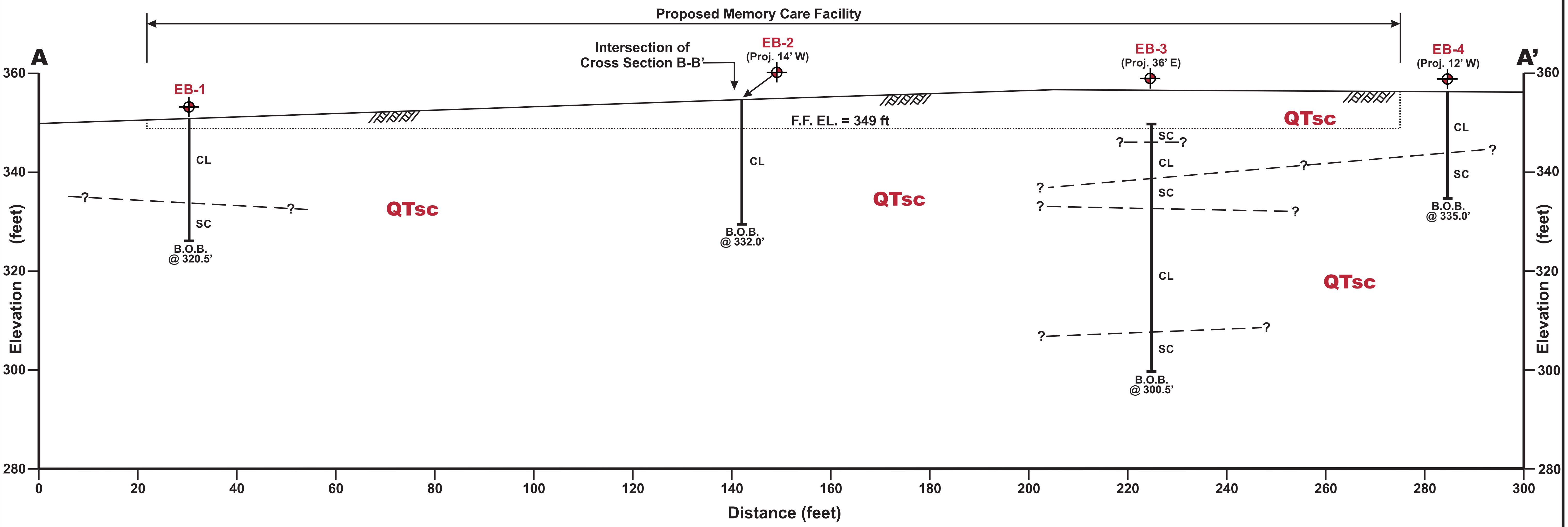

Section A-A'

(View Looking East)
1"=20' H:V

Notes:
1) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
2) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced borings. Actual subsurface conditions may vary significantly between borings.
3) See Figure 6A for location of cross section.

Explanation

Geologic Units		**Symbols**	
QTsc	Santa Clara Formation (lower Pleistocene and upper Pliocene)	CL	Lean Clay
		SC	Clayey Sand
		⊕	Approximate location of exploratory boring (EB)

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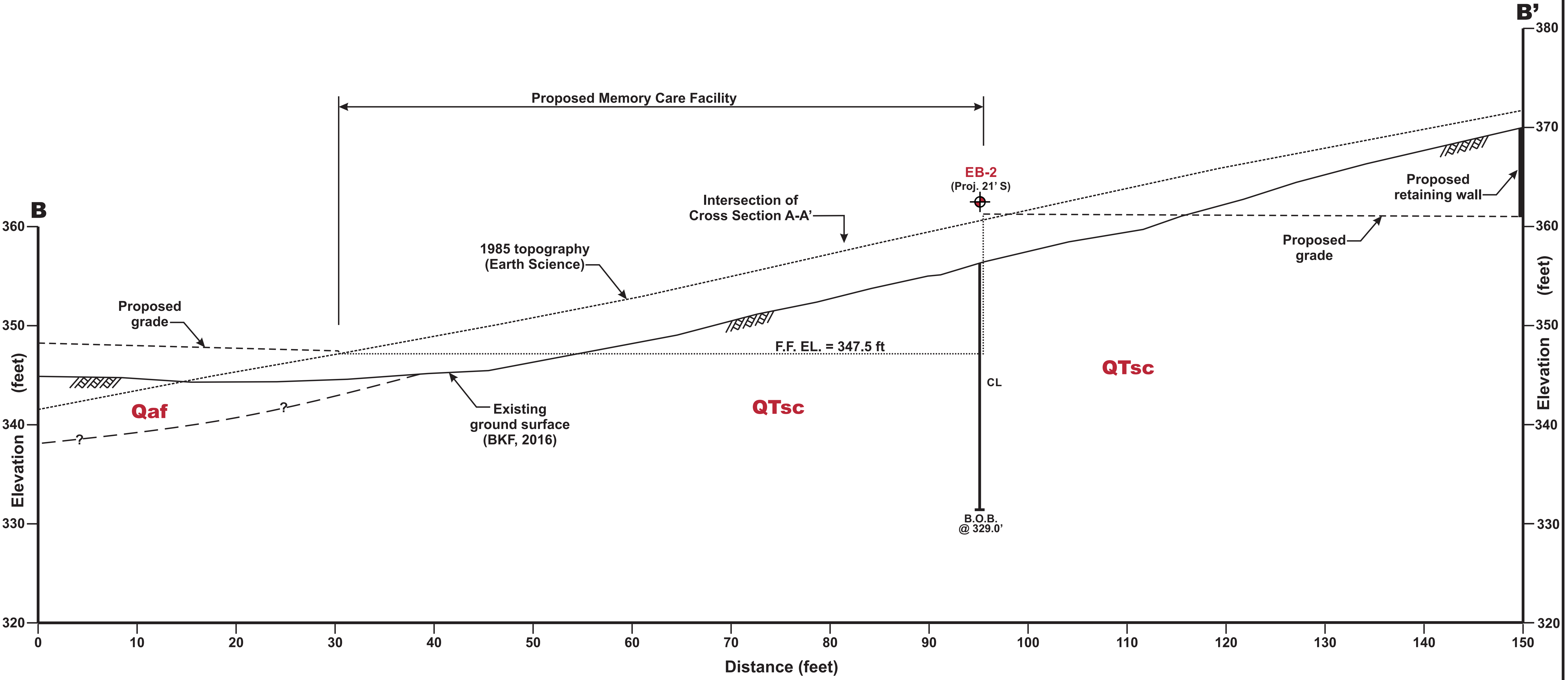
Geologic Cross Section A-A' - Memory Care Facility

The Forum at Rancho San Antonio
Cupertino, CA

Project Number 905-1-1
Figure Number Figure 7A
Date March 2017
Drawn By RRN



Section B-B'

(View Looking North)
1"=10' H:V

Notes:
1) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
2) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced borings. Actual subsurface conditions may vary significantly between borings.
3) See Figure 6A for location of cross section.

Explanation

Geologic Units

QTsc	Santa Clara Formation (lower Pleistocene and upper Pliocene)
Qaf	Articifial (man-made) fill

Symbols

CL	Lean Clay or sandy clay
⊕	Approximate location of exploratory boring (EB)

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EARTH GROUP

Geologic Cross Section B-B' - Memory Care Facility

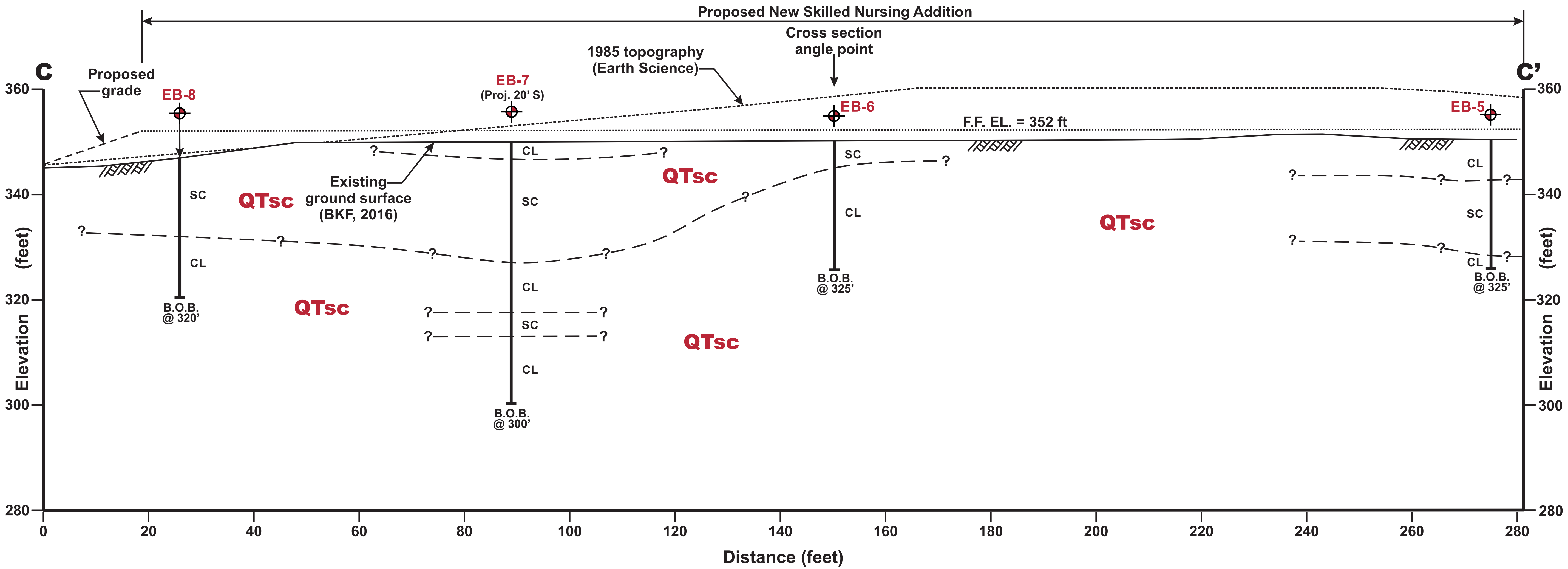
**The Forum at Rancho San Antonio
Cupertino, CA**

Project Number
905-1-1

Figure Number
Figure 7B

Date
March 2017

Drawn By
RRN



Section C-C'

(View Looking North)
1"=20' H:V

Notes:
1) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
2) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced borings. Actual subsurface conditions may vary significantly between borings.
3) See Figure 6C for location of cross section.

Explanation

Geologic Units

QTsc Santa Clara Formation (lower Pleistocene and upper Pliocene)

Symbols

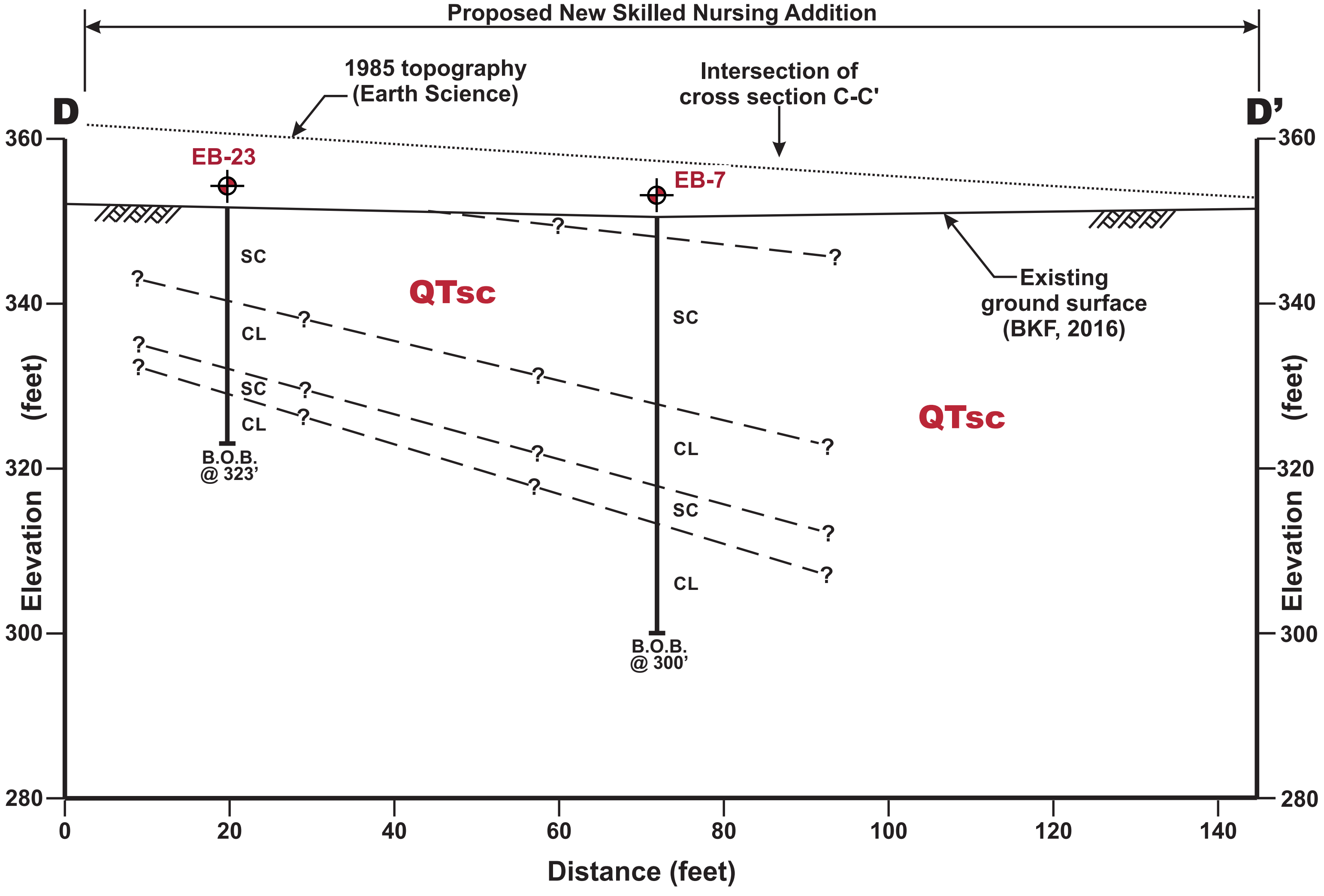
CL	Lean Clay
SC	Clayey Sand
⌖	Approximate location of exploratory boring (EB)

CORNERSTONE EARTH GROUP

Geologic Cross Section C-C' - Skilled Nursing Addition

The Forum at Rancho San Antonio
Cupertino, CA

Project Number 905-1-1
Figure Number Figure 7C
Date March 2017
Drawn By RRN



Section D-D'

(View Looking Southeast)
1"=20' H:V

Notes:
1) Surficial fills associated with existing pavements, landscaping or utilities are not shown.
2) The subsurface profile is conceptual and is based on limited subsurface data obtained from widely spaced borings. Actual subsurface conditions may vary significantly between borings.
3) See Figure 6C for location of cross section.

Explanation

Geologic Units

QTsc	Santa Clara Formation (lower Pleistocene and upper Pliocene)

Symbols

CL	Lean Clay
SC	Clayey Sand
⌖	Approximate location of exploratory boring (EB)

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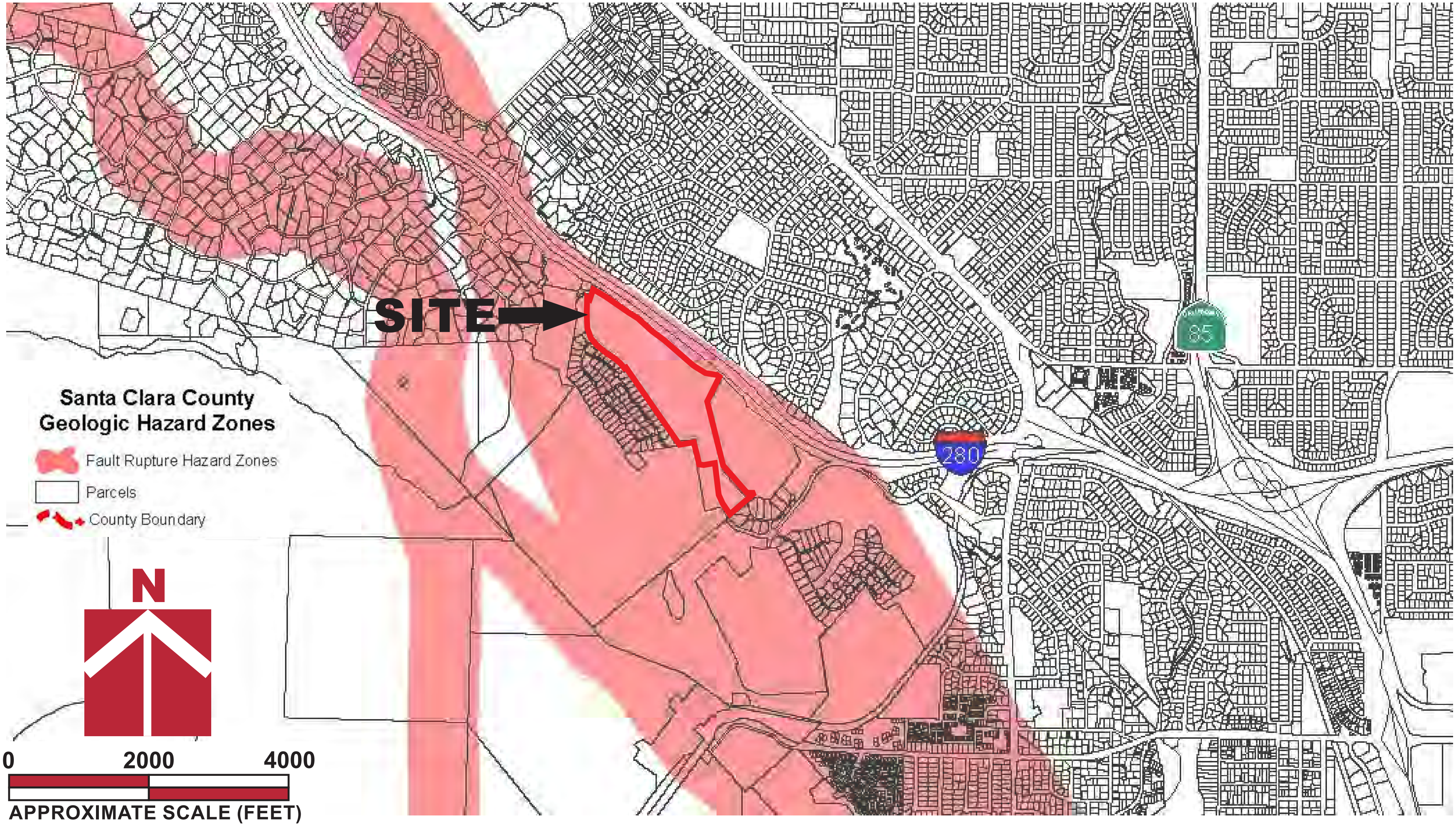
Geologic Cross Section D-D' - Skilled Nursing Addition

The Forum at Rancho San Antonio
Cupertino, CA

Project Number 905-1-1
Figure Number Figure 7D
Date April 2017
Drawn By RRN



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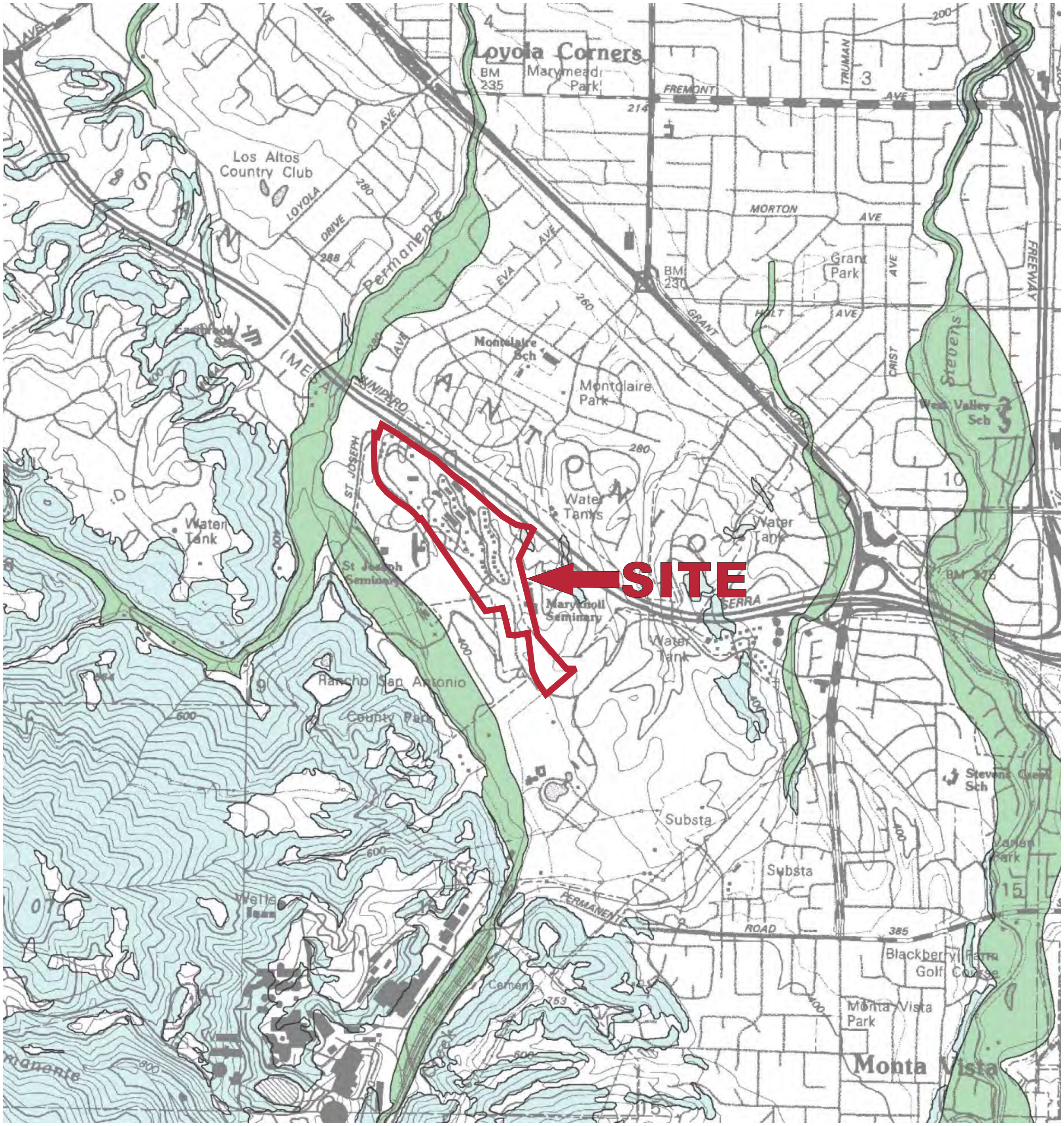
Santa Clara County Fault Hazard Map

The Forum at Rancho San Antonio
Cupertino, CA

Project Number	905-1-1
Figure Number	Figure 8
Date	March 2017
Drawn By	RRN



Explanation

Liquefaction

🟩 Areas where historic occurrence of liquefaction, or local geological, geotechnical and groundwater conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

Earthquake-Induced Landslides

🟦 Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

N

0 2000 4000

APPROXIMATE SCALE (FEET)

Base by State of California, Seismic Hazard Zones.

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State Seismic Hazard Zone Map

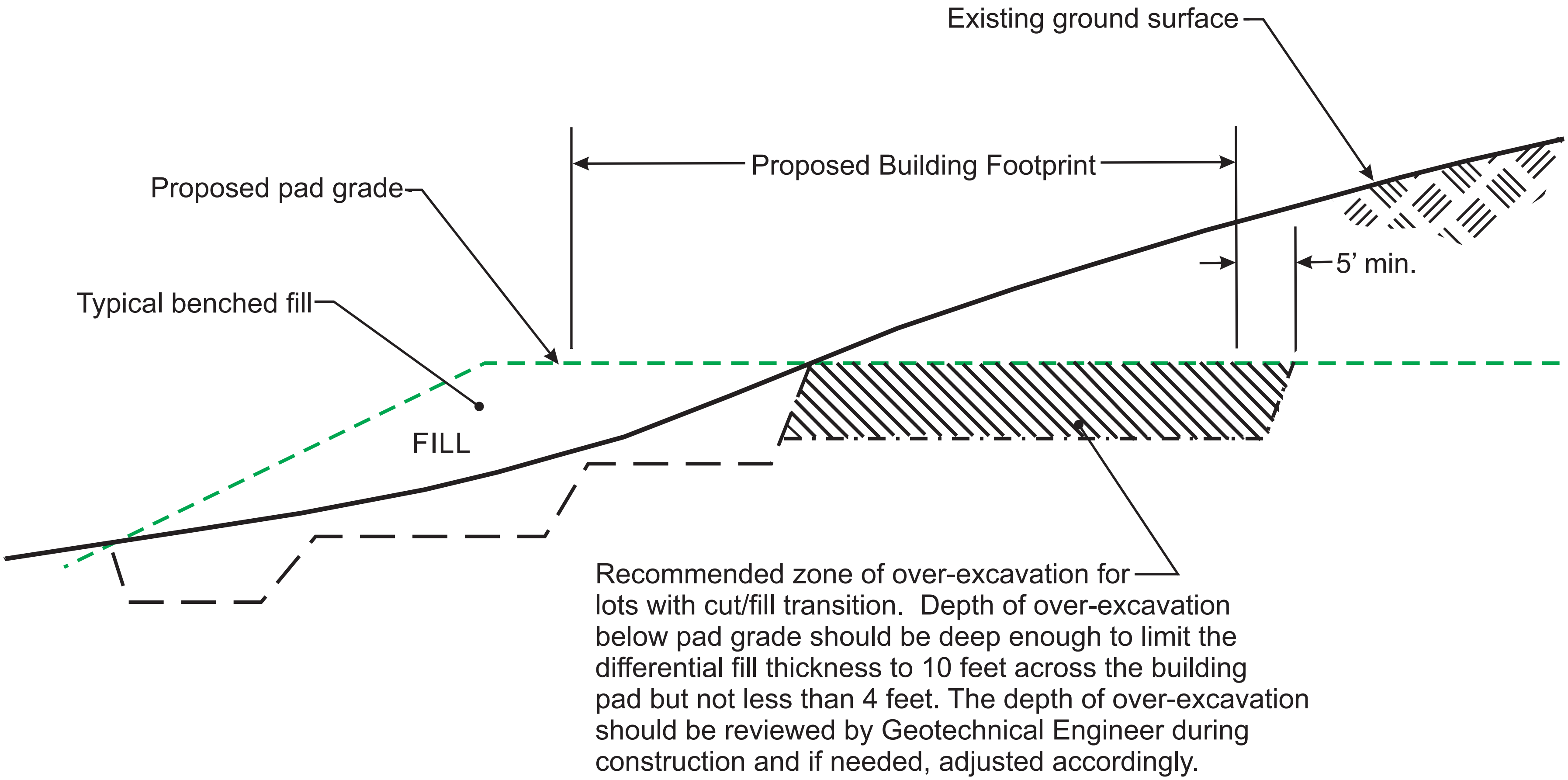
The Forum at Rancho San Antonio
Cupertino, CA

Project Number
905-1-1

Figure Number
Figure 9

Date
March 2017

Drawn By
RRN



Recommended zone of over-excavation for lots with cut/fill transition. Depth of over-excavation below pad grade should be deep enough to limit the differential fill thickness to 10 feet across the building pad but not less than 4 feet. The depth of over-excavation should be reviewed by Geotechnical Engineer during construction and if needed, adjusted accordingly.

Detail 3 - Conceptual Cut / Fill Transition Over-Excavation

Not to scale

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Cut/Fill Transition Over-Excavation Detail

The Forum at Rancho San Antonio
Cupertino, CA

Project Number
905-1-1

Figure Number
Figure 10

Date
March 2017

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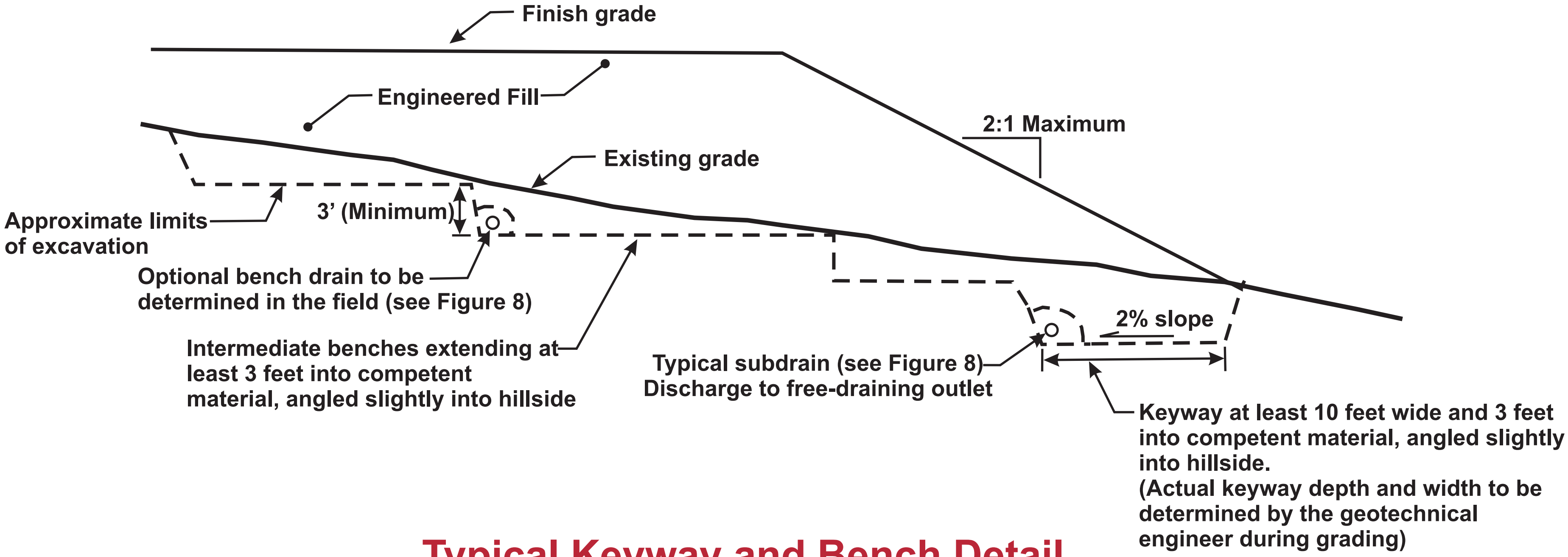

Typical Keyway and Bench Detail

Not to scale

Note: Fill slopes should be over-built at least 18 to 24 inches and trimmed to expose compacted fill.

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Typical Keyway and Bench Detail

The Forum at Rancho San Antonio
Cupertino, CA

Project Number 905-1-1

Figure Number Figure 11

Date March 2017

Drawn By RRN

DRAINAGE MATERIAL

Alternative 1

Class 2 Permeable Material
(Caltrans Standard Specs, latest edition)

Material shall consist of clean, coarse sand and gravel or crushed stone, conforming to the following gradation requirements:

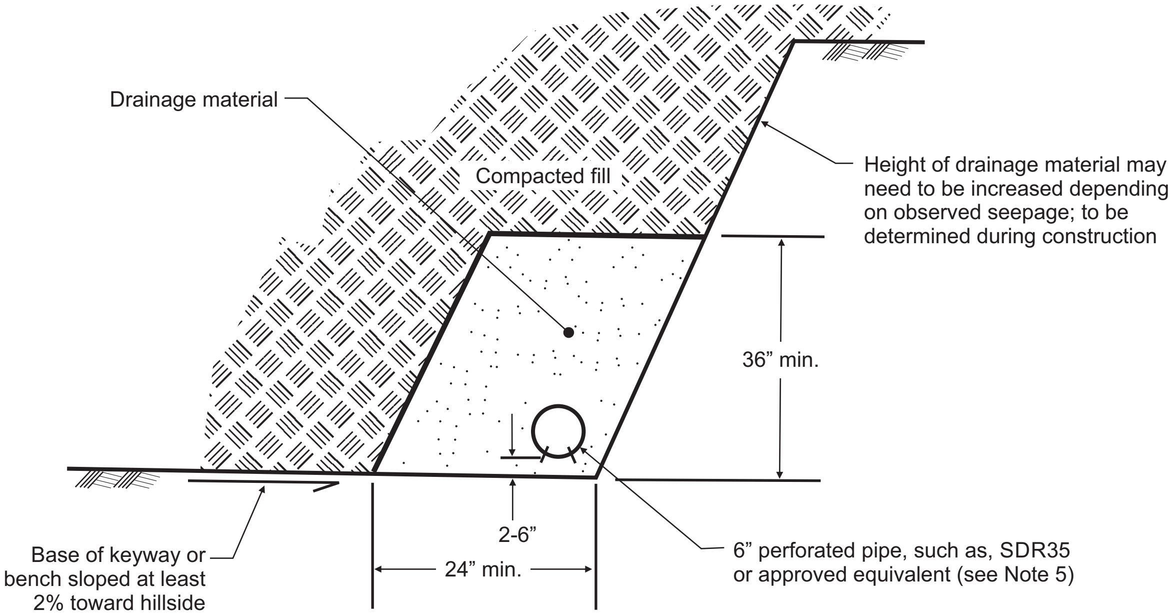
Sieve Size	% Passing Sieve
1”	100
3/4”	90-100
3/8”	40-100
#4	25-40
#8	18-33
#30	5-15
#50	0-7
#200	0-3

Alternative 2

½- to 3/4- inch Clean Crushed Rock or
Gravel Wrapped in Filter Fabric

All non-woven filter fabric shall meet the following minimum average roll values unless otherwise specified by Cornerstone Earth Group

Grab Strength (ASTM D-4632):	180 lbs.
Mass Per Unit Area (ASTM D-4751):	5 oz/yd
Apparent Opening Size (ASTM D-4751):	70-100 U.S. std. sieve
Flow Rate (ASTM D-4491):	80 gal/min/ft
Puncture Strength (ASTM D-4833):	80 lbs.



Notes:

1. 1% fall (minimum) along all keyways, benches and subdrain lines.
2. All perforated pipe placed perforations down.
3. All pipe joints shall be glued.
4. All subdrains should be discharged to a free draining outlet approved by the Civil Engineer.
5. Subdrain pipe (perforated or solid connector) should consist of SDR-35 PVC pipe when placed in fills less than 30 feet deep. SDR-23.5 PVC pipe should be used when fill is greater than 30 feet deep.

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Typical Keyway and Bench Subdrain Detail

The Forum at Rancho San Antonio
Cupertino, CA

Project Number
905-1-1

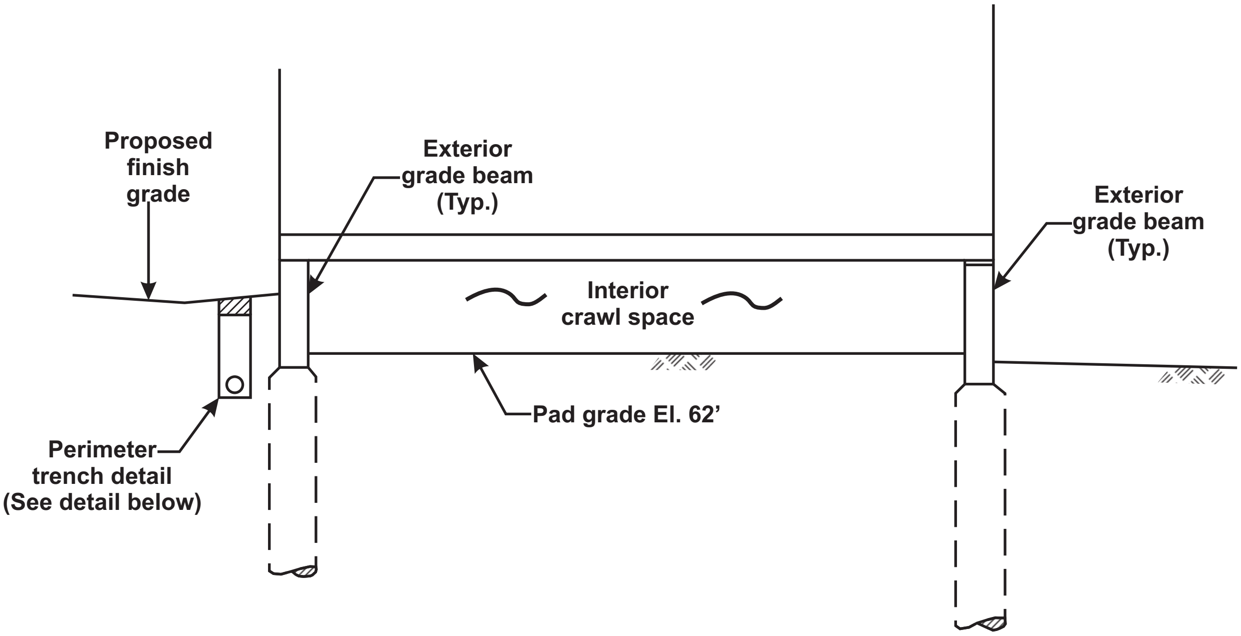
Figure Number
Figure 12

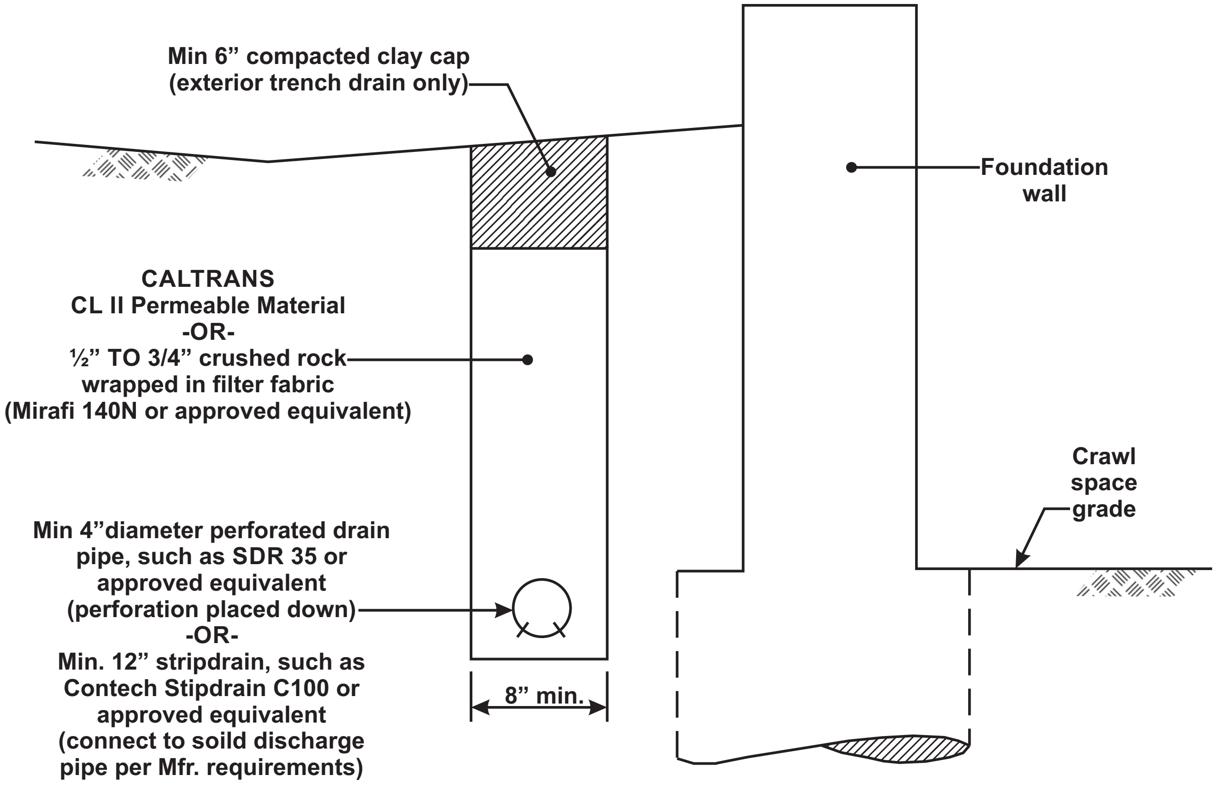
Date
March 2017

Drawn By
RRN

TYPICAL TRENCH SUBDRAIN DETAIL

(NOT TO SCALE)





NOTES:

1. Trench drain should discharge to free draining outlet.
2. Roof downspouts should not be connected to trench drain system.
3. Trench drain should be installed after foundation construction.
4. Trench depth (D) will likely range from 24” to 36” below finished grade; final depth to be determined in the field during construction.

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Typical Trench Subdrain Detail

The Forum at Rancho San Antonio
Cupertino, CA

Project Number
905-1-1

Figure Number
Figure 13

Date
March 2017

Drawn By
RRN

APPENDIX A: FIELD INVESTIGATION

The field investigation consisted of a surface reconnaissance and a subsurface exploration program using truck-mounted, hollow-stem auger and track-mounted, solid stem auger drilling equipment. Twenty-three (23) 8-inch-diameter exploratory borings were drilled between July 11 through 14, 2016, and on March 27, 2017, to depths of about 10 to 50 feet. The approximate locations of exploratory borings are shown on the various Site Plan and Geologic Maps for the development, Figures 6A through 6H. 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 locations were approximated using existing site boundaries, a hand held GPS unit, and other site features as references. Boring elevations were based on interpolation of plan contours. The locations and elevations of the borings 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 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 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

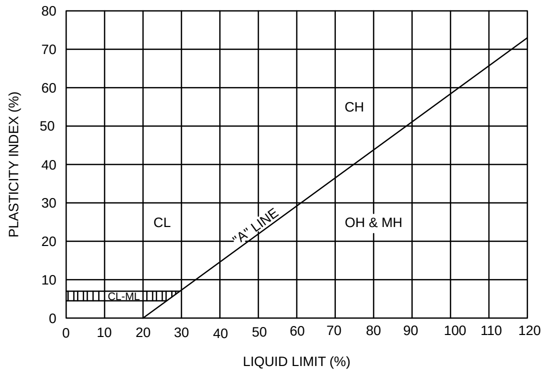
☒	SPT	■	Shelby Tube
⧗	Modified California (2.5" I.D.)	○	No Recovery
▯	Rock Core	✋	Grab Sample

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.

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LEGEND TO SOIL DESCRIPTIONS

Figure Number
A-1

BORING NUMBER EB-1

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME The Forum at Rancho San Antonio
PROJECT NUMBER 905-1-1
PROJECT LOCATION 23500 Cristo Rey Drive, Cupertino, CA
DATE STARTED 7/11/16 **DATE COMPLETED** 7/11/16
GROUND ELEVATION 351 FT +/- **BORING DEPTH** 24.5 ft.
DRILLING CONTRACTOR Britton Exploration, Inc.
LATITUDE 37.339015° **LONGITUDE** -122.087804°
DRILLING METHOD CME Track Rig, 6 inch Solid Flight Auger
GROUND WATER LEVELS:
LOGGED BY SDK
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

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

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
351.0	0	▨	**Sandy Lean Clay (CL)** hard, moist, brown, fine to coarse sand, some fine to coarse subangular to subrounded gravel, low to moderate plasticity	22	MC-1B	102	11			>4.5 ○
348.3		▨	**Lean Clay with Sand (CL)** hard, moist, brown, fine to medium sand, moderate plasticity	27	MC-2B	103	16			>4.5 ○
	5			83	MC-3B	116	11			>4.5 ○
345.0		▨	**Sandy Lean Clay (CL) [QTsc]** hard, moist, light brown with reddish brown mottles, fine to coarse sand, some fine subangular to subrounded gravel, moderate plasticity	87	MC-4B	108	16			>4.5 ○
	10									
				54	MC-5B	110	16			>4.5 ○
	15									
334.0		▨	**Clayey Sand with Gravel (SC) [QTsc]** very dense, moist, brown with reddish brown mottles, fine to coarse sand, fine to coarse subangular to subrounded gravel Liquid Limit = 42, Plastic Limit = 18	50/6"	MC-6B	113	9	24		
	20									
				50/6"	MC					
326.5	25		Bottom of Boring at 24.5 feet.							

BORING NUMBER EB-2

CORNERSTONE EARTH GROUP

PROJECT NAME The Forum at Rancho San Antonio
PROJECT NUMBER 905-1-1
PROJECT LOCATION 23500 Cristo Rey Drive, Cupertino, CA

DATE STARTED 7/11/16 **DATE COMPLETED** 7/11/16
GROUND ELEVATION 357 FT +/- **BORING DEPTH** 25 ft.
DRILLING CONTRACTOR Britton Exploration, Inc.
LATITUDE 37.338785° **LONGITUDE** -122.087550°
DRILLING METHOD CME Track Rig, 6 inch Solid Flight Auger
GROUND WATER LEVELS:
LOGGED BY SDK
∇ **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
1.0 2.0 3.0 4.0

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
357.0	0	▨	**Lean Clay with Sand (CL)** hard, moist, dark brown to brown, fine to medium sand, trace fine subangular gravel, moderate plasticity							
				20	MC-1B	99	15			>4.5 ○
				30	MC-2B	105	17			>4.5 ○
352.0	5	▨	**Sandy Lean Clay (CL) [QTsc]** hard, moist, light brown to brown with reddish brown mottles, fine to medium sand, some fine subangular to subrounded gravel, low plasticity	72	MC-3B	106	15			>4.5 ○
				50/6"	MC					>4.5 ○
	10									
				50/6"	MC					>4.5 ○
	15									
			fine to coarse sand	35	SPT-6		19		66	>4.5 ○
	20									
			color changes to dark brown to brown	26	SPT					>4.5 ○
332.0	25		Bottom of Boring at 25.0 feet.							

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-3

PAGE 1 OF 2

PROJECT NAME The Forum at Rancho San Antonio
PROJECT NUMBER 905-1-1
PROJECT LOCATION 23500 Cristo Rey Drive, Cupertino, CA
DATE STARTED 7/14/16 **DATE COMPLETED** 7/14/16
GROUND ELEVATION 350 FT +/- **BORING DEPTH** 49.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.338513° **LONGITUDE** -122.087699°
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
▽ **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)	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
350.0	0	**Clayey Sand (SC) [QTsc]** medium dense, moist, reddish brown and brown mottled, fine to medium sand, some fine to coarse subangular to subrounded gravel	47	MC-1B	113	16		25	
347.3		**Lean Clay with Sand (CL) [QTsc]** hard, moist, brown and reddish brown mottled, fine to medium sand, moderate plasticity	33	MC-2B	102	25			>4.5 ○
345.5	5	**Sandy Lean Clay (CL) [QTsc]** hard, moist, reddish brown, fine to medium sand, moderate plasticity Liquid Limit = 43, Plastic Limit = 21	45	3A MC 3B	103 100	17 26	22		>4.5 ○ >> ▲
344.0		**Lean Clay with Sand (CL) [QTsc]** hard, moist, reddish brown and brown mottled, fine to coarse sand, moderate plasticity	62	MC-4B	107	20			>4.5 ○
	10								
338.5		**Clayey Sand with Gravel (SC) [QTsc]** very dense, moist, light brown and reddish brown mottled, fine to medium sand, fine to coarse subangular to subrounded gravel	50/6"	MC-5B	117	10		22	
	15								
332.5		**Sandy Lean Clay (CL) [QTsc]** hard, moist, reddish brown with gray mottles, fine to medium sand, some fine gravel, low to moderate plasticity	46	MC-6B	106	21			>4.5 ○
	20								
			51	MC					>4.5 ○
	25								
324.0									

Continued Next Page

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-3

PROJECT NAME The Forum at Rancho San Antonio
PROJECT NUMBER 905-1-1
PROJECT LOCATION 23500 Cristo Rey Drive, Cupertino, CA

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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)	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
324.0		▨								
323.0		▨	**Lean Clay with Sand (CL) [QTsc]** hard, moist, reddish brown and brown mottled, fine to coarse sand, moderate plasticity							
		▨		60	MC-8B	99	26			>4.5 ○
	30	▨								
		▨		49	MC-9B	96	28			>4.5 ○
	35	▨								
		▨	some fine subangular to subrounded gravel	61	MC					>4.5 ○
	40	▨								
308.0		▨	**Clayey Sand with Gravel (SC) [QTsc]** very dense, moist, light brown with reddish brown mottles, fine to coarse sand, fine to coarse gravel, some cobbles							
		▨		50 6"	MC-11B	112	14			
	45	▨								
		▨		50 6"	MC					
300.5			Bottom of Boring at 49.5 feet.							
	50									
	55									

BORING NUMBER EB-4

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME The Forum at Rancho San Antonio
PROJECT NUMBER 905-1-1
PROJECT LOCATION 23500 Cristo Rey Drive, Cupertino, CA
DATE STARTED 7/11/16 **DATE COMPLETED** 7/11/16
GROUND ELEVATION 360 FT +/- **BORING DEPTH** 25 ft.
DRILLING CONTRACTOR Britton Exploration, Inc.
LATITUDE 37.338382° **LONGITUDE** -122.087431°
DRILLING METHOD CME Track Rig, 6 inch Solid Flight 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)	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)
360.0	0	**Sandy Lean Clay (CL) [Fill]** very stiff, dry, light brown, fine to coarse sand, moderate plasticity							
358.5		**Lean Clay with Sand (CL) [QTsc]** hard, moist, brown, some fine to coarse sand, trace fine subangular to subrounded gravel, moderate plasticity	25	MC-1B	111	13			>4.5 ○
357.5		**Lean Clay with Sand (CL) [QTsc]** hard, moist, light brown with reddish brown mottles, fine to medium sand, moderate plasticity	50 6"	MC-2B	100	21			>4.5 ○
	5		35	SPT					
	10		32	SPT-4		25			
	15		37	SPT-5		21			
344.0		**Clayey Sand with Gravel (SC) [QTsc]** very dense, moist, brown with reddish brown mottles, fine to coarse sand, fine subangular to subrounded gravel							
	20		54	SPT-6		14		16	
336.0		**Sandy Lean Clay (CL) [QTsc]** hard, moist, brown, fine to coarse sand, moderate plasticity	63	SPT-7		17			
335.0	25								

Bottom of Boring at 25.0 feet.

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 4/11/17 08:06 - P:\DRAFTING\GINT FILES\905-1-1 FORUM AT RANCHO SAN ANTONIO.GPJ

BORING NUMBER EB-5

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME The Forum at Rancho San Antonio

PROJECT NUMBER 905-1-1

PROJECT LOCATION 23500 Cristo Rey Drive, Cupertino, CA

DATE STARTED 7/14/16 **DATE COMPLETED** 7/14/16

GROUND ELEVATION 350 FT +/- **BORING DEPTH** 25 ft.

DRILLING CONTRACTOR Exploration Geoservices, Inc.

LATITUDE 37.338782° **LONGITUDE** -122.088480°

DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger

GROUND WATER LEVELS:

LOGGED BY DL

▽ **AT TIME OF DRILLING** 20 ft.

NOTES

▼ **AT END OF DRILLING** 15 ft.

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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
350.0	0	**Sandy Lean Clay (CL) [QTsc]** hard to very stiff, moist, brown with light brown and reddish brown mottles, fine to medium sand, some fine to coarse gravel, moderate plasticity						
			61	MC-1B	106	17		
			23	MC-2B	97	24		
	5		34	MC-3B	103	24		
342.5		**Clayey Sand with Gravel (SC) [QTsc]** medium dense, brown with reddish brown mottles, fine to medium sand, fine to coarse subangular to subrounded gravel						
			40	MC-4B	110	15		14
	10							
		dense	49	MC-5B	112	17		17
	▼ 15							
		very dense	50 6"	SPT-6		14		
	▽ 20							
327.5		**Sandy Lean Clay (CL) [QTsc]** very stiff, moist, brown with reddish brown mottles, fine to medium sand, some fine to coarse gravel, moderate plasticity						
			34	MC-7B		18		
325.0	25	Bottom of Boring at 25.0 feet.						

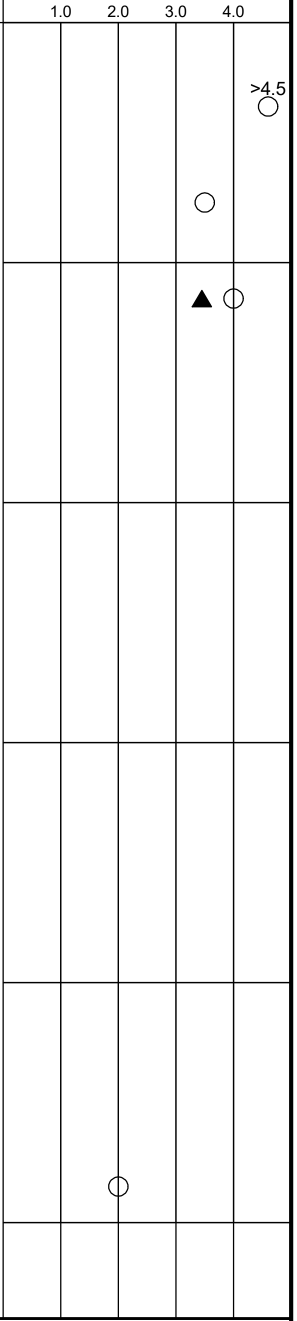
UNDRAINED SHEAR STRENGTH, ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL



BORING NUMBER EB-6

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME The Forum at Rancho San Antonio

PROJECT NUMBER 905-1-1

PROJECT LOCATION 23500 Cristo Rey Drive, Cupertino, CA

DATE STARTED 7/14/16 **DATE COMPLETED** 7/14/16

GROUND ELEVATION 351 FT +/- **BORING DEPTH** 25 ft.

DRILLING CONTRACTOR Exploration Geoservices, Inc.

LATITUDE 37.338975° **LONGITUDE** -122.088838°

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)	DESCRIPTION
351.0 350.8 350.7	0	2 inches asphalt concrete over 2 inches aggregate base
		Clayey Sand with Gravel (SC) [QTsc] medium dense, brown with reddish brown mottles, fine to medium sand, fine to coarse subangular to subrounded gravel
347.0	5	**Sandy Lean Clay (CL) [QTsc]** hard, moist, brown with light brown and reddish brown mottles, fine to medium sand, some fine to coarse gravel, moderate plasticity Liquid Limit = 46, Plastic Limit = 26
	10	
	15	increasing gravel content
333.5		**Lean Clay with Sand (CL) [QTsc]** hard, moist, reddish brown and brown mottled, fine to coarse sand, moderate plasticity
	20	
326.0	25	Bottom of Boring at 25.0 feet.

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

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
27	MC-1B	119	13			
49	MC-2B	102	21			>4.5 ○
51	MC-3B	108	16	20		>4.5 ○
26	MC-4B	97	28			>> ▲
50 6"	MC-5B	113	15			>4.5 ○
33	MC-6B	109	19			>4.5 ○
62	MC-7B	103	23			>4.5 ○

BORING NUMBER EB-7

PAGE 1 OF 2

CORNERSTONE EARTH GROUP

PROJECT NAME The Forum at Rancho San Antonio
PROJECT NUMBER 905-1-1
PROJECT LOCATION 23500 Cristo Rey Drive, Cupertino, CA
DATE STARTED 7/14/16 **DATE COMPLETED** 7/14/16
GROUND ELEVATION 350 FT +/- **BORING DEPTH** 49.9 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.339069° **LONGITUDE** -122.089038°
DRILLING METHOD Mobile B-40, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY OL
∇ **AT TIME OF DRILLING** 26.5 ft.
NOTES
▼ **AT END OF DRILLING** 26.5 ft.

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

DESCRIPTION

ELEVATION (ft)	DEPTH (ft)	DESCRIPTION
350.0 349.8	0	2 inches asphalt concrete over 5 inches aggregate base
349.4		**Sandy Lean Clay (CL) [QTsc]** hard, moist, brown and gray brown mottled, fine to coarse sand, low to moderate plasticity
347.3		**Clayey Sand with Gravel (SC) [QTsc]** very dense, moist, brown with reddish brown mottles, fine to coarse sand, fine subangular to subrounded gravel
	5	trace cobbles @ 5.5'
		becomes dense, no cobbles
	10	
335.5	15	**Sandy Lean Clay (CL) [QTsc]** very stiff, moist, brown with reddish brown mottles, fine to medium sand, low to moderate plasticity
334.0		**Clayey Sand with Gravel (SC) [QTsc]** very dense, moist, brown with reddish brown mottles, fine to coarse sand, fine subangular to subrounded gravel Liquid Limit = 36, Plastic Limit = 20
	20	
327.5		**Sandy Lean Clay (CL) [QTsc]** hard, moist, brown with reddish brown and gray mottles, fine to medium sand, moderate plasticity
	25	
324.0		

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
27	MC-1A	102	20			○
50 4"	MC-2A	115	10		16	
50 6"	MC-3B	120	7			
33	SPT-4		14			
46	MC-5B	118	12			○
50 5"	MC-6B	128	11	16		
71	MC-7B	108	21			>4.5 ○

Continued Next Page

CORNERSTONE
EARTH GROUP

BORING NUMBER EB-7

PAGE 2 OF 2

PROJECT NAME The Forum at Rancho San Antonio

PROJECT NUMBER 905-1-1

PROJECT LOCATION 23500 Cristo Rey Drive, Cupertino, CA

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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
324.0 ▼									
323.0		**Lean Clay with Sand (CL) [QTsc]** hard, moist, brown with reddish brown mottles, fine to medium sand, moderate plasticity							
	30		32	MC-8B	95	29			>4.5 ○
318.0		**Clayey Sand with Gravel (SC) [QTsc]** very dense, moist, brown with reddish brown mottles, fine to coarse sand, fine subangular to subrounded gravel							
	35		50 5"	MC-9B	118	14			
313.0		**Sandy Lean Clay (CL) [QTsc]** hard, moist, brown with reddish brown mottles, fine to medium sand, low to moderate plasticity							
	40		36	MC-10B	117	16			>4.5 ○
308.0		**Lean Clay with Sand (CL) [QTsc]** hard, moist, brown with reddish brown mottles, fine to medium sand, moderate plasticity							
	45		65	MC					>4.5 ○
			50 5"	MC					>4.5 ○
300.1	50	Bottom of Boring at 49.9 feet.							
	55								

BORING NUMBER EB-8

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME The Forum at Rancho San Antonio
PROJECT NUMBER 905-1-1
PROJECT LOCATION 23500 Cristo Rey Drive, Cupertino, CA
DATE STARTED 7/11/16 **DATE COMPLETED** 7/11/16
GROUND ELEVATION 345 FT +/- **BORING DEPTH** 25 ft.
DRILLING CONTRACTOR Britton Exploration, Inc.
LATITUDE 37.339054° **LONGITUDE** -122.089291°
DRILLING METHOD CME Track Rig, 6 inch Solid Flight 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)	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
345.0	0	**Clayey Sand with Gravel (SC) [QTsc]** medium dense to dense, moist, light brown with reddish brown mottles, fine to coarse sand, fine to coarse subangular to subrounded gravel							
			44	MC-1A	118	8			
			53	MC-2B	116	9			
	5		58	MC-3B	125	10			
			51	MC-4B	116	11			
	10								
330.5		**Sandy Lean Clay (CL) [QTsc]** hard, moist, brown with reddish brown mottles, fine to medium sand, low to moderate plasticity	42	MC-5B	108	19			
	15								
			55	MC					>4.5 ○
	20								
324.0		**Lean Clay with Sand (CL) [QTsc]** hard, moist, gray and reddish brown mottled, fine sand, moderate plasticity							
			34	MC-7B	96	30			>4.5 ○
320.0	25	Bottom of Boring at 25.0 feet.							

BORING NUMBER EB-9

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME The Forum at Rancho San Antonio
PROJECT NUMBER 905-1-1
PROJECT LOCATION 23500 Cristo Rey Drive, Cupertino, CA
DATE STARTED 7/12/16 **DATE COMPLETED** 7/12/16
GROUND ELEVATION 360 FT +/- **BORING DEPTH** 25 ft.
DRILLING CONTRACTOR Britton Exploration, Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD CME Track Rig, 6 inch Solid Flight Auger
GROUND WATER LEVELS:
LOGGED BY OL
▽ **AT TIME OF DRILLING** 18.5 ft.
NOTES
▼ **AT END OF DRILLING** 18.5 ft.

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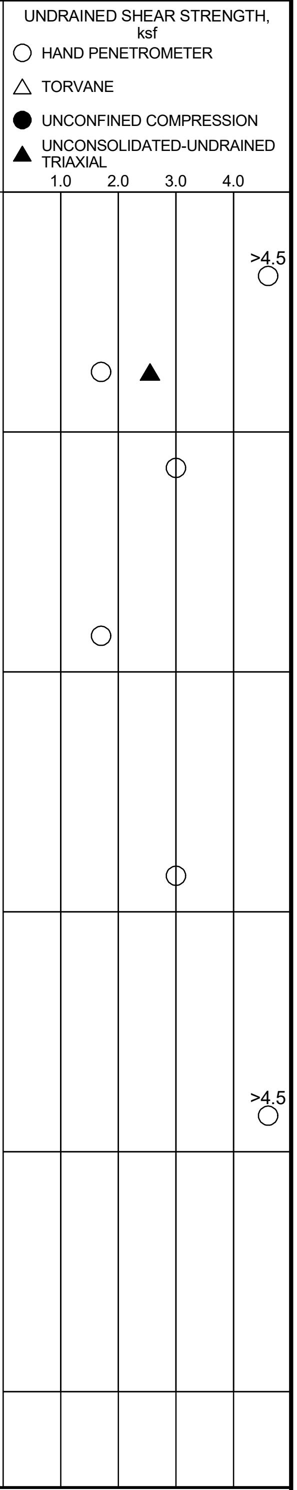
ELEVATION (ft)	DEPTH (ft)	SYMBOL	DESCRIPTION
360.0	0	▩	**Sandy Fat Clay (CH) [Fill]** hard, moist, brown and gray brown mottled, fine to coarse sand, some fine subangular to subrounded gravel, high plasticity Liquid Limit = 52, Plastic Limit = 23
357.0		▩	**Sandy Lean Clay (CL) [Fill]** stiff to very stiff, moist, light brown and gray mottled, fine to coarse sand, moderate plasticity
	5		
	10		
348.5		▨	**Lean Clay with Sand (CL)** very stiff, moist, dark brown, fine to coarse sand, some fine subangular to subrounded gravel, moderate plasticity
	15		
344.0		▨	**Sandy Lean Clay (CL) [QTsc]** hard, moist, light brown with reddish brown mottles, fine to coarse sand, some fine subangular to subrounded gravel, low plasticity
	▼		
	20		
339.0		▨	**Clayey Sand (SC) [QTsc]** dense, moist, brown with reddish brown mottles, fine to coarse sand, some fine to coarse subangular to subrounded gravel
335.0	25		Bottom of Boring at 25.0 feet.

N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
21	MC-1B	102	17	29	
24	MC-2C	110	18		
28	MC-3B	112	18		
18	MC-4B	108	17		
20	MC-5B	109	19		
56	MC-6B	98	24		
38	SPT				



CORNERSTONE
EARTH GROUP

BORING NUMBER EB-10

PAGE 1 OF 1

PROJECT NAME The Forum at Rancho San Antonio

PROJECT NUMBER 905-1-1

PROJECT LOCATION 23500 Cristo Rey Drive, Cupertino, CA

DATE STARTED 7/12/16 **DATE COMPLETED** 7/12/16

GROUND ELEVATION 359 FT +/- **BORING DEPTH** 25 ft.

DRILLING CONTRACTOR Britton Exploration, Inc.

LATITUDE **LONGITUDE**

DRILLING METHOD CME Track Rig, 6 inch Solid Flight 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 ○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0
359.0	0	▩	**Lean Clay with Sand (CL) [Fill]** stiff, moist, dark brown and brown mottled, fine to coarse sand, some fine subangular to subrounded gravel, trace organics, moderate plasticity	17	MC-1B	114	18			○
356.8		▨	**Sandy Lean Clay with Gravel (CL) [QTsc]** hard, moist, reddish brown, fine to coarse sand, fine subangular to subrounded gravel, low to moderate plasticity	50 3"	MC-2	97	17			>4.5 ○
	5			36	SPT					>4.5 ○
352.0		▨	**Clayey Sand with Gravel (SC) [QTsc]** dense, moist, reddish brown with brown mottles, fine to coarse sand, fine subangular to subrounded gravel	65	MC-4B	125	11			
	10									
345.3		▨	**Sandy Lean Clay (CL) [QTsc]** hard, moist, reddish brown with light brown mottles, fine to coarse sand, some fine subangular to subrounded gravel, low to moderate plasticity	70	MC-5B	106	18			>4.5 ○
	15									
				65	MC-6B	112	19			>4.5 ○
	20									
337.0		▨	**Clayey Sand with Gravel (SC) [QTsc]** dense, moist, reddish brown with brown mottles, fine to coarse sand, fine subangular to subrounded gravel	66	MC					
334.0	25		Bottom of Boring at 25.0 feet.							

BORING NUMBER EB-11

PAGE 1 OF 1

CORNERSTONE
EARTH GROUP

PROJECT NAME The Forum at Rancho San Antonio
PROJECT NUMBER 905-1-1
PROJECT LOCATION 23500 Cristo Rey Drive, Cupertino, CA

DATE STARTED 7/11/16 **DATE COMPLETED** 7/11/16
GROUND ELEVATION 337 FT +/- **BORING DEPTH** 20 ft.
DRILLING CONTRACTOR Britton Exploration, Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD CME Track Rig, 6 inch Solid Flight 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
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
337.0	0	**Clayey Sand with Gravel (SC) [Fill]** loose, moist, reddish brown with brown mottles, fine to coarse sand, fine subangular to subrounded gravel	17	MC-1B	113	10			
334.5		**Clayey Sand with Gravel (SC)** medium dense, moist, reddish brown with brown mottles, fine to coarse sand, fine subangular to subrounded gravel Liquid Limit = 33, Plastic Limit = 16	21	MC-2B	114	15	17		
332.0	5	**Sandy Lean Clay (CL) [QTsc]** hard, moist, reddish brown with brown mottles, fine to coarse sand, some fine subangular to subrounded gravel, low to moderate plasticity	28	MC-3B	104	19			>4.5 ○
			50 4"	MC-4B	100	26			>4.5 ○
	10								
			25	SPT					>4.5 ○
	15								
		becomes very stiff	27	SPT					○
317.0	20	Bottom of Boring at 20.0 feet.							
	25								

BORING NUMBER EB-12

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME The Forum at Rancho San Antonio

PROJECT NUMBER 905-1-1

PROJECT LOCATION 23500 Cristo Rey Drive, Cupertino, CA

DATE STARTED 7/14/16 **DATE COMPLETED** 7/14/16

GROUND ELEVATION 388 FT +/- **BORING DEPTH** 10 ft.

DRILLING CONTRACTOR Exploration Geoservices, Inc.

LATITUDE **LONGITUDE**

DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger

GROUND WATER LEVELS:

LOGGED BY DL

▽ **AT TIME OF DRILLING** Not Encountered

NOTES

▼ **AT END OF DRILLING** Not Encountered

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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
388.0 387.8	0	2 inches asphalt concrete over 6 inches aggregate base						
387.3		**Sandy Lean Clay (CL) [Fill]** very stiff, moist, dark brown to brown, fine to medium sand, trace fine angular to subangular gravel, moderate plasticity Liquid Limit = 48, Plastic Limit = 21	16	MC-1B	109	17	27	
384.3		**Lean Clay (CL) [Fill]** very stiff, moist, brown with gray mottles, some fine to medium sand, moderate plasticity	24	MC-2B	111	14		
	5		23	MC				
380.5		**Sandy Lean Clay (CL) [QTsc]** hard, moist, brown, fine to medium sand, trace fine angular to subangular gravel, moderate plasticity	21	MC-4B	105	20		
378.0	10	Bottom of Boring at 10.0 feet.						
	15							
	20							
	25							

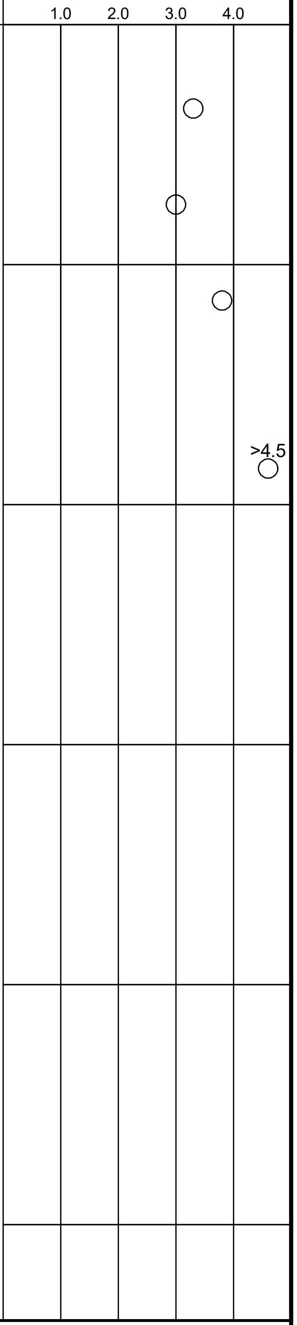
UNDRAINED SHEAR STRENGTH, ksf

○ HAND PENETROMETER

△ TORVANE

● UNCONFINED COMPRESSION

▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL



BORING NUMBER EB-13

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME The Forum at Rancho San Antonio
PROJECT NUMBER 905-1-1
PROJECT LOCATION 23500 Cristo Rey Drive, Cupertino, CA
DATE STARTED 7/12/16 **DATE COMPLETED** 7/12/16
GROUND ELEVATION 419 FT +/- **BORING DEPTH** 20 ft.
DRILLING CONTRACTOR Britton Exploration, Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD CME Track Rig, 6 inch Solid Flight Auger
GROUND WATER LEVELS:
LOGGED BY OL
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

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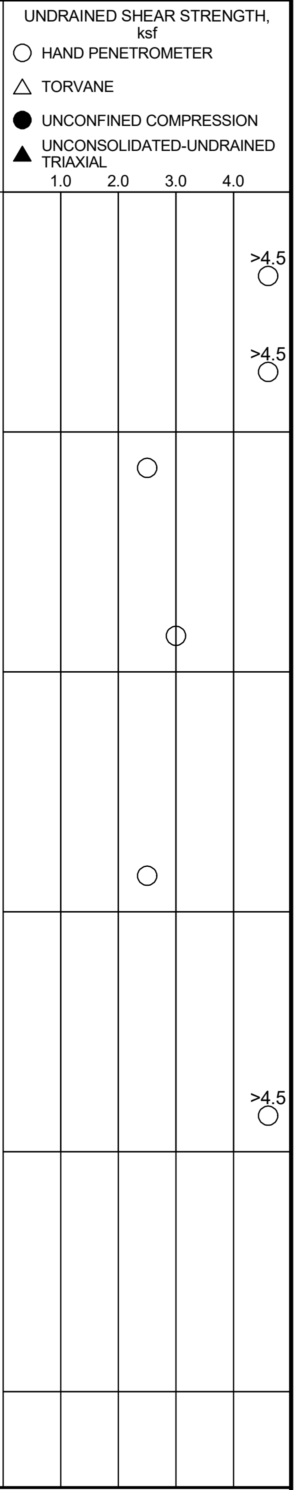
ELEVATION (ft)	DEPTH (ft)	DESCRIPTION
419.0	0	**Sandy Lean Clay (CL) [Fill]** hard, moist, brown with reddish brown mottles, fine to coarse sand, some fine subangular to subrounded gravel, moderate plasticity
414.5		**Lean Clay with Sand (CL) [Fill]** very stiff, moist, brown with reddish brown mottles, fine to medium sand, moderate plasticity
	5	
409.0	10	**Lean Clay with Sand (CL) [Qc]** very stiff, moist, brown and dark gray mottled, fine to coarse sand, trace fine subangular to subrounded gravel, moderate plasticity
	15	
402.0		**Sandy Lean Clay (CL) [QTsc]** hard, moist, reddish brown, fine to coarse sand, low plasticity
399.0	20	Bottom of Boring at 20.0 feet.
	25	

N-Value (uncorrected) blows per foot	SAMPLES TYPE AND NUMBER	DRY UNIT WEIGHT PCF	NATURAL MOISTURE CONTENT	PLASTICITY INDEX, %	PERCENT PASSING No. 200 SIEVE
38	MC-1B	115	14		
22	MC-2B	104	18		
12	MC-3B	100	19		
19	MC-4B	106	18		
18	MC-5B	92	24		
50 6"	MC				



BORING NUMBER EB-14

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME The Forum at Rancho San Antonio
PROJECT NUMBER 905-1-1
PROJECT LOCATION 23500 Cristo Rey Drive, Cupertino, CA
DATE STARTED 7/14/16 **DATE COMPLETED** 7/14/16
GROUND ELEVATION 388 FT +/- **BORING DEPTH** 19.9 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE **LONGITUDE**
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
388.0	0	▨	**Sandy Lean Clay (CL) [Qc]** very stiff, moist, dark brown to brown, fine to coarse sand, some fine subangular to subrounded gravel, moderate plasticity
384.5		▨	**Fat Clay with Sand (CH) [Qc]]** hard, moist, light brown with reddish brown mottles, fine sand, high plasticity Liquid Limit = 52, Plastic Limit = 27
383.0	5	▨	**Sandy Lean Clay (CL) [QTsc]** hard, moist, light brown with reddish brown mottles, fine to medium sand, trace fine angular to subangular gravel, moderate plasticity
	10		
	15		
368.1	20		Bottom of Boring at 19.9 feet.
	25		

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

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
15	MC-1B	103	17			○
12	MC-2B	97	23	25		>4.5 ○
36	MC-3B	112	16			>4.5 ○
36	MC					>4.5 ○
50 6"	MC-5B	109	18			>4.5 ○
50 5"	MC					>4.5 ○

CORNERSTONE EARTH GROUP

BORING NUMBER EB-15

PAGE 1 OF 1

PROJECT NAME The Forum at Rancho San Antonio
PROJECT NUMBER 905-1-1
PROJECT LOCATION 23500 Cristo Rey Drive, Cupertino, CA
DATE STARTED 7/14/16 **DATE COMPLETED** 7/14/16
GROUND ELEVATION 395 FT +/- **BORING DEPTH** 10 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
∇ **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)	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)
395.0	0	**Lean Clay with Sand (CL)** hard, moist, brown, fine to medium sand, moderate plasticity							
			17	MC-1B	91	17			>4.5 ○
			14	MC-2B	103	16			○
	5	becomes very stiff	21	MC-3B	93	19			○
388.5		**Clayey Sand with Gravel (SC) [QTsc]** medium dense, moist, brown with reddish brown mottles, fine to coarse sand, fine to coarse subangular to subrounded gravel, trace cobbles							
			34	MC					
385.0	10	Bottom of Boring at 10.0 feet.							
	15								
	20								
	25								

BORING NUMBER EB-16

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME The Forum at Rancho San Antonio
PROJECT NUMBER 905-1-1
PROJECT LOCATION 23500 Cristo Rey Drive, Cupertino, CA
DATE STARTED 7/14/16 **DATE COMPLETED** 7/14/16
GROUND ELEVATION 404 FT +/- **BORING DEPTH** 20 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
∇ **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
1.0 2.0 3.0 4.0

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
404.0	0	▩	**Sandy Lean Clay (CL) [Fill]** hard, moist, brown, moist, fine to medium sand, some fine subangular to subrounded gravel, moderate plasticity	26	MC-1A	110	15			>4.5 ○
				28	MC-2A	102	16			>4.5 ○
400.0		▨	**Clayey Sand with Gravel (SC) [QTsc]** medium dense, moist, brown and reddish brown mottled, fine to coarse sand, some fine to coarse subangular to subrounded gravel, some cobbles							
	5			51	MC-3B	103	24		35	
				50	MC					
	10									
			becomes dense	39	SPT-5		18		30	
	15									
				60	MC					
384.0	20		Bottom of Boring at 20.0 feet.							
	25									

BORING NUMBER EB-17

CORNERSTONE EARTH GROUP

PROJECT NAME The Forum at Rancho San Antonio
PROJECT NUMBER 905-1-1
PROJECT LOCATION 23500 Cristo Rey Drive, Cupertino, CA
DATE STARTED 7/14/16 **DATE COMPLETED** 7/14/16
GROUND ELEVATION 408 FT +/- **BORING DEPTH** 10 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
∇ **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
1.0 2.0 3.0 4.0

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
408.0	0	▨	**Sandy Lean Clay with Gravel (CL) [Fill]** hard, moist, brown, fine to coarse sand, fine subangular to subrounded gravel, moderate plasticity							
				28	MC-1A	92	14			>4.5 ○
				18	MC-2A	94	14			>4.5 ○
403.5	5	▨	**Sandy Lean Clay (CL)** hard, moist, brown, moist, fine to medium sand, some fine subangular to subrounded gravel, moderate plasticity	19	MC					>4.5 ○
401.0		▨	**Clayey Sand with Gravel (SC) [QTsc]** medium dense, moist, brown with reddish brown mottles, fine to coarse sand, some fine to coarse subangular to subrounded gravel							
				46	MC-4A	113	13			
398.0	10		Bottom of Boring at 10.0 feet.							
	15									
	20									
	25									

BORING NUMBER EB-18

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME The Forum at Rancho San Antonio

PROJECT NUMBER 905-1-1

PROJECT LOCATION 23500 Cristo Rey Drive, Cupertino, CA

DATE STARTED 7/14/16 **DATE COMPLETED** 7/14/16

GROUND ELEVATION 410 FT +/- **BORING DEPTH** 15 ft.

DRILLING CONTRACTOR Exploration Geoservices, Inc.

LATITUDE **LONGITUDE**

DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger

GROUND WATER LEVELS:

LOGGED BY DL

▽ **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

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
410.0	0	**Clayey Sand with Gravel (SC) [Fill]** medium dense, moist, brown with reddish brown mottles, fine to coarse sand, fine to coarse subangular to subrounded gravel, some cobbles							
			31	MC-1B	107	15		32	
			31	MC-2B	106	21			
405.5	5	**Sandy Lean Clay (CL) [QTsc]** hard, moist, brown with reddish brown mottles, fine to medium sand, trace fine gravel, moderate plasticity							
			38	MC-3B	106	22			>4.5 ○
	10		48	MC					>4.5 ○
			54	MC-5B	114	17			
395.0	15	Bottom of Boring at 15.0 feet.							
	20								
	25								

BORING NUMBER EB-19

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME The Forum at Rancho San Antonio
PROJECT NUMBER 905-1-1
PROJECT LOCATION 23500 Cristo Rey Drive, Cupertino, CA
DATE STARTED 7/12/16 **DATE COMPLETED** 7/12/16
GROUND ELEVATION 414 FT +/- **BORING DEPTH** 25 ft.
DRILLING CONTRACTOR Britton Exploration, Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD CME Track Rig, 6 inch Solid Flight 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
1.0 2.0 3.0 4.0

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
414.0	0		**Clayey Sand with Gravel (SC) [QTsc]** medium dense, moist, brown with reddish brown mottles, fine to coarse sand, fine to coarse subangular to subrounded gravel							
				25	MC-1B	107	13			
				59	MC-2B	120	11			
	5									
				53	MC-3B	118	11			
407.0			**Lean Clay with Sand (CL) [QTsc]** hard, moist, brown with reddish brown mottles, fine to coarse sand, moderate plasticity							
				35	MC-4B	109	20			>4.5 ○
	10									
				46	MC					>4.5 ○
	15									
				39	MC					>4.5 ○
	20									
392.0			**Lean Clay (CL) [QTsc]** hard, moist, gray with reddish brown mottles, some fine sand, moderate plasticity							
				31	MC					>4.5 ○
389.0	25		Bottom of Boring at 25.0 feet.							

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 9/8/16 14:12 - P:\DRAFTING\GINT FILES\905-1-1 FORUM AT RANCHO SAN ANTONIO.GPJ

BORING NUMBER EB-20

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME The Forum at Rancho San Antonio
PROJECT NUMBER 905-1-1
PROJECT LOCATION 23500 Cristo Rey Drive, Cupertino, CA
DATE STARTED 7/14/16 **DATE COMPLETED** 7/14/16
GROUND ELEVATION 425 FT +/- **BORING DEPTH** 15 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY DL
▽ **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
1.0 2.0 3.0 4.0

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
425.0	0	▩	**Sandy Lean Clay (CL) [Fill]** hard, moist, brown, fine to coarse sand, some fine subangular to subrounded gravel, moderate plasticity	20	MC-1B	98	16			>4.5 ○
				20	MC-2B	110	16			>4.5 ○
420.5	5	▩	**Clayey Sand with Gravel (SC) [Fill]** medium dense, moist, brown with reddish brown mottles, fine to coarse sand, fine to coarse subangular to subrounded gravel	20	MC-3B	112	13			
417.5		▨	**Sandy Lean Clay (CL) [QTsc]** hard, moist, brown with reddish brown mottles, fine to medium sand, fine to coarse gravel, low plasticity	28	MC-4B	106	19			>4.5 ○
	10									
412.5		▨	**Clayey Sand with Gravel (SC) [QTsc]** dense, moist, brown with reddish brown mottles, fine to coarse sand, fine to coarse subangular to subrounded gravel, trace cobbles	58	MC					
410.0	15		Bottom of Boring at 15.0 feet.							
	20									
	25									

BORING NUMBER EB-21

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME The Forum at Rancho San Antonio

PROJECT NUMBER 905-1-1

PROJECT LOCATION 23500 Cristo Rey Drive, Cupertino, CA

DATE STARTED 7/14/16 **DATE COMPLETED** 7/14/16

GROUND ELEVATION 430 FT +/- **BORING DEPTH** 10 ft.

DRILLING CONTRACTOR Exploration Geoservices, Inc.

LATITUDE **LONGITUDE**

DRILLING METHOD Mobile B-53, 8 inch Hollow-Stem Auger

GROUND WATER LEVELS:

LOGGED BY DL

▽ **AT TIME OF DRILLING** Not Encountered

NOTES

▼ **AT END OF DRILLING** Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

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 ○ HAND PENETROMETER △ TORVANE ● UNCONFINED COMPRESSION ▲ UNCONSOLIDATED-UNDRAINED TRIAXIAL 1.0 2.0 3.0 4.0
430.0	0	▩	**Sandy Lean Clay (CL) [Fill]** hard, moist, brown, fine to medium sand, some fine to coarse angular to subangular gravel, moderate plasticity	42	MC-1A	106	13			○ >4.5
427.5		▩	**Clayey Sand with Gravel (SC) [Fill]** medium dense, moist, brown, fine to coarse sand, fine to coarse subangular to subrounded gravel	21	MC-2B	92	11			
	5	▩		20	MC					
422.5		▨	**Sandy Lean Clay (CL) [QTsc]** very stiff, moist, brown with reddish brown mottles, fine to medium sand, moderate plasticity	38	MC-4B	110	16			○
420.0	10		Bottom of Boring at 10.0 feet.							
	15									
	20									
	25									

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 9/8/16 14:12 - P:\DRAFTING\GINT FILES\905-1-1 FORUM AT RANCHO SAN ANTONIO.GPJ

BORING NUMBER EB-22

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME The Forum at Rancho San Antonio
PROJECT NUMBER 905-1-1
PROJECT LOCATION 23500 Cristo Rey Drive, Cupertino, CA
DATE STARTED 7/12/16 **DATE COMPLETED** 7/12/16
GROUND ELEVATION 364 FT +/- **BORING DEPTH** 10 ft.
DRILLING CONTRACTOR Britton Exploration, Inc.
LATITUDE **LONGITUDE**
DRILLING METHOD CME Track Rig, 6 inch Solid Flight Auger
GROUND WATER LEVELS:
LOGGED BY OL
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

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

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
364.0	0	▨	**Sandy Lean Clay (CL) [Fill]** hard, dry, brown and reddish brown mottled, fine to coarse sand, some fine angular to subangular gravel, moderate plasticity							
362.5		▨	**Sandy Lean Clay (CL) [QTsc]** hard, moist, light brown with reddish brown mottles, fine to medium sand, some fine subangular to subrounded gravel, moderate plasticity	29	MC-1A	104	12			>4.5 ○
		▨		60	MC-2B	104	19			>4.5 ○
	5	▨		46	MC					>4.5 ○
		▨		72	MC-4A	96	27			>4.5 ○
354.0	10		Bottom of Boring at 10.0 feet.							
	15									
	20									
	25									

BORING NUMBER EB-23

PAGE 1 OF 1

CORNERSTONE EARTH GROUP

PROJECT NAME The Forum at Rancho San Antonio
PROJECT NUMBER 905-1-1
PROJECT LOCATION 23500 Cristo Rey Drive, Cupertino, CA
DATE STARTED 3/27/17 **DATE COMPLETED** 3/27/17
GROUND ELEVATION 351 FT +/- **BORING DEPTH** 28.5 ft.
DRILLING CONTRACTOR Exploration Geoservices, Inc.
LATITUDE 37.3391731° **LONGITUDE** -122.0889961°
DRILLING METHOD Mobile B-56, 8 inch Hollow-Stem Auger
GROUND WATER LEVELS:
LOGGED BY OL
▽ **AT TIME OF DRILLING** Not Encountered
NOTES
▼ **AT END OF DRILLING** Not Encountered

This log is a part of a report by Cornerstone Earth Group, and should not be used as a stand-alone document. This description applies only to the location of the exploration at the time of drilling. Subsurface conditions may differ at other locations and may change at this location with time. The description presented is a simplification of actual conditions encountered. Transitions between soil types may be gradual.

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
351.0 350.8	0	2½ inches asphalt concrete over 12 inches aggregate base							
349.8		**Clayey Sand with Gravel (SC) [Fill]** very dense, moist, brown with gray mottles, fine to coarse sand, fine subangular to subrounded gravel	50 4"	MC-1B	128	11			
348.8		**Clayey Sand with Gravel (SC) [QTsc]** dense to very dense, moist, brown with reddish brown mottles, fine to coarse sand, fine subangular to subrounded gravel	54	MC-2		18		18	
	5		50 6"	MC-3B	121	16			
	10		53	SPT-4		13		20	
339.5		**Sandy Lean Clay (CL) [QTsc]** hard, moist, brown with reddish brown mottles, fine to medium sand, low to moderate plasticity	44	SPT-5		22			>4.5 ○
	15								
334.5		**Lean Clay with Sand (CL) [QTsc]** very stiff, moist, brown with reddish brown mottles, fine to medium sand, moderate plasticity	35	SPT-6B		21			○
331.5	20	**Clayey Sand with Gravel (SC) [QTsc]** dense, moist, brown with reddish brown mottles, fine to coarse sand, fine subangular to subrounded gravel							
329.0		**Sandy Lean Clay (CL) [QTsc]** hard, moist, brown with reddish brown mottles, fine to medium sand, low to moderate plasticity	58	SPT-7		20			>4.5 ○
	25		65	MC-8B	114	18			>4.5 ○
322.5		Bottom of Boring at 28.5 feet.							

CORNERSTONE EARTH GROUP2 - CORNERSTONE 0812.GDT - 4/11/17 10:20 - P:\DRAFTING\GINT FILES\905-1-1 FORUM AT RANCHO SAN ANTONIO.GPJ

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 119 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 106 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.

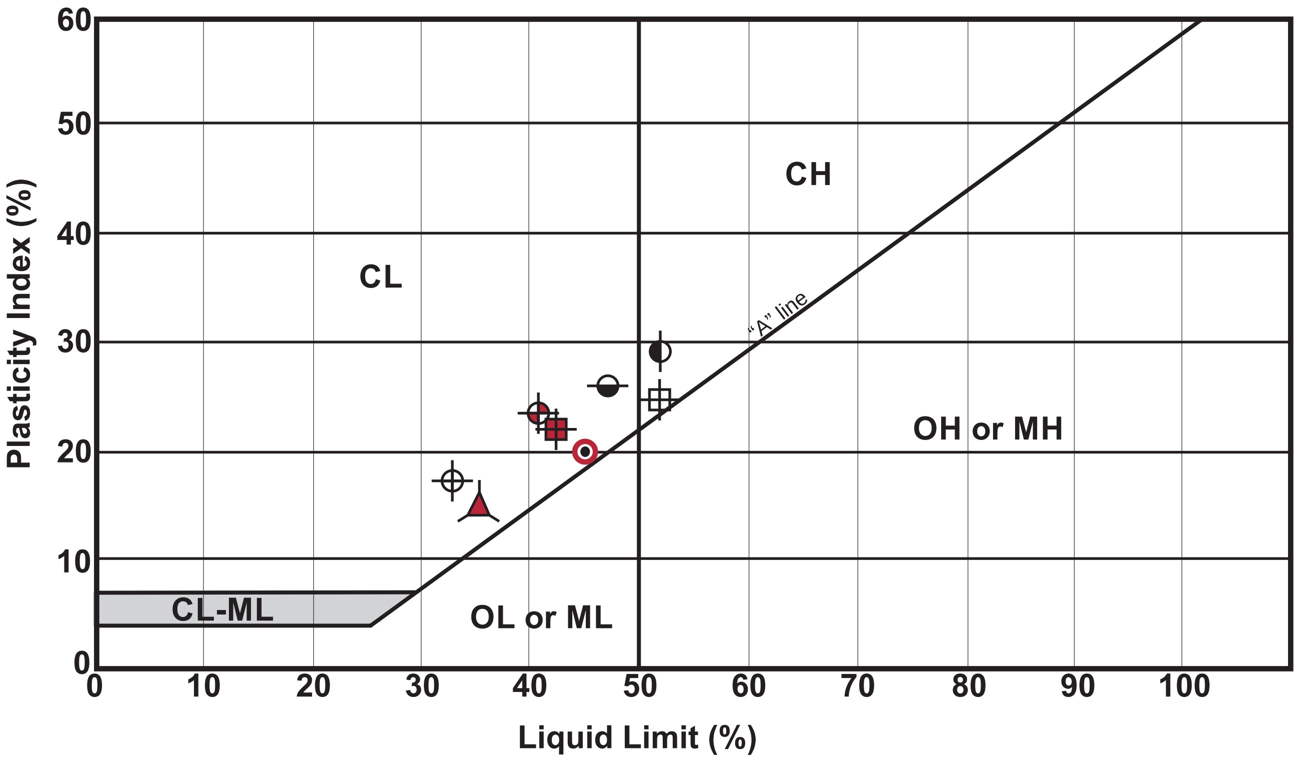
Washed Sieve Analyses: The percent soil fraction passing the No. 200 sieve (ASTM D1140) was determined on 12 samples of the subsurface soils to aid in the classification of these soils. Results of these tests are shown on the boring logs at the appropriate sample depths.

Plasticity Index: Eight 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.

Undrained-Unconsolidated Triaxial Shear Strength: The undrained shear strength was determined on four relatively undisturbed sample(s) by unconsolidated-undrained triaxial shear strength testing (ASTM D2850). The results of this test are included as part of this appendix.

Corrosion: Corrosion testing was completed on six soil samples collected from our exploratory borings between depths of 2 and 9 feet. The laboratory testing included pH, resistivity, chloride, and sulfate testing. An evaluation prepared by JDH Corrosion Consultants is included in Appendix C.

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	19.0	9	41	18	24	—	Clayey Sand (SC) [QTsc] (CL fines)
■	EB-3	5.5	17	43	21	22	—	Sandy Lean Clay (CL) [Qtsc]
◎	EB-6	6.0	16	46	26	20	—	Sandy Lean Clay (CL) [Qtsc]
▲	EB-7	19.0	11	36	20	16	—	Clayey Sand (SC) [QTsc] (CL fines)
◐	EB-9	2.0	17	52	23	29	—	Sandy Fat Clay (CH) [Fill]
⊕	EB-11	3.5	15	33	16	17	—	Clayey Sand (SC) (CL fines)
⊖	EB-12	2.0	17	48	21	27	—	Sandy Lean Clay (CL)
⊞	EB-14	4.0	23	52	27	25	—	Fat Clay with Sand (CH) [Qtsc]

CORNERSTONE
EARTH GROUP

Plasticity Index Testing Summary

The Forum at Rancho San Antonio
Cupertino, CA

Project Number
905-1-1

Figure Number
Figure B1

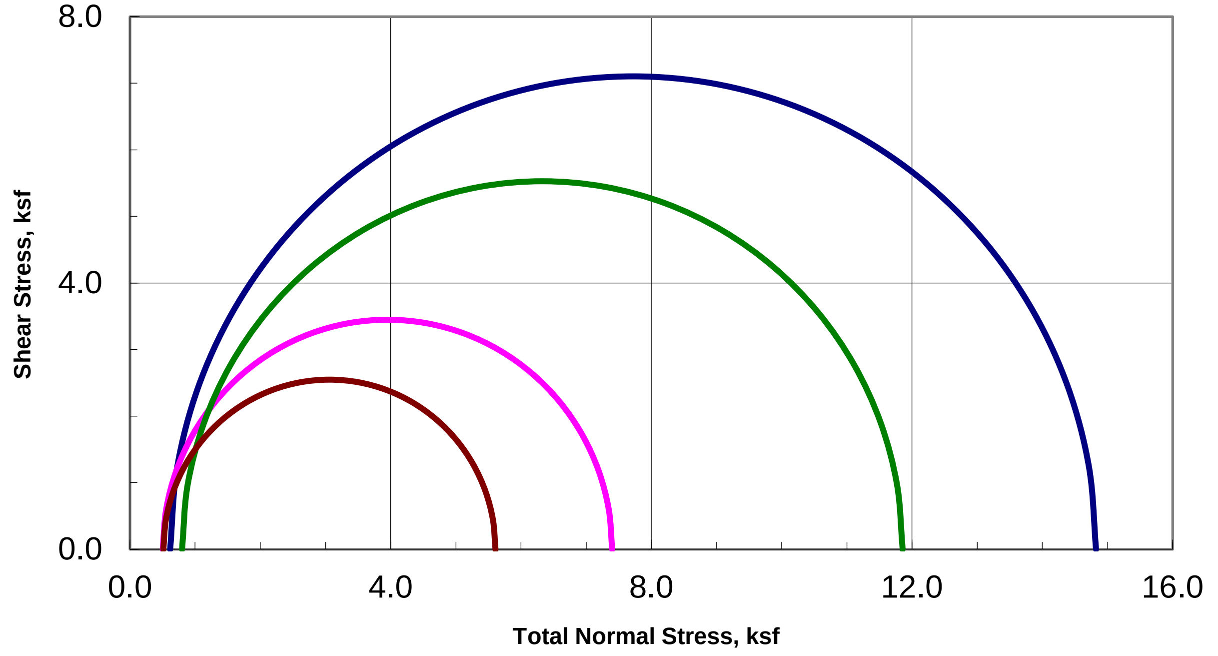
Date
August 2016

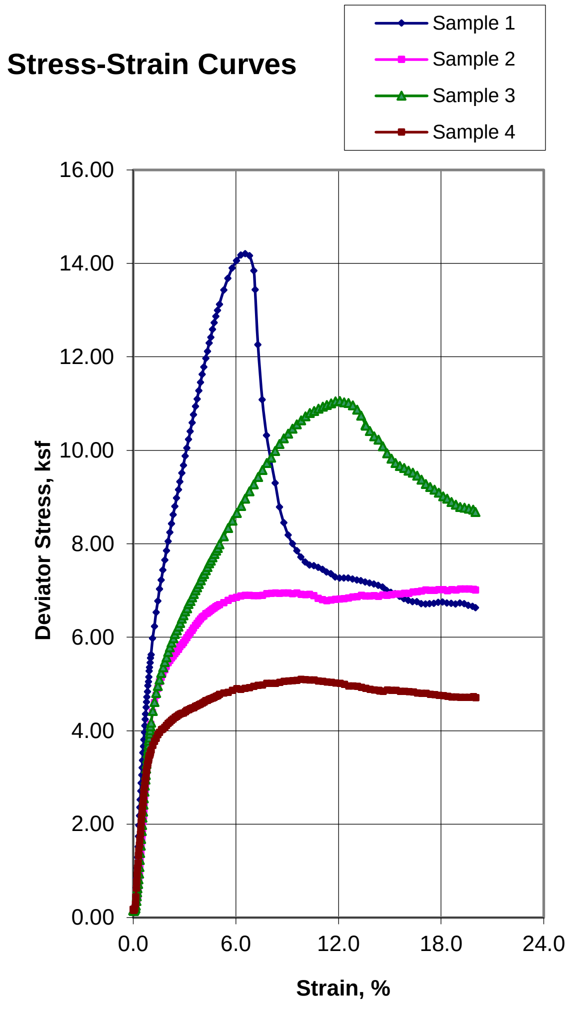
Drawn By
FLL

Cooper Testing Labs, Inc.
937 Commercial Street
Palo Alto, CA 94303

Unconsolidated-Undrained Triaxial Test

ASTM D2850




Stress-Strain Curves

Sample Data

	1	2	3	4
Moisture %	25.9	23.8	27.5	18.4
Dry Den,pcf	100.2	103.2	97.0	109.6
Void Ratio	0.714	0.663	0.769	0.566
Saturation %	99.8	98.6	98.5	89.5
Height in	5.03	5.05	5.07	5.02
Diameter in	2.42	2.41	2.42	2.40
Cell psi	4.3	3.5	5.6	3.6
Strain %	6.54	15.00	12.08	9.82
Deviator, ksf	14.207	6.896	11.057	5.096
Rate %/min	1.00	1.00	1.00	1.00
in/min	0.050	0.050	0.051	0.050
Job No.:	**640-1016**			
Client:	**Cornerstone Earth Group**			
Project:	**Forum at Rancho San Antonio - 905-1-1**			
Boring:	EB-3	EB-5	EB-6	EB-9
Sample:	3B	3B	4B	2C
Depth ft:	6.0	6.0	9.5	4.0

Visual Soil Description

Sample #	
1	Strong Brown Sandy CLAY
2	Strong Brown Clayey SAND, trace Gravel
3	Strong Brown Sandy CLAY
4	Strong Brown Clayey SAND w/ Gravel

Remarks:

Note: Strengths are picked at the peak deviator stress or 15% strain which ever occurs first per ASTM D2850.

APPENDIX C: SITE CORROSIVITY EVALUATION

JDH CORROSION CONSULTANTS REPORT DATED AUGUST 16, 2016

jdh corrosion consultants, inc.

Protecting the infrastructure
through innovative
Corrosion Engineering Solutions

August 16, 2016

Cornerstone Earth Group
1259 Oakmead Parkway
Sunnyvale, California 94085

Attention: **Mr. Paul Mateo, P.E.**
Project Engineer

Subject: **Site Corrosivity Evaluation**
Forum at Rancho San Antonio
Cupertino, CA
Job #: 905-1-1

Dear Paul,

In accordance with your request, we have reviewed the laboratory soils data for the above referenced project site. Our evaluation of these results and our corresponding recommendations for corrosion control for the above referenced project foundations and buried site utilities are presented herein for your consideration.

Soil Testing & Analysis

Soil Chemical Analysis

Six (6) soil samples from the project site were chemically analyzed for corrosivity by ***Cooper Testing Laboratories***. Each sample was analyzed for chloride and sulfate concentration, pH, resistivity at 100% saturation and moisture percentage. The test results are presented in Cooper Testing Laboratories *Corrosivity Test Summary* dated 8/1/2016. The results of the chemical analysis were as follows:

Soil Laboratory Analysis

Chemical Analysis	Range of Results	Corrosion Classification*
Chlorides	<2 – 11 mg/kg	Non-corrosive*
Sulfates	11 – 31 mg/kg	Non-corrosive**
pH	6.6 – 7.4	Non-corrosive *
Moisture (%)	8.9 – 25.2 %	Not-applicable
Resistivity at 100% Saturation	1,580 – 4,244 ohm-cm	**Corrosive to Moderately Corrosive***

* With respect to bare steel or ductile iron.
** With respect to mortar coated steel

Discussion

Reinforced Concrete Foundations

Due to the low levels of water-soluble sulfates found in these soils, there is no special requirement for sulfate resistant concrete to be used at this site. The type of cement used should be in accordance with California Building Code (CBC) for soils which have less than 0.10 percent by weight of water soluble sulfate (SO_4) in soil and the minimum depth of cover for the reinforcing steel should be as specified in CBC as well.

Underground Metallic Pipelines

The soils at the project site are generally considered to be “corrosive” to ductile/cast iron, steel and dielectric coated steel based on the saturated resistivity measurements. Therefore, special requirements for corrosion control are required for buried metallic utilities at this site depending upon the critical nature of the piping. Pressure piping systems such as domestic and fire water should be provided with appropriate coating systems and cathodic protection, where warranted. In addition, all underground pipelines should be electrically isolated from above grade structures, reinforced concrete structures and copper lines in order to avoid potential galvanic corrosion problems.

LIMITATIONS

The conclusions and recommendations contained in this report are based on the information and assumptions referenced herein. All services provided herein were performed by persons who are experienced and skilled in providing these types of services and in accordance with the standards of workmanship in this profession. No other warrantees or guarantees, expressed or implied, is provided.

We thank you for the opportunity to be of service to ***Cornerstone Earth Group*** on this project and trust that you find the enclosed information satisfactory. If you have any questions, or if we can be of any additional assistance, please feel free to contact us at (925) 927-6630.

Respectfully submitted,

Brendon Hurley

Brendon Hurley
JDH Corrosion Consultants, Inc.
Field Technician

Mohammed Ali

Mohammed Ali, P.E.
JDH Corrosion Consultants, Inc.
Principal

Registered Professional Engineer
Mohammed Ali
No. CR 1035
Exp. June 30, 2017
Corrosion
State of California

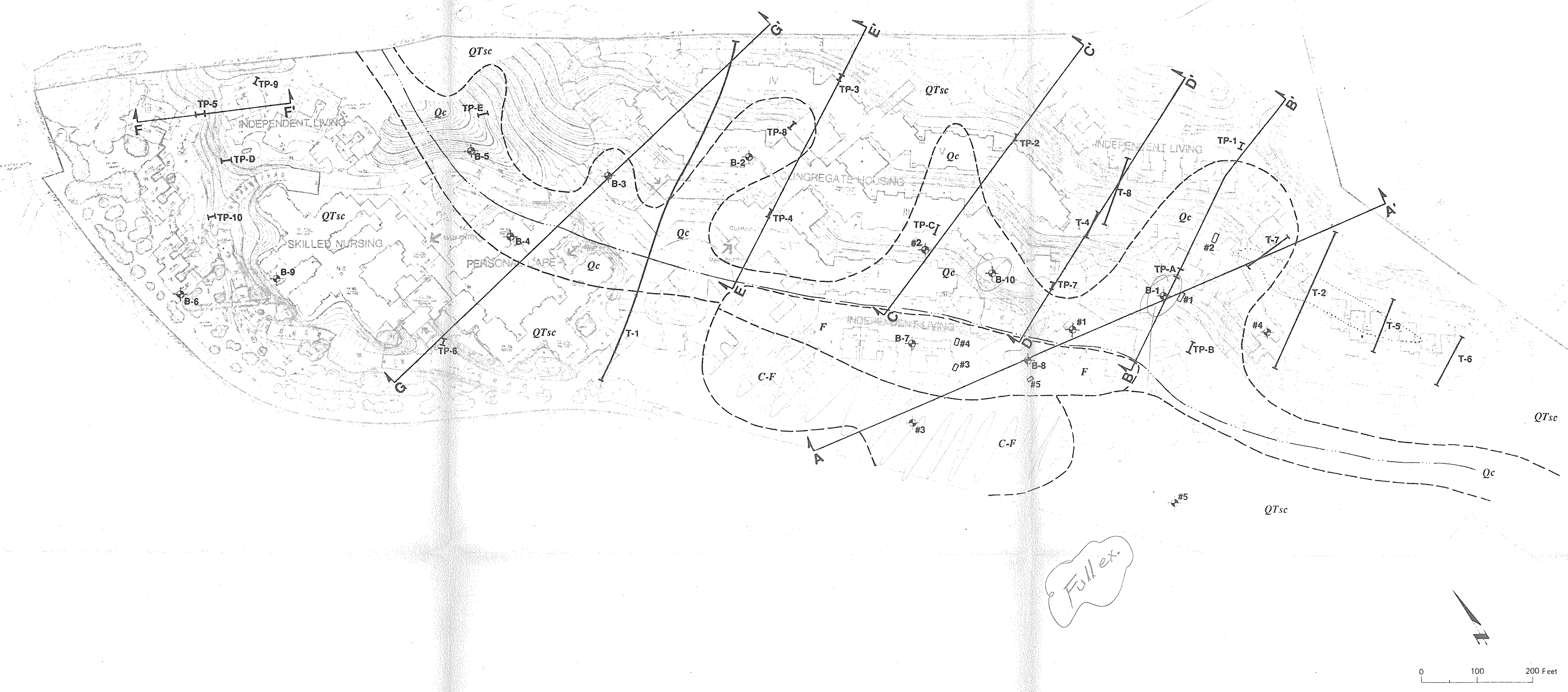
CC: File 16177

Corrosivity Tests Summary

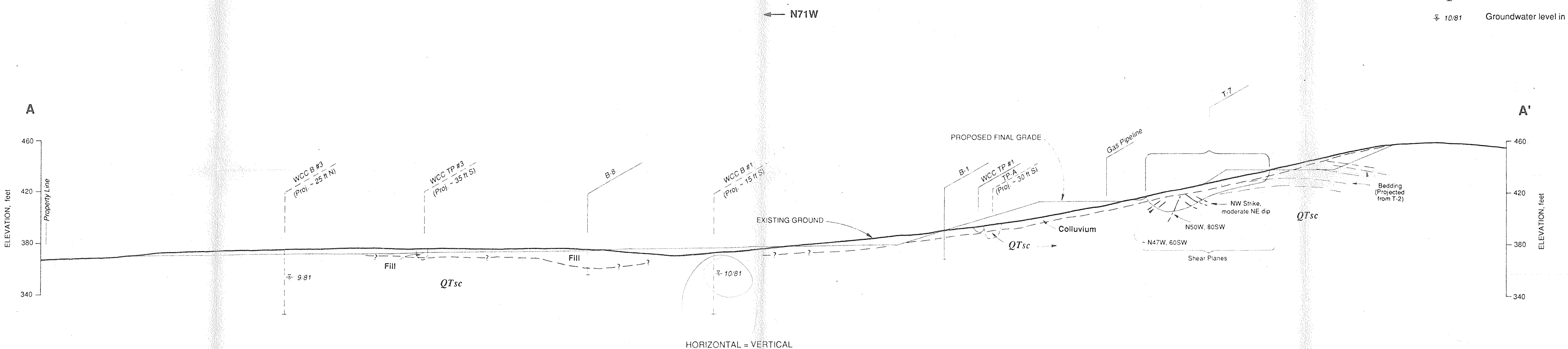
CTL #: 640-1016
Date: 8/1/2016
Tested By: PJ
Checked: PJ
Client: Cornerstone Earth Group
Project: Forum at Rancho San Antonio
Proj. No: 905-1-1
Remarks:

Sample Location or ID			**Resistivity @ 15.5 °C (Ohm-cm)**			**Chloride**	**Sulfate**		**pH**	**ORP**		**Sulfide**	**Moisture**	**Soil Visual Description**
			As Rec.	**Min**	**Sat.**	**mg/kg**	**mg/kg**	**%**		(Redox)		Qualitative	At Test	
						Dry Wt.	Dry Wt.	Dry Wt.		E_H (mv)	At Test	by Lead	**%**	
Boring	**Sample, No.**	**Depth, ft.**	ASTM G57	Cal 643	ASTM G57	ASTM D4327	ASTM D4327	ASTM D4327	ASTM G51	ASTM G200	Temp °C	Acetate Paper	ASTM D2216	
EB-3	2A	3.5	-	-	1,700	7	13	0.0013	7.3	-	-	-	25.2	Yellowish Brown Sandy CLAY
EB-3	4A	9.0	-	-	1,856	11	19	0.0019	7.2	-	-	-	22.4	Yellowish Brown Sandy CLAY
EB-8	1B	2.0	-	-	4,098	<2	11	0.0011	7.2	-	-	-	8.9	Yellowish Brown Sandy CLAY w/ Gravel
EB-8	3A	5.5	-	-	4,244	3	13	0.0013	7.4	-	-	-	10.2	Olive Silty SAND

APPENDIX D: PREVIOUS REPORTS BY EARTH SCIENCE ASSOCIATES AND EARTH SYSTEMS CONSULTANTS



Full ex.



HORIZONTAL = VERTICAL
1" = 40'

EXPLANATION

GEOLOGIC UNITS

F	Man-made fill
C-F	Areas modified by cut and fill activity
Qc	Colluvium; may include minor areas of alluvial stream channel deposits
QTsc	Santa Clara formation; clay, silt, sand, and gravel; massive to moderately well bedded; discontinuous beds

MAP SYMBOLS

	Geologic contact, approximately located
	Ephemeral stream course
T-3	Exploratory trench, ESA, 1985
TP-E	Exploratory test pit, ESA, 1985
TP-4	Exploratory test pit, ESA, 1988
B-4	Exploratory boring, ESA, 1988
#4	Exploratory test pit, Woodward-Clyde Consultants, 1981
#4	Exploratory boring, Woodward-Clyde Consultants, 1981
	Disturbed zone where shear planes are concentrated
A A'	Geologic section location

Base map from Conceptual Grading Plan by Brian Kangas Foulk

CROSS SECTIONS

	Original ground surface
	Proposed final grade
?	Geologic contact, queried where uncertain
	Apparent bedding in plane of section
	Shear planes in plane of section
T-7	Outline of exploratory trench, dashed where projected to plane of section
B-4	Exploratory boring, dashed where projected to plane of section
10/81	Groundwater level in borehole with date of measurement

Note

The logs of test pits and/or test borings and related information shown on this figure depict subsurface conditions only at those specific locations and at the particular time the excavation was made. The passage of time may result in a change in the soil and ground water at these locations. The geologic conditions shown between borings are interpretations, based on available data and made in accordance with accepted geotechnical principles and practices. The actual configuration of subsurface materials may differ widely from these interpretations.

C1025

Earth Sciences Associates
Palo Alto, California

FORUM LIFE CONTINUING CARE CENTER
CUPERTINO, CALIFORNIA
GEOLOGIC EXPLORATION MAP AND CROSS SECTION

Checked by T D Hunt Date 9-9-88
Approved by R C Harding Date 9-9-88
Project No. 3223A
Plate No. 1

Appendix A

FIELD EXPLORATION

The Phase II field exploration began on July 15, 1988 when a site reconnaissance was performed to locate and stake locations for the planned borings and test pits relative to the proposed grading plan. Generally, test pits were planned for areas of extensive cut in order to evaluate bedding characteristics and suitability of materials for fill; borings were located in areas proposed for fill to determine strength parameters of the in-place materials prior to burial.

Test pit excavations occurred on July 19 and 20, 1988, using a 24 inch wide bucket on a four-wheel drive "Extendahoe" backhoe. Ten test pits (TP-1 through TP-10) were excavated and ranged in depth from 10 to 17.3 feet. Hydraulic shoring was used to support the walls from caving during logging of the structural and lithologic properties. Bulk samples were taken from specific horizons for future lab analyses. Descriptive logs of the test pits with sampling depths are included in this Appendix.

Exploratory borings were drilled on July 25 and 26, 1988, with 5-1/4 inch diameter flight augers, using a track-mounted all-terrain type drill rig. Ten borings (B-1 through B-10) were drilled to depths ranging from 18.3 to 26.5 feet. Sampling of materials was accomplished with a 2-1/2 inch I.D. Modified California Sampler driven into the material by a 140-pound hammer falling 30 inches. Blow counts per 0.5 foot penetration were recorded, and brass liners with core samples were capped and retained. The borings were logged by inspection of cuttings and core samples. Bulk samples were gathered from specific horizons, and both these and the brass liners were retained for laboratory analyses. Logs of the borings are included in this Appendix.

All the backhoe pits and auger borings were backfilled with the excavated material after completion. The backhoe work was performed by Hatton's Backhoe Service of Pacifica. Hard Core Drilling of Oakland provided the drilling services. Supervision, logging, and sampling of the subsurface exploration was performed by T. D. Hunt, Senior Geologist with Earth Sciences Associates.

EXPLANATION FOR BORING LOGS

Classification Column

Classification of soils is based on the Unified Soils Classification System (USCS). Soils are identified in this column by hatching symbols on the left and USCS letter symbols on the right. Solid lines indicate contacts between major soil units; dashed lines indicate lenses within major units. Indurated material such as shale, sandstone, etc., is indicated by standard geologic hatching symbols (see Graphic Log Symbols).

Sample Column

Indicates sample number, type and depth interval of material sampled. The following symbols are used to indicate type of sample:

Bag	B	Core, wrapped and sealed	WC
Jar	J	Sample used for unconfined compressive strength test in field	UC
Shelby Tube	ST		
Box	X		
Liner	L		

On the right of the sample column is a small column for recording results of field tests, e.g., blow counts, penetrometer readings, torvane and vane shear test results.

Mode Column

Indicates method by which the hole is advanced, using the following symbols:

Flight Auger	AD	Pitcher Barrel sampling	PB
Bucket Auger	BD	Drive sampling	DR
Spin Auger	SD	Push sampling	P
Rotary Boring	RD	Coring	C

The recovery ratio (length of sample recovered/length of sample attempted) is shown in this column.

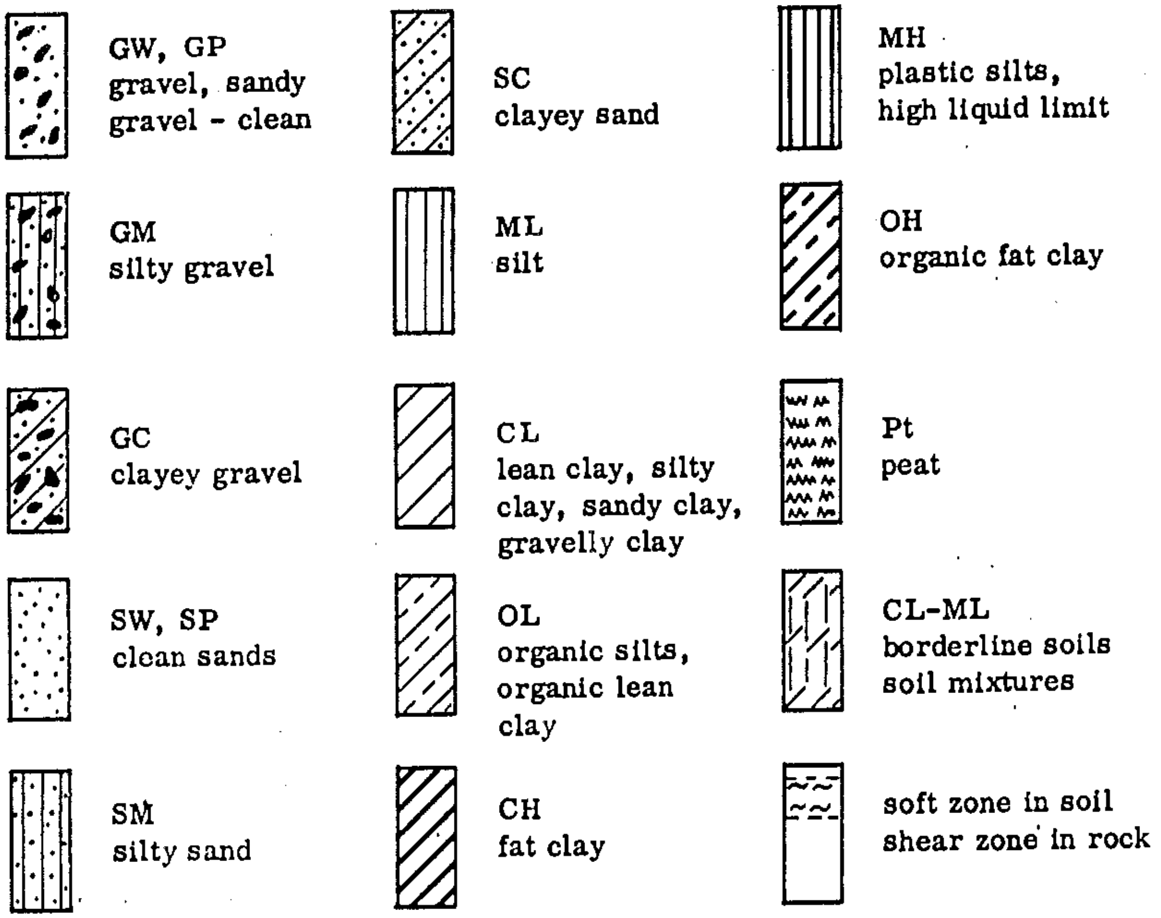
Remarks Column

Records pertinent comments not otherwise recorded in log, e.g.:

> Drilling methods and equipment, type and size of bit, etc.;
> sampling details;
> drilling difficulties;
> drill water circulation and water losses;
> casing used, piezometers installed, backfilling hole, reasons for terminating hole.

GRAPHIC LOG SYMBOLS

SOILS



GW, GP
gravel, sandy gravel - clean

SC
clayey sand

MH
plastic silts, high liquid limit

GM
silty gravel

ML
silt

OH
organic fat clay

GC
clayey gravel

CL
lean clay, silty clay, sandy clay, gravelly clay

Pt
peat

SW, SP
clean sands

OL
organic silts, organic lean clay

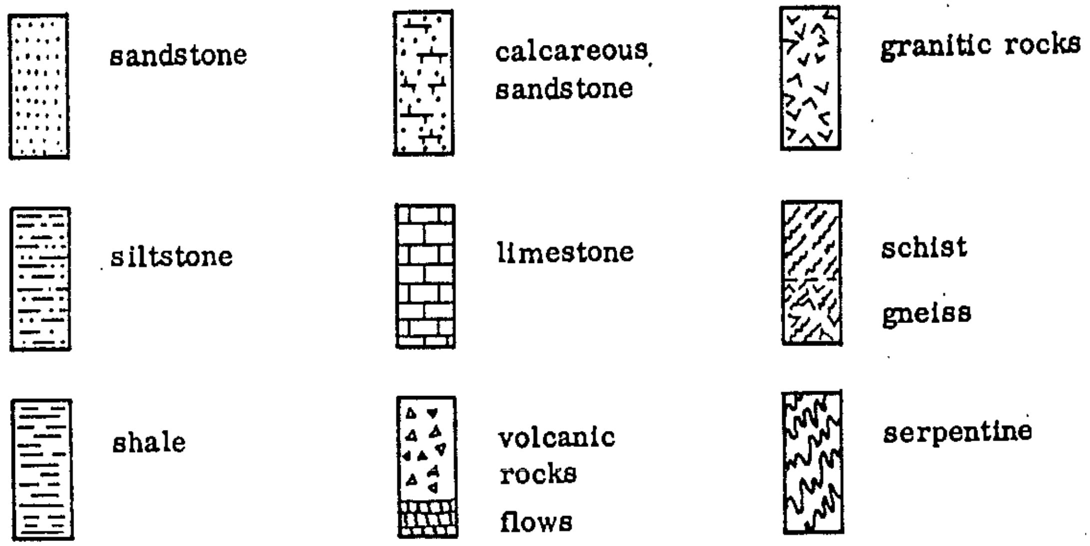
CL-ML
borderline soils
soil mixtures

SM
silty sand

CH
fat clay

soft zone in soil
shear zone in rock

ROCKS



sandstone

calcareous sandstone

granitic rocks

siltstone

limestone

schist

gneiss

shale

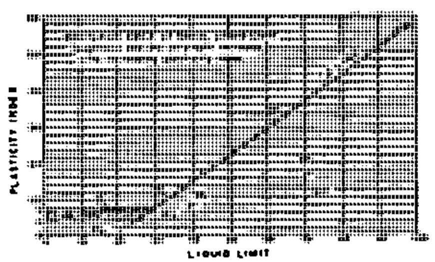
volcanic rocks

flows

serpentine

Earth Sciences Associates

UNIFIED SOIL CLASSIFICATION
INCLUDING IDENTIFICATION AND DESCRIPTION

<table>
<tr><th colspan="8">FIELD IDENTIFICATION PROCEDURES
(Excluding particles larger than 3 inches and basing fractions on estimated weights)</th><th>GROUP SYMBOLS a</th><th>TYPICAL NAMES</th><th>INFORMATION REQUIRED FOR DESCRIBING SOILS</th><th colspan="4">LABORATORY CLASSIFICATION CRITERIA</th></tr>
<tr><td rowspan="8">COARSE GRAINED SOILS
More than half of material is <u>larger</u> than No. 200 sieve size b</td><td rowspan="16">(The No. 200 sieve size is about the smallest particle visible to the naked eye)</td><td rowspan="4">GRAVELS
More than half of coarse fraction is larger than No. 4 sieve size</td><td rowspan="8">(For visual classification, the 1/4" size may be used as equivalent to the No. 4 sieve size.)</td><td rowspan="2">CLEAN GRAVELS
(Little or no fines)</td><td colspan="3">Wide range in grain size and substantial amounts of all intermediate particle sizes</td><td>GW</td><td>Well graded gravels, gravel-sand mixtures, little or no fines.</td><td rowspan="8">Give typical name; indicate approximate percentages of sand and gravel, max. size, angularity, surface condition, and hardness of the coarse grains; local or geologic name and other pertinent descriptive information; and symbol in parentheses
For undisturbed soils add information on stratification, degree of compactness, cementation, moisture conditions and drainage characteristics.
EXAMPLE:-
<u>Silty sand</u>, gravelly; about 20% hard, angular gravel particles ½-in maximum size; rounded and subangular sand grains coarse to fine; about 15% non-plastic fines with low dry strength; well compacted and moist in place; alluvial sand; (SM)</td><td rowspan="17">Use grain size curve in identifying the fractions as given under field identification</td><td rowspan="8">Determine percentages of gravel and sand from grain size curve.
Depending on percentage of fines (fraction smaller than No. 200 sieve size) coarse grained soils are classified as follows:
Less than 5% GW, GP, SW, SP.
More than 12% GM, GC, SM, SC.
5% to 12% <u>Borderline</u> cases requiring use of dual symbols.</td><td colspan="2">$C_u = \frac{D_{60}}{D_{10}}$ Greater than 4
$C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between one and 3</td></tr>
<tr><td colspan="3">Predominantly one size or a range of sizes with some intermediate sizes missing.</td><td>GP</td><td>Poorly graded gravels, gravel-sand mixtures, little or no fines.</td><td colspan="2">Not meeting all gradation requirements for GW</td></tr>
<tr><td rowspan="2">GRAVELS WITH FINES
(Appreciable amount of fines)</td><td colspan="3">Non-plastic fines (for identification procedures see ML below)</td><td>GM</td><td>Silty gravels, poorly graded gravel-sand-silt mixtures.</td><td>Atterberg limits below "A" line, or PI less than 4</td><td rowspan="2">Above "A" line with PI between 4 and 7 are <u>borderline</u> cases requiring use of dual symbols.</td></tr>
<tr><td colspan="3">Plastic fines (for identification procedures see CL below)</td><td>GC</td><td>Clayey gravels, poorly graded gravel-sand-clay mixtures.</td><td>Atterberg limits above "A" line with PI greater than 7</td></tr>
<tr><td rowspan="4">SANDS
More than half of coarse fraction is smaller than No. 4 sieve size.</td><td rowspan="2">CLEAN SANDS
(Little or no fines)</td><td colspan="3">Wide range in grain sizes and substantial amounts of all intermediate particle sizes</td><td>SW</td><td>Well graded sands, gravelly sands, little or no fines.</td><td colspan="2">$C_u = \frac{D_{60}}{D_{10}}$ Greater than 6
$C_c = \frac{(D_{30})^2}{D_{10} \times D_{60}}$ Between one and 3</td></tr>
<tr><td colspan="3">Predominantly one size or a range of sizes with some intermediate sizes missing.</td><td>SP</td><td>Poorly graded sands, gravelly sands, little or no fines.</td><td colspan="2">Not meeting all gradation requirements for SW</td></tr>
<tr><td rowspan="2">SANDS WITH FINES
(Appreciable amount of fines)</td><td colspan="3">Non-plastic fines (for identification procedures see ML below).</td><td>SM</td><td>Silty sands, poorly graded sand-silt mixtures.</td><td>Atterberg limits below "A" line or PI less than 4</td><td rowspan="2">Above "A" line with PI between 4 and 7 are <u>borderline</u> cases requiring use of dual symbols.</td></tr>
<tr><td colspan="3">Plastic fines (for identification procedures see CL below).</td><td>SC</td><td>Clayey sands, poorly graded sand-clay mixtures.</td><td>Atterberg limits above "A" line with PI greater than 7</td></tr>
<tr><td rowspan="8">FINE GRAINED SOILS
More than half of material is <u>smaller</u> than No. 200 sieve size.</td><td colspan="6">IDENTIFICATION PROCEDURES ON FRACTION SMALLER THAN No. 40 SIEVE SIZE</td><td rowspan="2"></td><td rowspan="2"></td><td rowspan="2"></td><td colspan="2" rowspan="2"></td></tr>
<tr><td colspan="3" rowspan="4">SILTS AND CLAYS
Liquid limit less than 50</td><td>DRY STRENGTH (CRUSHING CHARACTERISTICS)</td><td>DILATANCY (REACTION TO SHAKING)</td><td>TOUGHNESS (CONSISTENCY NEAR PLASTIC LIMIT)</td></tr>
<tr><td>None to slight</td><td>Quick to slow</td><td>None</td><td>ML</td><td>Inorganic silts and very fine sands, rock flour, silty or clayey fine sands with slight plasticity.</td><td rowspan="7">Give typical name; indicate degree and character of plasticity, amount and maximum size of coarse grains; color in wet condition, odor if any, local or geologic name, and other pertinent descriptive information; and symbol in parentheses.
For undisturbed soils add information on structure, stratification, consistency in undisturbed and remolded states, moisture and drainage conditions.
EXAMPLE:-
<u>Clayey silt</u>, brown; slightly plastic; small percentage of fine sand; numerous vertical root holes; firm and dry in place; loess; (ML)</td><td colspan="2" rowspan="7">
PLASTICITY CHART
FOR LABORATORY CLASSIFICATION OF FINE GRAINED SOILS</td></tr>
<tr><td>Medium to high</td><td>None to very slow</td><td>Medium</td><td>CL</td><td>Inorganic clays of low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays</td></tr>
<tr><td>Slight to medium</td><td>Slow</td><td>Slight</td><td>OL</td><td>Organic silts and organic silt-clays of low plasticity.</td></tr>
<tr><td colspan="3" rowspan="3">SILTS AND CLAYS
Liquid limit greater than 50</td><td>Slight to medium</td><td>Slow to none</td><td>Slight to medium</td><td>MH</td><td>Inorganic silts, micaceous or diatomaceous fine sandy or silty soils, elastic silts.</td></tr>
<tr><td>High to very high</td><td>None</td><td>High</td><td>CH</td><td>Inorganic clays of high plasticity, fat clays.</td></tr>
<tr><td>Medium to high</td><td>None to very slow</td><td>Slight to medium</td><td>OH</td><td>Organic clays of medium to high plasticity.</td></tr>
<tr><td colspan="5">HIGHLY ORGANIC SOILS</td><td colspan="3">Readily identified by color, odor, spongy feel and frequently by fibrous texture.</td><td>Pt</td><td>Peat and other highly organic soils.</td></tr>
</table>

a <u>Boundary classifications</u>.- Soils possessing characteristics of two groups are designated by combinations of group symbols. For example GW-GC, well graded gravel-sand mixture with clay binder.

b All sieve sizes on this chart are U.S. standard.

ADOPTED BY - CORPS OF ENGINEERS AND BUREAU OF RECLAMATION - JANUARY 1952

FIELD IDENTIFICATION PROCEDURES FOR FINE GRAINED SOILS OR FRACTIONS

These procedures are to be performed on the minus No. 40 sieve size particles, approximately 1/64 in. For field classification purposes, screening is not intended; simply remove by hand the coarse particles that interfere with the tests.

DILATANCY (Reaction to shaking)

After removing particles larger than No. 40 sieve size, prepare a pat of moist soil with a volume of about one-half cubic inch. Add enough water if necessary to make the soil soft but not sticky.

Place the pat in the open palm of one hand and shake horizontally, striking vigorously against the other hand several times. A positive reaction consists of the appearance of water on the surface of the pat which changes to a livery consistency and becomes glossy. When the sample is squeezed between the fingers, the water and gloss disappear from the surface, the pat stiffens, and finally it cracks or crumbles. The rapidity of appearance of water during shaking and of its disappearance during squeezing assist in identifying the character of the fines in a soil.

Very fine clean sands give the quickest and most distinct reaction whereas a plastic clay has no reaction. Inorganic silts, such as a typical rock flour, show a moderately quick reaction.

DRY STRENGTH (Crushing characteristics)

After removing particles larger than No. 40 sieve size, mold a pat of soil to the consistency of putty, adding water if necessary. Allow the pat to dry completely by oven, sun, or air drying, and then test its strength by breaking and crumbling between the fingers. This strength is a measure of the character and quantity of the colloidal fraction contained in the soil. The dry strength increases with increasing plasticity.

High dry strength is characteristic for clays of the CH group. A typical inorganic silt possesses only very slight dry strength. Silty fine sands and silts have about the same slight dry strength, but can be distinguished by the feel when powdering the dried specimen. Fine sand feels gritty whereas a typical silt has the smooth feel of flour.

TOUGHNESS (Consistency near plastic limit)

After removing particles larger than the No. 40 sieve size, a specimen of soil about one-half inch cube in size, is molded to the consistency of putty. If too dry, water must be added and if sticky, the specimen should be spread out in a thin layer and allowed to lose some moisture by evaporation. Then the specimen is rolled out by hand on a smooth surface or between the palms into a thread about one-eighth inch in diameter. The thread is then folded and rerolled repeatedly. During this manipulation the moisture content is gradually reduced and the specimen stiffens, finally loses its plasticity, and crumbles when the plastic limit is reached.

After the thread crumbles, the pieces should be lumped together and a slight kneading action continued until the lump crumbles.

The tougher the thread near the plastic limit and the stiffer the lump when it finally crumbles, the more potent is the colloidal clay fraction in the soil. Weakness of the thread at the plastic limit and quick loss of coherence of the lump below the plastic limit indicate either inorganic clay of low plasticity, or materials such as kaolin-type clays and organic clays which occur below the A-line.

Highly organic clays have a very weak and spongy feel at the plastic limit.

FIELD BOREHOLE LOGS

(See Plate 1 for approximate locations)

These logs of the test borings and related information depict subsurface conditions only at the specific location and at the particular time the boring was made. Soil conditions at other locations may differ from conditions occurring at these boring locations. Also, the passage of time may result in a change in the soil and ground-water conditions at these boring locations.

EARTH SCIENCES ASSOCIATES

DRILLING AND SAMPLING LOG

PROJECT Forum Group ; 3223A DATE DRILLED 7-25-88 HOLE NO. B-1
LOCATION West - facing slope near gas pipeline. GROUND SURFACE ELEV. 391' (topo.)
DRILLING CONTRACTOR Ball Bros. LOGGED BY TDH DEPTH TO GROUND WATER —
TYPE OF RIG Auger HOLE DIAMETER 5 1/4" HAMMER WEIGHT AND FALL 140 lbs / 30"
SURFACE CONDITIONS open slope WEATHER clear

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
0.0	CL	Topsoil, (cumulic?) 0.0 – ~5.0 GRAVELLY CLAY; dark brn. (7.5YR 4/4); moderate plasticity; sand and gravel 20-40%; clasts to 2", typic. subangular; hard; damp.		AD	Drilling w/ flight augers, 4-foot sections; sampling w/ 2½" I.D. Modified California Sampler.
2.0			Bulk		
4.0	CL	Santa Clara Fm. ~5.0 – 7.5 SANDY CLAY; gravel content grades less than 5%.			
6.0					
8.0	CH (CL, ML)	~7.5 – 22 SANDY CLAY; yel. brn. (10YR 5/4); highly plastic fines common, variable %; f. to c. gr. sand; generally fine gravel, decomposing clasts typical; interlensed gravelly and clayey or silty zones; damp-moist, dense - v. stiff.	L-1 L-2	DR	Drove Mod. Calif. Sampler, 8.5-9.9' 22/.5 44/.5 50/.4
10.0				AD	
12.0					Drove Mod. Calif. Sampler, 13.8 — 15.3'
14.0	CL	~15-15 silty clay; low plasticity	L-3	DR	20/.5 29/.5 42/.5
16.0				AD	
18.0		color grades to yel. red (5YR 4/6)			
20.0					

SHEET 1 OF 2

PROJECT 3223A DATE DRILLED 7-25-88 HOLE NO. B-1

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
20.0	CH (CL, ML)	(~7.5-22 SANDY CLAY, cont.)		AD	
22.0		B.H. 22.0'			Terminated hole @ 22.0'; backfilled

SHEET 2 OF 2

EARTH SCIENCES ASSOCIATES
DRILLING AND SAMPLING LOG

PROJECT Forum Group 3223A DATE DRILLED 7-25-88 HOLE NO. B-2
LOCATION west-facing slope, south of trench #1 GROUND SURFACE ELEV. 378' (topo.)
DRILLING CONTRACTOR Ball Bros. LOGGED BY TDH DEPTH TO GROUND WATER —
TYPE OF RIG Auger HOLE DIAMETER 5¼" HAMMER WEIGHT AND FALL 140 lbs / 30"
SURFACE CONDITIONS open slope WEATHER clear

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
0.0	CL	Colluvium or Santa Clara Fm. 0.0 – ~8 GRAVELLY CLAY; dk. brn (7.5YR 3/4); low-moderate plasticity fines 70% ±; f. to c. sand; f. gravel; damp; dense.		AD	Drilling w/ flight augers, 4-foot sections; Sampling w/ 2½" I.D. Modified California Sampler.
2.0			Bulk		
4.0					
6.0			L-1	DR	Drive Mod. Calif. Sampler 6.0 - 7.5' 16/.5 28/.5 38/.5
8.0	CL	Santa Clara Fm. ~8 – 12 SANDY CLAY; dk yel. brn. (10YR 4/4); low-moderate plasticity fines 70% ± f. to c. sand; gravel < 5%; v. stiff - hard damp.	Bulk	AD	
10.0					
12.0	CL-GC	~12 – 22.5 CLAYEY GRAVEL – GRAVELLY CLAY; yel. brn. (10YR 5/6) mottled; low to mod. plasticity fines variable %; abundant decomposing clasts; dense; damp.			
14.0			L-2	DR	Drove Mod. Calif. Sampler 14.0 - 15.5' 25/.5 38/.5 56/.5
16.0				AD	
18.0					
20.0					

SHEET 1 OF 2

PROJECT 3223A DATE DRILLED 7-25-88 HOLE NO. B-2

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
20.0	CL- GC	(~12-22.5 <u>GRAVEL - CLAY</u> MIX, cont.)		AD	
22.0		B. H. 22.5'			Terminated hole @ 22.5'; backfilled
24.0					

SHEET 2 OF 2

EARTH SCIENCES ASSOCIATES

DRILLING AND SAMPLING LOG

PROJECT Forum Group 3223A DATE DRILLED 7-25-88 HOLE NO. B-3
LOCATION west-facing slope N. of trench #1 GROUND SURFACE ELEV. 357' (topo.)
DRILLING CONTRACTOR Bell Bros. LOGGED BY TDH DEPTH TO GROUND WATER 16'
TYPE OF RIG Auger HOLE DIAMETER 5 1/4" HAMMER WEIGHT AND FALL 140 lbs./30"
SURFACE CONDITIONS open slope WEATHER clear

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
0.0	CH	Colluvium(?) 0.0 - 5.3 SANDY CLAY; brn (10YR 4/3); highly plastic fines 60-80%; fine to coarse grained sand; fine gravel < 5%		AD	Drilling w/ flight augers, 4 foot sections; sampling w/ Modified Calif. Sampler, 2 1/2" I.O.
2.0		Santa Clara Fm. ~ grades to lt. olv. brn (2.5Y 5/4)			
4.0				DR	Drive Mod. Calif. Sampler 4.0 - 5.5'
			L-1		25/.5 40/.5 55/.5
	CH	5.3 - ~10 GRAVELLY CLAY; as above, gravel 10-15%.		AD	
6.0			Bulk		
8.0		grades moist			
10.0	ML	~10-12.5 SILT; dk. yel. brn. (10YR 4/6); slightly plastic fines 90% + ; v.f. to f. gr. sand; moist; stiff.			
				DR	Drive Mod. Calif. Sampler 11.5 - 13.0'
12.0			L-2		14/.5 20/.5 22/.5
	ML	12.5-22 SANDY SILT; as above, w/fine to coarse sand + fine gravel 10-30%.		AD	
14.0			Bulk		
16.0		saturated below ~18'	▽		ground water @ 16' after 1 hr. open hole
18.0					Drive Mod. Calif. Sampler 18.0-19.5 22/.5 20/.5 44/.5 lost sample in hole
20.0					

SHEET 1 OF 2

PROJECT 3223A DATE DRILLED 7-25-88 HOLE NO. B-3

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
20.0	ML	(12.5-22 <u>SANDY SILT</u>, cont.)		AD	
22.0		B.H. 22'			Terminated hole @ 22'; back filled.

SHEET 2 OF 2

EARTH SCIENCES ASSOCIATES

DRILLING AND SAMPLING LOG

PROJECT Forum Group 3223A DATE DRILLED 7-25-88 HOLE NO. B-4

LOCATION NE-facing slope, SE of drainage path. GROUND SURFACE ELEV. 344' (topo.)

DRILLING CONTRACTOR Ball Bros. LOGGED BY TDH DEPTH TO GROUND WATER 18'

TYPE OF RIG Auger HOLE DIAMETER 5¼" HAMMER WEIGHT AND FALL 140 lbs. / 30"

SURFACE CONDITIONS open slope WEATHER clear

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
0.0	ML	colluvium(?) 0.0 – 6.0 SILT; dk brn. (10YR 3/3); slightly plastic fines 90% ±; f. to c. sand, fine gravel; dense; dry-damp.		AD	Drilling w/ flight augers, 4' sections; Sampling w/ 2½" I.D. Modified California Sampler.
2.0			Bulk		
4.0					
				DR	Drove Mod. Calif. Sampler 5.3' – 6.8' 16/.5 27/.5 25/.5
6.0	CL	Santa Clara Fm. 6.0~13 GRAVELLY CLAY; dk. yel. brn. (10YR 4/4); mod. plasticity fines 60 ± %; f. to c. gr sand, f. to c. gravel, rubbly locally, weathered clasts common; v. stiff - v. dense; grades moist w/ depth.	L-1		
				AD	
8.0					
10.0					
12.0		~12 gravel grades less			
	ML	~13 – 22 CLAYEY SILT; dk. yel. brn. (10YR 4/6); slight to low plasticity fines, variable %; f. gr. sand 10 - 30%; stiff; damp-moist, grades sandy + gravelly locally. ~15.3 - gravelly lens			
14.0			L-2	DR	Drove Mod. Calif. Sampler 14.0-15.5 17/.5 38/.5 42/.5 wet locally @ 15.5'
				AD	
16.0					
18.0			▽		H_2O @ 18' after open hole ½ hr.
20.0					

SHEET 1 OF 2

PROJECT 3223A DATE DRILLED 7-25-88 HOLE NO. B-4

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
20.0	ML	(~13-22 CLAYEY SILT, cont.)		AO	
22.0		B.H. 22.0'			Terminated hole @ 22.0'; backfilled.

SHEET 2 OF 2

EARTH SCIENCES ASSOCIATES
DRILLING AND SAMPLING LOG

PROJECT Forum Group 3223A DATE DRILLED 7-25-88 HOLE NO. B-5
LOCATION west-facing slope just east of drainage path. GROUND SURFACE ELEV. 341' (topo.)
DRILLING CONTRACTOR Ball Bros. LOGGED BY TDH DEPTH TO GROUND WATER 17' ?
TYPE OF RIG Auger HOLE DIAMETER 5¼" HAMMER WEIGHT AND FALL 140 lbs / 30"
SURFACE CONDITIONS open slope WEATHER clear

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
0.0	CL	colluvium (?) 0.0 – ~17 SANDY CLAY; dk. brn. (10YR 3/3); mod. plasticity; f. to c. sand + fine gravel 20 % ±; hard; dry - damp.		AD	Drilling w/ flight augers, 4' sections; Sampling w/ Modified California Sampler, 2½" I.D.
2.0		~2 – grades to dk. brn (7.5YR 3/4).			
4.0					
		Santa Clara Fm.			Drive Mod. Calif. Sampler 5.5 - 7.0' 18/.5 30/.5 50/.5
6.0		~6.5 fine gravel % increases; sand + gravel % 20 – 35 %; v. stiff; damp - moist, gradational zones of sandy silt, clayey gravel; decomposed clasts common.	L-1	DR	
8.0			Bulk	AD	
10.0					
12.0					
14.0					
16.0					wet locally on top of sample.
	GC	~17 – 18.3 CLAYEY GRAVEL; mottled colors; low plasticity fines 20 - 30%; f - c. sand, f. - c. gravel, cobbles, rubbly; abundant sandstone clasts, deeply weathered; dense - v. dense; damp - ~~moist~~.	▽?	DR	Drive Mod. Calif. Sampler 17.4 – 18.3' 42/.5 50/.4
18.0		B.H. 18.3'			Terminated hole @ 18.3'; backfilled.
20.0					

SHEET 1 OF 1

EARTH SCIENCES ASSOCIATES

DRILLING AND SAMPLING LOG

PROJECT Forum Group 3223A DATE DRILLED 7-26-88 HOLE NO. B-6
LOCATION near shrubs adjacent to road to St. Joseph's. GROUND SURFACE ELEV. 330' (topo.)
DRILLING CONTRACTOR Ball Bros. LOGGED BY TDH DEPTH TO GROUND WATER 13'
TYPE OF RIG auger HOLE DIAMETER 5¼" HAMMER WEIGHT AND FALL 140 lbs. / 30"
SURFACE CONDITIONS open flat WEATHER clear

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
0.0	ML	Santa Clara Fm. 0.0–2.0 **SANDY SILT**; dk. brn (7.5YR 3/4); low plasticity fines 50+%; typic. f. sand, scattered f. gravel < 5%.		AD	Drilling w/ flight augers, 4' sections; using 2½" I.D. Modified California Sampler.
2.0	CH	2.0–~10 **SANDY CLAY**; dk. brn (7.5YR 3/4); highly plastic fines 70% ±; f. to c. sand, c. grains predominate; f. gravel; damp, hard.	L-1	DR	Drove Mod. Calif. Sampler 2.0'-3.5' 16/.5 28/.5 44/.5
4.0				AD	
6.0	CL	~6 grades to dk. yel. brn. (10YR 4/4); mod. plasticity.	Bulk		
8.0	CL-GC	8–10 gravelly.			
10.0	GC CL	~10–19.4 **CLAYEY GRAVEL – GRAVELLY CLAY**; mottled strong brn. (7.5YR 5/6); mod. plasticity fines variable %; rubbly locally.			
12.0					
			▽	DR	Drove Mod. Calif. Sampler 13.3-14.8' 18/.5 28/.5 33/.5
14.0					
				AD	lost sample in hole. Ground water @ 13.0' after 15 min. open hole.
16.0					
18.0				DR	Drove Mod. Calif. Sampler 17.9'—19.4' 19/.5 23/.5 38/.5
			L-2		Terminated @ 19.4'; backfilled
20.0		BH 19.4'			

SHEET 1 OF 1

EARTH SCIENCES ASSOCIATES

DRILLING AND SAMPLING LOG

PROJECT Forum Group 3223A DATE DRILLED 7-26-88 HOLE NO. B-7
LOCATION NE-facing slope along drainage path GROUND SURFACE ELEV. 372' (topo.)
DRILLING CONTRACTOR Bell Bros. LOGGED BY TDH DEPTH TO GROUND WATER 24'
TYPE OF RIG Auger HOLE DIAMETER 5¼" HAMMER WEIGHT AND FALL 140 lbs. / 30"
SURFACE CONDITIONS open slope WEATHER clear

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
0.0	fill	<u>Fill</u> 0.0 – ~11 <u>SANDY SILT</u>; dk. yel. brn. (10YR 4/4); low plasticity fines 85% ±; f. gr. sand predom.; f. gravel < 3%; dry-damp; med. dense		AD	Drilling w/ flight augers; 4 foot sections; Sampling w/ 2½" I.D. Modified California Sampler.
2.0			Bulk		
4.0					
6.0		~6 grades to brn. (10YR 4/3); damp			Drove Mod Calif. Sampler 6.5-8.0' 8/.5 13/.5 17/.5
			L-1	DR	
8.0				AD	
10.0		<u>Santa Clara Fm.</u>			
	CH	~11 – ~15 <u>SANDY CLAY</u>; dk brn. (10YR 3/3), highly plastic; hard; scattered f. to c. sand; damp; 'A' soil horizon (?).			
12.0					
14.0					Drove Mod. Calif. Sampler 14.5-15.3'
	ML	~15 – ~21 <u>CLAYEY SILT</u>; strong brn. (7.5YR 4/6); low plasticity fines 90% ±; f. gr sand predom., f. gravel < 2%; hard - v. dense damp; massive	L-2	DR	30/.5 50/.3
16.0			Bulk	AD	
18.0					
20.0					

SHEET 1 OF 2

PROJECT 3223A DATE DRILLED * 7-26-88 HOLE NO. B-7

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
20.0	ML	(~15-21 CLAYEY SILT, cont.)		AD	
	CL	~21-26.5 SANDY CLAY; yel. brn (10YR 5/4); low plasticity fines 70% ±; f. to c. sand; scattered gravel < 3%; damp - moist; hard.			
22.0					
24.0	CL-GC	~24 hard, dense gravelly zone; grades wet below.	▽		Ground water @ 24.0', 25 min. after drilling.
26.0					Terminated hole @ 26.5'; backfilled
28.0		B.H. 26.5'			

SHEET 2 OF 2

EARTH SCIENCES ASSOCIATES

DRILLING AND SAMPLING LOG

PROJECT Forum Group 3223A DATE DRILLED 7-25-88 HOLE NO. B-8
LOCATION NE-facing slope adjacent to drainage path GROUND SURFACE ELEV. 376' (topo.)
DRILLING CONTRACTOR Ball Bros. LOGGED BY TDH DEPTH TO GROUND WATER —
TYPE OF RIG Auger HOLE DIAMETER 5¼" HAMMER WEIGHT AND FALL 140 lbs / 30"
SURFACE CONDITIONS open slope WEATHER clear

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
0.0	Fill	Fill 0.0–14 GRAVELLY CLAY; dk. yel. brn. (10YR4/4); low plasticity fines 70% +; sand and gravel mix; erratic hard angular gravel clasts of serpentine; dry-damp; med. dense.		AD	
2.0	(CL)		Bulk		
4.0			L-1	DR	Drove Mod. Calif. Sampler 4.3-5.8' 6/.5 11/.5 12/.5
6.0		~5.4' – grades to dark brn. (10YR3/3); high-mod. plasticity; hard. Buried modern 'A' soil, or fill		AD	
8.0					
10.0					
12.0		(base of modern 'A' soil, 14.0'?) Santa Clara Fm. (?)			Drive Mod. Calif. Sampler 14.0-15.5'
14.0	? Fill? (GC)	14.0–~17 CLAYEY GRAVEL; mottled dk. yel. brn. (10YR 4/6); mod. plasticity fines 40% ±; rubbly angular gravel, weathered + decomposing clasts 40% ±; dense.	L-2	DR	20/.5 31/.5 50/.5
16.0	CL ? CL	~16-17 dk. brn gravelly clay; ('A'?) Santa Clara Fm. ~17–18.5 SANDY CLAY, yel. brn. (10YR 5/4); low-mod. plasticity fines 60% ±; f. to c. gr. sand, deeply weathered grains, damp +; dense-v. stiff.		AD	Drove Mod. Calif. Sampler 18.0-19.5'
18.0	GC	18.5-19.5 CLAYEY GRAVEL, as above, 14-17'.	L-3	DR	14/.5 20/.5 35/.5 Terminated hole @ 19.5'; backfilled.
20.0		B.H. 19.5'			

SHEET 1 OF 1

EARTH SCIENCES ASSOCIATES

DRILLING AND SAMPLING LOG

PROJECT Forum Group 3223A DATE DRILLED 7-26-88 HOLE NO. B-9
LOCATION west-facing slope, edge of trees GROUND SURFACE ELEV. 347' (topo.)
DRILLING CONTRACTOR Bell Bros. LOGGED BY TOH DEPTH TO GROUND WATER —
TYPE OF RIG auger HOLE DIAMETER 5¼" HAMMER WEIGHT AND FALL 140 lbs / 30"
SURFACE CONDITIONS open shallow slope WEATHER clear

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
0.0	ML	<u>Santa Clara Fm.</u> 0.0-3.0 SANDY SILT; dk. brn. (7.5YR 3/4); slightly plastic fines 75-85%; f. sand predom.; f. gravel < 2%; damp; dense.		AD	
2.0					
	CL (GC)	3.0-~6 GRAVELLY CLAY; yel. brn. (10YR 5/6); mod. plasticity fines, variable %; abundant sand and weathered sandstone clasts, damp, dense.			
4.0					
6.0	CL	~6-12 SANDY CLAY; yel brn (10YR 5/4) low plastic fines 50% ±; typically f. sand scattered f. gravel < 5%; hard-dense; damp.	Bulk		
			L-1	DR	
8.0		~8- grades gravelly		AD	Drive Modified Calif. Sampler 7.5'-8.3' =7/.5 50/.3 sandstone cobble blocks drive.
10.0			Bulk		
12.0	CL-GC	~12-22.5 GRAVELLY CLAY-CLAYEY GRAVEL; yel. brn. (10YR 5/6); mod. plasticity fines, variable %; hard angular-subrnded Franciscan clasts; ~1½" recovered.			
14.0					
16.0					
18.0					
20.0					

SHEET 1 OF 2

PROJECT 3223A DATE DRILLED 7-26-88 HOLE NO. B-9

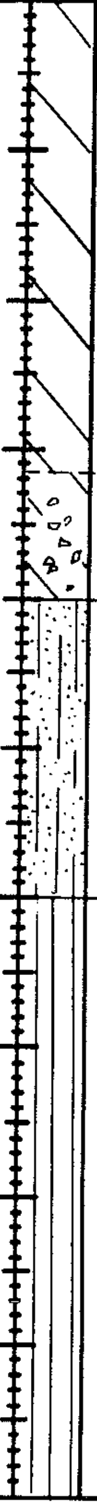
DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
20.0	CL-GC	(~12-22.5 <u>CLAY-GRAVEL</u> mix, cont.)		AD	
22.0		22.5' B.H.			Terminated hole @ 22.5'; backfilled.

SHEET 2 OF 2

EARTH SCIENCES ASSOCIATES

DRILLING AND SAMPLING LOG

PROJECT Forum Group 3223A DATE DRILLED 7-26-88 HOLE NO. B-10
LOCATION West-facing slope GROUND SURFACE ELEV. 377' (topo.)
DRILLING CONTRACTOR Ball Bros. LOGGED BY TDH DEPTH TO GROUND WATER —
TYPE OF RIG Auger HOLE DIAMETER 5 1/4" HAMMER WEIGHT AND FALL 140 lbs. / 30"
SURFACE CONDITIONS open slope WEATHER clear



DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
0.0	CL	Colluvium 0.0–~8 SANDY CLAY; dk. brn. (7.5YR 3/4); mod. plasticity fines 60-70%; f. to c. sand, f. to med. gravel; dry-damp; hard.	Bulk	AD	Drilling w/ flight augers, 4' sections; Sampling w/ 2½" I.D. Modified California Sampler.
2.0					
4.0		~5 – color grades to dk. yel brn. (10YR 4/6).			
6.0	GC	6.3–~8 clayey gravel; decomposed clasts	Bag #1 L-1	DR	Drove Mod. Calif. Sampler 6.0-7.5' 20/.5 30/.5 47/.5
8.0	ML-SM	Santa Clara Fm. ~8–12 SANDY SILT–SILTY SAND; yel. brn. (10YR 5/6); slightly plastic fines 30-70%; v.f. to f. gr. sand, well sorted; dense; damp+.		AD	
10.0			L-2 L-3	DR	Drove Mod. Calif. Sampler 10.0-11.5' 16/.5 21/.5 28/.5
12.0	ML	~12–22 SILT; dk. yel. brn (10YR 4/6); slight to low plasticity fines 90%+; f. gr. sand; massive; dense; damp - moist.	Bulk	AD	
14.0					
16.0					
18.0					
20.0					

SHEET 1 OF 2

PROJECT 3223A DATE DRILLED 7-26-88 HOLE NO. B-10

DEPTH	CLASS.	FIELD DESCRIPTION	SAMPLE	MODE	REMARKS
20.0	ML	(~12-22 SILT, cont.) grades to		AD	
22.0	ML	~22-26.5 SANDY SILT; as above; scattered f. to c. sand, fine gravel 10-15%; damp-moist; dense.			
24.0					
26.0					Terminated hole @ 26.5'; backfilled.
		B.H. 26.5'			
28.0					

SHEET 2 OF 2

TEST PIT LOGS

(See Plate 1 for approximate locations)

The logs of the test pits and related information depict subsurface conditions only at the specific location and at the particular time the test pit was made. Soil conditions at other locations may differ from conditions occurring at these locations. Also, the passage of time may result in a change in the soil and ground-water conditions at these locations.

TEST PIT LOGS

TP-1

0-1.5 ft SANDY CLAY: Brown (10 YR 4/3); low plasticity; 10-30% scattered sand and fine gravel; hard; damp; soil horizon.

1.5-12.5 ft SANDY SILT-SILTY SAND: Yellow-brown (10 YR 5/6); slightly plastic; typically very fine- to fine-grained sand; gravel to 1", typically 1/4-1/2"; decomposed and weathered clasts, sub-angular to angular; dry-damp; hard; dense; reddish 7.5 YR-5 YR clay skins, upper portion. Gravelly lenses <2', with angular cobbles of weathered Franciscan rocks; basalt, chert included; locally obscure thin silty clay beds, gradational.

Bedding of gravelly lens: N10W, 20° or less NE.

Bulk sample at 2-5'.

TP-2

0-2 ft SANDY CLAY: Dark brown (10 YR 3/3) with reddish 7.5 YR shade; moderate to highly plastic; hard; 10-20% scattered gravel and sand; blocky desiccation fracture.

2-5.7 ft CLAYEY GRAVEL: Brown (10 YR 4/3); 30-50% moderately plastic fines; fine- to coarse-grained sand; fine- to coarse-grained gravel; cobbles <1%; angular to sub-rounded clasts; typically sub-angular; dense-very dense; massive, chaotic; dry-damp. Base of unit is shear plane on southwall, depositional contact on opposite wall (shear lost in gravels).

Shear: N35W, 25SW; Trend of Striae: 33°, S35W

Bulk sample at 4.5-5.5'.

5.7- 10.5 ft SILT: Yellow brown (10 YR 5/6); non-plastic fines; local gradations to fine- and coarse-grained sandy silt; interbeds of clayey gravel (as unit above); dense; massive.

Bedding: ~N35W, 25SW.

Bulk sample at 5.5'.

TP-3

0-~2 ft GRAVELLY CLAY: Dark brown (7.5 YR 4/4); medium to high plasticity; 30%± scattered sand and gravel; hard, brittle; topsoil.

~2-6.5 ft CLAYEY SAND, GRAVEL: Yellow brown (10 YR 5/6); 20-30% medium plastic fines; ~40% fine- to coarse-grained sand; ~30% fine- to coarse-grained gravel and cobbles to 10" (cobbles ~5% of total); sub-rounded to angular, typically sub-angular; flattened clasts present; massive, chaotic; dense; abundant weathered buff-colored fine-grained sandstone clasts (Tertiary?).

TP-3 (Concluded)

~2-6.5 ft (Concluded)	Bulk sample at 2-5'.
6.5-10 ft	SILTY TO CLAYEY SAND: (color as above); non- to moderately plastic fines, variable %; fine- to coarse-grained sand; scattered fine gravel to 25%, a few cobbles, <1%; weak local stratification grades from silty or clayey sand to sandy silt. Bedding: ~N15E, 13NW. Bulk sample at 6.5-10'.

TP-4

0-1.5 ft	SANDY CLAY: Dark brown (10 YR 4/3); 75%± moderately plastic fines; scattered fine- to coarse-grained sand; brittle, hard.
1.5-7 ft	SAND AND SILT: Clayey sand with sandy silt interfingered lenses; strong brown (7.5 YR 5/6); 20-40% low plasticity fines; fine- to coarse-grained sand, vague zones of coarser sand; fine gravel, angular to sub-rounded clasts; dense to very dense; massive; damp; local weak stratification, discontinuous, with more silt or clay content. Bedding: ~N45W, 15SW. Bulk sample at 2-5'.
7-8 ft	SANDY SILT-SILTY SAND: Lens; N35W, 15SW; slight to low plasticity fines; fine-grained sand; dense; moist.
8-12.5 ft	CLAYEY GRAVEL: Strong brown (7.5 YR 5/6); mottled; less than 40% fines; 30%± gravel and cobbles; clasts to 5" common; medium dense-dense; moist. Bulk sample at 7-12.5'.

TP-5

0-1.5 ft	GRAVELLY CLAY: Brown (10 YR 4/3); low-medium plasticity; rubbly; ≤10% cobbles; porous; brittle; colluvial soil.
1.5-4± ft	SANDY CLAY: Above color; moderate plasticity; fine- to coarse-grained sand; hard; massive; gradational.
4±-8± ft	SANDY SILT; Above color; low plasticity; fine- to coarse-grained sand; hard; massive; scattered fine gravel; gradational.
8±-12± ft	SANDY CLAY: Yellow red (5 YR 4/6); very weak, gradational zones or lenses of gravel, sand, and clay mixtures; 60%± low plasticity fines; grades locally to silty gravel, ~30% fines, 40-50% gravel; fine gravel, sub-angular to sub-rounded clasts 1/4-1/2" commonly; discontinuous, approximate horizontal vague lenses; typically massive; very dense; gradational.

TP-5 (Concluded)

8±-12± ft (Concluded)	Bedding: ~horizontal.
12±-17.3 ft	SANDY AND GRAVELLY SILT: Dark yellow brown (10 YR 4/6); low plasticity fines; typically massive, vague gradations. Bulk sample at 10-14'.

TP-6

0-11 ft	GRAVELLY CLAY: Dark brown (7.5 YR 4/4); very weak 'A' soil horizon, top 1', darker (10 YR 4/3); variable low to high plasticity, typically moderate to high; locally clayey gravel; 40-70% fines typically; fine- to coarse-grained sand and gravel, typically angular clasts; 10%± angular cobbles; boulders to 14"; abundant buff fine-grained sandstone, angular cobbles, gravel; damp; very dense; characterized by massive, rubbly, chaotic form. ~4': Color grades to mottled dark yellow brown (10 YR 4/4). Bulk sample at 11'.

TP-7

0-2 ft	SANDY CLAY: Dark brown (7.5 YR 4.4); moderate plasticity; hard; scattered fine to coarse sand, fine gravel.
2-3 ft	SILTY SAND: Dark yellow brown (10 YR 4/6); non-plastic fines; fine-grained sand, very hard, dense; lens pinches out; base dips <5° to southwest.
3-10 ft	CLAYEY SAND AND GRAVEL: Dark yellow brown (10 YR 4/6) mottled with Fe-O stains; 20-30% moderately plastic fines, variable; fine to coarse sand and gravel; clasts typically deeply weathered; angular to sub-rounded; <3% cobbles, angular, to 1'; dense; damp-moist; chaotic, massive; faint suggestion of bedding by oxide-stained lens dips ~10-15° southwest. Bedding: (?) NW, ~10SW. Bulk sample at 6'.

TP-8

0-5 ft	SANDY CLAY: Brown (7.5 YR 4/4); moderate to high plasticity fines 80-90%; hard; tough; brittle; massive; deep blocky structure; damp; soil development on colluvial Santa Clara Formation unit, or post-Santa Clara Formation colluvium.
5-8 ft	CLAYEY GRAVEL: Light olive brown (2.5 Y 5/4); moderately plastic fines, variable %; fine to coarse sand; gravel typically subangular to 2", commonly <1"; very dense; massive; damp; gradational upper contact.

TP-8 (Concluded)

5-8 ft (Concluded)	Sharp lower contact, shear (?); gley clay-lined planes at contact; no slickensides observed; possible accumulation of clay through gravel unit deposited or underlying silt, clay also lines other sub-parallel planes along contact.
	Shear? Contact: N35W, 22SW.
8-10 ft	SILT: Brownish yellow (10 YR 6/8); slightly plastic fines; very stiff, very dense; massive, uniform; black oxide stains or fractures locally.
	Joint set: ~N35E, 90°.
	Bulk sample at 8-10'.

TP-9

0-1 ft	CLAYEY SAND AND GRAVEL: Brown (10 YR 4/3); soil profile developed on underlying unit.
1-4 ft	CLAYEY GRAVEL: Yellow brown (10 YR 5/8); mottled; 20-40% highly plastic fines; ~20% 3" and larger cobbles to 10"; gravel typically 1" or less; rubbly; very dense; crudely stratified.
	Bedding contact: ~N60E, 10NW.
4-5 ft	CLAYEY SAND: 30-40% moderately plastic fines; medium-grained sand; dense; gradational, poorly defined bed.
5-7.5 ft	CLAYEY GRAVEL: Yellow brown (10 YR 5/8); mottled; 20-40% highly plastic fines; cobbles to 10", ~20% 3" and larger; gravel typically 1" or less; rubbly; very dense; crudely stratified.
	Bulk sample at 6'.
7.5-8.5 ft	SILTY SAND: ~20% low plasticity fines; fine- to medium-grained sand; medium dense; gradational contacts above and below.
8.5-11 ft	SILT: Yellow brown (10 YR 5/8); non-plastic fines 100%; hard; very dense; massive; damp.
	Bedding: horizontal.
	Bulk sample at 8.5-11'.
11-12.5 ft	SILTY GRAVEL-SILTY SAND: Low plasticity fines, variable %; fine- to coarse-grained sand; fine gravel; dense; chaotic mix; very damp.

TP-10

0-1 ft	GRAVELLY CLAY: Dark brown (10 YR 4/3); moderately to highly plastic fines; 20%± scattered sand and gravel; hard, brittle.

TP-10 (Concluded)

1-4 ft	CLAYEY GRAVEL: Strong brown (7.5 YR 4/6) mottled; 40%+ highly plastic fines; fine to coarse sand; fine to coarse gravel; rubbly with buff fine-grained sandstone cobbles; dense-very dense; damp.
4-7.5 ft	CLAYEY SAND: As above; absence of buff sandstone cobbles, more sand. Bulk sample at 6'.
7.5-12 ft	SANDY SILT: Strong brown (7.5 YR 5/8); 70%± slightly plastic fines; fine to coarse sand, typically fine- to medium-grained; scattered fine gravel; massive; very dense; very damp; gradational gravelly lens, discontinuous. Bedding: contact @ 7.5' ~N45W, 10SW. Bulk sample at 7.5-11.5'.

Appendix B

LABORATORY INVESTIGATION

Appendix B

LABORATORY INVESTIGATION

A. General

Laboratory tests were performed on selected representative samples of the soils obtained during the field investigation of the Forum Life Continuing Care Center Project. The following types of tests were performed and the results are summarized in Table B-1:

1. Moisture and density tests.
2. Atterberg limits tests: Liquid Limit, Plastic Limit, and Plasticity Index.
3. Shrinkage limit tests.
4. Sieve analyses.
5. Free swell/swell pressure tests.
6. Compaction tests.
7. Unconfined compression tests.
8. Direct shear tests.
9. Consolidation tests.
10. R-value tests.
11. Pocket penetrometer/tor vane.

In addition to the above, Woodward-Clyde Associates (1981) in their preliminary geologic and geotechnical study for the proposed West Reservoir water storage facility obtained samples using the California Modified Sampler and performed laboratory tests. The number of blows required to take the sample, and results of moisture, density, and unconfined compression tests are summarized in Table B-2.

The laboratory studies performed during this investigation were directed toward establishing the engineering properties of the various soils in order to provide a basis for the design recommendations for the following:

1. Site grading including cuts, fills, and temporary trenches.
2. Drainage.
3. Foundations.
4. Retaining walls.
5. Roads and parking lots.

Laboratory tests for corrosion potential of the soil for buried pipes were beyond the scope of this study.

Results of the laboratory tests which provided a basis for the design recommendations are summarized in Tables B-1 and B-2 and a series of figures provided at the end of this Appendix. Test procedures are summarized in the following text.

B. Index and Identification

1. Visual Classification

Field classification was verified in the laboratory by visual examination of the samples in accordance with the Unified Soil Classification System and ASTM D2487 test method. When necessary to substantiate visual classifications, tests were conducted in accordance with the ASTM D2487 test method.

2. Atterberg Limits

Atterberg Limit Tests were performed on selected soil samples in order to estimate their plasticity and to aid in their classification. The testing procedure was in accordance with the ASTM D4318 test methods. Test results are summarized on Table B-1 and Figure B-1.

3. Shrinkage Limit

A shrinkage limit test was performed on a selected sample in order to estimate the swell potential. The testing procedure was in accordance with the ASTM D427 test method. Test results are summarized on Table B-1.

4. Grain Size Distribution

Grain size distribution tests were performed on representative samples of the various soils to assist in the soils classification and to correlate test data between various samples. Sieve analyses were performed on that portion of the sample retained on the No. 200 sieve in accordance with ASTM D422 test method. Results of these analyses are presented in Figure B-2 at the end of this Appendix.

5. Moisture Content

Moisture content determinations were performed on selected soil samples in accordance with the ASTM D2216 test method to assist in their classification and to estimate the in situ moisture content. Test results are presented on Table B-1 and on the figures presenting other test results when appropriate.

6. Unit Weight

Unit weight determinations were performed on selected undisturbed soil samples to assist in their classification and in the selection of samples for engineering properties testing. Samples were generally the same as those selected for moisture content determinations.

The test procedure entailed measuring specimen dimensions with a precision ruler or micrometer. Weights of the sample were then determined at natural moisture content. Total unit weight was computed directly from data obtained from the two previous steps. Dry density was calculated from the moisture content and the total unit weight. Results of the unit weight tests are presented as dry densities on Table B-1. Tests were performed on undisturbed samples of soil found at the site.

7. Free Swell/Swell Pressures

A swell pressure and free swell test was performed on selected undisturbed samples of cohesive, potentially expansive clays obtained from the site.

The free swell test was performed on a compacted sample obtained from boring B-1, bulk sample 2.0 ft to 6.0 ft. The free swell test was performed in general accordance with ASTM Test Procedure D4546-85, Method A. The sample was placed in the consolidometer and a small vertical confining load was applied to the sample and inundated with water. The sample was allowed to swell. The resulting one-dimensional swell of the sample was measured and recorded. The results and the expansion index are presented in Table B-1.

The swell pressure test was performed on the consolidation test sample (boring B-3, sample L-1) and consisted of placing the sample in a consolidometer, inundating the sample with water and preventing any swelling of the sample by adjusting (increasing) the vertical confining load. The vertical pressure at which no expansion (swell) of the sample occurred was recorded as the sample's swell pressure. The results are presented in Table B-1.

C. Engineering Properties

1. Moisture-Density Relations (Compaction Test)

Compaction tests were performed on various composite samples of soil. The results of these tests were used to estimate the densities at which the available materials might be placed as embankment fill and trench backfill during construction.

Tests were performed in accordance with ASTM D1557, Methods A and C. Test results are summarized in Figure B-3 and Table B-1.

2. Unconfined Compression

Unconfined compression tests were performed on selected samples of cohesive soils from the test borings for the purpose of evaluating the undrained, unconfined shear strength. For the cohesive soils, tests were performed in accordance with the ASTM D2166 test method. Results of the unconfined compression tests are presented in Table B-1.

3. Direct Shear Strength Tests

Direct shear tests were performed using a constant strain rate direct shear machine. Direct shear tests were performed on one relatively undisturbed samples of the sandy clay soils found at the site. The purpose of the test was to estimate the soil strength characteristics used in engineering analyses and evaluation for the embankment fill slopes and the foundation recommendations for the building.

The direct shear test to evaluate the peak strength parameters was performed by trimming and placing the compacted specimen (boring B-1 bulk, 2.0'-6.0') in the shear machine, and applying a specified normal load while allowing the sample to consolidate prior to shearing. The specimen was sheared until the maximum and residual strengths had developed under the applied normal stress. After the first specified normal load, another specimen (sample of similar material) was tested at a new normal load. The two test points establish the peak strength envelope for the material. Results of the direct shear test is summarized in Table B-1 and Figure B-4.

4. Consolidation Tests

A consolidation test was performed on a selected undisturbed sample of soil in order to estimate the settlement of areas requiring fill. The test procedure employed is in general accordance with ASTM D2435.

Porous stones were placed in contact with both sides of the specimens to permit ready addition or release of water. Loads were applied to the test specimens in several increments, and the resulting settlements recorded. Tests were performed on samples inundated with water. Results of consolidation tests are presented in Figure B-5.

5. R-Value Tests

Testing and Controls of Mountain View performed two R-value tests on representative samples according to ASTM D2844. The results are summarized in Table B-1 and are attached at the end of this Appendix.

Table B-1

SUMMARY OF LABORATORY TESTS, FORUM LIFE CONTINUING CARE CENTER

Boring/ Test Pit No.	Sample	Depth (ft)	Laboratory Soil Classification	Dry Density (pcf)	Moisture Content (%)	Percent Passing No. 200	Atterberg Limits (PI/ LL)	Shrinkage Limits	Unconfined Strength (psf)	Compaction Test		Direct Shear Strength Test Results		Free Swell/ Swell Pressure Tests	Other Tests*
										Maximum Dry Density (pcf)	Optimum Moisture Content (%)	ϕ (deg)	c (psf)		
B-1	L-1	8.7-9.2	CL	109.8	16.8				2,765						
B-3	L-1	4.8-5.3	SC	105.0	19.1									1500 psf	Consolidation test
B-4	L-2	14.3-14.8	SC	102.8	22.4				–						TV: 0.8, >1 PP: 2.25, 3.2, >4.5
B-5	L-1	5.8-6.3	CL	115.9	15.7				–						TV: >1 PP: >4.5
B-6	L-1	2.3-2.8	CL	116.3	14.4				18,878						TV: >1 PP: >4.5
B-7	L-2	15.3-15.8	SM	101.1	22.3				12,398						TV: 0.6, 1 PP: >4.5
B-9	L-1	7.6-8.1	SC	114.1	11.4				12,874						TV: .9 PP: >4.5
B-1	Bulk	2.0-5.0	CL-CH				26/50	12.1%							
B-3	Bulk	5.5-8.0	MH				23/57								
B-6	Bulk	4.0-8.0	CL				20/41								
TP-2	Bulk	4.5-5.5	GC												R-value = 22
TP-10	Bulk	7.5-11.5	CL												R-value = 21

Table B-1 (Concluded)

	Boring/ Test Pit No.	Sample	Depth (ft)	Laboratory Soil Classification	Dry Density (pcf)	Moisture Content (%)	Percent Passing No. 200	Atterberg Limits (PI/ LL)	Shrinkage Limits	Unconfined Strength (psf)	Compaction Test Maximum Dry Density (pcf)	Compaction Test Optimum Moisture Content (%)	Direct Shear Strength Test Results φ (deg)	Direct Shear Strength Test Results c (psf)	Free Swell/ Swell Pressure Tests	Other Tests*
Composite Sample	B-1	Bulk	2.0-6.0													
	B-2	Bulk	2.0-5.0	CL			60				120.5	14.5	36	1700	7.4%	UBC Expansion Index = 74
	B-3	Bulk	0.0-5.0													
Composite Sample	TP-6	Bulk	11.0	GC			13.9				129.2	11.8				
	TP-9	Bulk	6.0													
Composite Sample	TP-3	Bulk	2.0-5.0	SC			40				125.5	12.5				
	and above two composite samples															

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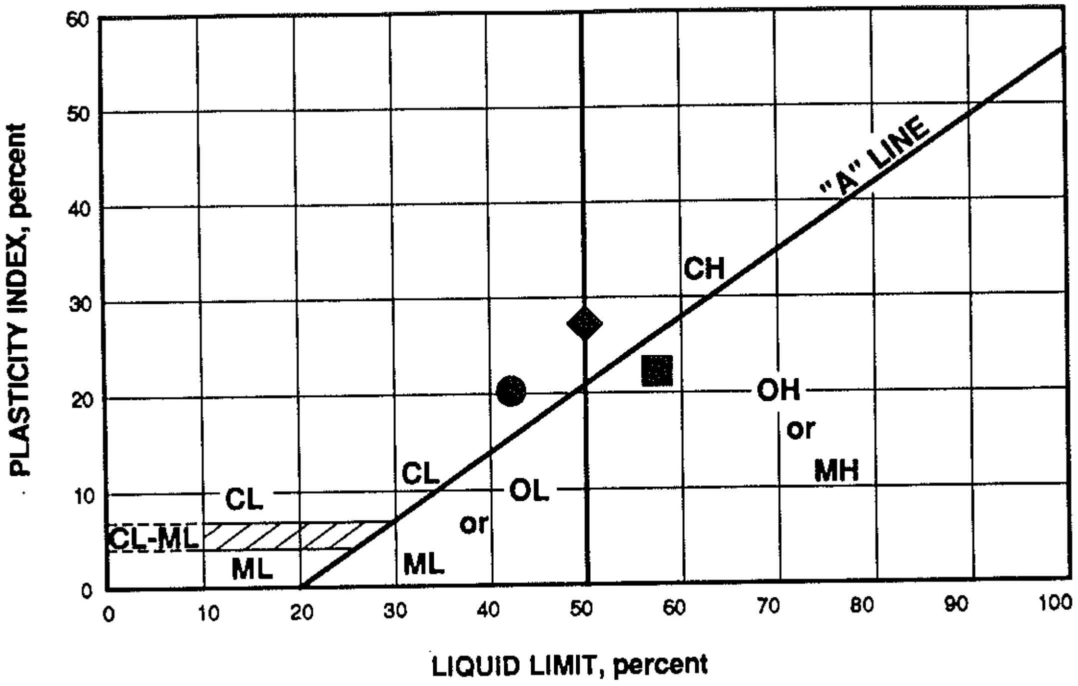
*TV = Tor Vanes in tons per square foot; PP = Pocket Penetrometer readings in tons per square foot.

Table B-2

SUMMARY OF LABORATORY TESTS BY WOODWARD-CLYDE CONSULTANTS

Boring No.	Sample No.	Depth (ft)	Soil Classification	Dry Density (pcf)	Moisture Content (%)	California Modified Blows	Unconfined Compression Test (psf)
B-1	1	2.0-3.5	CL-SC	--	--	19	--
	2	7.0-8.5	SC	116	17	55	6,820
	3	12.0-12.8	SC-GC	121	15	>50	5,980
	4	17.0-17.5	GC	118	15	42	--
	5	22.0-23.0	CL	103	24	45	4,900
	6	27.0-28.5	CL	103	24	59	12,210
	7	31.0-31.8	CL	103	23	>65	9,560
	8	37.0-37.5	CL	110	21	>40	4,020
	9	42.0-42.8	CL	117	17	>60	2,700
	10	47.0-47.3	GC	--	--	>50	--
B-2	1	3.0-3.5	CL	104	20	14	7,990
	2	7.0-7.5	SC	116	17	30	4,410
	3	12.0-13.5	CL	113	19	45	6,420
	4	17.0-17.5	CL	105	22	48	6,080
	5	22.0-23.5	SC	108	21	52	3,830
	6	27.0-27.9	SC	112	20	49	3,970
	7	31.0-32.0	SC	111	21	43	2,400
	8	37.0-38.0	CL	--	--	73	--
	9	42.0-43.5	CL	102	26	53	10,250
	10	47.0-48.0	CL	111	20	55	4,560
	11	52.0-53.0	CL	112	20	60	8,780
B-3	1	2.0-3.0	CL-GC	125	8	58	--
	2	7.0-8.0	GC	113	8	62	--
	3	12.0-12.5	CL	--	--	36	--
	4	17.0-17.5	CL	119	16	50	5,490
	5	22.0-23.5	CL	--	--	48	--
	6	27.0-27.5	CL	103	25	38	6,280
	7	31.0-31.5	CL	109	20	48	7,560
	8	37.0-37.5	CL	112	21	67	9,710
	9	42.0-43.5	CL	128	11	35	2,500
	10	49.0-49.9	CL	--	--	78	--
B-4	1	7.0-8.0	CL	110	15	50	6,130
	2	17.0-18.0	CL	111	13	51	2,840
	3	27.0-28.0	CL	--	--	47	--
	4	31.0-31.5	SC	112	14	23	2,450
	5	42.0-43.0	CL	108	15	62	4,170
	6	52.0-52.9	GC-GP	127	9	83	--
	7	60.0-60.7	CL	116	18	68	--
B-5	1	2.0-3.5	CL	110	16	47	7,550
	2	7.0-8.5	CL	106	22	36	6,330
	3	12.0-13.5	CL-GC	109	23	57	5,150
	4	17.0-18.5	CL	92	31	36	6,770
	5	22.0-23.0	GC	125	13	58	--
	6	27.0-28.0	GC	113	19	67	--
	7	31.0-32.0	GC	--	--	51	--
	8	37.0-38.0	GC	108	21	52	4,510
	9	42.0-43.0	GC	101	25	46	6,230
	10	49.0-50.0	GC	112	18	75	2,940



Symbol	Boring No.	Depth, ft.	Liquid Limit, %	Plasticity Index, %	USC Symbol
◆	B-1	2.0 - 6.0	50	26	CL - CH
■	B-3	5.5 - 8.0	57	23	MH
●	B-6	4.0 - 8.0	41	20	CL

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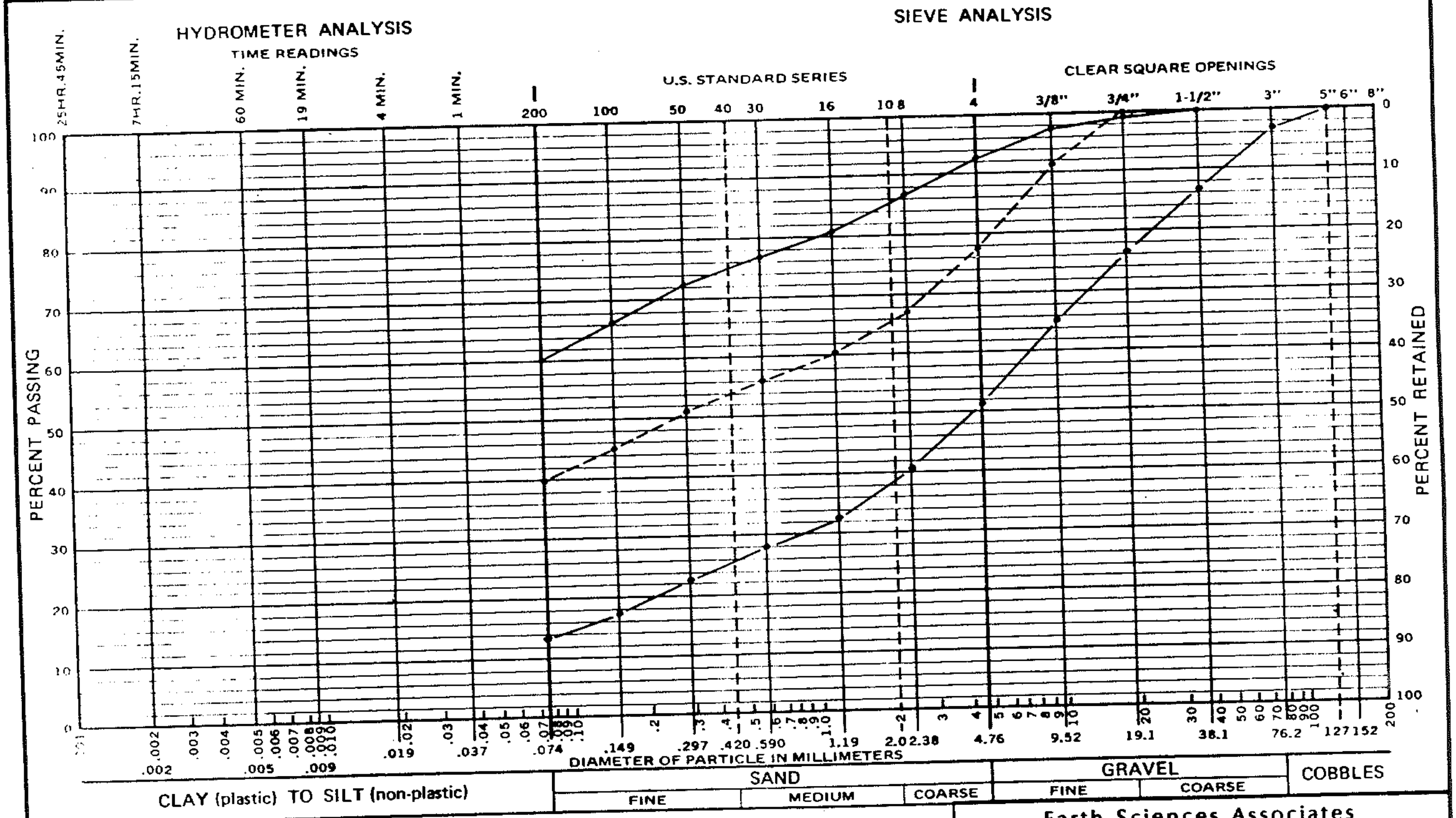
FORUM LIFE CONTINUING CARE CENTER
CUPERTINO, CALIFORNIA
ATTERBERG LIMITS

Checked By Date 9-9-88
Approved By R.C. Harding Date 9-9-88
Project No. 3223A
Figure No. B-1



Symbol	Sample*	Description
•———•	B-1, -2, -10 Composite	Sandy Clay
•———•	TP-6, -9 Composite	Silty, Clayey Gravel w/Sand
•- - - - -•	B-1, -2, -10/TP-3, -6, -9 Composite	Clayey Sand

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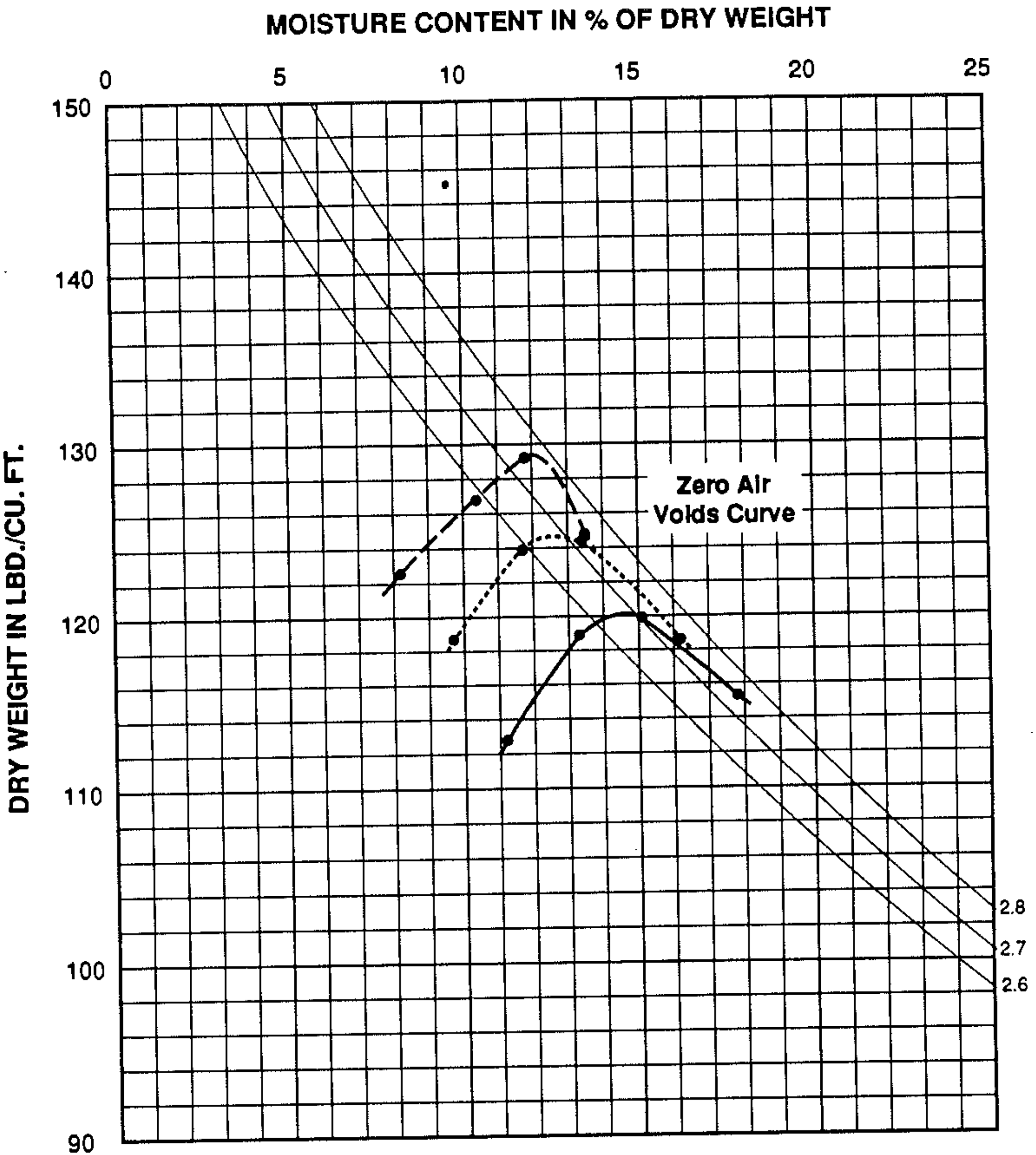
FORUM LIFE CONTINUING CARE CENTER
CUPERTINO, CALIFORNIA
SUMMARY OF GRADATIONS

Checked by Date 9-9-88		Project No.	Figure No.
Approved by R. C. Harding Date 9-9-88		3223A	B-2

* See Table B-1 for Sample Depth



Symbol	Sample*	γ d max., pcf	Optimum Moisture Content, %
———	B-1, -2, -10 Composite	120.5	14.5
— — — —	TP-6, -9 Composite	129.2	11.8
··············	B-1, -2, -10, TP-3, -6, -9 Composite	125.0	12.5

* See Table B-1 for Sample Depth

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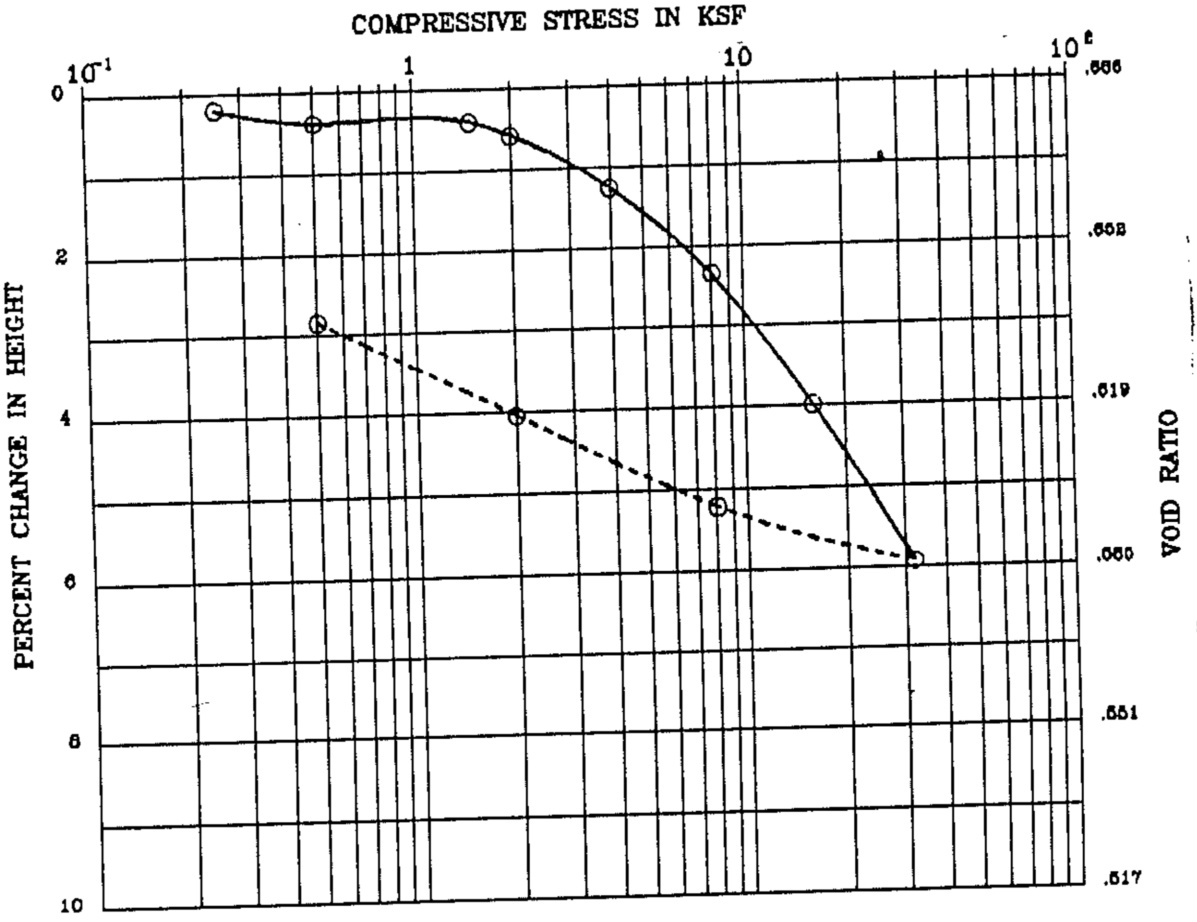
FORUM LIFE CONTINUING CARE CENTER
CUPERTINO, CALIFORNIA

SUMMARY OF COMPACTION TESTS

Checked By J. Y. Koneshiro Date 9-9-88
Approved By R. C. Harding Date 9-9-88
Project No. 3223A
Figure No. B-3



BORING	: B-3, L-1	DESCRIPTION	: clayey SAND, orange yellow brown
DEPTH (ft)	: 4.8-5.3 ft	LIQUID LIMIT	:
SPEC. GRAVITY	: 2.83	PLASTIC LIMIT	:

	MOISTURE CONTENT (%)	DRY DENSITY (pcf)	PERCENT SATURATION	VOID RATIO
INITIAL	19.1	105.0	79	.686
FINAL	22.5	108.0	100	.639

Remark : Undisturbed specimen Swell pressure was 1500 psf 8/88

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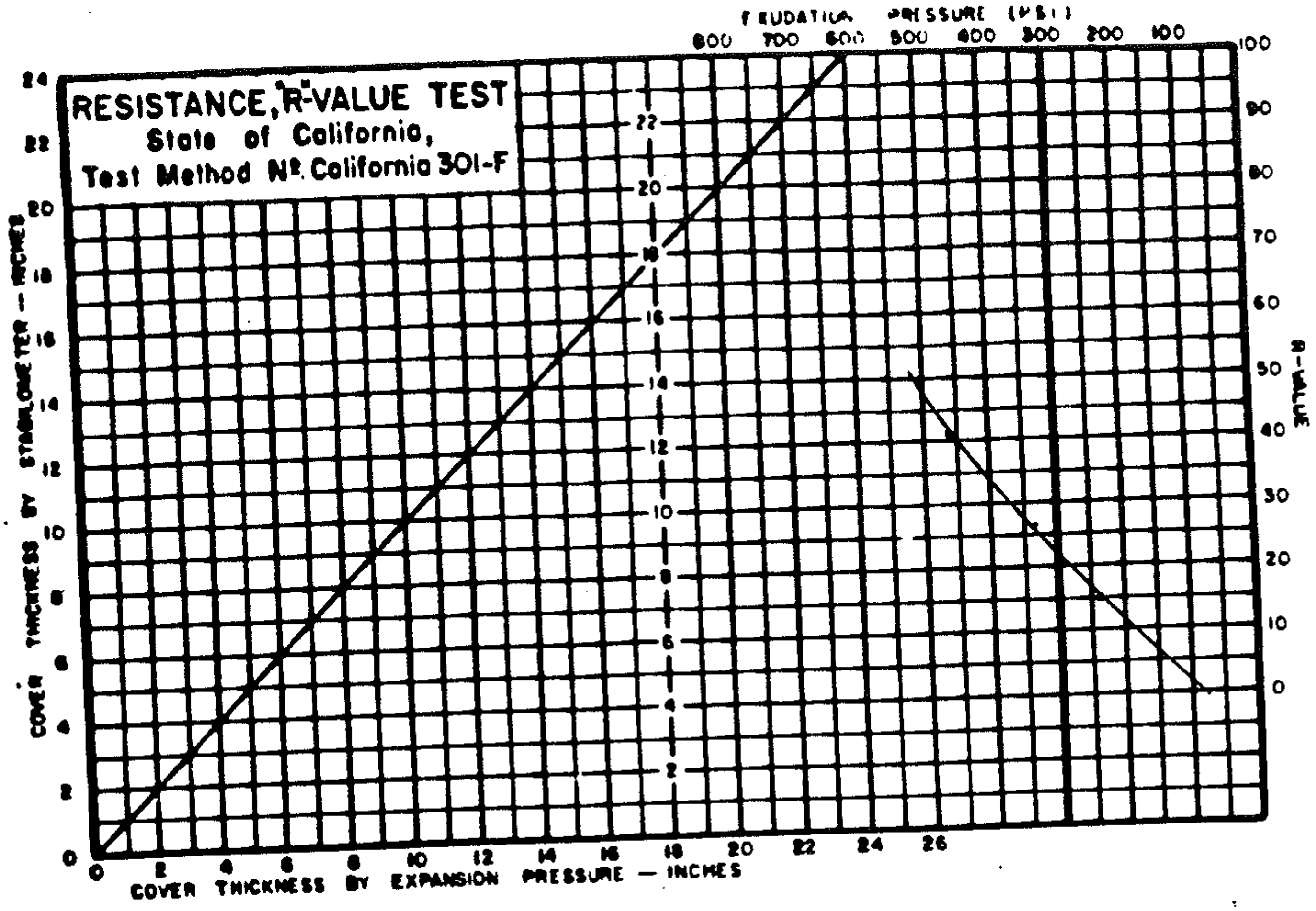
FORUM LIFE CONTINUING CARE CENTER
CUPERTINO, CALIFORNIA
CONSOLIDATION TEST

Checked by JYKaneshiro	Date 9-9-88	Project No.	Figure No.
Approved by RCHarding	Date 9-9-88	3223A	B-5

Test performed by Wahler Associates

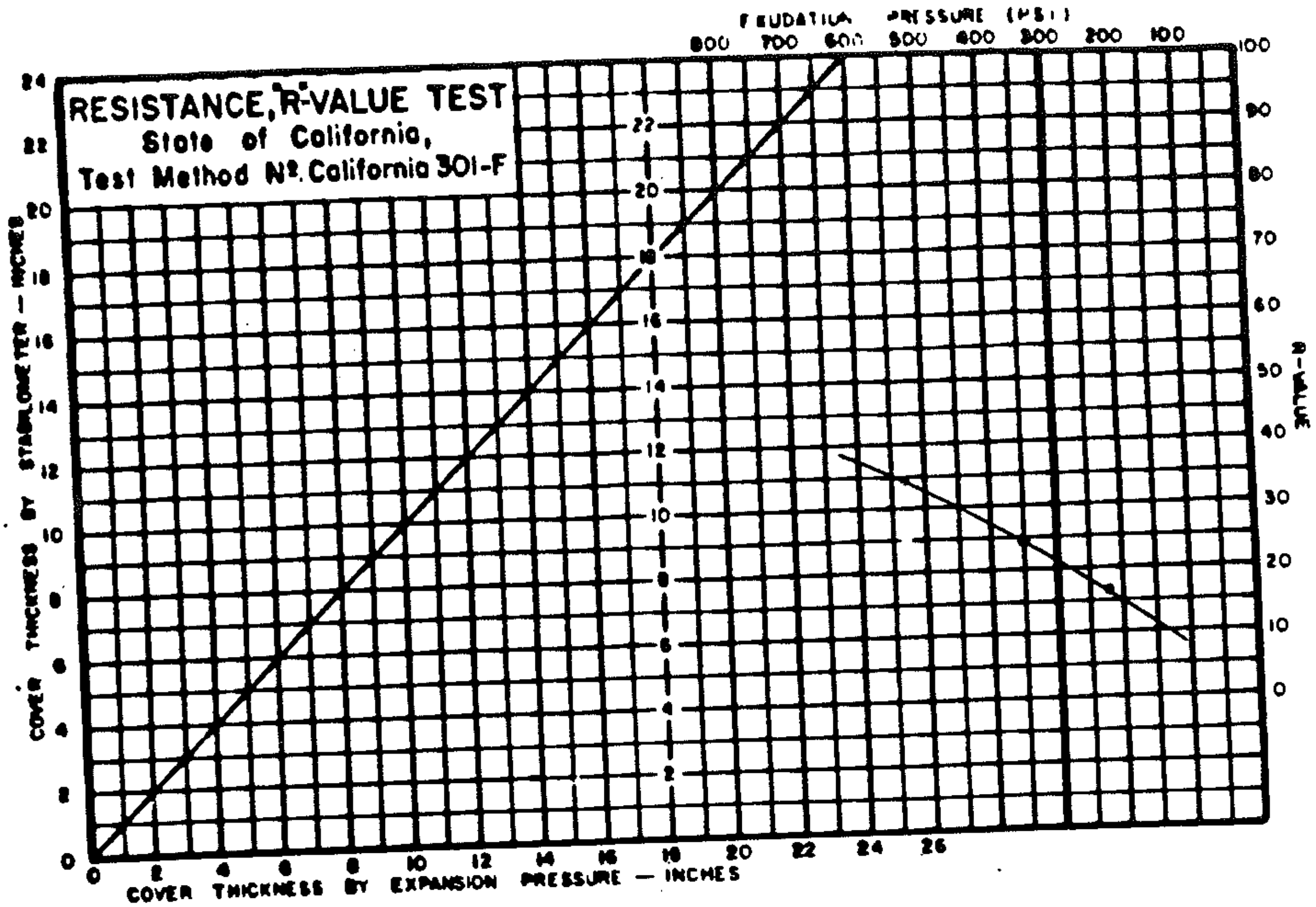
T3470-M01
518721
8-23-88



Sample: TP-2

Description: Reddish-brown sandy clay with gravel

Specimen	A	B	C
Exudation Pressure, p.s.i.	239	334	454
Expansion dial (.0001")	2	12	21
Expansion Pressure, p.s.f.	9	52	91
Resistance Value, "R"	15	26	41
% Moisture at Test	16.7	15.7	14.8
Dry Density at Test, p.c.f.	119.2	121.0	122.2
"R" Value at 300 p.s.i. Exudation Pressure	= (22)		



Sample: TP-10

Description: Reddish brown sandy clay with gravel

Specimen	A	B	C
Exudation Pressure, p.s.i.	223	350	534
Expansion dial (.0001")	6	13	30
Expansion Pressure, p.s.f.	26	56	130
Resistance Value, "R"	17	24	34
% Moisture at Test	21.4	20.5	19.5
Dry Density at Test, p.c.f.	107.4	107.9	111.4
"R" Value at 300 p.s.i. Exudation Pressure	= (21)		

NOTE: Trench 8 did not intersect the "disturbed zone" and was not, therefore, logged in detail.

PRINCIPAL GEOLOGIC UNITS

① TOPSOIL - SANDY CLAY: dark brown (7.5 YR 3/4).

② SANDY SILT: strong brown (7.5 YR 4/6) with prominant yellowish red (5 YR) ped faces in upper 1'; typically very fine-grained sand with slightly plastic fines, contains lenses of coarse sand; dense; hackly breakage.

③ GRAVELLY SILTY SAND: mottled brown (7.5 YR), low plasticity fines, 60-80% fine to coarse sand, 20%+ fine gravel and cobbles; crudely stratified; medium dense to dense.

④ CLAYEY SAND and GRAVEL: massive to weakly stratified; dense to very dense; moderate to highly plastic fines 10-40%.

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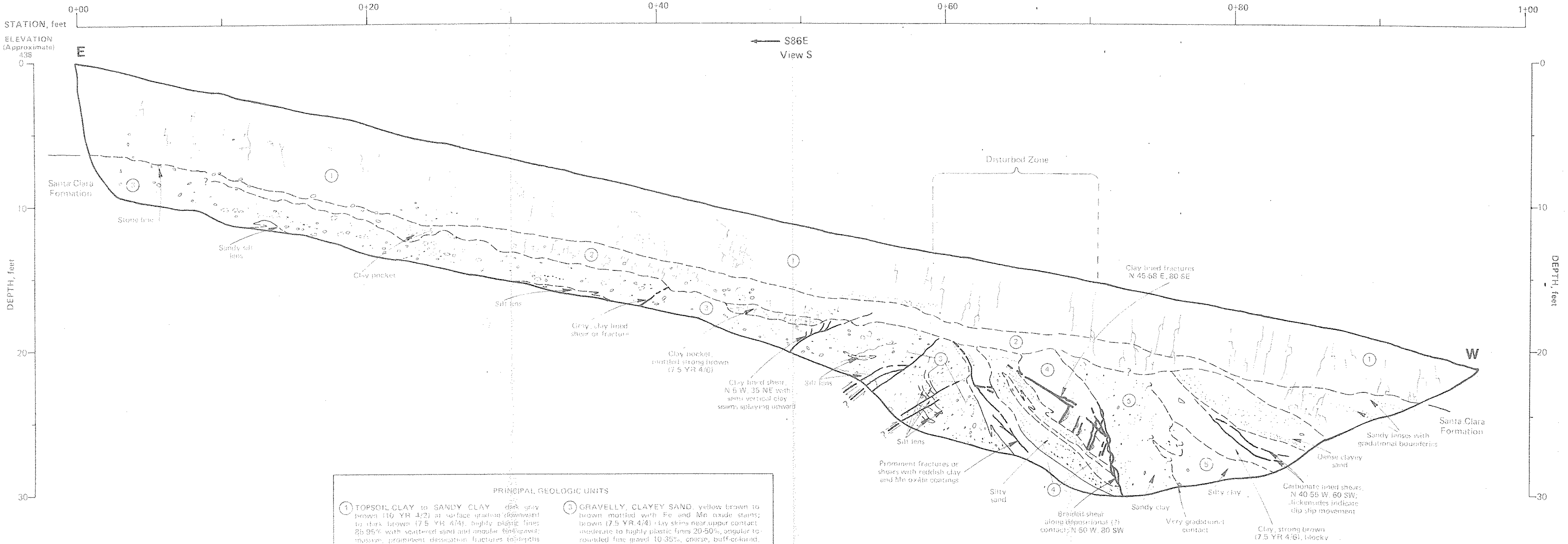
GEOLOGIC LOG OF
EXPLORATORY TRENCH T-8

Checked by	Date	Project No. 3223	Figure No. 1
Approved by	Date		



PRINCIPAL GEOLOGIC UNITS

① TOPSOIL CLAY to SANDY CLAY dark gray brown (10 YR 4/2) at surface grading downward to dark brown (7.5 YR 4/4); highly plastic fines 85-95% with scattered sand and angular fine gravel; massive; prominent dessication fractures to depths of 4'; blocky structure near base; very stiff to hard.

② COLLUVIUM CLAY yellowish brown (10 YR 5/4) to strong brown (7.5 YR 4/6) to the W; highly plastic; sand less than 10% with scattered gravel clasts to 3"; local well developed blocky structure.

③ GRAVELLY, CLAYEY SAND, yellow brown to brown mottled with Fe and Mn oxide stains; brown (7.5 YR 4/4) clay skins near upper contact moderate to highly plastic fines 20-50%, angular to rounded fine gravel 10-35%, coarse, buff-colored, deeply weathered sandstone clasts (Franciscan graywacke) up to 5%; massive; dense; damp.

④ SILT: yellowish brown (10 YR 5/6) with strong brown (7.5 YR) clay skins in upper part, contains beds of silty sand; non plastic; uniform; dense, brittle blocky fracture; abundant Mn oxide coated fractures.

⑤ SANDY CLAY mottled brown (7.5 YR) with grey and Fe and Mn oxide stains; moderately plastic fines 80%+; massive; very dense; becomes finer-grained up section to the W

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EL CAMINO HOSPITAL CONTINUING CARE CENTER
GEOLOGIC LOG OF
EXPLORATORY TRENCH T-7

Checked by	Date	Project No.	Figure No.
Approved by	Date	3223	2

PRINCIPAL GEOLOGIC UNITS

① TOPSOIL SANDY CLAY: brown (10 YR 4/3); scattered fine to coarse sand 10 - 30%, gravel less than 1%; moderately to highly plastic; hard, brittle; damp to dry granular grading to blocky downwards; abundant desiccation cracks.

② SILTY SANDY GRAVEL: yellow brown (10 YR 5/7), with lenses of sand and clayey silt; massive within beds; typically of low plasticity; dense; dry to damp.

③ SANDY CLAY: yellowish red (5 YR 4/6); low plasticity fines 40 - 50%, fine to coarse sand approximately 40% plus scattered gravel; massive; dense - very dense; hard; damp.

④ GRAVELLY CLAY: distinctive yellowish red (5 YR 4/6), approx. 80% low plasticity fines, approx. 20% fine sand and angular gravel; massive; hard; damp.

⑤ GRAVELLY SILTY SAND: yellow brown (10 YR 5/6) 35-60% slightly plastic fines, fine to coarse sand, scattered gravel; locally stratified; very dense; dry.

⑥ SANDY SILT to SILTY SAND: strong brown (7.5 YR 5/8) mottled with 5 YR clay skins on fracture surfaces and Fe oxide stained sand, local gravel lenses; dense; brittle; dry to damp.

⑦ CLAYEY SAND & GRAVEL: yellow brown (10 YR 5/8), mottled; slightly to moderately plastic fines 30 - 60%; weathered graywacke clasts to 3", typically 1"; massive;dense.

⑧ CLAYEY SAND & GRAVEL: yellow brown (10 YR 5/6); various mixtures of fine sand, coarse sand and gravel, slightly to moderately plastic fines 20 - 40%; grades locally to gravelly clay, typically massive; dense.

Scale: 1" = 5'
Horizontal = Vertical

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EL CAMINO HOSPITAL CONTINUING CARE CENTER
GEOLOGIC LOG OF
EXPLORATORY TRENCH T-2

Checked by	Date	Project No.	Figure No.
Approved by	Date	3223	3a

NOTE:
The deep exposures between stations 1+58 and 2+08 were excavated and logged 1/13-15/86.

See Figure 5a for Explanation of Principal Geologic Units.

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Scale: 1" = 5'
Horizontal = Vertical

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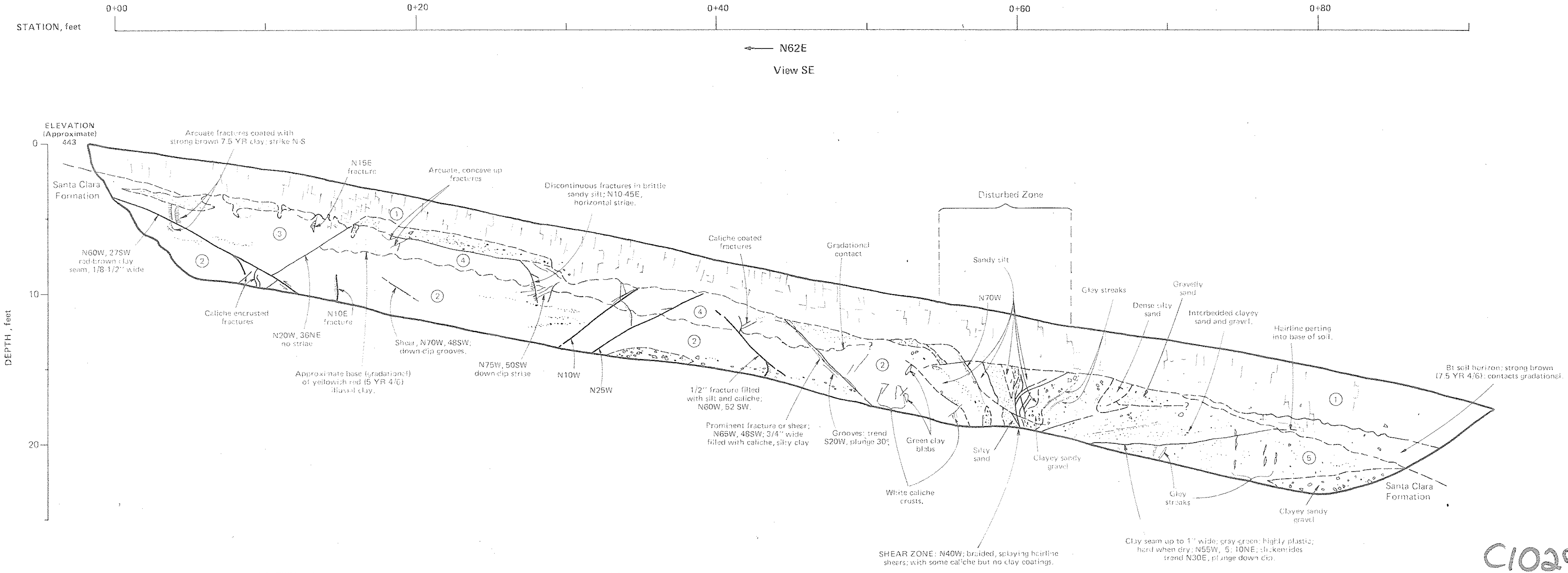
GEOLOGIC LOG OF
EXPLORATORY TRENCH T-2 (CONTINUED)

Checked by	Date	Project No.	Figure No.
Approved by	Date	3223	3b



PRINCIPAL GEOLOGIC UNITS

① TOPSOIL-SANDY CLAY: dark brown (7.5 YR 4/4) clay skins; blocky peds in lowest ½ - 1' of soil

② SANDY SILT: yellow brown (10 YR 5/4) mottled to dark yellowish brown (10 YR 4/6); uniform, medium dense to dense, damp.

③ SANDY SILT: no coarse grains or gravels; no peds, yellowish red (5 YR 5/6); clay coatings on fractures near upper contact.

④ SANDY SILT: with scattered coarse sand grains and fine gravel; very dense, slightly brittle, moderately well developed yellowish red (5 YR 4/6) illuvial clay on peds.

⑤ SANDY GRAVELLY CLAY: mottled 5 YR colors; fines approx. 60%; hard, brittle; very dense; grades to clayey sandy gravel.

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Scale: 1'' = 5'
Horizontal = Vertical

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GEOLOGIC LOG OF
EXPLORATORY TRENCH T-5

Checked by	Date	Project No.	Figure No.
Approved by	Date	3223	4

PRINCIPAL GEOLOGIC UNITS

(1a) TOPSOIL-SILTY CLAY: brown (10 YR 4/3) to dark brown (7.5 YR 4/2), less than 5% fine to coarse sand, less than 2% angular gravel up to 3"; moderately blocky, clay skins very thin to absent; moderate to high plasticity; stiff, hard; damp. Lateral variations of this unit include (1b) and (1c).

(1b) TOPSOIL-GRAVELLY, SANDY CLAY: brown (10 YR 4/3) mottled with MnO stains on blocky fractures; 20-40% fine to coarse sand, 5% coarse gravel with scattered cobbles; high plasticity, stiff; moist.

(1c) TOPSOIL-SANDY CLAY: dark brown (7.5 YR 3/4); low to moderate plasticity; firm; moist; abundant rootlet holes.

② SANDY CLAY: dark yellow brown (10 YR 4/4) mottled with Fe oxide stains and gley colors; stratification obscured by weathering; very dense; very hard; damp.

③ GRAVELLY, SANDY CLAY: yellow brown (10 YR 5/6) to yellowish red (5 YR 4/6) mottled with Fe oxide stains; 40 - 60%+ low plasticity fines, approx. 30% fine to coarse sand, approx. 10% fine to coarse gravel; weakly stratified; dense to very dense; damp.

④ CLAYEY SAND: yellow brown (10 YR 5/6), very fine to fine, arkosic; slight blocky structure; very dense; damp.

⑤ CLAYEY SAND: yellow brown (10 YR 5/6) mottled with Fe Oxides, approx. 60% coarse subangular sand, 5 - 10% gravel, moderately plastic fines; massive; dense, moist.

⑥ SILTY CLAY: yellow brown (10 YR 5/6) with yellow brown (10 YR 5/4) clay skins on blocky desiccation fractures; massive; moderately to highly plastic. (Bt Horizon ?)

⑦ CLAYEY, SANDY SILT: yellow brown (10 YR 5/6) with vertical gley streaks common; massive; deeply weathered; very dense; damp.

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Scale: 1" = 5'
Horizontal = Vertical

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GEOLOGIC LOG OF
EXPLORATORY TRENCH T-1

Checked by ________ Date ______
Approved by ________ Date ______

Project No.	Figure No.
3223	4A

C1025

Scale: 1" = 5'
Horizontal = Vertical

See Figure 4A for Explanation of Principal Geologic Units.

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GEOLOGIC LOG OF
EXPLORATORY TRENCH T-1 (CONTINUED)

Checked by ______________ Date ________	Project No.	Figure No.
Approved by ______________ Date ________	3223	4B

See Figure 4A for Explanation of Principal Geologic Units.

C1025

Scale: 1" = 5'
Horizontal = Vertical

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GEOLOGIC LOG OF
EXPLORATORY TRENCH T-1 (CONTINUED)

Checked by ______	Date ______	Project No.	Figure No.
Approved by ______	Date ______	- 3223	4C

See Figure 4A for Explanation of Principal Geologic Units.

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Scale: 1" = 5'
Horizontal = Vertical

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GEOLOGIC LOG OF
EXPLORATORY TRENCH T-1 (CONTINUED)

Checked by ______ Date ______ Approved by ______ Date ______	Project No. 3223	Figure No. 4D

See Figure 4A for Explanation of Principal Geologic Units.

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Scale: 1" = 5'
Horizontal = Vertical

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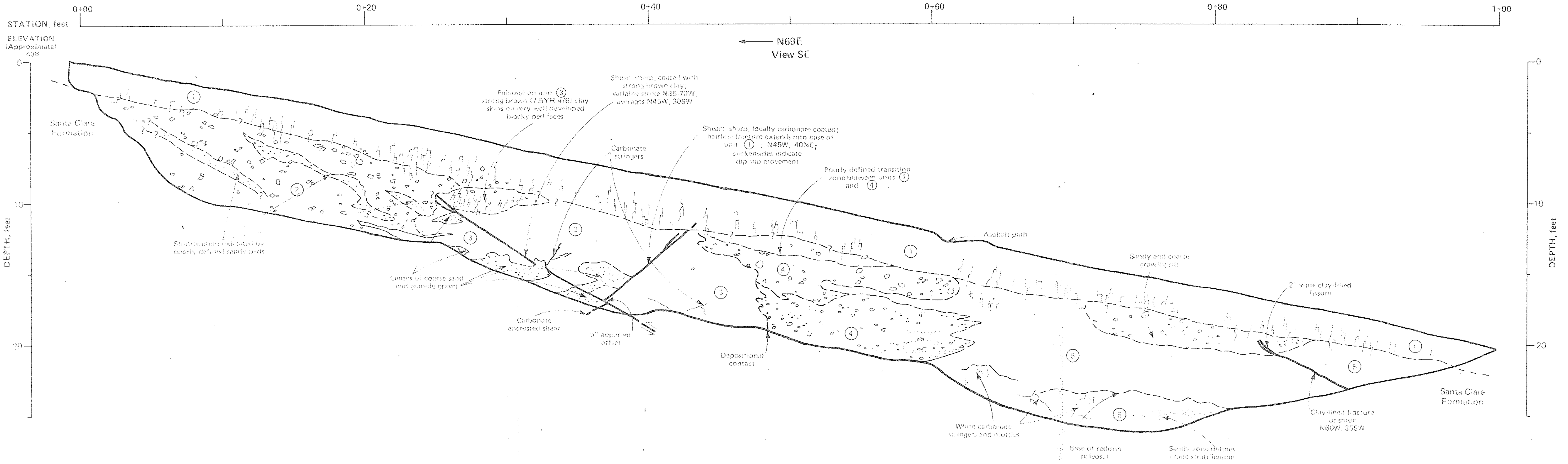
GEOLOGIC LOG OF
EXPLORATORY TRENCH T-1 (CONTINUED)

Checked by ____________ Date ______	Project No.	Figure No.
Approved by ____________ Date ______	3223	4E



PRINCIPAL GEOLOGIC UNITS

① TOPSOIL SANDY CLAY, dark brown (7.5 YR 3/4) with brown (7.5 YR 4/4) clay skins on abundant well developed blocky ped faces at base of unit; highly plastic; very stiff to hard; scattered angular pebbles, 5-10% fine to coarse sand; upper 1 to 2' disturbed by cultivation; damp to moist.

② GRAVELLY CLAYEY SAND: strong brown (7.5 YR 4/6); moderately to highly plastic fines 10-30%, very fine to coarse sand 60-80%, 5-15% fine to coarse angular gravel up to 5" most of which are 1-4", buff-colored, deeply weathered sandstone clasts (Franciscan graywacke); dense to very dense; massive to crudely bedded; damp.

③ SILTY SAND TO SANDY SILT: dark yellowish brown (10 YR 4/6) mottled with yellowish red (5 YR 4/6) clay coatings on fractures; low plasticity fines 40-60%, very fine sand 40-60%; dense to very dense; generally massive; damp.

④ GRAVELLY SAND strong brown (7.5 YR 4/6) clay matrix and clay skins dominate the color; medium to high plasticity fines 5-20%, fine to coarse sand 60%, fine to medium gravel 30%, gravel typically sub-rounded in lower zone; very dense; massive with crude stratification locally; damp to moist.

⑤ SANDY SILT: dark yellowish brown (10 YR 4/6) with well developed strong brown (7.5 YR 4/6) clay skins at contact with ① that die out 2' lower; slightly plastic fines 80%+, fine sand to fine gravel 20%+; dense to very dense; massive, damp.

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EL CAMINO HOSPITAL CONTINUING CARE CENTER
GEOLOGIC LOG OF
EXPLORATORY TRENCH T-6

Checked by	Date	Project No. 3223	Figure No. 5
Approved by	Date		

PRINCIPAL GEOLOGIC UNITS

① **TOPSOIL-SANDY CLAY:** brown (10 YR 4/3); scattered fine to coarse sand 10 - 30%, gravel less than 1%; moderately to highly plastic; hard, brittle; damp to dry granular grading to blocky downwards; abundant desiccation cracks.

② **SILTY SANDY GRAVEL:** yellow brown (10 YR 5/7), with lenses of sand and clayey silt; massive within beds; typically of low plasticity; dense; dry to damp.

③ **SANDY CLAY:** yellowish red (5 YR 4/6); low plasticity fines 40 - 50%, fine to coarse sand approximately 40% plus scattered gravel; massive; dense - very dense; hard; damp.

④ **GRAVELLY CLAY:** distinctive yellowish red (5 YR 4/6), approx. 80% low plasticity fines, approx. 20% fine sand and angular gravel; massive; hard; damp.

⑤ **SILTY SAND:** yellow brown (10 YR 5/6) 35 - 60% slightly plastic fines, fine to coarse sand locally stratified very dense; dry.

⑥ **SANDY SILT to SILTY SAND:** strong brown (7.5 YR 5/8) mottled with 5 YR clay skins on fracture surfaces and Fe oxide stained sand, local gravel lenses; dense; brittle; dry to damp.

⑦ **CLAYEY SAND & GRAVEL:** yellow brown (10 YR 5/8), mottled; slightly to moderately plastic fines 30 - 60%; weathered graywacke clasts to 3", typically 1"; massive; dense.

⑧ **CLAYEY SAND & GRAVEL:** yellow brown (10 YR 5/6); various mixtures of fine sand, coarse sand and gravel, slightly to moderately plastic fines 20 - 40%; grades locally to gravelly clay, typically massive; dense.

Scale: 1" = 5'
Horizontal = Vertical

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EL CAMINO HOSPITAL CONTINUING CARE CENTER
GEOLOGIC LOG OF
EXPLORATORY TRENCH T-2

Checked by ____________	Date ________	Project No.	Figure No.
Approved by ____________	Date ________	3223	5a

PRINCIPAL GEOLOGIC UNITS

① TOPSOIL-SANDY CLAY: dark yellowish brown (10 YR 3/4); moderately to highly plastic, fines approx. 70%; prominent desiccation fractures; hard; modern A horizon.

② Bt HORIZON-SANDY CLAY: dark brown (7.5 YR 4/4), very weakly developed clay skins on moderately well developed blocky peds.

③ CLAYEY SAND & GRAVEL: brown (7.5 YR 4/4); moderately to highly plastic, fines approx. 30%; fine to coarse subrounded sand; approx. 5% gravel; well graded; medium dense; damp-moist.

④ SANDY CLAY: yellow brown (10 YR 5/6) mottled with Fe oxide stains, gley colors locally; 30 - 40% very fine sand; massive; low plasticity, hard; damp; moderately well developed clay skins on local peds at top unit.

⑤ SILTY SAND: dark yellow brown (10 YR 4/6) grading downwards to light olive brown (2.5 YR 4); typically fine sand with 20 - 50% nonplastic fines; massive; very dense; damp to moist.

⑥ CLAYEY SAND & GRAVEL: brown (7.5 YR 4/4); 2 - 8% coarse gravel, locally rubbly, fine to coarse sand; low to highly plastic fines less than 35%; weakly stratified; medium dense to dense; moist.

⑦ SANDY CLAY: yellow brown (10 YR 5/6) to strong brown (7.5 YR 4/6) clay skins on peds; very fine sand, highly plastic fines approx. 50%; very dense.

⑧ SILTY CLAY: pale olive (5 Y 6/4) mottled with Fe oxides and gley colors; massive; low plasticity fines approx. 90%; very dense; hard; damp.

C1025

Scale: 1" = 5'
Horizontal = Vertical

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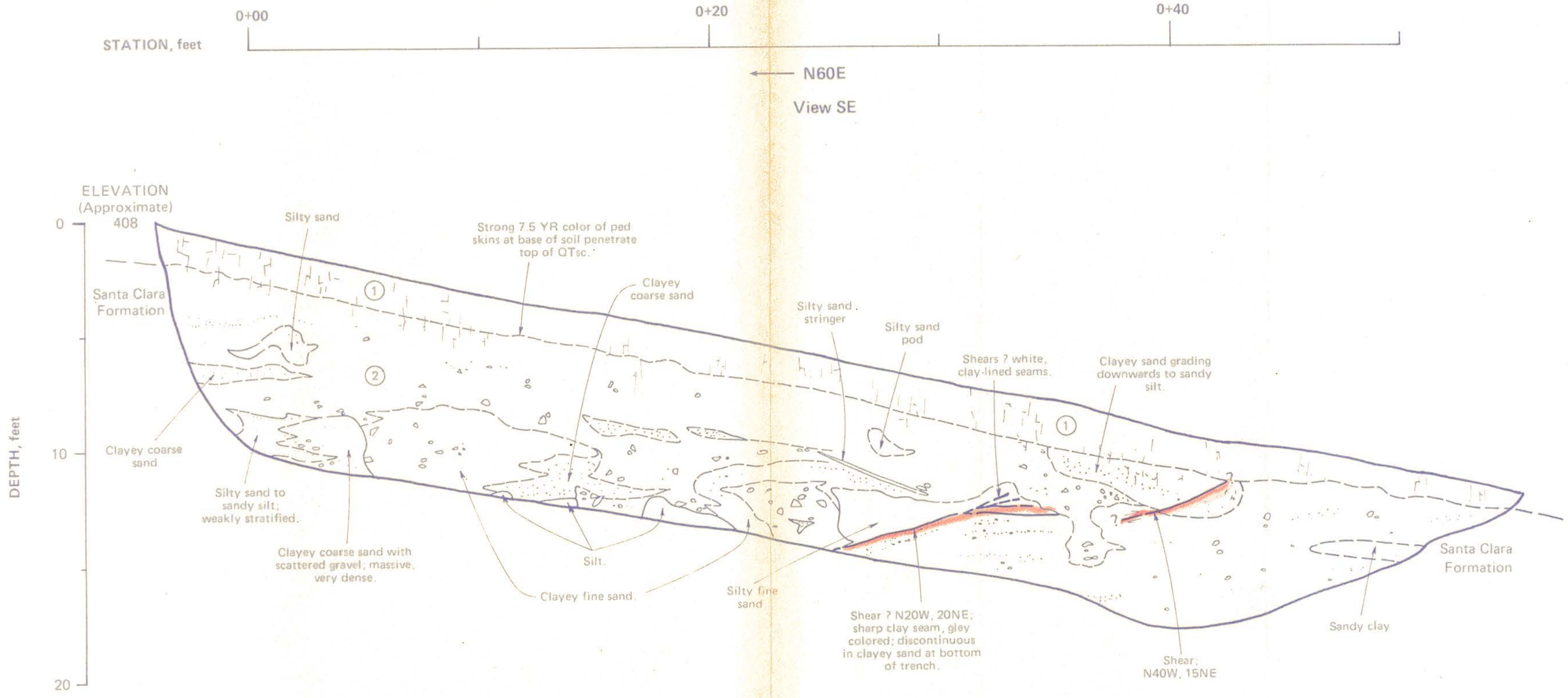
EL CAMINO HOSPITAL CONTINUING CARE CENTER
GEOLOGIC LOG OF
EXPLORATORY TRENCH T-3

Checked by ____________ Date ______
Approved by ____________ Date ______
Project No. 3223
Figure No. 6



C1025

Scale: 1" = 5'
Horizontal = Vertical

PRINCIPAL GEOLOGIC UNITS

① TOPSOIL-SANDY CLAY: dark brown (7.5 YR 4/4) clay skins; blocky peds in lowest ½ - 1' of soil

② CLAYEY SAND & GRAVEL: massive to weakly stratified; dense to very dense; moderately to highly plastic; fines = 10 - 40%.

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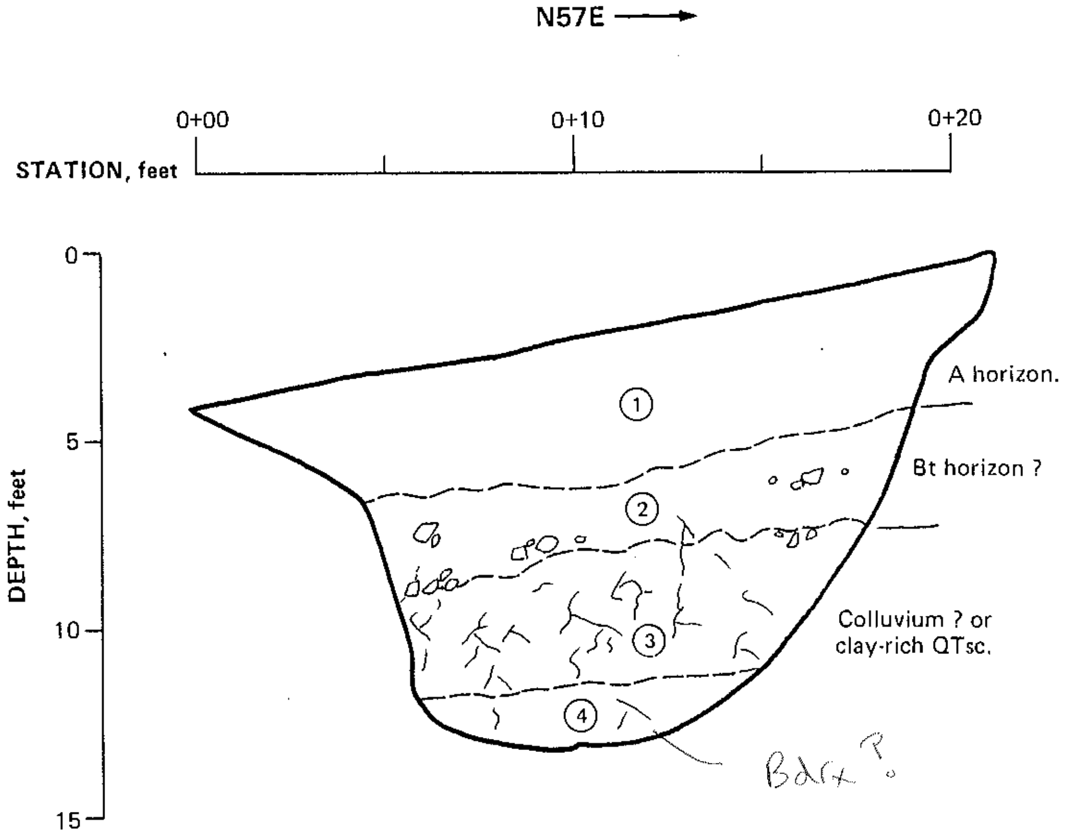
EL CAMINO HOSPITAL CONTINUING CARE CENTER
GEOLOGIC LOG OF
EXPLORATORY TRENCH T-4

Checked by ____ Date ____ Approved by ____ Date ____	Project No. 3223	Figure No. 7



① TOPSOIL: SANDY CLAY: dark reddish brown (5 YR 3/3), 85% moderately to highly plastic fines, scattered sand and gravel; porous; stiff; moist.

② SANDY CLAY: strong brown (7.5 YR 4/4) on well developed peds; 30% sand with scattered gravel; very stiff to hard, moist; shiny, clay-coated surfaces present.

③ SILTY CLAY: dark yellow brown (10 YR 4/4); pervasive, randomly oriented, shiny, striated clay skins on desiccation cracks; highly plastic; very stiff; moist.

④ VERY FINE SANDY SILT: yellow brown (10 YR 5/8) faintly mottled with Fe oxides and gley colors; massive; low plasticity fines, medium dense; moist; abundant shiny clay-coated surfaces.

Scale: 1″ = 5′
Horizontal = Vertical

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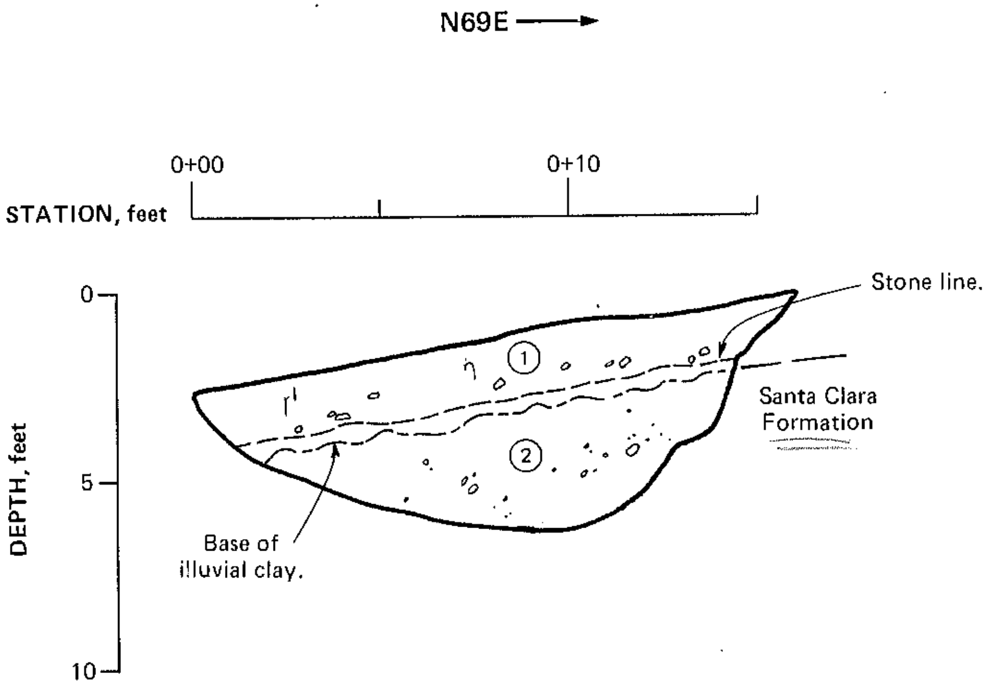
GEOLOGIC LOG OF
TEST PIT A

Checked by ______________ Date ________
Approved by ______________ Date ________

Project No. 3223
Figure No. 9



① TOPSOIL-SANDY CLAY: dark reddish brown (5 YR 3/3), 85%+ moderately to highly plastic fines, scattered sand and gravel; moderately developed desiccation cracks; brown (7.5 YR 3/4) clay skins well developed at base.

② CLAYEY SAND and GRAVEL: strong brown (7.5 YR 4/6), 30 - 50% moderately to highly plastic fines, remainder fine to coarse sand and fine angular gravel; massive, medium dense to dense; moist.

Scale. 1" = 5'
Horizontal = Vertical

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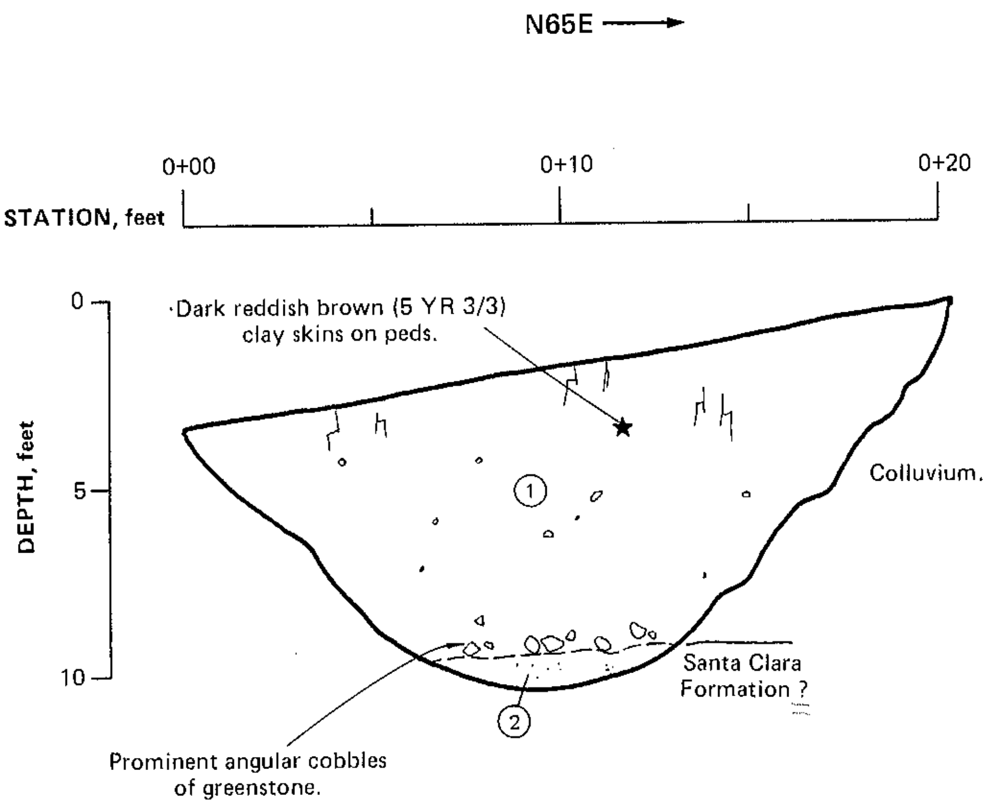
EL CAMINO HOSPITAL CONTINUING CARE CENTER
GEOLOGIC LOG OF
TEST PIT B

Checked by ______________ Date ______
Approved by ______________ Date ______
Project No. 3223
Figure No. 10



① COLLUVIUM-SANDY and GRAVELLY CLAY: dark brown (7.5 YR 3/4); 80%+ moderate to high plasticity fines; massive; stiff; weak blocky structure within 2-3′ of surface; damp to moist.

② SANDY SILT: yellow brown (10 YR 5/6), mottled; deeply weathered; 60%+ low plasticity fines; local thick yellowish red (5 YR 4/6) clay skins; dense; damp to moist.

Scale: 1″ = 5′
Horizontal = Vertical

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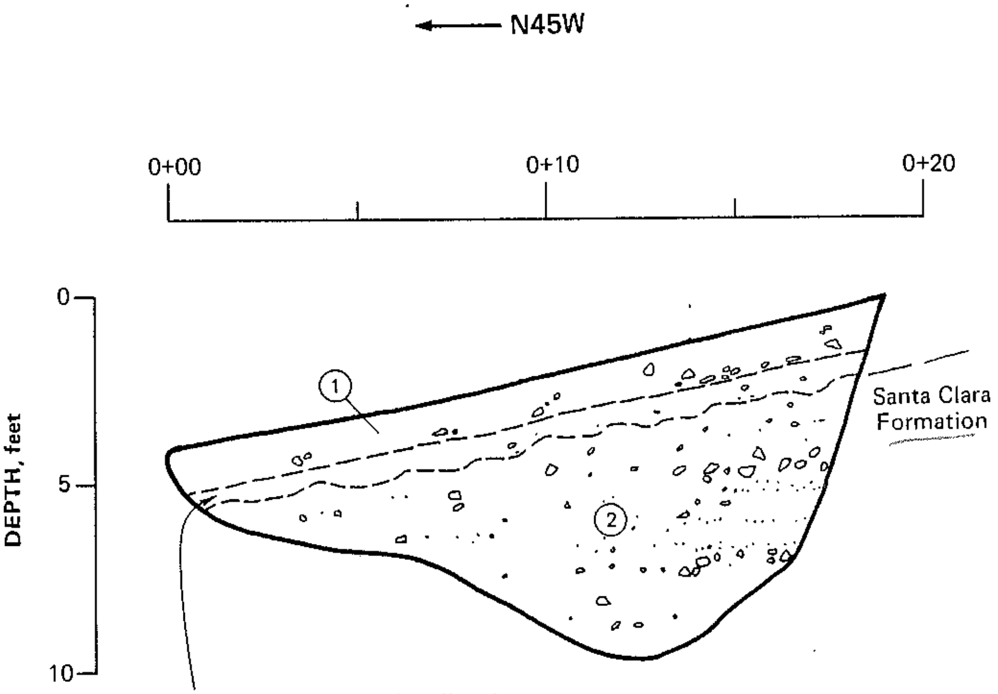
EL CAMINO HOSPITAL CONTINUING CARE CENTER
GEOLOGIC LOG OF
TEST PIT C

Checked by ____________	Date ______	Project No.	Figure No.
Approved by ____________	Date ______	3223	11



Transitional zone; dark yellow brown (10 YR 4/4); higher plasticity than ①, very stiff.

① TOPSOIL-GRAVELLY, SANDY CLAY: dark brown (7.5 YR 3/4), scattered sand and gravel with subrounded cobbles common near base; crumbly; low plasticity fines; weakly developed peds with dark reddish brown (5 YR 3/2) clay skins; damp.

② CLAYEY SAND and GRAVEL: strong brown (7.5 YR 5/6), 30 - 40% moderately plastic fines, fine to coarse sand, fine to coarse gravel; weakly stratified; medium dense; moist.

Scale: 1″ = 5′
Horizontal = Vertical

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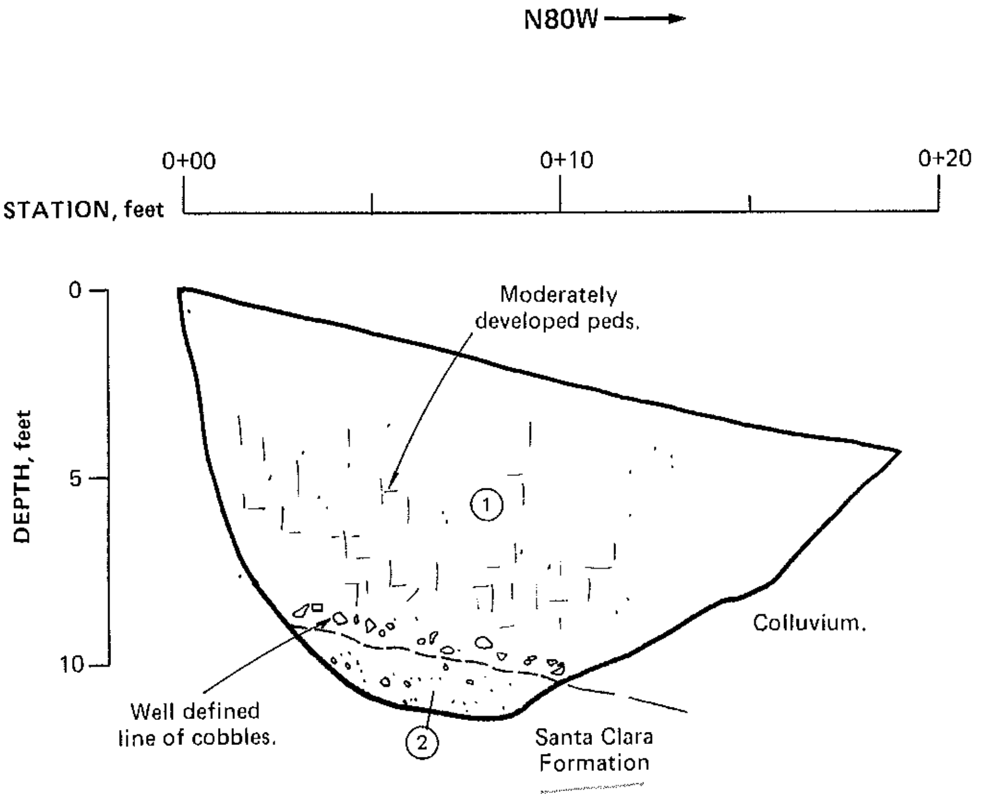
EL CAMINO HOSPITAL CONTINUING CARE CENTER
GEOLOGIC LOG OF
TEST PIT D

Checked by ______	Date ______	Project No.	Figure No.
Approved by ______	Date ______	3223	12



① COLLUVIUM-SANDY CLAY: dark brown (7.5 YR 3/4), 95% moderate to high plasticity fines; weak blocky peds becoming better developed with depth with strong brown (7.5 YR 4/6) clay skins; locally crumbly; hard; damp.

② GRAVELLY SILT and CLAY: light yellow brown (2.5 YR 6/4), very mottled; highly plastic fines near contact, becoming siltier with depth, 50% sand and gravel; deeply weathered; massive ?; dense; moist.

Scale: 1″ = 5′
Horizontal = Vertical

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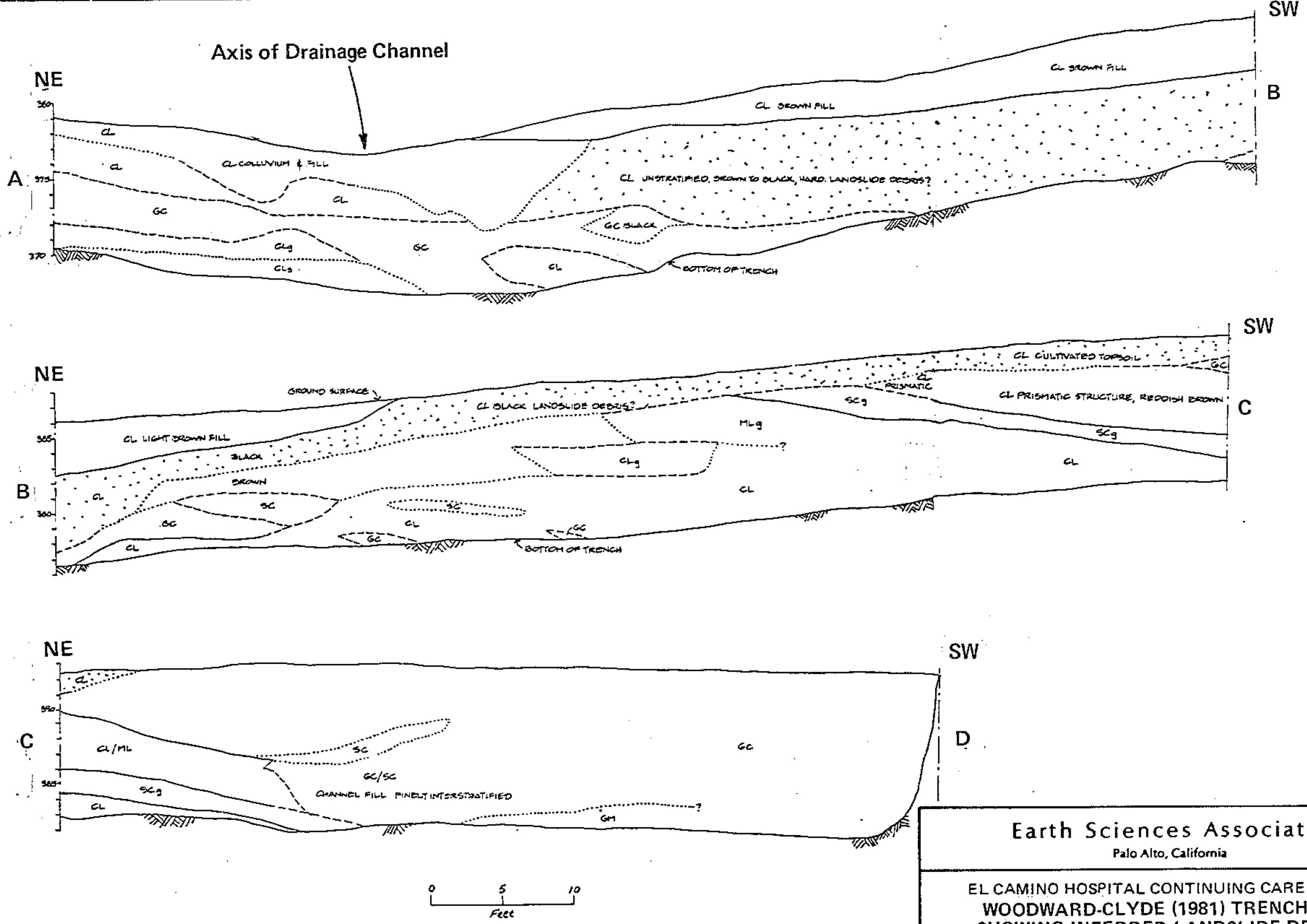
EL CAMINO HOSPITAL CONTINUING CARE CENTER
GEOLOGIC LOG OF
TEST PIT E

Checked by ______	Date ______	Project No.	Figure No.
Approved by ______	Date ______	3223	13



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EL CAMINO HOSPITAL CONTINUING CARE CENTER

WOODWARD-CLYDE (1981) TRENCH LOG
SHOWING INFERRED LANDSLIDE DEPOSIT

Checked by ______________ Date ______	Project No.	Figure No.
Approved by ______________ Date ______	3223	6

APPENDIX E: CBC SEISMIC DESIGN CRITERIA FOR SKILLED NURSERY FACILITY

For the Skilled Nursing Facility, we developed site-specific seismic design parameters in accordance with Chapters 16A and 18A of the 2016 California Building Code (CBC) and Chapters 11 and 21 of ASCE 7-10.

E.1 SITE CLASSIFICATION

Code-based site classification and ground motion attenuation relationships are based on the geology, engineering properties, and average shear wave velocity of the top 100 feet (30 meters) of the soil profile.

As discussed in Section 3, our borings encountered Santa Clara formation material at the ground surface and extending to a minimum depth of 50 feet, the maximum depth explored. The Santa Clara formation is a Pleistocene and Pliocene, non-marine sedimentary unit. The Santa Clara formation materials encountered at the site consisted of very stiff to hard sandy lean clays and medium dense to very dense clayey sands with varying amounts of gravel. Pocket penetrometer results indicate that the clays have unconfined compressive strengths of at least 4,000 psf and undrained shear strength greater than 2,000 psf for the entire soil profile. Standard Penetrations Tests were performed using an automatic trip hammer operating at an energy ratio of approximately 85%.

Wills et al (2000) map Plio-Pleistocene sedimentary units in the San Francisco Bay Area as Site Class C. Wills and Clahan (2006) compiled 18 shear wave velocity measurements for Quaternary to Tertiary (Plio-Pleistocene) sedimentary units, including the Santa Clara formation, and reported that the group had a mean V_{S30} of 455 m/s.

The USGS performed shear wave velocity measurements in a borehole located approximately 0.7 miles to the southwest of the project site. The “Maryknoll” boring encountered Santa Clara formation materials similar to those encountered at the project site. Shear wave velocity measurements indicate that the site had an average shear wave velocity of 485 meters per second for the top 100 feet (30 meters) of the profile (Gibbs et al, 1975).

Based on the conditions encountered in our borings, nearby V_S measurement, and available geologic data, the site may be classified as Soil Classification C, which is described as “very stiff soil and soft rock.” Our site-specific ground motion hazard analysis considered a V_{S30} of 400 m/s.

E.2 CODE-BASED SEISMIC DESIGN PARAMETERS

Code-based spectral acceleration parameters were determined based on mapped acceleration response parameters adjusted for the specific site conditions. Mapped Risk-Adjusted Maximum Considered Earthquake (MCE_R) spectral acceleration parameters (S_S and S_1) were calculated using the *U. S. Seismic Design Maps* on-line hazard calculator (USGS, 2014).

The mapped acceleration parameters were adjusted for local site conditions based on the average soils conditions for the upper 100 feet (30 meters) of the soil profile. Code-based MCE_R spectral response acceleration parameters adjusted for site effects (S_{MS} and S_{M1}) and design spectral response acceleration parameters (S_{DS} and S_{D1}) are presented in Table D1.

In accordance with CBC Section 1613A.3.5, Risk Category I, II, or III structures with mapped spectral response acceleration parameter at the 1-second period (S_1) greater than 0.75, are assigned Seismic Design Category E. In accordance with CBC 1616A.1.3, Seismic Design Category E structures require a site-specific ground motion hazard analysis. *Site-specific seismic design parameters are presented in Table D4, Section D.4.* ***The values in Table E1 should not be used for design.*** *Values are provided for determination of Seismic Design Category and comparison with minimum code requirements in our site-specific ground motion hazard analysis.*

Table E1: 2016 CBC Site Categorization and Site Coefficients

Classification/Coefficient	Design Value
Site Class	C
Site Latitude	37.338947°
Site Longitude	-122.088969°
Risk Category	I, II, or III
Seismic Design Category	E
Short Period Mapped Spectral Acceleration – S_S	2.268g
1-second Period Mapped Spectral Acceleration – S_1	0.819g
Short-Period Site Coefficient – F_a	1.0
Long-Period Site Coefficient – F_v	1.3
Short Period MCE Spectral Response Acceleration Adjusted for Site Effects – S_{MS}	2.268g
1-second Period MCE Spectral Response Acceleration Adjusted for Site Effects – S_{M1}	1.065g
Short Period, Design Earthquake Spectral Response Acceleration – S_{DS}	1.512g
1-second Period, Design Earthquake Spectral Response Acceleration – S_{D1}	0.710g
Long-Period Transition – T_L	12 seconds
Site Coefficient – F_{PGA}	1.0
Mapped Geometric Mean PGA – MCE_G	0.883g

E.3 SITE-SPECIFIC SEISMIC HAZARD ANALYSIS

We performed a site-specific hazard analysis in accordance with ASCE 7-10 Chapter 21.2 and 2016 CBC Section 1803A.6. Our analyses were performed using the computer program *EZ-*

Frisk, version 7.65.04 (Risk Engineering, 2015) and the 2008 USGS fault model (Petersen, et al., 2008).

Our analysis utilized the mean ground motions predicted by four of the Next Generation Attenuation (NGA) relationships: Boore-Atkinson (2008), Campbell-Bozorgnia (2008), Chiou-Youngs (2007), and Abrahamson-Silva (2007). Our analysis used the rotation factors specified in ASCE 7-10 Supplement No. 1 to calculate the maximum rotated component of ground motions (ASCE, 2013).

E.3.1 DETERMINISTIC MCE$_R$

We performed deterministic seismic hazard analyses in accordance with ASCE 7-10 Section 21.2.2. The deterministic MCE$_R$ acceleration response spectrum is defined as the largest 84th percentile ground motion in the direction of maximum horizontal response for each period for characteristic earthquakes on all known active faults within the region. Our analysis considered all known active faults within 200 kilometers of the site. As shown in Table 1 of the report (Section 2), the site is located within approximately 30 kilometers of five major faults. For periods up to 0.75 seconds, the largest deterministic ground motion resulted from a M_w 6.5 earthquake on the Monte Vista-Shannon Fault, located approximately 0.4 km from the site. For periods greater than 0.75 seconds, the largest deterministic ground motion resulted from a M_w 8.05 earthquake on the San Andreas Fault, located approximately 6.4 km from the site.

The 84th percentile ground motion in the direction of maximum horizontal response is presented on Figure E1 (green line). Spectral ordinates are tabulated in Table E2, Column 3.

ASCE 7-10 specifies that the deterministic MCE$_R$ shall not be less than the Deterministic Lower Limit MCE response spectrum (ASCE 7-10 Figure 21.2-1). The Deterministic Lower Limit spectrum is presented on Figure E1 (blue line). Spectral ordinates are tabulated in Table E2, Column 4.

The deterministic MCE spectrum was calculated by taking the greater of Table E2, Columns 3 and 4.

Spectral ordinates for the deterministic MCE$_R$ are tabulated in Table E2, Column 5, and presented graphically on Figure E1 (dashed black line).

E.3.2 PROBABILISTIC MCE$_R$

We performed a probabilistic seismic hazard analysis (PSHA) in accordance with ASCE 7-10 Section 21.2.1. The probabilistic MCE acceleration response spectrum is defined as the 5 percent damped acceleration response spectrum having a 2 percent probability of exceedance in a 50 year period (2,475-year return period). Our PSHA considered all known active faults within 200 kilometers of the site as well as a gridded seismic source modeled by the USGS (2008). The probabilistic MCE spectrum was multiplied by Risk Coefficients (C_R) to determine the probabilistic MCE$_R$. We used Risk Coefficients (C_{RS} and C_{R1}) of 0.974 and 0.926, respectively, based on ASCE 7-10 Section 21.2.1.1 - Method 1 and the USGS on-line calculator.

The resulting probabilistic MCE_R is presented on Figure E2 (red line). Spectral ordinates are tabulated in Table E2, Column 6.

E.3.3 SITE-SPECIFIC MCE_R

The site-specific MCE_R is defined by ASCE 7-10 Section 21.2.3 as the lesser of the deterministic and probabilistic MCE_R's at each period. Spectral ordinates for the site-specific MCE_R are tabulated in Table E2, Column 7, and presented graphically on Figure E2 (dashed black line).

Table E2: Development of Site-Specific MCE_R Spectrum

Period (seconds)	CBC General Spectrum (g)	Largest 84th Percentile Deterministic (g)	Deterministic Lower Limit (g)	Deterministic MCE_R (g)	Probabilistic MCE_R (g)	Site-Specific MCE_R (g)
0.000	0.605	0.997	0.600	0.997	1.057	0.997
0.050	1.088	1.245	1.033	1.245	1.283	1.245
0.094	1.512	1.648	1.413	1.648	1.783	1.648
0.100	1.512	1.699	1.465	1.699	1.853	1.699
0.104	1.512	1.729	1.500	1.729	1.894	1.729
0.200	1.512	2.124	1.500	2.124	2.312	2.124
0.300	1.512	2.161	1.500	2.161	2.318	2.161
0.400	1.512	2.081	1.500	2.081	2.151	2.081
0.469	1.512	1.981	1.500	1.981	2.048	1.981
0.500	1.420	1.937	1.500	1.937	2.006	1.937
0.520	1.365	1.897	1.500	1.897	1.975	1.897
0.600	1.183	1.781	1.300	1.781	1.868	1.781
0.750	0.946	1.550	1.040	1.550	1.658	1.550
1.000	0.710	1.307	0.780	1.307	1.360	1.307
2.000	0.355	0.773	0.390	0.773	0.775	0.773
3.000	0.237	0.560	0.260	0.560	0.538	0.538
4.000	0.177	0.425	0.195	0.425	0.407	0.407
5.000	0.142	0.347	0.156	0.347	0.341	0.341

E.3.4 DESIGN RESPONSE SPECTRUM

The site-specific Design Response Spectrum (DRS) is defined in ASCE 7-10 Section 21.3 as two-thirds of the site-specific MCE_R. Additionally, the DRS may not be less than 80% of the general design response spectrum. Spectral accelerations corresponding to ⅔ of the MCE_R are tabulated in Table E3, Column 2. Ordinates corresponding to 80% of the general Site Class C response spectrum are tabulated in Table E3, Column 3. Ordinates of the site-specific DRS are tabulated in Table E3, Column 4. Development of the site-specific DRS is presented graphically on Figure E3.

Table E3: Development of Site-Specific Design Response Spectrum

Period (seconds)	2/3 Site-Specific MCE_R (g)	80% CBC General Spectrum (g)	Design Response Spectrum (g)
0.000	0.664	0.484	0.664
0.050	0.830	0.870	0.870
0.094	1.099	1.210	1.210
0.100	1.133	1.210	1.210
0.104	1.153	1.210	1.210
0.200	1.416	1.210	1.416
0.300	1.441	1.210	1.441
0.400	1.387	1.210	1.387
0.469	1.320	1.210	1.320
0.500	1.291	1.136	1.291
0.520	1.265	1.092	1.265
0.600	1.188	0.946	1.188
0.750	1.034	0.757	1.034
1.000	0.871	0.568	0.871
2.000	0.516	0.284	0.516
3.000	0.359	0.189	0.359
4.000	0.272	0.142	0.272
5.000	0.228	0.114	0.228

E.4 DESIGN ACCELERATION PARAMETERS

Site-specific design acceleration parameters (S_{DS} and S_{D1}) were determined in accordance with Section 21.4 of ASCE 7-10. S_{DS} is defined as the design spectral acceleration at a period of 0.2 seconds, but not less than 90% of the spectral acceleration at any period greater than 0.2 seconds. S_{D1} is defined as the greater of the design spectral acceleration at a period of 1 second or two times the spectral acceleration at a period of 2 seconds.

Site-specific MCE_R spectral response acceleration parameters (S_{MS} and S_{M1}) are calculated as 1.5 times the S_{DS} and S_{D1} values, respectively, but not less than 80% of the code-based values presented in Table 2. Site-specific design acceleration parameters are summarized in Table E4.

When using the Equivalent Lateral Force Procedure, ASCE 7-10 Section 21.4 allows using the spectral acceleration at any period (T) in lieu of S_{D1}/T in Eq. 12.8-3. The site-specific spectral acceleration at any period may be calculated by interpolation of the spectral ordinates in Table E3, Column 4.

Table E4: Site-Specific Design Acceleration Parameters

Parameter	Value
S_{DS}	1.416
S_{D1}	1.032
S_{MS}	2.124
S_{M1}	1.548

E.5 MCE$_G$ PEAK GROUND ACCELERATION

We calculated the MCE Geometric Mean Peak Ground Acceleration (MCE$_G$) in accordance with ASCE 7-10 Section 21.5. The MCE$_G$ is calculated as the lesser of probabilistic and deterministic geometric mean PGA. The 2% in 50-year probabilistic geometric mean PGA is 0.986g. The deterministic MCE$_G$ is considered the greater of the largest 84th percentile deterministic geometric mean PGA (0.906g) or one-half of the tabulated F_{PGA} value from ASCE 7-10 Table 11.8.1. For the site, F_{PGA} is 1.0 and one half of the F_{PGA} is 0.5g; therefore, the deterministic MCE$_G$ is 0.906g. Additionally, the MCE$_G$ may not be less than 80% of the mapped PGA$_M$ determined from ASCE -10 Equation 11.8-1. The PGA$_M$ for the site is 0.883g; 80% of PGA$_M$ is 0.706g. Therefore, the MCE$_G$ for the site may be considered 0.906g.

SECTION E.6: REFERENCES

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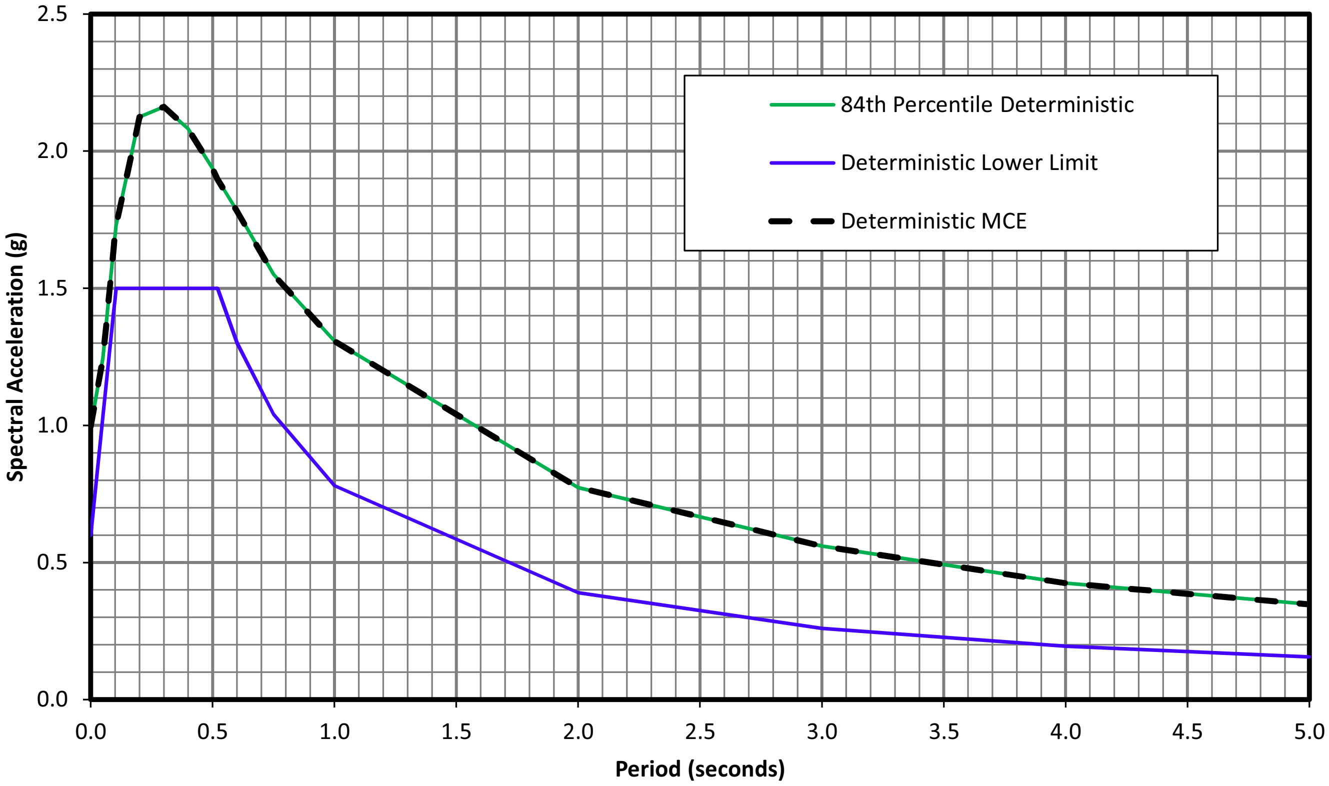


The Deterministic Maximum Considered Earthquake (MCE_R) is defined as the greater of the following at all periods:

- The largest 84th percentile ground motion from a characteristic earthquake on all known active faults, or
- The Deterministic Lower Limit MCE Spectrum – ASCE 7–10, Figure 21.2–1.

Spectral ordinates are presented in Table D2.

CORNERSTONE EARTH GROUP

Deterministic Maximum Considered Earthquake

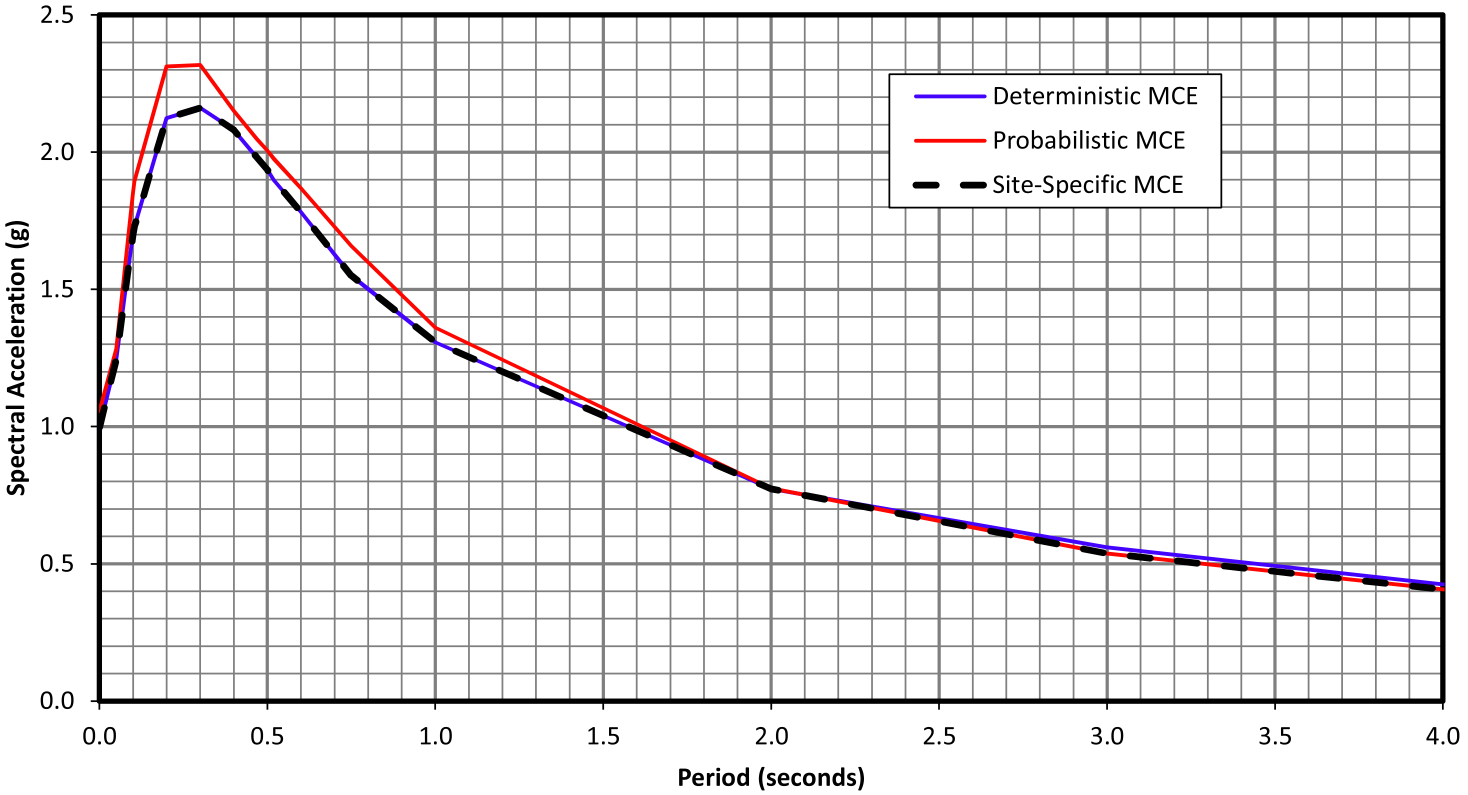
The Forum at Rancho San Antonio
Cupertino, CA

Project Number 905-1-1

Figure Number Figure E-1

Date April 2017

Drawn By RRN



The Site-Specific Maximum Considered Earthquake is defined as the lesser of the following at all periods:

- Deterministic MCE_R – developed on Figure E-1 or
- Probabilistic MCE_R – defined as the 2,475–year ground motion.

Spectral ordinates are presented in Table D2.

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Site-Specific Maximum Considered Earthquake

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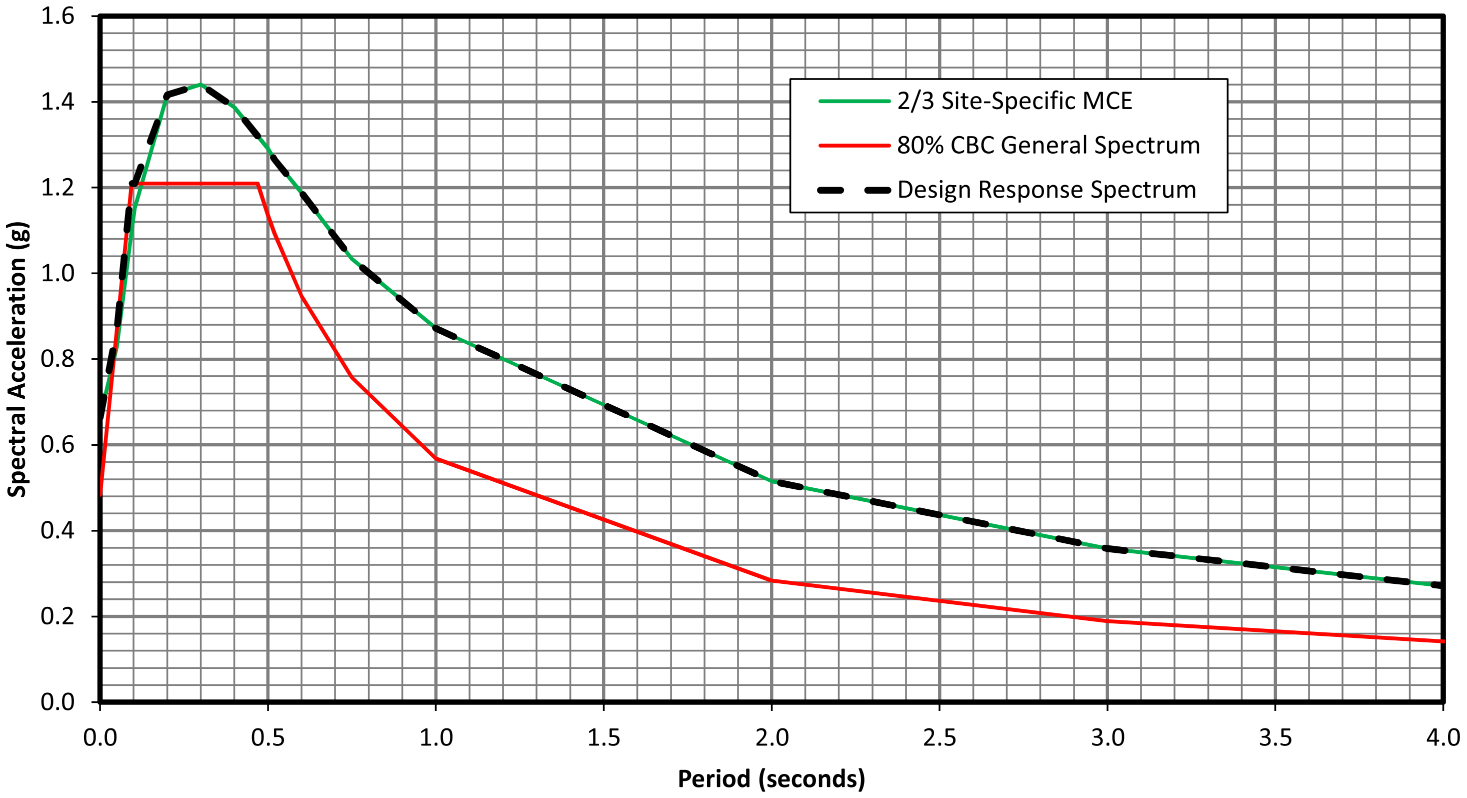
Cupertino, CA

Project Number	905-1-1
Figure Number	Figure E-2
Date	April 2017
Drawn By	RRN



The Site-Specific Design Response Spectrum is defined as the greater of the following at all periods:

- 2/3 of the Site-Specific MCE_R – developed on Figure E-2, or
- 80% of the CBC General Spectrum.

Spectral ordinates are presented in Table D3.

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Design Response Spectrum

The Forum at Rancho San Antonio
Cupertino, CA

Project Number	905-1-1
Figure Number	Figure E-3
Date	April 2017
Drawn By	RRN

APPENDIX F: FOUNDATION CALCULATIONS FOR SKILLED NURSERY FACILITY

Project Name: THE FORUM - SNF
Subject: DRILLED PIER ANALYSIS
Project No.: 905-1-1 Sheet No.: 1 of
By: Date: Chk'd by: Date:

THE FORUM - SKILLED NURSING FACILITY

STRUCTURE TYPE: 1-STORY, WOOD FRAME SLAB-ON GRADE ADDITION

SITE CONDITIONS: REFER TO FIG 6B CROSS SECTIONS C & D (FIGS 7C & 7D)

EXISTING CONDITIONS: PARKING LOT, LANDSCAPING

FOUNDATION TYPE: DRILLED, CAST-IN-PLACE FRICTION PIERS TO MATCH EXISTING AS-BUILT FOUNDATIONS AND TO MITIGATE DIFF. FILL SETTLEMENT AT WEST END OF BUILDING

SUBSURFACE PROFILE

1.) PROFILE BASED ON DATA FROM BORINGS EB-5, 6, 7, 8, 23 CROSS SECTIONS C-C', D-D' (FIGS 7C, 7D)

2.) BUILDING AREA UNDERLAIN BY SANTA CLARA FORMATION (QTsc) SOILS:

- interbedded very stiff to hard clay (CL)
- medium dense to very dense clayey sand (SC) with gravel (14 to 20 percent silt/clay fines)

3.) ASSUME SKIN FRICTION CAPACITY ONLY (NO END BEARING) DUE TO LIGHT BUILDING LOADS

4) DUE TO BOTH COHESIVE & COHESIONLESS QTsc WITHIN UPPER 10 - 20' OF SOIL PROFILE, ULTIMATE SKIN FRICTION WILL BE EVALUATE FOR BOTH

Project Name: FORUM - SNF

Subject: DRILLED PIERS

Project No.: 905-1-1 Sheet No.: 2 of

By: Date: Chk'd by: Date:

ANALYSIS BASED ON FHWA-NHI-10-016 DRILLED SHAFTS MANUAL (CH 3, 13)

ULTIMATE SKIN FRICTION - SIDE RESISTANCE

COHESIONLESS SOILS $(c = s_u = 0)$

(EQ 13.5) $R_{SN} = \pi B \Delta z_i f_{SN_i}$ B = pier diameter

Δz_i = thickness of soil layer

f_{SN_i} = nominal side unit resistance (skin friction)

$$f_{SN_i} = \sigma'_v K \tan(\delta)$$

σ'_v = effective stress

K = horizontal stress coeff.

$\delta = \phi'$ = shaft-soil interface friction angle

$\beta = K \tan \delta$ = side resistance coeff. (BETA METHOD)

$$\therefore f_{SN_i} = \sigma'_v \beta$$

FOR SC SOIL PROFILE FROM EB-5, 6, 7, 8, 23

$\gamma = 110$ to 120 pcf $\Rightarrow$ assume $\gamma_{avg} = 115$ pcf

$c \cong 0$ psf

$\phi' = 27.5 + 9.2 \log(N_{1,60})$ (EQ 3.8 FHWA)

(SEE ATTACHED $N_{1,60}$ CORRECTIONS FROM N_M FROM EB-5, 6, 7, 8, 23)

$N_{1,60}$ IN SC LAYER FROM UPPER 20' OF SOIL PROFILE RANGED FROM 38 TO 103 bpf

$N_{1,60}$ (AVG) = 68 bpf

CORNERSTONE EARTH GROUP

Project Name: THE FORUM - SNF

Subject: DRILLED PIERS

Project No.: 905-1-1 Sheet No.: ______ of ______

By: ______ Date: ______ Chk'd by: ______ Date: ______

FOR $N_{1,60}$ RANGING FROM 30 - 40 bpf

$\phi' = 27.5 + 9.2 \log (30 - 40)$

$\phi' = 41$ to 42°

Note: considering the percent fines ranging from 14 to 20 % use a lower ϕ' to account for some reduce friction

use $\underline{\phi'_{AVG} = 38^{\circ}}$

$\sigma_v' = 115$ pcf $(10') = 1150$ psf [at assumed avg max depth (mid-pt)]

(EQ. 13-11) $\sigma_p' = P_a \cdot 0.47 (N_{60})^{m}$

$P_a = 2116$ psf

P_a = atmospheric pressure

m = 0.8 for silty/clayey sand

N_{60} = N-value @ 60% hammer energy (assume $N_{60\,AVG} = 30$)

$\sigma_p' = (2116 \text{ psf})\, 0.47 (30)^{0.8}$

$\sigma_p' = 15,112$ psf

(EQ 13-9) $OCR = \frac{\sigma_p'}{\sigma_v'}$

$OCR = \frac{15,112 \text{ psf}}{1150 \text{ psf}} = 13.1$

$K_o = (1 - \sin \phi')(OCR)^{\sin \phi'} < K_p$ (EQ 13.8)

$\therefore K_o = 1 - \sin (38^{\circ}) (13.1)^{\sin 38^{\circ}}$

$K_o = 1.88 < K_p = 4.2$ $K_p = \tan^2(45 + \frac{\phi'}{2})$

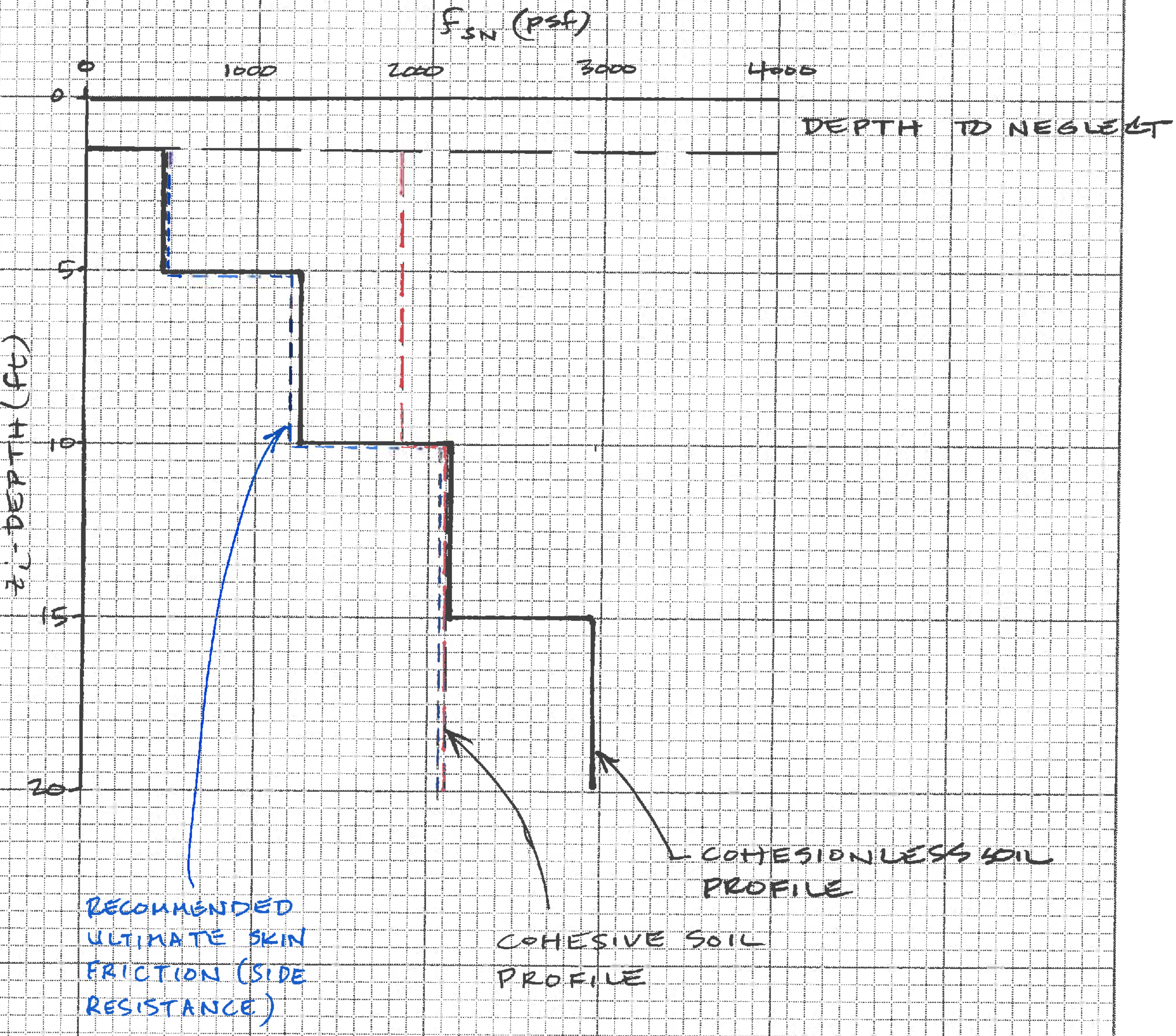
$\beta = K \tan \delta = (1.88) \tan (38) = 1.47$

$\therefore f_{SN} = \sigma_v' \beta = 115 \text{ pcf } \Delta z_i (1.47) = 169 \Delta z_i$ psf

CORNERSTONE
EARTH GROUP

Project Name: THE FORUM - SNF
Subject: DRILLED PIERS
Project No.: 905-1-1 Sheet No.: 4 of
By: Date: Chk'd by: Date:

DEPTH (ft)	z_i @ middle	f_{SN} psf
0-5	2.5	422
5-10	7.5	1270
10-15	12.5	2112
15-20	17.5	2960



CORNERSTONE EARTH GROUP

Project Name: THE FORUM -SNF
Subject: DRILLED PIERS
Project No.: 905-1-1 Sheet No.: 5 of
By: Date: Chk'd by: Date:

COHESIVE SOIL PROFILE

$$R_{SN} = \pi B \Delta z_i (\alpha S_u)_i$$

$$f_{SN_i} = (\alpha S_u)_i$$ (FHWA EQ 13-15)

α = UNIT SIDE RESISTANCE RELATIVE TO UNDRAINED SHEAR STRENGTH

S_u = UNDRAINED SHEAR STRENGTH

$\alpha = 0.55$ where $\frac{S_u}{P_a} \leq 1.5$ $P_a = 2116$ psf

$$\alpha = 0.55 - 0.1\left(\frac{S_u}{P} - 1.5\right)$$

FOR CLAY LAYERS WITH QTsc (EB-5, 6, 7, 8, 23)

EB-5 @ 6' (TxUU) $S_u \cong$ 3,400 psf

EB-6 @ 9' (TxUU) $S_u \cong$ 5,000 psf

Pocket penetrometers from all borings @ SNF ≥ 3 ksf

→ Assume 0-10' depth @ lower bound strength of $S_u \cong$ 3,400 psf

→ Assume 10-20' depth with S_u(avg) = 4,200 psf

DEPTH (ft)	S_u (psf)	α	f_{SN} (psf)
0 - 10	3400	0.54	1840
10 - 20	4200	0.5	2100

$$\alpha = 0.55 - 0.1\left(\frac{3400}{2116} - 1.5\right) = 0.54$$

$$\alpha = 0.55 - 0.1\left(\frac{4200}{2116} - 1.5\right) = 0.5$$

(SEE PLOT PG 4)

The Forum at Rancho San Antonio, Cupertino CA
905-1-1

Summary of SPT N-values at Skilled Nursing Facility (Borings EB-5, 6, 7, 8, 23)

Boring Number	Depth (ft)	Layer thickness (ft)	Measured N-Value[1] (bpf)	Fines Content (%)	C_E	C_B	C_R	C_S	N_{60} (bpf)	σ_{vo} (psf)	σ'_{vo} (psf)	C_N	$(N_1)_{60}$ (bpf)
5	9	4	24	15	1.33	1.15	0.85	1	31	1035	1035	1.23	38
5	14	3	29	15	1.33	1.15	0.85	1	38	1610	1610	1.08	41
5	19	6	60	15	1.33	1.15	0.95	1	87	2185	1935.4	1.02	89
6	4	3	29	15	1.33	1.15	0.75	1	33	460	460	1.49	50
7	4	2	60	15	1.33	1.15	0.75	1	69	460	460	1.49	103
7	6	3	60	15	1.33	1.15	0.8	1	74	690	690	1.34	99
7	9	6	33	15	1.33	1.15	0.85	1	43	1035	1035	1.21	52
7	19	8	60	15	1.33	1.15	0.95	1	87	2185	1935.4	1.02	89
7	34	5	60	15	1.33	1.15	1	1	92	3910	2724.4	0.94	86
8	3	2	26	15	1.33	1.15	0.75	1	30	345	345	1.61	48
8	4	2	31	15	1.33	1.15	0.75	1	36	460	460	1.49	53
8	6	2	34	15	1.33	1.15	0.8	1	42	690	690	1.34	56
8	9	6	30	15	1.33	1.15	0.85	1	39	1035	1035	1.21	47
23	4	2	32	15	1.33	1.15	0.75	1	37	460	460	1.49	55
23	6	3	60	15	1.33	1.15	0.8	1	74	690	690	1.34	99
23	9	3	53	15	1.33	1.15	0.85	1	69	1035	1035	1.21	83

AVG 68

[1] Includes 0.6 correction factor applied to Modified California sampler with 3" outside diameter (where applicable)