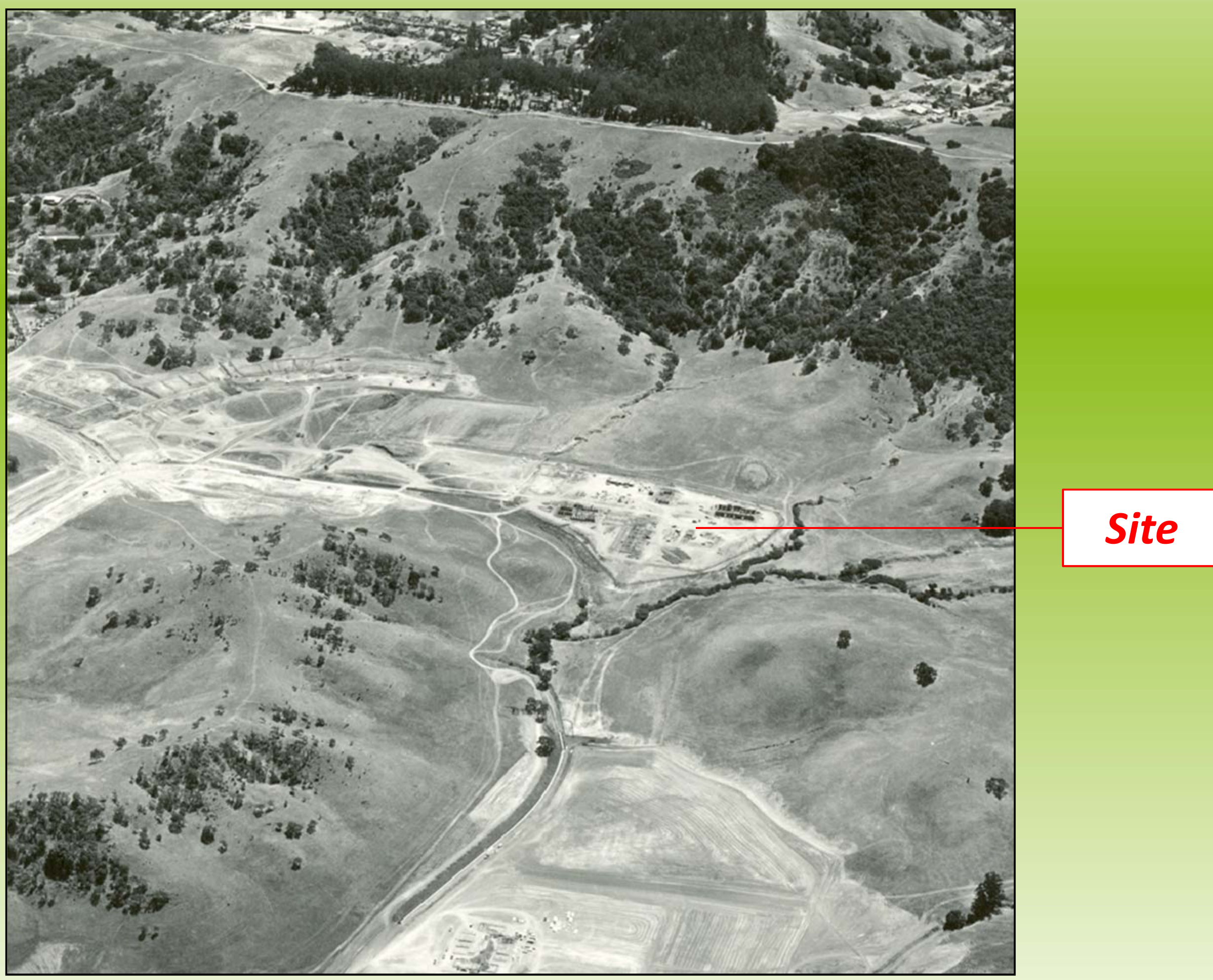
Preliminary Geotechnical Investigation and Geologic Hazards Study Report
Terra Linda High School
320 Nova Albion Way
San Rafael, Marin County, California



August 15, 1959 Aerial Photograph (looking southwest)

SUBMITTED TO:

San Rafael City Schools
c/o Pete Norgaard, Van Pelt Construction Services
310 Nova Albion Way, Room 505
San Rafael, CA 94903
pete@vpsconline.com

March 16, 2017

A3GEO

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San Rafael City Schools
c/o Pete Norgaard
Van Pelt Construction Services
310 Nova Albion Way, Room 505
San Rafael, CA 94903
pete@vpsconline.com

RE: Preliminary Geotechnical Investigation and Geologic Hazards Study Report
Terra Linda High School
320 Nova Albion Way
San Rafael, Marin County, California

Dear Mr. Norgaard,

This report presents the results of A3GEO's preliminary geotechnical investigation and geologic hazard study for planned improvements to Terra Linda High School (TLHS) in San Rafael, California. Our services were authorized by the San Rafael City Schools (District) under an Independent Consultant Agreement for Professional Services entered into on January 3, 2017. The planned improvements discussed in this report and shown on the attached figures are based on information obtained from the District's 2015 Facilities Master Plan (HY Architects, 2015).

The attached report provides information on geotechnical, geologic and seismic conditions, presents our preliminarily assessment of potential site hazards and constraints, and includes preliminary geotechnical recommendations intended for project evaluation and costing purposes. Please note that this report is not a design-level study and that supplemental investigation, testing and analyses would be needed prior to the design and construction of a future project at the TLHS campus.

The conclusions and recommendations presented in this report were developed in accordance with generally-accepted geotechnical principles and practices at the time that the report was prepared. Should you have questions or comments concerning our findings, the design concepts discussed, or our recommendations, please do not hesitate to call.

Sincerely,

A3GEO, Inc.

Wayne Magnusen, P.E., G.E.
Principal Engineer
Cell: (510) 325-5724

Dona Mann, P.E., G.E.
Principal Engineer
Cell: (415) 425-0247

Attachments: Report
Figures 1 – 8
Appendices A - G

REPORT

1.00 INTRODUCTION

1.01 Overview

This report presents the results of our preliminary (Phase 1) geotechnical investigation and geologic hazards study for Terra Linda High School in San Rafael, California. Our services were authorized by the San Rafael City Schools (District) under an Independent Consultant Agreement for Professional Services entered into on January 3, 2017. The location of Terra Linda High School (TLHS) indicated on the Site Location Map (Figure 1); the school address is 320 Nova Albion Way in San Rafael, California.

The District's 2015 Facilities Master Plan (HY Architects, 2015) indicates a variety of planned new improvements to the TLHS campus. This Phase 1 report addresses geotechnical, geologic and seismic conditions for the campus as a whole; our onsite investigations included borings and cone penetration tests (CPTs) near the locations of planned buildings and additions to existing buildings as shown on the Site Plan (Figure 2) and the accompanying Predevelopment Aerial Photograph (Plate 3). We understand that the new buildings and additions will generally be one or two stories tall and that at least two of the buildings will replace existing structures that are to be demolished. The elevation contours on the Site Plan (Figure 2) show that the portion of the campus in which existing buildings are located is approximately level and near Elevation +80 feet (NAVD88 datum). The planned new buildings and additions are all located in the nearly-level portion of the campus; a planned practice gym with basketball courts in the southeast part of the campus may be notched into an approximately 10-foot-high slope (Figure 2).

1.02 Purpose and Scope

The primary purpose this Phase 1 study report is to provide information on geotechnical, geologic and seismic conditions at the TLHS campus, preliminarily assess potential site hazards and constraints, and develop preliminary geotechnical conclusions and recommendations for project evaluation and costing purposes. The scope of our Phase 1 services consisted of:

- Compiling and reviewing geotechnical, geologic, seismic and historical information
- Exploring subsurface conditions with borings and CPTs
- Conducting laboratory tests on samples retrieved from the borings
- Assessing geologic hazard potential and effects using new and existing data
- Developing geotechnical conclusions and recommendations pertaining to planned construction
- Preparing this report

This Phase 1 study report is preliminary in nature and additional investigations and/or analyses will likely be needed before a design-level report in compliance with CGS Note 48 requirements can be prepared. Our scope of services did not include an environmental assessment or investigation of the site for the presence of toxic material in the soil, groundwater, or air.

1.03 Report Organization

The remainder of this report is organized as follows:

Section 2.00 describes our methods of investigation
Section 3.00 describes the geologic, seismic and historical setting of the site
Section 4.00 describes site-specific geotechnical and geologic conditions
Section 5.00 presents our preliminary assessment of potential geologic hazards
Section 6.00 discusses geotechnical considerations for planned future improvements
Section 7.00 presents our preliminary recommendations for planned future improvements
Section 8.00 outlines the limitations of our study
Section 9.00 presents a list of selected references

2.00 METHODS OF INVESTIGATION

2.01 Review of Geologic, Seismic and Historical Information

We reviewed a variety of references containing information on geologic, seismic and historical conditions. Selected references are described below; a list of references used in preparing this report is presented in Section 9.00.

The geologic references that we reviewed included maps prepared by Rice, Smith and Strand (1976); Blake, Graymer and Jones (2000); and Graymer, and others (2006). There are no zoned active faults within the USGS San Rafael 7.5 minute quadrangle so there is no official Alquist-Priolo Earthquake Fault Zones Map (A-P Map) for the site area. The CGS also prepares Seismic Hazard Zone maps delineating zones of required investigation for earthquake-induced landsliding and liquefaction, but no map has yet been issued for the site area.

Geologic hazard maps prepared for local General Plan are contained in California Division of Mines and Geology (CDMG) Open-File Report 76-2 (Rice, Smith and Strand, 1976); we reviewed the Slope Stability Map from this publication as well as the more recent map of Slides and Earth Flows in USGS Open-File Report 97-745C (Wentworth and others, 1997). The latest version of the Marin General Plan references the Liquefaction Susceptibility Map in USGS Open-File Report 00-444 (Knudsen and others, 2000), which we reviewed together with the accompanying Quaternary Deposits Map. We also reviewed the more recent liquefaction susceptibility and quaternary deposit maps by Witter and others (2006).

To evaluate flood hazards, we reviewed the Tsunami Inundation Map for Emergency Planning (CGS, 2009) and online flood maps prepared by the Federal Emergency Management Agency (FEMA, 2016).

The earliest historic map that we reviewed showing the site area was an 1873 map of Marin County (Austin, 1873). We also obtained historic aerial photographs of the TLHS campus area from Pacific Aerial Surveys (a Quantum Spatial Company) in Novato, California. In all, Pacific Aerial Surveys provided 10 vintages of geo-referenced aerial photographs taken between 1950 and 2016. The complete set of georeferenced aerial photographs with identifying information is attached in Appendix A. The 1950 aerial photograph is also reproduced on Figure 3. A 1959 aerial photograph of the school site during construction (Bradley, 1959) is presented on the cover of this report.

2.03 Cone Penetration Tests (CPTs)

On February 22, 2017, we advanced four cone penetration tests (CPT-1 through CPT-4) at the approximate locations shown on Figures 2 and 3. Gregg Drilling & Testing, Inc. (GREGG) of Martinez California performed the CPTs using truck-mounted equipment. Information on the depths of the CPTs follows (elevations shown derived from the available County-provided LiDAR dataset).
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Summary of CPTs

Location	Surface Elevation (feet)	CPT Depth (feet)	Bottom of CPT Elevation (feet)
CPT-1	80.7	26.6	54.1
CPT-2	76.8	21.3	55.5
CPT-3	79.1	22.6	56.5
CPT-4	80.6	18.2	62.4

All of the CPTs were advanced to practical refusal under the weight of a 30-ton truck.

GREGG's plots of measured cone tip resistance (q_t), sleeve friction (f_s) and pore water pressure (u) are presented on CPT logs attached in Appendix B. Also presented are geotechnical material descriptions interpreted based on the normalized soil behavior type (SBT_N) as prescribed by Robertson, 1990. The attached CPT logs present data and interpretations pertaining to subsurface conditions at the indicated locations at the time that the CPTs were performed; the passage of time may result in changes in the subsurface conditions. The CPT locations indicated on Figures 2 and 3 were determined by measuring from existing improvements and should be considered approximate. At the conclusion of the CPT investigation, the CPT holes were backfilled with grout.

2.04 Borings

On February 22, 2017, we drilled five borings (Borings B-1 through B-5) at the approximate locations shown on Figures 2 and 3. All of the borings were drilled by Gregg Drilling and Testing, Inc. (GREGG), of Martinez, California using a truck-mounted drill rig equipped with 8-inch hollow-stem augers. Information on the depths of the borings follows (elevations shown derived from the available County-provided LiDAR dataset).

Summary of Borings

Location	Surface Elevation (feet)	Boring Depth (feet)	Bottom of Boring Elevation (feet)
B-1	81.1	16.3	64.8
B-2	81.2	13.3	67.9
B-3	81.0	21.0	60.0
B-4	81.1	21.0	60.1
B-5	91.4	20.4	71.0

Soil samples were obtained using a 2-inch outside diameter Standard Penetration Test (SPT) sampler without liners and a 3-inch outside diameter Modified California (MC) sampler with liners. The samplers were driven using a standard 140-pound automatic hammer with an approximate 30-inch fall. The hammer blows required to drive the sampler the final 12 inches of each 18-inch drive are presented on the boring logs. Sampler blow counts obtained using the MC sampler were adjusted to approximate SPT N-values using a factor of 0.63 to account for differences in sampler end area. Where a full 12-inch drive could not be achieved, the number of blows and corresponding amount of sampler penetration are indicated on the logs.

During drilling, an A3GEO engineer visually/manually classified the soil in general accordance with ASTM D2488 classifications, which are based on the Unified Soil Classification System (USCS). Field classifications were subsequently checked and revised, where appropriate, based on laboratory test data. The logs of the borings are attached in Appendix C preceded by: 1) a Key to Exploratory Boring Logs that describes the USCS and the symbols used on the logs; and 2) a Key to Rock Descriptions. Groundwater depth measurements made during and after drilling are shown on the logs presented in Appendix C. Following our groundwater depth measurements, the boreholes were backfilled with grout.

The attached boring logs represent our interpretation of the subsurface materials at the boring locations at the time of drilling and the passage of time may result in changes in the subsurface conditions. The boring locations indicated on Figures 2 and 3 were determined by measuring from existing improvements and should be considered approximate.

2.05 Geotechnical Laboratory Tests

Samples from the borings were examined in the laboratory to check field classifications and assign laboratory tests. Our geotechnical laboratory testing program was directed toward a quantitative and qualitative evaluation of the physical properties of the soils that underlie the site. The following geotechnical laboratory tests were performed:

- Moisture content per ASTM Test Designation D-2216;
- Dry density per ASTM Test Designation D-2937;
- Atterberg limits per ASTM Test Designation D-4318; and
- Particle size distribution per ASTM Test Designation D6913.

The results of the tests are presented on the boring logs in Appendix C at the corresponding sample depths; laboratory data sheets are attached in Appendix D.

2.06 Previous Geotechnical/Geologic Report

We reviewed following geotechnical/geologic report provided to us by Miller Pacific Engineering Group (MPEG):

> ***MPEG, 2003 -*** Miller Pacific Engineering Group, 2003, "Geotechnical Investigation and Geologic Hazards Evaluation, Terra Linda High School, San Rafael, California," consulting report dated October 31, 2003, MPEG Project 779.12.

The referenced report includes the logs of borings and CPTs performed at the approximate locations shown on Figures 2 and 3. The logs of MPEG's borings and CPTs are attached in Appendix E.

2.07 Existing School Plans

We reviewed the following set of plans obtained from the District's archives:

> ***GM&P, 1958*** - Grommé Mulvin & Priestley (GM&P), 1958, *"New High School, San Rafael High School District for the Terra Linda Area, Marin County, California,"* 48-sheet plan set dated December 16, 1958.

Selected sheets from the referenced plan set are attached in Appendix F.

3.00 GEOLOGIC, SEISMIC AND HISTORICAL INFORMATION

3.01 Regional Geology

The geology of the San Francisco Bay Region includes three “basement” rock complexes; the Great Valley complex, the Franciscan Complex and the Salinian complex all of which are Mesozoic in age (225 to 65 million years old). Within the region, the Mesozoic basement rocks are locally overlain by a diverse sequence of Cenozoic Era (younger than 65 million years) sedimentary and volcanic rocks. Since their deposition, the Mesozoic and Cenozoic rocks have been extensively deformed by repeated episodes of folding and faulting. Significantly, the Bay Area experienced several episodes of uplift and faulting during late Tertiary Period (about 25 million to 2 million years ago) that produced the region’s characteristic northwest-trending mountain ranges and valleys.

World-wide climate fluctuations during the Pleistocene (about 1.8 million to 11 thousand years ago) resulted in several distinct glacial periods. A lowering of sea level accompanied each glacial advance as water became stored in vast ice sheets. Melting of the ice during warm intervals caused corresponding rises in sea level. High sea levels favored rapid and widespread deposition in the bay and surrounding floodplains. Low sea levels during glacial advances steepened the gradients of streams and rivers draining to the sea thereby encouraging erosional down cutting. The most recent glacial interval ended about 15,000 years ago. Evidence suggests that during the maximum extent of this latest glaciation, sea level was 300 to 400 feet below its present elevation and the valley now occupied by San Francisco Bay drained to the Pacific Ocean more than 30 miles west of the Golden Gate. Near the beginning of the Holocene age (about 11 thousand years ago) the rising sea re-entered the Golden Gate, and sediments accumulated rapidly beneath the rising San Francisco Bay and on the surrounding floodplains. The Holocene-age surface deposits are generally less dense and weaker than Pleistocene-age soils that predate the last sea level rise.

3.02 Regional Active Faults

Within the San Francisco Bay Region, the relative motion of the Pacific and North American crustal plates is presently accommodated by a series of northwest-trending active faults that exist over a width of more than 50 miles. Approximate distances and directions from the site to Bay Area active faults follow:

Approximate Distances and Directions to Bay Area Active Faults (Jennings and Bryant, 2010)

Fault System	Approximate Distance from Site	Approximate Direction from Site
San Andreas	8.5 miles	West-southwest
San Gregorio	9.0 miles	West-southwest
Hayward	10.0 miles	East-northeast
Rodgers Creek	12.5 miles	East-northeast
West Napa[1]	21.0 miles	East-northeast
Concord-Green Valley	26.5 miles	East-northeast
Calaveras	31.0 miles	Southeast
Greenville – Clayton - Marsh Creek	33.0 miles	East-southeast

[1] In 2014, a Magnitude 6.0 earthquake occurred on the West Napa fault and as a consequence the southern extent of this feature is presently being reevaluated.

Faults that are defined as active typically exhibit: 1) evidence of Holocene-age (younger than 11,000 years) displacement, 2) measurable aseismic fault creep, 3) close proximity to linear concentrations or trends of earthquake epicenters, and/or 4) prominent tectonic-related geomorphology. The major faults listed in the preceding table are near-vertical and generally exhibit right-lateral strike-slip movement (which means that the movement is predominantly horizontal and when viewed from one side of the fault, the opposite side of the fault is observed as being displaced to the right).

3.03 Regional Seismicity

Since 1836, six earthquakes of magnitude 6.5 or greater have occurred in the Bay Area (Bakun 1999); the dates, magnitudes (M) and epicentral locations of these six large earthquakes are summarized in the table that follows.

Magnitude 6.5 or Greater Earthquakes; 1836-1998
(Bakun 1999; Tuttle and Sykes, 1992)

Date	Magnitude	Epicenter Location
June 10, 1836	6.5	East of Monterey Bay
June 1838	6.8 – 7.2	Peninsula section of the San Andreas fault
October 8, 1865	6.5	Southwest of San Jose
October 21, 1868	6.8	Southern Hayward fault (Hayward Earthquake)
April 18, 1906	7.8	San Andreas fault (San Francisco Earthquake)
October 18, 1989	6.9	Santa Cruz Mountains (Loma Prieta Earthquake)

The Working Group on California Earthquake Probabilities (WGCEP) has developed authoritative estimates of the magnitude, location, and frequency of future earthquakes in California, which are published in Uniform California Earthquake Forecast (UCERF) reports. The most recent forecast (UCERF3) indicates the following likelihoods for one or more earthquake events of the specified magnitude occurring within the San Francisco region in the next 30 years (starting in 2014).

San Francisco Region UCERF3 Forecast
(WGCEP, 2013)

Earthquake Magnitude (greater than or equal to)	30-year Likelihood of one or more earthquake events
≥ 5.0	100%
≥ 6.0	98%
≥ 6.7	72%
≥ 7.0	51%
≥ 7.5	20%
≥ 8.0	4%

The WGCEP has also made estimates of the likelihood of earthquakes with magnitude greater than or equal to 6.7 occurring on specific faults; these probabilities are summarized in the table that follows..

San Francisco Region UCERF3 Forecast
(Aagaard et al., 2016)

Earthquake Fault	30-year Likelihood of one or more earthquake events with M≥6.7
Hayward - Rodgers Creek	33%
Calaveras	26%
San Andreas	22%
Hunting Creek, Berryessa, Green Valley, Concord, Greenville	16%
Maacama	8%
San Gregorio	6%

Compared to the previous forecast (UCERF 2; WGCEP, 2008) the likelihoods of moderate-sized earthquakes (magnitude 6.5 to 7.5) are generally lower whereas those of larger events are higher. This change reflects a better understanding of the regional fault system and the potential for multi-fault ruptures on many faults.

3.05 Local Geology

As illustrated on the cover of this report, the TLHS campus is situated on a gently-sloping plain bounded by hills. The hills southwest of the site are part of a continuous range extending northwest from downtown San Rafael with localized peaks at elevations above +600 feet (USGS, 2015). To the northeast of the site are smaller isolated hills the closest of which to the campus rises to above elevation +200 feet. As shown on the Site Plan (Figure 2), the nearly-level plain upon which TLHS is situated is about 2,000 feet wide.

The photograph on the cover of this report and the Predevelopment Aerial Photograph on Figure 3 show the plain upon which the school is presently located was once incised by streams emanating from the hills to the southwest. The hill directly northeast of the school (upper right-hand quadrant of the photograph on Figure 3) blocks the flow of streams causing the former natural stream courses to turn northwest and merge to create a northwest-trending channel in the vicinity of CPT-3, CPT-4 and Boring B-4 (Figure 3).

A recent USGS geologic map[2] (Blake, Graymer and Jones, 2000) showing the site area is presented on Figure 4. Blake, Graymer and Jones (2000) map the hills that surround the TLHS campus predominantly as Franciscan Complex Mélange described as follows:

> ***Mélange (map symbol fsr) -*** *A tectonic mixture of variably sheared shale and sandstone containing (1) hard tectonic inclusions largely of greenstone, chert, graywacke, and their metamorphosed equivalents, plus exotic high-grade metamorphic rocks and serpentinite and (2) variably resistant masses of graywacke, greenstone, and serpentinite up to several miles in longest dimension, and including minor discrete masses of limestone too small to be shown. Blocks and resistant masses have survived the extensive shearing evident in the mélange's matrix, and range in abundance from less than 1 to 50 percent or more of the rock mass. The degree of shearing in the unit ranges from gouge to unsheared rock, with resistant masses relatively unsheared and matrix sheared. Severely sheared shale is abundant in areas where blocks are abundant. Fresh, relatively unsheared rock is hard, the larger resistant masses are pervasively fractured, and blocks are commonly tough and relatively unfractured. Sandstone is graywacke, grayish green where fresh, weathering to brown, commonly medium to coarse grained, containing abundant angular lithic grains and no detrital potassium feldspar, except*

[2] Geologic maps generally show materials interpreted to be present at or near the ground surface.

> *rarely as much as 5 percent. Graywacke is locally veined with quartz and carbonate, and usually contains microscopic secondary pumpellyite. Topography of coherent masses resembles that of unit Kfs, whereas highly sheared matrix typically yields subdued, gently-rounded topography.*

A more detailed geologic map (Rice, Smith and Strand, 1976) shows the TLHS campus underlain by Quaternary (less than about 2.6 million year old) alluvium and colluvium; alluvium refers to deposits that have been deposited by streams whereas colluvium refers to soils that have moved downslope by gravity. Blake, Graymer and Jones (2000; Figure 4) map the TLHS campus predominantly as Quaternary alluvium (Figure 4), which they describe as follows:

> ***Alluvium, Quaternary (map symbol Qal)****—Sand, gravel, silt, and clay; loose to soft and friable*

As shown on Figure 4, Blake, Graymer and Jones (2000) also map a northwest-trending inferred fault below the front (northeast) part of the campus passing near Nova Albion Way. This inferred fault is the projection of a fault mapped in the hills farther to the northwest and is not considered active.

3.06 Liquefaction Mapping

Liquefaction is a phenomenon by which certain types of soils that are below groundwater can lose strength (liquefy), compress (settle) and gain mobility (flow) in response to earthquake groundshaking. Liquefaction is considered a geologic hazard and the California Geological Survey (CGS) has issued official seismic hazard maps showing “zones of require investigation” for liquefaction for many parts of California; however no such maps have yet been issued for Marin County.

The U. S. Geological Survey (USGS) has published maps of liquefaction susceptibility for the central San Francisco Bay Region (Knudsen and others, 2000; Witter and others, 2006). As shown on Figures 5a and 5b, both USGS maps show all of the TLHS campus within an area of “Moderate” liquefaction susceptibility. The summary description for this liquefaction susceptibility category (from Witter and others, 2006) follows.

> *Expect about 20 to 30 percent of future liquefaction effects to occur within geologic units assigned MODERATE susceptibility (with about 1 occurrence for every 50 square kilometers). Geologic map units within this category include latest Pleistocene to Holocene deposits from a variety of environments. Gravel quarries and percolation ponds (historical) are also assigned to this category. Together, units assigned MODERATE susceptibility cover 2,314 square kilometers of the central San Francisco Bay region. About 25 percent of historical liquefaction occurrences fall within map units assigned MODERATE susceptibility (about 0.02 occurrences per square kilometer).*

The referenced liquefaction susceptibility mapping by the USGS is based on accompanying regional-level maps of Quaternary deposits coupled with groundwater depth estimates, earthquake ground motion estimates, and documented accounts historical instances of liquefaction occurrence. As such, the USGS susceptibility maps (Figures 5a and 5b) are not “site-specific” as no onsite data was used in their development.

3.07 Landslide Mapping

Landsliding is considered a geologic hazard and the California Geological Survey (CGS) has issued official seismic hazard maps showing “zones of require investigation” for earthquake-induced landsliding for many parts of California; however no such maps have yet been issued for Marin County. The landslide map on Figure 6 (Wentworth and others, 1997) shows areas of “mostly landslides” at higher elevations in the hills southeast of the TLHS campus and areas of “few landslides” extending into adjacent residential neighborhoods and onto the far western portion of the TLHS campus. Generalized explanations of the mapping shown on Figure 6 follow.

> ***Mostly Landslides -*** *consists of mapped landslides, intervening areas typically narrower than 1500 feet, and narrow borders around landslides.*
>
> ***Few Landslides -*** *contains few, if any, large mapped landslides, but locally contains scattered small landslides and questionably identified larger landslides.*
>
> ***Surficial Deposits -*** *Slides and earth flows do not occur on nearly flat ground -- they require slopes that are steep and long enough to permit failure. We can thus exclude gently sloping ground from principal consideration. This boundary typically occurs at a slope of about 15 percent.*

Comparisons between Figure 6 (Landslide Map) and Figure 4 (Geology Map) suggest that the mapped area of “few landslides” within the TLHS campus correlates to the geologic mapping of Franciscan Mélange in the same area. A more detailed geologic map showing landslides (Rice, Smith and Strand, 1976) generally shows the TLHS campus as free of landslide deposits.

3.08 Site Development History

Sheet A1A of the 1958 plans for the school (GM&P; Appendix F) includes a Partial Site Plan and Profile at A-A’ that provide data on the pre-development topography and grading of the TLHS campus. These data generally show that the natural ground surface at the site sloped gently down from southwest to northeast and that the nearly-level pad upon which the new school buildings are located was constructed prior to December 1958 by cutting and filling. The drawing titled Profile at A-A’ on Sheet A1A (Appendix F) shows maximum cut and fill heights of about 9 feet and 7 feet, respectively. The photograph on the cover of this report shows that the grading to develop the TLHS campus was essentially complete in August of 1959. Appendix A includes two vertical aerial photographs taken prior to site development, the most recent of which is dated March 1, 1958.

Foundation drawings from the 1958 plans for the school (GM&P; Appendix F), generally show buildings with concrete slab-on-grade floors and spread footing foundations typically extending toe depths of 3 to 3-½ feet below the top-of-slab elevation. The Foundation Note on “Sheet S-2 of Building E” indicates:

> *Soil pressure does not exceed 2,000 lbs. per sq. ft. All footings to go to hard sand and clay or to shale rock at elevations shown. In necessary to reach firm material, lower footings below elevations shown as directed by the architect. Soil Data from R. S. Harding (Oct. 24, 1958).*

It is our understanding that the referenced Soil Data from R. S. Harding have not been located.

Appendix A includes vertical aerial photographs taken in 1970, 1975, 1982, 1986, 1992, 1996, 2000 and 2015. In general, it appears that the campus was fully-developed by 1970 with the exception of a single building that appears in the most recent (2015) photograph. This building was the subject of the 2003 geotechnical investigation and geologic hazard evaluation by report by MPEG; the Site Plan in MPEG’s report (included in Appendix E) identifies this building as a “New Performing Arts Facility.” The MPEG (2003) report includes recommendations for a shallow foundation alternative, a drilled pier foundation alternatives and concrete slab-on-grade floors.

4.00 SITE CONDITIONS

4.01 Surface Conditions

Surface conditions in the area of the TLHS campus are illustrated on the Site Plan (Figure 2). Topographically, the campus includes three terraces that step up from Nova Albion Way on the northeast towards Devon Drive on the southwest.

> ***Lower Terrace -*** The lower terrace is near Elevation +80 feet and includes the TLHS buildings, adjacent parking areas and patios, and a football field surrounded by an athletic track.
>
> ***Middle Terrace -*** The middle terrace is near Elevation + 90 feet and includes paved tennis and basketball courts.
>
> ***Upper Terrace -*** The upper terrace extends to just above Elevation +100 feet and includes a baseball diamond and play fields.

All three terraces are generally bounded on their upslope sides by steepened slopes; above the uppermost slope are the rear yards of residential properties that front on Devon Drive.

4.02 Preliminary Geologic Characterization

The Preliminary Site Geologic Map presented on Figure 7 shows the surficial geologic units we interpret to be present within the TLHS campus. As discussed in Section 3.08, grading to construct the campus involved cutting and filling. We used the Partial Site Plan on Sheet A1A of the 1958 plans for the school (GM&P; Appendix F) and the 1950 aerial photograph on Figure 3 to interpret the approximate lateral extent of artificial fill (map symbol Qaf). Within the campus, the +80-foot predevelopment elevation contour was used to approximate the southwestern lateral extent of the fill. Outside of the campus, we loosely interpreted the extent of artificial fill based on our review of topographic data and historical aerial photography. The hills that surround the campus are mapped as Franciscan Mélange, generally consistent with regional geologic mapping (e.g. Figure 4). Surficial deposits outside of the areas mapped as artificial fill or mélange are mapped as Quaternary alluvium/colluvium (map symbol Qa/Qf).

The Preliminary Site Geologic Map presented on Figure 7 includes the location of the one interpretive cross section developed for this study (Preliminary Geologic Cross Section A-A’, Figure 8). The preliminary cross section is taken at the same location as the A-A’ profile shown on Sheet A1A of the 1958 plans for the school (GM&P; Appendix F); both cross sections utilize an exaggerated vertical scale equal to twice the horizontal scale. Figure 8 includes graphic representations of the boring (MPEG, B-2) and CPTs (A3GEO CPT-2 and CPT-3) closest to the cross section. The surface of bedrock shown on the cross section is linearly interpreted using the data obtained from these three exploration points. Figure 8 also shows the interpreted thickness of artificial fill, the base of which was approximated using the predevelopment ground surface shown on the Profile at A-A’ from Sheet A1A of the 1958 plans for the school (GM&P; Appendix F);

4.03 Soil and Rock Conditions

The attached appendices contain subsurface data from five CPTs, six borings and associated geotechnical laboratory tests. As noted in Section 2.0 (Methods of Investigation), the CPT methodology produces an interpreted log of subsurface conditions with depth whereas the methodology used to in drill and sample the borings allows for the direct visual/manual examination of subsurface materials and produces samples that can be tested in the laboratory. Notably, the CPT methodology cannot directly distinguish artificial fill from generally similar natural soil deposits or distinguish dense/hard natural soil from bedrock. For this reason, the discussions in this section are based primarily on data from the borings, which we correlate (where appropriate) with relevant data from the CPTs.

All six borings extended through natural alluvium and/or colluvium and into bedrock, which was encountered at the depths/elevations indicated in the following table.

Bedrock Depths, Elevations and Descriptions

Boring (source)	Surface Elevation[3] (feet)	Bedrock Depth (feet)	Top of Bedrock Elevation (feet)	Bedrock Description
B-1 (A3GEO, 2017)	81.1	10.0	71.1	Clayey Sandstone
B-2 (A3GEO, 2017)	81.2	12.5	68.7	Shale
B-3 (A3GEO, 2017)	81.0	7.0	74.0	Claystone/Shale
B-4 (A3GEO, 2017)	81.1	18.5	62.6	Sandstone
B-5 (A3GEO, 2017)	91.4	18.5	72.9	Shale
B-1 (MPEG, 2003)	80.2	18.0	62.2	Sandstone
B-2 (MPEG, 2003)	81.2	12.0	69.2	Sandstone

Generalized descriptions of the materials encountered in the borings and CPTs follow.

Artificial Fill - Only one of the borings (B-4; Appendix C) was drilled in the area where artificial fill is shown on the geologic map and cross section (Figures 7 and 8, respectively). The fill at this location is interpreted to be about 10 feet deep and consists of lean clay (symbol CL) containing varying amounts of fine to medium sand. As shown on the log of Boring B-4 in Appendix C, the following data was obtained within the artificial fill layer: 1) adjusted sampler blow counts of 7 and 9 blows per foot; 2) pocket penetrometer uncompressive strength readings ranging from 0.75 to 1.25 tons per square foot (tsf); and 3) a plasticity index (PI) value of16. Four CPTs (CPT-1 through CPT-4; Appendix B) were advanced in the area where artificial fill is shown on Figures 7 and 8. The logs of CPT-2, CPT-3 and CPT-4 show increased tip resistance (qt) values below depths of about 10.5 feet , 16 feet and 16 feet, respectively; based on these data, we interpret the soils above these depths as "probable fill." The log of CPT-1 shows greater variation; based on the information currently available, we interpret that fill may only be 5 or 6 feet deep at this location.

Natural Alluvium/Colluvium – All of the borings and CPTs encountered natural alluvial/colluvial overlying bedrock. The logs of borings generally show natural soils that are mixtures of lean clay and fine to medium grained sand; these soils generally classify as either lean clay with sand (symbol CL) or clayey sand (symbol SC). As shown on the logs of borings in Appendix C, the following data was obtained within natural soils at the locations of B-1 through B-4: 1) adjusted sampler blow counts between 18 and 34 blows per foot; 2) pocket penetrometer uncompressive strength readings ranging from 3.0 to greater than 4.5 tsf; and 3) plasticity index values ranging from 16 to 21. B-5 is a special case as it was drilled on the Middle Terrace at a higher elevation. In Boring B-5, the soils encountered below a depth of about 10 feet are essentially similar to those encountered in the other borings (adjusted blow counts from 14 to 24 blows per foot and pocket penetrometer unconfined compressive strengths greater than 4.5 tsf). However, above the 10-foot depth Boring B-5 encountered soils that were generally less compact (adjusted blow counts of 6 and 10 blows per foot and pocket penetrometer unconfined compressive strengths between 1.5 and 2.0 tsf) and included a surficial layer of sandy silt, not present at the locations of B-1 through B-4. The logs of CPT-1 through CPT-4 (Appendix B) generally show adjusted blow count (N_{60}) values between about 10 and 30 blows per foot in the natural soils below the interpreted artificial fill layer.

[3] Elevations shown have been derived from the available LiDAR dataset and are considered approximate

Franciscan Complex Bedrock – All of the borings extended into bedrock materials comprised of sandstone or shale, which is consistent with Franciscan Complex Mélange described in Section 3.05. The bedrock materials encountered in Borings B-1 through B-5 (Appendix C) are typically described as weathered; adjusted sampler blow counts in bedrock generally increase with depth, ranging from 34 blows per foot (Boring B-3) to 50 blows for 5 inches (Boring B-5).

4.05 Groundwater Conditions

The logs of two borings drilled at the site in August of 2003 (MPEG, 2003: Appendix E) note “no groundwater was observed during drilling”; these borings (MPEG B-1 and B-2) were terminated in bedrock at depths of 23.5 and 16.0 feet (respectively).

Groundwater was observed in only one of the five borings drilled at the site in February 2017 (Boring B-4; A3GEO, 2017). The log of Boring B-4 (Appendix C) shows: 1) the boring was drilled from a ground surface elevation of 81.1 feet; 2) the boring was terminated in sandstone bedrock at a depth of approximately 21 feet; 3) groundwater was measured in the boring at a depth of 20 feet (Elevation +61 feet) shortly after the hole was completed; and 4) water was again measured in the boring at a depth of 10 feet (Elevation + 71.1 feet) before the hole was backfilled with grout. The groundwater conditions observed in Boring B-4 suggest that the fractured rock below the site may in some instances act as an aquifer, confined by an overlying aquitard comprised of natural clayey alluvium/colluvium.

Borings B1, B-2, B-3 and B-5 (A3GEO, 2017) were observed to be free of groundwater shortly before they were backfilled with grout. We note that groundwater measurements made in open boreholes are not necessarily representative of stabilized groundwater conditions at the time that the measurements were made, which is particularly true for holes drilled in low-permeability clayey soils. It should be anticipated that groundwater levels below the site may vary in response to rainfall or other factors. Groundwater may also be present below the site at times within seepage zones or due to a locally perched condition.

5.00 PRELIMINARY GEOLOGIC HAZARDS ASSESSMENT

5.01 Earthquake Ground Shaking

The San Francisco Bay Area is seismically active and it is likely that the TLHS campus will experience earthquake ground shaking within the foreseeable life of a future project. For this reason, structures at the site should be designed to resist strong ground shaking in accordance with the requirements of the California Building Code and local design practice. The seismic design provisions of the 2010 CBC include a methodology by which sites are classified as A through F in order to quantify site-specific ground shaking effects. Based on the available data, we judge that a Class D designation (Stiff Soil Profile) is appropriate for the campus, as a whole. Please refer to Section 7.01, Building Code Seismic Design Parameters, for applicable California Building Code seismic design parameters.

5.02 Surface Fault Rupture

Historically, earthquake fault rupture most often occurs along pre-existing active faults. The site is not located within or proximate to an Alquist-Priolo Earthquake Fault Zone and the closest known active fault (the San Andreas fault) is approximately 8.5 miles to the southwest (Section 3.02). Faults that have been mapped closer to the site, including all of the faults shown on Figure 4, are not considered active. In our opinion, the overall potential for surface fault rupture to affect the TLHS campus is negligible.

5.03 Liquefaction

The TLHS campus is mapped by the USGS (Figures 5a and 5b) within an area of “Moderate” liquefaction susceptibility. Soils that are most likely to experience liquefaction are loose (adjusted blow counts less than 10), relatively clean sands and gravels that are below groundwater. Similar soils that are medium dense (adjusted blow counts less than 30) can also experience liquefaction in some cases. Recent research has shown that cohesive soils can experience earthquake-induced strength loss that appears generally similar to liquefaction, provided that certain criteria are met. At this time, there appears to be a general consensus that cohesive soils with a plasticity index (PI) of 12 or greater can be considered highly resistant to liquefaction.

Liquefaction is only a concern where susceptible soils are submerged below groundwater at the time when an earthquake large enough to trigger liquefaction occurs. Except for one boring (Boring B-4; A3GEO, 2017), none of the borings drilled recently at the campus encountered groundwater. This observation generally suggests that the probability that soils at the site would be saturated at the time of an earthquake is likely to be low. In addition, recent borings drilled at the TLHS campus (MPEG, 2003; A3GEO, 2017) did not encounter clean sands or gravels and four Atterberg Limits determinations performed on samples of cohesive soils produced PI values greater than 16. Clayey sands with low plasticity fines are logged between depths of 4 and 10 feet in Boring B-1 and from 10 to 12.5 feet in Boring B-2; however these soils are generally dense and dense soils are not considered liquefiable. Based on the available data, it appears that the conditions needed for liquefaction to occur are generally absent and that the potential for significant liquefaction to affect the TLHS campus is very low.

5.04 Landsliding

The TLHS campus is located in an area of gently-sloping ground free of mapped landslide deposits (Rice, Smith and Strand, 1976), the soils that underlie the campus are not considered susceptible to seismic strength loss and bedrock is present below the campus at relatively shallow depths. Based on the available information, we judge that the overall potential for deep-seated landsliding within the TLHS campus to be low.

Grading of the TLHS campus has produced low (less than about 10 feet high) cut slopes that may be susceptible to shallow sliding, sloughing and/or surface erosion. Based on our review of historic aerial photography (Appendix A), it appears to us that the cut slopes within and surrounding the TLHS campus have performed relatively well since they were created almost 60 years ago (in 1958). Any future failures in these cut slopes would likely be very limited in lateral extent; the possibility of shallow slope failures having an impact on adjacent structures would best be addressed in a future phase on a location-specific basis for any planned structures proximate to steep slopes.

We also considered the possibility that landslides occurring the adjacent hills might in extreme circumstances extend onto the TLHS campus. In general, the residential neighborhoods that surround the site provide a buffer between the base of the hillslopes and the campus. To our knowledge the hillslopes that surround the site do not include deep deposits of materials that would likely experience dramatic reductions in strength following landslide initiation. Accordingly, we would expect any deep-seated landsliding triggered by wet weather or an earthquake to have limited runout potential and judge that the existing buffer zone between the campus and the hills is likely adequate. There is also the possibility that a fast-moving debris flow landslide that emanates from the hills could extend onto the TLHS campus. This potential hazard, if it exists, would appear to be greatest within the upper and middle terraces. Based on the information currently available, we judge the overall potential for landslides from the nearby hills to extend onto the lower terrace where TLHS buildings are located to be low.

5.05 Inundation

The site is near Elevation +80 feet and is more than a mile inland from the closest tsunami zone shown on the CGS Tsunami Inundation Map (CGS, 2009). The site's location in eastern Marin County would not be directly exposed to a tsunami from the Pacific Ocean, which would necessarily enter San Francisco Bay through the Golden Gate. The valley in which Terra Linda is located drains to the northeast towards San Pablo Bay and not towards the Golden Gate. . Accordingly, we judge that inundation by tsunami or seiche is not a concern.

To our knowledge, there are no significant reservoirs located upslope that could potentially pose a hazard to the TLHS campus. The Federal Emergency Management Agency (FEMA, 2016) maps the site within an "Area of Minimal Flood Hazard (Zone X)". As shown on Figure 3, several historic drainages previously existed in the vicinity prior to the development of the TLHS campus. Presumably, water from nearby upslope areas currently flows below the TLHS campus and adjacent residential neighborhoods in culverts, the condition of which are unknown. Based on the information available at this time, we judge that the overall potential for the TLHS campus to be flooded by water is low provided that existing drainage facilities in the area continue to function as intended.

6.00 PRELIMINARY GEOTECHNICAL ASSESSMENT

6.01 Feasibility

Based on the results of our preliminary investigation, we conclude that that the types of improvements outlined in the District's 2015 Facilities Master Plan (HY Architects, 2015) are feasible from a geotechnical standpoint. Our preliminary assessment of geotechnical considerations for future projects at the TLHS campus are discussed in the sections that follow.

6.02 Expansive Soils

The onsite soils include materials that are expansive and have the potential to damage overlying improvements unless mitigated. Alternative foundation types that are commonly used in the Bay Area to mitigate the potentially damaging effects of expansive soils on structures include: (1) shallow foundations (footings or mats) supported on a layer of compacted non-expansive fill; (2) deepened spread footings supported on natural soils below the zone of significant shrink/swell behavior; and (3) deep foundations that gain support at significant depths below the zone of shrink/swell behavior. Accordingly, the foundation support options discussed in Section 6.04 are considered effective in mitigating shrink-sell effects associated with expansive soils. New pavements and slabs-on-grade would best be underlain by a layer of engineered non-expansive fill to reduce the amount of movement and distress caused by expansive soils.

6.03 Undocumented Fill

As shown on the Preliminary Site Geologic Map (Figure 7) and Preliminary Geologic Cross Section (Figure 7), a portion of the campus is underlain by artificial fill. The fill shown on Figures 7 and 8 was placed during the original development of the school campus and, to our knowledge, there are no records available documenting that engineering controls were in place at the time that the site was cleared, the natural ground surface was prepared, and the fill was placed. The same is true for any localized fill that may have been placed in association with utility, sewer and and storm drain installations, building foundations, underground tanks or other below-grade features. Undocumented fill is considered generally unsuitable for the support of new buildings. Accordingly, the foundation support options discussed in Section 6.04 take into account the presence of undocumented fill in certain areas of the site. We judge that it is probably not feasible to remove all undocumented fill from beneath future pavements and slabs-on-grade; in this preliminary report, we recommend that all subgrades below the non-expansive fill layer supporting these types of improvements be checked to verify that they are capable of providing adequate support.

6.04 Building Foundations

As noted in Sections 6.02 and 6.03, expansive soils and artificial fill are key considerations in the selection of an appropriate foundation type for the planned new buildings. This preliminary report provides recommendations for two different foundation systems, the selection of which depends primarily upon whether a significant amount of artificial fill is present at the proposed building location (expansive soils are interpreted to be present throughout the TLHS campus). For planning and cost estimating purposes, we suggest assuming the following:

> ***Locations Underlain by Natural Soils -*** Future buildings located in the area where natural alluvium/colluvium (Qa/Qc) is mapped on Figure 7 can likely be supported on spread footing foundations. Section 7.00 of this report includes preliminary recommendations for deepened spread footings supported below the zone of significant shrink/swell behavior.
>
> ***Locations Underlain by Artificial Fill -*** Future buildings located in the area where artificial fill (Qaf) is mapped on Figure 7 would best be supported on deep foundations. Section 7.00 of this

report includes preliminary recommendations for drilled pier foundations that gain their support in natural soil and/or bedrock below the artificial fill. Uplift pressures from expansive soils also need to be considered in the design of drilled piers and grade beams.

Preliminary recommendations for deepened spread footings and drilled piers are presented in Sections 7.02 and 7.03, respectively. We note that other foundation scenarios may be feasible and cost-effective; selection of a final foundation design solution would best be made in a subsequent phase in consultation with the project Structural Engineer.

6.05 Construction Considerations

We anticipate onsite soils can be excavated with conventional earth-moving equipment. It is possible that excavations could encounter obstructions that would require jackhammering, hoe-ramming or equipment capable of cutting steel to excavate. Excavations deeper than 4 feet that will be entered by workers should be shored or sloped for safety in accordance with the California Occupational Safety and Health Administration (Cal-OSHA) standards. The near-surface materials may contain debris, wood and organic-laden materials that would not be suitable for onsite re-use.

Drilled piers should be installed by a qualified drilling contractor. We judge that the holes can likely be drilled using heavy auger drilling equipment; however, zones of relatively hard rock could be encountered. The contractor should be prepared to utilize suitable hard rock drilling techniques, if necessary. As noted in Section 4.05 (Groundwater Conditions), the bedrock below the site may in some cases function as a confined aquifer. The contractor should note that if water accumulates in the holes, it should be removed by pumping or bailing prior to concrete placement unless tremie methods are used. Concrete placement should start as soon as possible after the drilling and cleanout is complete. In all cases, holes for drilled piers should be concreted on the day they are drilled.

The contractor should anticipate that site excavations may need to be dewatered and that there may be environmental and regulatory aspects to the appropriate collection, storage and disposal of onsite water. The design, permitting, installation, monitoring, and abandonment of site dewatering and discharge systems are the contractor's responsibility; this includes whatever systems may be needed to handle water displaced or pumped from pier holes. The onsite soils may include materials that are wet of optimum, from an earthwork compaction standpoint. The contractor should anticipate that soils obtained from site excavations will likely include clayey materials that may need to be processed (e.g. by air drying) prior to being placed as engineered fill.

Although it is possible for excavation and/or construction to proceed during or immediately following the wet winter months, a number of geotechnical problems may occur which may increase costs and cause project delays. We advise that wet-weather issues be considered during project scheduling, noting that the contractor's responsibilities include onsite safety and construction means and methods.

7.00 PRELIMINARY RECOMMENDATIONS

7.01 California Building Code Seismic Parameters

Structures at the site should be designed to resist strong ground shaking in accordance with the applicable building code(s) and local design practice. This section provides mapped seismic design parameters per the California Building Code (Risk Category I/II/III). Site Class D is considered generally appropriate for the TLHS campus, however there may be instances where Site Class C could be justified. We recommend that Site Class should be checked and verified on a location-specific basis once updated plans pertaining to future buildings are available.

Site Class
D = Stiff Soil Profile

Latitude and Longitude
Latitude: 38.0000°N
Longitude: 122.5543°W

Maximum Considered Earthquake Spectral Response Accelerations (for Site Class D)
(Mapped Acceleration × Site Coefficient)
S_{MS} = 1.500g (MCE spectral acceleration at short periods)
S_{M1} = 0.900g (MCE spectral acceleration at 1-second period)

Design Spectral Response Acceleration (for Site Class D)
(Maximum Considered Earthquake Spectral Acceleration × 2/3)
S_{DS} = 1.000g (design spectral acceleration at short periods)
S_{D1} = 0.600g (design spectral acceleration at 1-second period)

The corresponding USGS Design Maps Summary Report and Design Maps Detailed Report are attached in Appendix G.

7.02 Deepened Spread Footings

For preliminary design and costing purposes, we recommend assuming that new buildings in areas where natural alluvial/colluvial soil (Qa/Qc) is mapped on Figure 7 will be supported on deepened spread footing foundations. Deepened spread footings should be at least 16 inches wide and should be founded at least 30 inches below lowest adjacent firm finished grade. We recommend that continuous deepened spread footings enclose the entire building perimeter in order to mitigate the potential for moisture changes beneath the interior ground floor concrete slab-on-grade. Deepened spread footings can be preliminarily evaluated using the bearing pressures in the following table (DL=Dead Loads; LL=Live Loads; Total=DL+LL+ wind or seismic).

Preliminary Foundation Allowable Bearing Pressures

Load Case	Bearing Pressure (psf)	Minimum Factor of Safety
DL Allowable	3000	3.0
DL+LL Allowable	4500	2.0
Total Allowable	6000	1.5

Resistance to lateral loads can be provided by passive pressures acting on the vertical faces of below-grade structural elements and by friction along the footing bottoms. Passive resistance can be preliminarily evaluated using an equivalent fluid weight of 300 pounds per cubic foot (pcf). This value can be increased by one-third for dynamic loading. A friction coefficient of 0.30 can be used to evaluate frictional resistance along the bottoms of footings. The above passive and frictional resistance values include a factor of safety of at least 1.5 and can be fully mobilized with deformations of less than 1/2- and 1/4-inch, respectively.

7.03 Drilled Piers

For preliminary design and costing purposes, we recommend assuming that new buildings in areas where artificial fill (Qaf) is mapped on Figure 7 will be supported on drilled piers. This section provides recommendations for drilled piers that are founded in bedrock.

Foundation piers should be at least 18 inches in diameter and spaced no closer than three pier diameters, center-to-center. Drilled pier groups should be structurally tied together at their tops by grade beams; grade beams and pile caps should be underlain by at least 18 inches of non-expansive fill to mitigate potential expansive soil uplift effects. Alternatively, a 6-inch minimum vertical space (void) could be provided between the ground surface and overlying structural elements.

The axial capacity of drilled piers can be preliminary evaluated using an allowable skin friction value of 600 psf in natural alluvium/colluvium and 1,800 psf in bedrock. These allowable skin friction values can be increased by one third for total compressive loads, including wind and/or seismic, but should not be increased for uplift loads. We recommend that skin friction in artificial fill be ignored in evaluating drilled pier axial capacity. We further recommend that any contribution to axial capacity from end bearing in bedrock be ignored due to difficulties associated with obtaining and/or assuring a clean bearing surface at the bottom of the pier holes. Drilled piers should extend at least 5 feet into bedrock, regardless of load. The preceding recommendations and the cross section on Figure 8 can be used to preliminarily estimate drilled pier lengths.

Resistance to lateral loads can be provided by passive pressures acting on the vertical faces of grade beams and the upper portions of drilled piers. Passive resistance can be evaluated using an equivalent fluid pressure of 300 pounds per cubic foot (pcf), which can be applied over two (horizontal) pier diameters over the upper 5 feet of the pier. The preceding passive resistance values include a factor of safety of at least 1.5. We recommend that the upper foot of soil be ignored in calculating passive resistance unless the surface of the soil is confined by pavement or a concrete slab-on-grade.

7.04 Retaining Walls

The following lateral pressure distributions can be used for the preliminary evaluation of retaining walls for a level backfill condition.

Lateral Pressures – Level Backfill Condition

Load Condition	Lateral Pressure
Lateral Pressure, Restrained Walls	60 (psf per foot of depth)
Lateral Pressure, Free-to-Rotate Walls	45 (psf per foot of depth)
Surcharge (vehicles)	100 psf (uniform) – applied over the upper 10 feet of the wall height
Surcharge (general)	0.5 times anticipated surcharge load (uniform)
Surcharge (earthquake)	18 H (psf), where H is the retained height of soil in feet.

Walls that are not free to rotate at their tops (including building walls) should be evaluated using the lateral pressure distribution for restrained walls.

The preceding lateral pressure distributions are based on the assumption that retaining walls will be fully drained to prevent the build-up of hydrostatic pressure. Wall drainage may consist of either: (1) holes, slots or gaps in the wall that allow water to freely drain through the wall face; or (2) a wall backdrainage system that collects water from behind the wall and drains it, by gravity, to an appropriate discharge location. Backdrainage should consist of either: (1) prefabricated drainage material (Miradrain or an approved alternative) installed in accordance with the manufacturer's recommendations, or (2) a drain rock layer at least 12 inches thick. Prefabricated drainage material should drain to a perforated plastic pipe or an approved prefabricated drainage conduit. Backdrainage should drain into a perforated plastic pipe installed (with perforations down) along the base of the walls on a 2-inch-thick bed of drain rock. Plastic pipe should be sloped to drain by gravity to a sump, relief wells or other suitable discharge and a cleanout should be provided at the pipe's upslope end. Perforated and non-perforated plastic pipe used in the drainage system should consist of 4-inch diameter Schedule 40 PVC or an approved equivalent. Drain rock should conform to Caltrans specifications for Class 2 permeable material. Alternatively, locally available, clean, ½- to ¾-inch maximum size crushed rock or gravel could be used, provided it is encapsulated in a non-woven geotextile filter fabric, such as Mirafi 140N or an approved alternative. The upper 2 feet of retaining wall backfill (above backdrainage) should be comprised of low-permeability soil to limit surface water infiltration into the retaining wall backdrainage system.

7.05 Engineered Fill

Preliminary geotechnical requirements for fill materials are presented below.

> ***General Fill -*** General fill material should have an organic content of less than 3 percent by volume and should not contain environmental contaminants or rocks or lumps larger than 6 inches in greatest dimension. From a geotechnical standpoint, onsite materials can be reused as General Fill if they meet or can be processed (e.g. by sorting and/or crushing) to meet the above requirements. General fill can be used anywhere except where non-expansive fill is required.
>
> ***Non-Expansive Fill -*** Non-expansive fill should conform to the requirements for General Fill, have a Plasticity Index no greater than 12, and a Liquid Limit no greater than 40. A non-expansive fill layer should be required beneath concrete slabs, pavements, and in cases where uplift pressures are a concern (e.g. below grade beams that are in direct contact with the ground).
>
> ***Imported Fill –*** Imported fill should conform to the requirements for Non-Expansive Fill and should be evaluated by our firm and the project environmental consultant prior to its importation to the site.

From a geotechnical standpoint, lime or cement treatment may be an appropriate means to process soils for use as Non-Expansive Fill; if lime or cement treatment is to be considered, A3GEO would assist the design team in developing an appropriate project- and site-specific specification for its use. Preliminary geotechnical requirements for fill placement and compaction are presented below (per ASTM D-1557 Test Methods):

- General Fill that is predominantly cohesive (>15 percent passing #200 sieve) should be moisture conditioned, as necessary, to between 3 and 5 percent over optimum moisture content and compacted to at least 90 relative compaction.
- General Fill that is predominantly granular (<15 percent passing #200 sieve) should be moisture conditioned, as necessary, to between 2 and 4 percent over optimum moisture content and compacted to at least 95 relative compaction.
- Non-Expansive Fill should be moisture conditioned, as necessary, to near optimum moisture content and compacted to at least 95 percent relative compaction.

All proposed fill materials should be approved by A3GEO and the project environmental consultant prior to use.

7.06 Interior Slabs-on-Grade

This section provides preliminary recommendations for the support of interior slabs-on-grade for buildings supported on deepened spread footings that provide full enclosure of the building perimeter. We recommend that interior slabs that are cast on-grade be underlain by at least 18 inches of Non-Expansive Fill. The upper 6 inches of this layer should consist of a moisture retarder comprised of 6 inches of compacted aggregate base overlain by a heavy-duty impermeable membrane (Stego® wrap 15-mil or an approved equivalent) installed and taped in accordance with the manufacturer's recommendations.

The crushed rock layer should be directly underlain by a 12-inch-thick layer of Non-Expansive Fill. Slab reinforcing should be provided in accordance with the anticipated use and loading of the slab. We recommend that interior slabs-on-grade be at least 5 inches thick and be reinforced with steel bar reinforcement.

7.07 Exterior Flatwork

We recommend exterior slabs-on-grade be supported on a minimum of 12 inches of Non-Expansive Fill. Slab reinforcing should be provided in accordance with the anticipated use and loading of the slab. We recommend that exterior slabs-on-grade be at least 4 inches thick and reinforced with steel bar reinforcement. Exterior slabs should be structurally independent from buildings. Concrete slabs that may be subject to vehicle loadings should be designed in accordance with the recommendations for rigid Portland cement concrete pavements.

Flexible asphalt concrete (AC) pavements may be used for parking areas and driveways. We developed the following recommended pavement sections for various traffic indices using the Caltrans R-value design method for flexible pavements. The pavement sections presented are based on an assumed subgrade R-value of 30 for Non-Expansive Fill.

Flexible Pavement Thickness Design for Subgrade R-Value = 30

Traffic Index	Asphalt Concrete (inches)	Caltrans Class 2 Aggregate Base (inches)	Total Thickness (inches)
4	2	6	8
5	3	6	9
6	3	9	12
7	3	12	15

For pavements, we recommend that the aggregate base be underlain by at least 12 inches of Non-Expansive Fill and that this layer extend at least 3 feet beyond the outside pavement edge unless a deepened curb or other moisture cutoff (at least 24 inches deep) is provided. The project civil engineer should choose the appropriate traffic indices for the pavement areas of the site and then use the given section for that traffic index. The upper 6 inches of subgrade beneath planned pavements should be compacted to at least 95 percent relative compaction per ASTM D-1557. Aggregate base for use in pavements should conform to Caltrans Standard Specifications for Class 2 Aggregate Base. The aggregate base used in pavement sections should be compacted to at least 95 percent relative compaction as determined by ASTM D-1557.

Rigid Portland cement concrete (PCC) pavements may also be used in driveway/loading areas. This section provides recommendations for Caltrans jointed plain concrete pavement (JPCP), which is engineered with longitudinal and transverse joints to control where cracking occurs. JPCPs do not contain steel reinforcement, other than tie bars and dowel bars. The project civil engineer should design and detail the JPCP per Caltrans specifications. We developed the following pavement thickness design using the Caltrans R-value design method for rigid pavements and an assumed traffic index. The PCC design that follows is appropriate for subgrade soils with an R-value between 10 and 40.

Portland Cement Concrete Pavement Thickness Design

Traffic Index	Portland Cement Concrete (inches)	Caltrans Class 2 Aggregate Base (inches)	Total Thickness (inches)
< 9	9	12	21

On a preliminary basis, we recommend that PCC pavements be underlain by at least 12 inches of non-expansive fill designed in accordance with the recommendations to this section to reduce the potential for adverse expansive soil effects.

7.08 Future Geotechnical Services

This report and interpretations, conclusions and recommendations it contains are preliminary in nature. Further investigations may be needed to characterize site-specific subsurface conditions at building locations and to comply with CGS Note 48 requirements. We should consult with District representatives and future Design Team members, as appropriate, about the opinions expressed in this report as well as our recommendations for further study. This preliminary report should not be used for final design.

Geotechnical services will also be required during the construction phase of future projects. This preliminary report was prepared to provide input for project evaluation and costing purposes. A subsequent design-level investigation report will contain recommendations for future geotechnical observation and testing services needed to: 1) ensure contract compliance; 2) check geotechnical design assumptions; 3) facilitate any design changes needed to address unforeseen conditions; and 4) provide the requisite construction reports to the Division of the State Architect (DSA).

8.00 LIMITATIONS

This report has been prepared for the exclusive use of the District and their consultants for specific application to the conceptual design of the TLHS improvements described herein. The opinions presented in this report were developed in accordance with generally-accepted geotechnical and engineering geologic principles and practices. No other warranty, expressed or implied, is made. In the event that any changes in the nature or design of the project are planned, the conclusions and recommendations contained in this report should not be considered valid unless the changes are reviewed and the conclusions of this report are modified or verified in writing.

The findings of this report are valid as of the present date. However, the passing of time will likely change the conditions of the existing property due to natural processes or the works of man. In addition, due to legislation or the broadening of knowledge, changes in applicable or appropriate standards will occur. Accordingly, this report should not be relied upon after a period of three years without being reviewed by this office.

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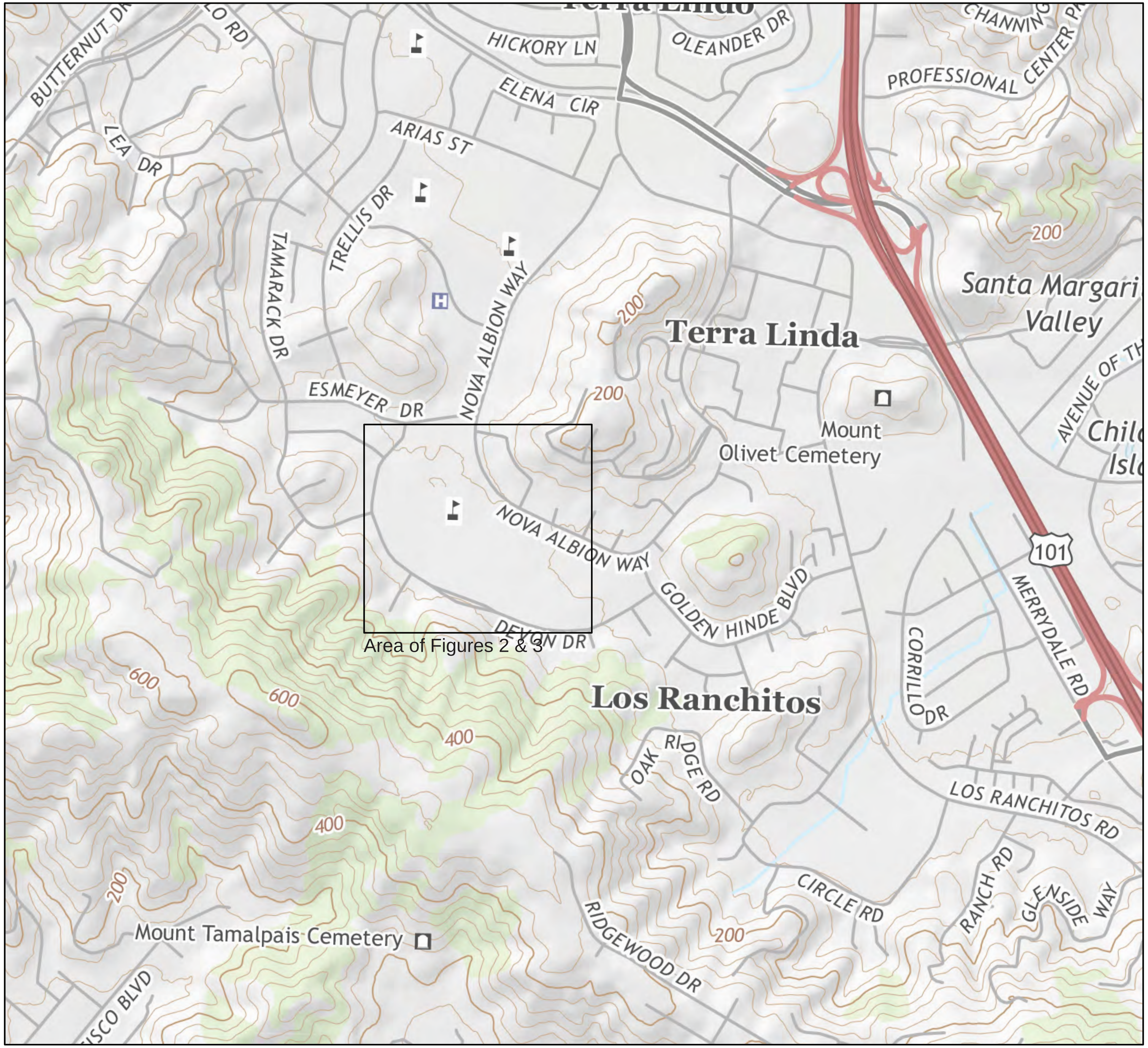
FIGURES

A3GEO

A3GEO, Inc.
PROJECT No. 1150-1A
TERRA LINDA HIGH SCHOOL
SAN RAFAEL



SCALE
0 1,000 2,000 FEET

A3GEO

FIGURE 1
SITE LOCATION MAP

A3GEO, Inc.
PROJECT No. 1150-1A
TERRA LINDA HIGH SCHOOL
SAN RAFAEL

A3GEO

LEGEND:

- B-5 — APPROXIMATE LOCATION OF EXPLORATORY BORING BY A3GEO (THIS STUDY)
- CPT-4 — APPROXIMATE LOCATION OF CPT BY A3GEO (THIS STUDY)
- B-2 — BORING (MPEG, 2003)
- CPT-1 — CPT (MPEG, 2003)
- PROPOSED BUILDING
- SITE BOUNDARY
- 100 — ELEVATION CONTOURS IN FEET (VERTICAL DATUM NAVD 1988)

SCALE
0 200 400 FEET

FIGURE 2
SITE PLAN

A3GEO, Inc.
PROJECT No. 1150-1A
TERRA LINDA HIGH SCHOOL
SAN RAFAEL

A3GEO

LEGEND:

- B-5 — APPROXIMATE LOCATION OF EXPLORATORY BORING BY A3GEO (THIS STUDY)
- CPT-4 — APPROXIMATE LOCATION OF CPT BY A3GEO (THIS STUDY)
- B-2 — BORING (MPEG, 2003)
- CPT-1 — CPT (MPEG, 2003)
- PROPOSED BUILDING
- SITE BOUNDARY

Air Photo Base: 11/8/1950

SCALE
0 200 400 FEET

FIGURE 3
PREDEVELOPMENT AERIAL PHOTOGRAPH

A3GEO, Inc.
PROJECT No. 1150-1A
TERRA LINDA HIGH SCHOOL
SAN RAFAEL

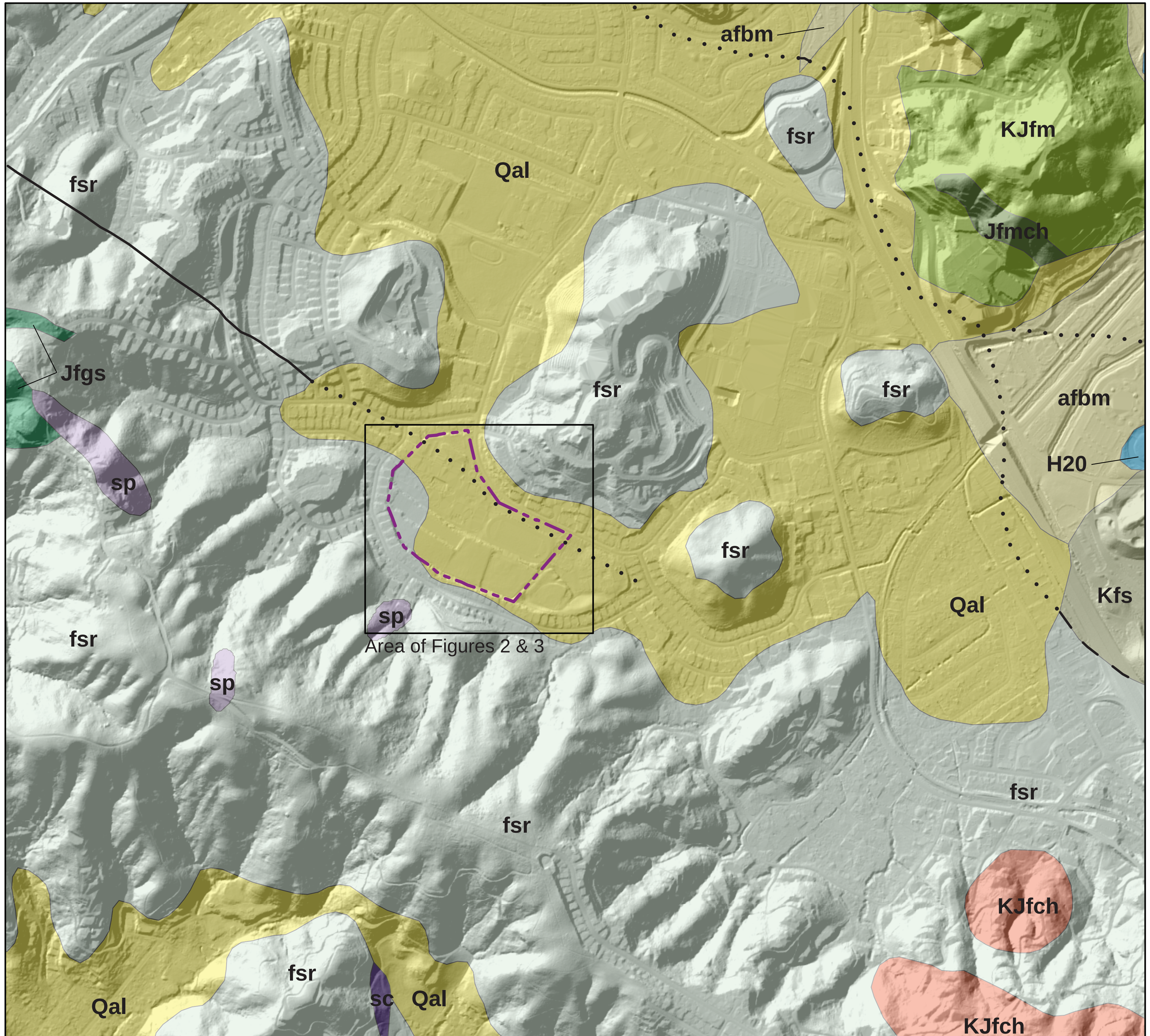
A3GEO

LEGEND:

afbm	ARTIFICIAL FILL OVER BAY MUD
Qal	ALLUVIUM
	FRANCISCAN COMPLEX
Kfs	SANDSTONE AND SHALE
KJfm	METAMORPHIC ROCK
KJfch	CHERT
Jfgs	GREENSTONE
sp	SERPENTINITE
sc	SILICA-CARBONATE ROCK
fsr	MÉLANGE

FAULT, DASHED WHERE APPROX. LOCATED, DOTTED WHERE CONCEALED

SITE BOUNDARY



Base map modified from: Miscellaneous Field Study MF-2337 for parts of Marin, San Francisco, and Contra Costa counties, 2000.

SCALE
0 1,000 2,000 FEET

FIGURE 4
GEOLOGY MAP

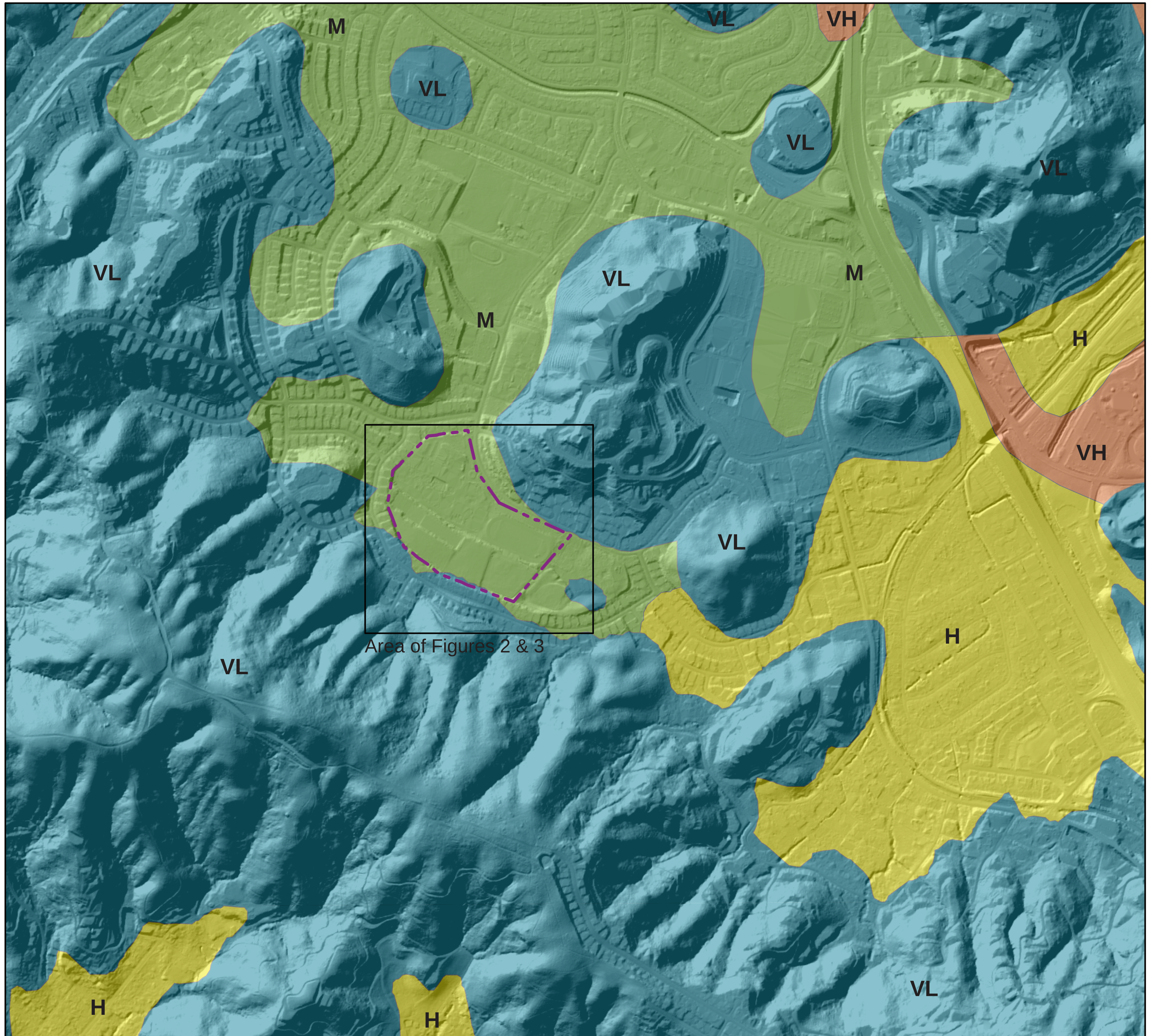
A3GEO, Inc.
PROJECT No. 1150-1A
TERRA LINDA HIGH SCHOOL
SAN RAFAEL

A3GEO

LEGEND:

LIQUEFACTION POTENTIAL

VL	VERY LOW
M	MODERATE
H	HIGH
VH	VERY HIGH
- - - -	SITE BOUNDARY



Base map modified from: U.S. Geological Survey Open File Report 00-444, 2000.

SCALE
0 1,000 2,000 FEET

FIGURE 5a
LIQUEFACTION MAP

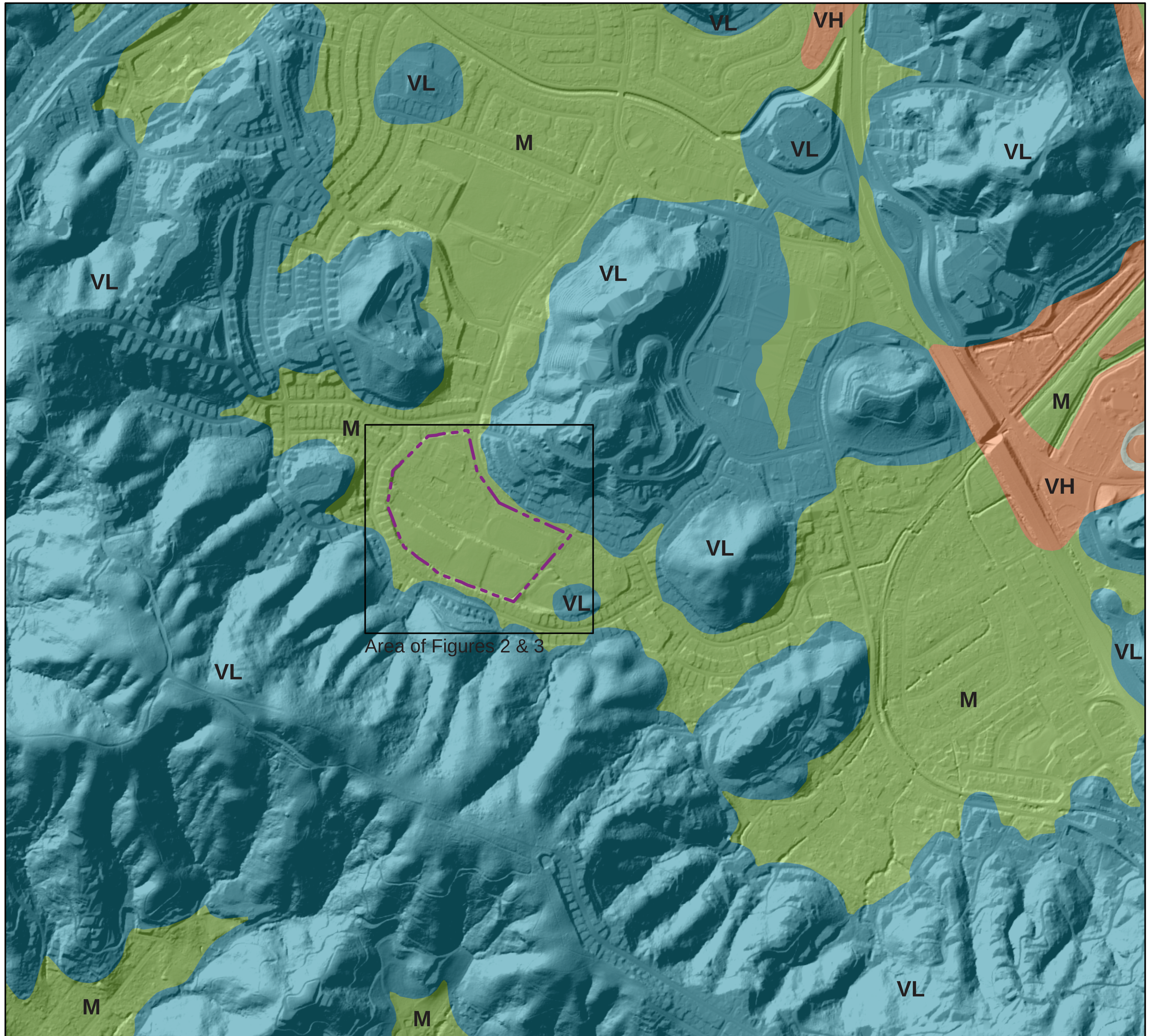
A3GEO, Inc.
PROJECT No. 1150-1A
TERRA LINDA HIGH SCHOOL
SAN RAFAEL

A3GEO

LEGEND:

LIQUEFACTION POTENTIAL

VL	VERY LOW
M	MODERATE
VH	VERY HIGH
- - -	SITE BOUNDARY



Base map modified from: U.S. Geological Survey Open File Report 06-1037, 2006.

SCALE

0 1,000 2,000 FEET

FIGURE 5b

LIQUEFACTION MAP

A3GEO, Inc.
PROJECT No. 1150-1A
TERRA LINDA HIGH SCHOOL
SAN RAFAEL

A3GEO

LEGEND:

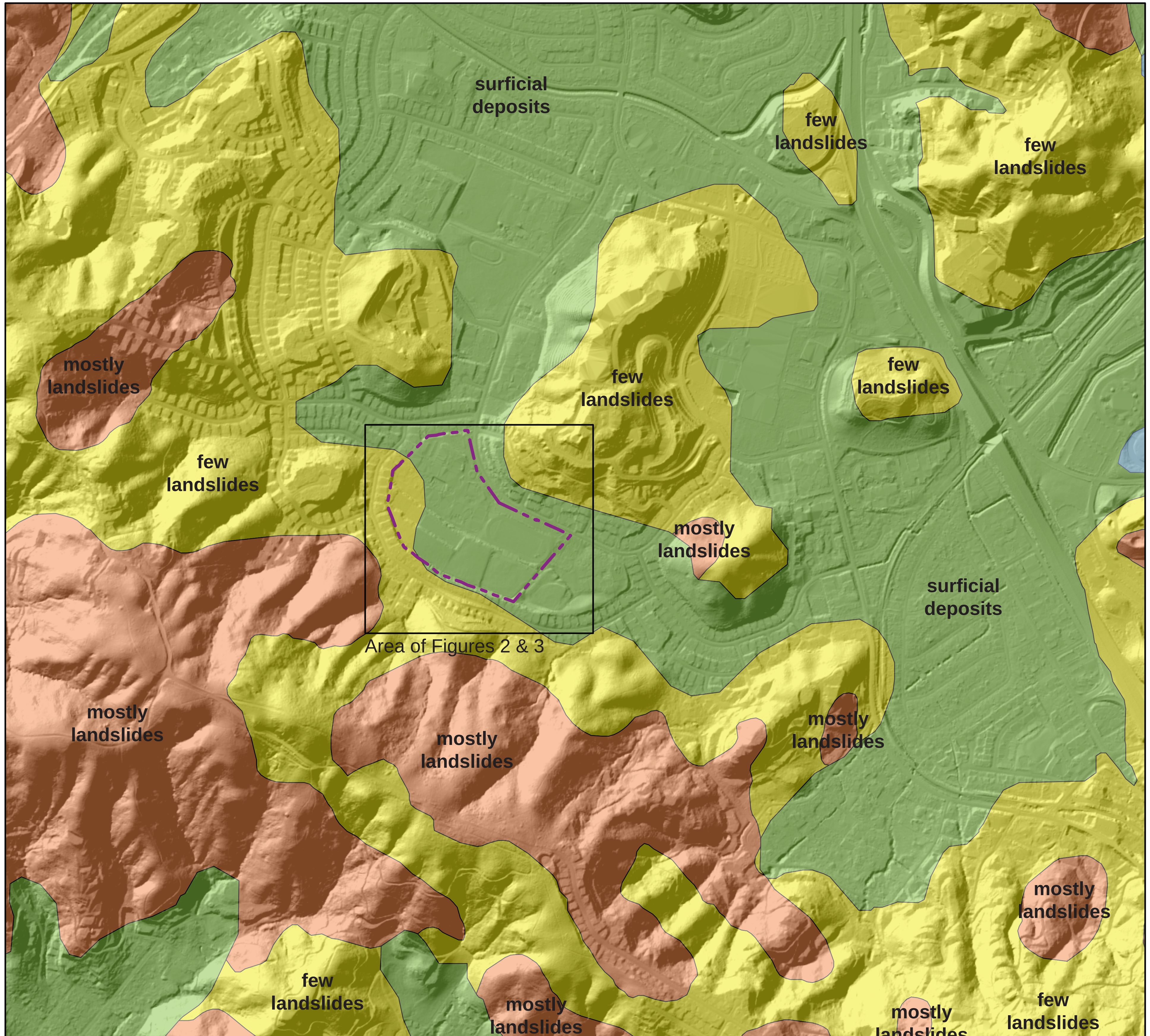
LANDSLIDE HAZARD

SURFICIAL DEPOSITS

FEW LANDSLIDES

MOSTLY LANDSLIDES

SITE BOUNDARY



Base map modified from: U.S. Geological Survey Summary Distribution of Slides and Earth Flows in the San Francisco Bay, 1997.

SCALE

0 1,000 2,000 FEET

FIGURE 6

LANDSLIDE MAP

A3GEO, Inc.
PROJECT No. 1150-1A
TERRA LINDA HIGH SCHOOL
SAN RAFAEL

A3GEO

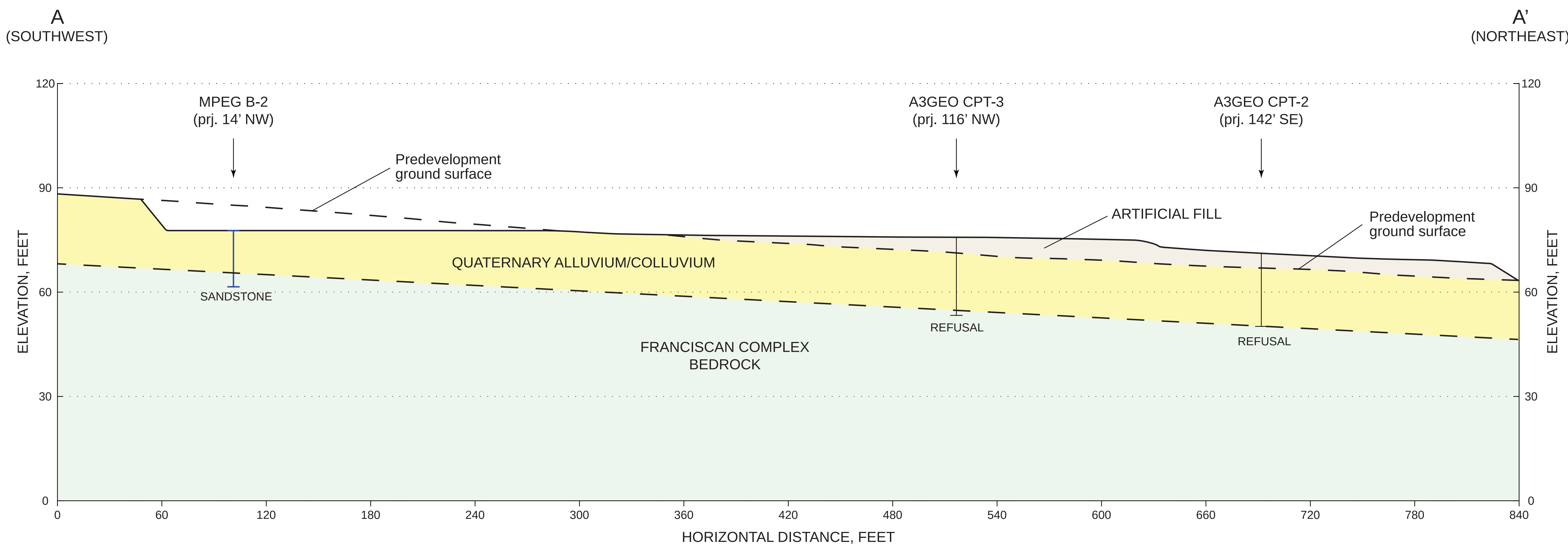
LEGEND:

Qaf	ARTIFICIAL FILL
Qa/Qc	ALLUVIUM/COLLUVIUM (QUATERNARY)
fsr	MÉLANGE
	CONTACT APPROXIMATELY LOCATED
A A'	LINE OF CROSS SECTION
B-5	APPROXIMATE LOCATION OF EXPLORATORY BORING BY A3GEO (THIS STUDY)
CPT-4	APPROXIMATE LOCATION OF CPT BY A3GEO (THIS STUDY)
B-2	BORING (MPEG, 2003)
CPT-1	CPT (MPEG, 2003)
	PROPOSED BUILDING
	SITE BOUNDARY

SCALE
0 200 400 FEET

FIGURE 7
PRELIMINARY SITE GEOLOGIC MAP

A3GEO, Inc.
PROJECT No. 1150-1A
TERRA LINDA HIGH SCHOOL
SAN RAFAEL



2:1 Horizontal to Vertical

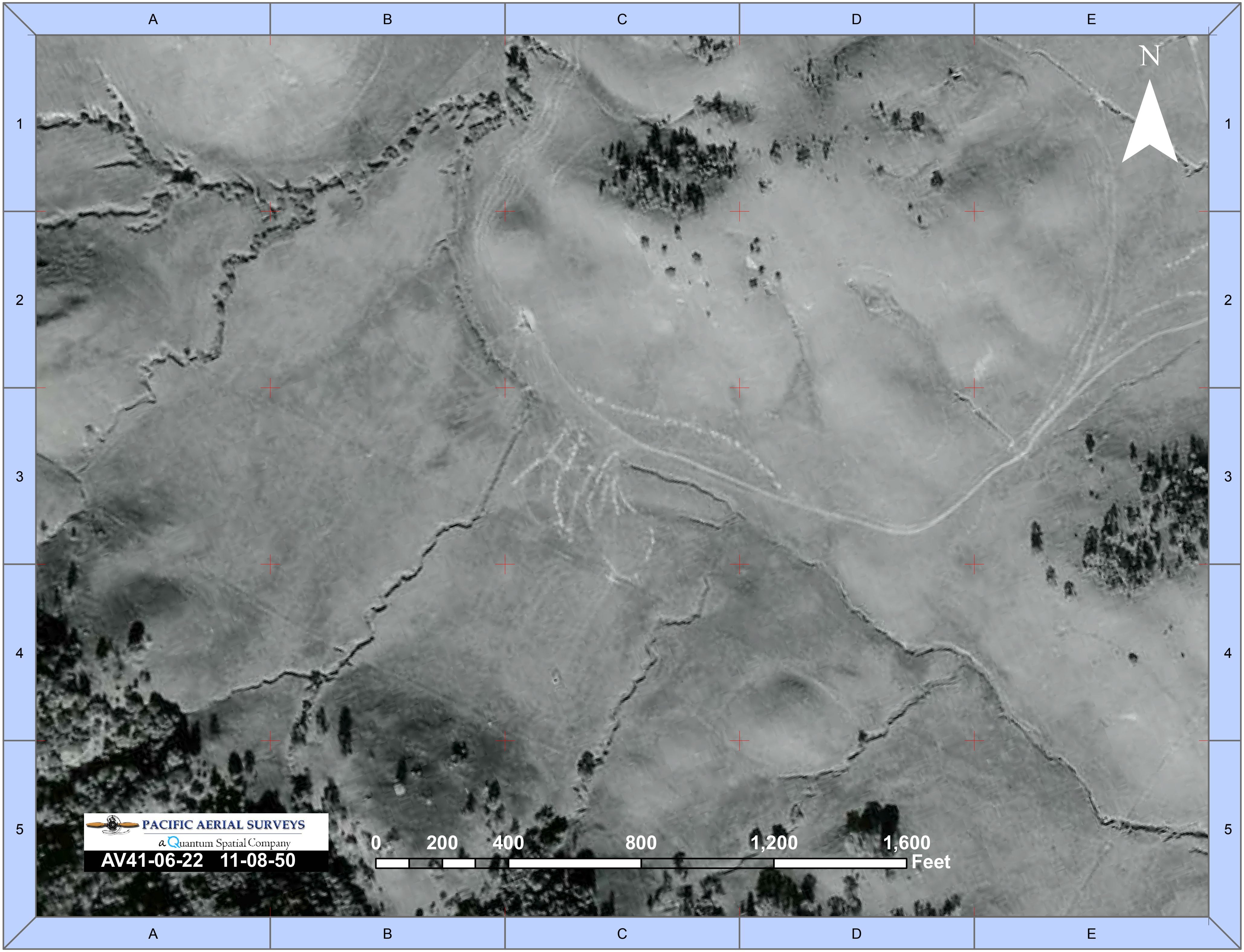
Note: Ground surface profiles shown were interpreted from profile at A - A' on Sheet A1A of the 1958 plans for the school (Appendix F) and are only approximate.

FIGURE 8

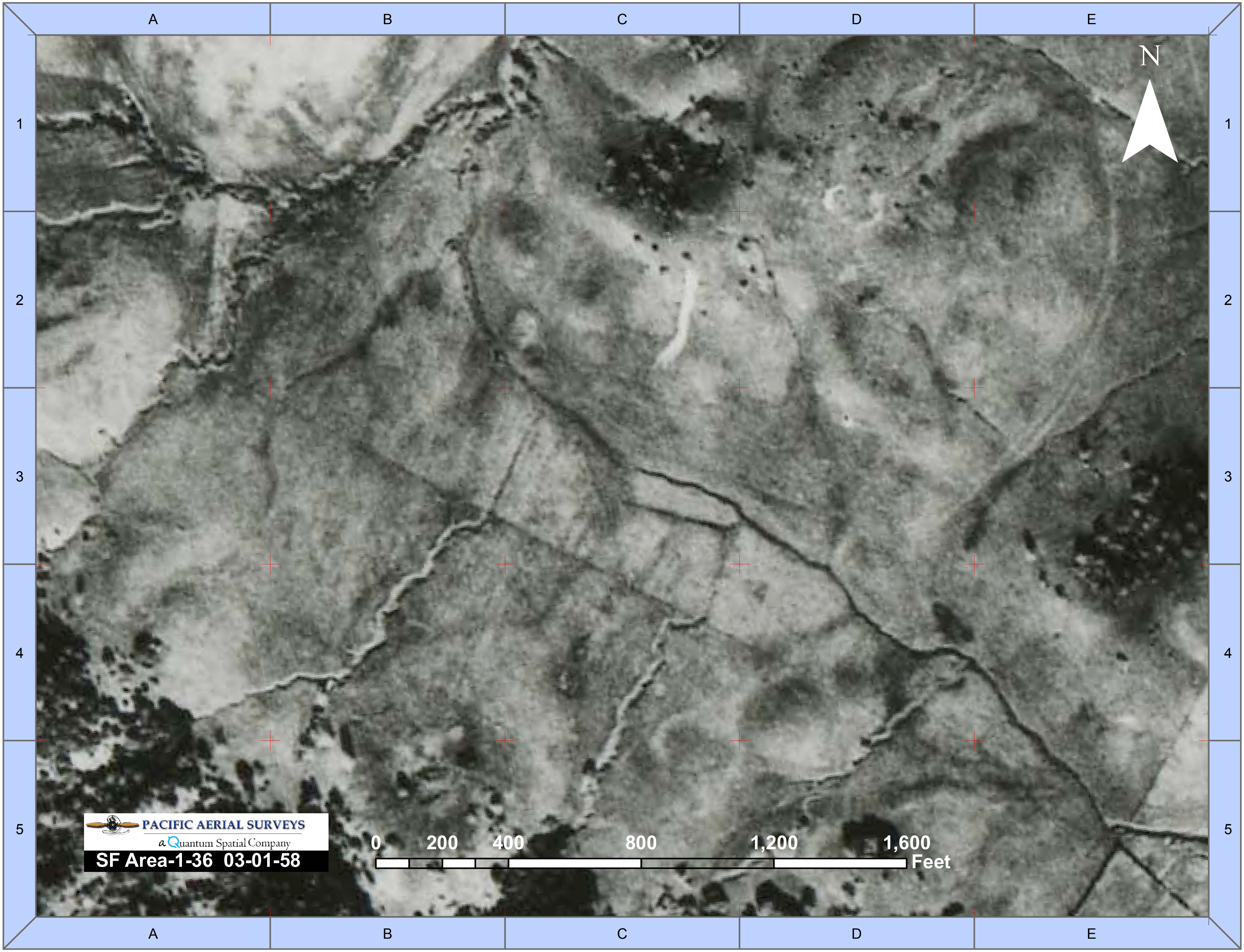
PRELIMINARY GEOLOGIC SECTION A - A'

APPENDIX A

Historic Aerial Photographs



A
B
C
D
E
1
2
3
4
5
N
PACIFIC AERIAL SURVEYS
a Quantum Spatial Company
AV41-06-22 11-08-50
0
200
400
800
1,200
1,600
Feet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a Quantum Spatial Company
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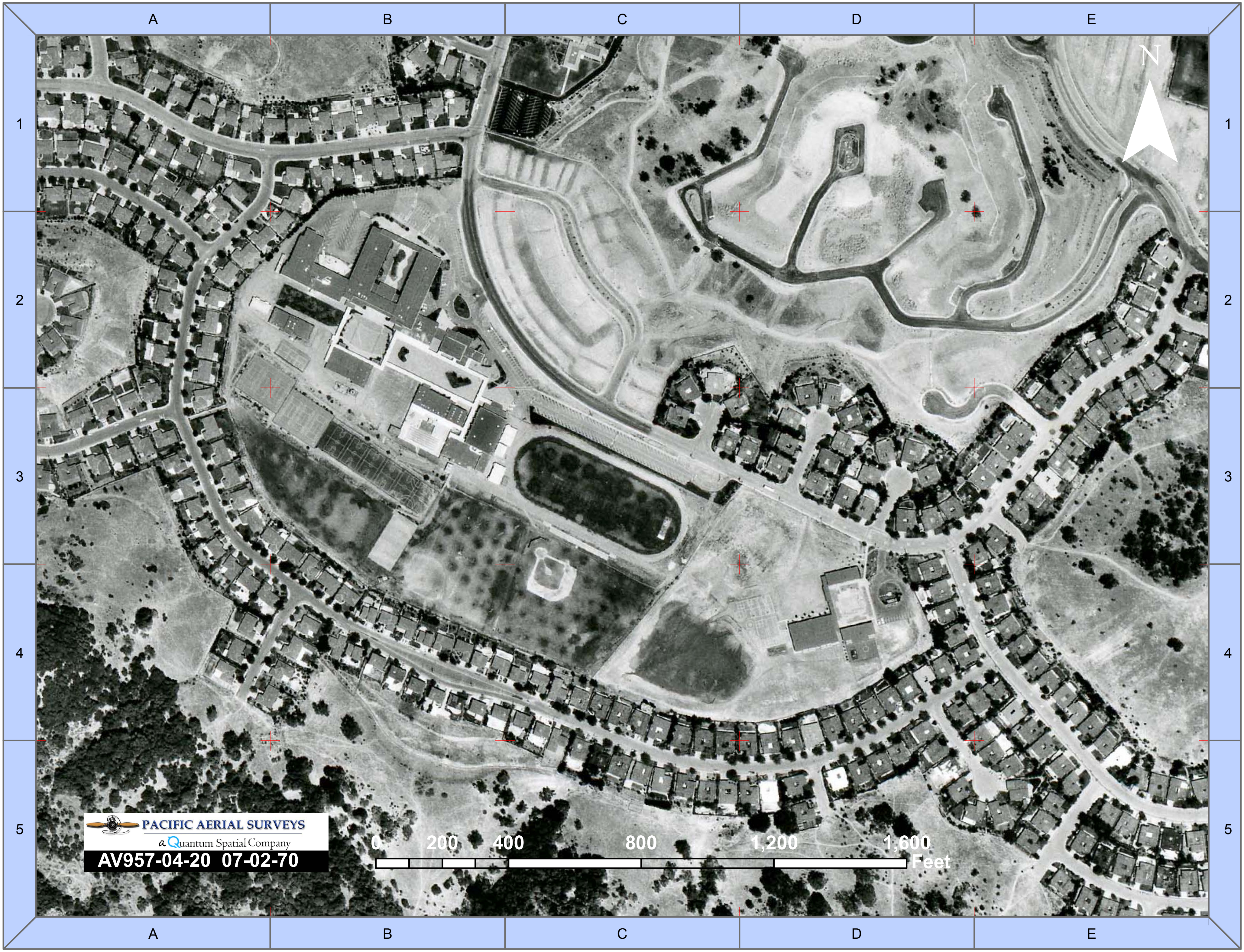
1,600
Feet



A
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a Quantum Spatial Company
AV957-04-20 07-02-70
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1,200
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Feet


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PACIFIC AERIAL SURVEYS
a Quantum Spatial Company
AV1187-04-20 04-17-75
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1,600
Feet


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PACIFIC AERIAL SURVEYS
a Quantum Spatial Company
AV2140-04-21 05-03-82
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Feet

A
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a Quantum Spatial Company
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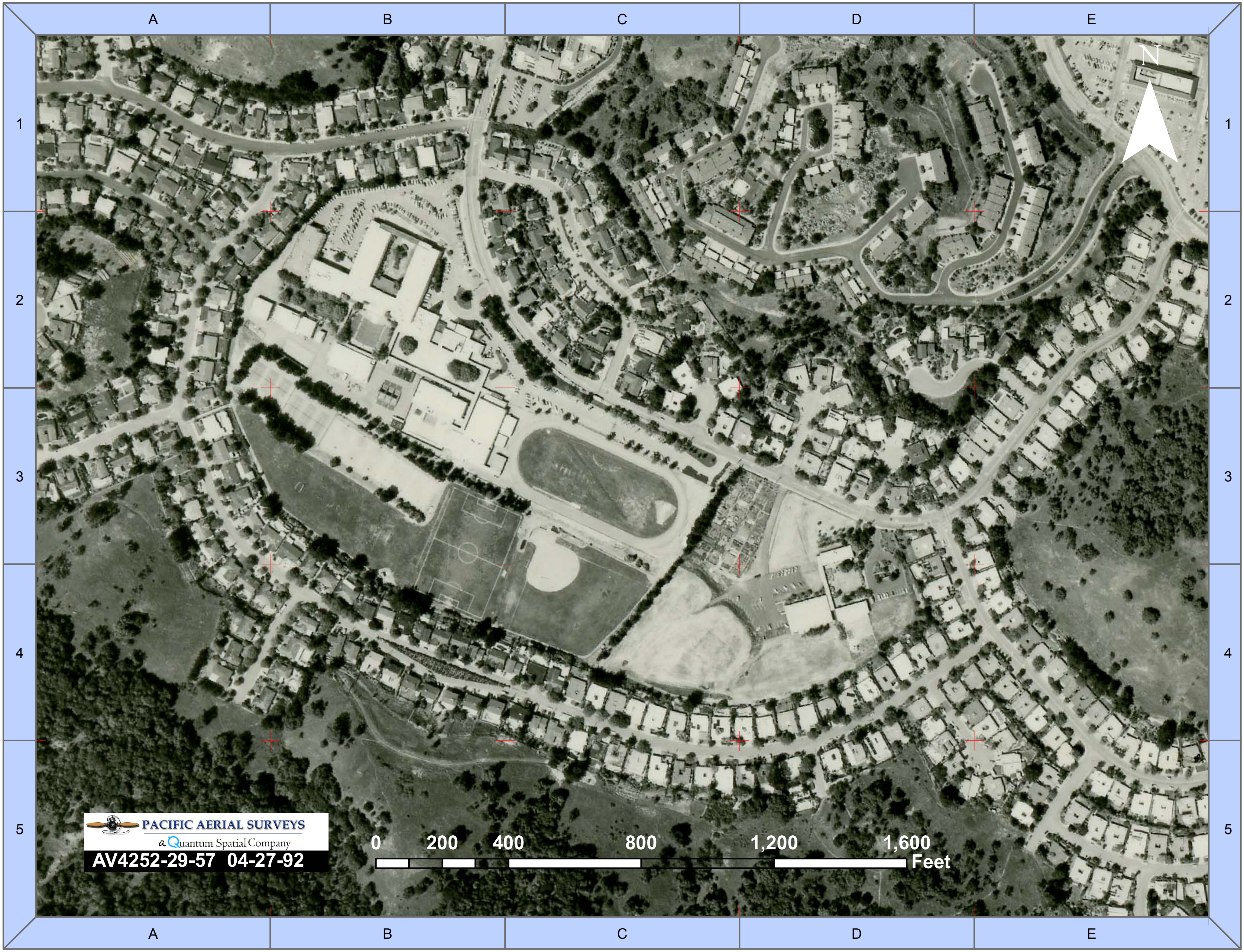
1,200
1,600
Feet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PACIFIC AERIAL SURVEYS
a Quantum Spatial Company
AV4252-29-57 04-27-92
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200
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800
1,200
1,600
Feet

A
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PACIFIC AERIAL SURVEYS
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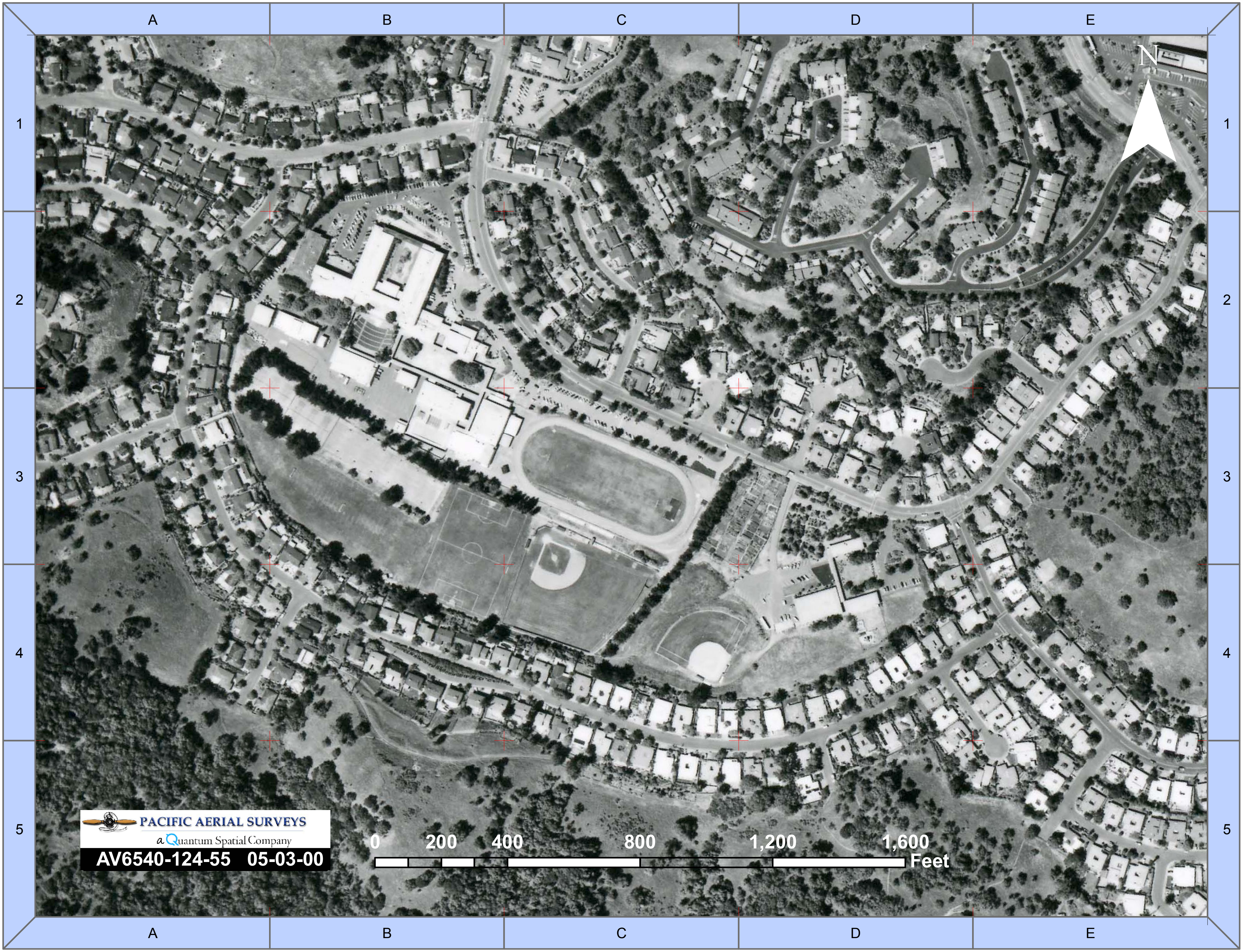
Feet


A
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PACIFIC AERIAL SURVEYS
a Quantum Spatial Company
AV6540-124-55 05-03-00
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800
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Feet


A
B
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5
N
PACIFIC AERIAL SURVEYS
a Quantum Spatial Company
Terra Linda HS 2015
0
200
400
800
1,200
1,600
Feet

APPENDIX B

Cone Penetration Test Data
(this study)

A3GEO

GREGG DRILLING & TESTING, INC.
GEOTECHNICAL AND ENVIRONMENTAL INVESTIGATION SERVICES

February 24, 2017

A3GEO
Attn: Wayne Magnusen

Subject: CPT Site Investigation
SRCS/Terra Linda High School
San Rafael, California
GREGG Project Number: 17-026MA

Dear Mr. Magnusen:

The following report presents the results of GREGG Drilling & Testing's Cone Penetration Test investigation for the above referenced site. The following testing services were performed:

1	Cone Penetration Tests	(CPTU)	☒
2	Pore Pressure Dissipation Tests	(PPD)	☒
3	Seismic Cone Penetration Tests	(SCPTU)	☐
4	UVOST Laser Induced Fluorescence	(UVOST)	☐
5	Groundwater Sampling	(GWS)	☐
6	Soil Sampling	(SS)	☐
7	Vapor Sampling	(VS)	☐
8	Membrane Interface Probe	(MIP)	☐
9	Vane Shear Testing	(VST)	☐
10	Dilatometer Testing	(DMT)	☐

A list of reference papers providing additional background on the specific tests conducted is provided in the bibliography following the text of the report. If you would like a copy of any of these publications or should you have any questions or comments regarding the contents of this report, please do not hesitate to contact our office at (925) 313-5800.

Sincerely,
GREGG Drilling & Testing, Inc.

Mary Walden
Operations Manager

950 Howe Rd • Martinez, California 94553 • (925) 313-5800 • FAX (925) 313-0302
www.greggdrilling.com

Cone Penetration Test Sounding Summary

-Table 1-

CPT Sounding Identification	Date	Termination Depth (feet)	Depth of Groundwater Samples (feet)	Depth of Soil Samples (feet)	Depth of Pore Pressure Dissipation Tests (feet)
CPT-01	2/22/17	25	-	-	24.6
CPT-02	2/22/17	21	-	-	-
CPT-03	2/22/17	23	-	-	-
CPT-04	2/22/17	18	-	-	17.7

Bibliography

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Copies of ASTM Standards are available through www.astm.org

Cone Penetration Testing Procedure (CPT)

Gregg Drilling carries out all Cone Penetration Tests (CPT) using an integrated electronic cone system, *Figure CPT*.

The cone takes measurements of tip resistance (q_c), sleeve resistance (f_s), and penetration pore water pressure (u_2). Measurements are taken at either 2.5 or 5 cm intervals during penetration to provide a nearly continuous profile. CPT data reduction and basic interpretation is performed in real time facilitating on-site decision making. The above mentioned parameters are stored electronically for further analysis and reference. All CPT soundings are performed in accordance with revised ASTM standards (D 5778-12).

The 5mm thick porous plastic filter element is located directly behind the cone tip in the u_2 location. A new saturated filter element is used on each sounding to measure both penetration pore pressures as well as measurements during a dissipation test (*PPDT*). Prior to each test, the filter element is fully saturated with oil under vacuum pressure to improve accuracy.

When the sounding is completed, the test hole is backfilled according to client specifications. If grouting is used, the procedure generally consists of pushing a hollow tremie pipe with a "knock out" plug to the termination depth of the CPT hole. Grout is then pumped under pressure as the tremie pipe is pulled from the hole. Disruption or further contamination to the site is therefore minimized.

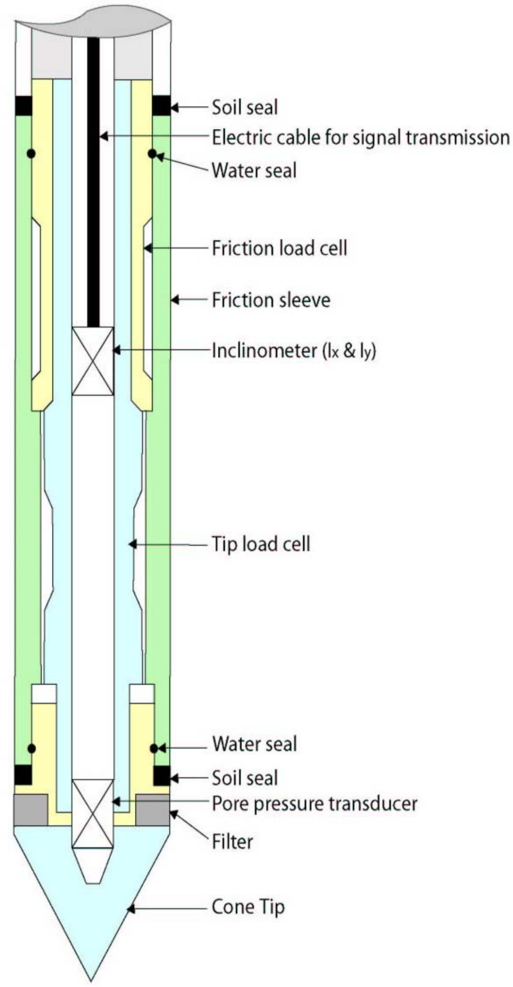


Figure CPT

Gregg 15cm^2 Standard Cone Specifications

Dimensions	
Cone base area	15 cm^2
Sleeve surface area	225 cm^2
Cone net area ratio	0.80
Specifications	
Cone load cell	
Full scale range	180 kN (20 tons)
Overload capacity	150%
Full scale tip stress	120 MPa (1,200 tsf)
Repeatability	120 kPa (1.2 tsf)
Sleeve load cell	
Full scale range	31 kN (3.5 tons)
Overload capacity	150%
Full scale sleeve stress	1,400 kPa (15 tsf)
Repeatability	1.4 kPa (0.015 tsf)
Pore pressure transducer	
Full scale range	7,000 kPa (1,000 psi)
Overload capacity	150%
Repeatability	7 kPa (1 psi)

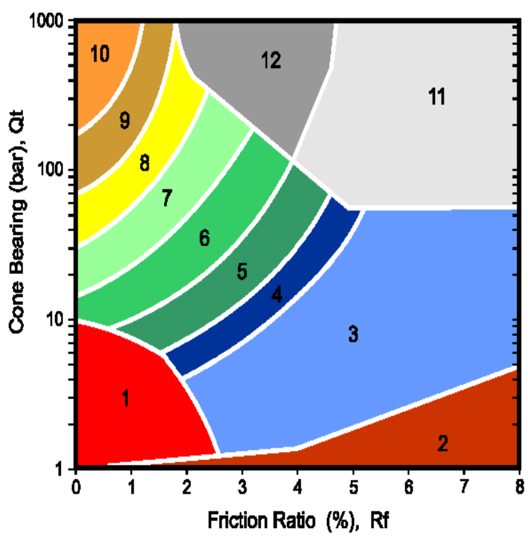
Note: The repeatability during field use will depend somewhat on ground conditions, abrasion, maintenance and zero load stability.

Cone Penetration Test Data & Interpretation

The Cone Penetration Test (CPT) data collected are presented in graphical and electronic form in the report. The plots include interpreted Soil Behavior Type (SBT) based on the charts described by Robertson (1990). Typical plots display SBT based on the non-normalized charts of Robertson et al (1986). For CPT soundings deeper than 30m, we recommend the use of the normalized charts of Robertson (1990) which can be displayed as SBTn, upon request. The report also includes spreadsheet output of computer calculations of basic interpretation in terms of SBT and SBTn and various geotechnical parameters using current published correlations based on the comprehensive review by Lunne, Robertson and Powell (1997), as well as recent updates by Professor Robertson (Guide to Cone Penetration Testing, 2015). The interpretations are presented only as a guide for geotechnical use and should be carefully reviewed. Gregg Drilling & Testing Inc. does not warranty the correctness or the applicability of any of the geotechnical parameters interpreted by the software and does not assume any liability for use of the results in any design or review. The user should be fully aware of the techniques and limitations of any method used in the software. Some interpretation methods require input of the groundwater level to calculate vertical effective stress. An estimate of the in-situ groundwater level has been made based on field observations and/or CPT results, but should be verified by the user.

A summary of locations and depths is available in Table 1. Note that all penetration depths referenced in the data are with respect to the existing ground surface.

Note that it is not always possible to clearly identify a soil type based solely on q_t, f_s, and u_2. In these situations, experience, judgment, and an assessment of the pore pressure dissipation data should be used to infer the correct soil behavior type.



ZONE		SBT
1		Sensitive, fine grained
2		Organic materials
3		Clay
4		Silty clay to clay
5		Clayey silt to silty clay
6		Sandy silt to clayey silt
7		Silty sand to sandy silt
8		Sand to silty sand
9		Sand
10		Gravely sand to sand
11		Very stiff fine grained*
12		Sand to clayey sand*

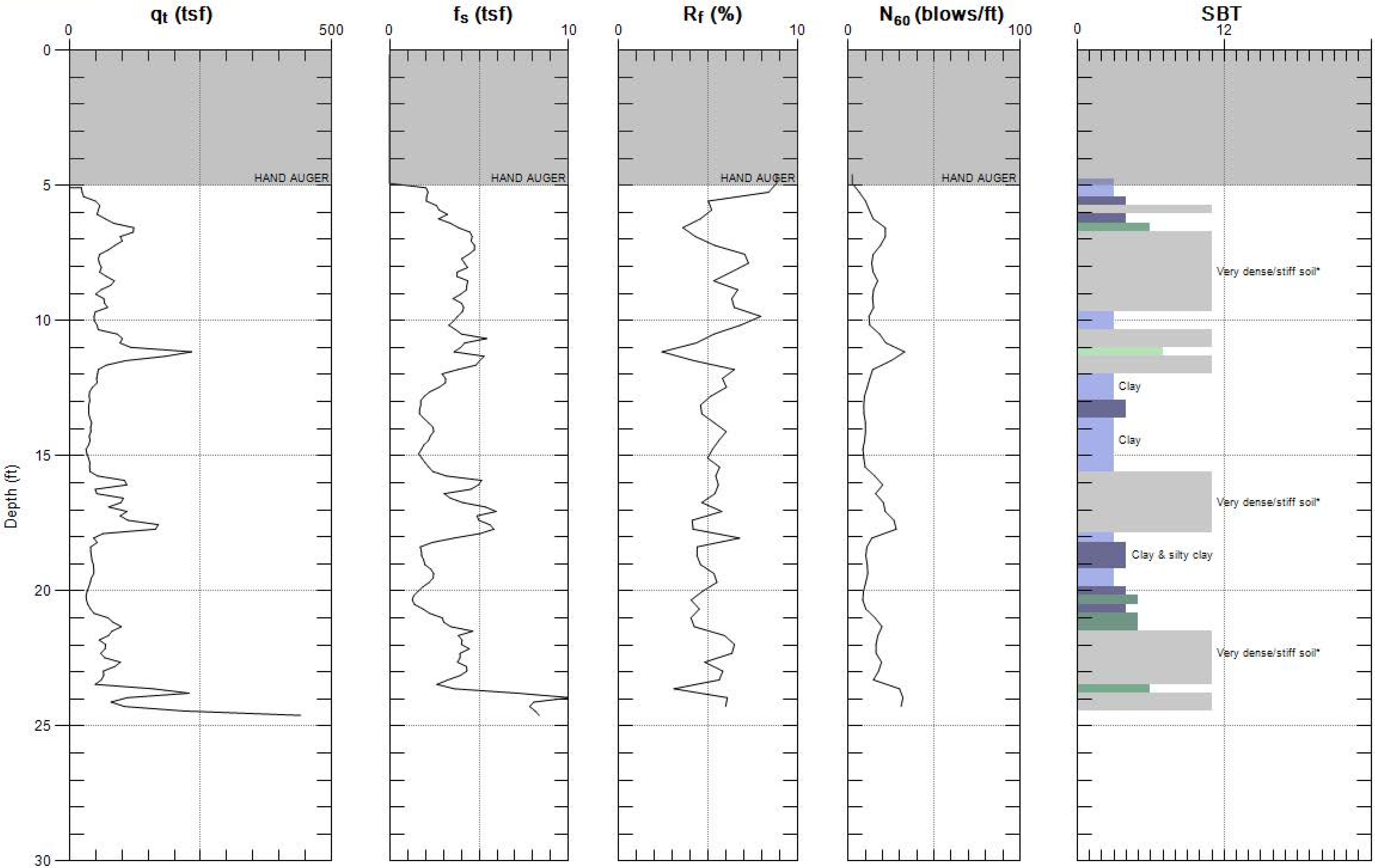
*over consolidated or cemented

Figure SBT (After Robertson et al., 1986) – Note: Colors may vary slightly compared to plots

GREGG **A3GEO**

Site: TERRA LINDA H.S.
Sounding: CPT-1A

Engineer: J.VAN DEN BERG
Date: 2/22/17 09:24



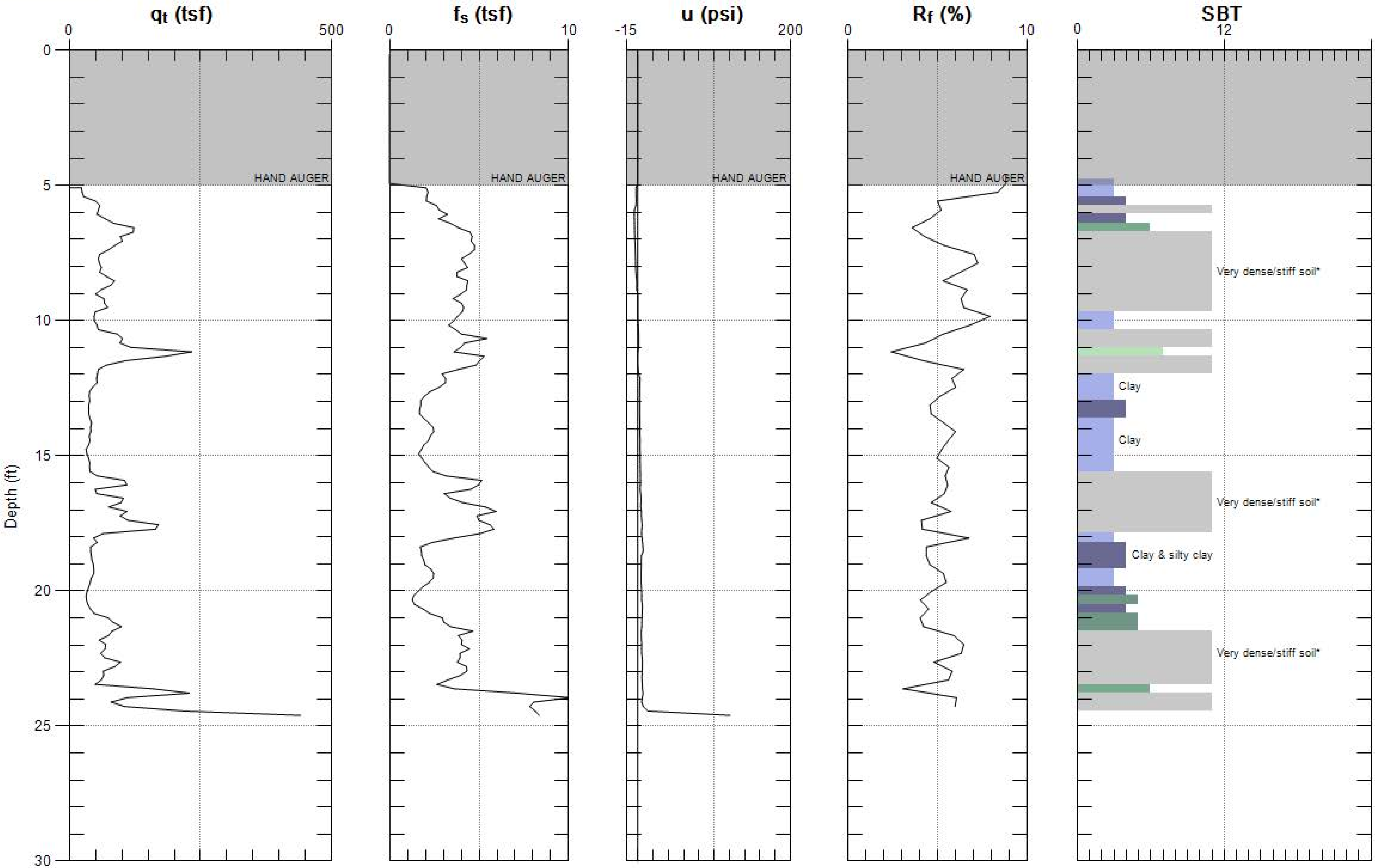
Max. Depth: 24.606 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

A3GEO

Site: TERRA LINDA H.S.
Sounding: CPT-1A

Engineer: J.VAN DEN BERG
Date: 2/22/17 09:24



Max. Depth: 24.606 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

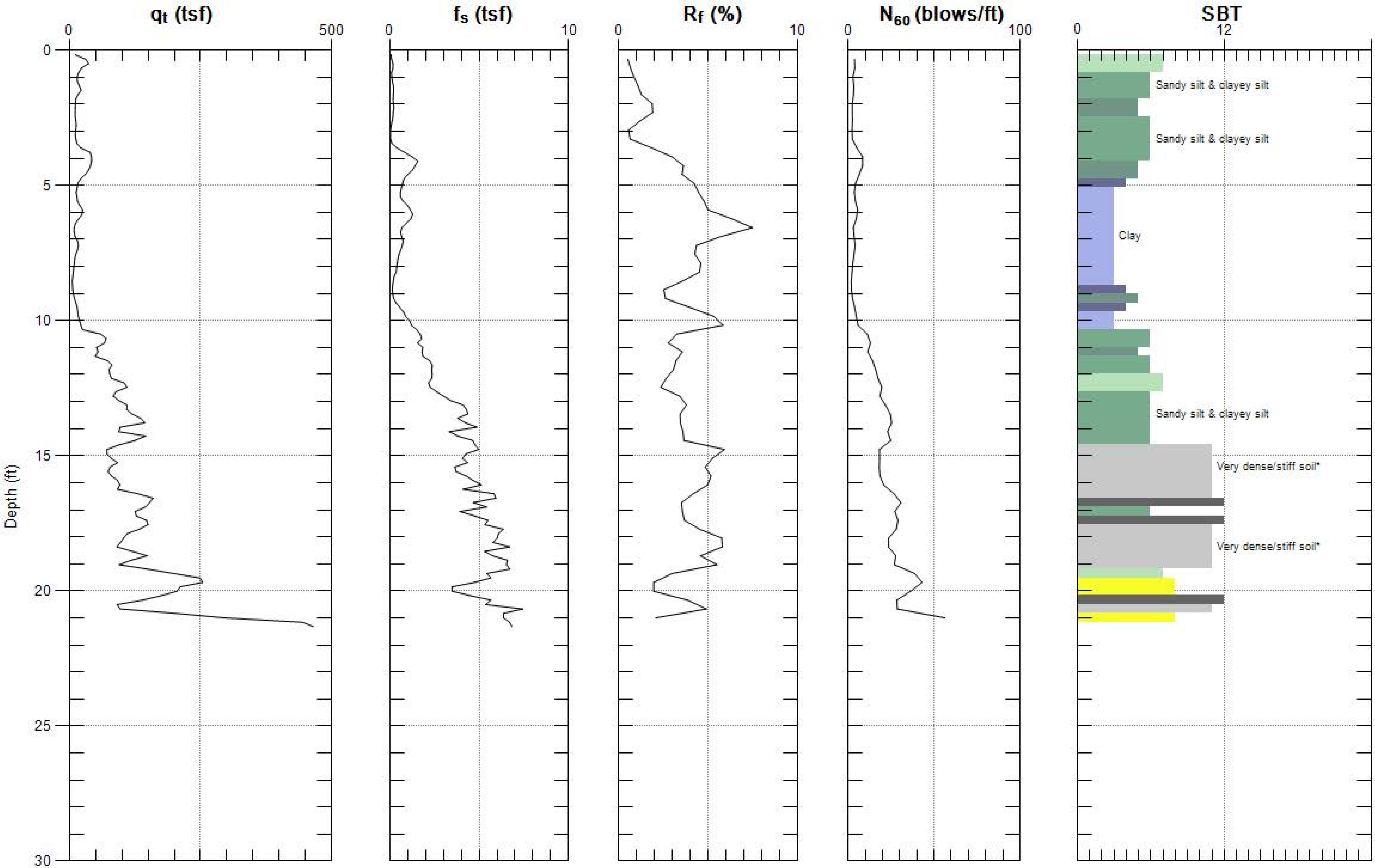
A3GEO

Site: TERRA LINDA H.S.

Sounding: CPT-02

Engineer: J.VAN DEN BERG

Date: 2/22/17 08:08



Max. Depth: 21.325 (ft)

Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

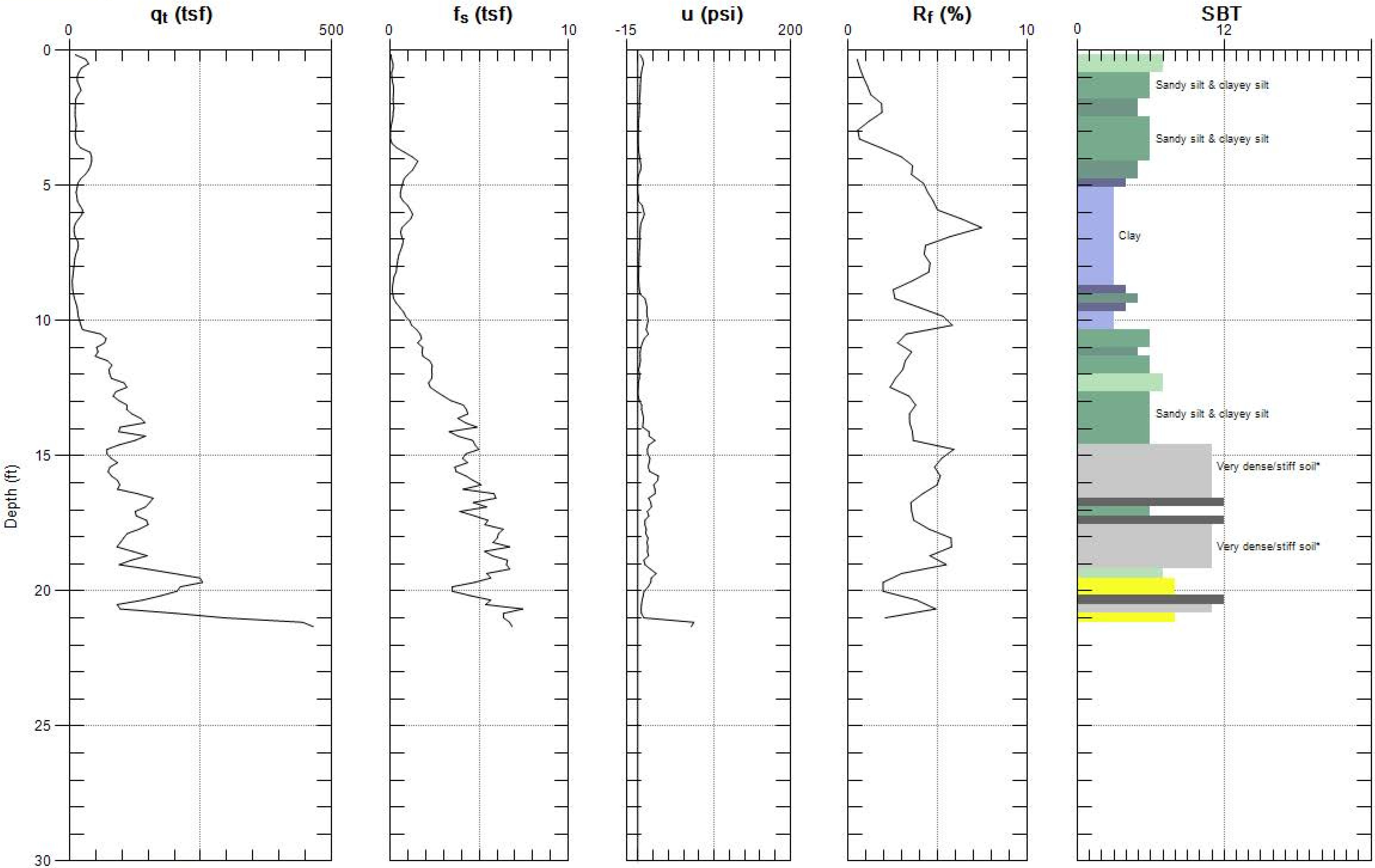
A3GEO

Site: TERRA LINDA H.S.

Sounding: CPT-02

Engineer: J.VAN DEN BERG

Date: 2/22/17 08:08



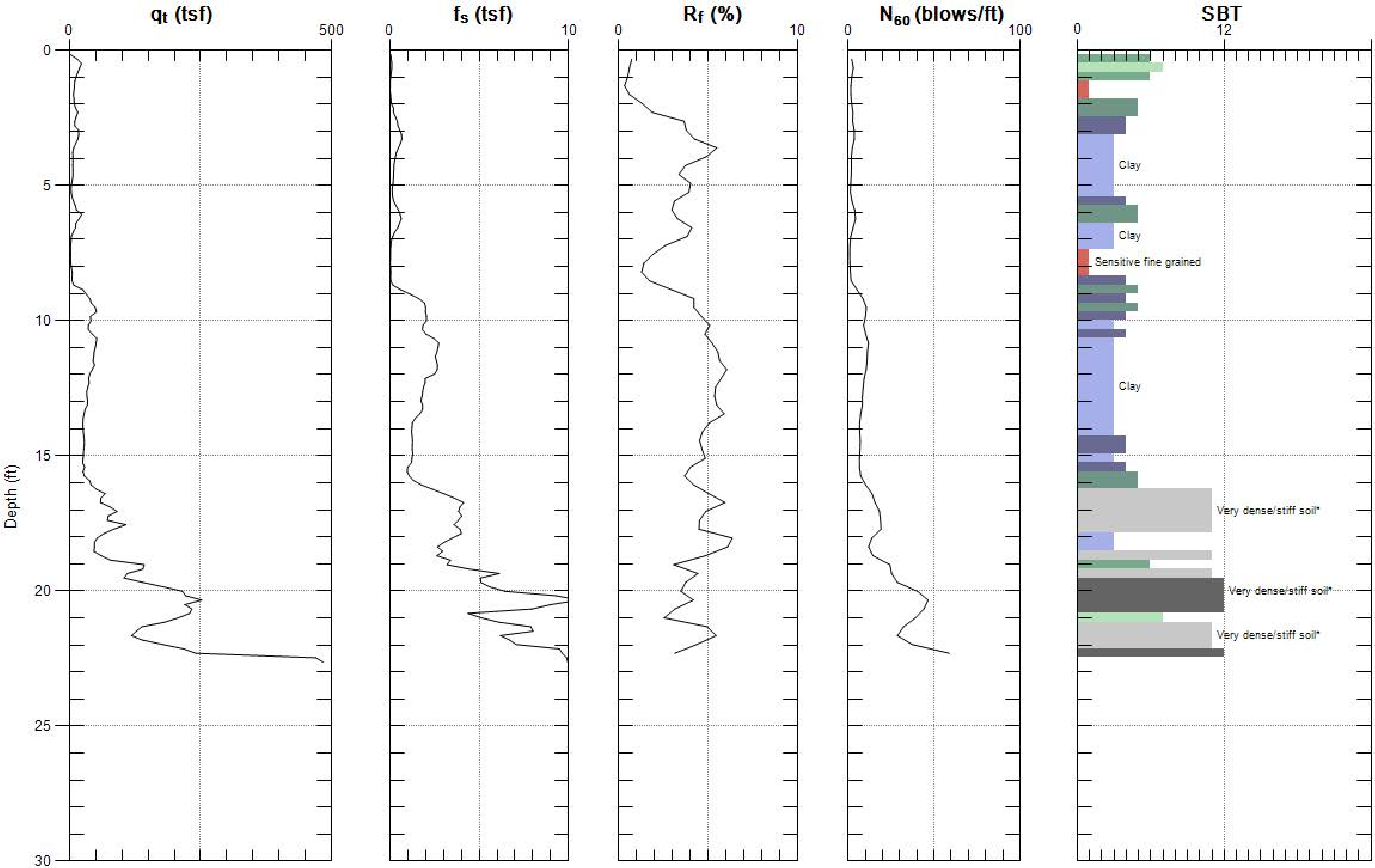
Max. Depth: 21.325 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG **A3GEO**

Site: TERRA LINDA H.S. **Engineer: J.VAN DEN BERG**

Sounding: CPT-03 **Date: 2/22/17 10:38**



Max. Depth: 22.638 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

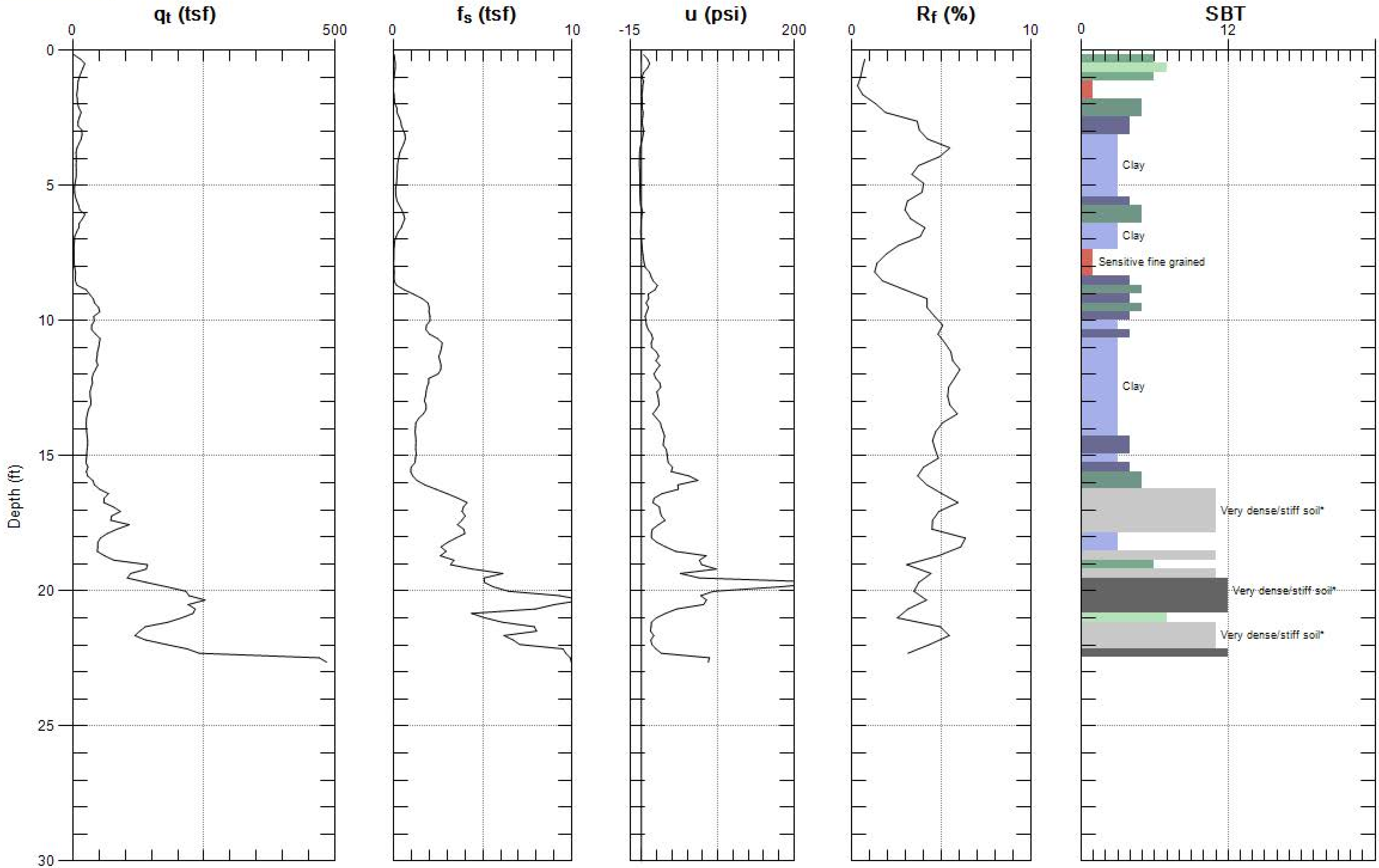
A3GEO

Site: TERRA LINDA H.S.

Engineer: J.VAN DEN BERG

Sounding: CPT-03

Date: 2/22/17 10:38



Max. Depth: 22.638 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

GREGG

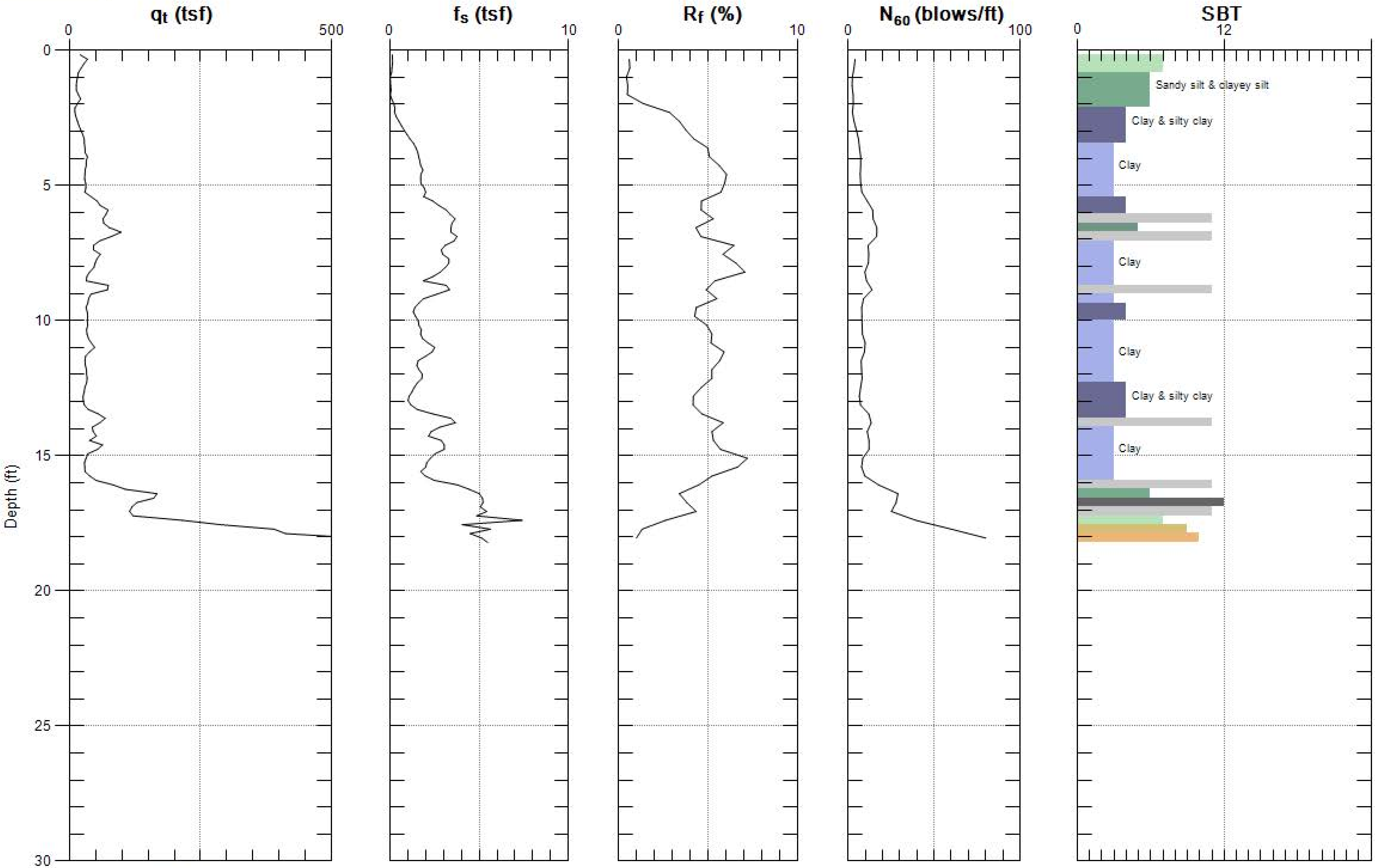
A3GEO

Site: TERRA LINDA H.S.

Sounding: CPT-04

Engineer: J.VAN DEN BERG

Date: 2/22/17 11:34



Max. Depth: 18.209 (ft)
Avg. Interval: 0.328 (ft)

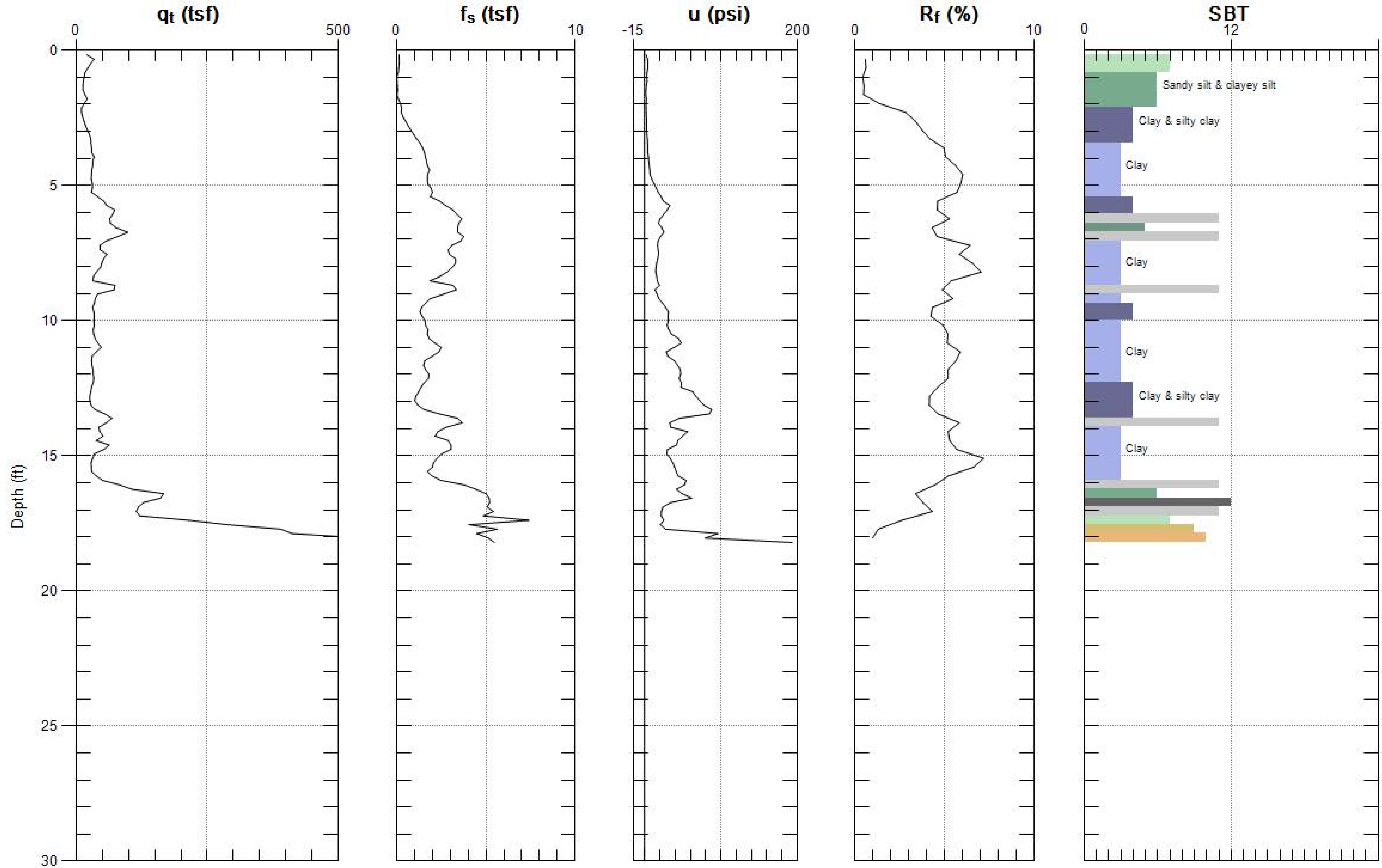
SBT: Soil Behavior Type (Robertson 1990)

GREGG

A3GEO

Site: TERRA LINDA H.S.
Sounding: CPT-04

Engineer: J.VAN DEN BERG
Date: 2/22/17 11:34



Max. Depth: 18.209 (ft)
Avg. Interval: 0.328 (ft)

SBT: Soil Behavior Type (Robertson 1990)

APPENDIX C

Logs of Borings
(this study)

A3GEO

UNIFIED SOIL CLASSIFICATION CHART

MAJOR DIVISIONS				TYPICAL NAMES
COARSE GRAINED SOILS: more than 50% retained on No. 200 sieve	COARSE GRAINED SOILS: 50% or more of coarse fraction on No. 4 sieve	CLEAN GRAVELS	GW	Well graded gravels and gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels and gravel-sand mixtures, little or no fines
		GRAVELS WITH SAND	GM	Silty gravels and gravel-sand-silt mixtures
			GC	Clayey gravels and gravel-sand-clay mixtures
	SANDS: more than 50% passing on No. 4 sieve	CLEAN SANDS	SW	Well graded sands and gravelly sand, little or no fines
			SP	Poorly graded sands and gravelly sand, little or no fines
		SANDS WITH FINES	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS: 50% or more passing No. 200 sieve	SILTS AND CLAY: Liquid Limit 50% or less		ML	Inorganic silts, very fine sands, rock flour, silty or clayey fine sands
			CL	Inorganic clays or low to medium plasticity, gravelly clays, sandy clays, silty clays, lean clays
			OL	Organic silts and organic silty clays of low plasticity
	SILTS AND CLAY: Liquid Limit 50% or greater		MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic clays
			CH	Inorganic clays of high plasticity, fat clays
			OH	Organic clays of medium to high plasticity
HIGHLY ORGANIC SOILS			PT	Peat, muck, and other highly organic soils

BOUNDARY CLASSIFICATION AND GRAIN SIZES

SILT OR CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	FINE	MEDIUM	COARSE	FINE	COARSE		
U.S. Standard Sieve Sizes	No. 200 0.075 mm	No. 40 0.425 mm	No. 10 2 mm	No. 4 3/16"	3/4"	3"	12"

SYMBOLS

Modified California (MC) Sampler (3" O.D.)	ROCK CORE (RC)	Disturbed Sample
Standard Penetration Test: SPT (2" O.D.)	Shelby Tube, pushed or used Ostenberg Sampler	Water Levels At time of drilling At end of drilling After drilling

ABBREVIATIONS

Item	Meaning
LL	Liquid Limit (%) (ASTM D 4318)
PI	Plasticity Index (%) (ASTM D 4318)
-200	Passing No. 200 (%) (ASTM D 1140)
TXCU	Laboratory consolidated undrained triaxial test of undrained shear strength (psf) (ASTM D 4767)
TXUU	Laboratory unconsolidated, undrained triaxial test of undrained shear strength (psf) (ASTM D 2850)
psf/tsf	pounds per square foot / tons per square foot
psi	pounds per square inch
OD	Outside Diameter
ID	Inside Diameter

NOTES

1. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.
2. Modified California (MC) blow counts were adjusted by multiplying field blow counts by a factor of 0.63.
3. Recorded blow counts have not been adjusted for hammer energy.

KEY TO EXPLORATORY BORING LOGS

BEDDING OF SEDIMENTARY ROCK		
SPLITTING PROPERTY	THICKNESS	STRATIFICATION
Massive	Greater than 4.0 feet	Very Thick-Bedded
Blocky	2.0 to 4.0 feet	Thick-Bedded
Slabby	0.2 to 2.0 feet	Thin-Bedded
Flaggy	0.05 to 0.2 feet	Very Thin-Bedded
Shaly or Platy	0.01 to 0.05 feet	Laminated
Papery	Less than 0.01 feet	Thinly Laminated

FRACTURING	
INTENSITY	SIZE OF PIECES IN FEET
Very Little Fractured	Greater than 4.0 feet
Occasionally Fractured	1.0 to 4.0 feet
Moderately Fractured	0.5 to 1.0 feet
Closely Fractured	0.1 to 0.5 feet
Intensely Fractured	0.05 to 0.1 feet
Crushed	Less than 0.05 feet

HARDNESS	
Soft	Reserved for plastic material alone
Low Hardness	Can be gouged deeply or carved easily by a knife blade
Moderately Hard	Can be readily scratched by a knife blade; scratch leaves a heavy trace of dust and is readily visible after the powder has been blown away
Hard	Can be scratched by a knife blade with difficulty; scratch produces little powder and is often faintly visible
Very Hard	Cannot be scratched by a knife blade; leaves a metallic streak

STRENGTH	
Plastic	Very low strength
Friable	Crumbles easily by rubbing with fingers
Weak	An unfractured specimen of such material will crumble under light hammer blows
Moderately Strong	Specimen will withstand a few heavy hammer blows before breaking
Strong	Specimen will withstand a few heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments
Very Strong	Specimen will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments

WEATHERING:	
— the physical and chemical disintegration and decomposition of rocks and minerals by natural processes such as oxidation, reduction, hydration, solution, carbonation, and freezing and thawing	
Deep	Moderate to complete mineral decomposition; extensive disintegration; deep and thorough discoloration; many fractures, all extensively coated or filled with oxides, carbonates and/or clay or silt.
Moderate	Slight change or partial decomposition of minerals; little disintegration; cementation little to unaffected. Moderate to occasionally intense discoloration. Moderately coated fractures.
Little	No megascopic decomposition of minerals; little or no effect on normal cementation. Slight and intermittent, or localized discoloration. Few stains on fracture surfaces.
Fresh	Unaffected by weathering agents. No discoloration or disintegration. Fractures usually less numerous than joints.

KEY TO ROCK DESCRIPTIONS

A3GEO, Inc.
1331 7th Street; Unit E
Berkeley, CA 94710
Telephone: 510-705-1664

BORING NUMBER B-1

CLIENT San Rafael City School District
PROJECT NAME Terra Linda High School - Preliminary Investigation
PROJECT NUMBER 1150-1A
PROJECT LOCATION San Rafael, CA
DATE STARTED 2/22/17 **COMPLETED** 2/22/17
GROUND ELEVATION 81.1 ft **HOLE SIZE** 6"
DRILLING CONTRACTOR Gregg Drilling and Testing, Inc.
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING ---
LOGGED BY RES **CHECKED BY** WM
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	% RECOVERED	OTHER LAB TESTS / NOTES
0		ASPHALTIC CONCRETE [3"]							
		AGGREGATE BASE [6"]							
		LEAN CLAY WITH SAND (CL): reddish brown, medium stiff to stiff, moderate plasticity, fine-medium sand , moist	GB						
		between 3'-4.25': some angular gravels, up to 3/4"	MC	20	>4.5	121	14	83%	32% Gravel 39% Sand -200 = 29%
5		CLAYEY SAND WITH GRAVEL (SC): olive brown, dense, well-graded sand, moist							
			MC	34	4.5 >4.5			83%	
10		CLAYEY SANDSTONE: reddish brown, friable, very deeply weathered, low hardness, fine-medium grained, moist	SPT	53				100%	
15		at 15': increased fine sand and silt content	SPT	81/10"				88%	

Bottom of borehole at 16.3 feet.
1. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.
2. Blow counts shown here for MC samples have been adjusted to SPT values by multiplying field blow counts by a factor of 0.63.
3. Ground surface elevation taken from county-provided LiDAR data (NAVD88 datum).
4. Groundwater was not encountered during drilling; hole was backfilled immediately after drilling.

BORING NUMBER B-2

A3GEO

A3GEO, Inc.
1331 7th Street; Unit E
Berkeley, CA 94710
Telephone: 510-705-1664

CLIENT San Rafael City School District
PROJECT NAME Terra Linda High School - Preliminary Investigation
PROJECT NUMBER 1150-1A
PROJECT LOCATION San Rafael, CA
DATE STARTED 2/22/17 **COMPLETED** 2/22/17
GROUND ELEVATION 81.2 ft **HOLE SIZE** 6"
DRILLING CONTRACTOR Gregg Drilling and Testing, Inc.
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING ---
LOGGED BY RES **CHECKED BY** WM
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	GRAPHIC LOG	MATERIAL DESCRIPTION	SAMPLE TYPE	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	% RECOVERED	OTHER LAB TESTS / NOTES
0		ASPHALTIC CONCRETE [3"]							
		AGGREGATE BASE [6"]							
		LEAN CLAY WITH SAND (CL): reddish brown, medium stiff, moderate plasticity, fine-medium sand, moist	GB				14		LL = 37 PI = 20
		SANDY LEAN CLAY (CL): reddish brown and dark grey, very stiff, low-moderate plasticity, fine-medium sand, heavy iron staining, moist	MC	20	>4.5 4.5	111	21	94%	1% Gravel 36% Sand -200 = 63%
5									
		at 7': dark olive brown and reddish brown with grey streaks - decreasing clay content with depth	MC	23				83%	
10		CLAYEY SAND (SC): yellowish brown, dense to very dense, well-graded, primarily fine-medium sand, low plasticity fines, moist at 12': very dense	MC	50/5"	3.5 >4.5			100%	
		SHALE: light olive brown, friable-weak, deeply weathered, low hardness, papery bedding, dry	SPT	50/5"				100%	

Bottom of borehole at 13.3 feet.
1. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.
2. Blow counts shown here for MC samples have been adjusted to SPT values by multiplying field blow counts by a factor of 0.63.
3. Ground surface elevation taken from county-provided LiDAR data (NAVD88 datum).
4. Groundwater was not encountered during drilling; hole was backfilled immediately after drilling.

A3GEO, Inc.
1331 7th Street; Unit E
Berkeley, CA 94710
Telephone: 510-705-1664

BORING NUMBER B-3

CLIENT San Rafael City School District
PROJECT NAME Terra Linda High School - Preliminary Investigation
PROJECT NUMBER 1150-1A
PROJECT LOCATION San Rafael, CA
DATE STARTED 2/22/17 **COMPLETED** 2/22/17
GROUND ELEVATION 81 ft **HOLE SIZE** 6"
DRILLING CONTRACTOR Gregg Drilling and Testing, Inc.
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING ---
LOGGED BY RES **CHECKED BY** WM
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	% RECOVERED	OTHER LAB TESTS / NOTES
0	ASPHALTIC CONCRETE [4"]							
	AGGREGATE BASE [3"]							
	LEAN CLAY WITH SAND (CL): olive brown and reddish brown, medium stiff, low-moderate plasticity, fine-medium sand, moist	GB						
	CLAYEY SAND (SC): olive brown, medium dense, low-moderate plasticity, fine-medium sand, some iron staining, some fine-coarse gravel, moist	MC	19		124	14	94%	LL = 37 PI = 21 14% Gravel 40% Sand -200 = 46%
5								
		MC	19	3.0			78%	
	CLAYSTONE: soft to low hardness, friable, deeply weathered, moist							
10	SHALE: dark olive brown, friable to weak, deeply weathered, low hardness, papery to platy bedding, dry	SPT	34				100%	
15		SPT	52				78%	
20		SPT	50/6"				100%	

Bottom of borehole at 21.0 feet.
1. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.
2. Blow counts shown here for MC samples have been adjusted to SPT values by multiplying field blow counts by a factor of 0.63.
3. Ground surface elevation taken from county-provided LiDAR data (NAVD88 datum).
4. Groundwater was not encountered during drilling; hole was backfilled immediately after drilling.

A3GEO, Inc.
1331 7th Street; Unit E
Berkeley, CA 94710
Telephone: 510-705-1664

BORING NUMBER B-4

CLIENT San Rafael City School District
PROJECT NAME Terra Linda High School - Preliminary Investigation
PROJECT NUMBER 1150-1A
PROJECT LOCATION San Rafael, CA
DATE STARTED 2/22/17 **COMPLETED** 2/22/17
GROUND ELEVATION 81.1 ft **HOLE SIZE** 6"
DRILLING CONTRACTOR Gregg Drilling and Testing, Inc.
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
▽ **AT TIME OF DRILLING** 20.00 ft / Elev 61.10 ft
LOGGED BY RES **CHECKED BY** WM
AT END OF DRILLING ---
NOTES
▼ **AFTER DRILLING** 10.00 ft / Elev 71.10 ft

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	% RECOVERED	OTHER LAB TESTS / NOTES
0	ASPHALTIC CONCRETE [5"]							
	AGGREGATE BASE [6"]							
	SANDY LEAN CLAY (CL): very dark brown to black with brown sand, medium stiff, low-moderate plasticity, primarily fine-medium sand, some construction debris (nail, staple), moist [FILL]	GB						
	LEAN CLAY WITH SAND (CL): dark grey to black with light grey streaks, medium stiff, low-moderate plasticity, fine-medium sand, some gravel, up to 2", moist (FILL)	MC	7	1.0 1.25	104	22		LL = 36 PI = 16
5								
	at 6.5': stiff, dark olive brown with some iron staining	MC	9	0.75 0.75				
	at 7': some subangular gravel in shoe of sampler, up to 1"							
10	▼							
	LEAN CLAY WITH SAND (CL): yellowish brown with dark brown streaks, stiff, some iron staining, moist	MC	18	>4.5 >4.5	111	18		
15	SANDY LEAN CLAY (CL): reddish brown and olive brown, some grey streaks, very stiff, primarily fine-medium sand, high sand content, moist	MC	19	4.25 >4.5				
	SANDSTONE: reddish brown and olive brown, friable-weak, deeply weathered, low hardness, crushed, dry							
20	▽	MC	50/3"					
		SPT	50/3"					

Bottom of borehole at 21.0 feet.
1. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.
2. Blow counts shown here for MC samples have been adjusted to SPT values by multiplying field blow counts by a factor of 0.63.
3. Ground surface elevation taken from county-provided LiDAR data (NAVD88 datum).
4. See report for discussion regarding groundwater; hole backfilled shortly after drilling complete.

A3GEO, Inc.
1331 7th Street; Unit E
Berkeley, CA 94710
Telephone: 510-705-1664

BORING NUMBER B-5

CLIENT San Rafael City School District
PROJECT NAME Terra Linda High School - Preliminary Investigation
PROJECT NUMBER 1150-1A
PROJECT LOCATION San Rafael, CA
DATE STARTED 2/22/17 **COMPLETED** 2/22/17
GROUND ELEVATION 91.4 ft **HOLE SIZE** 6"
DRILLING CONTRACTOR Gregg Drilling and Testing, Inc.
GROUND WATER LEVELS:
DRILLING METHOD Hollow Stem Auger
AT TIME OF DRILLING ---
LOGGED BY RES **CHECKED BY** WM
AT END OF DRILLING ---
NOTES
AFTER DRILLING ---

DEPTH (ft)	MATERIAL DESCRIPTION	SAMPLE TYPE	ADJUSTED BLOW COUNTS (N VALUE)	POCKET PEN. (tsf)	DRY UNIT WT. (pcf)	MOISTURE CONTENT (%)	% RECOVERED	OTHER LAB TESTS / NOTES
0	SANDY SILT (ML): dark brown, soft, low plasticity, fine-medium sand, moist							
		GB						
	at 3': medium stiff	MC	6	1.5 1.5	107	21		LL = 36 PI = 16
5	SANDY LEAN CLAY (CL): dark olive brown with olive brown and reddish brown spots, stiff, low-moderate plasticity, fine-medium sand, trace coarse sand, moist							
		MC	10	1.5 2.0	110	19		
	- increasing sand content with depth							
10	at 10': very stiff, increased sand content	MC	14	>4.5 >4.5 >4.5				
15	at 15': very stiff, slightly more sand	MC	24	>4.5 >4.5 >4.5				
	SHALE: dark brown, soft-weak, very deeply weathered, soft-low hardness, papery to platey bedding, crushed, some iron staining, damp							
20		SPT	50/5"					

Bottom of borehole at 20.4 feet.
1. Stratification lines represent the approximate boundaries between material types and the transitions may be gradual.
2. Blow counts shown here for MC samples have been adjusted to SPT values by multiplying field blow counts by a factor of 0.63.
3. Ground surface elevation taken from county-provided LiDAR data (NAVD88 datum).
4. Groundwater was not encountered during drilling; hole was backfilled immediately after drilling.

APPENDIX D

Laboratory Test Data
(this study)

B. HILLEBRANDT SOILS TESTING, INC.

29 Sugarloaf Terrace, Alamo, CA 94507 - Tel: (510) 409-2916 - Fax: (925) 891-9267 - Email: soiltesting@aol.com

LAB RESULTS SUMMARY FORM

Project Number: 1150-1A **Project Name:** Terra Linda High School **Results Due By:**
Requested By: DM **Request Date:** 2/28/17 **Throw Samples Out On:**

					Atterberg			-200			Compaction				
Boring #	Sample Depth (feet)	Dry Density (pcf)	Moisture Content (%)	Unconfined Compressive (ksf)	Liquid Limit	Plastic Limit	Plasicity Index	Passing #4 Sieve (%)	Passing #40 sieve (%)	Passing #200 sieve (%)	Maximum Dry Density (pcf)	Optimum Moisture (%)	Pocket Penetrometer (tsf)	Torvane (tsf)	Remarks
B-1	4.0	121	14.1					68	48	29					
B-2	1.0		14.1		37	17	20								
B-2	4.0	111	20.7					99	89	63					
B-3A	4.0	124	14.4		37	16	21	86	68	46					
B-4	3.5	104	21.9		36	20	16								
B-4	10.5	111	18.1												
B-5	4.0	107	20.8		36	20	16								
B-5	6.5	110	18.8												

B. HILLEBRANDT SOILS TESTING, INC.

29 Sugarloaf Terrace, Alamo, CA 94507 - Tel: (510) 409-2916 - Fax: (925) 891-9267 - Email: soiltesting@aol.com

MOISTURE CONTENT/DRY DENSITY

Job #: 1150-1A
Job Name: Terra Linda High School
Date: 2/28/17
Tested by: Brad Hillebrandt

Additional Tests:	FS	-200	PI, -200	PI		PI
Boring #:	B-1	B-2	B-3A	B-4	B-4	B-5
Depth:	4.0	4.0	4.0	3.5	10.5	4.0
Sample Description:	Yellowish brown clayey SAND with gravel	Dark yellowish brown and olive brown sandy CLAY	Dark yellowish brown and olive brown sandy CLAY with some gravel	Very dark gray lean CLAY with sand	Olive brown sandy CLAY	Dark brown sandy CLAY
Can #:	202	348	361	327	313	502
Wet Sample + can	1079.4	286.8	440.4	344.4	347.5	327.2
Dry Sample + can	979.2	244.2	389.4	289.3	300.2	276.7
Weight can	271.0	38.1	34.0	38.2	39.5	33.7
Weight water	100.2	42.6	51	55.1	47.3	50.5
Weight Dry Sample	708.2	206.1	355.4	251.1	260.7	243
WATER CONTENT (%)	**14.1%**	**20.7%**	**14.4%**	**21.9%**	**18.1%**	**20.8%**
Weight Sample + Liner	1221.8	1167.1	1210.5	1163.3	1127.7	1190.3
Weight Liner	249.4	254.5	211.1	274.2	262.2	274.6
Sample Length	6.0	5.8	6.0	5.95	5.6	6.0
Sample Diameter	2.39	2.39	2.39	2.39	2.39	2.39
DRY DENSITY (pcf)	**120.6**	**110.7**	**123.7**	**104.1**	**111.1**	**107.3**

B. HILLEBRANDT SOILS TESTING, INC.

29 Sugarloaf Terrace, Alamo, CA 94507 - Tel: (510) 409-2916 - Fax: (925) 891-9267 - Email: soiltesting@aol.com

MOISTURE CONTENT/DRY DENSITY

Job #: 1150-1A
Job Name: Terra Linda High School
Date: 2/28/17
Tested by: Brad Hillebrandt

Additional Tests:						
Boring #:	B-5					
Depth:	6.5					
Sample Description:	Dark brown sandy CLAY					
Can #:	363					
Wet Sample + can	331.7					
Dry Sample + can	284.5					
Weight can	33.6					
Weight water	47.2					
Weight Dry Sample	250.9					
WATER CONTENT (%)	**18.8%**					
Weight Sample + Liner	1180.9					
Weight Liner	259.2					
Sample Length	6.0					
Sample Diameter	2.39	2.39	2.39	2.39	2.39	
DRY DENSITY (pcf)	**109.8**	**#DIV/0!**	**#DIV/0!**	**#DIV/0!**	**#DIV/0!**	

B. HILLEBRANDT SOILS TESTING, INC.

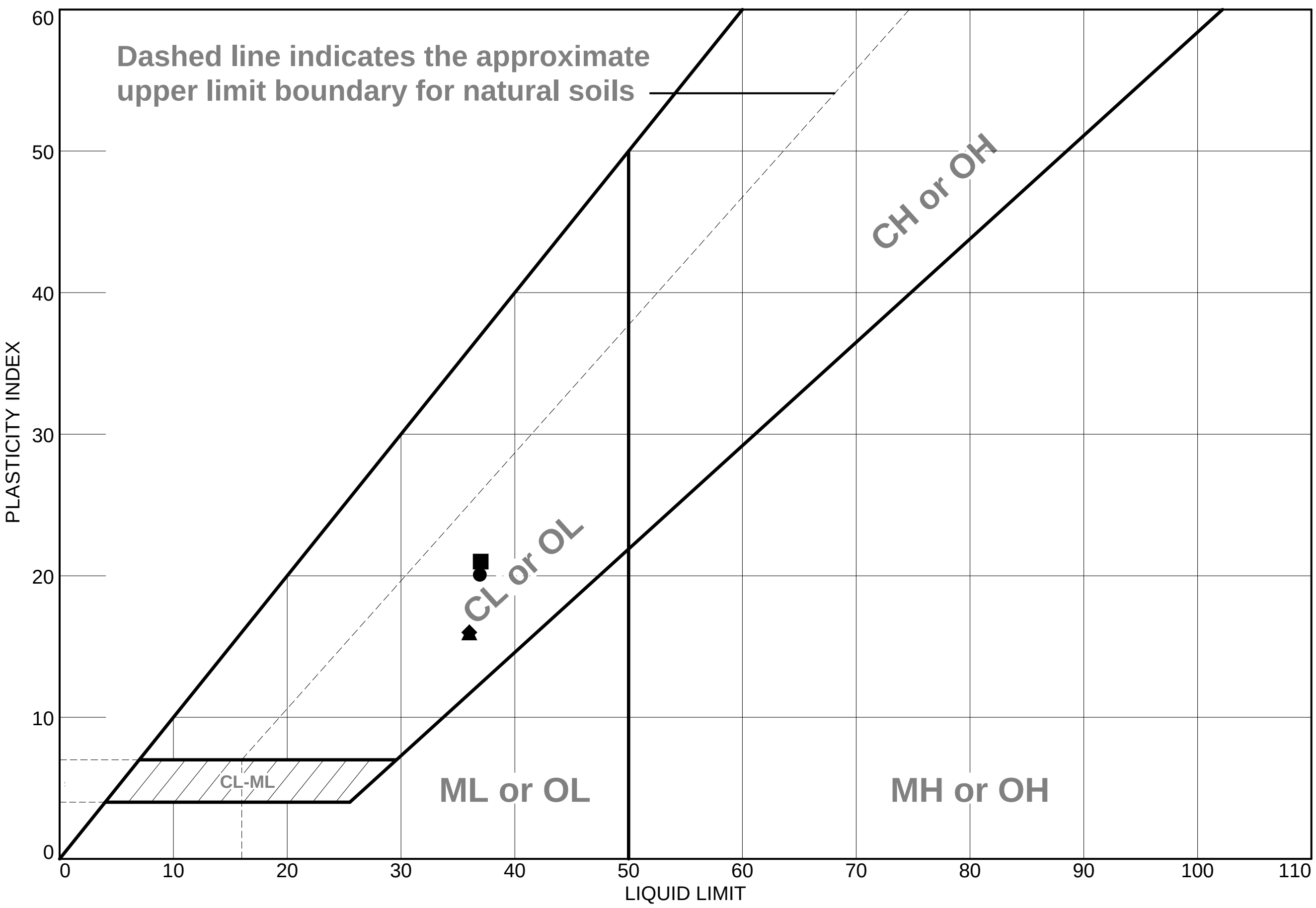
29 Sugarloaf Terrace, Alamo, CA 94507 - Tel: (510) 409-2916 - Fax: (925) 891-9267 - Email: soiltesting@aol.com

MOISTURE CONTENT WORKSHEET

Job #: 1150-1A
Job Name: Terra Linda High School
Date: 2/28/17
Tested by: B. Hillebrandt

Additional Tests:	PI								
Boring #:	B-2								
Depth:	1.0								
Sample Description:	Dark yellowish brown clayey SAND with some gravel								
Can #:	501								
Wet Sample + can	349.4								
Dry Sample + can	310.4								
Weight can	34.0								
Weight water	39								
Weight Dry Sample	276.4								
WATER CONTENT (%)	**14.1%**								

LIQUID AND PLASTIC LIMITS TEST REPORT



	MATERIAL DESCRIPTION	LL	PL	PI	%<#40	%<#200	USCS
●	Dark yellowish brown clayey SAND with some gravel	37	17	20			
■	Dark yellowish brown and olive brown sandy CLAY with some gravel	37	16	21	67.7	45.5	SC
▲	Very dark gray lean CLAY with sand	36	20	16			
◆	Dark brown sandy CLAY	36	20	16			

Project No. 1150-1A **Client:** A3Geo

Project: Terra Linda High School

●**Source of Sample:** B-2 **Depth:** 1.0 - 2.0'
■**Source of Sample:** B-3A **Depth:** 4.0'
▲**Source of Sample:** B-4 **Depth:** 3.5'
◆**Source of Sample:** B-5 **Depth:** 4.0'

Remarks:

B. HILLEBRANDT SOILS TESTING, INC.
+1 510-409-2816
SoilTesting@aol.com

Figure

Tested By: BH

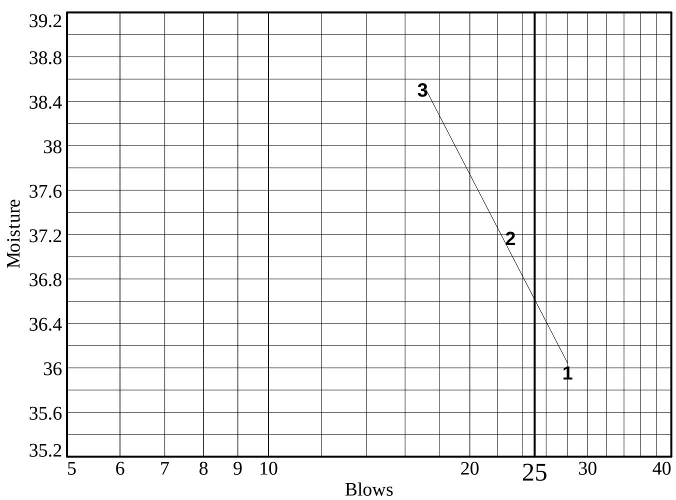
LIQUID AND PLASTIC LIMIT TEST DATA

3/3/2017

Client: A3Geo
Project: Terra Linda High School
Project Number: 1150-1A
Location: B-2
Depth: 1.0 - 2.0'
Material Description: Dark yellowish brown clayey SAND with some gravel
Tested by: BH

Liquid Limit Data

Run No.	1	2	3	4	5	6
Wet+Tare	30.18	30.26	27.05			
Dry+Tare	25.16	25.13	22.66			
Tare	11.20	11.33	11.26			
# Blows	28	23	17			
Moisture	36.0	37.2	38.5			



Liquid Limit= 37
Plastic Limit= 17
Plasticity Index= 20
Natural Moisture= 14.1
Liquidity Index= -0.1

Plastic Limit Data

Run No.	1	2	3	4
Wet+Tare	17.59	17.36		
Dry+Tare	16.67	16.47		
Tare	11.12	11.31		
Moisture	16.6	17.2		

Natural Moisture Data

Wet+Tare	Dry+Tare	Tare	Moisture
349.4	310.4	34.0	14.1

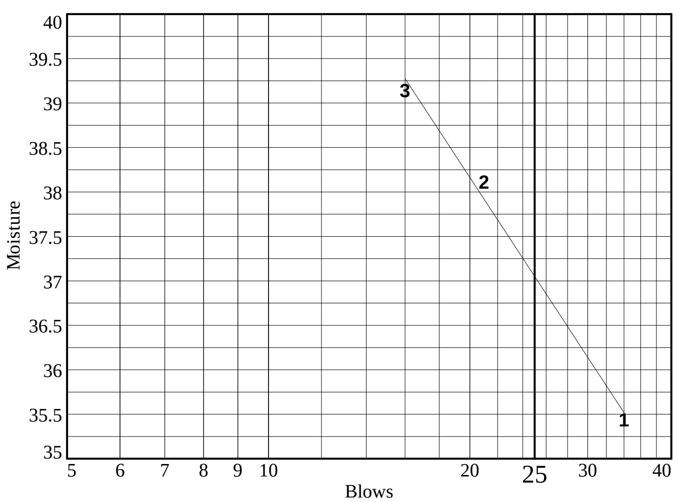
LIQUID AND PLASTIC LIMIT TEST DATA

3/3/2017

Client: A3Geo
Project: Terra Linda High School
Project Number: 1150-1A
Location: B-3A
Depth: 4.0'
Material Description: Dark yellowish brown and olive brown sandy CLAY with some gravel
%<#40: 67.7 **%<#200:** 45.5 **USCS:** SC **AASHTO:** A-6(5)
Tested by: BH

Liquid Limit Data

Run No.	1	2	3	4	5	6
Wet+Tare	26.87	30.50	27.73			
Dry+Tare	22.79	25.19	23.04			
Tare	11.28	11.26	11.06			
# Blows	34	21	16			
Moisture	35.4	38.1	39.1			



Liquid Limit= 37
Plastic Limit= 16
Plasticity Index= 21
Natural Moisture= 14.4
Liquidity Index= -0.1

Plastic Limit Data

Run No.	1	2	3	4
Wet+Tare	18.27	17.21		
Dry+Tare	17.3	16.36		
Tare	11.08	11.26		
Moisture	15.6	16.7		

Natural Moisture Data

Wet+Tare	Dry+Tare	Tare	Moisture
440.4	389.4	34	14.4

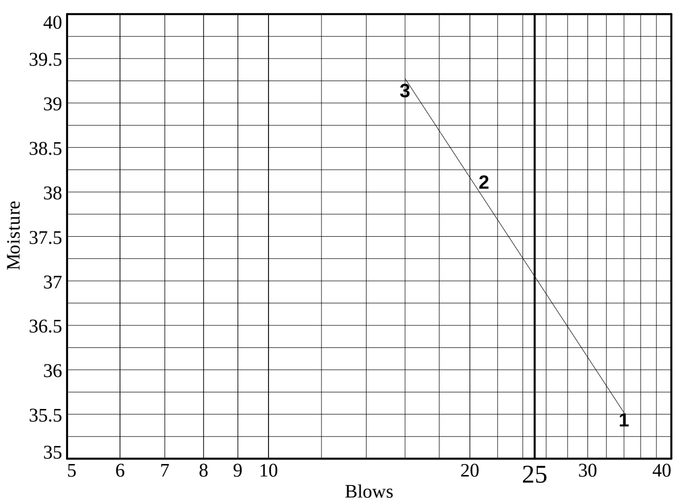
LIQUID AND PLASTIC LIMIT TEST DATA

3/3/2017

Client: A3Geo
Project: Terra Linda High School
Project Number: 1150-1A
Location: B-3A
Depth: 4.0'
Material Description: Dark yellowish brown and olive brown sandy CLAY with some gravel
%<#40: 67.7 **%<#200:** 45.5 **USCS:** SC **AASHTO:** A-6(5)
Tested by: BH

Liquid Limit Data

Run No.	1	2	3	4	5	6
Wet+Tare	26.87	30.50	27.73			
Dry+Tare	22.79	25.19	23.04			
Tare	11.28	11.26	11.06			
# Blows	34	21	16			
Moisture	35.4	38.1	39.1			



Liquid Limit= 37
Plastic Limit= 16
Plasticity Index= 21
Natural Moisture= 14.4
Liquidity Index= -0.1

Plastic Limit Data

Run No.	1	2	3	4
Wet+Tare	18.27	17.21		
Dry+Tare	17.3	16.36		
Tare	11.08	11.26		
Moisture	15.6	16.7		

Natural Moisture Data

Wet+Tare	Dry+Tare	Tare	Moisture
440.4	389.4	34	14.4

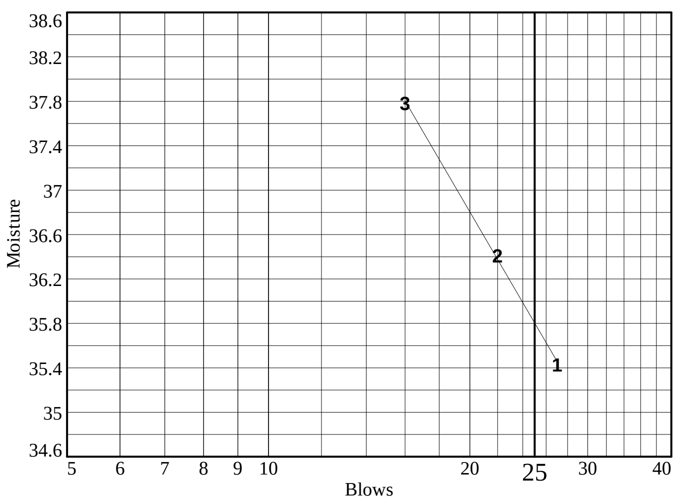
LIQUID AND PLASTIC LIMIT TEST DATA

3/3/2017

Client: A3Geo
Project: Terra Linda High School
Project Number: 1150-1A
Location: B-4
Depth: 3.5'
Material Description: Very dark gray lean CLAY with sand
Tested by: BH

Liquid Limit Data

Run No.	1	2	3	4	5	6
Wet+Tare	29.99	27.58	28.61			
Dry+Tare	25.10	23.17	23.86			
Tare	11.30	11.06	11.29			
# Blows	27	22	16			
Moisture	35.4	36.4	37.8			



Liquid Limit= 36
Plastic Limit= 20
Plasticity Index= 16
Natural Moisture= 21.9
Liquidity Index= 0.1

Plastic Limit Data

Run No.	1	2	3	4
Wet+Tare	17.48	17.05		
Dry+Tare	16.43	16.06		
Tare	11.21	11.27		
Moisture	20.1	20.7		

Natural Moisture Data

Wet+Tare	Dry+Tare	Tare	Moisture
344.4	289.3	38.2	21.9

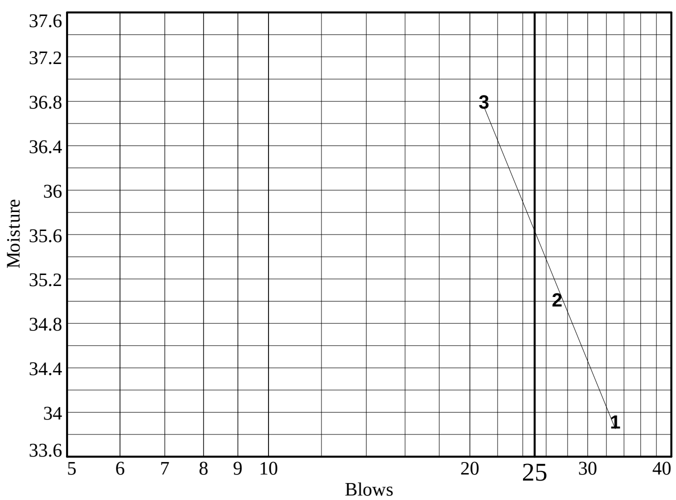
LIQUID AND PLASTIC LIMIT TEST DATA

3/3/2017

Client: A3Geo
Project: Terra Linda High School
Project Number: 1150-1A
Location: B-5
Depth: 4.0'
Material Description: Dark brown sandy CLAY
Tested by: BH

Liquid Limit Data

Run No.	1	2	3	4	5	6
Wet+Tare	24.05	26.27	29.78			
Dry+Tare	20.76	22.39	24.76			
Tare	11.06	11.31	11.12			
# Blows	33	27	21			
Moisture	33.9	35.0	36.8			



Liquid Limit= 36
Plastic Limit= 20
Plasticity Index= 16
Natural Moisture= 20.8
Liquidity Index= 0.1

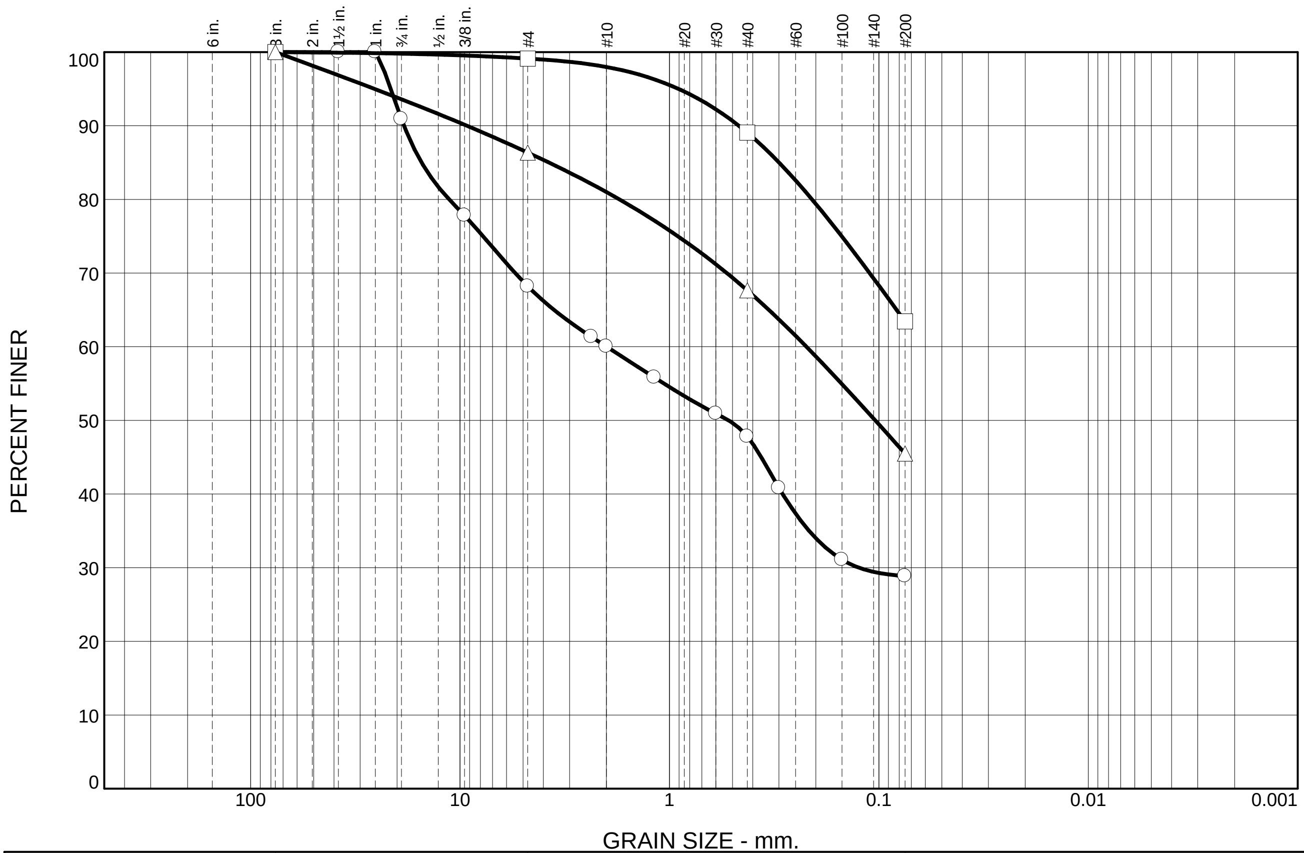
Plastic Limit Data

Run No.	1	2	3	4
Wet+Tare	17.27	17.79		
Dry+Tare	16.28	16.74		
Tare	11.33	11.28		
Moisture	20.0	19.2		

Natural Moisture Data

Wet+Tare	Dry+Tare	Tare	Moisture
327.2	276.7	33.7	20.8

Particle Size Distribution Report



% +3"	% Gravel		% Sand			% Fines	
	Coarse	Fine	Coarse	Medium	Fine	Silt	Clay

MATERIAL DATA

SYMBOL	SOURCE	SAMPLE NO.	DEPTH (ft.)	Material Description	USCS
○	B-1		4.0'	Yelowish brown clayey SAND with gravel	
□	B-2		4.0'	Dark yellowish brown and olive brown sandy CLAY	
△	B-3A		4.0'	Dark yellowish brown and olive brown sandy CLAY with some gravel	SC

B. HILLEBRANDT SOILS TESTING, INC.
+1 510-409-2816
SoilTesting@aol.com

Client: A3Geo
Project: Terra Linda High School

Project No.: 1150-1A **Figure**

Tested By: BH

GRAIN SIZE DISTRIBUTION TEST DATA

3/3/2017

Client: A3Geo
Project: Terra Linda High School
Project Number: 1150-1A
Location: B-1
Depth: 4.0'
Material Description: Yelowish brown clayey SAND with gravel
Tested by: BH

Sieve Test Data

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
979.20	271.00	0.00	3"	0.00	100.0
			1.5"	0.00	100.0
			1"	0.00	100.0
			3/4"	64.37	90.9
			3/8"	157.12	77.8
			#4	225.16	68.2
			#8	273.74	61.3
			#10	283.08	60.0
			#16	312.84	55.8
			#30	347.62	50.9
			#40	369.53	47.8
			#50	419.08	40.8
			#100	487.98	31.1
			#200	503.84	28.9

Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	9.1	22.7	31.8	8.2	12.2	18.9	39.3			28.9

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
				0.1246	0.2881	0.5221	1.9929	11.2655	15.2616	18.5168	21.4103

Fineness Modulus
3.23

GRAIN SIZE DISTRIBUTION TEST DATA

3/3/2017

Client: A3Geo
Project: Terra Linda High School
Project Number: 1150-1A
Location: B-2
Depth: 4.0'
Material Description: Dark yellowish brown and olive brown sandy CLAY
Tested by: BH

Sieve Test Data

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
244.20	38.10	0.00	3"	0.00	100.0
			#4	1.81	99.1
			#40	22.53	89.1
			#200	75.34	63.4

Fractional Components

Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	0.2	0.7	0.9	1.1	8.9	25.7	35.7			63.4

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
								0.2085	0.2997	0.4662	0.9050

Fineness Modulus
0.55

GRAIN SIZE DISTRIBUTION TEST DATA

3/3/2017

Client: A3Geo
Project: Terra Linda High School
Project Number: 1150-1A
Location: B-3A
Depth: 4.0'
Material Description: Dark yellowish brown and olive brown sandy CLAY with some gravel
USCS: SC
Tested by: BH

Sieve Test Data

Dry Sample and Tare (grams)	Tare (grams)	Cumulative Pan Tare Weight (grams)	Sieve Opening Size	Cumulative Weight Retained (grams)	Percent Finer
389.40	34.00	0.00	3"	0.00	100.0
			#4	48.48	86.4
			#40	114.96	67.7
			#200	193.68	45.5

Fractional Components

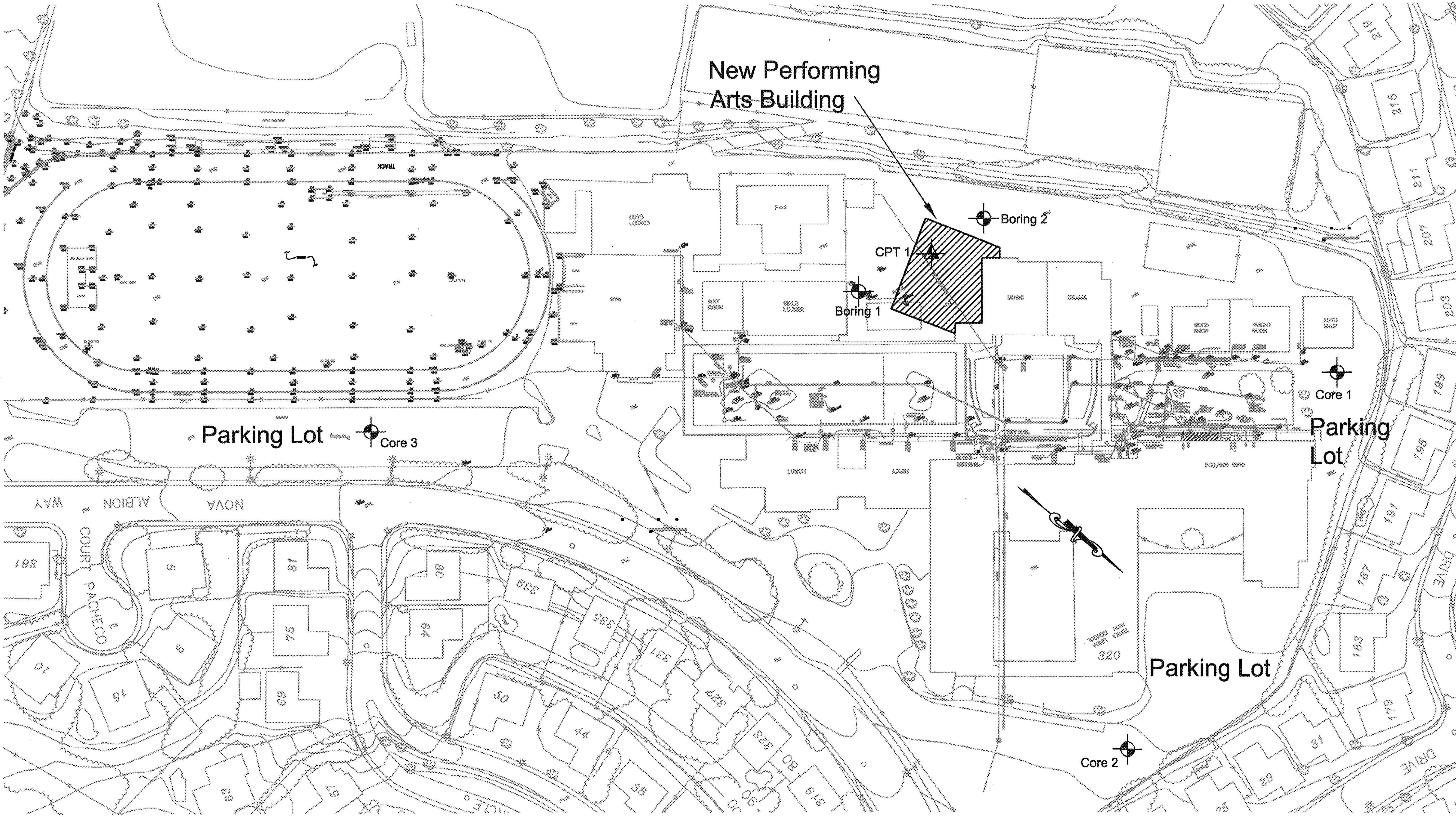
Cobbles	Gravel			Sand				Fines		
	Coarse	Fine	Total	Coarse	Medium	Fine	Total	Silt	Clay	Total
0.0	6.4	7.2	13.6	5.4	13.3	22.2	40.9			45.5

D_5	D_{10}	D_{15}	D_{20}	D_{30}	D_{40}	D_{50}	D_{60}	D_{80}	D_{85}	D_{90}	D_{95}
						0.1041	0.2219	1.7339	3.7597	9.2747	25.5894

Fineness Modulus
1.84

APPENDIX E

Site Plan and Data from Previous Investigation
(MPEG, 2003)



Boring Or Core By MPEG, August 2003

Boring By MPEG, August 2003

SCALE

0 50 100 200 FEET

FILE: 779.12/terralindasiteplan.dwg

Miller Pacific
ENGINEERING GROUP

SITE PLAN
San Rafael Schools - Terra Linda High
San Rafael, California

2

Project No. 779.12 | Date 09/03/03 | Approved By: | Figure

APPENDIX A

SUBSURFACE EXPLORATION AND LABORATORY TESTING

1.0 Subsurface Exploration - Borings

We explored subsurface conditions at the site by drilling 2 test borings and 3 pavement cores on August 14, 2003 at the locations shown on Figure 2. Test borings were drilled to depths between 16 and 25.5 feet using a hollow-stem auger with a diameter of 8 inches. The pavement cores were shallow and terminated in natural soils below the existing pavement section.

The soils encountered were logged and identified by our Engineer in general accordance with ASTM Standard D 2487, "Field Identification and Description of Soils (Visual-Manual Procedure)." This standard is briefly explained on Figure A-1, Soil Classification Chart and Key to Log Symbols and Figure A-2 Rock Classification Chart. The Boring Logs are presented on Figures A-3 to A-5.

We obtained "undisturbed" samples using a 3-inch diameter, split-barrel modified California sampler with 2.5 by 6-inch brass tube liners or with a 2-inch diameter, split-barrel Standard Penetration Test (SPT) sampler. The sampler was driven with a 140-pound hammer falling 30 inches. The number of blows required to drive the samplers 18 inches was recorded and is reported on the boring logs as blows per foot for the last 12 inches of driving. The samples obtained were examined in the field, sealed to prevent moisture loss, and transported to our laboratory.

2.0 Subsurface Exploration – Cone Penetration Testing

The Cone Penetration Test (CPT) is a special exploration technique that provides a continuous profile of data throughout the depth of exploration. It is particularly useful in defining stratigraphy, relative soil strength and in assessing liquefaction potential. We performed 1 CPT on August 20, 2003 at the locations shown on the Site Plan, Figure 2. The CPT equipment was mounted in a large rubber-tired van.

The CPT is a cylindrical probe, 35 mm in diameter, which is pushed into the ground at a constant rate of 2 cm/sec. The device is illustrated on Figure A-6. It is instrumented to obtain continuous measurements of cone bearing (tip resistance), sleeve friction and pore water pressure. The data is sensed by strain gages and load cells inside the instrument. Electronic signals from the instrument are continuously recorded by an on-board computer at the surface, which permits an initial evaluation of subsurface conditions during the exploration.

The recorded data is transferred to an in-office computer for reduction and analysis. The analysis of cone bearing and sleeve friction (i.e. friction ratio) indicates the soil type, the cone bearing alone indicates soil density or strength, and the pore pressure indicates the presence of clay. Variations in the data profile indicate changes in stratigraphy. This test method has been standardized and is described in detail by the ASTM Standard Test Method D3441 "Deep, Quasi-Static Cone and Friction Cone Penetration Tests of Soil." The interpretation of CPT data is illustrated on Figure A-7, and the CPT data log is presented on Figure A-8.

3.0 Laboratory Testing

We conducted laboratory tests on selected intact samples to verify field identifications and to evaluate engineering properties. The following laboratory tests were conducted in accordance with the ASTM standard test method cited:

- Laboratory Determination of Water (Moisture Content) of Soil, Rock, and Soil-Aggregate Mixtures, ASTM D 2216;
- Density of Soil in Place by the Drive-Cylinder Method, ASTM D 2937;
- Unconfined Compressive Strength of Cohesive Soil, ASTM D 2166;
- Liquid Limit, Plastic Limit, and Plasticity Index of Soils, ASTM D 4318;
- Resistance (R)-value/expansion pressure of untreated laboratory compacted soils/aggregates, D 2844.

The moisture content, dry density, and unconfined compression test results are shown on the exploratory Boring Logs, Figures A-3 through A-5. The R-value test is summarized on Figure A-9 and the Plasticity Index Test is summarized on Figure A-10.

The exploratory boring logs, description of soils encountered and the laboratory test data reflect conditions only at the location of the boring at the time they were excavated or retrieved. Conditions may differ at other locations and may change with the passage of time due to a variety of causes including natural weathering, climate and changes in surface and subsurface drainage.

SOIL CLASSIFICATION CHART

MAJOR DIVISIONS		SYMBOL	DESCRIPTION
COARSE GRAINED SOILS over 50% sand and gravel	CLEAN GRAVEL	GW	Well-graded gravels or gravel-sand mixtures, little or no fines
		GP	Poorly-graded gravels or gravel-sand mixtures, little or no fines
	GRAVEL with fines	GM	Silty gravels, gravel-sand-silt mixtures
		GC	Clayey gravels, gravel-sand-clay mixtures
	CLEAN SAND	SW	Well-graded sands or gravely sands, little or no fines
		SP	Poorly-graded sands or gravely sands, little or no fines
	SAND with fines	SM	Silty sands, sand-silt mixtures
		SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS over 50% silt and clay	SILT AND CLAY liquid limit <50%	ML	Inorganic silts and very fine sands, rock flour, silty or clayey fine sands or clayey silts with slight plasticity
		CL	Inorganic clays of low to medium plasticity, gravely clays, sandy clays, silty clays, lean clays
		OL	Organic silts and organic silt-clays of low plasticity
	SILT AND CLAY liquid limit >50%	MH	Inorganic silts, micaceous or diatomaceous fine sands or silts, elastic silts
		CH	Inorganic clays of high plasticity, fat clays
		OH	Organic clays of medium to high plasticity
HIGHLY ORGANIC SOILS		PT	Peat, muck, and other highly organic soils
ROCK			Undifferentiated as to type or composition

KEY TO BORING AND TEST PIT SYMBOLS

CLASSIFICATION TESTS

- AL ATTERBERG LIMITS TEST
- SA SIEVE ANALYSIS
- HYD HYDROMETER ANALYSIS
- P200 PERCENT PASSING NO. 200 SIEVE
- P4 PERCENT PASSING NO. 4 SIEVE

STRENGTH TESTS

- TV FIELD TORVANE (UNDRAINED SHEAR)
- UC LABORATORY UNCONFINED COMPRESSION
- TXCU CONSOLIDATED UNDRAINED TRIAXIAL
- TXUU UNCONSOLIDATED UNDRAINED TRIAXIAL

UC, CU, UU = 1/2 Deviator Stress

SAMPLER TYPE

UNDISTURBED CORE SAMPLE: MODIFIED CALIFORNIA OR HYDRAULIC PISTON SAMPLE

STANDARD PENETRATION TEST SAMPLE

X DISTURBED OR BULK SAMPLE

ROCK OR CORE SAMPLE

NOTE: Test boring and test pit logs are an interpretation of conditions encountered at the location and time of exploration. Subsurface rock, soil and water conditions may differ in locations and with the passage of time. Lines defining interface between differing soil or rock description are approximate and may indicate a gradual transition.

FILE: Soil Class 779-12.dwg

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SOIL CLASSIFICATION CHART
San Rafael City Schools - Terra Linda High
San Rafael, California

A-1

Project No. 779.12 Date 09/03/03 Approved By: [signature]

Figure

FRACTURING AND BEDDING

Fracture Classification	Spacing	Bedding Classification
Crushed	less than 3/4 inch	Laminated
Intensely fractured	3/4 to 2-1/2 inches	Very thinly bedded
Closely fractured	2-1/2 to 8 inches	Thinly bedded
Moderately fractured	8 to 24 inches	Medium bedded
Widely fractured	2 to 6 feet	Thickly bedded
Very widely fractured	greater than 6 feet	Very thickly bedded

HARDNESS

Low	Carved or gouged with a knife
Moderate	Easily scratched with a knife, friable
Hard	Difficult to scratch, knife scratch leaves dust trace
Very hard	Rock scratches metal

STRENGTH

Friable	Crumbles by rubbing with fingers
Weak	Crumbles under light hammer blows
Moderate	Indentations <1/8 inch with moderate blow with pick end of rock hammer
Strong	Withstands few heavy hammer blows, yields large fragments
Very strong	Withstands many heavy hammer blows, yields dust, small fragments

WEATHERING

Complete	Minerals decomposed to soil, but fabric and structure preserved
High	Rock decomposition, thorough discoloration, all fractures are extensively coated with clay, oxides or carbonates
Moderate	Fracture surfaces coated with weathering minerals, moderate or localized discoloration
Slight	A few stained fractures, slight discoloration, no mineral decomposition, no affect on cementation
Fresh	Rock unaffected by weathering, no change with depth, rings under hammer impact

NOTE: Test boring and test pit logs are an interpretation of conditions encountered at the location and time of exploration. Subsurface rock, soil and water conditions may differ in other locations and with the passage of time.

FILE: Rock Class.dwg

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ROCK CLASSIFICATION CHART
San Rafael City Schools - Terra Linda High
San Rafael, California

A-2

Project No. 779.12 | Date 09/03/03 | Approved By: [signature]

Figure

BORING 1

EQUIPMENT: CME 75 - 8" Hollow Stem Auger
DATE: 8/14/03
ELEVATION: 79.6 feet*
*REFERENCE: City of San Rafael Topo Map used for Elevation

OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DESCRIPTION
					2.5" ASPHALT CONCRETE 2" AGGREGATE BASE
	1600	42	15.2	112.0	SILTY SANDY CLAY (CL) (COLLUVIUM) mottled orange-red to brown, dry, stiff to very stiff, low plasticity, 15% - 20% fine to medium grained sands
	1100	50	12.3	116.0	SILTY SANDY CLAY WITH GRAVEL (CL) light brown to brown, dry, very stiff, low plasticity, 15% - 20% fine to coarse grained sands, 5% - 10% fine to medium graded gravel to 3/4"
	1100	29	14.9	110.0	Grades brown to dark brown
		50/1"	9.9		SANDSTONE (SS) moderately weathered, light brown to brown, fine to medium grained sands

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

FILE: Boring Logs 779-12.dwg

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BORING LOG
San Rafael City Schools - Terra Linda High
San Rafael, California

A-3

Project No. 779.12 Date 09/03/03 Approved By: [signature]

Figure

BORING 1 (CONTINUED)

OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (3)	DESCRIPTION
						20		SS	SANDSTONE (SS) moderately weathered, light brown to brown, fine to medium grained sands
					7			SS	
								SS	grades to slightly weathered, light gray to gray, fine to medium grained sands
		50/0"				25	☒	SS	
									Bottom of boring at 25.3' No groundwater observed during drilling
					8				
					9				
						30			
					10				
						35			
					11				
					12				
						40			

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

FILE: Boring Logs 779-12.dwg

Miller Pacific
ENGINEERING GROUP

BORING LOG
San Rafael City Schools - Terra Linda High
San Rafael, California

A-4

Project No. 779.12 | Date 09/03/03 | Approved By: [signature] | Figure

BORING 2

EQUIPMENT: CME 75 - 8" Hollow Stem Continuous Flight Auger
DATE: 8/14/03
ELEVATION: 79.6 feet*
*REFERENCE: City of San Rafael Topo Map, DPW, 1998

OTHER TEST DATA	UNDRAINED SHEAR STRENGTH psf (1)	BLOWS PER FOOT	MOISTURE CONTENT (%)	DRY UNIT WEIGHT pcf (2)	DEPTH meters / feet	SAMPLE	SYMBOL (3)	DESCRIPTION
					0 – 0			2.5" ASPHALT CONCRETE
								2" AGGREGATE BASE
	1400	34	19.9	103.0	1	■		SILTY SANDY CLAY (CL) (COLLUVIUM) mottled orange-red to brown, dry, stiff to very stiff, low plasticity, 15% - 20% fine to medium grained sands
					5 / 2			
	2200	61	13.8	119.0		■		SILTY SANDY CLAY w/ GRAVEL (CL) light brown to brown, dry, very stiff, low plasticity, 15% - 20% fine to coarse grained sands, 5% - 10% fine to medium graded gravel to 3/4"
					3 / 10			
		50/5"	12.7		4	■		SANDSTONE (SS) moderately weathered, light brown to brown, fine to medium grained sands
					15 / 5			Bottom of boring at 16' No groundwater observed during drilling
					6 / 20			

NOTES: (1) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) GRAPHIC SYMBOLS ARE ILLUSTRATIVE ONLY

FILE: Boring Logs 779-12.dwg

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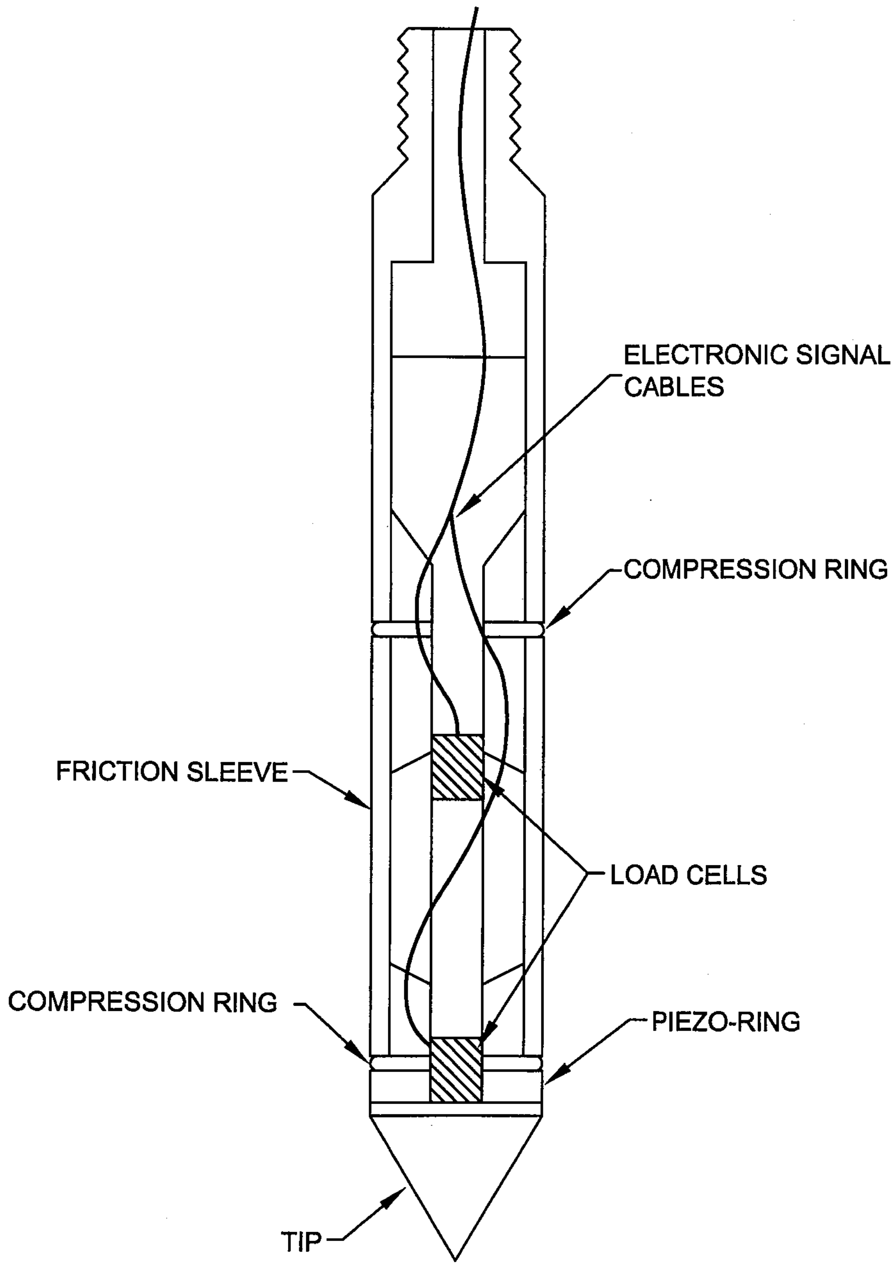
BORING LOG
San Rafael City Schools - Terra Linda High
San Rafael, California

A-5

Project No. 779.12 | Date 09/03/03 | Approved By: [signature] | Figure



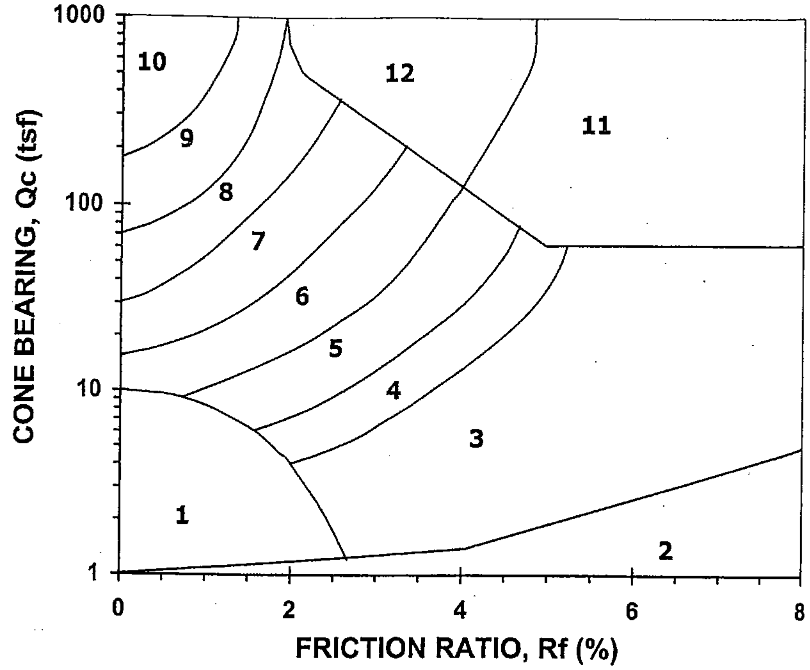
CONE PENETROMETER

(NO SCALE)

FILE: STD-075.779.12TL.dwt

Miller Pacific ENGINEERING GROUP	CONE PENETROMETER San Rafael City Schools - Terra Linda High San Rafael, California		A-6
Project No. 779.12	Date 10/29/03	Approved By: [signature]	Figure

SIMPLIFIED SOIL BEHAVIOR TYPE CLASSIFICATION FOR STANDARD ELECTRONIC CONE PENETROMETER



ZONE	Qc/N[1]	Su Factor (Nk)[2]	SOIL BEHAVIOR TYPE[1]
1	2		Sensitive Fine Grained
2	1	for Zones 1 to 6	Organic Material
3	1	10 for Qc <= 9 tsf	CLAY
4	1.5	12 for Qc = 9 to 12 tsf	Silty CLAY to CLAY
5	2	15 for Qc > 12 tsf	Clayey SILT to Silty CLAY
6	2.5		Sandy SILT to Clayey SILT
7	3	---	Silty SAND to Sandy SILT
8	4	---	SAND to Silty SAND
9	5	---	SAND
10	6	---	Gravelly SAND to SAND
11	1	15	Very Stiff Fine Grained (*)
12	2	---	SAND to Clayey SAND (*)

(*) Overconsolidated or Cemented

Qc = Tip Bearing
Fs = Sleeve Friction
Rf = Fs/Qc*100 = Friction Ratio

References: [1]Robertson, 1986, Olsen, 1988
[2]Bonaparte & Mitchell, 1979 (young bay mud Qc <= 9)
[2]Estimated from local experience (fine grained soils Qc > 9)

Note: Testing performed in accordance with ASTM D3441

John Sarmiento & Associates
Cone Penetrometer Testing Services

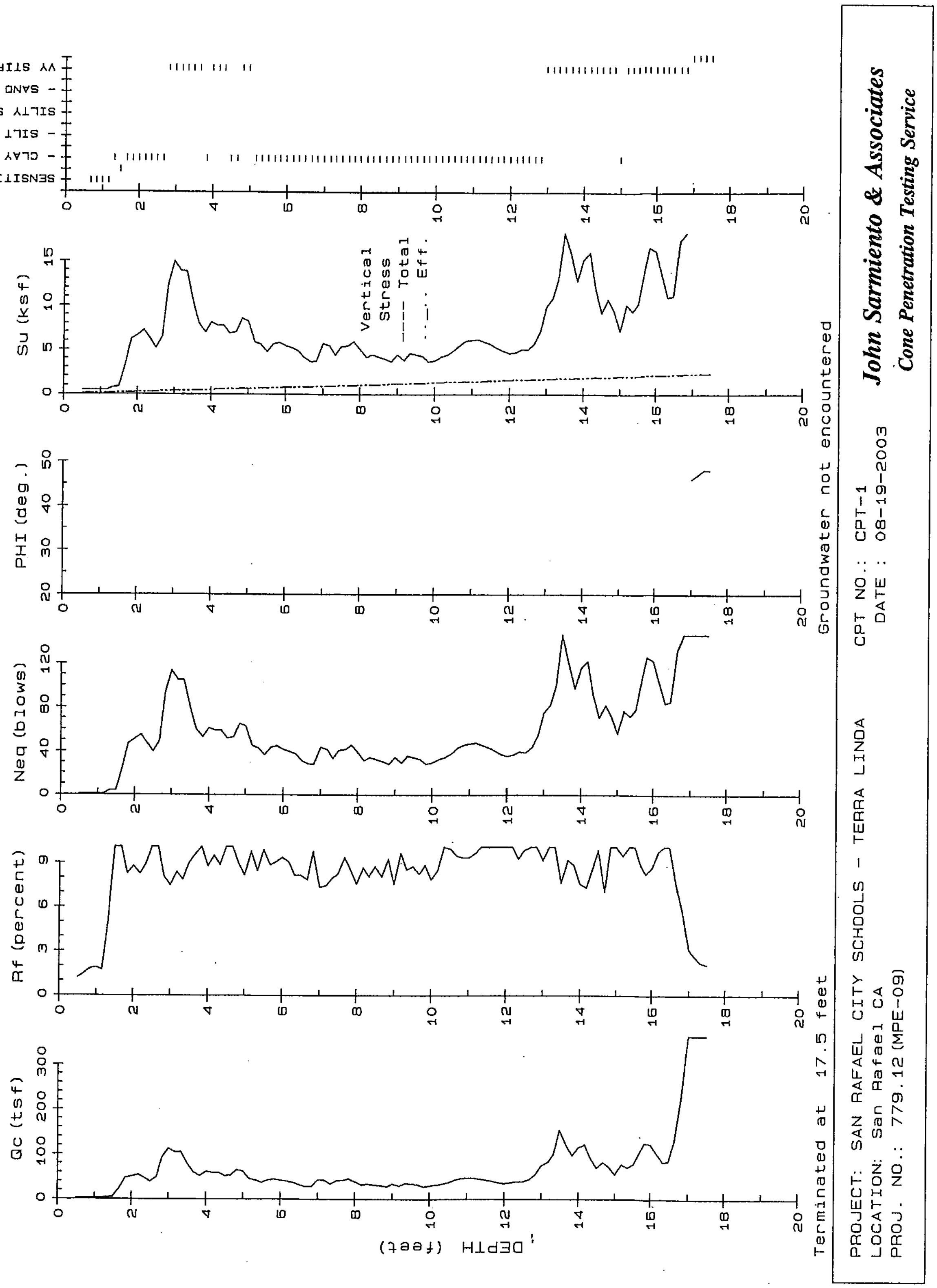
FILE: STD-075 cone 779.12TL.dwt

Miller Pacific
ENGINEERING GROUP

CPT SOIL INTERPRETATION CHART
San Rafael City Schools - Terra Linda High
San Rafael, California

A-7

Project No. 779.12 | Date 10/29/03 | Approved By: [signature] | Figure



FILE: CPT LOGS 779.12TL.dwg

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CONE PENETRATION TEST LOG
San Rafael City Schools - Terra Linda High
San Rafael, California

A-8

Project No. 779.12 | Date 10/29/03 | Approved By: [signature]

Figure

APPENDIX F

Selected Drawings from 1958 Plans for the School
(GM&P, 1958)

A3GEO

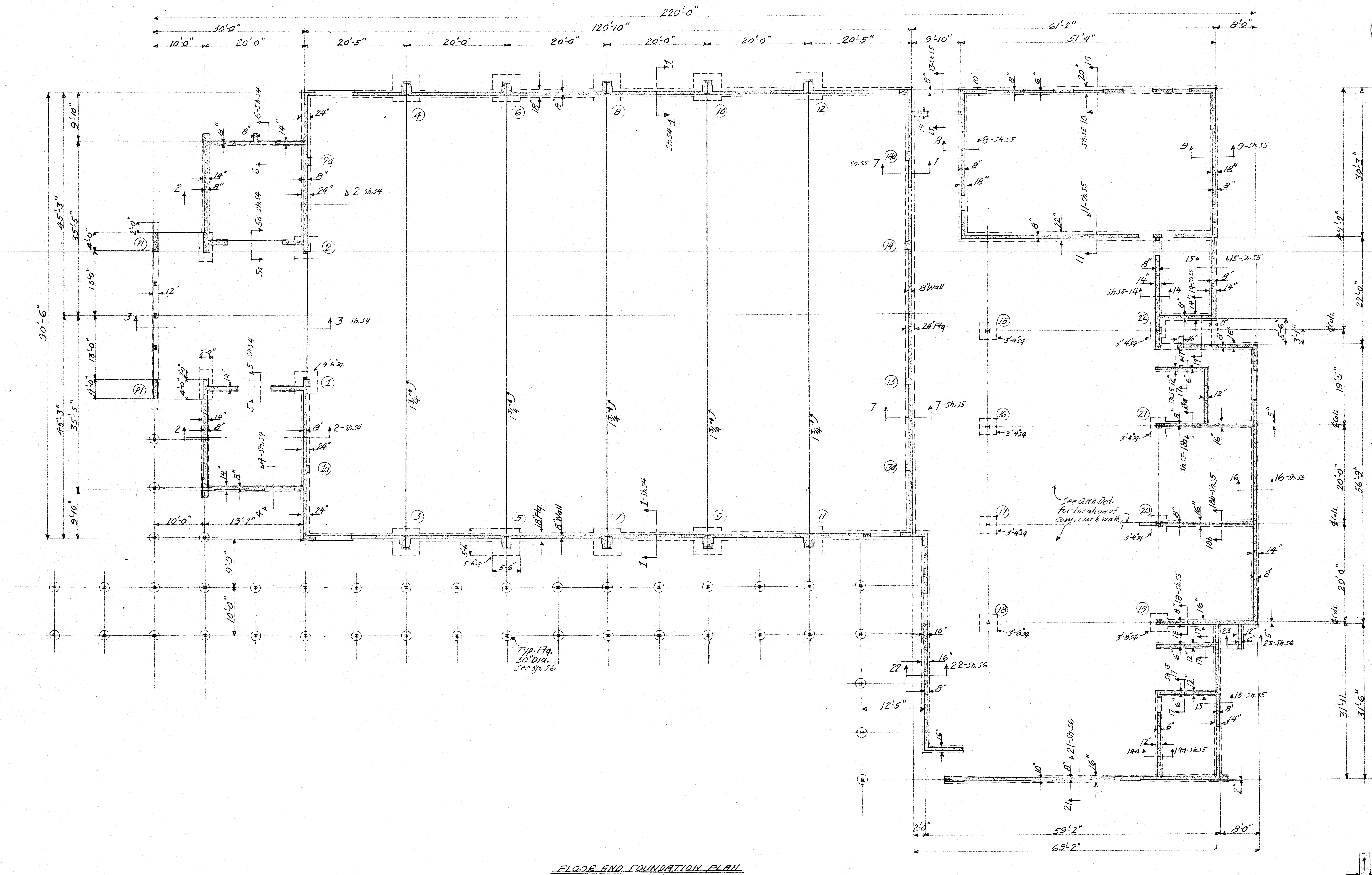
STATE DIVISION OF ARCHITECTURE	COUNTY SUPT. OF SCHOOLS
APPROVED MAR 16 1959	BOARD OF EDUCATION

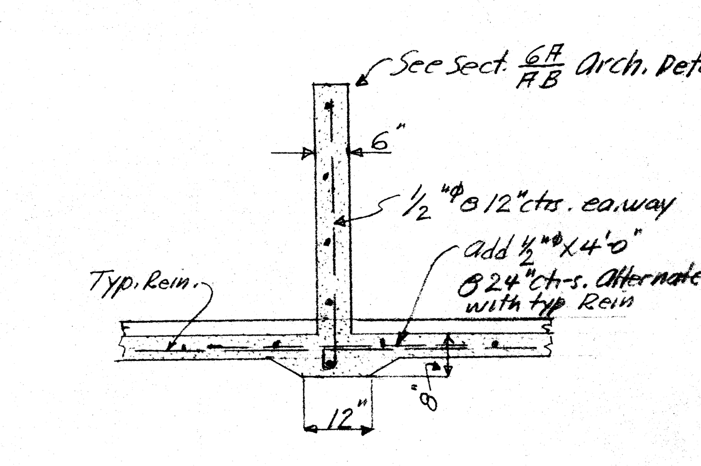
H. M. ENGLE
CIVIL & STRUCTURAL ENGINEER
1589 4TH ST. SAN RAFAEL
LICENSE No.

GROMMÉ MULVIN & PRIESTLEY
ARCHITECTS
1539 FOURTH STREET
SAN RAFAEL, CAL. GL. 3-3925

NEW HIGH SCHOOL
SAN RAFAEL HIGH SCHOOL DISTRICT
FOR THE TERRA LINDA AREA
MARIN COUNTY CALIFORNIA

JOB NO.	DATE: DEC. 16, 1958		DRAWING NO.
502-26	DRAWN BY D. SCHNELL	CHECKED BY J. WINNIE	A1A BLDGS. A.B.C.D.&E.
SET.	APPROVED		OF SHEETS





DETAIL LOCKER ROOM CURB WALLS
See Sect. 6A/AB

STATE DIVISION OF ARCHITECTURE	COUNTY SUPT. OF SCHOOLS						
APPROVED MAR 18 1959 Per M.W. Sahlberg	BOARD OF EDUCATION Victor W. Jones Secy.	ROBERT D. SUTTER	H. M. ENGLE	GROMMÉ, MULVIN & PRIESTLEY ARCHITECTS 1539 FOURTH STREET SAN RAFAEL, CAL. GL. 3-3925	NEW HIGH SCHOOL SAN RAFAEL HIGH SCHOOL DISTRICT FOR THE TERRA LINDA AREA MARIN COUNTY CALIFORNIA	JOB NO. 502-26. DATE: DEC. 16, 1958 DRAWN BY H.E. CHECKED BY SET. APPROVED	DRAWING NO. S1 BLDG. C OF SHEETS

STATE DIVISION OF ARCHITECTURE	COUNTY SUPT. OF SCHOOLS					
APPROVED MAR 16 1959	BOARD OF EDUCATION	ROBERT D. RITTER CONSULTING ENGINEER SAN RAFAEL, CALIF.	H. M. ENGLE CIVIL & STRUCTURAL ENGINEER SAN RAFAEL	GROMMÉ MULVIN & PRIESTLEY ARCHITECTS 1539 FOURTH STREET SAN RAFAEL, CAL. GL. 3-3925	NEW HIGH SCHOOL SAN RAFAEL HIGH SCHOOL DISTRICT. FOR THE TERRA LINDA AREA MARIN COUNTY CALIFORNIA	JOB NO. 502-26 DATE DEC. 16, 1958 DRAWN BY H.E. CHECKED BY APPROVED SET. DRAWING NO. S4 BLDG. "C" OF SHEETS

STATE DIVISION OF ARCHITECTURE

18695 APPROVED MAR 1 6 1939

COUNTY SUPT. OF SCHOOLS.

BOARD OF EDUCATION.

ROBERT P. RITTER
CONSULTING ENGINEER
1539 FOURTH STREET
SAN RAFAEL, CALIF.

H. M. ENGLE
CIVIL & STRUCTURAL ENGINEER
SAN RAFAEL

GROMMÉ, MULVIN & PRIESTLEY
ARCHITECTS
1539 FOURTH STREET
SAN RAFAEL, CAL. GL. 3-3925

NEW HIGH SCHOOL
SAN RAFAEL HIGH SCHOOL DISTRICT.
FOR THE TERRA LINDA AREA.
MARIN COUNTY CALIFORNIA.

JOB NO.	DATE: DEC. 16, 1938		DRAWING NO.
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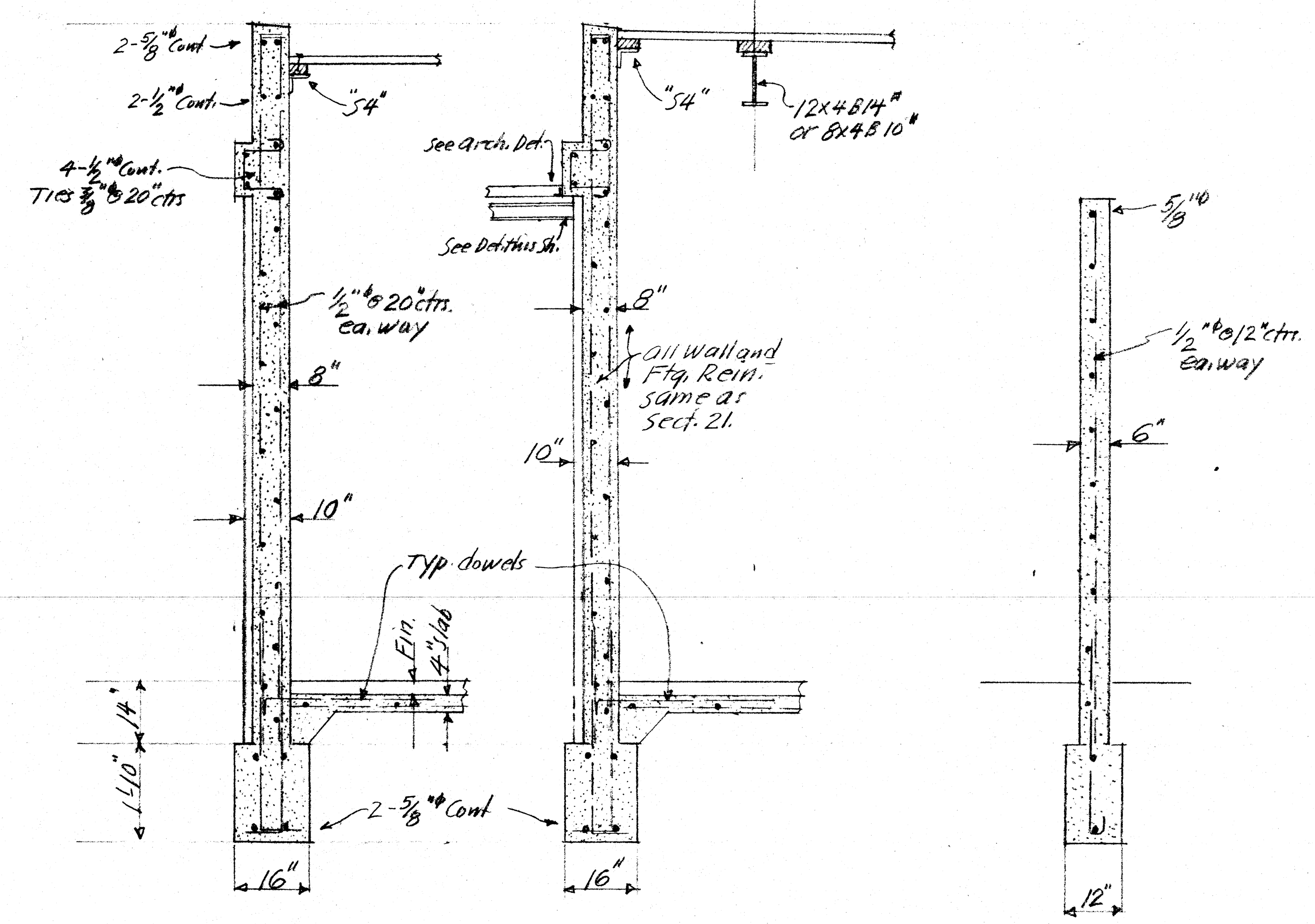
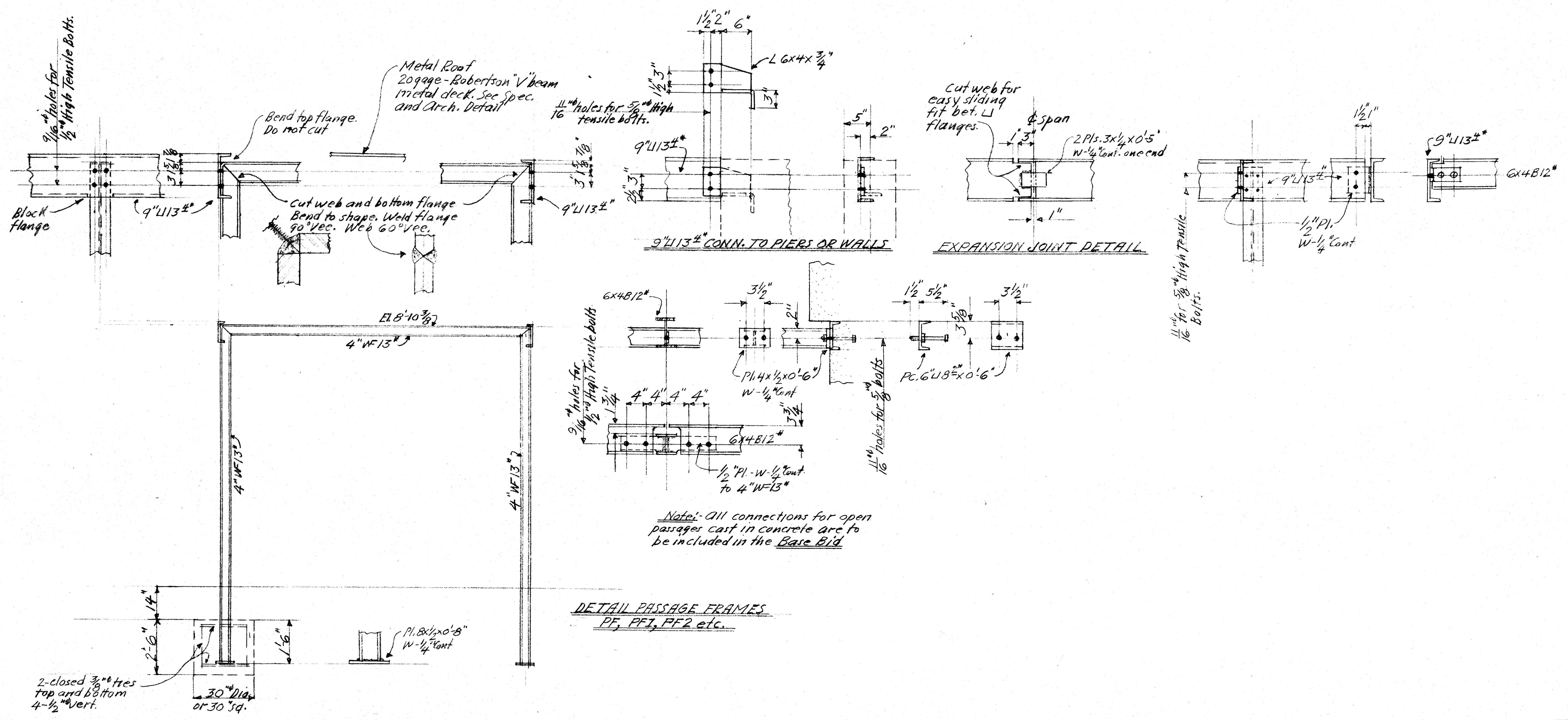
SET.	APPROVED		BLDG. C
			OF SHEETS



SECTION 21-21 SECTION 22-22 SECTION 23-23



9" U13# CONN. TO PIERS OR WALLS

EXPANSION JOINT DETAIL

Note: All connections for open passages cast in concrete are to be included in the Base Bid.

DETAIL PASSAGE FRAMES PF, PF1, PF2 etc.

STATE DIVISION OF ARCHITECTURE — APPROVED MAR 10 1959

COUNTY SUPT. OF SCHOOLS

BOARD OF EDUCATION

ROBERT ...

H. M. ENGLE

GROMMÉ, MULVIN & PRIESTLEY
ARCHITECTS
1539 FOURTH STREET
SAN RAFAEL, CAL. GL. 3-3925

NEW HIGH SCHOOL
SAN RAFAEL HIGH SCHOOL DISTRICT
FOR THE TERRA LINDA AREA
MARIN COUNTY CALIFORNIA

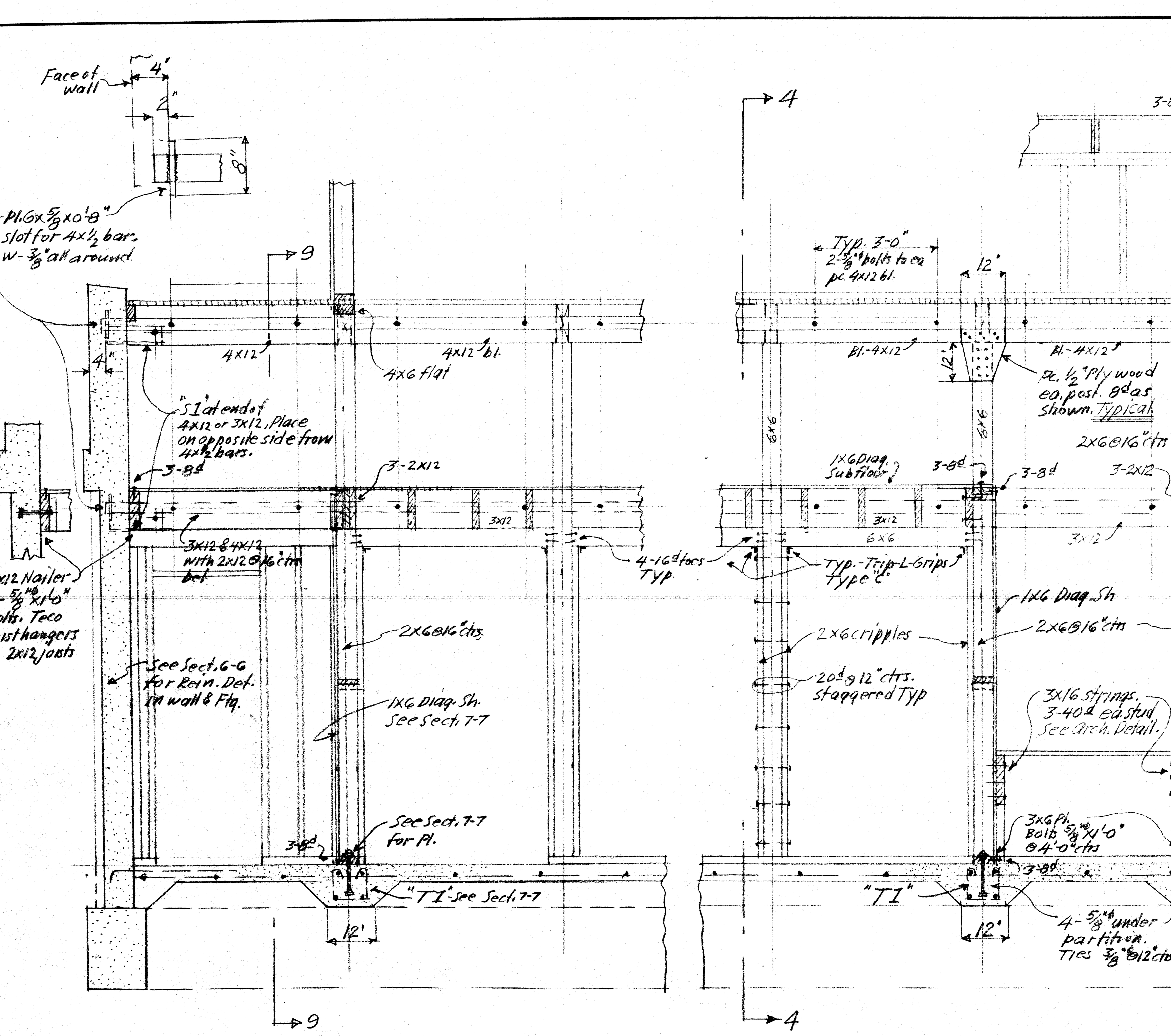
JOB NO.	DATE: DEC. 16, 1958		DRAWING NO.
502-26	DRAWN BY: H.E.	CHECKED BY	S6 BLDG "C"
SET.	APPROVED		OF SHEETS

STATE DIVISION OF ARCHITECTURE	COUNTY SUPT. OF SCHOOLS					JOB NO.	DATE: DEC. 16 1958		DRAWING NO.
18693 APPROVED MAR 16 1959	BOARD OF EDUCATION. Victor W. Jones Sec'y.	ROBERT P. HOLT	H. M. ENGLE CIVIL & STRUCTURAL ENGINEER SAN RAFAEL H. M. Engle	GROMMÉ MULVIN & PRIESTLEY ARCHITECTS 1539 FOURTH STREET SAN RAFAEL, CAL. GL. 3-3925	NEW HIGH SCHOOL SAN RAFAEL HIGH SCHOOL DISTRICT. FOR THE TERRA LINDA AREA. MARIN COUNTY CALIFORNIA.	507-26	DRAWN BY H.E.	CHECKED BY H.E.	S-1
						SET.	APPROVED		BLDG. D OF SHEETS

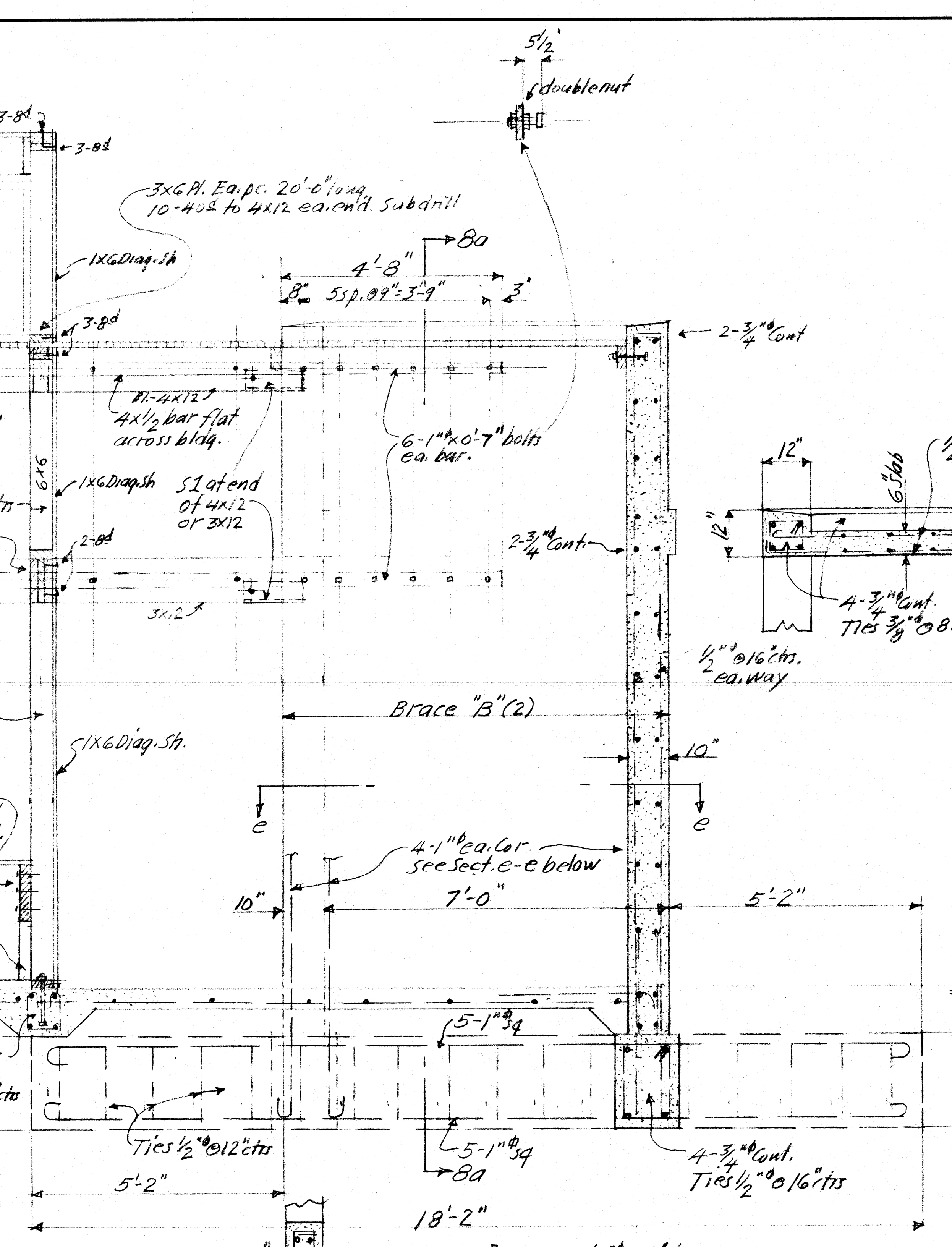
S-1

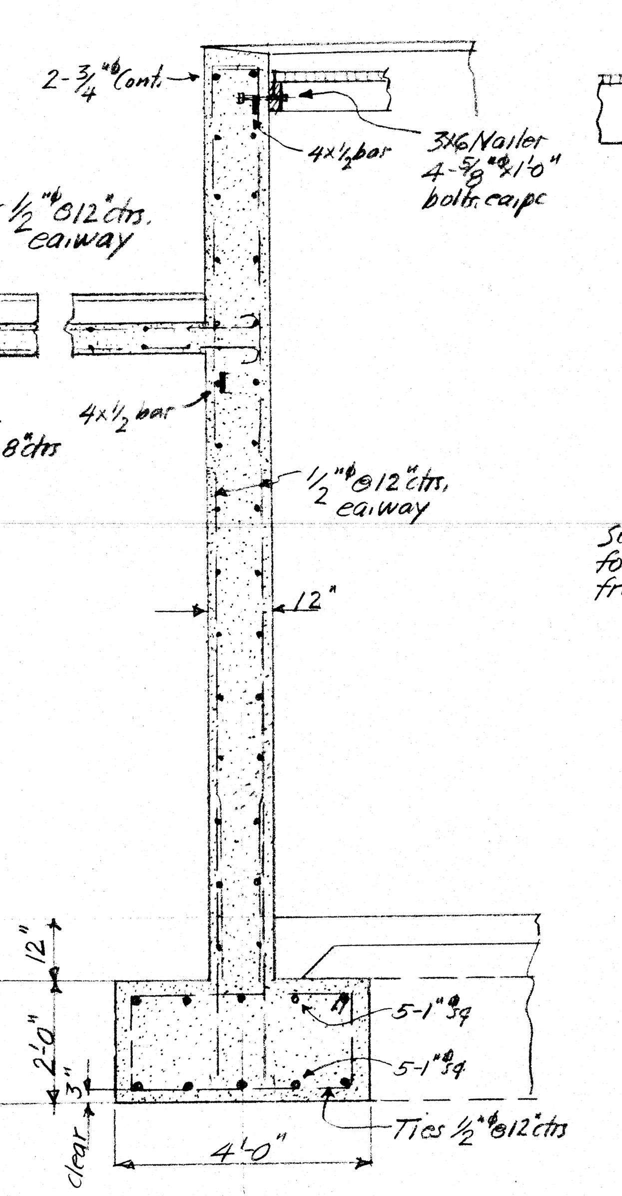
STATE DIVISION OF ARCHITECTURE 18005 APPROVED MAR 16 1959	COUNTY SUPT. OF SCHOOLS BOARD OF EDUCATION Victor W. Jones Secy.	ROBERT P. RITTER CONSULTING ENGINEER	H. M. ENGLE CIVIL & STRUCTURAL ENGINEER LICENSE NO. H. M. Engle	GROMMÉ MULVIN & PRIESTLEY ARCHITECTS 1539 FOURTH STREET SAN RAFAEL, CAL. GL. 3-3925	NEW HIGH SCHOOL SAN RAFAEL HIGH SCHOOL DISTRICT. FOR THE TERRA LINDA AREA. MARIN COUNTY CALIFORNIA.

JOB NO.	DATE	DRAWN BY	CHECKED BY	DRAWING NO.
502-26	DEC. 16, 1958	H.E.	H.E.	S-1. BLDG. E.
SET.	APPROVED			OF SHEETS

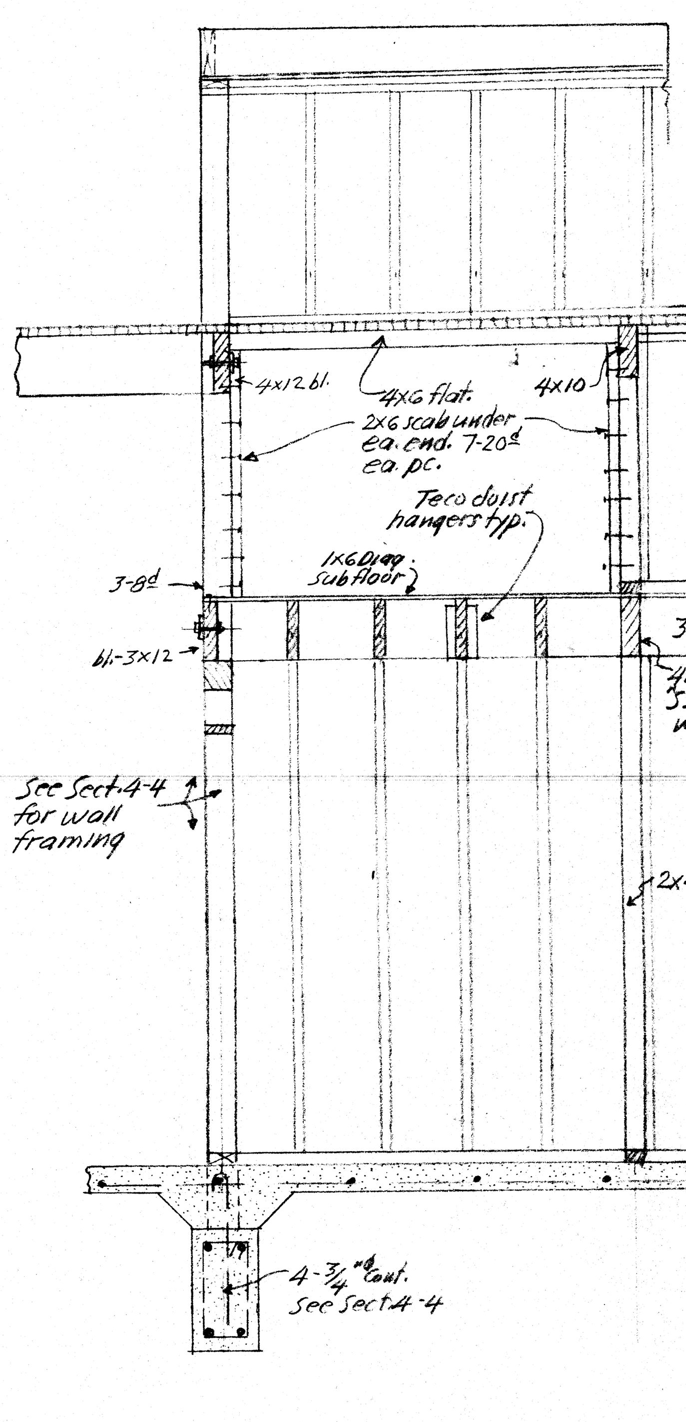


SECTION 8-8

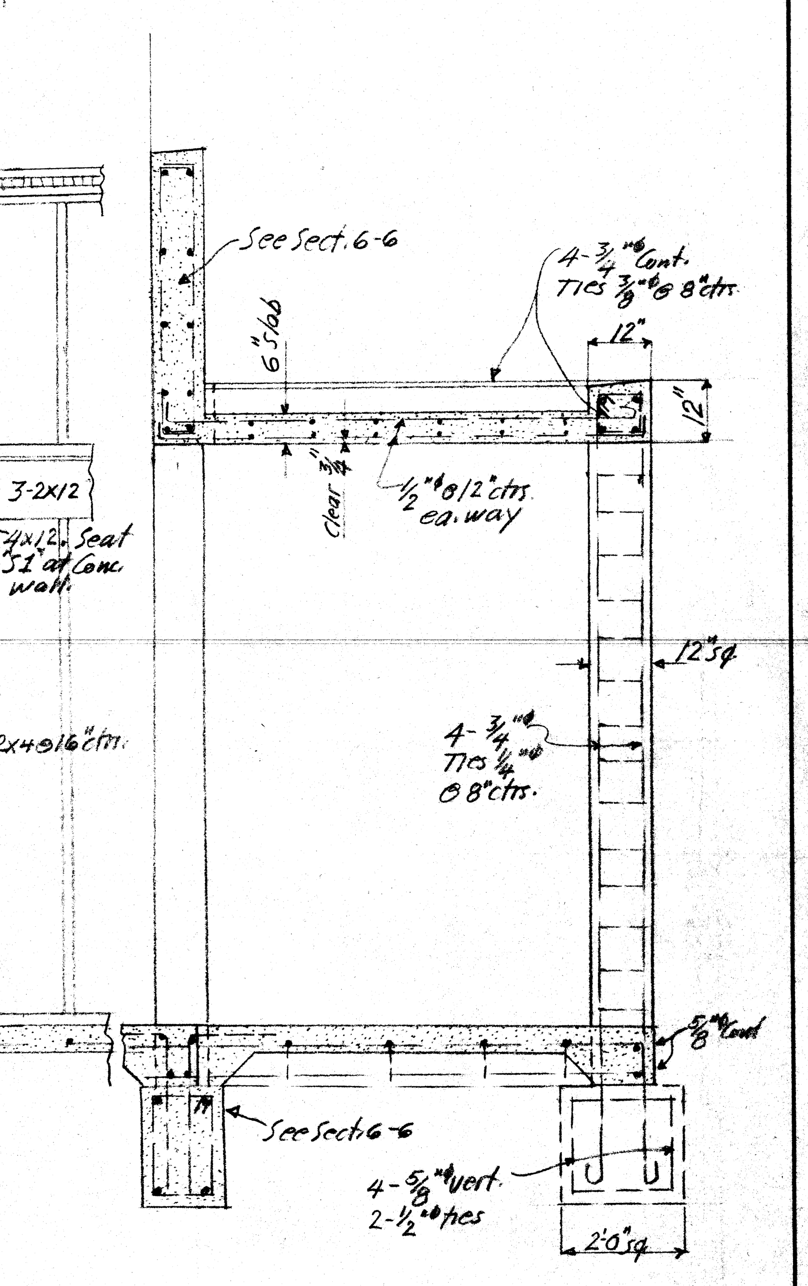




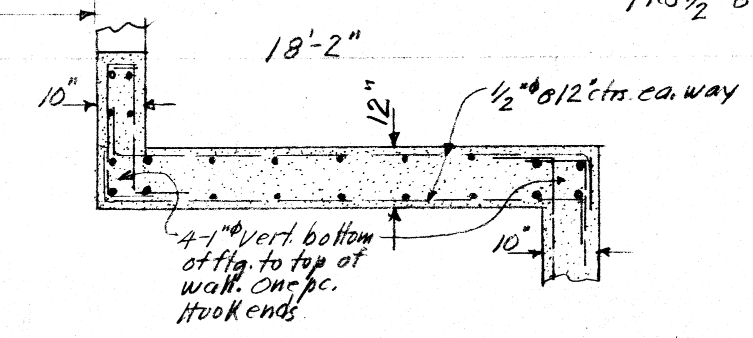
SECTION 8a-8a



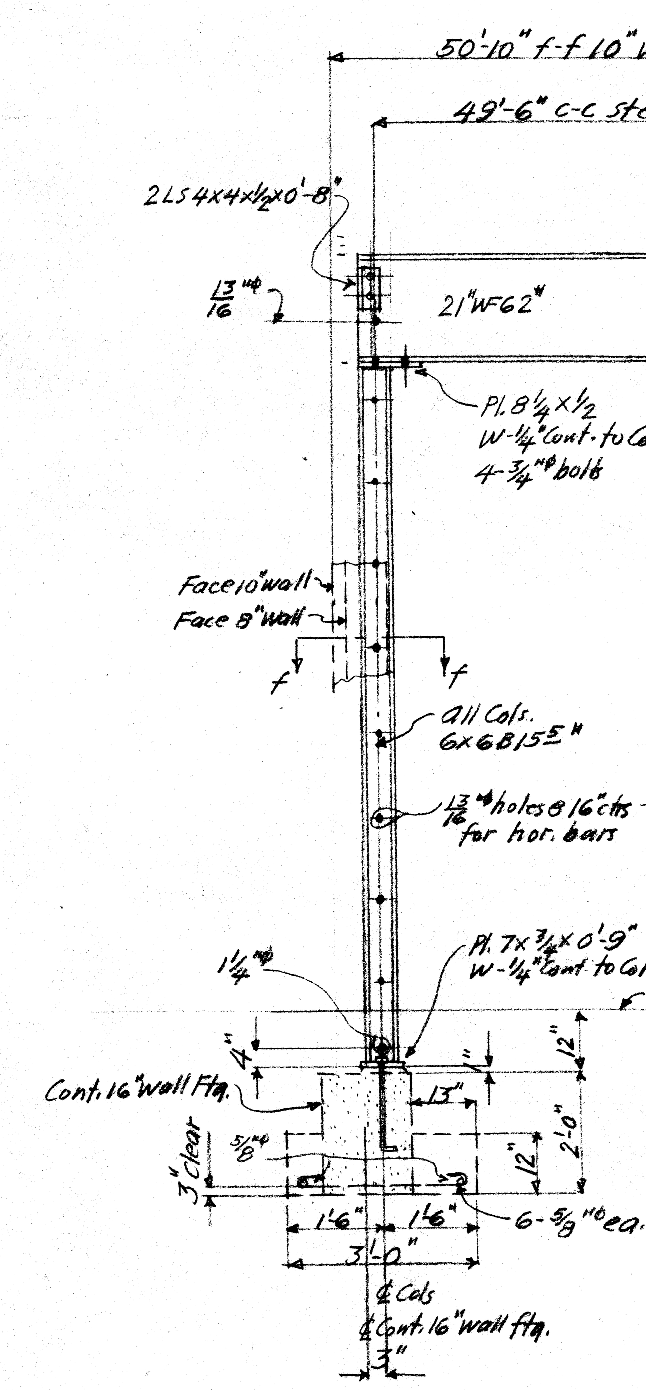
SECTION 9-9
Toilet ℄ Detail at ℄ same but opposite hand

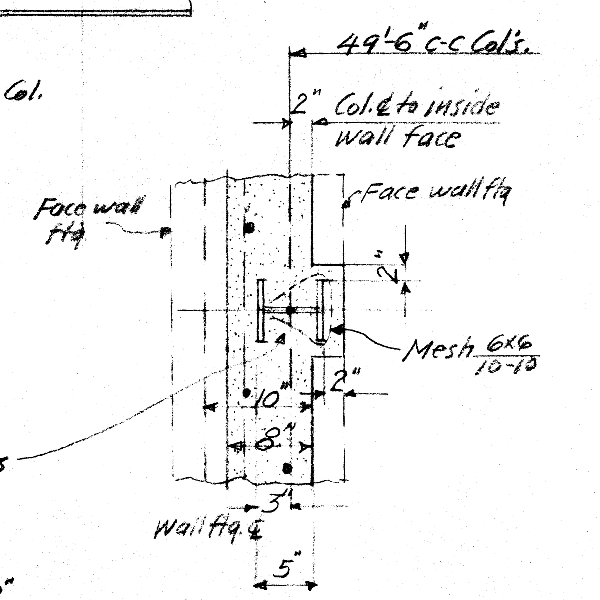


SECTION 10-10
SECTION 10a-10a Same but opposite hand.

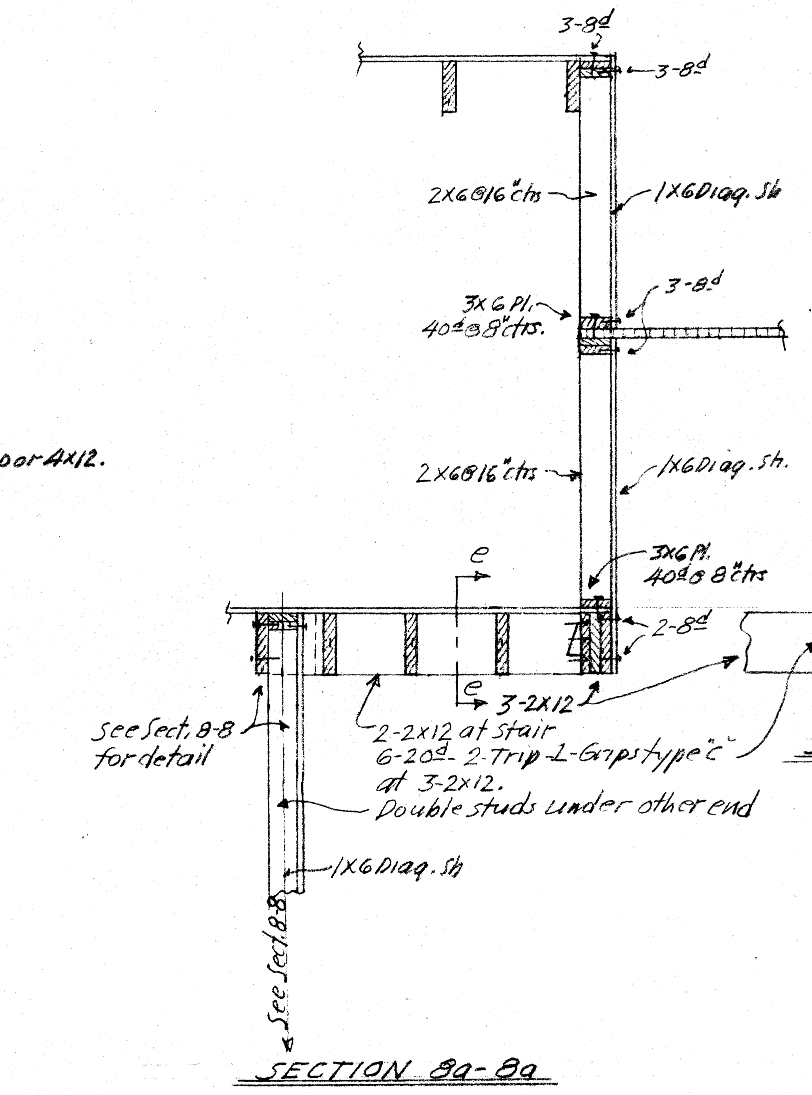


SECTION e-e

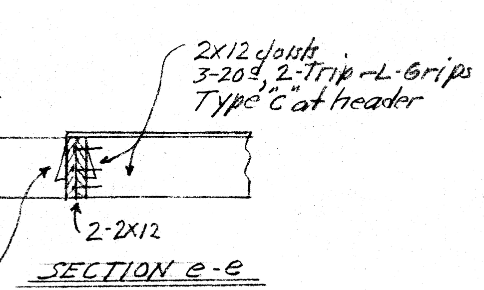




SECTION f-f



SECTION 8a-8a



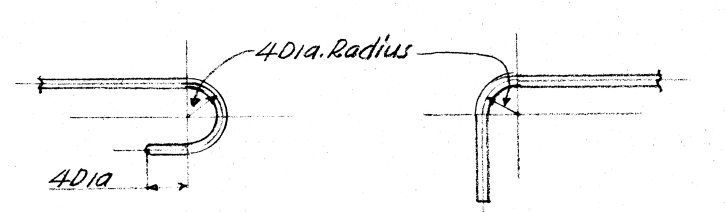
SECTION e-e

FOUNDATION NOTE:-

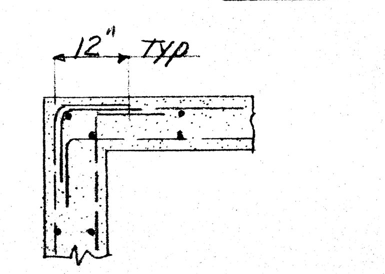
Soil Pressure does not exceed 2000 lbs. per sq. ft. All footings go to hard sand and clay or to shale rock at elevations shown. If necessary to reach firm material, lower footings below elevations shown as directed by the architect. Soil Data from R.S. Harding. (Oct. 24, 1958)

CONCRETE AND REINFORCING NOTE:-

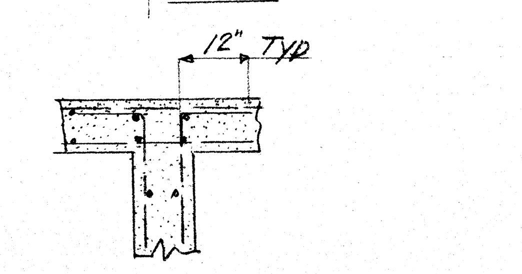
All bars to be deformed-Structural Grade. Bars to lap 48 Dia. at splices except as otherwise called for.
All Structural concrete to have a minimum strength in compression of 2500 lbs. per sq. inch at 28 days.
Bars marked as continuous (Cont.) to be in as long lengths as possible. Splices to be staggered.
All beam bars to be chaired up at not over 5'-0" ctrs. All bars in slabs over 3" thick shall be chaired up with continuous metal chairs at not over 3'-6" ctrs.



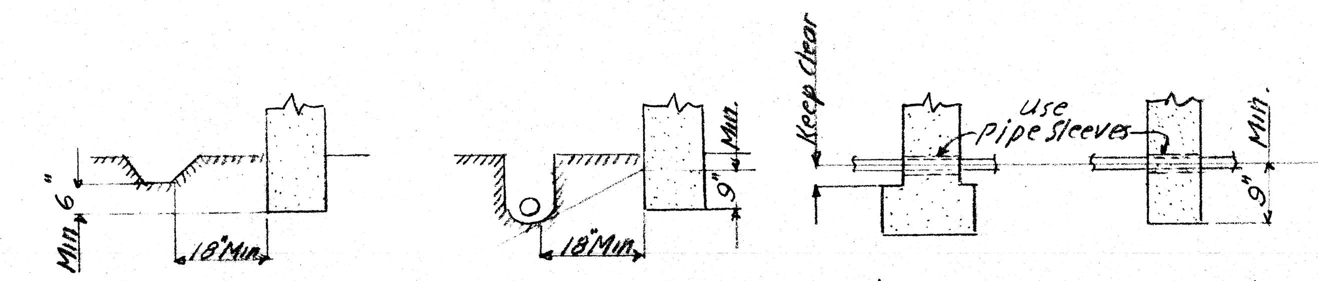
HOOK ← TYPICAL → BEND



TYPICAL CORNERS



TYPICAL INTERSECTIONS



OPEN TRENCH — FILLED TRENCH — PIPES PASSING THRU FOOTINGS

NAILING & FRAMING NOTE:

All nails to be common wire nails.
Double top plates:- 2-20d lower plate to each stud, 16d @ 18" ctrs. upper plate to lower plate. Stagger splices 4'-0" Min., 8-16d ea. side ea. cut. Lap plates at corners, with 3-16d upper to lower plate.
Multiple studs:- 16d @ 12" ctrs. - stagger for 2x6 studs or larger.
Frame stud wall corners and intersections solid.
Structural wall and roof sheathing to be laid diagonal at 45° to supports.
For 1x6 use 2-8d Min. at all bearings including blocking.
For 2x6 " 2-16d " " " " "
Use additional nailing as called for. Cut joints on a diagonal over and parallel to a bearing. Leave at least one bearing between joints in adjoining boards. For joints on same bearing, leave at least two boards between joints on vertical diaphragms, and at least one board between joints on horizontal diaphragms. Use 3-8d at ends of all sheathing boards, Min. Structural sheathing to be continuous behind blackboards, tack boards etc.
Joists or rafters to sides of studs:- as called for on plans - 3-16d Min.
" " " at all bearings:- 2-10d toes each side.
Studs to bearings - 2-10d toes ea. side, or as called for.
Blocking between joists or rafters - 2-10d toes ea. side, ea. end, 2-10d toes ea. side to bearing. Special nailing as called for.
Cross bridging between joists:- 2-8d toes each end.
Ceiling strips:- 1x4 - 2-8d to under side of joists per bearing. Thickness one inch net. One nail slant - one straight.
For Plaster Ceilings, strips to be "Saddle" tied to sides of joists with 16 ga. galvanized wire ties at 4'-0" ctrs.
Ribbons to studs - 1x4, 2-8d each bearing. 2" ribbons, nails as called for.
Built up beams:- 8" or less, 16d @ 12" ctrs. staggered. More than 8" 1/2" bolts at 2'-0" ctrs.
Double joists under partitions - 16d @ 12" ctrs., staggered.
Where nails tend to cause splitting, holes to be sub-drilled.

BOLT & LAG SCREW NOTE:-

Bolt holes to be bored to same diameter as nominal bolt diameter. Use Malleable Iron Washers (M.I.W.) under all heads and nuts bearing on wood. Washers to have a thickness not less than 1/2 bolt diameter, and an area 16 times nominal bolt area. Tighten all bolts at time of installation. Check and retighten before covering up. Check and tighten all accessable nuts before final acceptance.
Lag screws shall be screwed into place. Bore holes to same diameter and depth as shank. For threaded portion bore hole with bit not larger than diameter at root of thread.

STATE DIVISION OF ARCHITECTURE	COUNTY SUPT. OF SCHOOLS						
APPROVED MAR 16 1959	BOARD OF EDUCATION	ROBERT P. [illegible]	H. M. ENGLE	GROMMÉ MULVIN & PRIESTLEY ARCHITECTS 1539 FOURTH STREET SAN RAFAEL, CAL. GL. 3-3925	NEW HIGH SCHOOL SAN RAFAEL HIGH SCHOOL DISTRICT. FOR THE TERRA LINDA AREA. MARIN COUNTY CALIFORNIA.	JOB NO. 502.26 / DATE: DEC. 16 1958 / DRAWN BY H.E. / CHECKED BY H.E. / SET. / APPROVED	DRAWING NO. S-2 BLDG. E. OF SHEETS
M. W. Sahlberg [illegible]	[illegible]		H. M. Engle				

APPENDIX G

USGS Ground Motion Reports

A3GEO

Design Maps Summary Report

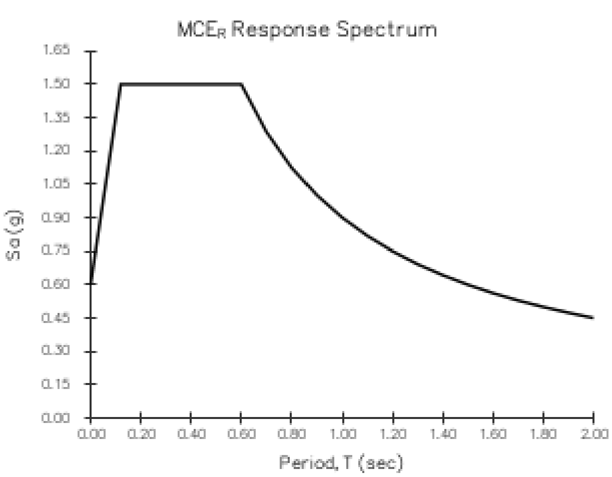
User–Specified Input

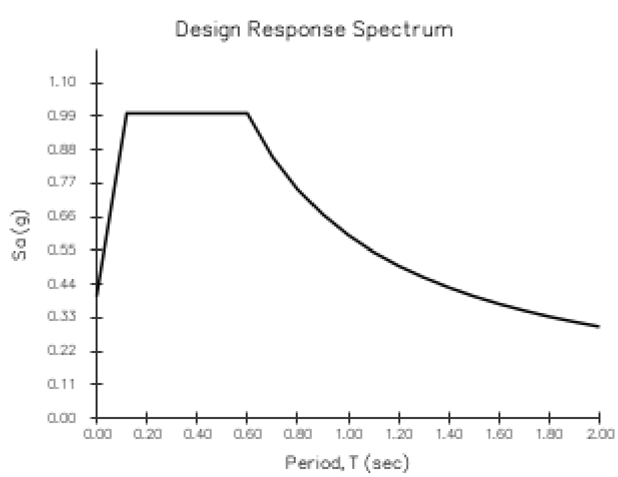
Report Title	Terra Linda High School Wed March 8, 2017 21:40:28 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	38°N, 122.5543°W
Site Soil Classification	Site Class D – "Stiff Soil"
Risk Category	I/II/III

USGS–Provided Output

S_S = 1.500 g	S_{MS} = 1.500 g	S_{DS} = 1.000 g
S_1 = 0.600 g	S_{M1} = 0.900 g	S_{D1} = 0.600 g

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





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

Design Maps Detailed Report

ASCE 7-10 Standard (38°N, 122.5543°W)

Site Class D – "Stiff Soil", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

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

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

Section 11.4.2 — Site Class

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

Table 20.3–1 Site Classification

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

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

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

Table 11.4–1: Site Coefficient F_a

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

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

For Site Class = D and S_s = 1.500 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

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

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

For Site Class = D and S_1 = 0.600 g, F_v = 1.500

Equation (11.4–1):	$S_{MS} = F_aS_S = 1.000 \times 1.500 = 1.500$ g
Equation (11.4–2):	$S_{M1} = F_vS_1 = 1.500 \times 0.600 = 0.900$ g

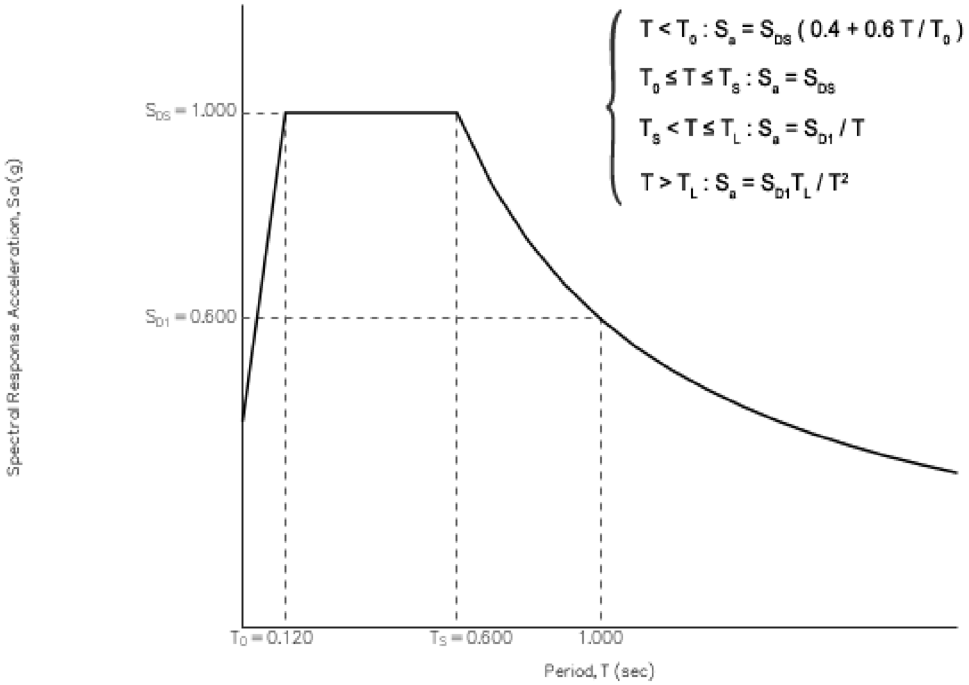
Section 11.4.4 — Design Spectral Acceleration Parameters

Equation (11.4–3):	$S_{DS} = ⅔\ S_{MS} = ⅔ \times 1.500 = 1.000$ g
Equation (11.4–4):	$S_{D1} = ⅔\ S_{M1} = ⅔ \times 0.900 = 0.600$ g

Section 11.4.5 — Design Response Spectrum

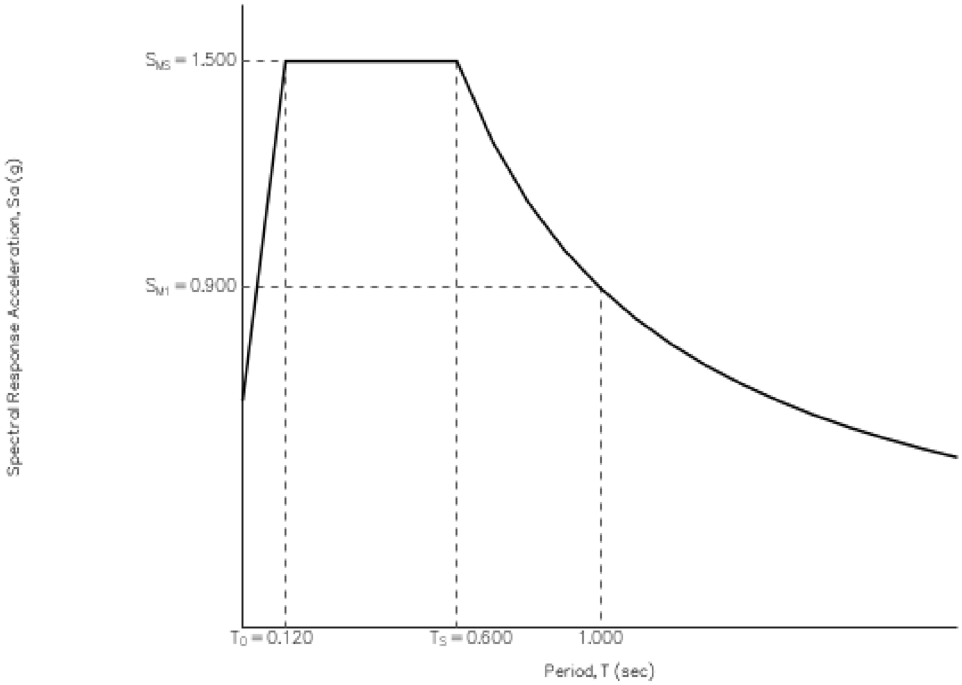
From Figure 22-12 [3]	T_L = 12 seconds

Figure 11.4–1: Design Response Spectrum



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

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



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

From Figure 22-7 [4] PGA = 0.500

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

Table 11.8–1: Site Coefficient F_{PGA}

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

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

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

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

From Figure 22-17 [5] C_{RS} = 1.070

From Figure 22-18 [6] C_{R1} = 1.024

Section 11.6 — Seismic Design Category

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

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

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

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

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

For Risk Category = I and S_{D1} = 0.600 g, Seismic Design Category = D

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

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

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

References

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

END OF REPORT