MILLER PACIFIC
ENGINEERING GROUP

PRELIMINARY GEOTECHNICAL INVESTIGATION
THE NEIGHBORHOOD DEVELOPMENT
LOS GAMOS AVENUE
SAN RAFAEL, CALIFORNIA

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Prepared for:
County Strategic Fund I, LLC
899 Northgate Drive, STE 301
San Rafael, California 94903

CERTIFICATION

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1.0 INTRODUCTION

This report presents the results of our preliminary geotechnical investigation for the proposed residential development in San Rafael, California. As shown on Figure 1, the project site is located on the west side Los Gamos Avenue, just north of the building at 1401 Los Gamos, and southwest of 1500 Los Gamos.

Our work was performed in accordance with our Agreement for Professional Services authorized May 7, 2020. The purpose of our investigation was to explore subsurface conditions and to develop preliminary geotechnical criteria for design and construction of the proposed improvements. The scope of our services includes:

- Reviewing published geologic mapping and geotechnical background information from our files, including existing geotechnical data from previous site investigations.
- Performing supplemental subsurface exploration with four borings located within the general vicinity of the planned improvements.
- Evaluating relevant geologic hazards including seismic shaking, settlement, slope instability and other hazards.
- Preparing geotechnical recommendations and design criteria related to building foundations, site grading, retaining walls, seismic design, and other geotechnical-related items.
- Preparing this preliminary geotechnical report which summarizes subsurface exploration, evaluation of relevant geologic hazards, and preliminary geotechnical recommendations and design criteria.

This report completes our Phase 1 services for the project. Subsequent phases of work should include possible supplemental exploration, design-level geotechnical report, geotechnical plan review and observation and testing of geotechnical-related work items during construction.

2.0 PROJECT DESCRIPTION

Based on our review of preliminary plans and discussions with the design team, we understand the project is expected to include developing the site with five, multi-story buildings which will provide about 180 units for residential use and a separate four-story community center building that will also include a market and office space. The residential units will occupy most of the site and will be up to five stories in height above basement level parking. The commercial building will be three to four stories in height, wood-framed and will include parking on the ground floor with a market and community center above. Ancillary improvements will include roadways, parking area, underground utilities, site drainage, play structure, exercise stations, and walking paths. The approximate building locations are shown on the Site Plan, Figure 2.

Moderate site grading is planned and includes excavation into the hillside to embed the structures. Retaining walls on the order of 30 feet tall will retain cuts and limit the extent of grading. Expected off-haul is expected to be around 54,000 CY. Fill placement is planned for roadways and to backfill some walls. Moderate to high foundation loads are expected for the structures.

3.0 SITE CONDITIONS

3.1 Regional Geology

The project site lies within the Coast Ranges geomorphic province of California. Regional topography within the Coast Ranges province is characterized by northwest-southeast trending mountain ridges and intervening valleys that parallel the major geologic structures, including the San Andreas Fault System. The province is also generally characterized by abundant landsliding and erosion, owing in part to its typically high levels of precipitation and seismic activity.

The oldest rocks in the region are the sedimentary, igneous, and metamorphic rocks of the Jurassic-Cretaceous age (190- to 65-million years old) Franciscan Complex. Within Marin County, a variety of sedimentary and volcanic rocks of Tertiary (1.8- to 65-million years old) and Quaternary (less than 1.8-million years old) age locally overlie the basement rocks of the Franciscan Complex. Tectonic deformation and erosion during late Tertiary and Quaternary time (the last several million years) formed the prominent coastal ridges and intervening valleys typical of the Coast Ranges province. The youngest geologic units in the region are Quaternary age (last 1.8 million years) sedimentary deposits, including alluvial deposits which partially fill most of the valleys and colluvial deposits which typically blanket the lower portions of surrounding slopes.

The proposed buildings are situated on a sloping hillside on the east facing side of a north-trending ridgeline. Regional geologic mapping (Rice, 1976) indicates the site is underlain by colluvial deposits of Quaternary age, mélange bedrock of the Franciscan Complex, and semi-schist, phillite, and schist bedrock. Much of the site is mapped as exhibiting continuous or intermittent downslope soil creep. A Regional Geologic Map and descriptions of the mapped geologic units are shown on Figure 3.

3.2 Seismicity

The project site is located within the seismically active San Francisco Bay Area and will therefore experience the effects of future earthquakes. Earthquakes are the product of the build-up and sudden release of strain along a “fault” or zone of weakness in the earth's crust. Stored energy may be released as soon as it is generated or it may be accumulated and stored for long periods of time. Individual releases may be so small that they are detected only by sensitive instruments, or they may be violent enough to cause destruction over vast areas.

Faults are seldom single cracks in the earth's crust but are typically comprised of localized shear zones which link together to form larger fault zones. Within the Bay Area, faults are concentrated along the San Andreas Fault zone. The movement between rock formations along either side of a fault may be horizontal, vertical, or a combination, and is radiated outward in the form of energy waves. The amplitude and frequency of earthquake ground motions partially depends on the material through which it is moving. The earthquake force is transmitted through hard rock in short, rapid vibrations, while this energy becomes a long, high-amplitude motion when moving through soft ground materials, such as Bay Mud.

3.2.1 Regional Active Faults

The California Geological Survey (previously known as the California Division of Mines and Geology), defines a “Holocene-active fault” as one that has had surface displacement within Holocene time (the last 11,700 years). CGS has mapped various faults in the region as part of their Fault Activity Map of California (CGS, 2010). Many of these faults are shown in relation to the project site on the attached Active Fault Map, Figure 4. The nearest known Holocene-active faults are the San Andreas, Hayward, and San Gregorio Faults. The San Andreas and San Gregorio Faults are located approximately 16.4 kilometers and 17.5 kilometers to the southwest[1], respectively. The Hayward Fault is located roughly 13.6 kilometers northeast of the site.

3.2.2 Historic Fault Activity

Numerous earthquakes have occurred in the region within historic times. The results of our USGS earthquake search catalogue indicates that at least 13 earthquakes with a Richter Magnitude of 5.0 or larger have occurred within 100 kilometers (62 miles) of the site between 1900 and 2019. The approximate locations of these earthquakes are shown on the Historic Earthquake Map, Figure 5.

3.2.3 Probability of Future Earthquakes

The site will likely experience moderate to strong ground shaking from future earthquakes originating on any of several active faults in the San Francisco Bay region. The historical records do not directly indicate either the maximum credible earthquake or the probability of such a future event. To evaluate earthquake probabilities in California, the USGS has assembled a group of researchers into the “Working Group on California Earthquake Probabilities” (USGS 2003, 2008, 2013) to estimate the probabilities of earthquakes on active faults. These studies have been published cooperatively by the USGS, CGS, and Southern California Earthquake Center (SCEC) as the Uniform California Earthquake Rupture Forecast, Versions 1, 2, and 3. In these studies, potential seismic sources were analyzed considering fault geometry, geologic slip rates, geodetic strain rates, historic activity, micro-seismicity, and other factors to arrive at estimates of earthquakes of various magnitudes on a variety of faults in California.

Conclusions from the most recent UCERF3 and USGS indicate the highest probability of an earthquake with a magnitude greater than 6.7 originating on any of the active faults in the San Francisco Bay region by 2043 is assigned to the Hayward/Rodgers Creek Fault system. The Hayward Fault is located approximately 13.6 kilometers northeast of the site and is assigned a probability of 33 percent. The San Andreas Fault, located approximately 16.4 kilometers southwest of the site, is assigned a 22 percent probability of an earthquake with a magnitude greater than 6.7 by 2043. Additional studies by the USGS regarding the probability of large earthquakes in the Bay Area are ongoing. These current evaluations include data from additional active faults and updated geological data.

[1] Distances to faults estimated using Caltrans ARS Online (v2.3.09), accessed July 13, 2020.

3.3 Surface Conditions

The project site encompasses an irregularly-shaped, approximately 10.24-acre parcel (APN 165-220-07) located southwest of the intersection of Los Gamos Drive and Marinwood Drive. The site is bordered by Los Gamos Drive to the east, by open space to the north and west, and by a commercial/office development to the south. The average slope of the parcel is about 3:1 (horizontal:vertical) but varies locally.

The ground surface within the planned building areas varies from approximately 50 feet elevation where the project site connects to Los Gamos to 200 feet at the upslope limit of the site[2]. The property is currently unimproved and is vegetated with native grasses, shrubbery, and a few trees. Hillside drainage swales or ravines cross the north and south ends of the property.

3.4 Subsurface Exploration and Laboratory Testing

We explored subsurface conditions on June 11, 2020, with four borings drilled with a track mounted Geoprobe to depths between about 40.5- and 41.5-feet below the ground surface. The approximate locations of our borings are shown on Figure 2. Our geologist logged the borings in the field and collected select soil samples for laboratory testing. Soil and Rock Classification Charts are presented along with the boring logs on Figures A-1 through A-11.

Laboratory testing of soil samples from the exploratory borings include determination of moisture content, dry density, unconfined compressive strength, undrained unconsolidated compressive strength, direct shear strength, expansion index, and plasticity index. Results from moisture content, dry density, unconfined compression undrained unconsolidated compressive strength, and direct shear strength are presented on the boring logs. The expansion index and plasticity index charts are presented on Figure A-12 through A-14. The laboratory testing program also is discussed in more detail in Appendix A.

3.5 Reference Geotechnical Data

Previous geotechnical investigations were completed by Salem Howes Associates Inc. for the originally planned development (Salem Howes, 1998 and 1999). These investigations included excavating seventeen test pits and four exploratory borings near the planned improvements. The report, including subsurface exploration and laboratory testing data, is presented in Appendix B. The exploration locations are also shown on Figure 2.

We also conducted an aerial photo review for the site using the photos hosted by Netronline Historic Aerials website and Google Earth. Between the photos taken in 1987 and 1993, there appears to be evidence of a landslide on the northern portion of the property.

3.6 Subsurface Conditions and Groundwater

Based on our recent exploration and review of reference data, the project site is generally underlain by between 4 to 9 feet of clayey colluvial soils over mélange and variably weathered bedrock of the Franciscan Complex. The clayey soils are generally medium stiff to stiff and are likely derived from the underlying mélange.

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[2] Surface elevations are based on those shown on the Design Review Plans by Tarnoff Engineering, Dated 11-1-20, Sheets C.1 through C.4

The bedrock encountered in the borings and test pits predominantly consists of pervasively sheared shale of the Franciscan Melange unit, which is generally highly to completely weathered and exhibit low to moderate hardness and weak to moderate strength. White and green mineralization was locally observed at varying depths in the bedrock.

Groundwater was not encountered in the subsurface exploration. However, because the test pits and borings were not left open for an extended period of time, a stabilized depth to groundwater may not have been observed. Groundwater elevations fluctuate seasonally with higher groundwater levels during periods of intense rainfall. Groundwater seepage will likely flow downslope along the soil to bedrock contact during winter and early spring.

4.0 GEOLOGIC HAZARDS

This section summarizes our review of commonly considered geologic hazards and discusses their potential impacts on the planned improvements. The primary geologic hazards which could affect the proposed development include strong seismic ground shaking, potential debris flow impact and slope instability. Other geologic hazards are judged less than significant regarding the proposed project. Geologic hazards, potential impacts and mitigation measures are discussed in further detail in the following sections.

4.1 Fault Surface Rupture

Under the Alquist-Priolo Earthquake Fault Zoning Act, the California Division of Mines and Geology (now known as the California Geological Survey) produced 1:24,000 scale maps showing known active and potentially active faults and defining zones within which special fault studies are required. The nearest known active fault to the site is the San Andreas Fault located approximately 16.4 kilometers to the southwest. The site is not located within an Alquist-Priolo Special Studies Zone. We therefore judge the potential for fault surface rupture in the development area to be low.

Evaluation: Less than significant. No mitigation measures are required.

4.2 Seismic Shaking

The site will likely experience seismic ground shaking similar to other areas in the seismically active Bay Area. The intensity of ground shaking will depend on the characteristics of the causative fault, distance from the fault, the earthquake magnitude and duration, and site-specific geologic conditions. Estimates of peak ground accelerations are based on either deterministic or probabilistic methods.

Deterministic methods use empirical attenuation relations that provide approximate estimates of median peak ground accelerations. A summary of the active faults that could most significantly affect the planning area, their maximum credible magnitude, closest distance to the center of the planning area, and probable peak ground accelerations are summarized in Table 1. The calculated accelerations should only be considered as reasonable estimates. Many factors (e.g., soil conditions, orientation to the fault, etc.) can influence the actual ground surface accelerations.

Table 1 – Deterministic Peak Ground Accelerations for Active Faults

Fault	Moment Magnitude for Characteristic Earthquake	Closest Estimated Distance (km)	Median Peak Ground Acceleration (g)	Median PGA +1 Std Dev (g)
San Andreas	8.0	16.4	0.26	0.47
Hayward	7.3	13.6	0.23	0.42
San Gregorio	7.4	17.5	0.20	0.36
Rodgers Creek	7.3	19.0	0.18	0.33
West Napa	6.9	30.5	0.09	0.18

Reference: Abrahamson & Silva, Boore & Atkinson, Campbell & Bozorgnia, and Chiou & Youngs (2008) NGA models using V_{s30} = 760 m/s.

Probabilistic Seismic Hazard Analysis analyzes all possible earthquake scenarios while incorporating the probability of each individual event to occur. The probability is determined in the form of the recurrence interval, which is the average time for a specific earthquake acceleration to be exceeded. The design earthquake is not solely dependent on the fault with the closest distance to the site and/or the largest magnitude, but rather the probability of given seismic events occurring on both known and unknown faults.

We calculated the peak ground acceleration for two separate probabilistic conditions; the two percent chance of exceedance in 50 years (2,475-year statistical return period) and the ten percent chance of exceedance in 50 years (475-year statistical return period). The peak ground acceleration values were calculated utilizing the USGS Unified Hazard Tool. The results of the probabilistic analyses are presented below in Table 2.

Table 2 – Probabilistic Peak Ground Accelerations for Active Faults

Probability of Exceedance	Statistical Return Period	Magnitude	Peak Ground Acceleration (g)
2% in 50 years	2,475 years	7.6	0.70
10% in 50 years	475 years	7.5	0.40

Reference: USGS Unified Hazard Tool (Dynamic: Conterminous U.S. 2008 v3.3.1) accessed July 13, 2020

Ground shaking can result in structural failure and collapse of structures or cause non-structural building elements (such as light fixtures, shelves, cornices, etc.) to fall, presenting a hazard to building occupants and contents. Compliance with provisions of the most recent version of the California Building Code (2019 CBC) should result in structures that do not collapse in an earthquake. Damage may still occur and hazards associated with falling objects or non-structural building elements will remain.

The potential for strong seismic shaking at the project site is high. Due to their proximity and historic rates of activity, the San Andreas and Hayward Faults present the highest potential for severe ground shaking. The significant adverse impact associated with strong seismic shaking is potential damage to structures and improvements.

Evaluation: *Less than significant with mitigation.*
Minimum recommendations include design of new structures in accordance with the provisions of the 2019 California Building Code or subsequent codes in effect when final design occurs. Recommended seismic design coefficients and spectral accelerations are presented in Section 5.1 of this report.

4.3 Liquefaction and Related Effects

Liquefaction refers to the sudden, temporary loss of soil strength during strong ground shaking. The strength loss occurs as a result of the build-up of excess pore water pressures and subsequent reduction of effective stress. While liquefaction most commonly occurs in saturated, loose, granular deposits, recent studies indicate that it can also occur in materials with relatively high fines content provided the fines exhibit lower plasticity. The effects of liquefaction can vary from cyclic softening resulting in limited strain potential to flow failure which cause large settlements and lateral ground movements.

Based on our subsurface exploration, the project site is underlain by a relatively thin layer of clayey soils over shallow Franciscan bedrock which are not susceptible to liquefaction. Therefore, we judge the likelihood of damage to the proposed improvements due to liquefaction is low.

Evaluation: *Less than significant. No mitigation measures are required.*

4.4 Settlement

Significant settlement can occur when new loads are placed over soft, compressible clays (e.g. Bay Mud) or loose soils. The medium stiff to stiff clayey soils and shallow Franciscan bedrock encountered in our borings are not highly compressible and new foundations will bear on bedrock, as discussed in Section 5.3. Therefore, we judge the risk of damage due to settlement induced by new structural loads is low.

Evaluation: *Less than significant. No mitigation measures are required.*

4.5 Seismic Densification

Seismic ground shaking can induce settlement in unsaturated, loose, granular soils. Settlement occurs as the loose soil particles rearrange into a denser configuration when subjected to seismic ground shaking. Varying degrees of settlement can occur throughout a deposit, resulting in differential settlement of structures founded on such deposits. Loose, granular soils were not encountered in our borings, so the risk of seismic densification impacting the new structures is generally low.

Evaluation: *Less than significant. No mitigation measures are required.*

4.6 Expansive Soils

Soil expansion occurs when clay particles interact with water causing seasonal volume changes in the soil matrix. The clay soil swells when saturated and then contracts when dried. This phenomenon generally decreases in magnitude with increasing confinement pressures at increasing depths. These volume changes may damage lightly loaded foundations, concrete slabs, pavements, retaining walls and other improvements. Expansive soils also cause soil creep on sloping ground. Laboratory testing on the near-surface soils indicate variable expansion potential. Plasticity Indexes (PI) on several samples tested were less than 20 (low plasticity), but one sample measured Expansion Index (EI) of 95 (medium-high). Expansion tests on the bedrock yielded an EI of 59 (medium expansion potential). Thus, there is a medium potential for damage due to expansive soils.

Evaluation: *Less than significant with mitigation.*
Soils subgrades and fills should be moisture conditioned above the optimum moisture content during site grading and maintained at this moisture content until imported aggregate base and/or surface flatwork is completed. Retaining structures should be designed with a soil creep load where walls retain sloping ground. Foundations should be designed to account for some expansive soil movement.

4.7 Erosion

Sandy soils on most slopes or clayey soils on steep slopes are susceptible to erosion when exposed to concentrated surface water flow. The potential for erosion is increased when established vegetation is disturbed or removed during normal construction activity.

The proposed improvements indicate that much of the site will be covered with new buildings, pavements, or concrete flatwork. Significant erosion is generally not anticipated within these areas. Drainage channels within the relatively steeply-sloping terrain show some active erosion, including gullies, localized small sloughs and raveling along the channel banks. Therefore, we judge the risk of erosion impacting the project is moderate.

Evaluation: *Less than significant with mitigation.*
Planned improvements or structures on shallow foundations should be setback from unimproved drainage channel. The recommended setback distance is a 3:1 inclination from the channel bed or 10 feet from top of bank, whichever is greater. The site drainage system should be designed to collect surface water from the maximum credible rainfall event and discharging it into an established storm drainage system. The project Civil Engineer is responsible for designing the site drainage system.

An erosion control plan could be developed prior to construction per the current guidelines of the California Stormwater Quality Association's Best Management Practice Handbook. Additionally, regular monitoring of the upslope areas should *be performed, particularly during and following periods of heavy rainfall. Regular maintenance of upslope areas should also be performed and should include maintaining vegetative cover on slopes, clearing debris from the v-ditches and drain inlets, and promptly repairing any erosion or shallow instabilities that occur.*

4.8 Slope Instability

The development will be located on a hillside which is locally inclined as steeply as about 2:1 but has an average slope of 3:1. Based on our aerial photo / topo review, reconnaissance, and exploration, there are areas of probable previous instability at the location shown on Figure 2. The depth of this probable instability is likely less than 10 feet. In other areas of the site, the surface soils are mapped as “creeping” and are prone to soil creep, occasional shallow sloughing, and debris flows in drainage channels which could result in debris impact to the rear of the structures. Deep excavations into the hillside can induce slope instability. We judge there will be a low risk of instability within the developed area of the site and a moderate risk of slope instability in the undeveloped areas within and upslope of the project site.

Evaluation: *Less than significant with mitigation.*
Supplemental exploration with exploratory trenches and geology site inspection/mapping further upslope should be performed to better evaluate the potential for instability. Most of the suspected areas of instability within the site will be removed as part of the planned excavation and building construction. Undeveloped areas of instability within the project site should be over-excavated, subsurface drainage installed, and backfilled with engineered fill. Global stability of the site should be checked as part of building wall design. Debris catchment structure or deflection wall/berm may be needed upslope of the planned buildings if debris flow paths cross planned structures, as discussed in Section 5.4.

4.9 Flooding

Flood Insurance Rate Maps prepared by the Federal Emergency Management Agency (FEMA, 2016) indicate the site is not mapped within a flood area. Based on the FEMA mapping, the risk of damage to future improvements due to flooding is considered low. The project Civil Engineer or Architect is responsible for site drainage and should evaluate localized flooding potential and provide appropriate mitigation.

Evaluation: *Less than significant. No mitigation measures are required. The project Civil Engineer is responsible for site drainage.*

4.10 Lurching and Ground Cracking

Lurching and associated ground cracking can occur during strong ground shaking. The ground cracking generally occurs along the tops of slopes where stiff soils are underlain by soft deposits, or along steep slopes or channel banks. These conditions do not exist at the site, therefore the risk of lurching and ground cracking at the project site is low.

Evaluation: *No significant impact.*
Mitigation: No mitigation measures are required.

4.11 Seiche and Tsunami

Seiche and tsunamis are short duration, earthquake-generated water waves in large enclosed bodies of water and the open ocean, respectively. The extent and severity of a seiche or tsunami would be dependent upon ground motions and fault offset from nearby active faults. The project site is at an increased elevation and not located near a large body of water. Therefore, seiche and tsunami events are not considered significant geologic hazards at the site.

Evaluation: *No significant impact.*
Mitigation: No mitigation measures are required.

4.12 Soil Corrosion

Corrosive soil can damage buried metallic structures, cause concrete spalling, and deteriorate rebar reinforcement. Laboratory testing was performed on representative samples of the near-surface site soils to evaluate pH, electrical resistivity, chloride and sulfate contents. These laboratory test results are presented on Figure A-8.

The results of our corrosivity testing indicate the upper soil layers have a pH of 6.32, a chloride concentration of 127.5 parts per million (ppm), and a sulfate concentration of 240 ppm. Per Caltrans Corrosion Guidelines (2003) a soil is considered corrosive if the pH level is less than 5.5, the chloride concentration is greater than 500 ppm, and/or the sulfate concentration is 2,000 ppm or greater. Therefore, based on the results of the corrosion testing, corrosive soil is not considered a significant geologic hazard at the project site.

Evaluation: *No significant impact.*
Mitigation: No mitigation measures are required.

4.13 Radon-222 Gas

Radon-222 is a product of the radioactive decay of uranium-238 and raduim-226, which occur naturally in a variety of rock types, mainly phosphatic shales, but also in other igneous, metamorphic, and sedimentary rocks. While low levels of radon gas are common, very high levels, which are typically caused by a combination of poor ventilation and high concentrations of uranium and radium in the underlying geologic materials can be hazardous to human health.

The project site is located in Napa County, California, which is mapped in radon gas Zone 3 by the United States Environmental Protection Agency (USEPA, 2018). Zone 3 is classified by the EPA as exhibiting a "low" potential for Radon-222 gas with average predicted indoor screening levels less than 2 pCi/L. Therefore, the potential for hazardous levels of radon at the project site is low.

Evaluation: *No significant impact.*
Mitigation: *No mitigation measures are required.*

4.14 Volcanic Eruption

Several active volcanoes with the potential for future eruptions exist within northern California, including Mount Shasta, Lassen Peak, and Medicine Lake in extreme northern California, the Mono Lake-Long Valley Caldera complex in east-central California, and the Clear Lake Volcanic Field, located in Lake County approximately 60-miles northeast of the project site. The most recent volcanic eruption in northern California was at Lassen Peak in 1917, while the most recent eruption at the nearest volcanic center to the project site, the Clear Lake Volcanic Field, was about 10,000 years ago. All of northern California's volcanic centers are currently listed under "normal" volcanic alert levels by the USGS California Volcano Observatory (USGS, 2018). While the aforementioned volcanic centers are considered "active" by the USGS, the likelihood of damage to the proposed improvements due to volcanic eruption is generally low.

Evaluation: *No significant impact.*
Mitigation: No mitigation measures are required.

4.15 Naturally Occurring Asbestos (NOA)

Naturally occurring asbestos is commonly found in association with serpentinite and associated ultramafic rock types. These rocks are a major constituent of the Franciscan Complex, which underlies vast portions of the greater San Francisco Bay Area. The site is underlain by relatively thick native alluvial soils. Therefore, the likelihood that significant deposits of naturally-occurring asbestos will be encountered at the site is low.

Evaluation: *No significant impact.*
Mitigation: No mitigation measures are required.

4.16 Hazardous Materials

Hazardous materials were not observed during our subsurface exploration. While environmental testing for hazardous materials was beyond the scope of our services, we did observe enclosures that contain HVAC units and other industrial equipment that has the potential for creating hazardous materials. Therefore, we judge the potential for hazardous materials being present on the project site, currently or in the future, is low to moderate.

Evaluation: *Less than significant with mitigation.*
Mitigation: The campus should comply with all local, state, and federal guidelines to minimize the exposure to hazardous materials. If a possible hazardous material spill occurs on campus, a qualified environmental specialist should be consulted.

4.17 Seiche and Tsunami

Seiche and tsunamis are short duration, earthquake-generated water waves in large enclosed bodies of water and the open ocean, respectively. The extent and severity of a seiche or tsunami would be dependent upon ground motions and fault offset from nearby active faults. The project site is at an increased elevation and not located near a large body of water. Therefore, seiche and tsunami events are not considered significant geologic hazards at the site.

Evaluation: *No significant impact.*
Mitigation: No mitigation measures are required.

5.0 CONCLUSIONS AND RECOMMENDATIONS

Based on the results of our investigation, we conclude the geologic and geotechnical site conditions are suitable for the proposed improvements. The primary geotechnical considerations will include designing the improvements to resist strong seismic ground shaking, excavation conditions, potential shoring, and potential instability of the upslope areas above the proposed development that could impact. Additional discussion and preliminary conclusions and recommendations addressing these and other considerations are presented in the following sections.

5.1 Seismic Design

Minimum mitigation of ground shaking includes seismic design of new structures in conformance with the provisions of the most recent edition (2019) of the California Building Code. The magnitude and character of these ground motions will depend on the particular earthquake and the site response characteristics. Based on the interpreted subsurface conditions and proximity of several nearby faults, we recommend the CBC coefficients and site values shown in Table 3, be used to calculate the design base shear of new improvements as applicable.

Table 3 – 2019 California Building Code Seismic Design Criteria

Parameter	Design Value
Site Class	C
Site Latitude	38.0139°N
Site Longitude	-122.5428°W
Spectral Response (short), S_S	1.5 g
Spectral Response (1-sec), S_1	0.6 g
Site Coefficient, F_a	1.0
Site Coefficient, F_v	1.0
Spectral Response (Short), S_{MS}	1.8 g
Spectral Response (1 sec), S_{M1}	0.84 g
Design Spectral Response (short), S_{DS}	1.2 g
Design Spectral Response (1 sec), S_{D1}	0.56 g
MCE_G PGA Adjusted, PGA_M	0.605 g

Reference: ATC Hazard by Location, accessed on July 13, 2020.

5.2 Site Grading

Site grading and earthwork should be performed in accordance with the recommendations and criteria outlined in the following sections.

5.2.1 Site Preparation

Clear over-sized debris and organic material from areas are to be graded. Debris, rocks larger than six inches, and vegetation are not suitable for structural fill and should be removed from the site.

Where fills or other structural improvements are planned, the subgrade surface should be scarified to a depth of eight inches, moisture conditioned to above the optimum moisture content, and compacted to at least 90 percent relative compaction. Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density, as determined by ASTM D1557. Subgrade preparation should extend a minimum of five feet beyond the planned building envelopes in all directions. The subgrade should be firm and unyielding when proof-rolled with heavy, rubber-tired construction equipment. If soft, wet or otherwise unsuitable materials are encountered at subgrade elevation during construction, we will provide supplemental recommendations to address the specific condition.

5.2.2 Excavations

Site excavations for new underground utilities, retaining walls, building foundations and other improvements will encounter from 3 to 10 feet of medium stiff clayey soils over Franciscan bedrock of variable weathering, strength and hardness. The bedrock encountered in our borings generally exhibited low to moderate hardness and strength and is highly to completely weathered. Temporary (steeper) cut slopes may be required during construction. For planning purposes, the soil layer may be designed for a Cal-OSHA Type "C" soil profile, and the underlying weathered bedrock as Cal-OSHA Type "A" soil profile.

Temporary, short (6-ft typical), vertical cuts are possible during dry conditions and for short term excavations, such as cuts for soil-nail wall construction. However, adversely bedded rock or seepage/weak soils near the ground surface may require lower cut heights, and/or temporary vertical supports for soil nails above the cut.

Based on our subsurface exploration, we judge that most of the site excavation can be performed with typical equipment, such as medium-size dozers and excavators. However, Franciscan bedrock contains inclusions and zones of harder, more resistant rock which cannot be efficiently excavated with typical equipment and requires specialized techniques or equipment to excavate (e.g. jackhammers or hoe-rams). Therefore, we recommend inclusion of a line item and clear definition for "hard rock excavation" in the project bid documents. If hard rock is encountered during construction which prohibits excavation to the required depths, we should be consulted to observe conditions and revise our recommendations and/or design criteria as appropriate. Reducing planned excavation depths will also reduce the volume of rock excavation and resulting costs.

5.2.3 Fill Materials, Placement and Compaction

Fill materials should consist of non-expansive materials that are free of organic matter, have a Liquid Limit of less than 40 (ASTM D 4318), a Plasticity Index of less than 20 (ASTM D 4318), and a minimum R-value of 20 (California Test 301). The fill material should contain no more than 50 percent of particles passing a No. 200 sieve and should have a maximum particle size of four inches. Onsite soils may be suitable for use as fill, provided they meet the criteria specified above. Any imported fill material needs to be tested to determine its suitability.

Fill materials should be moisture conditioned to above the optimum moisture content prior to compaction. Properly moisture conditioned fill materials should subsequently be placed in loose, horizontal lifts of eight inches-thick or less and uniformly compacted to at least 90 percent relative compaction. Where fill thicknesses are greater than five feet, fill materials should be compacted to at least 92 percent relative compaction. In pavement areas, the upper 12 inches of fill should be compacted to at least 95 percent relative compaction. The maximum dry density and optimum moisture content of fill materials should be determined in accordance with ASTM D1557.

5.2.4 Bulking and Shrinkage

During site grading, bulking or shrinkage can occur as the soil and bedrock is excavated and replaced as compacted fill. Bulking and shrinkage estimates are variable based on soil type, loading (thickness of fill) and degree of compaction. Some rough estimates are presented below. A laboratory testing program that includes compaction curves is recommended to refine estimates of grading quantities.

For excavation of on-site soil for use as fill placed and compacted 90% relative compaction, we estimated net volume change of 10% shrinkage. For on-site bedrock excavated for use as fill placed and compacted 90% relative compaction, we estimated net volume change of 5 to 10% bulking.

Due to significant bulking of materials placed in trucks for off-haul and disposal, we recommend excavated soil for removal and disposal be bid on a per ton basis.

For deep fills, some compression of the deep soil will occur from the overburden load and will likely cause some settlement at the ground surface. Long term compression settlement is estimated at 0.5 to 1% of the fill height and will typically occur within 5 to 10 years after construction.

5.3 Foundation Design

Bedrock is relatively shallow throughout the site, with about 3 to 10 feet of clayey soils overlying Franciscan Melange. Shallow foundations can be utilized provided they maintain uniform support on competent bedrock. Since the planned grading involves cutting into the hillside for building pads, the downslope sides of the building pads may expose soil, therefore, footings should be deepened to provide uniform bearing support on the weathered bedrock to minimize potential for differential settlement, the. Drilled, cast-in-place piers could also be utilized for the building foundation to extend through soils and into the underlying bedrock. Drilled piers or rock anchors can be utilized for overturning resistance. Geotechnical foundation design criteria are presented in Table 4.

Table 4 – Foundation Design Criteria

Shallow Spread Footings	
Minimum depth:[1]	18 inches
Allowable bearing capacity:[2]	
Weathered Bedrock	3,000 psf
Base friction coefficient:	0.35
Lateral passive resistance:[3,4]	
Sandy Clay Soils	300 pcf
Weathered Bedrock	450 pcf
Drilled Piers of Rock Anchors	
Min. Diameter:	
Drilled Pier	18 inches
Rock Anchor	6 inches
Minimum Pier Embedment into Bedrock:	10 feet
Allowable skin friction [2,5,6]:	
Sandy Clay Soils	1,000 psf
Weathered Bedrock	2,500 psf
Lateral passive resistance[7]:	
Sandy Clay Soils	250 pcf
Weathered Bedrock	400 pcf

Notes:
(1) Foundations to bear on weathered bedrock. Maintain at least 10 feet horizontal distance from base of footing to slope.
(2) May increase design values by 1/3 for total design loads including wind or seismic.
(3) Equivalent fluid pressure. Not to exceed 4000 psf.
(4) Ignore uppermost foot of resistance.
(5) Anchors should be specified with a minimum bonded length and minimum capacity. All rock anchors shall be double corrosion-protected anchors and should be tested to at least 1.33 times the design load per the "Recommendations for Prestressed Rock and Soil Anchors" by the Post-Tensioning Institute, Phoenix, Arizona.
(6) Use 80 percent of skin friction for uplift design.
(7) Apply lateral passive resistance over width of two pier diameters.

5.4 Retaining Walls

We understand retaining walls will be utilized to support roadway fill and stabilize cuts made to create level building pads. Taller site retaining walls can be constructed by laying back slopes, construction walls and backfilling, or by making vertical cuts supported with shotcrete-faced and soil walls. The soil nail walls can be designed as a temporary shoring wall, or could be part of a permanent building wall. Reinforced earth walls may be a good choice for site walls that support fills.

Retaining walls that can deflect at the top such as site walls can be designed using the unrestrained criteria shown in Table 5. Walls that are structurally connected at the top and not allowed to deflect, such as basement or tied-back walls are considered restrained. Restrained conditions are commonly designed using a uniform earth pressure distribution rather than an equivalent fluid pressure. Lateral support can be obtained from either passive soil resistance (i.e., keyways) or frictional sliding resistance of footings or from tiebacks. In addition to the soil loads, the retaining walls should be designed to resist temporary vehicular or seismic loads.

Table 5 - Retaining Wall Design Criteria

Foundations: See Table 4

Active Earth Pressure	Unrestrained[,2]	Restrained[,3]
Level Ground	40 pcf	30 X H psf
2:1 Slope	60 pcf	40 X H psf

Seismic Surcharge[3,4]	15 x H psf
Vehicular Surcharge[3,4]	50 psf upper 10 feet
Tiebacks or Soil Nails[5]:	
Minimum Diameter:	5 inches
Design Skin Friction:	2,500 psf
Unbonded Zone:	0.7 x Wall Height, 6 Feet Min

	Phi[6]	C (psf)[7]	Gamma (pcf)[8]
Sandy Clay Soils (upper 5’)	30°	750	125
Weathered Bedrock	32°	1,500	130

Notes:
(1) Interpolate earth pressures for intermediate slopes.
(2) Equivalent fluid pressure.
(3) Rectangular distribution. H = Wall Height = top of soil backfill to bottom of wall.
(4) The factor of safety for short-term seismic conditions can be reduced to 1.1 or greater.
(5) Tiebacks should be specified with a minimum bonded length and minimum capacity. All tiebacks shall be double corrosion protected anchors that are installed and tested to at least 1.33 times the design load per the “Recommendations for Pre-stressed Rock and Soil Anchors” by the Post-Tensioning Institute, Phoenix, Arizona.
(6) Angle of Internal Friction, effective stress.
(7) Apparent (effective) Cohesion, for seismic conditions 250 psf of additional cohesion may be included.
(8) Unit Weight of Soil
(9) Ignore skin friction within active wedge of wall (approximately equal to wall height).

All walls higher than 3-feet require drainage to prevent the build-up of hydrostatic pressure. Either Caltrans Class 1B permeable material within filter fabric, drainage panels, or Caltrans Class 2 permeable material can be used. The project Architect should design a water-proofing system for walls adjacent to living space. The drainage should be collected in 4-inch, perforated, Schedule 40 PVC drain line placed at the base of the wall or discharged through weep-holes in the case of soil nail or cast-in-place concrete walls. Seepage collected in the drains should be conveyed in a closed pipe system to a suitable discharge outlet well away from the structures.

To maintain the wall drainage system, clean-outs must be provided for perforated pipes at the upstream end. Sweep fittings should be used at all major changes in direction. A typical retaining wall drain detail is shown on Figure 6. Retaining wall backfill should be compacted in accordance with the recommendations presented in site grading.

5.5 Debris Barriers

As discussed above, debris impact with the planned structures could occur if instability upslope of the project results in the release of a sufficient volume of debris. Several methods to mitigate debris impact are available.

An earth berm could be constructed behind the proposed development area that could redirect any debris into the existing channels on to the north and south ends of the building area. The earth berm should be at least 8-feet high as measured from the existing ground surface. Side slopes should be no steeper than 2:1 (horizontal:vertical). The berm should be constructed of select fill, placed on a prepared subgrade, and compacted in lifts to 92% relative compaction as described in Section 5.2.3. A minimum 10-foot wide access route should be maintained to facilitate access for trucks and equipment to remove slide debris, maintain the berm, and maintain associated drainage facilities as needed. The access road should extend to form a spillway "notched" into the crest of the catchment berm. The access route should be graded with a minimum 5% cross-slope to force surface flow runoff into an adjacent infiltration trench or stormwater detention basin.

Another mitigation option could be a new debris-catchment structure with a minimum height of six feet and sited about 10 to 20 feet upslope from the planned buildings. While various structure types are feasible, a debris fence consisting of a combination of mesh, posts, and anchored cables would likely be relatively cost-effective and would allow for entrapment of debris upslope of the concrete v-ditch above the soil nail wall. Regular maintenance, including visual inspections and as-needed removal of debris would need to be performed to confirm the catchment structure is performing as intended.

5.6 Interior Concrete Slabs-On-Grade

Reinforced concrete slab-on-grade floors are judged to be appropriate for the proposed structures. The concrete slabs-on-grade may be poured monolithically or separated with a cold joint. We recommend that interior concrete slabs have a minimum thickness of five inches and be reinforced with steel reinforcing bars (not mesh) with rebar extending through crack control joints. Slabs should be placed on a moist subgrade to reduce potential for future shrink/swell behavior. The project Structural Engineer should specifically design the concrete slabs, including locations of crack control joints.

To reduce the potential for moisture to move upward through the slab, a four-inch layer of clean, free draining, ¾-inch angular gravel should be placed beneath interior concrete slabs to form a capillary moisture break. The gravel must be placed on a properly moisture conditioned and compacted subgrade that has been approved by the Geotechnical Engineer. A plastic membrane vapor barrier, 15 mils or thicker, should be placed over the compacted base rock. The vapor barrier shall meet the ASTM E1745 Class A requirements and be installed per ASTM E1643. Eliminating the capillary moisture break and/or plastic vapor barrier may result in excess moisture intrusion through the floor slabs resulting in poor performance of floor coverings, mold growth, or other adverse conditions.

We note that over time, placing sand between the vapor barrier and concrete is becoming less common because of elevated interior moisture contents. If sand is used, it should be dry, and if it is not used, the slab should be carefully designed with a lower water-cement ratio (generally less than 0.45) since eliminating the sand can cause cracking or “curling” of the new concrete. For slabs that are not sensitive to moisture vapor, we recommend at least four inches of Class 2 aggregate base (Caltrans, 2015) compacted to 95 percent relative compaction.

Where the gravel capillary break layer is placed beneath slabs, there is a possibility that water will tend to collect in the gravel layer and become trapped. If this condition occurs, the potential for moisture issues at the surface of the slab will be increased. One method of minimizing the potential for this to occur would be to construct a subdrain trench through and just below the gravel layer so that water collected in this area can escape. The subdrain should extend at least 12 inches below the base of the slab and 6 inches below the bottom of the gravel layer, and would consist of a four-inch-diameter, perforated pipe (Schedule 40 PVC) surrounded by gravel with non-woven filter fabric (Mirafi 140N or approved equal) lining the trench. The subdrain would connect to the gravel layer beneath the slab, and the pipe should lead (at a minimum 0.5 percent slope) to a storm drain or another suitable outlet point. The perforated pipe should transition to nonperforated pipe at a point three feet inside the perimeter footing of the structure. A compacted clayey soil plug should be used at the point where the outlet pipe penetrates the perimeter footing to prevent seepage from back-flowing into the underslab gravel layer.

5.7 Exterior Concrete Slabs

Exterior concrete walkway slabs and other concrete slabs that are not subjected to vehicle loads should be a minimum of four inches thick and underlain with four inches or more of Class 2 aggregate base. The aggregate base should be moisture conditioned to near optimum and compacted to at least 95 percent relative compaction. The upper eight inches of subgrade on which aggregate base is placed should be prepared as previously discussed under Section 5.2.

Where improved performance is desired (i.e., reduced risks of cracking or small movements), exterior slabs can be thickened to five inches and reinforced with steel reinforcing bars (not welded wire mesh). We recommend crack control joints no farther than six feet apart in both directions, and that the reinforcing bars extend through the control joints. Some movement or offset at sidewalk joints should be expected as the underlying soils expand and shrink from seasonal moisture changes.

5.8 Site and Foundation Drainage

New grading could result in adverse drainage patterns causing water to pond around the new buildings. Careful consideration should be given to design of finished grades at the site. We recommend that the building areas be raised slightly and that the adjoining landscaped areas be sloped downward at least 0.25 feet for five feet (five percent) from the perimeter of building foundations. Where hard surfaces, such as concrete or asphalt adjoin foundations, slope these surfaces at least 0.10 feet in the first five feet (two percent).

Roof gutter downspouts may discharge onto pavements but should not discharge onto landscaped areas immediately adjacent to the buildings. Provide area drains for landscape planters adjacent to buildings and collect downspout discharges into a tight pipe collection system that discharges well away from the building foundations. Site drainage should be discharged away from the building area and outlets should be designed to reduce erosion. Site drainage improvements should be connected into an established storm drainage system.

5.9 Underground Utilities

Site excavations for new underground utilities and other improvements will encounter up to about four to nine feet of medium stiff to stiff clayey soils over Franciscan bedrock of variable weathering, strength, and hardness. Trench excavations having a depth of five feet or more must be excavated and shored in accordance with OSHA regulations, as discussed in Section 5.2.

Unless otherwise recommended by the pipe manufacturer, pipe bedding and embedment materials should consist of well-graded sand with 90 to 100 percent of particles passing the No. 4 sieve and no more than five percent finer than the No. 200 sieve. Crushed rock or pea gravel may also be considered for pipe bedding. Provide the minimum bedding thickness beneath the pipe in accordance with the manufacturer's recommendations (typically three to six inches). Trench backfill may consist of on-site soils, provided that the soil meets the fill criteria outlined in Section 5.2. Trench backfill should be moisture conditioned and placed in thin lifts and compacted to at least 90 percent. The upper 12 inches of backfill should be compacted to at least 95 percent in new pavement areas. The Contractor should use equipment and methods that are suitable for work in confined areas without damaging utility conduits.

5.10 Pavements

5.10.1 Asphalt-Concrete Pavement Sections

New pavements are expected to include both rigid concrete pavements flexible asphalt pavements. We have calculated thicknesses for asphalt pavements in accordance with Caltrans procedures for flexible pavement design. Our calculations assume an R-value of 10 for subgrade soils and a range of Traffic Indices from 4.0 to 7.0 depending on the expected traffic loads for a twenty-year design life. In general, areas expected to experience loading from heavy vehicles should be designed using the higher Traffic Index, while parking areas and other lightly-loaded areas can utilize a thinner pavement section based on the lower Traffic Index. The recommended pavement sections are presented in Tables 6 and 7.

Table 6 – Preliminary Asphalt-Concrete Pavement Sections

Traffic Index[1]	Asphalt Concrete (inches)	Aggregate Base (inches)
4.0	3.0	7.0
5.0	3.5	8.0
6.0	5.0	8.5
7.0	5.0	13.0

(1) Traffic Index for final pavement design to be determined by the project Civil Engineer.

In areas where concrete pavement is planned, the concrete pavement design should conform to recommendations for rigid pavements from the Portland Cement Association (PCA, 1984). Concrete reinforcement should consist of No. 4 rebar (Grade 40 or higher) spaced at a maximum of 18 inches on center in both directions. Recommended design criteria for rigid pavements is summarized in Table 7.

Table 7 – Design Criteria for Concrete Pavements

Parameter	**Value**
Minimum Concrete Thickness	5 inches
Minimum Aggregate Base Thickness	4 inches
Modulus of Rupture (ASTM C78)	600 psi
Maximum Water-Cement Ratio (by weight)	0.45
Modulus of Subgrade Reaction	100 pci
Joint Spacing	12 to 15 feet

In pavement areas, the upper 12 inches of subgrade should be compacted to at least 95 percent relative compaction. The aggregate base and asphalt-concrete should conform to the most recent version of Caltrans Standard Specifications and should be compacted to at least 95 percent relative compaction. Additionally, the subgrade and aggregate base should be firm and unyielding under heavy, rubber-tired construction equipment.

5.10.2 Permeable Paver Section

Based on the results of our subsurface exploration and laboratory testing, we have performed pavement section analyses, taking into account both permeability and traffic loading. Based on our analyses, traffic loading appears to be the controlling design factor. Therefore, we have developed alternate structural sections for the new roadway using a variety of Traffic Indices (T.I.'s). We understand that a 40-year design life is desired, and we have thus shown slightly higher Traffic Indices than would be typical for a pedestrian thoroughfare. Structural sections were designed in general accordance with Caltrans procedures for flexible pavement design (1990) using a design R-Value for the subgrade soil of 10. The recommended permeable structural sections are presented below in Table 8.

Table 8 - Permeable Paver Sections

Permeable Pavers (3" minimum thickness)

T.I.	ASTM No. 8 Stone[1]	ASTM No. 57 Stone[2]	Subgrade[3]
4.0	2"	9"	95% R.C.
5.0	2"	12"	95% R.C.
6.0	2"	16"	95% R.C.
7.0	2"	21"	95% R.C.

1. ASTM No. 8 Stone shall conform to the ASTM grading and durability criteria and shall be crushed stone with 90% fractured faces (rounded gravel shall not be allowed). Caltrans Class 1A permeable material may be substituted provided all the criteria presented in Section 68 of the Caltrans Standard Specifications are met and the stone is crushed as discussed above.
2. ASTM No. 57 Stone shall conform to the ASTM grading and durability criteria and shall be crushed stone with 90% fractured faces (rounded gravel shall not be allowed). Caltrans Class 1B permeable material may be substituted provided all the criteria presented in Section 68 of the Caltrans Standard Specifications are met and the stone is crushed as discussed above.
3. The subgrade soil for the pavement section shall be moisture conditioned and compacted to at least 95% relative compaction to produce a firm and unyielding surface when proof rolled with heavy construction equipment. A triaxial geogrid, such as Mirafi TX-5 or equivalent, should be placed on the prepared subgrade to ensure stability when the subgrade is saturated.

6.0 SUPPLEMENTAL GEOTECHNICAL SERVICES

This report provides preliminary geotechnical recommendations and design criteria based on the current development plan. As the development plan is refined and to further evaluate geologic conditions as discussed, we should perform supplemental exploration and laboratory testing as needed to update this report for the design level final report.

As project plans are nearing completion, we should review them to confirm that the intent of our geotechnical recommendations has been incorporated. We can also consult with project team to supplement or clarify geotechnical recommendations, if needed. If requested, we can perform analyses and prepare plans, details, technical specifications, and calculation package for soil nail or tied-back retaining structures.

During construction, we should be present intermittently to observe foundation excavations, fill placement, trench backfill, retaining wall drainage and backfill and other geotechnical-related work items. The purpose of our observation and testing is to confirm that site conditions are as anticipated, to adjust our recommendations and design criteria if needed, and to confirm that the Contractor's work is performed in accordance with the project plans and specifications.

7.0 LIMITATIONS

This report has been prepared in accordance with generally accepted geotechnical engineering practices in the San Francisco Bay Area at the time the report was prepared. This report has been prepared for the exclusive use of the project Owner and/or their assignees specifically for this project. No other warranty, expressed or implied, is made. Our evaluations and recommendations are based on the data obtained during our subsurface exploration program and our experience with soils in this geographic area.

8.0 LIST OF REFERENCES

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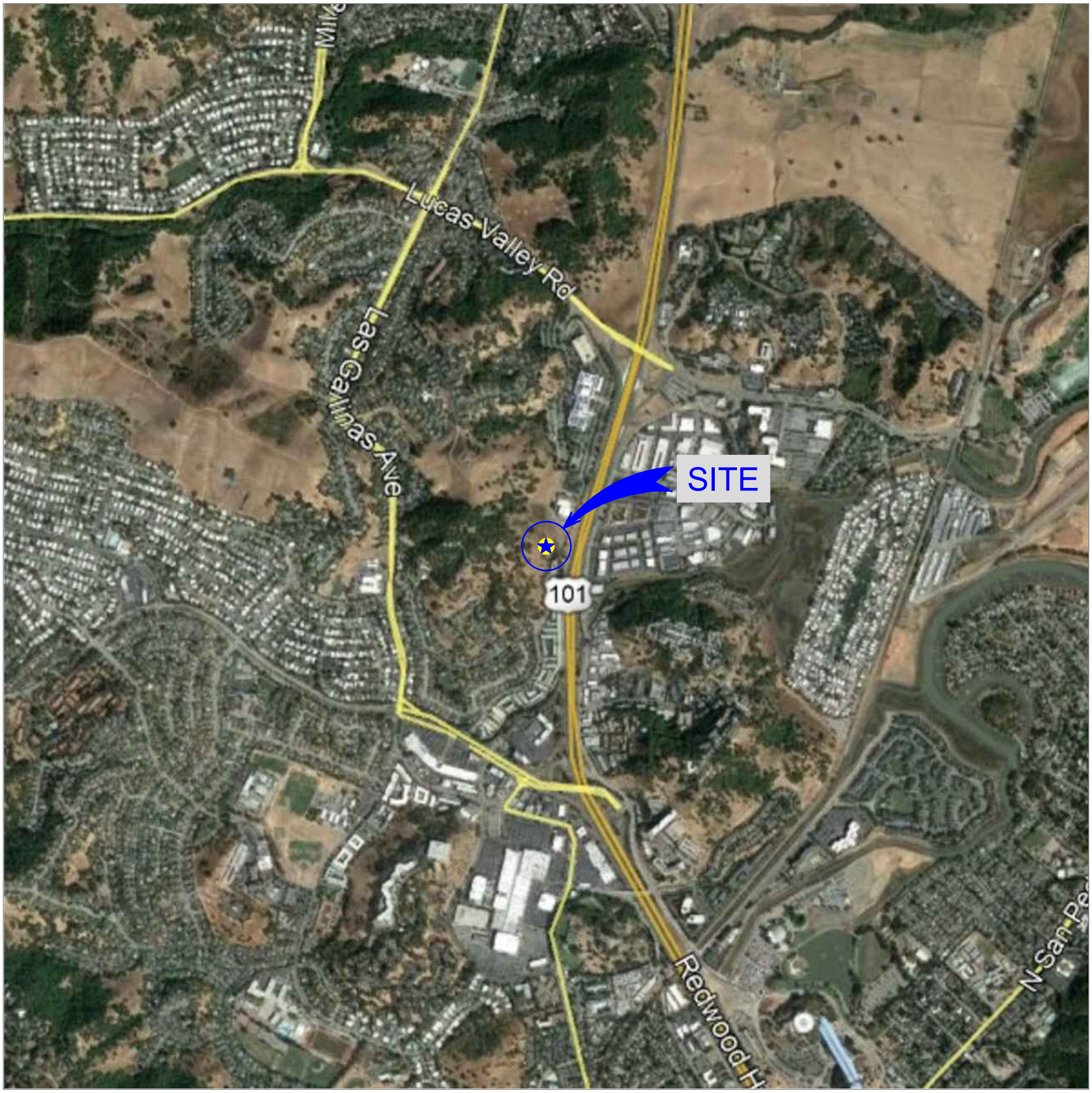

SITE COORDINATES
LAT. 38.0151°
LON. -122.5430°

SITE LOCATION
N.T.S.

NORTH

REFERENCE: Google Earth, 2019

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FILENAME: 3013.001 Standard Figures.dwg

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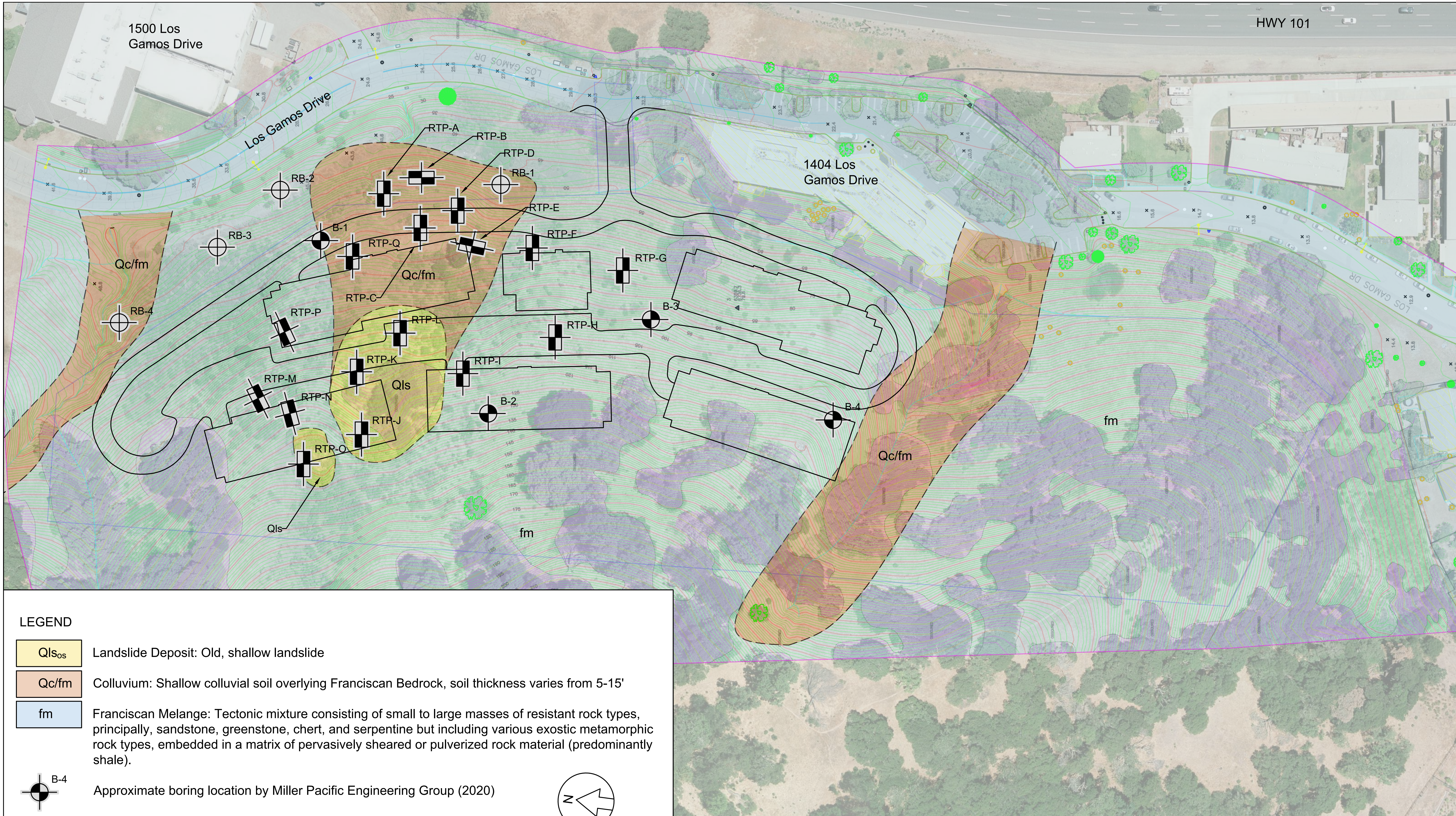
SITE LOCATION MAP

The Neighborhood at Los Gamos
San Rafael, California

Project No. 3013.001 Date: 7/13/2020

Drawn ZMS
Checked

1
FIGURE



LEGEND

Symbol	Description
Qls_{os}	Landslide Deposit: Old, shallow landslide
Qc/fm	Colluvium: Shallow colluvial soil overlying Franciscan Bedrock, soil thickness varies from 5-15'
fm	Franciscan Melange: Tectonic mixture consisting of small to large masses of resistant rock types, principally, sandstone, greenstone, chert, and serpentine but including various exostic metamorphic rock types, embedded in a matrix of pervasively sheared or pulverized rock material (predominantly shale).
B-4	Approximate boring location by Miller Pacific Engineering Group (2020)
RB-4	Approximate boring location by Salem Howes Associates (1998)
RTP-Q	Approximate test pit location by Salem Howes Associates (1998)

N

SCALE
0 50 100 200 FEET

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SITE PLAN AND PRELIMINARY GEOLOGIC MAP

The Neighborhood
San Rafael, California

Project No. 3013.001 Date: 07/20/2020

Drawn ZMS
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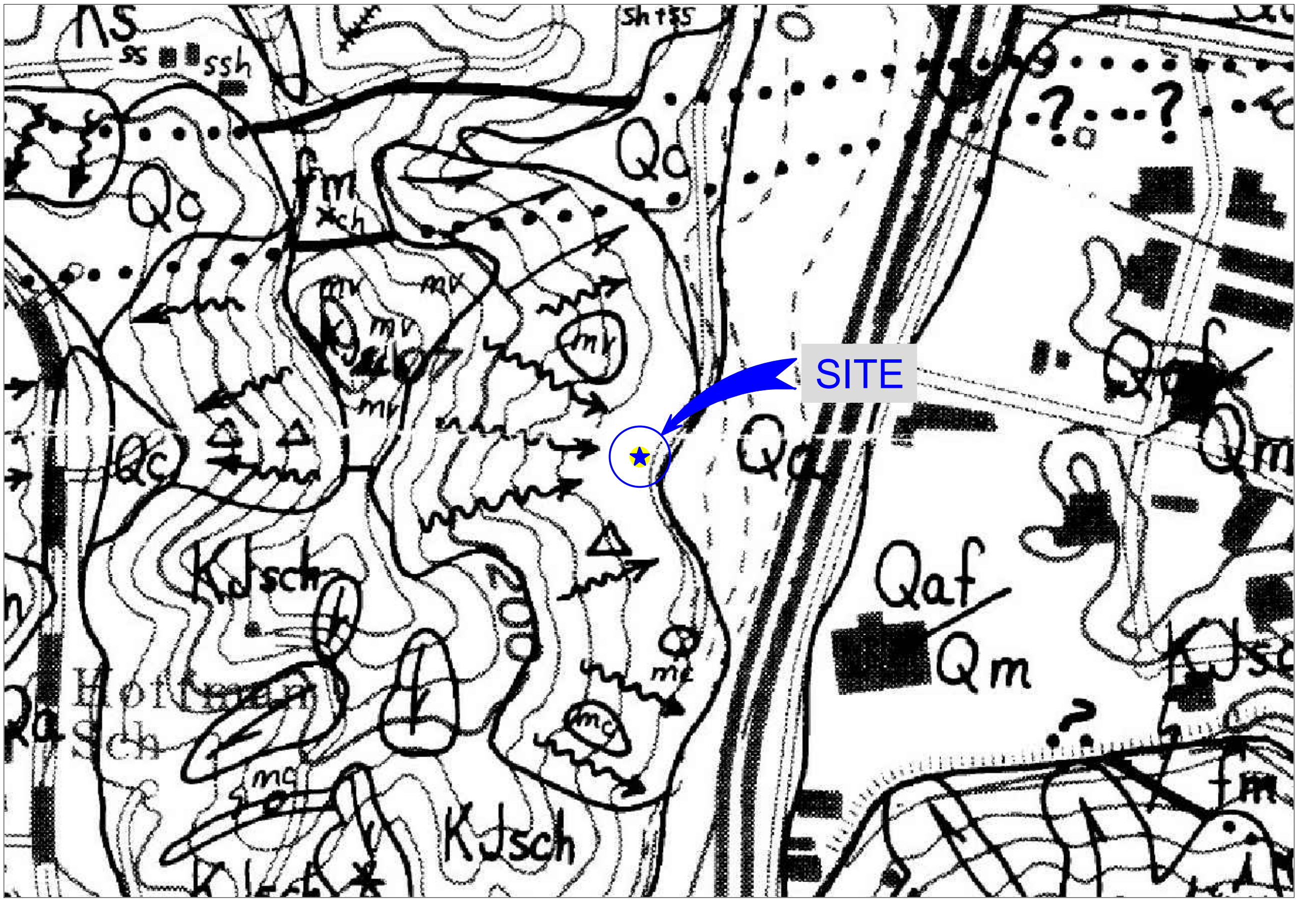
2
FIGURE



REGIONAL GEOLOGIC MAP

(NOT TO SCALE)

NORTH

LEGEND

Qa	Alluvium: Unconsolidated deposits of clay, silt, sand, and gravel underlying the bottom lands of the main stream valleys, consisting of materials transported and deposited by streams.
Qaf	Artificial Fill: Deposits of rock, soil, or garbage placed by man upon natural surfaces, mostly for engineering purposes.
Qm	Bay Mud: Marshlands, former marshlands, and mudflats. Thick deposits of unconsolidated, low density, semi-fluid, highly comporessible, highly impermeable, silty clay. Rich in organic material.
Kjsch	Schist, Phillite, and Semi-Schist: with associated meta-chert and volcanic rocks. Predominantly slightly to well-foliated or lineated metamorphosed sedimentary and volcanic rocks.
fm	Franciscan Melange: a tectonic mixture consisting of small to large masses of resistant rock types, principally sandstone, greenstone, chert, and serpentine, but including various exotic metamorphic rock types, embedded in a matrix of pervasively sheared or pulverized rock material.
	Creep Zones: Slopes exhibiting evidence of continuous or intermittent downslope creep of surface zone.

Reference: Rice, Salem J. "Geology of the Eastern Part of the San Rafael Area" in Geology for Planning in Central and Southeastern Marin County, California. OFR 76-2 SF Plate 1C. Map Scale 1:12000

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REGIONAL GEOLOGIC MAP

The Neighborhood at Los Gamos
San Rafael, California

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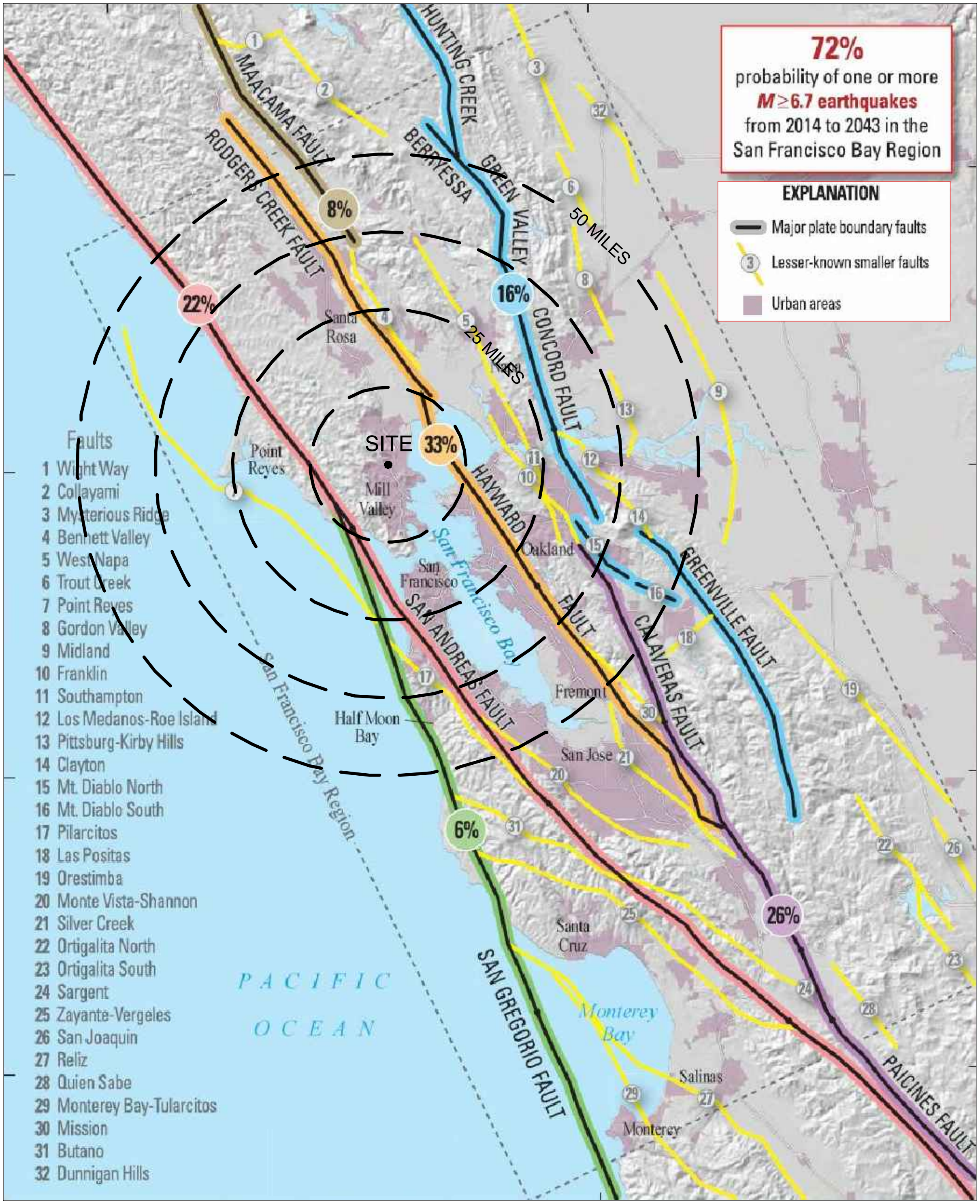
Project No. 3013.001 Date: 7/13/2020

FIGURE 3



SITE COORDINATES
LAT. 38.0151°
LON. -122.5430°

SCALE
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NORTH

DATA SOURCE:
1) U.S. Geological Survey, U.S. Department of the Interior, "Earthquake Outlook for the San Francisco Bay Region 2014-2043", Map of Known Active Faults in the San Francisco Bay Region, Fact Sheet 2016-3020, Revised August 2016 (ver. 1.1).

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FILENAME: 3013.001 Standard Figures.dwg

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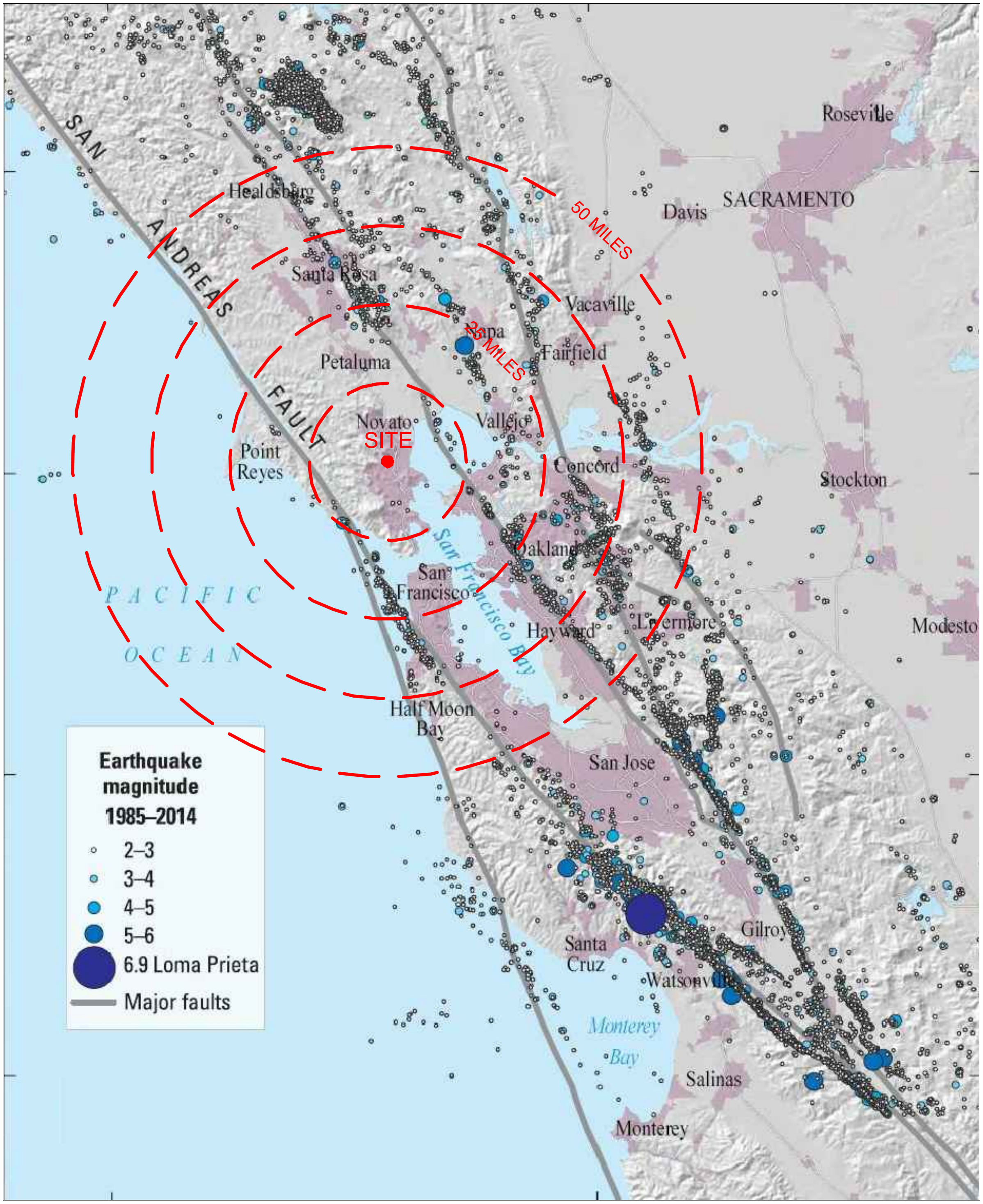
ACTIVE FAULT MAP

The Neighborhood at Los Gamos
San Rafael, California

Project No. 3013.001 Date: 7/13/2020

Drawn ZMS
Checked

4
FIGURE



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DATA SOURCE:
1) U.S. Geological Survey, U.S. Department of the Interior, "Earthquake Outlook for the San Francisco Bay Region 2014-2043", Map of Earthquakes Greater Than Magnitude 2.0 in the San Francisco Bay Region from 1985-2014, Fact Sheet 2016-3020, Revised August 2016 (ver. 1.1).

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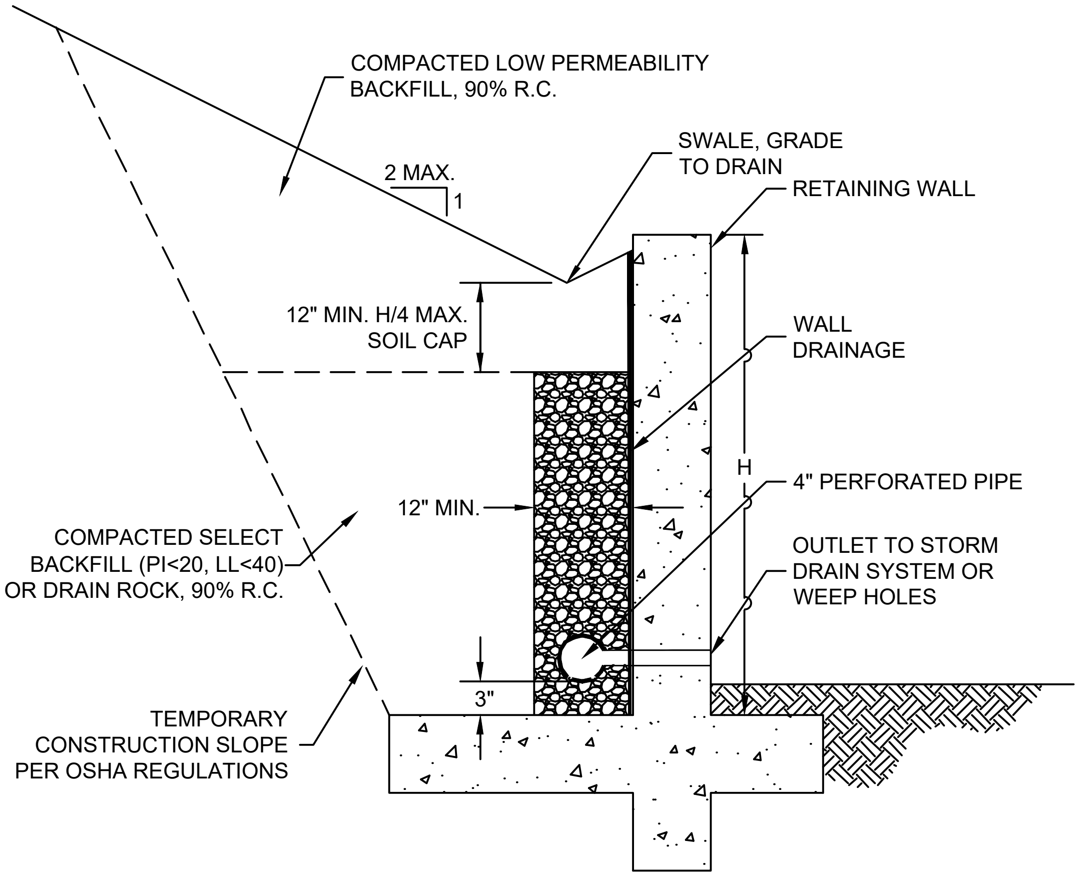
HISTORIC EARTHQUAKE MAP

The Neighborhood at Los Gamos
San Rafael, California

Project No. 3013.001 Date: 7/13/2020

Drawn ZMS
Checked

5
FIGURE



NOTES:

1. Wall drainage should consist of clean, free draining 3/4 inch crushed rock (Class 1B Permeable Material) wrapped in filter fabric (Mirafi 140N or equivalent) or Class 2 Permeable Material. Alternatively, pre-fabricated drainage panels (Miradrain G100N or equivalent), installed per the manufacturers recommendations, may be used in lieu of drain rock and fabric.

2. All retaining walls adjacent to interior living spaces shall be water/vapor proofed as specified by the project architect or structural engineer.

3. Perforated pipe shall be SCH 40 or SDR 35 for depths less than 20 feet. Use SCH 80 or SDR 23.5 perforated pipe for depths greater than 20 feet. Place pipe perforations down and slope at 1% to a gravity outlet. Alternatively, drainage can be outlet through 3" diameter weep holes spaced approximately 20' apart.

4. Clean outs should be installed at the upslope end and at significant direction changes of the perforated pipe. Additionally, all angled connectors shall be long bend sweep connections.

5. During compaction, the contractor should use appropriate methods (such as temporary bracing and/or light compaction equipment) to avoid over-stressing the walls. Walls shall be completely backfilled prior to construction in front of or above the retaining wall.

6. Refer to the geotechnical report for lateral soil pressures.

7. All work and materials shall conform with Section 68, of the latest edition of the Caltrans Standard Specifications.

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TYPICAL WALL BACKDRAIN

The Neighborhood at Los Gamos
San Rafael, California

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Project No. 3013.001 Date: 7/13/2020

FIGURE 6

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APPENDIX A – LABORATORY TESTING

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 gravelly sands, little or no fines
		SP	Poorly-graded sands or gravelly 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

PI	PLASTICITY INDEX
LL	LIQUID LIMIT
SA	SIEVE ANALYSIS
HYD	HYDROMETER ANALYSIS
P200	PERCENT PASSING NO. 200 SIEVE
P4	PERCENT PASSING NO. 4 SIEVE

SAMPLER TYPE

- MODIFIED CALIFORNIA
- HAND SAMPLER
- STANDARD PENETRATION TEST
- ROCK CORE
- THIN-WALLED / FIXED PISTON
- X DISTURBED OR BULK SAMPLE

NOTE: Test boring and test pit logs are an interpretation of conditions encountered at the excavation location during the time of exploration. Subsurface rock, soil or water conditions may vary in different locations within the project site and with the passage of time. Boundaries between differing soil or rock descriptions are approximate and may indicate a gradual transition.

STRENGTH TESTS

UC	LABORATORY UNCONFINED COMPRESSION
TXCU	CONSOLIDATED UNDRAINED TRIAXIAL
TXUU	UNCONSOLIDATED UNDRAINED TRIAXIAL
	UC, CU, UU = 1/2 Deviator Stress
DS (2.0)	DRAINED DIRECT SHEAR (NORMAL PRESSURE, ksf)

SAMPLER DRIVING RESISTANCE

Modified California and Standard Penetration Test samplers are driven 18 inches with a 140-pound hammer falling 30 inches per blow. Blows for the initial 6-inch drive seat the sampler. Blows for the final 12-inch drive are recorded onto the logs. Sampler refusal is defined as 50 blows during a 6-inch drive. Examples of blow records are as follows:

25	sampler driven 12 inches with 25 blows after initial 6-inch drive
85/7"	sampler driven 7 inches with 85 blows after initial 6-inch drive
50/3"	sampler driven 3 inches with 50 blows during initial 6-inch drive or beginning of final 12-inch drive

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SOIL CLASSIFICATION CHART

The Neighborhood at Los Gamos
San Rafael, California

Project No. 3013.001 Date: 7/16/2020

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A-1
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.

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ROCK CLASSIFICATION CHART

The Neighborhood at Los Gamos
San Rafael, California

Project No. 3013.001 Date: 7/16/2020

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A-2
FIGURE

BORING 1

EQUIPMENT: Track Mounted GeoProbe Drill Rig with 6" Hollow Stem Auger
DATE: 6/11/2020
ELEVATION: 71 - feet*
*REFERENCE: Google Earth, 2020

DEPTH meters	DEPTH feet	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	DRILL RATE (min/ft)
0	0	Sandy CLAY with Gravel (CL) Tan, dry to slightly moist, medium stiff, low to medium plasticity, fine to medium sand, percent clay varies within sample [Colluvium]	19	118	10.9		57.3% P200	
1	5	Trace angular gravel up to 1.0 inch diameter	19	117	12.1	UC 3225		
2, 3	10	Shale Melange Light to dark gray, low hardness, friable to weak, highly weathered, layers of more competent rock material encountered intermittently, typical thickness of 2.0 to 6.0 inches [Bedrock]	46		5.3			
4	15	Grades dark gray to black, waxy appearance, moderately strong	50/5"	118	6.0			
5		Harder drilling from 17-feet to 18-feet.						
6	20							

(continued on next page)

Water level encountered during drilling
Water level measured after drilling

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

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BORING LOG

The Neighborhood at Los Gamos
San Rafael, California

Project No. 3013.001 Date: 7/16/2020

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A-3
FIGURE

BORING 1
(CONTINUED)

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	DRILL RATE (min/ft)
	20	■		Shale Melange Light to dark gray, low hardness, friable to weak, highly weathered, layers of more competent rock material encountered intermittently, typical thickness of 2.0 to 6.0 inches [Bedrock]	45	133	5.7	TXUU (2000) 2800		10.0
7				Harder drilling from 21.0 to 24.0 feet, typ. drill rate for resistant material is approximately 5 min/ft						
8	25	■		Softer drilling at 25.0 feet, typ. drill rate for soft material is approximately 2 min/ft	78	140	5.3	UC 1350		
9	30	■			58	130	4.3	TXUU (4000) 3050		
10										4.0
11	35	■			53	110	3.4			
12	40	■		Bottom of boring at 40.5-ft. No groundwater encountered.	50/3"					

Water level encountered during drilling

Water level measured after drilling

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

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BORING LOG

The Neighborhood at Los Gamos
San Rafael, California

Project No. 3013.001 Date: 7/16/2020

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A-4
FIGURE

BORING 2

EQUIPMENT: Track Mounted GeoProbe Drill Rig with 6" Hollow Stem Auger
DATE: 6/11/2020
ELEVATION: 132 - feet*
*REFERENCE: Google Earth, 2020

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	DRILL RATE (min/ft)
0	0			Clayey SAND (SC) Medium brown, dry, medium dense, moderately plastic clay, fine to medium grained sand [Colluvium]	16	97	11.1		42.6% P200	
1										
	5			Shale Melange (Franciscan) Black to brown with green and white mineralization, low harness, friable, complete to high weathering, abundant mineralization, layers of more competent rock material encountered intermittently [Bedrock]	21	132	3.7	UC 325		
2										
3	10			Grades dark grey with minor mineralization.	36	123 129	11.2 9.4	TXUU (1500) 5500 UC 2475		
4										
	15				68	121	7.9	UC 650		
5										
				Easier drilling						0.3
6	20									

(continued on next page)

Water level encountered during drilling
Water level measured after drilling

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

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BORING LOG

The Neighborhood at Los Gamos
San Rafael, California

Project No. 3013.001 Date: 7/16/2020

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A-5
FIGURE

BORING 2
(CONTINUED)

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	DRILL RATE (min/ft)
	20			Shale Melange (Franciscan) Black to brown with green and white veins, low harness, friable, complete to high weathering, abundant mineralization, layers of more competent rock material encountered intermittently [Bedrock]						
7										
	25	■			50/0"					
8										2.0
				Hard shale layer from 24.5 feet to 28.0 feet depth.						
				Easier drilling at 28.0 feet						
9	30	■			50/5"	114	4.5			
10										
	35									
11										0.7
12										
	40	■		Bottom of boring at 40.5-ft. No groundwater encountered.	86/6"	112	9.4	UC 625		

▽ Water level encountered during drilling

▼ Water level measured after drilling

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
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BORING LOG

The Neighborhood at Los Gamos
San Rafael, California

Project No. 3013.001 Date: 7/16/2020

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A-6
FIGURE

BORING 3

EQUIPMENT: Track Mounted GeoProbe Drill Rig with 6" Hollow Stem Auger
DATE: 6/11/2020
ELEVATION: 97 - feet*
*REFERENCE: Google Earth, 2020

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	DRILL RATE (min/ft)
0	0			Sandy CLAY (CL) Light brown, dry, medium stiff, medium plasticity, fine to coarse grained sand, trace small gravels up to 1/8-inch diameter [Colluvium]	15	109	13.6		E.I. 95	
1	5			Grades to slightly moist.	17	126	11.0	UC 3375	68.7% P200	
2										
3	10			Shale Melange (Franciscan) Dark gray with abundant white mineralization, moderately hard, moderately strong, highly weathered, intensely fractured, waxy appearance [Bedrock]	46	126	9.6	UC 1500		
4	15			Grades increased hardness, color varies from gray to green	87	107	4.7			
5										
6	20									

(continued on next page)

▽ Water level encountered during drilling
▼ Water level measured after drilling

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
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BORING LOG

The Neighborhood at Los Gamos
San Rafael, California

Project No. 3013.001 Date: 7/16/2020

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A-7
FIGURE

BORING 4 (CONTINUED)

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	DRILL RATE (min/ft)
	20			Shale Melange (Franciscan) Black to brown with green and white mineralization, low harness, friable, complete to high weathering, abundant mineralization [Bedrock]	50/2" 50/3"		3.3			
7										0.7
	25			No recovery, sample grades more gray, some green mineralization	50/2" 50/3"		2.8			
8										0.7
9	30				50/2"					
										1.0
10										2.0
	35				50/3"		4.9			
11										0.5
12	40			Bottom of boring at 40.5-ft. No groundwater encountered.	94/10"					

▽ Water level encountered during drilling
▼ Water level measured after drilling

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
(3) METRIC EQUIVALENT STRENGTH (kPa) = 0.0479 x STRENGTH (psf)
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BORING LOG

The Neighborhood at Los Gamos
San Rafael, California

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Project No. 3013.001 Date: 7/16/2020

A-8
FIGURE

BORING 4

EQUIPMENT: Track Mounted GeoProbe Drill Rig with 6" Hollow Stem Auger
DATE: 6/11/2020
ELEVATION: 111 - feet*
*REFERENCE: Google Earth, 2020

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	DRILL RATE (min/ft)
0	0			Sandy CLAY (CL) Medium brown, dry, medium stiff, moderately plastic, approximately 20% fine grained sand, trace small gravel [Colluvium]	16	105	102	UC 1275	P.I. 12	
1										
	5			SAND with Clay (SC) Tan, dry, stiff, low plasticity, fine to medium grained, clay content varies with depth, low plasticity [Colluvium]	23		12.4	DS (1100) 5700		
2										
				Shale Melange (Franciscan) Gray and brown with green and white veining, highly weathered, weak to moderately strong, low hardness, pervasively sheared [Bedrock]						
3	10				56	103	16.2	UC 1600	E.I. 59	
4				Easy drilling						1.0
	15			Grades dark gray with green veining	39	118 124	12.2 9.6	UC 1450 TXUU (2000) 3850		
5										1.5
6	20			Grades increased strength						

(continued on next page)

▽ Water level encountered during drilling
▼ Water level measured after drilling

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
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BORING LOG

The Neighborhood at Los Gamos
San Rafael, California

Project No. 3013.001 Date: 7/16/2020

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A-9
FIGURE

BORING 4 (CONTINUED)

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)		BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	DRILL RATE (min/ft)
	20			Shale Melange (Franciscan) Gray and brown with green and white veining, highly weathered, weak to moderately strong, low hardness, pervasively sheared [Bedrock]	86	131	6.7	UC 2400		
7										
	25			Grades increased hardness	50/4"					
8										
9	30			Grades to black with glassy appearance, easier drilling	67	124 135	6.9 5.6	UC 350 TXUU (4000) 5600		
10										
11	35				48	135	7.0	UC 3200		
12	40									

(continued on next page)

Water level encountered during drilling

Water level measured after drilling

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
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San Rafael, California

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BORING 4
(CONTINUED)

DEPTH meters	DEPTH feet	SAMPLE	SYMBOL (4)	DESCRIPTION	BLOWS / FOOT (1)	DRY UNIT WEIGHT pcf (2)	MOISTURE CONTENT (%)	SHEAR STRENGTH psf (3)	OTHER TEST DATA	DRILL RATE (min/ft)
	40			Shale Melange (Franciscan) Gray and brown with green and white veining, highly weathered, weak to moderately strong, low hardness, pervasively sheared [Bedrock]	91	130	6.6	UC 1225		
13				Bottom of boring at 41.5-ft. No groundwater encountered.						
	45									
14										
15	50									
16										
	55									
17										
18	60									

Water level encountered during drilling

Water level measured after drilling

NOTES: (1) UNCORRECTED FIELD BLOW COUNTS
(2) METRIC EQUIVALENT DRY UNIT WEIGHT kN/m^3 = 0.1571 x DRY UNIT WEIGHT (pcf)
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BORING LOG

The Neighborhood at Los Gamos
San Rafael, California

Project No. 3013.001 Date: 7/16/2020

Drawn MNT
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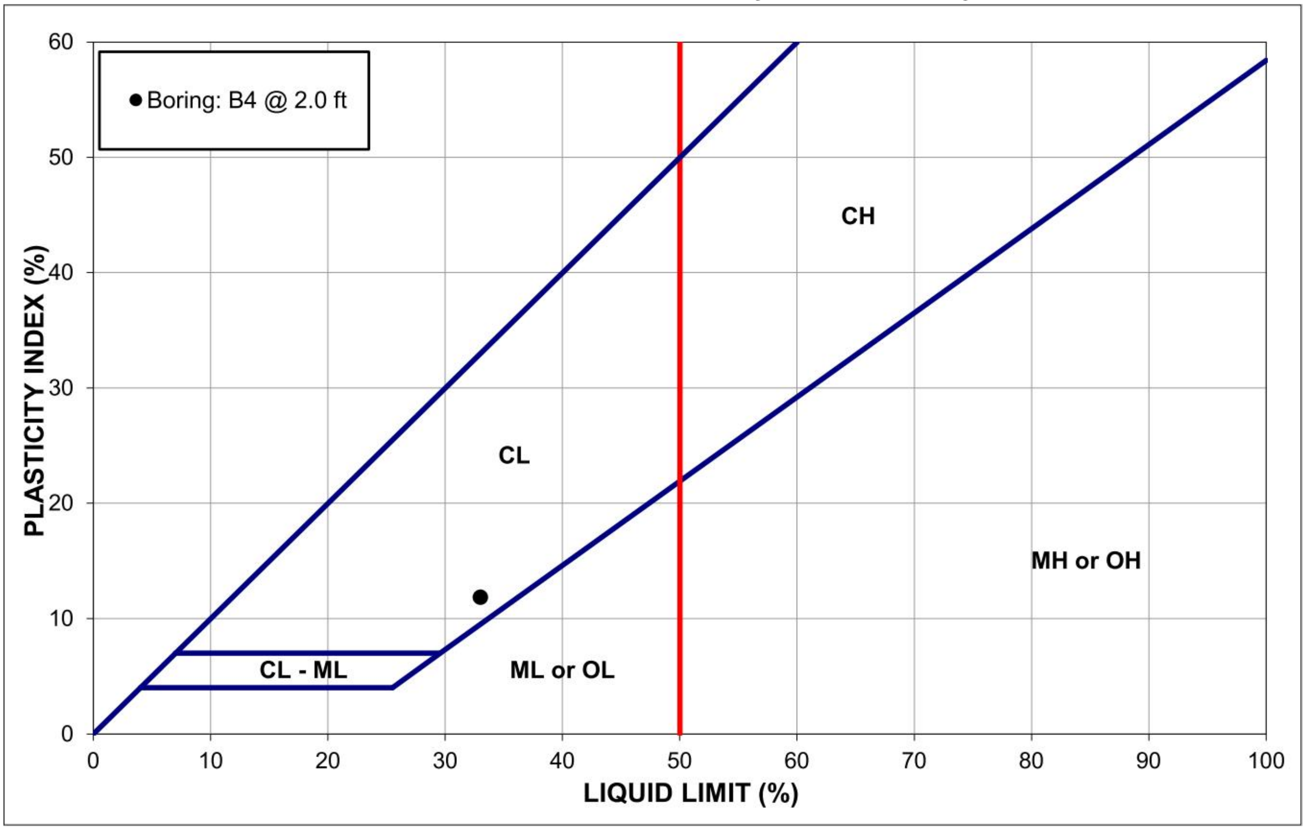
A-11
FIGURE

MILLER PACIFIC ENGINEERING GROUP

ATTERBERG LIMITS TEST (ASTM D 4318)



Sample	Classification	Liquid Limit (%)	Plastic Limit (%)	Plasticity Index (%)
Boring: B4 @ 2.0 ft	Sandy SILT Medium Brown	33	21	12

PI = 0-3: Non-Plastic
PI = 3-15: Slightly Plastic
PI = 15-30: Medium Plasticity
PI = >30: High Plasticity

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PLASTICITY INDEX CHART

The Neighborhood at Los Gamos
San Rafael, California

Project No. 3013.001 Date: 7/16/2020

Drawn MNT
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A-12
FIGURE

EXPANSION INDEX (ASTM D4829)

Project Name: THE NEIGHBORHOOD AT LOS GAMOS   Tested By: BPC   Date: 6/29/2020

Project Number: 3013.001   Sample Source: B4@11

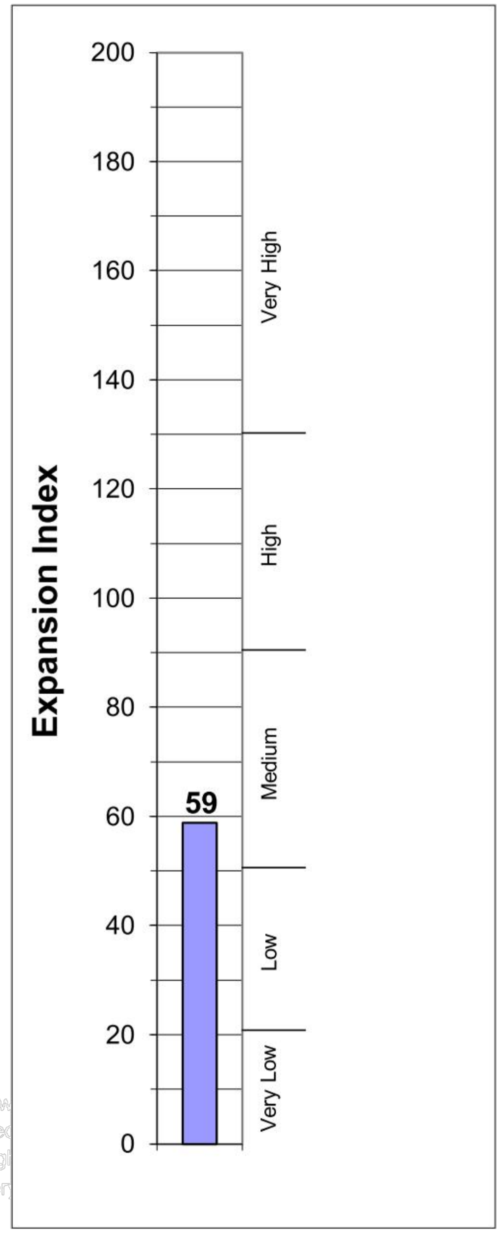
Sample Description: Shale, Gray/brown

Sample Height before Saturation (in.):	1.000
Sample/Ring Diameter: (in.):	4.000
Sample Volume before Saturation (cu.ft.):	0.007272
Ring Number:	EI-0
Ring Tare (gm):	366.2
Weight Ring + Moist Soil (gm):	736.6
Approx. Moisture Content (%):	
Estimated Specific Gravity (2.60-2.70):	2.68
Approx. EI Dry Density (pcf):	
Approximate Saturation:	50.0%

Dial Readings with 1 psi Load:

Start Time:	7:10	Dial Reading T0:	0.0000
Time 1:	7:11	Dial Reading T1:	0.0093
Time 2:	8:25	Dial Reading T2:	0.0557
Time 3:	10:20	Dial Reading T3:	0.0564
Time 4:	1:24	Dial Reading T4:	0.0579
Time 5:	4:00	Dial Reading T5:	0.0585

Final Height of Sample (in.):	1.059
Pan Identification:	25T
Weight of Ring + Wet Soil + Pan (gm):	1233.4
Pan Tare (gm):	451.9
Weight of Ring + Dry Soil + Pan (gm):	1146.4
Initial Moisture Content:	12.8%
Initial Dry Density (pcf):	99.5
Prepared Sample Saturation:	50.5%
Percent Expansion:	5.9%
Final Moisture Content:	26.5%
Final Dry Density (pcf):	94.0
EI $_{50}$**:**	59
Potential Expansion	Medium



MILLER PACIFIC ENGINEERING GROUP

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F 415 / 382-3450
www.millerpac.com

FILENAME: 3013.001 BL.dwg

EXPANSION INDEX CHART

The Neighborhood at Los Gamos
San Rafael, California

Drawn MNT
Checked

Project No. 3013.001   Date: 7/16/2020

A-13
FIGURE

EXPANSION INDEX (ASTM D4829)

Project Name: THE NEIGHBORHOOD AT LOS GAMOS   Tested By: BPC   Date: 6/29/2020

Project Number: 3013.001   Sample Source: B3 @2.0

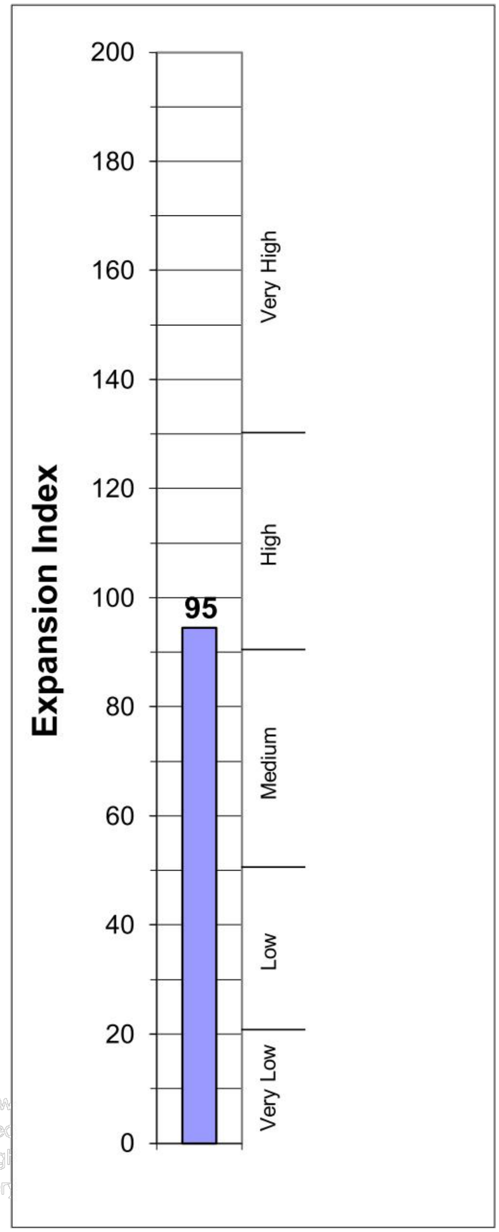
Sample Description: Sandy CLAY (CL), Light brown

Sample Height before Saturation (in.):	1.000
Sample/Ring Diameter: (in.):	4.000
Sample Volume before Saturation (cu.ft.):	0.007272
Ring Number:	EI-4
Ring Tare (gm):	367.1
Weight Ring + Moist Soil (gm):	748.6
Approx. Moisture Content (%):	
Estimated Specific Gravity (2.60-2.70):	2.68
Approx. EI Dry Density (pcf):	
Approximate Saturation:	50.0%

Dial Readings with 1 psi Load:

Start Time:	8:29	Dial Reading T0:	0.0000
Time 1:	8:30	Dial Reading T1:	0.0070
Time 2:	9:55	Dial Reading T2:	0.0976
Time 3:	12:55	Dial Reading T3:	0.0994
Time 4:	1:50	Dial Reading T4:	0.1002
Time 5:	3:50	Dial Reading T5:	0.1005

Final Height of Sample (in.):	1.101
Pan Identification:	25T
Weight of Ring + Wet Soil + Pan (gm):	1255.1
Pan Tare (gm):	451.8
Weight of Ring + Dry Soil + Pan (gm):	1167.3
Initial Moisture Content:	9.5%
Initial Dry Density (pcf):	105.6
Prepared Sample Saturation:	43.6%
Percent Expansion:	10.1%
Final Moisture Content:	25.2%
Final Dry Density (pcf):	96.0
***EI*$_{50}$:**	95
Potential Expansion	High



MPEG MILLER PACIFIC ENGINEERING GROUP	504 Redwood Blvd. Suite 220 Novato, CA 94947 T 415 / 382-3444 F 415 / 382-3450 www.millerpac.com	EXPANSION INDEX CHART The Neighborhood at Los Gamos San Rafael, California Project No. 3013.001 Date: 7/16/2020 Drawn MNT Checked A-14 FIGURE

FILENAME: 3013.001 BL.dwg

APPENDIX B – REFERENCE DATA

SalemHowesAssociates Inc.
GEOTECHNICAL ENGINEERS AND GEOLOGISTS

03 February 1999

~~Roland Mireles~~ and Dennis Hart (415)599-6986
RMD LLC
100 Larkspur Landing Circle ~~#110~~ #290
Larkspur, CA 94939

Job :9808105-H
File: Prelim 2 LVT Rpt

SUBJECT: Preliminary Site Evaluation
Geotechnical Investigation,
Lucas Valley Townhouses
Los Gamos Drive, San Rafael

This report presents the results of our geotechnical investigation of the proposed building site and our general recommendations for site development for your preliminary evaluation. The purpose of our investigation was to evaluate the geotechnical feasibility of the proposed development, assess the suitability of the building site, and provide detailed recommendations and conclusions as they relate to our specialty field of practice, geotechnical engineering and engineering geology. The scope of services specifically excluded any investigation needed to determine the presence or absence of issues of economic concern on the site, or of hazardous or toxic materials at the site in the soil, surface water, ground water, or air. Prior to the development of the site and environmental assessment may be necessary.

The field work consisted of reconnaissance mapping of exposed geologic features on the site and in the immediate surrounding area and the excavation and drilling of 21 test pits and borings. Selected samples were returned to the laboratory for testing. Field work was conducted in September of 1998. During this period we reviewed select geotechnical references pertinent to the area and examined stereo-paired aerial photographs of the site.

Site Geology and Slope Stability

The property is located on an east facing hillside which slopes upward at 20 to 30 percent from Los Gamos Avenue. A ten-foot high, 2:1, soil cutslope separates the property from Los Gamos Avenue. Native grasses and thistles presently cover the lower portion of the slope in the area of the proposed development. The colluvium* soils covering the slope exhibit the geomorphology of long-term downslope creep, with the soil typically shallow along the top of the slope (above the 100-foot contour) and increasing in thickness at the toe along Los Gamos Avenue. Published geologic maps show the creeping soils extending up to the 200-foot contour. However our air photo study followed by the test borings indicated that the slope area subject to significant soil creep is limited to the area designated on the geologic map of having a soil depth greater than five feet. A shallow historic landslide occupies the center of the property. This feature involves only the soil layer and does not indicate a gross instability of the underlying rock slope.

1202 GRANT AVENUE, SUITE F
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FAX 892-8568

Retaining Walls

All retaining walls should be supported on rock by pier type foundations. Where a cut is made into bedrock and the entire footing will be in rock, footing type foundations may be used.

Retaining walls should be designed for a coefficient of active soil pressure (K_a) equal to 0.50, or an equivalent fluid pressure[1] of 65 lbs/f^3. Any wall where the backfill is subject to vehicular loads should have the design pressure increased equivalent to a 200 lbs/ft^2 surcharge. If a uniform surcharge load q' acts on the soil behind the wall it results in a pressure Ps in lbs/ft. of wall equal to:

$$Ps = q' * (\text{height of wall}) * Ka$$

Allowable foundation bearing and lateral resistance to sliding should be obtained from the formulae in the respective sections on pier or footing foundations.

All retaining walls should have a backdrainage system consisting of, as a minimum, drainage rock in a filter fabric (e.g. Mirafi™ 140N) with at least three inch diameter perforated pipe laid to drain by gravity. If Caltrans specification Class 2 Permeable is used the filter fabric envelope may be omitted. The pipe should rest on the ground or footing with no gravel underneath. The pipe should be rigid perforated pipe PVC Schedule 40 or ABS with SDR no greater than 23.5. Pipes with perforations greater than 1/16 inch in diameter shall be wrapped in filter fabric. A bentonite seal should be placed at the connection of all solid and perforated pipes. All backdrainage shall be maintained in a separate system from roof and other surface drainage.

Retaining walls which are adjacent to living areas should have additional water proofing such as three dimensional drainage panels and moisture barriers (e.g. "Miradrain™ 6000" panels and "Paraseal™") and the invert of the drainage pipe should be a minimum of four inches below the adjacent interior finished floor elevation. All waterproofing materials must be installed in strict compliance with the manufacturer's specifications. The heel of the retaining wall footing should be sloped towards the hill to prevent ponding of water at the cold joint, the drainage pipe should be placed on the lowest point on the footing. The backslope of the retaining walls should be ditched to drain to avoid infiltration of surface run-off into the backdrainage system.

Notice: We will not accept the foundation for concrete placement if the pier holes or foundation grades are over 24 hours old, dried out or saturated and will require that they be overexcavated. The contractor may submit plans for remedial measures, such as spraying or covering the excavation, to extend this time period. However, acceptance is always subject to the condition of the foundation grade immediately prior to the pour.

Cuts and Fills

Unsupported cuts and fills are generally not recommended for this site. Fills behind retaining walls should be of material approved by the geotechnical engineer and compacted to a maximum dry density of 90 percent as determined by ASTM D-1157. Fills underlying pavements shall have the top 12 inches compacted to 95 percent maximum dry density.

Design Recommendations

Drilled Piers

Drilled, cast-in place, reinforced concrete piers should be a minimum of 18 inches in diameter and should extend at least six feet into competent bearing stratum as determined by the Engineer in the field. Additional depths may be imposed by the structural engineer. The piers shall extend into the bearing stratum six feet below a 30° line projected up from the bottom of the nearest cut slope or bank. Piers should be designed to resist forces from the gravitational creep of the soil layer. The height of the piers subject to the creep forces is equal to the depth to the top of rock. For design purposes on this project, this may be interpolated from the data on Drawing A/1. Creep forces should be calculated using a coefficient of active soil pressure (K_a) equal; to 0.50 with a soil unit weight of 130 lbs/ft^3 acting on two pier diameters, or an equivalent fluid pressure[1] of 65 lbs/ft^3 may be used. Because the bedrock is a discontinuous medium, for geotechnical considerations, the piers should be spaced no more than 12 feet on center and connected by tie and grade beams in a grid like configuration.

We recommend that piers be designed for lateral resistance using the method developed by Bowels [2] (attached as Table 1) for cantilever sheetpiling of a unit width. The unit width in the formula may be increased by 1.6 if the piers are greater than four diameters apart to account for single pier action. Lateral loads from transitory loading may be taken in the soil layer as a rectangular distribution of 0.30 kips/ft^2 acting on 1.6 pier diameters. A one-third increase may be used for all transitory loading. End bearing should be neglected

Design Parameters

Depth of fixity below top of bedrock surface:	Calculate from relationship in Bowels[2]
Soil active pressure:	$K_a = 0.50$
Soil unit weight:	0.130 kips/ft^3
Rock active pressure:	$K_a = 0.0$
Rock passive pressure:	$K_p = 8.0$
Rock unit weight:	0.135 kips/ft^3
Soil passive pressure for transitory loading only, neglect top foot:	$K_p = 3.0$
Angle of internal friction, in rock:	$\varnothing = 50°$
Adhesion in rock	900 lbs/ft^2

In order for these strength values to be realized, the sides of the pier holes must be scaled of any mudcake.

Notice: We will not accept the foundation for concrete placement if the pier holes are over 24 hours old and will require that they be redrilled. One should plan ahead and have the pier cages assembled prior to drilling the holes so that there is no delay in placing the concrete.

Ground water will be encountered in the drilled pier holes and it may be necessary to dewater, case the holes and/or place the concrete by tremie methods. Hard drilling may be necessary to reach the required depths. The contractor should be familiar with the local conditions in order to have the appropriate equipment on hand. The rock to be encountered in the drilling can be observed in the gully just north of the property.

Bedrock underlying the site has been mapped by others[1] as belonging to a schist and semi-schist [Jksch] unit of the Franciscan geologic formation. Rock is exposed in a gully along the north end of the property and was encountered in all of the test pits and borings. Local, discontinuous deposits of meta-chert were also encountered in the test pits and outcrop on the site. Slopes in this rock type are rated[1] moderate to high in fresh rock, but low for soils and deeply weathered rock. This formation is easily altered by weathering processes to depths as much as several tens of feet and clay-rich swelling soils are derived from the weathering.

Foundation Conditions and General Recommendations

A thick layer of creeping moderately to expansive soil generally underlies the area of development. The thickness of the soil deposit is shown on Drawing A, Site Plan and Location of Test Borings. This soil layer is undergoing continuous downslope creep that will be accelerated by any new loading or cutslopes. In addition it is expansive and will have a detrimental effect on slabs, pavements and sidewalks unless remedial measures, such as overexcavation and backfilling, are taken. Areas with colluvium soils over five feet in depth are subject to future instability activated by heavy rainfall or construction grading. Therefore, we recommend that all structures be founded on the underlying bedrock by drilled pier type foundation construction. Piers must be designed to resist the lateral forces resulting from the downslope creep of the colluvium soil layer above the rock. All soil cuts must be supported by retaining walls. Roadways must also be supported by retaining walls or excavated to the top of rock and built up to grade with engineered fill. The access road will require excavation to the top of rock and the soil replaced with an earth buttress fill to support the roadway and upslope colluvium soil. For design of earth retaining structures an active pressure of 65 lbs/ft^3 should be used.

The area identified as a historic landslide [Qls] on the Drawing A/1 is in a fragile state of stability and must be excavated to the top of rock and replaced with an engineered earth buttress fill. Colluvium slopes above the development contain two to three feet of soil on relatively steep slopes. Retaining walls and debris walls should be placed on the slope above the uppermost structures to as a safeguard against possible debris flows.

Very truly yours,

For SalemHowes Associates Inc.

E Vincent Howes

Geotechnical Engineer
GE #965 Exp.31 Mar 02
Engineering Geologist
CEG #1252 Exp. 19 Aug 98

Attachment: Drawing A, Site Plan and Location of Test Borings
Design Recommendations
Logs of Test Borings and Test Pits

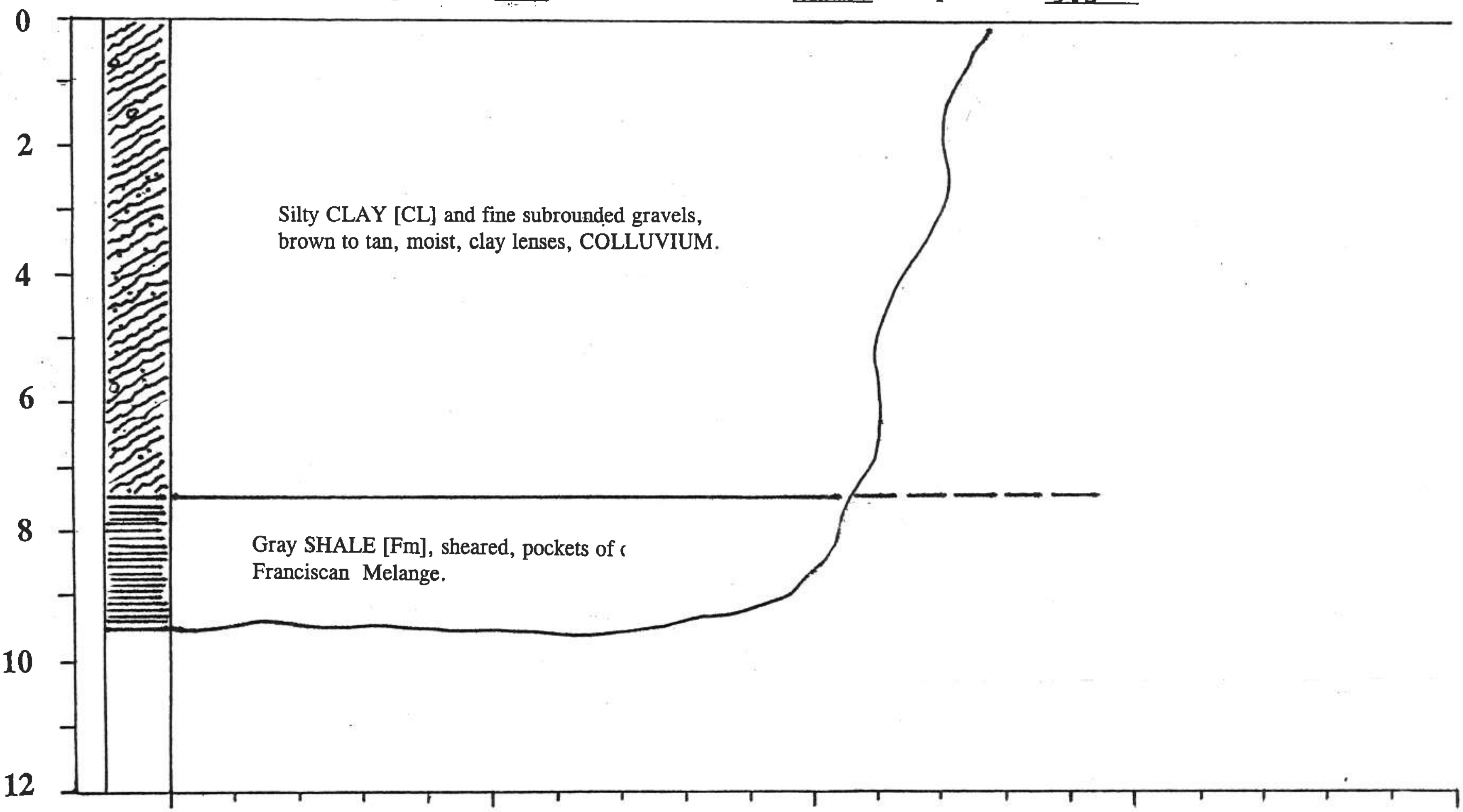
SalemHowesAssociates
170 E. Blithedale Avenue
Mill Valley, CA 94941
415/381-6146

Test Pit Log

Lucas Valley Townhouses

Date: 11 September 1998 **By:** E. Bowman

Log of Pit # A **Ground Elevation** 56' **Depth of Pit** 9.5'



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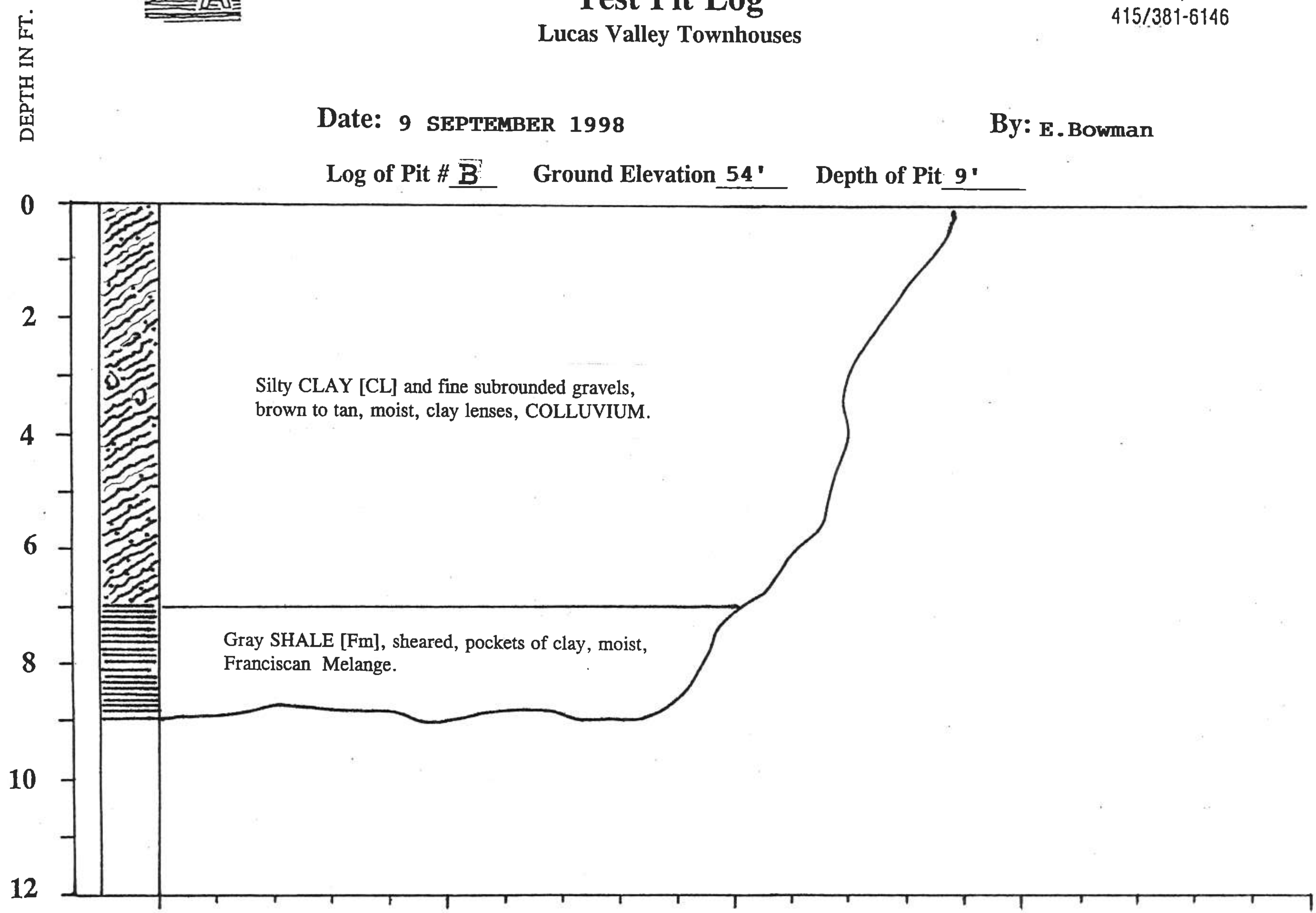
Test Pit Log

Lucas Valley Townhouses

Date: 9 SEPTEMBER 1998 **By:** E.Bowman

Log of Pit # B **Ground Elevation** 54' **Depth of Pit** 9'



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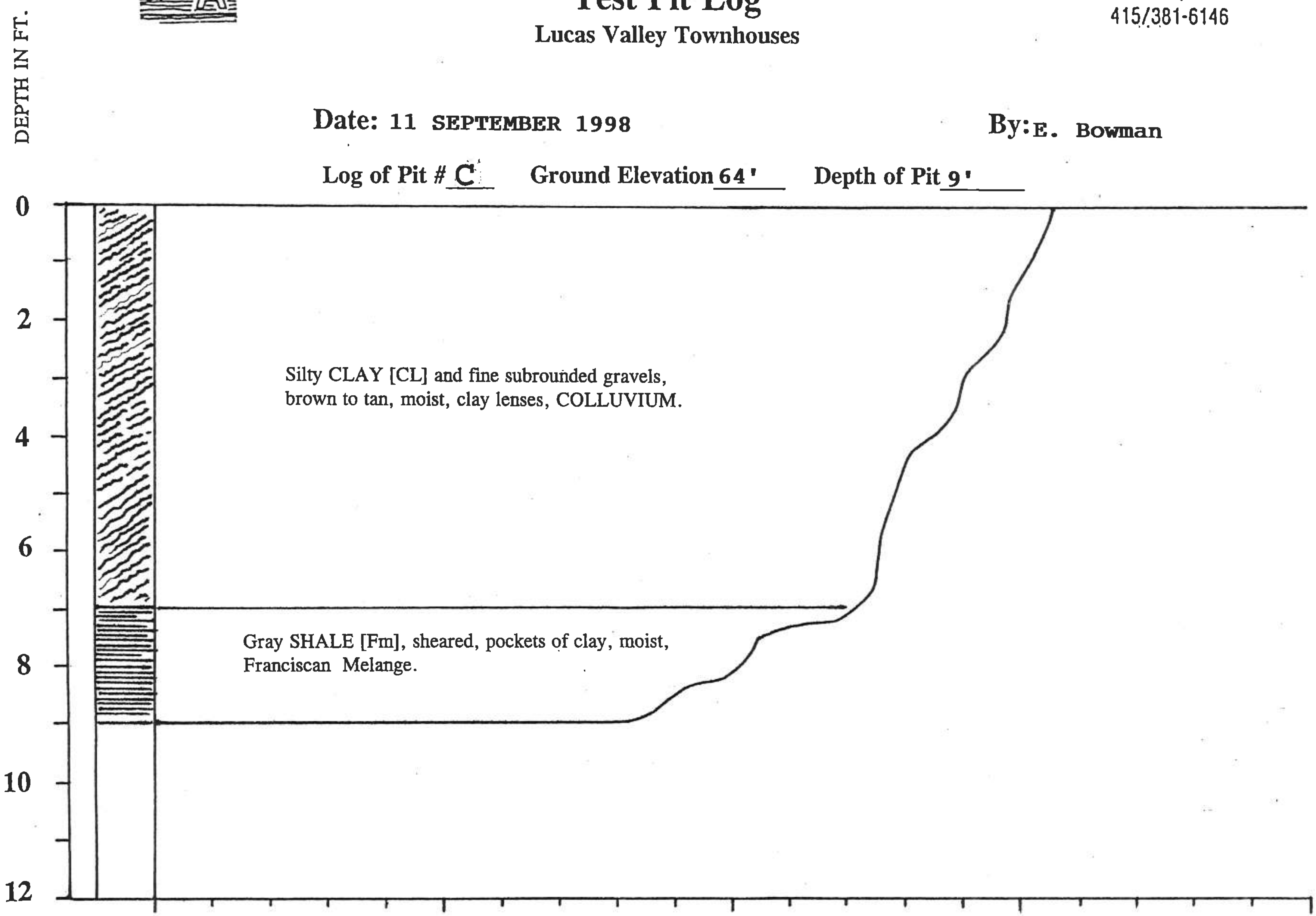
415/381-6146

Test Pit Log

Lucas Valley Townhouses

Date: 11 SEPTEMBER 1998 **By:** E. Bowman

Log of Pit # C **Ground Elevation** 64' **Depth of Pit** 9'



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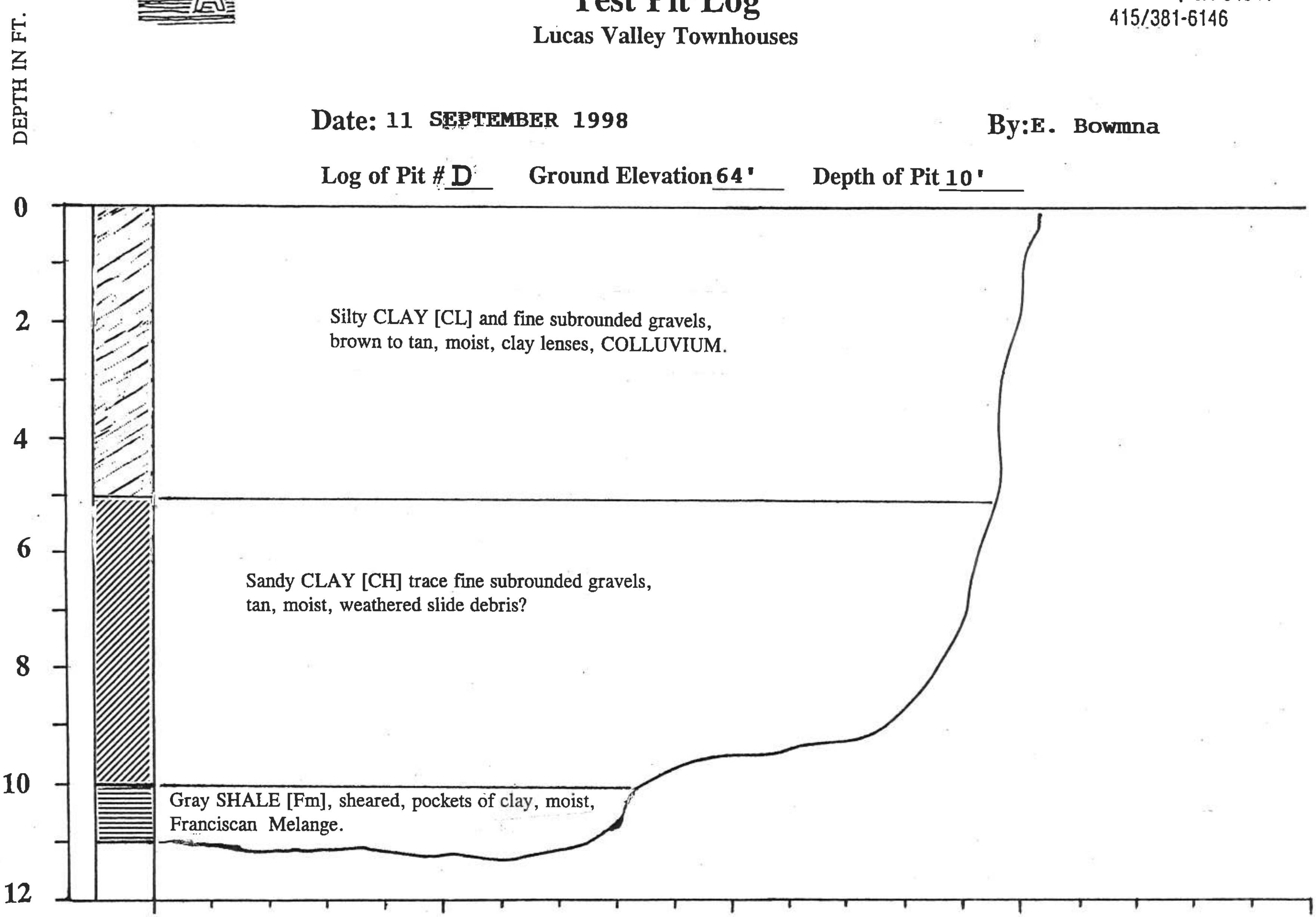
415/381-6146

Test Pit Log

Lucas Valley Townhouses

Date: 11 SEPTEMBER 1998 **By:** E. Bowmna

Log of Pit # D **Ground Elevation** 64' **Depth of Pit** 10'



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MILL VALLEY, CA 94941
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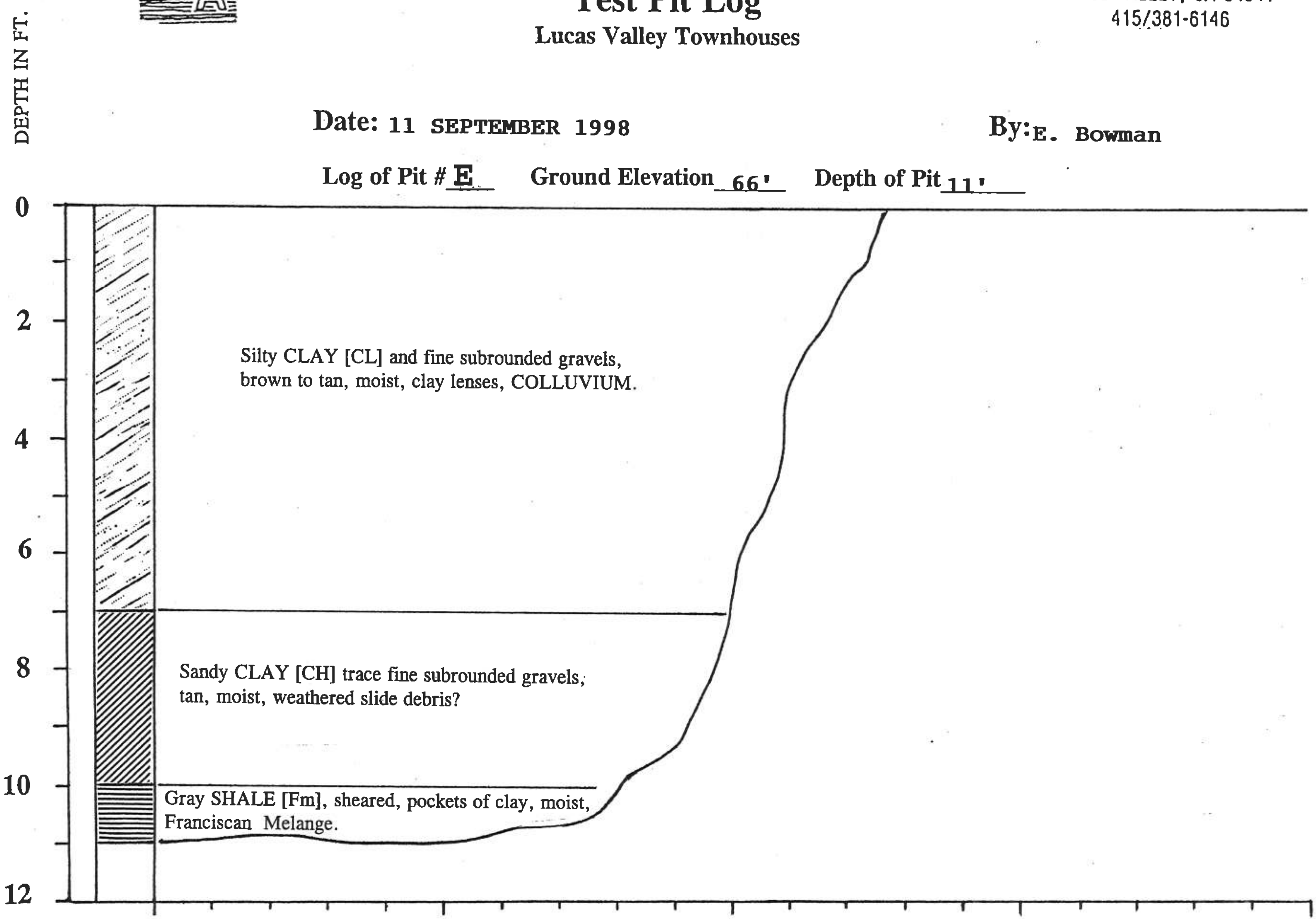
Test Pit Log

Lucas Valley Townhouses

Date: 11 SEPTEMBER 1998 **By:** E. Bowman

Log of Pit # E Ground Elevation 66' Depth of Pit 11'



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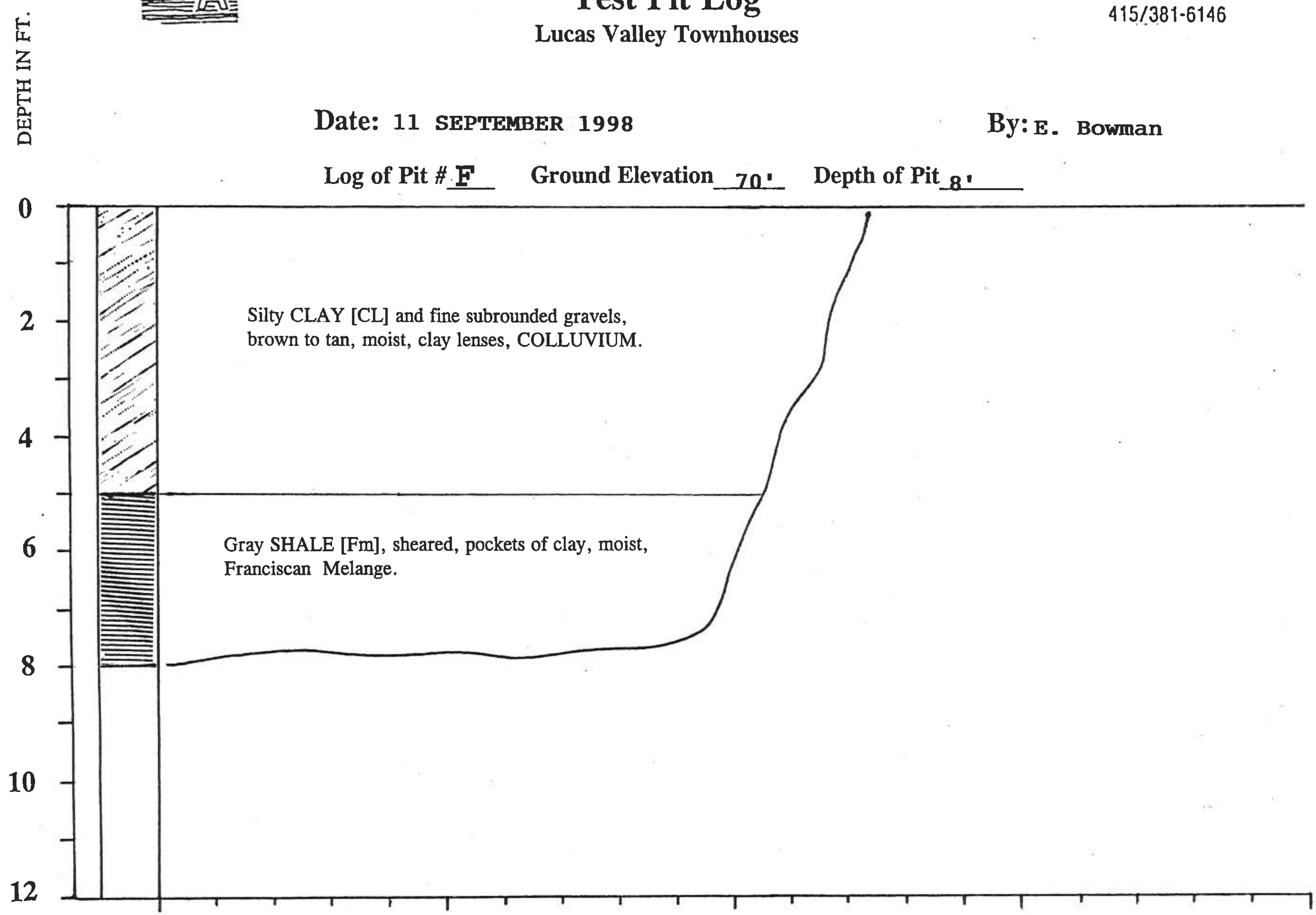
Test Pit Log

Lucas Valley Townhouses

Date: 11 SEPTEMBER 1998 **By:** E. Bowman

Log of Pit # F **Ground Elevation** 70' **Depth of Pit** 8'



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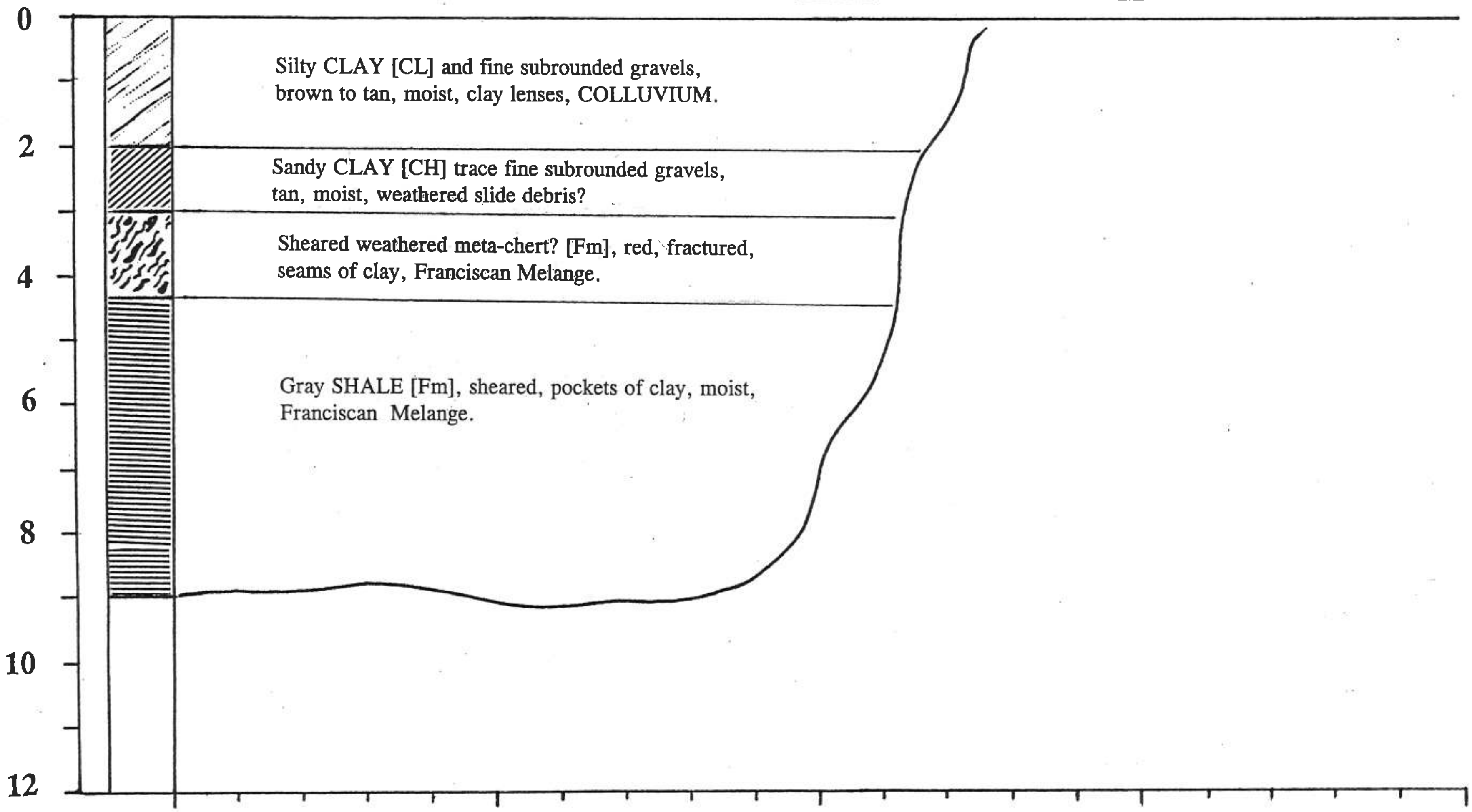
Test Pit Log

Lucas Valley Townhouses

Date: 11 SEPTEMBER 1998 **By:** E. BOWMAN

Log of Pit # G **Ground Elevation** 87' **Depth of Pit** 9'

DEPTH IN FT.



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Test Pit Log

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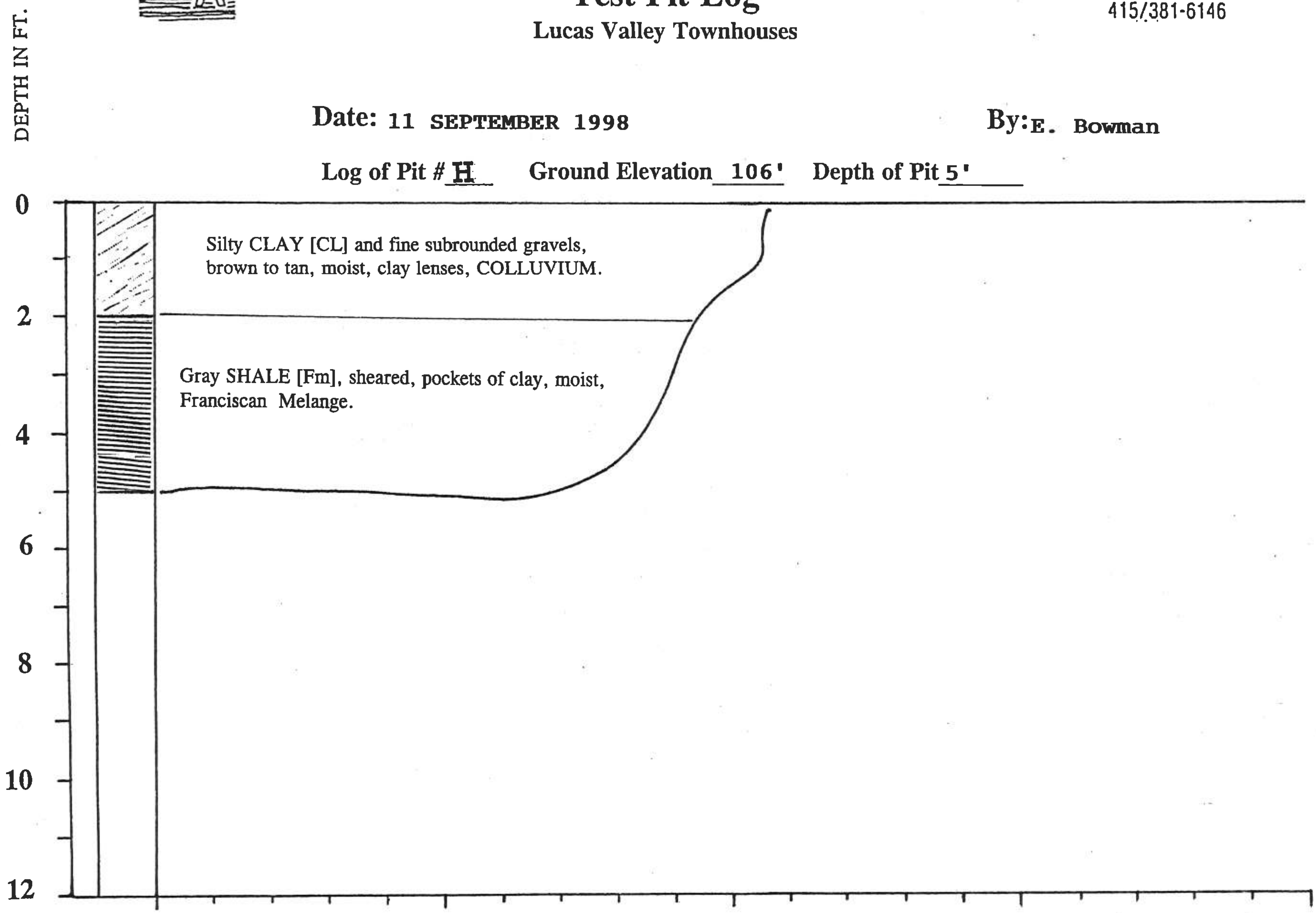
Date: 11 SEPTEMBER 1998 **By:** E. Bowman

Log of Pit # H **Ground Elevation** 106' **Depth of Pit** 5'



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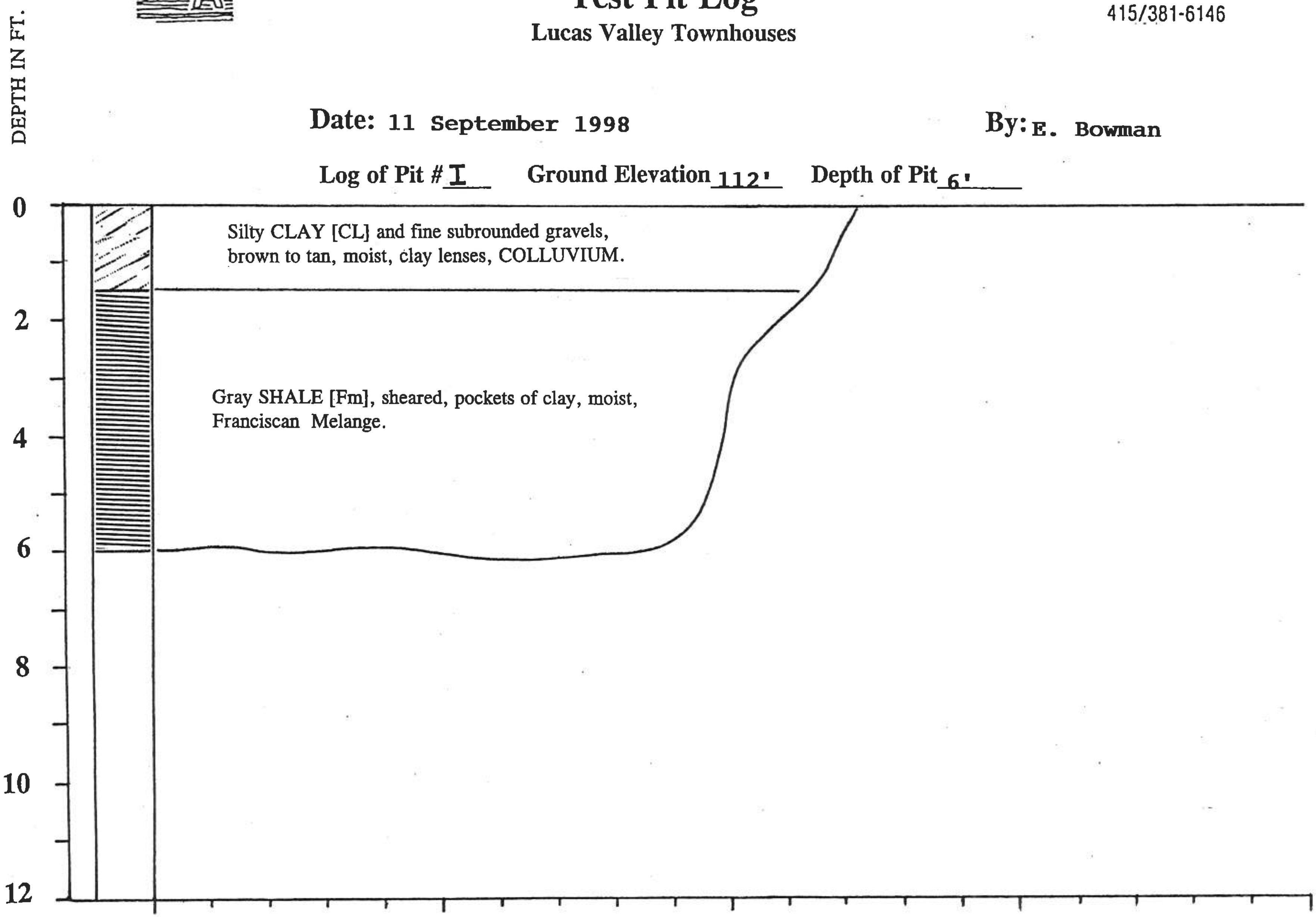
Test Pit Log

Lucas Valley Townhouses

Date: 11 September 1998 **By:** E. Bowman

Log of Pit # I **Ground Elevation** 112' **Depth of Pit** 6'



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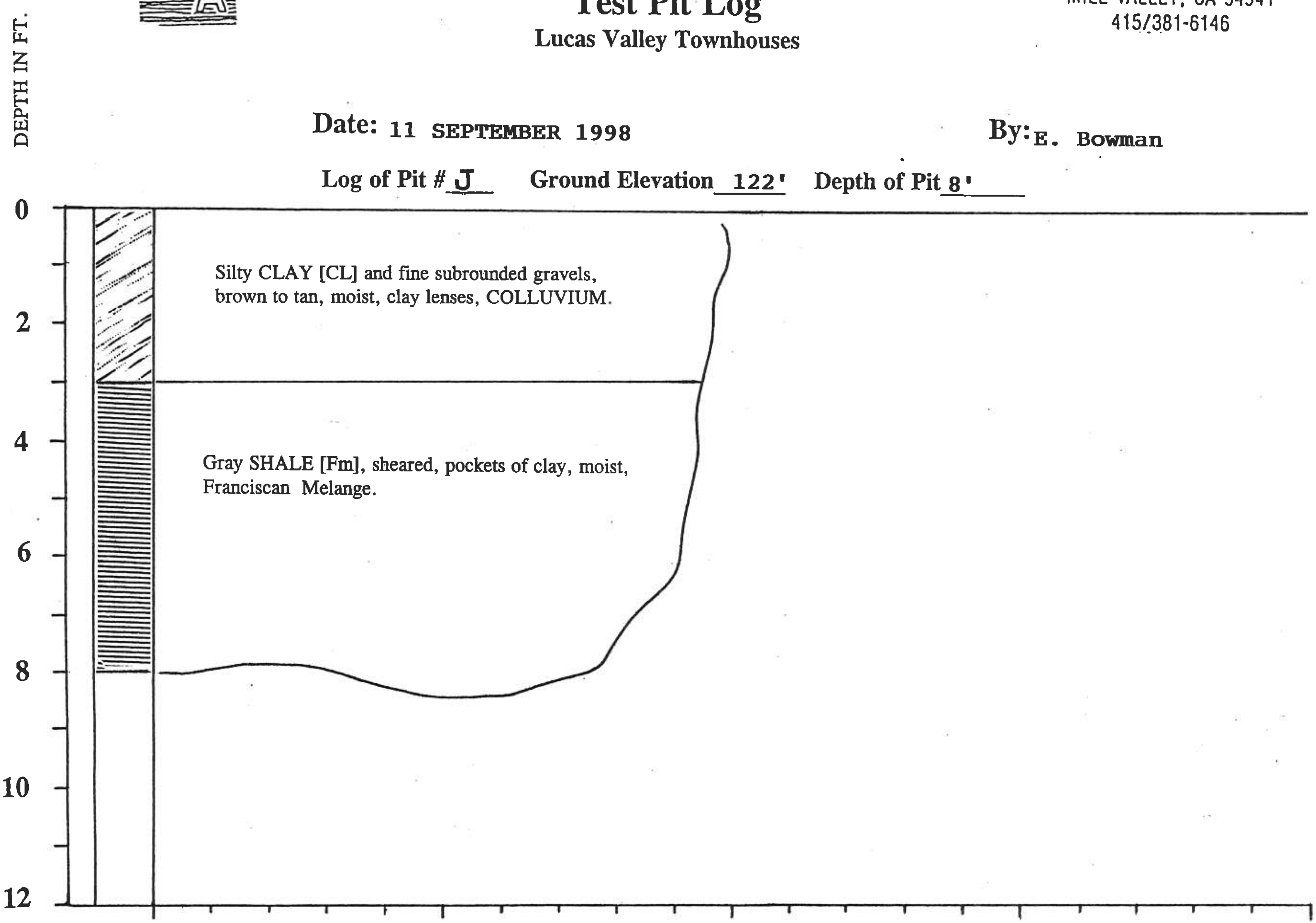
Test Pit Log

Lucas Valley Townhouses

Date: 11 SEPTEMBER 1998 **By:** E. Bowman

Log of Pit # J **Ground Elevation** 122' **Depth of Pit** 8'



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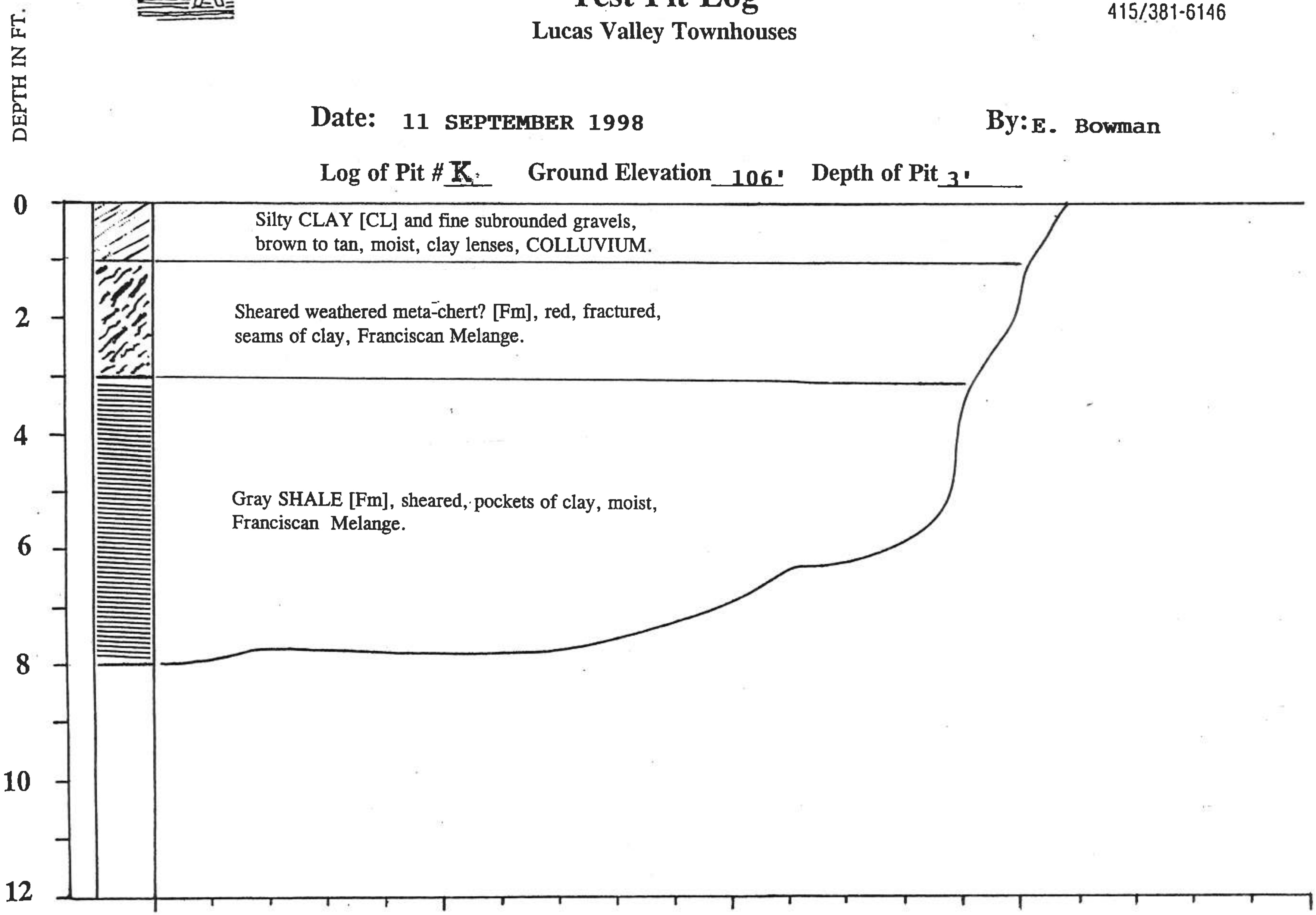
Test Pit Log

Lucas Valley Townhouses

Date: 11 SEPTEMBER 1998 **By:** E. Bowman

Log of Pit # K **Ground Elevation** 106' **Depth of Pit** 3'



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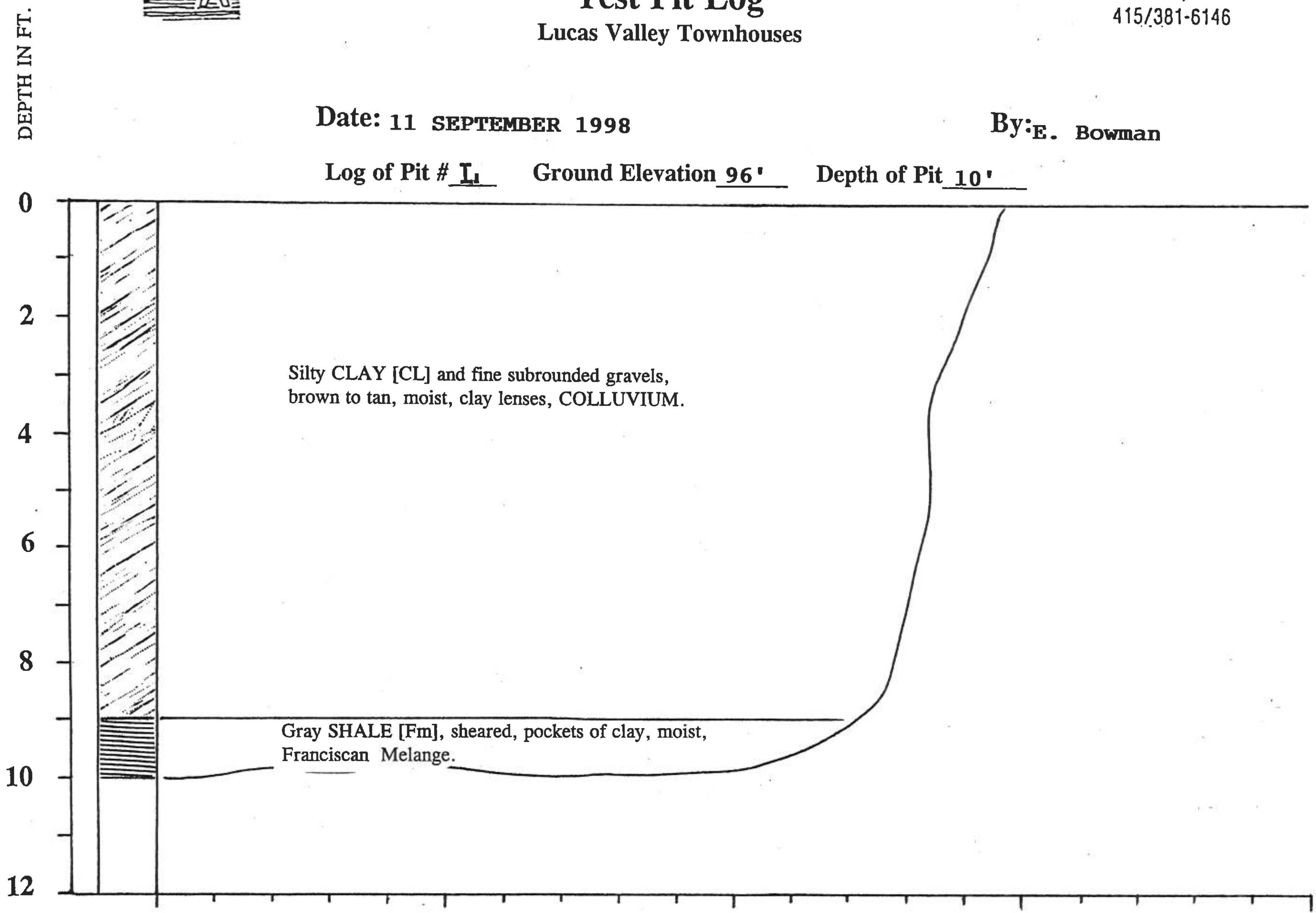
Test Pit Log

Lucas Valley Townhouses

Date: 11 SEPTEMBER 1998 **By:** E. Bowman

Log of Pit # L1 **Ground Elevation** 96' **Depth of Pit** 10'



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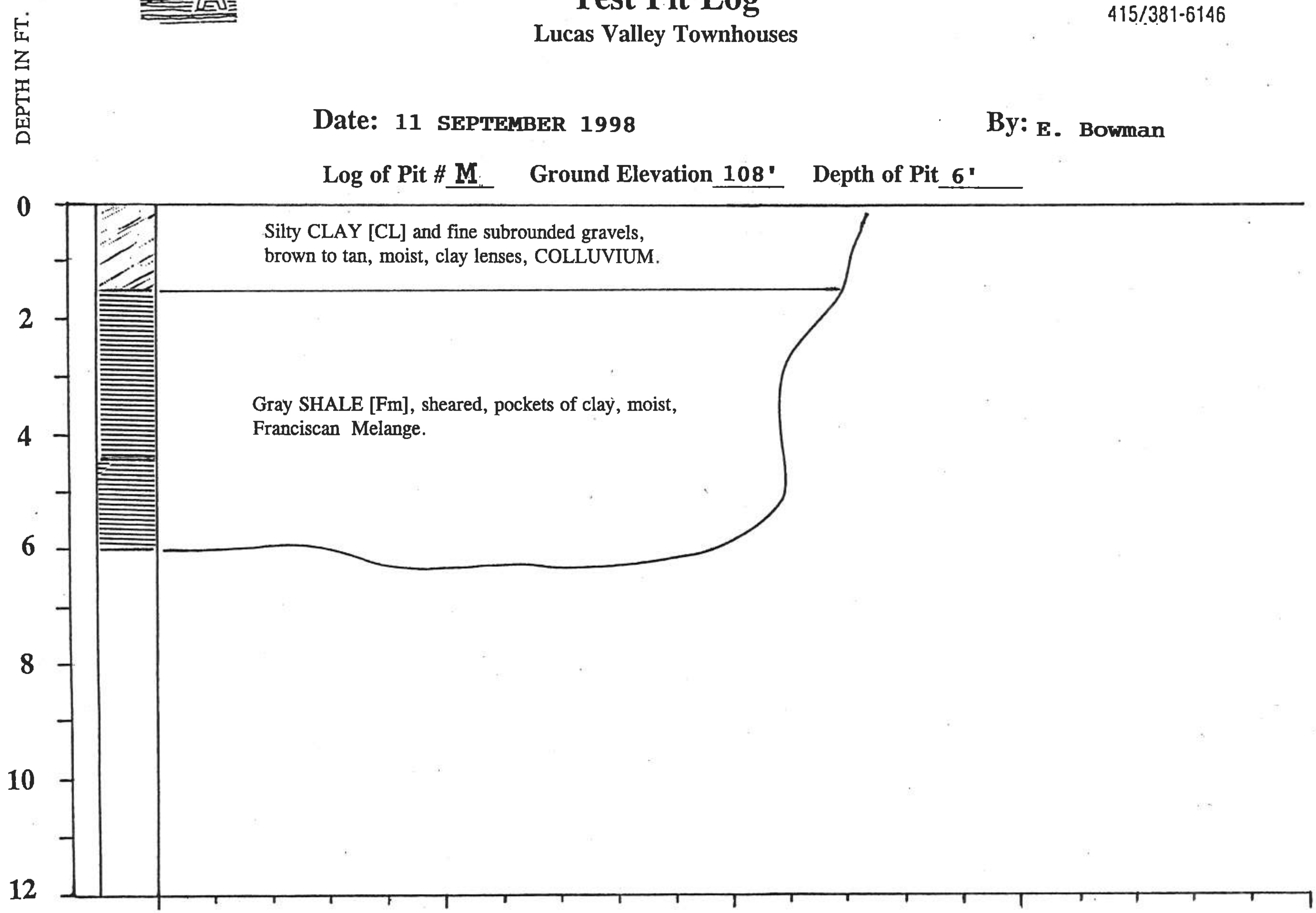
Test Pit Log

Lucas Valley Townhouses

Date: 11 SEPTEMBER 1998 **By:** E. Bowman

Log of Pit # M **Ground Elevation** 108' **Depth of Pit** 6'



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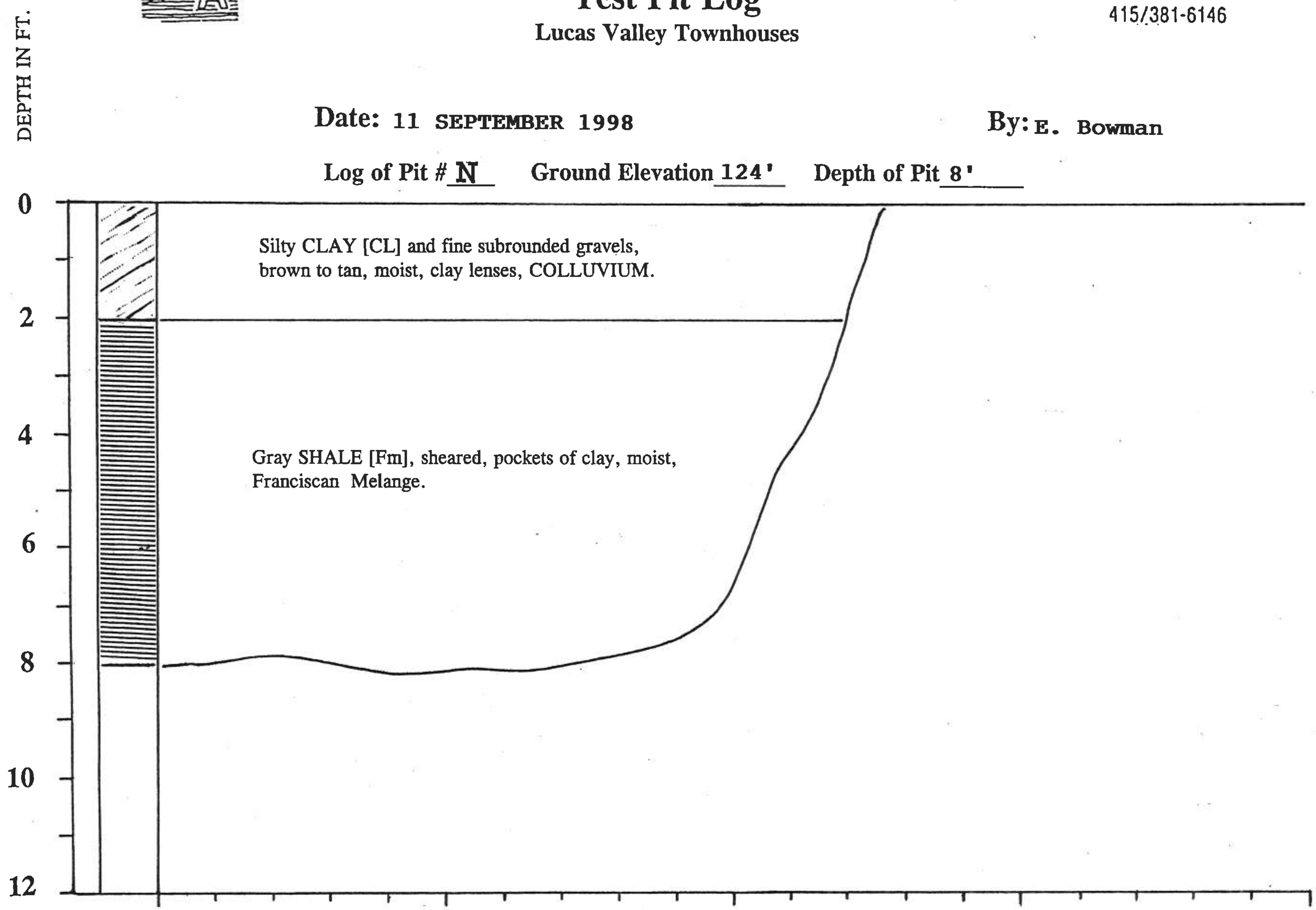
Test Pit Log

Lucas Valley Townhouses

Date: 11 SEPTEMBER 1998 **By:** E. Bowman

Log of Pit # N **Ground Elevation** 124' **Depth of Pit** 8'



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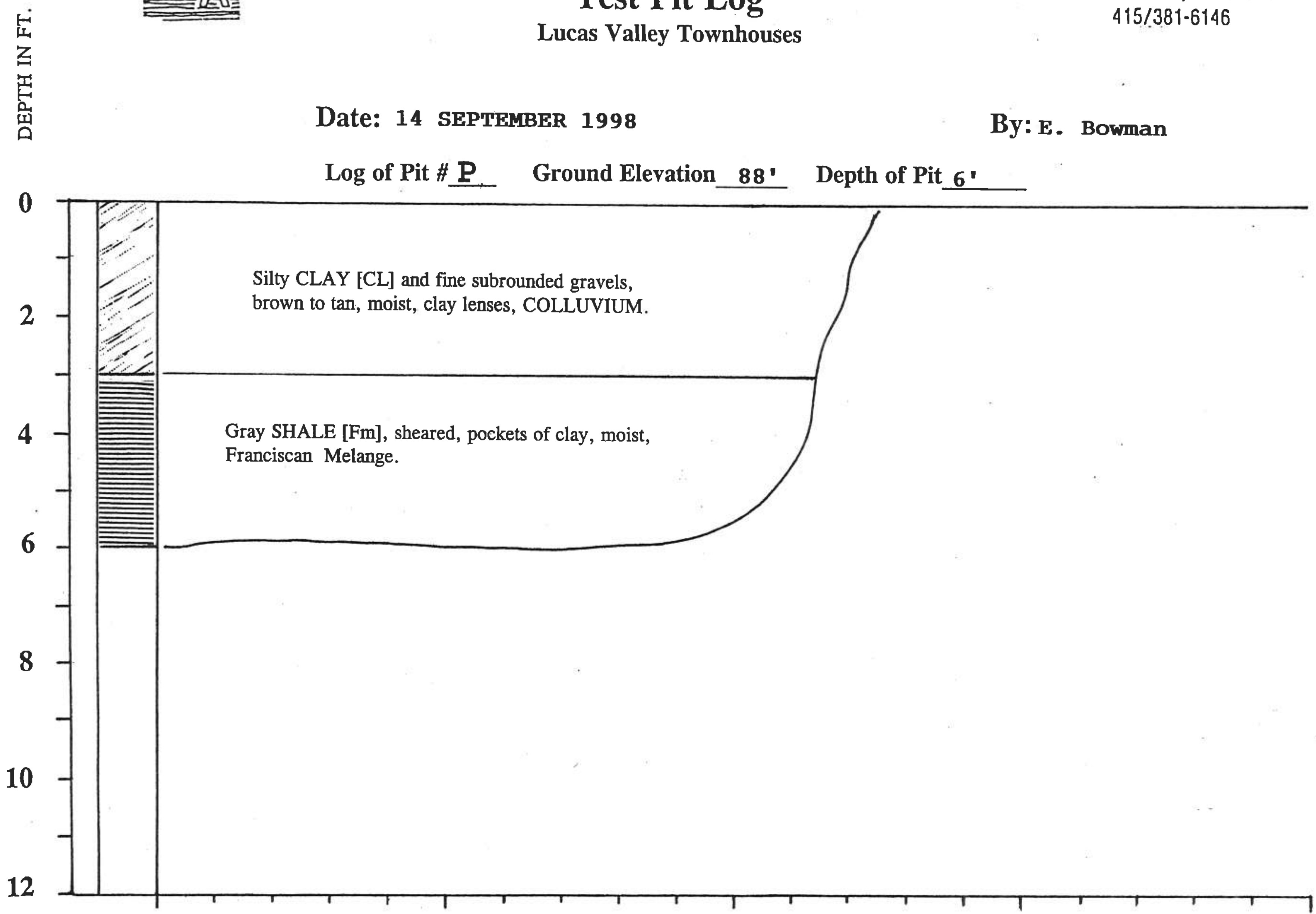
Test Pit Log

Lucas Valley Townhouses

Date: 14 SEPTEMBER 1998 **By:** E. Bowman

Log of Pit # P **Ground Elevation** 88' **Depth of Pit** 6'



Test Pit Log

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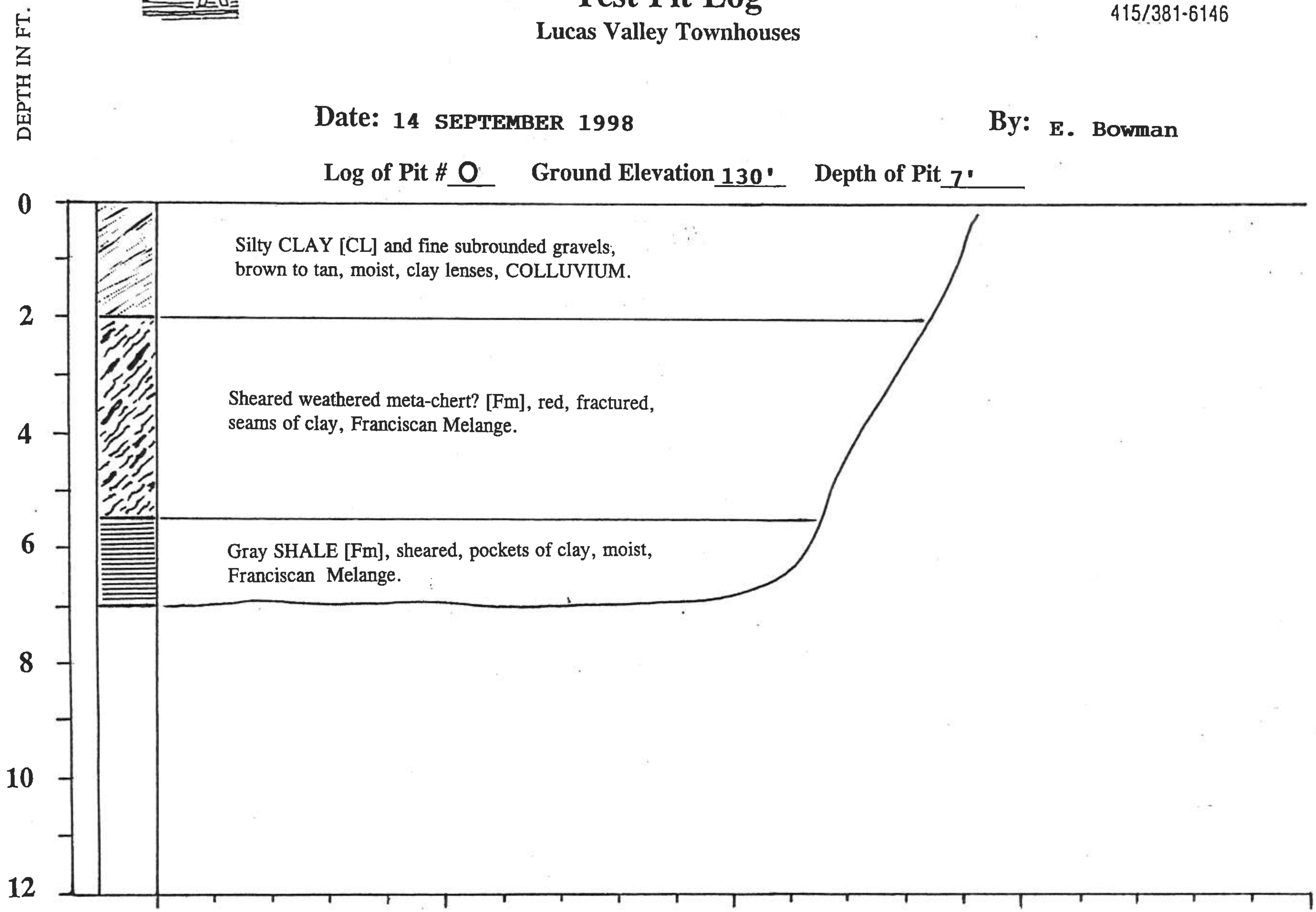
Date: 14 SEPTEMBER 1998 **By:** E. Bowman

Log of Pit # O **Ground Elevation** 130' **Depth of Pit** 7'



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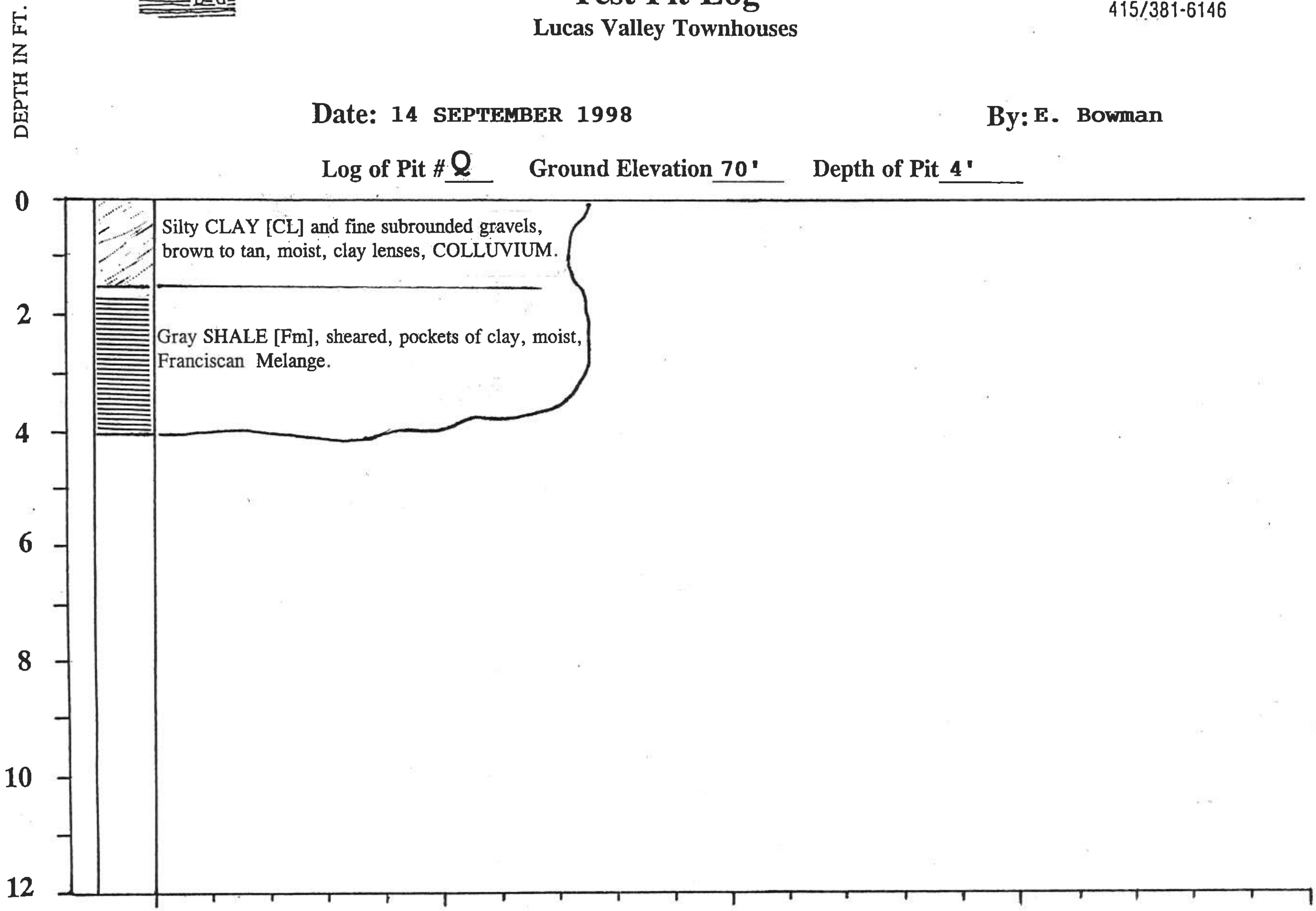
Test Pit Log

Lucas Valley Townhouses

Date: 14 SEPTEMBER 1998 **By:** E. Bowman

Log of Pit # Q **Ground Elevation** 70' **Depth of Pit** 4'



PROJECT: Lucas Valley Townhomes | **BORING:** B/1

ENGINEER: E.V. Howes | **LOGGED BY:** E. Bowman

JOB # : 9808105-H | **DATE:** 12 September98

MOISTURE CONTENT (%)	DRY DENSITY (pcf)	SAMPLE TYPE	Blows Per Foot	DEPTH (feet)	WATER LEVEL (date)	DESCRIPTIVE LOG	GRAPHIC LOG	REMARKS
				1				
				2				LL= 32, PI= 13
				3				
10		SPT	12 13 17	4				
				5				
				6				ϕ = 17°, C = .4
	117	SPT	6 7 7	7				
				8		COLLUVIUM Silty CLAY [CL-ML] and fine subrounded gravels, brown to tan, moist, colluvium.		
				9				QUC= 3.56 ksf
21	107	SPT	5 8 11	10				
				11				QUC= 3.11 ksf

DRLLED BY: Trans Bay Exploration | **EQUIPMENT:** Hydraulic Portable

BORING SIZE: 3.0" | **SHEET:** 1 of 2

PROJECT: Lucas Valley Townhomes | BORING: B/1
ENGINEER: E.V. Howes | LOGGED BY: E. Bowman
JOB # : 9808105-H | DATE: 12 September98

MOISTURE CONTENT (%)	DRY DENSITY (pcf)	SAMPLE TYPE	Blows Per Foot	DEPTH (feet)	WATER LEVEL (date)	DESCRIPTIVE LOG	REMARKS
16	117	SPT	9 11 13	12			
				13			
				14			
				15			$\phi = 8^{\circ}$, C = .45
19	110	SPT	8 9 13	16			
				17		FRANCISCAN MELANGE Gray Sheared SHALE [Fm], pockets weathered to clay, moist, hard, Franciscan Melange.	
				18			
				19			
15		SPT	REF	20			TOTAL DEPTH 20.0'
				21			
				22			

DRLLED BY: Trans Bay Exploration | EQUIPMENT: Hydraulic Portable

BORING SIZE: 3.0" | SHEET: 2 of 2

PROJECT: Lucas Valley Townhomes | BORING: B/2

ENGINEER: E.V. Howes | LOGGED BY: E. Bowman

JOB # : 9808105-H | DATE: 12 September98

MOISTURE CONTENT (%)	DRY DENSITY (pcf)	SAMPLE TYPE	Blows Per Foot	DEPTH (feet)	WATER LEVEL (date)	DESCRIPTIVE LOG	GRAPHIC LOG	REMARKS
				1				
				2				LL= 34, PI= 18
				3				
11		SPT	15 13 13	4				
				5		COLLUVIUM Silty CLAY [CL-ML] and fine subrounded gravels, brown to tan, moist, colluvium.		
				6				QUC= 4.01 ksf
18	111	SPT	13 21 25	7				No Water Encountered
				8				
				9				
		SPT	13 39 ref	10		FRANCISCAN MELANGE Gray Sheared SHALE [Fm], pockets weathered to clay, moist, hard, Franciscan Melange.		TOTAL DEPTH 10.0'
				11				

DRLLED BY: Trans Bay Exploration | EQUIPMENT: Hydraulic Portable

BORING SIZE: 3.0" | SHEET: 1 of 1

PROJECT: Lucas Valley Townhomes	**BORING:** B/3
ENGINEER: E.V. Howes	**LOGGED BY:** E. Bowman
JOB # : 9808105-H	**DATE:** 12 September98

MOISTURE CONTENT (%)	DRY DENSITY (pcf)	SAMPLE TYPE	Blows Per Foot	DEPTH (feet)	WATER LEVEL (date)	DESCRIPTIVE LOG	GRAPHIC LOG	REMARKS
				1				
				2				
				3		COLLUVIUM Silty CLAY [CL-ML] and fine subrounded gravels, brown to tan, moist, colluvium.		
				4				
				5				
				6				
		SPT	12 16 19	7				
				8				
				9				
				10				
				11				

DRLLED BY: Trans Bay Exploration	**EQUIPMENT:** Hydraulic Portable
BORING SIZE: 3.0"	**SHEET:** 1 of 2

PROJECT: Lucas Valley Townhomes	BORING: B/3
ENGINEER: E.V. Howes	LOGGED BY: E. Bowman
JOB # : 9808105-H	DATE: 12 September98

MOISTURE CONTENT (%)	DRY DENSITY (pcf)	SAMPLE TYPE	Blows Per Foot	DEPTH (feet)	WATER LEVEL (date)	DESCRIPTIVE LOG	GRAPHIC LOG	REMARKS
				12				
		SPT	22 26 29	13				
				14				
		SPT	REF	15	▼	FRANCISCAN MELANGE Gray Sheared SHALE [Fm], pockets weathered to clay, moist, hard, Franciscan Melange.		WATER LEVEL
				16				
				17				
				18				
		SPT	16 19 22	19				TOTAL DEPTH 19.5'
				20				
				21				
				22				

DRLLED BY: Trans Bay Exploration	EQUIPMENT: Hydraulic Portable
BORING SIZE: 3.0"	SHEET: 2 of 2

PROJECT: Lucas Valley Townhomes	BORING: B/4
ENGINEER: E.V. Howes	LOGGED BY: E. Bowman
JOB # : 9808105-H	DATE: 12 September98

MOISTURE CONTENT (%)	DRY DENSITY (pcf)	SAMPLE TYPE	Blows Per Foot	DEPTH (feet)	WATER LEVEL (date)	DESCRIPTIVE LOG	GRAPHIC LOG	REMARKS
				1				
				2				
				3				
				4				
				5		COLLUVIUM Silty CLAY [CL-ML] and fine subrounded gravels, brown to tan, moist, colluvium.		
				6				
				7				
				8				
				9				
		SPT	8 14 18	10				
				11				

DRLLED BY: Trans Bay Exploration	EQUIPMENT: Hydraulic Portable
BORING SIZE: 3.0"	SHEET: 1 of 2

PROJECT: Lucas Valley Townhomes	**BORING: B/4**
ENGINEER: E.V. Howes	**LOGGED BY: E. Bowman**
JOB # : 9808105-H	**DATE: 12 September98**

MOISTURE CONTENT (%)	DRY DENSITY (pcf)	SAMPLE TYPE	Blows Per Foot	DEPTH (feet)	WATER LEVEL (date)	DESCRIPTIVE LOG	GRAPHIC LOG	REMARKS
				12				
		SPT	9 14 19	13		FRANCISCAN MELANGE Gray Sheared SHALE [Fm], pockets weathered to clay, moist, hard, Franciscan Melange.		
				14				
		SPT	REF	15				TOTAL DEPTH 15.0'
				16				
				17				
				18				
				19				
				20				
				21				
				22				

DRLLED BY: Trans Bay Exploration	**EQUIPMENT: Hydraulic Portable**
BORING SIZE: 3.0"	**SHEET: 2 of 2**

SALEM HOWES ASSOCIATES INC.
GEOTECHNICAL CONSULTANTS

12 October 1998

[redacted] and Dennis Hart
RMD LLC
100 Larkspur Landing Circle #110
Larkspur, CA 94939

Job :9808105-H
File:Lucas Vly Townhouse Rpt

SUBJECT: Preliminary Site Evaluation
Geotechnical Investigation,
Lucas Valley Townhouses
Los Gamas Drive, San Rafael

This report presents the results of our geotechnical investigation of the proposed building site and our general recommendations for site development for your preliminary evaluation. The purpose of our investigation was to evaluate the geotechnical feasibility of the proposed development, assess the suitability of the building site, and provide detailed recommendations and conclusions as they relate to our specialty field of practice, geotechnical engineering and engineering geology. The scope of services specifically excluded any investigation needed to determine the presence or absence of issues of economic concern on the site, or of hazardous or toxic materials at the site in the soil, surface water, ground water, or air. Prior to the development of the site and environmental assessment may be necessary.

The field work consisted of reconnaissance mapping of exposed geologic features on the site and in the immediate surrounding area and the excavation and drilling of 21 test pits and borings. Selected samples were returned to the laboratory for testing. Field work was conducted in September of 1998. During this period we reviewed select geotechnical references pertinent to the area and examined stereo-paired aerial photographs of the site.

<u>Site Geology and Slope Stability</u>
The property is located on an east facing hillside which slopes upward at 20 to 30 percent from Los Gamos Avenue. A ten-foot high, 2:1, soil cutslope separates the property from Los Gamos Avenue. Native grasses and thistles presently cover the lower portion of the slope in the area of the proposed development. The colluvium* soils covering the slope exhibit the geomorphology of long-term downslope creep, with the soil typically shallow along the top of the slope (above the 100-foot contour) and increasing in thickness at the toe along Los Gamos Avenue. Published geologic maps show the creeping soils extending up to the 200-foot contour. However our air photo study followed by the test borings indicated that the slope area subject to significant soil creep is limited to the area designated on the geologic map of having a soil depth greater than five feet. A shallow historic landslide occupies the center of the property. This feature involves only the soil layer and does not indicate a gross instability of the underlying rock slope.

170 E. BLITHEDALE AVENUE
MILL VALLEY, CALIFORNIA 94941
415/381-6146
FAX 415/381-1360

Bedrock underlying the site has been mapped by others[1] as belonging to a schist and semi-schist [Jksch] unit of the Franciscan geologic formation. Rock is exposed in a gully along the north end of the property and was encountered in all of the test pits and borings. Local, discontinuous deposits of meta-chert were also encountered in the test pits and outcrop on the site. Slopes in this rock type are rated[1] moderate to high in fresh rock, but low for soils and deeply weathered rock. This formation is easily altered by weathering processes to depths as much as several tens of feet and clay-rich swelling soils are derived from the weathering.

Foundation Conditions and General Recommendations

A thick layer of creeping moderately to expansive soil generally underlies the area of development. The thickness of the soil deposit is shown on Drawing A, Site Plan and Location of Test Borings. This soil layer is undergoing continuous downslope creep that will be accelerated by any new loading or cutslopes. In addition it is expansive and will have a detrimental effect on slabs, pavements and sidewalks unless remedial measures, such as overexcavation and backfilling, are taken. Areas with colluvium soils over five feet in depth are subject to future instability activated by heavy rainfall or construction grading. Therefore, we recommend that all structures be founded on the underlying bedrock by drilled pier type foundation construction. Piers must be designed to resist the lateral forces resulting from the downslope creep of the colluvium soil layer above the rock. All soil cuts must be supported by retaining walls. Roadways must also be supported by retaining walls or excavated to the top of rock and built up to grade with engineered fill. The access road will require excavation to the top of rock and the soil replaced with an earth buttress fill to support the roadway and upslope colluvium soil.

The area identified as a historic landslide [Qls] on the Drawing A/1 is in a fragile state of stability and must be excavated to the top of rock and replaced with an engineered earth buttress fill. Colluvium slopes above the development contain two to three feet of soil on relatively steep slopes. Retaining walls and debris walls should be placed on the slope above the uppermost structures to as a safeguard against possible debris flows.

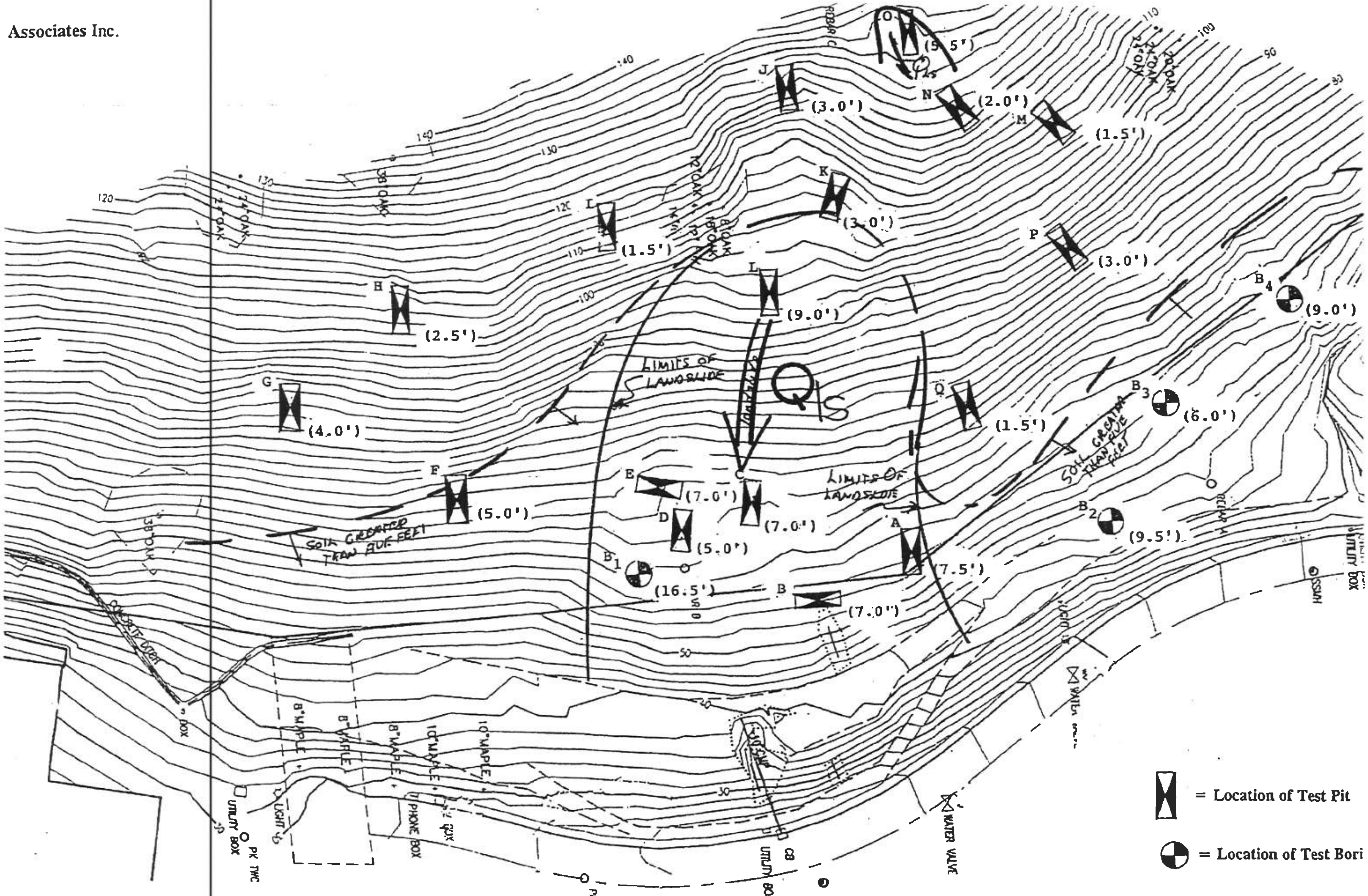
Very truly yours,

For SalemHowes Associates Inc.

E Vincent Howes

Geotechnical Engineer
GE #965 Exp.31 Mar 02
Engineering Geologist
CEG #1252 Exp. 19 Aug 98

Attachment: Drawing A. Site Plan and Location of Test Borings



= Location of Test Pit

= Location of Test Bori

(n.n') = Depth to Bearing Str:

Lucas Valley Townhomes

Location of Test Pits and Test Borings

Scale 1" = 50'