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WATER RESOURCES
CONSTRUCTION SERVICES

Project No.
6103.200.000

January 9, 2015

Mr. Mike O'Hara
Tim Lewis Communities
Suite 270
Roseville, CA 95661

Subject: Spotorno Property
Pleasanton, California

GEOTECHNICAL FEASIBILITY REPORT

Dear Mr. O'Hara:

We prepared this geotechnical feasibility report for the proposed residential development at the approximately 112-acre site referred to as the Spotorno Property. The subject property is located at 1000 Minnie Street in Pleasanton, California and is identified as Assessor Parcel Number (APN) 949-16-6. This report is prepared as outlined in our proposal dated December 29, 2014.

The accompanying report contains a summary of our document review, conclusions, and preliminary recommendations for the geotechnical aspects of the proposed development on the subject site. Based on our study, it is our opinion that the proposed development is feasible from a geotechnical standpoint provided the preliminary recommendations included in this report are incorporated into project planning and development.

We are pleased to be of service to you on this project and look forward to consulting further with you and your design team.

Sincerely,

ENGEO Incorporated

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

We prepared this geotechnical feasibility report for the proposed residential development at the subject site in Pleasanton, California. This report was prepared as outlined in our proposal dated December 29, 2014. Tim Lewis Communities authorized us to conduct the proposed scope of services, which included the following:

- Review of published geologic maps, previous geotechnical studies, and pre-development historical maps and aerial photographs for the site and adjacent to the site.

- Site reconnaissance visit to observe existing site conditions and perform preliminary geologic mapping of the site.

- Preparation of a report providing our preliminary findings and conclusions regarding the geotechnical aspects and feasibility of the project.

This report was prepared for the exclusive use of Tim Lewis Communities and their consultants for project planning and design. In the event that any changes are made in the character, design or layout of the development, we must be contacted to review the conclusions and recommendations contained in this report to determine whether modifications are necessary.

1.2 SITE LOCATION AND PROJECT DESCRIPTION

The study area is located east of Alisal Street and north of Westbridge Lane at 1000 Minnie Street in Pleasanton, California (Figure 1 and Figure 2) and is identified as Assessor Parcel Number (APN) 949-16-6. The study area has an irregular footprint that encompasses approximately 112 acres and consists of undeveloped land currently used for livestock pasture. It is our understanding that the proposed improvements consist of detached residential structures located on the western, relatively flat, portion of the property with open space consisting of hillsides planned in the eastern portions of the site. According to an Aerial Exhibit by Ruggeri Jensen Azar (RJA) dated December 19, 2014, the improvements are intended to be west of the approximate limit of a 25-percent slope. Based on information from RJA, cuts and fills 15 to 25 feet, respectively, are anticipated.

The highest elevation on the project area is approximately 683 feet near the top of a hill in the eastern portion of the study area and the lowest elevation is approximately 384 feet in the western end of the flatland area adjacent to Alisal Street.

1.3 PREVIOUS EXPLORATIONS

We previously performed a geotechnical exploration of the study area in 2004 including drilling and logging borings ranging in depth from about 11 to 20 feet below the existing ground surface and excavating and logging of 12 tests pits ranging from 5 to 16 feet deep. The proposed development at the time of our 2004 study included a by-pass road with 120 feet high cut slopes at gradients 3:1 (horizontal:vertical) and fill slopes up to 60 feet high at gradients 2½:1. The lower western portion was to be developed into 97 single-family lots.

We issued a revised report in 2008 utilizing previously collected data. The development plan was altered to 79 single-family lots with cuts and fills up to 15 feet with slopes between lots ranging up to 17 feet high with additional retaining walls.

Prior to our work, the study area was explored by Terrasearch as published in a fault study dated November 24, 1998, preliminary geotechnical investigation report dated March 18, 1999, and a geotechnical investigation report dated September 10, 1999. Terrasearch performed various field explorations including borings, tests pits, and rock cores. Prior to Terrasearch, Berlogar Geotechnical Consultants (BGC) performed a geologic reconnaissance as published in their report dated May 22, 1997. BGC's field exploration consisted of a field visit by an engineering geologist.

Known previous explorations located near and within the proposed area to be developed are shown in Figure 2.

2.0 GEOLOGY AND SEISMICITY

2.1.1 Regional Geology

The site is located within the Coast Ranges physiographic province of California. The Coast Ranges physiographic province is typified by a system of northwest-trending, fault-bounded mountain ranges and intervening alluvial valleys.

Bedrock in the Coast Ranges consists of igneous, metamorphic and sedimentary rocks that range in age from Jurassic to Pleistocene. The present physiography and geology of the Coast Ranges are the result of deformation and deposition along the tectonic boundary between the North American plate and the Pacific plate. Plate boundary fault movements are largely concentrated along the well-known fault zones, which in the area include the San Andreas, Hayward, and Calaveras faults, as well as other lesser-order faults.

2.1.2 Site Geology and Seismicity

Published geologic maps of the vicinity (Graymer, et al, 1996; Dibblee, 1980) map bedrock in the study area to be Pliocene to Pleistocene Livermore gravels (QTl), as shown in Figure 3. The Livermore gravels generally consist of stream terrace deposits of weakly consolidated, poorly sorted siltstone, sandstone and conglomerate. The Livermore gravels are mapped with a gentle (15 to 25 degrees) northeasterly dip. Alluvium (Qa), colluvium (Qc), and landslide deposits (Qls) are also mapped (Nilsen, 1975) in several locations within the study area, as shown on Figure 3.

The study area is located within a State of California Earthquake Fault Hazard Zone for the Verona Fault, which crosses the site adjacent to the 25-percent slope limit (Figure 6). The Calaveras and Las Positas faults are located approximately 2 miles southwest and 4 miles east of the study area, respectively. The Greenville fault is located approximately 10 miles to the east. The Hayward fault is located approximately 7¾ miles to the southwest. The San Andreas fault is located approximately 27 miles to the southwest. Each of these faults has produced earthquakes within the last 200 years.

The maximum earthquake for the region can be expected from the San Andreas fault, the major active fault within the Bay Area. Maximum earthquakes in the Moment Magnitude 6 to 7 range can be expected from the Hayward, Calaveras and Concord faults. The Greenville fault has been assigned a maximum Moment Magnitude of approximately 6.9. Figure 7 shows the study area in relation to the faults discussed. In general, the study area is located within a seismically active area.

The Uniform California Earthquake Rupture Forecast (UCERF, 2008) evaluated the 30-year probability of a Moment Magnitude 6.7 or greater earthquake occurring on the known active fault systems in the Bay Area, including the Calaveras fault. The UCERF generated an overall probability of 63 percent for the Bay Area as a whole, a probability of 31 percent for the Hayward fault, 7 percent for the Calaveras fault, and 3 percent for the Concord-Green Valley fault.

2.1.2.1 Artificial Fill

Based on our reconnaissance of the study area, it appears that past grading was limited to the construction of the dirt roads. Existing fill associated with the previous grading appears to be less than 3 to 4 feet thick and is located in the northern section of the study area and beyond the 25-percent slope limit.

2.1.2.2 Landslides

Regional landslide mapping of the vicinity by Nilsen (1975) (Figure 4) and by Majmundar (1991) shows numerous landslides within the study area.

We used black and white stereo-paired aerial photographs for the purpose of observing natural landforms in the study area. These photographs were used to study geomorphic features, interpret the relationships between landforms and the underlying rock, soil, and geologic structures and observe the presence, character, and activity of suspected slope failures on or adjacent to the study area. Based on our examination of aerial photographs, four possible landslide areas were identified on the southwestern portion of the study area near the 25-percent slope limit (identified as "L-1" through "L-4" on Figure 2). The four possible landslides were identified based on relatively subtle breaks in slope along spur ridges that may be indicative of landslides.

Test pits were previously excavated near the suspected head scarps and toes of two of the possible landslides. Evidence of landslides was observed in the test pits (Appendix A, TP-1 and TP-2 and Appendix B Terrasearch, TP-15 and TP-16). We previously observed drilling of two borings in the lower portion of the large mapped landslide in the central portion of the study area. The borings were drilled using hollow-stem augers and were sampled from about 3 to 25 feet deep using an interval drive sample method. Generally, landslide deposits encountered in the test pits and borings consisted of dark gray, very stiff to hard, silty clay 12 to 14 feet deep overlying several feet of light brown, moist to saturated, soft to stiff sandy clay, overlying poorly to unconsolidated, thinly bedded, friable to weak claystone, siltstone, or sandstone of the Livermore gravel formation. Shearing or other evidence of a landslide slip surface was limited to a sharp, sub-horizontal, soil/bedrock contact encountered in TP-1. Landslide deposits were encountered up to 20 feet thick in landslide area L-4 (Terrasearch TP-16). Findings of the test pits and borings indicate that the landslide areas appear to closely match landslide areas identified through regional mapping and aerial photograph review.

2.1.2.3 Residual Soil

In the upland areas of the study area, bedrock is capped with a relatively thin layer of residual soil, soil that develops essentially in-place from weathering of the underlying parent material. The United States Department of Agriculture maps residual soil as Linne clay loam having strongly calcareous and slightly plastic characteristics. Residual soil encountered at the study area consists predominately of dark brown, silty or sandy clay. The residual soil appears to be dry and hard without significant porosity. Based on visual examination, residual soil appeared to have moderate to high plasticity and may be highly expansive.

2.1.2.4 Colluvium

Regional mapping of surficial deposits in the vicinity by Nilsen (1975) shows much of the base of the west facing slope near the center of the study area to be underlain by colluvium (Figure 4). Colluvial deposits were found to range from silty clay to sandy clay with some fine gravel. The colluvial deposits were typically dark brown, very stiff to hard and varied from dry to moist. Based on the findings of our exploration and data by others, the colluvium appears to range up to about 16 feet in thickness (Terrasearch, 1998, TP-19).

2.1.2.5 Alluvium

The USDA maps alluvial topsoil in the study area as Rincon loam having neutral to mild alkalinity and very plastic characteristics and Pleasanton gravelly loam in the southern portion of the study area consisting of alluvium derived from sandstone and shale. Alluvial materials (Qal) consisting of relatively young, unconsolidated stream deposits were encountered in the borings

(Appendix A, B-1 through B-6) on the relatively flat western portion of the study area. Alluvial soils encountered were generally found to consist of silty or sandy clay with some gravel and minor layers of clayey sand and gravel. The alluvium encountered was primarily medium stiff to hard clay with some medium dense to dense silty or clayey sand and ranged in thickness up to about 18 feet. Laboratory tests on surficial soils in the alluvium resulted in Plasticity Indices (PI) as follows: Boring B-3 at 5 feet, PI = 14; Boring B-4 at 2 feet, PI = 29, and; Boring B-5 at 4 feet, PI = 33. The laboratory test results indicate that the surficial soils have low to high plasticity and corresponding moderate to high expansion potential.

2.1.2.6 Livermore Gravels

Portions of the study area underlain by Livermore Gravels at relatively shallow depths are indicated on the Geologic Map (Figure 3) using the symbol QTl. Where encountered in test pits and borings, the Livermore gravels were found to consist predominantly of light olive-gray and olive-brown siltstone and sandstone with some interbedded light olive-gray conglomerate. The bedrock encountered was friable to weak, and varied from thinly to thickly bedded. A few test pits encountered slightly cemented siltstone that was moderately difficult to excavate with the small excavator used for our exploration.

Bedrock structure noted in the test pits was striking generally northwest with dips ranging from 10 to 30 degrees to the northeast. Some beds at lower elevations were striking in a more northerly to northeasterly direction and had steeper dip angles ranging up to 50 degrees. Bedding planes appeared to be poorly developed and/or gradational.

2.1.2.7 Groundwater

Groundwater was encountered in Borings B-1, B-2, B-3 and B-8 drilled for this investigation at depths of 10, 12, 13½, and 19 feet, respectively. Groundwater was encountered in Test Pit TP-1 at a depth of about 12 feet and could be characterized as a strong seep from a thin, unconsolidated, “clean” sandstone bed. This groundwater could be a localized occurrence of shallow groundwater manifested by the presence of the Verona fault behaving as a groundwater barrier.

Terrasearch reported that groundwater was initially encountered in the lower-lying portion of the study area at depths ranging from 13 to 20 feet deep at the southwest end to 15 feet on the eastern side. A few hours after drilling, the groundwater rose to a depth of 2 feet in Boring 1 and gradually became deeper in other borings toward the eastern end of the study area up to a depth of 11 feet.

Groundwater levels should be expected to vary, depending on weather conditions and the time of year.

3.0 SITE RECONNAISSANCE

We conducted a site reconnaissance on Tuesday, January 6, 2015. From our review of existing information, we conducted our site reconnaissance with a focus on ground conditions and slope stability. Due to soft ground conditions, we limited our site reconnaissance to the area generally to the west of the 25-percent slope limit. It appeared that the upper foot of ground was generally soft from the recent rains and years of disking. We observed native soil at the ground surface in the relatively flat areas and grasses covering the hillsides. We observed surface evidence of the landslides previously identified and shown on Figure 2 near the 25-percent limit. It appears that the site conditions in the area generally west of the 25-percent limit line are similar to the our previous 2004 and 2008 study.

3.1 GROUNDWATER

No groundwater seepage or other indication of near surface groundwater was observed during our reconnaissance. Fluctuations in groundwater levels occur seasonally and over a period of years because of variations in precipitation, temperature, irrigation, or other factors.

4.0 CONCLUSIONS AND PRELIMINARY RECOMMENDATIONS

From a geologic and geotechnical standpoint, the study area is generally suitable for the proposed development, provided that the preliminary recommendations included in this report, along with other sound engineering practices, are incorporated in the design and construction of the project. The primary geologic and geotechnical considerations for this project are:

- Slope stability of the west facing slope at the southern portion of the site.
- Considerable ground shaking from an earthquake of moderate to high magnitude generated within the San Francisco Bay Region.
- Potential for fault rupture.
- Moderately to highly expansive soil.

4.1 LANDSLIDES AND SLOPE STABILITY

As previously discussed, landslides have been mapped throughout the study area. It is our understanding that the improvements are proposed on the western relatively flat portion of the property as delineated by the 25-percent. Four landslides have been identified along the eastern boundary of the proposed improvement area as shown on Figure 2. It does not appear that structures will be located on slide areas, however, structures could be located near the toe of the landslides risking damage to the structure. Recommendations for mitigation of this hazard should be provided in a design-level exploration for the proposed project pending the site plan. We provide the preliminary mitigation option of removal and replacement as engineered fill in

Section 5.5. Other options include avoidance of the area by revising the land plan or construction of a structural wall to protect upslope areas.

4.2 SEISMIC HAZARDS

Potential seismic hazards resulting from a nearby moderate to major earthquake can generally be classified as primary and secondary. The primary effect is ground rupture, also called surface faulting. The common secondary seismic hazards include ground shaking, ground lurching, and liquefaction. The following sections present a discussion of these hazards as they apply to the site.

4.2.1 Ground Rupture

In 1998, Terrasearch, Inc. conducted a supplemental fault investigation on the Verona Fault within the study area. Three fault trenches were excavated across the fault trace previously identified by literature review, aerial photograph interpretation and geophysical surveys. A fault zone, approximately 100 to 130 feet wide, was encountered in the trenches. The location of the fault zone is depicted on Figure 2. We concur with the findings developed by Terrasearch and recommend the establishment of a "Building Restriction Zone" that extends to 50 feet on both sides of the fault zone. We recommend that buildings for residential use not be located within this "Building Restriction Zone."

4.2.2 Ground Shaking

An earthquake of moderate to high magnitude generated within the San Francisco Bay Region could cause considerable ground shaking at the site, similar to that which has occurred in the past. To mitigate the shaking effects, all structures should be designed using sound engineering judgment and the latest California Building Code (CBC) requirements, as a minimum.

4.2.3 Liquefaction

The site is not located within a State of California Seismic Hazard Zone (CGS, 2008) for areas that may be susceptible to liquefaction (Figure 5). In addition, the United States Geological Survey (USGS) has mapped the area to have low susceptibility for liquefaction. Soil liquefaction results from loss of strength during cyclic loading, such as imposed by earthquakes. Soils most susceptible to liquefaction are clean, loose, saturated, uniformly graded, sand. Empirical evidence indicates that loose to medium dense gravel, silty sand, low-plasticity silt, and some low-plasticity clay are also potentially liquefiable.

The alluvial deposits encountered in explorations previously performed in the study area were generally found to be stiff sandy, gravelly clay with lenses of medium dense to dense clayey sand and gravel. According to the previous explorations, granular layers were generally less than three feet thick. Bedrock was encountered less than approximately 20 feet deep. Based on this information, it is our opinion that the potential for liquefaction is not a significant risk to the proposed improvements and will likely result in ground surface settlement 2 inches or less. We preliminarily recommend that building foundation design consider the potential for up to 1 inch of differential settlement due to extreme earthquake-induced liquefaction in design. This differential settlement should be assumed occur over a distance of approximately 50 feet.

4.2.4 Lateral Spreading

Lateral spreading is lateral displacement of sloping ground as a result of pore pressure buildup or liquefaction in a shallow, underlying soil deposit during an earthquake. Lateral spreading, as a result of liquefaction, occurs when a soil mass slides laterally on a liquefied layer, and gravitational and inertial forces cause the layer and the overlying non-liquefied material to move in a downslope direction. The magnitude of lateral spreading movements depends on earthquake magnitude, distance between the site and the seismic event, thickness of the liquefied layer, ground slope or ratio of free-face height to distance between the free face and structure, fines content, average particle size of the materials comprising the liquefied layer, and the density of the soil materials. Due the lack of significant free-faces and limited amount of potentially liquefiable material, it is our opinion that the risk of lateral spread is low.

4.2.5 Earthquake-Induced Landslides

Ground shaking associated with earthquake events can trigger landslides in weak geologic materials, caused by a wide range of mechanisms. Due to the presence of a landslides in the study area, especially near the 25-percent slop limit, the potential for earthquake-induced landslides is considered high. This geologic hazard can be mitigated by various methods including but not limited to removal of potentially unstable materials and replacement as engineered fill or construction of engineered fill buttresses.

4.3 EXPANSIVE SOIL

Expansive soil can shrink and swell as a result of moisture changes. This can cause heaving and cracking of slabs-on-grade, pavements, and structures founded on shallow foundations. Structures can be supported on structural reinforced mat foundations that are designed to accommodate shrinking and swelling subgrade soils.

Successful construction on expansive soil requires special attention during grading. It is imperative to keep exposed soil moist by occasional sprinkling. If the soil dries, it is extremely difficult to remoisturize the soil (because of their clayey nature) without excavation, moisture conditioning, and recompaction.

Conventional grading operations, incorporating fill placement specifications tailored to the expansive characteristics of the soil, and use of a mat foundation (either post-tensioned or conventionally reinforced) are common, generally cost-effective measures to address the expansive potential of the foundation soils. Based upon our initial findings, the effects of expansive soil are expected to pose a low impact when properly mitigated.

4.4 CORROSION POTENTIAL

An evaluation of possible corrosion impacts to study area improvements has not been conducted to date. We recommend that corrosion testing be conducted on the subgrade soil of the final building pads prior to building and utility construction. Clay soil typically has a low resistivity resulting in corrosion to buried metal in direct contact with the soil. Corrosion mitigation of buried metal, such as metallic pipes, will likely be necessary.

4.5 FLOODING

The Federal Emergency Management Agency has not yet mapped the study area for flood risk, the Civil Engineer should review pertinent information relating to possible flood levels for the subject site based on final pad elevations and provide appropriate design measures for development of the project, if recommended.

5.0 PRELIMINARY SITE RECOMMENDATIONS

The following preliminary recommendations are for initial land planning and preliminary estimating purposes. Final recommendations regarding site grading and foundation construction will be provided after additional design level geotechnical exploration has been undertaken.

5.1 GENERAL SITE CLEARING

Clear areas to be developed of all surface and subsurface deleterious materials including buried utility and irrigation lines, debris, and designated trees, shrubs, and associated roots. Clean and backfill excavations extending below the planned finished site grades with suitable material compacted to the recommendations presented in Section 5.8. Retain ENGEO to observe and test all backfilling.

Based on the vegetation observed at the time this report was prepared, we recommend stripping the site to remove surface organic materials, following clearing. Strip organics from the ground surface to a depth of at least 2 to 3 inches below the surface. Remove strippings from the site or, if considered suitable by the landscape architect and owner, use them in landscape fill. Depending on the specific vegetation conditions that exist at the time of grading, alternative recommendations for mowing and/or disking the site can be considered by the Geotechnical Engineer.

5.1.1 Demolition and Stripping

Site development should commence with the removal of and improvements and their foundations, and buried structures including abandoned utilities and their backfill. All debris or soft compressible soils should be removed from any location to be graded, from areas to receive fill or structures, and from those areas to serve as borrow. The depth of removal of such materials should be determined by the Geotechnical Engineer in the field at the time of grading.

Existing vegetation and any pavement (asphalt concrete/concrete and underlying aggregate base) should be removed from areas to receive fill, or structures, or those areas to serve for borrow. Tree roots should be removed down to a depth of at least 3 feet below existing grade. The actual depths of tree root removal should be determined by the Geotechnical Engineer's representative in the field. Subject to approval by the Landscape Architect, strippings and organically contaminated soils can be used in landscape areas. Otherwise, such soils should be removed from the study areas. Any topsoil that will be retained for future use in landscape areas should be stockpiled in areas where it will not interfere with grading operations.

All excavations from demolition and stripping below design grades should be cleaned to a firm undisturbed soil surface determined by the Geotechnical Engineer. This surface should then be scarified, moisture conditioned, and backfilled with compacted engineered fill. The requirements for backfill materials and placement operations are the same as for engineered fill.

No loose or uncontrolled backfilling of depressions resulting from demolition and stripping is permitted.

5.2 COLLUVIUM AND LANDSLIDE REMOVALS

We recommend removal of all existing colluvium and landslide debris within the development limit and within the west facing slope located near the 25-percent slope limit. Figure 8 and Figure 9 show cross-sections of the landslide with approximate removal areas.

5.3 ACCEPTABLE FILL

Onsite soil and rock material is suitable as fill material provided it is processed to remove concentrations of organic material, debris, and particles greater than 8 inches in maximum dimension.

5.4 SLOPES

5.4.1 Gradients

For planning purposes, slope gradients for proposed graded slopes should not be steeper than 3:1 (horizontal:vertical); slopes inclined steeper than 3:1 will require special evaluation and may require geogrid reinforcement. In addition, cut slopes may require reconstruction of the exposed slope as engineered fill if adverse conditions are encountered during excavation. The contractor is responsible to construct temporary construction slopes in accordance with CAL-OSHA requirements.

5.4.2 Fill Placed on Existing Slopes

We recommend keying and benching where fill is placed on original grade with a gradient of 6:1 or steeper. The contractor should do the following when keying and benching fill on existing slopes.

Construct a minimum 24-foot-wide keyway inward from the toe of the new fill slope as shown on Figure 10. Extend the keyway at least 3 feet below original grade into firm competent soil/rock, as determined in the field by the representative of the Geotechnical Engineer. Slope the keyway bottom at least 2 percent downward toward the heel of the keyway. Deeper keyways may be required based on actual soil/rock conditions observed during construction.

Cut benches into original grade after the keyway has been nearly filled with compacted engineered fill. Construct benches into original slope grade as filling proceeds every 2 feet vertically, to remove loose soil/rock. Deeper bench depths may be required depending on actual conditions observed during construction. Bench widths will vary depending on the original slope grade and actual bench depth.

5.4.3 Slope Setbacks

For planning purposes we recommend that buildings be set back from the top of slope in accordance with CBC requirements (a minimum of 1/3 the height of the slope or a maximum of 40 feet). Alternatively, deep foundations such as pier-and-grade-beam foundations should be anticipated for buildings close to the top of slopes.

5.5 CUT/FILL TRANSITION OR CUT LOTS

Building pads constructed in cut may encounter variably expansive subsurface conditions in the near-surface soil and rock; these pads may therefore be subject to damaging differential soil movements. Building pads that transition from cut to fill within the building pad area also can experience differential soil movements.

We recommend such building pads be reconstructed to create uniform subgrade conditions. This can be accomplished by subexcavating the soil on the building pads to a minimum depth of 2 feet below finished pad grade on cut lots or lots constructed over cut-and-fill transitions and replacing the subexcavated material with uniformly mixed compacted fill. The subexcavation should be performed over the entire flat pad area. We present overexcavation recommendations in Figure 11 to mitigate the effects of differential materials located under a structure.

5.6 DIFFERENTIAL FILL THICKNESS

Differential building movements may result from conditions where building pads have significant differentials in fill thickness. We recommend that the differential fill thickness across any lot be no greater than 10 feet. Local subexcavation of soil material and replacement with compacted fill may be needed to achieve this recommendation.

5.7 FILL PLACEMENT

For land planning and cost estimating purposes, the following compaction control requirements should be anticipated for general fill areas:

Test Procedures:	ASTM D-1557.
Required Moisture Content:	Not less than 4 percentage points above optimum moisture content.
Minimum Relative Compaction:	Not less than 90 percent relative compaction.

Relative compaction refers to the in-place dry density of soil expressed as a percentage of the maximum dry density of the same material.

Additional compaction requirements may be required for near-surface building pad foundation soils and retaining wall backfill soils. These additional requirements will be developed during our detailed exploration.

5.8 STATIC AND PERCHED GROUNDWATER

Based on the previous data, it appears that the static groundwater level beneath the study areas could affect proposed development. The highest groundwater elevation recorded was approximately 2 feet below the existing grade. In addition to impacting the construction of underground utilities, shallow or perched water can:

1. Impede grading activities.

2. Cause moisture damage to sensitive floor coverings.

3. Transmit moisture vapor through slabs causing excessive mold/mildew build-up, fogging of windows, and damage to computers and other sensitive equipment.

4. Cause premature pavement failure if hydrostatic pressures build up beneath the section.

Due to a relatively high groundwater table, groundwater will likely be encountered during construction of some underground utilities. Temporary construction dewatering should be anticipated during these construction activities.

5.9 SITE DRAINAGE

5.9.1 Surface Drainage

The project civil engineer is responsible for designing surface drainage improvements. With regard to geotechnical engineering issues, we provide the following minimum recommendation for surface drainage.

1. Slope pavement areas a minimum of 1 percent towards drop inlets or other surface drainage devices.

2. Slope finished grade away from building exteriors at a minimum of 5 percent for a distance of at least 10 feet.

3. Discharge roof downspouts into closed conduits and direct away from buildings to appropriate drainage devices.

5.9.2 Subsurface Drainage

Subsurface drainage systems should be installed in all keyways and swales or natural drainage areas. In addition, lot subdrains should be installed at the toe of cut or fill slopes above residential lots.

We recommend that we be retained to review the grading plans and show the approximate locations of recommended subdrains on a remedial grading plan. Depending on the actual conditions encountered during grading, similar subsurface drainage facilities may be recommended within low-lying areas. Subrains should also be added where wet conditions are encountered during grading.

5.9.3 Stormwater Infiltration and Stormwater Bioretention Areas

Due to the density of the site soil and fines content (percentage passing the No. 200 sieve) generally exceeding 30 percent, the near-surface site soil is expected to have a low to moderate permeability value for stormwater infiltration in grassy swales or permeable pavers, unless subdrains are installed. Therefore, Best Management Practices should assume that limited stormwater infiltration will occur at the site.

If bioretention areas are implemented, we recommend that, when practical, they be planned a minimum of 5 feet away from structural site improvements, such as buildings, streets, retaining walls, and sidewalks/driveways. In addition, we recommend that bioretention areas not be located at the tops of slopes.

5.10 FOUNDATIONS DESIGN

Based on the soil data and the proposed building type, it is our opinion that the structures can be supported on structural mat foundations.

A minimum mat thickness of 10 to 12 inches should be anticipated for preliminary purposes. A maximum allowable bearing pressure of 1,000 psf for dead-plus-live loads, which may be increased by 1/3 when considering total loads including wind or seismic, could also be incorporated for initial design purposes.

5.11 PRELIMINARY BUILDING CODE SEISMIC DESIGN

We provide the 2013 CBC seismic design parameters in Table 3.9-1 below, which include design spectral response acceleration parameters based on the mapped Risk-Targeted Maximum Considered Earthquake (MCE_R) spectral response acceleration parameters.

TABLE 5.11-1
2013 CBC Seismic Design Parameters
Latitude: 37.63579° Longitude: -121.86809°

Parameter	Value
Site Class	D
Mapped MCE_R Spectral Response Acceleration at Short Periods, S_S (g)	2.06
Mapped MCE_R Spectral Response Acceleration at 1-second Period, S_1 (g)	0.78
Site Coefficient, F_A	1.00
Site Coefficient, F_V	1.50
MCE_R Spectral Response Acceleration at Short Periods, S_{MS} (g)	2.06
MCE_R Spectral Response Acceleration at 1-second Period, S_{M1} (g)	1.18
Design Spectral Response Acceleration at Short Periods, S_{DS} (g)	1.37
Design Spectral Response Acceleration at 1-second Period, S_{D1} (g)	0.78
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.79
Long period transition-period, T_L	8 sec

5.12 PRELIMINARY PAVEMENT DESIGN

The following preliminary pavement section has been determined for a Traffic Indices of 4 through 7, an assumed R-value of 5, and in accordance to the design methods contained in Chapter 630 of Caltrans Highway Design Manual.

TABLE 5.12-1
Preliminary Pavement Section

Traffic Index	AC (inches)	AB (inches)
4.0	2.5	8.0
5.0	3.0	10.0
6.0	3.5	13.0
7.0	4.0	16.0

Note: AC – Asphalt Concrete
AB – Caltrans Class 2 aggregate base (R-value of 78 or greater)

The above preliminary pavement section is provided for estimating only. We recommend the actual subgrade material should be tested for R-value and the Traffic Index and minimum pavement section(s) should be confirmed by the Civil Engineer and the City of Pleasanton.

5.13 ADDITIONAL RECOMMENDATIONS

Based upon our findings and assuming that the project proceeds into the next phase of development, additional geotechnical studies will be necessary. These studies will include:

- A geotechnical exploration and report for the proposed development. The site exploration should include both exploratory borings and test pits, as appropriate. The exploration is necessary to characterize site-specific subsurface conditions, collect soil samples for laboratory analysis, and determine site-specific recommendations for construction.

- A review of final construction plans and specifications, including grading plans, foundation plans and calculations for conformance with our recommendations.

Although these studies were not included in our current scope of services, we believe that they are important in expediting approval by governing agencies and achieving cost-effective construction. We will be pleased to provide an estimate for these additional services once final plans are available.

6.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report presents geotechnical recommendations for design of the improvements discussed in Section 1.3 for the development project. If changes occur in the nature or design of the project, we should be allowed to review this report and provide additional recommendations, if any. It is the responsibility of the owner to transmit the information and recommendations of this report to the appropriate organizations or people involved in design of the project, including but not limited to developers, owners, buyers, architects, engineers, and designers. The conclusions and recommendations contained in this report are solely professional opinions and are valid for a period of no more than 2 years from the date of report issuance.

We strived to perform our professional services in accordance with generally accepted geotechnical engineering principles and practices currently employed in the area; no warranty is expressed or implied. There are risks of earth movement and property damages inherent in building on or with earth materials. We are unable to eliminate all risks or provide insurance; therefore, we are unable to guarantee or warrant the results of our services.

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Actual field or other conditions will necessitate clarifications, adjustments, modifications or other changes to ENGEO's documents. Therefore, ENGEO must be engaged to prepare the necessary clarifications, adjustments, modifications or other changes before construction activities commence or further activity proceeds. If ENGEO's scope of services does not include onsite construction observation, or if other persons or entities are retained to provide such services, ENGEO cannot be held responsible for any or all claims arising from or resulting from the performance of such services by other persons or entities, and from any or all claims arising from or resulting from clarifications, adjustments, modifications, discrepancies or other changes necessary to reflect changed field or other conditions.

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FIGURES

F
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G
U
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N

0 FEET 2000

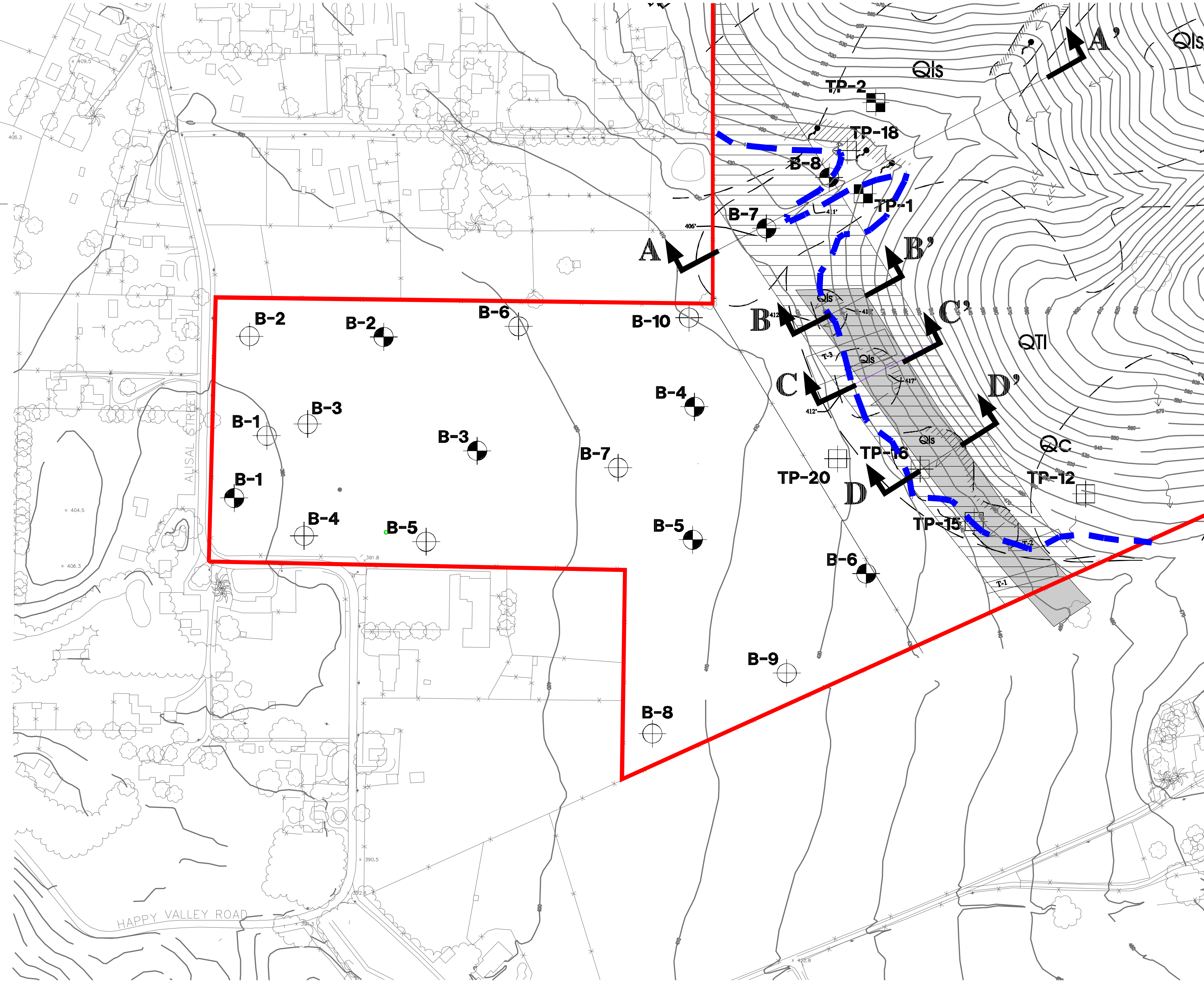
0 METERS 1000

BASE MAP SOURCE: GOOGLE EARTH PRO

ENGEO Expect Excellence	VICINITY MAP SPOTORNO PROPERTY PLEASANTON, CALIFORNIA	PROJECT NO.: 6103.200.000 SCALE: AS SHOWN DRAWN BY: SRP CHECKED BY: JF	FIGURE NO. 1

G:\Drafting\DRAFTING2_Dwg\6103\200\6103200000-1-VicinityMap-0115.dwg Plot Date: 1-07-15 spatters

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

	LIMIT OF 25% SLOPE
	GEOLOGIC CONTACT
	LANDSLIDE, HACHURES INDICATE SCARP LANDSLIDE AND COLLUVIUM AREA DESIGNATIONS ARE GIVEN)
B-10	BORING (ENGEO INCORPORATED, 2004)
B-9	BORING (TERRASEARCH INC., 1999)
TP-33	TEST PIT (TERRASEARCH, INC.)
TP-12	TEST PIT (ENGEO INCORPORATED, 2004) FAULT TRENCHES (TERRASEARCH INC., 1998)
T-3	FAULT TRENCHES (TERRASEARCH INC., 1998)
	FAULT ZONE (TERRASEARCH INC., 1998)
	BUILDING SETBACK ZONE
Qal	ALLUVIUM
Qc	COLLUVIUM
Qls	LANDSLIDE DEBRIS
QTl	LIVERMORE GRAVELS
Qaf	ARTIFICAL FILL
	SPRING
	GULLY
A A'	GEOLOGIC CROSS SECTION

N

0 FEET 300
0 METERS 150

BASE MAP SOURCE: UNKNOWN

ENGEO Expect Excellence

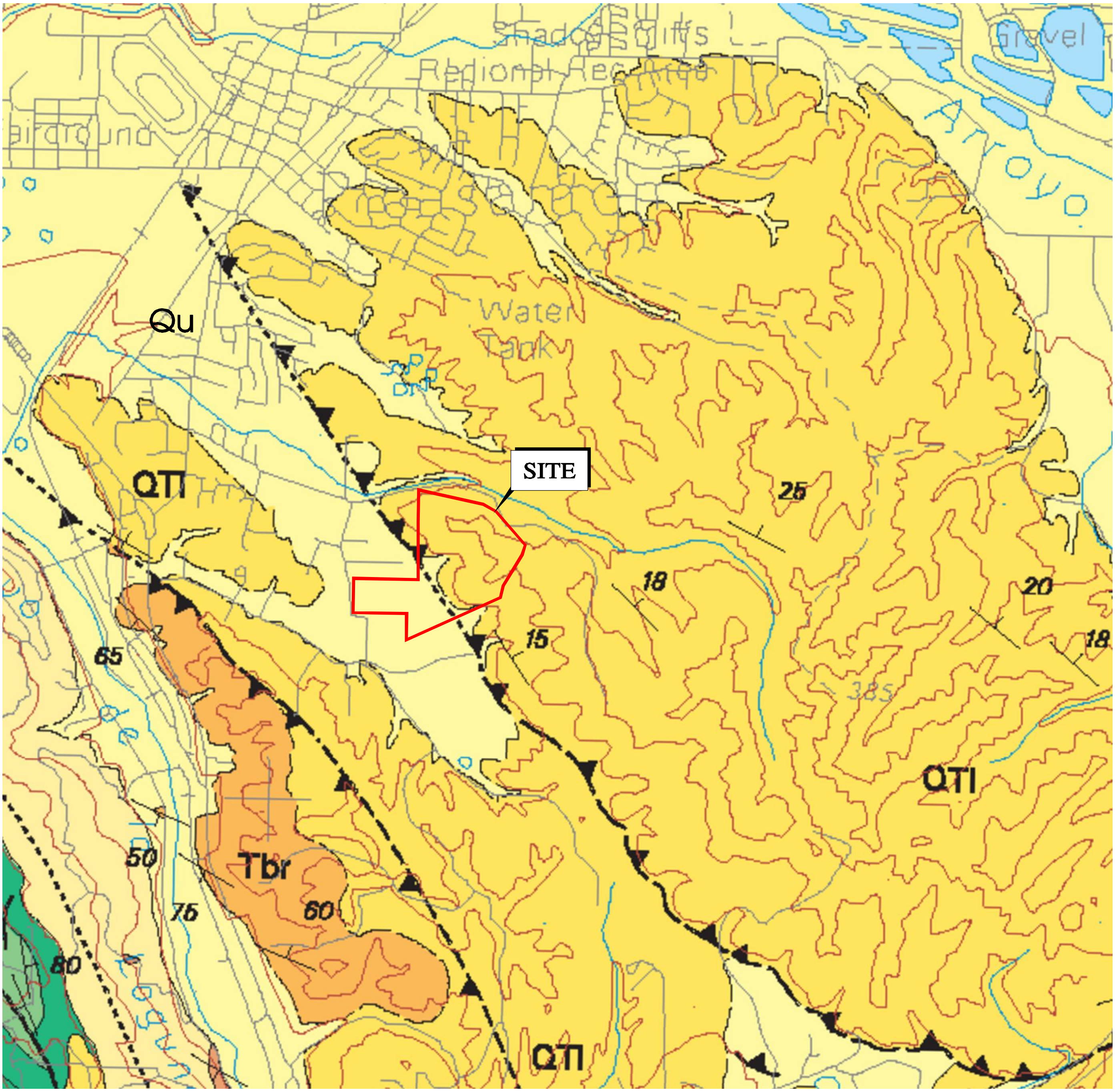
SITE PLAN
SPOTORNO PROPERTY
DUBLIN, CALIFORNIA

PROJECT NO.: 6103.200.000
SCALE: AS SHOWN
DRAWN BY: SRP | CHECKED BY: JF

FIGURE NO. 2

G:\Drafting\DRAFTING2_Dwg\6103\200\6103200000-2-SitePlan-0115.dwg Plot Date: 1-07-15 spatters

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

Qu	UNDIVIDED SURFICIAL DEPOSITS
QTl	LIVERMORE GRAVEL FORMATION
Tbr	BRIONES FORMATION
	OBLIQUE FAULT
25	STRIKE AND DIP OF BEDDING

BASE MAP SOURCE: GRAYMER, 1996

ENGEO — Expect Excellence

REGIONAL GEOLOGIC MAP

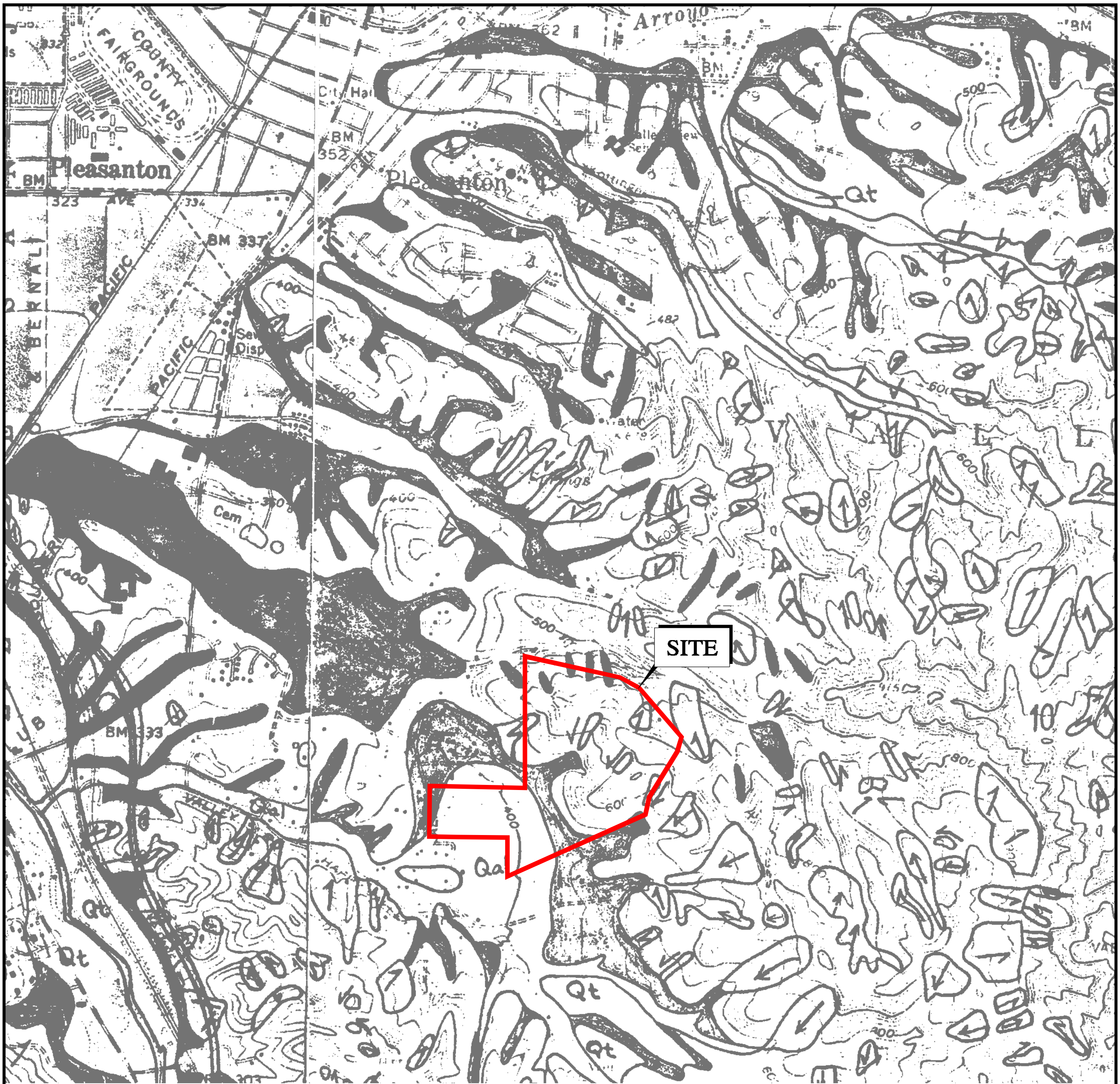
SPOTORNO PROPERTY

PLEASANTON, CALIFORNIA

PROJECT NO.: 6103.200.000	FIGURE NO. 3
SCALE: AS SHOWN	
DRAWN BY: SRP	CHECKED BY: JF

G:\Drafting\DRAFTING2_Dwg\6103\200\6103200000-3-RegionalGeology-0115.dwg Plot Date: 1-07-15 spatters

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

LANDSLIDE DEPOSIT. ARROWS INDICATE GENERAL DIRECTION OF DOWNSLOPE MOVEMENT. QUERIED WHERE UNCERTAIN

Qal ALLUVIAL DEPOSIT

Qt ALLUVIAL TERRACE DEPOSIT. QUERIED WHERE UNCERTAIN

COLLUVIAL DEPOSIT AND/OR SMALL ALLUVIAL FAN DEPOSIT

BEDROCK. QUERIED WHERE IDENTIFICATION UNCERTAIN

N

0 FEET 2000.

0 METERS 1000

BASE MAP SOURCE: NILSEN, 1975

ENGEO
Expect Excellence

REGIONAL SLOPE STABILITY

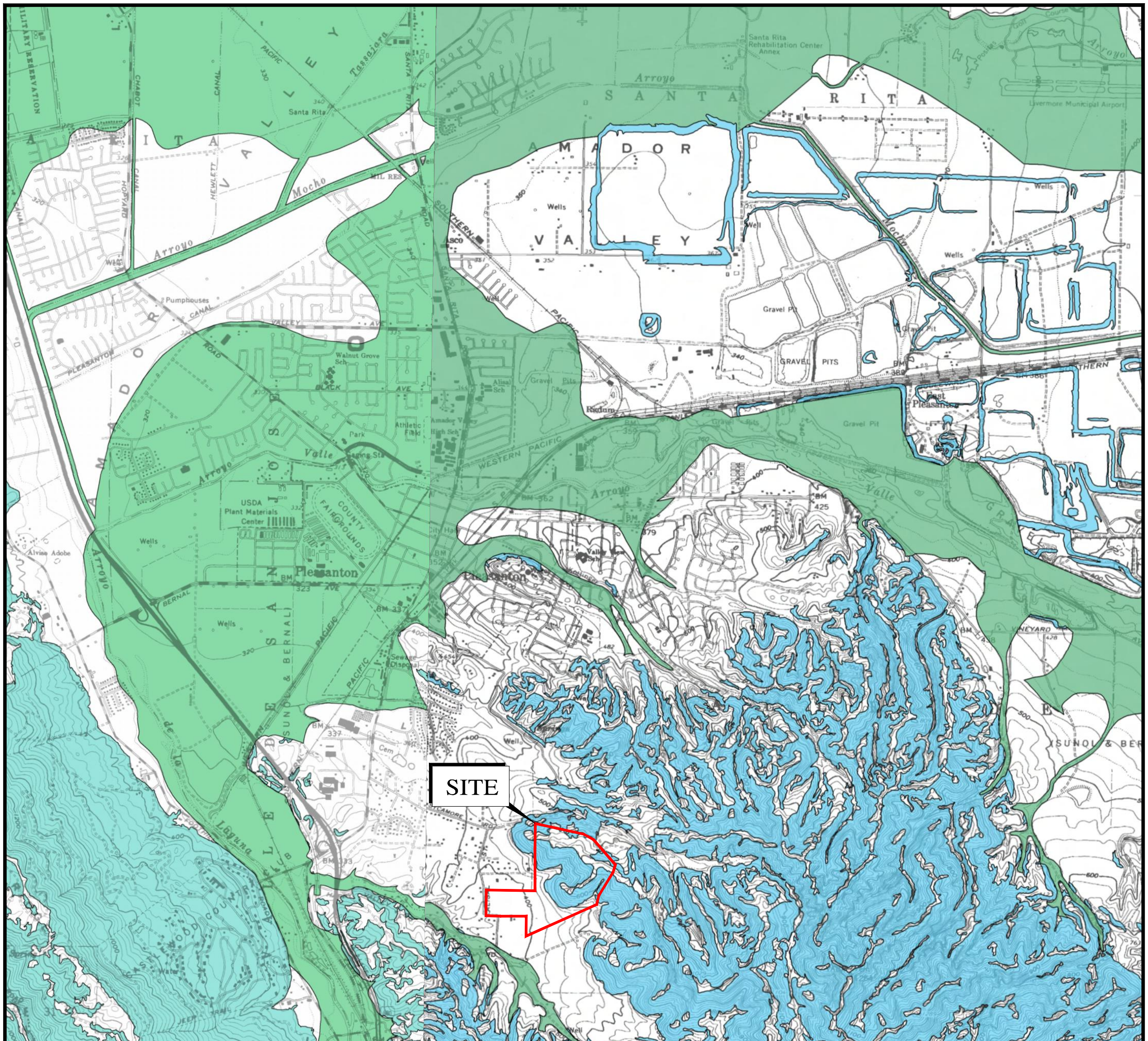
SPOTORNO PROPERTY

DUBLIN, CALIFORNIA

PROJECT NO.: 6103.200.000		FIGURE NO. 4
SCALE: AS SHOWN		
DRAWN BY: SRP	CHECKED BY: JF	

G:\Drafting\DRAFTING2_Dwg\6103\200\6103200000-4-SlopeStabilityMap-0115.dwg Plot Date: 1-07-15 spatters

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

LIQUEFACTION

AREAS WHERE HISTORIC OCCURRENCE OF LIQUEFACTION, OR LOCAL GEOLOGICAL, GEOTECHNICAL AND GROUNDWATER CONDITIONS INDICATE A POTENTIAL FOR PERMANENT GROUND DISPLACEMENTS SUCH THAT MITIGATION AS DEFINED IN PUBLIC RESOURCES CODE SECTION 2693(c) WOULD BE REQUIRED

EARTHQUAKE-INDUCED LANDSLIDES

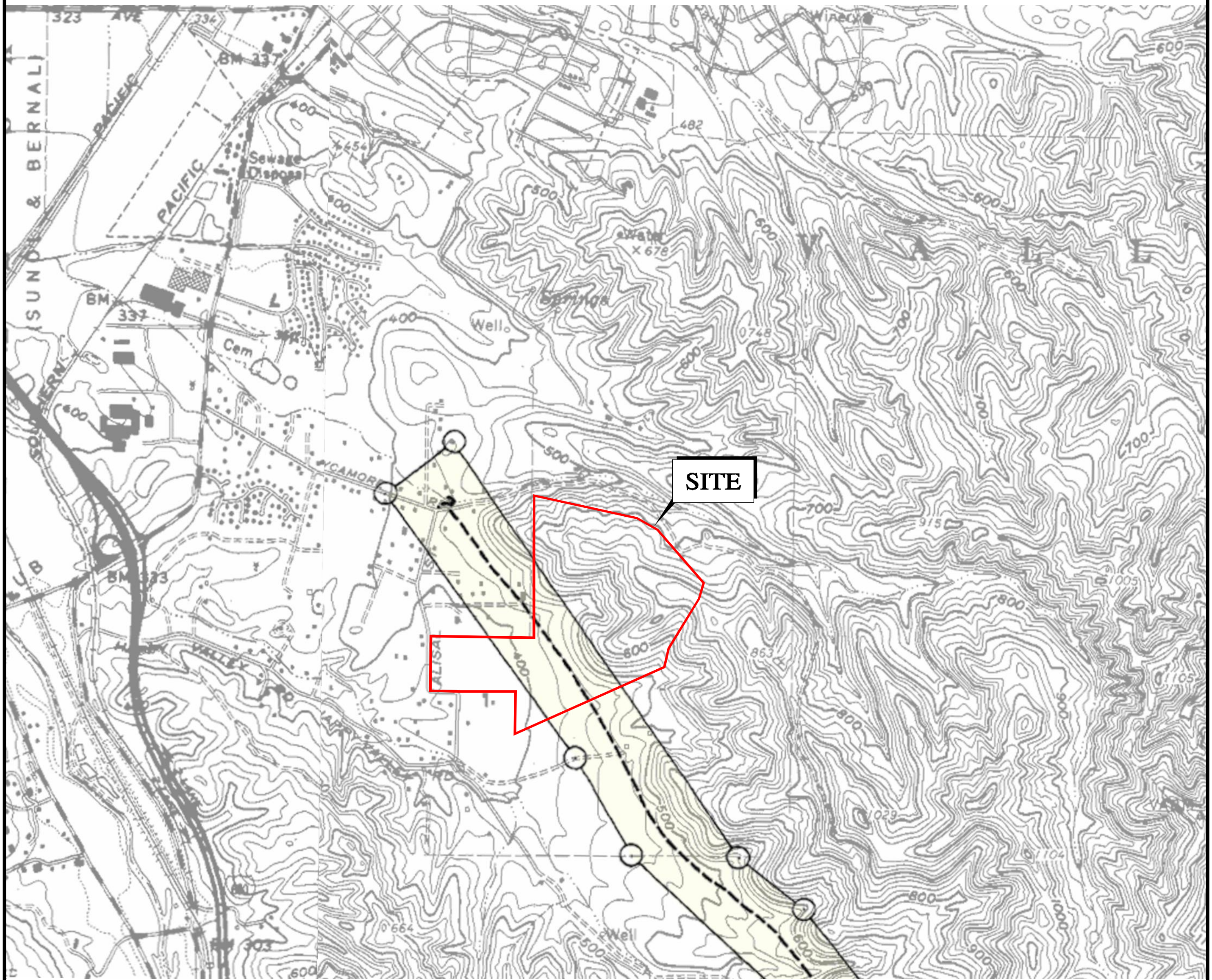
AREAS WHERE PREVIOUS OCCURRENCE OF LANDSLIDE MOVEMENT, OR LOCAL TOPOGRAPHIC, GEOLOGICAL, GEOTECHNICAL AND SUBSURFACE WATER CONDITIONS INDICATE A POTENTIAL FOR PERMANENT GROUND DISPLACEMENTS SUCH THAT MITIGATION AS DEFINED IN PUBLIC RESOURCES CODE SECTION 2693(c) WOULD BE REQUIRED

BASE MAP SOURCE: CALIFORNIA DEPARTMENT OF CONSERVATION, CALIFORNIA GEOLOGICAL SURVEY, 2006

ENGEO — Expect Excellence	SEISMIC HAZARD ZONES MAP SPOTORNO PROPERTY PLEASANTON, CALIFORNIA	PROJECT NO.: 6103.200.000 SCALE: AS SHOWN DRAWN BY: SRP · CHECKED BY: JF	FIGURE NO. 5

C:\Users\spatters\appdata\local\temp\AcPublish_12220\6103200000-5-SeismicHazardZonesMap-0115.dwg Plot Date: 1-07-15 spatters

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

1906 C

FAULTS CONSIDERED TO HAVE BEEN ACTIVE DURING HOLOCENE TIME AND TO HAVE A RELATIVELY HIGH POTENTIAL FOR SURFACE RUPTURE; SOLID LINE WHERE ACCURATELY LOCATED, LONG DASH WHERE APPROXIMATELY LOCATED, SHORT DASH WHERE INFERRED, DOTTED WHERE CONCEALED; QUERY (?) INDICATES ADDITIONAL UNCERTAINTY. EVIDENCE OF HISTORIC OFFSET INDICATED BY YEAR OF EARTHQUAKE-ASSOCIATED EVENT OR C FOR DISPLACEMENT CAUSED BY CREEP OR POSSIBLE CREEP

EARTHQUAKE FAULT ZONE BOUNDARIES; DELINEATED AS STRAIGHT-LINE SEGMENTS THAT CONNECT ENCIRCLED TURNING POINTS SO AS TO DEFINE EARTHQUAKE FAULT ZONE SEGMENTS

N

0 FEET 2000
0 METERS 1000

BASE MAP SOURCE: CDMG, 1993

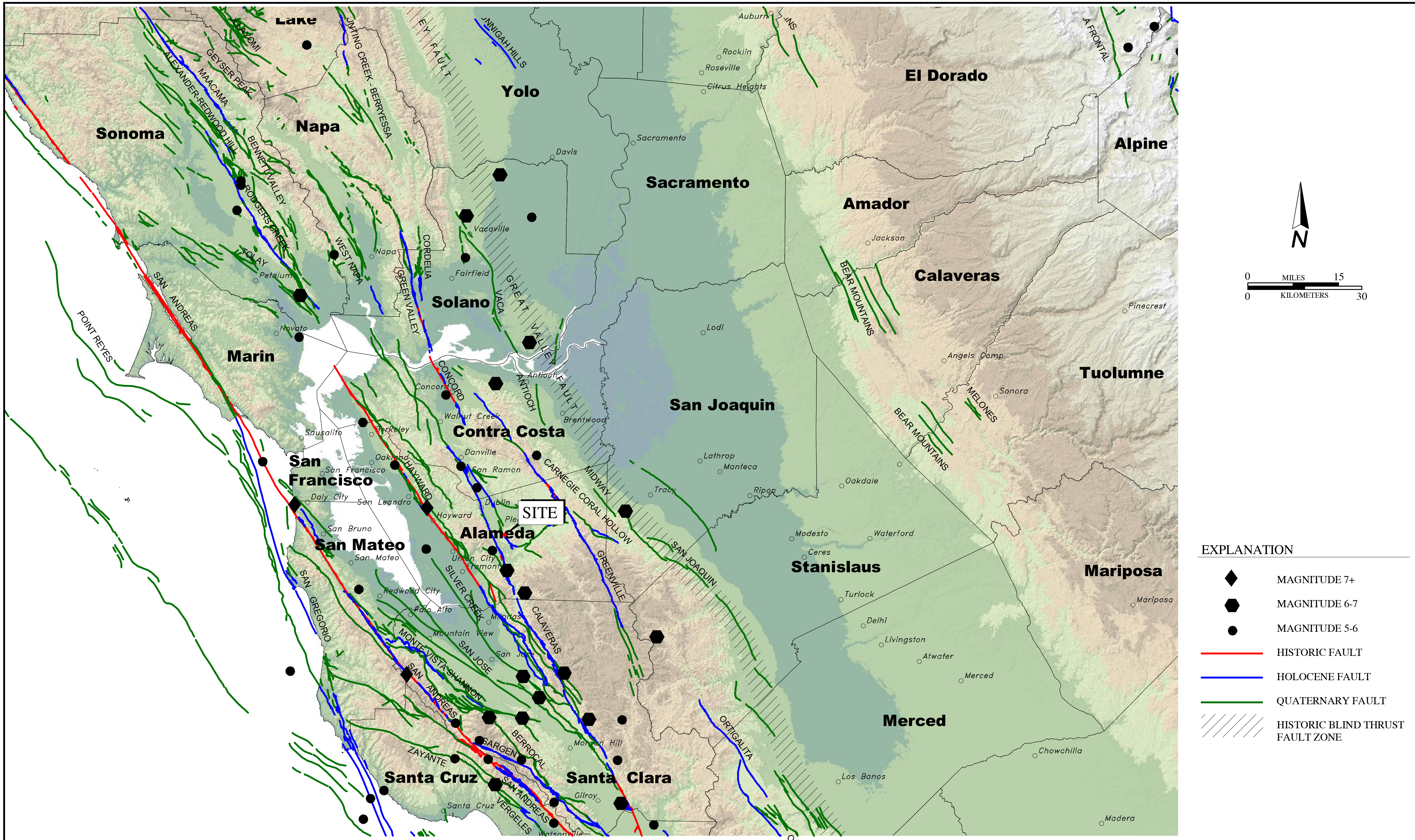
ENGEO
Expect Excellence

EARTHQUAKE FAULT ZONE MAP
SPOTORNO PROPERTY
PLEASANTON, CALIFORNIA

PROJECT NO.: 6103.200.000	FIGURE NO. 6
SCALE: AS SHOWN	
DRAWN BY: SRP	CHECKED BY: JF

G:\Drafting\DRAFTING2_Dwg\6103\200\6103200000-6-FaultZoneZonesMap-0115.dwg Plot Date:1-07-15 spatters

ORIGINAL FIGURE PRINTED IN COLOR



BASE MAP SOURCE:
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATASET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, NOVEMBER, 2010
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800–2000)

ENGEO Expect Excellence

REGIONAL FAULTING AND SEISMICITY
SPOTORNO PROPERTY
PLEASANTON, CALIFORNIA

PROJECT NO.:	6103.200.000
SCALE:	AS SHOWN
DRAWN BY: SRP	CHECKED BY: JF
FIGURE NO	7

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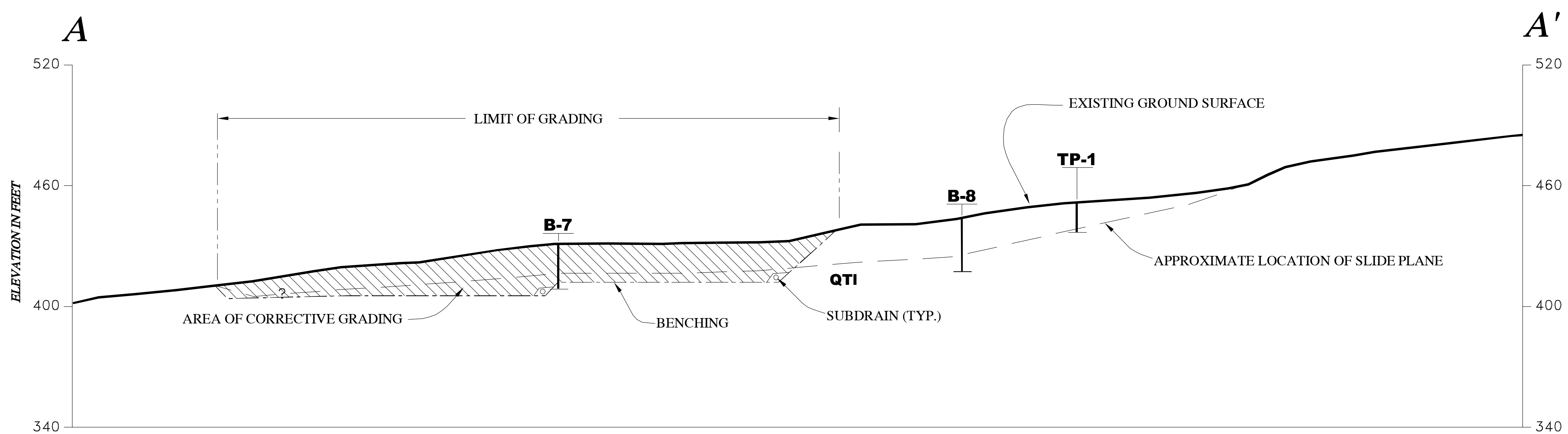
ORIGINAL FIGURE PRINTED IN COLOR



SECTION A-A'
1"=60'

EXPLANATION
ALL LOCATIONS ARE APPROXIMATE

Qls LANDSLIDE

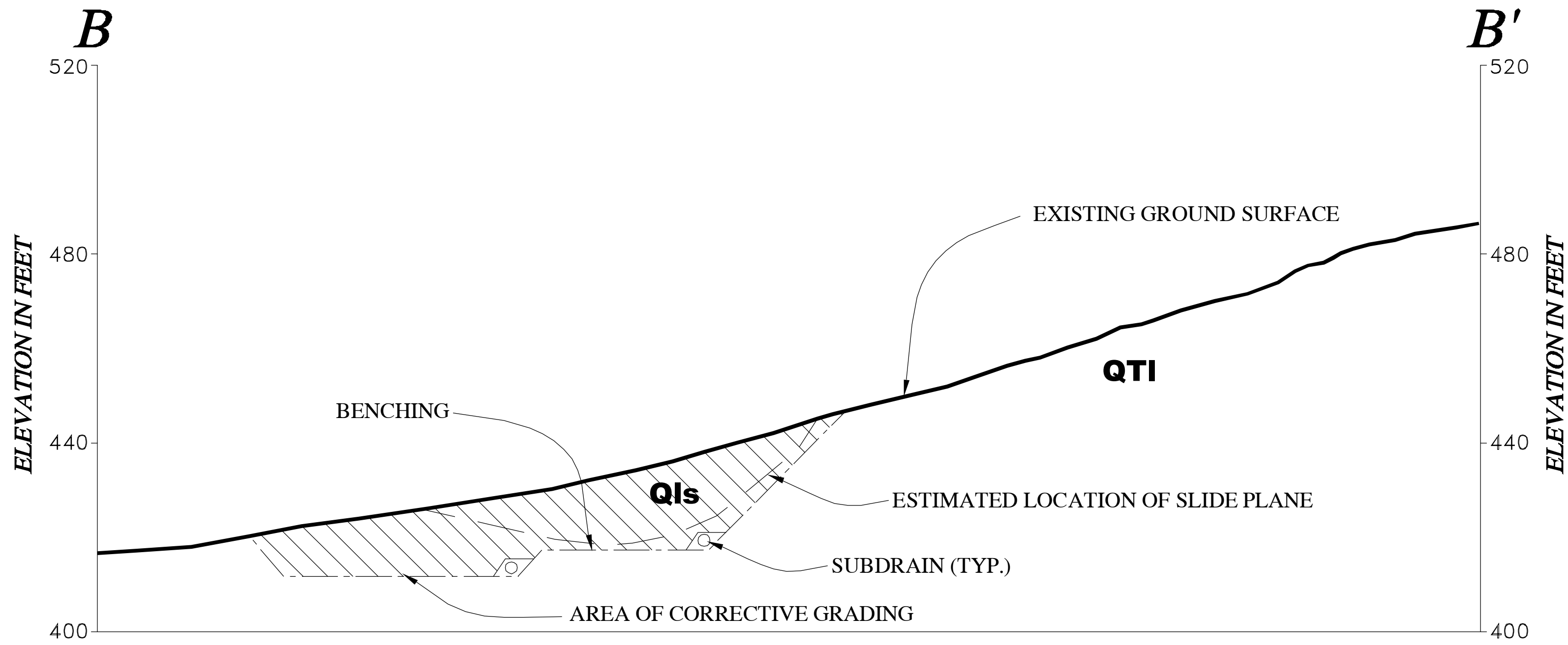
QTl LIVERMORE GRAVELS

TP1 TEST PIT (ENGEO, 2004)

B-7 BORING (ENGEO 2004

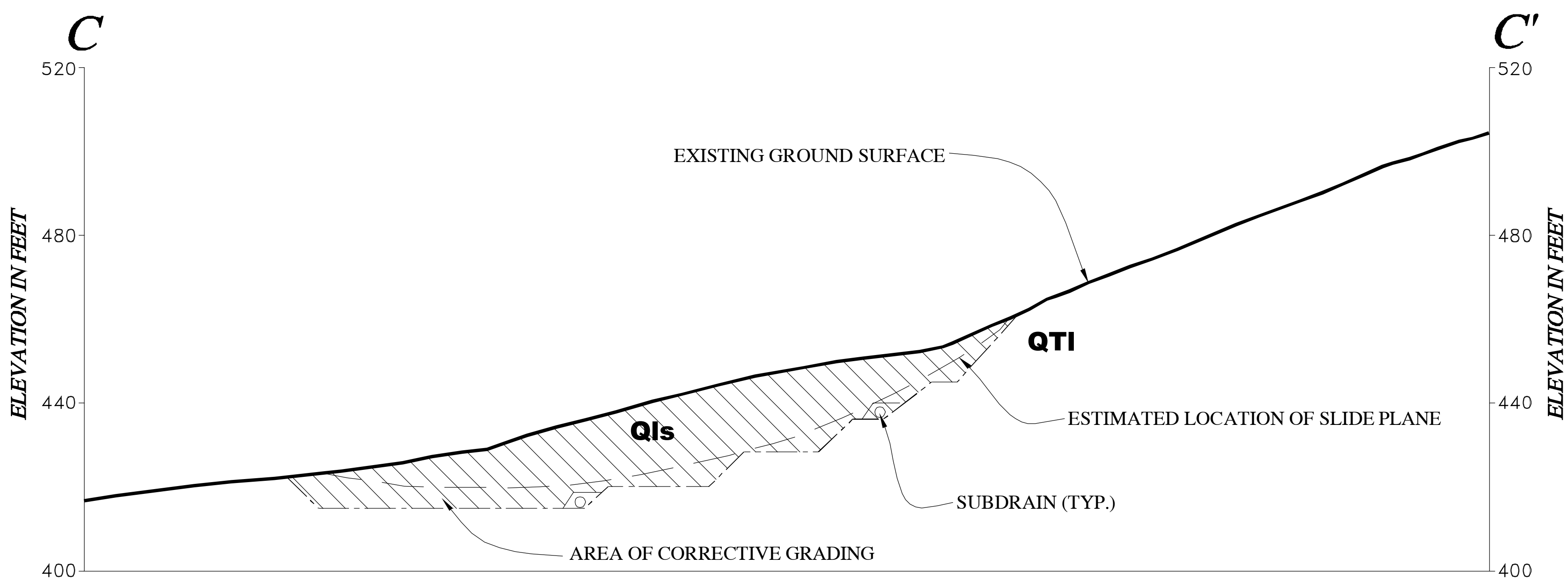
ENGEO Expect Excellence	GEOLOGIC CROSS SECTION A-A' SPOTORNO PROPERTY PLEASANTON, CALIFORNIA	PROJECT NO.: 6103.200.000 SCALE: AS SHOWN DRAWN BY: SRP CHECKED BY: JF
		FIGURE NO 8

G:\Drafting\DRAFTING2_Dwg\6103\200\610320000-8-SectionsDD_EE_FF-0115.dwg Plot Date:1-07-15 spatters



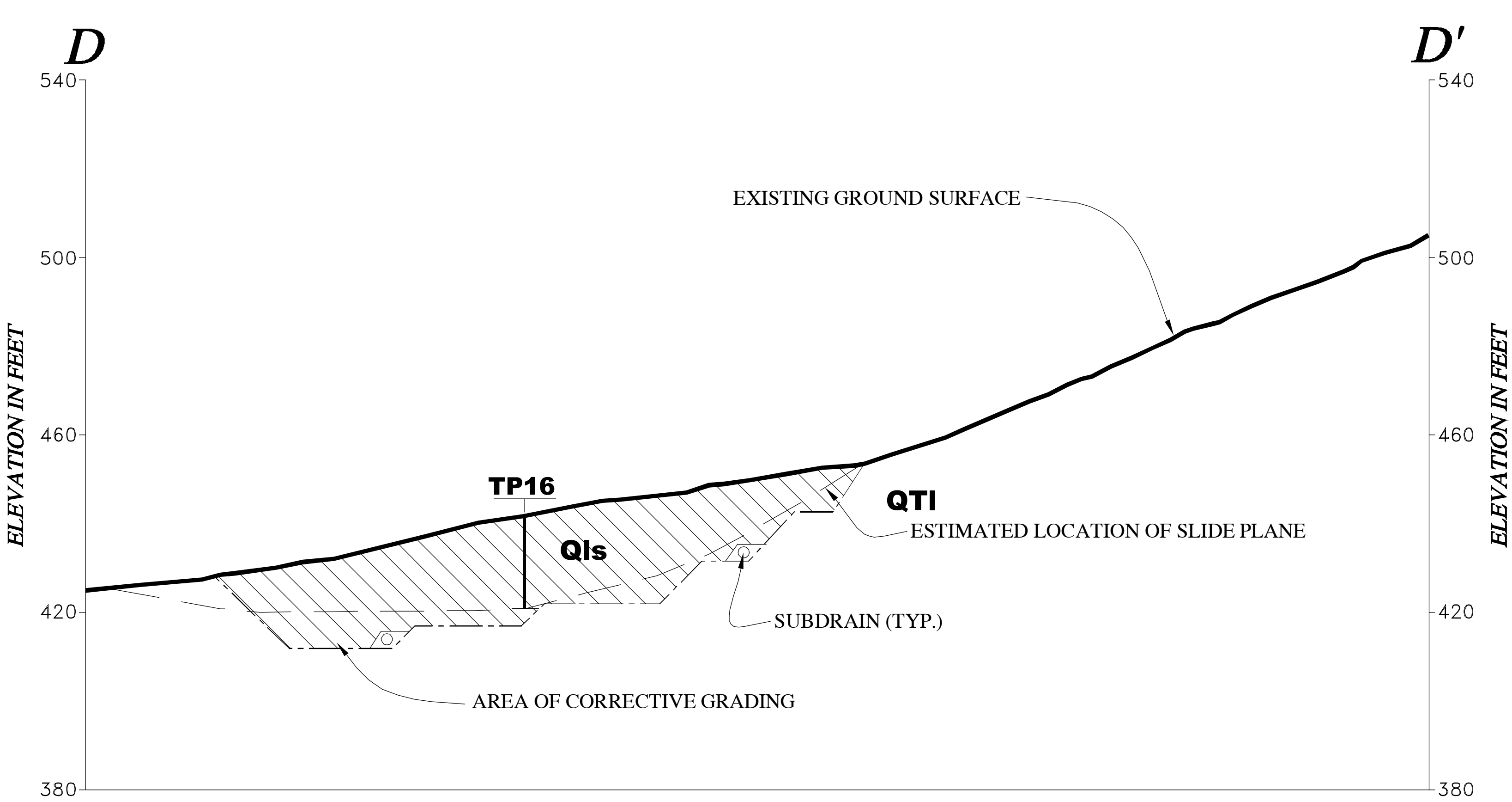
SECTION B-B'

1"=40'



SECTION C-C'

1"=40'



SECTION D-D'

1"=40'

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

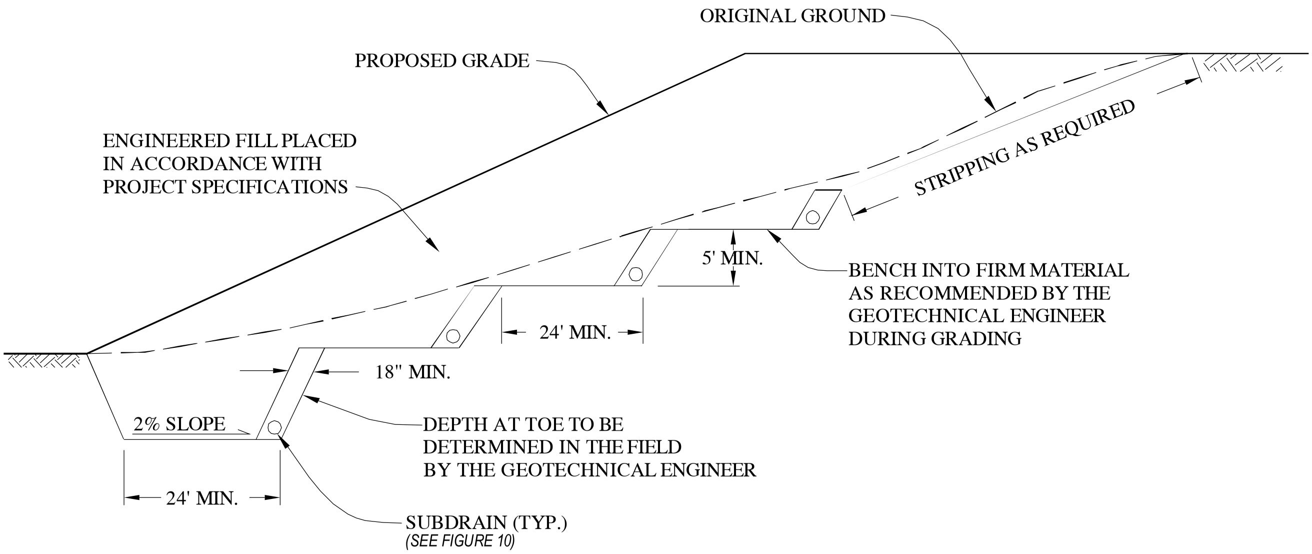
Qls LANDSLIDE

QTl LIVERMORE GRAVELS

TP16 TEST PIT (TERRASEARCH, INC.)

ENGEO Expect Excellence	GEOLOGIC CROSS SECTIONS B-B', C-C' AND D-D' SPOTORNO PROPERTY PLEASANTON, CALIFORNIA	PROJECT NO.: 6103.200.000 SCALE: AS SHOWN DRAWN BY: SRP CHECKED BY: JF	FIGURE NO 9

ORIGINAL FIGURE PRINTED IN COLOR



ENGEO Expect Excellence

TYPICAL SLOPE KEYWAY

SPOTORNO PROPERTIES

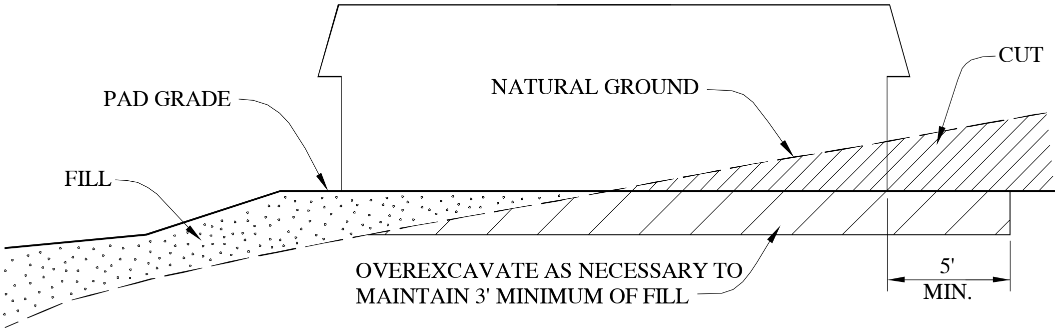
PLEASANTON, CALIFORNIA

PROJECT NO.: 6103.200.000	
SCALE: AS SHOWN	
DRAWN BY: SRP	CHECKED BY: JF

FIGURE NO. 10

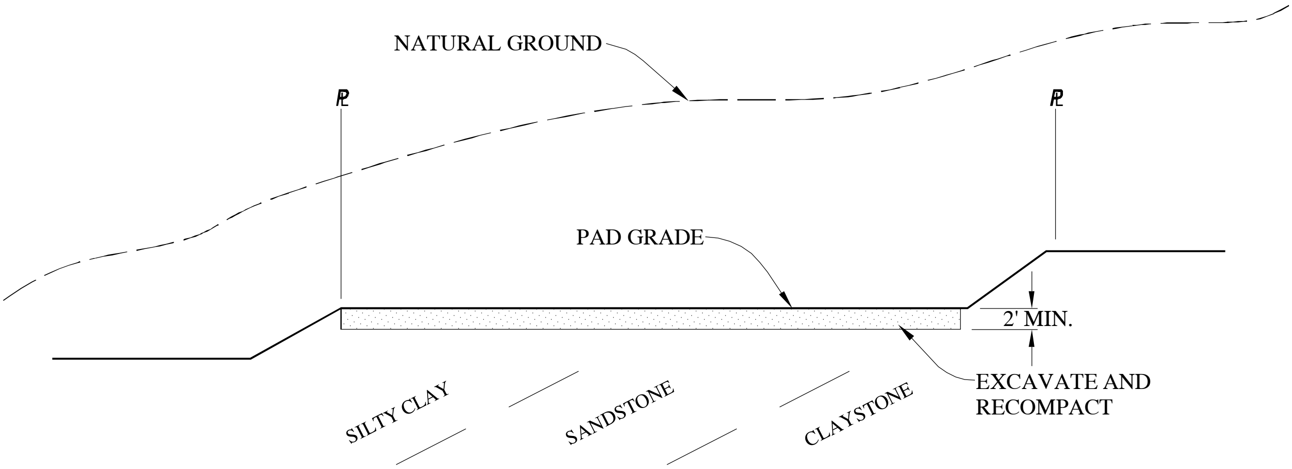
G:\Drafting\DRAFTING2_Dwg\6103\200\6103200000-10-WetlandSubdrnDtl-0115.dwg Plot Date:1-07-15 spatters

ORIGINAL FIGURE PRINTED IN COLOR



WHERE LOTS ARE PARTIALLY IN FILL, AND PARTIALLY IN CUT, THE CUT PORTION MUST BE OVEREXCAVATED AS SHOWN

CUT/FILL LOT



WHERE LOTS ARE COMPLETELY IN CUT, THE UPPER 2' SHOULD BE EXCAVATED AND RECOMPACTED AS SHOWN

CUT LOT

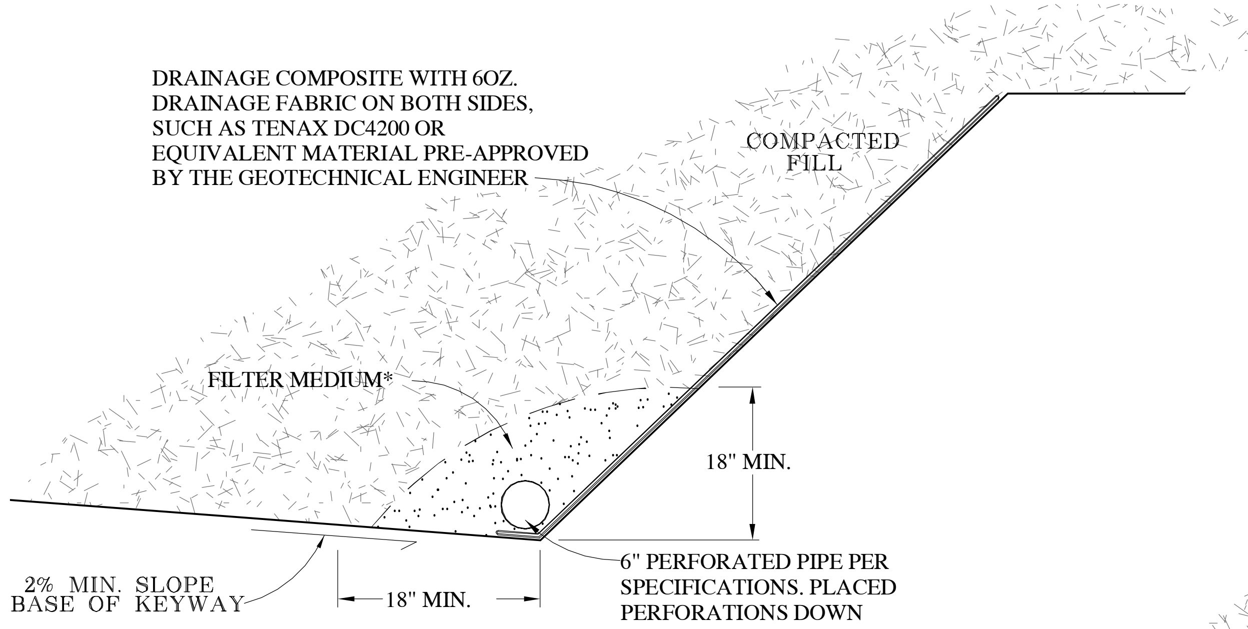
ENGEO
Expect Excellence

OVEREXCAVATION FOR CUT/FILL AND CUT LOTS
SPOTORNO PROPERTY
PLEASANTON, CALIFORNIA

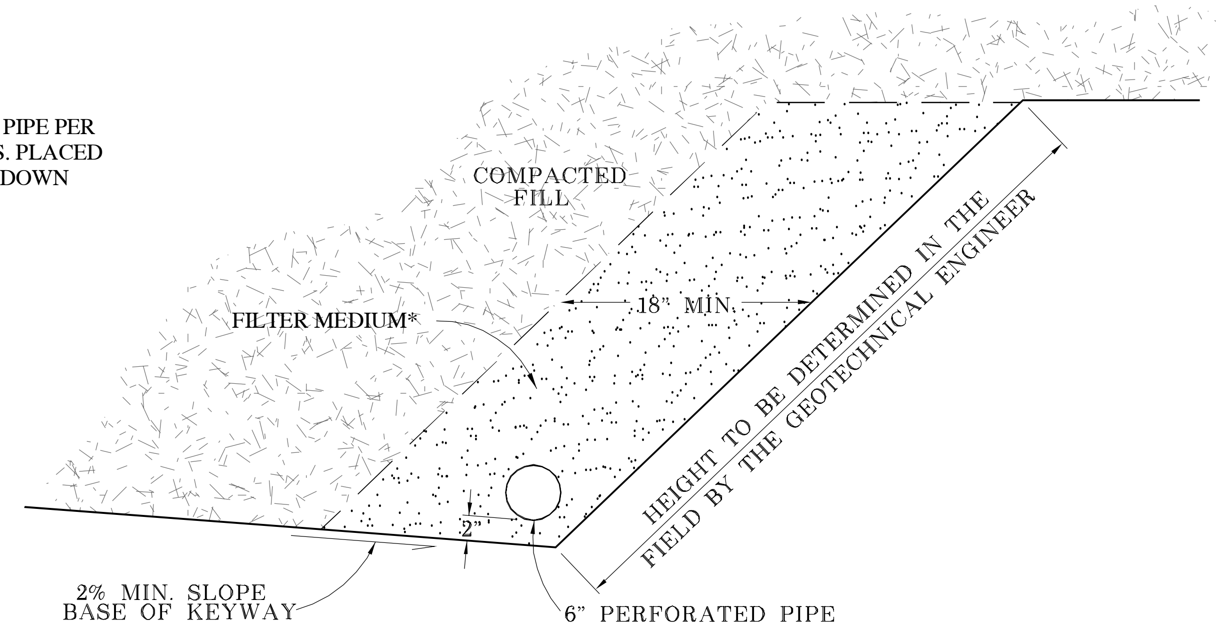
PROJECT NO.: 6103.200.000	
SCALE: AS SHOWN	
DRAWN BY: SRP	CHECKED BY: JF

FIGURE NO. 11

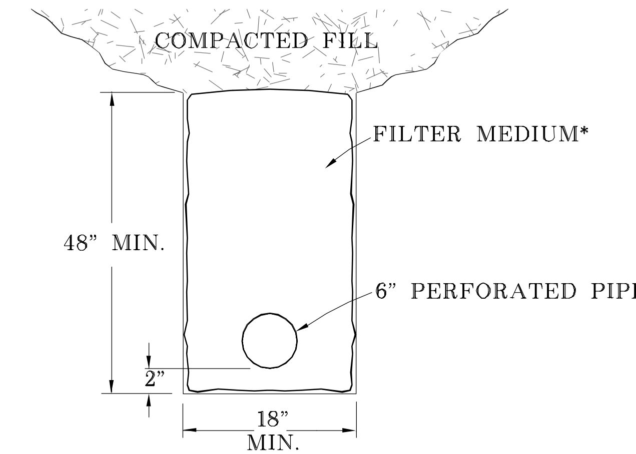
G:\Drafting\DRAFTING2_Dwg\6103\200\6103200000-11-OverexLots-0115-B.dwg Plot Date: 1-07-15 spatters



KEYWAY SUBDRAIN - OPTION 1



KEYWAY SUBDRAIN - OPTION 2



SWALE SUBDRAIN

*FILTER MEDIUM

ALTERNATIVE A

CLASS 2 PERMEABLE MATERIAL

MATERIAL SHALL CONSIST OF CLEAN, COARSE SAND AND GRAVEL OR CRUSHED STONE, CONFORMING TO THE FOLLOWING GRADING REQUIREMENTS:

SIEVE SIZE	% PASSING SIEVE
1"	100
3/4"	90–100
3/8"	40–100
#4	25–40
#8	18–33
#30	5–15
#50	0–7
#200	0–3

ALTERNATIVE B

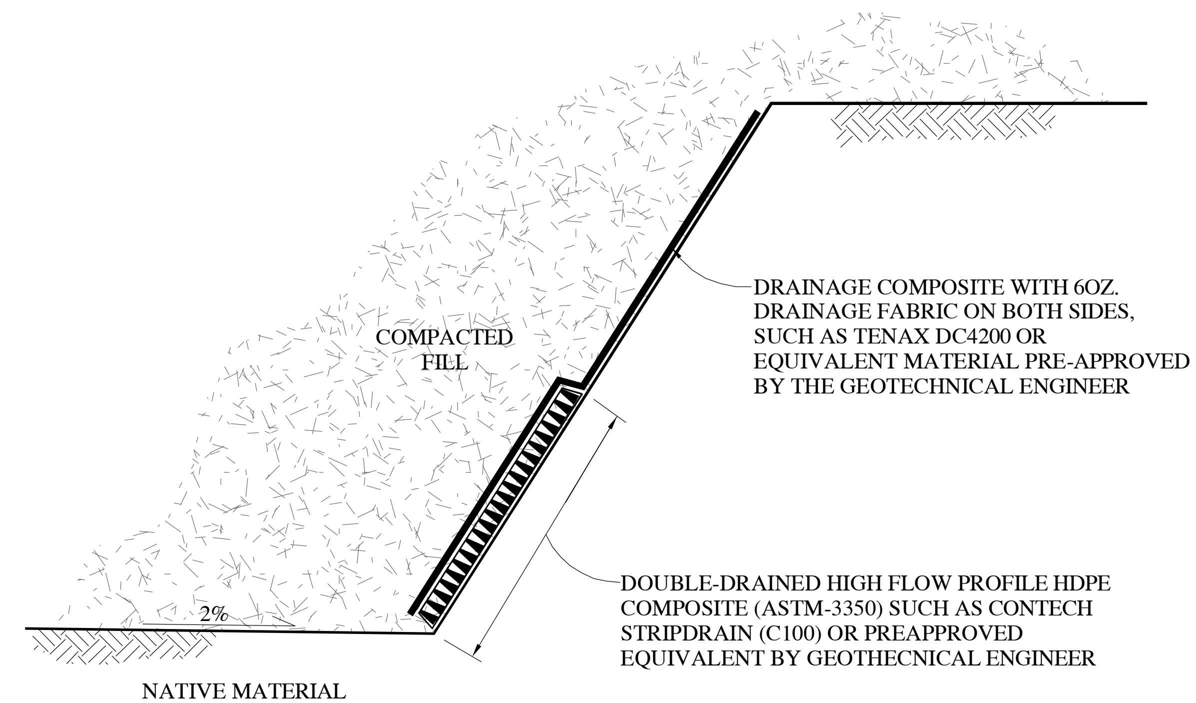
CLEAN CRUSHED ROCK OR GRAVEL WRAPPED IN FILTER FABRIC

ALL FILTER FABRIC SHALL MEET THE FOLLOWING MINIMUM AVERAGE ROLL VALUES UNLESS OTHERWISE SPECIFIED BY ENGEO:

GRAB STRENGTH (ASTM D-4632)	180 lbs
MASS PER UNIT AREA (ASTM D-4751)	6 oz/yd^2
APPARENT OPENING SIZE (ASTM D-4751)	70-100 U.S. STD. SIEVE
FLOW RATE (ASTM D-4491)	80 gal/min/ft
PUNCTURE STRENGTH (ASTM D-4833)	80 lbs

NOTES:

1. *ALL PIPE JOINTS SHALL BE GLUED*
2. *ALL PERFORATED PIPE PLACED PERFORATIONS DOWN*
3. *1% FALL (MINIMUM) ON ALL TRENCHES AND DRAIN LINES*



ALTERNATE KEYWAY SUBDRAIN - OPTION 3

(FOR DEPTHS LESS THAN 30 FEET)

ENGEO
Expect Excellence

TYPICAL SUBDRAIN DETAILS
SPOTORNO PROPERTY
PLEASANTON, CALIFORNIA

PROJECT NO.: 6103.200.000		FIGURE NO.
SCALE: AS SHOWN		12
DRAWN BY: SRP	CHECKED BY: JF	