Route 238 Bypass – Group 6

Hayward, California

PRELIMINARY GEOTECHNICAL EXPLORATION

Submitted to:
Mr. Kevin Briggs
City of Hayward
777 B Street, 2nd Floor
Hayward, CA 94541

Prepared by:
ENGEO Incorporated

January 9, 2017

Project No.
12843.000.000

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Mr. Kevin Briggs
City of Hayward
777 B Street, 2nd Floor
Hayward, CA 94541

Subject: Route 238 Bypass – Route 6
Hayward, California

PRELIMINARY GEOTECHNICAL EXPLORATION

Dear Mr. Briggs:

As requested, we completed this preliminary geotechnical exploration for Group 6 parcels in Hayward, California as outlined in our agreement dated August 24, 2016. The accompanying report presents our field exploration with our preliminary conclusions and recommendations for planning purposes of the proposed improvements at the parcel.

Our findings indicate that the study area is geotechnically feasible for the proposed development provided the recommendations and guidelines provided in this report are implemented during project planning. We are pleased to have been of service to you on this project and are prepared to consult further with you and your design team as the project progresses.

Sincerely,

ENGEO Incorporated

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1.0 INTRODUCTION

1.1 PURPOSE AND SCOPE

We prepared this preliminary geotechnical report in support of your evaluation of the subject parcels in Hayward, California. This report was prepared as outlined in our proposal dated August 24, 2016. The City of Hayward authorized ENGEO to conduct the proposed scope of services, which included the following:

- Review of available literature, previous reports, historic aerial images, and published geologic maps covering the study area.
- Geologic mapping of the site, including the cut slope in the southwest portion of the site.
- Subsurface exploration, consisting of 37 test pits.
- Geotechnical data analyses.
- Report preparation summarizing our preliminary conclusions and recommendations for planning purposes of the proposed park.

We performed a previous geologic reconnaissance at the site as referenced in our report titled "Geotechnical Feasibility Report for Route 238 Bypass – Group 6, Hayward, California" dated March 31, 2016.

We prepared this report exclusively for the City of Hayward (City) and its design team consultants. ENGEO should review any changes made in the character, design or layout of the development to modify the conclusions and recommendations contained in this report, as necessary.

1.2 SITE LOCATION AND PROJECT DESCRIPTION

The project site comprises approximately 30 acres of land currently utilized as open space that was a former quarry operation. The project site is located north of Carlos Bee Boulevard, south of Highland Boulevard and approximately 1,500 feet northeast of Route 238 (Mission Boulevard) and approximately 2,000 feet northwest of California State University, East Bay (Figure 1).

The current site topography can be characterized as a generally flat former quarry with elevations from approximately 290 feet in the northwest of the site to 300 feet in the southeast of the property. On the edges of the site there are moderate to steep cut faces remaining from the prior quarrying operations. The southeastern wall is the most significant of these, with a height of approximately 100 feet and a slope inclination of between 1:1 and 1.5:1 (horizontal:vertical). Along the northern boundary of the property is a steep sided drainage, which lies approximately 80 feet below the quarry floor. The drainage side slopes generally have inclinations ranging between 1.25:1 to 1.5:1. The property is generally undeveloped, though there is abandoned quarry infrastructure on-site, including debris piles and a few concrete slab foundations in the northwestern portion.

Based on the information provided, we understand that the City of Hayward is interested in acquiring the Group 6 parcel area from the State of California Department of Transportation (CalTrans) with plans of selling it for future development. No plans have been developed at this time showing potential grading and development concepts. We have assumed that cut and fill for design grading of the site will most likely be necessary, as well as potential landslide mitigation and repair, and non-engineered fill excavation and reprocessing.

1.2.1 Site Background and Aerial Photograph Review

As part of our scope, we reviewed relevant information regarding geotechnical and geological aspects of the site. These included aerial photographs from various years starting in 1946, available historic topographic maps from various years starting in 1899, and available published geologic maps and reports (see references for details).

Review of these materials indicated that quarrying at the site began sometime between 1957 and 1958 and ended by 1975. The topography at the site was altered significantly by quarrying, removing an estimated 160 feet of material from the previous hilltop down to the current grades. The previous site topography consisted of a rounded hilltop that dropped steeply toward the incised northern drainage. The Cross Sections presented in Figure 4 show the previous site topographic profiles. In the 1953 stereo photographs, a series of lineaments are observed crossing the hillslope and possibly altering the course of a small drainage. Other than quarrying operations, the site has generally remained undeveloped. Various support structures used in the quarrying operations were demolished by the 1980s.

Prior to the quarrying operations at the site, an adjacent parcel to the west of the site was also used as a quarry. Those quarrying operations created very steep grades, approximately 1:1, along a portion of the western edge of the site. Areas to the west and south of the site, including the quarry area, were developed for residential housing beginning around 1960. The apartment structures to east of the site were constructed in the late 1960s/early 1970s.

1.3 PREVIOUS GEOTECHNICAL STUDY

1.3.1 Geotechnical Feasibility Report, ENGEO, March 2016

In March, 2016, we performed a geotechnical feasibility investigation for residential development at the site. Our previous study included a review of geologic literature and maps, geologic reconnaissance of the site, review of aerial photographs, and preparation of a report. No subsurface investigation was undertaken for the preparation of the preliminary report. The previous study concluded that proposed residential development of the property is feasible provided that the project is appropriately designed for the geologic and geotechnical hazards identified in the report.

2.0 GEOLOGY AND SEISMICITY

2.1 GEOLOGIC SETTING

The site is located within the East Bay Hills just south of Castro Valley, and just east of the East Bay Plain. The East Bay Hills lie within the region of coastal California referred to by geologists as the Coast Ranges geomorphic province. The Coast Ranges have experienced a complex geological history characterized by Late Tertiary folding and faulting that has resulted in a series of northwest-trending mountain ranges and intervening valleys. The site is located on the western edge of an uplifted range of hills locally referred to as the East Bay Hills block (Buising and Walker, 1995; Graymer, 2000), bounded on the west by the active Hayward Fault and on the east by the active Calaveras Fault.

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.

2.1.1 Site Geology

According to published maps covering the site by Graymer et al. (1996), Graymer (2000) and Dibblee (1980), the site is primarily underlain by late Jurassic gabbro associated with the Coast Range Ophiolite Complex (Figure 2). The active, 90 km-long, approximately North-35-Degrees-West-trending active Hayward Fault is mapped approximately 1,500 to 2,000 feet southwest and west of the site (Herd, 1978). The Hayward Fault separates the Bay structural block from the East Bay Hills structural block (Buising and Walker, 1995; Wakabayashi, 1999; Graymer, 2000; Ponce et al., 2003).

Graymer et al. (1996) and Graymer (2000) map the majority of the bedrock on site as gabbro. We used the map from Graymer (2000) for this report. According to Ponce et al. (2003), the gabbro is part of an incomplete ophiolite sequence. The Coast Range Ophiolite Complex is an igneous complex that formed in an ancient sea-floor spreading tectonic environment, then later was scraped off the ocean floor during a long history of subduction from the late Jurassic to early Cretaceous. Ponce et al. (2003) refer to the gabbro on site as the San Leandro Gabbro, which, as a structural package, dips 75 to 80 degrees northeast in its current position along the Hayward Fault zone in this area. The present western boundary of the San Leandro Gabbro in the Hayward area marks the base of a pre-existing, shallow-angle roof-thrust which formed during subduction zone processes, now rotated near-vertically following multiple episodes of extension and attenuation, providing a preferred path of least resistance for the present tectonics now accommodated along the Hayward fault zone. Active seismicity occurs along the western contact of the San Leandro Gabbro along the Hayward fault because earthquakes concentrate near the edge of this massive igneous block rather than its interior due to the natural distribution of stress. The subduction to transform-fault tectonic episodic history is cause for the presently observed foliation, jointing and shear patterns reflected and mapped in the gabbro on site.

2.1.2 Geologic Mapping

During our exploration, a geologist from our firm performed geologic mapping at the site. Below are descriptions of the geologic units encountered during our exploration of the site (Figure 3).

2.1.2.1 Existing Fill (Qaf)

Fill soil identified onsite include material generated during quarrying as well as re-worked colluvium used for grading. Across the site, existing fill (Qaf) from quarrying activities is described as moist, loose to medium dense, clayey sand with angular gravel-sized gabbro fragments, and few cobble-sized gabbro fragments. Rock debris in the fill reflects the bedrock at the site, which ranges from highly weathered, weak gabbro to slightly weathered and strong gabbro. This variable weathering is due to differential tectonic mechanical breakdown, and differential chemical weathering. There are also non-engineered fill deposits that include concrete, asphalt and similar building/construction-related debris.

We encountered sandy, decomposed gabbro fill with building debris around the edges and across the floor of the old quarry. Much of the fill is placed in piles and does not extend much deeper than the base of the quarry floor or approximately 4 to 6 feet. Sandy, decomposed gabbro derived non-engineered fill devoid of building materials occurs along the north-facing slope of the gabbro cliff, along the southern border of the site. Fill thickness along this cliff face is approximately 2 to 5 feet.

The quarry floor is generally flat-lying and free from deep pits of non-engineered fill. However, at the north end of the quarry in the vicinity of TP-33, we encountered fill up to approximately 15 feet thick. This fill consisted of a dark reddish brown, stiff, sandy lean clay with gravel. The fill in the northern area of the quarry floor was thicker, and more clay rich and included much more vegetation than the fill we observed in the rest of the quarry floor. It is likely that a trench was previously excavated and backfilled at this area of the site.

Along the western and southern slopes bordering the quarry pit, we observed fill that consisted of dark brown, soft, sandy fat clay with scattered gravel. This fill is located adjacent to the original colluvium-mantled hillslope and was likely derived from re-working the colluvium for use in road-building and creating flat pads. The fat clay fill is as thick as approximately 4 to 6 feet.

2.1.2.2 Landslide Debris (Qls)

Previous landslide mapping by Nilsen (1975) and the California Geological Survey (CGS; 2003) combined show roughly three areas of the site which may be prone to sliding or should be evaluated for slope stability. These areas include:

- Old quarry bedrock slope/cliff located along the southern portion of the site. This prominent old quarry face trends roughly east-east and faces to the north.
- The steep-sided slope face associated with previous quarrying activities located along the western boundary of the site and
- The north/northeast steep-sided stream gully slope which bounds the site in this area.

Based on our subsurface exploration and detailed field mapping, we identified and evaluated these three areas of the site per CGS (2003). The following discussion describes these areas in more detail.

A quarry-related, abandoned cut into exclusively gabbro bedrock, is located along the southern portion of the site. The CGS (2003) shows this approximately east-west trending rock slope as an area of concern. We have mapped and documented this slope in terms of recording rock discontinuities (fracturing, jointing, foliation and shears) followed by data analysis. These data are presented in Appendix C.

A steep-sided slope exhibiting earthflow/slide mass accumulation is located in the western portion of the site (Figure 3). This earthflow/slide mass comprises silty clayey sand and we estimate it is approximately 15 to 20 feet thick. This area is also shown on the map by Nilsen (1975) as well as CGS (2003; Figure 3). The earthflow does not appear to exhibit signs of recent activity, such as cracking or displacement near the head scarp or sloughing of surficial soils. This steep slope is located immediately up slope of residential homes located at the end of Redstone Place, and in our opinion poses a slope stability hazard to the existing homes.

The northern-to northeastern portion of the site is bounded by a steeply-incised natural creek. This site-bounding slope, albeit steep and nearly 1:1, only appears to show signs of soil creep and hummocky topography indicative of soil creep. This area lacks the geomorphology often associated with active mass-wasting, land sliding or debris flows.

2.1.2.3 Quaternary Alluvial (Qao2/Qoa and Qal) and Colluviual Deposits (Qc)

Early to middle Pleistocene alluvium is mapped by Dibblee (1980), Graymer et al. (1996), Graymer (2000) and CGS (2003) along a ridge top located immediately north of the steeply-incised stream valley, which marks the eastern to northern boundary of the site (Figure 3). Since the majority of the site is essentially bedrock exposed at the surface with non-engineered fill areas we have mapped, the majority of Quaternary Alluvium is restricted to the active stream channel along the north-northeastern boundary of the project.

We observed colluvial deposits on the outward-facing slopes surrounding the quarry pit. On the west-facing slopes on the western edge of the site as well as in test pits on the slopes of the southern edge of the site, colluvium consisted of dark brown, stiff to hard, sandy fat clay up to approximately 6 feet. We mapped a thin deposit of colluvium or slope wash on the north-facing slope on the north-northeastern boundary of the site.

2.1.2.4 Jurassic Gabbro (Jgb)

According to published maps covering the site by Graymer et al. (1996), Graymer (2000), and Dibblee (1980), the project site is underlain by Jurassic gabbro (Jgb). Gabbro is the intrusive-plutonic equivalent to its eruptive counterpart, basalt, found immediately south of the project site, scattered across the Hayward fault zone.

Based on our mapping, bedrock at the site consists primarily of gabbro displaying a variety of discontinuity types. Fresh, moderately to very strong gabbro displays a greenish color in outcrop, owing to a weak heating pattern encountered during tectonic events and interaction with sea water. Weathered and weakly- to moderately strong gabbro on site exhibits a greenish-brown color, due intense weathering and oxidization. The rock on site contains a variety of discontinuities which are described below.

We mapped a foliation pattern or structural fabric on site displaying a consistent northwest strike and steep easterly dip, in general, similar to the overall structural block dip observed by Ponce et al. (2003). This foliation pattern/structural fabric in the gabbro, observed on site and locally in the Hayward Hills, is tectonic in origin and a secondary feature. It is reflective of deformation overprinted on the primary gabbro structure attributed to tectonic activity in the spreading center and subduction zone, as opposed to a primary magmatic fabric of mineral alignment upon crystallization. An existing bedrock cut-slope within the gabbro, located just south of the site and immediately adjacent to the student housing and dorms on the southern side of CSUEB campus, displays a similar foliation pattern/structural fabric related to late-brittle or brittle-ductile, tectonic-derived heating. This results in a slightly metamorphic hornblende and subgrain rotation of plagioclase, and possible pyroxene, which align to exhibit the mapped foliation pattern. Carbonate-filled shears, from less than 1- up to 6-inches wide, within the gabbro bedrock units generally strike west–northwest to east-northeast and dip 30 to 60 degrees towards the south.

2.2 FAULTING AND SEISMICITY

The site is located adjacent to the Hayward fault zone; however, the property lies outside of the designated State of California Earthquake Fault Zone (Figure 5). The Hayward fault is considered an active fault with a general right-lateral sense of movement. The Hayward fault is a creeping fault that displays evidence (off-set curbs, en-echelon cracks within pavements, etc.) of fault creep along various segments.

In October 1868, a magnitude 6.8 earthquake occurred along the southern segment of the Hayward fault. As the earthquake occurred before the existence of seismographs, the epicenter is unidentified. Most of what is known about the earthquake is based on eyewitness accounts. Surface rupture was reported from Fremont to Berkeley, and damage was reported as far north as Santa Rosa and as far south as Gilroy and Santa Cruz (Brocher, 2008).

An active fault is defined by the California Geological Survey as one that has had surface displacement within Holocene time (about the last 11,000 years) (SP42 CGS, 2007). Because of the presence of nearby active faults, the Bay Area Region is considered seismically active. Numerous small earthquakes occur every year in the region, and large (greater than Moment Magnitude 7) earthquakes have been recorded and can be expected to occur in the future. Figure 7 shows the approximate location of active and potentially active faults and significant historic earthquake epicenters mapped within the San Francisco Bay Region. Based on the 2014 update of the national seismic hazards maps, the table below shows the nearest known active faults capable of producing significant ground shaking at the site.

TABLE 2.2-1: Known Active Faults Capable of Producing Significant Ground Shaking at the Site

Fault Name	Distance from Site (miles)	Maximum Moment Magnitude (Elsworth)
Hayward-Rogers Creek	0.1	7.3
Calaveras	7.6	7.0
Mount Diablo Thrust	12.2	6.7
Concord-Green Valley	17.5	6.5
San Andreas	18.5	7.9
Monte Vista-Shannon	18.6	6.5
Greenville	18.9	7.0
San Gregorio	25.8	7.5

The Uniform California Earthquake Rupture Forecast (UCERF3, 2014) 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 72 percent for the Bay Area as a whole, a probability of 18 percent for the

Hayward fault, 7.4 percent for the Calaveras fault, 6.4 for the Northern San Andreas fault, and 3.5 percent for the Concord-Green Valley fault.

3.0 FIELD EXPLORATION

The sections below summarize our field exploration activities as well as ground surface, subsurface, and groundwater conditions.

3.1 FIELD LOGGING

We conducted the field exploration for this study on October 26, October 31, and November 2, 2016. The field exploration consisted of excavating 37 test pits to a maximum depth of 15 feet below existing grade at the approximate locations shown on Figure 3. The test pits were performed using a track-mounted excavator. We established exploration locations by handheld GPS and visual sighting from existing features and should be considered accurately located only to the degree implied by the method used.

A geologist from our firm logged the test pits in the field. The field logs were then used to develop the report test pit logs, which are presented in Appendix A. The boring and test pit logs depict subsurface conditions at the time the exploration was conducted. Subsurface conditions at other locations may differ from conditions occurring at these locations, and the passage of time may result in altered subsurface conditions. In addition, stratification lines represent the approximate boundaries between soil types, and the transitions may be gradual.

3.2 SUBSURFACE CONDITIONS

The results of our subsurface exploration indicate that the project site is generally underlain by shallow bedrock. As described previously, the gabbro bedrock varies in strength due to degree of weathering. The test pits indicate that the majority of site topography and slopes reflect the bedrock surface due to relatively thin overlying surficial materials. Generally, a thin mantle of colluvium and fill surrounds the site and relatively low piles of fill cover the quarry floor.

The deepest depths to bedrock were encountered in Test Pits TP-31 and TP-33 at approximately 13½ feet below the ground surface and 15½ feet below the ground surface, respectively. TP-31 was located on the western edge of the site likely where a fill pad was created above native colluvium. We observed the thickest surficial material consisting of fill and colluvium at the western portion of the site and may be susceptible to slope movements. TP-33 was located on the floor of the quarry in the northern portion of the site. The relatively deep depth to bedrock suggests that there may have been some kind of cut at this location that was infilled at some later date.

Consult Figure 3 and the boring and test pit logs for specific subsurface conditions at each location. We include our exploration logs in Appendix A. The logs describe the soil type, color, consistency, and visual classification in general accordance with the Unified Soil Classification System (USCS). The logs graphically depict the subsurface conditions encountered at the time of the exploration.

3.3 GROUNDWATER

Groundwater was not encountered during our subsurface exploration. However, monitoring wells (data obtained from various public reports via GeoTracker, 2011-2015) around the site appear to indicate that local groundwater may occur between 27 to 48 feet bgs (below ground surface). CGS' mapping is generally consistent with this data but adds that the Hayward Fault acts as a groundwater barrier in this region. On the western side of the fault the groundwater can be much deeper (greater than 50 feet where San Lorenzo Creek crosses the fault). On the eastern side of the fault, groundwater tends to be 10 to 30 feet bgs in the flatland areas (CGS, 2003). Fluctuations in groundwater levels occur seasonally and over a period of years because of variations in precipitation, temperature, irrigation, and other factors.

4.0 DISCUSSION AND CONCLUSIONS

Based on the exploration results, from a geotechnical standpoint the site is feasible for potential development. We studied the site with respect to known geologic and other hazards common to the greater San Francisco Bay Region. The main geologic/geotechnical issues to be addressed at the site include the following. The recommendations in subsequent sections consider the hazards and concerns listed below.

- Strong Ground Shaking
- Slope stability
- Steep slopes
- Existing fill
- Resistant bedrock

4.1 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, soil liquefaction, lateral spreading, and densification. Based on topographic and lithologic data, risk from earthquake-induced regional subsidence/uplift and tsunamis and seiches is considered negligible at the site.

The following sections present a discussion of these hazards as they apply to the site.

4.1.1 Ground Rupture

As previously discussed, the site is not located within a State of California Earthquake Fault Zone. Based on our field mapping, review of aerial photographs and the results of our field exploration, it is our opinion that fault-related ground rupture is unlikely at the subject property.

4.1.2 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 current California Building Code (CBC) requirements, as a minimum. Seismic design provisions of current building codes generally prescribe minimum lateral forces, applied statically to the structure, combined with the gravity forces of dead-and-live loads. The code-prescribed lateral forces are generally considered to be substantially smaller than the comparable forces that would be associated with a major earthquake. Therefore, structures should be able to: (1) resist minor earthquakes without damage, (2) resist moderate earthquakes without structural damage but with some nonstructural damage, and (3) resist major earthquakes without collapse but with some structural as well as nonstructural damage. Conformance to the current building code recommendations does not constitute any kind of guarantee that significant structural damage would not occur in the event of a maximum magnitude earthquake; however, it is reasonable to expect that a well-designed and well-constructed structure will not collapse or cause loss of life in a major earthquake (SEAOC, 1996).

4.1.3 Ground Lurching

Ground lurching is a result of the rolling motion imparted to the ground surface during energy released by an earthquake. Such rolling motion can cause ground cracks to form in weaker soils. The potential for the formation of these cracks is considered greater at contacts between deep alluvium and bedrock. Such an occurrence is possible at the site as in other locations in the Bay Area, but based on the site location, it is our opinion that the offset is expected to be minor.

4.1.4 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 fine-grained sands. Empirical evidence indicates that loose to medium dense gravels, silty sands, low-plasticity silts, and some low-plasticity clays are also potentially liquefiable. The State of California Seismic Hazard Zones Map does not show areas susceptible to liquefaction within the property (Figure 6). Additionally, the majority of the site is underlain by bedrock at shallow depths. While the drainage on the northern boundary of the site potentially contains liquefiable materials in the stream channel, any proposed improvements at the site will likely be set back from and not be impacted by liquefiable materials.

4.1.5 Lateral Spreading

Lateral spreading involves lateral ground movements caused by seismic shaking. These lateral ground movements are often associated with a weakening or failure of an embankment or soil mass overlying a layer of liquefied or weak soils. Due to the presence of shallow bedrock at the majority of the site, the potential for lateral spreading is considered to be low.

4.1.6 Earthquake-Induced Landsliding

As stated previously, several landslides were mapped and observed at the site. The State of California Seismic Hazard Zones Map shows portions of the site as areas susceptible to earthquake-induced landslides (Figure 6). However, if the recommendations of this report are incorporated into planning and construction, the potential for impacts to future proposed improvements at the site from earthquake-induced landsliding is low.

4.2 SLOPE STABILITY

As mentioned previously, there are a number of very steep slopes around the perimeter of the quarry pit created by the quarrying operations both at the site and adjacent to the site. These steep quarry slopes include the southeastern high wall, the former quarry slope adjacent to the site on the western edge, and a ring of 1:1 inward-facing slopes on the north and western edges that are up to 30 feet high in the northwest corner. In general, these quarry slopes are steeper than building code minimums and lack features like benches or debris catchments as prescribed for general construction. In the following sections, we address the slope stability of the large southeastern high wall as well as provide recommendations for grading the existing slopes so that they conform to accepted standards.

As discussed in the Geologic Mapping section, there is a likely landslide located at the western edge of the site. This landslide mass is located precariously uphill from the over-steepened quarry slope on the western edge of the site with a residential community located in close proximity downslope. From inspection, this slope should be graded to mitigate the risk to downslope residences. We recommend in Section 5 below that this slide mass be removed and the upper portion of the slope be re-graded to a shallower inclination.

In addition to the over-steepened quarry slopes, along the northern edge of the site are very steep slopes that drop down to a deeply incised creek. In the next section, we address their slope stability and in Section 5 provide recommendations for slope setbacks.

4.2.1 Methods of Analysis

For preliminary planning purposes we performed two-dimensional limit-equilibrium slope stability analyses of critical slopes with the computer slope stability software Slide Version 6.0 using Spencer’s method (Spencer, 1967).

We selected two critical slopes, shown on Cross Sections 4-4’ and 5-5’, for preliminary slope stability analyses that focus on the global stability of the bedrock neglecting the shallow surficial soil. Cross Section 4-4’ is cut through a typical slope along the northern drainage. Cross Section 5-5’ represents a typical section of the southeastern high wall. Figure 3 shows the location of both cross sections and the profiles of the cross section are included on Figure 4. For Cross Section 4-4’, we assumed a conservative groundwater table at roughly 18 feet below the quarry floor that follows the ground surface topography. On Cross Section 5-5’, we assumed a conservative groundwater table that generally follows the topography and is as shallow as 10 feet below the toe of the slope.

4.2.2 Estimation of Shear Strength

The majority of the site is comprised of bedrock consisting of weathered gabbro. In order to estimate the shear strength of the gabbro, we used the Generalized Hoek-Brown shear-normal function. The existing fill and colluvium are generally shallow and we modeled their strengths with typical shear strengths correlated with their field classification.

TABLE 4.2.2-1: Summary Generalized Hoek-Brown Shear Strength Parameters

Material	Bedrock Strength Parameters			
	UCS (psf)	mb	s	a
Jurassic Gabbro (Jgb)	1,500,000	0.260	0.000197	0.516

4.2.3 Seismic Slope Deformation Analyses

The Note 48 Checklist for the Review of Engineering Geology and Seismology Reports for California Public Schools, Hospitals, and Essential Services Buildings (October 2013) by the California Geological Survey, advises the procedure recommended in SP117A in addition to using a design-level ground motion based on geometric mean and without risk coefficient (i.e. $PGA_M/1.5$). We determined PGA_M using the 2016 CBC. We then divided the PGA_M by 1.5 to yield a design-level PGA of 0.64g. SP117A states that slopes that have a pseudo-static factor of safety greater than 1.0 using a seismic coefficient derived from the screening analysis procedure of Stewart and others (2003) can be considered stable. We used a pseudo-static coefficient equal to 0.4PGA (0.27g) based on 15 cm threshold of displacement as recommended by Stewart and others (2003). The developers of this approach (as well as developers of similar approaches) consider the results of these analysis to be indices of expected seismic performance and not predictions of actual slope displacements.

4.2.4 Results of Slope Stability Analyses

Appendix B shows the results of our static and pseudo static stability analyses for Cross Sections 4-4' and 5-5'. The results are summarized in Table 4.2.3-1. The analyses indicate factors of safety above commonly accepted criteria for global stability failures within the Gabbro.

TABLE 4.2.4-1: Static Stability

SECTION	MIN STATIC FS	MIN PSUEDO-STATIC FS
4-4'	2.5	1.7
5-5'	2.9	2.0

4.2.5 Results of Stereonet Analyses

In addition to the global stability analyses of Sections 4-4' and 5-5' discussed above, we performed stereonet analyses addressing the northwest-facing quarry slope (Section 5-5'). The stereonet analysis was performed based on mappable discontinuities exposed in the slope. These results are included in Appendix C.

Based on our mapping and stereonet analysis, the discontinuities affecting the gabbro at the site include several joint sets, an overall structural fabric, here referred to as foliation, and several mapped shears observed in the quarry face (Figures 3 and 4). Based on our review of the stereonet data, the discontinuities are unlikely to form plane failures in the quarry slope. For screening purposes, we used a base friction angle of 30 degrees in our analysis. Our screening analysis did indicate a slight potential for wedge failures formed by the intersecting northwest dipping and northeast dipping joint sets J2 and J4 respectively. In addition, there is a slight potential for wedge failures formed by the intersection of several of the localized shears and joint sets J2 and J4. Observations in the field however did not appear to indicate the presence of significant wedge failures formed by the intersection of the joint sets or the shears within the quarry slope face. This is likely due to a shear strength much greater than the base friction angle along the discontinuities due to the rough undulations and asperities present along the discontinuities. The overall quarry face does not appear steep enough to be affected by toppling failures and no evidence of toppling failures were observed during our field mapping. The various intersecting discontinuities observed in the slope are closely spaced and appear to form relatively small blocks that can become dislodged and bounce, roll or slide down the steep slope. Recommendations to help mitigate the potential risks of dislodged rock masses are provided in Section 5.

4.3 OVER STEEPENED SLOPES

Due to the quarrying operations both at the site and adjacent to the site, there are a number of very steep slopes around the perimeter of the quarry pit. As explained above, these slopes are currently generally stable and have performed adequately since the time the quarry was closed. However, they are steeper than would be designed for general construction. We provide recommendations to reduce the inclination of the slopes in Section 5.0.

4.4 EXPANSIVE SOIL

We observed potentially expansive fat clay in various locations at the surface and in several test pits. Expansive soils change in volume with changes in moisture. They can shrink or swell and cause heaving and cracking of slabs-on-grade, pavements, and structures founded on shallow foundations. Building damage due to volume changes associated with expansive soils can be reduced by: (1) using a rigid mat foundation that is designed to resist the settlement and heave of expansive soil, (2) deepening the foundations to below the zone of moisture fluctuation, i.e., by using deep footings or drilled piers, and/or (3) using mat or footings at normal shallow depths but bottomed on a layer of select fill having a low expansion potential.

Successful performance of structures on expansive soils requires special attention during construction. It is imperative that exposed soils be kept moist prior to placement of concrete for foundation construction. It is extremely difficult to remoisturize clayey soils without excavation, moisture conditioning, and recompaction.

4.5 EXISTING FILL AND COMPRESSIBLE SOIL

Existing fill could undergo vertical movement that is not easily characterized and could ultimately be inadequate to effectively support potential building and fill loads. In general, existing fills should be overexcavated and replaced as engineered fill. Recommendations for existing fill removal are presented in Section 5.3.

The fill derived from quarrying that are located along the perimeter and across of the floor of the quarry pit are generally not extensive in thickness or lateral expanse. However, the site is large and may potentially generate a considerable amount of off-haul if the material is not re-used onsite. Construction debris also exists across the quarry pit and within these fills. Fill soils will need to be removed as part of site preparation for grading, however, the fill may be re-used onsite if it is adequately processed. An onsite processing area may be used to separate construction debris like wood, metal, and asphalt that cannot be placed in engineered fill and should be removed from the site.

The clayey fill and colluvium present on the western and southern slopes of the site will also likely require over excavation and replacement as engineered fill. This soil is potentially expansive and the slopes it is present on are oversteepened. As a part of our preliminary earthwork recommendation below, we recommend that these materials be removed and the slopes be laid back to shallower inclinations.

4.6 SHALLOW GROUNDWATER

Groundwater was not encountered at the time of our exploration. Due to the shallow bedrock onsite, shallow groundwater is not anticipated. However, zones of seepage may be encountered and site drainage may be difficult due to the low infiltration rates of bedrock. Additional consideration for adequate surficial drainage during grading should be considered.

4.7 CALIFORNIA BUILDING CODE SEISMIC PARAMETERS

The site has varying soil and bedrock conditions, which can be generally classified as Site Class D in accordance with the 2016 California Building Code (CBC). The following seismic design parameters can be used for preliminary design.

TABLE 4.7-1: 2016 CBC Seismic Information

Parameter	Design Value
Site Class	B
0.2 second Spectral Response Acceleration, S_S	2.47
1.0 second Spectral Response Acceleration, S_1	1.03
Site Coefficient, F_A	1.0
Site Coefficient, F_V	1.0
Maximum considered earthquake spectral response accelerations for short periods, S_{MS}	2.47
Maximum considered earthquake spectral response accelerations for 1-second periods, S_{M1}	1.03
Design spectral response acceleration at short periods, S_{DS}	1.65
Design spectral response acceleration at 1-second periods, S_{D1}	0.68
Mapped MCE Geometric Mean (MCE_G) Peak Ground Acceleration, PGA (g)	0.96
Site Coefficient, F_{PGA}	1.00
MCE_G Peak Ground Acceleration adjusted for Site Class effects, PGA_M (g)	0.96
Long period transition-period, T_L	8 sec

4.8 EXCAVATABILITY

Based on our field exploration, it is our opinion that some of the more resistant bedrock onsite may be marginally rippable and require heavy to very heavy equipment, such as a Caterpillar D-10 or larger. The most resistant rock is generally encountered in the flat quarry areas, which likely will not require extensive grading. However, excavating utilities or deep foundations may be very challenging in the resistant bedrock. As a part of the design report, the bedrock should be characterized for rippability through laboratory testing, geophysical survey and/or consultation with a grading contractor.

The resistant rock may also generate oversized material (greater than six inches in diameter). Onsite processing of oversized materials may be necessary to avoid difficulties during grading or having to off-haul the material.

We provide this information for general planning purposes only. This information is not intended for bidding purposes.

5.0 PRELIMINARY EARTHWORK RECOMMENDATIONS

The preliminary recommendations included in this report, along with other sound engineering practices, should be incorporated in the planning of the project.

5.1 SELECTION OF MATERIALS

With the exception of some construction debris (wood, brick, metal, etc.), trees, organically contaminated materials (soil which contains more than 3 percent organic content by weight), and environmentally impacted soils, we anticipate the site soils and bedrock derived materials are suitable for use as engineered fill. Other materials and debris, including trees with their root balls, should be removed from the project site.

Oversized soil or rock materials (those exceeding two-thirds of the lift thickness or 6 inches in dimension, whichever is less) should be removed from the fill and broken down to meet this requirement or otherwise off-hauled.

5.2 DEMOLITION AND STRIPPING

Site preparation should commence with removal of site vegetation, structures, and any surface and subsurface improvements, such as foundations or concrete pads. Following the demolition of existing improvements, site development should include removal of debris, loose soil, and soft compressible materials in any location to be graded. Any soft compressible soils should be removed from areas to receive fill or structures, or those areas to serve as borrow. Vegetation and debris should be separately stockpiled from soft compressible material and existing soil fill.

5.3 EXISTING FILLS AND COLLUVIUM

Existing fill and compressible soil are unsuitable to remain below any potential improvements and should be over-excavated to expose underlying competent native soils that are approved by the Geotechnical Engineer. The base of the excavations should be processed, moisture conditioned, as needed, and compacted in accordance with the subsequent recommendations for engineered fill.

5.4 LANDSLIDE REMOVAL

As discussed above, there are potential landslides present on the western edge of the site above an over-steepened slope. Landslide material should be removed to competent bedrock as observed by a Certified Engineering Geologist. It has been our experience that the method of the removal i.e. top down vs. bottom up, to be considered a contractor means and method. The sequencing is typically decided based on available cut, available stockpile space, risk and consequence of a triggered failure, etc.

5.5 QUARRY HIGH WALL

As discussed in previous sections, there is large quarry high wall on the southeastern potion of the site that was found to be globally stable, but is susceptible to surface raveling that could generate rock falls. We recommend creating a minimum 30-foot-wide bench at the base of this slope and providing for a debris catchment berm and/or rock debris fencing. Intermediate drainage terraces in accordance with CBC requirements should be constructed on the slope. Additionally, a rock fall analysis should be performed in the design phase to address hazards from rock falls.

5.6 PERIMETER QUARRY SLOPES

In addition to the quarry high wall, there are a number of remnant steep quarry slopes along the western and northern edges of the quarry floor. They are generally 1:1 (horizontal:vertical) and are up to 30 feet high from the quarry floor. We recommend that these slopes be graded following the recommendations below.

5.7 SLOPES

5.7.1 Graded Slopes

The stability of cut slopes in bedrock materials is largely dependent on the planned cut location and the orientation of the cut slope with respect to the bedrock structure or other planes of geologic weakness. An Engineering Geologist from our firm should examine all cut slope exposures in the field. If adverse bedrock structure or other zones of geologic weakness are encountered in the cut slopes during grading, overexcavation and rebuilding may be necessary.

With the exception of the existing quarry high face previously discussed graded slopes should be designed in conformance with the following recommendations:

TABLE 5.7.1-1: Fill Slope Gradient

Slope Material	Slope Height (feet)	Maximum Recommended Slope Gradient (Horizontal:Vertical)
Bedrock-derived fill	0 - 30	2:1
Bedrock-derived fill	Greater than 30	3:1
General fill	0 – 10	2:1
General fill	Greater than 10	3:1

TABLE 5.7.1-2: Cut Slope Gradients

Slope Material	Slope Height (feet)	Maximum Recommended Slope Gradient (Horizontal:Vertical)
Gabbro	0 - 30	2:1
Gabbro	Greater than 50	3:1

Where steeper slopes than those recommended above are necessary, special slope construction measures will be required. These measures could include construction of benches in bedrock slopes and using geogrid reinforcement in fill slopes.

5.7.2 Slope Setbacks

The recommended slope setbacks for habitable structures are variable depending on slope height and soil conditions. Slope setbacks are intended to reduce the risk of adverse impacts from potential slope movement under static or seismic loading conditions. Slope setbacks should be in accordance to current CBC standards.

We recommend that where a building pad is adjacent to a downhill slope, all permanent structures should be set back from the toe the equivalent distance of one-third the vertical slope height. The maximum required setback is not to exceed 40 feet from top of the slope.

For building pads adjacent to other uphill slopes, all permanent structures should be set back from the toe the equivalent distance of one-half the vertical slope height. The maximum required setback is 30 feet from the toe of slope.

5.7.3 Debris Benches and Drainage Terraces

We recommend that debris benches located between major graded slopes and residential lots should be a minimum of 30 feet wide. Debris benches located between major graded slopes and roadways should be a minimum of 8 feet wide. The main purpose of the debris benches is to catch sediment or debris that could be generated from upslope areas and to provide access to the slope to perform slope maintenance, if needed.

Drainage terraces should be provided on slopes at no greater than 30-feet intervals to provide drainage and site access for maintenance.

5.7.4 Toe Keyways

Construction of subdrained toe keyways will be required at the toes of any fill slopes or for landslide removals to mitigate potential slope stability hazards. We anticipate that typical keyway designs will consist of minimum 18- to 24-foot-wide keyways constructed to a minimum depth of at least 3 feet into competent bedrock. Subsurface drainage systems should be installed within the keyways as recommended in a subsequent section. A typical keyway detail is presented on Figure 8.

Actual subsurface mitigation configurations (size and depths) should be provided following a design level exploration. Fills should be adequately keyed/benched into competent material or bedrock materials, as determined by the Geotechnical Engineer during fill slope construction.

5.7.5 Subsurface Drainage Facilities

We recommend subsurface drainage systems for keyways, and at the base of removal areas, as a minimum. Secondary bench subdrains may also be required, depending upon the height of fill slopes and the slope of the underlying native terrain. In addition, observed seepage areas or suspected spring areas should be controlled in development areas through the use of subdrains. Positive fall of at least ½ (selectively) to 1 percent towards an approved outlet should also be provided for all subdrains.

General details of recommended subdrains are presented on Figure 9. Subdrain systems should consist of a minimum 6-inch-diameter perforated pipe encased in Caltrans Class 2 permeable material, or crushed rock wrapped in filter fabric. Discharge from the subdrains will generally be low but in some instances may be continuous. Subdrains should outlet into the storm drain system or other approved outlets, and their locations should be surveyed and documented by the project Civil Engineer for future maintenance.

Not all sources of seepage are evident during the time of field work because of the intermittent nature of some of these conditions and their dependence on long-term climatic conditions. Furthermore, new sources of seepage may be created by a combination of changed topography, manmade irrigation patterns and potential utility leakage. Since uncontrolled water movements are one of the major causes of detrimental soil movements, it is of utmost importance that a Geotechnical Engineer be advised of any seepage conditions so that remedial action may be initiated, if necessary.

5.8 PLACEMENT OF FILL

The following compaction recommendations can be used for planning purposes:

TABLE 5.8-1: Compaction and Moisture Content Requirements

Description	Materials	Minimum Relative Compaction (%)	Minimum Moisture Content (Percentage Points Above Optimum)
From 0 to 50 feet below finished grade and utilities	Expansive	90	4
	Non-expansive	95	2
Keyways and Greater than 50 feet below finished grade	Expansive	95	3
	Non-expansive	95	2

Relative compaction refers to in-place dry density of the fill material expressed as a percentage of the maximum dry density as determined by ASTM D-1557. Optimum moisture is the moisture content corresponding to the maximum dry density. We recommend that the fills be compacted at higher than optimum moisture contents as shown above to minimize the effects of swell and/or hydrocompression.

5.9 CUT, FILL, AND CUT-FILL TRANSITION LOTS

We recommend that the upper three feet of subgrade soil be processed by subexcavating and replacing as engineered fill. This condition will be achieved as a result of remedial grading operations. This requirement will provide a relatively uniformly moisture conditioned state for the foundation subgrade soils. Moisture and compaction recommendations are provided in a subsequent section of this report.

5.10 DIFFERENTIAL FILL THICKNESS

For subexcavation activities that create a differential fill thickness across individual building pads, mitigation to achieve a similar fill thickness across the pad is beneficial for the performance of a shallow foundation system. We recommend that a differential fill thickness of up to 10 feet is acceptable across individual building pads. For a differential fill thickness exceeding 10 feet across an individual pad, we recommend performing subexcavation activities to bring this vertical distance to within the 10-foot tolerance and that the material is replaced as engineered fill. As a minimum, the subexcavation area should include the entire structure footprint plus 5 feet beyond the edges of the building footprint.

5.11 SURFACE DRAINAGE REQUIREMENTS

Improper drainage may result in detrimental soil movements. It is very important that surface grades provided for rapid removal of surface water. Ponding of water near the top of or uncontrolled flow across graded slopes can be detrimental to long-term performance of the slope. Mid-slope drainage should be in accordance with current CBC requirements. Surface water should be directed to outlet into the storm drain system or other approved outlets. We recommend if post construction stormwater treatment facilities are planned, they not be located near the top of the slope unless lined with an impermeable geomembrane.

6.0 LIMITATIONS AND UNIFORMITY OF CONDITIONS

This report is issued with the understanding that it is the responsibility of the owner to transmit the information and recommendations of this report to developers, owners, buyers, architects, engineers, and designers for the project so that the necessary steps can be taken by the contractors and subcontractors to carry out such recommendations in the field. The conclusions and recommendations contained in this report are solely professional opinions.

The professional staff of ENGEO Incorporated strives to perform its services in a proper and professional manner with reasonable care and competence but is not infallible. There are risks of earth movement and property damages inherent in land development. We are unable to eliminate all risks or provide insurance; therefore, we are unable to guarantee or warrant the results of our services.

This report is based upon field and other conditions discovered at the time of preparation of ENGEO's report. This document must not be subject to unauthorized reuse that is, reusing without written authorization of ENGEO. Such authorization is essential because it requires ENGEO to evaluate the document's applicability given new circumstances, not the least of which is passage of time. 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 on-study area 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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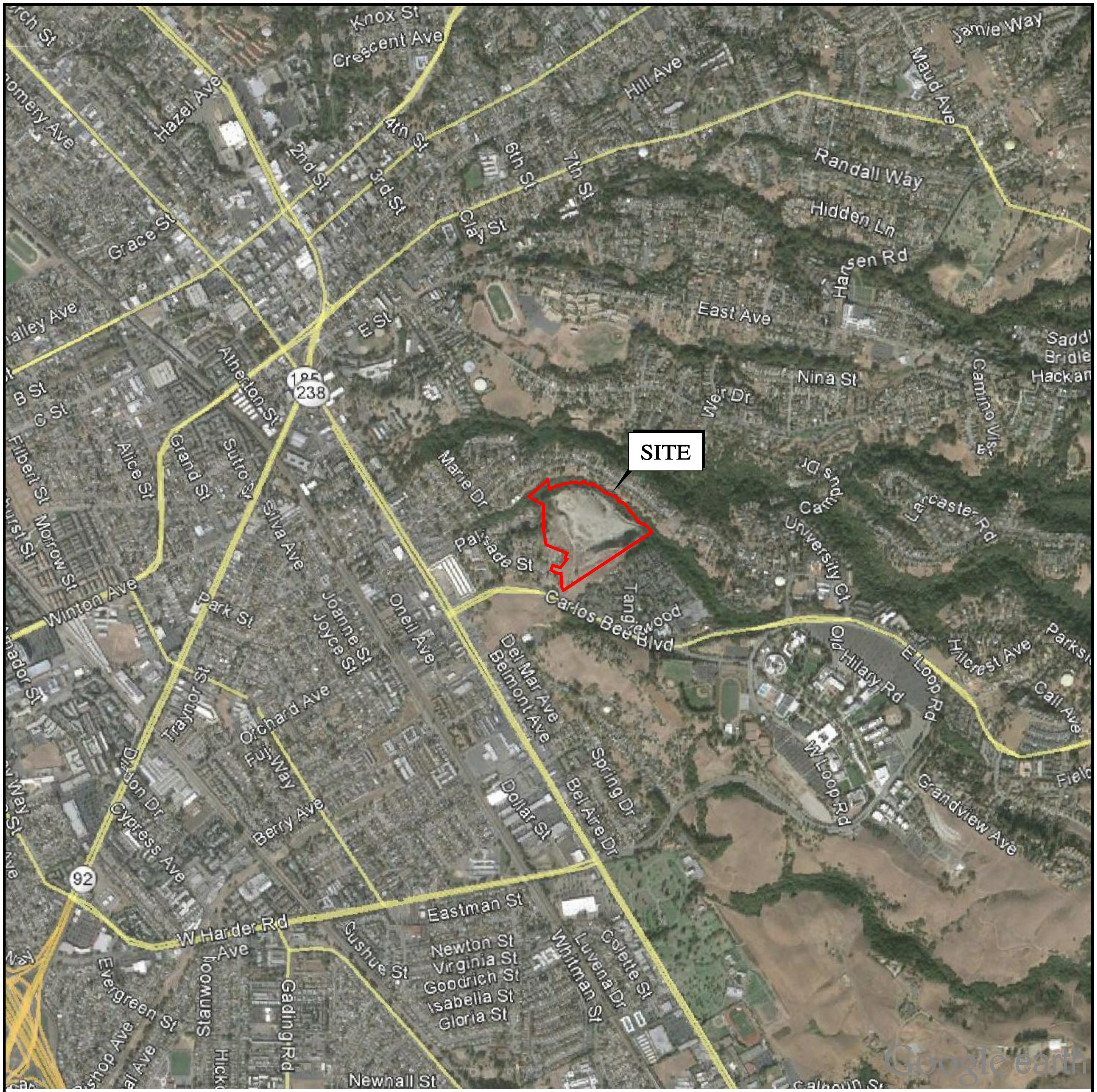
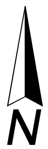
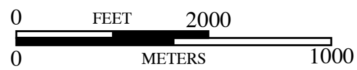
Pacific Aerial Surveys Photographs Reviewed

Date	Flight Number	Line	Photo(s)
4/21/1999	6100	19	47, 48, 49, 50, 51
7/7/1977	1377	7	51, 52
5/6/1975	1193	8	39, 40, 42, 43
5/2/1969	902	7	41, 42
5/2/1969	902	8	40, 41, 42
8/17/1953	119	21	25, 26, 27

LIST OF FIGURES







BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

ENGEO Expect Excellence	VICINITY MAP ROUTE 238 BYPASS - GROUP 6 HAYWARD, CALIFORNIA	PROJECT NO.: 12843.000.000 SCALE: AS SHOWN DRAWN BY: JCS CHECKED BY: RH	FIGURE NO. 1

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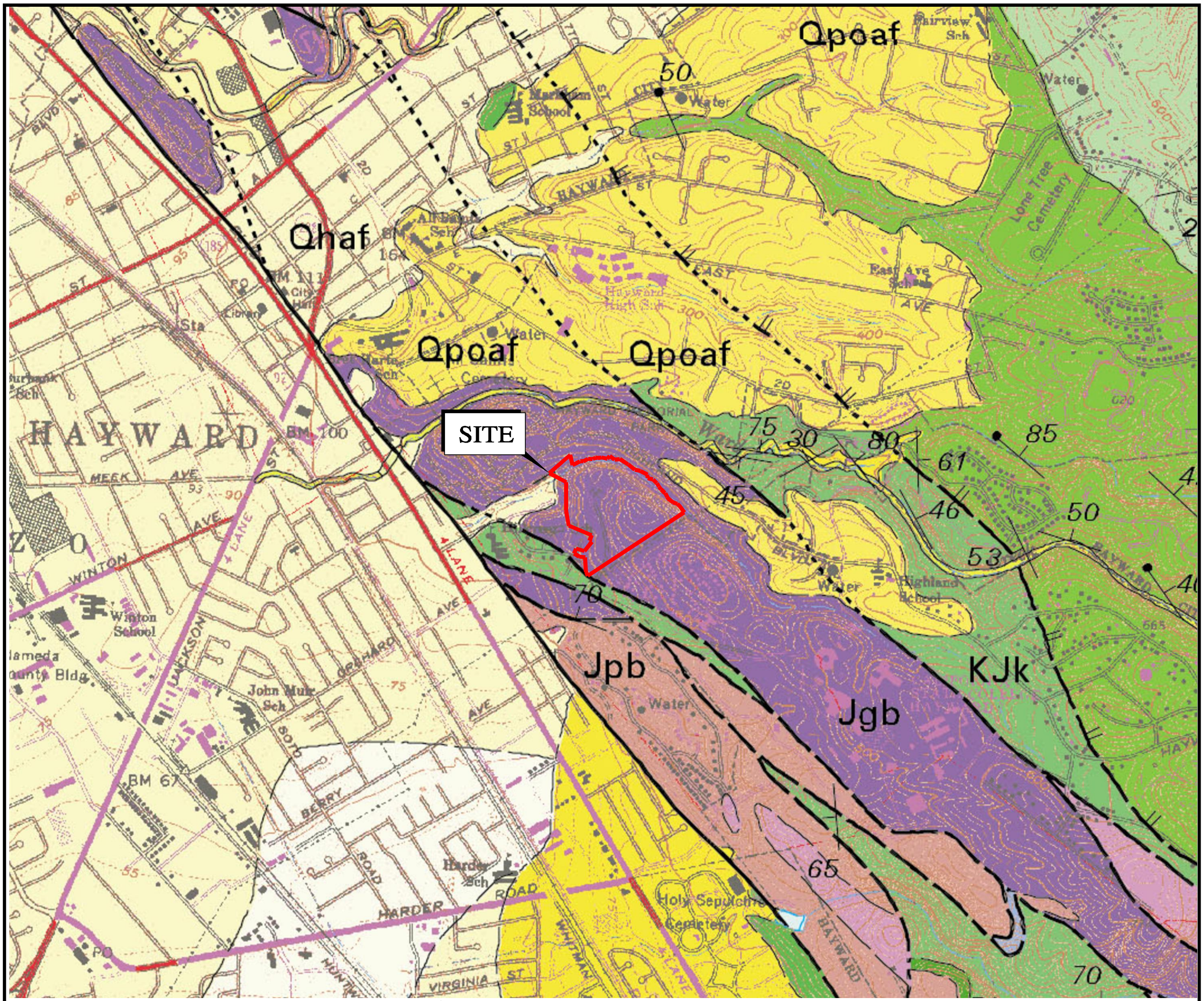


0 FEET 2000
0 METERS 1000

N

EXPLANATION

GEOLOGIC CONTACT - DEPOSITIONAL OR INTRUSIVE CONTACT, DASHED WHERE APPROXIMATELY LOCATED, DOTTED WHERE CONCEALED

FAULT - DASHED WHERE APPROXIMATELY LOCATED, SMALL DASHES WHERE INFERRED, DOTTED WHERE CONCEALED, QUERIED WHERE LOCATION IS UNCERTAIN.

NORMAL FAULT - DASHED WHERE APPROXIMATELY LOCATED, DOTTED WHERE CONCEALED

35 STRIKE AND DIP OF BEDDING

48 STRIKE AND DIP OF BEDDING, TOP INDICATOR OBSERVED

Qhaf ALLUVIAL FAN AND FLUVIAL DEPOSITS
Qpoaf OLDER ALLUVIAL FAN DEPOSITS
KJk KNOXVILLE FORMATION
Jpb PILLOW BASALT, BASALT BRECCIA, AND MINOR DIABASE
Jgb GABBRO

BASE MAP SOURCE: R.W. GRAYMER, 2000

ENGEO Expect Excellence	REGIONAL GEOLOGIC MAP ROUTE 238 BYPASS - GROUP 6 HAYWARD, CALIFORNIA	PROJECT NO.: 12843.000.000 SCALE: AS SHOWN DRAWN BY: JCS CHECKED BY: RH	FIGURE NO. 2

N

0 FEET 80
0 METERS 40

EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

Qc	COLLUVIUM
Qal	ALLUVIUM
Qaf	FILL
Qls/Qc	LANDSLIDE / COLLUVIUM
Jgb	GABBRO
	GEOLOGIC CONTACT
TP-37	TEST PIT (ENGEO, 2016)
87	BEDDING JOINT (ENGEO, 2016)
88	FOLIATION (ENGEO, 2016)
88	SHEAR PLANE (ENGEO, 2016)
5 5'	CROSS SECTION LOCATION

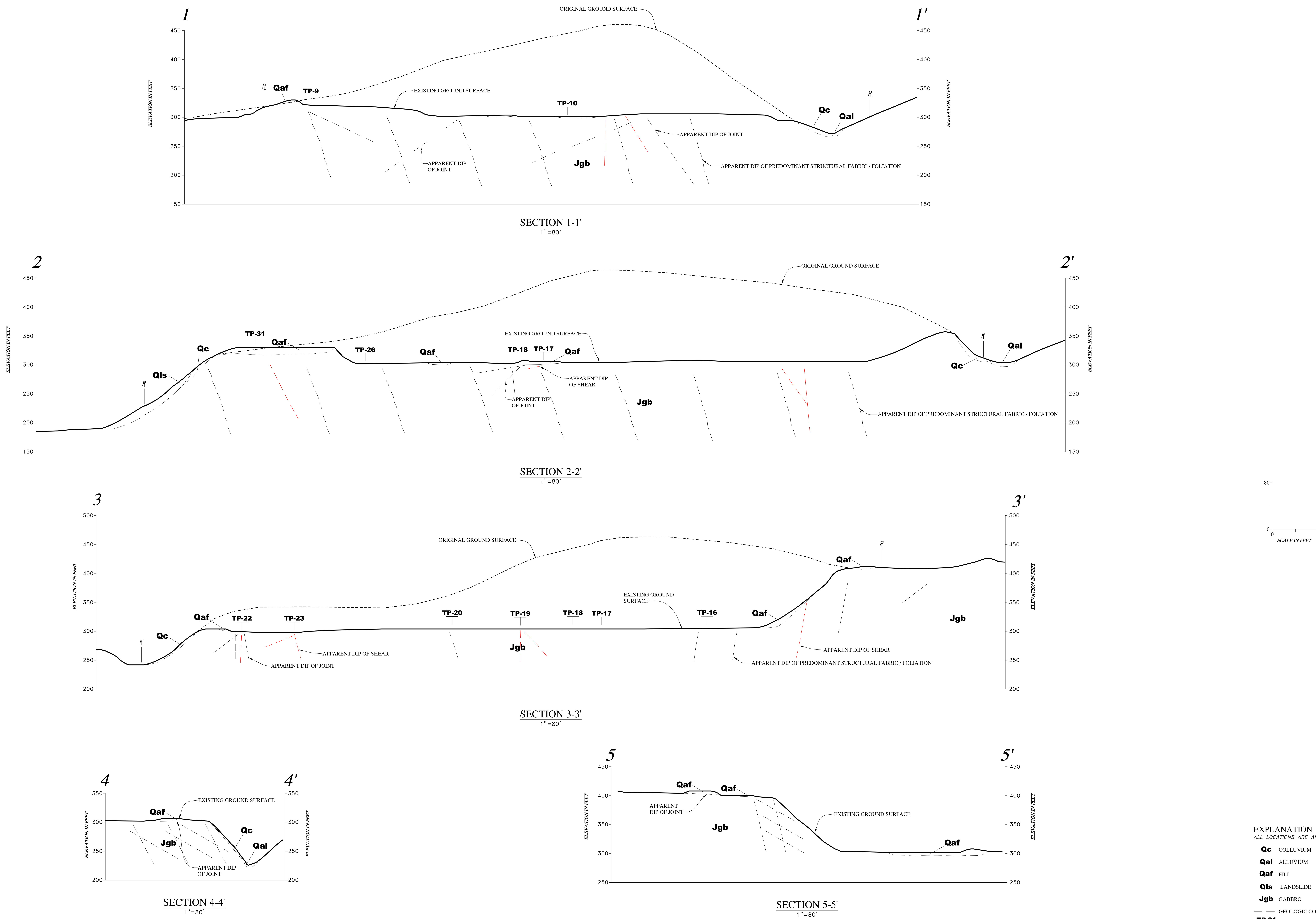
BASE MAP SOURCE: GOOGLE EARTH MAPPING SERVICE

ENGEO Expect Excellence

GEOLOGIC MAP
ROUTE 238 BYPASS - GROUP 6
HAYWARD, CALIFORNIA

PROJECT NO.: 12843.000.000
SCALE: AS SHOWN
DRAWN BY: JCS
CHECKED BY: RH
FIGURE NO. 3

ORIGINAL FIGURE PRINTED IN COLOR



EXPLANATION

ALL LOCATIONS ARE APPROXIMATE

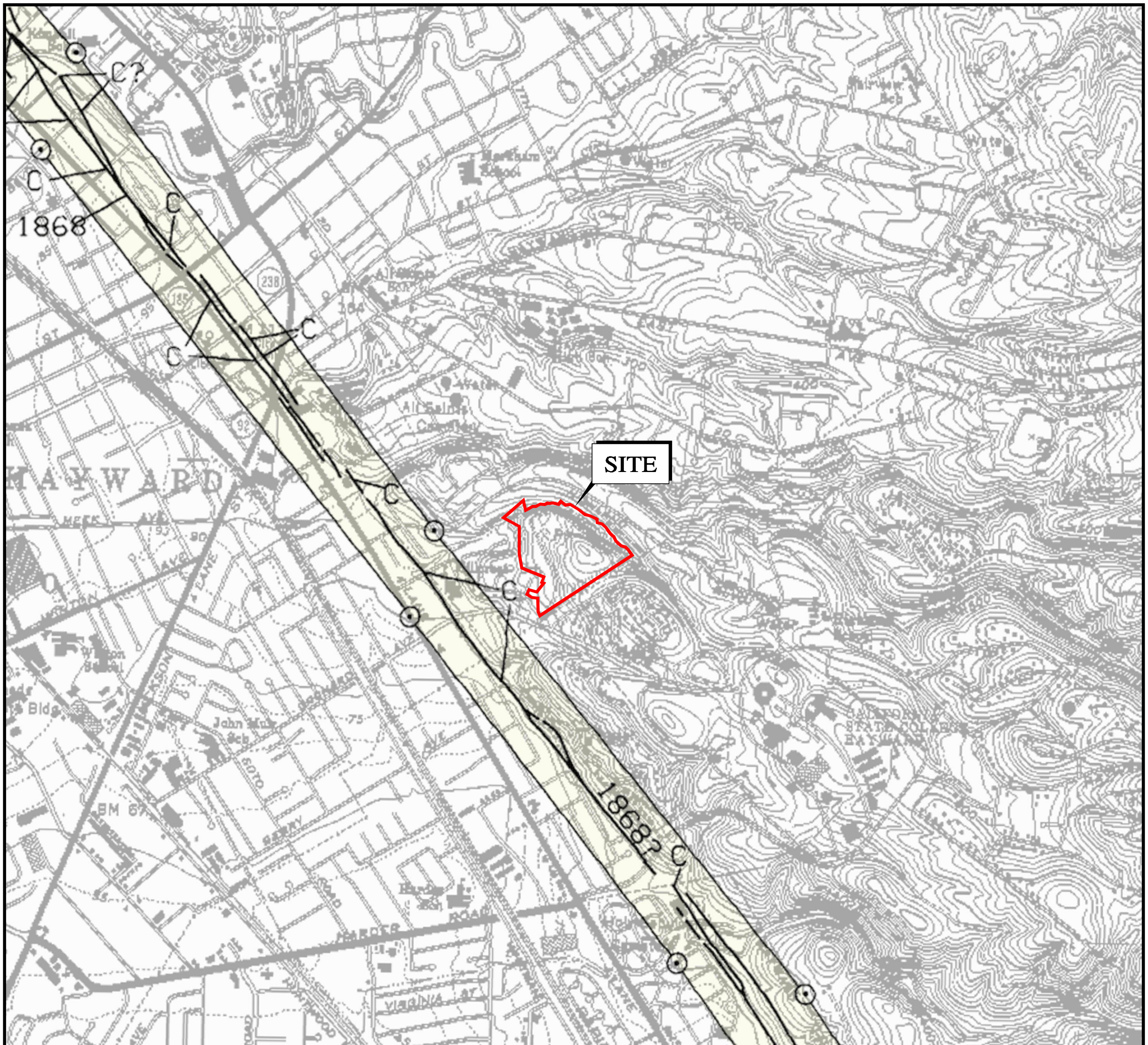
- **Qc** COLLUVIUM
- **Qal** ALLUVIUM
- **Qaf** FILL
- **Qls** LANDSLIDE
- **Jgb** GABBRO
- — — GEOLOGIC CONTACT
- **TP-31** TEST PIT

ENGEO Expect Excellence

GEOLOGIC CROSS SECTIONS
ROUTE 238 BYPASS - GROUP 6
HAYWARD, CALIFORNIA

PROJECT NO.: 12843.000.000
SCALE: AS SHOWN
DRAWN BY: JCS
CHECKED BY: RH
FIGURE NO. 4

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

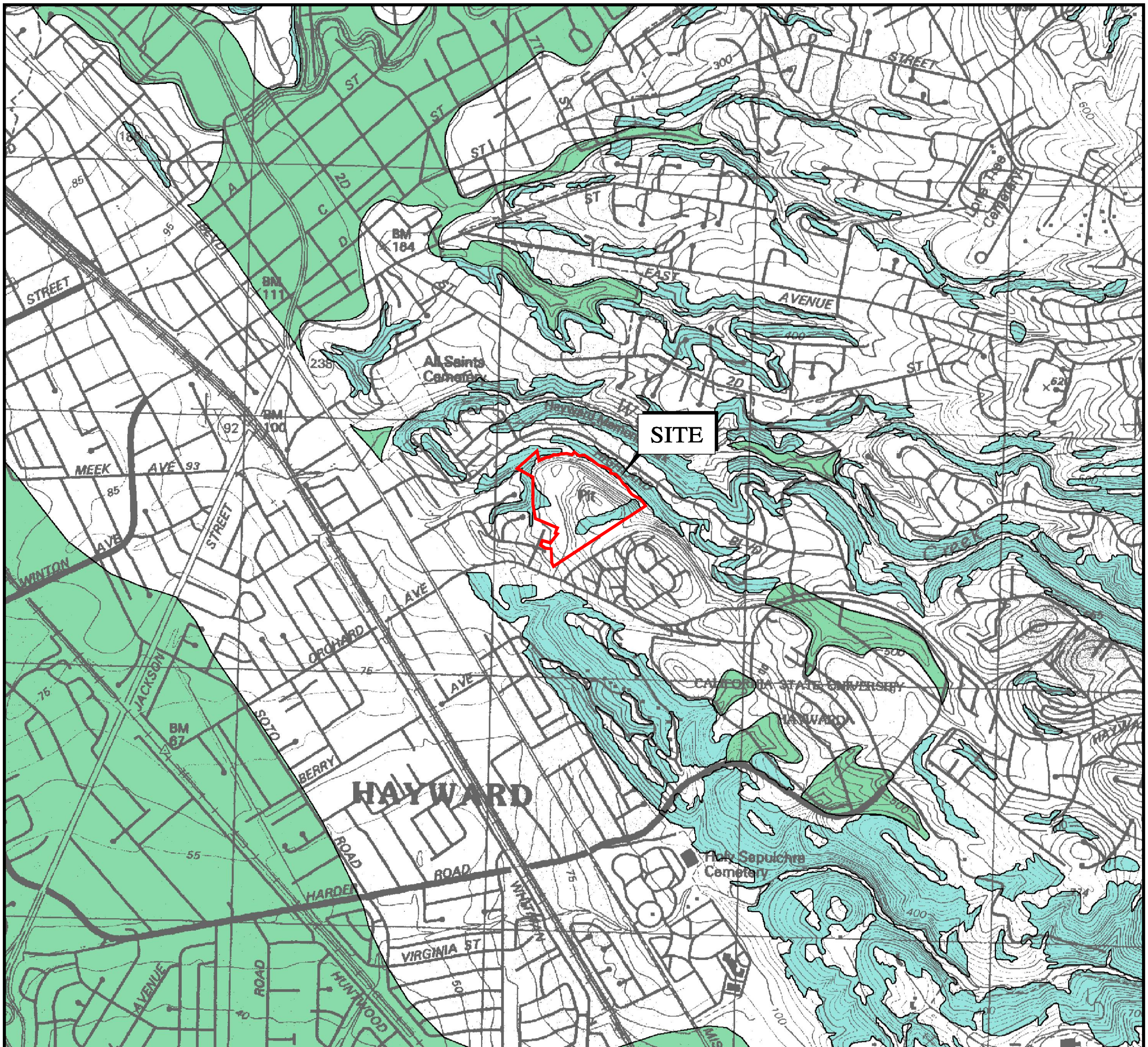
BASE MAP SOURCE: CDMG, 1993

ENGEO — Expect Excellence

EARTHQUAKE FAULT ZONE MAP ROUTE 238 BYPASS - GROUP 6 HAYWARD, CALIFORNIA	PROJECT NO.: 12843.000.000		FIGURE NO. 5
	SCALE: AS SHOWN		
	DRAWN BY: JCS	CHECKED BY: RH	

G:\Drafting\DRAFTING2_Dwg_10000 to 12999\12843\000\GEX\G6\12843000000-GEX-G6-5EQFaultZone-1216.dwg Plot Date: 12-20-16 JStephenson

ORIGINAL FIGURE PRINTED IN COLOR



N

0 FEET 2000

0 METERS 1000

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

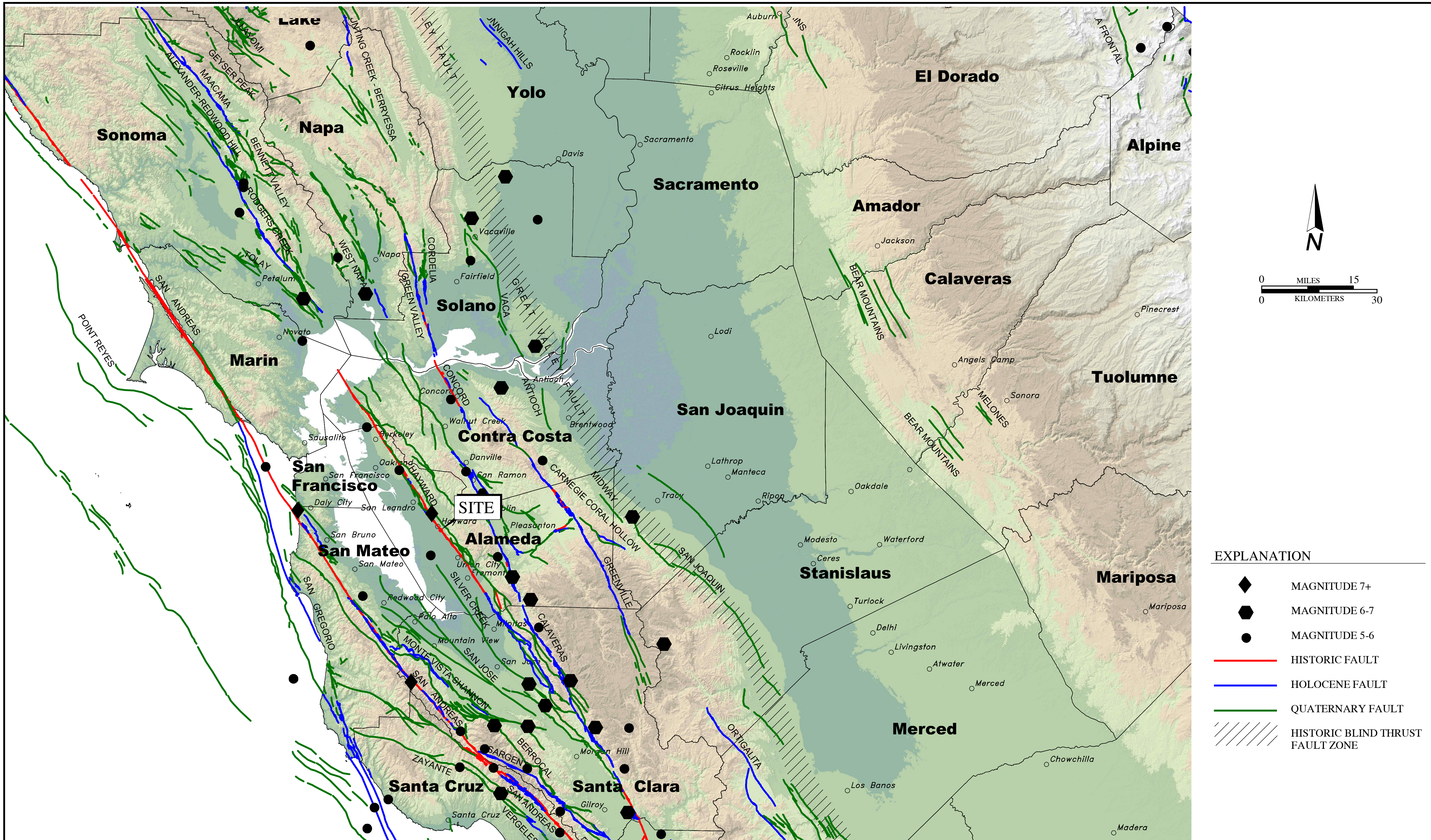
AREAS WHERE PREVIOUS OCCURRENCE OF LANDSLIDE MOVEMENT, OR LOCAL TOPOGRAPHIC, GEOLOGICAL, GEOTECHNICAL AND SUBSURFACE WATER CONDITIONS INDICATE A POTENTIAL FOR PERMANENT GROUND DISPLACEMENTS SUCH THAT MITIGATION AS DEFINED IN PUBLIC RESOURCES CODE SECTION 2693(c) WOULD BE REQUIRED

BASE MAP SOURCE: STATE OF CALIFORNIA SEISMIC HAZARD ZONES, HAYWARD QUADRANGLE, JULY 2003

ENGEO Expect Excellence	SEISMIC HAZARD ZONES MAP ROUTE 238 BYPASS - GROUP 6 HAYWARD, CALIFORNIA	PROJECT NO.: 12843.000.000 SCALE: AS SHOWN DRAWN BY: JCS CHECKED BY: RH	FIGURE NO. 6

G:\Drafting\DRAFTING2_Dwg_10000 to 12999\12843\000\GEX\G6\12843000000-GEX-G6-6SeismicMap-1216.dwg Plot Date: 12-15-16 JStephenson

ORIGINAL FIGURE PRINTED IN COLOR



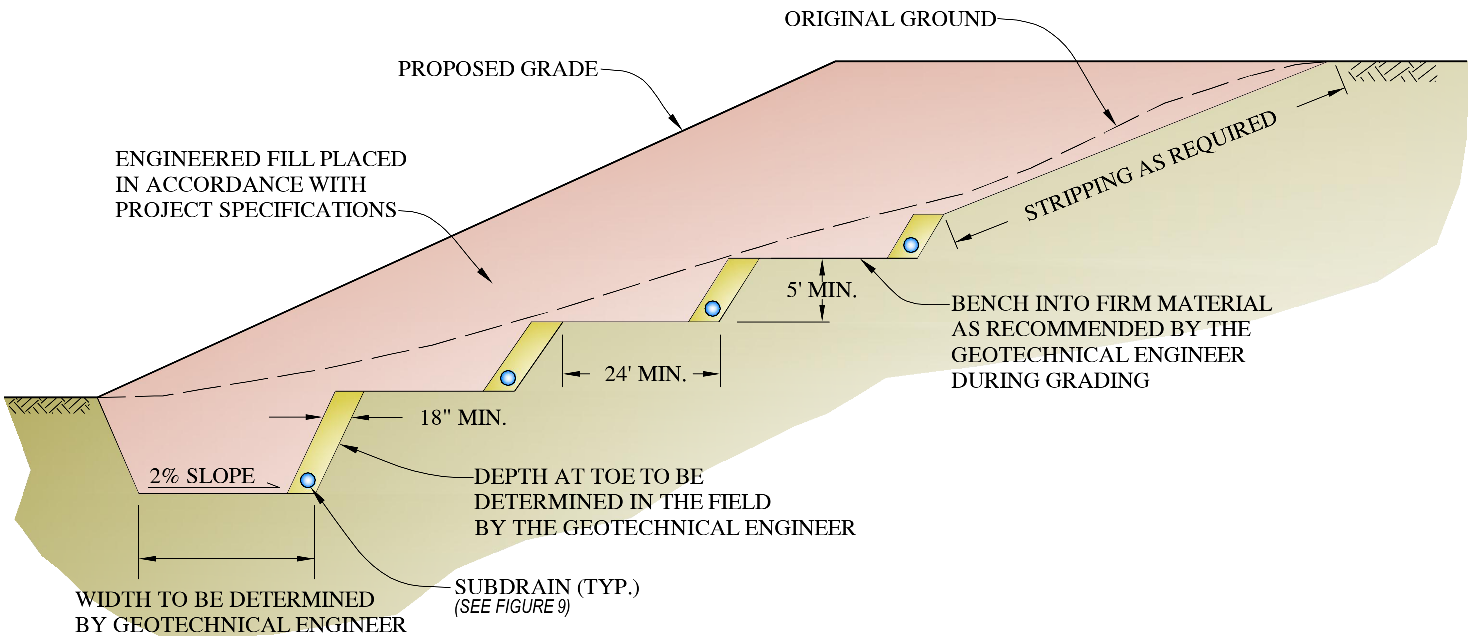
BASE MAP SOURCE:
COLOR HILLSHADE IMAGE BASED ON THE NATIONAL ELEVATION DATASET (NED) AT 30 METER RESOLUTION
U.S.G.S. QUATERNARY FAULT DATABASE, NOVEMBER, 2010
U.S.G.S. HISTORIC EARTHQUAKE DATABASE (1800–2000)

ENGEO
Expect Excellence

REGIONAL FAULTING AND SEISMICITY ROUTE 238 BYPASS - GROUP 6 HAYWARD, CALIFORNIA	PROJECT NO.: 12843.000.000	FIGURE NO
	SCALE: AS SHOWN	7
	DRAWN BY: JCS / CHECKED BY: RH	

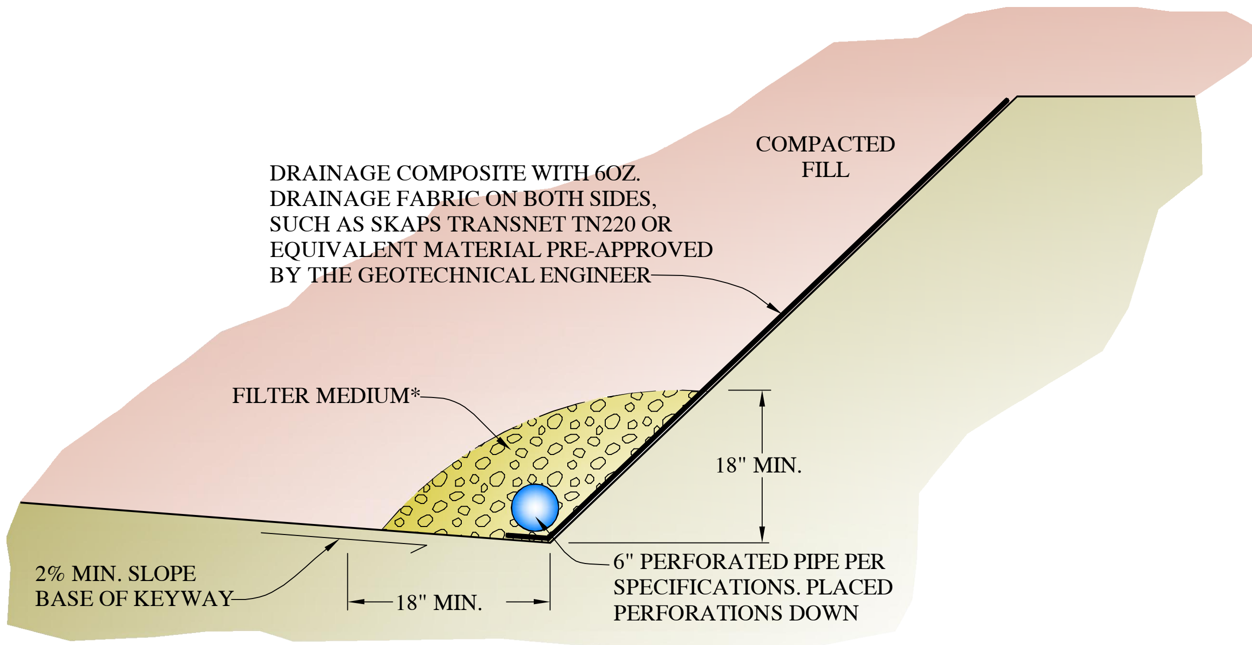
G:\Drafting\DRAFTING2_Dwg_10000 to 12999\12843\000\GEX\G6\12843000000-GEX-G6-7FaultMap-1216.dwg Plot Date:12-20-16 JStephenson

ORIGINAL FIGURE PRINTED IN COLOR

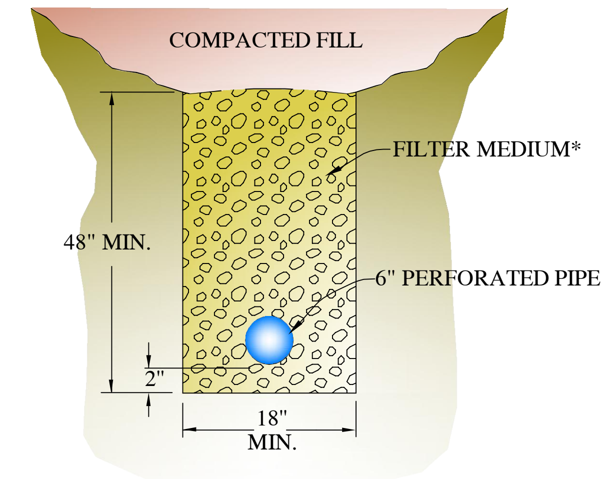


ENGEO Expect Excellence	TYPICAL KEYWAY DETAIL ROUTE 238 BYPASS - GROUP 6 HAYWARD, CALIFORNIA	PROJECT NO.: 12843.000.000 SCALE: NO SCALE DRAWN BY: JCS / CHECKED BY: RH	FIGURE NO. 8

ORIGINAL FIGURE PRINTED IN COLOR



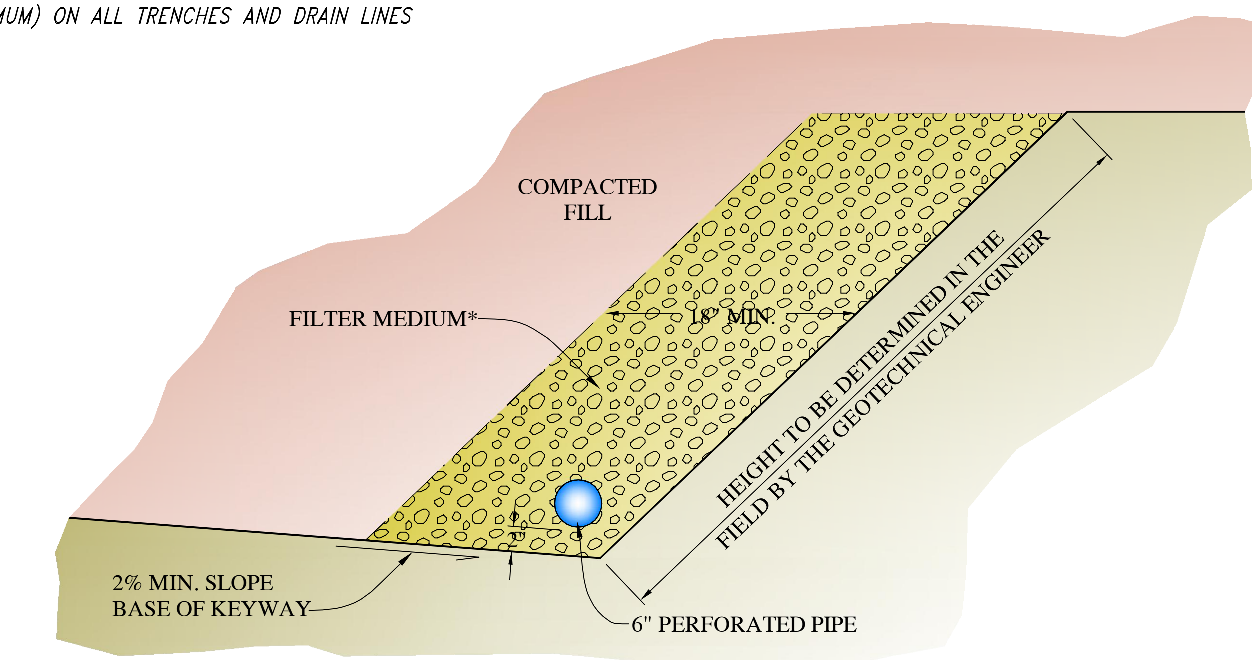
KEYWAY SUBDRAIN - OPTION 1



SWALE SUBDRAIN

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



KEYWAY SUBDRAIN - OPTION 2

*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

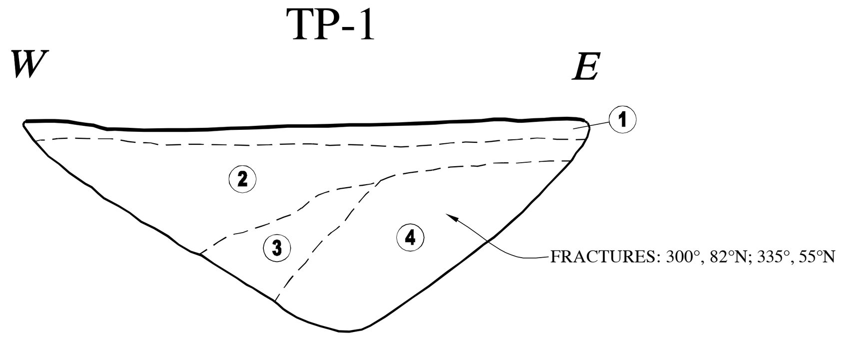
ENGEO
Expect Excellence

TYPICAL SUBDRAIN DETAILS
ROUTE 238 BYPASS - GROUP 6
HAYWARD, CALIFORNIA

PROJECT NO.: 12843.000.000	
SCALE: NO SCALE	
DRAWN BY: JCS	CHECKED BY: RH

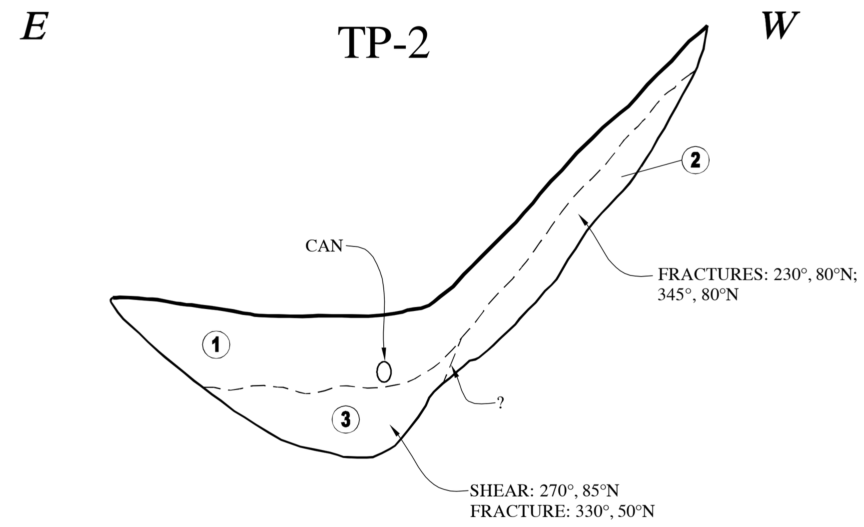
FIGURE NO
9

TP-1



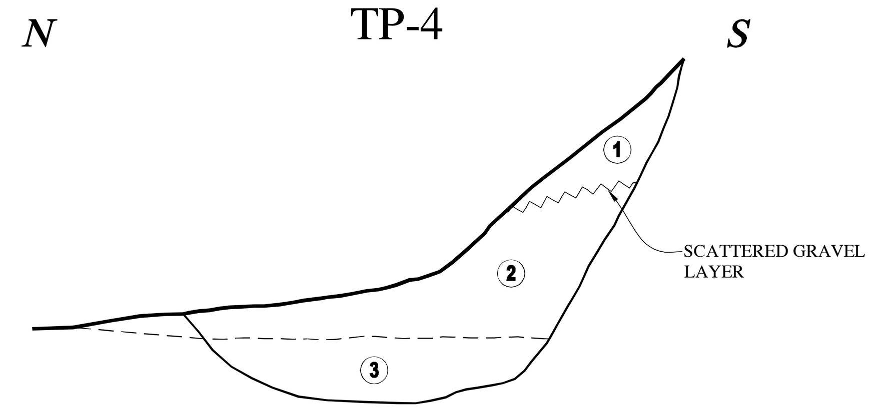
1. FAT CLAY (CH), dark brown, soft, moist, scattered sand and angular gravel (Fill).
2. SANDY LEAN CLAY (CL), light brown, medium stiff, moist (Fill).
3. CLAYEY GRAVEL (GC), yellowish brown, medium dense, moist, angular gravel (Colluvium).
4. COMPLETELY WEATHERED GABBRO (Jgb), white and yellow, friable, very closely fractured, completely wea

TP-2



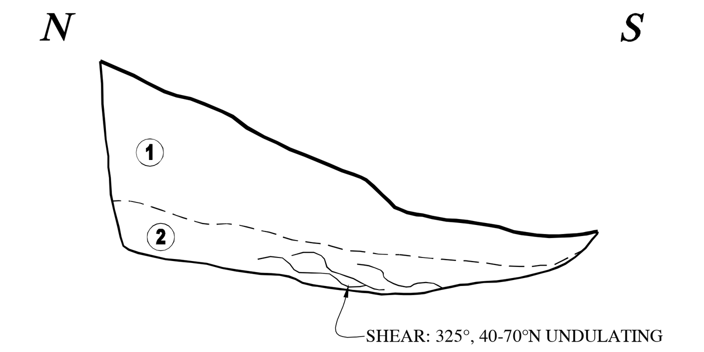
1. FAT CLAY (CH), dark brown, soft, moist, scattered sand and angular gravel (Fill).
2. COMPLETELY WEATHERED GABBRO (Jgb), white and yellow (10YR 5/8), friable, very closely fractured, completely weathered.
3. HIGHLY WEATHERED GABBRO (Jgb), dark bluish gray (10B 4/1), weak, shear banding, very closely fractured, highly weathered, calcium carbonate staining.

TP-4



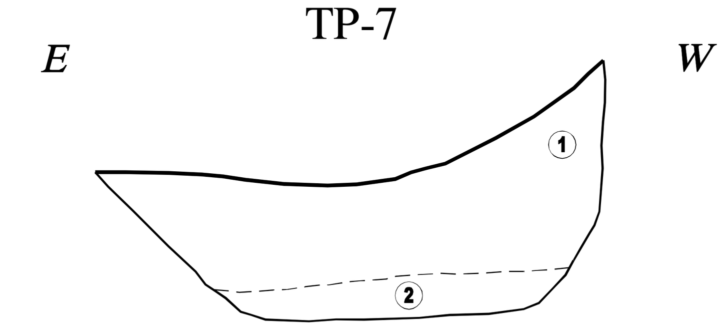
1. FAT CLAY with gravel (CH), dark brown, soft, moist, scattered angular gravel (Fill).
2. FAT CLAY with gravel (CH), dark brown, soft, moist, scattered angular gravel (Colluvium).
3. HIGHLY WEATHERED GABBRO (Jgb), light bluish gray, weak, shear banding, very closely fractured, highly weathered, calcium carbonate staining.

TP-5



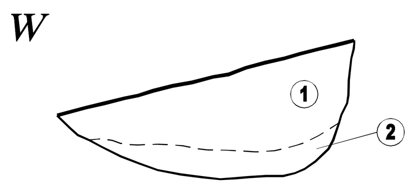
1. CLAYEY GRAVEL (GC), dark yellowish brown, medium dense, moist, subangular to angular gravel (Colluvium).
2. HIGHLY WEATHERED GABBRO (Jgb), light bluish gray, weak, shear banding, very closely fractured, highly weathered, calcium carbonate staining.

TP-7



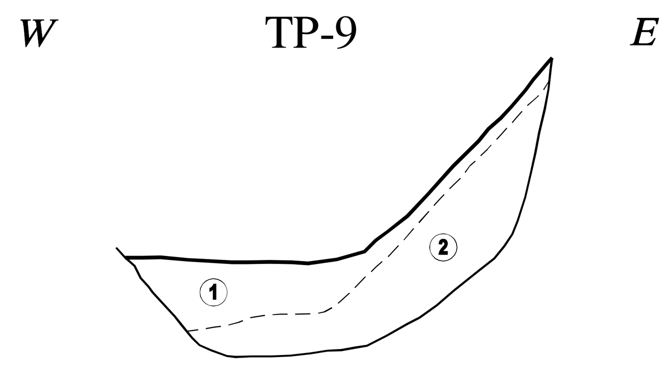
1. SANDY FAT CLAY WITH GRAVEL (CH), dark brown, soft, moist, scattered angular gravel, asphalt fragments (Fill).
2. HIGHLY WEATHERED GABBRO (Jgb), light bluish gray, weak, shear banding, very closely fractured, highly weathered, calcium carbonate staining.

TP-8



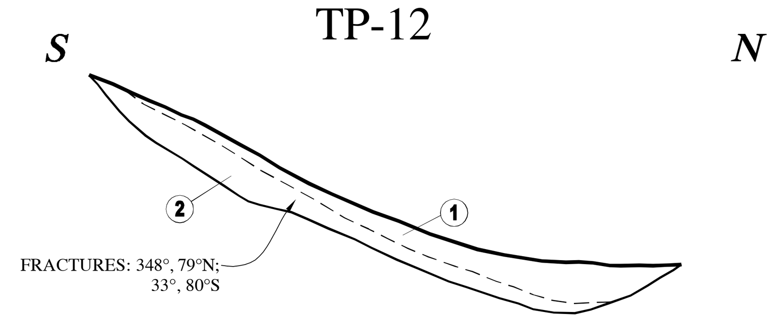
1. SANDY FAT CLAY WITH GRAVEL (CH), dark brown, soft, moist, scattered angular gravel, asphalt fragments (Fill).
2. HIGHLY WEATHERED GABBRO (Jgb), light bluish gray, weak, shear banding, very closely fractured, highly weathered, calcium carbonate staining.

TP-9



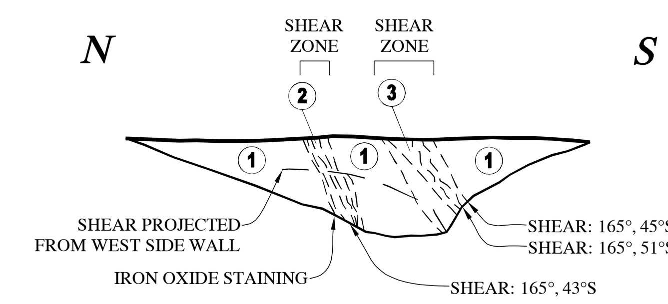
1. SANDY FAT CLAY WITH GRAVEL (CH), dark brown, soft, moist, scattered angular gravel, asphalt fragments (Fill).
2. COMPLETELY WEATHERED GABBRO (Jgb), white and yellow, friable, very closely fractured, completely weathered.

TP-12



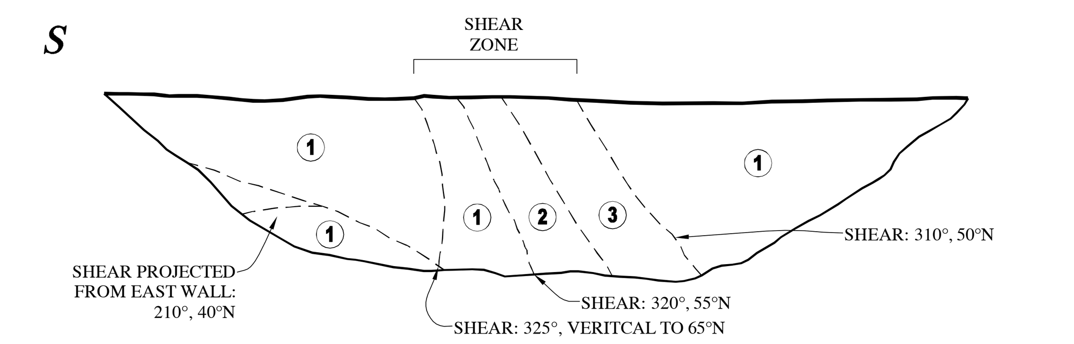
1. SILTY SAND WITH GRAVEL (SM), olive brown, loose, moist (Fill).
2. MODERATELY WEATHERED GABBRO (Jgb), dark greenish gray, moderately strong to strong, closely fractured, moderately weathered, iron oxide staining.

TP-15



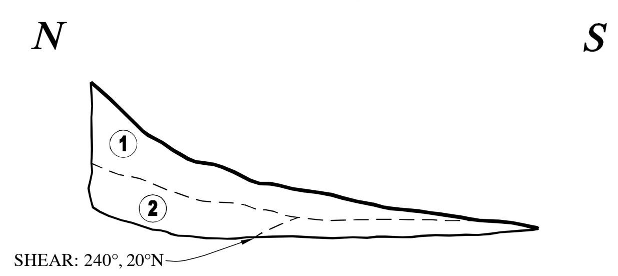
1. MODERATELY WEATHERED GABBRO (Jgb), dark greenish gray, moderately strong to strong, closely fractured, moderately weathered.
2. CLAY GOUGE (CH), light greenish gray (5GY 8/1).
3. CLAY GOUGE (CH), pale green (5G_/2 7/2).

TP-16



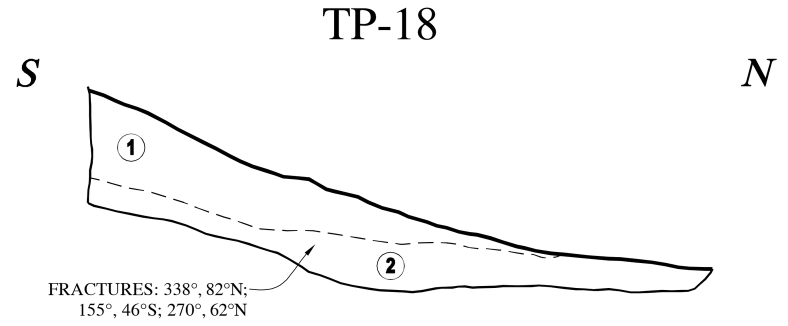
1. MODERATELY WEATHERED GABBRO (Jgb), dark greenish gray, moderately strong to strong, closely fractured, moderately weathered.
2. CLAY GOUGE (CH), olive yellow (2.5Y 5/6), STRIAE.
3. CLAY GOUGE (CH), greenish gray (5G_/1 6/1).

TP-17



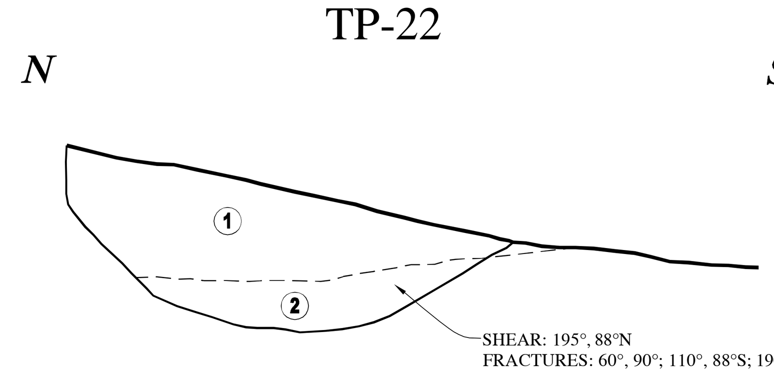
1. SILTY SAND WITH GRAVEL (SM), pale olive, loose to medium dense, moist, angular gravel and cobbles up to approximately 14-inches (Fill).
2. MODERATELY WEATHERED GABBRO (Jgb), bluish gray, moderately strong to strong, closely fractured, moderately weathered.

TP-18



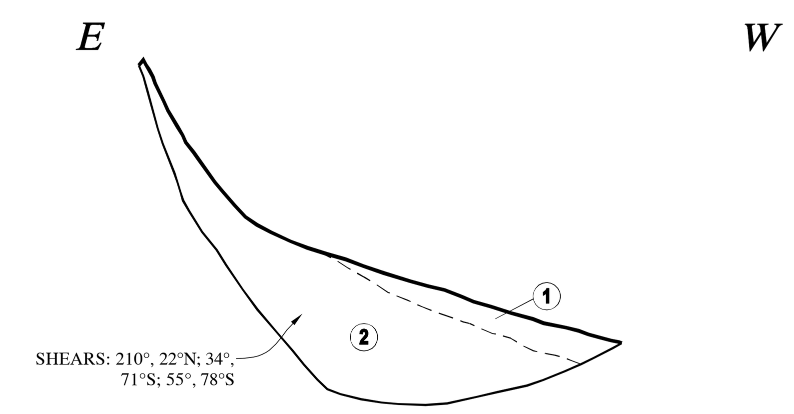
1. SILTY SAND WITH GRAVEL (SM), pale olive, loose to medium dense, moist, angular gravel and cobbles up to approximately 14-inches (Fill).
2. MODERATELY WEATHERED GABBRO (Jgb), bluish gray, moderately strong to strong, closely fractured, moderately weathered.

TP-22



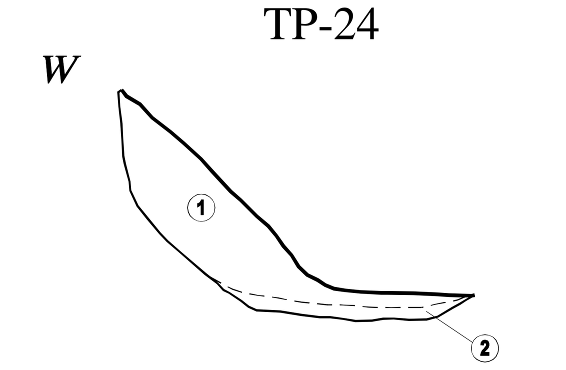
1. SANDY CLAY (CL), pale olive, soft to medium stiff, moist, scattered fine round to subrounded gravel (Fill).
2. HIGHLY WEATHERED GABBRO (Jgb), dark bluish gray, weak, shear banding, very closely fractured, highly weathered, calcium carbonate staining.

TP-23



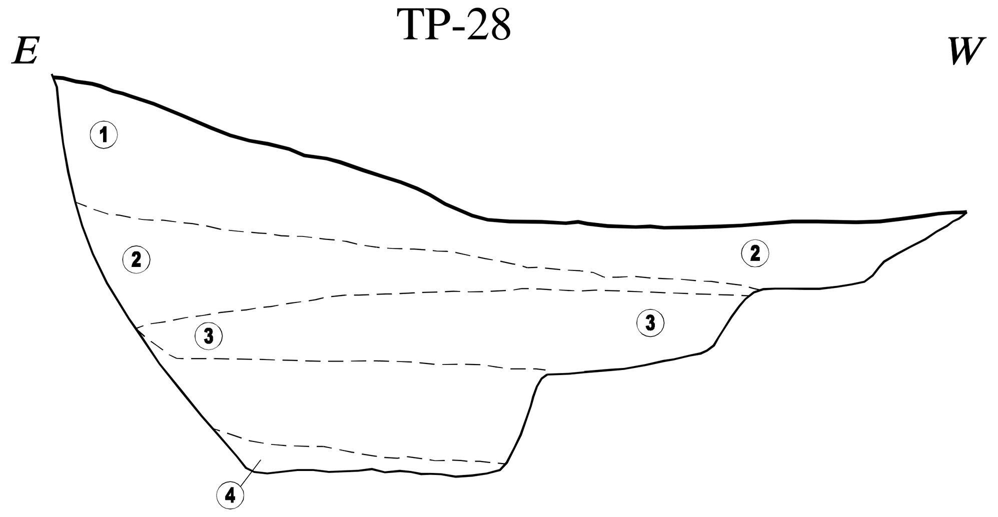
1. SILTY GRAVEL (GM), olive to yellowish brown, loose to medium dense, moist, construction debris at ground surface (Fill).
2. HIGHLY WEATHERED GABBRO (Jgb), dark bluish gray, weak, shear banding, very closely fractured, highly weathered, calcium carbonate staining.

TP-24



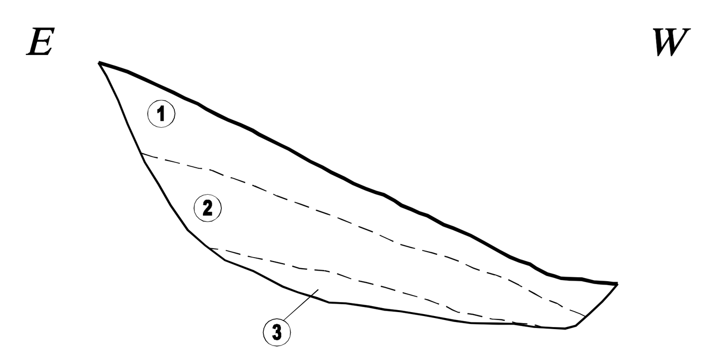
1. SANDY GRAVEL (GP), olive, loose to medium dense, moist, gabbro cobbles up to approximately 24-inches (Fill).
2. HIGHLY WEATHERED GABBRO (Jgb), dark bluish gray, weak, shear banding, very closely fractured, highly weathered, calcium carbonate staining.

TP-28



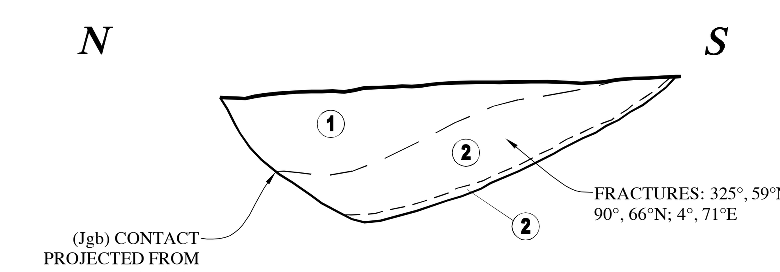
1. SANDY FAT CLAY (CH), dark brown, soft, moist, scattered gravel (Fill).
2. SILTY SAND (SM), pale yellow, loose, moist, scattered angular gravel (Fill).
3. SANDY FAT CLAY (CH), dark brown, very stiff to hard, moist (Colluvium).
4. HIGHLY WEATHERED GABBRO (Jgb), pale greenish yellow, friable to weak, shear banding, very closely fractured, highly weathered.

TP-30

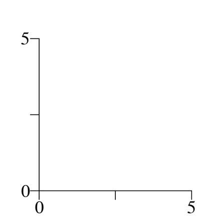


1. SANDY FAT CLAY (CH), dark brown, soft, moist, scattered gravel (Fill).
2. SILTY SAND (SM), pale yellow, loose, moist, scattered angular gravel (Fill).
3. SANDY FAT CLAY (CH), dark brown, very stiff to hard, moist (Colluvium).

TP-35



1. CLAYEY GRAVEL WITH SAND (GC), dark reddish brown, loose to medium dense, moist, angular to subangular gabbro gravel and cobbles up to approximately 14-inches (Fill).
2. MODERATELY WEATHERED GABBRO (Jgb), bluish gray, moderately strong to strong, closely fractured, moderately weathered.



NOTE: FOR TEST PITS NOT LISTED, SEE TEST PIT TABLES IN APPENDIX A.

LOGGED / REVIEWED BY: M. PARKS / JBR; DATES LOGGED: 10/28/2016, 10/31/2016, 11/02/2016

ENGEO Expect Excellence	TEST PIT GEOGRAPHICAL LOGS ROUTE 238 BYPASS - GROUP 6 HAYWARD, CALIFORNIA	PROJECT NO.: 12843.000.000 SCALE: AS SHOWN DRAWN BY: JCS CHECKED BY: RH	FIGURE NO. A1

ORIGINAL FIGURE PRINTED IN COLOR

APPENDIX A

Test Pit Logs

Expect Excellence	Test Pit Logs
Route 238 Bypass – Group 6 Preliminary Geotechnical Exploration Hayward, California 12843.000.000	Logged/Reviewed By: M. Parks/JBR Dates Logged: 10/26/2016, 10/31/2016, 11/2/2016

Test Pit Number	Depth (feet)	Description
TP-3	0 – 10	CLAYEY GRAVEL WITH SAND (GC), yellowish brown, loose to medium dense, dry, with fine gravel [Fill]
	10 – 10½	HIGHLY WEATHERED GABBRO (Jgb), light bluish gray, weak, shear banding, very closely fractured, highly weathered, calcium carbonate staining
		Total depth 10½ feet, no free groundwater encountered
TP-6	0 – 2	SANDY FAT CLAY (CH), dark brown, soft, moist, scattered gravel [Fill]
	2 – 2½	HIGHLY WEATHERED GABBRO (Jgb), light bluish gray, weak, shear banding, very closely fractured, highly weathered, calcium carbonate staining
		Total depth 2½ feet, no free groundwater encountered
TP-10	0 – 1½	SILTY CLAY WITH SAND (CL), pale yellowish brown, soft, wet [Fill]
	1½ – 2	MODERATELY WEATHERED GABBRO (Jgb), light bluish gray, moderately strong to strong, closely fractured, moderately weathered
		Total depth 2 feet, no free groundwater encountered
TP-11	0 – 1½	SILTY CLAY WITH SAND (CL), pale yellowish brown, soft, wet [Fill]
	1½ – 2	MODERATELY WEATHERED GABBRO (Jgb), light bluish gray, moderately strong to strong, closely fractured, moderately weathered
		Total depth 2 feet, no free groundwater encountered

ENGEO Expect Excellence	Test Pit Logs	
Route 238 Bypass – Group 6 Preliminary Geotechnical Exploration Hayward, California 12843.000.000	Logged/Reviewed By: M. Parks/JBR Dates Logged: 10/26/2016, 10/31/2016, 11/2/2016	

Test Pit Number	Depth (feet)	Description
TP-13	0 – 4	SILTY GRAVEL WITH SAND (GM), olive to bluish gray, loose to medium dense, moist [Fill]
	4 – 5	MODERATELY WEATHERED GABBRO (Jgb), bluish gray, moderately strong to strong, closely fractured, moderately weathered
		Fractures: 234°, 56°N; 0°, 68°E
		Total depth 5 feet, no free groundwater encountered
TP-14	0 – ¼	SILTY SAND (SM), pale olive, loose, moist [Fill]
	¼ – 1½	MODERATELY WEATHERED GABBRO (Jgb), dark greenish gray (5GY 4/1), moderately strong to strong, closely fractured, moderately weathered, iron oxide staining
		Fractures: 15°, 57°S; 280°, 75°N; 235°, 19°N
		Total depth 1½ feet, no free groundwater encountered
TP-19	0 – 1	SILTY SAND WITH GRAVEL (SM), pale olive, loose to medium dense, moist, angular gravel and cobbles [Fill]
	1 – 2	MODERATELY WEATHERED GABBRO (Jgb), dark greenish gray, moderately strong to strong, closely fractured, moderately weathered
		Shears: 199°, 38°N; 345°, 60°N
		Total depth 2 feet, no free groundwater encountered

ENGEO Expect Excellence	Test Pit Logs	
Route 238 Bypass – Group 6 Preliminary Geotechnical Exploration Hayward, California 12843.000.000	Logged/Reviewed By: M. Parks/JBR Dates Logged: 10/26/2016, 10/31/2016, 11/2/2016	

Test Pit Number	Depth (feet)	Description
TP-20	0 – ¼	SILTY SAND (SM), pale olive, loose, moist [Fill]
	¼ – 1½	MODERATELY WEATHERED GABBRO (Jgb), bluish gray, moderately strong to strong, closely fractured, moderately weathered, iron oxide staining
		Total depth 1½ feet, no free groundwater encountered
TP-21	0 – ¼	SILTY SAND WITH GRAVEL (SM), pale olive, loose, moist, gravel and cobbles up to approximately 6-inches [Fill]
	¼ – 1½	MODERATELY WEATHERED GABBRO (Jgb), dark bluish gray, moderately strong to strong, closely fractured, moderately weathered, iron oxide staining
		Fractures: 305°, 35°N; 295°, 66°N
		Total depth 1½ feet, no free groundwater encountered
TP-25	0 – ¼	SANDY GRAVEL (GP), olive, loose to medium dense, moist [Fill]
	¼ – 1	HIGHLY WEATHERED GABBRO (Jgb), dark bluish gray, weak, shear banding, very closely fractured, highly weathered, iron oxide staining
		Shears: 45°, 65°N; 10°, 66°S; 60°, 50°S
		Total depth 1 foot, no free groundwater encountered

Test Pit Logs

Route 238 Bypass – Group 6
Preliminary Geotechnical Exploration
Hayward, California
12843.000.000

Logged/Reviewed By: M. Parks/JBR
Dates Logged: 10/26/2016, 10/31/2016, 11/2/2016

Test Pit Number	Depth (feet)	Description
TP-26	0 – 1	CLAYEY SAND (SC), dark brown, loose, moist, scattered gravel [Fill]
	1 – 1½	MODERATELY WEATHERED GABBRO (Jgb), dark bluish gray, moderately strong to strong, closely fractured, moderately weathered
		Total depth 1½ feet, no free groundwater encountered
TP-27	0 – 1	SANDY CLAY (CL), dark brown, soft, moist [Fill]
	1 – 3	HIGHLY WEATHERED GABBRO (Jgb), bluish gray, weak, shear banding, very closely fractured, highly weathered
		Shears: 315°, 68°N; 25°, 88°S
		Total depth 3 feet, no free groundwater encountered
TP-29a Crest of hill	0 – 2	SANDY FAT CLAY (CH), dark brown, soft, moist, scattered gravel [Fill]
	2 – 2½	SILTY SAND (SM), pale yellow, loose, moist, scattered angular gravel [Fill]
	2½ – 3	SANDY FAT CLAY (CH), dark brown, very stiff to hard, moist [Colluvium]
		Total depth 3 feet, no free groundwater encountered

ENGEO Expect Excellence		# Test Pit Logs
Route 238 Bypass – Group 6 Preliminary Geotechnical Exploration Hayward, California 12843.000.000		Logged/Reviewed By: M. Parks/JBR Dates Logged: 10/26/2016, 10/31/2016, 11/2/2016
Test Pit Number	Depth (feet)	Description
TP-29b Base of hill	0 – 1½	SANDY FAT CLAY (CH), dark brown, soft, moist, scattered gravel [Fill]
	1½ – 1¾	SILTY SAND (SM), pale yellow, loose, moist, scattered angular gravel [Fill]
	1¾ – 3	SANDY FAT CLAY (CH), dark brown, very stiff to hard, moist [Colluvium]
		Total depth 3 feet, no free groundwater encountered
TP-31	0 – 2½	SANDY FAT CLAY (CH), dark brown, soft, moist, scattered gravel [Fill]
	2½ – 4	SILTY SAND WITH GRAVEL (SM), pale yellow, loose, moist, angular gravel [Fill]
	4 – 4¾	SANDY GRAVEL (GP), light yellowish brown, medium dense, moist, angular gravel [Fill]
	4¾ – 8½	SANDY FAT CLAY (CH), dark brown, hard, moist [Colluvium]
	8½ – 13¼	CLAYEY SAND WITH GRAVEL (SC), reddish brown, medium dense, moist, scattered weathered gravel [Colluvium]
	13¼ – 13½	HIGHLY WEATHERED GABBRO (Jgb), pale greenish yellow, friable to weak, shear banding, very closely fractured, highly weathered
		Total depth 13½ feet, no free groundwater encountered

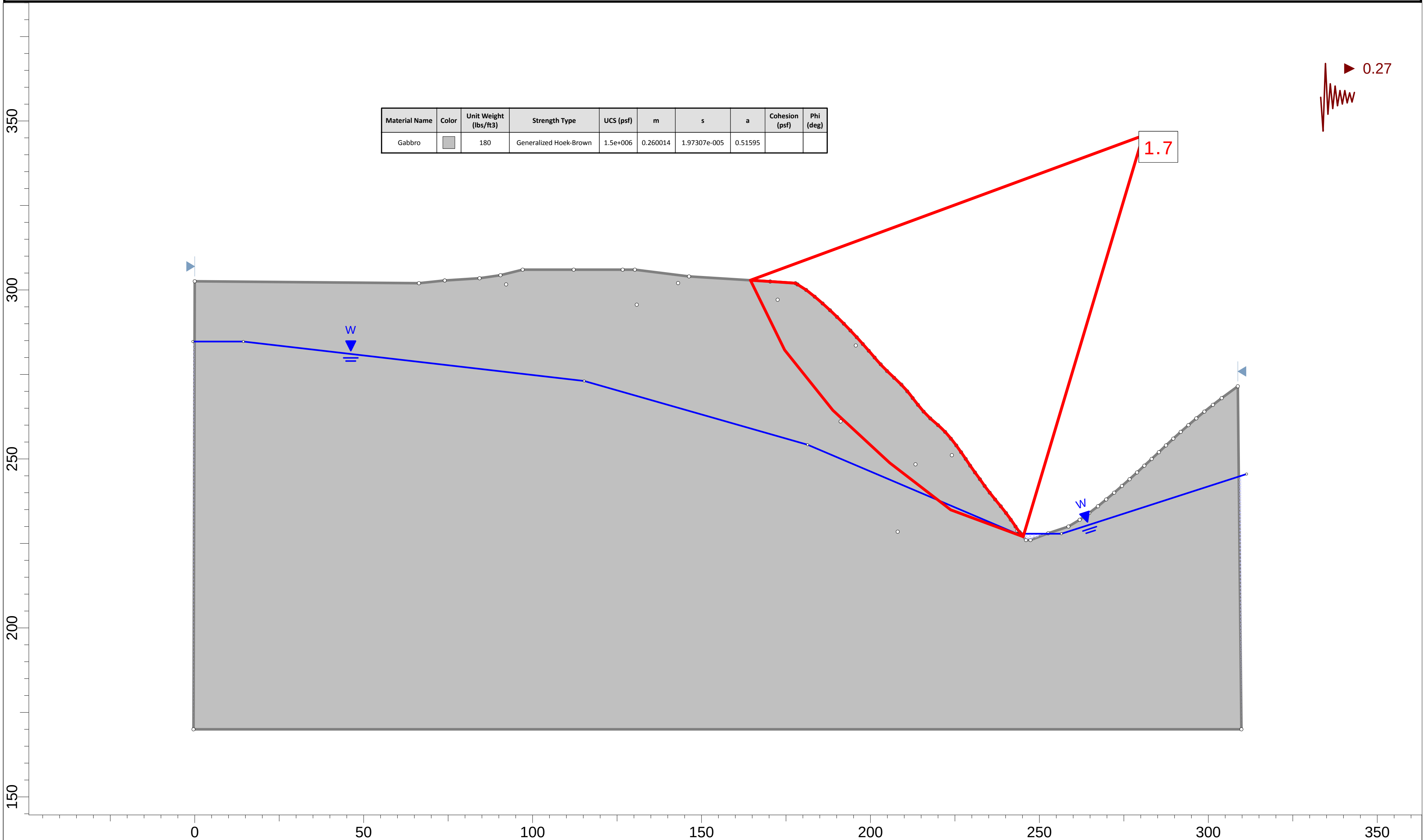
ENGEO Expect Excellence		Test Pit Logs
Route 238 Bypass – Group 6 Preliminary Geotechnical Exploration Hayward, California 12843.000.000		Logged/Reviewed By: M. Parks/JBR Dates Logged: 10/26/2016, 10/31/2016, 11/2/2016
Test Pit Number	Depth (feet)	Description
TP-32	0 – 2¾	SANDY FAT CLAY (CH), dark brown, soft, moist, scattered gravel [Fill]
	2¾ – 6	SANDY FAT CLAY (CH), dark brown, hard, moist [Colluvium]
	6 – 11	CLAYEY SAND WITH GRAVEL (SC), reddish brown, medium dense, moist, scattered weathered gravel [Colluvium]
	11 – 11½	HIGHLY WEATHERED GABBRO (Jgb), pale greenish yellow, friable to weak, shear banding, very closely fractured, highly weathered
		Total depth 11½ feet, no free groundwater encountered
TP-33	0 – 3½	SILTY GRAVEL WITH SAND (GM), yellowish brown, loose, moist, scattered gabbro gravel and cobbles up to approximately 6-inches [Fill]
	3½ – 4	MODERATELY WEATHERED GABBRO (Jgb), dark bluish gray, moderately strong to strong, closely fractured, moderately weathered
		Total depth 4 feet, no free groundwater encountered
TP-34	0 – 1½	SANDY LEAN CLAY WITH GRAVEL (CL), light brown, soft, moist [Fill]
	1½ – 15	SANDY LEAN CLAY WITH GRAVEL (CL), dark reddish brown, stiff to very stiff, moist, subangular gravel and cobbles up to approximately 3-inches; root at 10 feet below the ground surface
	15 – 15½	MODERATELY WEATHERED GABBRO (Jgb), dark bluish gray, moderately strong to strong, closely fractured, moderately weathered
		Total depth 15½ feet, no free groundwater encountered

ENGEO Expect Excellence		Test Pit Logs
Route 238 Bypass – Group 6 Preliminary Geotechnical Exploration Hayward, California 12843.000.000		Logged/Reviewed By: M. Parks/JBR Dates Logged: 10/26/2016, 10/31/2016, 11/2/2016
Test Pit Number	Depth (feet)	Description
TP-36	0 – 3	SANDY LEAN CLAY WITH GRAVEL (CL), dark reddish brown with pockets of yellowish brown to olive, soft to medium stiff, moist, angular to subangular gabbro gravel and cobbles up to approximately 7-inches [Fill]
	3 – 3½	MODERATELY WEATHERED GABBRO (Jgb), dark bluish gray, moderately strong to strong, closely fractured, moderately weathered
		Total depth 3½ feet, no free groundwater encountered
TP-37	0 – 3	SANDY FAT CLAY (CH), blackish brown, soft, moist, brick fragments just above gabbro contact [Fill]
	3 – 6	COMPLETELY WEATHERED GABBRO (Jgb), pale yellow and brownish yellow, friable, very closely fractured, completely weathered
		Total depth 6 feet, no free groundwater encountered

APPENDIX B

Slope Stability Analysis Results

Section 4-4' Seismic



Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	UCS (psf)	m	s	a	Cohesion (psf)	Phi (deg)
Gabbro		180	Generalized Hoek-Brown	1.5e+006	0.260014	1.97307e-005	0.51595		

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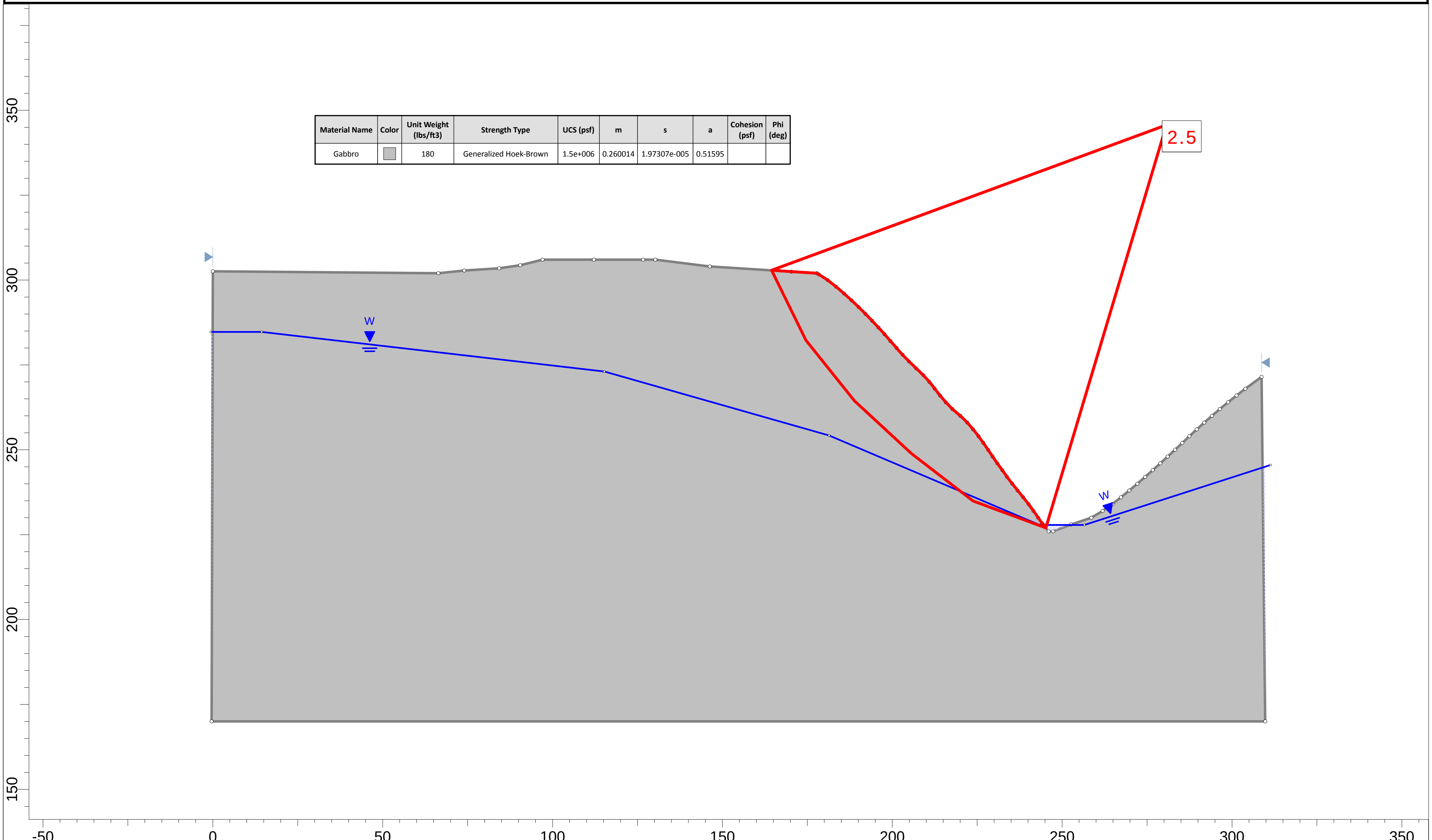
SLIDEINTERPRET 7.020

Project	Route 238 Group 6		
Scale 1:480	*Author* JBR, MCP	*Project No.* 12843.000.000	
Date 12/22/2016	*Condition* Seismic		

Section 4-4' Static

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	UCS (psf)	m	s	a	Cohesion (psf)	Phi (deg)
Gabbro		180	Generalized Hoek-Brown	1.5e+006	0.260014	1.97307e-005	0.51595		



Project	Route 238 Group 6		
Scale 1:480	Author JBR, MCP	Project No. 12843.000.000	
Date 12/22/2016	Condition Static		

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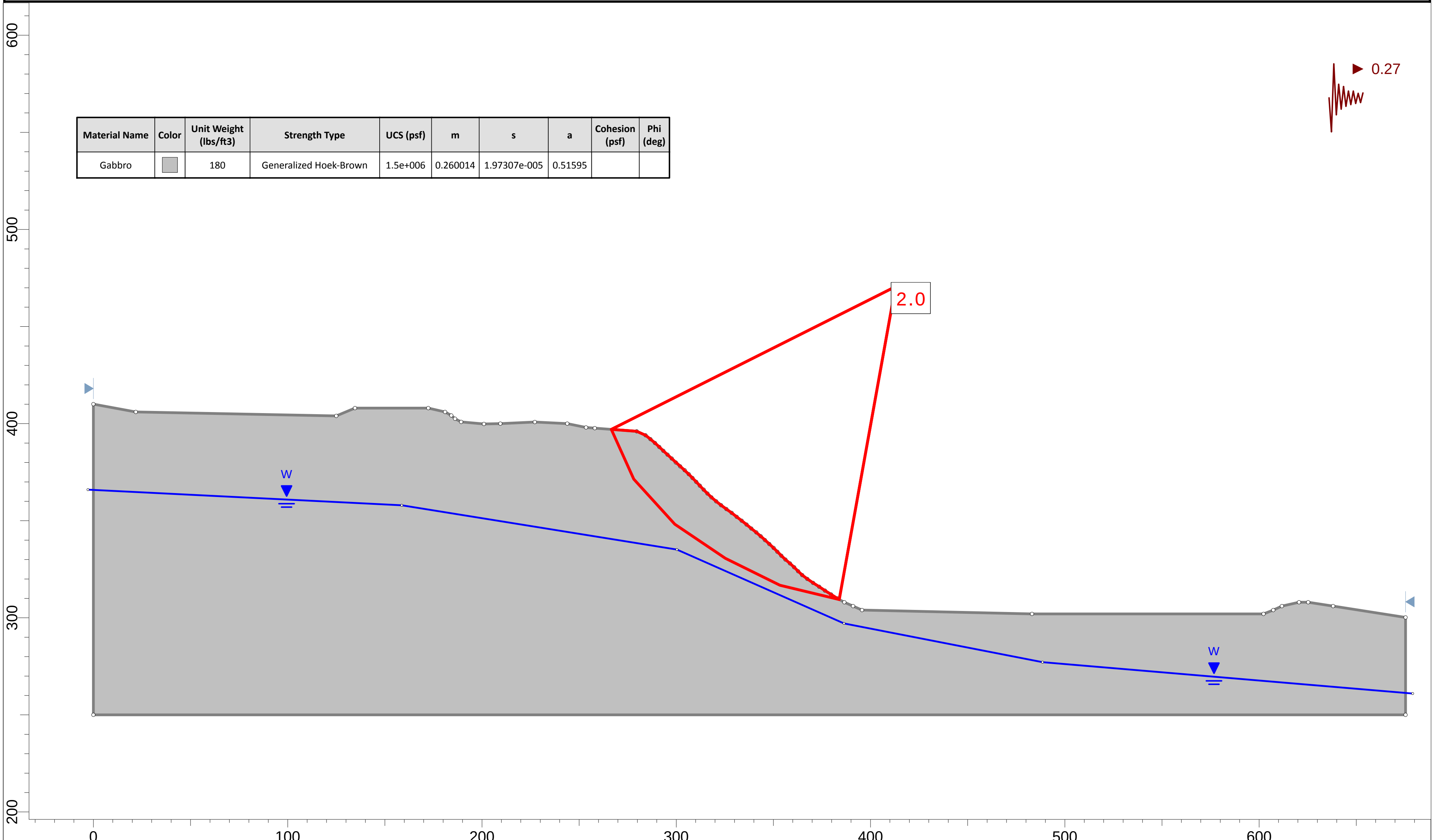
SLIDEINTERPRET 7.020

Section 5-5' Seismic

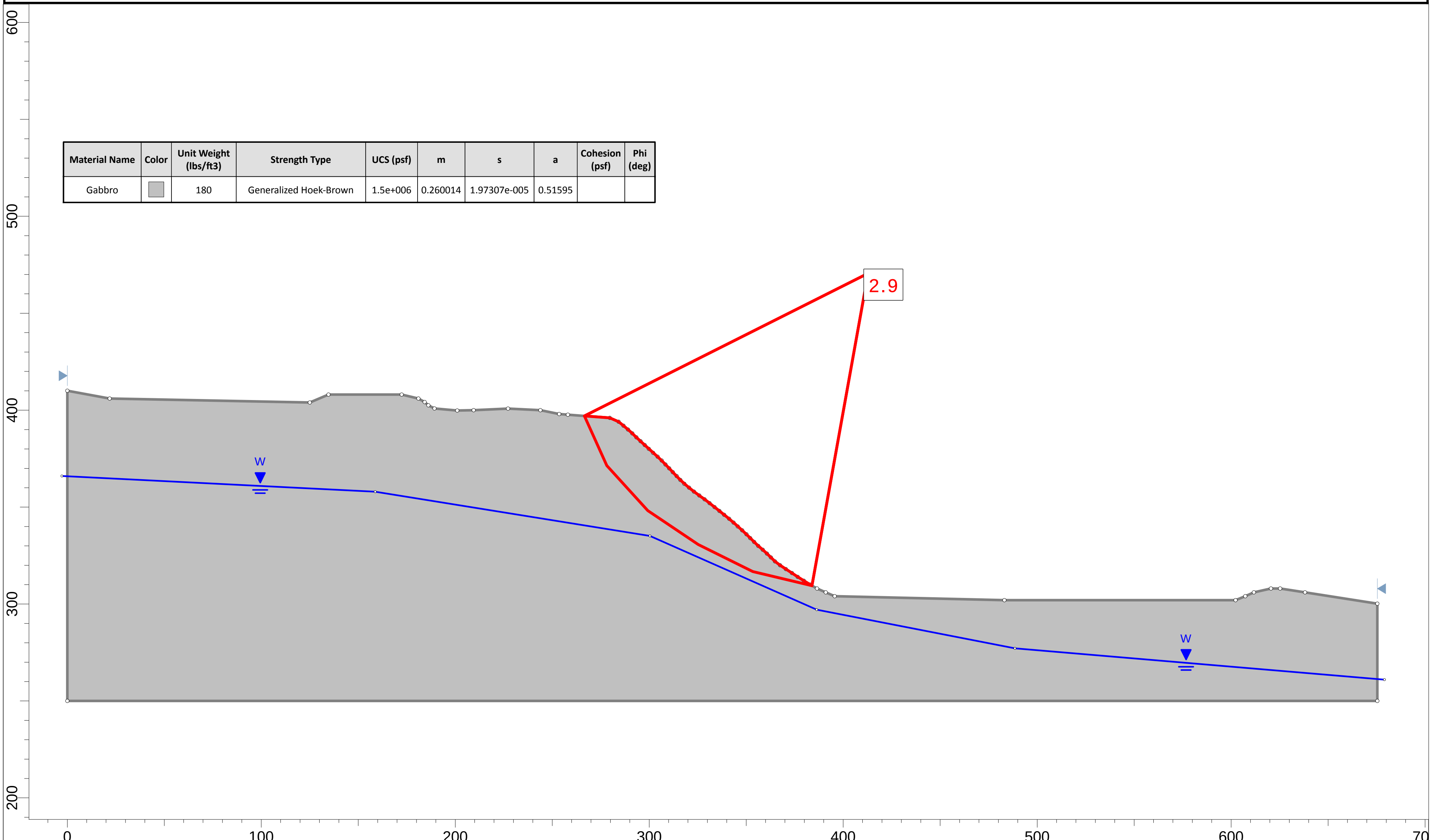
Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	UCS (psf)	m	s	a	Cohesion (psf)	Phi (deg)
Gabbro		180	Generalized Hoek-Brown	1.5e+006	0.260014	1.97307e-005	0.51595		



ENGEO Expect Excellence	*Project* Route 238 Group 6		
	Scale 1:840	*Author* JBR, MCP	*Project No.* 12843.000.000
SLIDEINTERPRET 7.020	*Date* 12/22/2016	*Condition* Seismic	

Section 5-5' Static

Material Name	Color	Unit Weight (lbs/ft3)	Strength Type	UCS (psf)	m	s	a	Cohesion (psf)	Phi (deg)
Gabbro		180	Generalized Hoek-Brown	1.5e+006	0.260014	1.97307e-005	0.51595		



ENGEO Expect Excellence	Project: Route 238 Group 6		
	Scale: 1:840	Author: JBR, MCP	Project No.: 12843.000.000
SLIDEINTERPRET 7.020	Date: 12/22/2016	Condition: Static	

USGS Design Maps Summary Report

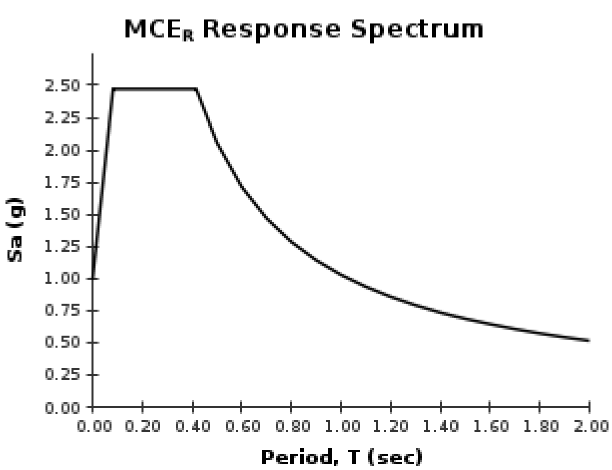
User–Specified Input

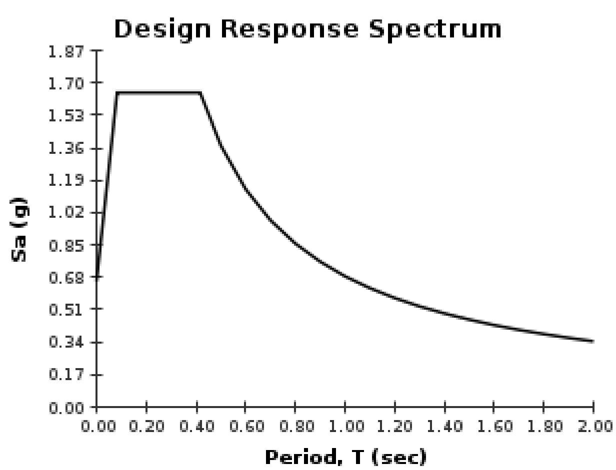
Report Title	Quarry Fri December 2, 2016 00:07:36 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	37.6636°N, 122.067°W
Site Soil Classification	Site Class B – “Rock”
Risk Category	I/II/III

USGS–Provided Output

S_S =	2.471 g	S_{MS} =	2.471 g	S_{DS} =	1.648 g
S_1 =	1.028 g	S_{M1} =	1.028 g	S_{D1} =	0.685 g

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





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

Although this information is a product of the U.S. Geological Survey, we provide no warranty, expressed or implied, as to the accuracy of the data contained therein. This tool is not a substitute for technical subject-matter knowledge.

Design Maps Detailed Report

ASCE 7-10 Standard (37.6636°N, 122.067°W)

Site Class B – "Rock", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

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

From Figure 22-1 [1] $S_S = 2.471$ g

From Figure 22-2 [2] $S_1 = 1.028$ g

Section 11.4.2 — Site Class

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

Table 20.3–1 Site Classification

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

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

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

Table 11.4–1: Site Coefficient F_a

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

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

For Site Class = B and S_S = 2.471 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

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

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

For Site Class = B and S_1 = 1.028 g, F_v = 1.000

Equation (11.4–1): $S_{MS} = F_aS_S = 1.000 \times 2.471 = 2.471$ g

Equation (11.4–2): $S_{M1} = F_vS_1 = 1.000 \times 1.028 = 1.028$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

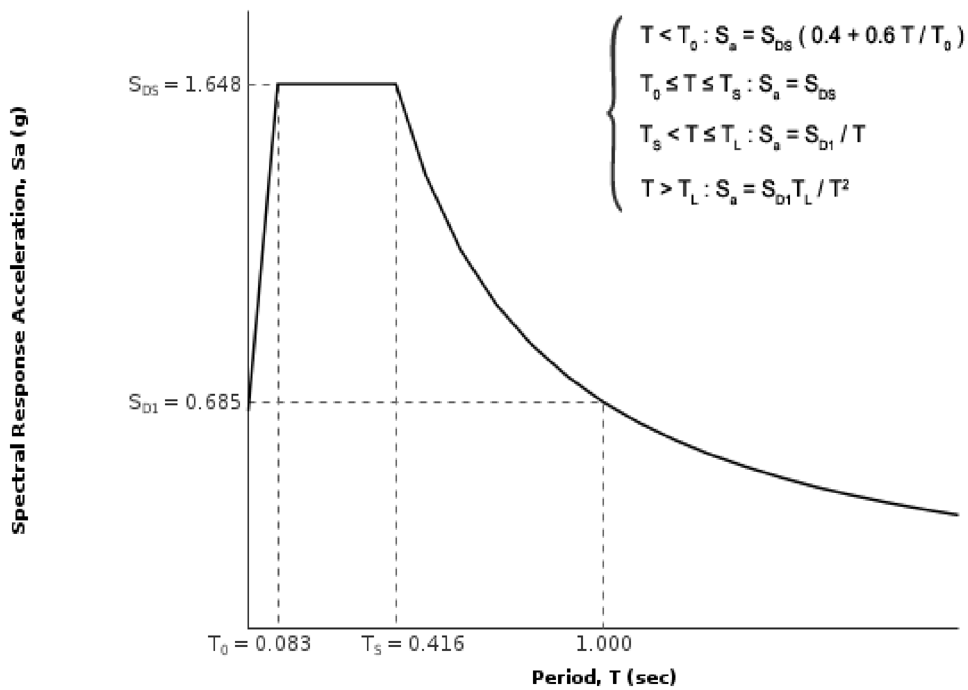
Equation (11.4–3): $S_{DS} = ⅔ S_{MS} = ⅔ \times 2.471 = 1.648$ g

Equation (11.4–4): $S_{D1} = ⅔ S_{M1} = ⅔ \times 1.028 = 0.685$ g

Section 11.4.5 — Design Response Spectrum

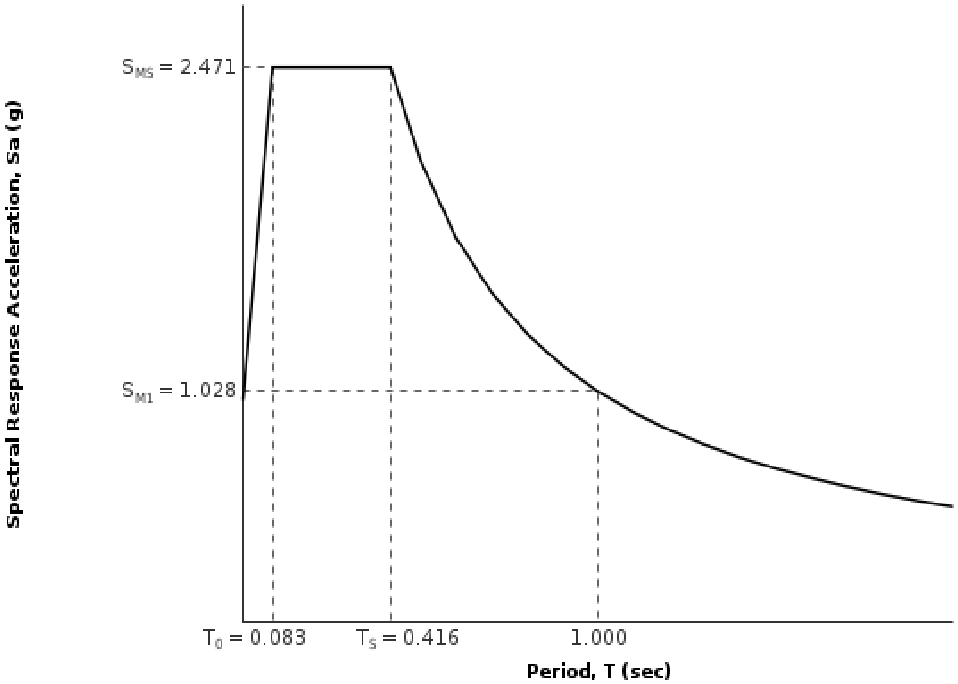
From Figure 22-12 [3] $T_L = 8$ seconds

Figure 11.4–1: Design Response Spectrum



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

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



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

From Figure 22-7 [4] PGA = 0.957

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.957 = 0.957$ g

Table 11.8–1: Site Coefficient F_{PGA}

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

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

For Site Class = B and PGA = 0.957 g, F_{PGA} = 1.000

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

From Figure 22-17 [5] C_{RS} = 0.989

From Figure 22-18 [6] C_{R1} = 0.964

Section 11.6 — Seismic Design Category

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

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

For Risk Category = I and S_{DS} = 1.648 g, Seismic Design Category = D

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

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

For Risk Category = I and S_{D1} = 0.685 g, Seismic Design Category = D

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

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

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

References

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

Design Maps Summary Report

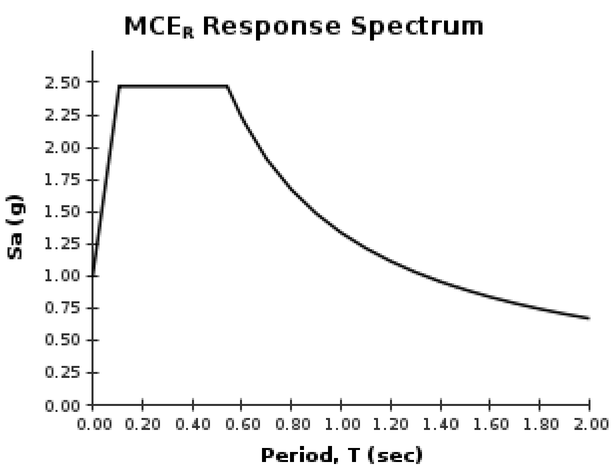
User–Specified Input

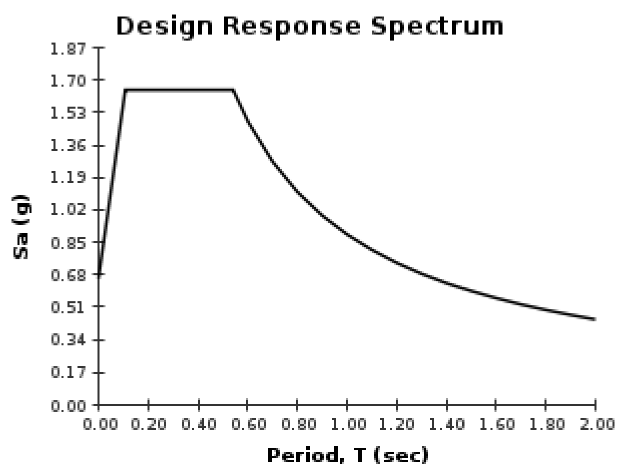
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	37.6636°N, 122.067°W
Site Soil Classification	Site Class C – "Very Dense Soil and Soft Rock"
Risk Category	I/II/III

USGS–Provided Output

S_S =	2.471 g	S_{MS} =	2.471 g	S_{DS} =	1.648 g
S_1 =	1.028 g	S_{M1} =	1.336 g	S_{D1} =	0.891 g

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





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

Design Maps Detailed Report

ASCE 7-10 Standard (37.6636°N, 122.067°W)

Site Class C – "Very Dense Soil and Soft Rock", Risk Category I/II/III

Section 11.4.1 — Mapped Acceleration Parameters

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

From Figure 22-1 [1] $S_S = 2.471$ g

From Figure 22-2 [2] $S_1 = 1.028$ g

Section 11.4.2 — Site Class

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

Table 20.3–1 Site Classification

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

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

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

Table 11.4–1: Site Coefficient F_a

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

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

For Site Class = C and S_S = 2.471 g, F_a = 1.000

Table 11.4–2: Site Coefficient F_v

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

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

For Site Class = C and S_1 = 1.028 g, F_v = 1.300

Equation (11.4–1): $S_{MS} = F_aS_S = 1.000 \times 2.471 = 2.471$ g

Equation (11.4–2): $S_{M1} = F_vS_1 = 1.300 \times 1.028 = 1.336$ g

Section 11.4.4 — Design Spectral Acceleration Parameters

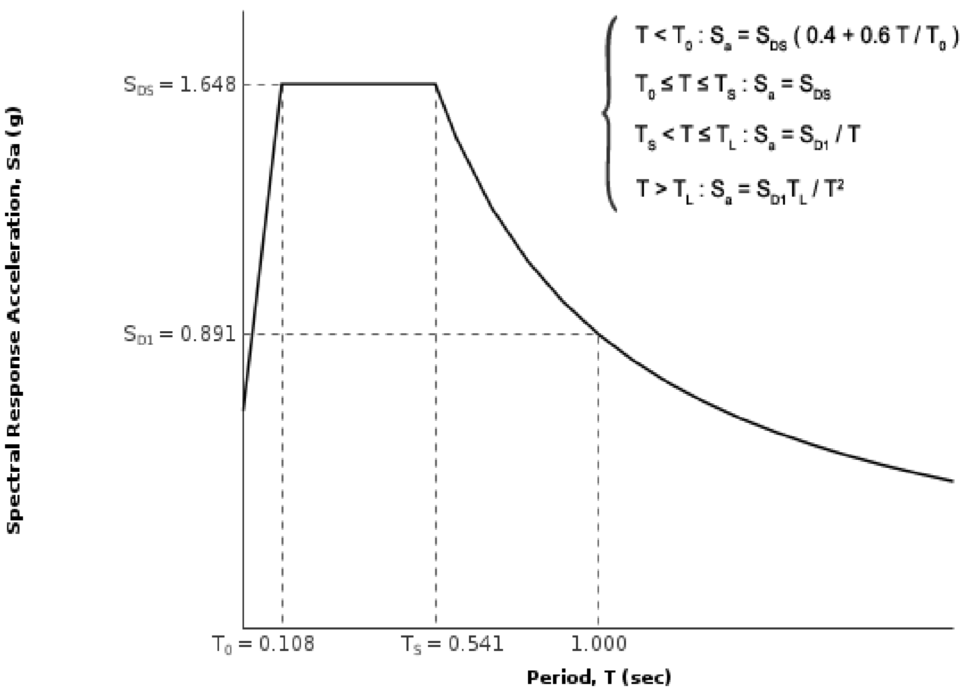
Equation (11.4–3): $S_{DS} = ⅔ S_{MS} = ⅔ \times 2.471 = 1.648$ g

Equation (11.4–4): $S_{D1} = ⅔ S_{M1} = ⅔ \times 1.336 = 0.891$ g

Section 11.4.5 — Design Response Spectrum

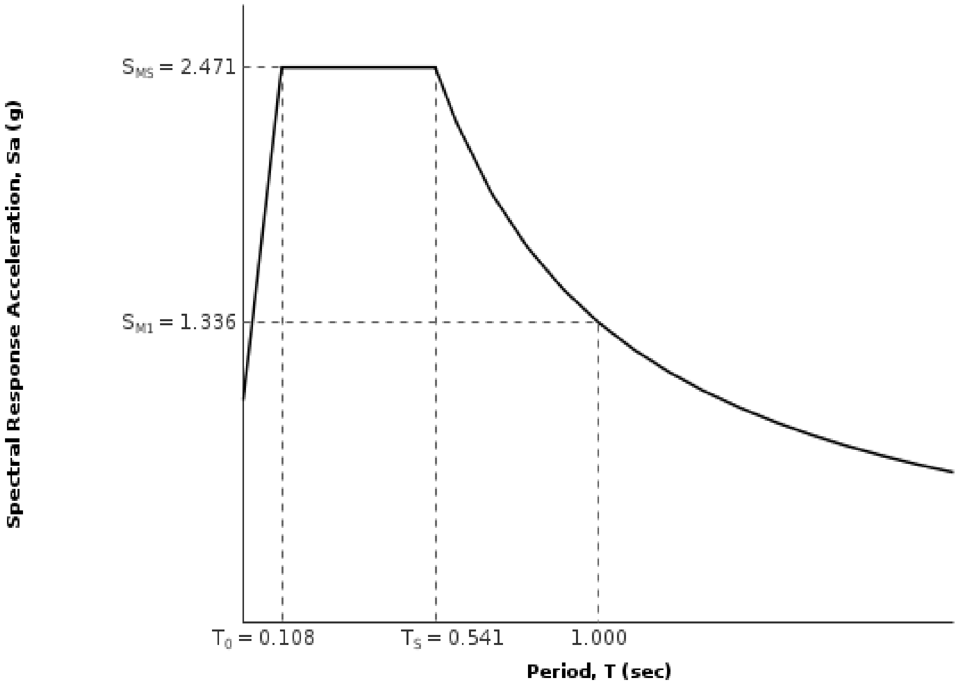
From Figure 22-12 [3] $T_L = 8$ seconds

Figure 11.4–1: Design Response Spectrum



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

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



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

From Figure 22-7 [4] $PGA = 0.957$

Equation (11.8–1): $PGA_M = F_{PGA}PGA = 1.000 \times 0.957 = 0.957$ g

Table 11.8–1: Site Coefficient F_{PGA}

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

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

For Site Class = C and PGA = 0.957 g, F_{PGA} = 1.000

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

From Figure 22-17 [5] $C_{RS} = 0.989$

From Figure 22-18 [6] $C_{R1} = 0.964$

Section 11.6 — Seismic Design Category

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

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

For Risk Category = I and S_{DS} = 1.648 g, Seismic Design Category = D

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

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

For Risk Category = I and S_{D1} = 0.891 g, Seismic Design Category = D

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

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

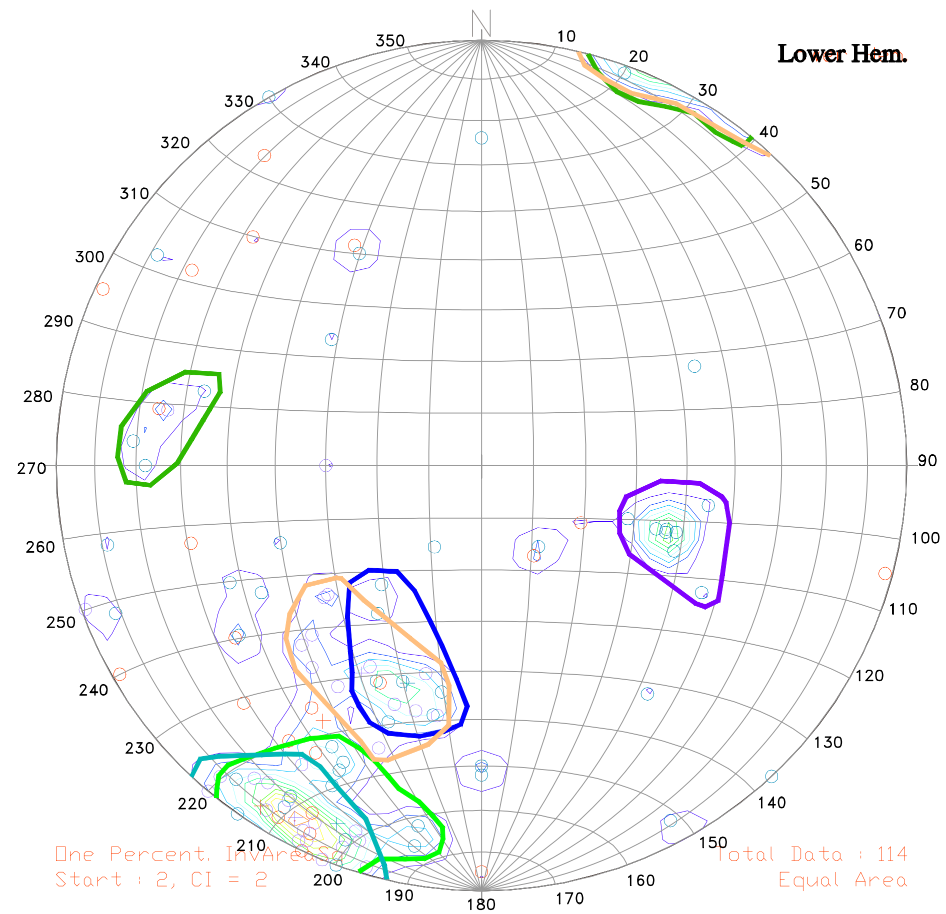
Note: See Section 11.6 for alternative approaches to calculating Seismic Design Category.

References

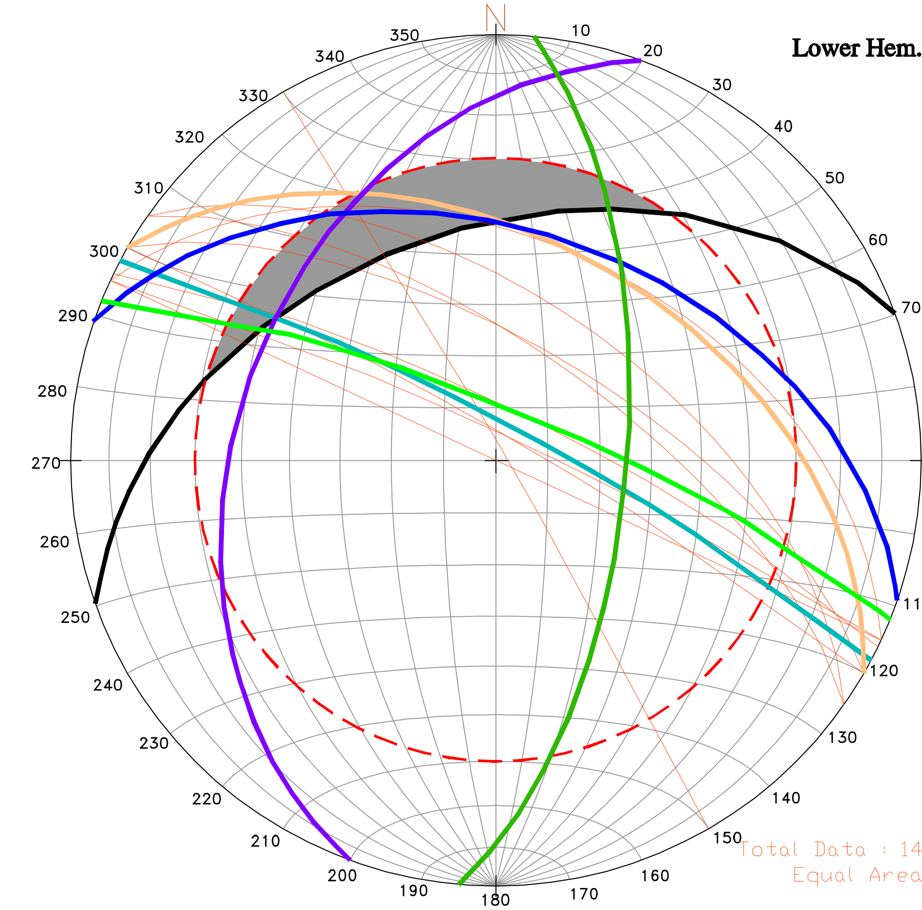
1. *Figure 22-1*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-1.pdf
2. *Figure 22-2*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-2.pdf
3. *Figure 22-12*: http://earthquake.usgs.gov/hazards/designmaps/downloads/pdfs/2010_ASCE-7_Figure_22-12.pdf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

APPENDIX C

Stereonet Analysis Results



NORTHWEST FACING QUARRY SLOPE MAPPABLE
JOINTS SETS, SHEARS AND FOLIATION



NORTHWEST FACING QUARRY SLOPE MAPPABLE
COMPILED - SHEARS, JOINT SETS AND FOLIATION
POTENTIAL WEDGE FAILURE ENVELOPES

STEREONETS

- POLE OF MAPPABLE JOINT ON STEREONET PLOT
- POLE OF MAPPABLE STRUCTURAL FABRIC ON STEREONET PLOT
- POLE OF MAPPABLE SHEAR ON STEREONET PLOT
- APPROXIMATE JOINT SET J1 CLUSTER BOUNDARY ON STEREONET PLOT
- APPROXIMATE JOINT SET J2 CLUSTER BOUNDARY ON STEREONET PLOT
- APPROXIMATE JOINT SET J3 CLUSTER BOUNDARY ON STEREONET PLOT
- APPROXIMATE JOINT SET J4 CLUSTER BOUNDARY ON STEREONET PLOT
- APPROXIMATE BEDROCK FOLIATION F1 CLUSTER BOUNDARY ON STEREONET PLOT
- APPROXIMATE BEDROCK FOLIATION F2 CLUSTER BOUNDARY ON STEREONET PLOT

APPROXIMATE GREAT CIRCLE OF FISHER AXIS OF JOINT SET, BEDDING AND SHEARS ON STEREONET

- JOINT SET J1
- JOINT SET J2
- JOINT SET J3
- JOINT SET J4
- FOLIATION SET F1
- FOLIATION SET F2
- SHEAR PLANES

- APPROXIMATED OVERALL EXISTING FACE OREINTATION ON STEREONET PLOT
- FRICTION CIRCLE ON STEREONET PLOT
- ZONE OF OVERALL PIT WALL DAYLIGHTING WEDGE INTERSECTIONS ON STEREONET PLOT

ENGEO Expect Excellence	STEREONET ANALYSIS ROUTE 238 BYPASS - GROUP 6 HAYWARD, CALIFORNIA	PROJECT NO.: 12843.000.000 SCALE: NO SCALE DRAWN BY: JBR CHECKED BY: PJS	FIGURE NO. 1