Appendix E:

The Project Geotechnical Investigation Report by Henry Justiniano & Associates

Project No. T-123-01
November 4, 2010

Mr. Hue Tran
4584 Ewing Road
Castro Valley, CA 94546

SUBJECT: GEOTECHNICAL INVESTIGATION
Proposed 24 Lot Subdivision, Tentative Track Map 8053
Plus Adjacent Lots 3 and 4, Fronting Proctor Road
Proctor Road
Castro Valley, California

Dear Mr.Tran:

Our geotechnical report for the proposed 24 Lot subdivision and adjacent two Lots, is herewith submitted. The report presents the results of our explorations and geologic literature review, along with our evaluations and recommendations for site grading, retaining wall design, and other earthwork related elements of the project.

In our opinion, the site is suitable for the proposed improvements, provided the recommendations presented in this report are incorporated into the design and adhered to during construction.

If you should have any questions or need further assistance, please do not hesitate to contact this office.

Respectfully Submitted,

HENRY JUSTINIANO & ASSOCIATES

Henry Justiniano, P.E.
Calif. No. C-42347
Exp. 3/31/2012

Enclosures

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Project No. T-123-01
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1.0 INTRODUCTION

1.1 PURPOSE

This report presents the results of our field exploration of the subject property that supplement our previous geologic evaluation reports.

General engineering design and geotechnical recommendations are provided, based upon the physical and strength characteristics of the subsurface materials, and take into consideration the proposed project's requisites.

1.2 SITE LOCATION

The subject property is located on the northern periphery of Castro Valley, in Alameda County, California. Specifically, the proposed building site is located along the southern side of Proctor Road, approximately 750 feet east of its intersection with Ewing Road. The precise location is illustrated on the site location map, Figure 1.

1.3 SITE CHARACTERIZATION

The approximately 6-acre subject property spans across a south trending small knoll ridge and a swale (Figure 2). No notable cuts or fills appear present, as there are uniformly contoured slopes that conform to the natural topography. In general, the terrain offers gentle gradients, except for segments of the swale, where the sides approach 2 horizontal to 1 vertical slopes.

At the time of our exploration, the ridge was blanketed with natural grasses, while the lower reaches of the swale hosted sapling oaks and a pond-like hummocky topography that lies adjacent to a storm water culvert inlet projecting into the fill slope supporting Joseph Drive.

Runoff from the majority of the site flows through the swale discharging into the storm water inlet beneath Joseph Drive, near the southeastern property corner.

1.4 SCOPE

The scope of our work included a literature research of available and applicable geological and geotechnical data, exploratory trenches, and logging of the soils encountered during the field investigation, and geologic interpretation. Representative soil samples were retrieved during the field exploration, to be tested in the laboratory for strength and classification. The compiled soil data was analyzed in support of the recommendations presented herein.

1.5 PROPOSED IMPROVEMENTS

The current subdivision plan designates 24 lots with approximately 20-feet of fill across a swale in the south-central portion of the property. Fill material would be generated from cutting up to 10-feet, into the ridge crest and north-central portion of the swale (Figure 2). The proposed development includes street improvements with underground utilities and a detention/treatment pond at the southeastern property corner.

1.6 SUMMARY OF RESULTS

Based upon the results of our evaluations, we conclude that there are no geotechnical nor geologic considerations that would preclude the proposed development. Information from our review of geological maps, published geotechnical reports, the existing topography, and our exploration program, indicates that the desired building locations would be within acceptably stable terrain, and that the site would be feasible for construction of the proposed residences, provided that the recommendations presented herein are incorporated into the design, and adhered to during the construction phases of the project.

2.0 GEOLOGY

2.1 REGIONAL GEOLOGY

The project site is situated within the central portion of the Coast Range Province of Northwestern California. The Coast Range Province is characterized by a structural domain that is locally controlled by north to northwest trending, subparallel mountain ridges and narrow valleys. The internal structures are often complex folds that are associated with structural deformations that have been created by a compressional regime during the Middle Mesozoic through Early Cenozoic Eras.

Tectonic features of the region reflect a deep crustal, northwestward movement of the Pacific Plate, relative to the North American Plate. Surface displacement is largely recognized along the San Andreas Fault Zone. However, the plate boundary movement is distributed among several faults between the Pacific Ocean and Western Nevada. These major faults are often characterized by a series of parallel anastomosing fault splays that develop at the surface, in response to the differential subsurface movement.

Historically, the active faults in the San Francisco Bay Region are, from west to east, the San Gregorio, the San Andreas, the Hayward, and the Calaveras Faults. These faults remain locked and quiet over periods of tens to hundreds of years. During quiet periods, strain builds up by gradual deformation of the crust adjacent to the fault. This strain is relieved periodically in sudden fault displacements that produce earthquakes. The displacement for Bay Area faults is dominantly right-lateral strike slip, with minor oblique slip component movements.

2.2 SITE GEOLOGY

Previous mapping by Dibblee (1980) and Graymer et al (2000, Figure 3) indicate that the site is underlain by Cretaceous-aged sedimentary rock that Dibblee mapped as Panoche Formation and Graymer et al. mapped as the Joaquin Miller Formation, consisting predominantly of fine sandstone and minor shale. Conglomerates and sandstones of the Oakland Formation are mapped several hundred feet northeast of the site. Mapping by Graymer indicates an inactive thrust fault east of the site, and the East Chabot and the active Hayward faults, are mapped to the southwest. Bedrock attitudes are shown to predominantly strike to the northwest. Northeast of the site, the bedrock is shown to dip moderately steeply (60 to 70 degrees) northeast.

2.3 SLOPE STABILITY/LANDSLIDING

Previous mapping by the USGS (Nilsen, 1975) does not depict any landslides within the area. Official Mapping by the State of California delineating Seismic Hazard Zones (Figure 4), does not assign the subject site as on an area with a potential for earthquake induced landsliding. During our reconnaissance, we did not observe any evidence of slope instability at the site.

However, the exploration exposed rocks that are flexed and with variable dip, from near horizontal to both the southwest and northeast. Out of slope bedding was observed in several locations, as well as shallow-dipping out of slope fractures, were noted.

2.4 ACTIVE FAULTING

The property is not within the current Alquist Priolo Earthquake Hazard Zone (formerly an Alquist Priolo Special Studies Zone) of the active Hayward Fault. The previous mapping by Dibblee (1980) and Graymer et al (2000) indicate that the Hayward fault is located approximately 1.8 miles (2.9 km) west of the site. During our reconnaissance, we did not observe any geomorphic conditions within the property that would suggest the trace of an active fault extends through the site. However, based on the mapped location of the Hayward fault, there is a potential for very strong seismic shaking in the area.

2.5 SEISMICITY AND SEISMIC SHAKING

Table 1 below presents an assessment of the faults that contribute the most significant ground-motion hazard to the site. Included in the Table is the shortest distance between the site and each fault (as measured in kilometers from the surface trace projection of the fault); the maximum moment magnitude (Mw) for the Upper Bound Earthquake (UBE) estimated for each fault.

TABLE 1
FAULT DISTANCE - MAGNITUDE - ACCELERATION

Active Fault System	Distance		Upper Bounds Magnitude (Mw)
	Miles	Kilometers	
Hayward	1.8	2.9	7.1
Calaveras	7.1	11.4	6.8
Concord-Green Valley	12.9	20.8	6.9
Greenville-Marsh Creek	16.9	27.2	6.9
San Andreas (Northern)	20.2	32.5	7.9

(Mw):Estimated Moment Magnitude from CDMG (1996) Open File Report 96-08.

The Design Basis Earthquake (DBE) ground motion is defined to have a 10% chance of exceedance in 50 years (475 year return period). Development of the DBE ground motion value requires a site specific Probabilistic Seismic Hazard Analysis (PSHA). A peak ground acceleration (PGA) estimate of 0.689 for the Design Basis Earthquake (10% probability of exceedance in 50 years) is presented in the California Geological Survey's web site for a Probabilistic Seismic Hazards Assessment for the site (Figure 4). The subject area is assigned a high hazard rating, due to its proximity to several faults . . . in particular, the Hayward and San Andreas Faults.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

3.1 FIELD INVESTIGATION

On October 11, 2010, our Consulting Engineering Geologist explored the subsurface conditions in the subject property, with ten test pits. The test pit were excavated with a rubber-tire backhoe to a maximum depth of 10-feet, at the approximate location shown on Figure 2. The test pits location were established by our Consulting Engineering Geologist, who logged the exposed conditions. An attempt to explore the site with a truck mounted drill rig was abandoned, after one borehole, due to access limitations.

The logs of test pits are presented on Figures 6 thru 8, and the borehole as Figure 9. Soils are described in accordance with the Unified Soil Classification System, and bedrock descriptions in Engineering Geology, Rock Terms. The test pit log shows our interpretation of subsurface conditions at the date and locations indicated. Conditions may vary at other locations and times.

3.2 LABORATORY TESTING

Laboratory testing was performed on selected samples, in order to identify some of their engineering properties. Testing was conducted to establish Atterberg limits and sieve analyses for soil classification.

The determination of Atterberg limits is used to correlate consistency changes with moisture variation, which is indicative of the expansion potential of the soil (ASTM D-4943). Atterberg limits testing was performed on representative samples of the near surface soils. Three tests yielded liquid limits between 30 and 33, and plasticity indexes between 12 and 15, which, correspond to clays of low plasticity. An additional test performed on a representative sample of a relatively shallow, thin layer (approx. 18-inches thick) reddish colored clayey soil, from the western side of the ridge, yielded a liquid limit of 40, and a plasticity index of 22, which corresponds to a clay of moderate to high plasticity.

Sieve analyses were conducted to obtain grain size distribution and to classify the encountered near surface layers (Figure 10). In general, the grain size distribution curves, combined with Atterberg limits, classify the near surface soils as low to moderately expansive, inorganic clays.

4.0 SUBSURFACE CONDITIONS

4.1 GENERAL

The results of the exploration revealed that the upper swale regime traversing along the eastern side of the property, is underlain by less than 5-feet of alluvial fan and colluvial soil deposits, and the swale's side slopes are blanketed by a thin (less than 2-feet thick) layer of topsoil and residual soils. These near surface soils were classified as silty clays that poses low to moderate shrink/swell potential.

In the lowermost swale area, near the Joseph Drive terminus, the hummocky topography was found to be underlain by stiff gravels, in a clayey matrix. No bedrock was encountered to a depth of 10-feet.

In general, the western ridge is blanketed by less than 2-feet of silty gravel topsoil; however, a relatively thin (approx. 18-inches thick) layer of moderate to highly expansive clay was revealed in test pit No. 5. The topsoil, overlies siltstone/sandstone bedrock, which is typically weak and very closely fractured. Adverse bedding and fractures were noted in several locations.

No free groundwater was encountered to the depths explored.

4.2 LIQUEFACTION AND LURCHING

Liquefaction occurs when a loose, saturated, granular deposit changes from a solid to a liquid state, due to particle densification, and increased pore pressures, during seismic shaking. Liquefaction is a relatively rare phenomenon because three conditions must be met for it to occur: 1) the sediments must be saturated; 2) sediments must be loosely packed, allowing pore pressure to be increased by a disturbance (i.e., seismic shaking); and 3) soil particles must be of a certain size and distribution.

Official Mapping by the State of California, delineating Seismic Hazard Zones (2003, Figure 5), does not assign the subject site to an area with a potential for seismically induced settlement and liquefaction.

Our investigation indicates that clayey soils overlie sandstone/siltstone bedrock and no groundwater was encountered during the exploration. As such, we judge there is an insignificant risk with respect to seismically induced liquefaction.

Lurching and lateral spreading results from movement toward steep, unsupported embankments during seismic shaking. These conditions are not present on the subject property or adjacent areas. As such, we judge this risk to be insignificant.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

Our exploration and evaluations indicate that the near surface soils and the underlying strata, possess acceptable geotechnical characteristics to receive the proposed residential improvements. The proposed improvement plan shows mass grading to accomplish access roads to the future Lots and a storm water detention/treatment pond (Figure 2). As such, grading in the residential area will be limited to that needed to accommodate the roadways. Plans for future grading on the individual Lots, to produce building pads where they are deemed necessary, will be developed at a later date, during the home design phase of the project.

From a Geotechnical Engineering perspective, there are four noteworthy issues that will require special consideration with respect to the overall development of the property:

1. The presence of out of slope bedding attitudes and fractures of the rock, may promote slope instability. However, the fill that is designated to the swale, will offer a buttressing effect to the adjacent slopes. Nevertheless, above the fill areas, unsupported cuts are proposed that should be reviewed by our Engineering Geologist during excavations, to evaluate whether there are indications of potentially unstable materials. Cuts that are in excess of 3 to 4-feet may require support from retaining walls or to be reconstructed as engineered fill. In addition, the limits of the proposed cuts may need to be expanded laterally, in order to stabilize potentially unstable slope areas.
2. The proposed cut in the swale, near the common boundary of Lots 15 and 16, will likely undermine soft swale deposits. Minor corrective grading under the direction of the project soils engineer, should be anticipated to stabilize the swale deposits above.
3. The proposed building pad and fill designated to Lot 10 and the access road to the pond, will require an extensive keyway/over-excavation procedure to accomplish a stable fill, access road and building site (see Figure 11).
4. Per the Preliminary Plan, the detention and treatment pond banks will require minor fills and cuts. Some over-excavation of soft soil deposits should be anticipated, as it will be necessary for the fills to be keyed into firm non-yielding materials, to promote stability to the side slopes above. Although limited in volume, expansive clays from the west-central side of the ridge, may be utilized as a natural pond liner material.

As proposed, we anticipate that the mass grading will produce building sites with a wide variety of conditions. Lots 1 thru 8, will offer a gently sloping transition from cut to native grades, and relatively shallow bedrock. Building sites for Lots 9, 10 and 11, will consist mainly of engineered fill. In Lots 12 and 13, the grading will produce transitions from fill to relatively steep slopes. Lots 14, 15 and 16, will offer transitions from cuts to relatively steep, natural slopes. Lots 17, 18 and 19 will consist of relatively deep and steep cuts into bedrock. Lots 20, 21 and 22, will have relatively gentle slopes and shallow bedrock. Lot 24 and the adjacent two Lots to the west that front Proctor Road, will not be disturbed by the grading and will offer relatively gentle slopes and shallow bedrock.

Based upon the above described variable building site conditions and conceivable future localized grading to generate building pads, we can anticipate that both, conventional footing and pier and grade beam foundation systems, could be appropriate. Conventional footings may be applicable to relatively level (less than 5 horizontal to 1 vertical) building pads that offer uniform near surface bedrock. Building pads with slope gradients exceeding 5 horizontal to 1 vertical, expansive soils, fill and transitions from cut to fill, will require pier and grade beam foundations. Relatively high retaining walls combined with pier and grade beam foundations, will be required on the steeper Lots, where stepped pad configurations are anticipated.

The detention and treatment pond area has accumulated soft soil deposit during similar service, over many decades. The proposed pond will be approximately 5-feet deep, but the depth of water impoundment would be minimal. Provided that the fill and cut banks are stabilized, the pond would be benign, with respect to impacting the adjacent improvements.

The recommendations presented in this report are for the soil/rock conditions encountered during our exploration. Should other soil or rock conditions be uncovered during construction, due to non-uniformity of the geological formations, we should be contacted to evaluate the need for revision of the recommendations presented herein.

The structural design should incorporate current seismic code requirements. Seismically induced ground shaking with possible minor structural damage, should be expected to occur within the economic life of the structures. Nevertheless, the hazard of seismic shaking is shared throughout the San Francisco Bay region.

5.2 SEISMIC DESIGN

Based on the results of our investigation, we recommend that the following seismic design criteria be implemented in accordance with the California Building Code (2007):

Site Class	C
F_a	1.0
F_v	1.3
S_{ds}	1.204
S_{d1}	0.585

5.3 GRADING RECOMMENDATIONS

The initial site preparations should commence with stripping of root and organically contaminated soil from the areas to be disturbed during grading. The stripped materials may be stockpiled for beneficial use during landscaping, or hauled off the site.

It is recommended that the southern, eastern and western pond embankments be over-excavated to remove soft soil deposits and accomplish a bench-like configuration that penetrates into bedrock or firm, non-yielding soil, as approved in the field by the Engineer. The excavation must be approved by the Engineer, prior to fill placement for embankment construction. Similarly, over-excavation of soft soil deposits from the bottom of the ponds, should be performed.

Grading preparations for the main fill designated to the southern end of the subdivision, should commence with an excavation at the northern pond embankment, to remove soft soil deposits. The excavation should be extended laterally to the north until bedrock is intercepted. Initially, in the pond area, the depth of excavation is anticipated to be approximately 5-feet to penetrate into uniform, firm non-yielding materials, but will have to intercept and penetrate into the bedrock as it approaches the southern edge of the access roadway, as interpreted in the field by the Engineer. As the fill is placed in the lower excavation and commencing the fill prism upslope for the roadway and continuing into the future building site on Lot 10 and the main fill, continuous benching should be accomplished into the hillside, to remove the soil deposits and produce a base keyway / bench-like configuration that penetrates into the bedrock, as illustrated in Figure 11. The aforementioned benching and fill placement procedures should continue, extending well into the cut areas above, as directed in the field by the Engineer. The Engineer may render some soil deposits in the central swale as suitable to remain in place, provided that the fill is benched into the adjacent side slopes.

The fill slopes should not exceed 2H:1V (horizontal:vertical) gradients. Cuts should be planned with maximum gradients of 2H:1V, and should be evaluated by a Certified Engineering Geologist, to determine if remedial grading or a retaining wall is warranted.

Subdrain placement will constitute an essential factor in the stability of the lower fill slope. The precise locations, extent, and depths of subdrains should be determined in the field, by the Engineer, based upon the materials encountered and the configuration of the excavations. Conceptual subdrain locations are depicted in the attached Figure 2, and a standard subdrain detail is provided in Figure 12.

The subdrain pipes should consist of a 4-inch minimum diameter (rigid wall SDR 35 or equivalent), perforated pipe, placed at the heel base of the keyway, and surrounded with 3 cubic feet of Class II filter rock per foot of pipe. A clean-out riser should remain at one of the terminus of each subdrain that traverses the fill. The subdrain should be sloping at a minimum of 2 percent, and extend to drain at a point of daylight, at the northern pond embankment.

The engineered fill materials should be placed in thin, moisture conditioned lifts not exceeding 8-inches in uncompacted thickness, prior to receiving compaction efforts intended to accomplish a minimum 90 percent relative compaction, based on ASTM Test Procedure D1557. Moisture conditioning should accomplish between 0 and 3% above the established optimum moisture content. If the fill material contains rocks or rubble, no rocks larger than 6-inches in their greatest dimension should be allowed. Following the conclusion of the grading activity, all disturbed slope areas should be track-walked, and seeded, to mitigate erosion.

All grading operations must be under the supervision of the Engineer, in addition to the compaction testing procedures conducted by a Field Technician.

5.4 FOUNDATION DESIGN RECOMMENDATIONS

Foundation design recommendations must be based on the topography and subsurface conditions. It is anticipated that the Lots will be graded in the future, to develop building pads. Recommendations for foundations design are purposely omitted, due to the current phases of planning not yet having reached that level of planning detail. Foundation and retaining wall recommendations can be provided during the individual Lot design phases of the project, as an addendum to this report.

5.5 RETAINING WALL SUPPORTING THE FIRE TRUCK TURNAROUND

Retaining wall systems are numerous and should be customized for each application. Depending on the location, configuration and local conditions, the forces acting on a retaining wall and the available foundation bearing capacity, will vary significantly. Nevertheless, the conditions associated with the proposed retaining wall that is assigned to support the fire truck turnaround, can be predicted.

In our opinion, the most appropriate retaining wall for this condition, is a system implementing a segmental masonry block wall that derives support laterally from a reinforced earth backfill and vertically from a pier and grade beam foundation.

Soil reinforcement should be achieved by the installation of continuous sheets of geogrid with aperture geometry and rib junction cross-sections sufficient to permit mechanical interlock with the backfill soil, so as to allow the structure to be analyzed as a gravity wall. Backfill materials, placement procedures and compaction specifications should conform to the recommendations prescribed in Section 5.3 of this report.

1. External Stability

Analysis of the retaining wall structure should assume that the reinforced soil mass behaves as a rigid body. Computations assessing overall bearing, and potential external slip circles will not be required, provided that the masonry facing derives support from a pier and grade beam foundation system. Although piers will be provided, they will be rather slender, rigid, structural elements, embedded in fill, within the outer slope face, and therefore cannot carry significant lateral loads that act perpendicularly to their axis. In recognizing these limitations, we recommend that the piers be assigned only axial loads in determining the diameter required to carry the estimated axial loads. In computing the pier loading capacity, the following table summarizes our recommended design criteria:

Pier Diameter	Minimum 16-inches.
Pier Depth	Minimum of 12-feet, or as determined in the field by a representative from this office, during drilling.
Bearing Capacity	Maximum friction value of 500 psf commencing 5-feet below the existing grade. These values may be increased by 1/3 for wind and seismic loads.
Grade Beams	Minimum reinforcement of two No. 5 bars, top and bottom.

2.0 Internal Stability

Analysis of all areas relating to internal behavior mechanisms, stresses within the structure, arrangement and spacing of the reinforcement, durability of the reinforcement, and insitu soil properties, should be considered by the designer. The reinforced soil mass should extend into the slope as necessary to meet the pullout resistance of the reinforcement elements and lateral stability criteria. The number, size, strength, spacing and length of the reinforcing elements necessary to insure stability of the structure, must be determined.

3.0 Design Parameters

Because the foundation soils, the retained soils and the reinforced soils will be from a common fill material, they should be assumed to have common parameters. The following table presents the parameters that may be implemented in the analysis:

Density	125 pcf.
Phi (angle of internal friction)	27^{0}
Cohesion	0 psf.

5.6 LOT DRAINAGE

Concrete lined V-ditches will be needed along the uphill side of the Lots, to capture surface waters from the hillside above.

It will be important to divert surface run-off away from the foundation perimeter, concrete flat work, or any other improvement that is founded near the surface and is susceptible to displacements resulting from expansive soils. Downspouts should be connected to conduits that will transport their effluent to a discharge point away from structural element-bearing soils. A slope gradient of 3 percent down and away from the building perimeter, for a minimum of 5 feet, should be provided to the finish grade. Yard areas should be sloped toward catch-basins that are designated to low points.

5.7 UTILITY TRENCHES

Utility trenches that are parallel to the sides of house foundations, should be placed so that they do not extend below a line sloped down and away at a 2:1 (horizontal:vertical) slope from the bottom outside edge of the footing/grade beam.

All trenches should be backfilled with native materials compacted uniformly to a 90% relative compaction. Jetting of trench backfill should be avoided as it may result in an unsatisfactory degree of compaction.

5.8 PAVEMENTS

Based on the nature of the subgrade material that are expected, in conjunction with the anticipated traffic along the access roadway, we recommend minimum pavement sections consisting of 2-inches of Asphaltic Concrete on 8-inches of Class II Aggregate Baserock for the upper, cut into bedrock segment of roadway, and 3-inches of Asphaltic Concrete over 11-inches of Class II Aggregate Baserock for the lower, fill areas. The Engineer should establish the location of the transition from bedrock to fill, based on the conditions produced by the grading.

The performance of the final pavement will depend upon the quality of workmanship and materials. The following summarizes the recommended construction procedure to be followed:

1. Scarify the subgrade surface to a minimum of 6-inches, to properly moisture condition the soil to near the optimum moisture content, and produce a smooth-drum-rolled surface that is compacted to a minimum 95 percent of maximum dry density.
2. Provide the necessary gradient to prevent the ponding of water.
3. Place the baserock in lifts that are within the compaction capabilities of the compaction equipment, and compact to 95 percent of maximum density.
4. Place the Asphaltic Concrete during fair weather only, and at a temperature within its' prescribed limits.

6.0 GENERAL CONDITIONS

6.1 PLAN REVIEW

Prior to the submission of design drawings and construction documents for approval by the appropriate local agency, copies of these documents should be reviewed by our firm to evaluate whether or not the recommendations contained in this report have been effectively incorporated into the design of the project.

6.2 CONSTRUCTION OBSERVATIONS

A representative of this firm must be present during grading of the site. This item is necessary to properly evaluate the quality of the materials and their relative compaction. Foundation excavations must be inspected by a representative of this firm, in order to make the necessary adjustments as a result of localized irregularities.

At the completion of the earthwork related construction, a report will be submitted summarizing our observations, including the results of the compaction testing program.

To allow for proper scheduling, we request a minimum of 48 hours notice prior to the commencement of earthwork operations requiring our presence.

6.3 LIMITATIONS

This report has been prepared by HENRY JUSTINIANO & ASSOCIATES for the exclusive use of Mr.Hue Tran and his representatives, for consideration of the proposed improvements to the property described in this report.

The interpretations and recommendations presented in this report are professional judgements, and are based on our evaluations of the technical information obtained during this investigation, on our understanding of the characteristics of the planned improvements to the structure, and on our general experience with similar subsurface conditions in other areas. We do not guarantee the performance of this project in any respect, only that our engineering work and judgements meet the standards of care normally exercised by our profession.

It is assumed that the test pits are representative of the subsurface conditions throughout the areas designated to receive improvements. Unanticipated soil conditions are commonly encountered and cannot be fully determined by performing exploratory borings. If, during construction, subsurface conditions different from those indicated in this report, are encountered or appear to be present beneath excavations, HENRY JUSTINIANO & ASSOCIATES should be advised at once so we can review these conditions and reconsider our recommendations, when necessary.

If more than 18 months have elapsed between the submission of this report and the start of work at the site, or if conditions have changed because of natural causes or construction operations at or adjacent to the site, we recommend that this report be reviewed to determine the applicability of the conclusions and recommendations, considering the time lapse or changed conditions.

The scope of our services did not include an environmental assessment, or an investigation of the presence or absence of hazardous, toxic or corrosive materials in the soil, surface water, groundwater, or air, on, below, or around this site.

REFERENCES

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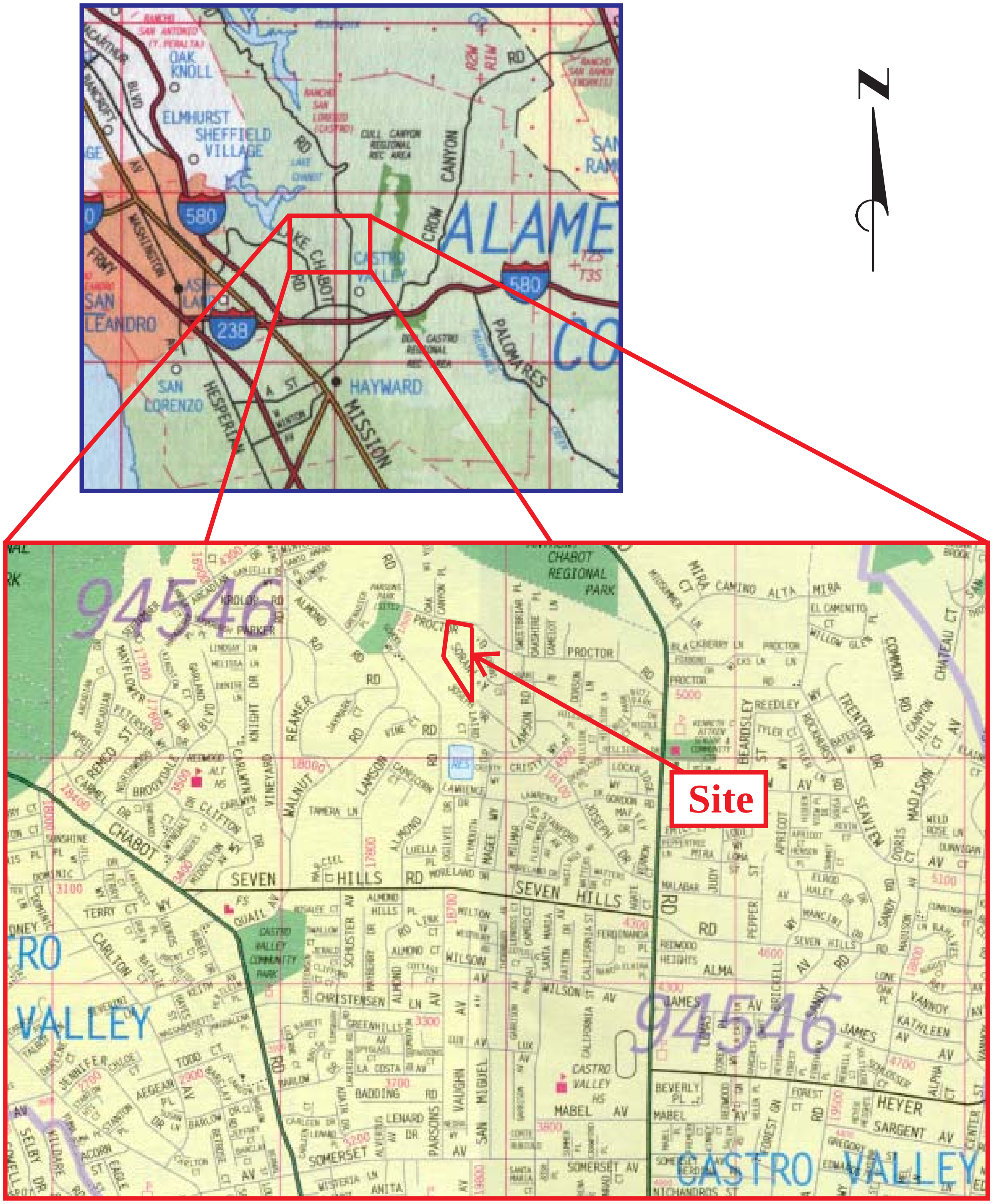
California Geological Survey (CGS), State of California Seismic Hazard Zones, Hayward Quadrangle Official Map, July 2, 2003.



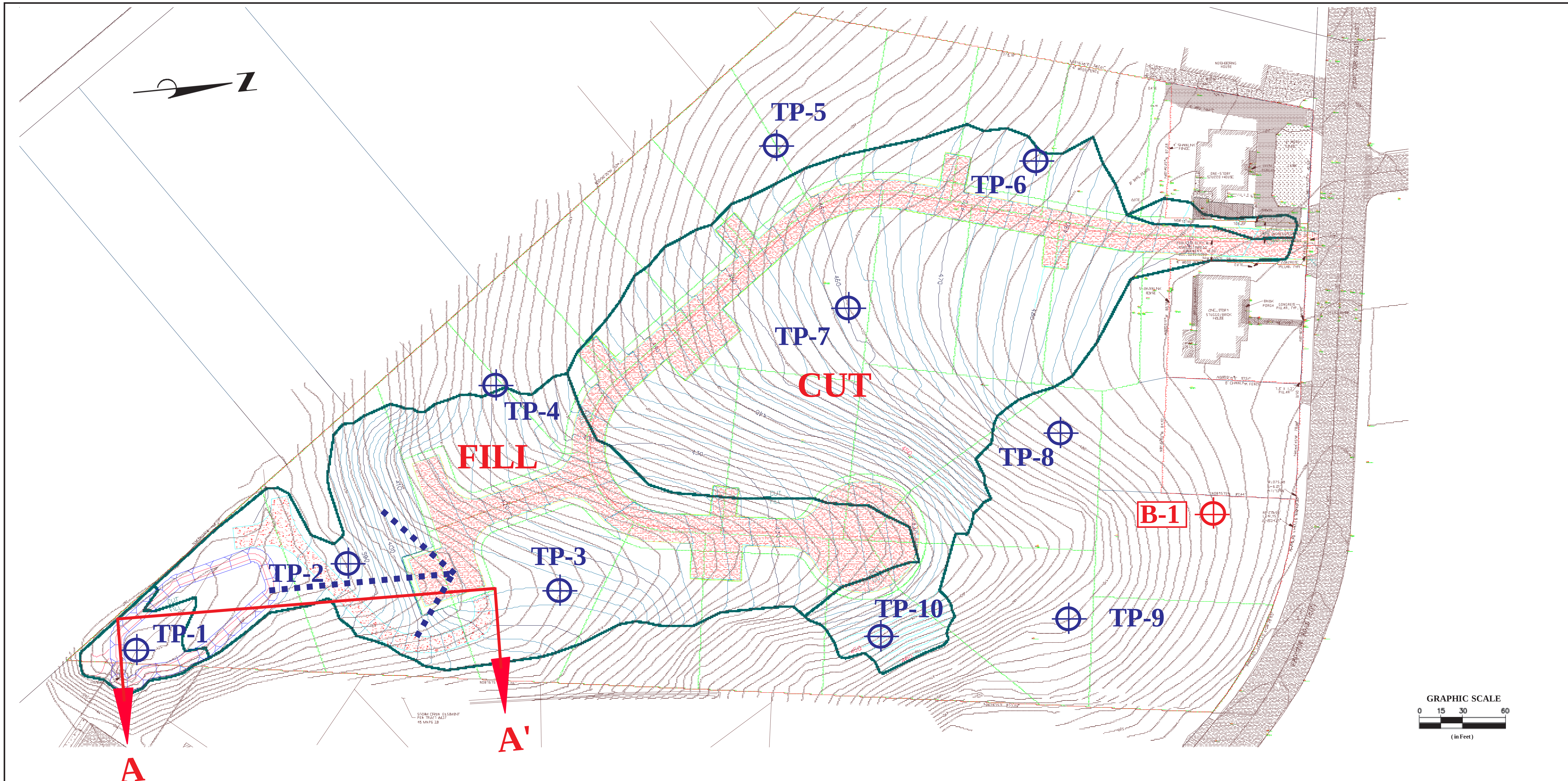
Project No.: T-123-01	Date: 11-04-10	Scale: NTS

SITE LOCATION

Source: Thomas Guide, 1999
Google © 2007

Henry Justiniano & Associates
Soils and Foundation Engineering

Figure No. 1



Explanation

- Approximate Location of Test Pits
- Approximate Location of Borehole
- Subdrain

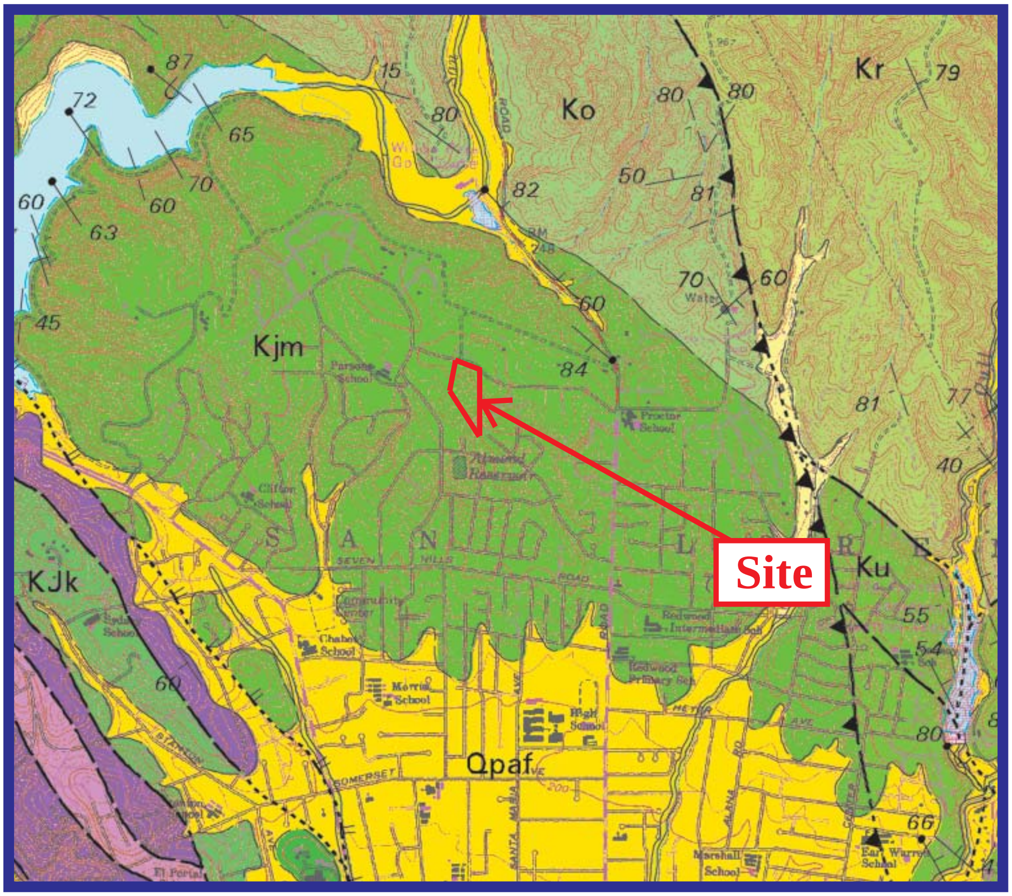
SITE PLAN

Source: Lea & Braze Engineering, Inc.

Project No. T-123-01	Date: 11-04-10	Scale: As Shown

Henry Justiniano & Associates
Soils and Foundation Engineering

Figure No. 2



EXPLANATION

Qpaf	Alluvial fan and fluvial deposits (Pleistocene)	Jsv	Keratophyre and quartz keratophyre (Late Jurassic)
Kjm	Joaquin Miller Formation (Late Cretaceous, Cenomanian)	Jgb	Gabbro
Ko	Oakland Conglomerate (Late Cretaceous, Turonian and/or Cenomanian)	Kr	Redwood Canyon Formation (Late Cretaceous, Campanian)
		Ku	Unnamed sedimentary rocks (Late Cretaceous, Turonian and Cenomanion)

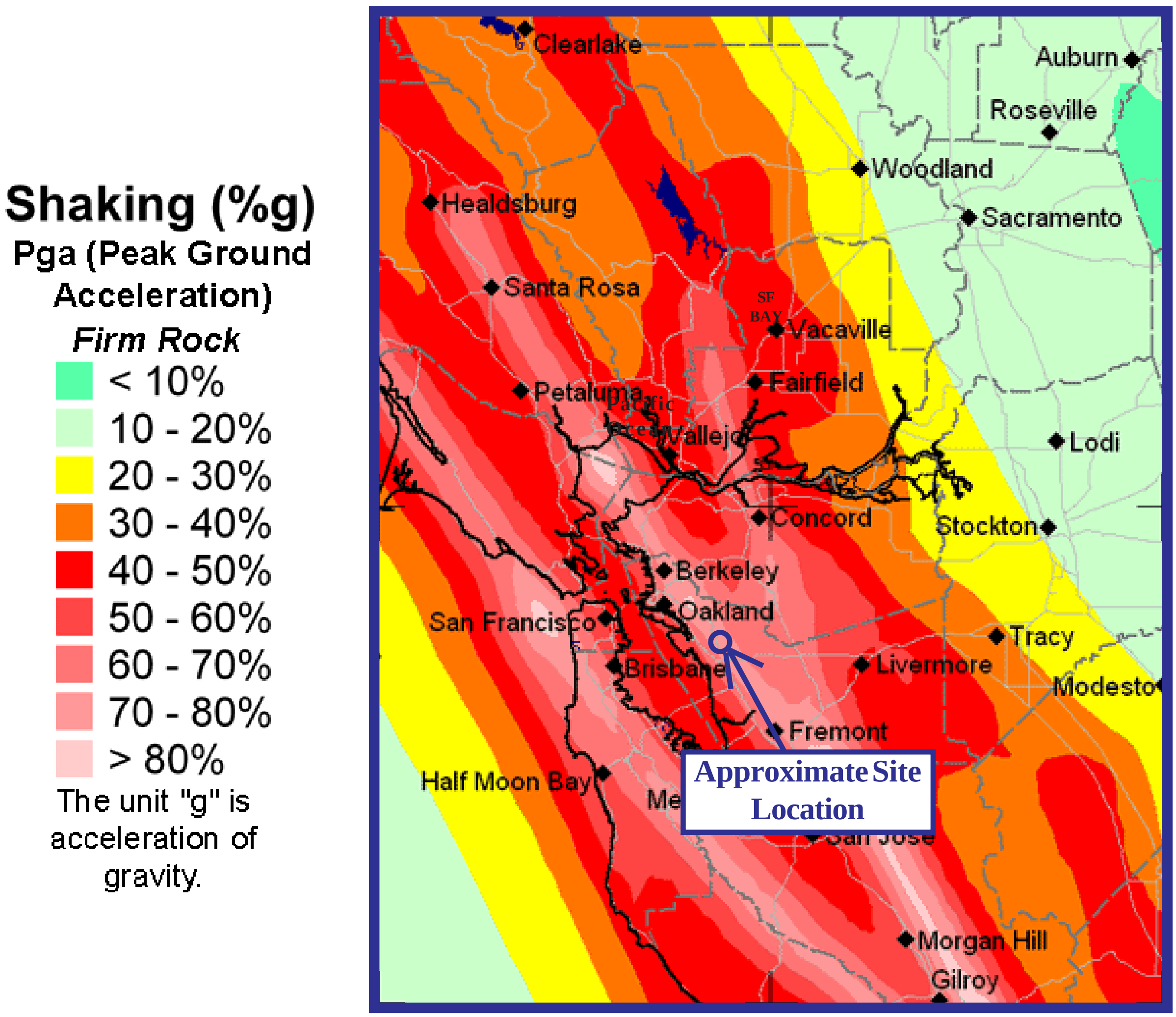
GEOLOGY MAP

R.W. Graymer, 2000

Project No.: T-123-01	Date: 11-04-10	Scale: NTS

Henry Justiniano & Associates
Soils and Foundation Engineering

Figure No. 3



PROBABILISTIC SEISMIC HAZARD MAP (Modified)

(10% Probability of Exceedance in 50 Years)
Peak Horizontal Ground Acceleration
Firm-Rock Site Condition

Based on the USGS/CGS Probabilistic Seismic Hazards Assessment (PSHA)
(revised 2003)

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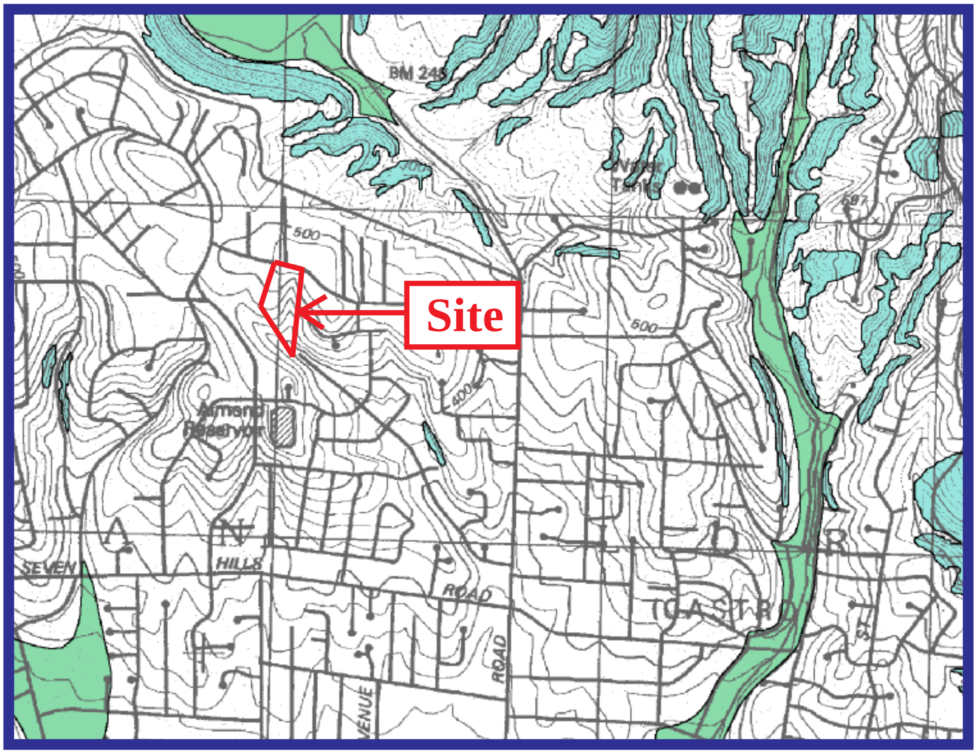
Soils and Foundation Engineering

Figure No. 4



N

EXPLANATION

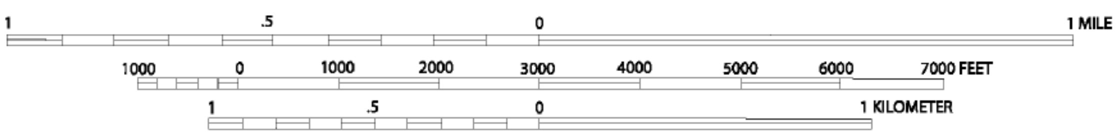
Liquefaction
Areas where historical occurence of liquefaction, or local geological, geotechnical and ground-water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

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.

STATE OF CALIFORNIA SEISMIC HAZARD ZONES

HAYWARD QUADRANGLE OFFICIAL MAP

RELEASED JULY 2, 2003 (MODIFIED)



Scale 1:24,000

Project No.: T-123-01	Date: 11-04-10	Scale: As Shown

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Figure No. 5

TEST PIT LOGS

Test Pit No.	Depth (Feet)	Description
TP 1	0.0-3.5	Dark Gray/Brown Silty Clay (CL); soft; very wet; porous with abundant organics.
	3.5-7.1	Dark Gray/Brown Sandy Clay (CL); with occasional fine gravels; wet; soft to medium stiff.
	7.1-8.9	Becoming more gravelly; grading to Orange/Brown; gravels semi-rounded; stiff; wet.
	8.9-10.0	Orange/Brown to Brown Clayey Gravel to Gravelly Clay; stiff; wet.
TP 2	0.0-1.4	Gray/Brown Silty Sand (SM); occasional gravels; with roots and organics; loose to medium dense; moist; porous.
	1.4-2.5	Yellow/Brown Silty Clay (CL); medium stiff; moist; residual soil.
	2.5-4.6	Gray/Yellow/Brown Silty Gravel (GM); medium dense to dense; moist; deeply weathered rock.
	4.6-5.2	Gray/Brown Interbedded Siltstone and Sandstone; very closely fractured; weak?; bedding dipping @ 17 deg. Into slope, with near vertical fractures.
TP 3	0.0-1.5	Gray/Brown Silty Sand (SM); occasional gravels; with roots and organics; loose to medium dense; moist; porous.
	1.5-4.1	Gray/Brown Sandy Clay (CL); medium stiff; moist.
	4.1-6.2	Gray/Yellow/Brown Gravelly Slit (ML) to Silty Gravel (GM); medium dense to dense; moist; residual soil.
	6.2-8.1	Yellow/Brown Clayey Gravel (GC); dense; moist to wet; deeply weathered rock.
	8.1-9.5	Increasing rock fragments with faint rock structure; dense

Figure 6

TEST PIT LOGS

Test Pit No.	Depth (Feet)	Description
TP 4	0.0-1.4	Brown Silty Gravel (GM); medium dense; dry; with roots; porous.
	1.4-2.2	Brown Silty Gravel (GM); abundant rock fragments; dense; dry; deeply weathered rock.
	2.2-4.3	Yellow/Brown Sandstone; very closely to moderately fractured; weak to moderately strong; bedding N60W, 35SW.
TP 5	0.0-0.8	Brown Silty Gravel (GM) to Gravelly Silt (ML); loose to medium dense; dry; with roots and organics; porous.
	0.8-2.0	Red/Brown Gravelly Clay (CL);stiff; moist; some expansion cracks.
	2.0-3.3	Gray/Yellow/Brown Silty to Clayey Gravel (GM-GC); dense; moist; residual soil and deeply weathered rock.
	3.3-4.2	Yellow/Brown Interbedded Siltstone and Sandstone; very closely fractured; weak.
TP 6	0.0-1.3	Brown Silty Gravel (GM); medium dense; dry; with roots and organics; porous.
	1.3-3.7	Yellow/Brown Interbedded Siltstone and Sandstone; very closely fractured; weak; bedding show slight flexure; near horizontal to slightly out of slope.
TP 7	0.0-1.2	Brown Silty Gravel (GM); medium dense; dry; with roots and organics; porous.
	1.2-3.2	Yellow/Brown Interbedded Siltstone and Sandstone; very closely fractured; weak; bedding orientation indistinct.

TEST PIT LOGS

Test Pit No.	Depth (Feet)	Description
TP 8	0.0-0.8	Brown Silty Gravel (GM); medium dense; dry; with roots; porous; abundant weathered rock fragments.
	0.8-1.4	Yellow/Brown Silty Gravel (GM); abundant rock fragments; dense; dry; deeply weathered rock.
	1.4-2..3	Yellow/Brown Sandstone; very closely to moderately fractured; weak to moderately strong.
TP 9	0.0-2.4	Brown Silty Gravel (GM); medium dense; dry; with roots; porous.
	2.4-3.1	Orange/Brown Silty Gravel (GM); abundant rock fragments; dense; dry; deeply weathered rock.
	3.1-4.4	Orange/Brown Sandstone; very closely to moderately fractured; weak to moderately strong; bedding subhorizontal; slightly flex to out of slope.
TP 10	0.0-0.5	Brown Silty Gravel (GM); medium dense; dry; with roots; porous.
	0.5-4.6	Brown Gravel (GM); abundant rock fragments; dense; dry; deeply weathered rock.
	4.6-7.2	Gray/Brown Siltstone and Sandstone; very closely to moderately fractured; weak to moderately strong; bedding N30W, 53NE; Prominent fracture set 1, N70E, vertical; fracture set 2, N30W, 40SW (out of slope with clay films on fractures).

Figure 8

Exploration Boring Log by:

Henry Justiniano & Associates

Boring Log No.: B-1

Project: Proctor Road

Client: Tran

Date Drilled: 10/11/10

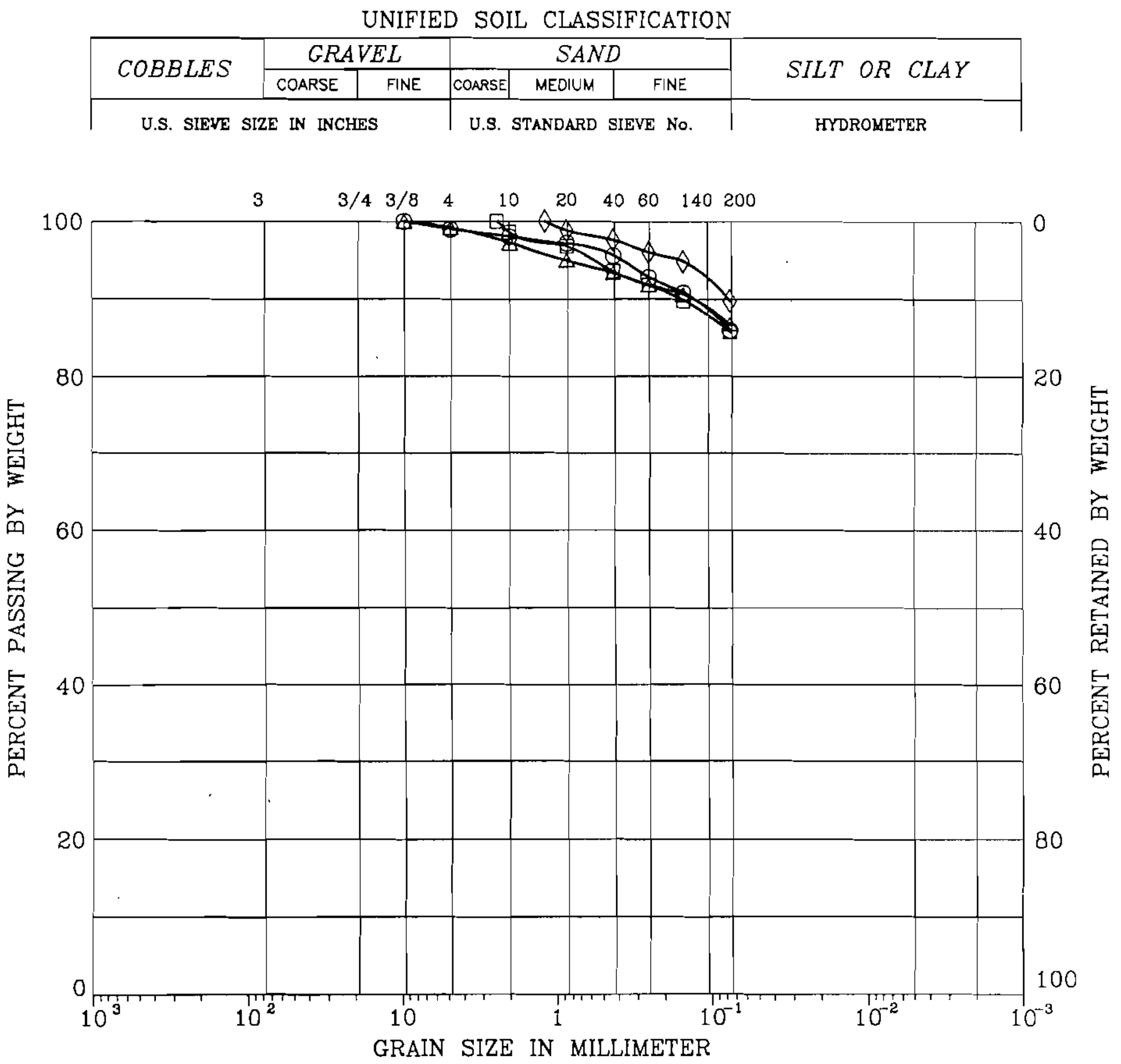
Equipment Used: Mobile Drill, 140 Lb., 30 inch Drive, 4.5" Continuous Flight, Samplers As Noted.

Location: 48' S of Proctor Road, 99' E of Fence

Depth (in Feet)	Other Laboratory Tests	Dry Density (pcf)	Moisture Content %	Blow Count per 12 inch Drive	Sample Number & Type	Groundwater	Description of Material
0–3.5							Tan/Grey Silty CLAY Dry, Stiff
3.5–4.5				65	B-1-A SPT		SANDSTONE BEDROCK Borehole Terminated @ 4.5 Feet

Depth scale: 1, 2, 3, 4, 5, 6, 7, 8, 9, 10

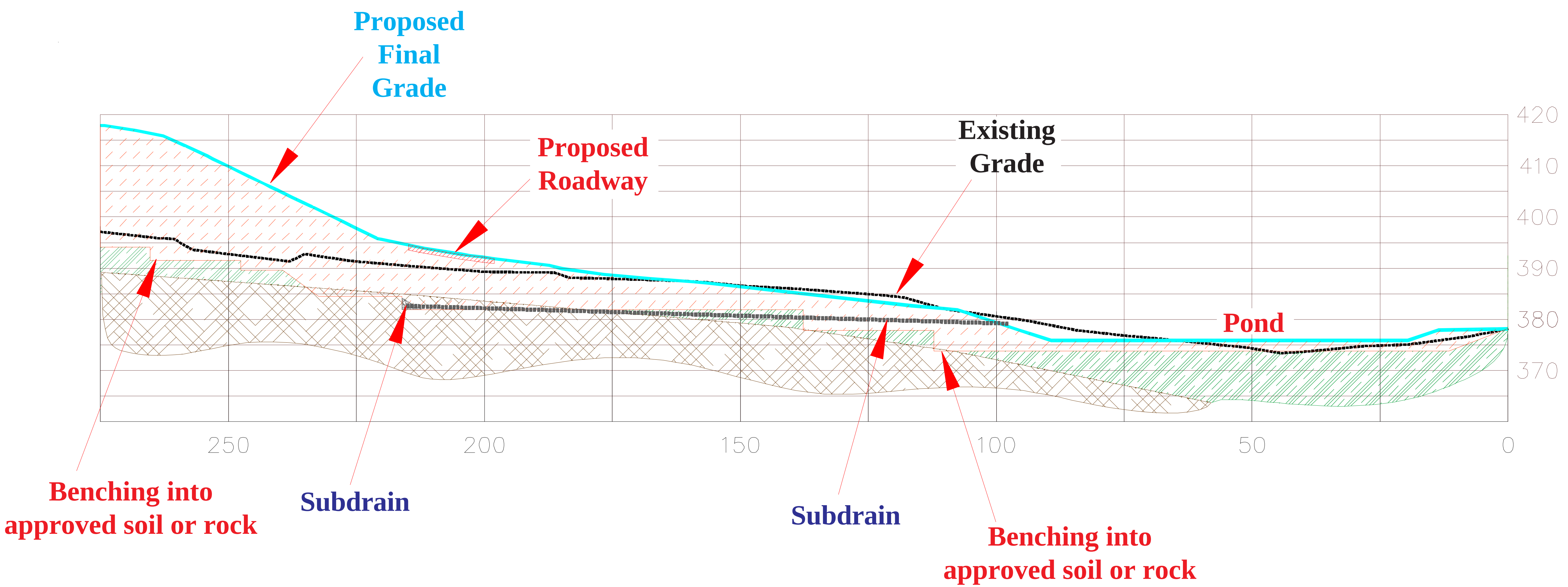
Figure No. 9



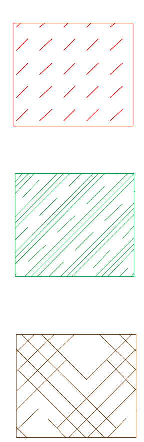
SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
○	TP-1	3.5	30	12	INORGANIC CLAYS (CL)
□	TP-2	1.5	32	14	INORGANIC CLAYS (CL)
△	TP-3	2	33	15	INORGANIC CLAYS (CL)
◇	TP-5	1.5	40	22	INORGANIC CLAYS (CL)

Remark :

Project No.T-12301	Proctor Rd., Castro Valley
H. Justiniano And Associates	GRAIN SIZE DISTRIBUTION Figure No. 10



Legend



Proposed Engineered Fill

Stiff Natural Deposits

Bedrock

X-Section A - A'
Conceptual Keyway and Over-excavations

Project No. : T-123-01	Date: 11-04-10	Scale: As Shown

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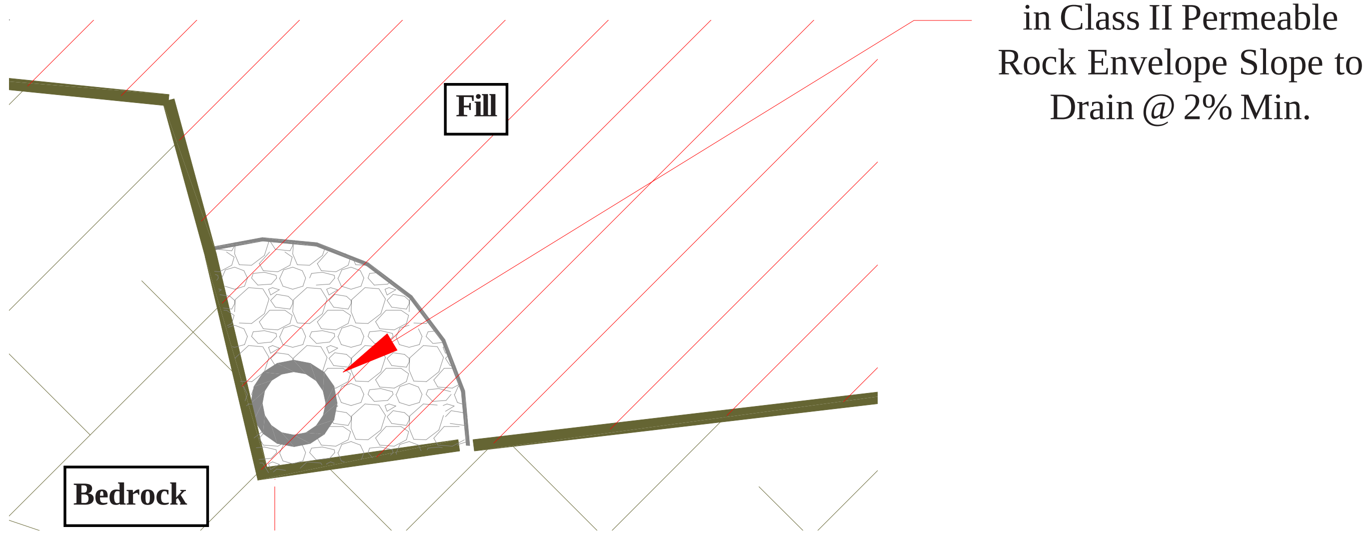
Figure No. 11

Note:

Keyway Excavation and Subdrain Placement Must be Approved by the Soils Engineer Prior to Commencement of Fill Placement



SUBDRAIN PLACEMENT ON SLOPE

(TYPICAL)

Project No. T-123-01	Date: 11-04-10	Scale: NTS

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Figure No. 12