APPENDIX G

GEOTECHNICAL INVESTIGATION REPORT AND UPDATES, HENRY JUSTINIANO & ASSOCIATES, AUGUST 10, 2015

HENRY JUSTINIANO & ASSOCIATES
GEOTECHNICAL ENGINEERING

August 10, 2015
Project No. C-149-03

Crawford Development Inc.
ATTN: Mr. Mark Crawford
P.O. Box 2151
Castro Valley, CA 94546

SUBJECT: GEOTECHNICAL INVESTIGATION REPORT AND UPDATES
Proposed 31 Single Family Residences
3231 & 3247 D Street, Tract 8296
3289 & 3291 D Street, Tract 8297
Hayward, California

Dear Mr. Crawford:

As requested, we present herein the results of our site explorations and the review of published geologic maps, as well as the review of previous geotechnical reports prepared by Geotechnical Engineering Inc., (GEI) and United Soil Engineering, Inc., (USE), along with peer review comments from Engeo Inc., that addressed an earlier development concept for Tact No. 8297. As such, this report includes updates to the previous geotechnical reports prepared by GEI and expands the study area to incorporate Tact No. 8296. In addition, this report presents our recommendations for street improvements, house foundation and retaining wall designs, as well as other earthwork related elements for the development of the two subject Tracts.

In our opinion, the properties are suitable for the proposed residential development, 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

REGISTERED PROFESSIONAL ENGINEER
HENRY JUSTINIANO
C042347
CIVIL
STATE OF CALIFORNIA

Henry Justiniano, P.E.
Calif. No. C-42347
Exp. 3/31/2016

Reviewed by: Donn Ristau, Ph.D., C.E.G. 1155
Enclosures

P.O. Box 2338 * San Ramon, CA 94583
(925) 831-9092 * e-mail - justapiano@sbcglobal.net

TABLE OF CONTENTS

1.0 INTRODUCTION

1.1 PURPOSE

This report presents the results of our investigation of the subject properties, along with the review of the published geological data pertaining to the general area and site specific geotechnical 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 properties are located in a section of the Hayward Hills that corresponds to the unincorporated Fairview District, of Alameda County. Specifically, the sites lie along the southern side of "D" Street, approximately 900-feet to the northeast of its intersection with Maud and Fairview Avenues. The approximate location is illustrated on the site location map, Figure 1.

1.3 SITE CHARACTERIZATION

The subject two Tracts have their layout partitioned near the center of the project, by a wedge-like shaped property that serves as a care facility (see Figures 1 and 2).

The eastern section of the project is designated Tract No. 8297 (Figure 3). It has an approximate area of 5.25 acres, with a higher elevation relative to Tract No. 8296 and hosts two older single family dwellings.

The western side of the project is designated Tract No. 8296 (Figure 4). It has an approximate area of 5 acres and at the time of our explorations, was mostly vacant with short natural grasses.

Topographically, the upper Tract (8297) offers a ridge-crest environment with a faint saddle-like feature near its center. From the saddle area, a broad swale projects downward to the east, with a slight increase in vegetation and somewhat hummocky appearance. Further eastward, there are single family residences belonging to a neighboring subdivision. To the west, the ridge is abruptly interrupted by a steep slope that is supported at the base, by a 5 to 12-feet high retaining wall.

The lower Tract (8296) is smoothly contoured, gently sloping to the southeast with a gradient of approximately 6 horizontal to 1 vertical.

1.4 SCOPE

The scope of our work included a literature research and review of available and applicable geological and geotechnical data, exploratory test pits, sample collection, laboratory testing and logging of the foundation soils encountered during the field investigation. The soil data compiled was analyzed in support of the recommendations presented herein.

1.5 PROPOSED IMPROVEMENTS

In accordance with the information furnished to this office, it is proposed to perform mass grading, establish street improvements and construct thirty one, wood-framed, single family dwellings.

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 residential improvements. Information from our review of geological maps and exploration program, indicates that the desired building locations are within stable terrain and that the site would be feasible to receive the proposed thirty one 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 SITE GEOLOGY

Previous mapping by Graymer (2000, Figure 5) depicts the site as being within a unit of Late Cretaceous sedimentary rocks described as the Oakland Conglomerate. This unit is shown to be in thrust-faulted with un-named sandstone, conglomerate and shale of the Castro Valley area. To the southeast, the Oakland Conglomerate is shown to be in depositional contact with the Joaquin Miller Formation. During our subsurface exploration, the bedrock unit that was frequently encountered consisted of a yellow/brown, weak to moderately strong, sandstone. Rocks characteristic of conglomerate, were not encountered. This is consistent with previous geologic investigations that have been performed on the property. The sandstone did not display obvious bedding and in only one Test Pit was a prominent fracture orientation noted. Structural orientations shown on Graymer's map to the south, indicate variable strike with dips ranging from 25 to 60 degrees. The orientations suggest that the folds are folded.

2.2 LANDSLIDING / SLOPE STABILITY

Nilsen (1975, Figure 6) mapped a series of colluvial and/or alluvial fan deposits within the lower slopes of the southwest portion of the project site. This appears consistent with the subsurface conditions encountered in Test Pits 2, 5, 7 and 8 where the depth to rock or deeply weathered rock (residual soil) was substantially deeper than in other portions upslope of these areas. Landslides have not been mapped previously within the site. However, a large swale within the northeastern portion of the site, where previous subsurface explorations were performed, apparently contains deep soil deposits (13-14 feet) and the topography appears irregular and possibly may contain old slide deposits. Areas where clayey sands were encountered in the test pits were moist and may be subject to creep (a gradual, downslope soil movement).

2.3 FAULTING/SEISMICITY

The site is not within a current Earthquake Hazard Zone (formerly Alquist - Priolo Special Studies Zone) and during our reconnaissance, we did not observe geomorphic evidence suggestive of active faulting within the site. However, the subject area is assigned a high seismic rating, due to its proximity to several faults . . . in particular, the Hayward Fault.

Table I 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) and the maximum moment magnitude (Mw) for the Upper Bound Earthquake (UBE) estimated for each fault.

TABLE 1
FAULT DISTANCE - MAGNITUDE

Fault System	Distance		Upper Bounds Magnitude (Mw)
	Miles	Kilometers	
Calaveras	6.3	10.1	6.8
Concord-Green Valley	14.6	23.5	6.9
Hayward	1.4	2.3	7.1
San Andreas (Northern)	19.9	32.0	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 0.685, 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 7).

2.4 GEOLOGICAL HAZARDS

Mapping by the California Geological Survey (2012, Figure 8) for the State of California Earthquake Zones of Required Investigation, does not include the subject site within an area labeled as potentially susceptible to earthquake induced landsliding.

Based on the relatively shallow depth to rock and limited soil cover, we consider the risk of slope instability affecting the project site to be low and specific mitigation measures do not appear to be warranted.

Other risks related to the potential for strong seismic shaking include liquefaction, densification, lateral spreading, lurching and seismically induced slope failure. Based on the hillside building envelope locations and the bedrock lithologies the risks of liquefaction and densification are considered to be insignificant. Likewise, there are no steep, unsupported banks that potentially could be influenced by lurching or lateral spreading. Seismically-induced slope failure may occur in hillside areas, especially when sites are in close proximity to earthquake epicenters. Based on the relatively gentle nature of the site topography and shallow depth to relatively strong rock, we consider that this risk would be insignificant and far below the range of acceptability that would commonly be associated with hillside construction in the Hayward Hills area.

3.0 FIELD INVESTIGATION AND LABORATORY TESTING

3.1 FIELD INVESTIGATION

On July 10, 2015, our Certified Engineering Geologist explored the subsurface conditions in the western Tract with eight test pits and one test pit on the eastern Tract. The test pits were excavated with a track mounted excavator to a maximum depth of 7.3-feet, at the approximate location shown on Figure 2. The test pit locations were established by our Consulting Engineering Geologist, who logged the exposed conditions. Our explorations also served to complement/confirm the conditions reported in previous geotechnical investigations, performed by others.

The logs of the test pits performed by this office, are presented on Figures 9 thru 11. The logs of test pits and borings performed by GEI, are provided in Appendix A, at the back of this report. Soils are described in accordance with the Unified Soil Classification System, and bedrock descriptions in Engineering Geology, Rock Terms. Our test pit log show 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 and creep potential of the soil (ASTM D-4943). Atterberg limits testing was performed on a representative near surface samples of the soils. The testing yielded a liquid limits of 32 and 42 with a plasticity indexes of 19 and 27, which corresponds to moderate to highly expansive and creep susceptible clays.

Sieve analyses were conducted to obtain grain size distribution and to classify the encountered stratigraphic layers (Figure 12). In general, the grain size distribution curves, combined with Atterberg limits, classify the near surface soils as silty clays.

4.0 SUMMARY OF DATA FROM PREVIOUS GEOTECHNICAL STUDIES AND PEER REVIEWS (TRACT 8297)

4.1 REPORTS BY GEI Inc.

An "Updated Report, Preliminary Soil Investigation" (2006) and "Final Report - Additional Investigation Including Incorporation of Subsurface Data From Preliminary Investigation" (2007), prepared by GEI, for the eastern, Tract 8297, were available for our review. The report documents seven borings and three test pits, along with some laboratory tests results. Their findings are summarized as "merely from a geotechnical standpoint the site would be suitable for construction of the planned residence." It then goes on to recommend that the fill encountered in the easterly projecting swale be "subexcavated, keyed into underlying competent rock, backfilled and properly compacted." It also recommends the use of pier and grade beam foundations. The maximum recommended slope gradient for cut and fill slopes is 2 horizontal:1 vertical.

4.2 REPORTS BY USE Inc.

In a "Geotechnical Clarifications" letter dated November 17, 2008, United Soil Engineering, Inc. refers to a September 2008, submittal to Alameda County, of a Geotechnical Engineering of Record affirmation, for the previous project. In addition, USE proposes the use of piers to support a retaining wall and minimize the impacts and stability of the slope and existing retaining wall, along the western property boundary, in consideration of an existing 5 to 12 feet high retaining wall on the adjacent property. Subsequently, in November 2008, USE presents a "Grading and Drainage Plan Review of Tentative Tract Map 7303." In February 2009, USE presents a "Geotechnical Clarifications" report that presents the results of stability analysis computations for the proposed improvements along the western property boundary.

4.3 PEER REVIEW COMMENTS

In their first "Geotechnical Review," Engeo Inc., presents comments that relate, primarily to existing and proposed fills, drainage and stability of slopes. In their second review, most of the items remain unresolved. In the third review, most items remain unresolved and some input from USE is mentioned. The fourth review, Engeo expresses concern that a USE stability analysis is incomplete and additional keyways and subdrains are warranted. On the fifth review of March 2009, Engeo acknowledges their review of pressure diagrams provided by USE and other miscellaneous items that were pending and approves the project.

4.4 SUMMARY OF PEER REVIEW PROCESS AND IMPACTS ON CURRENT PROJECT

The previous project presented complications with regard to the designation of fill to the top of a rather steep configuration along the western property boundary that is common with the neighboring care facility. The care facility's buildings are very close to a retaining wall with a height of 5 to 12 feet that is followed by a relatively steep slope. The current project does not propose fill or any other disturbance to this area (Figure 3).

A minor fill and relatively soft soils in a swale area located in the east-central area of the Tract, will require sub-excavation, keyways and subdrains, prior to fill placement to achieve the proposed pad grades. The required subdrain outflow presented complications due to its depth. Following discussions and design revisions, it was determined that the subdrain could be connected to the storm system. The current project proposes less but similar depth of fill to establish building pads with similar elevations to the previous design, hence, there will be a need to find an appropriate solution to this subdrain outflow.

According to a Plate labeled G1, prepared by GEI, other minor fills are present on the site. Nevertheless, the new design (Figure 3) shows relatively deep cuts to considerable portions of the site, including the areas that have been documented as having "undocumented" fills. It is therefore safe, to assume that all existing fills will be removed.

5.0 CONCLUSIONS AND RECOMMENDATIONS

5.1 GENERAL

Our investigation and the data gathered for the project site, indicate that stable bedrock materials can be accounted for at relatively shallow depths. No geological hazards were disclosed and the California Geological Survey (CGS) mapping does not assign the site as having a risk of earthquake induced landslide hazard. From a geotechnical engineering viewpoint the following items are the main considerations for the development of the project:

1. The is a need to over-excavate fill, soft soils deposits and residual soils from the area of Lots 4 thru 6, in Tract 8297. Subsequently, a subdrain will be required as delineated in Figure 3 and extended to daylight. The design elevations are similar to the previous development's conceptual plan and the subdrain was connected to a storm line. Engineered fill would then be placed to accomplish the pads for Lots 4 thru 6.

2. The excavations along the property boundary common to the care facility, for a proposed 5-feet high retaining wall that is designated to the top of a cut slope, along the rear of Lots 1 thru 3, or east side of Tract 8296 (Figure 4), could destabilize the existing retaining walls immediately above. In addition, due to the overall height of the retained soil and the steepness of the ground in front of the proposed new wall, the design of these walls will require that the combined pressure from the two walls be considered as being transmitted to large diameter and relatively deep piers.

3. Mechanically Stabilized Earth Wall systems (MSE Walls) should be considered for the retaining walls proposed to the base of up to 20 feet of fill, along the western side of Tract 8296, designated Lots 10 thru 15. The system will no doubt prove cost efficient, esthetically pleasing and allow for continuation of the planned fill placement above the walls.

4. As proposed, a majority of the building pads will be excavated to a significant depth, such that we can anticipate that they will expose the underlying sandstone at the pad surface. However, some will be established by a significant fill thickness. As such, we believe that it is appropriate to have two different foundation systems to support the proposed residences. The cut pads exposing bedrock at the surface, would be adept to conventional footing foundations, while the fill pads should implement cast-in-place concrete piers, integrated with grade beams.

Generally, grading is most economically performed during the summer months when on-site soils are usually dry of optimum moisture content. Delays should be anticipated in site grading performed during the rainy season or early spring, due to excessive moisture in on-site soils. Special and relatively expensive construction procedures should be anticipated if grading must be completed during the winter and early spring.

In order to avoid saturation of foundation bearing soils resulting from surface flows, the drainage at each Lot must be planned so that the foundations are not allowed to saturate, and no ponding of water takes place near the foundation.

Detailed recommendations regarding grading, foundation design criteria and other pertinent considerations, are presented in the following sections of this report.

The recommendations presented in this report are for the soil conditions encountered in 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.

Based on the available geologic maps, it is our opinion that the subject site is not located astride an active fault. It must be understood by the owners, that all risk of geologic hazards cannot be eliminated, due to uncertainties of geologic conditions and unpredictability of seismic activity in the Bay Area. The structural design should incorporate current seismic code requirements. Seismically induced ground shaking with possible structural damage, should be expected to occur within the economic life of the structure. Nevertheless, the hazard of seismic shaking is shared throughout the 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 (2013):

Site Class	B
S_{ds}	1.428
S_{d1}	0.588

5.3 GRADING RECOMMENDATIONS

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

Subdrain placement will constitute an essential factor in the stability of any fill slope. The precise locations, extent, and depths of subdrains should be determined in the field, by the soils engineer, based upon the materials encountered and the configuration of the excavations. A conceptual subdrain location is depicted in the attached Figure 3.

In Tract 8297, grading procedures should commence with an over-excavation of fill, soft soils deposits and residual soils from the area of Lots 4 thru 6. The excavation is anticipated to be approximately 12-feet deep and should penetrate into and expose a uniform surface of firm non-yielding materials, as interpreted in the field by the Engineer. Subsequently, a subdrain pipe should be provided at the heel-base of the excavation or in a trench that is excavated through approved compacted fill and into the bedrock. The subdrain should consist of a 4-inch minimum diameter (rigid wall SDR 35 or equivalent), perforated pipe that is covered by Class II permeable rock that adheres to Caltrans specifications. A clean-out riser should be provided at a minimum, at one of the terminus of each subdrain that traverses a fill. The subdrain outlets should be provided at the low point, and may be daylighted on slope surfaces, since only minor volume of water effluent is anticipated.

As the fill materials are placed commencing the fill prism upslope, a continuous benching should be established into the hillside. The fill and cut slopes should not exceed a 2 horizontal:1vertical gradient.

The engineered fill materials should be placed in thin, moisture conditioned lifts not exceeding 8-inches in uncompacted thickness, prior to receiving compaction efforts to accomplish a minimum 90 percent relative compaction, based on ASTM Test Procedure D1557. If the fill material contains rocks or rubble, no rocks larger than 6-inches in their greatest dimension should be allowed. On-site materials are suitable for fill provided that they are free from organic matter or other deleterious substances. 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 FOUNDATIONS

5.4.1 Foundations in Cut Pads

In excavated, level building pads that expose bedrock materials at the surface, geotechnical conditions would be acceptable for implementation of conventional strip footing foundations that are structurally integrated to slab-on-grade floors. All footings should be at least 12-inches in width, and should have their bases located no less than 18-inches below the lowest adjacent finished subgrade. Footings constructed to the given criteria, may be designed for an allowable bearing capacity of 2,000 psf for dead load, and 2,500 psf for dead load plus live load condition. These values may be increased by one-third to accommodate short duration seismic or wind loading conditions.

The footings should contain steel reinforcement over their entire length, with reinforcement as directed by the project Structural Engineer. In no case, however, should the exterior footing contain less than two No. 5 reinforcing bars, both top and bottom.

All slabs should be a minimum thickness as set forth by the Structural Engineer, but should not be less than 5-inches thick, and reinforced by a minimum of No. 4 bars, spaced at 18-inches each way, and centered within the entire slab.

5.4.2 Foundations in Fill Pads

It is recommended that where level building pad grades have been established by the placement of fill, a foundation system that employs drilled, cast-in-place reinforced concrete piers that extend into the underlying bedrock materials, be utilized. Structural loads should determine pier spacing.

The piers should contain steel reinforcement over their entire length, with reinforcement as directed by the project Structural Engineer. The following table summarizes our recommended criteria for foundation design:

FOUNDATION DESIGN CRITERIA

Pier Diameter	Minimum 12-inches.
Pier Depth	Minimum of 10-feet, or as determined in the field by a representative from this office, during drilling.
Bearing Capacity	Maximum friction value of 600 psf commencing 1-foot 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, both top and bottom.

5.5 CONCRETE SLAB-ON-GRADE

Concrete slabs-on-grade will provide satisfactory floor area for the garage and patio areas. In order to reduce the potential for slab cracking, the following recommendations are presented:

1. Scarify the subgrade surface to a minimum of 6-inches, to properly moisture condition the soil to near the optimum moisture content, and compact it to a minimum of 90 percent of maximum dry density.

2. The slabs should consist of a floating type of slab system. Complete isolation of the floor, from bearing walls, columns, nonbearing partitions, stairs, and utilities, should be provided, to allow the slab to move with minimum damage to the structural integrity of the building. A flexible felt joint should be provided between the grade beam and the slab, to fill the void and prevent moisture infiltration.

3. Provide the necessary gradient to prevent the ponding of water.

4. Concrete slabs should include crack control joints for normal lineal shrinkage of the concrete materials. Where large areas of concrete slab are placed, with irregular projections or inserts within the slab area, stress concentrations will result, causing uncontrolled crack patterns. Where possible, crack control joints should be placed at stress locations where projections from a main slab or where inserts occur, in order to control the resultant crack pattern.

5. All slabs should be a minimum thickness as set forth by the Structural Engineer, but should not be less than 5-inches in total thickness when placed.

6. All concrete slabs-on-grade should be underlain by a 4-inch thick capillary break of "pea gravel" or clean crushed rock (no fines). It is recommended that Class 2 baserock not be employed as the capillary break material. If vapor transmission is undesirable, it is recommended that an impermeable membrane of 10-mil minimum thickness be placed upon the capillary break material, and overlain by 2 inches of clean sand, to assist in proper curing of the slab. The specified 4-inch thickness of the capillary break cannot be reduced, because of the use of sand.

7. Reinforcement of the concrete slabs shall be as directed by the project Structural Engineer, but in no event should it consist of less than No. 3 bars at 18-inches each way, centered within the slab.

5.6 RETAINING WALLS

According to preliminary plans (Figures 3 and 4), retaining walls are proposed at:

1. The base of a deep cut into the hillside and thus, into sandstone bedrock on Lots7, 8 and 9, on Tract 8297

2. Along the top of a cut slope and below an existing retaining wall, on Lots 1, 2 and 3, on Tract 8296

3. The base of a 15 to 20-foot thick, sliver fill, along Lots 10 thru 15, on Tract 8296.

4. Structural retaining walls at the split level transition in pads 9 through 16, on Tract 8296.

The above described four distinct conditions for the materials and configurations that are to be retained, require specific design parameters for each condition, as appropriate.

We recommend that all retaining walls have a drain blanket consisting of Class II Permeable material (conforming to Caltrans specifications) of minimum 12-inches in width or a Geo-composite drain, extending for the full height of the wall, except for 18-inches of compacted soil cover at the surface. A 4-inch perforated subdrain line (SDR 35) should be provided near the base of the drain blanket, with a suitable discharge location away from all structural improvements.

Where the retaining wall is used as part of a living structure, and in order to reduce the potential for moisture transmission through the retaining wall, it is recommended that the stem wall be waterproofed, in accordance with manufacturer's specifications. This should include the heel of the footing and down face of the heel. A "can't strip" or equivalent, should be provided on the exterior of the walls, at the joint between the retaining wall footing and the stem (wall).

5.6.1 RETAINING WALLS AT THE BASE OF CUT AT REAR OF LOTS 7, 8 AND 9, TRACT 8297

A retaining wall designated to the base of a cut into the hillside that would expose bedrock, may be designed for a drained condition and to resist lateral pressures exerted from soils having an equivalent fluid weight of 40 pcf. The active lateral force may be resisted by a conventional footing with shear key, or piers. For conventional walls that extend to a minimum depth of 4 feet below current existing grades, a maximum toe bearing pressure of 2,500 psf combined with a passive force equal to the resistance provided by an equivalent fluid weight of 450 pcf, may be implemented. Additional lateral resistance may be provided by a friction factor of 0.45 between the bottom of the footing and the soil.

5.6.2 RETAINING WALL AT TOP OF CUT AND BELOW EXISTING RETAINING WALL ON LOTS 1, 2 AND 3, TRACT 8296

There are three important issues to consider with this retaining wall:

1. The potential for the excavations to accommodate the proposed wall to undermine the existing wall

2. The additional (surcharge) pressures being transmitted to the proposed wall from the existing wall above

3. The limited support to the wall foundation, due to the sloping terrain in front of the wall

As such, we recommend that a "soldier beam wall" option be selected for this application, as it is able to be constructed in phases. This would avoid the undermining of the wall above and the drilled pier support can be designed neglecting the upper portion of pier embedment. The wall construction can begin with the excavations of slots, to accommodate the drilling of the piers and installation of steel beam supports. Subsequently, additional excavations can be undertaken to place the perforated pipe, lagging and drain rock, on individual segments, prior to proceeding to the next segment. With the foregoing, we present the following recommendations for the design of "soldier beam wall":

> The wall should be designed to resist lateral pressures exerted from soils having an equivalent fluid weight of 60 pcf, plus a 200 psf uniform surcharge to account for the upper wall loads. Retaining wall support should be derived from piers that are designed assuming that a passive force equivalent to that caused by a fluid weighing 400 pcf commences 4-feet below the bottom of the wall. The passive force can be assumed to have a tributary horizontal width equal to 2 pier diameters.

5.6.3 MSE (MECHANICALLY STABILIZED EARTH) RETAINING WALLS AT THE BASE OF FILL, LOTS 10 THRU 15, TRACT 8296

Modular Concrete Units Walls with Geogrid Reinforced Backfill (i.e., Keystone, Allan Block, etc.) are purposely omitted, due to the current phase of planning has not yet reached that level of details. This type of wall should be designed by the Soils Engineer of Record, for the project. This office can provide this service expeditiously, upon the client's request.

5.6.4 STRUCTURAL RETAINING WALLS AT THE SPLIT LEVEL TRANSITION IN PADS 9 THROUGH 16, TRACT 8296.

Wall in the interior foundation footprint, used to retain a vertical configuration in the step between upper and lower pads, on Lots 9 through 16, on Tract 8296, should be designed for a drained condition and to resist lateral pressures exerted from soils having an equivalent fluid weight of 55 pcf. The active lateral force may be resisted by a passive force commencing a minimum of one foot below the lowest adjacent grade in front of the wall, equal to the resistance provided by an equivalent fluid weight of 350 pcf.

For conventional walls, a maximum toe bearing pressure of 2,000 psf may be implemented for dead load plus live load criteria. This value may be increased by one-third for seismic loading. Additional lateral resistance may be provided by a friction factor of 0.3 between the bottom of the footing and the soil.

5.7 DRAINAGE

It is important to direct surface runoff away from the foundation perimeters, concrete flat work, or any other improvement that is founded near the surface. Downspouts should be connected to conduits that will transport their effluent to a discharge point away from structural element-bearing soils. Area drains should be provided to capture, collect and transport surface waters around the dwelling.

5.8 STREET PAVEMENTS

Based on the nature of the subgrade soil, in conjunction with the anticipated traffic along the private driveway, we recommend a minimum pavement section consisting of 2.5-inches of Asphaltic Concrete over 8-inches of Class II Aggregate Baserock.

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 1-foot, to properly moisture condition the soil to near the optimum moisture content, and compact it 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.

5.9 UTILITY TRENCHES

Utility trenches parallel to the sides of the grade beams 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 grade beam.

All trenches should be backfilled with native materials compacted uniformly to a 90% relative compaction. If local building codes require the use of sand or other permeable trench backfill, all utility trenches entering the building must be provided with an impervious seal of either cohesive soil or lean concrete, where the trench passes under the building perimeter. The impervious plug should extend 4 feet into, and out of, the building perimeter.

Jetting of trench backfill should be avoided as it may result in an unsatisfactory degree of compaction.

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. Mark Crawford 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 borings 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

Dibblee, T. W., 1980, Preliminary Geologic Map of the Hayward 7.5' Quadrangle, Alameda County, California, U.S. Geol. Survey Open-file Report 80-540.

Davis, J., 1982, State of California, Special Studies Zones, Revised Official Map, Hayward 7.5' Quadrangle, Alameda County, California.

Nilsen, T.H., 1975, Preliminary Photointerpretation of Landslide and other Surficial Deposits of the Hayward 7 1/2' Quadrangle, Contra Costa & Alameda Counties, California: U.S. Geol. Survey Open-file Map 75-277-19.

U.S.G.S., Geologic Map and Map Database of the Oakland Metropolitan Area, Alameda, Contra Costa, and San Francisco Counties, California, by R. W. Graymer, Miscellaneous Field Studies, MF-2342, Version 1.0, 2000.

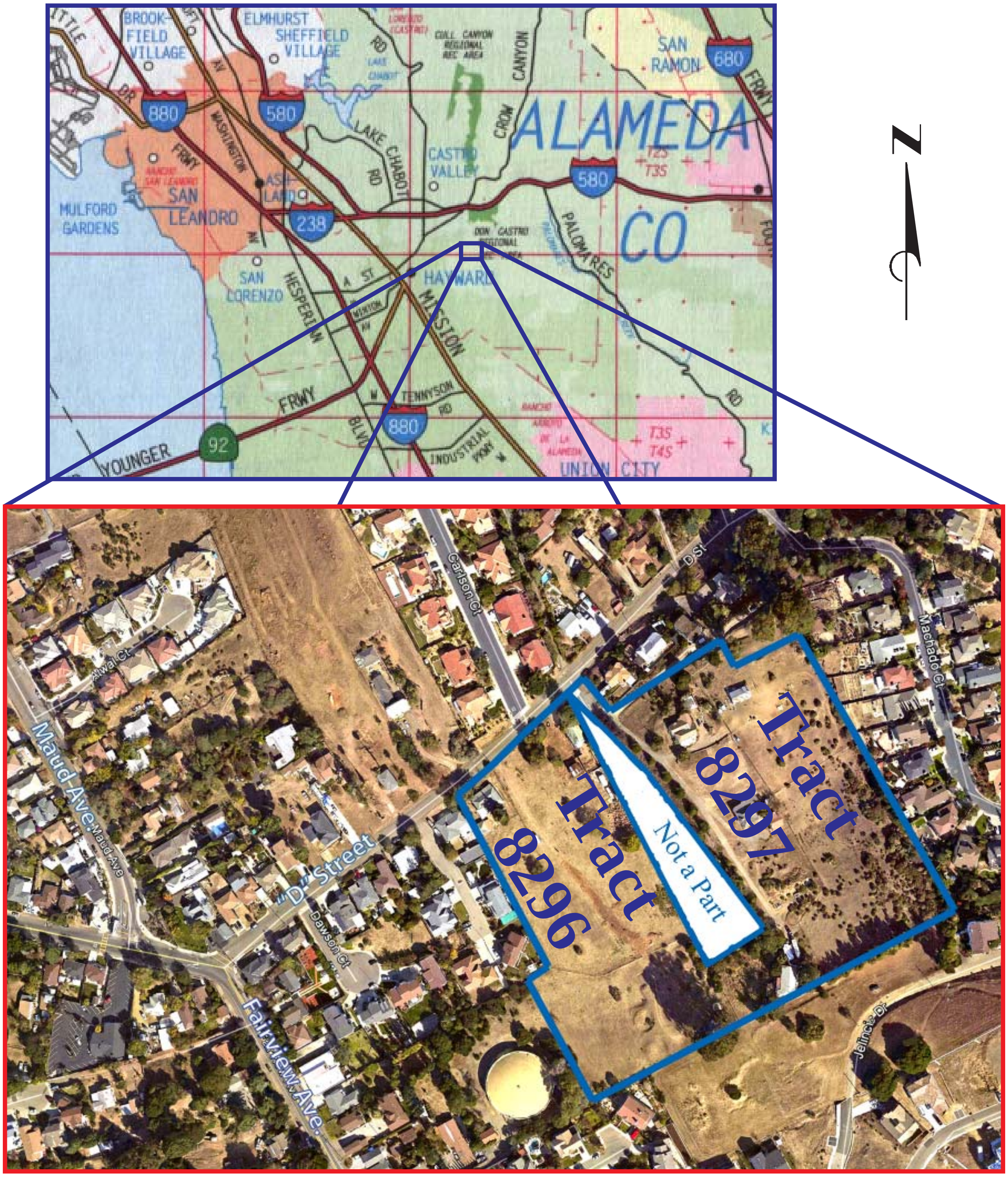
Petersen, et al. (1996, and 2003 Revisions), Probabilistic Seismic Hazard Assessment for the State of California, U.S.G.S. Open-File Report 96-706, D.M.G. Open-File Report 96-08.

California Geological Survey, Earthquake Zones of Required Investigation, Hayward Quadrangle, 2012.

Geotechnical Engineering, Inc., Updated Report, Preliminary Soil Investigation, Planned 17 New Residences, 3297 D Street, Hayward, Alameda County, California, Job No. 111382AA, Dated March 31, 2006. Final Report - Additional Soil Investigation Including Incorporation of Subsurface Data from Preliminary Investigation, Planned 16 Lots Subdivision, 3297 D Street, Hayward, Alameda County, California, Job No. 111382B, Dated October 16, 2007.

Engeo Inc., Getechnical Review, Bassard Property, Tract 7303, Hayward, California, Project No. 5505.105.301, Dated January 30, 2008. Second Geotechnical Peer Review, Dated January 30, 2008, Revised May 28, 2008. Third Geotechnical Peer Review, Dated October 15, 2008. Fourth Geotechnical Peer Review, Dated January 7, 2009. Fifth Geotechnical Peer Review, Dated March 16, 2009.

United Soil Engineering, Inc., Geotechnical Clarifications, Proposed Residential Subdivision, Bassard Property, Tract 7303, 3297 D Street, Hayward, California, File No. 5936-S1, Dated November 17, 2008. Grading and Drainage Plan Review for Tentative Tract Map 7303, Dated November 24, 2008. Geotechnical Clarifications, Dated February 19, 2009.



SITE LOCATION

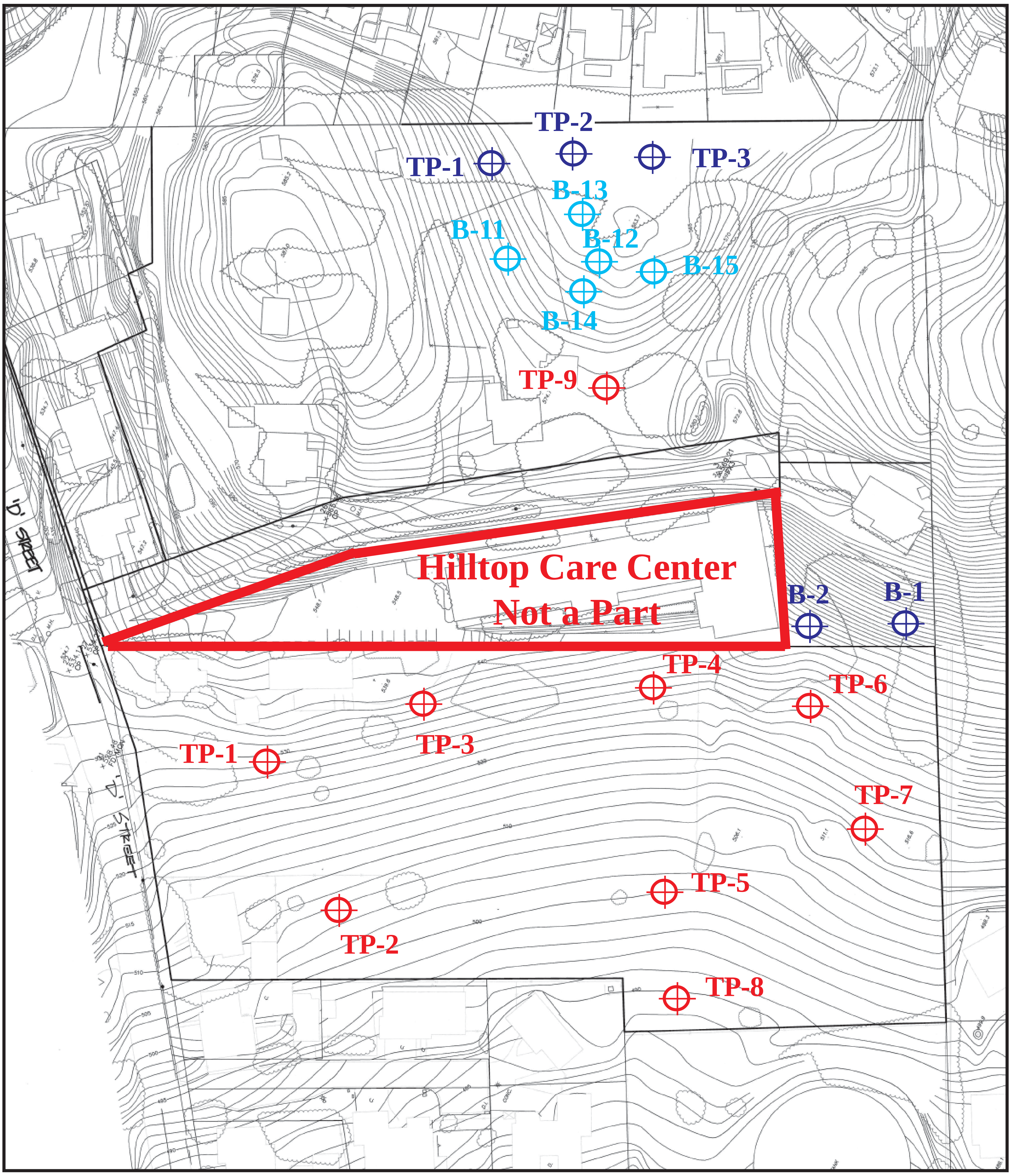
Sources: Thomas Guide and Google Earth

Project No.: C-149-03	Date: 08-10-15	Scale: NTS

Henry Justiniano & Associates

Soils and Foundation Engineering

Figure No. 1



Explanation

Approximate Borehole and
Test Pit Locations and Dates
by GEI and HJ&A

GEI Inc., March, 2006

GEI Inc., October, 2007

HJ & Assoc., June, 2015

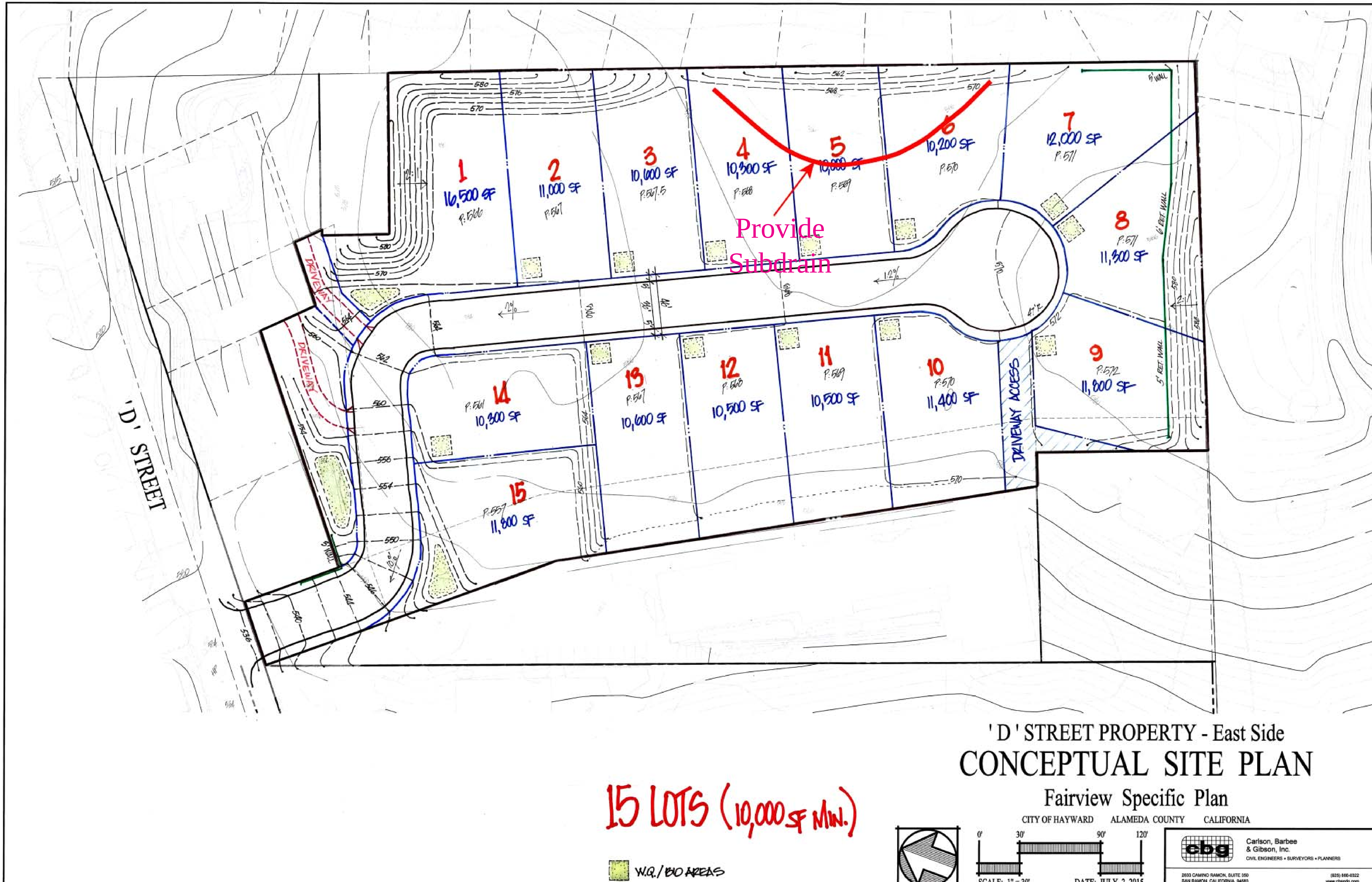
Source: Crawford Development

SITE PLAN

Project No.: C-149-03	Date: 08-10-15	Scale: NTS

Henry Justiniano & Associates
Soils and Foundation Engineering

Figure No. 2



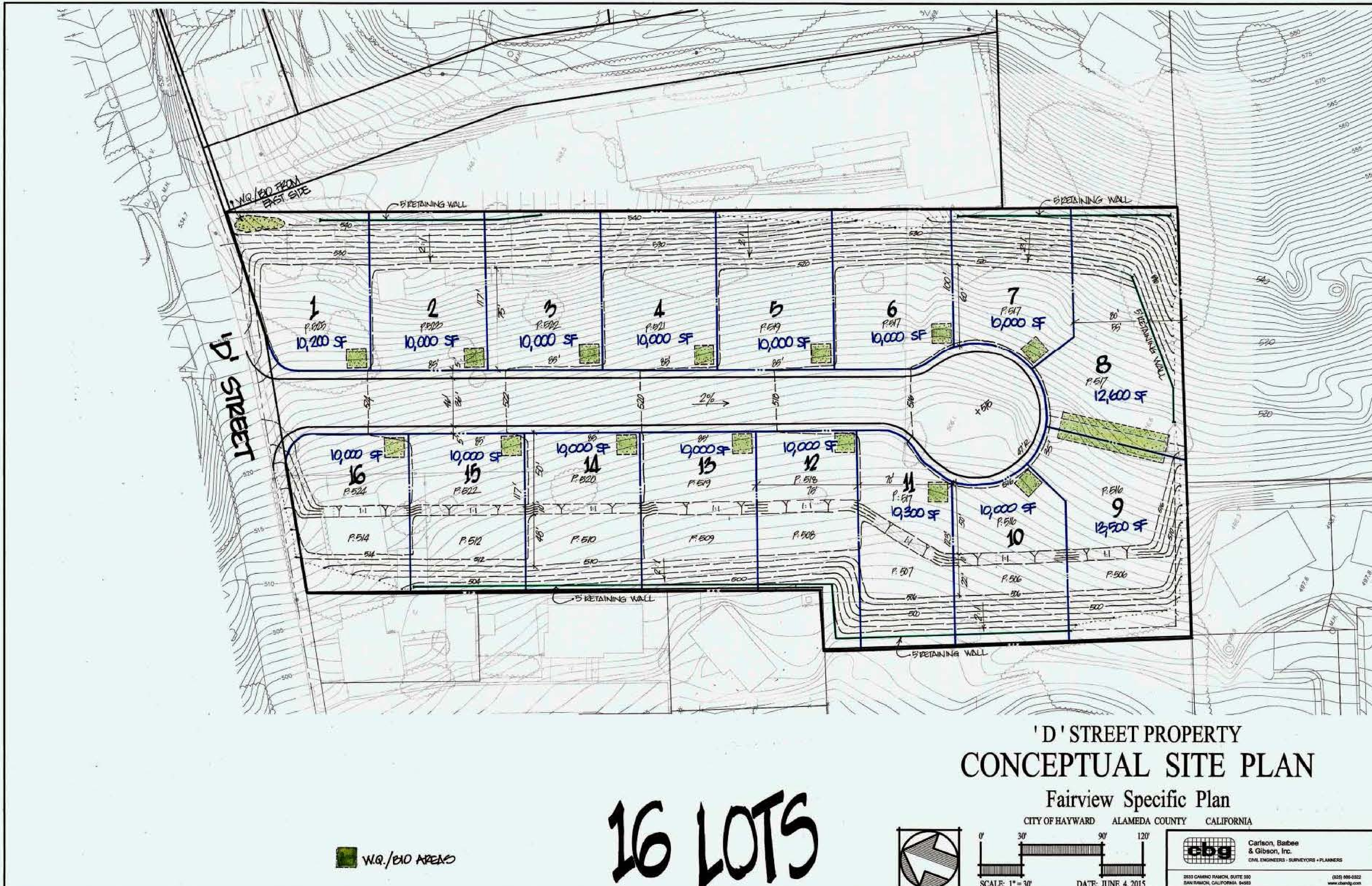
'D' STREET PROPERTY - East Side

CONCEPTUAL SITE PLAN

Fairview Specific Plan

CITY OF HAYWARD ALAMEDA COUNTY CALIFORNIA

Figure 3



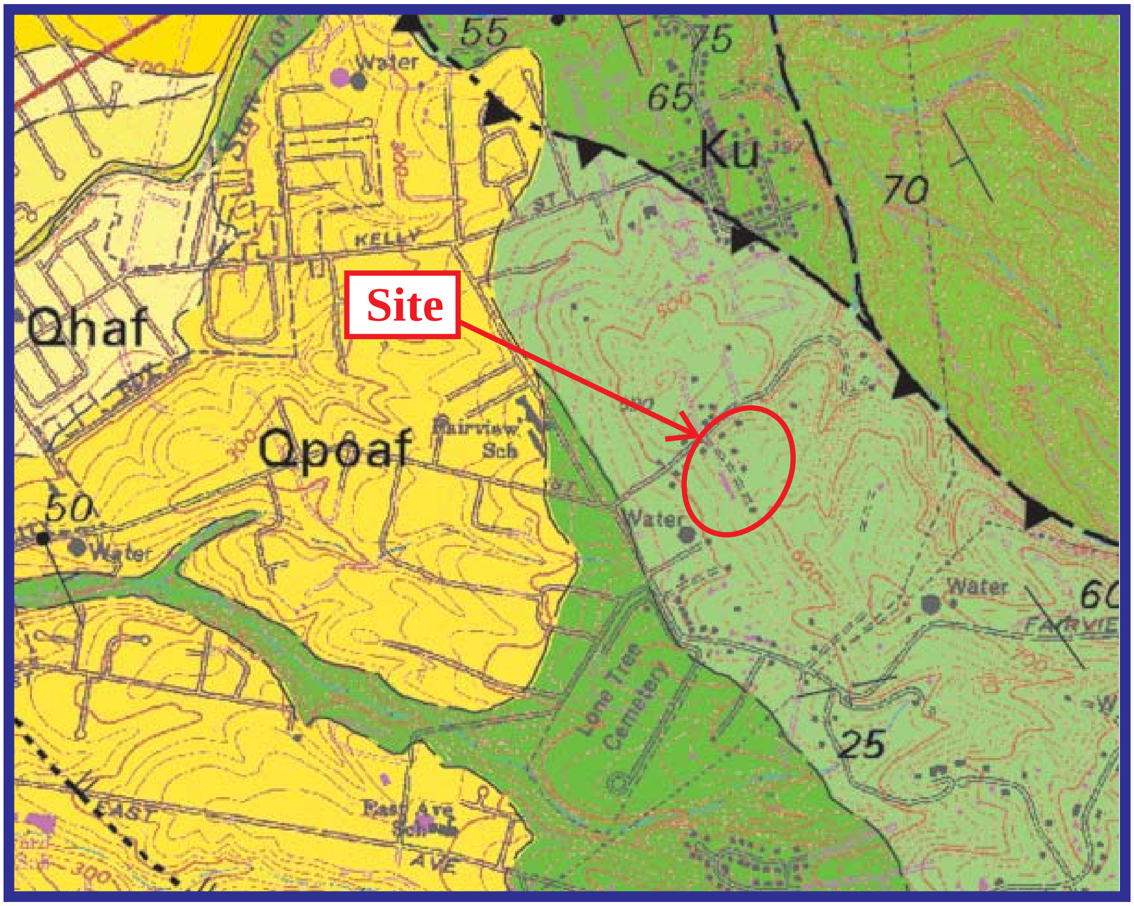
'D' STREET PROPERTY

CONCEPTUAL SITE PLAN

Fairview Specific Plan

CITY OF HAYWARD ALAMEDA COUNTY CALIFORNIA

Figure 4



EXPLANATION

Qhaf	Alluvial fan and fluvial deposits (Holocene)	Kcv	Unnamed sandstone, conglomerate, and shale of the Castro Valley area (Late Cretaceous, Turonian and younger (?))
Qpoaf	Older alluvial fan deposits (Pleistocene)	Kjm	Joaquin Miller Formation (Late Cretaceous, Cenomanian)
Ko	Oakland Conglomerate (Late Cretaceous, Turonian and/or Cenomanian)		
Ku	Undivided Great Valley complex rocks (Cretaceous)		

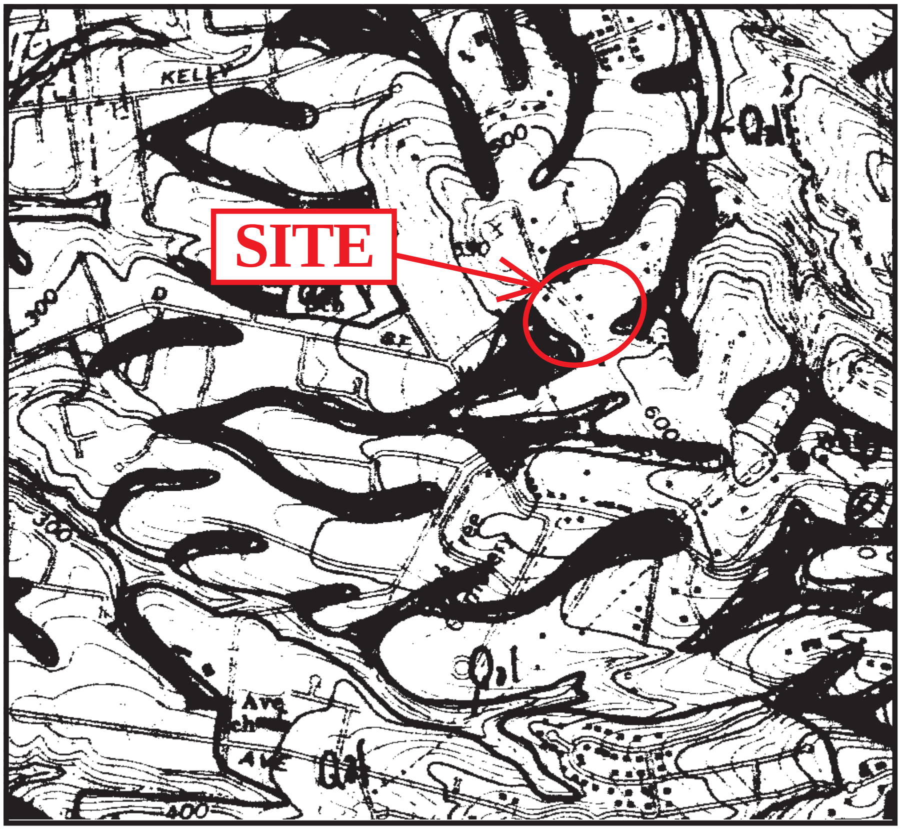
GEOLOGY MAP

R.W. Graymer, 2000

Project No.: C-149-03	Date: 08-10-03	Scale: NTS

Henry Justiniano & Associates
Soils and Foundation Engineering

Figure No. 5



EXPLANATION

Qal — Alluvial deposit

Qt — Alluvial terrace deposit Queried where uncertain.

Colluvial deposit and/or small alluvial fan deposit

Qaf — Artificial fill

Bedrock Queried where identification uncertain.

Landslide & Surficial Deposits

Source: **T. H. Nilsen, 1975**

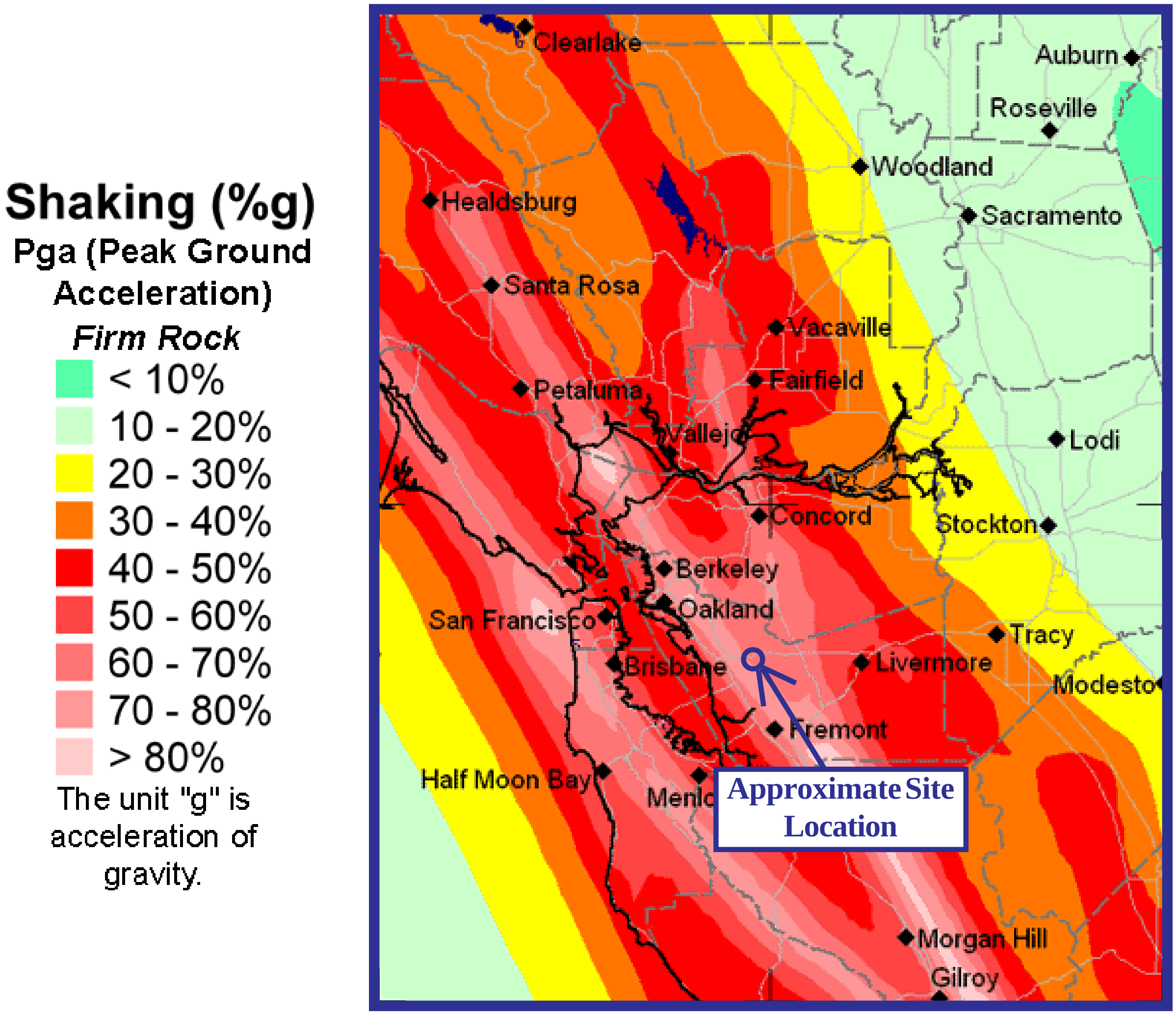
N

Project No. C-149-03	Date: 08-10-15	Scale: NTS

Henry Justiniano & Associates

Soils and Foundation Engineering

Figure No. 6



PROBABILISTIC SEISMIC HAZARD MAP (Modified)

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

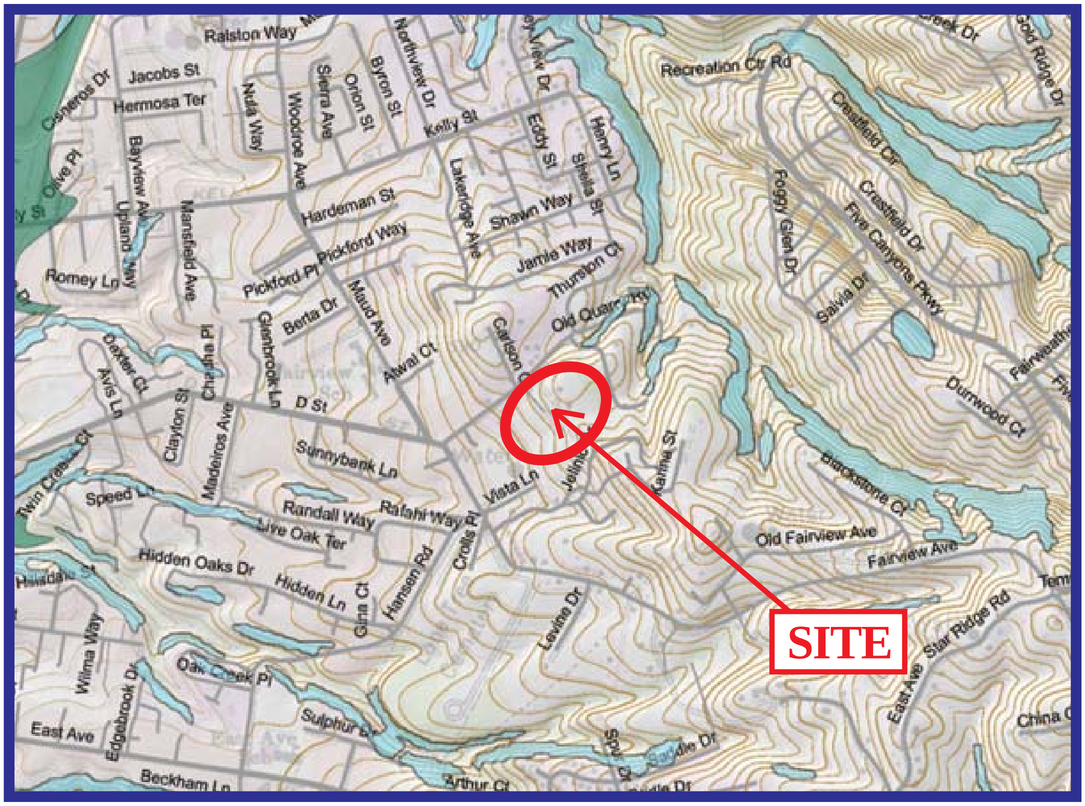
Based on the USGS/CGS Probabilistic Seismic Hazards Assessment (PSHA)
(revised 2003)

N

Project No. C-149-03	Date: 08-10-15	Scale: NTS

Henry Justiniano & Associates
Soils and Foundation Engineering

Figure No. 7



EXPLANATION

Liquefaction
Areas where historical occurrence 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.

CALIFORNIA GEOLOGICAL SURVEY

EARTHQUAKE ZONES OF REQUIRED INVESTIGATION

HAYWARD QUADRANGLE OFFICIAL MAP

RELEASED SEPTEMBER 21, 2012 (MODIFIED)

Project No.: C-149-03	Date: 08-10-15	Scale: As Shown

Henry Justiniano & Associates
Soils and Foundation Engineering

Figure No. 8

Project No C-1149-03
August 10, 2015

TEST PIT LOGS

Test Pit No.	Depth (Feet)	Description
TP 1	0.0 - 0.4	Brown Silty Sand (SM); loose to medium dense; porous; roots to 4"; Topsoil
	0.6 - 1.1	Brown Silty Sand; medium dense; dry; Residual Soil
	1.1 - 2.5	Yellow Brown Sandstone; weak to moderately strong; closely fractured; dry; prominent fracture orientation (N24E, 55NW); bedding not apparent
TP 2	0.0 - 1.8	Brown Silty Sand (SM); loose to medium dense; porous; roots to 6"; Topsoil
	1.8 - 2.7	Brown Silty Clay; Stiff; dry; Colluvial Soil; Liquid Limit 32; Plasticity Index 19
	2.7 - 4.3	Chocolate Brown Silty Clay; Stiff; moist
	4.3 - 7.3	Yellow Brown Silty Clay; Very Stiff; appears to be deeply weathered sandstone; moderately difficult to excavate
TP 3	0.0 - 1.7	Brown Silty Sand (SM); loose to medium dense; porous; roots to 4"; Topsoil
	1.7 - 3.0	Dark Brown Sandstone; deeply weathered; friable; dry:
	3.0 - 4.3	Yellow Brown Sandstone; moderately strong; moderately fractured with no prominent orientations; bedding not evident.

Figure 9

TEST PIT LOGS

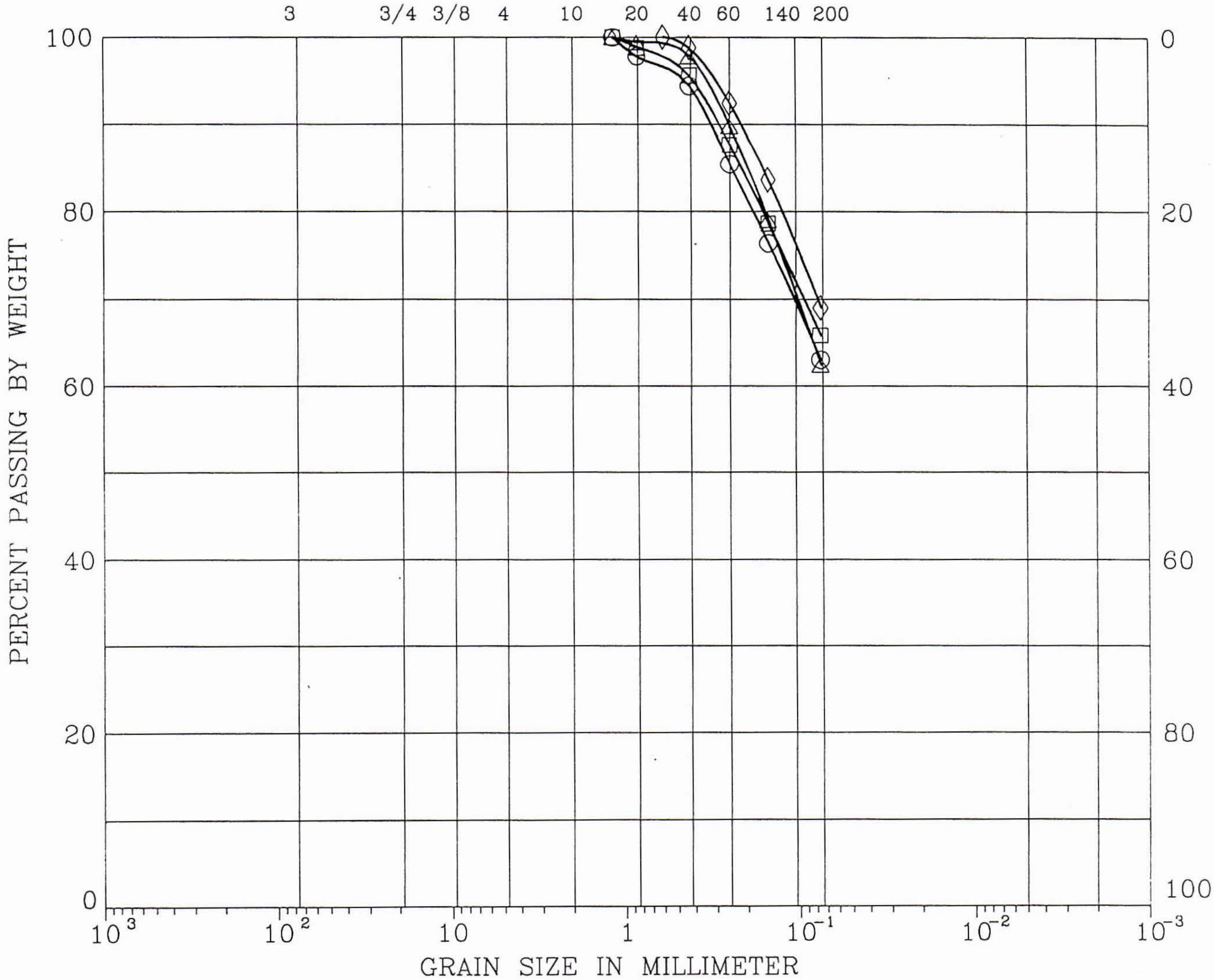
Test Pit No.	Depth (Feet)	Description
TP 4	0.0 - 2.0	Brown Silty Sand (SM); loose to medium dense; porous; roots to 5"; Topsoil
	2.0 - 3.1	Mottled Gray/Yellow/Brown Silty Clay; Stiff; moist; Liquid Limit 42; Plasticity Index 27
	3.1 - 4.2	Dark Brown Clayey Sand with scattered sandstone fragments; dense; moist
	4.2 - 4.9	Yellow/Brown Sandstone; moderately strong; moderately fractured with no prominent orientations; bedding not evident
TP 5	0.0 - 2.5	Brown Silty Sand (SM); loose to medium dense; porous; roots to 5"; Topsoil
	2.5 - 5.0	Yellow/Brown Silty Sand; medium dense to dense; moist; Colluvial Soil ?
	5.0 - 6.3	Mottled Yellow/Dark Brown Clayey Sand; medium dense; moist
	6.3 - 6.9	Dark Brown Clayey Sand; slight increase in clay content; medium dense; moist; Residual Soil / Deeply Weathered Sandstone ?
TP 6	0.0 - 0.2	Brown Silty Sand (SM); loose to medium dense; porous; roots to 2"; Topsoil
	0.2 - 1.4	Yellow/Brown Sandstone; moderately strong; moderately fractured with no prominent orientations; bedding not evident; moderately difficult to excavate

TEST PIT LOGS

Test Pit No.	Depth (Feet)	Description
TP 7	0.0 - 1.2	Brown Silty Sand (SM); loose to medium dense; porous; roots to 4"; Topsoil
	1.2 - 2.1	Dark Brown Clayey Sand; medium dense to dense; dry
	2.1 - 6.6	Chocolate Brown Silty Sand; medium dense; moist with increasing moisture at 6 ft.
	6.6 - 7.1	Yellow/Brown Clayey Sand; dense; moist; Residual Soil ?
TP 8	0.0 - 0.7	Brown Silty Sand (SM); loose to medium dense; porous; roots to 4"; Topsoil
	0.7 - 1.5	Brown Sand; medium dense to dense; dry
	1.5 - 3.3	Yellow/Brown Clayey Sand; medium dense; moist with increasing moisture at 3 ft.
	3.3 - 7.2	Dark Brown Sand; dense; moist; Residual Soil ?
TP 9	0.0 - 1.0	Brown Silty Sand (SM); loose to medium dense; porous; roots to 4"; Topsoil
	1.0 - 1.6	Dark Brown Silty/Clayey Sand; medium dense to dense; dry
	2.1 - 4.6	Brown Silty Sand; medium dense; moist with slight increase in moisture at 4 ft.
	4.6 - 7.0	Yellow/Brown Clayey Sand; dense; moist; Residual Soil ?

UNIFIED SOIL CLASSIFICATION

COBBLES	GRAVEL		SAND			SILT OR CLAY
	COARSE	FINE	COARSE	MEDIUM	FINE	
U.S. SIEVE SIZE IN INCHES			U.S. STANDARD SIEVE No.			HYDROMETER



SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
○	TP 2	2	32	19	INORGANIC CLAYS (CL)
□	TP 2	3			INORG. SILTS AND CLAYS (ML–CL)
△	TP 2	7.25			INORG. SILTS AND CLAYS (ML–CL)
◇	TP 4	2.5	42	27	INORGANIC CLAYS (CL)

Remark :

Project No.C–14903	D Street, Hayward
H. Justiniano And Associates	GRAIN SIZE DISTRIBUTION Figure No. 12

Appendix A

GEI's

Logs of Borings
and
Test Pit Logs

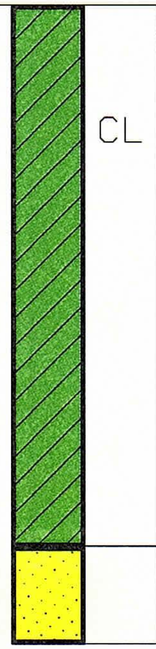
BORING 1

1" Diameter Percussion Hole

Drilled 4/25/97

Depth (Ft.)

0 —



6" Top Soils

Brown & dark brown silty clay, with few sand, stiff, slightly plastic average moisture content=14%

1 —

LL=32%, PL=18%, PI=14

2 —

With few gravel

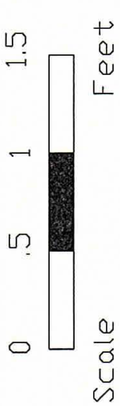
3 —

Light yellow brown, sandstone medium, weathered, hard, average moisture content=9%

Refusal

Note: Free Ground Water Not Encountered

LOG OF BORING

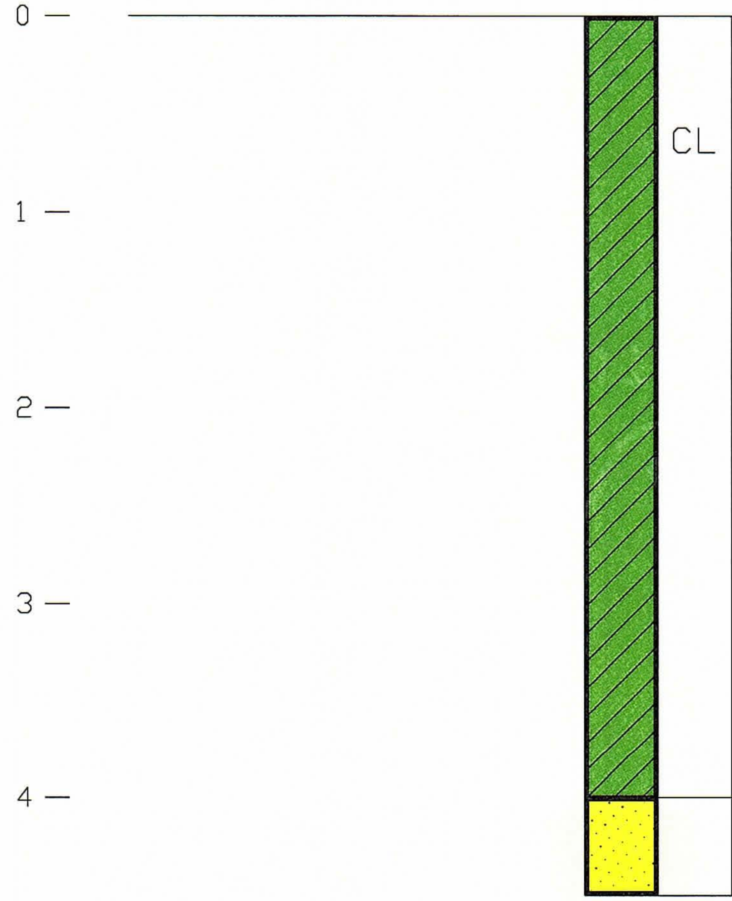


BORING 2

1" Diameter Percussion Hole

Drilled 4/25/97

Depth (Ft.)



6" Top Soils

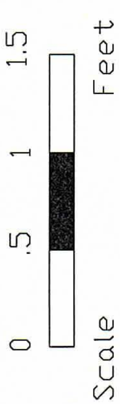
Brown & dark brown silty clay, with few sand, stiff, slightly plastic average moisture content=10%

With few gravel

Light yellow brown, sandstone, medium, weathered, hard, average moisture content=13%

Refusal

Note: Free Ground Water Not Encountered



LOG OF BORING

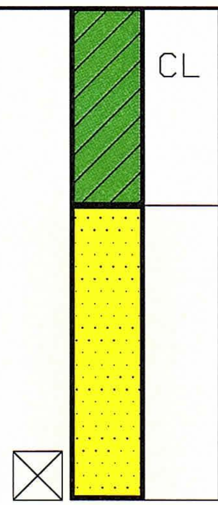
GEOTECHNICAL ENGINEERING, INC.

BORING 11

1″ Diameter Percusion Hole

Drilled 9/21/07

Depth (Ft.)



Depth (Ft.)	Symbol	Description
0 – 2	CL	6″ Top Soil Brown, silty clay, probable fill with some organics, loose
2 – 5		Light yellow brown, sandstone weathered, hard Slow drilling, very hard

Ave. Moisture Content = 6%

Refusal @ 5′

Note: Free ground water not encountered

LOG OF BORING

Scale 0 1 2 3 Feet

GEOTECHNICAL ENGINEERING, INC.

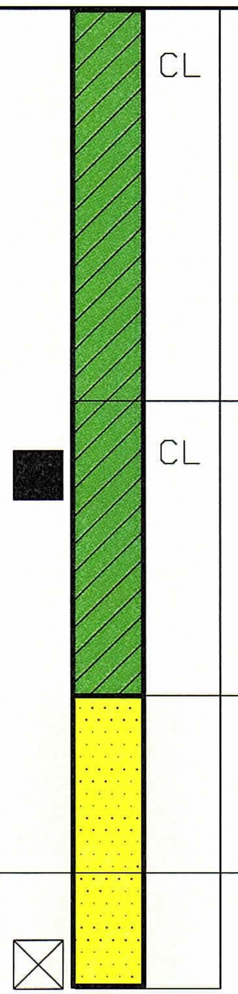
PLATE 2

BORING 12

1" Diameter Percusion Hole

Drilled 9/21/07

Depth (Ft.)



Depth (Ft.)	Sample data	Soil	Description
0–4		CL	6" Top Soil Light brown & brown, silty clay, fill, loose
4–7	15% - 112 (68) *	CL	Brown, sandy clay, natural, stiff
7–10	- - (N=50/4")		Light yellow brown, sandstone weathered, hard Standard penetration test N=Blows Per Foot Slow drilling, very hard

Refusal @ 10'

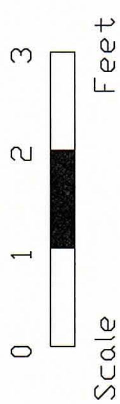
*140-lbs. weight falling 30-ins.
Note: Free ground water not encountered

15% - 112 (68) Undisturbed Sample

Blows per Foot

Field Dry Density (p.c.f.)

Natural Moisture Content (%)



GEOTECHNICAL ENGINEERING, INC.

LOG OF BORING

PLATE 3

BORING 13

1″ Diameter Percusion Hole

Drilled 9/21/07

Depth (Ft.)

0 — CL 6″ Top Soil

3 —

Dark brown, silty clay fill, medium to stiff

4 —

LL=25% PL=16% PI=9

12% - 105 (14)

9 — CL Dark brown, silty clay with some sand, slightly plastic, stiff

14% - 113 (46)

12% - (N=50/5.5″)

12— Light yellow brown, sandstone weathered, very hard

Slow drilling

Refusal @ 12′

Note: Free ground water not encountered

Scale 0 1.5 3 4.5 Feet

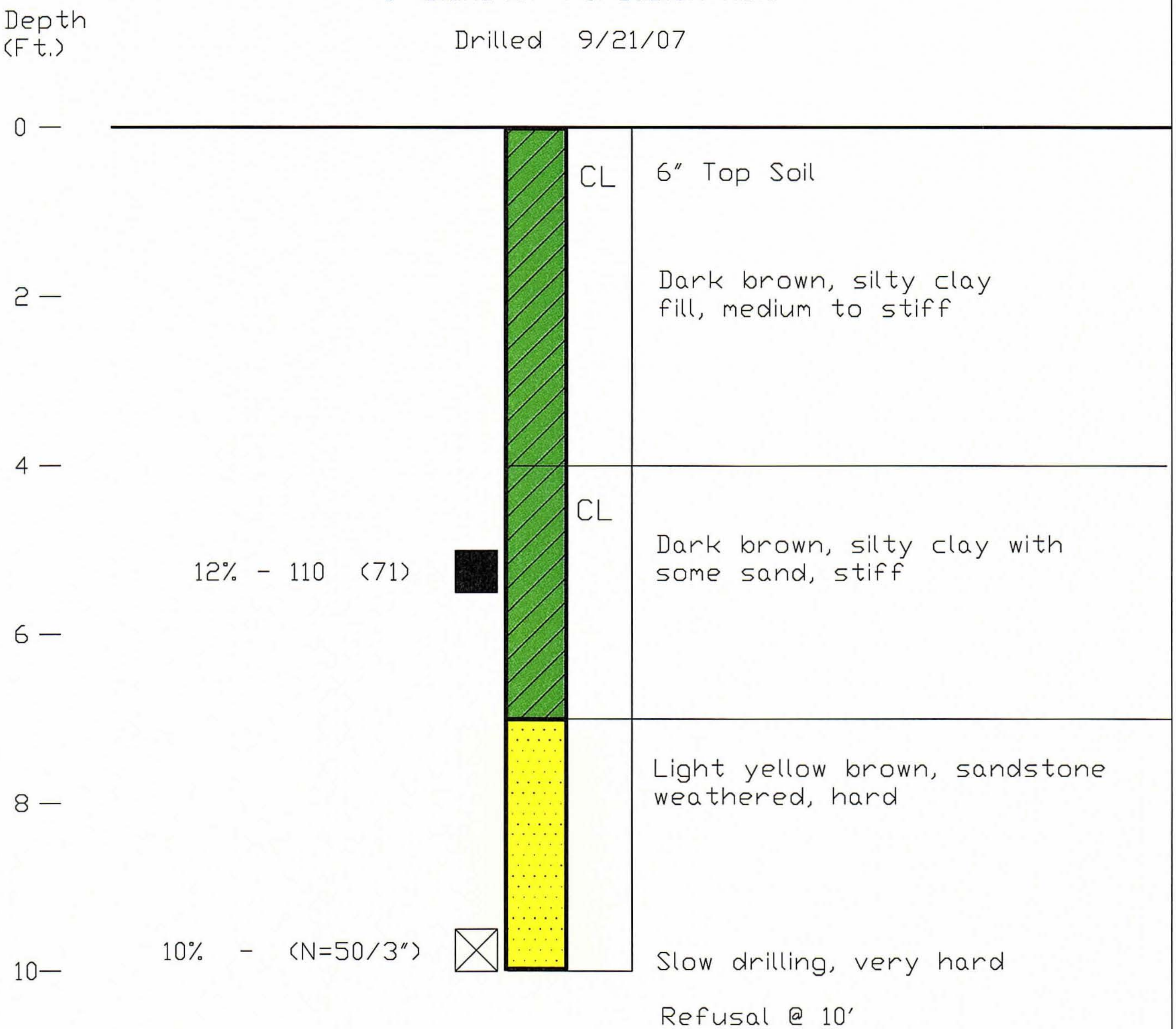
LOG OF BORING

GEOTECHNICAL ENGINEERING, INC.

BORING 14

1" Diameter Percusion Hole

Drilled 9/21/07



Note: Free ground water not encountered

LOG OF BORING

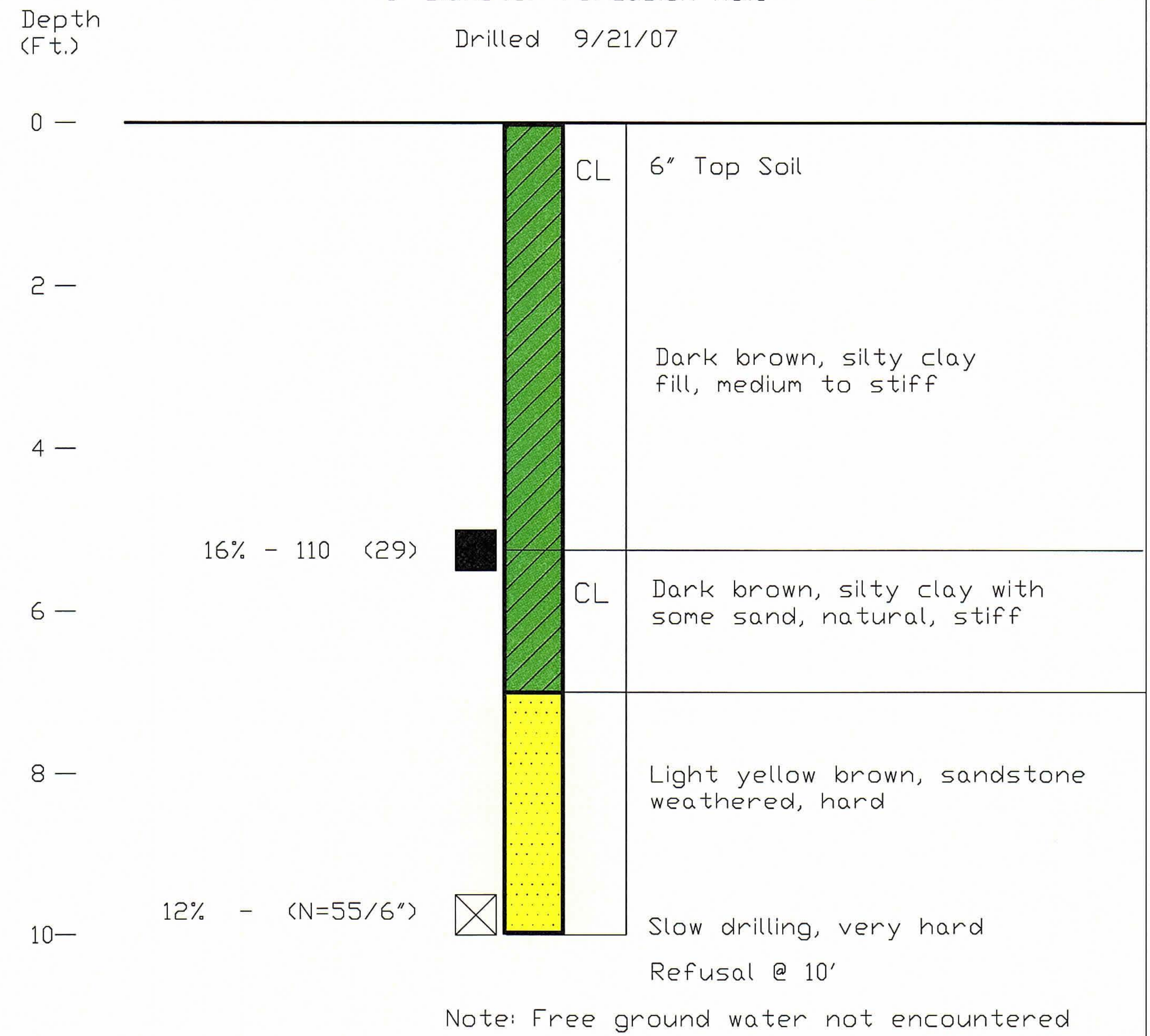
GEOTECHNICAL ENGINEERING, INC.

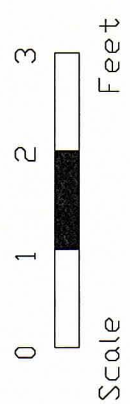
PLATE 5

BORING 15

1" Diameter Percusion Hole

Drilled 9/21/07

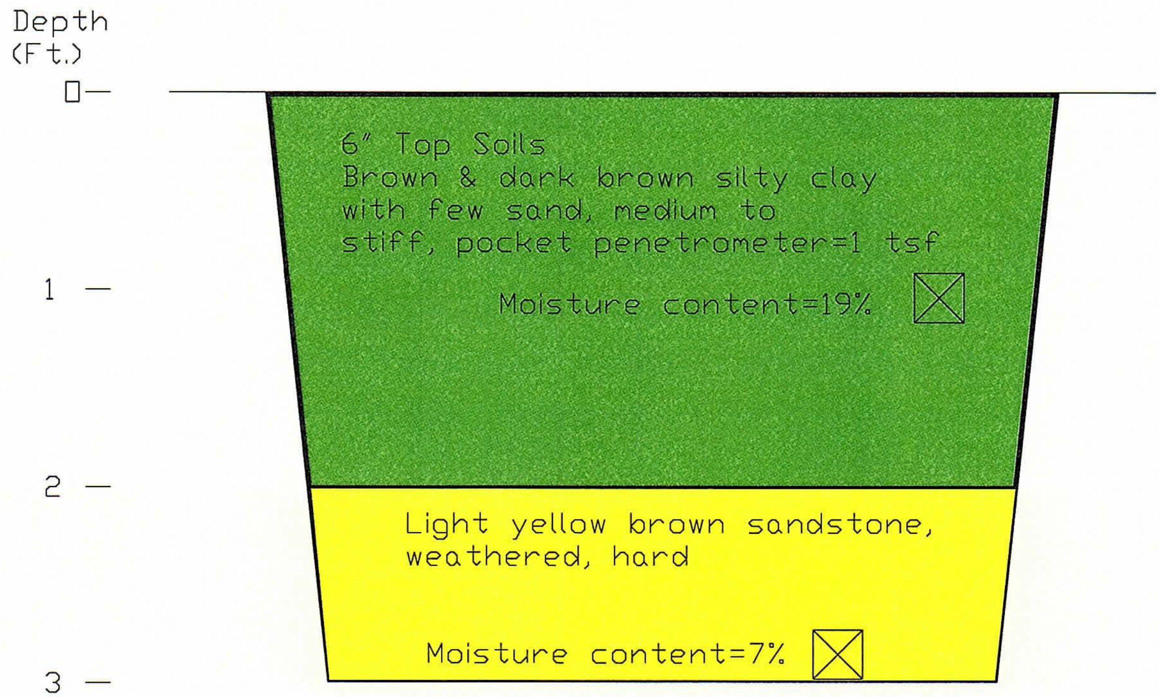




LOG OF BORING

GEOTECHNICAL ENGINEERING, INC.

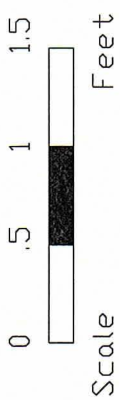
PLATE 6



Note: Free ground water not encountered

4

LOG OF TEST PIT 1

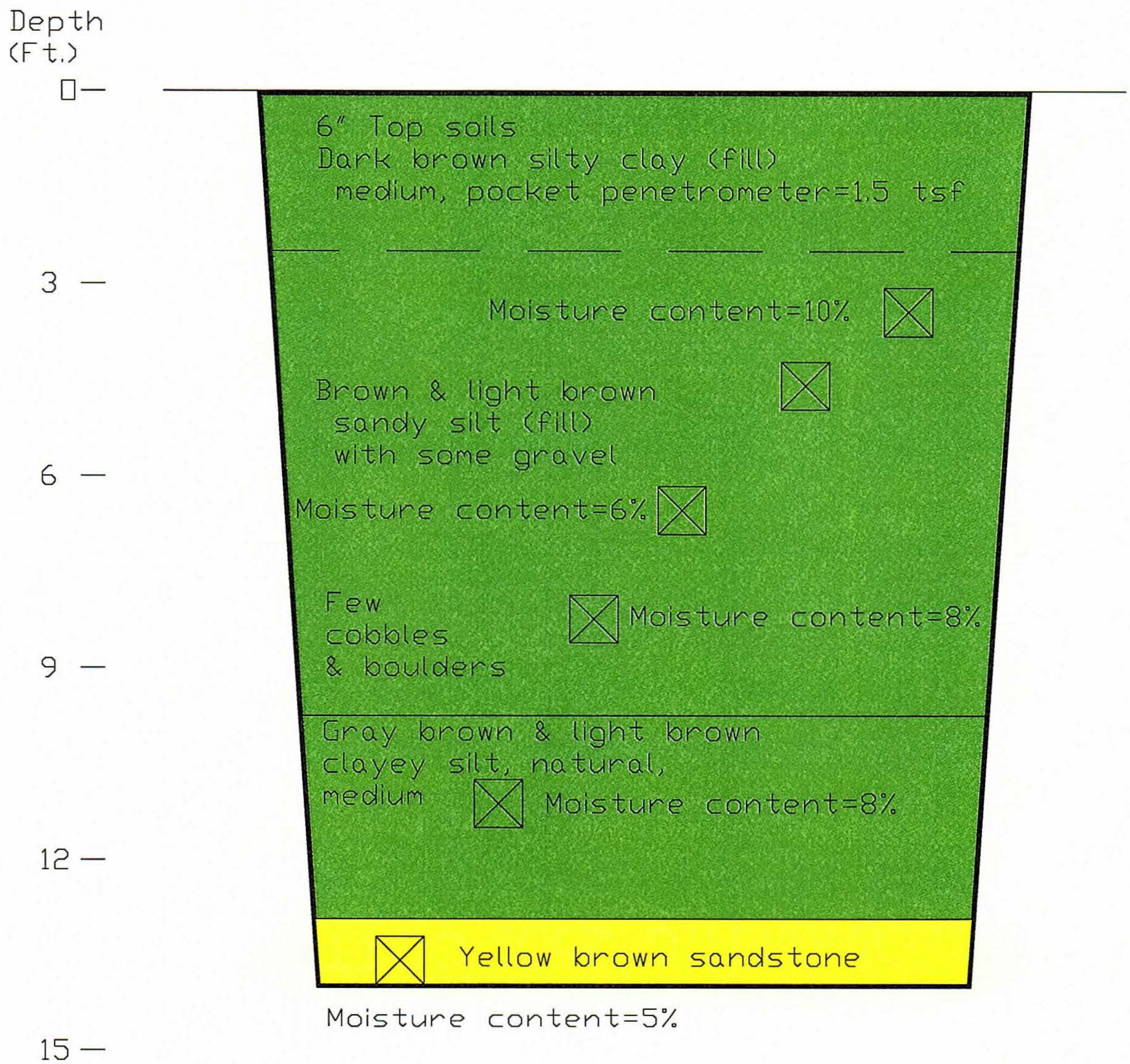


(SAMPLE LOCATIONS & LABORATORY DATA)

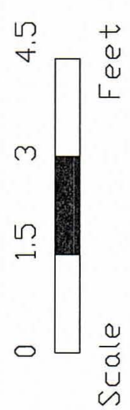
DUG 12/11/1997

GEOTECHNICAL ENGINEERING, INC.

PLATE 9



Note: Free ground water not encountered



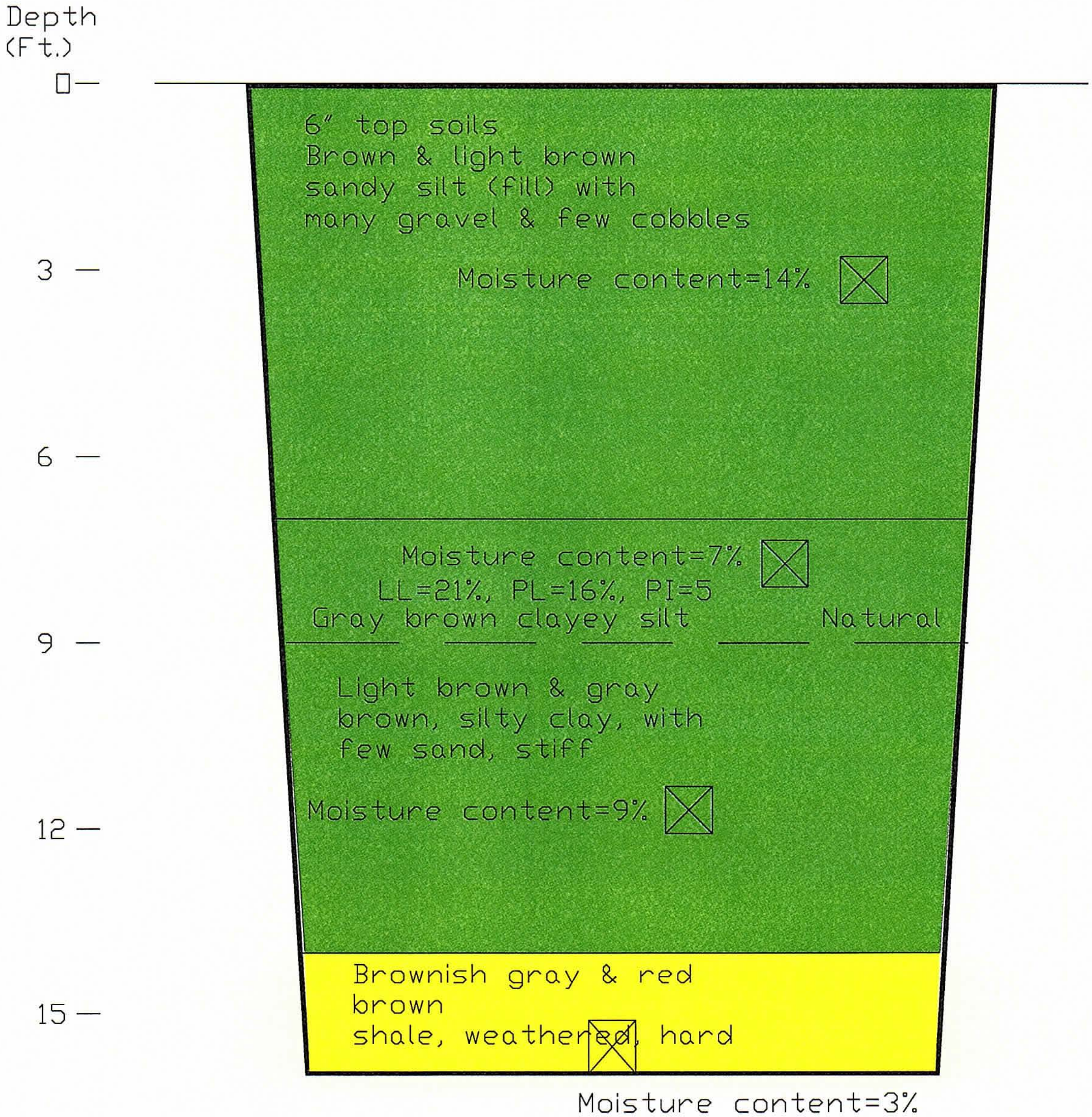
LOG OF TEST PIT 2

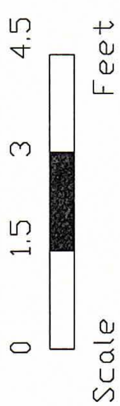
(SAMPLE LOCATIONS & LABORATORY DATA)

DUG 12/11/1997

GEOTECHNICAL ENGINEERING, INC.

PLATE 10





Note: Free ground water not encountered

LOG OF TEST PIT 3

(SAMPLE LOCATIONS & LABORATORY DATA)

DUG 12/11/1997

GEOTECHNICAL ENGINEERING, INC.

PLATE 11

Appendix B

Technical
Specifications

TECHNICAL SPECIFICATIONS
APPENDIX-B

1.0 INTRODUCTION

1.1 GENERAL

The work shall conform to applicable Federal, State, County, and local regulations. Test procedures shall conform to applicable ASTM standards, as documented in the edition of the standards in force at the start of the work, or by the specific standards quoted in these specifications.

1.2 CONFLICTS

Where a conflict exists between these specifications and applicable codes, the design drawings, other project specifications, or manufacturer's recommendations, the more stringent criteria shall apply unless otherwise directed in writing by the Engineer.

1.3 INSPECTION OF WORK

Inspection of all construction activities in these specifications, will be provided by the Owner and the Engineer while work is in progress. All work done by the Contractor shall be done in a workmanlike manner and conform to the best recognized practice to achieve a neat and functional construction. In addition, all work performed by the Contractor must meet the approval of the Engineer, but the detailed manner and methods of doing work shall be the responsibility of the Contractor.

1.4 SUSPENSION AND RESUMPTION OF OPERATIONS

The contractor shall suspend fill placing and foundation preparation operations whenever, in the opinion of the Engineer, conditions for such operations are unsatisfactory due to rain or any other reason.

1.7 CONTRACTOR RESPONSIBILITIES

The Contractor shall examine the technical specifications and construction drawings to be aware of all conditions and the site, affecting execution of the work. These conditions include:

A. The Contractor shall be responsible for the design and strength of all temporary supports and shoring which may be required for the sides of the excavations, or for protection of adjacent existing improvements. The adequacy of such systems shall be the complete responsibility of the Contractor, and shall conform to current OSHA standards.

B. The contractor shall maintain benchmarks, monuments and other reference points. If disturbed or destroyed, they must be replaced as directed.

C. Expose and verify location of all underground utilities prior to commencement of excavations. The Contractor shall be responsible for protecting existing underground utilities.

D. Applicable safety and health regulations.

C. Soil conditions.

2.0 EARTHWORK

2.1 SITE PREPARATION

Site preparations shall be performed within limits delineated in fill and borrow area in the construction drawings. These shall commence with clearing operations including, but not limited to, removal of old foundations, rubbish, abandoned pipelines, septic tanks and leach fields; cutting trees and stumps to approximately ground level, and followed by removal of all growth, stumps, brush, roots, and similar organic and deleterious matter within the borrow and fill area limits delineated in the construction drawings, and to the satisfaction of the engineer. The cleared material shall be hauled offsite prior to commencement of fill operations.

Clearing operations shall be followed by stripping. Stripping shall consist of removal and stockpiling all top soil down to suitable material as determined by the Engineer. The stripped material shall be removed from the stripped area and placed in the topsoil stockpile area from which it may later be reclaimed for landscape use. The topsoil stockpile area will be determined by the Owner.

2.2 EXCAVATION

After clearing and stripping, all surfaces to receive fill shall be scarified to a minimum 6-inch depth and recompacted to the same requirements as the fill to be placed over the prepared foundation.

Areas deemed soft or unsuitable by the Engineer, shall be excavated to accomplish a firm non-yielding foundation and backfilled in accordance with Section 2.5.

Excavations shall be graded and properly maintained to provide adequate drainage at all times. Work shall be suspended when the site is wet, muddy, or in any other condition when the area cannot be properly maintained.

Subgrade excavation shall be performed as required to achieve the lines and grades shown on the drawings. Material removed below grade shall be replaced with approved material and compacted to the requirements for structural fills, unless otherwise directed by the Engineer.

2.3 KEYWAY EXCAVATION

At the toe of side slope fills that are designated to terrain that slopes steeper than 5 horizontal to 1 vertical, a 10 foot wide base keyway shall be excavated a minimum of 3 feet into firm non-yielding material and sloped into the hillside at a gradient of no less than 5%. A 4-inch perforated pipe shall be placed at the hillside base of the keyway and shall be surrounded with 3 cubic feet of Class II filter rock per foot of pipe. The configuration of the keyway and subdrain system shall permit gravity flow to a discharge point downhill that will be subsequently connected into a line discharging to an approved outlet.

2.4 FILL MATERIALS

Fill materials shall be obtained from designated borrow areas or areas designated by the Engineer. Placement of fill shall be made only in areas approved by the Engineer for fill placement. All fill materials shall consist of durable, nonperishable, weather resistant soil/rock mixture and be free of organics or other deleterious matter. Should import material be required, it must be approved by the Engineer prior to transporting it to the project and must adhere to the following specifications.

1. Plasticity index not to exceed 15.
2. Should not contain rocks larger than 8-inches in their greatest dimension.
3. Not more than 15% passing the No. 200 sieve.

2.5 FILL PLACEMENT

After areas designated to receive fill have been cleared, grubbed and stripped, as specified in Section 2.1, they should be compacted as specified in Section 2.6. The Engineer shall approve the compacted surface prior to placement of the fill. Fill shall be placed on the compacted surface in loose lifts not exceeding 8-inches in thickness. These materials should be moisture conditioned to near optimum and compacted to a minimum of 90% of the maximum density as determined by ASTM Test Method D-1557 (Modified Proctor). Boulders in excess of 8 inches, or greater in size than 3/4 the thickness of the lift, whichever is smaller, shall be removed. All fill should be evenly brought up. Lifts shall be uniform in thickness and moisture shall be evenly mixed throughout the fill. Any portions of previous lifts exhibiting pumping or yielding shall be removed and replaced prior to placement of subsequent lifts.

2.6 COMPACTION

Where compaction is referred to within these specifications or on the design drawings, it shall mean the relative compaction as determined by comparing the in-place dry density to the laboratory maximum dry density as determined by ASTM Test Method D-1557 (Modified Proctor). The field in-place dry density shall be as determined by ASTM D-2922 (nuclear) methods.

During the compaction operation of all fill material, the surface of the fill and the material being placed will be maintained within the moisture content range required (+/- 3%) to permit proper compaction to the specified density. The moisture shall be uniformly distributed throughout each layer.

Compaction tests will be made by the Engineer during the placement of the fill, and optimum moisture content and the maximum dry density will be determined.

The Contractor will furnish and operate the necessary types of equipment required to obtain the specified compacted dry density. After each layer of fill is placed and uniformly wetted, it will be compacted by passing compaction equipment over the entire surface a sufficient number of times to obtain the density specified. The compactive effort shall be uniform and consistent.

The degree of compaction of the placed fill will be determined by comparing field density test results to the Laboratory Maximum Dry Density as obtained by the ASTM D-1557 Test Method. A minimum of 1 compaction test per 200 cubic yards of in-place fill, is recommended. More frequent testing may be justified if deemed necessary by the Engineer, due to special circumstances.

2.6 UTILITY TRENCH BACKFILL

Materials for trench backfill shall consist of imported materials meeting the criteria specified in the drawings and approved by the Engineer, and native materials that are free of organics, rocks exceeding 4-inches in their greatest dimension, or other deleterious substances.

All utility trench backfill shall be compacted to a minimum of 90% of the Laboratory Maximum Dry Density as obtained by the ASTM D-1557 Test Method, except for the final 12-inches measured from the subgrade elevation in areas designated to receive pavements, where 95% relative compaction will govern.

Prior to pipe installations, the proper bedding shall be provided in accordance with the local authority's standards, but shall be a minimum of 6-inches thick that meets the above reference import material specifications. A minimum of 12-inches of protective cover implementing imported materials shall be provided prior to commencement of compaction efforts. Subsequent lifts may employ native materials.

The Engineer shall observe and periodically test the backfill compaction during the underground construction phase to assess compliance with these specifications.