ATTACHMENT VII

GEOTECHNICAL INVESTIGATION
Proposed Single Family Residence
Parkside Dr., APN 426-430-005
Hayward, California

Prepared for:

Mr. Conan Zhang

CNY Inc.

20683 Center St.

Castro Valley, CA 94546

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PLANNING DIVISION

PROJECT NUMBER
201505614 SPR

Prepared by:

HENRY JUSTINIANO & ASSOCIATES

July, 2015

HENRY JUSTINIANO & ASSOCIATES
GEOTECHNICAL ENGINEERING

Project No. Z-107-01
July 28, 2015

Mr. Conan Zhang
CNY Inc.
20683 Center St.
Castro Valley, CA 94546

SUBJECT: GEOTECHNICAL INVESTIGATION
Proposed Single Family Residence
Parkside Dr., APN 426-430-005
Hayward, California

Dear Mr. Zhang:

Our geotechnical report for the proposed new residence at the above subject property, is herewith submitted. The report presents the results of our explorations and geologic literature review, along with our evaluations and recommendations for foundation design, as well as other earthwork related elements of the project.

In our opinion, the property is suitable for the proposed residence, 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
C-042347
STATE OF CALIFORNIA

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

Enclosures

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1.0 INTRODUCTION

1.1 PURPOSE

This report presents the results of our investigation of the subject property, and the review of the published geological data pertaining to the general area.

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 in the Hayward Hills, approximately 0.3 miles to the east of the East Bay, California State University Campus. Specifically, the site lies along the northern side of Parkside Drive, approximately 250-feet to the northwest of its intersection with Hillcrest and Tribune Avenues. The approximate location is illustrated on the site location map, Figure 1.

1.3 SITE CHARACTERIZATION

The property is a rectangularly shaped, densely wooded, vacant, essentially one-acre parcel, with approximately 100-feet fronting Parkside Drive (see Figure 1). In the project vicinity, there are several residences along the common downslope segment of Parkside Drive, with relatively modern neighboring residences, both to the east and west. Topographically, the building area is relatively steep, with a slope gradient of approximately 2.5 horizontal to 1 vertical.

There is a 38-inch oak on the lower portion of the proposed building area that is amongst the many younger sapling oaks and various other trees. At the time of our exploration, there were also remnants of an old foundation on the west side, half way down the proposed building area.

To the north, where the steep downward projecting slope extends beyond the proposed building area, there is densely vegetated terrain, with a hummocky, unstable appearance. Nevertheless, no slope instabilities were noted on the proposed building area or the neighboring residences.

1.4 SCOPE

The scope of our work included a literature research of available and applicable geological data, exploratory boreholes, 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 construct a wood-framed, two-story, single family dwelling.

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 residence. Information from our review of geological maps and exploration program, indicates that the desired building location is within stable terrain and that the site would be feasible for construction of a residence, 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

The site geology has been mapped previously by Graymer et al. (2000, Figure 3) and Dibblee (1980, Figure 4). Graymer indicates the site is underlain by rocks that are part of a Cretaceous-age sequence of sediments that are part of the Joaquin Miller Formation (Kjm), which is described as fine sandstone and shale with minor thin sandstone beds. Graymer's mapping shows a faulted contact immediately southwest of the site that separates Knoxville Formation sandstone and shale from the Joaquin Miller Formation.

Dibblee's mapping indicates the rocks are Cretaceous-aged sediments (Kp) consisting of shale with minor thin sandstone beds. The mapping by Dibblee indicates the rocks strike to the northwest, and locally dip from 45 to 55 degrees to the northeast. The Chabot Fault is mapped by Dibblee several hundred feet southwest of the site.

Data from our subsurface investigation indicates that the underlying bedrock consists predominantly of a tan sandstone that is typically moderately fractured and weak. The presence of this material suggests that localized areas of moderately strong and better indurated rocks may be encountered during construction, especially in areas of deep cuts.

2.2 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.

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 0.726, 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 5).

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	6.7	10.8	6.8
Concord-Green Valley	15.9	25.6	6.9
Hayward	1.2	2.6	7.1
San Andreas (Northern)	19.2	30.9	7.9

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

2.3 SLOPE STABILITY

Mapping by the California Geological Survey (2012, Figure 6) for the State of California Earthquake Zones of Required Investigation, includes the proposed building site within the southern margin of an area labeled as potentially susceptible to earthquake induced landsliding. However, landslide mapping by California Division of Mines and Geology, Majmundar (1996, Figure 7) does not show any landslide or earthflow in the area downslope of the proposed building site.

Previous landslide mapping by Nilsen (1975, Figure 8) does not depict any slides along the top of slope, common to the subject property. During our field reconnaissance and investigation, we did not observe any indication of active or dormant landslide deposits within the proposed building site and there does not appear to be a significant risk of landsliding from off-site areas impacting the site.

Based on the bedrock exposures in the area, the relatively shallow depth to rock and limited soil cover, we consider the risk of slope instability affecting the building 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 relatively low and within 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 June 5, 2014, our Engineer explored the subsurface conditions within the proposed building area, with two drilled borings. The borings were advanced with a portable drill rig utilizing 4-inch O.D. augers, to a maximum depth of 4.5-feet.

The specimens collected during our investigation, consisted of relatively undisturbed samples obtained by advancing into undisturbed soil, a Standard Penetration split barrel sampler. The samples were obtained through the action of a 140-pound hammer falling a distance of 30 inches.

The in-situ strength characteristics of the underlying soil, are indicated by correlating the blow counts required to drive the sampler the lower 12-inches of an 18-inch sample attempt. The soils encountered were examined and logged in the field by an Engineer from this office. The soil profiles are presented as Figures 9 and 10.

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 representative near surface samples of the soils. The testing yielded a liquid limit of 36 with a plasticity index of 19, which corresponds to moderately expansive and creep susceptible clays.

Sieve analyses were conducted to obtain grain size distribution and to classify the encountered stratigraphic layers (Figure 11). In general, the grain size distribution curves, combined with Atterberg limits, classify the near surface soils as sandy clays.

4.0 CONCLUSIONS AND RECOMMENDATIONS

4.1 GENERAL

Our investigation of the proposed building site indicates that stable bedrock materials can be accounted for at depths of approximately 3 to 4-feet. While the California Geological Survey (CGS) mapping is not sufficiently detailed (large scale) to accurately define the limits of the area below the site that it labels as having a significant risk of earthquake induced landslide hazard, generalized maps of this kind are notoriously conservative. In this particular instance, if the subject site were to fall inside the hazards zone area of the map, it would contradict the superior detail map, published by the California Division of Mines and Geology, formerly California Geological Survey (the same agency with a name change by the State Legislature, in 2006) and the referenced USGS, Nilsen (1975) landslide map, which is a very reliable source of existing landsliding potential, as it enjoyed the benefits of less developed landscape during its compilations. In addition, The 38-inch diameter oak and adjoining residences on either side, attest to a stable hillside, during recent times. As such, it is our professional opinion that the site can be regarded stable and suitable to receive the proposed residence. However, the proposed residence design will have to contend with the steep topography.

We believe that the most appropriate foundation system to support the proposed structure, will be a system comprising relatively deep, cast-in-place concrete piers, integrated with grade beams. The piers will offer inherent flexibility to adjust their depths, as warranted by the subsurface conditions encountered during drilling, to insure proper penetration into competent materials.

In order to avoid saturation of foundation bearing soils resulting from surface flows, the site drainage 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 site preparations, 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.

4.2 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}	1.646
S_{d1}	0.685

4.3 SITE PREPARATIONS

According to the information furnished to this office, the proposed residence will be constructed, essentially on the existing grades, with the exception of possible retained cuts into the hillside.

Nevertheless, areas designated to receive improvements should be cleared of all vegetative matter, and the upper 2-inches of soil containing roots, organics, or other deleterious material, should be hauled away from the property.

The loosely backfilled depressions that may result from activities such as the removal of tree stumps, old foundations and other deleterious items, should be over-excavated and backfilled with compacted native soils. It is recommended that backfill be placed in horizontal lifts no greater than 8-inches in loose, uncompacted thickness, and moisture conditioned as necessary prior to receiving compactive effort.

All fill should be compacted to a minimum of 90% of the maximum dry density, as determined by ASTM compaction test designation: D1557-78 (Modified Proctor).

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

4.4 FOUNDATIONS

Geotechnical conditions demand the construction of drilled, cast-in-place reinforced concrete piers that extend into the underlying stable rock materials. 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. In addition to the structural lateral loading, an active lateral force equal to that resulting from an equivalent fluid weighing 50 pcf over the upper 3-feet, acting over a tributary horizontal width equal to 1-1/2 pier diameters, should be considered. This active force may be resisted by a passive force commencing 3-feet below the top of the piers, equivalent to that caused by a fluid weighing 450 pcf. The passive force may be assumed to have a tributary horizontal width equal to 2 pier diameters. In no case, however, should these piers contain less than four No. 4 reinforcing bars, with two bars tied to the top steel bars in the grade beam.

The following table summarizes our recommended criteria for foundation design:

FOUNDATION DESIGN CRITERIA

Pier Diameter	Minimum 16-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 700 psf commencing 3-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, both top and bottom.

4.5 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.

4.6 RETAINING WALLS

It is anticipated that retaining walls may be warranted at the base of cuts into the hillside. Retaining walls designated to support significant volume of fill, are not recommended due to the site steepness and should be avoided.

A retaining wall designated to the base of a cut into the hillside would expose bedrock and 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. A pier support alternative can assume a passive force commencing at the base of the cut, equivalent to that caused by a fluid weighing 400 pcf. The passive force may be assumed to have a tributary horizontal width equal to 2 times the pier diameter.

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 manufacture's specifications.

4.7 DRAINAGE

It is important to direct surface runoff away from the foundation perimeter, 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.

Along the uphill side, slightly above the foundation perimeter, as a minimum, an earth-lined V-ditch with area drains should be provided to capture, collect and transport surface waters around the dwelling.

5.0 GENERAL CONDITIONS

5.1 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.

5.2 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.

5.3 LIMITATIONS

This report has been prepared by HENRY JUSTINIANO & ASSOCIATES for the exclusive use of Mr. Conan Zhang 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.

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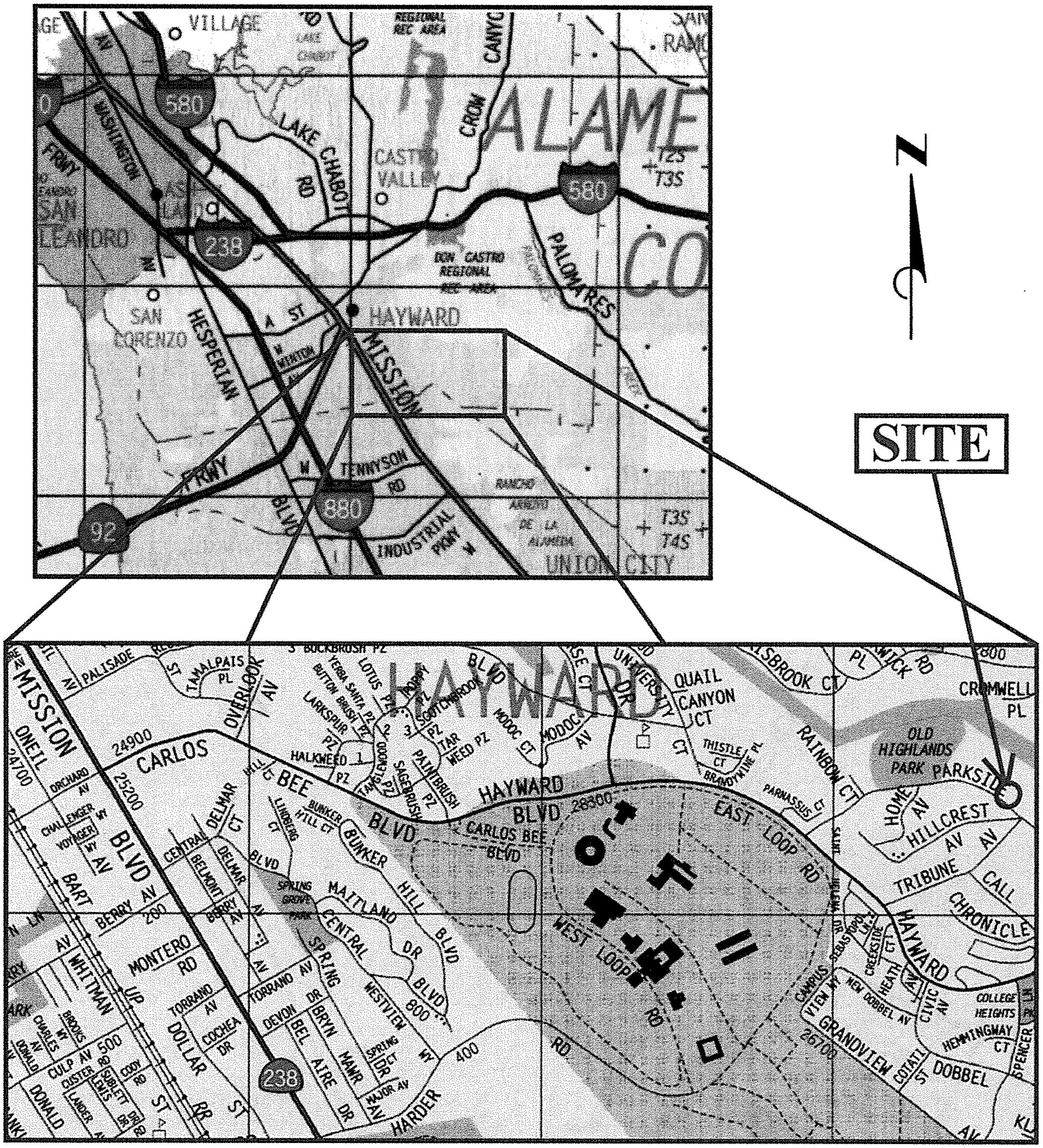
Petersen, et al. (1996, and 2003 Revisions), Probabilistic Seismic Hazard Assessment for the State of California, U.S.G.S. Open-File Report 96-706, D.M.G. Open-File Report 96-08.

Majmundar, H. H., 1996, Landslide Hazards in the Hayward Quadrangle and Parts of the Dublin Quadrangle, Alameda and Contra Costa Counties, California, DMG Open-file Report 95-14



SITE LOCATION

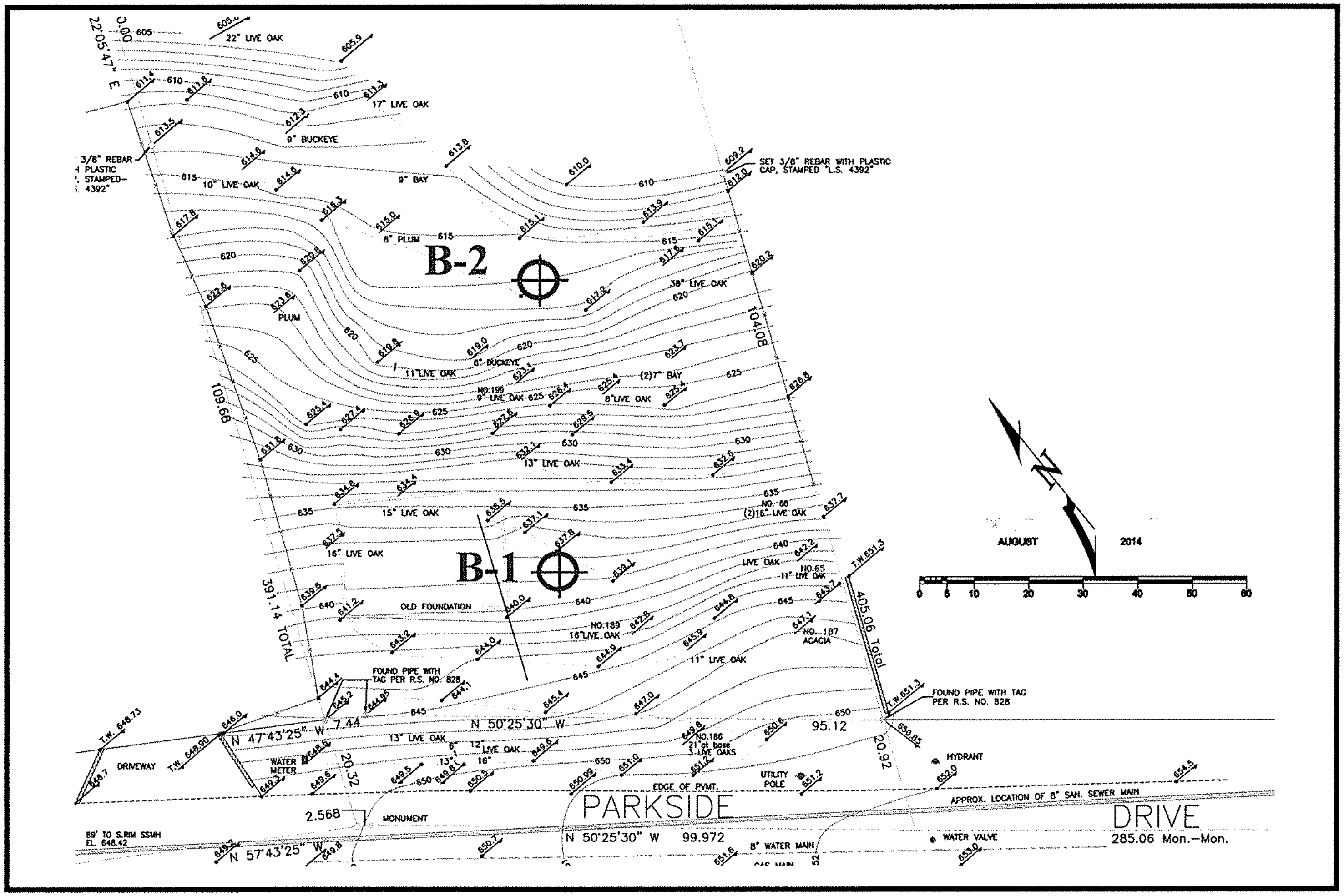
Project No.: Z-107-02	Date: 07-28-15	Scale: NTS

Henry Justiniano & Associates

Soils and Foundation Engineering

Source: Thomas Guide

Figure No. 1



SITE PLAN

Project No. : Z-107-02	Date: 07-28-15	Scale: NTS

Explanation

Approximate Location of Borehole

Source: **Bruce W. Starr, L.S.**

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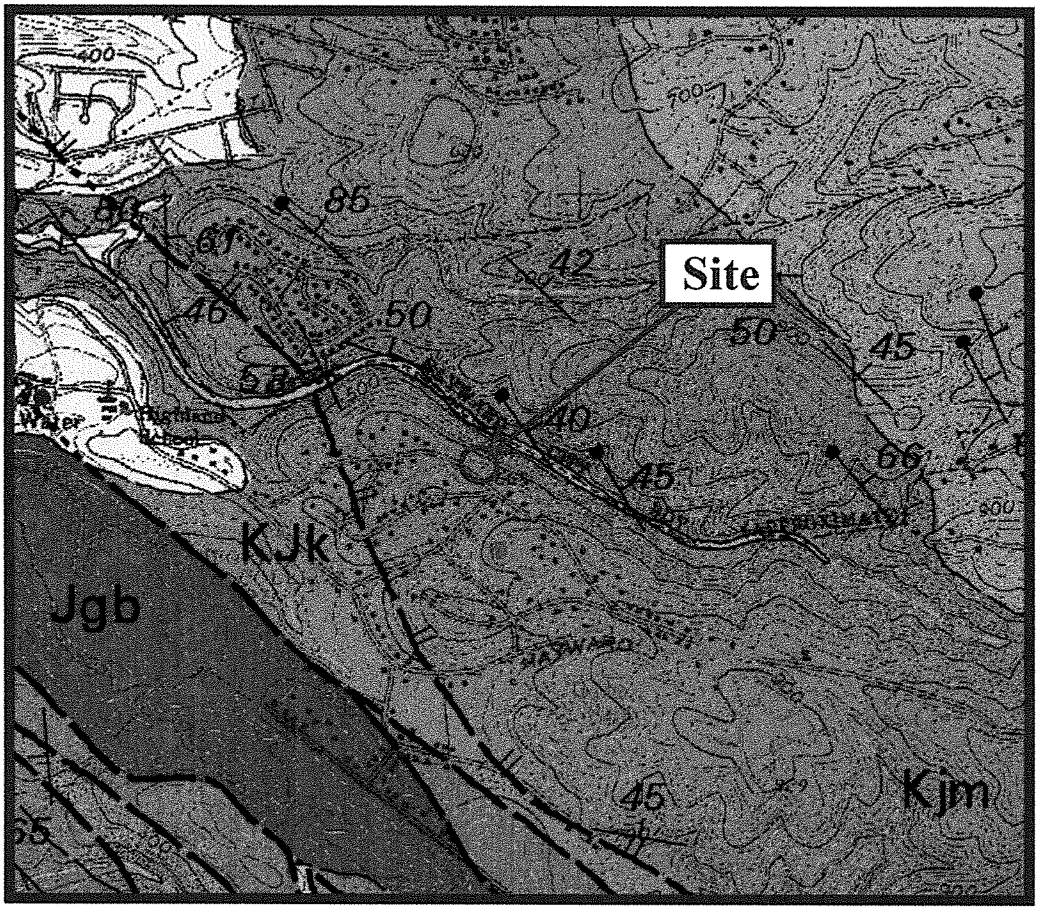
Soils and Foundation Engineering

Figure No. 2



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EXPLANATION

Qpoaf	**Older alluvial fan deposits (Pleistocene)**
Ko	**Oakland Conglomerate (Late Cretaceous, Turonian and/or Cenomanian)**
Kjm	**Joaquin Miller Formation (Late Cretaceous, Cenomanian)**
KJk	**Knoxville Formation (Early Cretaceous and Late Jurassic)**
Jsv	**Keratophyre and quartz keratophyre (Late Jurassic)**
Jpb	**Pillow basalt, basalt breccia, and minor diabase**
Jgb	**Gabbro**

GEOLOGY MAP

R.W. Graymer, 2000

Project No.: Z-107-02	Date: 07-28-15	Scale: NTS

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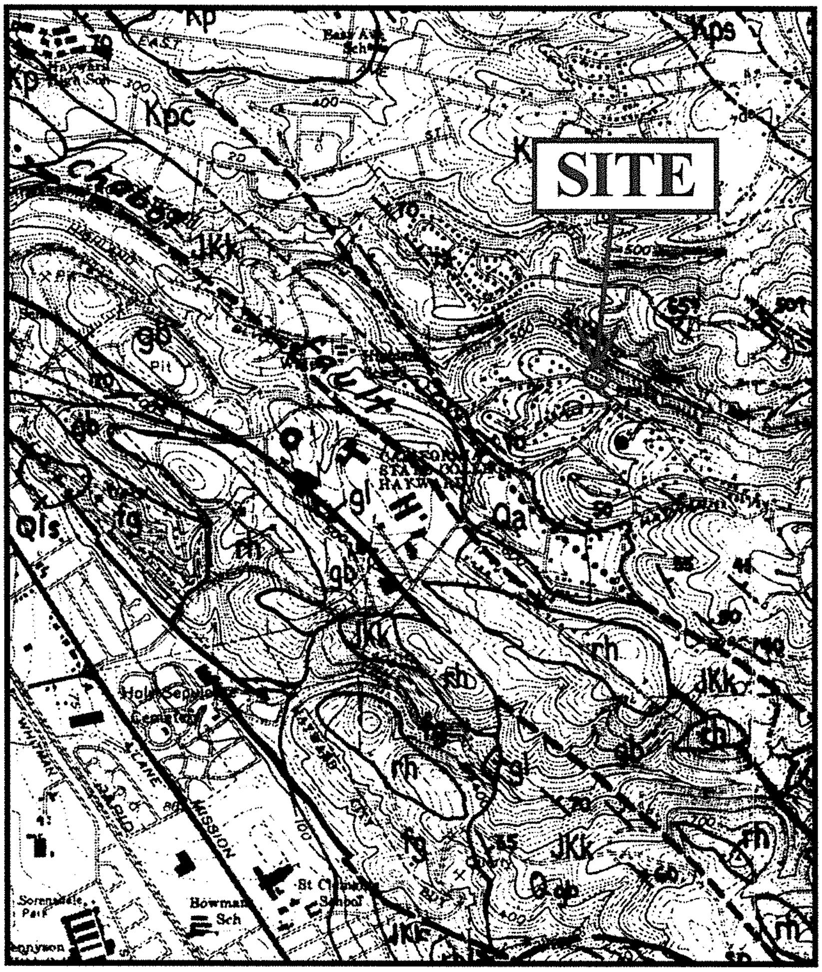
Figure No. 3



EXPLANATION

Contact
dashed where gradational or approximately located

Fault
dashed where inferred; dotted where concealed; queried where existence doubtful; double arrows indicate strike-slip movement; U – upthrown side, D – downthrown side relatively

anticline
syncline
Axis of fold
arrow on axis indicates direction of plunge

inclined
inclined (approximate)
vertical
overturned
Strike and dip of strata

shear zone
sandstone bed

Kp Clay Shale, gray micaceous, argillaceous to silty, minor thin sandstone bed

Kpc conglomerate composed of cobbles of granitic and dioritic rocks, porphyritic rocks, quartzite and black chert in matrix of brown sandstone

gb gabbro-diabase, partly serpentinized

JKk dark, micaceous shale, minor thin sandstone

Qa alluvium

GEOLOGY MAP

Source: **T. W. Dibblee, 1980**

N

Project No. Z-107-02	Date: 07-28-15	Scale: NTS

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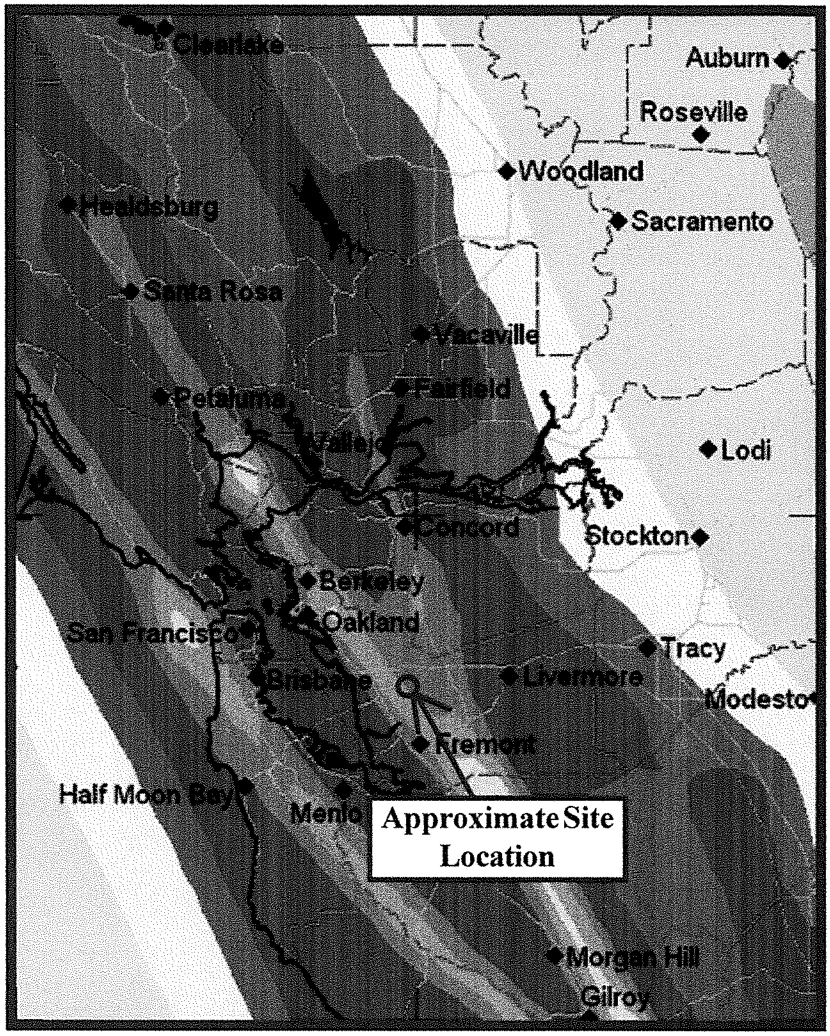
Figure No. 4

Shaking (%g)
Pga (Peak Ground Acceleration)
Firm Rock
- < 10%
- 10 - 20%
- 20 - 30%
- 30 - 40%
- 40 - 50%
- 50 - 60%
- 60 - 70%
- 70 - 80%
- > 80%

The unit "g" is acceleration of gravity.



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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Project No. Z-107-02	Date: 07-28-15	Scale: NTS

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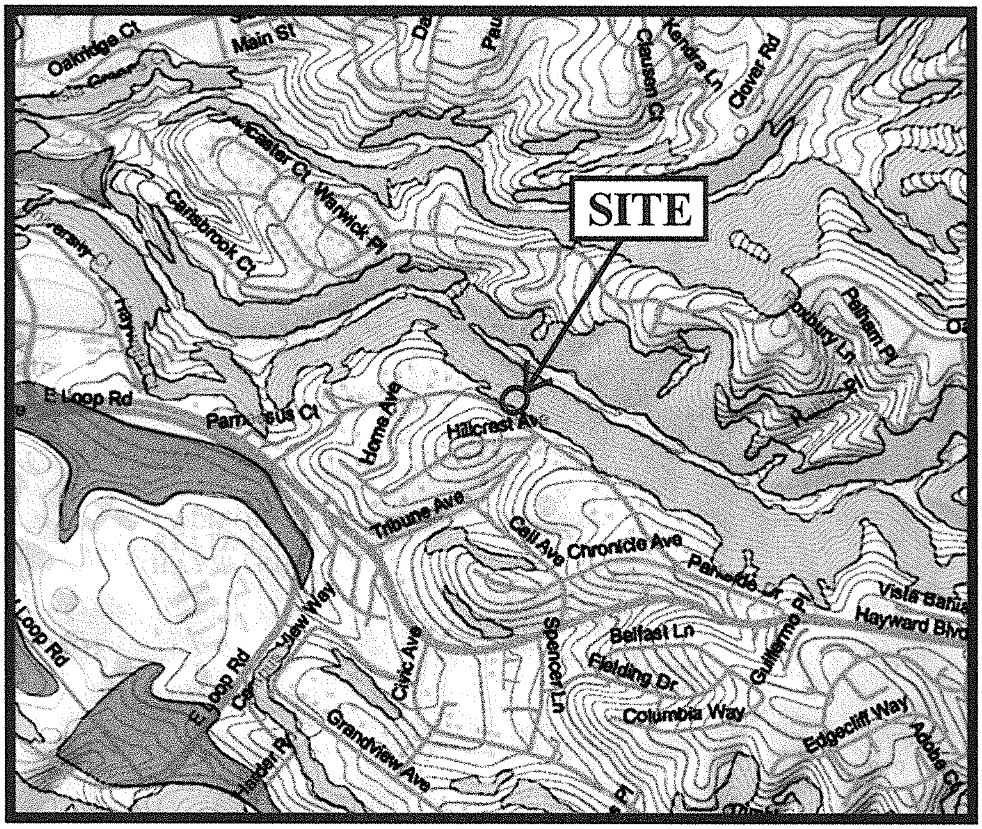
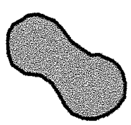
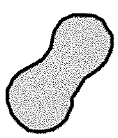
Figure No. 5



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

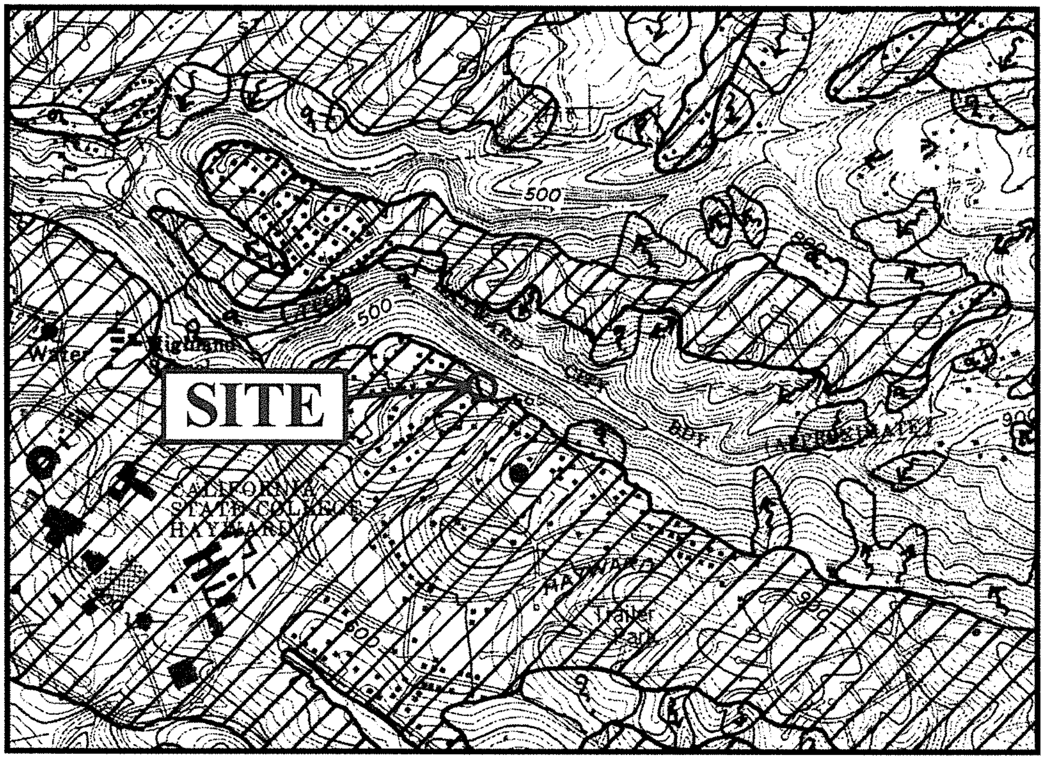
HAYWARD QUADRANGLE OFFICIAL MAP

RELEASED SEPTEMBER 21, 2012 (MODIFIED)

Project No.: Z-107-02	Date: 07-28-15	Scale: As Shown

Henry Justiniano & Associates
Soils and Foundation Engineering

Figure No. 6



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EXPLANATION

DEFINITE or PROBABLE LANDSLIDE. Exhibits several to nearly all of the diagnostic features, including but not limited to headwall scarps, cracks, rounded toes, well-defined benches, closed depressions, springs, and irregular or hummocky topography that are common to landslides and are indicative of mass movement of slope materials. Continuous, single-barbed arrows indicate general direction of movement. Scarp (headwall of slump or block glide) indicated by hachures where mapped.

EARTHFLOW. Relatively shallow deposit of soil or other colluvial material that has oozed downslope, commonly at a rate too slow to observe except over long duration. Source area shown by hachures where mapped. Area immediately upslope of failure typically unravels due to successive small slumps that occur in the oversteepened banks left by movement of the main body away from the source area. Wiggle arrow shows general direction of movement.

Landslide Hazards Map

Source: **Hasmukhrai H. Majmundar, 1996**

Project No. Z-107-02	Date: 07-28-15	Scale: NTS

Henry Justiniano & Associates
Soils and Foundation Engineering

Figure No. 7

EXPLANATION

Landslide deposit
Arrows indicate general direction of downslope movement. Queried where uncertain

Qal
Alluvial deposit

Qt
Alluvial terrace deposit
Queried where uncertain.

Colluvial deposit and/or small alluvial fan deposit

Qaf
Artificial fill

Bedrock
Queried where identification uncertain.

Landslide & Surficial Deposits

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

N

Project No. Z-107-02	Date: 07-28-15	Scale: NTS

Henry Justiniano & Associates
Soils and Foundation Engineering

Figure No. 8

Exploration Boring Log by:

Henry Justiniano
& Associates

Boring Log No.: B-1
Project: Parkside Dr., Hayward
Client: Zhang
Date Drilled: 6/5/14
Equipment Used: Portable Drill, 140 Lb. Hammer, 30 inch Drop
Location: 9' E, 5' N of Eastern End of Concrete Wall

Depth (in Feet)	Other Laboratory Tests	Dry Density (pcf)	Moisture Content %	Blow Count per 12 inch Drive	Sample Number & Type	Groundwater	Description of Material
0–1							Topsoil, Brown, Silty, CLAY
1	**Atterberg Limits** Liquid Limit = 36 Plasticity Index = 19 **Sieve**			36	B-1-A SPT		Tan, Clayey, SAND w/Highly Weathered Sandstone Fragments Residual Soil Moist, Dense
2							
3	**Sieve**			56	B-1-B SPT		Tan, Silty CLAY Highly Weathered Sandstone Moist, Very Dense Borehole Terminated @ 4-Feet
4							
5							
6							
7							
8							
9							
10							

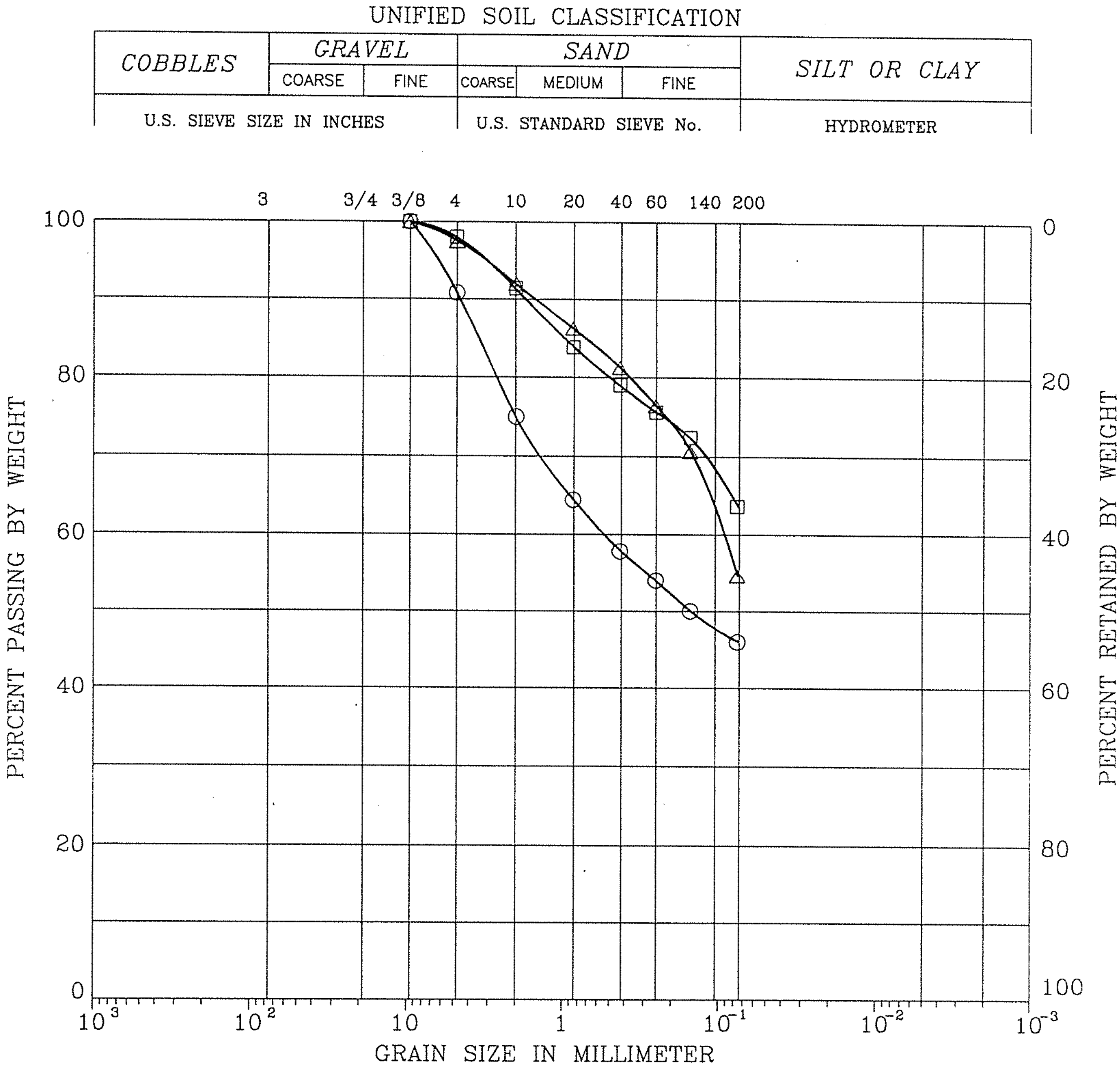
Figure No. 9

Exploration Boring Log by:
Henry Justiniano
& Associates

Boring Log No.: B-2
Project: Parkside Dr., Hayward
Client: Zhang
Date Drilled: 6/5/14
Equipment Used: Portable Drill, 140 Lb. Hammer, 30 inch Drop
Location: 17' West of 36" Oak Lower Building Area

Depth (in Feet)	Other Laboratory Tests	Dry Density (pcf)	Moisture Content %	Blow Count per 12 inch Drive	Sample Number & Type	Groundwater	Description of Material
							Topsoil, Brown, Silty, CLAY
1							Tan, Clayey, SAND w/Highly Weathered Sandstone Fragments Residual Soil Moist, Medium Dense to Dense
				28	B-1-A SPT		
2							
3				73/7"	B-1-B SPT		
	Sieve						Tan, Silty CLAY Highly Weathered Sandstone Highly Fractured Moist, Very Dense Borehole Terminated @ 4.5-Feet
4				50/6"	B-1-B SPT		
5							
6							
7							
8							
9							
10							

Figure No. 10



SYMBOL	BORING	DEPTH (ft)	LL (%)	PI (%)	DESCRIPTION
○	B-1-A	1-2	36	19	CLAYEY SAND (SC)
□	B-1-B	3-4			INORG. SILTS AND CLAYS (ML-CL)
△	B-2-B	3.5-4			INORG. SILTS AND CLAYS (ML-CL)

Remark :

Project No.Z-10702	Parkside, Hayward
H. Justiniano And Associates	GRAIN SIZE DISTRIBUTION Figure No. 11