Update of the
Geotechnical Report

For the
Vacant Lot nest to 2609 Dryden Court
APN 81D-2086-64
Proposed Single Family Residence
Hayward, California

Submitted February 2016
Bijan Mashaw

Update Report in October 2015 by

SUMMIT ENGINEERING

General Civil Engineering and Geotechnical Consultants
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PROJECT NUMBER
201600993 SPR

SUMMIT ENGINEERING

- General Civil Engineering.
- Land Surveying, Parcel Maps, Subdivisions.
- Storm Drain, Sanitary Sewer Design.
- Hydrology and Creek Protection Studies.
- Grading Drainage Plans.
- Soil Reports.

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Bijan Mashaw, Owner
2626 Dreyden Court
Hayward, CA 94542

October 28th, 2015

Re : **Geotechnical Report for the Vacant Lot next to 2609 Dryden Court, Hayward, California.**

Dear Sir / Lady :

We have inspected the subject property above where a new residence will be built in the near future. The land appears to substantially remain unchanged with respect to the condition when the original geotechnical study (Ref. 1) was prepared. Therefore, except for the following additions and modifications, all recommendations in the above geotechnical report remain valid to this day.

INTRODUCTION

This report presents the results of a supplemental investigation and update of the original geotechnical study (Ref. 1) of a vacant parcel adjacent to 2609 Dryden Court in the city of Hayward, California (Fig. 1). .

PROPOSED CONSTRUCTION

The current owners plan to build a new, 3000-ft2, two-storey over garage, single-family home. The new foundation will consist of, either a drilled-pier and grade-beam system, or shallow footings, provided all elements to be embedded in the underlying dense, sandstone bedrock.

SCOPE

The scope of this investigation included:

1. A geologic reconnaissance of the surrounding area;
2. A review of the original soils report, as well as other reports from our files relevant to the site;
3. Prepare an additional report to supplement, update and/ or revise the original report.

SITE TOPOGRAPHY

The lot's front portion has a steep up-sloping topography from the road, having a gentler 50% gradient in the mid and rear areas.

FIELD INVESTIGATION

Field investigation included the drilling of three site borings as shown in Ref. 1 Borings, B-1, B-2, and B-3, were ended to refusal at depths of 2.5 feet, 6 feet, and 3.5 feet respectively. Depths to bedrock were 2 feet, 5 feet, and 3 feet respectively.

SEISMICITY

The seismically-active Hayward Fault is mapped 0.83 miles SW of the site (Figure 2), which falls outside the Alquist-Priolo Special Studies Zone. The equally active Calaveras, Concord, and San Andreas Faults are located 6 miles E, 16 miles NNE, and 19 miles SW of the site, respectively. Seismic stability maps show the site outside areas of seismic liquefaction or seismic slope failure.

Additional 2013 California Building Code (CBC) Seismic Parameters.
The existing soil profile is classified as a **B** soil type, i.e. ' Rock'. Site coordinates are 37.650267 deg N, 122.042422 deg W (NAD27). The USGS ground response curves are shown in the following page. The proposed building will have a II occupancy category. Since S1 = 0.672g < 0.750, the new building will have a **D** seismic design category. See the Lateral Earth Pressures Section for seismic pressures.

CONCLUSIONS

Based on our field and office studies, it is our opinion that from a geotechnical engineering standpoint, the site is suitable for the proposed new residence, provided that the recommendations given in the report and this update are incorporated into the design and construction of the proposed structure.

It must be emphasized that in order to minimize differential settlements, and provide greater seismic stability, the new foundation supports should be extended to reach hard bedrock. The new supports may consist of either footings, or concrete drilled pier and grade beam systems with a 4-foot minimum length of embedment in the Hard bedrock which means a minimum 9-foot pier length for the primary structures from the original ground surface.

RECOMMENDATIONS

Because the recommendations are partly general and partly specific to certain items of concern identified above, recommendation implementation should be discussed with SUMMIT ENGINEERING, including :

USGS Design Maps Summary Report

User–Specified Input

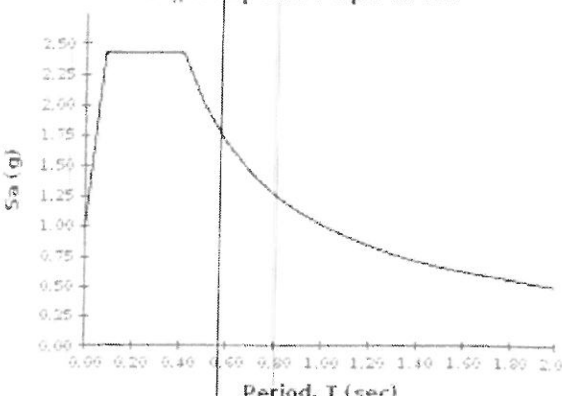
Report Title	Lot Adjacent to 2609 Dryden Ct, Hayward CA Mon November 9, 2015 18:33:57 UTC
Building Code Reference Document	ASCE 7-10 Standard (which utilizes USGS hazard data available in 2008)
Site Coordinates	37.65027°N, 122.04242°W
Site Soil Classification	Site Class B – "Rock"
Risk Category	I/II/III

USGS–Provided Output

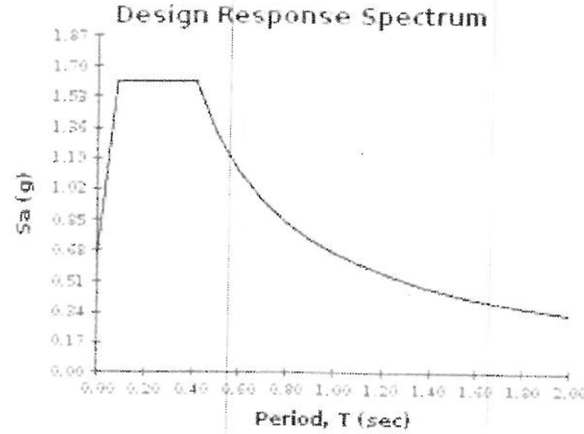
S_S =	2.423 g	S_{MS} =	2.423 g	S_{DS} =	1.615 g
S_1 =	1.008 g	S_{M1} =	1.008 g	S_{D1} =	0.672 g

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

MCE_R Response Spectrum



Design Response Spectrum



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

- Review the foundation, grading, and drainage plans prior to construction.
- Update this report if necessary because of observed changes or delays.
- Inspect the excavation operations, particularly those for drilled pier foundations; the placement of fill and backfill materials; and the installation of surface drains and sub-drains behind retaining walls.
- Prepare a Final Soils Engineer's Report that indicates whether construction was done according to expected soils characteristics, or new features were encountered which required special engineering considerations.

These recommendations are contingent upon SUMMIT ENGINEERING being allowed to inspect and test the grading work, drainage work, and foundation construction. This will allow comparison of the exposed subsurface soil conditions with those assumed in preparation of this report.

C. Drilled Pier Foundations

Drilled, friction piers may be used for the house foundation. All piers should have a minimum diameter of 16 inches. Piers for the primary foundation should penetrate a minimum of 4 feet into sandstone bedrock, with a minimum pier length of 8 feet. Pier lengths may be 6 feet for secondary structures (stairs, etc.). . The following summary table must be used for pier design :

Table 1 - Soil Parameters for Foundation Design

Depth (ft)	Soil Charact.	Skin Frict (psf)	Pullout Res.(psf)	Passive Resistance (*)	Bearing Pressure (ksf)
0					
	Fill Soils Disregard (Creep Layer)	0	0	0	
≥ 4					4+
	Sandstone/ Shale	600	300	500	

The depth to bedrock, 4 ft, is an average, as it likely varies between 2.5 and 6 feet at the site. The (*) means passive resistance is applied over 1.5 pier diameters. The allowable bearing pressure, qa, inside the bedrock mass is given by :

$$qa = 4 (1 + 0.1 (H)) \text{ ksf}$$

and H = pier length (ft) inside the bedrock mass.

When embedded in bedrock, and provided that pier bore-holes will be clean from all debris, piers may be designed to account for the end-bearing effect, with an allowable bearing pressure of 4 kips per square foot, plus 10 % per foot of penetration starting from the top of bedrock, to a maximum of 12 ksf for dead plus live load, with a one-third increase for all loads including wind or seismic. With end-bearing effects included, the total pile capacity will be calculated as follows :

Pier capacity = Shaft resistance + Tip resistance / 3

The following pier capacities have been calculated for foundation design (Linear interpolation is acceptable for intermediate depths). For given pier loads, pier length and diameter may be obtained quickly from this table :

Table 2 - Vertical Pier Capacities in kips (1 kip = 1,000 lbs)

D (in)	H(ft) 4	6	8	10	12	14
16	12	18	23	29	34	39
18	14	20	26	33	39	45

where :

D = Pier diameter in inches

H = pier length (ft) inside the bedrock mass.

Creep Pressures on Piers

Creep pressures will be accounted for by designing piers drilled thru the Creep Layer to resist a rectangular lateral pressure diagram equal to 60 psf thru the assumed 4 feet. Then, the Total Creep Force = 4 x 60 = 240 lbs/ foot of pier width, applied 2.0 feet below ground surface. Creep pressures will be applied over one pier diameter. Passive Soil Resistance, (acting only below the Creep Layer of 4 feet), will be applied over 1.5 diameters and it is given on Table 1 above and in Section H below (Figure 5). If the flagpole formula is used for pier bending, assume the pier is restrained at the top.

Pier Construction

All piers must be tied together with tie beams or grade beams that extend up and down the slope between the piers as well as across the slope between the piers. We recommend to design pier-grade beam connections to resist bending. The spacing of the piers should be determined by the Structural Engineer, but pier center-to-center spacing should not be closer than five pier diameters. Pier foundations become more efficient when using fewer, widely spaced piers. We recommend to extend pier re-bars continuously into the grade beams, and connect to top grade beam re-bars, rather than cutting the steel re-bars at the pier top and using dowels

All pier holes should be dry and reasonably free of loose cuttings and falling debris prior to installing reinforcing steel and placing concrete. Some of the pier holes may encounter different soil conditions that assumed thru their design depths; such piers will be evaluated on an individual basis at the time of construction.

In addition, care must be taken during the pier hole drilling operation to verify that boulders or locally unconfined rock outcrops are avoided. Pier boreholes should preferably not be left un-poured over 24 hours and by no means over 48 hours. If water is encountered in any of the pier excavations, pumping may be required to remove the mud from the holes. If open boreholes are caving-in, a drill-and-pour technique should be implemented.

D. Shallow Foundations

Footings may be used for secondary structures, or for the primary foundation and as a supplement to drilled piers for primary retaining walls, provided that all footings rest on bedrock. Retaining walls may be designed using piers for vertical and horizontal (passive) loads supplemented by short footings of, say, 2 to 4 feet to provide a rigid bending joint at the base of the wall. The footings may be designed for maximum allowable bearing pressures as follows :

	Depth of Soil Cut (ft)	Allowable Pressure (psf)
	2.0	1,200
	3.0	1,500
On Claystone	$\geq$ 4.0	3,000

Alternatively, foundations for the primary retaining walls reaching sandstone bedrock may be designed using footing/ keyway combinations. Keyway design may use a passive resistance of 300 pcf-efw.

Passive pressures are assumed to be exerted by a horizontal soil mass. Where the soil mass slopes away from the foundation, passive pressures are assumed to be fully mobilized at depths where a horizontal line intercepts the slope at least 5 feet from the foundation. At shorter distances, a linear interpolation is acceptable (Figure 5A).

The allowable bearing pressures may be used for dead plus live loads, with a one-third increase for all loads including wind and seismic. The allowable bearing pressures are net values; therefore, the weight of the footings can be neglected for design. Footing friction resistance on bedrock is given by 0.40 x dead weight and it may be used in addition to passive resistance.

G. Lateral Earth Pressures

Back-drained retaining walls must be designed for the following active soil pressures :

Wall Type	Active Pressure(pcf-efw)(*)
Unrestrained, Slope ≤ 4:1	40
Unrestrained, Slope < 2:1	50
Unrestrained, Slope < 1.5:1	60
Restrained	Add an additional uniform lateral pressure equal to 8 x H (psf), where H = height of backfill above retaining wall foundation in feet.

(*) Pounds per cubic foot-equivalent fluid weight

- A 30% pressure increase should be used for shoring design.
- For unrestrained walls, add an additional uniform pressure equal to one-third the maximum surcharge load applied to the wall backfill.
- For restrained walls this additional uniform design pressure should be one-half the maximum surcharge.

Construction equipment and transport vehicle surcharges should also be anticipated in design, and the construction specifications should provide for adequate concrete curing time before allowing the back-fill to be compacted. Sliding friction and passive resistance should be taken as discussed in Section H below.

Retaining walls behind non-living spaces should have a wall drainage system including conventional drain material (such as Miradrain), and a sub-drain installed as a "drainage burrito". Retaining walls behind living spaces must be drained and thoroughly water-proofed, preferably by hot-mopping, or better yet, with sealing synthetic membranes such as PermaSeal, Bituthene, Miradri, HLM 5000, or equivalent.

Lined surface ditches must be provided behind any wall having an exposed sloping surface draining towards it. These ditches will collect runoff water from the slopes and should be sloped to drain to suitable discharge facilities. The top of the walls should extend at least 6 inches of free board above the ditch in order to retain minor erosion or sloughing materials. Do not introduce surface runoff into perforated sub-drains (Fig. 6).

The walls can be best supported on pier/ footing combinations designed to take bending stresses in accordance with the re-commendations presented previously under Section C, "Drilled Pier Foundations" and Section D, "Shallow Foundations". Lateral load resistance can be developed in accordance with the recommendations given below.

Seismic Earth Pressures

According to the 2012 International Building Code, Section 1803.5.12, the determination of seismic pressures will be limited to retaining walls higher than 6 ft. For seismic retaining wall pressures for Category **D** Design and level backfill, consider the following :

Per the California Building Code (Ref. 4), $PGA_M = F_{PGA} \; PGA$

From Table 11.8.1, $F_{PGA} = 1.0$

Take $PGA = S_{D1} = 0.672$ g (from attached USGS spectral curves)

From Table 6.17 (Ref. 5), calculate dynamic earth pressure coefficients at time of maximum dynamic both wall moments and earth pressures on stiff and flexible walls.

For Stiff Walls, $\underline{\wedge}$ Kae = 0.342
For Flexible Walls, $\underline{\wedge}$ Kae = 0.232

Assuming a 120 pcf backfill, the seismic pressure increases are :

For Stiff Walls, $\underline{\wedge}$ p = 41 pcf
For Flexible Walls, $\underline{\wedge}$ p = 28 pcf

To account for sloping backfill, often imposed by lot topography, use the following table which is an extension of the Los Angeles Code for non-expansive backfill (Ref. 6) :

Backfill Slope	Coefficient, C
Level	1.00
5 : 1	1.13
4 : 1	1.17
3 : 1	1.27
2 : 1	1.43
1.5: 1	1.62

Therefore,

For Stiff Walls, $\underline{\wedge}$ p = C x 41 pcf
For Flexible Walls, $\underline{\wedge}$ p = C x 28 pcf

These seismic stresses will be added to corresponding active earth pressures. Remember that for static and dynamic load combinations, the soil parameters may be increased by 1/3.

LIMITATIONS

The recommendations presented herein are based on the soil conditions revealed by test borings and laboratory procedures according to generally accepted geo-technical engineering principles and practices. This warranty is in lieu of all other warranties either expressed or implied.

It must be understood that for this report to be valid, the owner should ensure that necessary steps are taken to carry out the recommendations of the report in the field. Any added risk incurred by the choice of alternative construction methods which depart from our recommendations will be borne by the owner. Further, this report must not be construed as any guarantee or insurance against any type of soil failure.

The recommendations in this report are general in nature and are subject to adaptation or revision as the construction circumstances warrant. We should be notified for additional recommendations should unusual situations be encountered during construction. We may be consulted for supplemental advice, or to provide assistance in interpreting our findings and recommendations, or to inspect various aspects of construction.

Our recommendations are valid as of the present time. However, future conditions may change due to legislation, improvement of engineering knowledge, natural process, or man's works. Therefore, this report is subject to review and its validity may decrease with the passage of time.

Finally, careful design and construction cannot guarantee that damage will not occur if a disaster strikes. Disaster may strike, for instance, in the form of a destructive landslide, or a significant, nearby earthquake. The owner alone undertakes such risk, and therefore, the owner should obtain home insurance if available against landslide and earthquake damage.

Please, call us with any questions.

Sincerely,

AGM

Al G. Masso
GE-2089

REFERENCES

1. Geotechnical Engineering, Inc. (1990), "Report, Soil Investigation, Planned Single Family Residence Adjacent to 2609 Dryden Court, Hayward, California". July 23, 1990.

2. State of California, 1982, Special Studies Zones, Hayward, Official Map. Effective January 1, 1982.

3. State of California, 2003, Seismic Hazard Zones, Official Map, Hayward Quadrangle, July 2, 2003.

4. California Building Code, 2013 Edition, ASCE Standard ASCE / SEI 7-10.

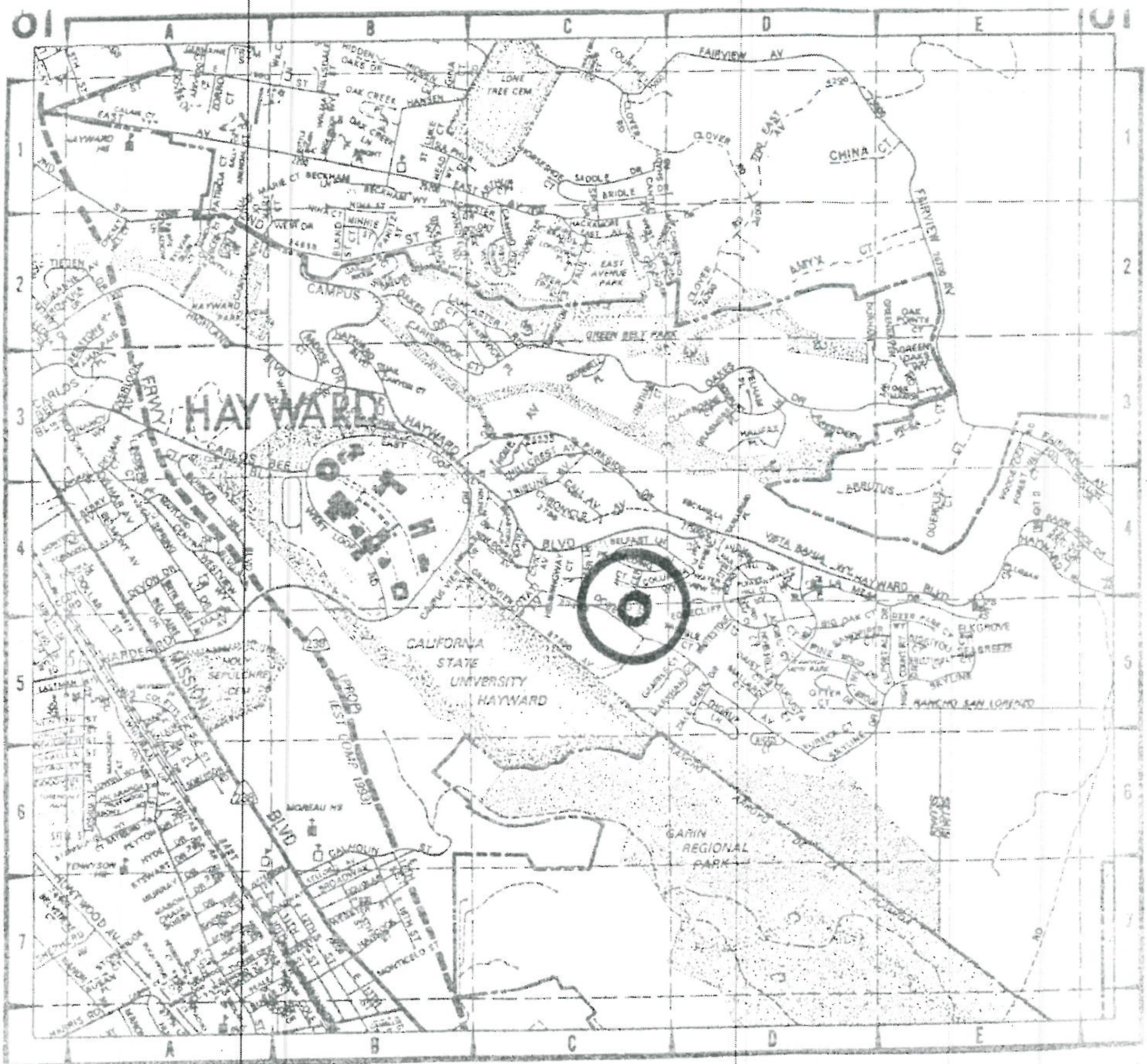
5. "Experimental and Analytical Study of the Seismic Performance of Retaining Structures" (Al Atik and Sitar), PEER Report 2008/14, Pacific Earthquake Engineering Research Center, University of California, Berkeley, March 2009.

6. "Residential Code Manual, County of Los Angeles, Department of Public Works, Building and Safety Division, Based on the 2011 LACRC", Article 1, 10/25/12.

7. Boundary and Topographic Survey by Hendrik Van Depol, June 12, 2015.

8. SUMMIT ENGINEERING, 2014, Geotechnical Report for Four Lots Between Overlook Avenue and Tamalpais Place, Hayward , California, April 17, 2014.

9. SUMMIT ENGINEERING, 2005, Geotechnical Report for Parcel 2, Parcel Map 3280, New Dobbel Avenue, Hayward , California, May 25, 2005.



Ref: Thomas Brothers Map
Alameda County.

Scale : 1" = 2,200'

FIGURE 1 - SITE LOCATION

SUMMIT ENGINEERING

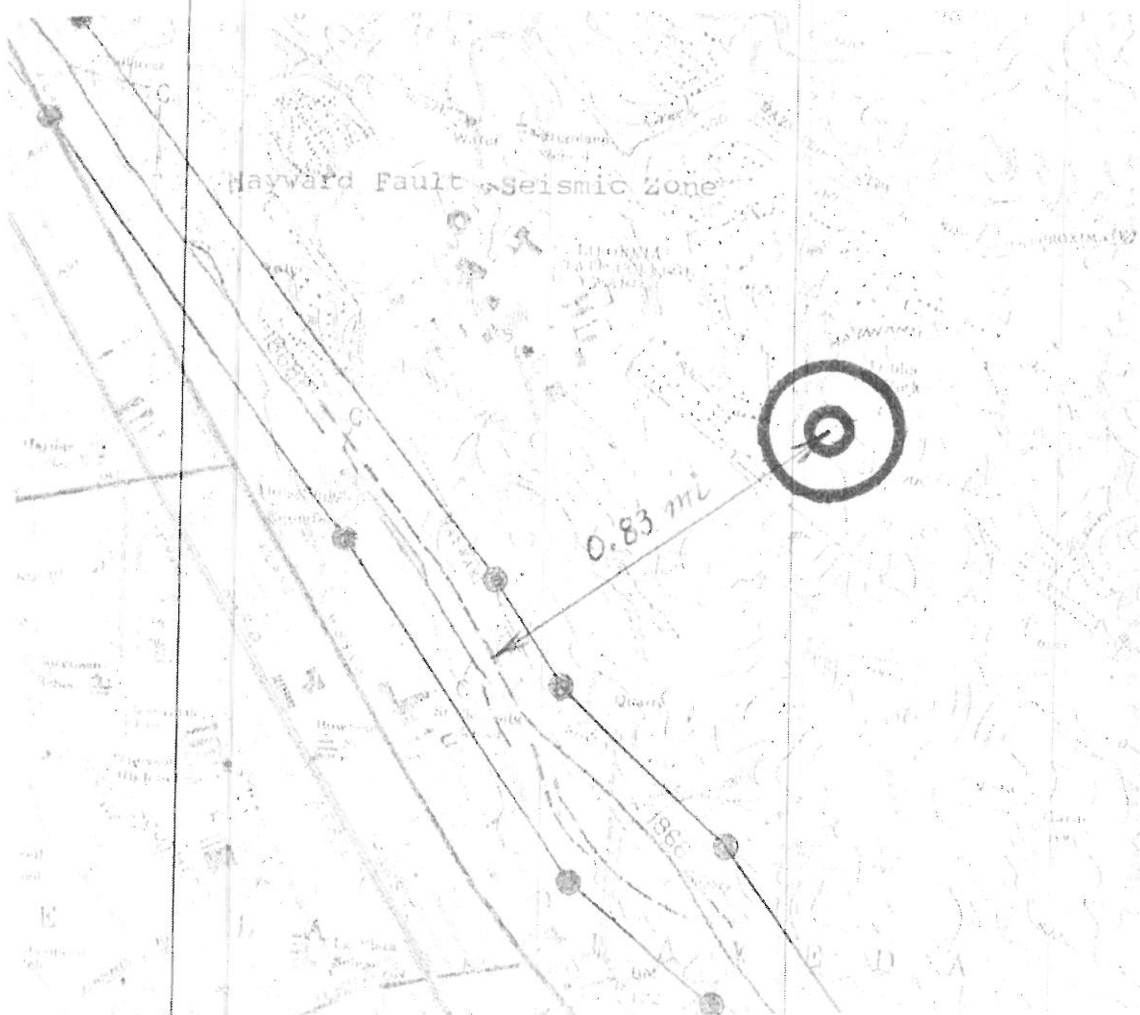
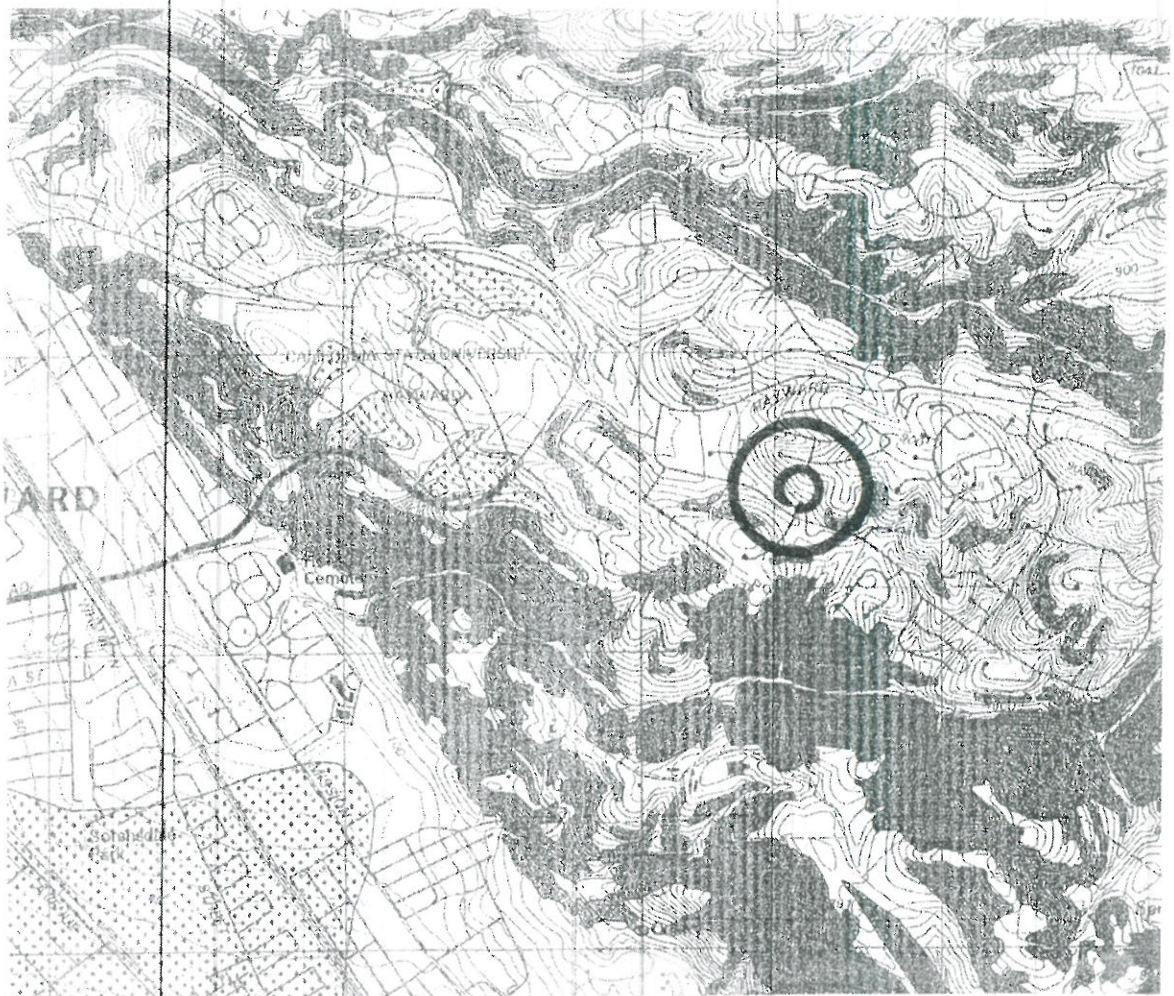


FIGURE 2 - LOCAL SEISMICITY

SUMMIT ENGINEERING



MAP EXPLANATION

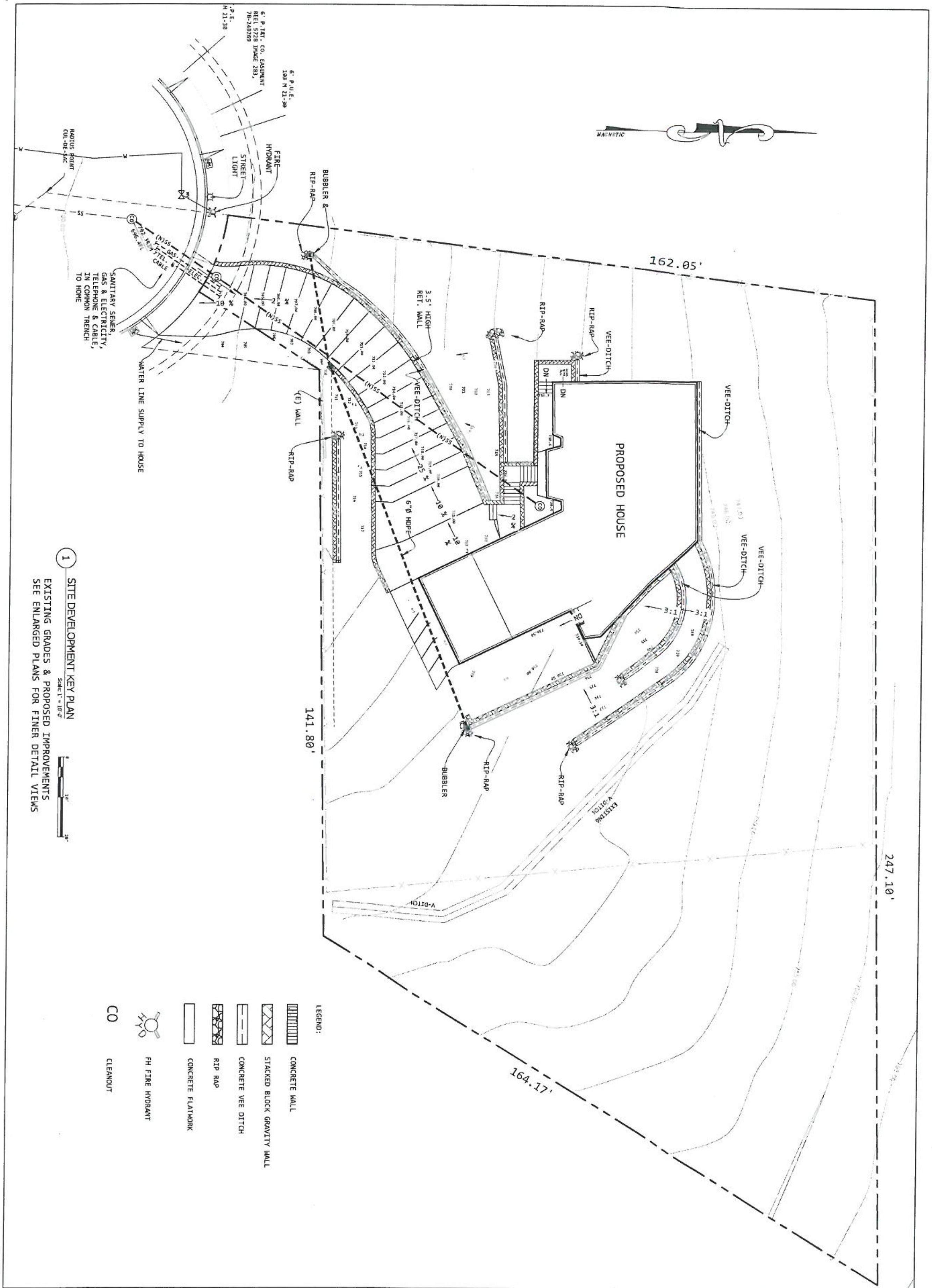
Zones of Required Investigation:

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

Earthquake-Induced Landslides
Areas where previous occurrence of landslide movement, or local topographic, geological, geotechnical and subsurface water conditions indicate a potential for permanent ground displacements such that mitigation as defined in Public Resources Code Section 2693(c) would be required.

FIGURE 3 - SEISMIC LAND STABILITY

SUMMIT ENGINEERING



REVISIONS	BY

1/19/2016

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New Single Family Home
2626 Dryden Court
Hayward, CA
94542

SITE PLAN

DATE: 1-DEC-15
SCALE: AS SHOWN
DRAWN: ---
PAGE:

C1.0

REPORT
SOIL INVESTIGATION
PLANNED SINGLE FAMILY RESIDENCE
ADJACENT TO 2609 DRYDEN COURT
HAYWARD, CALIFORNIA

July 23, 1990

Our Job No. 110793

TABLE OF CONTENTS

SUMMARY

This summary has been prepared only for the general familiarization of the reader with this report. The text of the report should be consulted for design purposes. The scope of the soil investigation was undertaken to explore the general subsurface conditions and provide recommendations for earthwork and foundation design parameters for the planned residence. The results of our investigation can be summarized as follows:

1. Beneath top soils, the site is underlain by stiff silty clay overlying weathered but hard shale.

2. Ground water was not encountered in the exploratory borings.

3. The planned residence and any retaining walls should be supported on properly designed and constructed concrete pier and grade beam foundations, as detailed in the text of this report.

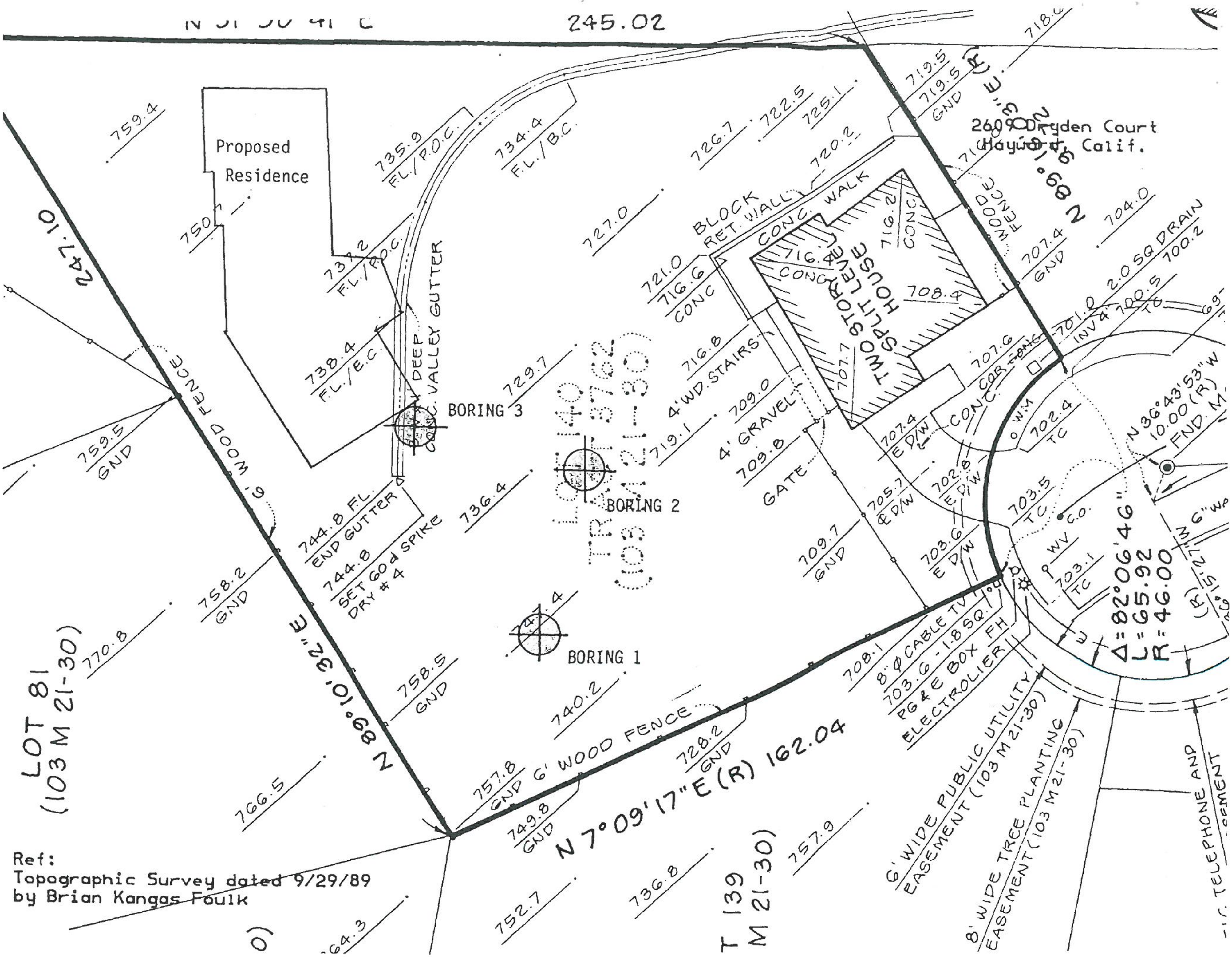
4. An allowable friction value of up to 600 pounds per square foot may be used for design of piers <u>in competent bedrock</u>.

5. The estimated settlements of foundations designed and constructed in accordance with the recommendations of this report are expected to be on the order of 0.5 inch.

6. Depending upon the slope of backfill material, appropriate values of active earth pressure are presented.

7. A passive pressure of 300 pounds per cubic foot for competent bedrock and a coefficient of friction of 0.3 may be used for resisting lateral loads as detailed herein.

8. Criteria for site preparation and compaction, earthwork, drainage and garage slabs are also provided.


245.02
247.10
2609 Dryden Court
Hayward, Calif.
Proposed
Residence
TWO STORY
SPLIT LEVEL
HOUSE
BORING 1
BORING 2
BORING 3
LOT 140
TRACT 3762
(103 M 21-30)
LOT 81
(103 M 21-30)
6' WOOD FENCE
N 89°10'32"E
N 7°09'17"E (R) 162.04
CONC. VALLEY GUTTER
BLOCK RET. WALL
CONC. WALK
4' WD. STAIRS
4' GRAVEL
GATE
END GUTTER
SET 60 d SPIKE
DRY # 4
8" φ CABLE TV
PG & E BOX FH
ELECTROLIER
6' WIDE PUBLIC UTILITY EASEMENT (103 M 21-30)
8' WIDE TREE PLANTING EASEMENT (103 M 21-30)
Δ = 82°06'46"
L = 65.92
R = 46.00
N 36°43'53"W
10.00 (R)
FND. M.
2.0 SQ DRAIN
WOOD FENCE
Ref:
Topographic Survey dated 9/29/89
by Brian Kangas Foulk

BORING 1

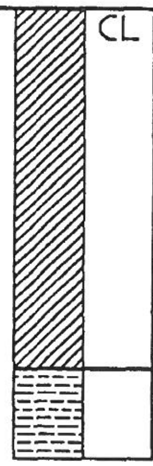
1" Diameter Percussion Hole

Drilled 7/5/90

Depth (Ft.)

Elev. (Ft.)

Elevation 745'



Depth (Ft.)	Elev. (Ft.)	Description
0		6" Top Soil
	745	Brown SILTY CLAY with sand, medium plastic, porous, stiff, average moisture content=14%
1		
	744	Ave. moisture content=12%
2		Brown & Gray SHALE, weathered, hard
	743	Refusal Note: Ground water not encountered
3		

Scale 1" = 1'

LOG OF BORING

GEOTECHNICAL ENGINEERING, INC.

PLATE 2

BORING 2

SAMPLE 1

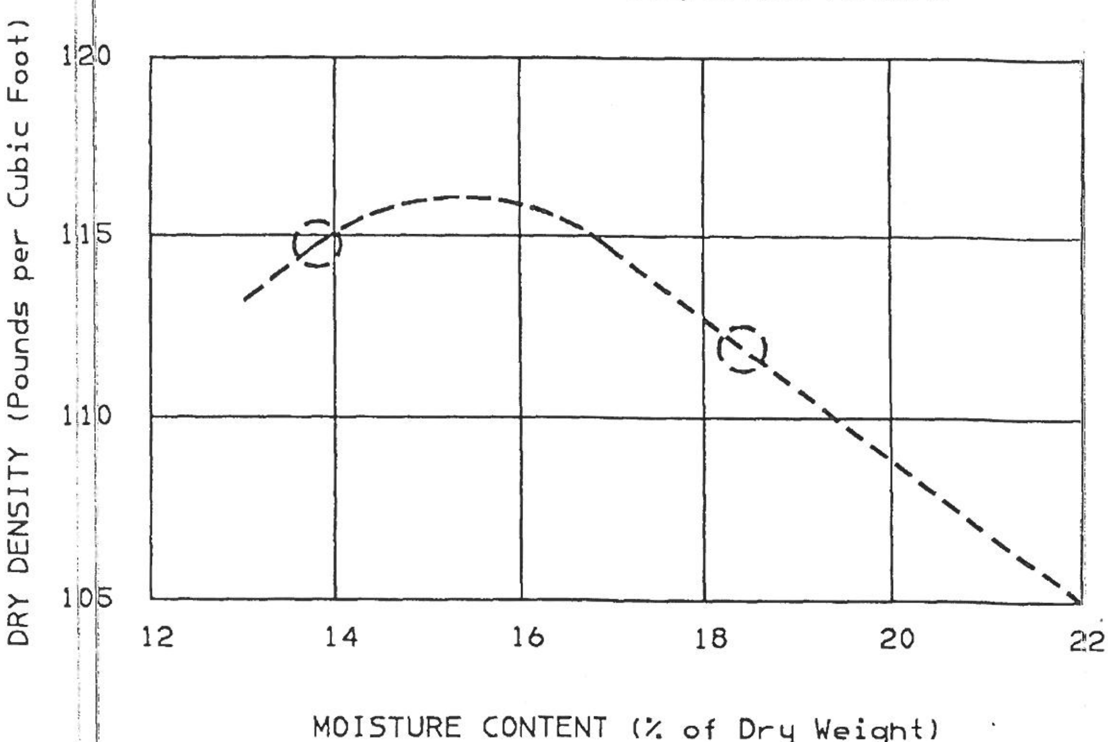
DEPTH 0.5'

MAXIMUM DRY DENSITY = 116 Pounds Per Cubic Foot

OPTIMUM MOISTURE CONTENT = 16 Percent

AMERICAN SOCIETY FOR TESTING & MATERIALS DESIGNATION: D:1557-78

(Modified Proctor Compaction Method)



COMPACTION TEST DATA

GEOTECHNICAL ENGINEERING, INC.