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SHEET NO. ______ OF ______
CALCULATED BY GJC DATE ______
CHECKED BY ______ DATE ______
SCALE TYPE 2 PL WALL

6'-0" MAX RET HT.   W = 3'-10"   Z = 0'-6"

OVERTURNING

		ARM	MOMENT
$W_w = 150(\frac{10}{12})6 = $	750	0.83	622 ft-lb
$W_b = 110(6)2.5 = $	1650	2.58	4257
$W_f = 150(\frac{16}{12})3.83 = $	766	1.91	1467
$W_k = $			
Σ	3166 lb		Σ 6346 ft-lb

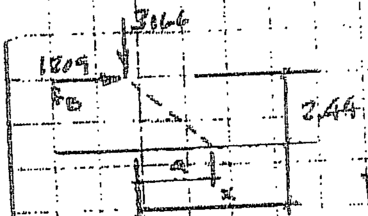
$S.F. = \frac{6346}{2954} = 2.15 \geq 2.0$ ok!

CHECK SLIDING

$F = (.35)3166 + 250\left(\frac{1.5}{2}\right)^2 = 1389^{lb} \leq 1.5(1209_{f_B}) = 1814^{lb}$

$1814 - 1389 = 425(10) = 4247^{lb}/\text{PILE} = 18"\phi$ PILE

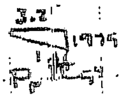
SOIL PRESSURE



$a = \frac{2954}{3166} = .93'$

$x = \frac{6346}{3166} = 2.00 - .93 = 1.07'(3) = 3.20'$

$P_{max} = 3166(2/3.20) = 1979$ psf - SUPPORT TOE OF WALL WITH 18" ϕ PILES



$P_P = 1979(3.2/2)\frac{3.2(2)}{3}(1/3.2-.75) = 2757$ plf $(10') = 27570^{lb}/\text{PILE}$

∴ 18" ϕ × 12' DP @ 10' oc

TOE DESIGN

$M_{MAX} = 1975\left(\frac{.5}{2}\right)^2 = 247$ ft-lb

$A_s = \frac{247(12)1.7}{60000(13 - \frac{.93}{2}).9} = .01 in^2$

USE #4 @ 18" o.c.

HEEL DESIGN

$M_{max} = (660+200)2.5^2/2 = 2688$ ft-lb

$A_s = \frac{2688(12)1.7}{60000(13 - \frac{.93}{2}).9} = .08 in^2$

USE #4 @ 18" o.c.

HORIZ. @ FTG = .0020(12)3.83(16) = 1.47 in²   USE 5 #5 CONT

HORIZ. @ STEM = .0020(10)6(12) = 1.44 in²   USE 7 #4 CONT.

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SHEET NO. 1 OF
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CHECKED BY DATE
SCALE TYPE 2 PL WALL

7'-0" MAX RET HT. W= 4'-6" Z= 0'-6"

OVERTURNING

		ARM	MOMENT
$W_w = 150(7)^{10}/_{12} =$	875	0.83	726 ft-lb
$W_s = 110(7)3.17 =$	2441	2.92	7127
$W_F = 150(^{14}/_{12})4.5 =$	900	2.25	2025
$W_{LL} =$			
	Σ 4216 lb		Σ 9878 ft-lb

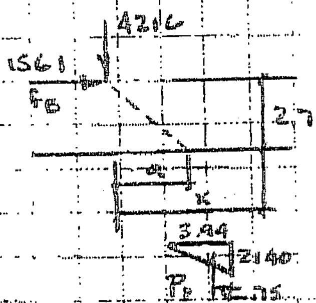
$S.F. = \frac{9878}{4335} = 2.28 \geq$ OK!

CHECK SLIDING

$F = (.35)4216 + 250(\frac{1.5}{2})^2 = 1757^{lb} \leq 1.5(\underset{f_B}{1561}) = 2342^{lb}$

$2342 - 1757 = 585(8') = 4681^{lb}/\text{pier}$ @ 18" ø PIER

SOIL PRESSURE



$a = \frac{4335}{4216} = 1.03'$

$x = \frac{9878}{4216} = 2.34' - 1.03 = 1.31(3) = 3.94'$

$P_{max} = 4216(^2/_{3.94}) = 2140$ psf – SUPPORT TOE OF WALL WITH 18" ø PIER

$P_P = 2140(3.94/2)\frac{3.94(2)}{3}(3.94 - .75) = 3471$ plf $(8') = 27770$ lb/pier

∴ 18" ø × 12' DP @ 8' oc

TOE DESIGN

$M_{MAX} = 2140(\frac{.5}{2})^2 = 268$ ft-lb

$A_s = \frac{268(12)1.7}{60000(13 - \frac{.73}{2}).9} = .01$

USE #4 @ 18" o.c.

HEEL DESIGN

$M = (770 + 200)3.17^2/2 = 4874$ ft-lb

$A_s = \frac{4874(12)1.7}{60000(13 - \frac{.73}{2}).9} = .15 in^2$

USE #4 @ 12" o.c.

HORIZ @ FTNG = .0020(12)4.5(16) = 1.73 in²

USE 6 #5 CONT

HORIZ @ STEM = .002(10)7(12) = 1.68 in²

USE 8 #4 CONT

JAN-10-2002 17:31 FROM-TEMPLETON-LEVERETTE-COMPANY +5106520114 T-492 P.005/009 F-581

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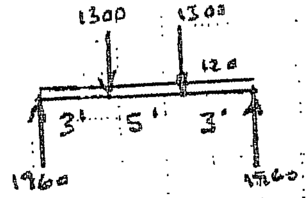
SHEET NO REV OF
CALCULATED BY GJC DATE 2-24-99
CHECKED BY DATE
SCALE

UPPER FLOOR M BD & BATH REV.

HDR @ BD TO DECK
SPAN = 11' w_1 = 120 PLF
P = 1300



M = 1960 (5.5) - 1300 (2.5) - 120 $\frac{(5.5)^2}{2}$ = 5715 #-ft
S = 12M/1000 = 69 in³ DF
2000 = 27 in³ PL

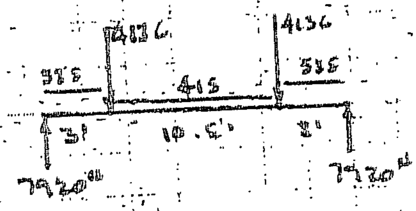
V = 1960 # A = 3V/2(135) = 35 in² DF
285 = 11 in² PL

I = $\left(\frac{5(120)11^3}{384} + \frac{1300(3)(3(11)^2 - 4(3)^2)}{24(11)}\right)\frac{144(360)}{1600000}$ = 224 in⁴

USE 3 1/8" × 11 7/8" PARALLAM OR Fb = 2800

CHECK SHEAR LINE 2
V = $\frac{8285}{24'}$ = 345 PLF 19 OK! LT SIMP HST 27 @ EACH WALL END
P = 1960 + 2176 = 4136 #

SUPPORT BM @ MAIN FLOOR BEE SHT 8 CALCS
SPAN = 16.5'
w_1 = 535 PLF P = 1960 + 2176 = 4136 #
w_2 = 535 - 120 = 415 PLF



M = 7920 (8.25) - 4136 (5.25) - 415 $\frac{(5.25)^2}{2}$ - 535 (6.75) 3 = 27073 #-ft
S = 12M/1350 = 136 in³ GLB
2400 = 116 in³ PL
21600 = 15 in³ STL

V = 7920 # ≈ 7890 # A = 3V/2(165) = 72 in² GLB
285 = 42 in² PL

I = $\left(\frac{5(535)16.5^3}{384} + \frac{4136(3)(3(16.5)^2 - 4(3)^2)}{24(16.5)}\right)\frac{144(240)}{2000000}$ = 963 in⁴ PL
1700000 = 1133 in⁴ GLB
29000000 = 69 in⁴ STEEL

BEAM AS DESIGNED OK! SEE SHT 8

JAN-10-2002 17:31 FROM-TEMPLETON-LEVERETTE-COMPANY +5106520114 T-492 P.006/008 F-501

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JOB
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CHECKED BY DATE
SCALE

CHECK HDR @ CEIL OVER F.P.
SPAN = 4' P=1360# W=128 PLF
M_{max} = 1615 ft# S = 20 in³
V_{max} = 1360(1-.5/4) + 128(4/2) = 1446 lb A = 3V/2(85) = 26 in²

USE 4x10 DF 1 OR BETTER

HDR @ BRG WALL TO RR
SPAN = 4' W_{max} = 400 PLF
M = 400(4)²/8 = 800 ft# S = 12M/1000 = 10 in³ DF
V = 400(4/2) = 800# A = 3V/2(85) = 14 in²

USE 4x8 DF 1

CHECK SUPPORT BM BELOW
RAFT 12
SPAN = 8.5'
W = 702 PLF P₁ = 2510 P₂ = 800 lb

800 2510
702
4.25' 1' 3.25'
4343 4934 lb
V = 0; x = 3.45'

M = 4934(3.45) - 2510(2) - 702(3.45)²/2 = 7232 ft#
S = 12M/260 = 57 in³ PL
V = 4934 lb A = 3V/2(185) = 26 in² PL

3½ x 11⅞" PL OR 5¼ x 9½" PL OK!

EXT. HDR @ RAISED FLOOR (KIT/BRK)
SPAN = 3.5' W = 1016 PLF P_{pt} = 1643 lb
M = 1016(3.5)²/8 + 1643(3.5/4) = 3168 ft# S = 12M/1000 = 38 in³ DF
V_{max} = 1016(3.5 - .75/2)/2 + 1643(2.69/3.5) = 2570 lb A = 3V/2(85) = 45 in² DF
205 = 14 in² PL

USE 6x10 DF 1 OR 3½ x 9½" PARALLAM OR E = 2500

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SHEET NO. E OF
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SCALE

CHECK SHEAR @ LINE D MAIN FLOOR
$\Sigma L = 21$ SAME AS EXIST - DESIGN 21 ∴ $v = 218$ plf
∴ NO CHANGE TO EXIST DESIGN

CHECK SHEAR @ LINE C
$\Sigma L = 21$ $v = \frac{7129}{21} = 339$ plf A or 11 OK!

CHECK HDR @ DIN/KIT
SPAN: 5.5' REF CALC SHT 12
w = 1130 plf P = 5016#
$M = 5016(5.5/4) + 1130(5.5)^2/8 = 11170$ ft-lb $S = \frac{12M}{2600} = 52$ in³
$V = 5016(1/2) + 1130(5.5/2) = 5616$# $A = 3V/2(285) = 30$ in²

USE 5¼ × 9½" PL or 3½ × 11⅞" PL HDR

CHECK HDR @ GARAGE TO GUEST REF 13 CALCS
SPAN = 4' w = 2040 plf P = 5616#
$M = 2040(4)^2/8 + 5616(4/4) = 9696$ ft-lb $S = \frac{12M}{2600} = 45$ in³
$V = 2040(4/2) + 5616(3/4) = 8292$# $A = 3V/2(285) = 44$ in²

∴ USE 5¼ × 9½" PL HDR OK!

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SHEET NO ______ OF ______
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SCALE ______ FAX 510 653 4015

FIELD MEMO

8-19-99

CITY OF OAKLAND
BUILDING INSPECTOR
C/O MARK BECKER
134 DRACENA, OAKLAND, CA.

PROJECT: 6101 CONTRA COSTA
OAKLAND, CA.

ITEM: 3' SECTION OF 10 SHEAR REMOVED AT LINE C
UPPER FLOOR SHEAR WALLS

FOR $V_{WALL} = 4662^{lb}$ $v = 186$ plf $L = 25'$

REVISED $L = 22'$ $v = \frac{4662}{22} = 212$ plf   10 OR 11 OK!

EXIST SHEAR WALL WITH NAILING AT 4" O.C. EDGE AND
12" O.C. FIELD OK AT THIS WALL LINE.
NO CHANGE TO SILL NAILING OR OVERTURNING REQ'D!

GREGORY J. COOK RCE 31570
PROJECT ENGINEER.

8-20-99

PRODUCT ...

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SHEET NO. EF 4 OF
CALCULATED BY GJ DATE 4-16-99
CHECKED BY DATE
SCALE CONTRA COSTA

ROOF BM @ M BD DOOR
SPAN = 16' $w = 40(9^5/2(6)) = 270$ PLF

$M = 270\frac{(16)^2}{8} = 8640$ ft-lb $S = \frac{12M}{1800} = 80$ in³ DF
$\frac{}{2400} = 40$ in³ PL

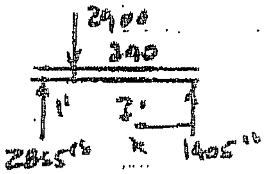
$V = 270(16/2) = 2160$#

$A = 3V/2(95) = 38$ in² DF
$285 = 12$ in² PL

$I = \frac{5(270)16^3(1728)360}{384(1600000)} = 467$ in⁴ DF
$2000000 = 374$ in⁴ PL

USE 6x12 DF #1 OR 5¼x11⅞" P.L. F_b = 2800
OR 3½x11⅞ PL

M BD WDW
SPAN = 4' $w = 40(17/2) = 340$ PLF P = 2900#



$M = 2855(1) - 340\frac{(1)^2}{2} = 2685$ ft-lb $S = \frac{12M}{1000} = 32$ in³ DF
$2600 = 13$ in³ PL

V = 2855# $A = 3V/2(85) = 51$ in² DF
$285 = 15$ in² PL

USE 6x12 DF #1 OR 3½x9¼" PARALLAM OR F_b = 2800

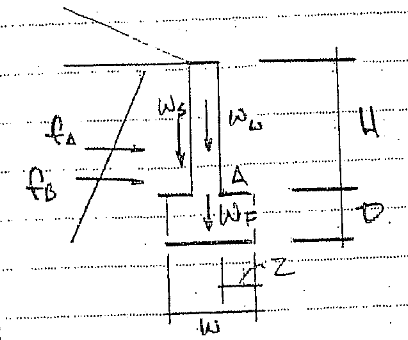
CEIL BM @ BR 2 SPAN = 6' $w = 180$ PLF
$M = 180(6)^2/8 = 810$ ft-lb $S = 12M/1000 = 10$ in³
USE 4x10 DF #1 OR 3½x9¼ PL

FLR JST @ PA PA = 180(8/2) = 720# SPAN = 13'
$M = 720(13/4) = 2340$ ft-lb $S = 12M/2600 = 11$ in³ ML

ADD 1¾x11⅞" JOIST LVL @ EA POST ABOVE

4-16-99

RETAINING WALL DESIGN



EFW = 45 pcf

SOIL BRNG = 1500 psf

fRICT = .35

PPASS = 250 pcf

$f = 45\frac{(H)^2}{2}$ $\quad M = 45\frac{(H)^3}{6}$

WALL HT. H	D	f_A	M_A	f_B	M_B	t_W	d	a $\frac{A_s f_y}{.85(f'_c)b}$	$A_{S\,REQ.}$ $\frac{M(12)1.7}{.9(f_y)(d-a/2)}$	VERT REINF
										#4 @ 18"
3'-0"	16"	202	202	422	609	10"	7"	.21	.01	#4 @ 18"
4'-0"	16"	360	480	639	1136	"	"	.21	.03	#5 @ 18"
5'-0"	16"	563	937	902	1903	"	"	.49	.05	#5 @ 18"
6'-0"	16"	810	1620	1209	2954	"	"	.73	.09	#5 @ 12"
7'-0"	16"	1102	2572	1561	4335	"	"	.73	.15	#5 @ 12"
8'-0"	16"	1440	3840	1959	6091	10	7"	.73	.22	

M_{MAX} @ 18" ϕ PIER FOR KEY = 20769 ft-lb

$V_{4'}$ = 2092" (10'-5.33) = 9770 ft-lb OK

$V_{6'}$ = 6555 (10'-7.33) = 17502 ft-lb OK

$V_{8'}$ = 11784 (10-9) = 11784 ft-lb OK

USE 18" ϕ PIER @ 8' MAX O.C. @ RET WALL

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CALCULATED BY GJC DATE ____
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SCALE ____

4'-0" MAX RET HT. $W = 2'-10"$ $Z = 1'-4"$

OVERTURNING

		ARM	MOMENT
$W_W = 150(10/12)4 =$	500	1.75	875 ft-lb
$W_S = 110(4).67 =$	295	2.50	738
$W_F = 150(16/12)2.83 =$	566	1.42	800
$W_K =$			
	Σ 1361 lb		Σ 2414 ft-lb

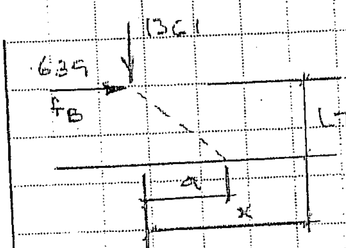
$S.F. = \frac{2414}{1136} = 2.12 \geq 2.0$ OK!

CHECK SLIDING

$F = (.35)1361 + 250\left(\frac{1.33^2}{2}\right) = 697^{lb}$ $1.5(639) = 958^{lb}$ (f_B)

$958 - 697 = 262(8) = 2092^{lb}$ / PIER ∴ 18" ø PIER OK!

SOIL PRESSURE



$a = \frac{1.78(639)}{1361} = .83$

$x = \frac{2414}{1361} = 1.77 - .83 = .94 \geq \frac{2.83}{3} = .94$ OK MID 1/3

$P_{MAX} = 1361(2/2.83) = 962$ PSF < 1500 OK!

TOE DESIGN

$M_{MAX} = 962\frac{(1.33)^2}{2} = 851$ ft-lb

$A_S = \frac{851(12)1.7}{60000(12-\frac{.73}{2}).9} = .03$ in²

USE #4 @ 18" o.c.

HEEL DESIGN

$M = (440+200).67^2/2 = 144$ ft-lb

$A_S = \frac{144(12)1.7}{60000(12-\frac{.73}{2}).9} = 0$

USE NO ADDED REINF

HORIZ @ FTNG = $.0020(12)2.83(16) = 1.09$ in² USE 4 #5 CONT

HORIZ @ STEM = $.002(10)4(12) = .96$ in² USE 4 #4 CONT

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JOB ____
SHEET NO. E4 OF ____
CALCULATED BY GJC DATE ____
CHECKED BY ____ DATE ____
SCALE ____

6'-0" MAX RET HT. W = 4'-0" Z = 2'-2"

OVERTURNING	ARM	MOMENT
$W_W = 150(10/12)6 = 750$	2.59	1943 ft-lb
$W_S = 110(6)1 = 660$	3.5	2310
$W_F = 150(15/12)4 = 800$	2.0	1600
$W_K =$		
Σ 2210 lb		Σ 5853 ft-lb

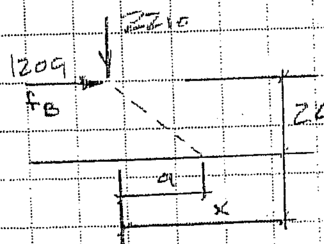
$S.F. = \frac{5853}{2953} = 1.98 \cong 2.0$ OK!

CHECK SLIDING

$F = (.35)2210 + 250\frac{(1.33)^2}{2} = 995^{\#} \leq 1.5(1209_{f_B}) = 1814$

$1814 - 995 = 819(8) = 6555^{\#}$ @ 18"ø PIER

SOIL PRESSURE



$a = \frac{2.44(1209)}{2210} = 1.34$

$x = \frac{5853}{2210} = 2.65 - 1.34 = 1.31(3) = 3.93'$

$P_{MAX} = 2210(2/3.93) = 1125$ psf < 1500 psf OK!

TOE DESIGN

$M_{MAX} = 1125\frac{(2.17)^2}{2} = 2649$ ft-lb

$A_s = \frac{2649(12)1.7}{60000(12 - .73/2).9} = .09$ in²

USE #5 @ 18" o.c.

HEEL DESIGN

$M = (660 + 200)1^2/2 = 430$ ft-lb

$A_s = \frac{430(12)1.7}{60000(12 - .73/2).9} = .02$ in²

USE #4 @ 18" o.c.

HORIZ @ FTNG = .0020(12)4(16) = 1.5 in²

USE 5 #5 CONT

HORIZ @ STEM = .002(10)12(6) = 1.4 in²

USE 8 #4 CONT

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SCALE

8'-0" MAX RET HT. W= 5'-4" Z= 3'-0"

OVERTURNING

	ARM	MOMENT
$W_W = 150(10/12)8 = 1000$	3.42	3420 ft-lb
$W_S = 110(8)1.5 = 1320$	4.58	6045
$W_F = 150(16/12)5.33 = 1066$	2.67	2841
$W_K =$		
Σ 3386 lb		Σ 12306 ft-lb

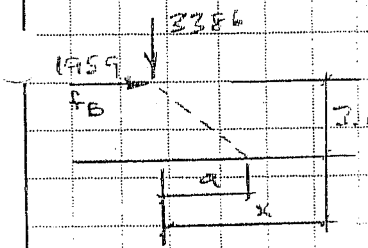
$S.F. = \frac{12306}{6091} = 2.02 \geq 2.0$ ok!

CHECK SLIDING

$F = (.35)3386 + 250(1.5)^2/2 = 1466$ $\quad$ $\leq 1.5(1959) = 2939$ lb (f_B)

$2939 - 1466 = 1473(8) = 11784$ lb @ 18" ϕ PIER

SOIL PRESSURE



$a = \frac{1959(3.11)}{3386} = 1.80'$

$x = \frac{12306}{3386} = 3.63 - 1.80 = 1.83 > 5.33/3 = 1.78$ OK! MID 1/3

$P_{max} = 3386(2/5.33)1.73/1.78 = 1271$ psf ≤ 1500 psf OK!

TOE DESIGN

$M_{MAX} = 1271\frac{(3)^2}{2} = 5717$ ft-lb

$A_s = \frac{5719(12)1.7}{60000(12 - \frac{.73}{2}).9} = .19$

USE #5 @ 12" o.c.

HEEL DESIGN

$M = (880 + 200)1.5^2/2 = 1215$ ft-lb

$A_s = \frac{1215(12)1.7}{60000(12 - \frac{.73}{2}).9} = .04$ in²

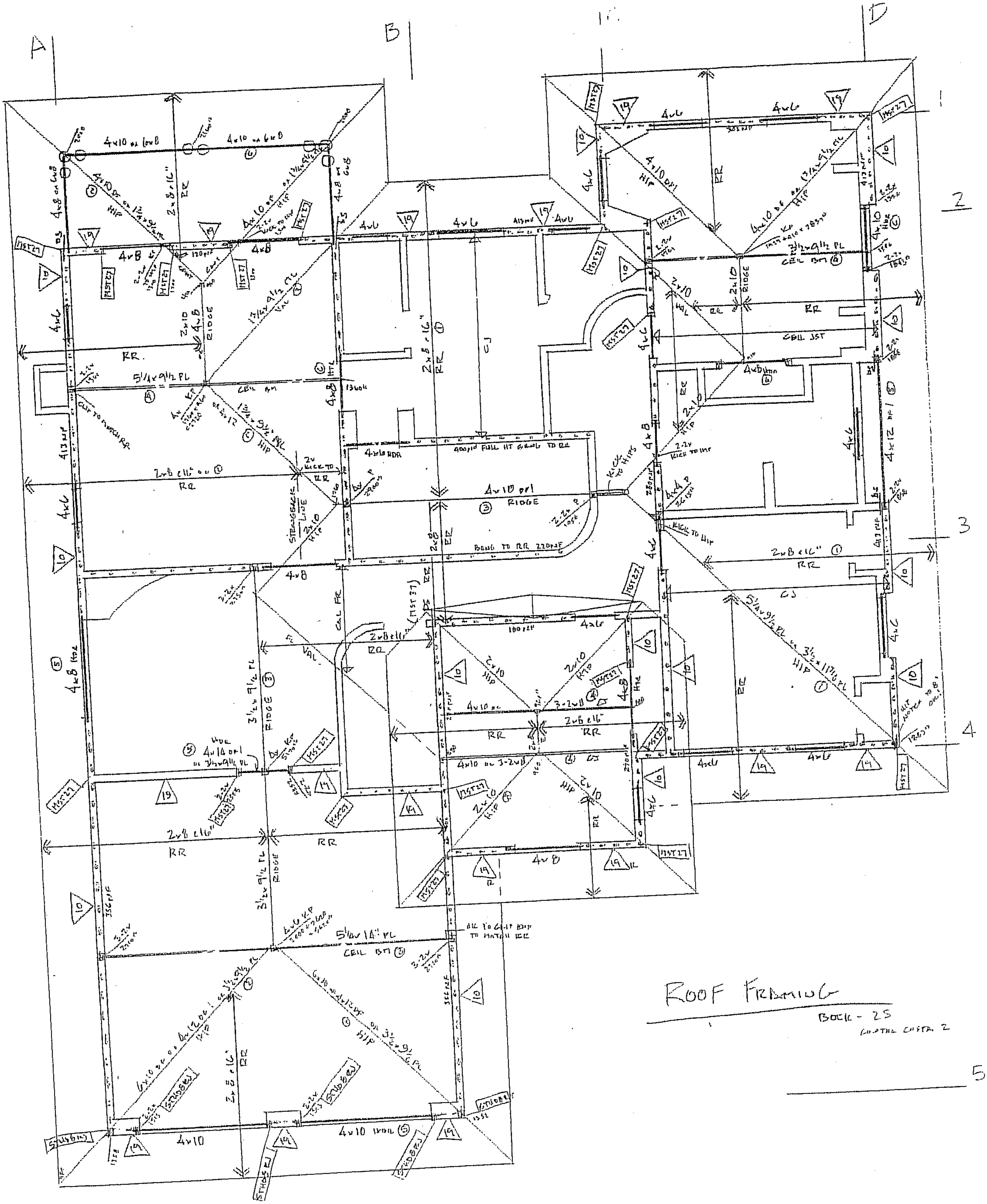
USE #4 @ 18" o.c.

HORIZ @ FTNG = .0020(12)16(5.33) = 2.0 in²

USE 6 #5 CONT

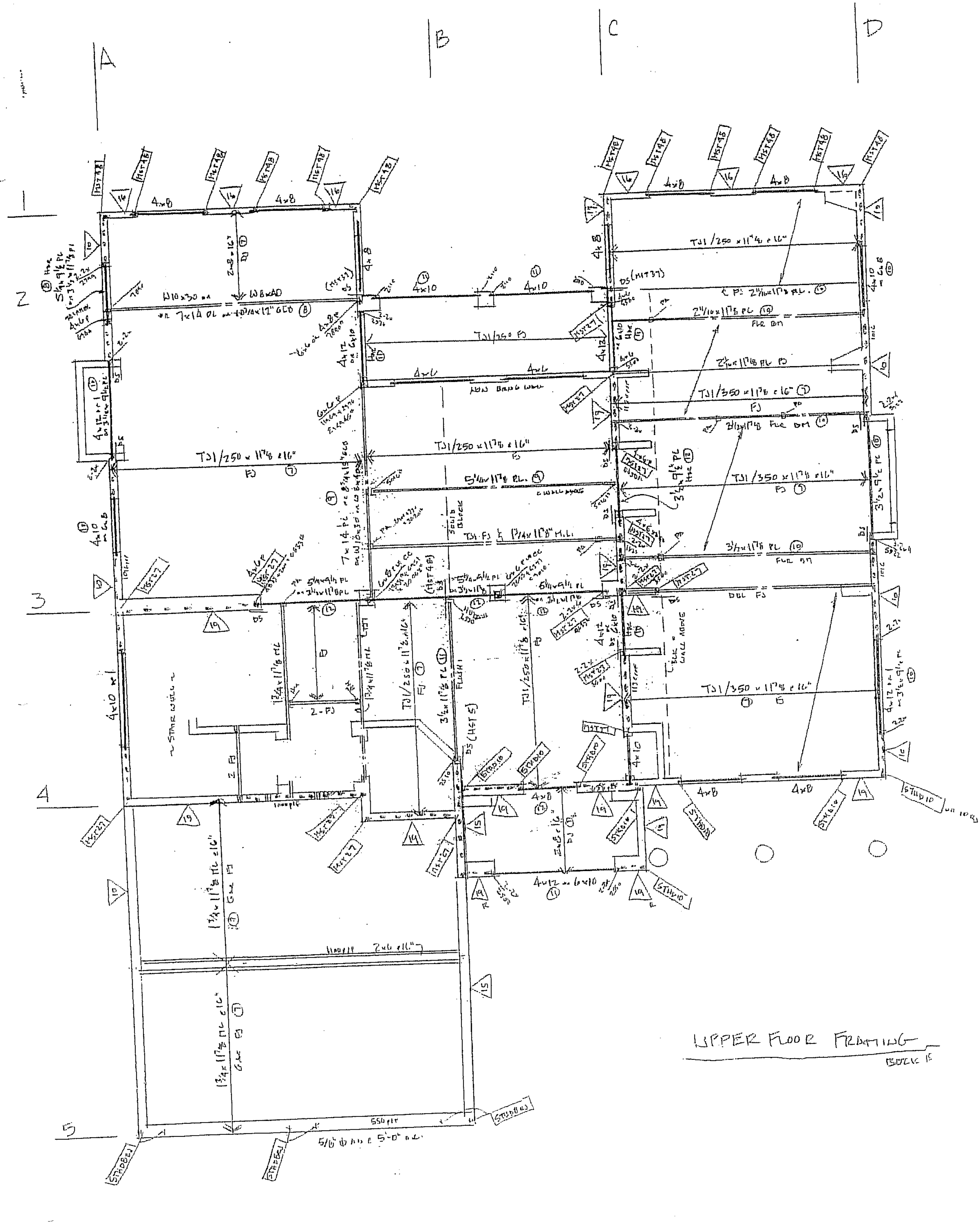
HORIZ @ STEM = .002(8)10(12) = 1.9 in²

USE 10 #4 CONT

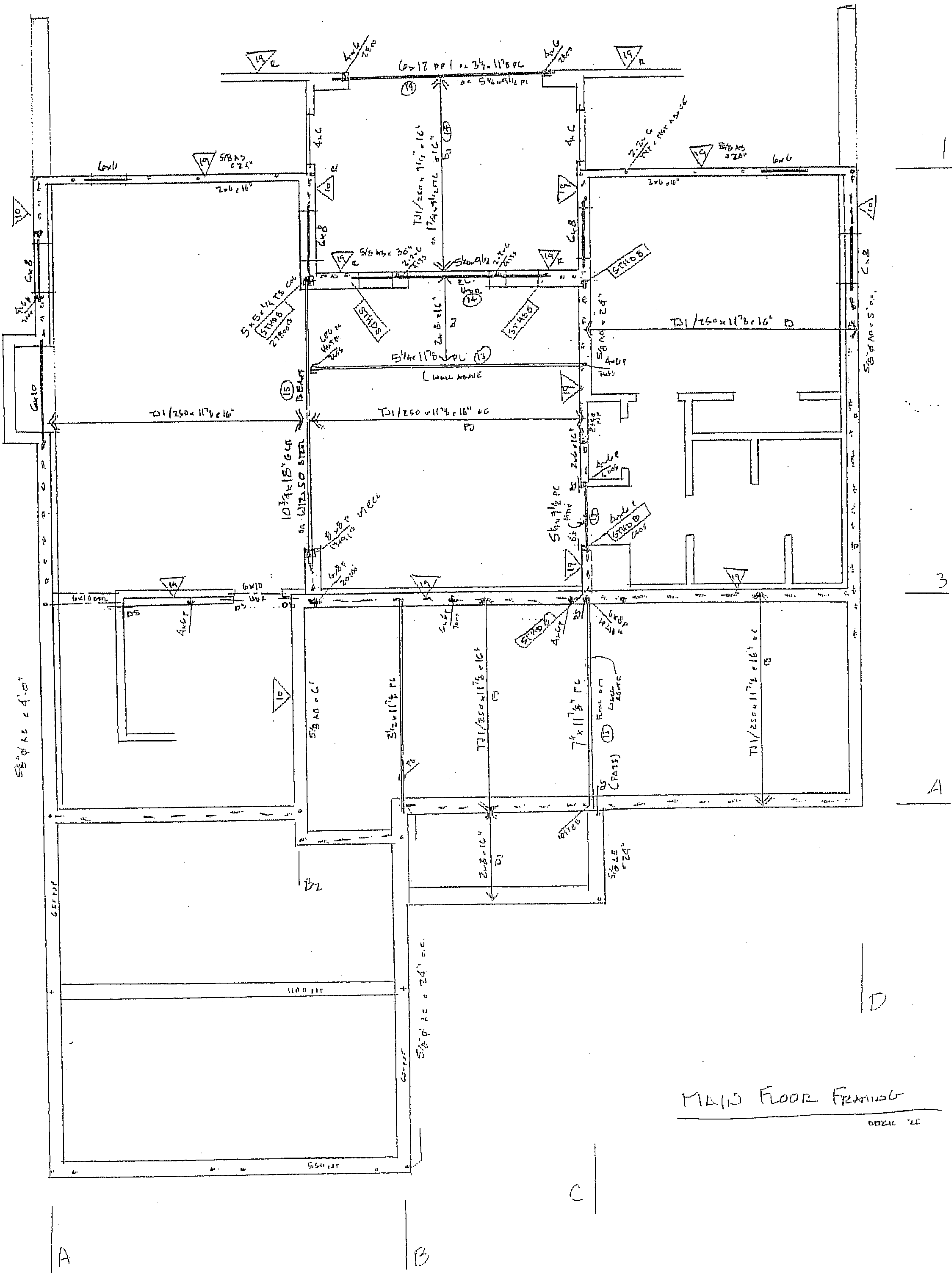


ROOF FRAMING

BOCK - 25



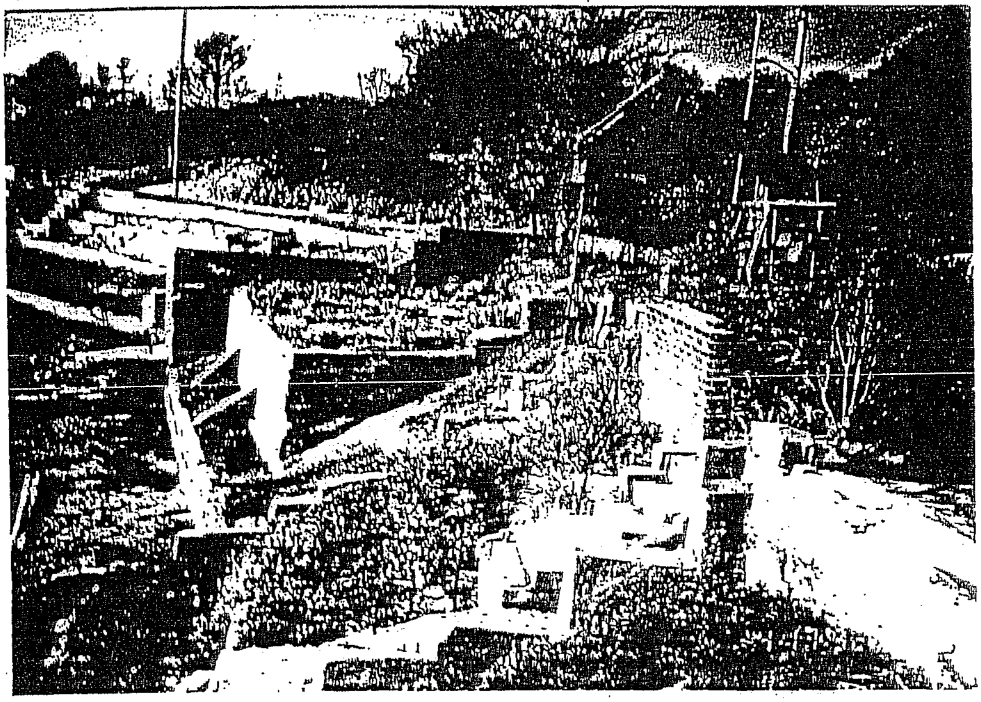
UPPER FLOOR FRAMING



MAIN FLOOR FRAMING

: 'y 11, 1992
F1le No. 1-25-1fd

GEOENGINEERING, INC.



GEOTECHNICAL EVALUATION
REPLACEMENT RESIDENCE
FIRE STORM AREA
DOWNSLOPE BUILDING SITE
6101 CONTRA COSTA ROAD
OAKLAND, CALIFORNIA

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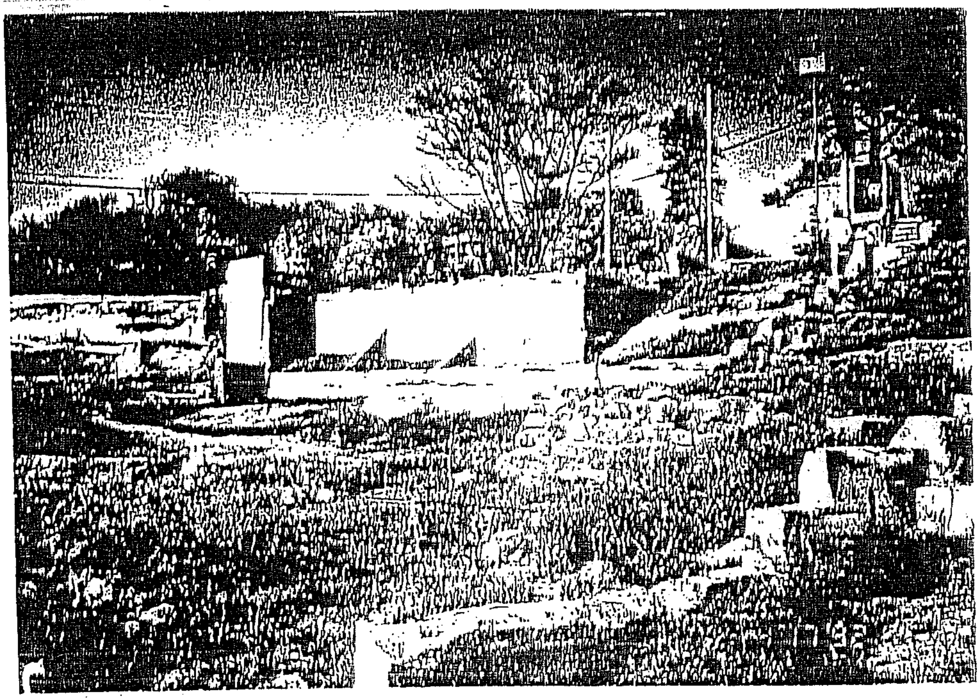

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Orinda, CA 94563

May 11, 1992
File No. 1-25-1fr

GEOTECHNICAL EVALUATION
REPLACEMENT RESIDENCE
FIRE STORM AREA
DOWNSLOPE BUILDING SITE
6101 CONTRA COSTA ROAD
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1. SUMMARY AND KEY POINTS

This property lies just below the ridgeline that is traversed by Contra Costa Road. It occupies a portion of a subdued swale that falls westerly at slopes averaging about 3h:1v. Essentially all of the previous foundation system remains.

Weathered bedrock was encountered at depths ranging from 9 to 14 feet. Because of the swale setting and deeper bedrock, the foundation requirements for this property are greater than those for the nearby properties to the south, which lie outside the swale.

A drilled pier and interconnected grade beam foundation system, penetrating into the weathered bedrock and designed for creep forces within the upper soils appears to be the most feasible means of foundation support. Alternatively, a non-drilled system penetrating up to 5 feet deep with stepped bottoms might also suffice. However, the costs for excavating such a system would preclude the savings recognized through elimination of pier drilling. In order to keep the design options open, we discuss both drilled and non-drilled foundation systems.

As for all comparable projects, we must approve the foundation plans and monitor excavation and drilling.

INTRODUCTION, WORK SCOPE, & PROJECT DESCRIPTION

Our firm has been retained by the addressee to perform the entitled services. The topics and figures contained herein are indexed in the preceding Table of Contents.

This study was undertaken to provide the geotechnical information necessary to select and plan the most feasible foundation system for the proposed residence. The design of the structural systems are discussed but are specifically excluded from our work scope.

The information and recommendations contained herein are based on a subsurface investigation performed on 4/17/92 that included three borings drilled with a backhoe mounted auger and the second investigation on 5/1/92 that included three borings in the lower portion of the property using portable equipment. The densities of the upper soils were evaluated using a portable percussion probe. This data was correlated with the topography, exposed geology, and information developed by us during a previous investigation for the property approximately 70 feet to the south. We also consulted the available Geologic Maps.

Design plans for this project are not available but we expect that it will be a two or three story wood frame structure with no special foundation requirements. It will occupy a similar footprint but it will extend several feet further upslope.

3. SITE DESCRIPTION

3.1 SETTING AND SURFICIAL FEATURES

As the Topographic Vicinity Map (Fig 3) shows, this property lies just west of the ridgeline, that overlooks Lake Temescal and is traversed by Contra Costa Road. It occupies a subdued swale that originates just below the ridgeline crest and extends downslope roughly 600 feet where it merges with other hillside features.

As the Site Plan (Fig 1) shows, the original foundation system and terracing remain, thus leaving variations in the topography. On the average, the ground surface falls westerly at slopes averaging about 3h:1v (33% or 19 degrees). Ground elevations range from +540 Contra Costa Road to +512 at the lower property line.

3.2 GEOLOGY, SOIL CONDITIONS, & BORING LOGS

Moderately well compacted clayey sand fill has been placed over the upper portion of the property. It is similar in composition and difficult to distinguish from the underlying natural soils but we judge it to be up to 5 feet thick at the upper portion of the site.

The upper 3 to 5 feet of natural colluvium consists of dense to very dense brown clayey sands with random rock fragments. Its lower portion consists of very stiff to hard sandy clays, also containing rock fragments.

Underlying the colluvium and separating it from the bedrock, we identified a 1 to 3 foot thick stratum of stiff to hard residual clayey sands and sandy clays. Residual soils can be described as bedrock that has been weathered in place to the consistency of a hard soil.

Bedrock was encountered 9-1/2 to 15 feet deep in the upper portion of the property and 10 feet deep within the lower portion. It consists of weathered sandstones and shales, similar to those exposed on the upslope cut for Contra Costa Road just above this property.

Our measured bedrock depths are indicated on the Site Plan (Fig 1) at the respective boring locations. Logs describing the subsoils encountered in our test borings are described below--Borings A, B, and C were drilled with a backhoe mounted auger; Boring C and D were drilled with portable equipment--all measurements are in feet

BORING A (30 N & 70 Ft W of SE Prop Crnr--El +531)

0-3	FILL--Brown dense to very dense clayey/ silty sand fragments, renders very hard sounding resistance.*
3-8.5	COLLUVIUM--Brown very dense clayey sand with rock fragments--renders refusal to our sounding device after 1 foot of penetration.
8.5-12	COLLUVIUM--Tan very stiff to hard silty clay--renders sounding refusal after 1-1/2 foot of penetration, PP=6**.

12-14 RESIDUAL--Tan very stiff to hard sandy clay with rock fragments.

14-15.5 BEDROCK--Tan and gray, weathered and fractured sandstone/shale.

BORING B (25 Ft S & 30 Ft W of SE Prop Crnr--El +540)

0-7 FILL--Dark brown dense to very dense clayey sand with rock fragments--render sounding refusal after 1-1/2 foot of penetration.

7-11 COLLUVIUM--Brown dense to very dense clayey sand with rock fragments.

11.5-13 COLLUVIUM--Tan very stiff to hard sandy clay with rock fragments.

13-15 RESIDUAL--Tan very stiff to hard sandy clay with rock fragments.

15-16.5 BEDROCK--Tan highly weathered sandstone/ shale.

BORING C (40 Ft S & 11 Ft W of NE Prop Crnr--El +538)

0-3 FILL--Dark brown dense clayey sand with rock fragments.

3-4.5 COLLUVIUM--Brown dense clayey sand with rock fragments.

3.5-5 COLLUVIUM--Tan very stiff silty clay, PP=6.

7-9.5 RESIDUAL--Tan very stiff to hard silty clay with rock fragments.

9.5-11 BEDROCK--Tan and gray very highly weathered siltstones/shale.

BORING D (30 Ft S & 70 Ft W of NE Prop Crnr--El +520)

0-8 COLLUVIUM--Brown very dense clayey sand with rock fragments--renders very hard resistance to portable drilling equipment which encountered refusal at 8 feet. Renders sounding refusal after typical penetrations of 1 foot.

BORING E (90 Ft W & 35 Ft S of NE Prop Crnr--El +512)

0-6	COLLUVIUM--Brown very dense clayey sand--renders very hard drilling resistance and sounding refusal after typical penetrations of 1 foot.
6-8	COLLUVIUM--Very stiff to hard silty clay with rock fragments.
8-10	RESIDUAL--Very stiff to hard silty clay.
10	BEDROCK-RESIDUAL--Very highly weathered sandstone/shale or hard residual soils--this stratum could not be penetrated with our portable equipment and the bedrock definition is based on our judgment from the drilling behavior.

*The sounding device is a 1/2 inch rod driven by impact with a 6 pound sleeve hammer developing an equivalent fall of about 15 feet. It can sometimes penetrate several inches into highly weathered bedrock.

**PP=The hand penetrometer readings approximately correspond to laboratory unconfined compressive strengths measured in tons per square foot. Values greater than the 4-1/2 foot limitation of the device as estimated.

Groundwater was not encountered in our test borings which were drilled about 1 month after the rainy season. Experience shows that perched groundwater sometimes collects over the surface of relatively impervious residual soils or bedrock during periods of heavy rainfall and/or prolonged irrigation.

As indicated by our exploration data, the subdued swale in which this property lies, has been filled with colluvium and thus represents different stratigraphic section than the nearby properties to the north and south. Nonetheless, except for some shallow slippage features, we found no indication that the colluvium mantle has experienced significant movement or creep. Its dense condition supports this view although the absence of soil creep cannot be confirmed.

This site lies within about 700 feet of the Hayward Fault (see Geologic Map--Fig 4) which is considered capable of developing a major earthquake. In view of this, we recommend a conservative interpretation of the seismic design criteria outlined in the Uniform Building Code.

The dense mantle of colluvium and fill, although not susceptible to earthquake induced liquefaction, could experience downhill movement during a major earthquake and, this has been considered in our recommendations.

4. DISCUSSION AND RECOMMENDATIONS (Summarized in Sect 1)

4.1 DESIGN REVIEW AND MONITORING SERVICES

Foundation and grading plans should be approved by us before finalization. If the recommendations contained herein pose any costly design or construction penalties, we should be notified. In this case, we would review our design parameters and, if possible, modify our recommendations to avoid unnecessary cost.

Foundation excavation and/or drilling must be monitored by the geotechnical engineer and this must be stated on the foundation plans. It is the owner's/contractor's responsibility to provide at least three days notice to the geotechnical engineer. If monitoring services are performed by others, that person must be sufficiently qualified to implement any foundation changes that might be deemed necessary by unanticipated soil conditions. Our monitoring services would be billed at our hourly rate unless other arrangements are made.

4.2 FOUNDATIONS

4.2.1 DRILLED INTERCONNECTED FOUNDATIONS (Opt 1)

Drilled piers and grade beam foundations may be designed as follows--the general design scheme is sketched on Fig 5:

1. Pier penetrations will be finalized by the geotechnical engineer during drilling and will be based on properties of the soils/bedrock encountered. We anticipate penetrations of 6 feet into weathered bedrock for total penetration of 15 to 20 feet below existing grades.

2. The piers should be at least 18 inches wide unless smaller sizes are approved by the geotechnical engineer.

3. Piers should be laid out on a grid with center to center spacing up to 14 feet. Greater spacing might be used to fit the

framing plan but should be approved by the geotechnical engineer. When possible, they should be aligned in the slope direction to optimize stress transfer.

4. All drilled piers should be tied in the uphill-downhill direction with grade beams intersecting each pier. Exceptions for closely spaced piers can be made, but the designers should avoid unnecessarily close spacing.

5. Steel reinforcement will be designed by the structural engineer but we suggest the following minimum criteria: at least two #5 bars or three #4 bars uphill and downhill.

   If feasible, the pier cages should be elongated upslope-downslope (Fig 5) to maximize the bending strength in the direction of potential soil movement: for 18 inch piers, cage dimensions of 5 by 11 inches would be appropriate. The narrow dimension could facilitate the bends into the upslope-downslope grade beams.

   The pier bars shall extend to the top grade beam steel and be bent to achieve transfer of moment stresses to the grade beams. <u>In no case shall they be cut below the top grade beam steel.</u>

   Grade beams should be least 8 inches wide by 18 inches deep, and should be reinforced to at least the same degree as the piers. They should be tied as directed by a structural engineer. This reinforcing scheme is intended to prevent the grade beam from being the weak link of the foundation system but must also be checked for support capabilities.

6. This system should be designed to resist creep forces within a soil mantle having a thickness of 8 feet, plus thickness of any fill. In cut areas, this creep mantle thickness may be reduced by the depth of soil excavated -- i.e., if 2 feet is removed, the mantle may be reduced by 2 feet.

This mantle should be assumed to develop pressures equal to a fluid weighing 30 pounds per cubic foot (pcf) (equivalent fluid pressure) acting downhill against the grade beam and against projected diameters 2 feet greater than the respective piers.

Our criteria for pier interconnection and minimum steel must always be met. Depending on our observations during pier drilling, it may be necessary to increase the design creep zone, which could mandate an increase in reinforcing steel. This, however, is unlikely.

7. The soils below the creep zone may be assumed to resist creep forces with *ultimate equivalent fluid pressures of 600 pcf. These pressures should act from the creep zone bottom and against projected diameters 2 feet greater than the respective piers.

This resistance may be assumed to develop 3 feet below grade for wind forces, and earthquake forces acting uphill or parallel to the contours. Resistance to downhill earthquake forces should act from the creep zone bottom.

*As for all lateral restraint parameters, these are ultimate values and must be applied with the 1.5 code safety factor.

8. The requirements to sustain the indeterminate lateral creep forces, rather than building loads, will govern pier penetrations. Consequently, the piers must still meet the minimum criteria outlined above.

The soils below the creep zones may be assumed to resist vertical pier loads using allowable friction values of 800 psf for dead and permanent loads such as retaining walls--they may be increased to 1,200 psf to include code live loads and to 1,800 psf to include earthquake and wind forces. These values may be doubled in bedrock.

*As for all vertical loading parameters, these are allowable values and require no further safety factors.

The minimum depths criteria outlined above must be maintained. Friction within the creep zone and end-bearing cannot be used. The weight of the piers may be neglected when computing their capacities.

9. The upper exterior grade beams should penetrate at least 1 foot below the upslope grade to act as a moisture barrier. The remaining exterior members should penetrate at least 1/2 foot below their exterior grades to avoid voids that often develop below the grade beams from surficial soil creep.

10. Water should be available to facilitate pier drilling and aid in extraction of the cuttings from the pier holes. Plywood covers should also be available to keep the holes free of debris. The pier and grade beams need not be poured monolithically.

11. If water accumulates in the pier holes, it may be displaced by pumping the concrete mix to the hole bottom but this method should be approved by the engineer. If water is pumped from the holes, they should be carefully checked for caving -- if caving is observed, the displacement technique must be used.

4.2.2 NON-DRILLED GRID FOUNDATIONS (Opt 2)

These criteria may be relaxed for typical ancillary structures such as decks and minor detached retaining walls, pending our approval:

1. Foundations for the residence should be interconnected and tied upslope-downslope at maximum intervals of 12 feet. They should be capable of spanning at least 12 feet across zones of non-support and their corners should be capable of cantilevering at least 6 feet along the intersecting members.

   All foundations should be reinforced with at least two #5 bars top and bottom. The bars may be bundled in pairs if acceptable to the structural engineer.

2. Foundation subgrades must be approved by the engineer but they should penetrate:

(a) At least 3-1/2 feet below lowest adjacent final grade.

(b) Below imaginary planes projected upward at 2h:1v from the base of detached retaining walls, permanent excavations, swimming pools or septic systems.

(c) Below imaginary horizontal planes intersecting the slope at least 6 feet from the respective foundation edges.

3. The foundation bottoms should slope no more than 10%.

4. Such foundations may be sized for allowable* soil pressures of 1,400 psf for dead and permanent applied loads such as retaining wall foundations. They may be increased to 1,800 psf to <u>include code live loads</u> and to 2,400 psf for all loads including those caused by wind or earthquake forces. The weight of foundation concrete below grade may be excluded in computing soil pressures.

   *As for all axial and vertical loading parameters, these values require no further safety factors.

5. Sustained horizontal forces, such as active earth pressures, may be resisted using <u>ultimate*</u> friction factors of 1/2 between the foundation concrete and sub soils.

   Additional sliding resistance may be developed by assuming that subsoils resist foundation movement with an <u>*ultimate</u> passive equivalent fluid pressure of 450 pcf acting against the foundation edges. For foundations bearing on downslopes, confinement should begin below an imaginary plane intersecting the hillside 5 feet from the foundation edges. A uniform value of 300 psf may be added in bedrock. If both friction and lateral restraint are used, one should be reduced by 1/3.

For transient horizontal forces, such as those caused by wind or earthquake, the above values may be increased by 1/3.

*As for all lateral restraint parameters, a 1.5 code safety factor should be included in design.

4.2.3 FOUNDATION DRAINAGE & PROTECTION

1. Upslope exterior foundation members on the slope should extend at least 2 feet above the exterior grade. This measure is recommended to protect the framing from possible but unlikely earth/debris flow. They may be designed to the subsequent retaining wall criteria. Normal 30 inch doorway openings cut within the concrete would be acceptable pending our approval.

   This measure might be relaxed through regrading or construction of an upslope diversion wall with at least 2 feet of freeboard and designed to the criteria outlined in Section 5.4, pending our approval.

2. Upslope exterior foundations members should be provided with backdrains extending to their bottoms or to the bedrock surface. They may consist of bottom-perforated pipe placed in drainrock. The trench bottom should slope at least 1% to the flanks of the structure. In most cases the subdrain could discharge onto a landscaped area. Discharge onto pavement might result in a prolonged wet surface.

   The drainrock should be separated from the adjacent soils by non-woven geotextile filter cloth and backfilled with a low permeability clayey soil to prevent migration of surface water into the drain structure. The on-site soils would be suitable for this backfill.

3. The lower intersection of the foundation members should be provided with 1 inch weepholes, just above the ground line for drainage. <u>All subfloor grades should slope downhill</u>, and when possible, should be no lower than the adjacent exterior grades.

4. The subfloor ground surfaces should be covered with 6 mil plastic or non-structural concrete (rat proofing). This measure is recommended to mitigate increased crawlspace humidity from increases in ground moisture that results from the building confinements. This is in addition to a complete venting system.

4.3 RETAINING WALLS AND BULKHEADS

4.3.1 LATERAL PRESSURES AND FOUNDATIONS

Retaining walls may be designed for allowable <u>active lateral earth pressures</u> equal to the following:

1. A fluid with a density of 45 pcf (equivalent fluid pressure or efp) for walls that retain cuts with no backfill except drain rock -- for backfilled walls it should be increased to 55 pcf.

   Where the ground above the wall rises, it should be increased in proportion to one half of the upslope rise; for example, an upslope rise of 40% (2.5h:1v), corresponds to a pressure increase of 20%.

   This pressure may be reduced by 15% for walls that support no pavement or structure. In no case need it exceed 60 pcf.

2. A uniform lateral pressure equal to one third of any anticipated surcharge pressure but at least 50 psf for walls supporting streets, driveways, or garage slabs. This is in addition to the equivalent fluid pressure.

If drilled piers are used with conventional "L" or inverted "T" foundations, <u>the foundation toes</u> (downslope edges) must be pier supported unless approved by us.

Drilled piers that support retaining walls which also act as foundation grade beams may be designed as recommended in Sect 5.2. For drilled piers that support detached retaining walls piers, the designated creep zone pressures may be eliminated on downslopes flatter than 3h:1v pending our approval--lateral support could then be assumed to develop below the designated creep zone.

4.3.2 BACKFILL AND BACKDRAINAGE

Retaining walls that support (or are integrated with) other structures should be backfilled before framing or subsequent construction. This measure is recommended to avoid the effects of initial wall deflections that often accompany backfill placement.

Retaining walls shall be backdrained and provided with separate surface drainage. Lined V-ditches along the tops would minimize infiltration of surface water into the backdrain. When acting as building or garage walls, they must also be waterproofed.

Backdrains may consist of conventional 3 or 4 inch bottom-perforated pipe in the drainrock blankets. The pipe should be placed about 2 inches above the wall bottom and sloped toward the flanks at about 2%. In most cases subdrain drainage may be discharged on normal landscaping. If discharged onto pavements, it could result in prolonged wetness on the paved surface.

The drainrock blanket should be at least 4 inches wide and separated from the adjacent soil with a non-woven geotextile filter cloth. It should extend from the wall bottom upward for 1-1/2 to 3 feet depending on the wall height. The remainder of the backfill should be a clayey soil with a low permeability to prevent migration of surface water into the backdrain.

Weepholes, which are more reliable but still require drainrock, may be used in lieu of (or in conjunction with) perforated pipe, where wall seepage is acceptable. They should be about 1 inch wide and spaced at about 2 feet intervals along the base of the wall.

Structured backdrain material (such as Miradrain) may be used in lieu of drainrock behind the retaining wall stem. It can be placed against a relatively smooth cut with its filter cloth back against the soil while its plastic surface can act as the rear form. This eliminates the need for back forms and the related over-excavation. Since form ties cannot be used, the forward forms must be braced externally. The fabric side of the structured backdrain panels must be against the earth.

Structured backdrain still requires weepholes and/or perforated pipe in a drainrock conduit. Weepholes must penetrate through the backdrain and be provided with localized drainrock pockets (or equivalent drainage medium) to allow passage from the structured medium to the openings. If used with perforated pipe in drainrock, the structural backdrain must penetrate to the drainrock -- its geotextile liner should then be extended to the bottom of the drainrock, and wrapped around over the pipe to allow transfer of seepage water. Drainrock around the perforated pipe can be completely eliminated by application of a special technique -- plans for this scheme can be supplied on request but we recommend that it not be used for walls that border living areas.

4.3.3 POST AND WOOD LAGGING WALLS

Post and wood lagging walls up to 4 feet high may be built to the standard county design if the supporting grades slope less than 10% (10h:1v). Walls up to 5 feet high may be designed to the criteria used by other municipalities that have such designs (ie, the City of Novato).

Walls not meeting these criteria may be designed using the lateral earth pressures outlined in Section 5.3.1. The drilled posts may be designed for ultimate equivalent fluid passive earth pressures of 300 (both acting against a diameter 2 feet greater than the drilled sockets). If the supporting grade slopes more than 10%, the wall bottom should be assumed to be at an imaginary horizontal plane intersecting the hillside 4 feet from the face of the wall.

Lagging shall be spaced at approximate 1/2 inch intervals to allow drainage and retard deterioration, and provided with drainrock as recommended above. All wood shall be approved for earth contact.

4.4 GRADING, DRAINAGE, AND SLIDE PROTECTION

Site grading should be limited as much as possible in order to minimize its effects on hillside stability.

The ground surface should be sloped for rapid drainage away from building areas. In the areas upslope from the dwelling, it may be channeled around the structure or into a separate system.

Roof drainage should be collected in downspouts and channeled away from the structure to the pavement. Drainage to multiple discharge points is preferable to concentrated discharge (which should be avoided when feasible). If this is not feasible, erosion protection could be achieved by discharging through multiple outlets over 6 to 12 inch rip rap rock. Horizontal drainage spreaders or flumes that allow uniform spillage such as lined swales or perforated pipe (sketched on Fig 6) would also suffice. Discharge into dry wells (gravel filled unlined excavations) is not acceptable.

Drainage onto adjacent or downslope properties should be avoided. If this is not possible, drainage should be dispersed over a large area in "sheetflow." This can be achieved as discussed above.

Even with the above outlined drainage measures, erosion can be expected. Considering this, all exposed unpaved areas should be provided with a vegetative cover. Courts have ruled that property owners are responsible for slide and erosion damage to downslope or adjacent properties, even when natural and without artificial influences.

Surface water should never be introduced into backdrains or other subterranean drainage system that utilizes perforated pipe or drainrock. Such systems are intended only for removal of relatively small quantities of groundwater and are likely to become blocked if used for surface drainage.

The existing retaining walls and foundation members may be left in place but cannot be integrated with the design for the new system. In this case, new walls could be poured against the existing walls using the backdrainage and waterproofing measures outlined in Sect 5.3.2 for walls poured directly against cuts.

4.5 EXCAVATION AND ENGINEERED FILL

Indications are that excavation can be achieved with conventional methods. Temporary cuts higher than 6 feet should be cut to safe slopes which we expect would be about 1-1/2h:1v in soil and 1/4h:v in bedrock. In any case, procedures must comply with OSHA standards and protection should be provided to workmen.

Areas to receive engineered fill (fill placed below building and pavement areas) must be cleared of vegetation and debris, and stripped of top soil. The stripping depths should be determined during earthwork but we expect it will range up to 4 inches.

After stripping, the fill areas should be benched to slopes sufficiently flat to allow operation of compaction equipment. The exposed subgrade should be scarified, moisture conditioned and compacted to at least 90% of the maximum dry density as determined by the Modified AASHTO test.

Engineered fill should be approved by the geotechnical engineer, spread in approximate 8 inch lifts, and moisturized and compacted as recommended for the subgrade. The on-site soils can be used as engineered fill, pending our approval.

As a general criteria, permanent fill, and cut slopes in soil, should not exceed 50% (2h:1v). These requirements may be relaxed in bedrock or small slopes, or slopes provided with a rip-rap cover.

4.6 SLABS AND PAVEMENTS

The subgrades below slabs and pavements should be prepared as recommended above, and approved by the geotechnical engineer.

Prior to placement of baserock or concrete, the subgrade for interior slabs cut into the hillside should be sloped at 1% (1 inch in 8 feet) for drainage, compacted as recommended above, and rolled to smooth surface.

At least 4 inches of free draining baserock should be placed and compacted over the subgrade to act as a capillary break and to provide subslab drainage for potential groundwater at the low corners of the base rock blanket. Drain outlets through the low foundation intersections should be provided.

An impervious barrier should be placed over the drainrock to prevent moisture permeation unless slab wetness is acceptable. It should be covered with 2 inches of clean sand for protection from puncturing and to aid in concrete curing.

Floor slabs within living areas will require extra precautions with respect to drainage and water-proofing, especially if they abut retaining walls. In view of the seepage problems inherit with such slabs, we recommend that they be provided with pressure treated plywood covering bearing on pressure treated fir 2 by 4 "sleepers". This is in addition to the other recommended waterproofing and drainage measures.

4.7 EARTHQUAKE DESIGN CRITERIA

All structures should be designed to the seismic criteria outlined for Zone 4 in the Uniform Build-ing Code, as well as any local codes. This is the most severe earthquake designation in the code and includes most of the San Francisco Bay Region.

In view of the proximity to the Hayward fault, we recommend a conservative interpretation of this criteria. No special earthquake or fault studies were performed for this investigation.

5. CLOSURE AND LIMITATIONS

By accepting this report the client and other recipients acknowledge their understanding and acceptance of the following terms and conditions. It is also acknowledged that no verbal guarantees were made by the undersigned.

Even though we see no reason to suspect that the soil or foundation behavior will differ from our predictions, one must recognize that factors contributing to hillside and foundation instabili-ty, surface and ground water seepage, and other geotechnical related problems cannot always be detected. This report represents our best judg-ment based on the available information and com-plies with current standards of practice for comparable studies. No forms of warranty or insurance coverage are expressed or implied in our reports or other communications.

It is also understood that certain risks must be assumed for all types of foundation and earth systems. These risks can always be lessened by upgrading these systems even though the margin of additional safety may be small compared to the additional costs involved. Although the engineer may assist in selection of the optimum balance between safety and economy, the client and all recipients understand that the risk is their own.

If a claim is made against GeoEngineering, Inc. for any act relating to our professional services, the initiator(s) of the claim shall pay for all costs and lost time associated with our defense. This includes (but is not limited to) attorney fees and our time which would be charged at the prevailing hourly billing rate. In order to discourage frivolous lawsuits against our profession, we would pursue charges for such action against the attorneys and plaintiffs if we feel a basis exists. In any case, our liability cannot exceed our fee for this project. We carry no errors and omission insurance.

We trust that this report provides you with the information required at this time. You may contact the undersigned as questions and the need for design clarification arise.

Respectfully submitted,

GEOENGINEERING, INC.

Robert H Settgast

Robert H. Settgast
Professional Geotechnical Engineer

RHS:ceb

REGISTERED PROFESSIONAL ENGINEER
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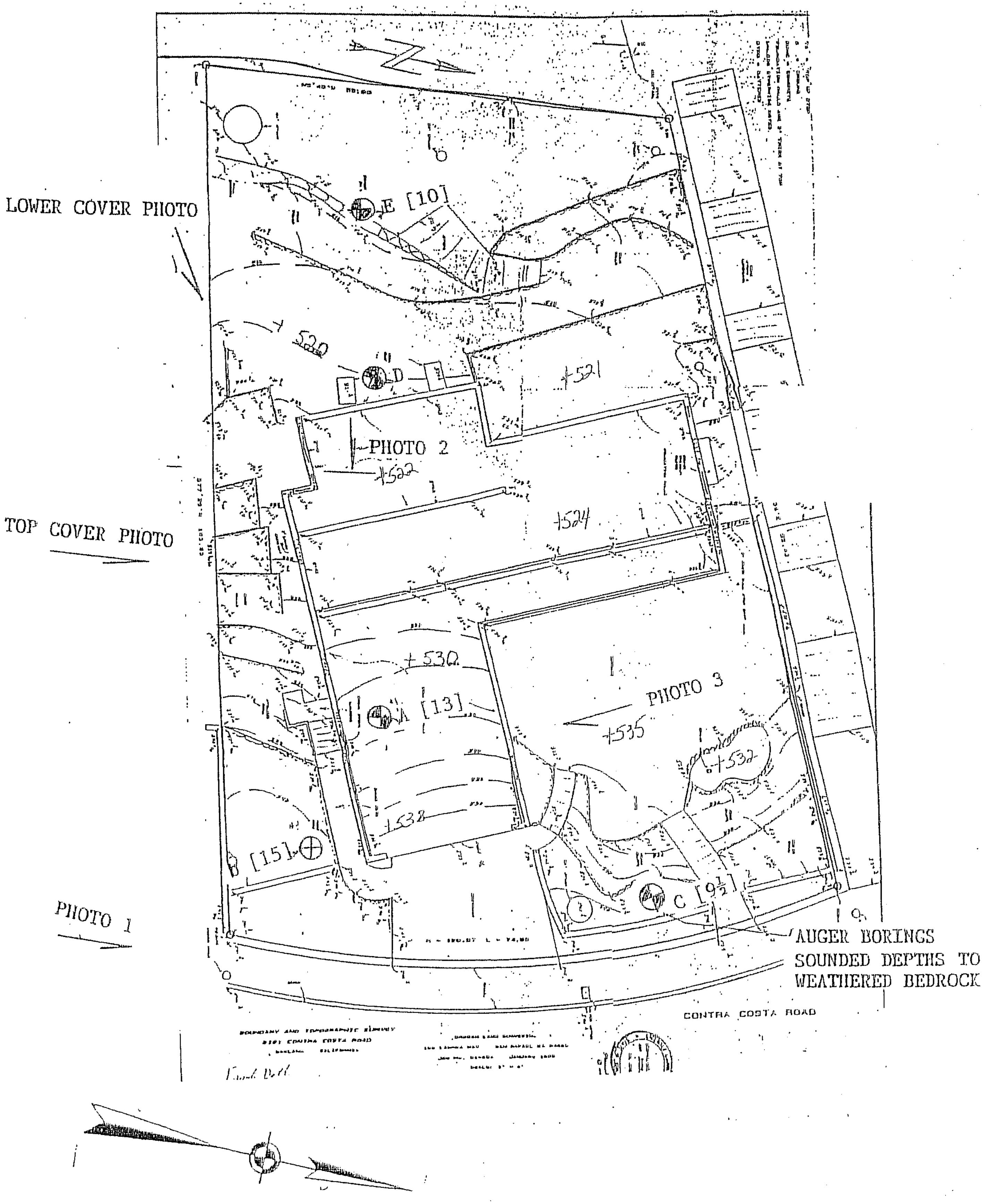
STATE OF CALIFORNIA



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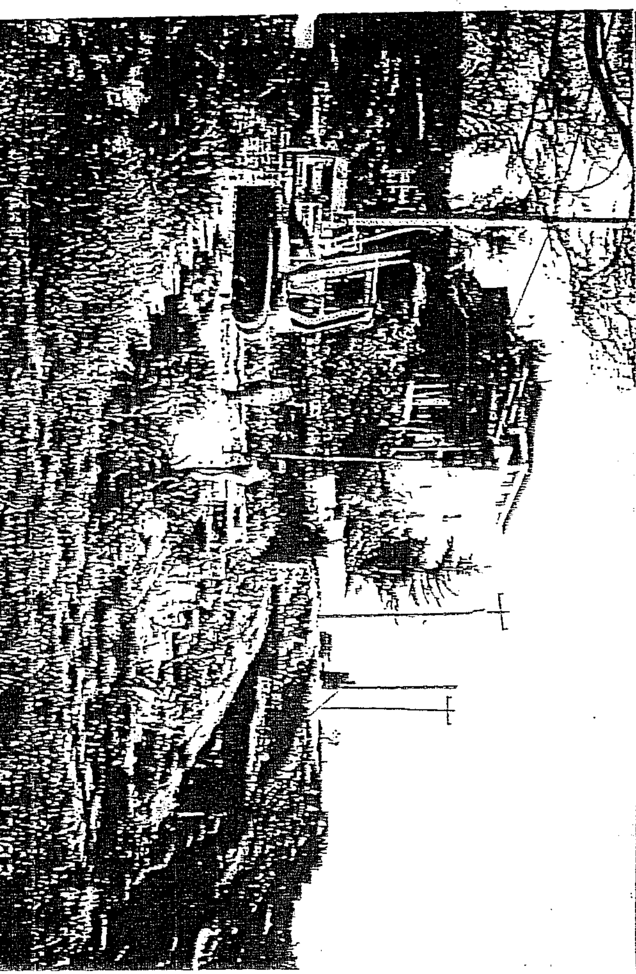

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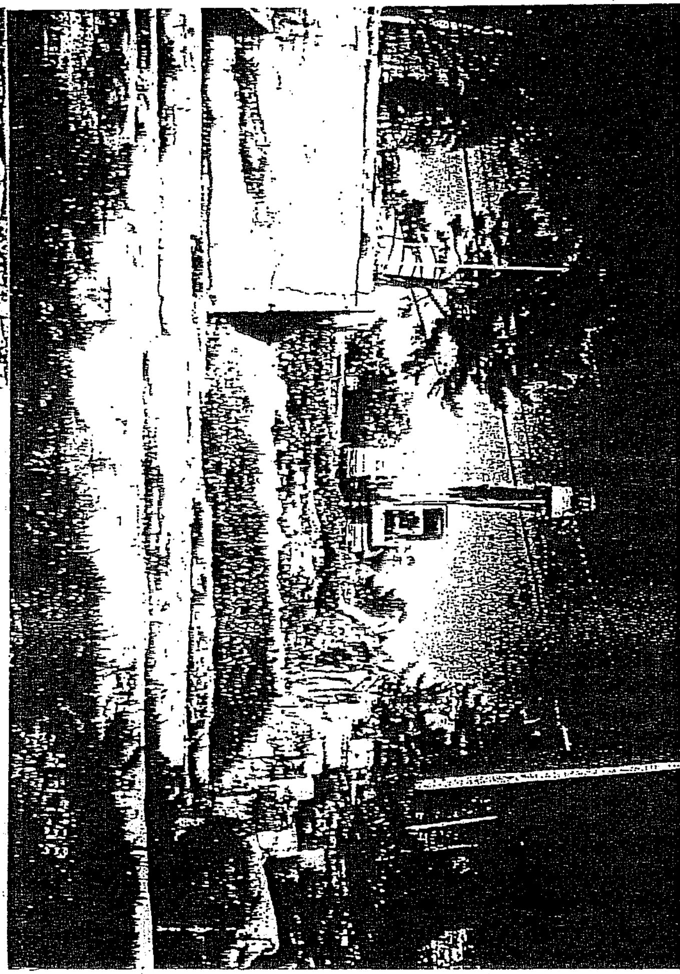

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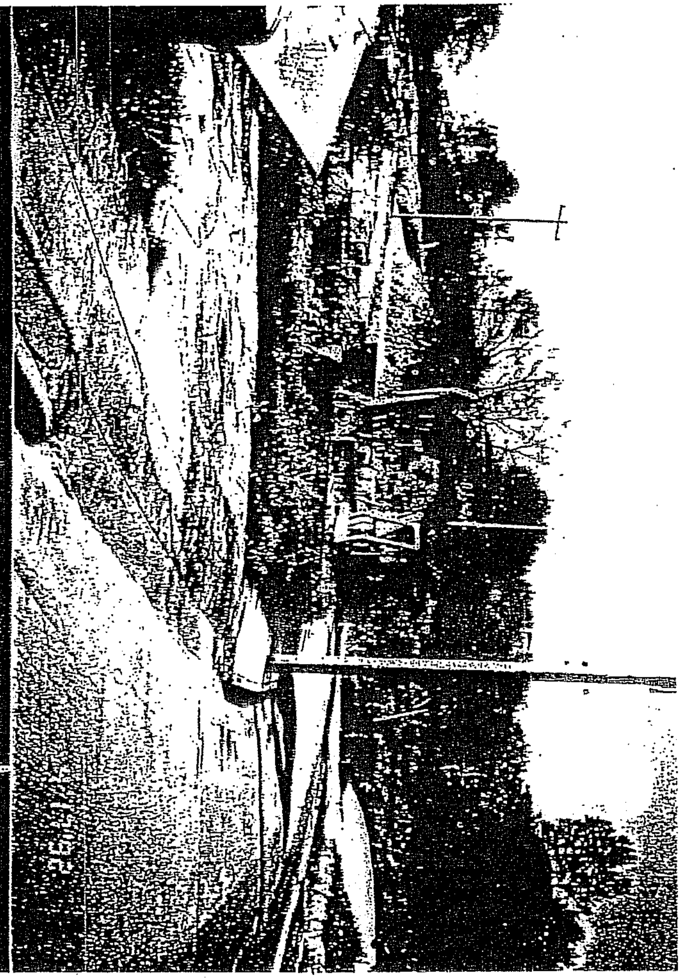

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Photographs Keyed To Site Plan

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FIG 2

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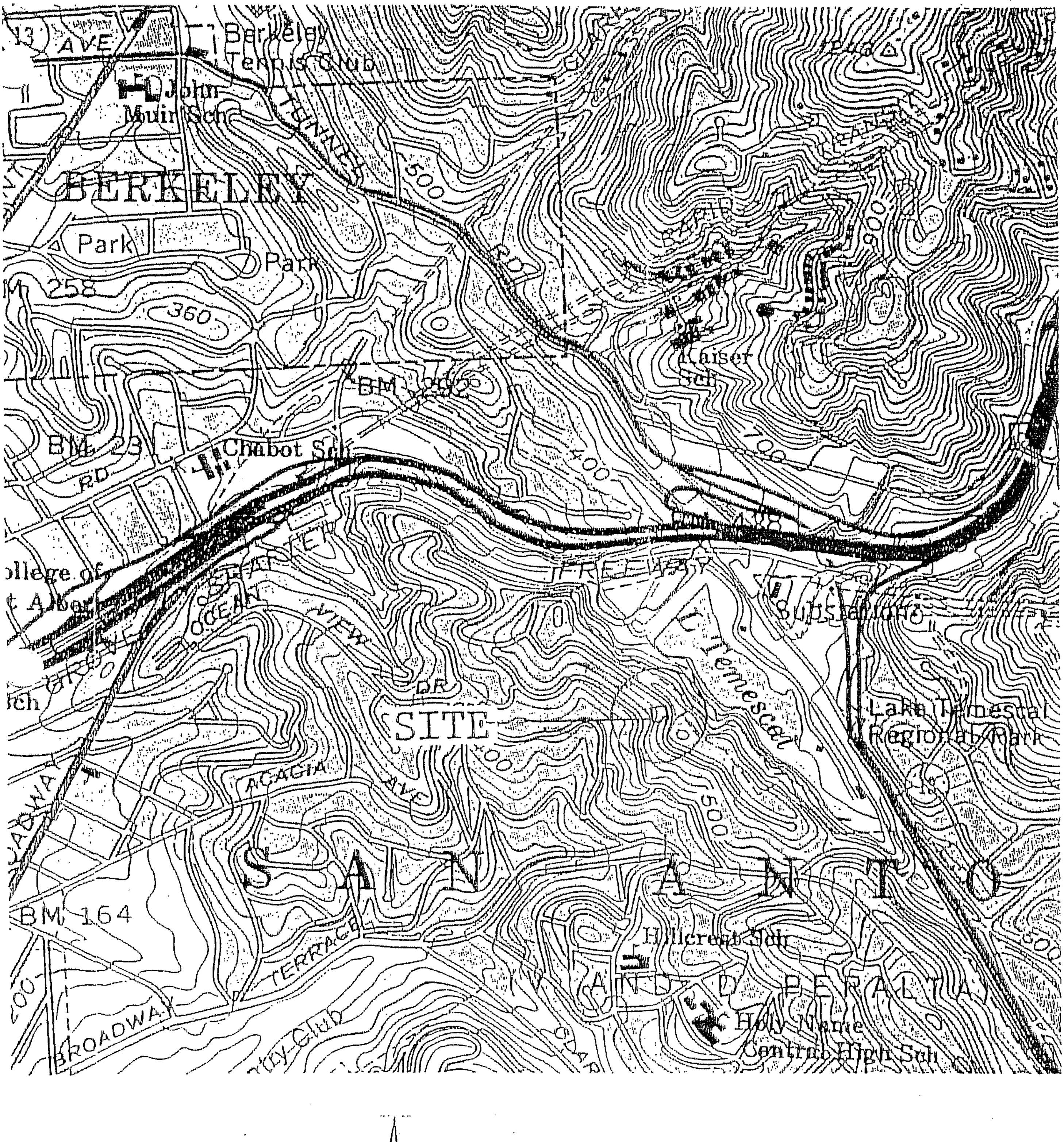

1 in = 1000 ft
20 ft contours

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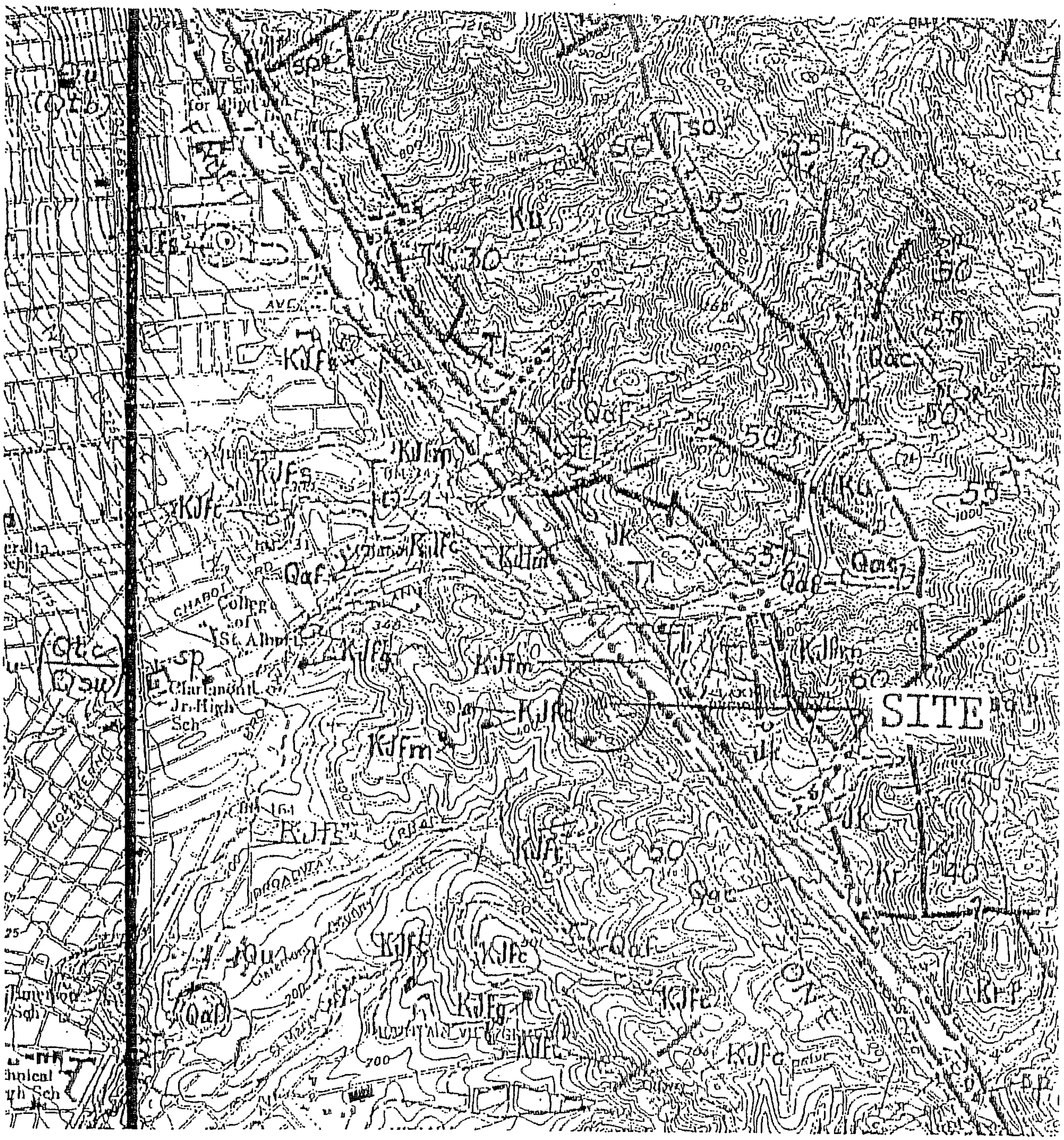
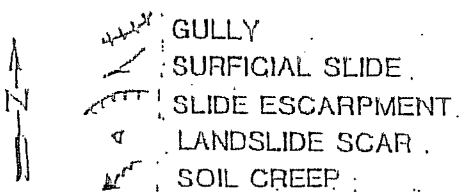
TOPOGRAPHIC VICINITY MAP FIG 3

GEOENGINEERING, INC



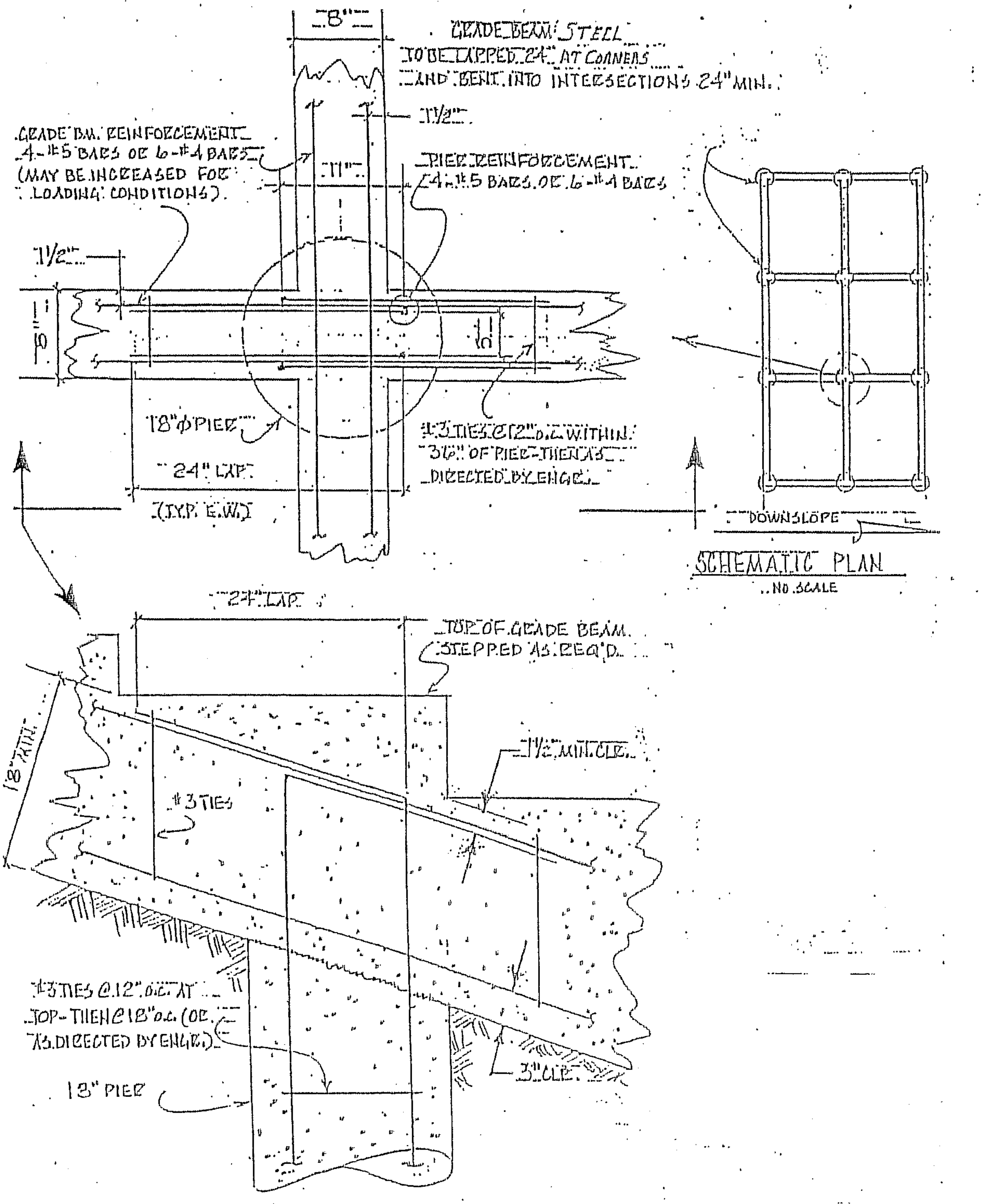
LEGEND



1 in = 2000 ft

KJfs	Franciscan sandstone and shale
KJfc	Franciscan chert and shale
KJfg	Franciscan greenstone
KJfm	Franciscan metamorphic rock

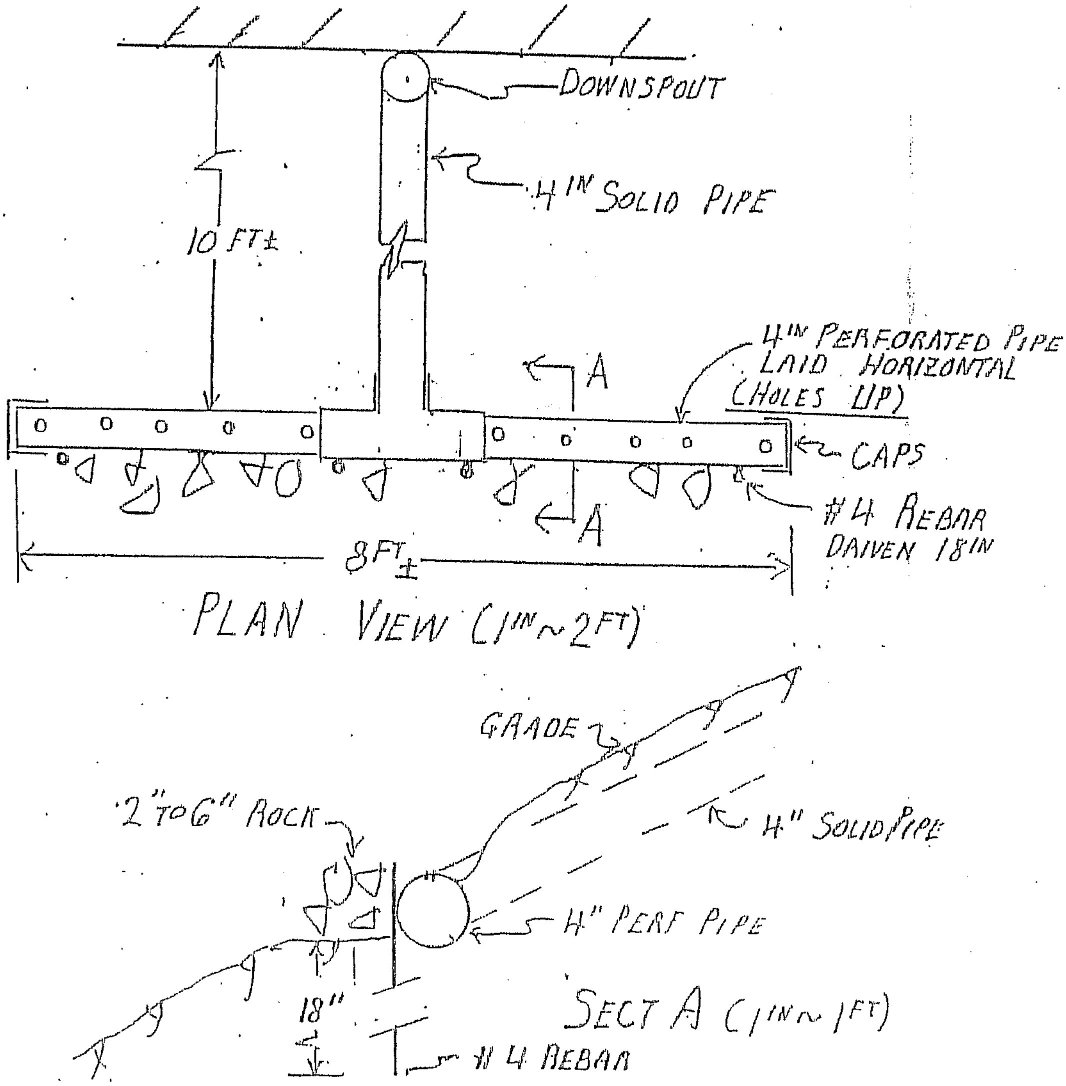
GEOENGINEERING, INC



Guidelines For Drilled Pier And
Grade Beam Foundation Systems

FIG 5

GEOENGINEERING, INC



Perforated Pipe Dispersion Flume

1-800-345-7334

SCALE IN 1/10 OF AN INCH

ASSESSOR'S MAP 48A

Code Area Nos. 17-001

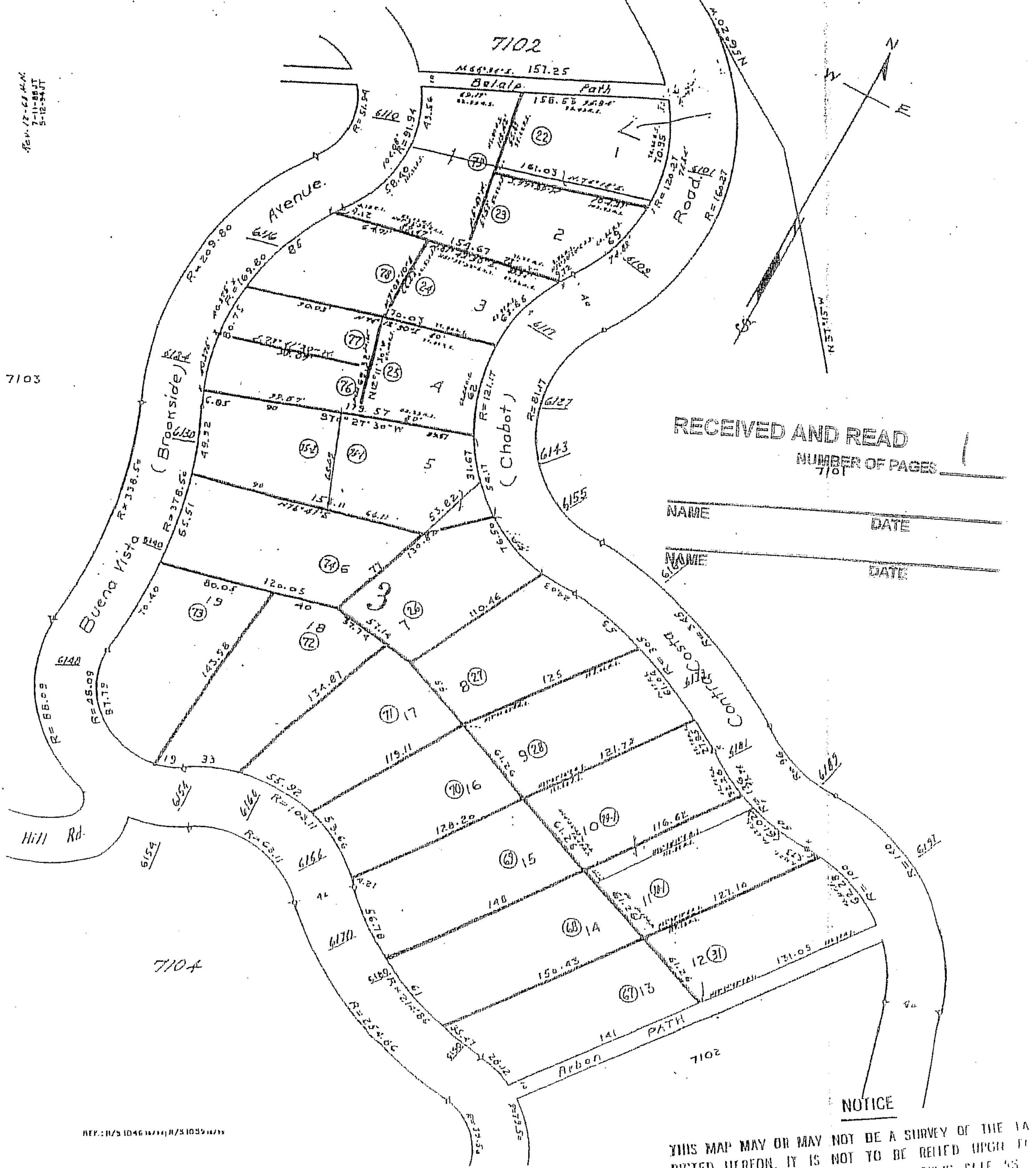
7102
4373

Page 3

Resub of Blocks 9,10,11,12,13,14. and a portion of Block 16 Rock Ridge Terrace.

Scale 1in = 50ft.

Rev. 12-62 H.N. 7-11-85 JT 5-12-94 JT



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NUMBER OF PAGES 1

NAME DATE

NAME DATE

NOTICE

THIS MAP MAY OR MAY NOT BE A SURVEY OF THE LA
PICTED HEREON. IT IS NOT TO BE RELIED UPON F
PURPOSE OTHER THAN ORIENTATING ONE'S SELF AS
GENERAL LOCATION OF THE PARCEL OR PARCELS OF
COMPANY ASSUMES NO LIABIL

Title 24 Report for:

MARK BECKER

B.N.H. PARTNERSHIP
OAKLAND, CA.

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NUMBER OF PAGES 4

NAME DATE

NAME DATE

October 24, 1998

The California Residential Energy Conversation Standards have been reviewed and the design, drawings, and calculations comply substantially with these Standards.

Building Mechanical Systems Inc.
6 Morgan, Suite 118
Irvine, CA 92718
Phone: (949) 587-1551
Fax: (949) 380-7446

PROJECT SUMMARY

WALL INSULATION	R-13
ROOF INSULATION	R-30
FLOOR INSULATION	R-19
AC EFFICIENCY	12.00 SEER
FURNACE EFFICIENCY	90.0% AFUE
HVAC DUCT INSULATION	R-4.2 (IN UNCONDITIONED SPACE)
GLAZING	DUAL PANE WOOD FRAME WINDOWS
SLAB-EDGE	NO SLAB-EDGE INSULATION REQUIRED

Project Title.......... B.N.H. PARTNERSHIP
Project Address........ 6101 CONTRA COSTA RD.
OAKLAND, CA. 94611
umentation Author... OMID TOHIDIAN
Building Mechanical Systems, Inc.
6 Morgan, Suite 100
Irvine, CA 92718
714-587-3070
Climate Zone........... 03
Compliance Method...... MICROPAS4 v4.50 for 1995 Standards by Enercomp, Inc.

****
v4.50
****

Date........ 10/24/98

Building Permit #

Plan Check / Date

Field Check/ Date

MICROPAS4 v4.50 File-BNH Wth-CTZ03S92 Program-FORM CF-1R
User#-MP1692 User-Building Mechanical Syste Run-New

GENERAL INFORMATION

Conditioned Floor Area.....	3414 sf
Building Type..............	Single Family Detached
Construction Type	New
Building Front Orientation.	Front Facing 315 deg (NW)
Number of Dwelling Units...	1
Number of Stories..........	2
Floor Construction Type....	Raised Floor
Glazing Percentage.........	21 % of floor area
Average Glazing U-value....	0.6 Btu/hr-sf-F

BUILDING SHELL INSULATION

Component ype	Frame Type	Cavity R-value	Sheathing R-value	Insul R-value	Assembly U-value	Location/Comments
Wall	Wood	R-13	R-0	R-13	0.088	TO OUTSIDE, TO GARAGE
Roof	Wood	R-30	R-0	R-30	0.035	ATTIC
Floor	Wood	R-19	R-0	R-19	0.037	RAISED FLOOR
Door	n/a	R-0	R-n/a	R-0	0.330	TO OUTSIDE, TO GARAGE

FENESTRATION

Orientation			Area (sf)	U-Value	# of Pan-es	Interior Shading/ Description	Exterior Shading	Over-hang/ Fins	Framing Type
Window	Front	(NW)	50.0	0.600	2	Drapes.Std	None	None	Wood
Window	Front	(NW)	64.0	0.600	2	Drapes.Std	None	Yes	Wood
Window	Front	(NW)	30.0	0.600	2	Drapes.Std	None	Yes	Wood
Window	Left	(NE)	110.0	0.600	2	Drapes.Std	None	None	Wood
Window	Left	(NE)	32.0	0.600	2	Drapes.Std	None	Yes	Wood
Window	Back	(SE)	166.0	0.600	2	Drapes.Std	None	None	Wood
Window	Back	(SE)	60.0	0.600	2	Drapes.Std	None	Yes	Wood
Window	Back	(SE)	52.0	0.600	2	Drapes.Std	None	Yes	Wood
Window	Right	(SW)	121.0	0.600	2	Drapes.Std	None	None	Wood
Window	Right	(SW)	32.0	0.600	2	Drapes.Std	None	Yes	Wood

Project Title.......... B.N.H. PARTNERSHIP Date........ 10/24/98

MICROPAS4 v4.50 File-BNH Wth-CTZ03S92 Program-FORM CF-1R
User#-MP1692 User-Building Mechanical Syste Run-New

THERMAL MASS

Type	Exposed	Area (sf)	Thickness (in)	Location/Comments
InteriorHorz	Yes	110	1.0	kitchen counters

HVAC SYSTEMS

Equipment Type	Minimum Efficiency	Duct Location	Duct R-value	Thermostat Type
Furnace	0.900 AFUE	Attic	R-4.2	Setback
ACSplit	12.00 SEER	Attic	R-4.2	Setback

WATER HEATING SYSTEMS

Tank Type	Heater Type	Distribution Type	Number in System	Energy Factor	Tank Size (gal)	External Insulation R-value
Storage	Gas	Recirc/TimeTemp	1	0.544 EF	75	R-0

SPECIAL FEATURES/REMARKS

NEW FURNACE FOR MAIN FLR. TO BE PAYNE 350MAV048080
80000 INPUT, 74,000 BTUH OUTPUT, 92% AFUE.
NEW A/C FOR MAIN FLR. TO BE PAYNE 563C042, 12 SEER
NEW FURNACE FOR UPPER FLR. TO BE PAYNE 350MAV048080,
80,000 INPUT, 74,000 BTUH OUTPUT, 92% AFUE.
NEW A/C UPPER FLR. TO BE PAYNE 563C042, 12 SEER
NEW WATER HEATER TO BE A.O.SMITH PGC-75, 75,000 BTUH
PROVIDE COOLING COILS IN THE FURNACES.

roject Title.......... B.N.H. PARTNERSHIP Date........ 10/24/98

MICROPAS4 v4.50 File-BNH Wth-CTZ03S92 Program-FORM CF-1R
User#-MP1692 User-Building Mechanical Syste Run-New

COMPLIANCE STATEMENT

This certificate of compliance lists the building features and performance specifications needed to comply with Title-24, Parts 1 and 6 of the California Code of Regulations, and the administrative regulations to implement them. This certificate has been signed by the individual with overall design responsibility. When this certificate of compliance is submitted for a single building plan to be built in multiple orientations, any shading feature that is varied is indicated in the Special Features/ Remarks section.

DESIGNER or OWNER

Name.... MARK BECKER
Company. ________________
Address. 134 DRACENA AVE.
PIEDMONT CA
Phone... (510) 658-6889
License. ________________

Signed.. ________________ (date)

DOCUMENTATION AUTHOR

Name.... OMID TOHIDIAN
Company. Building Mechanical Systems, Inc
Address. 6 Morgan, Suite 100
Irvine, CA 92718
Phone... 714-587-3070

Signed.. ________________ 10-29-98 (date)

ENFORCEMENT AGENCY

Name.... ________________
Title... ________________
Agency.. ________________
____
Phone... ________________

Signed.. ________________ (date)