APPENDIX G-1

GEOTECHNICAL FEASIBILITY EVALUATION, JULY 2012

HERZOG
GEOTECHNICAL
CONSULTING ENGINEERS

July 20, 2012
Project Number 2368-01-08

Berg Holdings
Attention: Mr. Skip Berg
2330 Marinship Way, Suite 301
Sausalito, California 94965

RE: Geotechnical Feasibility Evaluation
Upper Road Land Division - Vesting Tentative Map
Assessor's Parcel 073-011-26
Ross, California

Dear Mr. Berg:

This presents the results of our geotechnical update/feasibility evaluation of the proposed Upper Road Land Division at in Ross, California. Herzog Associates previously performed a geotechnical investigation at the site and presented results in their reports dated October 12, 1989, August 9, 1990, and July 12, 1993. Herzog Geotechnical has been retained as the geotechnical engineer or record for the project.

SCOPE OF WORK

The scope of our work was to conduct a site reconnaissance, review the previous Herzog Associates reports, conduct engineering analyses, and produce a letter containing conclusions regarding the feasibility of the proposed project and regarding the applicability of the October 12, 1989 report. Our work was performed in accordance with the terms and conditions outlined in our proposal dated December 12, 2008.

PROJECT DESCRIPTION

The project will consist of subdividing the property into three parcels for development of single-family residences. The parcels will be accessed by an asphalt paved driveway extending from Upper Road. The project is shown on the Vesting Tentative Map submittal by CSW/Stuber-Stroeh Engineering Group dated May 7, 2012.

WORK PERFORMED

We reviewed the following information as part of our work:

- CSW/Stuber-Stroeh Engineering Group, May 7, 2012, *Vesting Tentative Map, Upper Road Land Division,* Sheets C1 though C16.

- Herzog Associates, October 12, 1989, *Geotechnical Investigation: Proposed Property Subdivision, Upper Road, Ross, California,* Job Number 1385.2-0-1.

- Herzog Associates, August 9, 1990, *Access Road Exploration, Monte Bello Subdivision, Ross, California,* Job Number 1385.2-0-1.

- Herzog Associates, July 12, 1993, *Geotechnical Report: Geological Hazards Investigation, Lot 3 Monte Bello Subdivision, Ross, California,* Job Number 1385.2-0-1.

- Phoenix Consultants, November 16, 2001, Geologic Review and Update, Proposed Monte Bello Subdivision, Upper Road, Ross, Project 2026-1.

On July 2, 2012, our Certified Engineering Geologist and Principal Engineer performed a reconnaissance of the property. No additional subsurface exploration was performed as part of our scope of work.

CONCLUSIONS

Based on the results of our review and reconnaissance, we judge that the proposed project depicted on the May 7, 2012 *Vesting Tentative Map* submittal is feasible from a geotechnical standpoint. Prior to preparation of *Final Map,* we should be retained to prepare a design-level geotechnical report for the project including subsurface investigation within building envelopes which have been modified from prior studies.

LIMITATIONS

Our services consist of professional opinions and conclusions developed in accordance with generally-accepted geotechnical engineering principles and practices. We provide no other warranty, either expressed or implied. Our conclusions and recommendations are based on the information provided us regarding the proposed construction, the results of the field exploration and laboratory testing programs, and professional judgment. Verification of our conclusions and recommendations is subject to preparation of a design-level geotechnical report, our review of the project plans and specifications, and our observation of construction.

Our work 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, ground water or air, on or below, or around the site, nor did it include an evaluation or investigation of the presence or absence of wetlands.

We appreciate the opportunity to be of service to you. If you have any questions, please call.

Sincerely,
HERZOG GEOTECHNICAL

Craig Herzog, G.E.
Principal Engineer

Registered Professional Engineer
Craig W. Herzog
No. 002383
Exp. 9/30/13
Geotechnical
State of California

Reviewed by Donn Ristau, C.E.G. #1155

cc. CSW/Stuber-Stroeh Engineering Group, Inc.
Attention: Mr. Wayne Leach, P.E.
45 Leveroni Court
Novato, California 94949

PHOENIX Consultants
Engineering Geology

November 16, 2001

Skip Berg
c/o Berg Holdings
2330 Marinship Way Suite 301
Sausalito, California 94965

RE: Geologic Review and Update
Proposed Monte Bello Subdivision
Upper Road, Ross

Project: 2026-1

This letter confirms that we have reviewed the current Preliminary Grading and Drainage Plan, Sheets 5, 6, 12 and 13, the proposed Monte Bello subdivision in Ross, California, by CSW/Stuber-Stroeh. Prior to our review, we provided consultation and recommendations for modifications to the roadway and driveway alignments. These modifications have been incorporated into the plan and result in the following:

* avoidance of areas of potential instability, thereby reducing the amount of grading, over-excavation, and reconstruction of unstable slopes.

* reduction in extent of retaining wall construction.

* optimization of the use of conventional and/or geogrid-reinforced fills, thereby reducing the amount of off-site offhaul.

Our previous work indicated that the site conditions were essentially unchanged from the 1989 report and 1993 reconnaissance. During our previous reconnaissance we did not observe any new areas of instability along the road alignment or within the anticipated building envelopes. The only noteworthy change was that vegetation density has increased in several areas that were formerly more open.

Our previous work also indicated that the geotechnical recommendations are judged to be in conformance with current standards of practice and are considered to be applicable to the current subdivision plan.

Site specific foundation design criteria for structures was not included in the previous work because structures have not been proposed and specific building envelopes and the associated grading has not been delineated. Detailed foundation design recommendations will be dependant on site specific grading and type of construction, and should be developed when building locations have been finalized. At that time, a site specific investigation should be performed, and may involve additional subsurface exploration.

Once the Preliminary development concept has been approved the geotechnical recommendations concerning the construction of the retaining walls and bridge footings should be reviewed relative to the actual location and heights.

All cuts should be evaluated for stability by a Certified Engineering Geologist. Keyway excavations for construction of engineered fills should also be evaluated and the fills constructed as stipulated in the previous geotechnical reports. Reconstruction (or retention) of some cuts may be required depending on site conditions.

Roadway grading and access road construction should be reviewed and observed during construction by a qualified Geotechnical Engineer and Certified Engineering Geologist. This work should be documented in a final construction observation letter.

LIMITATIONS

This review has been prepared by Phoenix Consultants for the exclusive use of Mr. Skip Berg and his representatives with respect to a review of the preliminary design concept and geologic conditions within the subject site. Our services consist of professional opinions and conclusions of an Engineering Geologist developed in accordance with our understanding of generally accepted geotechnical principles and practices. We provide no other warranty, either express or implied. Our conclusions and recommendations are based on our review of previous work, previous field reconnaissance and professional judgement.

If you have any questions related to these issues, please contact us.

Sincerely

Donn Ristau
Engineering Geologist - 1155

August 9, 1990

1385.02-02-3

Mr. Robert Toigo
Institute for Fiduciary Education
1112 "I" Street, Suite #210
Sacramento, California 95814

RE: Access Road Exploration
Monte Bello Subdivision
Ross, California

This presents the results of our additional subsurface exploration along two portions of the access road/driveway alignment within the proposed Monte Bello Subdivision in Ross, California.

The purpose of our additional subsurface exploration was to provide for a more detailed analysis of the extent of grading (over-excavation and slope buttressing) within two areas where our previous work indicated a potential for slope instability.

This work was done as part of a request by the Ross Town Council concerning the impacts the proposed grading may have on issues related to ground disturbance, tree removal and soil off-haul. The data is being submitted to facilitate a review of whether this particular issue should be included in a proposed focused EIR.

The subsurface exploration will also be used for developing the design criteria for the Final Map Improvement Plans, as per the recommendation by Miller Pacific in their review letter of June 14, 1990.

BACKGROUND DISCUSSION

Our previous report of October 12, 1989 identified two locations where the proposed road alignment would cross through areas that contained potentially unstable soil/rock materials. The first area is immediately south of the planned bridge crossing. The second area is where the access roadway transitions to the private driveway to Lot 5.

On July 12 and 13, 1990, thirteen (13) test pits were excavated within the two areas of concern. Plate 1 shows the test pit locations as well as the locations of nearby test pits from our previous work. Plates 2 through 14 present descriptive logs of the most-recently excavated test pits.

Our letter of April 19, 1990 presented our estimates of the aerial extent of reworking for the alignment in the two areas. Area 1 (south of the bridge) was estimated at 150 feet (upslope/downslope) by 100 feet (across slope). Area 2 was estimated at 60 feet (upslope/downslope) by 150 feet (across slope).

SITE CONDITIONS

Area 1: Test pits B1 and B2 encountered weak sheared shale and greenstone down to a depth of 12 feet. Seepage was encountered at a depth of 10 feet. These conditions are similar to those encountered in Test Pit 1 of our 1989 investigation where a weak greenstone rock and seepage were evident at 13 feet. The depth of excavation for the fills and cut slopes within the area of Test Pit 1 and Test Pits B1 and B2 will be approximately 12 to 14 feet.

Within Test Pits B3, B4 and B5, we did not encountered the weak, sheared rock found in Test Pit 1 and in B1 and B2, nor was water encountered. As such, the depth of excavation for the keyways and benches in this area will be approximately eight (8) to nine (9) feet. Because the weak rock conditions and seepage were not encountered, it is our conclusion that reconstruction of the proposed cut slope will not be required within the areas of Test Pits B4 and B5.

The additional test pit exploration indicates that approximately 75 linear feet of the cut slope area (instead of 100 feet) will need to be constructed as an engineered fill. Within the proposed fill slope, approximately 75 feet will require a keyway that is 12 to 14 feet deep. The remaining portion of the keyway would transition to a depth of approximately eight (8) feet.

Area 2: Test Pits B6, B7, B8, B9 and B10 encountered very dense to very stiff colluvial soils and relatively shallow sandstone bedrock in the northern and central portions of the alignment. Although bedrock was not evident in B7 and B8, these soils appear sufficiently strong to act as a base for the keyways for fills. Based on the conditions encountered, the keyway excavations would extend to approximately six (6) to eight (8) feet, and there does not appear to be any justification to require buttressing the cuts or slopes on the upslope side of the roadway in this area. Thus, the proposed grading/reconstruction for the northern half of the alignment will be substantially less than originally expected.

Within Test Pits B12 and B13, weak, potentially unstable materials were encountered between depths of six (6) to twelve (12) feet, and the southern 75 feet of the alignment will require buttressing/reconstruction in the upslope cut area. The depth of excavations for the keyway and benches would be approximately twelve (12) to thirteen (13) feet.

CONCLUSIONS

Based on our additional test pit exploration, we conclude that the amount of over-excavation and reconstruction along the upslope sides of the roadway would be substantially reduced from our previous estimate, and that the proposed cuts for the southern one-third (1/3) to one-half (1/2) of Area 1 and the northern one-half (1/2) of Area 2 may be made as planned without reconstruction. The depth of excavation for keyways would also be less than originally estimated for fills in these areas; on the order of six (6) to eight (8) feet, instead of ten (10) to twelve (12) feet.

However, slope reconstruction work will still be required for upslope portions of the roadway/driveway alignment, as originally proposed. In Area 1, the linear extent would be approximately 75 feet and in Area 2, it would be approximately 80 feet. The depth of excavation for keyways and benches will be on the order of 12 to 14 feet.

Based on the conditions encountered and the limited extent of slope reconstruction that would be required to stabilize uphill roadway banks, it is our opinion that these conditions do not pose a significant impact with respect to grading and that a focused EIR concerning the geotechnical elements of the proposed subdivision does not appear to be warranted. This conclusion is consistent with that presented by the Town's Geotechnical Review and Planning Staff.

We trust this provides the information you require at this time. If you have any questions, or wish to discuss this further, please do not hesitate to call.

Very truly yours,

HERZOG ASSOCIATES

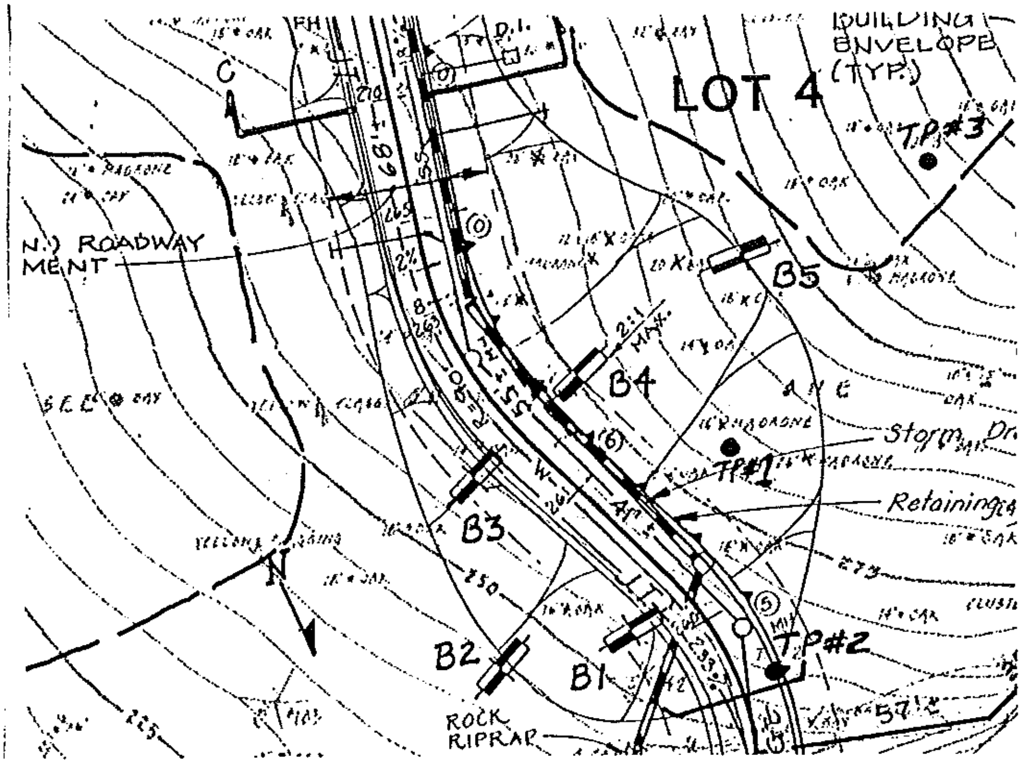
Donn A. Ristau
Certified Engineering Geologist - 1155

DAR:cms(S-14.20)

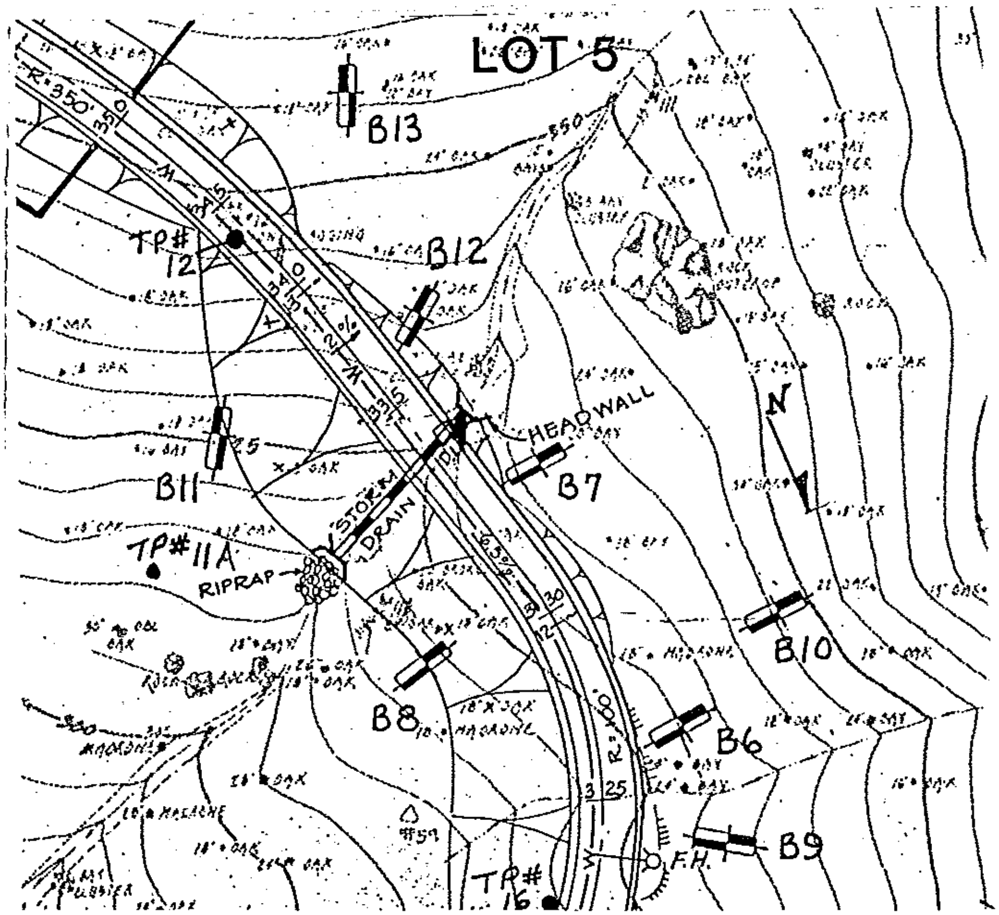
Three Copies Submitted

Attachments: Plates 1 through 16

AREA 1



AREA 2



Herzog Associates
Geoscientists

Job No: 1385.02-02-3
Appr: Jr
Drwn: cms
Date: Aug 1990

SITE PLAN

MONTE BELLO SUBDIVISION

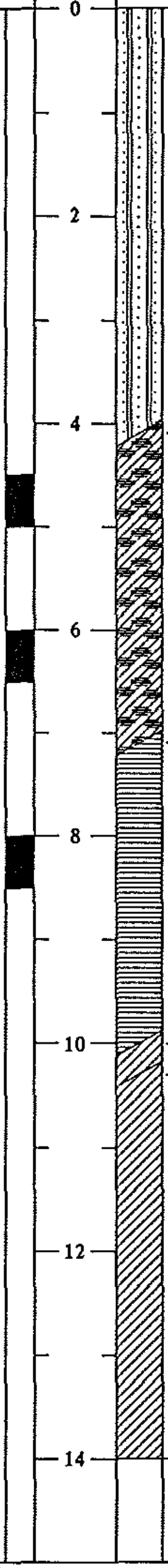
Ross, California

PLATE

1

Equipment: Trackhoe 24" Bucket
Logged By: Noble
Elevation: 255.0 **
Start Date: 7-12-90
Finish Date: 7-13-90

Other Laboratory Tests
Dry Density (pcf)
Moisture Content (%)
Depth (feet)



RED BROWN SILTY SAND (SM), medium dense, slightly moist, slightly porous, with occasional gravels and organics.

GREY TO BROWN CLAYEY GRAVEL TO GRAVELLY CLAY (GC/CL), stiff, moist, with occasional larger boulders; grading to grey in color at 5.5'.

GREY SHEARED SHALE, weak to friable, wet, with occasional resistant graywacke boulders.

Very wet, seepage.

BROWN SANDY CLAY (CL), medium stiff, moist to wet, no planes evident.

RED BROWN SANDY CLAY TO CLAYEY SAND (CL), stiff to dense, wet, grading to orange brown at 13', deeply weathered greenstone

Bottom of test pit at 14.0 feet.

** Elevation taken from Tentative Map Monte Bello, Ross, CA., by Stuber-Stroeh Associates, Inc., sheet 1 of 9, dated Sept 1989.

Herzog Associates
Geoscientists

Job No: 1385.02-02-3
Appr:
Drwn: cms
Date: Aug 1990

LOG OF TEST PIT B1

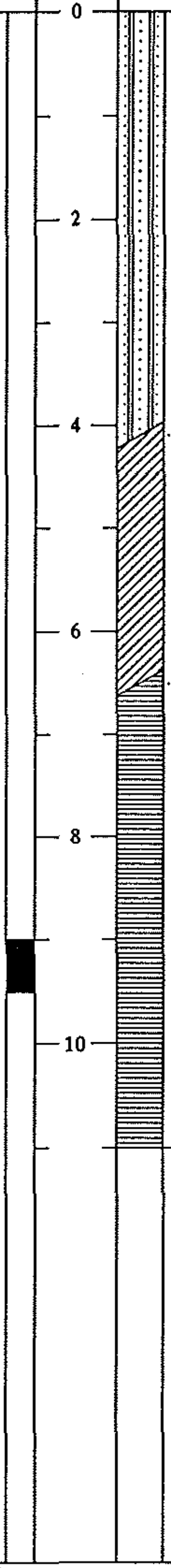
MONTE BELLO SUBDIVISION

Ross, California

PLATE
2

Equipment: Trackhoe 24" Bucket
Elevation: 245.0 **
Logged By: Noble
Start Date: 7-12-90
Finish Date: 7-13-90

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)



RED BROWN SILTY SAND (SM), medium dense, slightly moist, slightly porous, with occasional gravels.

GREY BROWN SANDY CLAY (CL), medium stiff, moist to wet, becoming soft between 5.5' and 6.5', seepage present.

GREY SHEARED SHALE, weak to friable, highly weathered, with occasional resistant graywacke boulders, becoming more competent at 9.0', stronger, less weathered.

Bottom of test pit at 11.0 feet.

Herzog Associates
Geoscientists

Job No: 1385.02-02-3
Appr:
Drwn: cms
Date: Aug 1990

LOG OF TEST PIT B2

MONTE BELLO SUBDIVISION

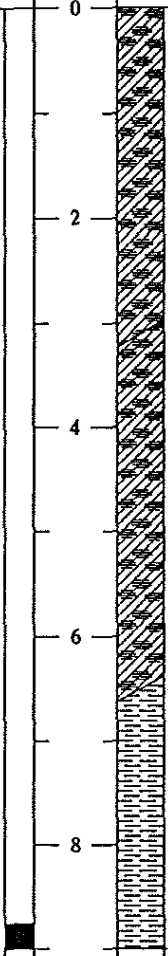
Ross, California

PLATE

3

Equipment: Trackhoe 24" Bucket
Elevation: 260.0 **
Logged By: Noble
Start Date: 7-12-90
Finish Date: 7-13-90

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Depth (feet)



BROWN CLAYEY GRAVEL TO GRAVELLY CLAY (GC/CL), medium dense to medium stiff, porous.

BROWN CLAYEY GRAVEL (GC), dense, moist, gravels consist of greenstone, relatively sharp contact at 6.5', dipping to north.

LIGHT BROWN SILTSTONE, with interbedded sandstone, extremely closely spaced fractures, friable to weak, highly weathered with soil between fracture surfaces, becoming more competent at 9.0'.

Bottom of test pit at 9.0 feet.
No free water encountered.

Herzog Associates
Geoscientists

Job No: 1385.02-02-3
Appr:
Drwn: cms
Date: Aug 1990

LOG OF TEST PIT B3

MONTE BELLO SUBDIVISION

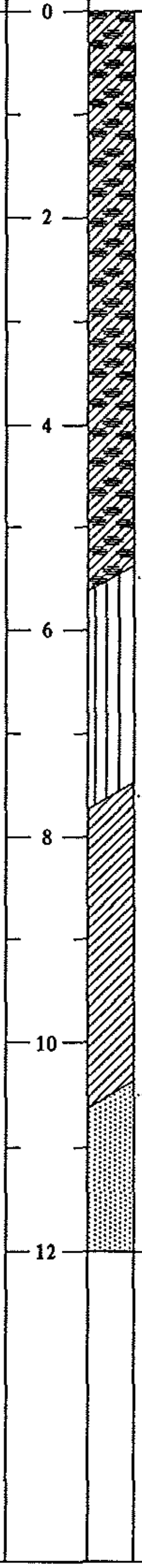
Ross, California

PLATE
4

Equipment: Trackhoe 24" Bucket
Logged By: Noble

Elevation: 275.0 **
Start Date: 7-12-90
Finish Date: 7-13-90

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Depth (feet)



RED BROWN CLAYEY GRAVELS (GC), moist, medium dense to dense, slightly porous upper 2', gravels consist of greenstone.

LIGHT BROWN SANDY SILT (ML), stiff, slightly moist, at 6.7' grey clay present, continuous across trench, dipping to northwest at 27 degrees.

GREY BROWN SANDY CLAY (CL), very stiff, moist, with occasional gravels, grey clay at 8.4', oriented same direction as clay above.

BROWN TO GREY SILTY SANDSTONE, friable to weak, highly weathered, becoming more competent at 12.0'

Bottom of test pit at 12.0 feet.
No free water encountered.

Herzog Associates
Geoscientists

Job No: 1385.02-02-3
Appr:
Drwn: cms
Date: Aug 1990

LOG OF TEST PIT B4

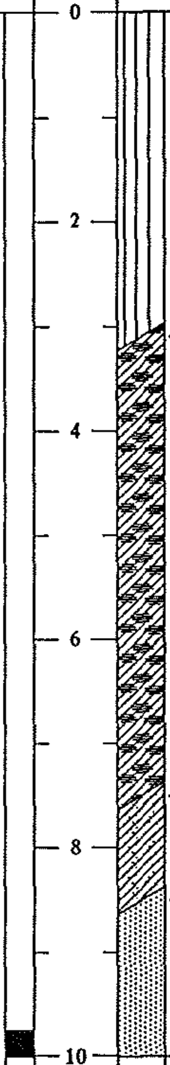
MONTE BELLO SUBDIVISION

Ross, California

PLATE
5

Equipment: Trackhoe 24" Bucket
Elevation: 295.0 **
Logged By: Noble
Start Date: 7-12-90
Finish Date: 7-13-90

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Depth (feet)



BROWN SANDY SILT/SILTY SAND (ML/SM), medium stiff, moist, slightly porous.

RED BROWN CLAYEY GRAVEL (GC0, dense, moist, gravel consists of greenstone.

RED BROWN CLAYEY SAND (SC), very dense, moist. (Residual)

YELLOW BROWN SILTY SANDSTONE, extremely closely spaced fractures, weak, highly weathered.

Bottom of test pit at 10.0 feet.
No free water encountered.

Herzog Associates
Geoscientists

Job No: 1385.02-02-3
Appr:
Drwn: cms
Date: Aug 1990

LOG OF TEST PIT B5

MONTE BELLO SUBDIVISION

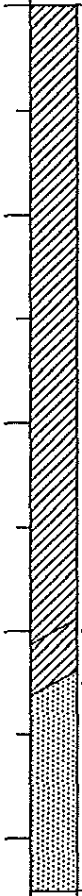
Ross, California

PLATE
6

Equipment: Trackhoe 24" Bucket
Logged By: Noble

Elevation: 330.0 **
Start Date: 7-12-90
Finish Date: 7-13-90

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Depth (feet)
			0
			2
			4
			6
			8



BROWN SANDY CLAY TO GRAVELLY CLAY (CL/GC), stiff to dense, moist below 2', gravels consist of greenstone and chert.

DARK GREY SANDY CLAY (CL), stiff, moist (appears to be highly weathered sheared shale), sharp contact with upper and lower units, slickensided surfaces, dipping downslope at 2:1.

BROWN SANDSTONE, closely spaced fractures, weak to moderately strong, highly weathered.

Bottom of test pit at 8.5 feet.
No free water encountered.

Herzog Associates
Geoscientists

Job No: 1385.02-02-3
Appr:
Drwn: cms
Date: Aug 1990

LOG OF TEST PIT B6

MONTE BELLO SUBDIVISION

Ross, California

PLATE
7

Equipment: Trackhoe 24" Bucket

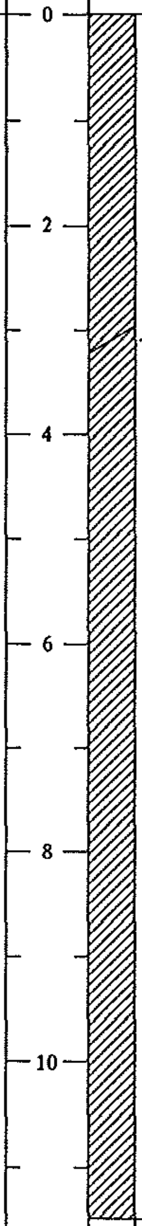
Logged By: Noble

Elevation: 335.0 **

Start Date: 7-12-90

Finish Date: 7-13-90

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Depth (feet)



BROWN SANDY CLAY (CL), medium stiff, moist, slightly porous top 2'.

DARK BROWN TO BROWN GRAVELLY CLAY (CL), stiff, moist, increasing gravels below 8', consisting of chert and greenstone, becoming very stiff to very dense, difficult to excavate at 8.0'.

Bottom of test pit at 11.5 feet.
No free water encountered.

Herzog Associates
Geoscientists

Job No: 1385.02-02-3

Appr:

Drwn: cms

Date: Aug 1990

LOG OF TEST PIT B7

MONTE BELLO SUBDIVISION

Ross, California

PLATE

8

Equipment: Trackhoe 24" Bucket

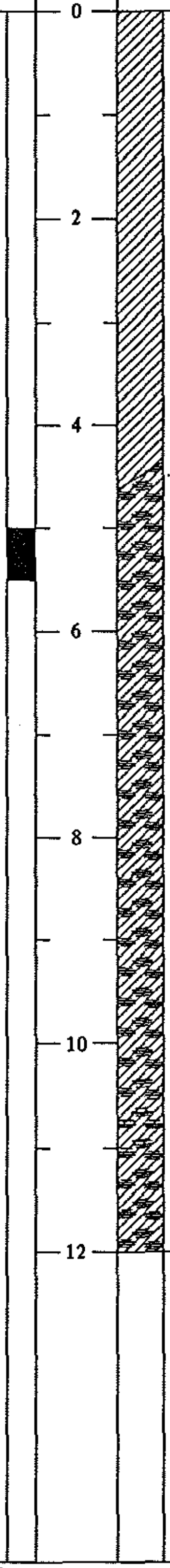
Logged By: Noble

Elevation: 315.0 **

Start Date: 7-12-90

Finish Date: 7-13-90

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Depth (feet)



BROWN SANDY CLAY (CL), medium stiff, slightly moist, with occasional gravels, becoming stiff at 4'.

MOTTLED BROWN SANDY CLAY TO CLAYEY GRAVELS (CL/GC), stiff, dense, gravels consist of greenstone and chert, becoming very stiff/very dense at 9.0', difficult to excavate.

Bottom of test pit at 12.0 feet.
No free water encountered.

Herzog Associates
Geoscientists

Job No: 1385.02-02-3

Appr:

Drwn: cms

Date: Aug 1990

LOG OF TEST PIT B8

MONTE BELLO SUBDIVISION

Ross, California

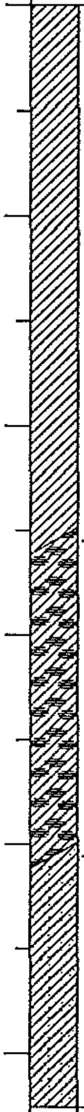
PLATE

9

Equipment: Trackhoe 24" Bucket
Logged By: Noble

Elevation: 330.0 **
Start Date: 7-12-90
Finish Date: 7-13-90

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Depth (feet)
			0
			2
			4
			6
			8
			10



BROWN SANDY CLAY (CL), medium stiff to stiff, moist, at 4.5' thin continuous plane dipping downslope approximately 20 degrees.

DARK GREY TO BROWN CLAYEY GRAVEL TO GRAVELLY CLAY (GC/CL), dense to stiff.

ORANGE BROWN CLAYEY SAND (SC), dense, moist with abundant greenstone gravels, dense, hard, difficult to excavate.

Bottom of test pit at 10.5 feet.
No free water encountered.

Herzog Associates
Geoscientists

Job No: 1385.02-02-3
Appr:
Drwn: cms
Date: Aug 1990

LOG OF TEST PIT B9

MONTE BELLO SUBDIVISION

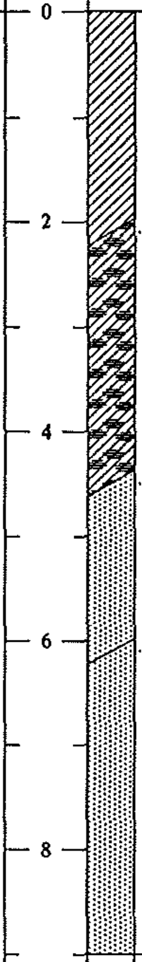
Ross, California

PLATE
10

Equipment: Trackhoe 24" Bucket
Logged By: Noble

Elevation: 347.0 **
Start Date: 7-12-90
Finish Date: 7-13-90

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Depth (feet)



BROWN SANDY CLAY (CL), medium stiff, slightly moist, slightly porous.

REDDISH BROWN CLAYEY GRAVEL (GC), dense, slightly moist, gravels consist of sandstone.

GREY BROWN SANDSTONE with interbedded shale, extremely closely spaced fractures, weak, highly weathered.

BROWN SANDSTONE, extremely closely spaced fractures, moderately strong, highly weathered.

Bottom of test pit at 9.0 feet.
No free water encountered.

Herzog Associates
Geoscientists

Job No: 1385.02-02-3
Appr:
Drwn: cms
Date: Aug 1990

LOG OF TEST PIT B10

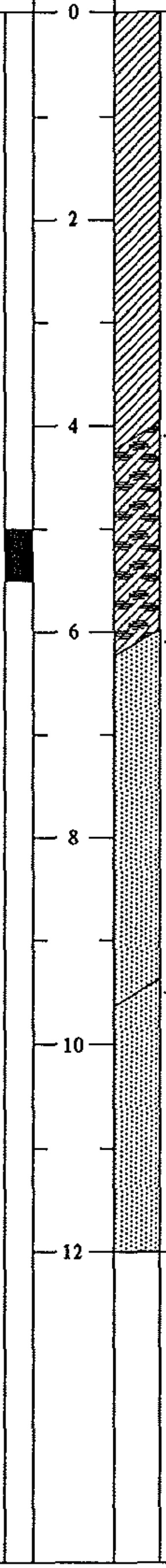
MONTE BELLO SUBDIVISION

Ross, California

PLATE
11

Equipment: Trackhoe 24" Bucket
Logged By: Noble
Elevation: 327.0 **
Start Date: 7-12-90
Finish Date: 7-13-90

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Depth (feet)



BROWN SANDY CLAY (CL), medium stiff to stiff, moist.

RED BROWN CLAYEY GRAVEL TO GRAVELLY CLAY (GC/CL), stiff, dense, gravel consists of sandstone.

YELLOW BROWN SANDSTONE, closely spaced fractures, moderately strong, highly weathered, with soil between fractures, occasional interbedded shale, extremely closely spaced fractures, shear surface at 9.5', dipping approximately 24 degrees, adversely, possibly highly weathered shale bed.

YELLOW BROWN SANDSTONE and interbedded shale, closely spaced fractures, moderately strong, highly weathered.

Bottom of test pit at 12.0 feet.
No free water encountered.

Herzog Associates
Geoscientists

Job No: 1385.02-02-3
Appr:
Drwn: cms
Date: Aug 1990

LOG OF TEST PIT B11

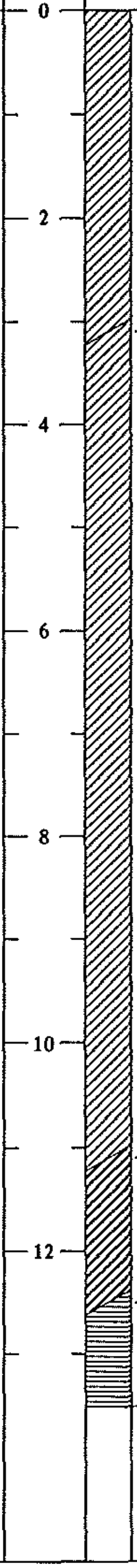
MONTE BELLO SUBDIVISION

Ross, California

PLATE
12

Equipment: Trackhoe 24" Bucket
Logged By: Noble
Elevation: 340.0 **
Start Date: 7-12-90
Finish Date: 7-13-90

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Depth (feet)



BROWN SANDY CLAY (CL), medium stiff, moist, slightly porous, with occasional greenstone gravels and cobbles.

LIGHT BROWN TO RED BROWN SANDY CLAY (CL), stiff, moist, with occasional to abundant greenstone gravels.

GREY SILTY CLAY (CH), medium stiff, moist, shear plane dipping to approximately N25, striations present on surface, sharp contact with upper unit.

GREY SHEARED SHALE, friable to weak, highly weathered to clay consistency.

Bottom of test pit at 13.5 feet.
No free water encountered.

Herzog Associates
Geoscientists

Job No: 1385.02-02-3
Appr:
Drwn: cms
Date: Aug 1990

LOG OF TEST PIT B12

MONTE BELLO SUBDIVISION

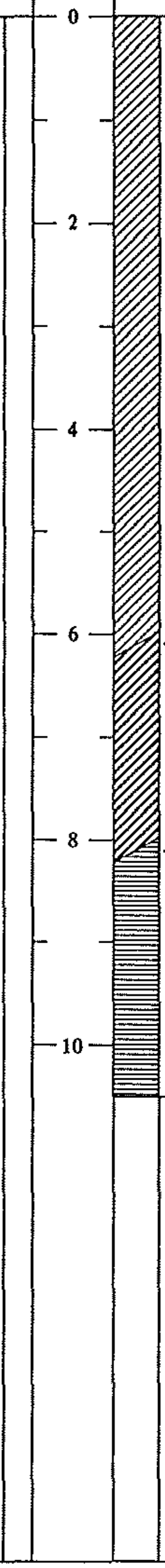
Ross, California

PLATE
13

Equipment: Trackhoe 24" Bucket
Logged By: Noble

Elevation: 355.0 **
Start Date: 7-12-90
Finish Date: 7-13-90

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Depth (feet)



BROWN SANDY CLAY TO GRAVELLY CLAY (CL), stiff, moist, sharp abrupt planar contact at 6.0', dipping N25.

GREY SILTY CLAY (CH), stiff, moist with abundant white staining, highly weathered, sheared shale.

DARK GREY SHEARED SHALE, weak, friable, highly weathered.

Bottom of test pit at 10.5 feet.
No free water encountered.

Herzog Associates
Geoscientists

Job No: 1385.02-02-3
Appr:
Drwn: cms
Date: Aug 1990

LOG OF TEST PIT B13

MONTE BELLO SUBDIVISION

Ross, California

PLATE

14

MAJOR DIVISIONS					TYPICAL NAMES
COARSE GRAINED SOILS More than Half > #200 sieve	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	▤	VELL GRADED GRAVELS, GRAVEL-SAND MIXTURES
			GP	▤	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES
		GRAVELS WITH OVER 12% FINES	GM	▤	SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES
			GC	▨	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	░	VELL GRADED SANDS, GRAVELLY SANDS
			SP	░	POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH OVER 12% FINES	SM	▥	SILTY SANDS, POOORLY GRADED SAND-SILT MIXTURES
			SC	▨	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS More than Half < #200 sieve	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	▥	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS VITH SLIGHT PLASTICITY
			CL	▨	INORGANIC CLAYS OF LOV TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	▥	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOV PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	▥	INORGANIC SILTS, MICACEOUS OR DIATOMACIOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	▨	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	░	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			Pt	▩	PEAT AND OTHER HIGHLY ORGANIC SOILS

UNIFIED SOIL CLASSIFICATION SYSTEM

Shear Strength, psf

Confining Pressure, psf

Consol	Consolidation	Tx	2630 (240)	Unconsolidated Undrained Triaxial
LL	Liquid Limit (in %)	Tx sat	2100 (575)	Unconsolidated Undrained Triaxial, saturated prior to test
PL	Plastic Limit (in %)	DS	3740 (960)	Consolidated Drained Direct Shear
PI	Plasticity Index	FVS	1320	Field Vane Shear
Gs	Specific Gravity	UC	4200	Unconfined Compression
SA	Sieve Analysis	LVS	500	Laboratory Vane Shear
■	Undisturbed Sample	SS	Shrink Swell	
☒	Bulk or Disturbed Sample	EI	Expansion Index	
▦	Standard Penetration Test	P	Permeability	
☐	Sample Attempt with No Recovery	SE	Sand Equivalent	

KEY TO TEST DATA

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Job No: 1385.02-02-3
Appr:
Drwn: cms
Date: Aug 1990

SOIL CLASSIFICATION CHART AND KEY TO TEST DATA

MONTE BELLO SUBDIVISION

Ross, California

PLATE

15

ROCK SYMBOLS

▤ SHALE OR CLAYSTONE

▦ SILTSTONE

▒ SANDSTONE

▦ CONGLOMERATE

░ CHERT

▦ PYROCLASTIC

░ VOLCANIC

▩ PLUTONIC

▩ SERPENTINITE

▩ METAMORPHIC ROCKS

░ ALTERED ROCKS

▨ SHEARED ROCKS

LAYERING

MASSIVE	Greater than 6 feet
THICKLY BEDDED	2 to 6 feet
MEDIUM BEDDED	8 to 24 inches
THINNLY BEDDED	2-1/2 to 8 inches
VERY THINNLY BEDDED	3/4 to 2-1/2 inches
CLOSELY LAMINATED	1/4 to 3/4 inches
VERY CLOSELY LAMINATED	Less than 1/4 inch

JOINT, FRACTURE, OR SHEAR SPACING

VERY WIDELY SPACED	Greater than 6 feet
WIDELY SPACED	2 to 6 feet
MODERATELY SPACED	8 to 24 inches
CLOSELY SPACED	2-1/2 to 8 inches
VERY CLOSELY SPACED	3/4 to 2-1/2 inches
EXTREMELY CLOSELY SPACED	Less than 3/4 inch

HARDNESS

SOFT - Pliable; can be dug by hand

FIRM - Can be gouged deeply or carved with a pocket knife

MODERATELY HARD - Can be readily scrached by a knife blade; scratch leaves heavy trace of dust and is readily visable after the powder has been blown away

HARD - Can be scratched with difficulty; scratch produces little powder and is often faintly visable

VERY HARD - Cannot be scratched with pocket knife; leaves a metallic streak

STRENGTH

PLASTIC - Capable of being molded be hand

FRIABLE - Crumbles by rubbing with fingers

WEAK - An unfractured specimen of such material will crumble under light hammer blows

MODERATELY STRONG - Specimen will withstand a few heavy hammer blows before breaking

STRONG - Specimem will withstand a few heavy ringing hammer blows and usually yields large fragments

VERY STRONG - Rock will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments

DEGREE OF WEATHERING

HIGHLY WEATHERED - Abundant fractures coated with oxides, carbonates, sulphates, mud, etc., thourough discoloration, rock disintegration, mineral decomposition

MODERATELY WEATHERED - Some fracture coating, moderate or localized discoloration, little to no effect on cementation, slight mineral decomposition

SLIGHTLY WEATHERED - A few stained fractures, slight discoloration, little or no effect on cementation, no mineral decomposition

FRESH - Unaffected by weathering agents, no appreciable change with depth

Herzog Associates
Geoscientists

Job No: 1385.02-02-3
Appr:
Drwn: cms
Date: Aug 1990

Engineering Geology Rock Terms & Test Pit Explanation
MONTE BELLO SUBDIVISION
Ross, California

PLATE
16

July 12, 1990

1385.02-01-3

Mr. Robert Toigo
Institute for Fiduciary Education
1112 "I" Street, Suite 210
Sacramento, California 95814

RE: Geotechnical Report
Geological Hazards Investigation
Lot 3 Monte Bello Subdivision
Ross, California

This report presents the results of our supplemental subsurface investigation for Lot 3 of the proposed Monte Bello subdivision in Ross, California. Our previous reports of October 7, 1982 and October 12, 1989 have discussed the geotechnical conditions and aspects of the proposed 5-lot subdivision. Lot 3 was originally shown as Lot 5 on Plate 1 of the October 12, 1989 Herzog Associates Report. Subsequent reorganization of property lines was performed and the area in question was designated as Lot 3.

The purpose of the supplemental work was to address the comments by Miller Pacific Engineering Group, dated June 14, 1990, concerning their geotechnical review of our previous work.

According to the Miller Pacific review letter, additional work (for this phase of planning) was recommended only for Lot 3. This recommendation was based on the conditions that:

> "the building envelope is in a topographic lobe at the lower end of an unstable area. Test Pit TP-19 shows questionable material to a depth of 10 feet. Additional exploration should be carried out to confirm the lot's suitability."

On July 2, 1990 we explored the subsurface conditions within the proposed building area of Lot 3 with nine (9) test pits. We also performed a geologic reconnaissance of the drainage ravine south-southeast of the lot, down to the main creek and along the ridge flanks north and east of the lot.

The test pits were located and logged by our Certified Engineering Geologist (CEG) and our Staff Geologist. Our CEG performed the geologic reconnaissance. Test pits were excavated with a track-mounted backhoe and ranged in depth from 6.5 to 11.5 feet. Logs of the test pits are presented on Plates 2 through 10. Test pit locations are shown on Plate 1, and were referenced and taped from existing stakes and story poles. The locations were not surveyed and should be considered approximate.

BACKGROUND DISCUSSION

As part of our October 7, 1982 report, one test pit (TP-19) was excavated within the area of Lot 3. The log of that test pit indicates a variety of apparent soil layers that differ primarily with respect to color (yellow/brown to red/brown to gray). A layer at two and one half (2 1/2) to three and one half (3 1/2) feet was questionably logged as slide debris. Another layer at nine (9) to ten (10) feet was questionably considered old topsoil. The exact spatial relationships of the layers was not evident or conclusive, and this area was not mapped as containing slide deposits. Because of the depth to definable sandstone/shale bedrock, the original mapping depicted the lot as being within a deposit of colluvial soils. A lobe of composite colluvium/slide debris was mapped in the south-central portion of the lot.

As part of our October 12, 1989 work, one test pit (TP-6A) was excavated in the lobed area of previously identified colluvial soil/slide debris. That test pit encountered gray/brown to red/brown sandy clay and clayey gravels that we interpreted as representing colluvial soils. Residual soils were described from nine (9) to eleven-and-one-half (11-1/2) feet and sandstone bedrock was encountered at 11-1/2 feet.

Within Test Pit 6-A there were no well defined planar contacts or slickensided surfaces indicative of slide debris, although the lobate topography is suggestive of materials that may have been transported downslope. All of the soils exposed in the test pit were either stiff or dense, suggesting moderately strong soil properties. Wet zones or other zones of potential weakness were not evident.

Based on our 1982 and 1989 work, we concluded that the lot was geotechnically suitable for development. We also noted that any development within the lot would require a foundation system designed to resist the potential effects of soil creep above the bedrock (sandstone/shale) contact.

SUPPLEMENTAL WORK

Slope Reconnaissance

Our examination of the incised drainage ravine south of the proposed site indicates that gray/brown to yellow/brown sandstone and shale is present in the ravine sides and bottom, approximately 50 feet south-southwest of the southeast corner of the building envelope.

The sandstone is very closely to closely fractured and moderately strong to strong and the resistant nature of the sandstone has created a five (5) to six (6) foot high vertical drop in the ravine. The shale bedding has been deformed between the sandstone and is very closely fractured and weak to moderately strong. This rock outcrop extends for approximately 25 feet down the ravine bottom. The disposition and appearance of the rock and integrity of the shale unit suggests that the outcrop is in place. The rock composition is similar to the non-melange sandstone/shale units that are present within other portions of the property and appears to correlate with the typical Cretaceous sandstone (Ks) unit mapped locally by Rice et al (1976).

East of the sandstone outcrop, the ravine bottom and banks are lined with massive metagraywacke that may be part of the typical Franciscan melange assemblage. In places, the graywacke forms eight (8) to ten (10) foot high vertical faces and the rocks are smooth and water-worn, suggesting that the graywacke is in place and forming a resistant erosional base. The sandstone/shale unit and metagraywacke rocks exposed in the ravine appear to form the ridge spur that is present in the eastern portion of the building envelope of Lot 3.

The flanks of the ridge spur that extend east and north of the proposed building are relatively uniform and during our reconnaissance we did not observe evidence of any slope failure that would indicate the eastern or northern portion of the ridge is unstable.

A small slump with lobed topography is evident north-northwest of the proposed building envelope, but does not appear to be of significant extent such that it would pose a constraint to development. Depending upon the extent of potential development in this area, the slump may require removal or buttressing.

However, if construction is located away from the failure, mitigation may not be necessary. Detailed recommendations concerning mitigation would have to be developed at the time construction is proposed for the area.

Subsurface Investigation

Data from our supplemental work indicates that the soil/rock conditions within Lot 3 are structurally complex, but that the stability conditions appear comparable to those previously discussed, and that the development of the lot is geotechnically feasible.

Our current work indicates that the eastern portion of the proposed building envelope is underlain by sandstone and shale bedrock at depths that range from three (3) to eight (8) feet. The rock exposed in these pits is lithologically similar to that exposed in the ravine to the south-southwest.

Toward the central portion of the building envelope, the sandstone/shale unit is overlain by a thickening wedge of melange greenstone, metagraywacke and sheared shale. The contact between the melange/non-melange rocks is sheared and irregular but it is not a planar contact. In some areas, the contact dips to the east and north, in others it dips west and north.

The melange rocks are deeply weathered and altered to sandy clay or clayey gravel. The weathering of the rock gives the impression of a soil deposit and where the greenstone/sandstone/shale units are intermixed, it appears that several layers of soil could be present. The presence of rock fragments in the weathered melange matrix also is similar to the appearance of colluvial soils, except that the rock fragments from specific melange units tend to be of a homogeneous lithology.

Within Test Pits S-5, S-6 and S-7 the soil/rock deposit is a thick clayey gravel with abundant greenstone fragments and the materials appear as a uniform deposit of deeply weathered rock. The homogeneity of this material, especially when compared to test pits to the south and east, where heterogenous conditions exist, also tends to indicate that the deposit is weathered melange. The presence of laterally grading material types between Test Pits S-4, S-9 and S-5 at comparable elevations would not be expected if this material were slide debris.

Our review of the conditions described from TP-19 in our 1982 report is compatible with the supplemental subsurface information. The areas of yellow/brown sandy clay correlate with sheared weathered rock or residual soil from the sandstones. Areas of red/brown gravelly clay correspond to sheared greenstone and gray silty or sandy clays appear to represent weathered sheared shales. Test Pit S-5 was excavated west of Test Pit 19 and did not encounter any soil deposits that would correlate with the unit previously mapped as "Old Topsoil". Based on our current supplemental information we would consider the horizon encountered in Test Pit 19 as part of the weathered melange.

Within Test Pit 19, sandstone bedrock that is similar to shallower rocks to the east, was present at a depth of approximately 10-1/2 feet. The sandstone is overlain by a thick sequence of weathered melange containing several different rock types.

CONCLUSIONS

Based on a review of our original work and our supplemental reconnaissance and subsurface investigation, we conclude that the proposed building envelope for Lot 3 is geotechnically feasible and that the building area does not contain deep slide deposits or other materials that would preclude its consideration for development.

Bedrock exposures in the ravine to the south and exposures from the test pit exploration indicate the eastern portion of the building area is underlain at a relatively shallow depth by bedded sandstone and shale. The central portion of the lot contains a mixed zone of sandstone and shale overlain by a wedge of weathered melange. The western portion of the lot appears to be deeply weathered greenstone.

We did not encounter continuous planar contacts, or prominent zones of weakness that would suggest the soil/rock materials are inherently unstable. Likewise, slope failures along the eastern and northern portions of the ridge spur, downslope of the building envelopes, are not prevalent and the spur appears relatively stable.

Based on the conditions encountered in the test pits, we conclude that, with proper engineering design, grading and drainage control, the lot is suitable for development in its present condition.

We trust this provides the information you require at this time. If you have any questions, or wish to discuss this further, please do not hesitate to call.

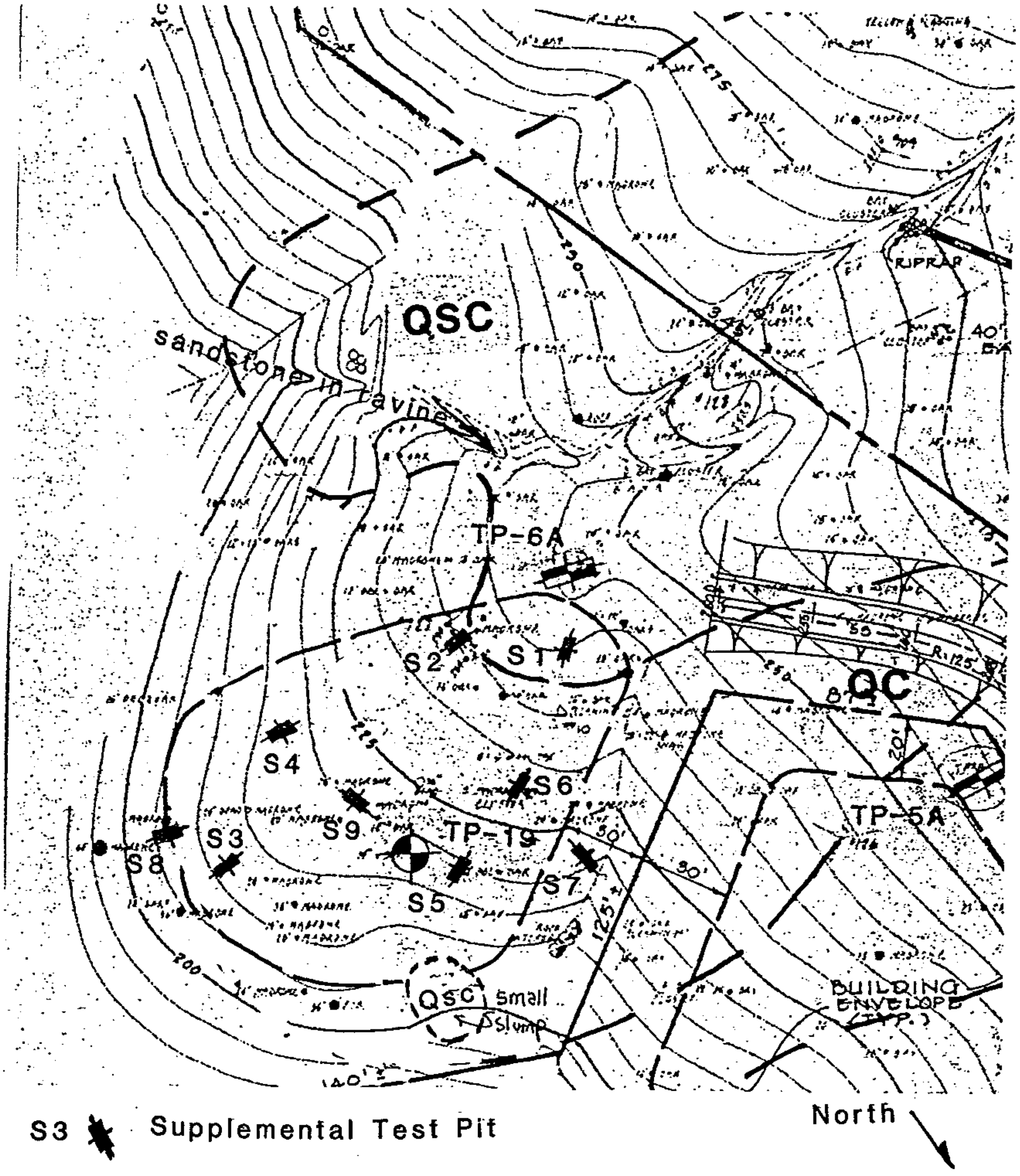
Very truly yours,

HERZOG ASSOCIATES

Donn A. Ristau, Ph.D.
Certified Engineering Geologist - 1155

DAR:mth:cms(S-14.1)

Attachments: Plates 1 through 12
Distribution List



Topographic base, property line and building envelope form Plate 1 of 1989 Herzog Associates report. Property lines and building envelope were later revised slightly.

scale 1":50'

Herzog Associates
Geoscientists

Job No: 1385.02-01-3
Appr: dr
Drwn: cms
Date: Jul 1990

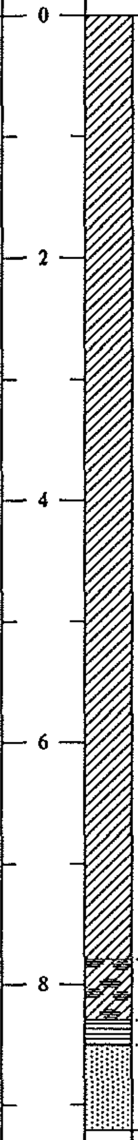
SITE PLAN

LOT 3 - MONTE BELLO SUBDIVISION

Ross, California

PLATE
1

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Depth (feet)	Equipment: Trackhoe; 24" Bucket Logged By: J. Noble	Elevation: ** Start Date: 7-2-90 Finish Date: 7-2-90



BROWN SANDY CLAY (CL), stiff, moist, with occasional greenstone gravels

sharp contact

VIOLET/BLUE-GRAY CLAYEY GRAVEL (GC), dense, moist, sharp contact, vague planar surface sloping 17 degrees to north.

GRAY SHEARED SHALE, firm, friable, highly weathered to soil consistency

BROWN SANDSTONE, moderately strong to strong, highly weathered

Bottom of test pit at 9.0 feet. No free water encountered.

** Reference: Existing ground surface at time of excavation.

Herzog Associates
Geoscientists

Job No: 1385.02-01-3
Appr: JL
Drwn: cms
Date: Jul 1990

LOG OF TEST PIT S- 1

LOT 3 - MONTE BELLO SUBDIVISION

Ross, California

PLATE
2

Equipment: Trackhoe; 24" Bucket

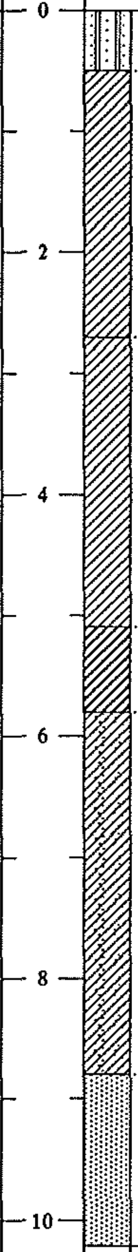
Logged By: J. Noble

Elevation: **

Start Date: 7-2-90

Finish Date: 7-2-90

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Depth (feet)



BROWN SILTY SAND (SM), medium dense, dry, slightly porous

BROWN SANDY CLAY (CL), stiff, moist, with occasional gravels

MOTTLED RED-BROWN SANDY CLAY (CL), slightly porous, sharp contact with upper unit, non planar, contact dipping to north (Colluvium)

BROWN-GRAY SILTY CLAY (CH), stiff, moist.

BROWN SANDY CLAY TO CLAYEY SAND (CL/SC), stiff/dense, slightly porous (Residual Soil)

YELLOW-BROWN SANDSTONE, extremely closely spaced fractures, weak to moderately strong, highly weathered, with occasional near vertical gray colored shears.

Bottom of test pit at 10.2 feet.
No free water encountered.

Herzog Associates
Geoscientists

Job No: 1385.02-01-3
Appr: Jr
Drwn: cms
Date: Jul 1990

LOG OF TEST PIT S- 2

LOT 3 - MONTE BELLO SUBDIVISION

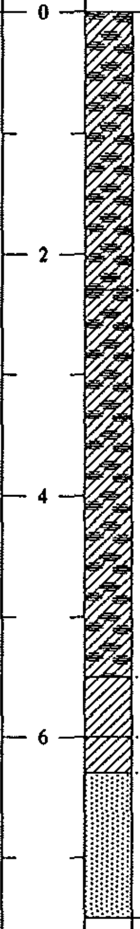
Ross, California

PLATE

3

Equipment: Trackhoe; 24" Bucket
Elevation: **
Logged By: J. Noble
Start Date: 7-2-90
Finish Date: 7-2-90

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Depth (feet)



RED-BROWN CLAYEY GRAVEL (GC), dense, moist, slightly porous

RED-BROWN SANDY CLAY TO CLAYEY GRAVEL (CL/GC), stiff/dense, moist

GRAY SANDY CLAY (CL), medium stiff, moist, with occasional planar features, no striation or slickensided present, upper contact dipping to NE 16 degrees (Residual)

RED-BROWN SANDY CLAY (CL), stiff, moist

YELLOW-BROWN SILTSTONE AND SANDSTONE, extremely closely spaced fractures, weak to moderately strong, highly weathered

Bottom of test pit at 7.5 feet.
No free water encountered.

Herzog Associates
Geoscientists

Job No: 1385.02-01-3
Appr: jr
Drwn: cms
Date: Jul 1990

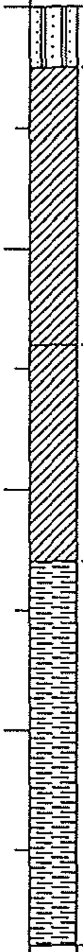
LOG OF TEST PIT S- 3

LOT 3 - MONTE BELLO SUBDIVISION

Ross, California

PLATE
4

Equipment: Trackhoe; 24" Bucket
Elevation: **
Logged By: J. Noble
Start Date: 7-2-90
Finish Date: 7-2-90



Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Depth (feet)	
			0	BROWN SILTY SAND (SM), medium dense, moist, slightly porous (Top Soil)
				GRAY-BROWN SANDY CLAY (CL), medium stiff to stiff, moist, non-continuous planar contact dipping to southeast, (highly weathered sheared shale).
			2	
				RED-BROWN SANDY CLAY (CL), stiff, moist, slightly porous, with occasional greenstone gravels
			4	
				RED-BROWN SILTSTONE, closely spaced fractures, weak to moderately strong, highly weathered
			6	
				Bottom of test pit at 7.8 feet. No free water encountered.

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Geoscientists

Job No: 1385.02-01-3
Appr: dr
Drwn: cms
Date: Jul 1990

LOG OF TEST PIT S- 4

LOT 3 - MONTE BELLO SUBDIVISION

Ross, California

PLATE
5

Equipment: Trackhoe; 24" Bucket
Elevation: **
Logged By: J. Noble
Start Date: 7-2-90
Finish Date: 7-2-90

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Depth (feet)	
			0	ORANGE-BROWN CLAYEY GRAVEL (GC), dense, moist (Colluvium)
			2	
			4	
				PURPLE TO BROWN CLAYEY GRAVEL (GC), very dense, moist, with abundant greenstone gravels (Residual soil or deeply weathered melange greenstone).
			6	
			8	
			10	
				Bottom of test pit at 10.3 feet. No free water encountered.

Herzog Associates
Geoscientists

Job No: 1385.02-01-3
Appr: dr
Drwn: cms
Date: Jul 1990

LOG OF TEST PIT S- 5

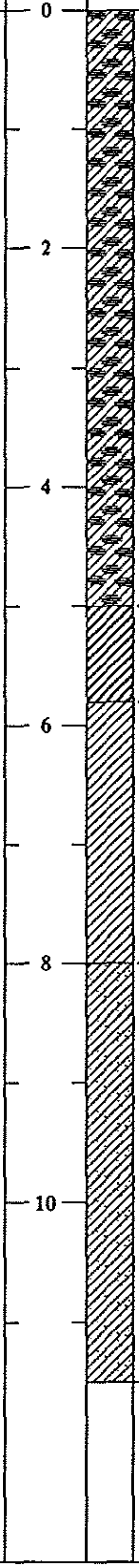
LOT 3 - MONTE BELLO SUBDIVISION

Ross, California

PLATE
6

Equipment: Trackhoe; 24" Bucket
Elevation: **
Logged By: J. Noble
Start Date: 7-2-90
Finish Date: 7-2-90

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Depth (feet)



BROWN TO RED-BROWN CLAYEY GRAVEL (GC), medium dense to dense, slightly porous

RED-BROWN SANDY CLAY (CH), stiff, moist, with occasional gravels of greenstone (Residual Soil)

RED-BROWN SANDY CLAY (CL), stiff, moist, with fragments of greenstone.

BROWN CLAYEY SAND (SC), very dense, moist, with fragments of greenstone.

Bottom of test pit at 11.5 feet.
No free water encountered.

Herzog Associates
Geoscientists

Job No: 1385.02-01-3
Appr:
Drwn: cms
Date: Jul 1990

LOG OF TEST PIT S- 6

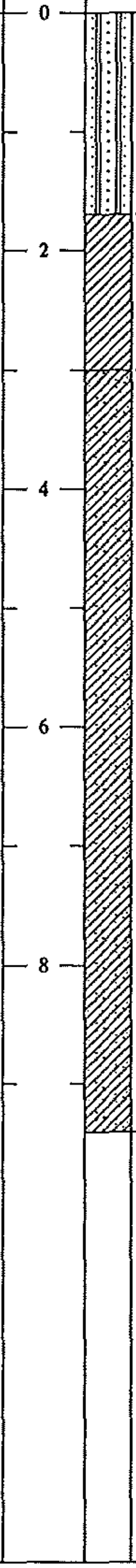
LOT 3 - MONTE BELLO SUBDIVISION

Ross, California

PLATE
7

Equipment: Trackhoe; 24" Bucket
Elevation: **
Logged By: J. Noble
Start Date: 7-2-90
Finish Date: 7-2-90

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Depth (feet)



BROWN SILTY SAND (SM), medium dense, moist, porous (Top Soil)

DARK RED-BROWN SANDY CLAY (CL), stiff to very stiff, moist, with fragments of greenstone (Residual Soil)

RED-BROWN CLAYEY SAND TO SANDY CLAY (SC/CL), dense/stiff, moist, with fragments of greenstone (deeply weathered greenstone).

Bottom of test pit at 9.4 feet.
No free water encountered.

Herzog Associates
Geoscientists

Job No: 1385.02-01-3
Appr:
Drwn: cms
Date: Jul 1990

LOG OF TEST PIT S- 7

LOT 3 - MONTE BELLO SUBDIVISION

Ross, California

PLATE
8

Equipment: Trackhoe; 24" Bucket

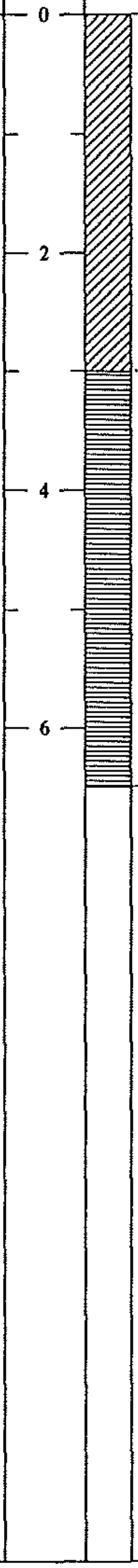
Logged By: J. Noble

Elevation: **

Start Date: 7-2-90

Finish Date: 7-2-90

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Depth (feet)



BROWN SANDY CLAY (CL), medium stiff, moist (Top Soil/Residual Soil)

BROWN-GRAY SHALE, extremely closely spaced fractures, weak, highly weathered

Bottom of test pit at 6.5 feet.
No free water encountered.

Herzog Associates
Geoscientists

Job No: 1385.02-01-3
Appr:
Drwn: cms
Date: Jul 1990

LOG OF TEST PIT S- 8

LOT 3 - MONTE BELLO SUBDIVISION

Ross, California

PLATE
9

Equipment: Trackhoe; 24" Bucket
Logged By: J. Noble
Elevation: **
Start Date: 7-2-90
Finish Date: 7-2-90

Other Laboratory Tests	Dry Density (pcf)	Moisture Content (%)	Depth (feet)	
			0	BROWN SANDY CLAY (CL), medium stiff, moist, slightly porous
				GRAY-PURPLE CLAYEY SAND TO GRAVELLY CLAY (SC/CL), dense/stiff, moist, with abundant gravels of greenstone
			2	
			4	
				sharp contact with respect to color; contact dips to northwest
				DARK GRAY SANDY CLAY (CL), stiff, moist, with occasional graywacke fragments (deeply weathered sheared shale/metagraywacke).
			6	
				Contact is irregular and dips to northwest.
				BROWN SANDSTONE, extremely closely spaced fractures, weak, highly weathered, with occasional metagraywacke blocks
				Bottom of test pit at 7.8 feet. No free water encountered.

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Job No: 1385.02-01-3
Appr:
Drwn: cms
Date: Jul 1990

LOG OF TEST PIT S- 9

LOT 3 - MONTE BELLO SUBDIVISION

Ross, California

PLATE
10

MAJOR DIVISIONS					TYPICAL NAMES
COARSE GRAINED SOILS More than Half > #200 sieve	GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	▤	VELL GRADED GRAVELS, GRAVEL-SAND MIXTURES
			GP	▤	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES
		GRAVELS WITH OVER 12% FINES	GM	▤	SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES
			GC	▨	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES
	SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	░	VELL GRADED SANDS, GRAVELLY SANDS
			SP	░	POORLY GRADED SANDS, GRAVELLY SANDS
		SANDS WITH OVER 12% FINES	SM	▥	SILTY SANDS, POOORLY GRADED SAND-SILT MIXTURES
			SC	▨	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
FINE GRAINED SOILS More than Half < #200 sieve	SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	▥	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS VITH SLIGHT PLASTICITY
			CL	▨	INORGANIC CLAYS OF LOV TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
			OL	▥	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOV PLASTICITY
	SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	▥	INORGANIC SILTS, MICACEOUS OR DIATOMACIOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
			CH	▨	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
			OH	░	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS			Pt	▩	PEAT AND OTHER HIGHLY ORGANIC SOILS

UNIFIED SOIL CLASSIFICATION SYSTEM

Shear Strength, psf

Confining Pressure, psf

Consol	Consolidation	Tx	2630 (240)	Unconsolidated Undrained Triaxial
LL	Liquid Limit (in %)	Tx sat	2100 (575)	Unconsolidated Undrained Triaxial, saturated prior to test
PL	Plastic Limit (in %)	DS	3740 (960)	Consolidated Drained Direct Shear
PI	Plasticity Index	FVS	1320	Field Vane Shear
Gs	Specific Gravity	UC	4200	Unconfined Compression
SA	Sieve Analysis	LVS	500	Laboratory Vane Shear
■	Undisturbed Sample	SS	Shrink Swell	
☒	Bulk or Disturbed Sample	EI	Expansion Index	
▒	Standard Penetration Test	P	Permeability	
☐	Sample Attempt with No Recovery	SE	Sand Equivalent	

KEY TO TEST DATA

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Appr:
Drwn: cms
Date: Jun 1990

SOIL CLASSIFICATION CHART AND KEY TO TEST DATA
LOT 3 - MONTE BELLO SUBDIVISION
Ross, California

PLATE
11

ROCK SYMBOLS

▤ SHALE OR CLAYSTONE

▤ SILTSTONE

░ SANDSTONE

▦ CONGLOMERATE

░ CHERT

▤ PYROCLASTIC

▒ VOLCANIC

▩ PLUTONIC

▩ SERPENTINITE

▩ METAMORPHIC ROCKS

░ ALTERED ROCKS

▨ SHEARED ROCKS

LAYERING

MASSIVE	Greater than 6 feet
THICKLY BEDDED	2 to 6 feet
MEDIUM BEDDED	8 to 24 inches
THINNLY BEDDED	2-1/2 to 8 inches
VERY THINNLY BEDDED	3/4 to 2-1/2 inches
CLOSELY LAMINATED	1/4 to 3/4 inches
VERY CLOSELY LAMINATED	Less than 1/4 inch

JOINT, FRACTURE, OR SHEAR SPACING

VERY WIDELY SPACED	Greater than 6 feet
WIDELY SPACED	2 to 6 feet
MODERATELY SPACED	8 to 24 inches
CLOSELY SPACED	2-1/2 to 8 inches
VERY CLOSELY SPACED	3/4 to 2-1/2 inches
EXTREMELY CLOSELY SPACED	Less than 3/4 inch

HARDNESS

SOFT - Pliable; can be dug by hand

FIRM - Can be gouged deeply or carved with a pocket knife

MODERATELY HARD - Can be readily scrached by a knife blade; scratch leaves heavy trace of dust and is readily visable after the powder has been blown away

HARD - Can be scratched with difficulty; scratch produces little powder and is often faintly visable

VERY HARD - Cannot be scratched with pocket knife; leaves a metallic streak

STRENGTH

PLASTIC - Capable of being molded be hand

FRIABLE - Crumbles by rubbing with fingers

WEAK - An unfractured specimen of such material will crumble under light hammer blows

MODERATELY STRONG - Specimen will withstand a few heavy hammer blows before breaking

STRONG - Specimem will withstand a few heavy ringing hammer blows and usually yields large fragments

VERY STRONG - Rock will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments

DEGREE OF WEATHERING

HIGHLY WEATHERED - Abundant fractures coated with oxides, carbonates, sulphates, mud, etc., thourough discoloration, rock disintegration, mineral decomposition

MODERATELY WEATHERED - Some fracture coating, moderate or localized discoloration, little to no effect on cementation, slight mineral decomposition

SLIGHTLY WEATHERED - A few stained fractures, slight discoloration, little or no effect on cementation, no mineral decomposition

FRESH - Unaffected by weathering agents, no appreciable change with depth

10☐
10

Herzog Associates
Geoscientists

Job No: 1385.02-01-3
Appr:
Drwn: cms
Date: Jun 1990

Engineering Geology Rock Terms & Test Pit Explanation
LOT 3 - MONTE BELLO SUBDIVISION
Ross, California

PLATE
12

DISTRIBUTION LIST

Hoffman & Albritton
Attention: Roy Hoffman
24 Commercial Boulevard
Suite B
Novato, California 94949

Ross Town Council
Town of Ross
P. O. Box 320
Ross, California 94957

Neuman Planning Associates
Attention: Lisa Neuman
2201 Mulberry Terrace
San Rafael, California 94903

Mr. Skip Berg
10 Captains Landing
Tiburon, California 94920

Miller Pacific
Attention: Gene Miller
165 North Redwood Drive
Suite 120
San Rafael, California 94903

GEOTECHNICAL INVESTIGATION PROPOSED PROPERTY SUBDIVISION UPPER ROAD ROSS, CALIFORNIA

STUBER-STROEH
ASSOCIATES
OCT 25 1989
RECEIVED

Prepared for:

Mr. Robert Toigo
Institute for Fiduciary Education
1112 "I" Street, Suite 210
Sacramento, California 95814

Prepared by:

HERZOG ASSOCIATES, INC.
275 Miller Avenue
Mill Valley, California 94941
(415) 383-7740

Job Number 1385.2-0-1

Donn A Ristau, Ph.D.
Engineering Geologist - 1155

Donald Herzog,
Geotechnical Engineer - 395

No. 000395
Exp. 6/30/93

12 October 1989

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HERZOG

TABLE OF CONTENTS

EXECUTIVE SUMMARY

This summarizes the results of our geotechnical investigation performed for the proposed 18½ acre subdivision off Upper Road in Ross, California. The subdivision layout and improvement plans are shown on Plate 1. In developing our evaluation of the project, we have incorporated our previous work (1982) within the site as well as performing additional subsurface investigation.

This report is intended to satisfy the requirements of the Town of Ross Hillside Lot/Hazard Zone Application; Section A(5); Items a - f. The geologic conditions discussing these various items are included in our October 7, 1982 (which is included herein as an Appendix) and this current report.

The main geotechnical considerations we have identified within the property related to the Tentative Map submittal include:

1. The presence of old dormant landslide deposits and weak soils within several areas where roadways are proposed, and the potential that reactivation of slides could impact roadway improvements.
2. The presence of slide deposits where reconstruction of the slope appears necessary, within or adjacent to two building envelopes.
3. The presence of creeping soils on natural slopes steeper than 5:1 in areas where improvements are planned.
4. The presence of expansive soils throughout various portions of the site. Expansive soils may be encountered during grading and the shrinking and swelling of these soils may disrupt slabs, foundations, and roadways unless mitigated.

Based on our 1989 reconnaissance mapping, we did not observe any major surficial changes in the slope stability conditions from those identified in our 1982 no active slides

report, and we did not observe any active slide conditions that would indicate that new failures have occurred within proposed building areas or the roadway alignment since 1982. This condition represents a positive condition with respect to the proposed construction, considering that Marin County experienced numerous areas of slope failure between 1982 and 1986.

The road alignment that has been proposed is situated to minimize crossing slide areas and to minimize creek crossings. As such, this alignment and the building envelope layout represent a configuration that would reduce the amount of grading and ground disturbance associated with slide mitigation and slope reconstruction. The proposed alignment utilizes only one major creek crossing, and this crossing is within an area where slope stability problems appear to be minimal. A bridged crossing is proposed for this area, and thus the potential risks associated with flooding and potential for impact from debris slides have been substantially reduced.

Likewise, building envelopes have been proposed for areas where the soil/rock conditions appear relatively shallow and stable. In an attempt to avoid ground disturbance, tree removal, and massive grading, the building envelopes have been proposed mainly for the central portions of the property, away from ravines and outside massive slide areas. Slope reconstruction work would apparently only be necessary within the southern portion of Lot 2 and within the northeast portion of Lot 4.

All of the proposed building envelopes appear to be within areas that can be developed utilizing standard hillside construction methods. Areas where slope reconstruction would be required may be constructed with conventional engineered fill buttresses. Based on the configuration of the property, we anticipated that slope reconstruction work could be completed using onsite materials and off haul or import of fill will probably not be required.

Detailed geotechnical analysis and design recommendations for the general subdivision improvements are included in the following sections of this report and the attached Appendix.

INTRODUCTION

This report presents the results of our geotechnical investigation for a proposed subdivision of an 18½ acre parcel located west of Upper Road in Ross, California. The property subdivision and proposed building envelopes are shown on the Tentative Parcel Map, dated October, 1989 by Stuber-Stroeh Associates. As shown on Plate 1, the property is to be split into six lots. Single family residences presumably will be proposed for construction within each of the lots.

We previously performed a geotechnical evaluation of the property, and presented the results of that evaluation in our report dated October 7, 1982. For that evaluation, 21 test pits were excavated throughout the site to explore the general subsurface conditions along a possible access road alignment and within the proposed lots. Subsequently, a portion of the property was sold and the various lot configurations, building envelopes, and access road alignment were modified.

The purpose of our current investigation was to evaluate the geotechnical considerations that could affect the lot split and to assess the suitability of each of the potential building sites.

Our scope of work is outlined in our confirming agreement dated July 10, 1989, and included a detailed reconnaissance by our Certified Engineering Geologist (CEG) and Geotechnical Engineer, exploration of the subsurface conditions within areas of general subdivision improvements, and the preparation of this report.

Our report provides geotechnical recommendations regarding construction of the access roadways and general subdivision improvements and includes the following geotechnical information:

1. A description of the soil and geologic conditions observed.
2. A discussion of the potential geologic hazards and recommended mitigation measures.
3. Site grading recommendations for general subdivision improvements.
4. Retaining wall design criteria for access roadways.
5. Preliminary design criteria for pavement thickness.
6. Soil engineering drainage recommendations.
7. Recommended supplemental services.

Our scope of work did not include evaluation of any potential hazardous waste contamination of the soil or groundwater at the site. As indicated, we are providing a discussion regarding the suitability of the proposed lots and construction of the access roadways. It is intended that additional investigations would be performed for grading and foundation recommendations for the proposed residences when specific designs have been formulated.

This report has been prepared in accordance with generally accepted geotechnical engineering principles and practices for the specific use of

Mr. Robert Toigo and his representatives as an aid in the design of the proposed subdivision. No other warranty, either express or implied, is given.

WORK PERFORMED

Prior to our investigation, we reviewed stereo-paired aerial photographs, selected geotechnical references, and our previous report relating to the local geologic and slope stability conditions within the vicinity of the site. Our 1982 report has been included as an Appendix to this report. A complete listing of the reference material reviewed is presented at the end of this report.

On July 18, 1989, our CEG performed a detailed geologic reconnaissance and mapping of the area. Surficial lithologic features and geomorphic terrain suggestive of active or potential slope instability were mapped on a topographic map of the site. The geologic mapping is shown on Plate 1. At that time, locations for the subsurface investigations were established.

On July 25, 1989, we explored the subsurface conditions to the extent of 17 test pits within the proposed road alignments and the building envelopes not previously investigated. The pits were excavated with a track-mounted backhoe and ranged in depth from 5 to 17 feet.

Our CEG determined the test pit locations based upon the proposed road alignments, lot layout, and in areas where topographic conditions suggested possible slope instability. The approximate locations of the test pits are shown on Plate 1. The locations of the recently excavated pits were plotted by the surveyors who prepared the topographic map. The locations of the pits excavated in 1982 were transposed onto Plate 1 from our earlier work.

Our staff geologist and CEG observed the excavation of the test pits, logged the conditions encountered, and obtained representative soil and rock samples for visual examination and classification. Logs of the recently excavated pits are presented on Plates 2 through 10. The soil and rock materials are described in accordance with the criteria presented on Plates 11 and 12.

The test pit logs show our interpretation of the subsurface conditions on the dates and at the locations indicated. It is not warranted that they are representative of the subsurface conditions at other locations or at other times. The breaks between various soil types and rock lithologies depicted on the logs represent approximate boundaries; the actual transitions may be gradual or uneven.

The test pits were backfilled upon completion of our field investigation. The backfill material was compacted by tamping with the backhoe bucket. As such, the backfill will be subject to settlement. Detailed recommendation relating to construction within areas of test pit excavation are presented in the grading section of this report.

On September 21, 1989, our CEG met with Mr. William Schenck of Stuber-Stroeh Associates to discuss the proposed road alignment and several of the building locations. The Tentative Map reflects those discussions and incorporates our recommendations as to the feasible locations of improvements.

The road alignment that has been proposed is situated to minimize crossing slide areas and to minimize creek crossings. As such, this alignment and the building envelope layout represent a configuration that would reduce the amount of grading and ground disturbance associated with slide mitigation and slope reconstruction. The proposed alignment utilizes only one major creek crossing, and this crossing is within an area where slope stability problems appear to be minimal. A bridged crossing is proposed for this area, and thus the potential risks associated with flooding and potential for impact from debris slides have been substantially reduced.

Likewise, building envelopes have been proposed for areas where the soil/rock conditions appear relatively shallow and stable. In an attempt to avoid ground disturbance, tree removal, and massive grading, the building envelopes have been proposed mainly for the central portions of the property, away from ravines and outside massive slide areas. Slope reconstruction work would apparently only be necessary within the southern portion of Lot 2 and within the northeast portion of Lot 4.

SITE CONDITIONS

Project Description

Grading for the general subdivision improvements will be performed for the access roadway and for slope reconstruction work within portions of Lots 2 and 4. Grading for building pads within the other lots is not proposed.

Access to the site will extend off Upper Road, along an existing paved driveway. Widening of the driveway to 22 feet is planned.

The construction of the access roadway will require the use of both cuts and fills. In order to reduce the effects of ground disturbance and tree removal, retaining walls are proposed for various segments of the inboard (upslope) side of the roadway. In areas where walls are proposed, the slopes above the walls are proposed to be cut at a gradient of 2:1. The Tentative Map by Stuber-Stroeh Associates depicts the proposed grading and extent of cuts, fills, and retaining walls. The site conditions for the property have been discussed in detail in our 1982 report, which has been included as an Appendix to this report. For a discussion of those conditions, please review the Appendix Section.

Based on our 1989 reconnaissance mapping, we did not observe any major surficial changes in the slope stability conditions from those identified in our 1982 report. As is a result of performing additional test pit exploration, the limits of some slides have been modified from our 1982 report. We did not observe any active slide conditions that would indicate that new failures have occurred within proposed building areas or the roadway alignment since 1982.

CONCLUSIONS

Based upon the results of our investigation, we judge that from a geotechnical standpoint, the proposed access roadway and building envelopes are suitable for development provided that the recommendations presented in this report are incorporated into the design and construction of the project.

The main geotechnical considerations we have identified within the property related to the Tentative Map submittal include:

1. The presence of old dormant landslide deposits and weak soils within several areas where roadways are proposed, and the potential that reactivation of slides could impact roadway improvements.
2. The presence of slide deposits where reconstruction of the slope appears necessary, within or adjacent to two building envelopes.
3. The presence of creeping soils on natural slopes steeper than 5:1 in areas where improvements are planned. Improvements must be designed for creep forces.
4. The presence of expansive soils throughout various portions of the site. Expansive soils may be encountered during grading and the shrinking and swelling of these soils may disrupt slabs, foundations, and roadways unless mitigated. The effects of expansive soils can be mitigated by avoiding slab-on-grade construction and providing for well drained non-expansive roadway sections.

In order to mitigate the risk of potential disturbance where the access roadway crosses areas of existing slide deposits, portions of the roadway must be constructed as a compacted fill buttress in accordance with the recommendations presented in the following section of this report. The areas where these conditions exist are adjacent to the southern end of the bridge crossing of the

ROAD PROB'S

main creek, and where the driveway to Lot 6 crosses the broad drainage ravine.

South of the bridge crossing, the roadway is shown to consist of a cut and fill condition. However, because of the relatively deep and potentially unstable soils, the roadway in this area (including the cut slope) should be constructed as an engineered fill. The reconstruction of the roadway within this area also should extend upslope and onto the northeast portion of the building envelope of Lot 4. Dormant slide deposits that extend into the building envelope of Lot 2 will also require reconstruction as a compacted fill buttress.

The driveway alignment leading to Lot 6 will cross an area of expansive colluvial soils and old slide debris. These soils do not appear to be suitable for the construction of roadways or structural improvements. In areas where the proposed access road is to cross these areas, cuts and fills will need to be stabilized and constructed with an engineered fill buttress.

In areas where improvements are not planned, the risk of future instability related to creep, erosion, or possible reactivation of slides does not appear to pose a risk to the proposed building envelopes. Massive grading and reconstruction of these slides does not appear warranted.

New fills should be stable when keyed into suitable bedrock materials, drained, and compacted. The construction of stable fills will require the excavation of keyway into rock or approved very stiff or dense soils, placement of subdrains (as necessary), and reconstruction of slope with compacted materials.

The main consideration with respect to the construction of engineered fills is the depth of keying required to mitigate potential creep or fill failure. Although fills of only a few feet may be proposed, in some instances the presence of underlying weak material may require deep keyways.

All of the proposed building envelopes appear to be within areas that can be developed utilizing standard hillside construction methods. As is typical for most Marin County hillsides, it will be necessary to extend foundations into rock, and to design the foundations to resist forces imposed by creeping of the soils above the rock. The depth of the creep zone will vary significantly from lot to lot and will have to be defined.

All hillside areas, and particularly known slide areas, have the potential to be subject to future sliding under certain hydrologic and/or seismic conditions. The proposed building envelopes for Lots 1, 3, 5, and 6 and do not appear to be within or adjacent to any slide areas that would pose a risk to those envelopes. Lots 2 and 4 are adjacent to slides that could encroach into the building envelope and reconstruction of portions of these lots is recommended.

As shown, the access driveways to the various lots generally do not appear to cross slides or areas of steep terrain that may be potentially unstable. Major grading and/or slope reconstruction is not expected for these areas. However, some of the lots contain slide deposits that are relatively active (pre-1982) and future slope failures may tend to occur within and/or across property lines.

Building envelopes and roadway improvements have been laid out in a manner to reduce the risk of impact from potential slides. However, it is imperative that all future property owners be made aware of the presence of, and potential consequences associated with slides within the various lots, even though the slides may not impact the building site. When construction is proposed within each lot, the soils report prepared for the development should include an evaluation of the presence of slides within the property and their relationships to the proposed construction.

The firm bedrock on the site is suitable to support foundations or fills. However, most of the soils within the site are weak and compressible when wet, experience slow downhill creep, and are unsuitable for support of foundations and fills. On slopes steeper than 5:1 it will be necessary to key fills into firm bedrock and to extend foundations into bedrock. It will be necessary to design foundations to resist the lateral forces caused by soil creep. We anticipate that spread footing foundations may be suitable if grading is performed to expose bedrock. However, in sloping areas where deep soils are present, we judge that drilled, cast-in-place, reinforced concrete piers extending well into underlying bedrock will probably be the most suitable foundation system. Detailed design recommendations for the construction of residences are not included in this report.

Detailed geotechnical recommendations for design of fill buttresses and associated roadway grading are presented in the following sections of this report.

All conclusions and recommendations presented in this report are contingent upon Herzog Associates being retained to: 1) review the soil and engineering aspect of the final grading and retaining wall foundation plans prior to construction; and 2) observe construction of the project as outlined under the Supplemental Services section of this report.

RECOMMENDATIONS

Grading

Areas to be graded should be cleared of vegetation, debris, and any other materials that would pose a difficulty to construction. These areas should be stripped of the upper few inches containing organic matter. Large roots and other organic debris should be separated and removed from the site. Organic soils may be stockpiled for later use to aid in revegetation of the fill buttress.

Prior to the placement of fill, our Field Technician should collect representative samples of the soil/rock materials that will be used as fill. The samples should be laboratory tested, and compaction curves established to determine their maximum dry densities. The maximum dry densities may then be compared to the in place field densities obtained during construction testing to evaluate the relative compaction of the fill.

We anticipate that with the exception of organic matter and rocks or lumps larger than 8 inches in diameter, the excavated material would be suitable for re-use as compacted fill. Because the existing weak, low density soils are being replaced with higher density fill, completion of the fill buttress may require importing some fill material. If imported fill is required, it should be non-expansive materials with a plasticity index of 15 or less. The imported fill should also be free of organic matter and materials larger than 8 inches in diameter.

A keyway should be excavated along the toe of the proposed fill slopes. Where roads cross slides and above grade fills are not planned, the keyway should be excavated at least 8 feet downslope and upslope from the edges of the road. The keyways should be at least 12 feet wide and should extend at least 2 feet into rock along the outboard (downslope) edge of the excavation. In areas flatter than 5:1, and where suitable soils are present, the fills may be keyed into stiff soils at least 3 feet below existing ground level. The base of the keyways should be sloped to the rear and sloped to drain to an outlet by gravity. The actual depth and extent of the keyways should be recommended in the field during construction by our Engineering Geologist.

A chimney subdrain should be installed along the rear of keyways. The drains should consist of a 2-inch basal layer of drain rock (3/4 to 1.5 inch diameter) or other approved permeable free draining material, upon which a 6-inch diameter perforated heavy-walled plastic pipe is bedded. The pipe should have a SDR of 23.5 or better. The pipe should be covered by a 1.5 foot wide (minimum) chimney of drain rock that extends at least 5 to 6 feet up the rear wall of the keyway excavation. If clean drain rock is used, the drain rock should be separated from contact with the rear wall of the excavation with a layer of geotextile filter cloth (Mirafi 140N or equivalent). If Class II permeable material is used, the filter cloth may be eliminated. A cleanout riser should be provided for the subdrain. The perforated pipe should outlet into a solid line that discharges into an erosion resistant downslope from the toe of the fill buttress.

After the subdrain has been installed, fill material should be spread in 8-inch thick loose lifts, moisture conditioned as necessary, and compacted to at least 90 percent relative compaction, as determined by the ASTM D-1557-70(C) laboratory compaction test procedure. Where roads are constructed of compacted fill, the upper 2 feet of the subgrade should be compacted to 95 percent. As the fill continues upslope, it should be continually benched into rock.

Subdrains should be installed every 15 vertical feet on intermediate benches, where evidence of seepage is observed, or as recommended by the Geotechnical Engineer in the field during construction. A chimney subdrain should be installed on the last (uppermost) major bench of the buttress. Intermediate drains should be constructed in a manner similar to that of the keyway drain.

Cutslopes in the dense/stiff soils and weak and/or intensely fractured bedrock should be inclined at a 2:1 slope. Cuts that expose very weak soils or slide debris should be constructed as a compacted fill buttress. Cuts within the strong, moderately fractured bedrock may be inclined at 1.5:1 if minor sloughing of the rock face is acceptable. In areas where sloughing or maintenance of the cut is not desirable, then a small (3-foot high) slough wall may be constructed, the slope inclined at a flatter gradient, or a retaining wall constructed. For planning purposes, all cutslopes should be planned to be inclined at 2:1. During construction, cutslope inclinations may be modified in the field by the Engineering Geologist, if conditions warrant.

Based on our experience with similar soil and rock types, we judge that the excavations can be performed without blasting.

Conventional fill slopes should not be constructed steeper than 2:1. Fills do not appear to be proposed within areas where the existing slope gradients are steeper than 2:1. However, if the need arises and the construction of typical 2:1 fill slopes would not catch, the use of reinforced fills and/or retaining walls may be used to develop the fills. Reinforced fills that utilize welded wire or plastic mesh may be constructed at slopes of up to 1:1 in accordance with the manufacturer's recommendations.

Compacted fill typically settles about one percent of the total thickness, and this settlement may produce asphalt cracking where compacted fill transitions to stiff native soils or rock. For this reason, we recommend that abrupt transitions between cuts and deep fills be avoided.

The shrinking and swelling of expansive soil will cause pavements to experience edge cracking. Therefore, the roadway subgrade should be evaluated for the presence of expansive soil/rock areas by our Geotechnical Engineer. If expansive materials are encountered, they should be over-excavated to a depth of two (2) feet and replaced with non-expansive fill. If edge cracks do occur, they should be sealed. Concrete paving should be reinforced to reduce cracking and should be provided with frequent joints to control cracking. If possible, during the construction of roadway fills, non-expansive soil/rock should be separated and selectively used for the upper 2 feet of roadway subgrade.

The test pits were backfilled upon completion by tamping with the bucket and track-walking the area. As such, settlement of test pit backfill should be expected. If structural or landscaping improvements are to be located in areas of test pits, we recommend that the improvements be designed to mitigate the risk of settlement, or that the backfill be excavated and replaced with fill compacted to 90% of its maximum dry density as determined by the ASTM D-1557-70(c) laboratory compaction test procedure.

Finished fill slopes should not be steeper than 2:1. The finished fill slope should be hydroseeded or planted with some other fast growing ground cover to reduce the erosion potential of the buttress. The stripped organic material may also be spread on the finished slope to facilitate re-vegetation of the area. For extensive areas of graded slopes with bare soil, gully erosion and rilling may occur prior to the growth of the vegetative cover. In this instance, additional erosion control measures may be required. These may involve the use of hay bales or silt fences to retard sediment transport and the use of small rip-rap to control and reduce erosion in gullies that may develop.

Because of the presence of the subdrains beneath the compacted fill, we recommend having the drain line locations surveyed and staked prior to excavating cut areas or drilling piers for walls. The construction should be planned and excavated in a manner that would not disrupt the drains, or a provision made that would restore the drains after the excavation.

Pavements

For planning purposes, we recommend that the following, based on an assumed R-value of 18 and an assumed traffic index value of pavement sections be used:

Location	Asphalt Concrete	Class II Aggregate Base[1]
Assess Road	3.0 in.	8.0 in.
Driveways	2.5 in.	6.0 in.

If a heavy volume of truck traffic for residential construction is anticipated, the above values should be increased. The actual R-value of subgrade soils should be established after rough grading, and the pavement design modified as necessary.

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[1] Aggregate base and subbase materials shall conform to the requirements specified in Section 25 of the CalTrans Standard Specifications, State of California Department of Transportation, published July 1984).

Pavement thicknesses should be computed using Method 301-F of the CalTrans Pavement Design Manual and should be based on a pavement life of 20 years.

After utility trenches are properly backfilled, compacted, and tested, pavement subgrade should be prepared by scarifying to a depth of at least 6 inches, moisture-conditioning to wet of optimum, and compacting to at least 95 percent relative compaction. Finished subgrade should be smooth and nonyielding. Aggregate base material should then be spread, moisture-conditioned as necessary, and compacted to at least 95 percent relative compaction. The aggregate base material should also be smooth and non-yielding.

These recommendations are intended to provide support for auto and light truck traffic only. These recommendations are not intended to provide for heavy construction equipment or concentrated storage loads such as parked truck-trailers, or for concentrated wheel loads such as forklifts or self-loading garbage trucks.

In areas where concentrated storage and/or wheel loads are anticipated, the slabs and pavements should be designed to support these loads. Support could be provided by increasing pavement sections or by providing reinforced concrete slabs. Loading areas for self loading garbage trucks should be provided with reinforced concrete slabs at least 6 inches thick, and reinforced with #4 bars at 12 inch centers each way.

We recommend that the pavements be constructed during the dry season to avoid the saturation of the subgrade and base materials which often occurs during the wet winter months. Our experience indicates that pavements constructed during the dry season generally have a longer service life and require less maintenance than those constructed during the wet season.

If pavements are constructed during the winter, a cost increase relative to drier weather construction should be anticipated. Unstable areas should be overexcavated to remove soft soils. The excavations will probably require backfilling with imported crushed rock. The soils engineer should be consulted for recommendations at the time of construction if this condition is encountered.

Where pavements will abut landscaped areas, the pavement baserock layer and subgrade soils should be protected against saturation from irrigation and rainwater by means of a concrete curb and gutter, redwood header-board, a subdrain, or a thickened asphalt concrete section. The curb and gutter, header-board, subdrain, or thickened asphalt should extend to a depth of at least 6 inches below the bottom of the baserock layer.

Retaining Walls

Retaining walls constructed at the site must be designed to resist lateral earth pressures plus additional lateral pressures that may be caused by surcharge loads applied at the ground surface behind the walls. Retaining walls supporting level backfill should be designed to resist an equivalent fluid pressure of 45 pcf acting in a triangular pressure distribution. Where the backfill slopes up steeper than 3:1, the walls should be designed for an equivalent fluid pressure of 60 pcf. Retaining walls restrained from movement at the top should be designed for equivalent fluid pressures of 60 pcf and 80 pcf for level backfill and backfill steeper than 3:1, respectively. Where an imaginary 1½:1 line projected down from foundations intersects retaining walls, the portions of the retaining walls below the intersection should be designed for an additional horizontal surcharge load. Where retaining wall backfill is subject to vehicular traffic, the walls should be designed to resist an additional surcharge pressure equivalent to 2 feet of additional backfill.

Retaining walls should be supported on drilled piers or spread footings, as applicable, designed in accordance with the recommendations presented in the following section of this report. A minimum factor of safety of 1.5 against overturning and sliding should be used in the design of retaining walls.

Retaining walls should be fully backdrained. The backdrains should consist of 4-inch diameter, rigid perforated pipe embedded in drain rock. The pipe should be PVC Schedule 80 or ABS with an SDR of 35 or better, and the pipe should be sloped to drain to outlets by gravity. Drain rock should consist of clean, free-draining crushed rock or gravel. The rock should be wrapped in filter fabric such as Mirafi 140N or equivalent. The top of the pipe should be at least 8 inches below lowest adjacent grade. The crushed rock or gravel should extend to within 1 foot of the surface. The upper 1 foot should be backfilled with compacted soil to exclude surface water. The ground surface behind retaining walls should be sloped to drain.

Where migration of moisture through retaining walls would be detrimental, retaining walls should be waterproofed. Retaining walls will yield slightly during backfilling. Therefore, walls should be backfilled prior to building on or adjacent to the walls.

Spread Footing Foundations

Conventional continuous and isolated spread footing foundations may be used wherever level excavations expose strong bedrock. Spread footings should be at least 12 inches wide and should extend at least 12 inches into undisturbed rock. The footings should be stepped as necessary to produce level tops and bottoms. Footings should be deepened as necessary to provide at least 7 feet of horizontal confinement between the footing bottoms and the face of the nearest slope.

Footings installed in accordance with these recommendations may be designed using allowable bearing pressures of 2000, 3000, and 4000 pounds per square foot (psf), for dead loads, dead plus code live loads, and total loads (including wind and seismic), respectively.

The portion of spread footing foundations extending into rock and at least 7 horizontal feet from the face of the nearest slope may impose a passive equivalent fluid pressure and a friction factor of 350 pcf and 0.40 respectively, to resist sliding.

Drilled Piers

We recommend that drilled, cast-in-place reinforced concrete piers be used to support retaining walls wherever level cuts do not extend through the soil and expose rock. The piers should be designed by the project structural engineer. However, all piers should be reinforced with at least four No. 5 bars and should be tied together with grade beams. The grade beams should be designed to span between the piers in accordance with structural requirements. The portion of the piers extending into undisturbed rock impose an allowable skin friction of 800 pounds per square foot (psf). The portion of the piers in compacted fill or dense/stiff soil beneath the colluvium may impose an allowable skin friction of 600 psf. End bearing should be neglected because of the difficulty of cleaning out small diameter pier holes, and the uncertainty of mobilizing end bearing and skin friction simultaneously.

Lateral loads on piers will be resisted by passive pressure in the fill and rock. An equivalent fluid pressure of 350 pcf for rock and 250 pcf for compacted fill or stiff soil, acting on two pier diameters, should be used. The stability of the system should be calculated using a minimum factor of safety of 1.5. Confinement for passive pressure may be assumed from 2 feet below the roadway surface if rock is not exposed as a result of the cutting. Where rock is exposed, the confinement for passive pressure may begin at the roadway grade.

If groundwater is encountered, it may be necessary to dewater the holes and/or place the concrete by the tremie method. If caving soils are encountered, it may be necessary to case the holes. Hard drilling may be required to achieve the required penetration.

Because of the potential that retaining walls could be used in areas of compacted fill, we recommend having the subdrain line locations surveyed and staked prior to pier drilling. Drilled piers should be located so that they do not encroach within 5 feet of the surveyed line. If drainrock and subdrain lines are encountered during pier drilling, the wall design and layout may need to be modified.

Utility Lines

All utility lines, including power, water, sewer, and gas must be moderately flexible to accommodate potential differential settlement between areas of compacted fill and native soils or rock. Where utilities are located in creeping soils, it will be necessary to provide flexible joints to accommodate creep movement. Lines that extend through engineered fills should not be subject to significant creep, and these fills are considered as being suitable for utility line construction. If utilities extend through unrepaired slide areas, it will be necessary to extend the utilities into firm rock beneath the potential zone of movement.

General Foundation Recommendations

We anticipate that buildings constructed on level areas excavated into rock can be supported on continuous and interconnected spread footings. Level pads of properly compacted, non-expansive engineered fill of uniform thickness on slopes flatter than 5:1 may also provide adequate support for spread footings for residential structures. Spread footings should not be used to span areas between rock and compacted fill or native soils.

On slopes steeper than 5:1, or where there is a potential for differential settlement because of variable fill/soil/rock conditions across the building area, drilled pier and grade beam foundations should be used. Detailed foundation design recommendations will have to be evaluated on a lot-by-lot basis at the time development of structures is proposed.

Erosion Protection

Construction and grading will expose areas of weak soil/rock which may be sensitive to erosion and/or sloughing. Erosion protection measures should be utilized during and after construction to reduce the risk of induced instability. Erosion protection measures include the use of seeding or hydromulch and the installation of hay bales and/or silt fences to retard sedimentation. Detailed erosion protection recommendations should be developed when grading plans are finalized and should be implemented immediately after construction has been performed.

Soil Engineering Drainage

Surface water should be diverted away from slopes either by grading or with the use of lined ditches. Ditches should be provided behind the tops of all retaining walls. All subdrain outlets and retaining wall backdrain outlets should discharge into either erosion resistant rip-rap areas within the creeks or swales, or into lined ditches that tie into the storm drain system. Surface drains and subdrains (or retaining wall backdrains) should be constructed separately. Surface water should not discharge into subsurface drainage improvements.

Maintenance

Periodic land maintenance will be required. Surface and subsurface drainage facilities should be checked frequently and cleaned and maintained as necessary. A dense growth of deep-rooted ground cover must be maintained on all slopes to reduce sloughing and erosion. Sloughing and erosion that occurs must be repaired promptly before it can enlarge into sliding.

SUPPLEMENTAL SERVICES

We should review the final plans for conformance with the intent of our recommendations. During construction, we should observe the conditions encountered in construction excavations and modify our recommendations, if warranted.

We should observe and approve overexcavations, keyways, and subdrain installations. We should provide intermittent soil engineering observation and density testing during fill placement and compaction operations. Our Engineering Geologist should be consulted regarding the extent of grading and trimming of cuts especially above retaining walls. We should approve subgrade and baserock compaction prior to application of asphaltic concrete paving. We should observe keyway and footing excavations, and pier drilling operations for the retaining walls to determine the actual depths required.

Our services during construction are limited to observation of soil and bedrock conditions, depth of excavation or drilling, and the condition of excavations or pier holes prior to concrete placement. Our services do not include observation or approval of steel, concrete, or asphalt; nor do they include establishing or verifying construction lines and grades. This should be performed by the appropriate party. Upon completion of the project, we should perform a final observation and summarize the results of this work in a final report.

These supplemental services are performed on an as-requested basis, and we cannot accept responsibility for items that we are not notified to observe. These supplemental services are in addition to this soil investigation, and are charged for on an hourly basis in accordance with our Schedule of Charges.

A qualified Geotechnical Engineer should be consulted as project plans are developed for the construction of the residences and other site specific improvements. When building locations and designs are finalized, a geotechnical consultant should perform a detailed site specific investigation including, as necessary, subsurface exploration, sampling, laboratory testing, and engineering analysis to develop conclusions and recommendations regarding:

1. Soil, rock, and groundwater conditions.
2. Foundation and retaining wall design criteria.
3. Site grading, including cut and fill slope recommendations.
4. Support for slab-on-grade, as appropriate.
5. Pavement design.
6. Soil engineering drainage control.
7. Supplemental services.

LIMITATIONS

This report is prepared for the specific use of Robert Toigo and his representatives for construction of the proposed subdivision improvements described in this report. Our services consist of professional opinions, conclusions developed by a consulting Geotechnical Engineer and Engineering Geologist in accordance with generally accepted principles and practices. This warranty is in lieu of all other warranties, either express or implied.

Our scope of work did not include evaluation of potential hazardous material contamination of soil or groundwater. Should the need arise, we can provide a separate proposal to perform such studies upon request.

We judge that construction in accordance with these recommendations will be stable, and that the risk of future instability is within the range generally associated with construction on steep hillsides in the Ross area. However, there is an inherent risk of instability associated with all hillside construction. Therefore, we are unable to guarantee the stability of any hillside construction. For houses constructed on hillsides, we recommend that mudflow and earthquake insurance be obtained.

If conditions different from those described in this report are encountered during construction or if the project is revised, we should be notified immediately so that we may modify our recommendations, if warranted.

Soil conditions and standards of practice change. Therefore, we should be consulted to update this report if construction is not performed within 18 months.

DAR:ts/R40-2

REFERENCES

Borcherdt, R.D., 1975; Studies for Seismic Zonation of the San Francisco Bay Region: U.S. Geological Survey Professional Paper 941-A, 102 pp.

Hays, W.W., 1980; Procedures for Estimating Earthquake Ground Motions: U.S. Geological Survey Professional Paper 1114, 77 p.

Herzog, Donald and Associates, 1982; Report Preliminary Geotechnical Investigation.

Jennings, C.W., 1975; Fault Map of California with Locations of Volcanoes, Thermal Springs, and Thermal Wells: California Division of Mines and Geology, Geological Data Map No. 1, Scale 1:750,000.

Ploessel, M.R. and Slosson, J.E., 1974; Repeatable High Ground Accelerations from Earthquakes: California Geology, September, 1974, pp. 195-199.

Seed, H.R. and Idriss, 1982; Ground Motion and Soil Liquefaction During Earthquakes: Earthquake Engineering Research Institute.

AERIAL PHOTOGRAPHS AV 2140-03-22, 23, and 24; 5/3/82.

Blake, M. D., Jr., and others, 1974; Preliminary Geologic Map of Marin and San Francisco Counties, and Parts of Alameda, Contra Costa, and Sonoma Counties, California: U. S. Geological Survey Miscellaneous Field Studies Map, MF-574, BDC 64, Scale 1:62,500.

Smith, T.C.; Rice, S.J.; and Strand, R.G., 1976; Relative Stability of the Upland Slopes in the Upper Ross Valley and the Western Part of the San Rafael Area, Marin County, California: California Division of Mines and Geology, Open File Report 76-3SF, Plate 2-B, Scale 1:12,000.

LIST OF PLATES

TEST PIT NO.	DEPTH(Feet)	DESCRIPTION
1A	0 - 1.5	BROWN-RED BROWN SILTY GRAVEL TO GRAVELLY SILT(GM/ML), loose to medium dense, moist, roots in upper 18 inches.
	1.5 - 6.7	RED BROWN CLAYEY GRAVEL (GC), stiff to hard, dry, gravels consist of greenstone rock fragments.
	6.7 - 17.0	RED BROWN CLAYEY GRAVEL(GC), dense, moist, gravels consist of greenstone; becoming very wet and soft at 13' to 15'. At 17 becoming gravelly and stiffer. (Slide debris/Colluvium)
2A	0 - 3	BROWN GRAVELLY CLAY(CL), medium stiff, moist.
	3 - 11	GREY SHEARED SHALE, friable to weak, highly weathered to stiff to very stiff, gravelly clay, large blocks of meta sandstone. Layer of brown, moist, gravelly clay/clayey gravel at 9.

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LOG OF TEST PIT 1A & 2A

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PLATE
2

TEST PIT NO.	DEPTH(Feet)	DESCRIPTION
3A	0 - 7.5	REDDISH BROWN GRAVELLY CLAY(CL), medium stiff to stiff, gravels consist of greenstone. (Colluvium)
	7.5 - 8.5	BROWN CLAYEY GRAVEL(GC), dense, moist. (Colluvium)
	8.5 - 11	RED BROWN SANDY CLAY(CL), stiff, moist with occasional gravels. Well defined textural change. (Residual)
	11 - 15	LIGHT BROWN CLAYEY GRAVEL(GC), dense, moist, gravels consist of sandstone/siltstone, very friable, highly weathered (Residual)
	15 - 16	LIGHT BROWN SILTSTONE, very closely spaced fractured, weak, plastic, highly weathered to clayey silt, with resistant blocks of siltstone.
4A	0 - 1	BROWN SILTY SAND(SM), medium, dense, dry, porous w/roots. (Topsoil)
	1 - 4	RED BROWN GRAVELLY CLAY(CL), stiff, slightly moist. (Colluvium)
	4 - 5.5	RED BROWN TO GREY SILTY SAND(SM), very dense, dry, with gravels (Residual)
	5.5 - 9.5	BROWN FINE GRAINED SANDSTONE, closely spaced fractures, friable, highly weathered in areas sheared, becoming stronger below 7.5

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LOG OF TEST PIT 3A & 4A

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PLATE
3

TEST PIT NO.	DEPTH(Feet)	DESCRIPTION
5A	0 - 1.5	BROWN SILTY SAND(SM), medium dense, dry, porous, with occasional gravels (Topsoil)
	1.5 - 4.5	LIGHT RED BROWN SANDY CLAY(CL/CH), medium stiff to stiff moist (Colluvium)
	4.5 - 6.0	RED BROWN TO GREY CLAYEY SAND(SC), dense, moist (Residual)
	6 - 7.5	LIGHT BROWN TO GREY SHEARED SANDSTONE, friable to moderately strong, highly weathered
6A	0 - 1	BROWN SILTY SANDY(SM), medium dense, slightly moist, porous, roots. (Topsoil)
	1 - 3.5	BROWN TO GREY SANDY CLAY(CL), stiff, slightly moist (Colluvium)
	3.5 - 9	REDDISH BROWN CLAYEY SAND(SC), dense to very dense, slightly moist, with occasional gravels of greenstone. (Colluvium)
	9 - 11.5	BROWN CLAYEY GRAVELS/GRAVELLY CLAY(GC/CL), dense, stiff, slightly moist (Residual)
	11.5 - 12.5	GREY BROWN SANDSTONE, closely spaced fractures, moderately strong, highly weathered.

TEST PIT NO.	DEPTH(Feet)	DESCRIPTION
7A	0 - 1.5	BROWN SILTY SAND(SM), medium dense, slightly moist to dry, porous w/roots. (Topsoil)
	1.5 - 14	REDDISH BROWN CLAYEY SAND/SANDY CLAY(CL/SC), medium stiff, medium dense, slightly moist, with occasional to abundant greenstone gravels, porous, becoming stiff at 9', still porous at 13'. (Colluvium)
8A	0 - 0.5	BROWN SILTY SAND(SM), medium dense, dry, porous with roots. (Topsoil)
	0.5 - 7	RED BROWN GRAVELLY CLAY/CLAYEY GRAVEL(CL/GC), medium stiff, medium dense, slightly moist, slightly porous, gravel consists of greenstone. (Colluvium)
	7 - 9	MOTTLED GREY BROWN SANDY CLAY, stiff, slightly moist with abundant sandstone fragments. (Residual)
	9 - 13	LIGHT BROWN SANDSTONE, extremely closely spaced fractures, plastic to weak, highly weathered, with red staining between fracture surfaces.

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LOG OF TEST PIT 7A & 8A
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PLATE
5

TEST PIT NO.	DEPTH(Feet)	DESCRIPTION
9A	0 - 1	BROWN SILTY SAND(SM), medium dense, dry (Topsoil).
	1 - 2.5	MOTTLED BROWN SANDY CLAY(CL), stiff, slightly moist (Residual)
	2.5 - 6	DARK GREY BROWN SANDSTONE, closely spaced fractures, medium strong to strong, highly weathered.
10A	0 - 1	BROWN SILTY SAND(SM), medium dense, dry with roots. (Topsoil)
	1 - 3.5	MOTTLED YELLOW BROWN SANDY CLAY TO CLAYEY SAND(CL/SC), medium dense, medium stiff, slightly moist with occasional sandstone gravels (Residual)
	3.5 - 6	YELLOW BROWN SANDSTONE, closely spaced fractures, firm, weak, friable, highly weathered.

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LOG OF TEST PIT 9A & 10A

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PLATE
6

TEST PIT NO.	DEPTH(Feet)	DESCRIPTION
11A	0 - 1	BROWN SILTY SAND(SM), medium dense, slightly moist, porous with roots. (Topsoil)
	1 - 5	DARK BROWN SANDY CLAY(CL), stiff, slightly moist with occasional greenstone and chert gravels. (Slide debris)
	5 - 5.8	GREY SANDY CLAY(CH), medium stiff to stiff, moist; sharp contact with upper and lower unit. Sheared texture, occasional slickensided facets. (Slide plane)
	5.8 - 7	MOTTLED RED/BROWN/GREY SANDY CLAY(CL), stiff, moist, slightly sheared texture (Slide debris).
	7 - 8	GREY BROWN CLAYEY GRAVEL(GC), dense, moist gravels consist of sandstone. (Slide debris)
	8 - 9.2	GREY BROWN SANDY CLAY(CL), stiff, moist with occasional gravel. (Slide debris)
	9.2 - 9.3	GREY SILTY CLAY(CH), medium stiff, moist; continuous striated planar contact. (Slide plane)
	9.3 - 11	BROWN GREY SANDY CLAY(CL), stiff, moist, slightly sheared texture. (Residual)
	11 - 12	GREY BROWN SANDSTONE, closely spaced fractures, moderately strong to strong, highly weathered.

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LOG OF TEST PIT 11A

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PLATE
7

TEST PIT NO.	DEPTH(Feet)	DESCRIPTION
12A	0 - 1	BROWN SANDY SILT(SM), medium stiff, dry with roots. (Topsoil)
	1 - 3	GREY BROWN SANDY CLAY(CL), stiff, slightly moist, with occasional shale fragments. (Residual)
	3 - 8	GREY TO BROWN SEMI SHEARED SHALE, extremely closely spaced fractures, friable, weak, with occasional sandstone inclusions.
	8 - 9	BROWN SANDSTONE, moderately spaced fractures, strong, highly weathered.
13A	0 - 1	BROWN SILTY SAND(SM), medium dense, dry. (Topsoil)
	1 - 5.5	BROWN CLAYEY SAND TO SANDY CLAY(SC/CL), stiff/dense, slightly moist (Colluvium)
	5.5 - 7.5	REDDISH PINK SILTY CLAY(CH), stiff, moist, sheared texture, expansive. (Residual)
	7.5 - 8	RED TO BROWN CHERT, closely spaced fractures, strong moderately weathered.

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LOG OF TEST PIT 12A & 13A

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Marin County California

PLATE
8

TEST PIT NO.	DEPTH(Feet)	DESCRIPTION
14A	0 - 1	BROWN SILTY SAND(SM), medium dense, dry with roots (Topsoil).
	1 - 3.5	MOTTLED GREY BROWN SANDY CLAY(CL), stiff, slightly moist. (Residual)
	3.5 - 5.5	YELLOW BROWN SANDSTONE, closely spaced fractures, weak to moderately strong, semi-sheared, highly weathered.
15A	0 - 1.5	BROWN SILTY SANDY SAND(SM), medium dense, dry, with roots. (Topsoil)
	1.5 - 3	DARK BROWN CLAYEY GRAVEL(GC), dense, dry (Residual)
	3 - 5	YELLOW BROWN SANDSTONE, closely spaced fractures, weak to moderately strong, highly weathered.

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LOG OF TEST PIT 14A & 15A

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PLATE
9

TEST PIT NO.	DEPTH(Feet)	DESCRIPTION
16A	0 - 1.5	BROWN SILTY SAND(SM), medium dense, dry, with roots. (Topsoil)
	1.5 - 7	RED BROWN CLAYEY GRAVEL(GC), dense, slightly moist, gravels consist of sandstone (Colluvium)
	7 - 10	RED BROWN SEMI SHEARED SANDSTONE AND SHALE, extremely closely fractures, weak highly weathered.
17A	0 - 2	BROWN SILTY SAND(SM), medium dense, dry, slightly porous w/roots (Topsoil).
	2 - 4.3	BROWN CLAYEY SAND(SC), dense slightly moist with abundant shale fragment (Colluvium)
	4.3 - 5.5	BROWN CLAYEY GRAVEL(CG), dense, slightly moist, gravels consist of shale. (Residual)
	5.5 - 7	REDDISH BROWN SHALE, extremely closed spaced fracture, friable to weak, highly weathered.

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LOG OF TEST PIT 16A & 17A

TOIGO PROPERTY SUBDIVISION

PLATE

10

MAJOR DIVISIONS				TYPICAL NAMES
GRAVELS MORE THAN HALF COARSE FRACTION IS LARGER THAN NO. 4 SIEVE	CLEAN GRAVELS WITH LITTLE OR NO FINES	GW	░	WELL GRADED GRAVELS, GRAVEL-SAND MIXTURES
		GP	░	POORLY GRADED GRAVELS, GRAVEL-SAND MIXTURES
	GRAVELS WITH OVER 12% FINES	GM	░	SILTY GRAVELS, POORLY GRADED GRAVEL-SAND-SILT MIXTURES
		GC	▨	CLAYEY GRAVELS, POORLY GRADED GRAVEL-SAND-CLAY MIXTURES
SANDS MORE THAN HALF COARSE FRACTION IS SMALLER THAN NO. 4 SIEVE	CLEAN SANDS WITH LITTLE OR NO FINES	SW	░	WELL GRADED SANDS, GRAVELLY SANDS
		SP	░	POORLY GRADED SANDS, GRAVELLY SANDS
	SANDS WITH OVER 12% FINES	SM	▥	SILTY SANDS, POOORLY GRADED SAND-SILT MIXTURES
		SC	▨	CLAYEY SANDS, POORLY GRADED SAND-CLAY MIXTURES
SILTS AND CLAYS LIQUID LIMIT LESS THAN 50		ML	▥	INORGANIC SILTS AND VERY FINE SANDS, ROCK FLOUR, SILTY OR CLAYEY FINE SANDS, OR CLAYEY SILTS WITH SLIGHT PLASTICITY
		CL	▨	INORGANIC CLAYS OF LOW TO MEDIUM PLASTICITY, GRAVELLY CLAYS, SANDY CLAYS, SILTY CLAYS, LEAN CLAYS
		OL	▥	ORGANIC CLAYS AND ORGANIC SILTY CLAYS OF LOW PLASTICITY
SILTS AND CLAYS LIQUID LIMIT GREATER THAN 50		MH	▥	INORGANIC SILTS, MICACEOUS OR DIATOMACIOUS FINE SANDY OR SILTY SOILS, ELASTIC SILTS
		CH	▨	INORGANIC CLAYS OF HIGH PLASTICITY, FAT CLAYS
		OH	░	ORGANIC CLAYS OF MEDIUM TO HIGH PLASTICITY, ORGANIC SILTS
HIGHLY ORGANIC SOILS		Pt	▦	PEAT AND OTHER HIGHLY ORGANIC SOILS

UNIFIED SOIL CLASSIFICATION SYSTEM

			Shear Strength, psf Confining Pressure, psf	
Consol	Consolidation	Tx	630 (2400)	Unconsolidated Undrained Triaxial
LL	Liquid Limit (in %)	TxCU	240 (2100)	Consolidated Undrained Triaxial
PL	Plastic Limit (in %)	DS	3740 (1200)	Consolidated Drained Direct Shear
PI	Plasticity Index	FVS	320	Field Vane Shear
Gs	Specific Gravity	UC	4200	Unconfined Compression
SA	Sieve Analysis	LVS	500	Laboratory Vane Shear
■	Undisturbed Sample	SS	Shrink Swell	
☒	Bulk or Disturbed Sample	EXP	Expansion	
▩	Standard Penetration Test	P	Permeability	
☐	Sample Attempt with No Recovery			

KEY TO TEST DATA

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Job No: 1385.2.0.1
Appr:
Drwn: JN
Date: Oct. 89

SOIL CLASSIFICATION CHART AND KEY TO TEST DATA
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Marin County, California

PLATE
11

ROCK SYMBOLS

▤	SHALE OR CLAYSTONE	△	CHERT	▥	SERPENTINITE
▦	SILTSTONE	⁘	PYROCLASTIC	▨	METAMORPHIC ROCKS
▒	SANDSTONE	⁛	VOLCANIC	░	ALTERED ROCKS
◌	CONGLOMERATE	∴	PLUTONIC	▧	SHEARED ROCK

LAYERING

MASSIVE	Greater than 6 feet
THICKLY BEDDED	2 to 6 feet
MEDIUM BEDDED	8 to 24 Inches
THINLY BEDDED	2-1/2 to 8 inches
VERY THINLY BEDDED	3/4 to 2-1/2 inches
CLOSELY LAMINATED	1/4 to 3/4 inches
VERY CLOSELY LAMINATED	Less than 1/4 inch

JOINT, FRACTURE, OR SHEAR SPACING

VERY WIDELY SPACED	Greater than 6 feet
WIDELY SPACED	2 to 6 feet
MODERATELY SPACED	8 to 24 Inches
CLOSELY SPACED	2-1/2 to 8 Inches
VERY CLOSELY SPACED	3/4 to 2-1/2 Inches
EXTREMELY CLOSELY SPACED	Less than 3/4 Inch

HARDNESS

Soft - pliable; can be dug by hand

Firm - can be gouged deeply or carved with a pocket knife

Moderately Hard - can be readily scratched by a knife blade; scratch leaves heavy trace of dust and is readily visable after the powder has been blown away

Hard - can be scratched with difficulty; scratch produces little powder and is often faintly visible

Very Hard - cannot be scratched with pocket knife, leaves a metallic streak

STRENGTH

Plastic - capable of being molded by hand

Friable - crumbles by rubbing with fingers

Weak - an unfractured specimen of such material will crumble under light hammer blows

Moderately Strong - specimen will withstand a few heavy hammer blows before breaking

Strong - specimen will withstand a few heavy ringing hammer blows and usually yields large fragments

Very Strong - rock will resist heavy ringing hammer blows and will yield with difficulty only dust and small flying fragments

DEGREE OF WEATHERING

Highly Weathered - abundant fractures coated with oxides, carbonates, sulphates, mud, etc., thorough discoloration, rock disintegration, mineral decomposition

Moderately Weathered - some fracture coating, moderate or localized discoloration, little to no effect on cementation, slight mineral decomposition

Slightly Weathered - a few stained fractures, slight discoloration, little or no effect on cementation, no mineral decomposition

Fresh - unaffected by weathering agents, no appreciable change with depth

Herzog Associates
Geoscientists

Job No: 1386.2.0.1
Appr:
Drwn: JN
Date: Oct. 89

ENGINEERING GEOLOGY
ROCK TERMS
TOIGO PROPERTY SUBDIVISION
Marin County, California

PLATE
12

APPENDIX

October 7, 1982

1385.1

Roth Properties
833 Market St.
San Francisco, Ca.

Attention: Dwight Johnson
Vice President

Dear Mr. Johnson:

Preliminary Investigation
Geotechnical Feasibility
Site Development
Roth Properties
Ross, California

INTRODUCTION

This report presents the results of our geotechnical investigation of a proposed residential subdivision within the Roth Properties, in Ross, California. The purpose of our investigation was to assess the feasibility and stability of five proposed building sites and an access road alignment, as shown on a schematic layout dated September, 1982 by CHNMB.

The purpose of our investigation was to develop the following geotechnical conclusions and recommendations:

1. A description of the surface and subsurface soil and rock conditons.
2. An evaluation of potential geologic hazards and mitigation measures.
3. Development feasibility.
4. Generalized recommendations for grading and construction.

The investigation is intended to satisfy the requirements for the Tentative Map stage of planning development. Specific issues relating to Section 18.39.030, Ordinance 435 of the Ross Municipal Code are also addressed.

WORK PERFORMED

The site was initially inspected by our Principal Engineer and our Registered Geologist, in conjunction with the Project Planners and Property Manager. Based upon that inspection, a subsurface field exploration program was established. Prior to the field exploration, selected geotechnical references pertinent to the area were reviewed (Smith et. al., 1976; Blake et. al., 1974; Wentworth and Frizzell, 1975).

Subsurface conditions were explored to the extent of 21 backhoe-dug test pits. The pits were located and logged in the field by our Registered Geologist. The five proposed building sites and portions of the road alignment and access driveways were explored. Representative bulk soil and rock samples were collected. Logs of the test pits are presented in the Appendix. Descriptions are primarily based on an *in situ* examination of the materials. The locations of the test pits as well as bedrock types and depths to competent rock are shown on a topographic base map of the area (Plate 1).

SITE CONDITIONS

The Roth Property lies on the eastern side of Bald Hill, and consists of several southeast trending ridges and three major drainage swales. The northern and eastern property boundaries border privately owned land. The western and southern boundaries border Marin Municipal Water District land.

The central drainage swale is spring-fed, and was flowing at the time of investigation. Slopes are variable, with nearly level benched areas along ridge crests, and, with steep-sided ravine flanks along portions of the drainage swales. The area is densely covered with hardwood trees and scattered brush. The site reportedly was logged in the early 1900's, and logging skid-trails are still evident. The ground surface is covered with abundant organic debris, and areas of bare soil are rare. The site appears well drained, and there was no evidence of excessive surface erosion. The major drainage channels typically are steep-sided and incised. Bedrock is occasionally exposed along portions of the channel bottoms. Generally, rock exposures are rare, except throughout the higher western elevations. Most of the area is mantled with soil.

Franciscan bedrock within the area has been mapped as sandstone and shale in contact with greenstone (Smith, et al., 1976), and as melange (Blake et. al. 1974); a hetergeneous mixture of blocks of sandstone, greenstone, chert and serpentinite in a matrix of sheared shale. The entire area has also been depicted as being within a complex massive landslide deposit (Smith, et. al., 1976; Wentworth and Frizzell, 1975).

Data from our subsurface exploration indicates that bedrock conditions (lithology and depth to rock) vary markedly throughout the site. The areas of proposed development are underlain predominantly by sandstone and shale. Melange zones of sheared shale and areas of massive greenstone were encountered intermittently. The sandstone and shale typically are moderately strong, closely to intensely fractured, deeply weathered and non-expansive. Melange matrix material normally was weak (for rock), sheared, and deeply weathered. Highly altered areas are clayey and expansive. Resistant inclusions of graywacke and greenstone within the sheared shales are strong.

The depth to competent bedrock varied from 1 to over 13 feet below existing ground surface. Topographically benched areas and breaks in slope often appeared to correlate with changes in bedrock compositon.

The soils throughout the site are also highly variable in extent and composition. Organic rich topsoil horizons varied from 1 to 2-1/2 feet thick, and generally consisted of dry, and loose, compressible sands and silts, with varying amounts of gravel-sized rock fragments. Areas of extensive surface soil cracking, indicative of expansive soil, were not evident.

Deep colluvial soils of stiff sandy silts, sandy clays, and clayey sands, were common in many areas. Generally, these soils appeared well consolidated, only slightly compressible, and non-expansive.

Areas of extensive landslide debris were predominantly found along the drainage channels. These soils varied from dry and stiff clayey-gravels, to wet and soft expansive sandy clays.

Groundwater was not encountered in any of the test pits. However, shallow subsurface groundwater within the soils overlying bedrock may be expected during wetter months.

Landslide deposits appeared to be relatively "old" meta stable features. There was one area of recent activity within the older deposits. This area is below the existing cabin, approximately 65 feet downslope from the proposed road alignment. A fresh scarp, from 1/2 to 2 feet high, and extensive ground disturbance was apparent. The failure appears to be a shallow-seated slump and portions of the toe have encroached upon the creek. There did not appear to be any significant slope failures within areas of proposed development or in areas upslope from the property, that could be attributed to the intense winter storms of 1981-1982. Minor slumping and erosion was evident along some portions of the steep-sided ravine banks. However, these failures are not within propsed building areas, and should pose no constraint to development.

Slide debris within the central drainage ravine appears to have been derived from an area 1500 feet upslope, and outside of the property. Slide debris along the southern ravine appears to have been derived from an area of massive greenstone near the western property boundary.

There were no landforms within the area that would indicate the presence of active faults. Offsets within subsurface bedrock and soil layers were not evident within any of the test pits. The site is not within any current Alquist-Priolo Special Studies Zone. Presumed fault contacts between the sandstone and greenstone, and within melanage zones, resulted from structural deformation during pre-Quaternary time.

The site is within the California Coast Range Province, which is known to be a region of high seismicity. The nearest known active fault traces (Jennings, 1975), are 7-1/2 miles to the west (San Andreas Fault) and 12 miles to the east (Hayward Fault). Maximum predicted earthquake magnitudes (Richter Scale) for the San Andreas and Hayward Faults are 8.3 and 7.0 respectively.

CONCLUSIONS

Based upon the results of our investigation, we judge that the proposed schematic layout is feasible from a geotechnical standpoint. A well designed and engineered development would enhance the stability of the site, and locally improve surface and subsurface drainage.

The soils which blanket most of the slopes are relatively weak and compressible; experience slow downhill creep (on the order of a small fraction of an inch per year) as is typical of hillsides in Marin County; and are unsuitable for support of structures or fills. It will be necessary to construct fills on level keyways and benches founded in firm material beneath the soil. Where water is concentrated in swales, it will be necessary to drain the colluvium with subdrains. In some roadway areas, it will probably be necessary to reconstruct about the outer 8 feet of cut banks as drained compacted earth buttresses to support the upslope colluvium.

Constructing roads on the steep hillsides will require retaining structures, and may require side hill bridges. It will be necessary to support the foundations in firm rock below the colluvium. In sloping areas, it will be necessary to design foundations to resist lateral forces caused by downhill creep of the soils above the rock. It will be necessary to found retaining structures on firm rock, and to provide lateral confinement between the retaining structure foundations and the face of downhill slopes. Retaining structure foundations on the downhill side of roads will be much deeper than those on the uphill sides of roads. Therefore, it will be more economical to install retaining walls on the upslope sides of roads.

There are no known active faults within the site, and the potential for surface rupture is considered low. The maximum peak bedrock acceleration and repeatable ground acceleration anticipated are 0.6g and 0.4g respectively (Hays, 1980; Ploessal and Slosson, 1974). Assuming a causative earthquake of 8.3 magnitude on the San Andreas Fault, we believe a site period of 0.3 seconds is applicable to the site. These values are within the typical range for Marin County hillsides.

The areas of landslide deposits do not appear to be as extensive as previously mapped. The ridges consist of competent bedrock at a relatively shallow depth. Benched topographic features appear related to compositional changes or structural contacts between bedrock units, rather than to large scale landsliding.

Massive debris flow deposits are present within the ravines. Where explored, the depths to competent rock was as much as 14 feet below ground surface. Deep colluvial soil deposits, 8 to 11 feet thick, were also found in some swales. Although these deposits are inherently weak and potentially unstable, we judge that hazards may be mitigated with proper grading and/or structural design. Construction activities are not expected to cause deepseated reactivation of the large debris flow deposits. The type of failures that might be expected would consist of shallow-seated slumps and flows within the upper few feet of surfical soil, and can be mitigated.

The five proposed building sites are located within relatively stable areas. The depth to competent bedrock for three of the sites is shallow (less than 5 feet). The remaining two sites contain deep, but well consolidated, very stiff, colluvial soil. We judge that the soil and/or rock within all site locales will provide adequate support for typical residential structures. In areas of deep soil, drilled pier foundations should be used. In areas of shallow bedrock, conventional spread footings may be appropriate. Large resistant melange inclusions may pose difficulty for drilling or excavation.

Several areas indicative of springs were observed at the site, and other springs may be encountered during construction. Seepage can reduce slope stability, and can adversely affect pavement and foundation performmance. In the improvement areas, it will be necessary to intercept seepage with subsurface drainage facilties.

Because there are areas of steep terrain underlain by deep, inherently weak soil, the threat of debris slide activity must be regarded as a potential hazard. However, the propsed building envelopes are not within the path of any apparent landslide deposits, and there are no indications that the existing slides are enlarging or encroaching upon the sites. Most sites are situated on relatively level ridge areas, away from steep slopes. The threat of upslope debris slides impacting these areas is considered low. The central-most lot site is near a drainage swale with a moderately steep upslope area. The slope configuration is such that the western portion of this lot site may need catchment or diversion walls to protect against potential upslope failures.

The proposed road alignment will cross areas of old slide debris. Portions of the road will cross areas where there is a potential impact from upslope or downslope failures. Special design and mitigation measures within these areas are advisable, and are feasible from a geotechnical standpoint. Possible mitigation measures include replacing the slide debris beneath and immediately upslope of the roads as a compacted buttress; supporting the road and the area immediately upslope with crib walls; or constructing the road on a side hill bridge design to resist a creep force and to allow passage of slide debris.

Retaining walls and/or side hill bridges may be supported on drilled, cast-in-place, reinforced concrete piers. In landslide areas, the landslides should be stabilized as buttresses prior to installing piers, or the piers designed to resist slide forces. In other areas, the piers can be installed through the natural soils, and designed to resist downhill creep of the soil above the rock.

Roadways outside of slide areas, or on reconstructed landslides should tolerate anticipated minor creep with no more cracking than is typical for Marin County roadways. It will be necessary to construct utilities with flexible pipe, or to provide with frequent joints to accommodate minor creep movement.

Compacted fill generally settles about one percent of its thickness. Roadway grades and utilities should be designed to accommodate this settlement.

Existing site conditions indicate the area is well drained. Problems associated with excessive surface erosion or ponding are not evident. Construction and grading will expose areas of deep, weak soil and slide debris, which may be sensitive to erosion and/or slope failure. Erosion protection measures during and after construction should be utilized to reduce the risk of induced instability.

RECOMMENDATIONS

Site Grading

Areas to be developed should be cleared of vegetation and of the upper few inches of soil containing organic matter. The strippings should be removed or stockpiled for reuse as topsoil. Excavation should then be performed as necessary. We anticipate that with the exception of organic matter and of rocks or lumps larger than six inches in diameter, the excavated material will be suitable for reuse as compacted fill. Organic matter should be disposed of off of the site. Larger material should be disposed of outside of improvement areas. Areas to receive fill should be prepared by cutting level keyways extending into rock. The outboard edge of the keyway excavation should intercept a 1:1 line projected down from the toe of the planned fill.

Subsurface drainage facilities should be installed at the rear of keyways as recommended by the Soil Engineer. The depth and extent of keyways and subdrains should be determined and approved by the Soil Engineer in the field during construction.

Where slope stabilization measures are needed, excavation of the landslides during site grading operations, or reconstructing the landslides as compacted earth buttresses with subsurface drainage facilities, offer the greatest reduction of risk. Excavating the landslides consists of removing all the landslide debris, and exposing a relatively flat slope in firm material beneath the slide plane. Buttressing consists of excavating the slide debris; cutting wide level keyways into firm underlying rock; installing subsurface drainage facilities; and backfilling the excavation with compacted fill.

Cut and fill banks generally should be no steeper than two horizontal to one vertical (2:1). Where sloughing is acceptable, cutbanks in rock may be 1-1/2:1. Cuts in weak, expansive melange soils or slide debris should be protected, retained or rebuilt. Where the finished slope must be steeper, or will not "catch", it will be necessary to use crib walls, bin walls, reinforced earth, or other retaining structures. These retaining structurues must be founded on firm rock beneath the zone of weakness or else on compacted fill founded on firm rock beneath the weak soils.

As an alternative to repairing an entire landslide area, only the portion of the landslide affecting the planned improvements could be repaired. Improvements can be protected from downslope landslides by extending a buttress down at 2:1 to firm rock beneath the slide plane, or by supporting the downhill edge of the buttress with a retaining structure founded on firm rock beneath the slide plane. The unrepaired portion(s) of the slide downslope of the buttress may continue to move, but should not adversely affect the buttress.

Improvements can be protected from upslope slides by retaining the slides with a retaining structure, or with a partial earth buttress underlain by subsurface drainage systems. If the upslope is steep enough to allow debris flows to overtop the retaining structure or buttress, it will be necessary to enhance the stability of the upslope area with subsurface drains, and/or to provide catchment areas for possible slough debris and mudflows.

Crib Walls

Crib walls may be used to either support fills, cuts or landslides. Crib walls should be of reinforced concrete construction and should conform to the California Department of Transportation Specifications. Crib walls should be battered at least one foot for every six feet of height. Crib walls should be founded on firm rock or on engineerred fill founded on firm rock. The wall toe should be founded at least 18 inches into rock for walls less than 10 feet high; at least 30 inches deep for walls 10 to 15 inches high; and at least 3 feet deep for walls over 15 feet high. The toe of the walls should also be deepened as necessary to provide at least 7 feet of horizontal confinement between the toe of the walls and the face of the nearest slope. Subsurface drainage should be provided from the rear of wall foundation excavations.

Reinforced Earth Walls

Reinforced Earth (patented by the Reinforced Earth Company) consists of thin facing panels connected to strips extending into the backfill. The strips are generally galvanized steel about 1/8 inch thick and 2 to 3 inches wide. The strips are located a few feet apart horizontally and vertically, and extend back a distance equivalent to about 80 percent of the wall height. Areas to receive reinforced earth are prepared by excavating a level bench into firm material, as previously described for Concrete Crib Walls. Strips are extended across the bottom of the excavation. A row of facing panels a few feet high is then placed along the outboard edge of the planned fill, and attached to the strips. The panels are then backfilled with granular material. Strips are then attached to the tops of these panels, and extended back across the granular fill. A second row of panels is placed, and compacted granular backfill placed over the top of the strips. Another row of strips is placed, panels installed, and additional backfilled placed. The resulting walls are generally more economical than crib walls or reinforced concrete retaining walls for heights greater than about 15 feet. The major disadvantage is that the walls require clean granular backfill in order to bond adequately to the reinforcing strips.

Properly constructed reinforced earth walls can support roadways and can retain landslides. As with retaining walls and crib walls, reinforced earth walls should be fully backdrained.

Utilities

Utilities in slide areas should be constructed in compacted earth buttresses founded on firm material beneath the slide plane, or else should extend into firm rock beneath the slide plane. Utilities founded in colluvium on slopes will experience differencial lateral movement as the soils above the rock slowly creep downslope (on the order of a small fraction of an inch per year). Frequent joints should be provided to accommodate the anticipated creep movement.

Seepage will accumulate in utility trench backfill. Gravity flow outlet pipes should be provided from the bedding material at each low point, and at least every 500 feet to prevent buildup of hydrostatic pressure in utility trenches.

Where utility benches are steep enough to erode, check dams should be provided. Where utility trenches are steep enough to erode, check dams and/or rip rap should be provided.

Pavement

The pavement design should be based upon traffic indices provided by the City of Ross Engineer, and upon R Value tests on representative soils exposed at subgrade level after rough grading operations. All utilities should be installed and property backfilled prior to subgrade preparation.

Some of the on-site soil is expansive, and will swell when wet and shrink when dry. The shrinking and swelling will cause pavements to experience edge cracking. Edge cracks must be sealed as they occur. Concrete paving should be reinforced to reduce cracking, and provided with frequent joints to control cracking.

Building Foundations

We anticipate that the portion of buildings constructed on level areas excavated into firm soil or rock can be supported on continuous and interconnected spread footings. Drilled piers extending into rock will be necessary on and near slopes, and may be used everywhere. It will be necessary to design piers to resist lateral forces caused by downhill creep of soils above the rock.

Soil Engineering Drainage

Surface runoff should be diverted away from cut and fill banks. Subsurface drainage facilities should be installed beneath fills; behind retaining walls; where springs are observed; and in other areas as determined by the Soil Engineer during site grading operations.

Surface water should be diverted away from slopes and slide areas by means of concrete lined interceptor ditches. Surface and subsurface drainage facilities should be maintained entirely separate. Drains should outlet into erosion resistant areas, and should not concentrate water above neighboring property.

Portions of landslides above buttresses, and other unrepaired landslides where reactivation would be detrimental, should be enhanced with surface and subsurface drainage improvements. The subsurface drainage improvements should consist of a subdrain extending down the central axis of the landslide, with laterals extending to each side at about 50 foot intervals. Roadways that cross areas of deep, weak soil, or excessively wet areas, may need subdrains along the inboard (uphill) side. Subdrains should extend below the soil/rock contact, and should be sloped to drain by gravity.

Supplemental Services

This is a preliminary investigation for evaluating project feasibility. Additional investigations will be necessary to develop geotechnical design criteria for actual design and construction.

We should review the master plan for conformance with the intent of this investigation.

Limitations

We have performed this preliminary investigation in accordance with current standards of engineering practice. We offer no other guarantees or warranties either exposed or implied.

We trust this provides the information you require at this time. If you have questions, please call.

Yours very truly,

DONALD HERZOG & ASSOCIATES, INC.

Donn Ristau

Donn A. Ristau,
Senior Staff Geologist
Registered Geologist - 3634

Donald Herzog,
Principal Engineer
Civil Engineer - 18093

DAR:pb

4 copies submitted

Plate 1

Appendix A

References

REFERENCES

Blake, M. C., Jr. and others, 1974, Preliminary Geologic Map of Marin, and San Francisco Counties and Parts of Alameda, Contra Costa and Sonoma Counties, California: U.S. Geological Survey Miscellaneous Field Studies Map MF-574, BDC 64, Scale 1:62,500.

Wentworth, C. M. and Frizzell, V. A., 1975, Reconnaissance Landslide Map of Parts of Marin and Sonoma Counties, California, San Rafael Sheet: U.S. Geological Survey, Scale 1:24,000.

Smith, T. C.; Rice, S. J.; and Strand, R. G., 1976, Geology of the Upper Ross Valley and the Western Part of the San Rafael Area, Marin County, California: California Division of Mines and Geology, OFR 76-2SF, Plate 1B, Scale 1:12,000.

Smith, T. C.; Rice, S. J.; and Strand, R. G., 1976, Relative Stability of the Upland Slopes in the Upper Ross Valley and the Western Part of the San Rafael Area, Marin County, California: California Division of Mines and Geology, OFR 76-2SF, Plate 2B, Scale 1:12,000.

Jennings, C. W., 1975, Fault Map of California With Locations of Volcanoes, Thermal Springs, and Thermal Wells: California Division of Mines and Geology, Geol. Data Map No. 1, Scale 1:750,000.

Hays, W. W., 1980, Procedures for Estimating Earthquake Ground Motions: U. S. Geological Survey Professional Paper 1114, 77 p.

Ploessel, M. R., and Slosson, J. E., 1974, Repeatable High Ground Accelerations from Earthquakes, California Geology, September, pp. 195-199.

APPENDIX A

Test Pit	Depth (inches)	Description
1.	0 - 15	RED BROWN SILTY SAND, dry, loose; TOPSOIL
	15 - 84	MOTTLED YELLOW-WHITE-BROWN SILTY SAND with abundant fragments of deeply weathered greenstone, dry, very stiff; COLLUVIUM
	84 - 86	GREENSTONE, deeply weathered, gray clay seams, weak, dry; DEEPLY WEATHERED ROCK
2.	0 - 11	BROWN SANDY SILT, dry, loose; TOPSOIL
	11 - 78	MOTTLED ORANGE-BROWN SANDY SILT, with occasional rock fragments, dry, very stiff; COLLUVIUM
	78 80	GREENSTONE, hard, very strong, deeply weathered with clay seams in places; ROCK
3.	0 - 15	BROWN SILT with cobbles of greenstone and sandstone, dry, loose; TOPSOIL
	15 - 126	MOTTLED YELLOW-WHITE-BROWN SANDY SILT with fragments of sandstone, shale and greenstone, dry to moist, very stiff; COLLUVIUM

Test Pit	Depth (inches)	Description
4.	0 - 9	BROWN SANDY SILT with rock fragments, dry, loose; TOPSOIL
	9 - 73	MOTTLED YELLOW-RED-BROWN SANDY SILT with abundant rock fragments of greenstone and sandstone, dry, very stiff; COLLUVIUM
5.	0 - 32	BROWN SANDY SILT with occasional shale fragments, dry, loose; TOPSOIL
	32 - 48	MOTTLED YELLOW-BROWN CLAYEY SAND, moist, stiff; COLLUVIUM
	48 - 60	DARK GRAY-BROWN SANDY CLAY with abundant shale framents, moist, very stiff; RESIDUAL SOIL
	60 - 78	GRAY-BROWN SHALE, intensely fractured, moderately strong, deeply weathered; ROCK
6.	0 - 19	BROWN SILTY SAND, occasional rock fragments, dry, loose; TOPSOIL
	19 - 57	BROWN-ORANGE SANDY SILT with rock fragments, dry, very stiff; COLLUVIUM
	57 - 67	LIGHT BROWN SANDSTONE, closely fractured, deeply weathered, moderately strong; ROCK
7.	0 - 15	BROWN SILTY SAND, occasional rock fragments, dry, loose; TOPSOIL
	15 - 21	MOTTLED GRAY-BROWN-YELLOW SANDY CLAY, moist, very stiff; COLLUVIUM
	21 - 70	MOTTLED BROWN-ORANGE CLAYEY SAND with abundant greenstone fragments, moist, very stiff; RESIDUAL SOIL

Test Pit	Depth (inches)	Description
	70 - 90	GREENSTONE, intensely fractured, strong, red-brown clay seams; DEEPLY WEATHERED ROCK
8.	0 - 31	BROWN SANDY SILT, occasional rock fragments, dry, loose; TOPSOIL
	31 - 60	MOTTLED GRAY-BROWN-YELLOW GRAVELLY CLAY, moist, very stiff; FILL (?)
	60 - 78	RED-BROWN CLAYEY GRAVEL, moist, very stiff; SLIDE DEBRIS
	78 - 85	DARK GRAY SANDY CLAY with occasional rock fragments, moist, stiff; SLIDE DEBRIS
	85 - 94	INTENSLY FRACTURED BLOCKS OF SILICEOUS META-GRAYWACKE, META-VOLCANICS,voids, gray clay in fractures, wet, soft; SLIDE DEBRIS
	94 - 132	GRAY SANDY CLAY with abundant rock fragments, wet, stiff; SLIDE DEBRIS
	132 - 144	RED CLAY with rounded greenstone fragments, wet, stiff; SLIDE DEBRIS
	144 - 156	MOTTLED GREEN-YELLOW-RED-BROWN-GRAY CLAY/SANDY CLAY, wet, soft to moderately stiff; SLIDE DEBRIS
	156 158	DARK GRAY SANDY CLAY with rock fragments, appears to be deeply weathered melange material; DEEPLY WEATHERED ROCK

Test Pit	Depth (inches)	Description
9.	0 - 20	BROWN SANDY SILT, abundant rock fragments, dry, loose; TOPSOIL
	20 - 144	BROWN SILTY SAND, dry, stiff to very stiff; COLLUVIUM
	144 - 147	GREENSTONE FRAGMENTS, minor clay seams, deeply weathered, friable; WEATHERED ROCK
10.	0 - 14	BROWN SILTY SAND, minor rock fragments, dry, loose; TOPSOIL
	14 - 28	MOTTLED YELLOW-BROWN SANDSTONE AND SHALE, friable, dry; DEEPLY WEATHERED ROCK
	28 - 52	MOTTLED YELLOW-BROWN SANDSTONE, GRAY SHALE intensely fractured, weak, dry, deeply weathered; ROCK
11.	0 - 17	BROWN SILTY SAND dry, loose; TOPSOIL
	17 - 26	MOTTLED RED-BROWN SILTY SAND, dry to moist, stiff; COLLUVIUM
	26 - 48	MOTTLED YELLOW-BROWN SHALE, friable; DEEPLY WEATHERED ROCK
	48 - 83	YELLOW-BROWN SHALE, and SANDSTONE, friable, intensely fractured, moist; ROCK
12.	0 - 17	LIGHT GRAY-BROWN SAND, dry, loose; TOPSOIL
	17 - 38	YELLOW-BROWN SANDSTONE with sand, dry; DEEPLY WEATHERED ROCK

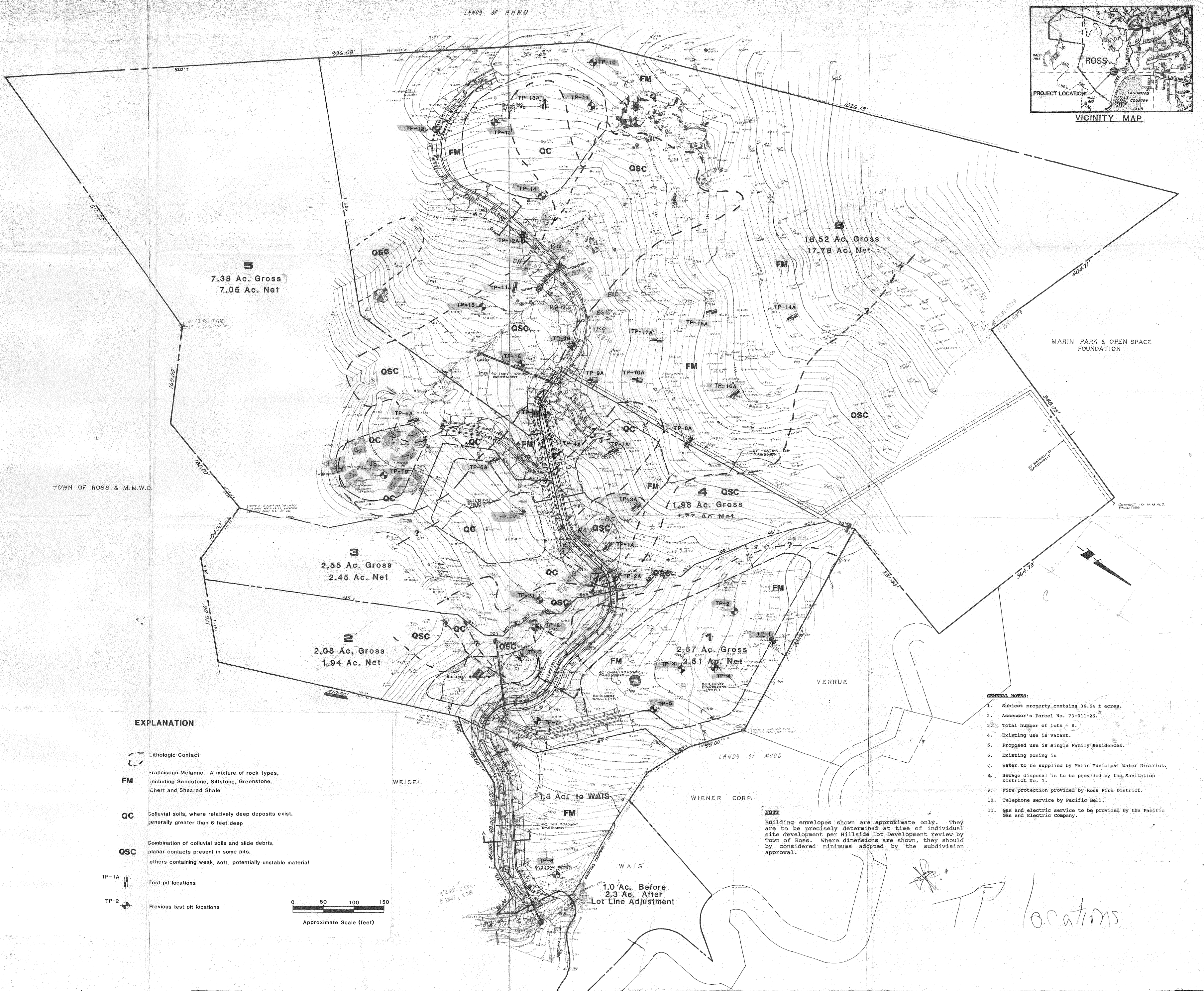
Test Pit	Depth (inches)	Description
	38 - 74	YELLOW BROWN SANDSTONE, weak, closely fractured, dry; ROCK
13.	0 - 29	BROWN SANDY SILT, dry, loose; TOPSOIL
	29 - 38	YELLOW-BROWN SANDY SILT, dry, stiff; COLLUVIUM
	38 - 65	DARK GRAY SANDY CLAY with graywacke rock fragments, moist, very stiff; RESIDUAL MELANGE SOIL
	65 - 78	DARK GRAY SHEARED SHALE AND GRAYWACKE, friable, moist, deeply weathered; MELANGE ROCK
14.	0 - 16	BROWN SANDY SILT, occasional rock fragments, dry loose; TOPSOIL
	16 - 27	YELLOW-BROWN SANDY CLAY with sandstone fragments, moist, stiff; COLLUVIUM
	27 - 49	GRAY-BROWN SANDY CLAY with graywacke rock fragments, grades to gray sandy clay, moist, very stiff; RESIDUAL MELANGE SOIL
	49 - 102	GRAY SHEARED SHALE AND GRAYWACKE, with inclusions of sheared yellow brown sandstone, friable, moist; MELANGE/SANDSTONE SHEAR ZONE, ROCK
15.	0 - 11	BROWN SANDY SILT, dry, loose; TOPSOIL
	11 - 73	MOTTLED YELLOW-BROWN SANDY CLAY, abundant rock fragments, grades to red-brown sandy clay, moist, very stiff; SLIDE DEBRIS

Test Pit	Depth (inches)	Description
	73 - 108	GRAY BROWN TO RED-BROWN SANDY CLAY, moist, very stiff; SLIDE DEBRIS
	108 - 121	MOTTLED YELLOW-RED SANDY CLAY with abundant rock fragments, moist very stiff; SLIDE DEBRIS
	121 - 134	GRAY-BROWN SANDY CLAY, occasional graywacke fragments, moist to dry, very stiff; DEEPLY WEATHERED SHEARED SHALE/RESIDUAL SOIL
16.	0 - 21	BROWN SILTY SAND, dry, loose; TOPSOIL
	21 - 34	RED-BROWN SANDY CLAY, minor rock fragments, dry, very stiff; COLLUVIUM
	34 - 42	RED-BROWN SANDSTONE with sand; DEEPLY WEATHERED ROCK
	42 - 64	YELLOW-BROWN SANDSTONE, intensely fractured, moderately strong, dry, deeply weathered; ROCK
17.	0 - 11	BROWN SILTY SAND, dry, loose; TOPSOIL
	11 - 52	BROWN SANDSTONE AND GRAY-BROWN SHALE, slightly sheared, intensely fractured, weak to moderately strong, dry, deeply weathered; ROCK
18.	0 - 10	BROWN SANDY SILT, occasional rock fragments, dry, loose; TOPSOIL
	10 - 48	BROWN SANDY SILT, dry, moderately stiff; COLLUVIUM
	48 - 94	MOTTLED BROWN SANDY CLAY, abundant greenstone and sandstone fragments, dry, stiff; SLIDE DEBRIS

Test Pit	Depth (inches)	Description
	94 - 95	GRAY SILTY CLAY, moist, very stiff; DEEPLY WEATHERED SILTSTONE
	95 - 106	YELLOW BROWN SANDSTONE, moderately fractured, strong, moderately weathered; ROCK
19.	0 - 17	BROWN SANDY SILT, dry, loose; TOPSOIL
	17 - 28	MOTTLED YELLOW-BROWN SANDY CLAY, abundant greenstone gragments, dry, very stiff; COLLUVIUM
	28 - 44	RED-BROWN GRAVELLY CLAY, moist, stiff; SLIDE DEBRIS (?)
	44 - 55	YELLOW-BROWN SANDY CLAY, with occcasional rock fragments, moist, stiff; COLLUVIUM
	55 - 64	DARK GRAY SANDY CLAY/SILTY CLAY, rock fragments, yellow sandy layer with sandstone at 60 inches, moist, very stiff; COLLUVIUM
	64 - 68	RED-BROWN GRAVELLY CLAY/CLAYEY GRAVEL, moist, very stiff; COLLUVIUM
	68 - 111	ORANGE-YELLOW-BROWN SANDY CLAY, occasional rock fragments, moist to dry, very stiff; COLLUVIUM
	111 - 120	MOTTLED RED-BROWN GRAVELLY SAND, minor clayey areas, abundant rock fragments, loose pockets of material, dry; OLD TOPSOIL (?)
	120 - 129	GRAY SILT, very stiff, dry, may be deeply weathered siltstone; ROCK

Test Pit	Depth (inches)	Description
	129 - 148	YELLOW BROWN SANDSTONE AND GRAY SHALE, closely fractured, moderately strong, deeply weathered; ROCK
20.	0 - 27	BROWN SILTY SAND, dry, loose; TOPSOIL
	27 - 56	RED-BROWN CLAYEY SAND, with abundant sandstone fragments, moist, moderately stiff; COLLUVIUM
	56 - 87	RED-BROWN SANDY SILT, occasional rock fragments, moist, stiff to very stiff; COLLUVIUM
	87 - 89	GRAY SANDY CLAY, moist, very stiff; COLLUVIUM
	89 - 118	MOTTLED ORANGE-BROWN-GRAY SANDY CLAY, occasional rock fragments, moist, very stiff; COLLUVIUM
	118 - 132	MOTTLED GRAY-YELLOW-BROWN CLAYEY SAND, abundant sandstone and shale fragments, very stiff; RESIDUAL SOIL/DEEPLY WEATHERED ROCK
21.	0 - 12	DARK BROWN SANDY SILT, moist, loose; TOPSOIL
	12 - 57	MOTTLED YELLOW-BROWN-GRAY-RED SANDY CLAY, moderately stiff; moist, with weathered sandstone and greenstone fragments; SLIDE DEBRIS
	57 - 94	MOTTLED BLUE-GREEN-YELLOW-BROWN SANDY CLAY, wet, soft; SLIDE DEBRIS
	94 - 97	RED CLAY, with rounded gravel and larger cobbles, wet, soft; SLIDE DEBRIS

Test Pit	Depth (inches)	Description
	97 - 120	DARK GRAY SANDY CLAY, abundant rock fragments of meta-graywacke and sandstone, wet, moderately stiff; COLLUVIUM/RESIDUAL SOIL (?)
	120 - 130	BROWN GRAVELLY CLAY/SANDY CLAY, with large boulders of meta-graywacke, wet, moderately stiff; RESIDUAL SOIL
	130 - 156	DARK GRAY CLAY, abundant boulders and cobbles of meta-graywacke, moist to wet, stiff, deeply weathered sheared shale melange; DEEPLY WEATHERED ROCK



EXPLANATION

Lithologic Contact

FM Franciscan Melange. A mixture of rock types, including Sandstone, Siltstone, Greenstone, Chert and Sheared Shale

QC Colluvial soils, where relatively deep deposits exist, generally greater than 6 feet deep

QSC Combination of colluvial soils and slide debris, planar contacts present in some pits, others containing weak, soft, potentially unstable material

TP-1A Test pit locations

TP-2 Previous test pit locations

0 50 100 150

Approximate Scale (feet)

GENERAL NOTES:

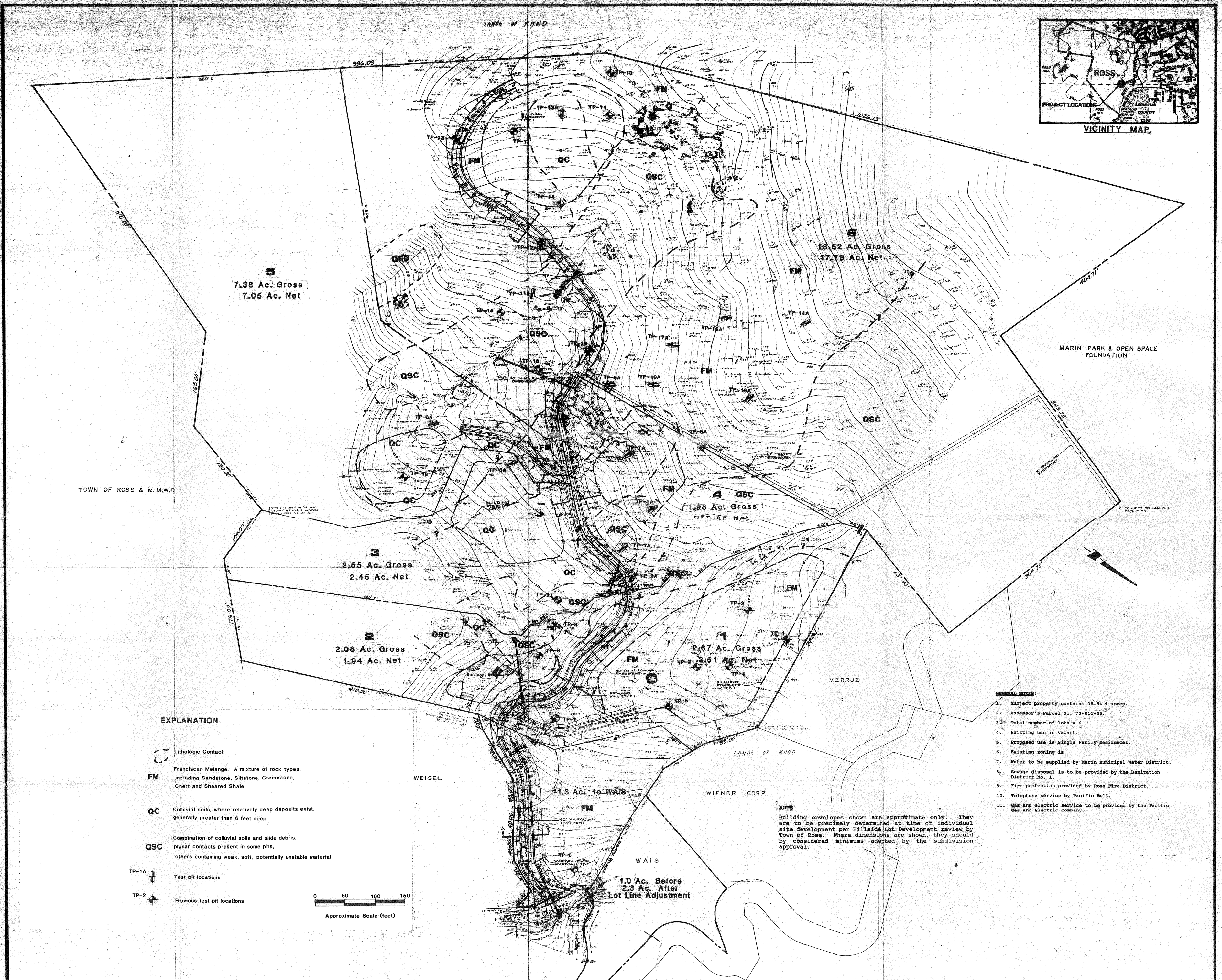
1. Subject property contains 36.54 ± acres.
2. Assessor's Parcel No. 73-011-26.
3. Total number of lots = 6.
4. Existing use is vacant.
5. Proposed use is Single Family Residences.
6. Existing zoning is
7. Water to be supplied by Marin Municipal Water District.
8. Sewage disposal is to be provided by the Sanitation District No. 1.
9. Fire protection provided by Ross Fire District.
10. Telephone service by Pacific Bell.
11. Gas and electric service to be provided by the Pacific Gas and Electric Company.

NOTE

Building envelopes shown are approximate only. They are to be precisely determined at time of individual site development per Hillside Lot Development review by Town of Ross. Where dimensions are shown, they should by considered minimums adopted by the subdivision approval.

TP locations

DRAWN BY: J.N.	DONALD HERZOG & ASSOCIATES, INC. GEOTECHNICAL CONSULTANTS	PREPARED UNDER THE DIRECTION OF: DATE Oct.1989	GEOLOGY MAP TOIGO PROPERTY SUBDIVISION MARIN COUNTY, CALIFORNIA	PLATE 1
DESIGNED BY:				JOB NO. 1385.2.0.1
CHECKED BY:				
SCALE: 1"=50'				
DATE:				



EXPLANATION

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REVISION BLK.				
NUMBER	DATE	DESCRIPTION	BY	
1				
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DRAWN BY: J.N.
DESIGNED BY:
CHECKED BY:
SCALE: 1" = 50'
DATE:
F.B. REF.

DONALD
HERZOG
& ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

PREPARED UNDER THE DIRECTION OF:

DATE Oct.1989

GEOLOGY MAP
TOIGO PROPERTY SUBDIVISION
MARIN COUNTY, CALIFORNIA

PLATE
1

JOB NO. 1385.2.0.1

May 4, 1983

1228.4

Dr. John Mudd
8 Springs Road
Kentfield, Ca. 94904

Dear Dr. Mudd:

Geotechnical Review
Subdivision Feasibility
Upper Road, Ross

This presents the results of our geotechnical review and reconnaissance for a proposed subdivision of 6.6 acres in Ross, California. The purpose of our work was to evaluate the project feasibility from a geotechnical standpoint. Our report is intended to satisfy the requirements of the Application for Informal Subdivison of Three Lots or Less, Section 10 202.5 of the Ross Municipal Code, specifically Note 3; Hillside Lot Application, Section 10 110.02f of the Ross Municipal Code. The proposed subdivision is shown on a topographic map of the area prepared in 1982 by J. Grippi and Associates. As shown, Parcel 1 contains an existing residence, and Parcels 2 (2.5 acres) and 3 (2.0 acres) are proposed for residential construction. It is our understanding that further development of Parcel 1 is not planned.

WORK PERFORMED

A substantial amount of prior work has been performed throughout the area. On June 9, 1981, we prepared a report that discussed the general conditions of the area and site suitability. Our Certified Engineering Geologist performed a reconnaissance of the area. The subsurface conditions at 6 locales were explored with a portable power auger.

On October 8, 1982 we prepared a report addressing the geotechnical feasibility of two lots within the area. One of these lots has subsequently been included in a proposed offer of dedication to the Town of Ross for Open Space. The other lot that was discussed corresponds to Parcel 2 of the proposed land division.

Additional work in the area that has been used for this review includes our subsurface investigation of a proposed lot site on the Roth property; adjacent to and on the same ridge spur as the southwest side of Parcel 3.

On April 29, 1983, our Certified Engineering Geologist performed a brief reconnaissance of the sites to determine if the stability characteristics of the slopes had changed during the winter of 1982-1983.

SITE CONDITIONS

The parcels are situated on the eastern side of Bald Hill, and upslope (west) of Upper Road in Ross. Two major southeast trending ridge spurs are separated by a steep-sided drainage gulley that extends through Parcel 3. Runoff in this gulley is seasonally intermittant.

A minor drainage swale that shows minimal evidence of concentrated flow is present within the north-central portion of Parcels 1 and 2.

A paved driveway extends up the northern ridge spur to the existing house. An unpaved access road extends up from the house to a water tank near the northwest corner of the Mitchell property. The driveway, building pad, and tank site were all developed by cutting on the uphill sides and placing fill on the downhill sides. The driveway cuts vary from nearly vertical to about one horizontal to one vertical (1:1). Although these cuts are much steeper than generally recommended today, they are performing satisfactorily and have only experienced very minor sloughing. The fill bank on the downhill side of the driveway slopes at about 1:1, which is much steeper than the 2:1 generally recommended. However, the driveway has only experienced very minor longitudinal cracking along the cut-fill line, and the fill is apparently performing satisfactorily.

The area is densely covered with hardwood trees and brush. The ground surface is covered with abundant organic debris, and areas of bare soil are rare, except along road cuts. All sites appear well drained, and there was no evidence of excessive surface erosion, or ponded water.

The geology and slope stability have been mapped previously by state and federal agencies (Smith et.al., 1976; Blake et. al., 1974; Wentworth and Frizzell, 1975). The area has previously been mapped as being within a massive, complex landslide deposit.

Bedrock exposures are common along the private driveway and dirt road. Bedrock consists of several types of sandstone, with minor amounts of interbedded siltstone and shale. Data from our subsurface exploration indicates that bedrock conditions (lithology and depth to rock) vary markedly throughout the area. Generally, the rock is blanketed by only a few inches of top soil and colluvium. Within the drainage swales, the colluvial soil deposits are deeper. Data from the subsurface investigation indicate topsoil horizons from 1/2 to 3 feet thick and colluvial soils from 2-1/2 to over 6 feet thick.

The fill, topsoil, and colluvium are porous sandy silts and clays, and clayey sands, which become weak and compressible when wet. The bedrock is relatively firm and incompressible. No free water was encountered in the test borings. However, ground water conditions vary with rainfall, and subsurface seepage may occur. Areas of extensive soil cracking, indicative of expansive soil, were not evident.

There did not appear to be any major slope failures within the potential building sites, or in areas upslope from the sites. Existing slide deposits appear to be meta-stable features. There was no evidence of recent slope failure that could be attributed to the intense winter storms of 1981-1982, or 1982-1983.

Small landslides were observed on the slope above the turn-around area; in the southern swale near the western property line; in the cut bank of an unimproved road at about elevation 270 (MSL); and in the northern swale. The preliminary geologic map of the Upper Ross Valley (Smith et al, 1976) indicates a large block landslide in the area. We discussed this with Mr. Rice and Mr. Smith, and were informed that the mapping was predominantly based on aerial photo interpretation with perhaps minor field reconnaissance.

During our work at the site we observed slope steepening due to differential erosion of different rock types in the area indicated as the landslide scarp. We also observed rubble in the Upper Road cut bank below the house immediately north of the driveway entrance. This rubble is from debris flows on either side of the bedrock nose. We observed relatively uniform strike and dip in numerous cut banks throughout the site, indicating that the bedrock at the site is intact.

There were no land forms within the area that would indicate the presence of active faults. Offsets within subsurface bedrock and soil layers were not evident within the test pits. The site is not within any current Alquist-Priolo Special Studies Zone.

The site is within the California Coast Range Province, which is known to be a region of high seismicity. The nearest known active fault traces (Jennings, 1975) are 7-1/2 miles to the west (San Andreas Fault) and 12 miles to the east (Hayward Fault). Maximum predicted earthquake magnitudes (Richter Scale) for the San Andreas and Hayward Faults are 8.3 and 7.0, respectively.

CONCLUSIONS

Based upon the results of our investigation, we judge that development of the proposed lots is feasbile from a geotechnical standpoint. A well designed and engineered development would enhance the stability of the site, and locally improve surface and subsurface drainage.

The soils which blanket most of the slopes are relatively weak and compressible; experience slow downhill creep (on the order of a small fraction of an inch per year) as is typical of hillsides in Marin County; and are unsuitable for suppport of structures or fills. It will be necessary to construct fills on level keyways and benches founded in firm material beneath the soil. Where water is concentrated in swales, it will be necessary to drain the colluvium with subdrains. In some roadway areas, it may be necessary to reconstruct about the outer 8 feet of cut banks as drained compacted earth buttresses to support the upslope weak soil.

There are no known active faults within the site, and the potential for surface rupture is considered low. The maximum peak bedrock acceleration and repeatable ground acceleration anticipated are 0.6g and 0.4g respectively (Hays, 1980;

Ploessel and Slosson, 1974). Assuming a causative earthquake of 8.3 magnitude on the San Andreas Fault, we believe a site period of 0.5 seconds is applicable to the site. These values are within the typical range for Marin Country hillsides.

Deep colluvial soil deposits, up to 8 feet thick, are expected within Parcel 3. Although these deposits are inherently weak we judge that potential hazards may be mitigated with proper grading and/or structural design. Construction activities are not expected to cause deep-seated earth failures. The type of failures that might be expected would consist of shallow-seated slumps and flows within the upper few feet of surficial soil, and these can be mitigated.

The areas of landslide deposits do not appear to be as extensive as previously mapped. The ridges consist of competent bedrock at a relatively shallow depth. Benched topographic features appear related to compositional changes or structural contacts between bedrock units, rather than to large scale landsliding.

The potential building sites for Parcels 2 and 3 are located within relatively stable areas. The depth to competent bedrock for both sites appears to be from 2 to 8 feet below existing ground. We judge that the rock within both site locales will provide adequate support for typical residential structures. In areas of deep soil, drilled pier foundations should be used. In areas of shallow bedrock, conventional spread footings may be appropriate.

Because there are areas of steep terrain underlain by deep, inherently weak soil, the threat of debris slide activity must be regarded as a potential hazard. However, the potential building sites are not within the path of any apparent landslide deposits, and there are no indications that existing slides are enlarging or encroaching upon the sites. The sites are situated on ridge crests, and the threat of upslope debris slides impacting these areas is considered low.

Springs may be encountered during construction. Seepage can reduce slope stability, and can adversely affect pavement and foundation performance. In the improvement areas, it will be necessary to intercept seepage with subsurface drainage facilities.

Portions of the road will cross areas where there is a potential for upslope or downslope failures. Special design and mitigation measures within these areas are adviseable, and are feasible from a geotechnical standpoint. Possible mitigation measures include replacing the weak soil beneath and immediately upslope of the roads as a compacted buttress; supporting the road and the area immediately upslope with crib walls; or constructing the road on a side hill bridge designed to resist a creep force and to allow passage of potential slide debris.

Constructing roads on steep hillsides will require retaining structures, and may require side-hill bridges. Retaining structure foundations should be supported in firm rock and designed to resist lateral forces caused by downhill creep of the soils above the rock. Retaining structure foundations on the downhill side of roads would be much deeper than those on the uphill sides of roads, and therefore, it would be more economical to design roads with retaining walls on the upslope sides.

Retaining walls and/or side hill bridges may be supported on drilled, cast-in-place, reinforced concrete piers. Piers can be installed through the natural soils, and should be designed to resist downhill creep of the soil above the rock.

Roadways should tolerate anticipated minor creep with no more cracking than is typical for Marin County roadways. It will be necessary to construct utilities with flexible pipe, or to provide frequent joints to accommodate minor creep movement.

Compacted fill generally settles about one percent of its thickness. Roadway grades and utilities should be designed to accommodate this settlement.

Existing site conditions indicate the area is well drained. Problems associated with excessive surface erosion or ponding are not evident. Construction and grading will expose areas of deep, weak soil and slide debris, which may be sensitive to erosion and/or slope failure. Erosion protection measures during and after construction should be utilized to reduce the risk of induced instability. Excavation and construction should be performed during the summer months.

RECOMMENDATIONS

Grading and Buttressing

Areas to be developed should be cleared of vegetation and of the upper few inches of soil containing organic matter. The strippings should be removed or stockpiled for reuse as topsoil. Excavation should then be performed as necessary. We anticipate that with the exception of organic matter and of rocks or lumps larger than six inches in diameter, the excavated material will be suitable for reuse as compacted fill. Organic matter should be disposed of off of the site. Larger material should be disposed of outside of improvement areas. Areas to receive fill should be prepared by cutting level keyways extending into rock. The outboard edge of the keyway excavation should intercept a 1:1 line projected down from the toe of the planned fill.

Subsurface drainage facilities should be installed at the rear of keyways as recommended by the Soil Engineer. The depth and extent of keyways and subdrains should be determined and approved by the Soil Engineer in the field during construction.

Where slope stabilization measures are needed, excavating the landslides during site grading operations, or reconstructing the landslides as compacted earth buttresses with subsurface drainage facilities, offer the greatest reduction of risk. Excavating the landslides consists of removing all the landslide debris, and exposing a relatively flat slope in firm material beneath the slide plane. Buttressing consists of excavating the slide debris; cutting wide level keyways into firm underlying rock; installing subsurface drainage facilities; and backfilling the excavation with compacted fill.

Cut and fill banks generally should be no steeper than two horizontal to one vertical (2:1). Where sloughing is acceptable, cut banks in rock may be 1-1/2:1. Cuts in weak soils or slide debris should be protected, retained or rebuilt. Where the finished slope must be steeper, or will not "catch", it will be necessary to use crib walls, bin walls, reinforced earth, or other retaining stuctures. These retaining structures must be founded on firm rock beneath the zone of weakness, or else on compacted fill founded on firm rock beneath the weak soils.

Crib Walls

Crib walls may be used to support fills, cuts or landslides. Crib walls should be of reinforced concrete construction and should conform to the California Department of Transportation Specifications. Crib walls should be battered at least one foot for every six feet of height. Crib walls should be founded on firm rock or on engineered fill founded on firm rock. The wall toe should be founded at least 18 inches into rock for walls less than 10 feet high; at least 30 inches deep for walls 10 to 15 feet high; and at least 3 feet deep for walls over 15 feet high. The wall bottom should also be deepened as necessary to provide at least 7 feet of horizontal confinement between the toe of the walls and the face of the nearest slope. Subsurface drainage should be provided from the rear of wall foundation excavations.

Reinforced Earth Walls

Reinforced Earth (patented by the Reinforced Earth Company) consists of thin facing panels connected to strips extending into the backfill. The strips are generally galvanized steel about 1/8 inch thick and 2 to 3 inches wide. The strips are located a few feet apart horizontally and vertically, and extend back a distance equivalent to about 80 percent of the wall height. Areas to receive reinforced earth are prepared by excavating a level bench into firm material, as previously described for Concrete Crib Walls. Strips are extended across the bottom of the excavation. A row of facing panels a few feet high is then placed along the outboard edge of the planned fill, and attached to the strips. The panels are then backfilled with granular material. Strips are then attached to the tops of these panels, and extended back across the granular fill. A second row of panels is placed, and compacted granular backfill placed over the top of the strips. Another row of strips is placed, panels installed, and additional backfill placed. The resulting walls are generally more economical than crib walls or reinforced concrete retaining walls for heights greater than about 15 feet. The major disadvantage is that the walls require clean granular backfill in order to bond adequately to the reinforcing strips.

Properly constructed reinforced earth walls can support roadways and can retain landslides. As with retaining walls and crib walls, reinforced earth walls should be fully backdrained.

Utilities

Utilities in weak, unstable areas should be constructed in compacted earth buttresses founded on firm material beneath the weak area, or else should extend into firm rock. Utilities founded in colluvium on slopes will experience differential lateral movement as the soils above the rock slowly creep downslope (on the order of a small fraction of an inch per year). Frequent joints should be provided to accommodate the anticipated creep movement.

Seepage will accumulate in utility trench backfill. Gravity flow outlet pipes should be provided from the bedding material at each low point, and at least every 500 feet to prevent build up of hydrostatic pressure in utility trenches.

Where utility benches are steep enough to erode, check dams should be provided. Where utility trenches are steep enough to erode, check dams and/or rip-rap should be provided.

Pavement

The pavement design should be based upon traffic indices provided by the City of Ross Engineer, and upon R Value tests on representative soils exposed at subgrade level after rough grading operations. All utilities should be installed and properly backfilled prior to subgrade preparation.

Some of the on-site soil may be expansive, and may swell when wet and shrink when dry. The shrinking and swelling will cause pavements to experience edge cracking. Edge cracks must be sealed as they occur. Concrete paving should be reinforced to reduce cracking, and provided with frequent joints to control cracking.

Building Foundations

We anticipate that the portion of building constructed on level areas excavated into firm rock can be supported on continuous and interconnected spread footings. Drilled piers extending into rock will be necessary on and near slopes, and may be used everywhere. It will be necessary to design piers to resist lateral forces caused by downhill creep of soils above the rock.

Soil Engineering Drainage

Surface runoff should be diverted away from cut and fill banks. Subsurface drainage facilities should be installed beneath fills; behind retainig walls; where springs are observed; and in other areas as determined by the Soil Engineer during site grading operations.

Surface water should be diverted away from slopes and weak soil areas by means of concrete-lined interceptor ditches. Surface and subsurface drainage facilities should be maintained entirely separate. Drains should outlet into erosion resistant areas, and should not concentrate water above neighboring property.

Roadways that cross areas of deep, weak soil, or excessibly wet areas, may need subdrains along the inboard (uphill) side. Subdrains should extend below the soil rock contact, and should be sloped to drain by gravity.

SUPPLEMENTAL SERVICES

This is a preliminary investigation for evaluating project feasibility. Additional investigation will be necessary to develop geotechnical criteria for actual design and construction.

LIMITATIONS

We have performed this preliminary investigation in accordance with current standards of engineering practice. We offer no other guarantees or warranties, either expressed or implied.

We trust this provides the information you require at this time. If you have questions, please call.

Yours very truly,

DONALD HERZOG & ASSOCIATES, INC.

Donn Ristau

Donn A. Ristau, PhD.
Senior Staff Geologist
Engineering Geologist - 1155

Donald Herzog,
Principal Engineer
Civil Engineer - 18093

DAR:DH:pbc

3 copies submitted

REFERENCES

Blake, M. C., Jr. and others, 1974, Preliminary Geologic Map of Marin, and San Francisco Counties and Parts of Alameda, Contra Costa and Sonoma Counties, California: U.S. Geological Survey Miscellaneous Field Studies Map MF-574, BDC 64, Scale 1:62,500.

Wentworth, C. M. and Frizzell, V. A., 1975, Reconnaissance Landslide Map of Parts of Marin and Sonoma Counties, California, San Rafael Sheet: U.S. Geological Survey, Scale 1:24,000.

Smith, T. C.; Rice, S. J.; and Strand, R. G., 1976, Geology of the Upper Ross Valley and the Western Part of the San Rafael Area, Marin County, California: California Division of Mines and Geology, OFR 76-2SF, Plate 1B, Scale 1:12,000.

Smith, T. C.; Rice, S. J.; and Strand, R. G., 1976, Relative Stability of the Upland Slopes in the Upper Ross Valley and the Western Part of the San Rafael Area, Marin County, California: California Division of Mines and Geology, OFR 76-2SF, Plate 2B, Scale 1:12,000.

Jennings, C. W., 1975, Fault Map of California With Locations of Volcanoes, Thermal Springs, and Thermal Wells: California Division of Mines and Geology, Geol. Data Map No. 1, Scale 1:750,000.

Hays, W. W., 1980, Procedures for Estimating Earthquake Ground Motions: U. S. Geological Survey Professional Paper 1114, 77 p.

Ploessel, M. R., and Slosson, J. E., 1974, Repeatable High Ground Accelerations from Earthquakes, California Geology, September, pp. 195-199.

September 6, 1983

1228.4

Dr. John Mudd
8 Spring Road
Kentfield, California 94904

Geotechnical Investigation
Driveway Fill
15 Upper Road
Ross, California

This presents the results of our geotechnical investigation of fill for the driveway leading to the residence at 15 Upper Road, Ross.

The purpose of our investigation was to assess the stability of the existing fill, and provide recommendations for pavement design of the proposed new access driveway. The property is presently being considered for a lot split in which two new single-family house sites would be generated. The access driveway would serve the three residences, and under normal use would be subject to car and light truck traffic. Heavy construction traffic would presumably use the driveway during the development of the lots.

The existing driveway shows several areas of extensive cracking and settlement, generally between the outboard edge of the asphalt and the middle of the road. The outboard edge of the asphalt and fill shows no apparent signs of distress.

On August 22, 1983, we collected samples of the fill material below the asphalt and subgrade adjacent to an area of roadway distress. A portion of that material was laboratory tested to determine its R-value. The remaining portion of that material was laboratory tested to define a compaction curve and to establish a maximum dry density and optimum moisture content.

On August 30, 1983, we explored the subsurface conditions of the roadway fill adjacent to areas of distress with four test borings. The holes were located and the drilling was observed by our Certified Engineering Geologist.

Subsurface soil samples were collected from one hole with a 2.43-inch diameter split-barrel sampler. The samples were tested in our laboratory for dry density and moisture content.

Laboratory Test Results

Soil Sample at four inches to one foot below existing asphalt subgrade:

R-value - 42
Maximum Dry Density - 128 pcf
Optimum Moisture Content - 10 percent

Test Boring Samples at one foot and three feet below asphalt level:

Moisture Content - 8.5 percent and 7.1 percent, respectively
Dry Density - 107 and 90 pcf, respectively

The other three holes were drilled as probes to establish the depth of fill.

CONCLUSIONS

Based upon the results of our surface reconnaissance and subsurface exploration, we judge that the existing roadway fill slope is performing satisfactorily. The areas of pavement distress are associated within the location of a utility trench that runs down the road. The cracking and settlement of the asphalt appears to have been caused by differential settlement of the utility trench backfill.

The existing roadway fill along the driveway appears to vary from one to five feet in depth. The material below the fill appears to be a sandy to silty gravel residual soil overlying the sandstone and shale bedrock. The fill was dry in all of the test hole borings. Original topsoil horizons were not evident beneath the fill, and apparently the site was stripped prior to fill placement. Our tests indicated that the fill below the asphalt subgrade has a relative dry density of 70 to 85 percent of its maximum dry density, and is 1-1/2 to 2 percent below optimum moisture content. At the time of our investigation, we did not observe evidence of lateral spreading of the fill slope, or other active slope failure associated with the fill slope.

RECOMMENDATIONS

We recommend that any road widening be performed by excavating into the bank on the uphill side and retaining the excavation with a retaining structure. No additional fill should be placed on the downhill side of the road.

The new pavement edge should be located at least one foot and preferably two feet from the outboard edge of the roadway bench. The existing utility trench should be moisture-conditioned and compacted to at least 90 percent relative compaction per the ASTM D-1557-70(C) laboratory compaction test procedure. The roadway subgrade should then be prepared by scarifying to a depth of six inches, moisture-conditioning as necessary, and compacting to at least 95 percent relative compaction. The finished subgrade should be smooth and nonyielding.

Utilizing a resistance value of 40 and a traffic index of 4.5, the new roadway would need to be constructed with two inches of asphaltic concrete over five inches of aggregate base rock. The required new roadway would even be less because of the presence of existing high R-value base rock. However, in order to be conservative and to accommodate construction traffic, we recommend that the roadway be constructed with three inches of asphaltic concrete over six inches of aggregate base rock.

Class II aggregate base rock should be spread, moisture-conditioned as necessary, and compacted to at least 95 percent relative compaction. The base rock should be smooth and nonyielding. The asphaltic concrete should then be placed and compacted. The work should conform to the requirements of the City of Ross and to the current edition of the California Standard Specifications.

We should be notified to provide observation and field and laboratory density testing to ascertain that the work is being performed in accordance with the intent of our recommendations. We should observe the conditions encountered and modify our recommendations as necessary.

LIMITATIONS

Our work is performed in accordance with generally accepted standards of engineering practice. We offer no other guarantees or warranties, either expressed or implied.

We trust this provides the information you require at this time. If you have questions or wish to discuss this further, please call.

Yours very truly,

DONALD HERZOG & ASSOCIATES, INC.

Donn Ristau

Donn A. Ristau, Ph.D.
Senior Staff Geologist
Engineering Geologist - 1155

Donald Herzog,
Principal Engineer
Civil Engineer - 18093

DAR:DH:cal

Six copies submitted

DONALD
HERZOG
& ASSOCIATES, INC.
GEOTECHNICAL CONSULTANTS

Donald Herzog
John Hom
Jere Giblin

August 10, 1983

1228.4

Town of Ross
Civic Center
Ross, California 94957

Attention: Jorgen Lunding
Director of Public Works

Gentlemen:

Mudd Properties
Upper Road
Ross, California

This letter is in response to two letters; one dated July 27, 1983, from Ted Smith of the California Division of Mines and Geology (CDMG) and another from the Town of Ross dated May 27, 1983, regarding the Mudd property along Upper Road in Ross, California.

We previously submitted a geotechnical report dated May 4, 1983. The scope of our May 4, 1983, report initially was to perform a reconnaissance study of the area and assess whether the proposed lot split was feasible. Our reconnaissance was intended only to address overall site suitability. Detailed site investigations and mapping were recommended by us, when actual building layouts, locations, and design are determined. This scope of work is consistent, if not beyond, the level of effort we have performed on similar projects in the Town of Ross which have been accepted by the Town prior to establishment of the new ordinance.

We subsequently sent the report to CDMG for review with respect to the question of Stability Zonation. Our intent was to establish: 1) whether Ted Smith or Salem Rice had provided the mapping in this area, and 2) if the geologist had actually walked through the site in question and made a field evaluation, or whether the data presented for that area on the 1976 Geology and Slope Stability maps had been developed from air photo interpretation.

Main Office ☐
275 Miller Avenue
Mill Valley, California 94941
(415) 383-7740

Branch Office ☐
3060 Cleveland Avenue
Santa Rosa, California 95401
(707) 523-3880

Soil Engineering, Engineering Geology
and Laboratory Testing for Buildings,
Dams, Landfills, Bridges and Roads

The report was not submitted to CDMG for a detailed analysis, as was implied, simply because the State has no specific involvement in the review process. For this reason, the topographic map delineating the locations of our subsurface exploration, and potential building sites, was not submitted along with the report. Our inquiry into this matter was made in order to develop background information that would allow us to address the new requirements of Ordinance 440 with respect to the Stabilty Zonation. The new ordinance (#440) of Section 18.39.030 of the Ross Municipal Code, now requires a geotechnical review for property in areas designated Zones 3 or 4 on the Slope Stability map. Because the subject parcel lies in a Zone 3, we were was interested in the level of investigation that was used by the state to make that determination.

As was pointed out in a letter from the Town of Ross (May 27, 1983), a paradox exists in that actual residences are not being proposed for development, but the level of investigation required for approval of the lot split is more detailed and site-specific than the initial reconnaissance-level study.

In response to the May 27, 1983, letter, and to the CDMG letter which was sent, unsolicited, to the Town of Ross, we offer the following information.

According to a conversation between Don Herzog and both Salem Rice and Ted Smith in 1981, we were informed that the mapping (within the subject area) was predominantly based on aerial photo interpretation. The statement in our May 4, 1983, report, reflects these comments. This statement is not intended as a commentary on the mapping project as a whole. As was pointed out in the CDMG (July 27, 1983) letter, streets were walked but backyards were not field-checked and this mapping was not intended to be site-specific.

Most of the requirements presented in Chapter 18.39.03 subsection A(3), and the newly added Ordinance 440 subsection A(6) Section 2 of the Ross Municipal Code, have been thoroughly addressed in our report (see enclosure). Our conclusions that development of the proposed lots is feasible from a geotechnical standpoint and that the potential building sites (for both parcels) are located within relatively stable areas indicates that there are buildable sites within each parcel. The overall site stability has been studied by three different Certified Engineering Geologists during the last two and a half years, and each concluded the sites were stable. The actual locations of the proposed potential building envelopes were field-checked during our latest study. This field work plus the previous subsurface investigations leads us to conclude that the potential sites, as shown on the Site Layout Plan, Proposed Land Division dated June 30, 1983, are suitable and can be safely developed. In fact, the geologic conditions throughout both sites are such that the building envelopes could be shifted to other locations and still be practical and safe for development.

Because actual building plans for a specific design have not been developed, it is difficult to assess potential construction impacts on the area. Obviously, a residence designed for foundations with conventional spread footings, large, level landscaped areas, and retaining walls would require cuts and fills and more slope disturbance than would a residence founded on drilled piers with supported decks and limited yards and/or patios. While both types of design potentially could be constructed safely, the costs associated with the development of the former could prove to be very expensive. The most practical method of development would probably involve drilled-pier foundation systems with limited cuts and fills for landscaping. In this case, the potential impact in terms of erosion and induced slope instability would be minimal and should not pose a constraint to construction.

The one area that apparently needed further discussion, as a result of the new Ordinance deals with the Stability Zone Evaluation.

It is our opinion that our on-site reconnaissance and subsurface exploration, in conjunctiuon with the reconnaissance work on adjoining properties, supply a broad information base upon which to evaluate the site stability. As Smith pointed out, the feature he interpreted as a degraded scarp of a large block landslide within the area could have been caused by differential erosion. He also concluded that, based on his evalution of the area, there did not appear to be any immediate threat to the site and that the (interpreted) landslide debris appears relatively stable. Based upon our work and Smith's comments, it is our opinion that the site stability is good and that potential residences could be safely developed.

I trust this clarifies any confusion that may have arisen regarding the status of the property. If you have any questions relating to this matter, please call.

Yours very truly,

DONALD HERZOG & ASSOCIATES, INC.

Donn Ristau

Donn A. Ristau, Ph.D.
Senior Staff Geologist
Engineering Geologist - 1155

DAR:pbc

Three copies submitted

Copy to: Roy Hoffman, City Engineer

Dr. John Mudd
8 Spring Road
Kentfield, CA 94904

ENCLOSURE

The following attachment delineates various sections in our May 4, 1983 report that deals with the issues outlined in Ordinance 440, Section 2, amendment to subsection (A)(6).

(6) see:	Page (p.) 4; paragraph (par.) 4 p. 5; par. 2 p. 5; par. 4 p. 6; par. 1-4
6(a) see:	p. 3; par. 5 & 6 p. 4; par. 6 p. 5; par. 3
6(b) see:	p. 2; par. 4 & 5 p. 5; par. 5
6(c) see:	p. 3; par. 1
6(d) see:	p. 6; par. 6
6(e) see:	p. 3; par. 4 p. 5; par. 6 p. 6; par. 4
6(f) see:	p. 4; par. 3, 4 & 6 p. 5; par. 1

October 8, 1982

1228.2

Dr. John Mudd
8 Spring Road
Kentfield, California 94904

Dear Dr. Mudd:

Geotechnical Feasibility
Investigation
Mudd Property
Ross, California

This report presents the results of our geotechnical investigation of two proposed building sites within the Mudd Property in Ross. The purpose of the investigation was to assess the stability and the geotechnical feasibility of developing the proposed sites, as shown on a schematic layout dated September 1982 by CHNMB. The results of the investigation were used to develop the following conclusions and recommendations:

1. A description of the surface and subsurface soil and rock conditions observed.

2. An evaluation of potential geologic hazards and mitigation measures.

3. Development feasibility.

4. General grading and design recommendations.

This investigation is intended to satisfy the minimal geotechnical requirements for the Tentative Map stage of development. Specific issues relating to Section 18.39.030, Ordinance 435 of the Ross Municipal Code are addressed.

WORK PERFORMED

The site was initially inspected by our Principal Engineer and Engineering Geologist. A subsequent inspection by the property owner and our Registered Geologist was made on September 10, 1982, and a subsurface exploration program was established. Prior to the field exploration, we reviewed our previous work within the property, as well as selected geotechnical references pertinent to the area (Smith et. al., 1976; Blake et.al., 1974; Wentworth and Frizzell, 1975).

Subsurface conditions were delineated within the upper site (Parcel 3) with four backhoe-dug test pits, and from soil and rock exposures in a road cut along the eastern margin of the property. The pits were located and logged in the field by our Registered Geologist. Test Pit logs are presented in the following section. The location of the test pits, as well as bedrock type and the depth to competent rock, are shown on Plate 1. Soil and rock descriptions are based on an in situ examination of the material.

Soil and rock conditions within the lower lot (Parcel 2) were determined from a test boring drilled previously, and from road cut exposures above and below the building site. Limited access restricted the subsurface exploration within this area.

SITE CONDITIONS

The two proposed lot sites are on the eastern side of Bald Hill; on an east-facing ridge. Drainage ravines skirt the southwest margins of both parcels.

The sites presently are undeveloped, but are separated by an existing single-family residence. Access to this residence is via a paved private driveway, which crosses through the lower parcel. A dirt road extends from the residence to a water storage tank several hundred feet upslope. Access to the upper parcel will, in part, utilize the existing dirt road.

The slopes along the ridge crest are generally moderately steep (2 horizontal to 1 vertical - 2:1) to steep (1-1/2:1). The drainage swales are shallow and apparently do not carry large volumes of run-off. Both lots are densely covered with hardwood trees and brush. The ground surface is covered with abundant organic debris, and areas of bare soil are rare, except along road cuts. Both sites appear well drained, and there was no evidence of excessive surface erosion.

The geology and slope stability have been mapped previously by state and federal agencies (Smith et. aL, 1976; Blake et. aL, 1974; Wentworth and Frizzell, 1975). The area has previously been mapped as being within a massive, complex landslide deposit.

Bedrock exposures are common along the private driveway and dirt road. Bedrock consists of several types of sandstone, with minor amounts of interbedded siltstone and shale. Data from our subsurface exploration indicates that bedrock conditions (lithology and depth to rock) vary markedly throughout the area. Rocks within the upper parcel include strong graywacke sandstone, weak sandstone, siltstone, and shale. Rocks within the lower parcel include moderately strong sandstone and weak shale. The depth to rock within the proposed building sites is expected to range from 1 to 4 feet.

The soils throughout the area also are variable in extent and composition. Organic-rich topsoil horizons ranged from 5 to 10 inches thick, and generally consisted of dry and loose compressible sands and silts with varying amounts of rock fragments. Areas of extensive soil cracking, indicative of expansive soil, were not evident.

Deep colluvial soils of dry, stiff clayey sand were encountered in several areas. Generally these soils appeared well consolidated, slightly compressible, and nonexpansive.

Landslide deposits contained material that ranged from moist and very stiff gravelly clays, to wet and moderately stiff expansive clays. Expansive soils undergo volumetric changes with changes in moisture content.

Ground water was not encountered in any of the Test Pits or Test Boring. However, shallow subsurface ground water within soils above the bedrock contact may be expected during wetter months.

There did not appear to be any major slope failures within the proposed building sites, or in areas upslope from the sites. Existing slide deposits appear to be meta-stable features. There was no evidence of recent slope failure that could be attributed to the intense winter storms of 1981-1982.

There were no land forms within the area that would indicate the presence of active faults. Offsets within subsurface bedrock and soil layers were not evident within any of the test pits. The site is not within any current Alquist-Priolo Special Studies Zone.

The site is within the California Coast Range Province, which is known to be a region of high seismicity. The nearest known active fault traces (Jennings, 1975), are 7-1/2 miles to the west (San Andreas Fault) and 12 miles to the east (Hayward Fault). Maximum predicted earthquake magnitudes (Richter Scale) for the San Andreas and Hayward Faults are 8.3 and 7.0, respectively.

LOGS OF TEST PITS

Test Pit #	Depth (inches)	Description
1.	0 - 9	BROWN SANDY SILT, dry, loose; TOPSOIL
	9 33	BROWN CLAYEY SAND, minor rock fragments, moderately stiff, dry; COLLUVIUM
	33 - 98	MOTTLED, RED-BROWN CLAYEY SAND, abundant rock fragments, very stiff, dry; COLLUVIUM
	98 - 111	YELLOW-BROWN SILTSTONE, closely fractured, deeply weathered, weak, dry; ROCK
2.	0 - 10	BROWN SAND, loose, dry; TOPSOIL

Test Pit #	Depth (inches)	Description
	10 - 42	YELLOW-BROWN SANDSTONE WITH SAND, dry; DEEPLY WEATHERED ROCK
	42 - 74	YELLOW-BROWN SANDSTONE, intensely fractured, friable to weak, dry, deeply weathered; ROCK
3.	0 - 9	BROWN SILTY SAND, minor rock fragments, loose, dry; TOPSOIL
	9 - 30	BROWN SAND, moderate rock fragments, loose, dry; RESIDUAL SOIL
	30 - 40	INTERBEDDED GRAY-BROWN SANDSTONE, GRAY CLAYSTONE, BROWN SILTSTONE, intensely to closely fractured, weak to moderately strong, deeply weathered; ROCK
4.	0 - 5	DARK BROWN SANDY CLAY, loose, dry; TOPSOIL
	5 - 60	MOTTLED BROWN GRAVELLY CLAY, stiff, moist, abundant rock fragments and occasional boulders; SLIDE DEBRIS
	60 - 120	GRAY-BROWN SANDY CLAY, some rock fragments, very stiff, moist; SLIDE DEBRIS

Test Pit #	Depth (inches)	Description
	120-144	MOTTLED BROWN-WHITE CLAY, very stiff, moist to wet; SLIDE DEBRIS
	144-150	MOTTLED ORANGE-BROWN CLAY, stiff, wet; SLIDE DEBRIS
	150-168	MOTTLED GRAY-BLUE-GREEN CLAY, around shale fragments, moist, stiff; RESIDUAL SOIL
	168-172	MOTTLED YELLOW-BROWN SANDSTONE AND SILTSTONE, gray clay seams, intensely fractured, deeply weathered, weak to moderately strong; ROCK

CONCLUSIONS

Based upon the results of our investigation, we judge that development of the proposed lots is feasible from a goetechnical standpoint. A well designed and engineered development would enchance the stability of the site, and locally improve surface and subsurface drainage.

The soils which blanket most of the slopes are relatively weak and compressible; experience slow downhill creep (on the order of a small fraction of an inch per year) as is typical of hillsides in Marin County; and are unsuitable for support of structures or fills. It will be necessary to construct fills on level keyways and benches founded in firm material beneath the soil. Where water is concentrated in swales, it will be necessary to drain the colluvium with subdrains. In some roadway areas, it will probably be necessary to reconstruct about the outer 8 feet of cut banks as drained compacted earth buttresses to support the upslope weak soil.

There are no known active faults within the site, and the potential for surface rupture is considered low. The maximum peak bedrock acceleration and repeatable ground acceleration anticipated are 0.6g and 0.4g respectivelv (Hays, 1980; Ploessel and Slosson, 1974). Assuming a causative earthquake of 8.3 magnitude on the San Andreas Fault, we believe a site period of 0.3 seconds is applicable to the site. These values are within the typical range for Marin County hillsides.

A massive debris flow deposit is present within the southwest portion of the upper parcel. Where explored, the depth to competent rock was as much as 14 feet below ground surface. Deep colluvial soil deposits, up to 8 feet thick, were also found within both parcels. Although these deposits are inherently weak and potentially unstable, we judge that hazards may be mitigated with proper grading and/or structural design. Construction activities are not expected to cause deep-seated reactivation of the large debris flow deposits. The type of failures that might be expected would consist of shallow-seated slumps and flows within the upper few feet of surfical soil, and these can be mitigated.

The areas of landslide deposits do not appear to be as extensive as previously mapped. The ridges consist of competent bedrock at a relatively shallow depth. Benched topographic features appear related to compositional changes or structural contacts between bedrock units, rather than to large scale landsliding.

The two proposed building sites are located within relatively stable areas. The depth to competent bedrock for both sites appears to be shallow (less than 4 feet). We judge that the rock within both site locales will provide adequate support for typical residential structures. In areas of deep soil, drilled pier foundations should be used. In areas of shallow bedrock, conventional spread footings may be appropriate.

Because there are areas of steep terrain underlain by deep, inherently weak soil, the threat of debris slide activity must be regarded as a potential hazard. However, the proposed building envelopes are not within the path of any apparent landslide deposits, and there are no indications that the existing slides are enlarging or encroaching upon the sites. The sites are situated on ridge crests, and the threat of upslope debris slides impacting these areas is considered low.

Several areas indicative of springs or surface seepage were observed within the southwest portion of the upper parcel. Other springs may be encountered during construction. Seepage can reduce slope stability, and can adversely affect pavement and foundation performance. In the improvement areas, it will be necessary to intercept seepage with subsurface drainage facilities.

The proposed access road alignment for the upper parcel will cross areas of deep soil and old slide debris. Portions of the road will cross areas where there is a potential impact from upslope or downslope failures. Special design and mitigation measures within these areas are advisable, and are feasible from a geotechnical standpoint. Possible mitigation measures include replacing the slide debris beneath and immediately upslope of the roads as a compacted buttress; supporting the road and the area immediately upslope with crib walls; or constructing the road on a side hill bridge designed to resist a creep force and to allow passage of slide debris.

Constructing roads on steep hillsides will require retaining structures, and may require side hill bridges. Retaining structure foundations should be supported in firm rock and designed to resist lateral forces caused by downhill creep of the soils above the rock. Retaining structure foundations on the downhill side of roads will be much deeper than those on the uphill sides of roads. Therefore, it will be more economical to install retaining walls on the upslope sides of roads.

Retaining walls and/or side hill bridges may be supported on drilled, cast-in-place, reinforced concrete piers. In landslide areas, the landslides should be stabilized as buttresses prior to installing piers, or the piers designed to resist slide forces. In other areas, the piers can be installed through the natural soils, and designed to resist downhill creep of the soil above the rock.

Roadways outside of slide areas, or on reconstructed landslides should tolerate anticipated minor creep with no more cracking than is typical for Marin County roadways. It will be necessary to construct utilities with flexible pipe, or to provide frequent joints to accommodate minor creep movement.

Compacted fill generally settles about one percent of its thickness. Roadway grades and utilities should be designed to accommodate this settlement.

Existing site conditions indicate the area is well drained. Problems associated with excessive surface erosion or ponding are not evident. Construction and grading will expose areas of deep, weak soil and slide debris, which may be sensitive to erosion and/or slope failure. Erosion protection measures during and after construction should be utilized to reduce the risk of induced instability.

RECOMMENDATIONS

Grading and Buttressing

Areas to be developed should be cleared of vegetation and of the upper few inches of soil containing organic matter. The strippings should be removed or stockpiled for reuse as topsoil. Excavation should then be performed as necessary. We anticipate that with the exception of organic matter and of rocks or lumps larger than six inches in diameter, the excavated material will be suitable for reuse as compacted fill. Organic matter should be disposed of off of the site. Larger material should be disposed of outside of improvement areas. Areas to receive fill should be prepared by cutting level keyways extending into rock. The outboard edge of the keyway excavation should intercept a 1:1 line projected down from the toe of the planned fill.

Subsurface drainage facilities should be installed at the rear of keyways as recommended by the Soil Engineer. The depth and extent of keyways and subdrains should be determined and approved by the Soil Engineer in the field during construction.

Where slope stabilization measures are needed, excavating the landslides during site grading operations, or reconstructing the landslides as compacted earth buttresses with subsurface drainage facilities, offer the greatest reduction of risk. Excavating the landslides consists of removing all the landslide debris, and exposing a relatively flat slope in firm material beneath the slide plane. Buttressing consists of excavating the slide debris; cutting wide level keyways into firm underlying rock; installing subsurface drainage facilities; and backfilling the excavation with compacted fill.

Cut and fill banks generally should be no steeper than two horizontal to one vertical (2:1). Where sloughing is acceptable, cut banks in rock may be 1-1/2:1. Cuts in weak soils or slide debris sould be protected, retained or rebuilt. Where the finished slope must be steeper, or will not "catch", it will be necessary to use crib walls, bin walls, reinforced earth, or other retaining structures. These retaining structures must be founded on firm rock beneath the zone of weakness, or else on compacted fill founded on firm rock beneath the weak soils.

As an alternative to repairing the entire landslide areas, only the portion of the landslide affecting the planned improvements could be repaired. Improvements can be protected from downslope landslides by extending a buttress down at 2:1 to firm rock beneath the slide plane, or by supporting the downhill edge of the buttress with a retaining structure founded on firm rock beneath the slide plane. The unrepaired portion(s) of the slide downslope of the buttress may continue to move, but should not adversely affect the buttress.

Improvements can be protected from upslope slides by retaining the slides with a retaining structure, or with a partial earth buttress underlain by subsurface drainage systems. If the upslope is steep enough to allow debris flows to overtop the retaining structure or buttresses, it will be necessary to enhance the stability of the upslope area with subsurface drains, and/or to provide catchment areas for possible slough debris and mudflows.

Crib Walls

Crib walls may be used to support fills, cuts or landslides. Crib walls should be of reinforced concrete construction and should conform to the California Department of Transportation Specifications. Crib walls should be battered at least one foot for every six feet of height. Crib walls should be founded on firm rock or on engineered fill founded on firm rock. The wall toe should be founded at least 18 inches into rock for walls less than 10 feet high; at least 30 inches deep for walls 10 to 15 inches high; and at least 3 feet deep for walls over 15 feet high. The toe of the walls should also be deepened as necessary to provide at least 7 feet of horizontal confinement between the toe of the walls and the face of the nearest slope. Subsurface drainage should be provided from the rear of wall foundation excavations.

Reinforced Earth Walls

Reinforced Earth (patented by the Reinforced Earth Company) consists of thin facing panels connected to strips extending into the backfill. The strips are generally galvanized steel about 1/8 inch thick and 2 to 3 inches wide. The strips are located a few feet apart horizontally and vertically, and extend back a distance equivalent to about 80 percent of the wall height. Areas to receive reinforced earth are prepared by excavating a level bench into firm material, as previously described for Concrete Crib Walls. Strips are extended across the bottom of the excavation. A row of facing panels a few feet high is then placed along the outboard edge of the planned fill, and attached to the strips. The panels are then backfilled with granular material. Strips are then attached to the tops of these panels, and extended back across the granular fill. A second row of panels is placed, and compacted granular backfill placed over the top of the strips. Another row of strips is placed, panels installed, and additional backfill placed. The resulting walls are generally more economical than crib walls or reinforced concrete retaining walls for heights greater than about 15 feet. The major disadvantage is that the walls require clean granular backfill in order to bond adequately to the reinforcing strips.

Properly constructed reinforced earth walls can support roadways and can retain landslides. As with retaining walls and crib walls, reinforced earth walls should be fully backdrained.

Utilities

Utilities in slide areas should be constructed in compacted earth buttresses founded on firm material beneath the slide plane, or else should extend into firm rock beneath the slide plane. Utilities founded in colluvium on slopes will experience differential lateral movement as the soils above the rock slowly creep downslope (on the order of a small fraction of an inch per year). Frequent joints should be provided to accommodate the anticipated creep movement.

Seepage will accumulate in utility trench backfill. Gravity flow outlet pipes should be provided from the bedding material at each low point, and at least every 500 feet to prevent build up of hydrostatic pressure in utility trenches.

Where utility benches are steep enough to erode, check dams should be provided. Where utility trenches are steep enough to erode, check dams and/or rip rap should be provided.

Pavement

The pavement design should be based upon traffic indices provided by the City of Ross Engineer, and upon R Value tests on representative soils exposed at subgrade level after rough grading operations. All utilities should be installed and properly backfilled prior to subgrade preparation.

Some of the on-site soil is expansive, and will swell when wet and shrink when dry. The shrinking and swelling will cause pavements to experience edge cracking. Edge cracks must be sealed as they occur. Concrete paving should be reinforced to reduce cracking, and provided with frequent joints to control cracking.

Building Foundations

We anticipate that the portion of building constructed on level areas excavated into firm soil or rock can be supported on continuous and interconnected spread footings. Drilled piers extending into rock will be necessary on and near slopes, and may be used everywhere. It will be necessary to design piers to resist lateral forces caused by downhill creep of soils above the rock.

Soil Engineering Drainage

Surface runoff should be diverted away from cut and fill banks. Subsurface drainage facilities should be installed beneath fills; behind retaining walls; where springs are observed; and in other areas as determined by the Soil Engineer during site grading operations.

Surface water should be diverted away from slopes and weak soil areas by means of concrete-lined interceptor ditches. Surface and subsurface drainage facilities should be maintained entirely separate. Drains should outlet into erosion resistant areas, and should not concentrate water above neighboring property.

Portions of landslides above buttresses, and unrepaired landslides where reactivation would be detrimental, should be enhanced with surface and subsurface drainage improvements. The subsurface drainage improvements should consist of a subdrain extending down the central axis of the landslide, with laterals extending to each side at about 50 foot intervals. Roadways that cross areas of deep, weak soil, or excessively wet areas, may need subdrains along the inboard (uphill) side. Subdrains should extend below the soil rock contact, and should be sloped to drain by gravity.

SUPPLEMENTAL SERVICES

This is a preliminary investigation for evaluating project feasibility. Additional investigation will be necessary to develop geotechnical design criteria for actual design and construction. We should review the master plan for conformance with the intent of this investigation.

LIMITATIONS

We have performed this preliminary investigation in accordance with current standards of engineering practice. We offer no other guarantees or warranties, either expressed or implied.

We trust this provides the information you require at this time. If you have question, please call.

Yours very truly,

DONALD HERZOG & ASSOCIATES, INC.

Donn Ristau

Donn A. Ristau,
Senior Staff Geologist
Registered Geologist - 3634

Donald Herzog,
Principal Engineer
Civil Engineer - 18093

DAR/DH/gh

4 copies submitted

Attachment: Plate 1

REFERENCES:

Blake, M. C., Jr. and others, 1974, Preliminary Geologic Map of Marin, and San Francisco Counties and Parts of Alameda, Contra Costa and Sonoma Counties, California: U.S. Geological Survey Miscellaneous Field Studies Map MF-574, BDC 64, Scale 1:62,500.

Wentworth, C. M. and Frizzell, V. A., 1975, Reconnaissance Landslide Map of Parts of Marin and Sonoma Counties, California, San Rafael Sheet: U.S. Geological Survey, Scale 1:24,000.

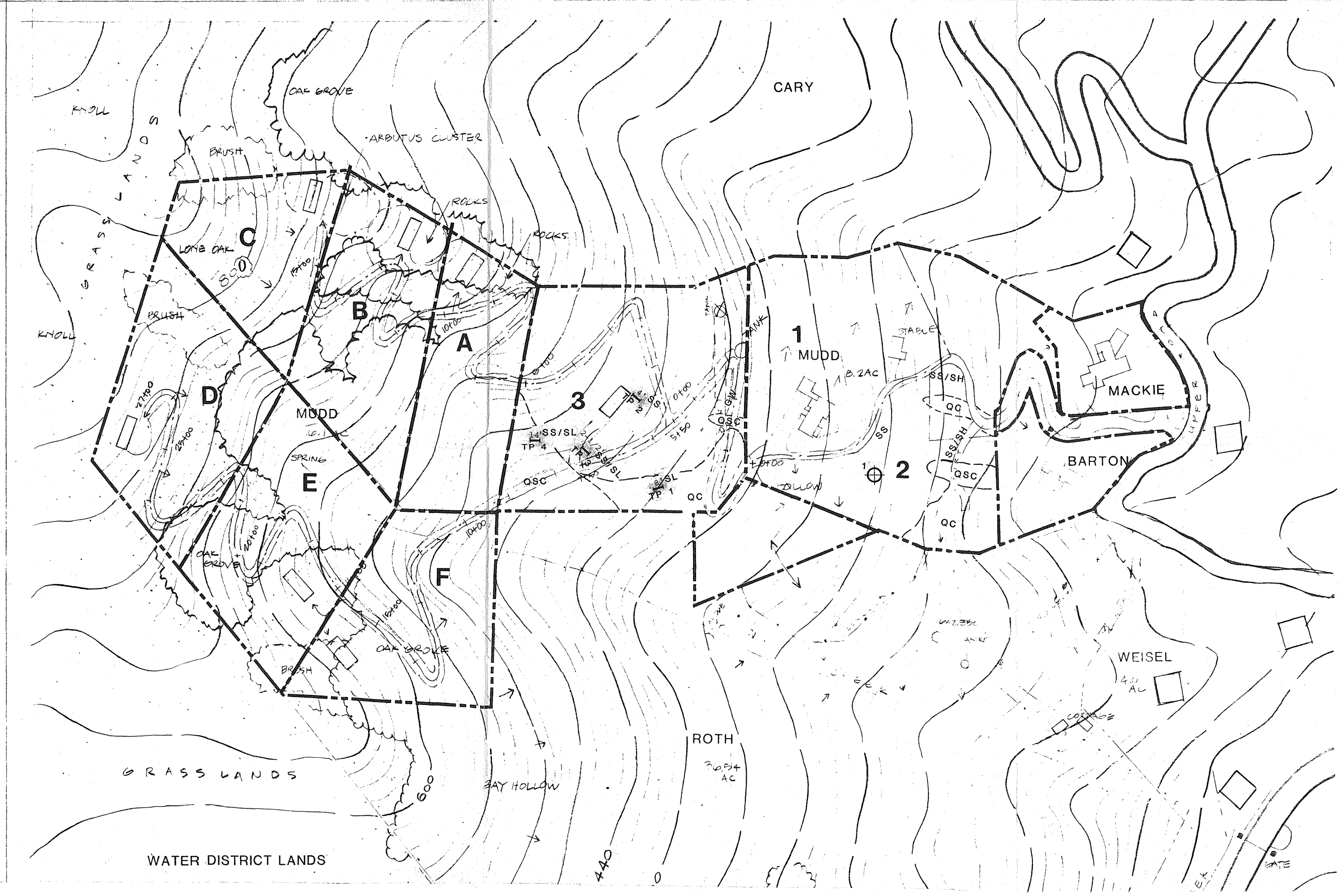
Smith, T. C.; Rice, S. J.; and Strand, R. G., 1976, Geology of the Upper Ross Valley and the Western Part of the San Rafael Area, Marin County, California: California Division of Mines and Geology, OFR 76-2SF, Plate 1B, Scale 1:12,000.

Smith, T. C.; Rice, S. J.; and Strand, R. G., 1976, Relative Stability of the Upland Slopes in the Upper Ross Valley and the Western Part of the San Rafael Area, Marin County, California: California Division of Mines and Geology, OFR 76-2SF, Plate 2B, Scale 1:12,000.

Jennings, C. W., 1975, Fault Map of California With Locations of Volcanoes, Thermal Springs, and Thermal Wells: California Division of Mines and Geology, Geol. Data Map No. 1, Scale 1:750,000.

Hays, W. W., 1980, Procedures for Estimating Earthquake Ground Motions: U. S. Geological Survey Professional Paper 1114, 77 p.

Ploessel, M. R., and Slosson, J. E., 1974, Repeatable High Ground Accelerations from Earthquakes, California Geology, September, pp. 195-199.



EXPLANATION

SS/SL : SANDSTONE and SILTSTONE
SS/SH : SANDSTONE and SHALE
GW : GRAYWACKE SANDSTONE
SS : SANDSTONE
QC : DEEP COLLUVIUM
QSC : SLIDE DEBRIS and COLLUVIUM
ESTIMATED LIMITS of DEEP SOIL

DEPTH TO ROCK
2'
SS/SH
ROCK TYPE
TP 1
TEST PIT #

TEST BORING
DEPTH TO ROCK

DONALD HERZOG & ASSOCIATES
consulting engineers
Job No: 1228.2 Appr: DR Date 10-7-82

Subsurface Exploration
Locations and Soil and Rock
Conditions.
Mudd Properties: Ross, Calif

PLATE
1

MUDD PROPERTY ASSESSMENT

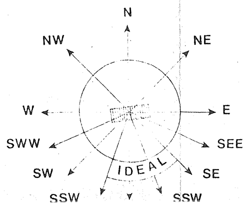
CITY OF ROSS MARIN CALIF

CHNMB
Carter Hull Nishita McCulley Associates
Urban Design • Landscape Architecture • Planning
2789 25th Street, San Francisco, California 94110 (415) 821 3500

15 SEPT 82

1":100'

NORTH



○ DEVELOPMENT POTENTIAL

○ CONSERVATION AREAS

○ OWNED BY OTHERS

October 8, 1982

1228.2

Dr. John Mudd
8 Spring Road
Kentfield, California 94904

Dear Dr. Mudd:

Geotechnical Feasibility
Investigation
Mudd Property
Ross, California

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WORK PERFORMED

The site was initially inspected by our Principal Engineer and Engineering Geologist. A subsequent inspection by the property owner and our Registered Geologist was made on September 10, 1982, and a subsurface exploration program was established. Prior to the field exploration, we reviewed our previous work within the property, as well as selected geotechnical references pertinent to the area (Smith et. al., 1976; Blake et.al., 1974; Wentworth and Frizzell, 1975).

Subsurface conditions were delineated within the upper site (Parcel 3) with four backhoe-dug test pits, and from soil and rock exposures in a road cut along the eastern margin of the property. The pits were located and logged in the field by our Registered Geologist. Test Pit logs are presented in the following section. The location of the test pits, as well as bedrock type and the depth to competent rock, are shown on Plate 1. Soil and rock descriptions are based on an in situ examination of the material.

Soil and rock conditions within the lower lot (Parcel 2) were determined from a test boring drilled previously, and from road cut exposures above and below the building site. Limited access restricted the subsurface exploration within this area.

SITE CONDITIONS

The two proposed lot sites are on the eastern side of Bald Hill; on an east-facing ridge. Drainage ravines skirt the southwest margins of both parcels.

The sites presently are undeveloped, but are separated by an existing single-family residence. Access to this residence is via a paved private driveway, which crosses through the lower parcel. A dirt road extends from the residence to a water storage tank several hundred feet upslope. Access to the upper parcel will, in part, utilize the existing dirt road.

The slopes along the ridge crest are generally moderately steep (2 horizontal to 1 vertical - 2:1) to steep (1-1/2:1). The drainage swales are shallow and apparently do not carry large volumes of run-off. Both lots are densely covered with hardwood trees and brush. The ground surface is covered with abundant organic debris, and areas of bare soil are rare, except along road cuts. Both sites appear well drained, and there was no evidence of excessive surface erosion.

The geology and slope stability have been mapped previously by state and federal agencies (Smith et. aL, 1976; Blake et. aL, 1974; Wentworth and Frizzell, 1975). The area has previously been mapped as being within a massive, complex landslide deposit.

Bedrock exposures are common along the private driveway and dirt road. Bedrock consists of several types of sandstone, with minor amounts of interbedded siltstone and shale. Data from our subsurface exploration indicates that bedrock conditions (lithology and depth to rock) vary markedly throughout the area. Rocks within the upper parcel include strong graywacke sandstone, weak sandstone, siltstone, and shale. Rocks within the lower parcel include moderately strong sandstone and weak shale. The depth to rock within the proposed building sites is expected to range from 1 to 4 feet.

The soils throughout the area also are variable in extent and composition. Organic-rich topsoil horizons ranged from 5 to 10 inches thick, and generally consisted of dry and loose compressible sands and silts with varying amounts of rock fragments. Areas of extensive soil cracking, indicative of expansive soil, were not evident.

Deep colluvial soils of dry, stiff clayey sand were encountered in several areas. Generally these soils appeared well consolidated, slightly compressible, and nonexpansive.

Landslide deposits contained material that ranged from moist and very stiff gravelly clays, to wet and moderately stiff expansive clays. Expansive soils undergo volumetric changes with changes in moisture content.

Ground water was not encountered in any of the Test Pits or Test Boring. However, shallow subsurface ground water within soils above the bedrock contact may be expected during wetter months.

There did not appear to be any major slope failures within the proposed building sites, or in areas upslope from the sites. Existing slide deposits appear to be meta-stable features. There was no evidence of recent slope failure that could be attributed to the intense winter storms of 1981-1982.

There were no land forms within the area that would indicate the presence of active faults. Offsets within subsurface bedrock and soil layers were not evident within any of the test pits. The site is not within any current Alquist-Priolo Special Studies Zone.

The site is within the California Coast Range Province, which is known to be a region of high seismicity. The nearest known active fault traces (Jennings, 1975), are 7-1/2 miles to the west (San Andreas Fault) and 12 miles to the east (Hayward Fault). Maximum predicted earthquake magnitudes (Richter Scale) for the San Andreas and Hayward Faults are 8.3 and 7.0, respectively.

LOGS OF TEST PITS

Test Pit #	Depth (inches)	Description
1.	0 - 9	BROWN SANDY SILT, dry, loose; TOPSOIL
	9 33	BROWN CLAYEY SAND, minor rock fragments, moderately stiff, dry; COLLUVIUM
	33 - 98	MOTTLED, RED-BROWN CLAYEY SAND, abundant rock fragments, very stiff, dry; COLLUVIUM
	98 - 111	YELLOW-BROWN SILTSTONE, closely fractured, deeply weathered, weak, dry; ROCK
2.	0 - 10	BROWN SAND, loose, dry; TOPSOIL

Test Pit #	Depth (inches)	Description
	10 - 42	YELLOW-BROWN SANDSTONE WITH SAND, dry; DEEPLY WEATHERED ROCK
	42 - 74	YELLOW-BROWN SANDSTONE, intensely fractured, friable to weak, dry, deeply weathered; ROCK
3.	0 - 9	BROWN SILTY SAND, minor rock fragments, loose, dry; TOPSOIL
	9 - 30	BROWN SAND, moderate rock fragments, loose, dry; RESIDUAL SOIL
	30 - 40	INTERBEDDED GRAY-BROWN SANDSTONE, GRAY CLAYSTONE, BROWN SILTSTONE, intensely to closely fractured, weak to moderately strong, deeply weathered; ROCK
4.	0 - 5	DARK BROWN SANDY CLAY, loose, dry; TOPSOIL
	5 - 60	MOTTLED BROWN GRAVELLY CLAY, stiff, moist, abundant rock fragments and occasional boulders; SLIDE DEBRIS
	60 - 120	GRAY-BROWN SANDY CLAY, some rock fragments, very stiff, moist; SLIDE DEBRIS

Test Pit #	Depth (inches)	Description
	120-144	MOTTLED BROWN-WHITE CLAY, very stiff, moist to wet; SLIDE DEBRIS
	144-150	MOTTLED ORANGE-BROWN CLAY, stiff, wet; SLIDE DEBRIS
	150-168	MOTTLED GRAY-BLUE-GREEN CLAY, around shale fragments, moist, stiff; RESIDUAL SOIL
	168-172	MOTTLED YELLOW-BROWN SANDSTONE AND SILTSTONE, gray clay seams, intensely fractured, deeply weathered, weak to moderately strong; ROCK

CONCLUSIONS

Based upon the results of our investigation, we judge that development of the proposed lots is feasible from a goetechnical standpoint. A well designed and engineered development would enchance the stability of the site, and locally improve surface and subsurface drainage.

The soils which blanket most of the slopes are relatively weak and compressible; experience slow downhill creep (on the order of a small fraction of an inch per year) as is typical of hillsides in Marin County; and are unsuitable for support of structures or fills. It will be necessary to construct fills on level keyways and benches founded in firm material beneath the soil. Where water is concentrated in swales, it will be necessary to drain the colluvium with subdrains. In some roadway areas, it will probably be necessary to reconstruct about the outer 8 feet of cut banks as drained compacted earth buttresses to support the upslope weak soil.

There are no known active faults within the site, and the potential for surface rupture is considered low. The maximum peak bedrock acceleration and repeatable ground acceleration anticipated are 0.6g and 0.4g respectivelv (Hays, 1980; Ploessel and Slosson, 1974). Assuming a causative earthquake of 8.3 magnitude on the San Andreas Fault, we believe a site period of 0.3 seconds is applicable to the site. These values are within the typical range for Marin County hillsides.

A massive debris flow deposit is present within the southwest portion of the upper parcel. Where explored, the depth to competent rock was as much as 14 feet below ground surface. Deep colluvial soil deposits, up to 8 feet thick, were also found within both parcels. Although these deposits are inherently weak and potentially unstable, we judge that hazards may be mitigated with proper grading and/or structural design. Construction activities are not expected to cause deep-seated reactivation of the large debris flow deposits. The type of failures that might be expected would consist of shallow-seated slumps and flows within the upper few feet of surfical soil, and these can be mitigated.

The areas of landslide deposits do not appear to be as extensive as previously mapped. The ridges consist of competent bedrock at a relatively shallow depth. Benched topographic features appear related to compositional changes or structural contacts between bedrock units, rather than to large scale landsliding.

The two proposed building sites are located within relatively stable areas. The depth to competent bedrock for both sites appears to be shallow (less than 4 feet). We judge that the rock within both site locales will provide adequate support for typical residential structures. In areas of deep soil, drilled pier foundations should be used. In areas of shallow bedrock, conventional spread footings may be appropriate.

Because there are areas of steep terrain underlain by deep, inherently weak soil, the threat of debris slide activity must be regarded as a potential hazard. However, the proposed building envelopes are not within the path of any apparent landslide deposits, and there are no indications that the existing slides are enlarging or encroaching upon the sites. The sites are situated on ridge crests, and the threat of upslope debris slides impacting these areas is considered low.

Several areas indicative of springs or surface seepage were observed within the southwest portion of the upper parcel. Other springs may be encountered during construction. Seepage can reduce slope stability, and can adversely affect pavement and foundation performance. In the improvement areas, it will be necessary to intercept seepage with subsurface drainage facilities.

The proposed access road alignment for the upper parcel will cross areas of deep soil and old slide debris. Portions of the road will cross areas where there is a potential impact from upslope or downslope failures. Special design and mitigation measures within these areas are advisable, and are feasible from a geotechnical standpoint. Possible mitigation measures include replacing the slide debris beneath and immediately upslope of the roads as a compacted buttress; supporting the road and the area immediately upslope with crib walls; or constructing the road on a side hill bridge designed to resist a creep force and to allow passage of slide debris.

Constructing roads on steep hillsides will require retaining structures, and may require side hill bridges. Retaining structure foundations should be supported in firm rock and designed to resist lateral forces caused by downhill creep of the soils above the rock. Retaining structure foundations on the downhill side of roads will be much deeper than those on the uphill sides of roads. Therefore, it will be more economical to install retaining walls on the upslope sides of roads.

Retaining walls and/or side hill bridges may be supported on drilled, cast-in-place, reinforced concrete piers. In landslide areas, the landslides should be stabilized as buttresses prior to installing piers, or the piers designed to resist slide forces. In other areas, the piers can be installed through the natural soils, and designed to resist downhill creep of the soil above the rock.

Roadways outside of slide areas, or on reconstructed landslides should tolerate anticipated minor creep with no more cracking than is typical for Marin County roadways. It will be necessary to construct utilities with flexible pipe, or to provide frequent joints to accommodate minor creep movement.

Compacted fill generally settles about one percent of its thickness. Roadway grades and utilities should be designed to accommodate this settlement.

Existing site conditions indicate the area is well drained. Problems associated with excessive surface erosion or ponding are not evident. Construction and grading will expose areas of deep, weak soil and slide debris, which may be sensitive to erosion and/or slope failure. Erosion protection measures during and after construction should be utilized to reduce the risk of induced instability.

RECOMMENDATIONS

Grading and Buttressing

Areas to be developed should be cleared of vegetation and of the upper few inches of soil containing organic matter. The strippings should be removed or stockpiled for reuse as topsoil. Excavation should then be performed as necessary. We anticipate that with the exception of organic matter and of rocks or lumps larger than six inches in diameter, the excavated material will be suitable for reuse as compacted fill. Organic matter should be disposed of off of the site. Larger material should be disposed of outside of improvement areas. Areas to receive fill should be prepared by cutting level keyways extending into rock. The outboard edge of the keyway excavation should intercept a 1:1 line projected down from the toe of the planned fill.

Subsurface drainage facilities should be installed at the rear of keyways as recommended by the Soil Engineer. The depth and extent of keyways and subdrains should be determined and approved by the Soil Engineer in the field during construction.

Where slope stabilization measures are needed, excavating the landslides during site grading operations, or reconstructing the landslides as compacted earth buttresses with subsurface drainage facilities, offer the greatest reduction of risk. Excavating the landslides consists of removing all the landslide debris, and exposing a relatively flat slope in firm material beneath the slide plane. Buttressing consists of excavating the slide debris; cutting wide level keyways into firm underlying rock; installing subsurface drainage facilities; and backfilling the excavation with compacted fill.

Cut and fill banks generally should be no steeper than two horizontal to one vertical (2:1). Where sloughing is acceptable, cut banks in rock may be 1-1/2:1. Cuts in weak soils or slide debris sould be protected, retained or rebuilt. Where the finished slope must be steeper, or will not "catch", it will be necessary to use crib walls, bin walls, reinforced earth, or other retaining structures. These retaining structures must be founded on firm rock beneath the zone of weakness, or else on compacted fill founded on firm rock beneath the weak soils.

As an alternative to repairing the entire landslide areas, only the portion of the landslide affecting the planned improvements could be repaired. Improvements can be protected from downslope landslides by extending a buttress down at 2:1 to firm rock beneath the slide plane, or by supporting the downhill edge of the buttress with a retaining structure founded on firm rock beneath the slide plane. The unrepaired portion(s) of the slide downslope of the buttress may continue to move, but should not adversely affect the buttress.

Improvements can be protected from upslope slides by retaining the slides with a retaining structure, or with a partial earth buttress underlain by subsurface drainage systems. If the upslope is steep enough to allow debris flows to overtop the retaining structure or buttresses, it will be necessary to enhance the stability of the upslope area with subsurface drains, and/or to provide catchment areas for possible slough debris and mudflows.

Crib Walls

Crib walls may be used to support fills, cuts or landslides. Crib walls should be of reinforced concrete construction and should conform to the California Department of Transportation Specifications. Crib walls should be battered at least one foot for every six feet of height. Crib walls should be founded on firm rock or on engineered fill founded on firm rock. The wall toe should be founded at least 18 inches into rock for walls less than 10 feet high; at least 30 inches deep for walls 10 to 15 inches high; and at least 3 feet deep for walls over 15 feet high. The toe of the walls should also be deepened as necessary to provide at least 7 feet of horizontal confinement between the toe of the walls and the face of the nearest slope. Subsurface drainage should be provided from the rear of wall foundation excavations.

Reinforced Earth Walls

Reinforced Earth (patented by the Reinforced Earth Company) consists of thin facing panels connected to strips extending into the backfill. The strips are generally galvanized steel about 1/8 inch thick and 2 to 3 inches wide. The strips are located a few feet apart horizontally and vertically, and extend back a distance equivalent to about 80 percent of the wall height. Areas to receive reinforced earth are prepared by excavating a level bench into firm material, as previously described for Concrete Crib Walls. Strips are extended across the bottom of the excavation. A row of facing panels a few feet high is then placed along the outboard edge of the planned fill, and attached to the strips. The panels are then backfilled with granular material. Strips are then attached to the tops of these panels, and extended back across the granular fill. A second row of panels is placed, and compacted granular backfill placed over the top of the strips. Another row of strips is placed, panels installed, and additional backfill placed. The resulting walls are generally more economical than crib walls or reinforced concrete retaining walls for heights greater than about 15 feet. The major disadvantage is that the walls require clean granular backfill in order to bond adequately to the reinforcing strips.

Properly constructed reinforced earth walls can support roadways and can retain landslides. As with retaining walls and crib walls, reinforced earth walls should be fully backdrained.

Utilities

Utilities in slide areas should be constructed in compacted earth buttresses founded on firm material beneath the slide plane, or else should extend into firm rock beneath the slide plane. Utilities founded in colluvium on slopes will experience differential lateral movement as the soils above the rock slowly creep downslope (on the order of a small fraction of an inch per year). Frequent joints should be provided to accommodate the anticipated creep movement.

Seepage will accumulate in utility trench backfill. Gravity flow outlet pipes should be provided from the bedding material at each low point, and at least every 500 feet to prevent build up of hydrostatic pressure in utility trenches.

Where utility benches are steep enough to erode, check dams should be provided. Where utility trenches are steep enough to erode, check dams and/or rip rap should be provided.

Pavement

The pavement design should be based upon traffic indices provided by the City of Ross Engineer, and upon R Value tests on representative soils exposed at subgrade level after rough grading operations. All utilities should be installed and properly backfilled prior to subgrade preparation.

Some of the on-site soil is expansive, and will swell when wet and shrink when dry. The shrinking and swelling will cause pavements to experience edge cracking. Edge cracks must be sealed as they occur. Concrete paving should be reinforced to reduce cracking, and provided with frequent joints to control cracking.

Building Foundations

We anticipate that the portion of building constructed on level areas excavated into firm soil or rock can be supported on continuous and interconnected spread footings. Drilled piers extending into rock will be necessary on and near slopes, and may be used everywhere. It will be necessary to design piers to resist lateral forces caused by downhill creep of soils above the rock.

Soil Engineering Drainage

Surface runoff should be diverted away from cut and fill banks. Subsurface drainage facilities should be installed beneath fills; behind retaining walls; where springs are observed; and in other areas as determined by the Soil Engineer during site grading operations.

Surface water should be diverted away from slopes and weak soil areas by means of concrete-lined interceptor ditches. Surface and subsurface drainage facilities should be maintained entirely separate. Drains should outlet into erosion resistant areas, and should not concentrate water above neighboring property.

Portions of landslides above buttresses, and unrepaired landslides where reactivation would be detrimental, should be enhanced with surface and subsurface drainage improvements. The subsurface drainage improvements should consist of a subdrain extending down the central axis of the landslide, with laterals extending to each side at about 50 foot intervals. Roadways that cross areas of deep, weak soil, or excessively wet areas, may need subdrains along the inboard (uphill) side. Subdrains should extend below the soil rock contact, and should be sloped to drain by gravity.

SUPPLEMENTAL SERVICES

This is a preliminary investigation for evaluating project feasibility. Additional investigation will be necessary to develop geotechnical design criteria for actual design and construction. We should review the master plan for conformance with the intent of this investigation.

LIMITATIONS

We have performed this preliminary investigation in accordance with current standards of engineering practice. We offer no other guarantees or warranties, either expressed or implied.

We trust this provides the information you require at this time. If you have question, please call.

Yours very truly,

DONALD HERZOG & ASSOCIATES, INC.

Donn Ristau

Donn A. Ristau,
Senior Staff Geologist
Registered Geologist - 3634

Donald Herzog,
Principal Engineer
Civil Engineer - 18093

DAR/DH/gh

4 copies submitted

Attachment: Plate 1

REFERENCES:

Blake, M. C., Jr. and others, 1974, Preliminary Geologic Map of Marin, and San Francisco Counties and Parts of Alameda, Contra Costa and Sonoma Counties, California: U.S. Geological Survey Miscellaneous Field Studies Map MF-574, BDC 64, Scale 1:62,500.

Wentworth, C. M. and Frizzell, V. A., 1975, Reconnaissance Landslide Map of Parts of Marin and Sonoma Counties, California, San Rafael Sheet: U.S. Geological Survey, Scale 1:24,000.

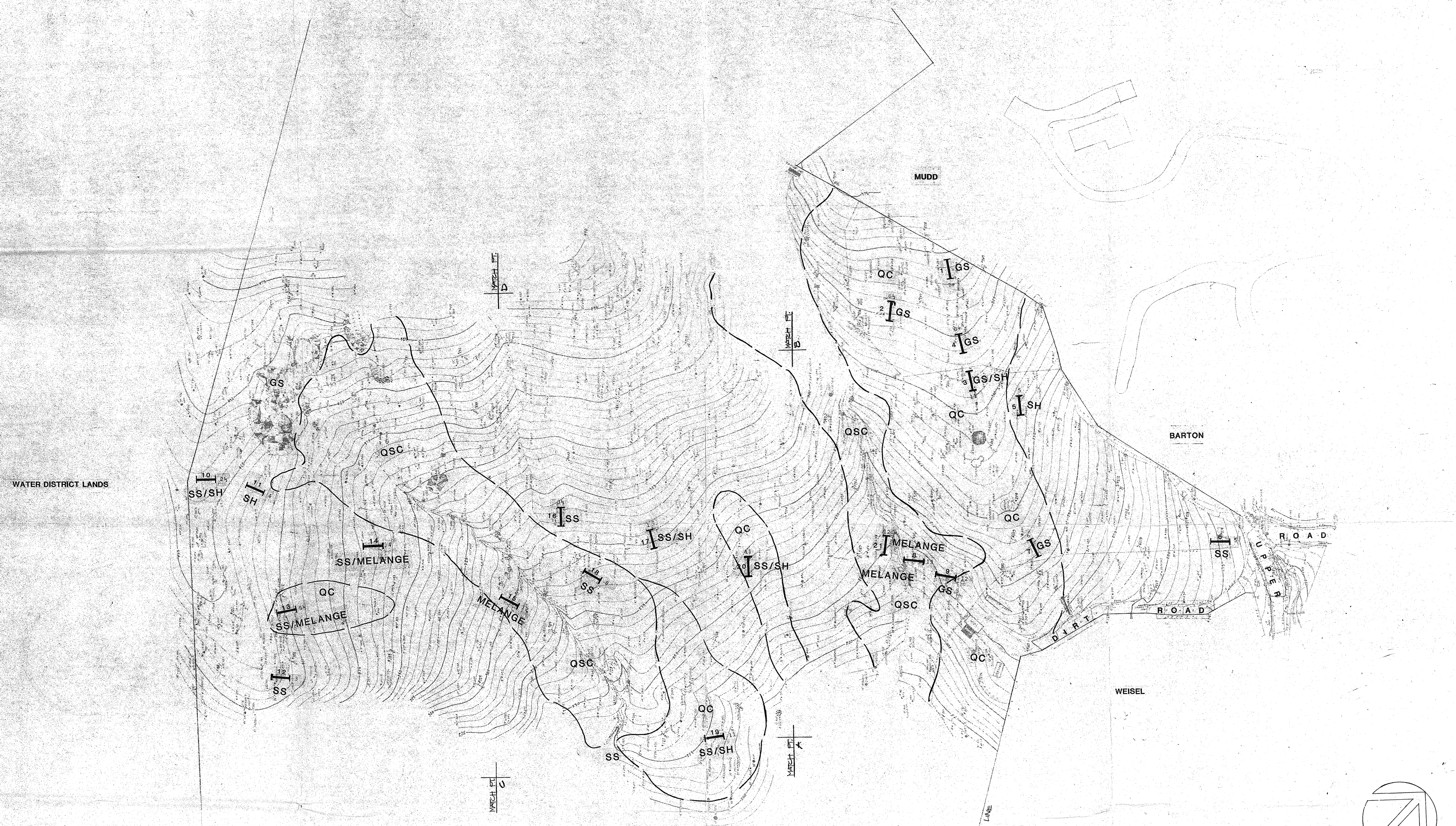
Smith, T. C.; Rice, S. J.; and Strand, R. G., 1976, Geology of the Upper Ross Valley and the Western Part of the San Rafael Area, Marin County, California: California Division of Mines and Geology, OFR 76-2SF, Plate 1B, Scale 1:12,000.

Smith, T. C.; Rice, S. J.; and Strand, R. G., 1976, Relative Stability of the Upland Slopes in the Upper Ross Valley and the Western Part of the San Rafael Area, Marin County, California: California Division of Mines and Geology, OFR 76-2SF, Plate 2B, Scale 1:12,000.

Jennings, C. W., 1975, Fault Map of California With Locations of Volcanoes, Thermal Springs, and Thermal Wells: California Division of Mines and Geology, Geol. Data Map No. 1, Scale 1:750,000.

Hays, W. W., 1980, Procedures for Estimating Earthquake Ground Motions: U. S. Geological Survey Professional Paper 1114, 77 p.

Ploessel, M. R., and Slosson, J. E., 1974, Repeatable High Ground Accelerations from Earthquakes, California Geology, September, pp. 195-199.



EXPLANATION

SS : SANDSTONE
SH : SHALE
GS : GREENSTONE
MELANGE : sheared shale matrix-resistant graywacke and greenstone inclusions
QC : DEEP COLLUVIUM
QSC : SLIDE DEBRIS and COLLUVIUM
◯ ESTIMATED LIMITS OF DEEP SOIL

TEST PIT #
DEPTH TO ROCK
ROCK TYPE
6 SS/SH

DONALD HERZOG & ASSOCIATES consulting engineers Job No: 1385.1 Appr: ______ Date 10-4-82	TEST PIT LOCATIONS AND SOIL AND ROCK CONDITIONS Roth Properties, Ross, Ca.	PLATE 1

DESIGN DEVELOPMENT

ROTH PROPERTY ASSESSMENT

1": 50'-0"

CHNMB
Carter Hull Nishita McCulley Associates
Urban Design • Landscape Architecture • Planning
2789 25th Street, San Francisco, California 94110 (415) 821-3500

H. 435-2692
call w Nancy 382-0544
Bill
Repts
Skip Berg
500 Palm Dr.
Novato CA 94949
Bob Toigo

BASE DATA FROM TOPOGRAPHIC SURVEY, APRIL 1982 SURVEY NO.1113 BY
JOSEPH GRIPPI & ASSOC. INC. 1817 GRAND AVE., SAN RAFAEL, CALIFORNIA