APPENDIX

BORING B-1

DATE DRILLED: November 24, 1998
EQUIPMENT USED: 24" - Diameter Bucket
ELEVATION: 155 Ft. (Mean Sea Level)

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	BLOW COUNT* (blows/ft.)	SAMPLE LOC.	USCS	DESCRIPTION
						ML	ARTIFICIAL FILL CLAYEY SILT - some fine Sand, few Gravel, some wood and steel fragments, slightly moist, firm, brown
				2	■		
							Some wood fragments, some Cobbles (to 8-inches), brown Encountered 3-inch P.V.C. pipe
150	5			<1	■		
						SM	COLLUVIUM SILTY SAND - some bedrock fragments, reddish brown Sandstone fragment (7-inch thick)
				<1	■		
							Reddish brown to yellowish gray and black
145	10						
				1	■		
							Some Cobbles (to 6 inches), some angular bedrock fragments (to 12 inches) Seepage encountered on top of sandstone bed
140	15			2	■		
							LANDSLIDE DEBRIS SANDSTONE - massive, fine to medium-grained, highly weathered, very hard, yellowish brown Contact (top of sandstone bed): N70E, 32SE
							CLAYEY SILTSTONE - highly weathered, moist, soft, light brown and yellow Moist, moderately firm, light brown
135	20			2	■		
130	25			5	■		Moderately bedded, sheared, dark gray
							Moist, moderately firm, dark bluish gray, unoxidized
125	30			4	■		
							Some slickensides, sheared Well bedded layer
120	35			20 for 10"	■		SANDSTONE - fine to medium-grained, damp, hard, gray Slightly cemented
							END OF BORING AT 38'.

* Number of blows required to drive the Crandall sampler 12 inches for depths of:
0 to 25' using a 1600 pound kelly bar falling 12 inches;
Below 25' using a 765 pound kelly bar falling 12 inches.

NOTE: Water seepage encountered at depths of 13' and 29'. Caving at 13' and below. Boring backfilled and tamped. Downhole logged by SFK.

Note: The log of subsurface conditions shown hereon applies only at the specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

70131-8-0618.0001 DATE 12/31/1998 F.T. AR DR ___ O.E. JJB CHKD JJB

LOG OF BORING

LAW/CRANDALL

FIGURE A-1.1

BORING B-2

DATE DRILLED: November 25, 1998
EQUIPMENT USED: 24" - Diameter Bucket
ELEVATION: 163 Ft. (Mean Sea Level)

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	BLOW COUNT* (blows/ft.)	SAMPLE LOC.
				2	■
160					
	5			<1	■
155				1	■
	10			3	■
150					
	15			8	■
145					
	20			10	■
140					
	25			15 for 10"	■
135					
	30			25	■
130					
	35				
125					

5" Asphalt Paving

ARTIFICIAL FILL

ML — CLAYEY SILT - medium brown, firm, moist, some 1/4 to 3-inch brick fragments in upper foot, root hairs and roots to 1/8-inch in diameter

TERRACE DEPOSITS

ML — CLAYEY to fine SANDY SILT - mottled light yellow to medium brown, soft to firm, very moist, mineralized rootlets, some pin-hole porosity

BEDROCK - Interbedded light gray to light yellow brown sandstone and light to medium gray siltstone and shale, moist, thinly to thickly bedded, friable, slightly cementeed sandstone with intermittent 3 to 6 inch concretions, gypsum crystals within the siltstone
8½' - Bedding: N25E, 32SE
10' - Fault: N15W, 75NE

13½' - Carbonate filled joints 1/16 to 1/8-inch, some light red iron staining

15½' - Bedding: N85E, 50NW - intermittent gypsum crystals to 1/4-inch thick along bedding

17' - Bedding: EW, 84N - slickensides along bedding

21½' - Well cemented lenticular concretion 2 to 3-inch thick
22' - Shear: N28W, 62SW - 1/4-inch wide clay seam
Unable to correlate strata across shear

25' - Bedding: N10E, 46SE - 2 to 3-inch gouged clay seam with 1/4-inch thick gypsum along bedding, groundwater seepage along east clay seam contact

32' - Color change to dark blue gray, intermittent well cemented layers from 2 to 6-inch thick

34' - Bedding: N24E, 50SE; 3-inch thick well cemented Sandstone layer

NOTE: Slight groundwater seepage at 25'. No caving. Boring backfilled and tamped. Downhole logged by JJB.

BORING TERMINATED AT 39' DUE TO REFUSAL IN WELL CEMENTED SANDY SILTSTONE.

70131-8-0618.0001 DATE 12/31/1998 F.T. AR DR ___ O.E. JJB CHKD JJB

Note: The log of subsurface conditions shown hereon applies only ... specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

LOG OF BORING

LAW/CRANDALL

FIGURE A-1.2

BORING B-3

DATE DRILLED: November 25 1998
EQUIPMENT USED: 24" - Diameter Bucket
ELEVATION: 136 Ft. (Mean Sea Level)

ELEVATION (ft.)	DEPTH (ft.)	MOISTURE (% of dry wt.)	DRY DENSITY (lbs./cu. ft.)	BLOW COUNT* (blows/ft.)	SAMPLE LOC.	USCS	DESCRIPTION
135				5	■	ML-CL	**ARTIFICIAL FILL** CLAYEY SILT to SILTY CLAY - mottled light yellow brown to dark brown, moist, firm, abundant roots in upper foot to maximum 1/4-inch diameter
						ML	**TERRACE DEPOSITS** CLAYEY SILT - with Sand, medium brown, moist, firm, some intermittent gravel to 2-inch maximum diameter
130	5			7	■		Dark reddish brown, moist, firm to stiff, massive, some minor root hairs, some intermittent gravel to a maximum 2-inch diameter; 6' - Cobbles and Boulders in 2-foot thick layer up to 18-inch diameter, loose sandy matrix, ravelling and belling of hole in this zone from 2 to 3 feet, clasts predominately of granitic and metamorphic composition
							BEDROCK - Weathered Basalt; mottled light yellow brown, light reddish brown and dark gray, massive, friable, randomly oriented carbonate veins; 9' - Shear: N52E, 85NW
125	10			17	■		
120	15			25 for 10"	■		Becoming harder and less weathered
115	20			20	■		

BORING TERMINATED AT 22' DUE TO REFUSAL IN HARD BASALT.

NOTE: Groundwater not encountered. Raveling from 6 to 8 feet. Boring backfilled and tamped. Downhole logged by JJB.

70131-8-0618.0001 DATE 12/31/1998 F.T. AR DR. O.E. JJB CHKD JJB

Note: The log of subsurface conditions shown hereon applies only a. .e specific boring location and at the date indicated. It is not warranted to be representative of subsurface conditions at other locations and times.

LOG OF BORING

LAW/CRANDALL

FIGURE A-1.3

MAJOR DIVISIONS			GROUP SYMBOLS	TYPICAL NAMES
COARSE GRAINED SOILS (More than 50% of material is LARGER than the No.200 sieve size)	GRAVELS (More than 50% of coarse fraction is LARGER than the No.4 sieve size)	CLEAN GRAVELS (Little or no fines)	GW	Well graded gravels, gravel-sand mixtures, little or no fines
			GP	Poorly graded gravels or gravel-sand mixtures, little or no fines
		GRAVELS WITH FINES (Appreciable amount of fines)	GM	Silty gravels, gravel-sand-silt mixtures
			GC	Clayey gravels, gravel-sand-clay mixtures
	SANDS (More than 50% of coarse fraction is SMALLER than the No.4 sieve size)	CLEAN SANDS (Little or no fines)	SW	Well graded sands, gravelly sands, little or no fines
			SP	Poorly graded sands or gravelly sands, little or no fines
		SANDS WITH FINES (Appreciable amount of fines)	SM	Silty sands, sand-silt mixtures
			SC	Clayey sands, sand-clay mixtures
FINE GRAINED SOILS (More than 50% of material is SMALLER than the No.200 sieve size)	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 silts and organic silty clays of low plasticity
	SILTS AND CLAYS (Liquid limit GREATER than 50)		MH	Inorganic silts, micaceous or diatomaceous 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

BOUNDARY CLASSIFICATIONS: Soils possessing characteristics of two groups are designated by combinations of group symbols.

PARTICLE SIZE LIMITS

SILT OR CLAY	SAND			GRAVEL		COBBLES	BOULDERS
	Fine	Medium	Coarse	Fine	Coarse		

No. 200 No. 40 No. 10 No. 4 3/4 in. 3 in. (12 in.)

U. S. STANDARD SIEVE SIZE

UNIFIED SOIL CLASSIFICATION SYSTEM

REFERENCE:
The Unified Soil Classification System, Corps of Engineers, U.S. Army Technical Memorandum No. 3-357, Vol. 1, March, 1953. (Revised April, 1960).

LAW/CRANDALL

FIGURE A-2

The J. Byer Group, Inc.

Geologic and Soils Engineering Exploration
Proposed Landslide Repair and Multi Unit Condo and
Townhome Buildings
Tentative Tract 52928
17331-17333 Tramonto Drive
Pacific Palisades, California
For
Palisades Landmark, LLC
August 16, 2000

Copy 2

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GEOLOGIC AND SOILS ENGINEERING EXPLORATION
PROPOSED LANDSLIDE REPAIR AND
MULTI UNIT CONDOMINIUM AND TOWN HOME BUILDINGS
TENTATIVE TRACT 52928

17331 - 17333 TRAMONTO DRIVE

PACIFIC PALISADES, CALIFORNIA

FOR PALISADES LANDMARK, LLC.

THE J. BYER GROUP, INC. PROJECT NUMBER JB 18457-I

AUGUST 16, 2000

GEOLOGIC AND SOILS ENGINEERING EXPLORATION
PROPOSED LANDSLIDE REPAIR AND
MULTI UNIT CONDOMINIUM AND TOWN HOME BUILDINGS
TENTATIVE TRACT 52928
17331 - 17333 TRAMONTO DRIVE
PACIFIC PALISADES, CALIFORNIA
FOR PALISADES LANDMARK, LLC.
THE J. BYER GROUP, INC. PROJECT NUMBER JB 18457-I
AUGUST 16, 2000

INTRODUCTION

This report has been prepared per our signed agreement dated June 13, 2000 and summarizes findings of The J. Byer Group, Inc. geologic and soils engineering exploration performed on the site. The purpose of this study is to evaluate the nature, distribution, engineering properties, relative stability, and geologic structure of the earth materials underlying the site with respect to stabilizing a portion of the "Revello Drive" landslide and redeveloping the site with multi unit condominium and townhome buildings.

INTENT

It is the intent of this report to assist in the design and completion of the proposed project. The recommendations are intended to reduce geotechnical risks affecting the project. The professional opinions and advice presented in this report are based upon commonly accepted standards and are subject to the general conditions described in the NOTICE section of this report.

PROJECT SCOPE

The scope of the project was determined from consultation with Ken Kahan of Palisades Landmark, LLC. The preliminary plans prepared by Nadel Architects dated February 8, 2000, were considered prior to beginning work on this project. This report is limited to the area of the proposed project as shown on the enclosed Geologic Map and cross sections. Conditions affecting portions of the property outside the area explored, are beyond the scope of this report.

Exploration was conducted near the toe of the Revello Drive landslide between March 31 and April 2, 1997 and March 1 through 24, 2000. The exploration included grading an access road to the northwestern portion of the property and drilling five borings. The borings were advanced to depths of 43 to 65 feet with the aid of hillside drill rig. The borings were downhole logged by the undersigned engineering geologist.

Crandall Consultants, Inc. (Crandall), drilled three borings within the study area on November 24 and 25, 1998. Boring 1 was drilled within the Revello Drive slide, while Borings 2 and 3 were drilled near the central and eastern portions of the slide. Recent borings were also drilled under the direction of Lockwood-Singh & Associates (April, 1999), near the head and western margins of the Revello Drive slide. Numerous borings were drilled by Pacific Soils as part of the original development of the Ocean Woods Terrace apartment and to investigate the Revello Drive slide.

Office tasks for this report included reviewing previous laboratory testing, reviewing the City of Los Angeles grading records, preparing the Geologic Map and cross sections, and performing engineering analysis. Earth materials exposed in the borings are described on the enclosed Log of Borings. Appendix I contains a discussion of the laboratory testing procedures and results.

The proposed project, surface geologic conditions, and the location of the borings are shown on the Geologic Map. Subsurface distribution of the earth materials, projected geologic structure, and the proposed project are shown on Sections A through P.

RESEARCH - PREVIOUS REPORTS

Crandall Consultants, Inc. (geotechnical engineer) and Glen A. Brown, Consulting Geologist, explored the subject property in 1998. The following reports were prepared for Palisades Landmark Group by Crandall Consultants, Inc.:

> *Report of Geotechnical Consultation, Conceptual Development Considerations, Vesting Tentative Tract No. 52928, Tramonto Drive, Pacific Palisades, California,* dated December 15, 1999, and
>
> *Supplemental Information*, dated January 27, 2000.

The Grading Section of the city of Los Angeles Department of Building and Safety reviewed these reports and issued a letter dated February 7, 2000, requesting additional information.

Numerous reports were prepared by The J. Byer Group, Inc., for the adjacent downslope property between 1997 and 2000 for the previous and existing property owners. The following reports were prepared for Coler Construction and Investment Company:

> *Geologic and Soils Engineering Exploration, Proposed Sea Vista Apartments,* dated May 6, 1997;
>
> *Addendum Geologic and Soils Engineering Exploration,* dated June 30, 1997;
>
> *Additional Comments, Calculations and Recommendations,* dated August 11, 1997;
>
> *Additional Comments, Calculations and Recommendations #2,* dated October 9, 1997;
>
> *Additional Calculations and Recommendations,* dated November 12, 1997;
>
> *Additional Comments and Recommendations,* dated November 14, 1997;
>
> *Additional Calculations and Recommendations,* dated December 10, 1997;
>
> *Geologic and Soils Engineering Exploration Update,* dated November 23, 1998;

Addendum Geologic and Soils Engineering Report, dated January 29, 1999;

Geotechnical Engineering Memorandum, dated February 22, 1999 and

Geotechnical Engineering Memorandum, dated August 3, 1999.

The following reports were prepared for G.H. Palmer and Associates:

Geologic and Soils Engineering Exploration Update, Proposed Sea Vista Condominiums, Portion of Lot D, Castellammare Tract, 17325 Castellammare Drive, Pacific Palisades, California, dated December 17, 1999 and

Addendum Geologic and Soils Engineering Exploration, Proposed Condominium Building, dated April 19, 2000.

Research at the City of Los Angeles Department of Building and Safety was performed for the initial exploration and report. The records contain numerous geotechnical reports pertaining to studies of the 1965 Revello Drive landslide and development of adjacent properties. The data contained in the following reports was reviewed and considered as part of our work on this project:

Report by Pacific Soils Engineering:

Investigation of the Pacific Palisades Landslide, Revello to Castellammare Drives, Los Angeles, California, dated November 5, 1965.

Reports by AAKO Geotechnical Engineering Consultants, Inc.:

Geotechnical Engineering Investigation, Planned Single Family Dwelling, 17474 Revello Drive, Pacific Palisades Area of Los Angeles, California, February 8, 1989;

Geotechnical Engineering Review of Grading Plan, Planned Single Family Dwelling, 17474 Revello Drive, Pacific Palisades Area of Los Angeles, California, February 25, 1991;

Response to the City of Los Angeles Grading Division, Geotechnical Report Review Letter for 17474 Revello Drive, Pacific Palisades Area of Los Angeles, California, July 25, 1991;

Response #2 to City of Los Angeles Grading Division, Geotechnical Report Review Letter, January 20, 1992;

Response #3 to City of Los Angeles Grading Division, Geotechnical Report Review Letter, May 28, 1992; and

Response #4 to City of Los Angeles Grading Division, Geotechnical Report Review Letter, August 24, 1992.

Reports by Harley Tucker:

Preliminary Engineering Geologic Investigation, Proposed Commercial Development, Portion of Lot 5, Tract # 10238, Vicinity of Northwest Corner Pacific Coast Highway and Sunset Boulevard, Los Angeles, California, (17381 Sunset Boulevard), August 5, 1982;

Supplemental Engineering Geologic Report, October 1, 1982;

Supplemental Engineering Geologic Report #2, November 18, 1982; and

Supplemental Engineering Geologic Report #3, September 11, 1985.

Reports by G. C. Masterman & Associates, Inc.:

Soils and Foundation Investigation, Sunset Pacific Plaza, 17381 Sunset Boulevard, Pacific Palisades, California, August 6, 1982;

Addendum #1, October 5, 1982;

Addendum #2, November 22, 1982;

Addendum #3, November 30, 1982;

Additional Information, January 21, 1983;

Addendum #4, May 25, 1984;

Retaining Wall Design Recommendation Letter, June 8, 1984;

Design Load Recommendations for Retaining Wall, August 9, 1984;

Retaining Wall Design Values, August 14, 1984;

Revised Wall Design Analysis Letter, October 22, 1984;

Addendum #5, October 25, 1984;

Explanation of Soldier Pile Analysis, October 25, 1984;

Retaining Wall Calculations, November 5, 1984;

Calculations for Lower Retaining Wall, November 8, 1984;

Proposed Retaining Wall Vertical Drains, November 12, 1984;

Wall Drain Equivalency Letter, November 16, 1984; and

Temporary Excavation Stability and Vehicular Surcharge Design, November 27, 1984.

Report by Kovacs-Byer & Associates, Inc.:

Debris Wall Design Parameters, 17321 Castellammare Drive, Pacific Palisades, California, May 2, 1980.

Reports by Leroy Crandall & Associates:

Geotechnical Exploration, Proposed Soldier Piles, Ocean Woods Terrace Apartments, 17331-17333 Tramonto Drive, Tract 29827, November 7, 1980; and

Inspection of Foundation Excavations and Inspection and Testing of Compacted Backfill, Repair of Slope Failures below Pool and Deck, February 1, 1982.

Reports by the City of Los Angeles Bureau of Engineering:

(Design of Bulkhead) 17329 Castellammare Drive, 300 feet west of Sunset Boulevard - 1980 storm damage, July 24, 1980 and

Addendum Report, 17327-17329 Castellammare Drive-300 feet west of Sunset Boulevard - 1980 storm damage, March 27, 1981.

The Revello Drive landslide occurred in the spring of 1965 and destroyed single family homes and an apartment building. Numerous geotechnical consultants and the city of Los Angeles Bureau of Engineering drilled borings through the landslide and performed geotechnical analyses of the slide and adjacent properties. The history of the landslide and the sequence of events leading up to the slope failure are documented in the referenced reports. Briefly, the Revello Drive landslide occurred in the spring of 1965, after a 1:1 to 1½:1 cut slope was created below the westernmost Ocean Woods Terrace apartment building. The slope failed onto and bulldozed over a level pad at 17325 Castellammare Drive. The westernmost Ocean Woods Terrace apartment building and upslope dwellings within the slide were damaged and subsequently demolished. Foundation remnants still litter the slope on and below the site. Through the years, additional movement of the original slide mass, and secondary failures, have caused the slide to enlarge and impact the street and 17321 Castellammare Drive. Concrete and I-beam soldier piles with a wood bulkhead on top were constructed by the Bureau of Engineering to protect the property at 17321 Castellammare Drive and the street. The remaining two apartment buildings of the Ocean Woods Terrace development have been occupied since the 1960's.

The referenced J. Byer Group reports for 17321 Castellammare Drive were prepared for stabilizing the toe of the Revello Drive landslide and constructing a multi-unit condominium building. Review of the project is on going and additional information is currently being provided to the Grading Section for the project.

The referenced AAKO geotechnical exploration and reports were prepared to provide recommendations for constructing a residence at the head of slide (17474 Revello Drive). The recommendations, approved by the Grading Division, included stabilizing the landslide with soldier piles tied back to concrete deadmen. The project was never built.

The referenced LeRoy Crandall reports were prepared to provide recommendations for installing soldier piles to support the west side of the Ocean Woods Terrace apartment building at 17333 Tramonto Drive, which is adjacent to the eastern margin of the slide. The pile supported retaining wall is present below (south) of the existing pool. The Crandall report, dated February 1, 1982, contains the as-built depths of soldier piles and the results of compaction testing of retaining wall backfill.

Harley Tucker and G.C. Masterman prepared numerous reports for the construction of the large commercial/retail building that is located along the downhill side of Castellammare Drive, just south of the subject property. This building is significant for the subject property because it has substantial retaining walls that support excavations, which extend to near the elevation of Sunset Boulevard, and an extensive subdrain system. The building was constructed in the middle to late 1980's.

Kovacs-Byer and Associates (KBA) prepared their report to provide recommendations for protecting the property at 17321 Castellammare Drive with soldier piles. John Merrill, engineering geologist, performed the geologic analysis and downhole logging of borings drilled as part of the KBA exploration. Geotechnical data and active earth pressure recommendations for soldier piles contained in the KBA report were used, at least in part, by the city of Los Angeles Bureau of Engineering for their design of the soldier pile/bulk head along Castellammare Drive. The upper 20 feet of the soldier piles are designed to retain an equivalent fluid pressure of 80 pounds per cubic foot.

The slide re-activated during the above normal rainfall year of 1997/1998. The limits of the reactivated slide were delineated by earth cracks and horizontal and vertical offsets in pavement, hardscaping, fences, and the ground surface. The margins of the slide were observed in November, 1998, from the terminus of Posetano Road, from Revello Drive, Castellammare Drive, and from the Oceanwoods Terrace apartments. The limits of the recent slide are similar to the limits shown on The J. Byer Group Geologic Map dated November 14, 1997. The recent slide toed up above the existing City of Los Angeles bulkhead along Castellammare Drive. There is no evidence of deeper slide movement or distress to the street and the property between Castellammare Drive and Pacific Coast Highway.

As a consequence of the 1998 slide movement, a lawsuit was filed against the city of Los Angeles. Geotechnical borings were drilled near the head and western margins of the slide by geotechnical consultants representing the plaintiffs. Logs of borings drilled under the direction of Lockwood-

Singh & Associates (April, 1999) were provided to The J. Byer Group as a courtesy by the city attorney. A report that contains all of the logs of subsurface exploration performed within and near the Revello Drive slide was compiled by Grover-Hollingsworth and Associates (geotechnical consultant to the city attorney). Copies of this report (*Preliminary Results of Geologic and Geotechnical Research, Revello Drive Landslide, Los Angeles, California*, dated February 29, 2000) were provided to the Department and The J. Byer Group.

PROPOSED DEVELOPMENT

Information concerning the proposed project was provided by Ken Kahan of Palisades Landmark, LLC. The preliminary plans prepared by Nadel Architects, dated February 8, 2000 and the aerial topographic survey prepared by Grimes Surveying, Inc., dated April 1999, were considered prior to beginning work on this project. It is proposed to stabilize the portion of the Revello Drive landslide on the subject property and re-develop the site with six multi-story condominium and town home buildings. The existing apartment buildings will be torn down. Four of the buildings are to be located east of the existing apartment buildings and pool. Two of the buildings will be located in the area of the property underlain be slide debris. The buildings are numbered to facilitate discussion. Access to the site and the proposed buildings will continue to be from Tramonto Drive.

In general, the elevations of the buildings will follow the natural topography to minimize the amount of grading and retaining walls. Retaining walls on the order of 20 feet high are contemplated to support excavations into the hillside, and retaining wall backfill. The existing slide will be removed to stable bedrock and replaced as compacted fill. The compacted fill be keyed and benched into bedrock, and supported laterally by soldier piles. The soldier piles will also be utilized to support temporary excavations, in order to remove the slide debris. Formal plans have not been prepared and await the conclusions and recommendations of this report.

SITE DESCRIPTION

The subject property consists of a partially graded and developed parcel that contains approximately 3.8 acres of hillside terrain. The study area is on the eastern edge of Castellammare Mesa, approximately 400 feet west of the intersection of Castellammare Drive and Sunset Boulevard, in the Pacific Palisades section of the city of Los Angeles, California. Two apartment buildings, known as the Ocean Woods Terrace apartments are present in the central portion of the site. The western margin of the property includes the central portion of the active Revello Drive landslide. A third building within the Ocean Woods Terrace apartment development was destroyed by the slide.

The Ocean Woods Terrace apartments are accessed from Tramonto Drive via a private driveway. Carports for the apartments are located at the elevation of the access drive. The buildings are built over the slope below the carports. A pool and wood deck extend over the slope between the two buildings. The study area has access to Castellammare Drive via narrow strips. The properties below the study area are developed with apartment and commercial buildings, except the area of the slide, which is presently vacant. Properties above and west of the subject property are developed with single family residences and apartment buildings, except the slide, which is vacant.

Vegetation on the site consists a moderately thick assemblage of native and cultured trees, chaparral, shrubs, and grasses. Surface drainage within the slide on the subject property is uncontrolled and generally flows down the contours of the land toward Castellammare Drive. Depressions within the eastern, more active portion of the slide, captures and concentrates runoff. Drainage within the developed portions of the property is moderately to poorly controlled. Numerous terrace drains, paved swales, and drainage pipes are present on the slopes below the apartment buildings. The swales and drainage devices are in various stages of repair and functionality. Generally, the swales and drainage devices convey runoff to Castellammare Drive.

GROUNDWATER

Perched groundwater zones and seeps were encountered during exploration. Moderate to heavy seeps were encountered within the slide mass as noted in the Log of Borings. In general, the water is perched on top of the clayey gouge along the base of the upper and lower slides. A significant amount of water was also reported by previous geotechnical consultants who have drilled borings within the Revello Drive landslide. Outside of the slide to the east, a slight seep was encountered in Crandall Boring 2 at 25 feet, while Boring 3 was dry to a depth of 22 feet. Seasonal fluctuations in groundwater levels may occur due to variations in climate, irrigation and other factors not evident at the time of the exploration. Fluctuations in groundwater levels may also occur across the site. Rising groundwater can saturate earth materials, causing instability of slopes. Recommendations are presented in the "Conclusions and Recommendations" section of this report to control subsurface drainage.

EARTH MATERIALS

Fill

Fill reported by Crandall in their borings outside of the slide was less than two feet thick. At least 10 feet of approved retaining wall backfill, which was the subject referenced February 1, 1982 Leroy Crandall report, is likely present behind the pile supported retaining wall on the slope below the deck. Pacific Soils reported seven feet of fill near Boring PS3B, beneath the entry drive on the eastern portion of the site. Here, the fill may thicken to 10 to 12 feet along the downhill edge of the driveway. Fill is also was likely present as retaining wall backfill and along the downhill margins of the pads. The Regional Geologic Map shows the limits of cut and fill in the vicinity of the site. In general the fill consists of silty clay and clayey sand that is mottled grey and dark brown.

Soil - Colluvium

Natural residual soil and colluvium likely blanket the bedrock on the natural slopes east of the slide. The residual soil likely consists of dark brown sandy clay that is on the order of three feet thick. Colluvium may be present in the swale below the access drive on the eastern portion of the study area.

Alluvial Terrace

Alluvial terrace caps the bedrock on the eastern portion of the site as shown on the Geologic Map. Pacific Soils borings PS2B and PS3B encountered up to 15 feet of terrace. In general, the terrace consists of sand and sandy gravel with cobbles that is tan to brown, dense, and structureless.

Landslide Debris

The western portion of the study area is mantled by landslide debris of variable thickness. Offsite to the south, landslide debris encountered during exploration ranges from 25 feet in the vicinity of Boring 3 to 48 feet in Boring 4. The base of the slide within the subject property and upslope were defined by recent and older boring logs found in the records. There are clearly two main slide planes; an 'upper' and 'lower'. It is believed that the 'upper' plane failed onto the 1965 pad cut on the adjacent downslope property, just above Castellammare Drive. Through time, the slide enlarged in size and depth to include the 'lower' slide plane. The limits of the slide shown on the Geologic Map represent the intersection of the 'lower' slide plane the ground surface. Back calculations indicate a lower shear strength for the 'lower' slide plane. Boring Logs by Public Works, drilled in Revello Drive in 1965 upslope from the upper (shallower) slide, indicate that the larger (deeper) slide was the re-activation of an older slide. Crandall boring 1 (C1), which was terminated at a depth of 38 feet due to caving, reported a slide plane at 35 feet. This slide plane correlates with the 'upper' slide plane.

In general, the slide debris consists of admixtures of siltstone and sandstone with a silty sand matrix that is gray-brown, tan and slightly moist to moist. The deeper slide debris consists of contorted and discontinuous siltstone and sandstone that is orange-brown, gray-brown, tan, slightly moist to saturated, medium dense to dense, highly oxidized, and has a chaotic structure.

The base of the slides are marked by a one to 24 inch thick zone of clay gouge and intensely sheared siltstone that is dark grey-brown to blue-gray, plastic, moist to saturated and contains slicks. The base of the slide acts as an aquiclude and has perched the groundwater above.

Bedrock

Bedrock underlying the site and encountered in the borings below the landslide debris consists of siltstone, sandstone, and occasional basalt and conglomerate. Previous geotechnical consultants working on the subject and adjacent properties have mapped bedrock beneath the site as part of the Martinez Formation of Eocene age. J. T. McGill, 1989 (*Geologic Maps of the Pacific Palisades Area*, CDMG Map I-1828), based upon the presence of Middle Miocene micro-fossils located near the subject property, has mapped bedrock beneath the subject property as part of the Topanga Formation. This is consistent with the likely Topanga age basalt on the eastern portion of the study area. In general, the siltstone and sandstone bedrock is thinly to thickly bedded, moderately hard, contorted, and sheared. Cemented conglomerate that is blue-grey to light grey, massive, and slightly weathered was encountered in Boring 3. Where exposed near the ground surface the bedrock is oxidized to a light tan to gray-brown color. Below the slide plane, the bedrock is dark gray to blue-gray with little oxidation. Hard basalt was encountered in Crandall Boring 3 (C3).

GEOLOGIC STRUCTURE

The bedrock described is common to this area of the Pacific Palisades near the base of the south flank of the Santa Monica Mountains. Bedding mapped on the subject property and offsite by other consultants is warped and folded. However, the majority of bedding planes mapped strike to the northwest and dip moderately to steeply to the northeast. The geologic structure observed in the borings and reported by other consultants is consistent with that mapped by McGill, 1989 (see Regional Geologic Map). Faults were not encountered during exploration. However, bedding plane shears were observed in the bedrock below the slide, which likely formed during regional folding of the bedrock.

The Revello Drive landslide is a 'strength of materials' failure, which occurred within the upper weathered portion of the siltstone and sandstone bedrock, and is not related to the geologic structure. As determined by this exploration and as shown on numerous cross sections by other consultants, the base of the slide dips between 10 and 15 degrees toward the southeast. Based upon 1928 and 1949 Spence oblique photographs, and 1952 aerial stereo-pair photographs, it appears that the subject property was underlain by an ancient landslide prior to development. Subsequent development, introduction of water, and grading in 1965 reactivated the ancient landslide. The toe of the 1965 landslide bulldozed across a pad graded approximately 10 feet above Castellammare Drive. Slide debris was not encountered in retaining wall excavations for the referenced project at 17318 Sunset Boulevard, which is across Castellammare Drive from the slide.

The above normal rainfall year of 1997/1998 caused a reactivation of the Revello Drive landslide. The limits of the recent slide are similar to the limits shown on pre-1998 Geologic Maps and represent likely movement along the lower slide plane. The slide toed up above and was impounded by the existing City of Los Angeles bulkhead along Castellammare Drive. There is no evidence of deeper slide movement (below the lower slide plane) or distress to the street and the property between Castellammare Drive and Pacific Coast Highway.

Below the landslide, the geologic structure of the bedrock is favorably oriented for stability of the site and the proposed project. Recommendations to remove and recompact landslide debris on the subject property and support the upslope offsite landslide with soldier piles are presented in the conclusions and recommendations section of this report.

GENERAL SEISMIC CONSIDERATIONS

Southern California is located in an active seismic region. Moderate to strong earthquakes can occur on numerous local faults. The United States Geological Survey, California Division of Mines and Geology, private consultants, and universities have been studying earthquakes in southern California for several decades. Early studies were directed toward earthquake prediction and estimation of the effects of strong ground shaking. Studies indicate that earthquake prediction is not practical and not sufficiently accurate to benefit the general public. Governmental agencies are shifting their focus to earthquake resistant structures as opposed to prediction. The purpose of the code seismic design parameters is to prevent collapse during strong ground shaking. Cosmetic damage should be expected.

Within the past 25 years, southern California and vicinity have experienced an increase in seismic activity beginning with the San Fernando earthquake in 1971. In 1987, a moderate earthquake struck the Whittier area and was located on a previously unknown fault. Ground shaking from this event caused substantial damage to the City of Whittier, and surrounding cities.

The January 17, 1994, Northridge earthquake was initiated along a shallowly dipping, previously unrecognized fault below the San Fernando Valley. The energy released by the earthquake propagated to the southeast, northwest, and northeast in the form of shear and compression waves, which caused the strong ground shaking in portions of the San Fernando Valley, Simi Valley, City of Santa Clarita, and City of Santa Monica.

Southern California faults are classified as: active, potentially active, or inactive. Faults from past geologic periods of mountain building, but do not display any evidence of recent offset, are considered "inactive" or "potentially active". Faults that have historically produced earthquakes or show evidence of movement within the past 11,000 years are known as "active faults".

Based upon the "*Maps of Known Active Fault, Near Source Zones in California and Adjacent Portions of Nevada*", dated February 1988, (Part of the 1997 Uniform Building Code), the site is located within two kilometers of a known seismic source. The nearest known active fault is the Malibu Coast, located within one kilometer. From a 1997 Uniform Building Code (Chapter 16) standpoint, the Malibu Coast fault is classified as a Type "B" fault. The following table lists the applicable 1997 UBC seismic coefficients for the project:

1997 UNIFORM BUILDING CODE SEISMIC COEFFICIENTS	
Earth Materials	**Sandstone and Siltstone Bedrock**
Soil Profile Type	S_c
Seismic Coefficient (C_A)	$0.40N_A$
Seismic Coefficient (C_V)	$0.56N_V$
Near-Source Factor (N_A)	1.3
Near-Source Factor (N_V)	1.6

The principal seismic hazard to the subject property and proposed project is strong ground shaking from earthquakes produced by local faults. Modern, well-constructed buildings are designed to resist ground shaking through the use of shear walls and reinforcement. Additional precautions may be taken to protect personal property and reduce the chance of injury, including strapping water heaters and securing furniture. It is likely that the subject property will be shaken by future earthquakes produced in southern California. However, secondary effects such as surface rupture, and liquefaction should not occur at the subject property.

SLOPE STABILITY

Slopes on and adjacent to the subject property include the Revello Drive landslide, which extends approximately 50 feet in elevation above the rear property line, and steep slopes below the existing building, east of the slide. Within the study area, it is proposed to remove and recompact the slide debris to create suitable building sites. A computerized version of the Modified Spencer's method (REAME ©1999) developed by the Civil Engineering Software Center of the University of Kentucky was used to model non-circular planar failures, while the Simplified Bishop's method was used for circular failures.

The geometry of the Revello Drive landslide is shown in Sections A, B, and C, which are believed to be the most critical. The sections are oriented parallel to the direction of movement, which is based upon movement vectors contained within a John D. Merrill Geologic Map, dated September 28, 1971. The legend of Merrill's map indicates that the movement vectors were determined from survey measurements pre and post slide. The ground surfaces depicted in the sections are mostly computer generated based upon the 1999 aerial topographic survey. The recent topographic survey is believed to accurately depict the slide that moved in 1998.

Back-Calculations

Back calculations (Calculation Sheets 1 through 10) were performed to determine the shear strength along the base of the 'upper' and 'lower' slide planes. The back-calculations assume no groundwater table and utilize a resisting force due to the bulkhead. The research and Public Works drawings indicate that the bulkhead is designed to resist an equivalent fluid pressure of 80 pounds per foot. The assumptions and external forces used are noted on the calculation output sheets. Section B does not pass through the bulkhead.

Back calculations of the main slide mass result in a phi angle that varies between 16 and 17 degrees with zero cohesion. Groundwater was certainly significant in causing sliding. Therefore, the back-calculated strengths include a groundwater table high enough to cause slide movement. Thus, the calculated strengths are considered the lower bound shear strength for the slide plane since groundwater was ignored and resisting forces due to the city bulkhead were used. The back calculation of the shallower secondary slide (Section G) results in a shear strength of 19 degrees phi and zero cohesion (Calculation Sheets 9 and 10).

Shoring Design

In order to remove and recompact the slide debris, the upslope portion of the landslide must be supported. Typical soldier piles, spaced 10 feet on center, are shown on the Geologic Map. The piles are numbered for discussion purposes. The design loads on soldier piles supporting the upslope portion of the Revello Drive landslide were calculated based upon Sections A through C and Calculation Sheets 11 through 16. The external force used to model the soldier piles was increased until the safety factor was at least 1.25. Based upon Section A, temporary load on soldier piles P1 through P10 is 170 kips per foot. For Sections B and C, the force required to raise the safety factor to 1.25 is 145 kips per foot. Section A coincides with pile number 10, while Section C coincides with pile 17. Between piles P10 and P17, the design force should decrease linearly from 170 to 145 kips per foot. From P17 to P35, the recommended design force is 145 kips per foot. The point of application is assumed to be ⅓ the retained height of the pile. Piles P36 through P40 should be designed for an equivalent fluid pressure of 65 pounds per cubic foot per Calculation Sheet #41. Piles along the downslope property line should be designed for an equivalent fluid pressure of 30 pounds per cubic foot. Piles along the downslope property may be omitted if soldier piles and/or a retaining wall have been installed on the adjacent property (Palmer Project).

Permanent Soldier Piles

Permanent soldier piles will be required to support the future compacted fill along the downslope property line. Development plans for the adjacent downslope property includes permanent retaining walls, which will be the full height of the slide. If the offsite retaining wall is not present, soldier piles along the southern property line should be designed for an equivalent fluid pressure of 65 pounds per cubic foot (Calculation Sheets 38). Assuming that slide debris is left in place upslope from the subject property, soldier piles will also be required along the upslope property lines. Soldier piles along the upslope property lines should be designed for a permanent equivalent fluid pressure of 30 pounds per cubic foot.

Safety Factor of the Repair

The safety factor of the repair was calculated assuming that the slide plane and slide debris on the subject property are replaced with compacted fill, and that soldier piles along the uphill and downhill sides of the repair have soldier piles designed for an equivalent fluid pressure of 30 and 65 pounds per cubic foot, respectively. Calculation Sheets 15 through 25 show that the repair will have a safety factor greater than 1.5. The analysis assumes that the groundwater will rise at a 1:1 slope from the rear subdrain as shown in the sections. The safety factor of potential failures in the bedrock just below the repair is greater than 1.5, based upon Calculation Sheets 26 through 34. The calculations assume the worst case groundwater depicted in the sections. For a conservative analysis, the residual strength of the bedrock (Shear Diagram #4) was assumed.

Safety Factor of Slopes East of Revello Drive Slide

Steep slopes are present below the existing apartment building east of the Revello Drive slide as shown in Section M. A pile supported retaining wall with a 2:1 backslope is recommended between the slide and extending east of Section P. The locations of typical piles are shown on the Geologic Map. So as to not surcharge the downslope structures, the piles should retain the earth above elevation 65 as shown in Section M. The soldier piles and retaining wall above elevation 65 should be designed to retain an equivalent fluid pressure of 43 pounds per cubic foot (calculation sheet 42). Based upon calculation sheets 35 through 37, slopes near Section M will be grossly stable after the soldier piles and 2:1 backslope.

CONCLUSIONS AND RECOMMENDATIONS

General Findings

The conclusions and recommendations of this exploration are based upon subsurface exploration, field geologic mapping, research of available records, consultation, years of experience observing similar properties in similar settings and review of the development plans. It is the finding of The J. Byer Group, Inc. that Tentative Tract 52928, the proposed landslide repair, and redevelopment of the site with multi unit condominium and town home buildings is feasible from a geologic and soils engineering standpoint provided the advice and recommendations contained in this report are included in the plans and are implemented during construction.

The recommended bearing materials are the bedrock, approved compacted fill, and alluvial terrace. Geotechnical issues affecting the site include the Revello Drive landslide, deep removal excavations, and non-conforming slopes below the existing apartment building. Six separate multi-unit buildings are planned for the project. For discussion purposes, the buildings are numbered 1 through 6.

Revello Drive Landslide Stabilization

It is proposed to remove the existing slide debris to stable bedrock and place compacted fill up to the planned grades for Buildings 1 and 2. The compacted fill will be used as primary structural fill to support the proposed buildings. The removal depths could be up to 60 feet. It is planned to support vertical excavations along the north, west, and south sides of the removal with soldier piles. As discussed above, the temporary load on soldier piles P1 through P10 is 170 kips per foot. From P17 to P35, the recommended design force is 145 kips per foot. Between piles P10 and P17, the design force should decrease linearly from 170 to 145 kips per foot. The point of application is assumed to be ⅓ the retained height of the pile. Piles P1 through P35 should be embedded in the bedrock below the base of the slide.

Piles P36 through P40 extend beyond the limits of the slide and are intended to support the existing properties along Revello Drive. Piles P36 through 40 should be founded below a 1½:1 plane projected up from the base of the slide as shown in Sections L, K, and O. The recommended design equivalent fluid pressure is 65 pounds per cubic foot for the portion of the pile between the ground surface and the 1½:1 setback plane. Piles along the upslope property line may also be utilized to support temporary vertical excavations to construct the required rear yard retaining walls. Slopes may be trimmed to reduce the heights of shored excavations if permission is granted from the offsite property owner(s).

The owner of the downslope property (17325 Castellammare Drive) is presently designing a project and processing plans to stabilize the slide and develop the toe of the Revello Drive slide. Soldier piles will be required along the common property line if either this, or the downslope project proceeds without the other. North-facing excavations along the southern property line should be supported with soldier piles designed for an equivalent fluid pressure of 65 pounds per cubic foot. Permanent soldier piles designed for an equivalent fluid pressure of 65 pounds per cubic foot will also be required to support the compacted fill placed within the landslide removal void. Building 2, located upslope from the soldier piles should be founded in approved compacted fill below a 1:1 plane projected up from the downhill base of the slide removal. Deepened foundations consisting of friction piles tied with gradebeams will be required to support portions of Building 2 (the southerly building). Floor slabs within the setback area need not be supported by deepened foundations and may utilize the at-grade future compacted fill for support.

South-facing excavations may be supported with soldier piles or trimmed at a 1:1 gradient back toward the existing apartment buildings, which will presumably be torn down prior to repairing the slide. The eastern portions of Buildings 1 and 2 will transition across the limits of the slide into the area of the temporary 1:1 back-cut. To create a more uniform bearing condition, it is recommended that the portions of the proposed building located on bedrock outside the limits of the slide be over-excavated 10 feet below the bottom of footings and replaced with compacted fill.

Subdrains will be required at the base of the repair. Ideally, the subdrains should discharge to the atmosphere via gravity. Discharging the subdrain system to Castellammare Drive via a pipe through the strip of land between the adjacent downslope properties will require a deep trench or drilled excavation. Alternatively, a sump pump system will be required.

Building 4

Building 4 is located near the westerly existing apartment building. Slopes between proposed building 4 and the southern property line are steeper than 2:1 and considered non-conforming. A pile supported retaining wall which supports a 2:1 backfill, is recommended below building 4, as shown on the Geologic Map. Piles should derive support below elevation 65 as shown on Section M. The recommended bearing material for building 4 is the bedrock, which can be reached with conventional and deepened foundations.

Access Drive East of Building 6

The access drive east of building 6 will cross a swale that may contain up to 10 or more feet of fill over colluvium and soil. A pile supported retaining wall is recommended to support the downhill side of the access drive. Existing fill, soil, and colluvium located upslope from the pile supported wall should be removed and replaced as compacted fill.

SITE PREPARATION

Surficial materials consisting of fill, soil, colluvium and landslide debris are present on the site, as discussed above. Remedial grading is recommended to improve site conditions.

General Grading Specifications

The following guidelines may be used in preparation of the grading plan and job specifications. The J. Byer Group would appreciate the opportunity of reviewing the plans to insure that these recommendations are included. The grading contractor should be provided with a copy of this report.

A. The areas to receive compacted fill should be prepared by removing all vegetation, debris, existing fill, soil, colluvium and slide debris. The exposed excavated area should be observed by the soils engineer or geologist prior to placing compacted fill. The exposed grade should be scarified to a depth of six inches, moistened to optimum moisture content, and recompacted to 90 percent of the maximum density.

B. The proposed building site for buildings 1 and 2 shall be excavated to a minimum depth of 10 feet below the bottom of all footings. The excavation shall extend a minimum of 10 feet beyond the building footprint. The excavated areas shall be observed by the soils engineer or geologist prior to placing compacted fill.

C. Fill, consisting of soil approved by the soils engineer, shall be placed in horizontal lifts and compacted in six inch layers with suitable compaction equipment. The excavated onsite materials are considered satisfactory for reuse in the controlled fills. Any imported fill shall be observed by the soils engineer prior to use in fill areas. Rocks larger than six inches in diameter shall not be used in the fill.

D. The fill shall be compacted to at least 90 percent of the maximum laboratory density for the material used. The maximum density shall be determined by ASTM D 1557-91 or equivalent.

E. Field observation and testing shall be performed by the soils engineer during grading to assist the contractor in obtaining the required degree of compaction and the proper moisture content. Where compaction is less than required, additional compactive effort shall be made with adjustment of the moisture content, as necessary, until 90 percent compaction is obtained. One compaction test is required for each 500 cubic yards or two vertical feet of fill placed.

Fill Slopes

Compacted fill slopes may be constructed at a 2:1 gradient and should be keyed and benched into bedrock or supported laterally with retaining walls or soldier piles.

Subdrain

Seeps and groundwater were encountered in the borings during exploration. A subdrain system is recommended at the back of the proposed repair. The subdrain at the back of the removal excavation should consist of an eight inch perforated pipe surrounded by five cubic feet of gravel per foot of subdrain. It was observed in the boring and reported by other consultants, that most of the seepage within the landslide is along the base of the slide. The intent of the large subdrain at the base of the slide is to prevent the blockage of groundwater flow and the development of hydrostatic pressure. Gravel 'chimney' drains are recommended along the uphill sides of the repair. The gravel chimney drains should consist of a 12 inch wide strip of ¾ inch gravel placed between the compacted fill and the shored excavation. The chimney drains should have hydraulic connectivity to the main subdrain.

Excavation Characteristics

The borings did encounter occasional hard, cemented layers within the slide debris and bedrock. Excavation difficulty is a function of the degree of weathering and amount of fracturing within the bedrock. The bedrock generally becomes harder and more difficult to excavate with increasing depth. Hard cemented layers are also known to occur at random locations and depths and may be encountered during foundation excavation. Should a hard cemented layer be encountered, coring or the use of jackhammers may be necessary. Groundwater and caving zones may also be encountered in soldier pile excavations. Casing and/or drilling muds may be required should caving zones be encountered.

FOUNDATION DESIGN

General Conditions

The following foundation recommendations are minimum requirements. The structural engineer may require footings that are deeper, wider, or larger in diameter, depending on the final loads.

Spread Footings

Continuous and/or pad footings may be used to support the proposed buildings and garage retaining walls provided they are founded in bedrock, approved compacted fill (buildings 1 and 2) or alluvial terrace. Continuous footings should be a minimum of 12 inches in width. Pad footings should be a minimum of 24 inches square. The following chart contains the recommended design parameters.

Bearing Material	Minimum Embedment Depth of Footing (Inches)	Vertical Bearing (psf)	Coefficient of Friction	Passive Earth Pressure (pcf)	Maximum Earth Pressure (psf)
Bedrock	12	4,000	0.35	500	6,000
Alluvial Terrace	12	1,500	0.3	300	3,000
Future Compacted Fill	18	1,500	0.3	300	3,000

Increases in the bearing values of the compacted fill, terrace and bedrock are allowable at a rate of 20 percent for each additional foot of footing width or depth to a maximum of 3,000 pounds per square foot for the fill and terrace and 6,000 pounds per square foot for the bedrock. For bearing calculations, the weight of the concrete in the footing may be neglected.

The bearing values shown above are for the total of dead and frequently applied live loads and may be increased by one third for short duration loading, which includes the effects of wind or seismic forces. When combining passive and friction for lateral resistance, the passive component should be reduced by one third.

All continuous footings should be reinforced with a minimum of four #4 steel bars; two placed near the top and two near the bottom of the footings. Footings should be cleaned of all loose soil, moistened, free of shrinkage cracks and approved by the geologist prior to placing forms, steel or concrete.

Deepened Foundations - Friction Piles

Drilled, cast in place concrete friction piles are recommended to support portions of the proposed buildings located over deep fill and adjacent to slopes to achieve the required slope setbacks. Also, piles are recommended to support the southern portion of Building 2 below the 1:1 setback plane. Piles should be a minimum of 24 inches in diameter and a minimum of eight feet into bedrock or eight feet into fill below the setback plane. Piles may be assumed fixed at three feet into bedrock or three feet into fill below the setback plane. The piles may be designed for a skin friction of 700 and 500 pounds per square foot for that portion of pile in contact with the bedrock and compacted fill, respectively. All piles should be tied in two horizontal directions with grade beams.

Lateral Design

The existing fill and soil on the site are subject to downhill creep. Pile shafts are subject to lateral loads due to the creep forces. Pile shafts should be designed for a lateral load of 1,000 pounds per linear foot for each foot of shaft exposed to the existing fill and soil. Friction piles supporting the portion of Building 2 within the foundation zone should be designed for an arbitrary creep force of 5 kips, with a point of application at the ground surface.

The friction values are for the total of dead and frequently applied live loads and may be increased by one third for short duration loading, which includes the effects of wind or seismic forces. Resistance to lateral loading may be provided by passive earth pressure within the bedrock.

Passive earth pressure may be computed as an equivalent fluid having a density of 380 pounds per cubic foot. The maximum allowable earth pressure is 6,000 pounds per square foot. For design of isolated piles, the allowable passive and maximum earth pressures may be increased by 100 percent. Piles spaced more than 2½ pile diameters on center may be considered isolated.

Foundation Settlement

Settlement of the foundation system is expected to occur on initial application of loading. A settlement of ¼ to ½ inch may be anticipated. Differential settlement should not exceed ¼ inch.

Foundation Setback

The Building Code requires that foundations be a sufficient depth to provide horizontal setback from a descending slope steeper than 3:1. The required setback is ⅓ the height of the slope with a minimum of five feet and a maximum of 40 feet measured horizontally from the base of the foundation to the slope face.

Toe of Slope Clearance

The Building Code requires a level yard setback between the toe of an ascending slope and the rear wall of the proposed structure of one half the slope height to a maximum 15 feet clearance for slopes steeper than 3:1. For retained slopes, the face of the retaining wall is considered the toe of the slope.

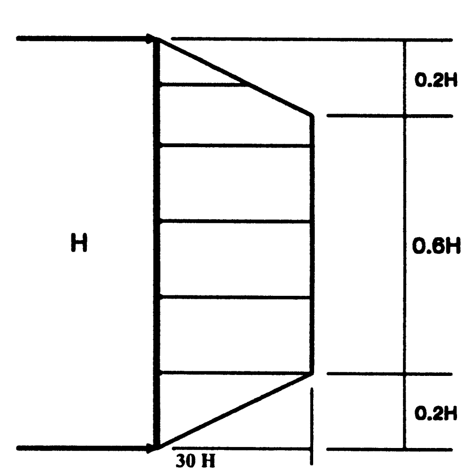
RETAINING WALLS

General Design

Cantilevered retaining walls up to 15 feet high, supporting compacted fill with backslopes between level and 2:1, may be designed for an equivalent fluid pressure of 43 pounds per cubic foot. Cantilevered retaining walls higher than 15 feet will require specific calculations based upon the backslope and surcharge conditions. Restrained basement and parking garage walls, where wall deflection is limited, should be designed for a pressure of 30H, where H is the height of the restrained wall in feet. The pressure distribution on restrained walls is assumed to be trapezoidal as shown in the figure. Retaining walls should be provided with a subdrain or weepholes covered with a minimum of 12 inches of ¾ inch crushed gravel.

Backfill

TRAPEZOIDAL DISTRIBUTION OF PRESSURE



Retaining wall backfill should be compacted to a minimum of 90 percent of the maximum density as determined by ASTM D 1557-91, or equivalent. Where access between the retaining wall and the temporary excavation prevents the use of compaction equipment, retaining walls should be backfilled with ¾ inch crushed gravel to within two feet of the ground surface. Where the area between the wall and the excavation exceeds 18 inches, the gravel must be vibrated or wheel-rolled, and tested for compaction. The upper two feet of backfill above the gravel should consist of a compacted fill blanket to the surface. Retaining wall backfill should be capped with a paved surface drain.

Foundation Design

Retaining wall footings may be sized per the "Deepened" and "Spread Footings" sections of this report.

Freeboard

Retaining walls surcharged by a sloping condition should be provided freeboard for slough protection. For manufactured 2:1 slopes, a minimum of 12 inches of freeboard is recommended. For retaining walls supporting existing or natural slopes, the recommended freeboard is 18 inches. An open "V" drain should be placed behind the wall so that all upslope flows are directed around the structure to the street or approved location.

TEMPORARY EXCAVATIONS - SOLDIER PILES

Temporary excavations will be required for removing the landslide and constructing the proposed buildings and retaining walls. Soldier piles are also recommended as part of the stabilization plan to support the compacted fill laterally and to increase the safety factor. Southeast facing vertical excavations are not recommended in the slide debris. All southeast facing excavations in the slide debris should be trimmed to 1:1 or along other flatter planes of weakness. Non-southeast facing temporary excavations in the slide debris may be created vertically up to five feet high. Where non-southeast facing vertical excavations in the slide debris exceed five feet in height, the upper portion should be trimmed to 1:1 (45 degrees). Northeast-facing excavations in the bedrock will unsupport bedding in the down-dip direction. Northeast-facing excavations should be trimmed to 1:1, or shored.

Soldier piles will be required to support temporary excavations and the landslide along the uphill property line and to support offsite properties (Soldier Piles P1 through P40 on the Geologic Map). Soldier piles will also be required to support excavations along the downhill (southern) property line. Soldier piles should be spaced a maximum of 10 feet on center. Piles may be assumed fixed at 10 feet into bedrock below the slide debris, below the 1½:1 setback plane, or below the base of the excavation, whichever is deeper.

As discussed above, the temporary load on soldier piles P1 through P10 is 170 kips per foot. From P17 to P35, the recommended design force is 145 kips per foot. Between piles P10 and P17, the design force should decrease linearly from 170 to 145 kips per foot. The point of application is assumed to be ⅓ the retained height of the pile. Piles P1 through P35 should be embedded in the bedrock below the base of the slide.

Piles P36 through P40 extend beyond the limits of the slide and are intended to support the existing properties along Revello Drive. Piles P36 through 40 should be founded below a 1½:1 plane projected up from the base of the slide as shown in Sections L, K, and O. The recommended design equivalent fluid pressure is 65 pounds per cubic foot for the portion of the pile between the ground surface and the 1½:1 setback plane. Piles along the upslope property line may also be utilized to support temporary vertical excavations to construct the required rear yard retaining walls.

Due to the large forces and high retaining heights, cantilevered piles may not be feasible. Bracing, rakers, tie-back anchors, and additional row(s) of soldier piles, may be used to assist the property line retaining walls. Slopes may be trimmed offsite to reduce the heights of shored excavations with permission from the offsite property owner. The installation of tie-back anchors offsite will also require permission from the offsite property owner.

Lateral Design - Soldier Piles

Resistance to lateral loading may be provided by passive earth pressure within the bedrock. Passive earth pressure may be computed as an equivalent fluid having a density of 380 pounds per cubic foot. The maximum allowable earth pressure is 6,000 pounds per square foot. For design of isolated piles, the allowable passive and maximum earth pressures may be increased by 100 percent. Piles spaced more than 2½ pile diameters on center may be considered isolated.

Tie-back Anchors

Tie-back earth anchors may be used to assist the soldier piles in resisting the lateral loads. Either friction anchors or belled anchors may be used. However, it has been our experience that friction anchors involve fewer installation problems and provide more uniform support than belled anchors.

For design purposes, the active wedge within the slide debris is defined by the base of the slide as shown in the cross sections. For earth anchors remote to the slide, it is assumed that the active wedge adjacent to the shoring is defined by a plane drawn at 35 degrees with the vertical through the bottom of the excavation. Friction anchors should extend at least 25 feet beyond the potential active wedge, or to a greater length if necessary to develop the desired capacities.

Testing

The capacities of the anchors should be determined by testing of the initial anchors as outlined in a following section. For preliminary design purposes, it is estimated that drilled friction anchors will develop an average value of 400 pounds per square foot. Only the frictional resistance developed beyond the active wedge should be considered in resisting lateral loads. If the anchors are spaced at least six feet on center, no reduction in the capacity of the anchors need be considered due to group action.

The frictional resistance between the soldier piles and the retained earth may be used in resisting a portion of the downward component of the anchor load. The coefficient of friction between the soldier piles and the retained earth may be taken as 0.35. (This value is based on the assumption that uniform full bearing will be developed between the steel soldier beam and the lean-mix concrete and between the lean-mix concrete and the retained earth). In addition, the soldier piles below the excavated level may be used to resist downward loads. The downward frictional resistance between the concrete soldier piles and the soils below the excavated level may be taken as equal to 700 pounds per square foot.

The anchors may be installed at angles of 20 to 40 degrees below the horizontal. Caving and sloughing of the anchor hole should be anticipated and provisions made to minimize such caving and sloughing. Groundwater and seeps should be anticipated for anchors drilled within the slide debris. The anchors should be filled with concrete placed by pumping through the auger from the tip out, and the concrete should extend from the tip of the anchor to the active wedge. To minimize chances of caving and sloughing, that portion of the anchor shaft within the active wedge should be backfilled with sand before testing the anchor. This portion of the shaft should be filled tightly and flush with the face of the excavation. The sand backfill should be placed by pumping; the sand may contain a small amount of cement to facilitate pumping.

Our representative should select at least eight of the initial anchors for a 24-hour 200% test and eight additional anchors for quick 200% tests. The purpose of the 200% tests is to verify the friction value assumed in design. The anchors should be tested to develop twice the assumed friction value. Anchor rods of sufficient strength should be installed in these anchors to support the 200 percent test loading. Where satisfactory tests are not achieved on the initial anchors, the anchor diameter and/or length should be increased until satisfactory test results are obtained. The total deflection during the 24-hour 200% test should not exceed 12 inches. During the 24-hour test, the anchor deflection should not exceed 0.75 inch measured after the 200% test load is applied. If the anchor movement after the 200% load has been applied for 12 hours is less than 0.5 inch, and the movement over the previous four hours has been less than 0.1 inch, the 24-hour test may be terminated.

For the quick 200% tests, the 200% test load should be maintained for 30 minutes. The total deflection of the anchor during the 200% quick tests should not exceed 12 inches; the deflection after the 200% test load has been applied should not exceed 0.25 inch during the 30-minute period.

All of the anchors should be pretested to at least 150% of the design load; the total deflection during the test should not exceed 12 inches. The rate of creep under the 150% test should not exceed 0.1 inch over a 15-minute period for the anchor to be approved for the design loading.

After a satisfactory test, each anchor should be locked-off at the design load. The locked-off load should be verified by rechecking the load in the anchor. If the locked-off load varies by more than 10% from the design load, the load should be resent until the anchor is locked-off within 10% of the design load.

The installation of the anchors and the testing of the completed anchors should be observed by our firm.

Lagging

Continuous lagging is anticipated for shoring piles supporting slide debris. The soldier piles should be designed for the full anticipated lateral pressure. However, the pressure on the lagging will be less due to arching in the soils. Lagging should be designed for the recommended earth pressure, but may be limited to a maximum value of 400 pounds per square foot.

Rakers

Rakers may be used to internally brace the soldier piles. The raker bracing could be supported laterally by temporary concrete footings (deadmen) or by the permanent interior footings. For design of temporary footings or deadmen, poured with the bearing surface normal to rakers inclined at 45 degrees, a bearing value of 4,000 pounds per square foot may be used, provided the shallowest point of the footing is at least one foot below the lowest adjacent grade.

Deflection

Some deflection of the shored embankment should be anticipated. If excessive deflection occurs during construction, additional bracing may be necessary to minimize deflection. If desired to reduce the deflection of the shoring, a greater active pressure could be used in the shoring design. Monitoring of the performance of the shoring system is recommended. The monitoring should consist of periodic surveying of the lateral and vertical locations of the tops of all the soldier piles. Also, some means of periodically checking the load on selected anchors may be necessary.

The geologist should be present during grading to see temporary slopes. All excavations should be stabilized within 30 days of initial excavation. Water should not be allowed to pond on top of the excavations nor to flow toward it. No vehicular surcharge should be allowed within three feet of the top of the cut.

FLOOR SLABS, DECKING AND PAVING

Concrete floor slabs and concrete decking should be cast over bedrock or approved compacted fill and reinforced with a minimum of #4 bars on 16 inch centers, each way. Slabs which will be provided with a floor covering should be protected by a polyethylene plastic vapor barrier. The barrier should be covered with a thin layer of sand, about one inch, to prevent punctures and aid in the concrete cure.

Decking which caps a retaining wall should be provided with a flexible joint to allow for the normal one to two percent deflection of the retaining wall. Decking which does not cap a retaining wall should not be tied to the wall. The space between the wall and the deck will require periodic caulking to prevent moisture intrusion into the retaining wall backfill.

It should be noted that cracking of concrete floor slabs is very common during curing. The cracking occurs because concrete shrinks as it dries. Crack control joints which are commonly used in exterior decking to control such cracking are normally not used in interior slabs. The reinforcement recommended above is intended to reduce cracking and its proper placement is critical to the slab's performance. The minor shrinkage cracks which often form in interior slabs generally do not present a problem when carpeting, linoleum, or wood floor coverings are used. The slab cracks can, however, lead to surface cracks in brittle floor coverings such as ceramic tile. A mortar bed or slip sheet is recommended between the slab and tile to limit, the potential for cracking.

Paving

Paving should be placed over bedrock, terrace, or approved compacted fill. Base course should be compacted to at least 95 percent of the of the maximum dry density. Trench backfill below paving, should be compacted to 90 percent of the maximum dry density. Irrigation water should be prevented from migrating under paving. The following table shows the recommended pavement sections:

Service	Pavement Thickness (Inches)	Base Course (Inches)
Light Passenger Cars	3	4
Moderate Trucks (Storage, etc.)	4	6

DRAINAGE

Control of site drainage is important for the performance of the proposed project. Roof gutters are recommended for the proposed structures. Pad and roof drainage should be collected and transferred to the street or approved location in non-erosive drainage devices. Drainage should not be allowed to pond on the pad or against any foundation or retaining wall. Drainage should not be allowed to flow uncontrolled over any descending slope. Planters located within retaining wall backfill should be sealed to prevent moisture intrusion into the backfill. Planters located next to raised floor type construction also should be sealed to the depth of the footings. Drainage control devices require periodic cleaning, testing and maintenance to remain effective.

WATERPROOFING

Interior and exterior retaining walls are subject to moisture intrusion, seepage, and leakage and should be waterproofed. Waterproofing paints, compounds, or sheeting can be effective if properly installed. Equally important is the use of a subdrain that daylights to the atmosphere. The subdrain should be covered with ¾ inch crushed gravel to help the collection of water. Yard areas above the wall should be sealed or properly drained to prevent moisture contact with the wall or saturation of wall backfill.

Construction of raised floor buildings where the grade under the floor has been lowered for joist clearance can also lead to moisture problems. Surface moisture can seep through the footing and pond in the underfloor area. Positive drainage away from the footings, waterproofing the footings, compaction of trench backfill and subdrains can help to reduce moisture intrusion.

PLAN REVIEW

Formal plans ready for submittal to the Building Department should be reviewed by The J. Byer Group. Any change in scope of the project may require additional work.

SITE OBSERVATIONS DURING CONSTRUCTION

The Building Department requires that the geotechnical company provide site observations during construction. The observations include foundation excavations, tie-back excavations, shoring piles, keyways for fill, benching, and temporary slopes. All fill that is placed should be tested for compaction and approved by the soils engineer prior to use for support of engineered structures. The City of Los Angeles requires that all retaining wall subdrains be observed by a representative of the geotechnical company and the City Inspector.

Please advise The J. Byer Group, Inc. at least 24 hours prior to any required site visit. The agency approved plans and permits should be at the jobsite and available to our representative. The project consultant will perform the observation and post a notice at the jobsite of his visit and findings. This notice should be given to the agency inspector.

FINAL INSPECTION

Many projects are required by the agency to have final geologic and soils engineering reports upon completion of the grading.

CONSTRUCTION SITE MAINTENANCE

It is the responsibility of the contractor to maintain a safe construction site. When excavations exist on a site, the area should be fenced and warning signs posted. All pile excavations must be properly covered and secured. Soil generated by foundation and subgrade excavations should be either removed from the site or properly placed as a certified compacted fill. Soil must not be spilled over any descending slope. Workers should not be allowed to enter any unshored trench excavations over five feet deep.

GENERAL CONDITIONS

This report and the exploration are subject to the following NOTICE. Please read the NOTICE carefully, it limits our liability.

NOTICE

In the event of any changes in the design or location of any structure, as outlined in this report, the conclusions and recommendations contained herein may not be considered valid unless the changes are reviewed by us and the conclusions and recommendations are modified or reaffirmed after such review.

The subsurface conditions, excavation characteristics, and geologic structure described herein and shown on the enclosed cross sections have been projected from excavations on the site as indicated and should in no way be construed to reflect any variations that may occur between these excavations or that may result from changes in subsurface conditions.

Fluctuations in the level of groundwater may occur due to variations in rainfall, temperature, irrigation, and other factors not evident at the time of the measurements reported herein. Fluctuations also may occur across the site. High groundwater levels can be extremely hazardous. Saturation of earth materials can cause subsidence or slippage of the site.

If conditions encountered during construction appear to differ from those disclosed herein, notify us immediately so we may consider the need for modifications. Compliance with the design concepts, specifications or recommendations during construction requires the review of the engineering geologist and geotechnical engineer during the course of construction.

THE EXPLORATION WAS PERFORMED ONLY ON A PORTION OF THE SITE, AND CANNOT BE CONSIDERED AS INDICATIVE OF THE PORTIONS OF THE SITE NOT EXPLORED.

This report is issued and made for the sole use and benefit of the client, is not transferable and is as of the exploration date. Any liability in connection herewith shall not exceed the fee for the exploration. No warranty, expressed or implied, is made or intended in connection with the above exploration or by the furnishing of this report or by any other oral or written statement.

THIS REPORT WAS PREPARED ON THE BASIS OF THE PRELIMINARY DEVELOPMENT PLAN FURNISHED. FINAL PLANS SHOULD BE REVIEWED BY THIS OFFICE AS ADDITIONAL GEOTECHNICAL WORK MAY BE REQUIRED.

The J. Byer Group appreciates the opportunity to provide our service on this project. Any questions concerning the data or interpretation of this report should be directed to the undersigned.

Respectfully submitted,
THE J. BYER GROUP, INC.

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Shear Test Diagrams (9)
Regional Geologic Map
Log of Borings 1-5 (13 Pages)
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Log of Borings by AAKO (10)
Log of Borings by Lockwood-Singh (12)
Log of Borings by Pacific Soils (11)
Calculation Sheets (42)

In Pocket: Geologic Map
Sections A through P

xc: (5) Addressee
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(1) Gary Safronoff
(3) City of Los Angeles, Department of Building and Safety, Grading Section