55.00

Borehole logs for
Test well #1 (= WW1)
Test well #2 (= WW2a)

DEWATERING TEST RESULTS AND RELATED ANALYSES
Abalone Cove Landslide
Rancho Palos Verdes, California

Job No.: 1372-98
Log No.: 4462
August 16, 1979

PREPARED FOR:

City of Rancho Palos Verdes
30940 Hawthorne Boulevard
Rancho Palos Verdes, California 90274

Attention: Mr. Dennis J. Pikus
Public Works Director

Robert Stone & Associates

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Attention: Mr. Dennis J. Pikus
Public Works Director

Subject: DEWATERING TEST RESULTS AND RELATED ANALYSES
Abalone Cove Landslide
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Gentlemen:

This report presents the results of our investigation undertaken to determine the feasibility of dewatering the active Abalone Cove landslide and to determine the potential stabilizing effect dewatering should have on the landslide. Our investigation included:

1. Drilling, logging, casing, gravel packing and testing of two test wells. We also drilled and cased two monitoring wells.

2. An analysis of groundwater conditions within the active Abalone Cove landslide and the effectiveness of the proposed dewatering wells.

3. Laboratory testing of the residual shear strength of remolded bentonite from Test Well No. 2.

4. Calculations to determine the effect of dewatering on the stability of the landslide.

5. Analysis of data on slide movement and groundwater conditions which has been obtained since completion of our report dated 2/28/79.

6. Preparation of this report.

This report supplements our report entitled: FINAL REPORT, Geotechnical Investigation of Abalone Cove Landslide, Rancho Palos Verdes, Los Angeles County, California, dated 2/28/79.

TEST WELLS

Two dewatering test wells have been drilled and tested. Test well No. 1 is located 120 feet northeast of the intersection of Narcissa Drive and Ginger Root Drive and about 300 feet north of the head of the active landslide. Test Well No. 2 was originally planned for Lot 58 adjacent to Figtree Road. However, the location was changed after two attempts to drill this site failed because of loss of drilling fluid into underground cavities above the water table. Instead, Test Well No. 2 is located about 100 feet west of Narcissa Drive and 500 feet north of Palos Verdes Drive South.

Test Well No. 1

Test Well No. 1 was drilled to a total depth of 195 feet with a diameter of 12½ inches. It is fitted with 8-5/8 inch steel casing to 90 feet and gravel packed. The bottom 40 feet of casing is perforated. The static water level in the well stood at 51.5 feet below the ground surface on July 27. The well penetrates landslide debris consisting mainly of brecciated siliceous shale to a depth of 115 feet. Below this is weathered basalt and then unweathered basalt to a depth of 147 feet. Siliceous shale occurs from a depth of 147 feet to about 170 feet below which is bentonite and bentonitic tuff (Portuguese tuff).

An observation well located 28.3 feet from Test Well No. 1 was drilled to a depth of about 72 feet. The static water level stood at a depth of 52.4 feet below the ground surface on July 27.

Test Well No. 1 was tested on July 27 using a pump powered by an MM Twin City gasoline engine. The first pumping averaged 118 gallons per minute (gpm), but after 78 minutes the drawdown in the pumping well was only 0.5 feet. After 88 minutes the discharge was increased to 175 gpm and drawdown in the pumping well was 7.5 feet. The well pumped some sand. After 2 hours, 19 minutes the pump was shut off and adjusted. It was then pumped at the maximum rate of which the pump was capable, yielded 215 gpm with a drawdown of 7.5 feet. Total time of the test was 4 hours, 3 minutes.

The observation well did not show the progressive drawdown that is normal in a pumping test. Drawdown of nearly 2 feet was measured at one time, but at the end of the test drawdown was only 1 foot, and during much of the test there was no drawdown at all. The behavior of this well suggests that the principal aquifer contributing to the yield was open fractures in the basalt, extending from 115 to 147 feet deep. The observation well, only 72 feet deep, did not reach the basalt and, therefore, was not directly affected by the pumping.

Because of the behavior of the observation well, and the fact that the pumping well was pumped at various discharges up to its maximum, the best estimate of transmissivity is obtained from the modified Thiem formula $T = 1460\frac{Q}{s}$ developed by the U. S. Geological Survey and utilized by the California Department of Water Resources. In this "shortcut" formula for unconfined aquifers, T is transmissivity in gal/day/ft; Q is discharge in gpm, and s is drawdown in feet.

For this test, then,

$$T = 1460 \times \frac{215}{7.5} = 41{,}850 \text{ gal/day/ft, or in round numbers,}$$

$$= 42{,}000 \text{ gal/day/ft.}$$

These data suggest that Test Well No. 1 should yield 400 to 500 gpm without difficulty using a pump of appropriate size.

Test Well No. 2

Test Well No. 2 was drilled to a depth of 146 feet with a diameter of 12¼ inches. The well has 8-5/8 inch diameter casing to a depth of 102 feet and is gravel packed. The casing is perforated from 54 to 74 feet below the surface. The static water level stood at 52 feet below the surface on July 30. The well penetrated landslide debris consisting mainly of chert and shale fragments to a depth of 69 feet. Below that was mostly chert to 80 feet. Bentonite predominates from 80 to 89 feet followed by chert and siliceous shale to 141 feet. Bentonite and bentonitic tuff (Portuguese tuff) occur from 141 feet to the bottom at 146 feet.

An observation well was drilled 28.3 feet from Well No. 2 and reached a total depth of 100 feet. A 2 inch plastic pipe was set in the observation well, which was gravel packed.

A pumping test was conducted on July 30. Static level in the pumping well was 51.9 feet below ground surface, and in the observation well 51.1 feet below the top of the casing. Pumping was begun at 110 gpm, but the pumping well quickly drew down to the bowles, which were set at 90 feet according to the driller. Discharge was then intermittent.

The pump was then stopped, and re-started at a slower speed with the discharge valve partly closed. Discharge was then continuous at 19.4 gpm. Approximately one and a quarter hours later, the gasoline engine began to falter, and its speed was increased to maintain the pumping. The new rate of discharge, measured at 33 gpm, was sustained until the pump was shut off 3 hours and 2 minutes after the second start.

A good record of progressive drawdown was obtained in the observation well, although the drawdown was a little irregular at first because of the early heavier pumping. Transmissivity (T) and storage coefficient (S) were obtained by the Jacob method.

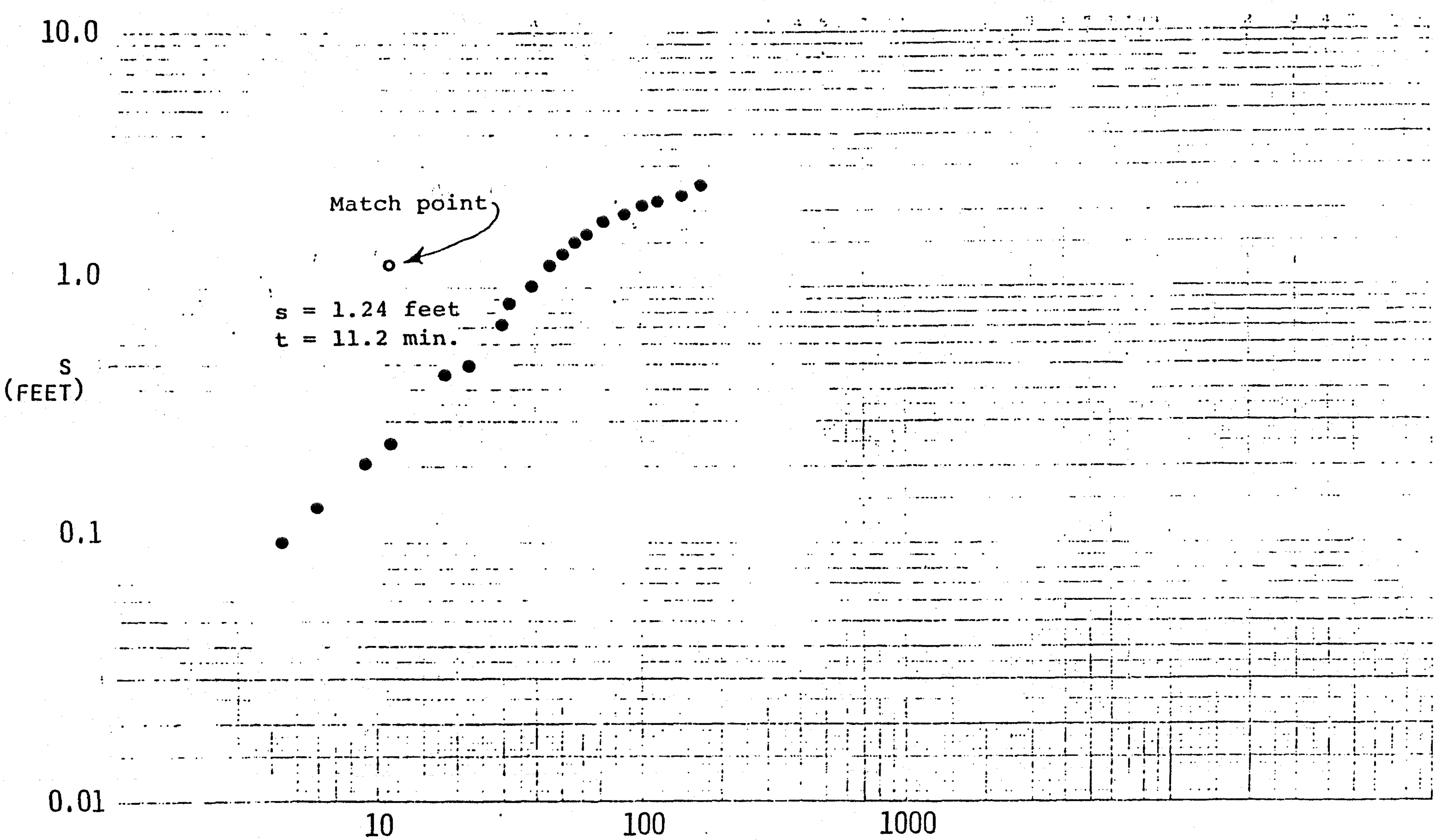
$$T = \frac{2.3\ Q}{4\Pi s} \log t$$

s in this formula is the change in drawdown over 1 log cycle in a semi-log plot (see Figure 1).
t is time, and units must be made consistent. The position and slope of the line is determined by the longer times during the test for two reasons: the formula is not valie for short times, and the shorter times were more greatly affected by the large drawdown at the early pumping rate that could not be maintained.

$$T = 2.3 \times 27 \times 1440 \times 1 = 3{,}558 \text{ gal/day/ft, or in round numbers}$$
$$= 3{,}500 \text{ gal/day/ft}$$

$$S = \frac{2.25\ T\ t_o}{r^2}$$

TEST WELL NO. 2



Here t_o is the intercept on the zero drawdown axis, and r is the distance between the two wells.

$$S = \frac{2.25 \times \frac{3500}{7.48} \times \frac{9}{1440}}{(28.3)^2} = 0.008$$

Transmissivity as found in this test is less than one-tenth that found at Test Well No. 1. The two principal reasons for this appear to be: the presence of a fractured basalt aquifer at Test Well No. 1; and the fact that only 20 feet of casing was perforated in Water Well No. 2, and that the material contains clay between rock fragments. Our test indicates that the well can produce at least 30 gpm as presently developed. The yield should be increased by perforating the lower 28 ft. of casing and by further test pumping for development.

The storage coefficient, S, should not be considered equivalent to specific yield in this test (see later section on storage capacity). A much longer test in an unconfined aquifer of this type would be needed for S to approach specific yield.

FEASIBILITY OF DEWATERING THE ABALONE COVE LANDSLIDE

The effectiveness of a dewatering system depends upon: (1) the volume of water stored within the slide; (2) the rate at which new water enters the slide; (3) the rate of natural outflow along the toe of the slide, and (4) the capacity of the dewatering system to remove water. Because the stability of the landslide is improved as the water table is lowered within the slide mass, the rate at which the water table is lowered is a measure of the effectiveness of the de-watering system.

Groundwater Stored in Landslide

The volume of water stored in the landslide area can be estimated by the equation: $V=A \cdot H \cdot Y$. Where V is volume of water, A is surface area of the landslide, H is the mean vertical distance between the slide plane and overlying water table (see Plate 3), and Y is the specific yield (the amount of water a unit volume of saturated material will yield). For the purpose of our calculations, the 30 acres of slide to the south of Palos Verdes Drive South is separated from the 50 acres to the north of Palos Verdes Drive South. For the 30 acres, the available data indicate that the water table averages about 20 feet above the base of the slide. We estimate the average specific yield to be only about 2½% because the area of highest water table is underlain by a bedrock block with a specific yield close to zero. (The groundwater is stored in fissures and fractures within blocks of bedrock and in openings between rock fragments in slide debris.) Thus the volume of water stored in the 30 acres is estimated to be 15 acre-feet ($V=30x20x0.025$). For the northern 50 acres, available data indicate the water table averages about 40 feet above the slide plane. The average specific yield is estimated to be 4% in this area. Thus, an estimated 80 acre-feet of groundwater is stored in the northern 50 acres.

Sources of New Water

New water enters the landslide by (1) migration of groundwater from upslope within the Altamira Canyon drainage system where it was derived from rainfall, (2) westward migration of groundwater from the head of the Portuguese Bend landslide where ponded runoff from Portuguese Canyon percolates into slide debris, (3) domestic water introduced primarily as sewage, and (4) direct percolation of rainfall and runoff down Altamira Canyon into the landslide during winter storms.

Of the four sources of water, that derived from domestic water can be estimated with the greatest precision. An analysis presented in our report of 2/28/79 estimates that 1.5 million cubic feet per year (34 acre-feet per year) of water are introduced as sewage by the 20 residences within the active slide and the 70 residences within the drainage area directly upslope from the slide. This amounts to an average of 0.02 acre-feet per day being introduced directly into the landslide and 0.07 acre-feet per day being introduced into the groundwater system above the slide. A small additional amount of water is introduced from outdoor watering during the dry season. To account for this our estimate of the average daily infiltration of domestic water is 0.03 acre-feet within the active landslide and 0.1 in the area above the landslide. Although leaks in water pipes have contributed some water in the past, we assume that efforts to control leaks have reduced leakage to an insignificant amount.

Of the other three sources of new water, that derived directly from rainfall and associated runoff should be zero for at least the next three months. After that, it will depend upon the nature of the rainy season. If rainfall is normal, about 1 foot of rain will fall during the season. If 20% of the rainfall percolates to the water table, 10 acre-feet of water will be added to the underground reservoir within the 50 acre slide area north of Palos Verdes Drive South and 6 acre-feet in the 30 acres south of Palos Verdes Drive South. If we assume an equal amount of water is added from percolation of discharge along Altamira Canyon within the slide area, the total annual introduction from these two sources would be 20 acre-feet to the north of Palos Verdes Drive South and 12 acre-feet to the south. When expressed as an average daily inflow this amounts to 0.05 acre-feet per day within the 50 acres and 0.03 acre-feet per day within the 30 acres. We consider this estimate to be on the high side for a year with average rainfall but on the low side for a year with significantly above average rainfall.

Subsurface inflow through the head of the active landslide consists mainly of groundwater derived from percolation of rainfall within the Altamira Canyon drainage area to the north of the slide. It also includes inflow from the head of the Portuguese Bend landslide and the previously estimated 0.1 acre-feet per day of domestic water from the 70 residences north of the active slide. The total subsurface inflow through the head of the slide is estimated from Darcy's law: $Q=T \cdot I \cdot W$. Where Q is inflow, T is transmissivity under 100%

hydraulic gradient, I is the water table gradient, and W is the width of the zone of inflow. Our estimate is based on T=3,500 gallons per day per foot, I is 15 feet per 100 feet (0.15) and W=1,800 feet. Of these values, T is least certain, I varies along the head of the slide but should be close to the assigned value and W is known with good precision. The results indicate an inflow of 945,000 gallons per day or 2.9 acre-feet per day (1 acre-foot = 326,000 gallons).

Groundwater migrating westward from the upper part of the active Portuguese Bend landslide joins groundwater of the Altamira Canyon groundwater system in the area northeast of the head of the Abalone Cove landslide. We believe the water is flowing through an ancient debris-filled graben that trends nearly westward from near the intersection of Sweetbay Road and Peppertree Drive. For our estimate we use: T=5,000 gallons per day per foot, I=5 feet per 100 feet (0.05), and W-300 feet. This equates to an inflow 0.2 acre-feet per day.

A check on groundwater inflow to the landslide from the Altamira Canyon drainage area to the north can be made by estimating the water available from that area. Assuming rainfall to be the same as at Los Angeles County Flood Control Station 43D, located at 340 Palos Verdes Drive West, average annual rainfall would be 11.38 inches (0.95 feet) on the area of about 790 acres of the Altamira Canyon drainage north of the active Abalone Cove landslide. Rainfall was 29.61 inches (2.47 feet) in 1977-78 and 17.15 inches (1.43 feet) to date in 1978-79. Annual rainfall over the 790 acres would total about 750 acre-feet during an average year, but was about 1950 acre-feet in

1977-78 and 1130 acre-feet to date this year. These figures are slightly conservative, since rainfall on the higher elevations of Altamira Canyon drainage is greater than at Station 43D near the Coast.

It is estimated that 15 percent of the rainfall in upper Altamira Canyon drainage percolates to groundwater. Recharge from this source above the landslide is thus about 55 acre-feet in an average year, but was 295 acre-feet in 1977-78 and so far this year has been 170 acre-feet.

Groundwater recharged during these two above-normal years now constitutes the great preponderance of subsurface inflow moving into the landslide from the north. If we estimate the recharge of 1977-78 to move in during the one year period following the end of the water year (September 30), subsurface inflow to the landslide from this source is presently 0.8 acre-feet per day. This suggests that the previous calculation of 2.9 acre-feet per day by Darcy's law is too high, even though the latter includes percolation of waste water north of the landslide and inflow from Portuguese Canyon. The discrepancy is probably the result of assigning too high a value to transmissivity at the head of the Abalone Cove landslide. Clay gouge along steeply inclined slip surfaces probably impedes groundwater inflow more than assumed in our initial calculations.

Based on all information available, we estimate that inflow along the head of the Abalone Cove landslide is about 2 acre-feet per day at the present time. This figure is probably on the high side. A further check on this estimate can be made by measuring the decline in the water table after Test Well No. 1 begins to dewater part of this area.

Outflow of Groundwater

Under long-term natural conditions, outflow of groundwater along the toe of the Abalone Cove landslide must equal inflow less minor losses to deep-rooted plants and seeps along Altamira Canyon. Most of the outflow occurs as springs in the surf zone at low tide. When averaged over a period of many years, we estimate outflow is about 0.5 acre-feet per day. The present outflow is probably higher than that. Our proposed dewatering system utilizes natural outflow to dewater the 30 acres of landslide south of Palos Verdes Drive South.

Proposed Dewatering System

The proposed dewatering system consists of 6 wells, 2 north of the active landslide and 4 within the upper half of the landslide. Test Well No. 1 and one other well north of the slide would serve two functions -- to intercept water before it can flow into the slide and to improve the gross stability of this area. We estimate 2½ acre-feet of water per day (an average of 565 gallons per minute) can be removed by the two wells during early stages of pumping. The rate of removal by pumping will decrease as the water table drops and should eventually stabilize at a rate equal to inflow within the area drained by the wells.

The four proposed wells within the active slide would be located in graben areas delineated on Plate 1. These areas contain highly fractured bedrock and fragmented rock debris which backfill areas where the slide mass has been pulled apart as a result of greater slide movement on the downhill side than the uphill side. Most of the groundwater within the slide mass is stored in openings between rock fragments within these areas. These are also areas of high permeability where wells have the best chance of achieving a high production.

Test Well No. 2 would be one of the four wells. The lower 28 feet of its casing should be perforated in order to improve production. The lower part of the casing is probably experiencing deformation and may eventually seal off. Available data indicate that the base of the presently active landslide is in bentonitic beds at a depth of about 80 feet. However, material involved in ancient sliding may extend to a depth of 146 feet as suggested by the presence of intensely sheared bentonite in a core sample taken at 146 feet. The gravel pack within Test Well No. 2 will allow groundwater to move upward from the zone below the active slide surface.

Of the other three proposed wells in the active slide area, we recommend that one be placed on Lot 58 along the north side of Figtree Road near boring R1. This location is recommended because loose-textured, highly fractured material was encountered below the water table in boring R1 and water flowed rapidly into the hole during drilling. The water table was 107 feet below the surface when boring R1 was drilled on 1/23/79 and is probably a few feet higher at the present time. The base of the landslide was not penetrated in boring R1 but is about 40 feet below the water table based on extrapolations from other borings. Thus, the well should be drilled to a depth of about 150 feet with perforations in the casing below a depth of about 110 feet.

We recommend that the other two dewatering wells be placed along the channel of Altamira Canyon. One well would be at an elevation of about 220 feet on Lot 60 to the east of the cul-de-sac on Figtree Road. The water table should be about 80 feet below the surface and the slide base about 110 feet below the surface at this location. An important function of this proposed well would be to intercept groundwater infiltrating along Altamira Canyon and groundwater which may be entering along a graben which extends eastward into the Portuguese Bend landslide. The other proposed well would be along the west side of Narcissa Drive between 200 and 300 feet northeast of Palos Verdes Drive South. The water table is about 50 feet below the surface and the slide base is about 100 feet below the surface. This proposed well and Test Well No. 2 should be able to remove most of the water which would otherwise flow into the slide area south of Palos Verdes Drive South.

No wells would be placed in the 30 acre area south of Palos Verdes Drive South. This area would continue to drain naturally. The water table would subside as pumping reduces inflow from the north.

We expect the four wells within the active landslide to have a combined production of 1½ to 2½ acre-feet per day (340 to 565 gpm). With all six of the proposed wells in operation, we estimate that there will be a net reduction (supply minus disposal) of about 1 acre-foot per day in the northern 50 acres of the active landslide during the early stages of dewatering. This would cause the water table to drop an average of about ½' per day within the 50 acre area during the early stages of dewatering. As dewatering proceeds, the rate of production will diminish. Part of the water is undoubtedly trapped in materials which cannot readily drain to the wells. Therefore, we estimate that the drop in the average height of the water table will decline to about ¼ foot per day after the water table has been lowered an average of 10 feet below its original position. Production will continue to decline with further lowering of the water table. We estimate that with the proposed pumping the water table can be lowered 20 feet in 2 to 3 months and that it can be lowered 30 feet in 4 to 6 months. It is problematical as to whether the proposed wells could lower the water table much more than 30 feet below its present position.

Although we believe the six proposed dewatering wells will be effective in dewatering the Abalone Cove landslide, there is no way of assuring their effectiveness in advance. Once pumping begins, the effectiveness of the system can be evaluated based on the amount of water removed and the rate at which the water table declines. Several monitoring wells (4" rotary borings cased with 2 inch plastic pipe) should be installed in order to determine the effect pumping has on the water table at a considerable distance from the pumping wells. It may be desirable to establish additional pumping wells if the proposed system is not as effective as anticipated. Any additional wells would most likely be placed in the northwestern part of the landslide.

EFFECT OF DEWATERING ON SLIDE MOVEMENT

In order to evaluate the effect of dewatering on the stability of the active Abalone Cove landslide, we have (1) performed multicycle direct shear tests on remolded bentonite to determine the shear strength of materials along the slide plane; (2) prepared cross sections A-A' and B-B' (Plate 2) through slide mass; and (3) analyzed the stability of the slide mass in the cross sections.

Shear Strength Along Slide Plane

Where exposed in the surf zone, the base of the active Abalone Cove landslide occurs in bentonite within the stratigraphic interval known as the Portuguese tuff. The base of the active Portuguese Bend landslide also occurs within this interval. The bentonite along shear surfaces typically consists of exceptionally fine-grained waxy clay. It is common knowledge that such clay has a low residual shear strength.

For the purpose of testing the residual shear strength, we selected samples of bentonite from a core taken at a depth of 146 feet in Test Well No. 2. The bentonite consists of gray waxy clay, part of which is intensely sheared. Although the sample comes from below the base of the active landslide as shown in cross section B-B', we consider it representative of material which occurs along the slip surface.

Laboratory Testing - Repetitive Shear Tests on Slide Plane Materials

Shear strength parameters of the probable slide plane materials were established by performing repetitive direct shear tests on the samples of bentonite taken at a depth of 146 feet in Test Well No. 2. The samples were sheared at a low rate of strain under several axial loads. Each sample was sheared along the same pre-cut failure surface through several repetitions until a stabilized lower value of shear strength was obtained for a specific axial load. The failure planes were remoistened as necessary during shearing to restore any moisture which might be lost by evaporation. The results of these tests used in our calculations are as follows:

$$\phi = 8^{o}$$

$$C = 150 \text{ psf}$$

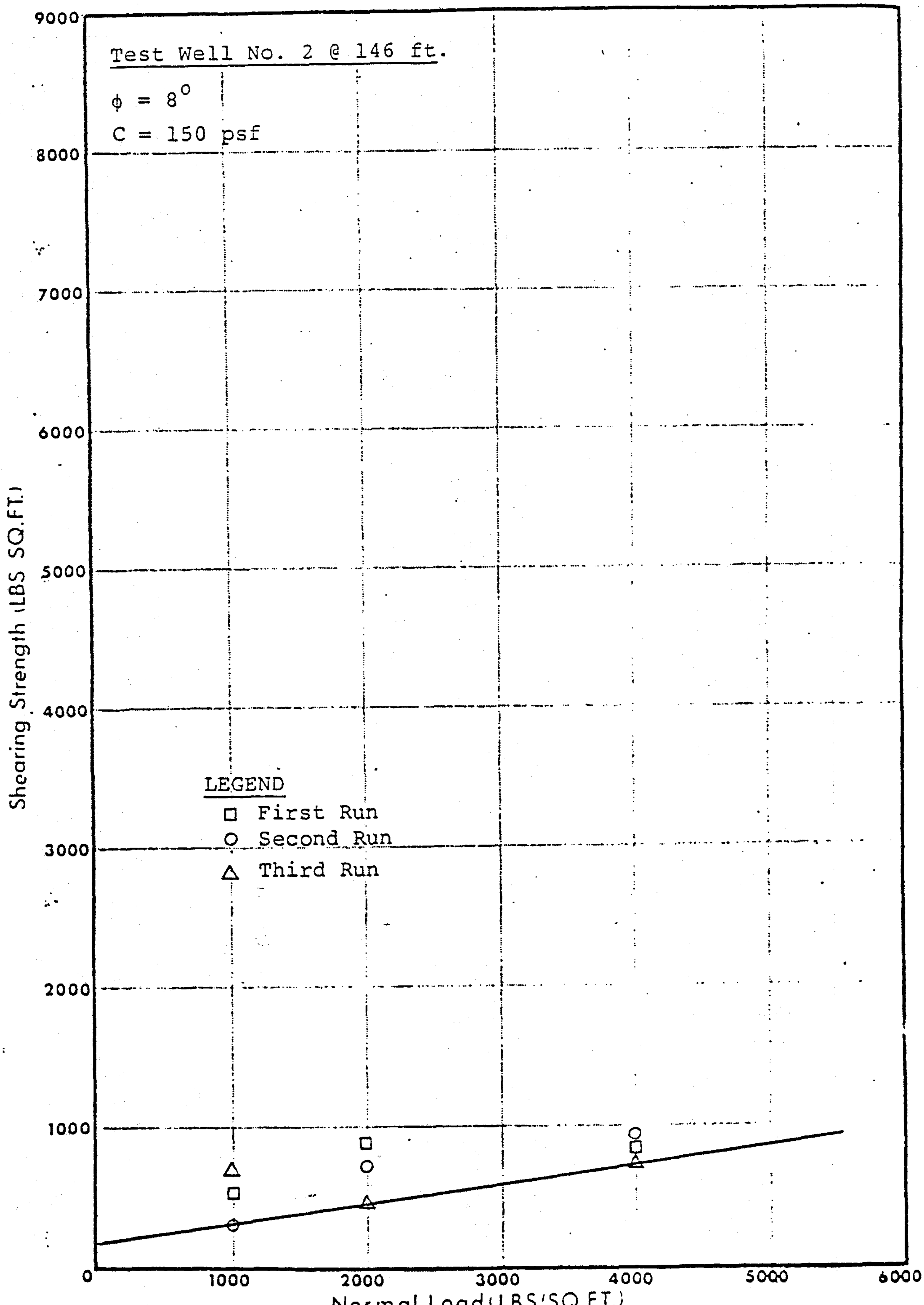
(See Figure 2)

Stability Analysis

Cross sections A-A' and B-B' show the configurations of the western and eastern parts of the slide, respectively. The translational stability of the active landslide has been analyzed along both cross sections using the residual shear strength of $\phi = 8^{o}$ and C = 150 psf obtained in our laboratory tests. Our calculations (Table 1) indicate that the active landslide would have a factor of safety of 1.04 along cross section A-A' and 1.20 along cross section B-B' if the water table were at or below the slide plane. A factor of safety of 1.11 is obtained for the total slide by combining the results obtained from the two cross sections.

RESULTS OF SHEARING STRENGTH TESTS

Undisturbed, Saturated Samples



EXPLANATION:

B-9@12' = Sample taken from Boring 9 at 12 Feet in Depth

ROBERT STONE & ASSOCIATES, INC.

Job No. 1372-98

Analysis of Translational Stability

Abalone Cove Landslide, Rancho Palos Verdes

Slide density = 125 pcf
$\phi = 8^{0}$
C = 150 psf

Cross Section and Segment	Area 10^2 ft.	Weight (w) in 10^3 lbs.	Dip of base (θ)	Driving Force W sin θ	Normal Force W cos θ	Frictional Resistance Fn tan φ	Effect of H_2O: buoyant weight	W sin θ	W Cos θ tan φ	
A I	0.43	54	-30	-27	47	7	27	-14	3	Calculated for Water Table at Sea Level
A II	2.50	312	0	0	312	44	156	0	22	
A IIIa	230.0	28,750	5.5	2756	28,618	4022	1785'x62.4cos5.5= 45 kips			reduction in normal force by 1 vertical foot of groundwater
A IVa	12.9	1,612	25	681	1,461	205	68'x62.4cos25= 4.2 kips			
A Va	8.0	1,000	45	707	707	99	47'x62.4cos45= 2.1 kips			
A VIa	4.1	512	65	464	216	30	37'x62.4cos45= 1.0 kips			
				4581 kips		4407 kips	118.3 kips			
						+ 336 kips cohesion				horizontal length of segment
						4743 kips				

Base length = 2240 ft.
x 150 cohesion
336 kips

Factor of safety = $\frac{4743}{4581}$ = $\boxed{1.04}$

Cross Section and Segment	Area 10^2 ft.	Weight (w) in 10^3 lbs.	Dip of base (θ)	Driving Force W sin θ	Normal Force W cos θ	Frictional Resistance Fn tan φ	buoyant weight	W sin θ	W Cos θ tan φ	
B I	0.67	84	-30	-42	73	10	42	-21	5	Calculated for Water Table at Sea Level
B II	5.1	637	0	0	637	90	468	0	66	
B III	153.0	19,125	4.5	1501	19,066	2680	1493'x62.4cos4.5= 93 kips			reduction in normal force by 1 vertical foot of groundwater
B IV	67	8,375	6.5	948	8,321	2269	453'x62.4cos6.5= 28 kips			
B V	8.5	1,062	25	449	962	135	54'x62.4cos25= 3.1 kips			
B VI	6.8	850	45	601	601	84	53'x62.4cos45= 2.3 kips			
B VII	2.6	325	65	295	137	19	36'x62.4cos65= 0.9 kips			
				3752 kips		4187 kips	127.3 kips			
						+ 352				
						4539 kips				

118.3
127.3
245.6 tan 8 = 34.5 kips

Base length 2350 ft.
x150
352 kips cohesion

Factor of safety = $\frac{4539}{3752}$ = $\boxed{1.21}$

Combined factor of safety A+B = $\frac{4743+4539}{4581+3752} = \frac{9282}{8333} = \boxed{1.11}$

Adjustment for segments I & II

Section A

Adjusted driving force=4594 kips

Adjusted resisting force= 4717 kips

Section B

Adjusted driving force=3773 kips

Adjusted resisting force= 4510 kips

Sum resisting forces
4717+4510 = 9227

Sum driving forces
4594+3773=8367 kips

difference = 860 kips

Height water table needs to be above slide surface for resisting force to equal driving force:
860÷34.5= $\boxed{24.9 \text{ feet}}$

Height of groundwater needed to decrease factor of safety 1%
$\frac{8367}{100 \times 34.5} = \boxed{2.4 \text{ feet}}$

Robert Stone & Associates, Inc.