U. S. FOREST SERVICE
NORTH WEST Research Note
FOREST AND RANGE EXPERIMENT STATION • U.S. DEPARTMENT OF AGRICULTURE • PORTLAND, OREGON

PNW-68 *November 1967*

SOIL-WATER PIEZOMETRY IN A SOUTHEAST ALASKA LANDSLIDE AREA

by

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INTRODUCTION

Soil-water piezometry started in the summer of 1964 and continued in 1965 as part of studies to determine debris avalanche causes. The measurements provide information on development and distribution of soil water-pore pressures and their effect on soil shear strength.

Mass movements of rock and soil in southeast Alaska are common on steep slopes resulting from glacial erosion and recent mountain building. These slopes are covered by thin, undifferentiated soils over bedrock or podzols developed on glacial tills of variable composition and thickness. Mass movements developed in these mantle materials are classified as debris avalanches or debris flows (Sharpe 1960; Highway Research Board 1958), [1/] depending on their overall water content and speed of movement. They result from loss of soil shear strength caused by both external and internal changes of stability. External changes include seismic movements, slope undercutting,

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[1/] Names and dates in parentheses refer to Literature Cited, p. 17.

or deposition on upper slopes. Internal conditions include piezometric head increase causing increased pore-water pressures and corresponding reductions in soil cohesion (Terzaghi 1950, pp. 83-123).

Bishop and Stevens (1964) indicated that clearcutting old-growth forests apparently accelerated debris avalanche and flow occurrence in the Maybeso Creek valley, Prince of Wales Island, Alaska (fig. 1). These mass movements developed on steep slopes in a glacial till soil, apparently triggered by high rainfall.

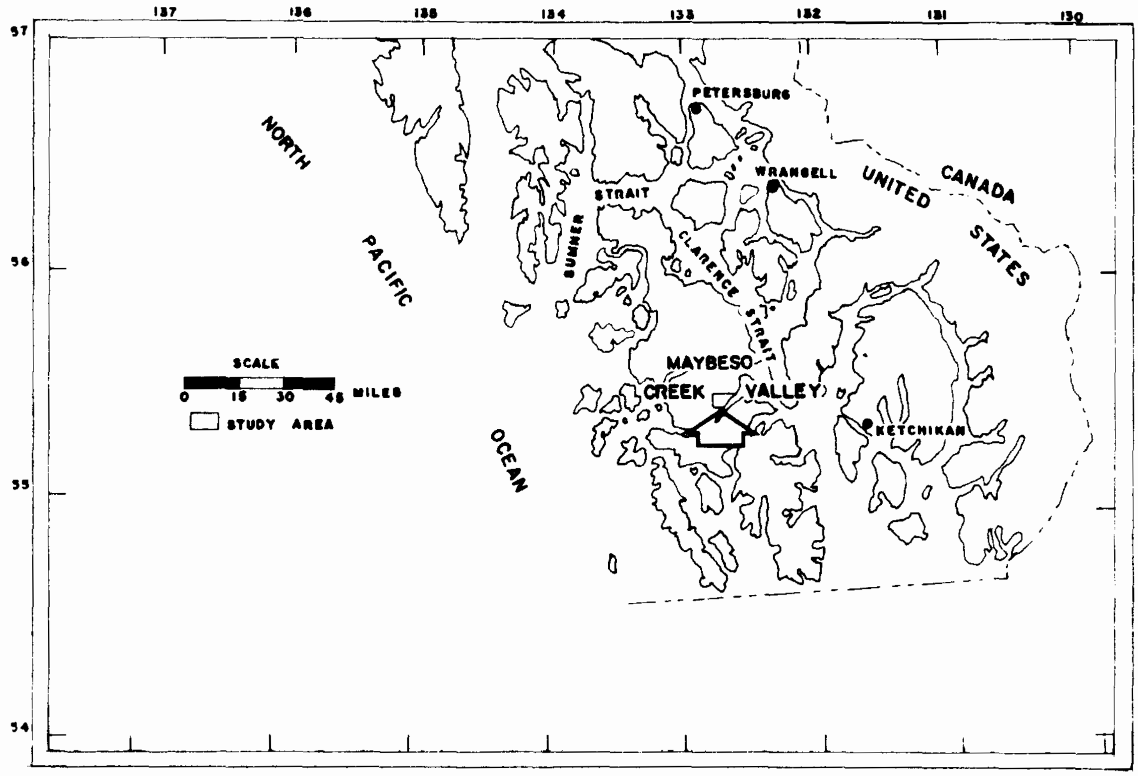

Figure 1.–Map of southeast Alaska showing location of Maybeso Creek valley.

STUDY AREA

Climate in the Maybeso Creek valley is cool, wet, and temperate. The mean annual temperature is 44° F. The mean annual precipitation is about 108 inches; rainfall is highest during the months of March, April, October, and November. Snow usually appears on upper slopes about mid-October. Recent debris avalanches and flows are associated with high rainfall in October (Bishop and Stevens 1964).

The drainage is a U-shaped glacial valley, floored by a variable thickness of glacial till extending to about 1,200-foot elevation. The valley is oriented in a west-northwest direction along the strike of the underlying bedrock. Bedrock is interbedded graywacke, shale, and black graphitic slate, dipping southwest into the valley at a variable angle. The southwest-facing slope is smooth and uniformly till covered, cut by several deep, narrow, V-shaped ravines. On the northeast-facing slope, indipping bedrock layers create a slope surface broken by benches and ridges with very few crosscutting ravines. The majority of debris avalanches and flows apparently affected by logging occur on the southwest-facing slope, either along the sides of the ravines or in incipient drainage depressions on the open slope.

The glacial till soils are in the Karta series.[2/] They are well-drained, permeable soils with textures ranging from gravelly to silty loam. A representative Karta soil profile from the study area is described in table 1. A thick organic A horizon is underlain by a yellow-brown iron-stained horizon. Compact, unweathered till lies below the iron-stained horizon. Maximum thickness of the soil on these slopes is 3 feet. Mass movements begin in the iron-stained horizon and slide along the unweathered till surface.

Principal drainage is through deep, V-shaped ravines into Maybeso Creek. Little additional surface runoff occurs. Rain falling on the slope infiltrates the soil and moves along the upper surface of impermeable, unweathered till, concentrating in subsurface drainageways developed on the unweathered till surface. The shallow depressions on open till slopes are surface manifestations of these drainageways. Only during periods of very heavy rainfall has surface flow in

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[2/] Gass, C. R., Billings, R. S., Stevens, M. E. and Stephens, F. R. Hollis area soil management report. (In preparation for publication, U.S. Forest Service, Region 10.)

these depressions been observed. It is probable that these drainage depressions were formed initially by surface runoff following deglaciation and the beginning of Karta soil formation. These depressions have been subsequently enlarged by more rapid leaching and oxidation in the subsurface zones of maximum water concentration and by incipient slumping and minor erosion during infrequent periods of surface flow.

Table 1.--A typical Karta soil profile in the Maybeso Creek valley[1/]

Horizon	Depth (inches)	Representative profile in study area
01	5- 4	Living mosses and undecomposed leaves and twigs.
02	4- 0	Reddish-black (10 R 2/1) peat; wet, nonsticky, nonplastic; many fine, medium, and few coarse roots; pH 3.5; abrupt wavy boundary.
A2	0- 1	Pale-brown (10 YR 6/3) gravelly silt loam; weak, fine, subangular blocky structure; wet, slightly sticky, slightly plastic, coarse fragments 40 percent by volume; many fine, medium, and few coarse roots, common fine and medium pores; pH 4.0; abrupt wavy boundary.
B2hir	1-14	Yellowish-red (5 YR 5/6) gravelly silt loam with dark-brown (7.5 YR 3/2) variegations; gravelly silt loam; weak, fine, and medium subangular blocky structure; wet, slightly sticky, slightly plastic; coarse fragments 40 percent by volume; common fine, few medium, and coarse roots, common fine and medium pores; pH 4.5; clear wavy boundary.
C1	14-17	Olive (5 Y 5/3) gravelly silt loam; massive; weakly cemented; 75 percent coarse fragments, few fine roots, pH 5.0; clear wavy boundary.
C2	17-21+	Olive (5 Y 5/3) gravelly silt loam; massive; strongly cemented by $CaCO_3$, with pockets of noncemented, very porous glacial till; pH 5.5, 75 percent coarse fragments by volume. In transition zone to underlying unweathered till.

[1/] From Charles Gass, soil scientist, Tongass National Forest, Alaska.

METHODS

Ten piezometers were installed during the 1964 and 1965 field seasons. The piezometer is a 1-1/2-inch, outside diameter, porous cylinder 6 or 12 inches long and connected to 1/2-inch, outside diameter, polyethylene tubing (fig. 2).

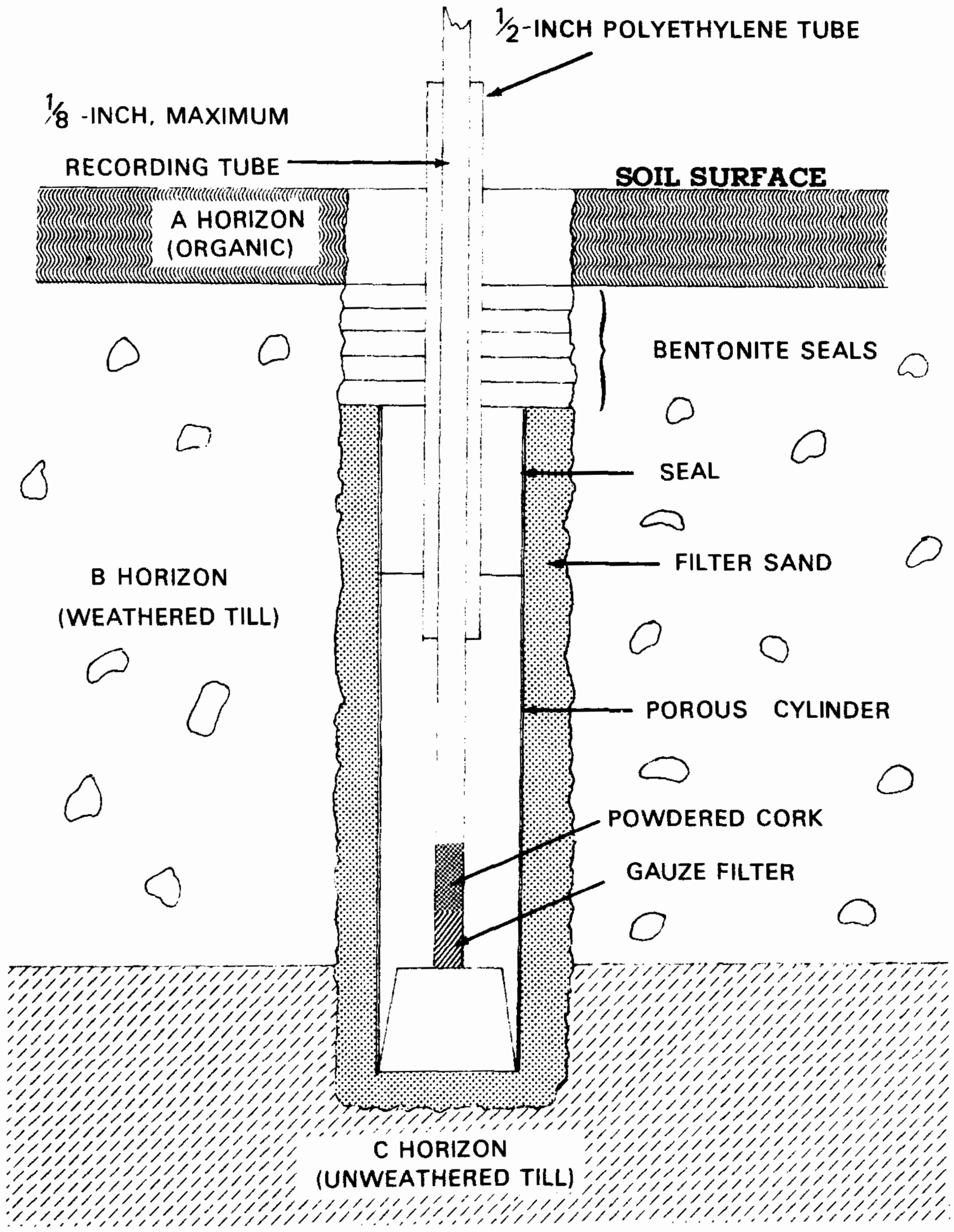


Figure 2.–Diagram of standard piezometer installation showing positioning of maximum recording tube.

The sites were on open slopes above or within the linear depressions. Slopes ranged between 35 and 40 degrees (about 70- to 85-percent grade). Piezometer locations are shown in figure 3. Several auger holes were drilled at each site to determine thickness of the weathered till. If water flowed into the hole freely and rose above the seepage zone, the site was judged satisfactory for piezometer installation. The hole was washed and filled with clean water, and 1 foot of clean, saturated sand was poured in. The porous cylinder was then inserted to the bottom of the hole with an excess head of water maintained in the cylinder at all times. After the height from the porous cylinder top to the ground surface was measured, saturated sand was poured in to fill the open space around the cylinder. A 6-inch layer of sand was poured in above the porous cylinder top, and the hole was then sealed with five layers of bentonite, each 1 inch thick. Piezometric head was measured by inserting a two-conductor wire into the polyethylene tube. When the wires touched the water surface, a circuit was completed and an ohmmeter needle deflected. Piezometric head was the difference between the length of wire inserted and the length from the open end of the polyethylene tube to the bottom of the porous cylinder.

The piezometers were read at time intervals determined by access difficulty and weather conditions. In the fall of 1964, readings were made in areas 1, 2, and 3 (fig. 3) every 3 days. More frequent readings of a modified network of piezometers was begun in 1965. Readings were limited to area 1 and vicinity, where the largest number of wells were located, and area 4 on the north-facing slope of Maybeso Creek valley.

In 1965, a reliable method was devised for recording absolute maximum piezometric rise. A polyethylene tube, about 1/8 inch inside diameter, with powdered cork in the bottom, was inserted into each operating piezometer (fig. 2). As the piezometric surface rose, powdered cork was carried upward on the water surface. When the water level fell, cork adhered to the tube walls at the point of maximum rise.

In 1964, rainfall was measured at a sea-level weather station near the mouth of Maybeso Creek. In 1965, rain was also measured with a recording gage placed in area 1, at an elevation of about 1,000 feet.

RESULTS

Figures 4 and 5 show the relationship of piezometric head to rainfall during the 1964 and 1965 fall rainy seasons. The curves of

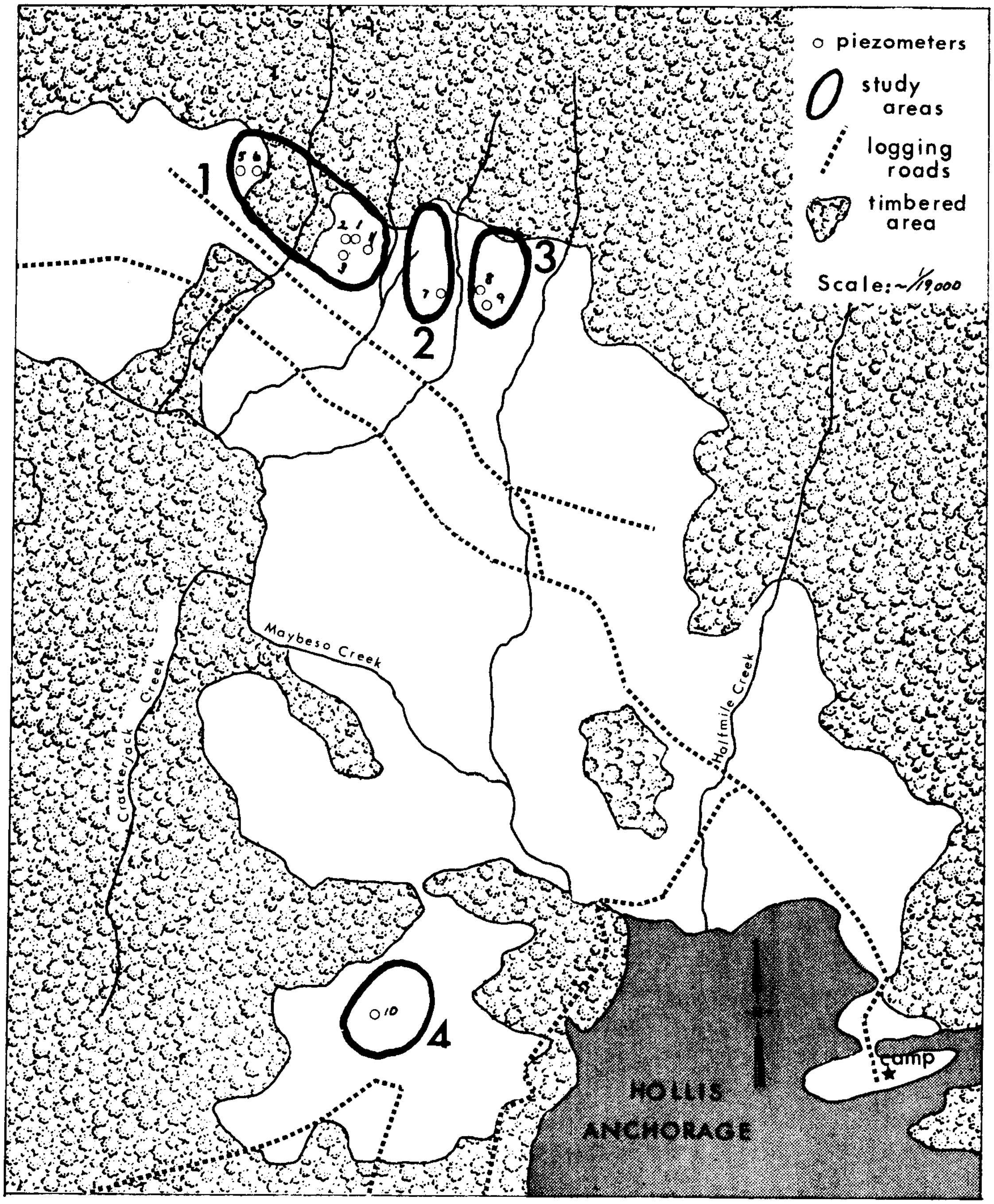


Figure 3.–Planimetric map of lower Maybeso Creek valley, Alaska, showing piezometer locations.

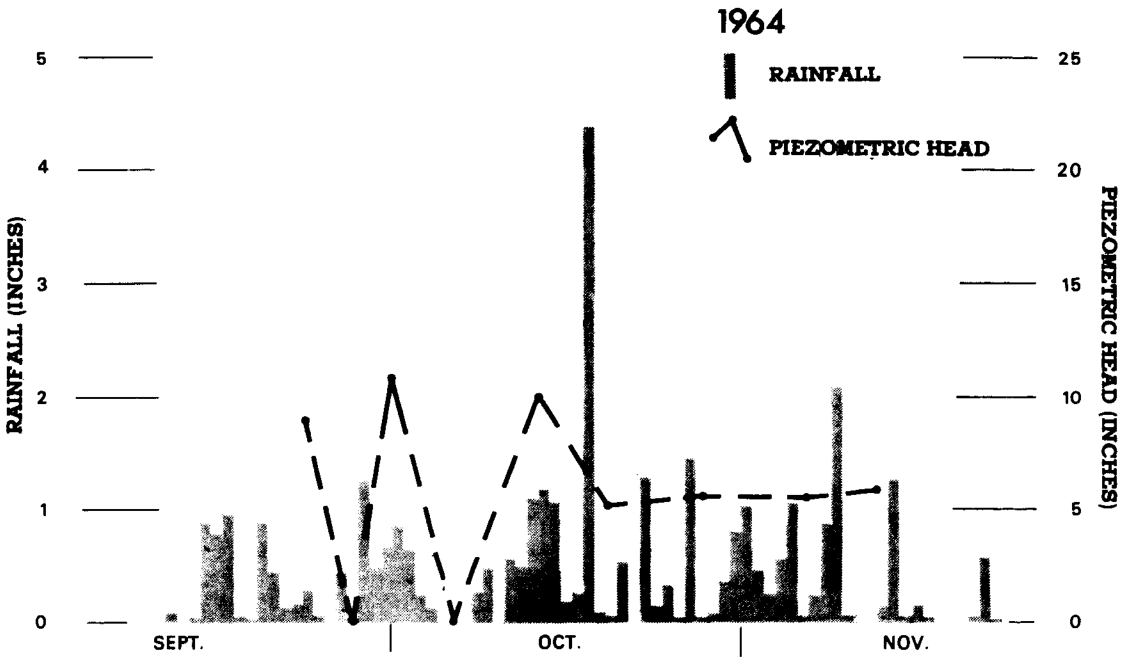


Figure 4.–Representative curve showing relationship between rainfall and piezometric head during the 1964 field season.

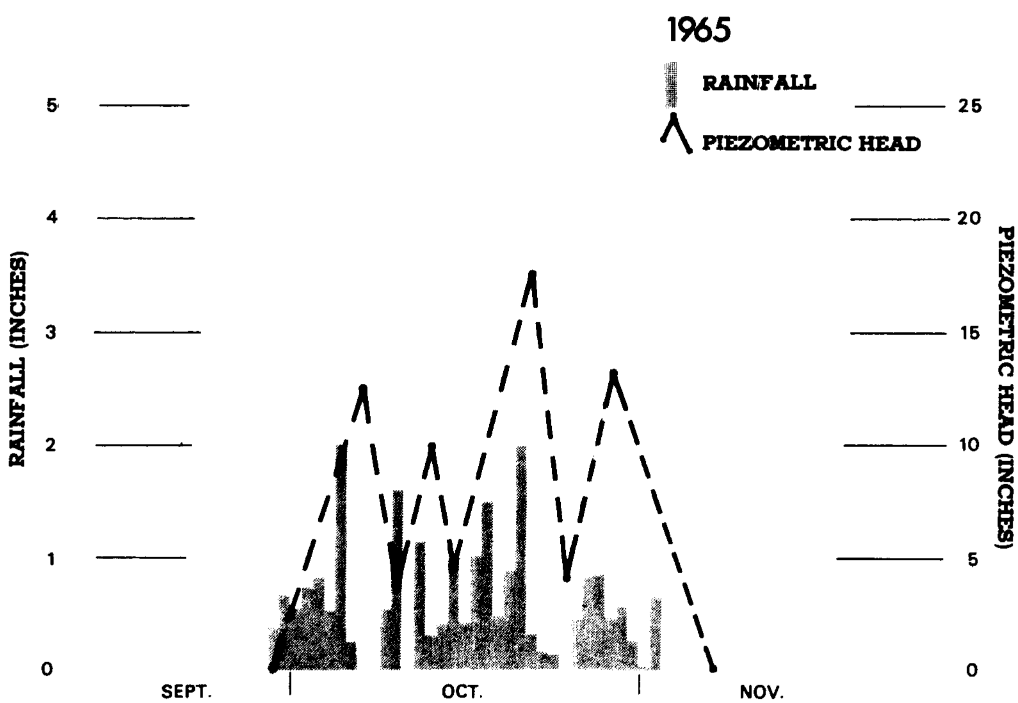


Figure 5.–Representative curve showing relationship between rainfall and piezometric head during the 1965 field season.

individual wells, although varying in magnitude of piezometric head, are markedly similar in relation to rainfall variations. All wells showed rapid fluctuations with rainfall variations, indicating relatively high permeability coefficients for the soil at the well sites. A short lag in piezometric rise occurred at the beginning of the rainy season for most wells, reflecting the time necessary for the soils to reach saturation at the interface between the B horizon and the compact till.

The 1964 piezometric head gradually leveled off late in the season despite undiminished rainfall. This coincides with the appearance of a permanent snow cover on the ridge above the well sites late in October. The 1965 season was short, and a permanent snow cover was absent during the recording period. During this time, no leveling off of the piezometric head curves occurred.

The more frequent reading of the 1965 piezometer network provided enough data for a regression analysis to be made (fig. 6). The resulting relationship is curvilinear, the basic units being average piezometric head and accumulated rainfall per day. The mathematical relationship is the polynomial:

$$Y = a + \frac{b}{X} + cX \qquad (1)$$

where

Y = piezometric head in inches,

and

X = rainfall in inches per day.

Curve A is from the group of piezometers on the upper slope, at the top or just outside of the drainage depressions. Curve B is from piezometers lower on the slope and within drainage depressions. The correlation coefficients (r) for the points surrounding each curve are high. The following tabulation shows the regression equation and corresponding correlation coefficient for each curve:

	Equation	r
Curve A	$Y = 5.17 - \frac{0.232}{X} + 1.85X$	0.81
Curve B	$Y = 7.66 - \frac{0.331}{X} + 6.51X$	0.86

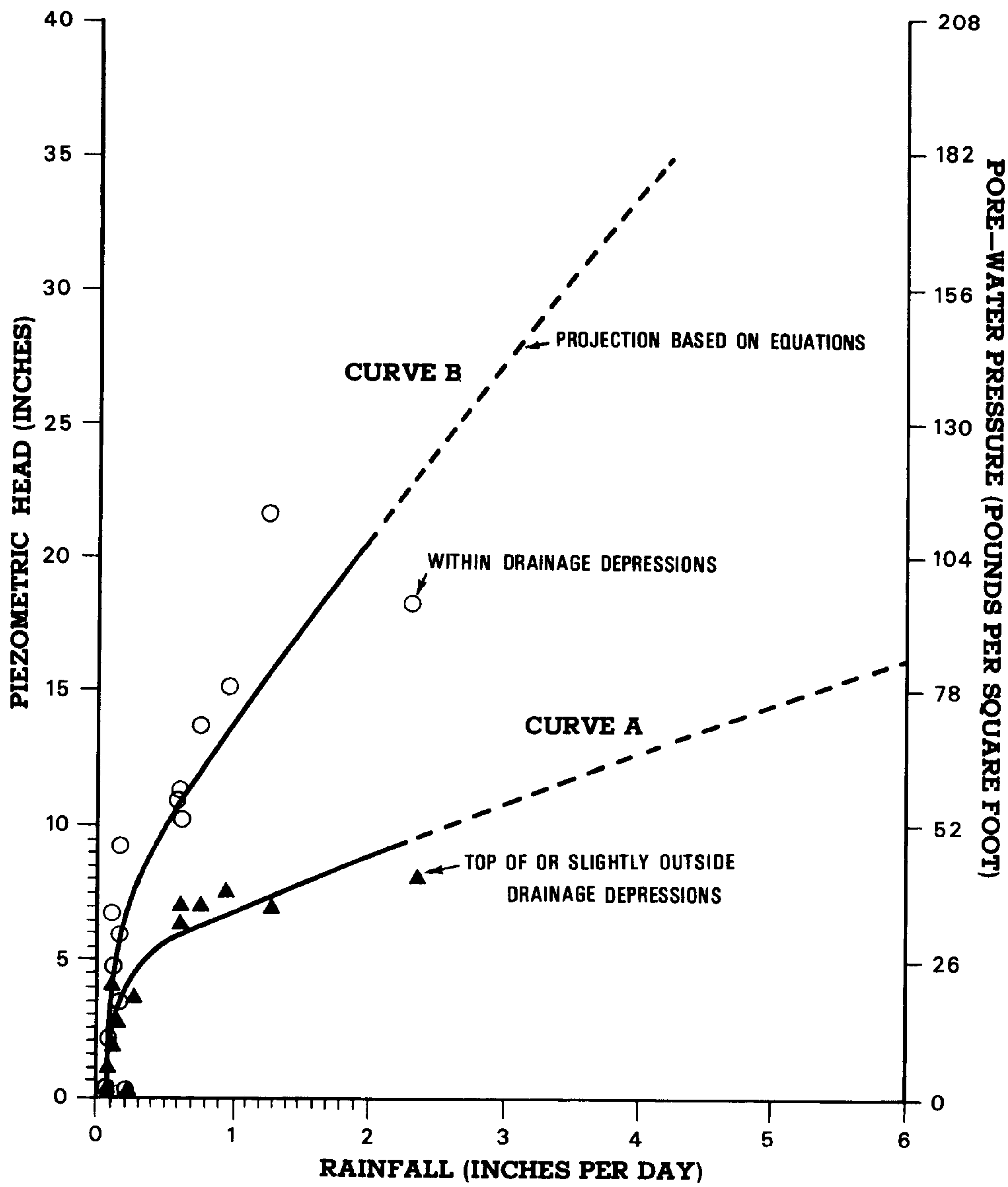


Figure 6.–Graph of average piezometric head vs. accumulated rainfall per day for two locations within preexisting drainage depressions.

Maximum piezometric heads for 1965 and the resultant pore-water pressures are shown in table 2.

Table 2.--Maximum piezometric head and pore-water pressure recorded for each well in the Maybeso Creek study sites during the 1965 field season

Well number	Maximum piezometric head during 1965 field season	Maximum pore-water pressure
	Inches	Pounds per square foot
1	32.0	166.4
2	10.5	54.6
3	18.5	96.2
4	7.0	43.4
5	20.0	104.0
6	10.6	55.12
7	11.0	57.2
8	6.0	31.2
9	6.0	31.2
10	27.5	143.0

The relationship between piezometric head and pore-water pressure is:

$$\mu = h_p \gamma_w \qquad (2)$$

in which μ = pore-water pressure (pounds per square foot)

h_p = piezometric head (feet)

γ_w = unit weight of water (62.4 pounds per cubic foot)

Maximum values for piezometers 1, 3, 5, and 10, all located well within drainage depressions, are conspicuously high. The remaining piezometers occur above the depressions on the open slope.

DISCUSSION

Figure 6 demonstrates the effect of rainfall and slope position on piezometric head and the increase in piezometric head within drainage depressions. Since pore-water pressure is directly related to piezometric rise (equation 2) the correlations hold true also for pore-water pressure. Both curves begin at the same x-axis point, approximately 0.05 inches of rain. Beyond this point, piezometric head increases rapidly with increasing rainfall. After approximately 0.05 to 0.10 additional inch of rain, piezometric head increases more slowly. The curves are reliable for rainfall rates of 2.5 inches per day or less. Beyond this point, the curves are approximate, based on the curve equations. In actual practice, the upper limit of both curves is determined by complete saturation or thickness of the soil profile. In the case of curve A, this upper limit is not reached for the range of rainfall intensities considered in figure 6. For curve B, the limit is about 36 inches, or the average thickness of the soil in the slope area considered. Curve B shows greater piezometric head increase than curve A, indicating higher piezometric head due to increased hydrostatic pressure and flow volume downslope and within the depressions.

The magnitude of recorded and predicted pore-water pressures is great enough to strongly affect shearing strength of slide-prone soils in the Maybeso Creek valley. As an example, assume a soil 3 feet thick, completely saturated. At saturation, the piezometric head in the zone of failure is about 36 inches. From equation 2, we find the resultant pore-water pressure at the base of a column of this soil is 187 pounds per square foot. The effect of this pressure on soil shear strength can be determined by the Mohr-Coloumb theory of soil failure (Wu 1962).

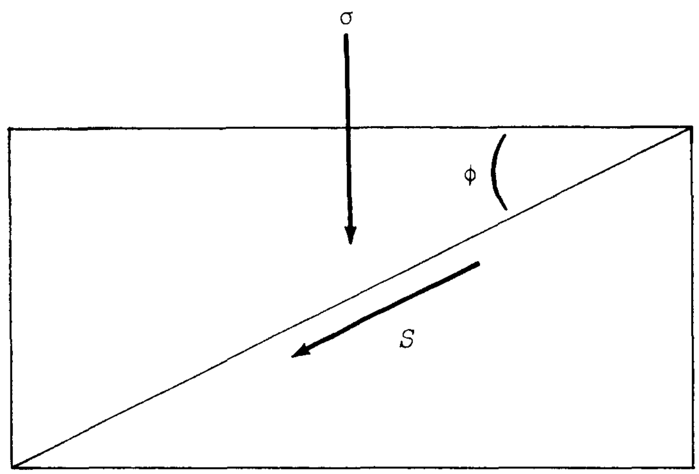
The Mohr theory of rupture (Wu 1962, *p. 71*; Terzaghi 1963, *p. 23*, states essentially that failure in a plastic soil material occurs if the shear stress on any plane equals the shear strength of the material, and that the shear strength, S, is a function of the normal stress, σ, on that plane. Thus,

$$S = f(\sigma) \tag{3}$$

With Coloumb's definition of the function, f, as a linear function of the normal stress or the stress acting at an angle, ϕ, to the shear stress, the equation $S = f(\sigma)$ can be written as

$$S = c + \sigma \tan \phi$$

In this equation, known as Coloumb's equation, S is shear strength, c is a constant called cohesion reflecting the resistance of the soil to shear along a surface, σ is total normal stress, and ϕ is the angle of internal friction (Wu 1962, *p. 71*; Terzaghi 1963, *p. 19*) (fig. 7).



ϕ = angle of internal friction
σ = saturated unit weight of soil
S = shear stress on block

Figure 7.–Diagram of stress relationships on a unit block of soil.

This equation represents the strength as a factor of total normal stress applied to the soil mass at failure, which includes both an effective stress transmitted through points of contact between individual soil grains and a neutral stress produced by the weight of the water, often called pore-water pressure. Since we are primarily interested here in the effect of pore-water pressure variations, Coloumb's equation can be expressed under conditions of effective stress where effective stress, $\bar{\sigma}$, equals total normal stress, σ, minus pore-water pressure, μ. Coloumb's initial equation becomes

$$S = \bar{c} + (\bar{\sigma}) \tan \bar{\phi} \qquad (4)$$

where S = soil shear strength (pounds per square foot)

$\bar{c}$ = effective cohesion of soil (pounds per square foot)

$\bar{\sigma}$ = effective stress

$\bar{\phi}$ = effective angle of internal friction of soil degrees

The above equation shows how variations in pore-pressure affect the strength of the soils. An increase in pore-water pressure decreases the total normal stress and thus decreases the shear stress of the soil. The reverse is true for a decrease in pore-water pressure, assuming c and ϕ remain constant in both cases. Karta soil is cohesionless with a 37° angle of internal friction and a unit weight of 95 pounds per cubic foot. Weight of the 1- by 1- by 3-foot soil mass is 285 pounds (95 pounds per cubic foot x 3).

Applied to equation 4, these values at zero pore-water pressure equal 214 pounds per square foot soil shear strength.

$$S = 0 + 285\ (0.75355) = 214$$

With the addition of pore-water pressure, soil shear strength is reduced to 74 pounds per square foot.

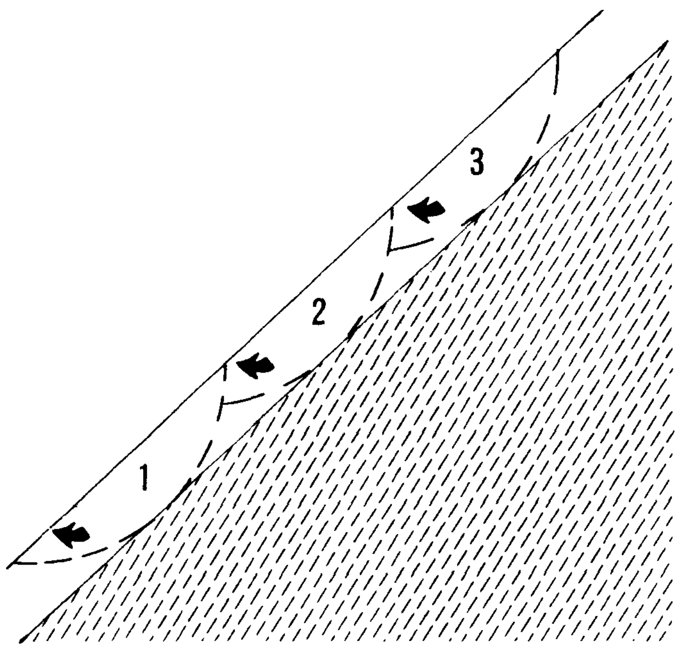
$$S = 0 + (285-187)\ 0.75355 = 74$$

Shear strength of the soil mass affected by pore-water pressure is decreased by 65 percent.

Development of a fairly constant piezometric head late in 1964 and its rapid disappearance following the accumulation of a permanent snow cover above the well sites has implications. Permanent snow on the ridges may modify large piezometric fluctuations by maintaining a fairly constant supply of water to the slope soils. This means a fairly constant flow rate and piezometric head for the saturated zone of soil water. Flow rate in the zone of saturation is large enough to compensate for most additions of water to the soil by natural rainfall. The constant water supply compensates for most decreases resulting from decreasing rainfall. The fairly low pore-water pressures maintained following permanent snow accumulation may effectively terminate the fall period of most active sliding.

Within drainage depressions, piezometric heads are high and increase downslope. This is due to pore-water concentration and resultant increases in volume. The increased pore-water pressures downslope probably are the cause of numerous small "slip-outs" and debris avalanches occurring in the lower parts of many drainage depressions. Thus, drainage depressions may develop and enlarge by small-scale slides occurring progressively upslope. A slip-out in a subsurface drainage channel probably first occurs low on the slope where pore-water pressures are greatest. Above this slip-out, later ones may occur due to high pore-water pressures and reduced shearing strength in the soil mass caused by undercutting (fig. 8).

Figure 8.–Diagram illustrating the possible development of progressive slope failure upslope within areas of concentrated subsurface water flow.



The regression equations for curves A and B (fig. 6) may be used to forecast piezometric head for rainfall amounts up to 2.5 inches at given slope locations. These equations also provide an approximation of piezometric levels at slope failure. Bishop and Stevens (1964) reported a 6.41-inch rain accumulation during October 14-15, 1961, a period of massive landslide occurrence. Five inches of rain fell in one day. Since the slides occurred within the drainage depressions, a calculation using curve B equation indicates a corresponding piezometric head in excess of 45 inches. Since soils in the area of initial slope failure do not exceed 36 inches in thickness, it is probable that the soils were thoroughly saturated when sliding occurred. Rainfall frequency data for Alaska (Miller 1963) indicate a 2- to 5-year occurrence interval for rainfall of this magnitude. The frequency of occurrence agrees with the other extensive mass movements in 1959 (Bishop and Stevens 1964).

CONCLUSIONS

The present study indicates a close relationship between rainfall and pore-water pressure development. As rainfall increases, pore-water pressure increases, rapidly at first, but at a decreasing rate as rainfall continues, reaching an upper limit determined by thickness of the soil profile. Pore-water pressures also increase downslope and within drainage depressions.

A modifying effect on high pore-water pressures with the arrival of a semipermanent snow cover above the till slopes is also suggested. Thus, the arrival of the first fall snows may terminate the season of maximum slide activity.

The regression equations based on data relating piezometric head and pore-water pressure to rainfall provide a means for estimating head pressures for given rainfall amounts and slope locations. Calculation of piezometric head for the maximum recorded rainfall during past slide activity in the study area indicates that the sliding soils were probably completely saturated and therefore subjected to maximum pore-water pressures at time of failure.

Finally, the effect of pore-water pressure on slide susceptibility of the glacial till soils is indicated by a calculated decrease in shear strength of 65 percent at total saturation.

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