Mechanics of Landslides

Thesis by

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INTRODUCTION

Purpose Of Investigation

The purpose of this investigation was to determine whether landslides could be predicted for hill slopes of known inclinations from data secured by laboratory tests performed on samples of the ground under consideration. Specifically, the investigation was to show whether a correlation existed between experimentally determined values for friction and cohesion of ground and calculated values based upon the configuration of earth masses that had slid. The ability to determine the stability of slopes from experimental data is of obvious significance.

Principles Of Soil Mechanics Involved

The combination of forces that resist movement of an earth mass along a boundary surface are due to cohesion and friction. Coulomb in 1781 formulated the laws governing the forces which act in masses of homogeneous earth. Coulomb's laws as stated by William Cain are:

"Law I. The maximum frictional resistance along any portion of a plane in the interior of a mass of earth equals the normal pressure on the portion of the plane considered, multiplied by f, the coefficient of friction, where f is a constant for the earth considered. The friction is thus independent of the area pressed.

"Law 2. The maximum cohesion equals the area considered under compression, multiplied by a constant c, the coefficient of cohesion or cohesion per unit area. The cohesion is thus independent of the normal pressure." (1).

Hence $p = fN + ca$

Where p = the force acting parallel to the slip surface to produce movement at the instant of impending motion.

f = coefficient of friction

N = force normal to slip surface

c = coefficient of cohesion

a = area of slip surface.

Coulomb's laws have been found to hold in laboratory experiments and they have been accepted by modern workers for practical application (2).

Cohesion

The force of cohesion is so variable within a large earth mass that many students of soil mechanics have preferred to neglect it. Presumably cohesion is a function of area and is independent of pressure but since increasing pressure brings more of the surface of the particles in contact, cohesion is indirectly effected by pressure.

It is important that one recognize the presence and appreciate the function of cohesion in giving stability to slopes inclined more steeply than the angle of repose which the material would possess in the loose granular state. In the case of cohesionless materials the angle of repose is approximately equal to ϕ , the angle of internal friction. If a slope in either cohesive or non-cohesive material is cut at an angle equal to or smaller than ϕ, it will be stable to any height because the frictional resistance to sliding varies with the height of slope in the same proportion as the force activating sliding.

In ground endowed with cohesion it is not safe to assume that because a slope steeper than ϕ stands to a height h, stability will be maintained if the slope is extended to greater heights. Such an assumption is erroneous because as the height of the slope is augmented, the force which activates sliding increases more rapidly than the cohesive force which restrains sliding. Ultimately, as the height of slope is increased, the activating force would surpass the restraining force and a slide would occur. It is unsafe therefore to extend steep slopes in cohesive ground much beyond the heights for which they have been known to be stable.

Internal Friction.

Internal friction for granular material is a rather complex phenomenon different in many respects from friction between smooth surfaced solids. In a granular material sliding is resisted by friction which arises from the particles rolling, sliding, and abrading one another at a boundary surface. The coefficient of friction for a given granular material may vary, depending upon the density of the packing and the thickness of the layer involved in rolling. Terzaghi states:

"For perfectly cohesionless material (clean dry sand or the like) the angle of internal friction depends to a large extent on the density of the structure. For high densities its value appreciably decreases with pressure while for low densities the effect of the pressure on the value of the coefficient of internal friction is very small." (3).

For all practical purposes the coefficient of friction is assumed to be independent of pressure.

Hydrodynamic Stress.

Under conditions of free drainage the normal static friction that a material inherently possesses may be expected to act to resist motion. When drainage is impeded, an increase in pressure or a sudden decrease in porosity of a saturated soil will induce a hydrodynamic stress within the material. The load will then be supported partly by the skeleton of solid material and partly by the trapped water that cannot escape quickly enough. The frictional resistance will be a function only of that portion of the load supported by the skeleton of solid material. In cohesive, homogeneous, water-saturated, air-free soils, any change of loading produces an instantaneous change in the stress of the pore water and of the solid material. (4).

If p_1 = the increase in load

w = the hydrostatic pressure induced in the pore water

p = the portion of p_1 carried by the solid skeleton after time interval t.

f = normal static coefficient of friction.

f_x = hydrodynamic coefficient of friction.

Then under the assumption that the pore-water hydrostatic pressure was zero prior to the change in loading,

$$p = p_1 - w \qquad 1.$$

Only the portion p of the additional load p_1, may increase the pressure between grains and thereby raise the frictional resistance; therefore the coefficient of hydrodynamic internal friction amounts to

$$f_x = f \frac{(p_1 - w)}{p_1} \qquad 2.$$

Increase of the hydrostatic pressure acts to lessen friction and reduction of hydrostatic pressure increases friction. As the water drains away with the passage of time, the stress w is reduced to zero and the normal static friction is capable of acting. The physical cause of slipperiness on the surface of fat clays is the hydrostatic lift produced by the rapid application of load. "If one steps suddenly on a very slightly inclined clay-surface the foot slips, although even a very fat clay has a fairly large internal friction (friction angle at least 11 deg.) In the rapid application of the pressure of the foot, the greatest part of the weight of the body is compensated for by hydrostatic pressure, and the friction produced by the remaining weight is not sufficient to prevent the slip." (5).

Many landslides may be attributed to the reduction of frictional resistance arising from an induced hydrodynamic stress. Terzaghi and Casagrande have described several slides attributable to this cause (4), (6). Generally this stress is not induced by an increase in the external loading but more often by a decrease in volume of the saturated material. Casagrande has shown that "The density in the loose state of many cohesionless soils, particularly medium and fine, uniform sands, is considerably above their critical density.* Such materials

* Critical density is the state in which a soil can undergo any amount of deformation without volume change.

in their loose state tend to reduce their volume if exposed to continous deformation. If the voids are filled with water and the water cannot escape as quickly as the deformation is produced, then a temporary transfer of load on to the water takes place." (6). Deformations sufficient to cause volume changes and consequent hydrodynamic stresses in saturated soils may be produced through jarring. The jarring action may be considered as the trigger force that initiates the slide. In itself it would be insufficient to cause slides but under the proper conditions it may initiate the series of physical phenomena that end in a landslide.

Theory Of Slip-Surfaces

For purposes of facilitating calculations pertaining to stability it is helpful to know the shape of the surface along which earth masses break.

Coulomb's Hypothesis.

Coulomb and many of the workers following him assumed the slip surface to be a plane. Laboratory and field observation show this assumption to be wrong except for materials entirely devoid of cohesion, like loose dry sand.

Resal's Hypothesis. (7).

Resal in 1910 assumed a curved slip surface and derived a differential equation for the curve.

$$\frac{dy}{dx} = \tan\,(i - \alpha) \qquad 3.$$

Where i = inclination of slope to horizontal

α = angle formed between the slope and the tangent drawn to the slip-curve.

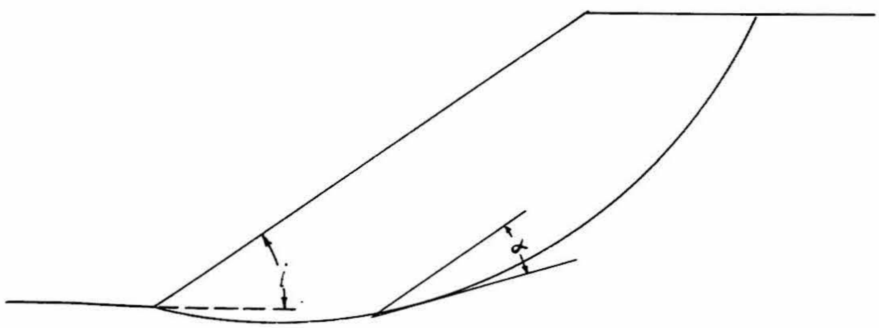


Fig. 1-a, Slip-curve, Resal's method.

According to Terzaghi (3), Resal did not succeed in solving his equation but on the basis of several assumptions arrived at an approximate formula for determining the critical height of a slope.

$$h_1 = \frac{c \sin i \cos \phi}{\Delta \sin^2 \left(\frac{i - \phi}{2}\right)} \qquad 4.$$

Where h_1 = critical height of slope

c = coefficient of cohesion

i = angle of inclination of slope to horizontal

ϕ = angle of internal friction

Δ = wt. of earth per unit volumne.

Frontard's Hypothesis. (8).

Several years later Frontard succeeded in solving Resal's differential equation for the slip surface curve and from it developed the following equation for the critical height of slopes:

$$h_1 = \frac{2c \sin i \cos \phi}{\Delta \sin (i - \phi)} \left[\frac{\cos \phi}{\sin i (1 - \sin \phi)} + \frac{\text{arc} \cos \frac{\sin^2 i - \sin \phi}{\sin i (1 - \sin \phi)}}{\sqrt{\sin (i - \phi) \sin (i + \phi)}} \right] \qquad 5.$$

According to Terzaghi this equation is useless because by making certain fundamental assumptions regarding the direction of the forces acting, the problem became overdeveloped (3).

Becker's Hypothesis. (9)

In 1916, Becker, after a study of the Panama Canal slides derived an equation for the curve along which slides occur. However, as Terzaghi has pointed out, "Becker's theory includes several misinterpretations of the laws of applied mechanics--." (3).

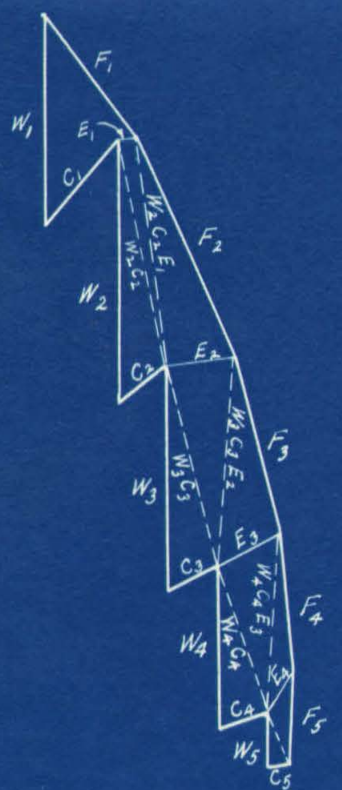
Petterson's Hypothesis. (10)

Petterson in Sweden assumed that the slip-surface was cylinderical and that the trace of the slip-surface on a vertical plane was the arc of a circle. This assumption has been checked by thorough field investigations (11) and found to be approximately true. It has been accepted by recent workers in the field of soil mechanics as corresponding closely enough to the actual field conditions.

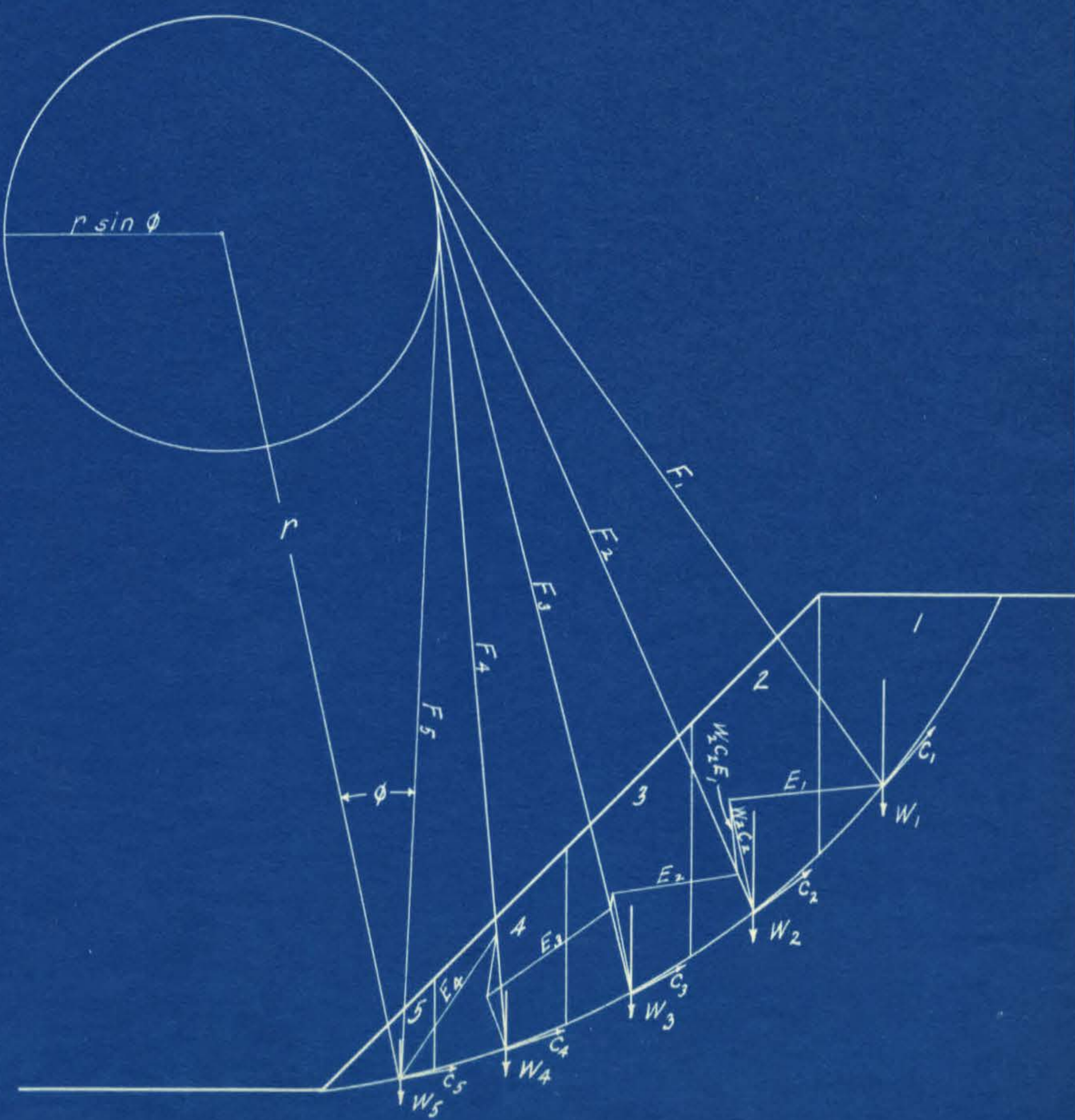
In this investigation the writer's calculations are based on the assumption of a cylindrical slip surface.

Methods Of Calculation

Several methods may be employed to determine whether a mass will slide along a given curved surface when the coefficients of cohesion and friction are known. A section of unit thickness is taken through the slope and an assumed slip-curve is drawn. The section is commonly divided into segments and the forces for each segment are treated separately. Fig. 1 illustrates the method used by Fellenius (12). The section is divided into a number of suitable



Force Diagram



Space Diagram

Figure 1. Graphical method for determining stability of slopes, after Fellenius. Assumptions: Ground possesses cohesion and friction; slip-curve is circular and passes through toe of slope.

segments and a graphical solution is performed to find whether the block is in equilibrium. The directional paths of action of the known forces W, C, and F are drawn for each of the segments.

W = weight of segment

C = cohesion exerted on segment

F = resultant between tangential force of friction and force N, acting normal to slip-surface.

$F = N/\cos \phi$

6.

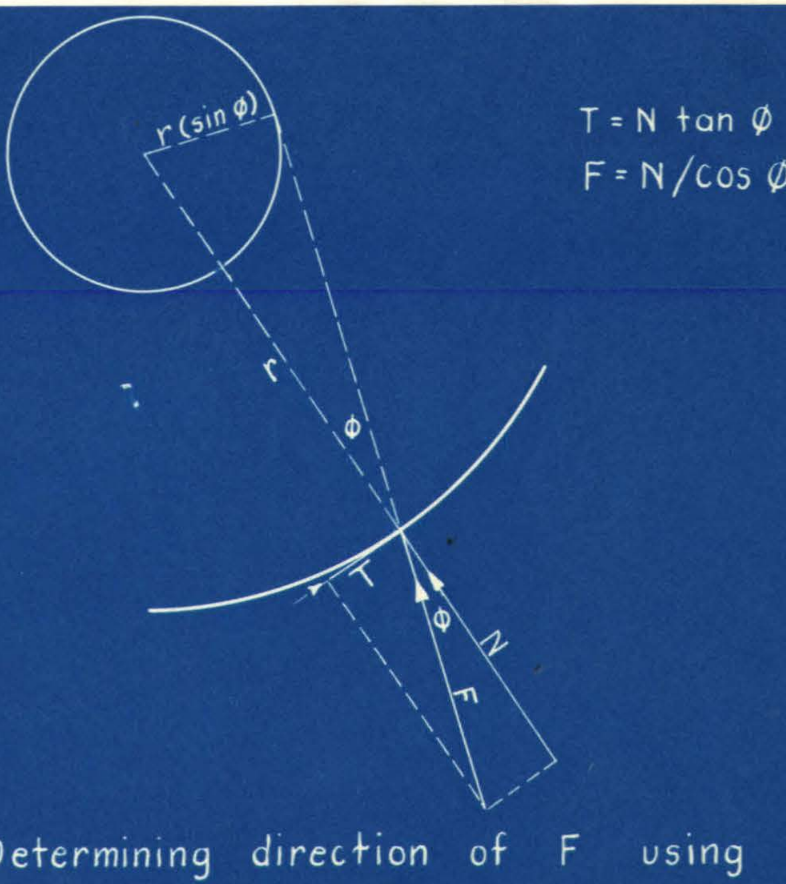


Fig. 2. Determining direction of F using circle of radius r(sin ∅) circumscribed about center of slide-arc.

The direction of F is along a line which is tangent to a circle with radius R sin ϕ circumscribed about the center of the slip-curve and which passes through the point where W intersects the slip-curve.

Starting with forces acting on segment I, a force diagram is constructed. The E force represents the resultant sideward thrust exerted by each block upon the lower adjacent one. If the section is in equilibrium the force diagram will close for the last segment with E equal to zero and the space diagram will show the segment to be free from rotation.

In the force diagram the broken lines represent resultants of combined forces. The directions of these resultants are used in the space diagram for determining the point through which the lateral thrust E acts.

In order to determine whether a slope is stable it is necessary to perform this graphical calculation for a series of probable slip-arcs. This is a tedious and time consuming task.

Krey simplified the calculations considerably by assuming that the E force could be neglected and that for equilibrium the ground reaction was equal and opposite to the weight of the section (13). Fig. 3. Concerning the E forces, Terzaghi stated:

"However, experience has shown that the effect of the nature of these assumptions (position and magnitude of E forces) has but little influence on the final result

$W = Q$

$R = Lc + N \tan \phi$

where c = coefficient of cohesion

$\tan \phi$ = coefficient of friction

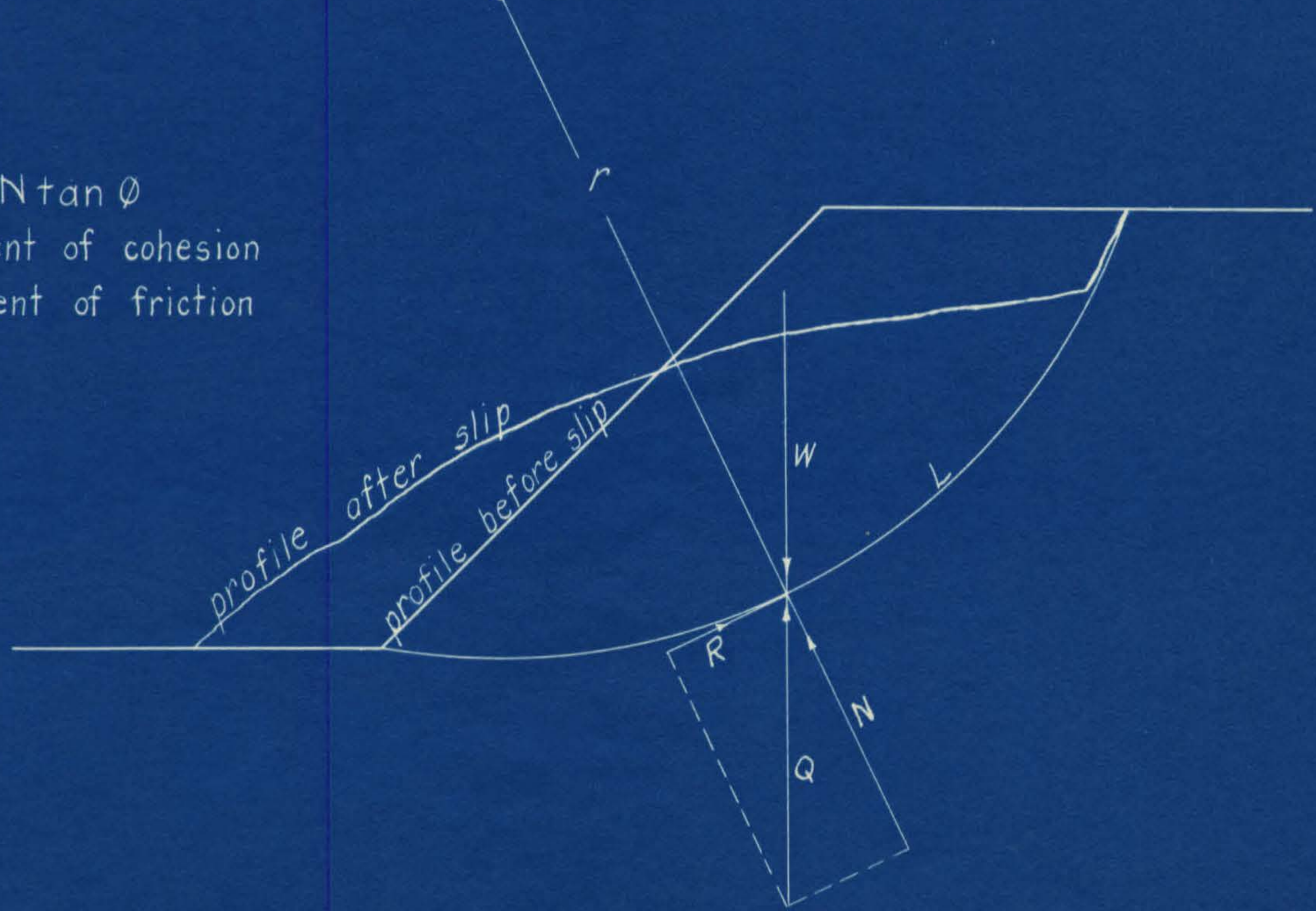


Figure 3. Illustrating forces acting on section to maintain equilibrium.

of the computation. This is essentially due to the fact the forces, E, are internal forces of the sliding wedge of earth which in turn leads to both the horizontal and vertical components of the forces, E, balancing each other within the wedge. The sum of all vertical forces acting on the sliding surface is always equal to the weight of the wedge, regardless of what the value and the direction of the E forces may be. Hence, the assumptions concerning E merely influence to a certain extent the distribution of the vertical forces over the sliding surface." (3).

With Krey's simplification the problem of determining stability along various arcs may be conveniently solved by setting up an equilibrium equation for the moments acting to revolve the section about the center O of the slide-arc.

$$r(\Sigma T) = r(Lc + N \tan \phi) \qquad 7.$$

Where r = radius of slide-arc

T = tangential component of W, (wt. of segment)

N = normal component of W

L = total length of arc

c = coefficient of cohesion

$\tan \phi$ = coefficient of friction

The writer found by trial that the results of calculations made by dividing the whole section into segments differed only slightly from those attained by treating the entire section as a single unit. Therefore, since the latter method was much less time consuming, the sections in this study were treated as single units. The center of gravity was determined experimentally by cutting the section which was drawn to scale, out of a uniform grade of cardboard and suspending it successively from three different points. The point of intersection of the vertical lines drawn from the points of suspension represented the position of the center of gravity for the section (14).

It was assumed for the calculations that the ground was homogeneous with regard to density, friction, and cohesion. The weight of a section was given in terms of its volume which was equivalent to its area, since it was assumed that the sections were of unit thickness. The area for each of the sections was determined with the aid of a planimeter.

$$W = a\ n\ s\ d = a\ K \qquad 8$$

Where W = weight of section

a = planimeter reading

n = planimeter constant of multiplication for converting reading into terms of sq. inches of area measured = 1.11

s = number of sq. ft. represented by one sq. inch of drawing. (dependent on scale)

d = density of ground per cubic foot = 120 lbs

K = (n s d).

The coefficients of cohesion and friction that acted at the instant of impending motion for each of the slides investigated were determined graphically. Fig. 3. First the original ground profile was reconstructed from survey data gathered in the field. A series of possible slip-curves were drawn to pass either through the toe of the reconstructed slope or the lower point where the break appeared to have occurred. One of these curves was constructed to pass along the slip-surface exposed at the upper edge of the landslide. This curve represented the approximate trace of the actual slip-surface on a vertical plane of the section. The center of gravity for the section bounded by each slip-curve was located experimentally. The weight, W, of the section was graphically represented as a vector acting on the slip-curve along a line passing vertically through the center of gravity. At the point of intersection with the slip-curve, the vertical reaction Q was resolved into two components, a normal force N and a tangential force R. R represents the combined resistance of cohesion and friction along the slip surface.

$$R = L\,c + N \tan \phi \qquad 9.$$

Where L = length of slip-arc

c = coefficient of cohesion

$\tan \phi$ = coefficient of friction.

For static equilibrium

$$Q = W$$

R is the resistance to sliding along the slip-curve when motion is imminent. The writer was interested in graphically calculating the values for ϕ and c that the slide-ground possessed, in order to make comparisons with the values for the coefficients secured by laboratory experiment.

A method described by Terzaghi for determing ϕ and c was used. The value of R for each slip curve was determined graphically by constructing an equilibrium force triangle and resolving Q into components. A series of values for ϕ was assumed and a corresponding series of c values was computed for each slip curve, using equation 9. See Figure 43. The distances between the centers, O_1, O_2, O_3, of the various slip-curves were plotted as abscissas. For each slip curve the values of c corresponding to different assumed ϕ values were plotted as ordinates. The points corresponding to the same value of ϕ were joined in a curve. Each curve shows a maximum point. The abscissa passing through a maximum represents the position of the center of the most dangerous slip-arc for material possessing the given ϕ value. The curve whose maximum falls on the abscissa of the actual slip-arc serves to determine the values of ϕ and c that acted when sliding was imminent.

Good judgment is more important in these computations than a high degree of accuracy for as Terzaghi has stated:

"The assumptions that the sliding surface is strictly cylinderical and that natural soil deposits are perfectly homogeneous give the preceding computations the character of estimates. Hence no great accuracy is required in the numerical computations and the results obtained with an ordinary slide rule are sufficiently accurate." (3).

The chief disadvantage of hypothesizing various arcs as possible slip-curves is that an infinite number may be drawn through any one or two points. However this disadvantage was circumvented by means of a diagram developed by Fellenius which is here reproduced. Fig.4. Fellenius made a detailed study of numerous arcs passing through the toe of a plane slope which terminated in horizontal surfaces at the toe and at the crest. From this study he was able to construct a diagram which makes it possible to locate the centers of the most dangerous slip-curves for slopes of different inclination and for ground possessing various relative proportions of friction and

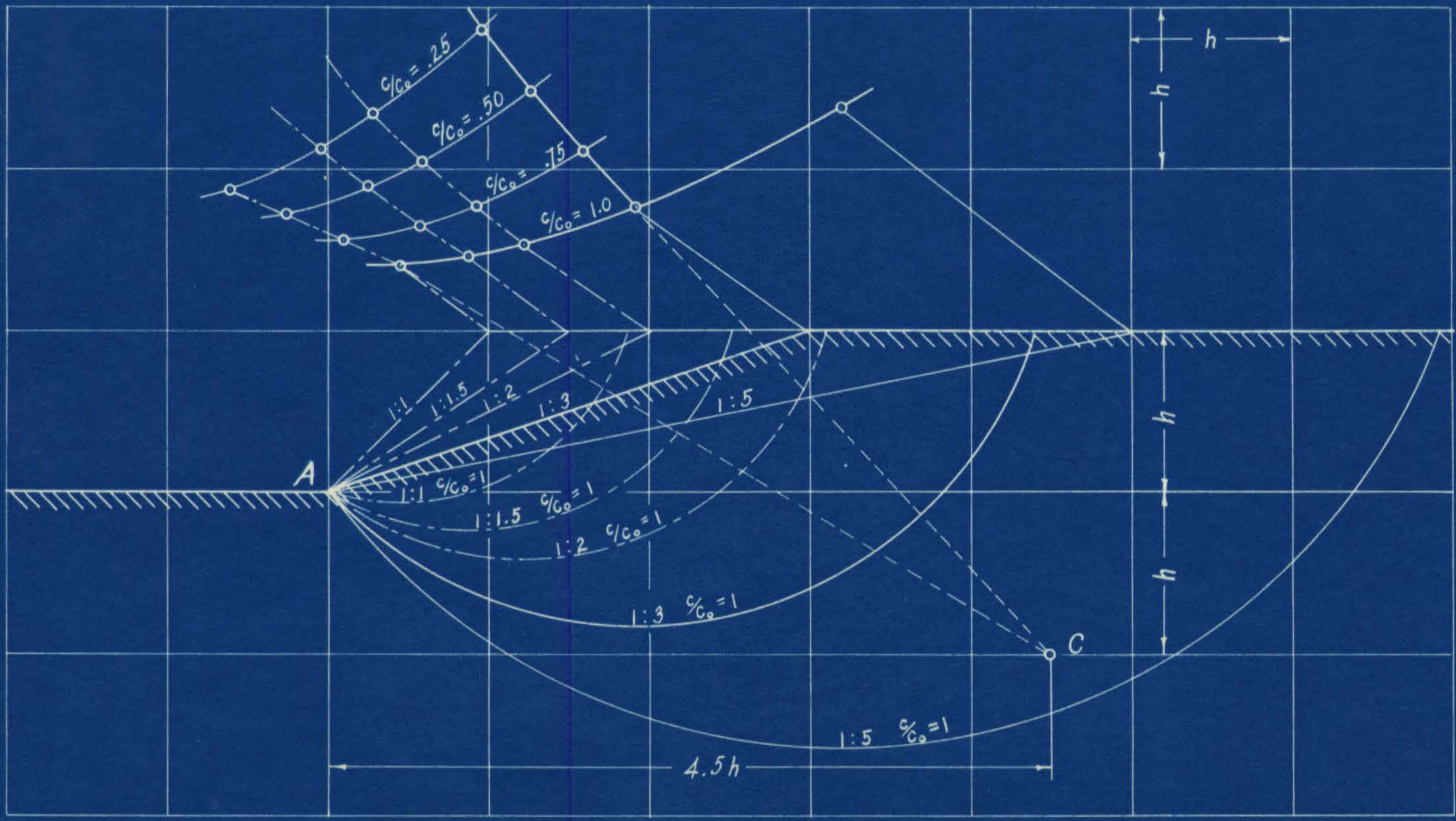


Figure 4. Loci of center points of slip-curves for various inclinations of slope and various relative proportions of cohesion; slip-curves through toe of slope. c_o = requisite cohesion for equilibrium when ϕ = zero. After Fellenius.

cohesion. The following is a free translation from Fellenius with regard to this diagram:

(The positions of the various slip-curves and the migration of the slip-curve centers corresponding to changes in the angle of slope and in the relative values of cohesion are shown in Fig. 4. In this figure the toe (A) of the slope is regarded as fixed.

The center points lie on curves which pass outward from the center of the most dangerous arc for the case when only cohesion is present. These curves asymptotically approach the normals drawn to the mid-points of the respective hill slopes. If the ground possesses friction but no cohesion, the slip surface becomes a plane and the ground slope must be coincident with this plane for equilibrium.

The center point curves have not been determined for the more steeply inclined slopes because in such cases plane slip-surfaces may be assumed without much error.

With the aid of Fig. 4 it is possible to determine by eye the position of the center of the most dangerous slip-curve passing through the toe of slopes having inclinations varying from 1:1 to 1:4 and for ground having various relative proportions of cohesion and friction. It is not necessary to determine the center point more accurately than can be done with this diagram since small displacements to one side or the other have negligible effects upon the calculated results.) (12).

Fellenius also made a critical study of slip-curves which pass beneath the toe of a slope and break out on the lower surface which intersects the slope. He calculated that for ground which possessed only cohesion the center of the most dangerous arc lay on a vertical line removed from the crest of the slope a distance (a), and as the proportion of friction relative to cohesion increased the center of the most dangerous curve moved outward and downward from a point on this line. Figs 5, 6.

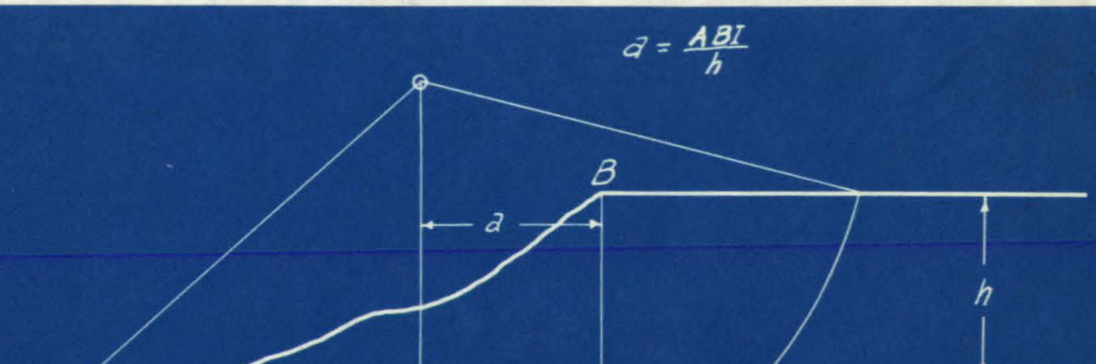


Fig. 5. Illustrating position of line along which center of critical arc falls. Assumption, cohesion only. After Fellenius.

Of the curved slip-surfaces passing beneath the toe of the slope Fellenius says:

$\frac{d}{h}$	Center Point Coordinates			ϕ, when c/c_0 =				
	Cohesion Only	Friction Only		1.00 Cohesion Only	.75	.50	.25	0 Friction Only
	Z	X	y					
0.5	0.31	0.25	0.26	0°	5.1°	9.0°	12.9°	16.8°
1.0	0.59	0.33	0.41	0°	3.2°	5.9°	8.4°	11.0°

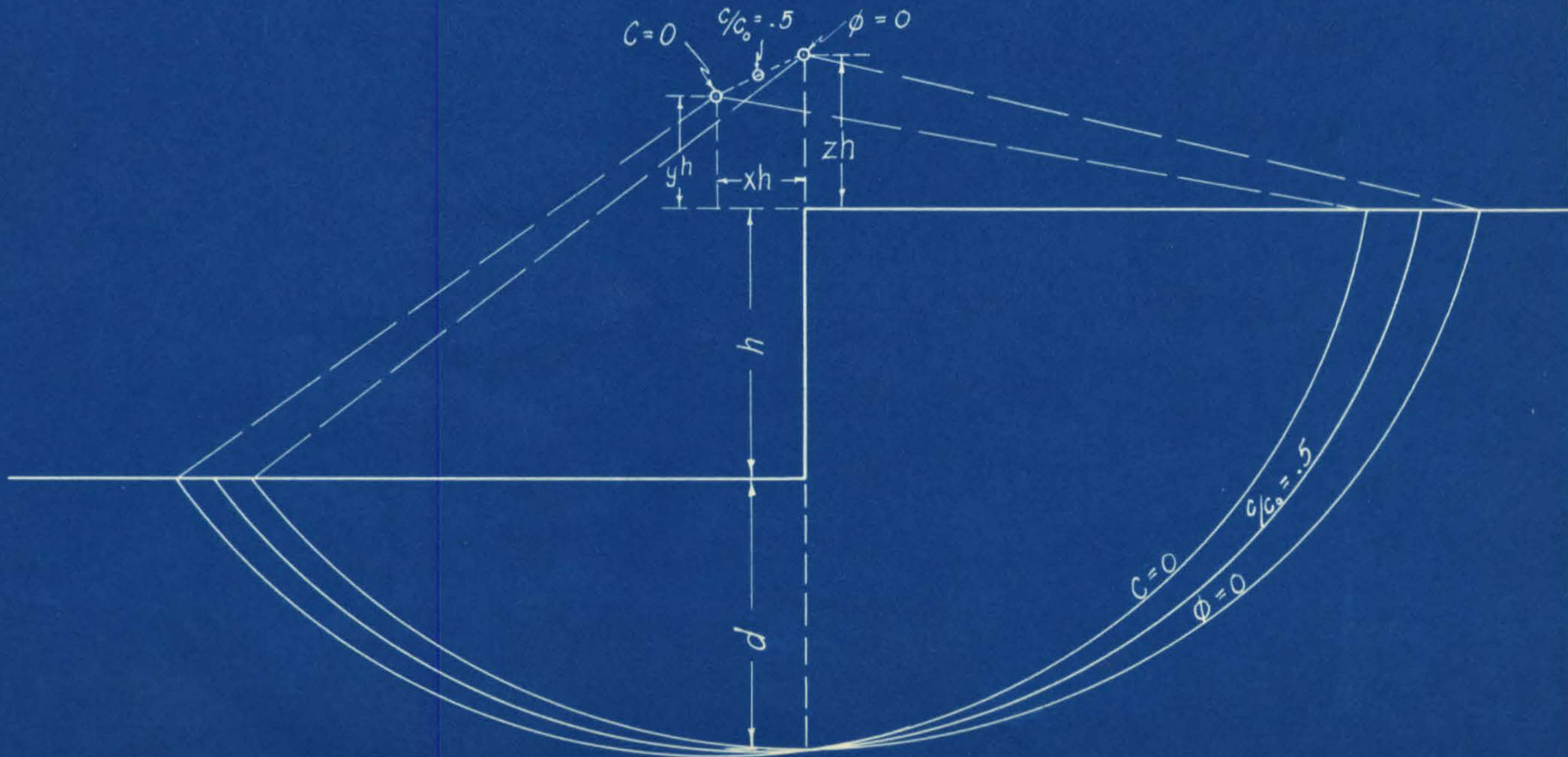


Fig. 6 . Illustrating shift of critical slip-surfaces with changes in the relative value of cohesion; one point on the slip-surface is fixed. After Fellenius.

(We have found that for ideal cohesive ground devoid of friction the most dangerous slip-surface lies theoretically at an infinite depth. However the requisite cohesion for equilibrium along a surface lying at shallow depth beneath the foot of the slope is only slightly less than the cohesion required along the infinitely deep lying surface.

$$c = \frac{\gamma h}{4} (0.723) \text{ for infinitely deep surface.}$$

Where γ = wt. of ground per unit volume

h = height of slope

c = coefficient of cohesion

The presence of the slightest amount of friction in the ground causes the most critical slip-surface to migrate upwards to a position of comparatively shallow depth.)*

* Free translation from Fellenius (12).

In figure 6 the coefficient of cohesion for equilibrium along a surface at depth $d = h$ in ground devoid of friction is $\frac{\gamma h}{4}$ 0.720 which is only slightly less than the requisite cohesion for an infinitely deep lying surface.

With the assistance of the diagrams developed by Fellenius the writer was able to choose critical slip-curves for which to carry out cohesion and friction calculations.

GENERAL RESEARCH PROCEDURE

Field Methods

The field procedure for the examination of landslides was comparatively simple. Owing to the fact that core drilling equipment was not available to the writer no direct investigations of subsurface conditions were performed. A slide was examined from several points of observation and the major axis and direction of slip were determined as accurately as possible. A line of levels was run along the major axis of the slide parallel to the direction of movement. This was done using a clinometer and tape. The unbroken ground surrounding the landslide was carefully observed and slope measurements were taken. This latter information was used to reconstruct the profile of the original ground.

A geologic examination was made of each slide. This consisted mainly in a determination of the local structural relations, the attitude of the beds, the lithology and general character of the ground.

In the early part of this investigation the writer endeavored to secure representative samples of undisturbed ground in which the slides had occurred. This was done for the Huntington slide by collecting large irregular blocks of siltstone which were trimmed down in the laboratory to suitable size for testing. The cohesive strength of these samples proved to be so high that had the original hillside possessed such strength no landslide would have occurred. It became obvious on reexamination of the landslide locality that the hill mass, because of the thorough fracturing of the ground, had never possessed the cohesive strength of the individual fragments. The high cohesive strength of the tested samples was a consequence of the adhesion between the particles plus the capillary tension of internal water plus the shearing strength of any cement which may have been present. Terzaghi has shown that internal capillary tension may exert tremendous forces on soils, comparable to strong external compression (4), (5). By introducing fractures into the ground all three of the agents which may have acted to produce a high cohesive strength were weakened. Along the fissures the particles were not in close enough contact to adhere strongly to one another, the openings were too large for capillary forces to act, and the cement was broken.

Many of the landslides examined were in shale ground that was badly fractured. It was impossible to cut an undisturbed sample from this type of ground with the apparatus that the writer had available and valueless to measure the cohesion of the unfractured lumps. Samples were therefore collected from this type of ground simply by digging with a garden trowel. In this way a few pounds of material were collected from each of these slide localities for testing in the laboratory.

General Laboratory Procedure

The principal laboratory test was to determine the angle of internal friction possessed by the different samples of ground. For this purpose the apparatus shown in Fig. 7 was constructed.

The apparatus is of brass. It consists of two rings which contain the sample and fit into an upper and lower block respectively. A plate fits into the upper ring and rests on the sample. A load may be transmitted to the sample through this plate by means of lever A. The upper block is held stationary. The lower block is free to roll on ball bearings except for the resistance to shear exerted by the sample being tested. A shearing force may be applied by means of lever B.

The samples of undisturbed ground were prepared for shearing by a method described below. It was found too difficult to cut cylinderical samples that would fit smoothly into the rings of the shearing apparatus. The Huntington Slide ground from which the first samples of undisturbed material were taken is of well indurated siltstone difficult to saw and rather brittle. The samples were therefore prepared in the following manner.

A lump of material was taken and broken into fragments small enough to fit into the rings of the shearing apparatus. A fragment was then placed in a mold illustrated in Fig. 8. The mold was constructed of two sections A and B. The sample was first placed in section B as shown and a ring of Wood's metal, W, (melting point 65.6°C) was poured around its lower part. Above the metal the mold was filled to the surface with plaster of paris, P. By means of a trowel, the surface of the plaster was smoothed. Thin copper strips were fitted around the sample to form a plane of separation between the plaster

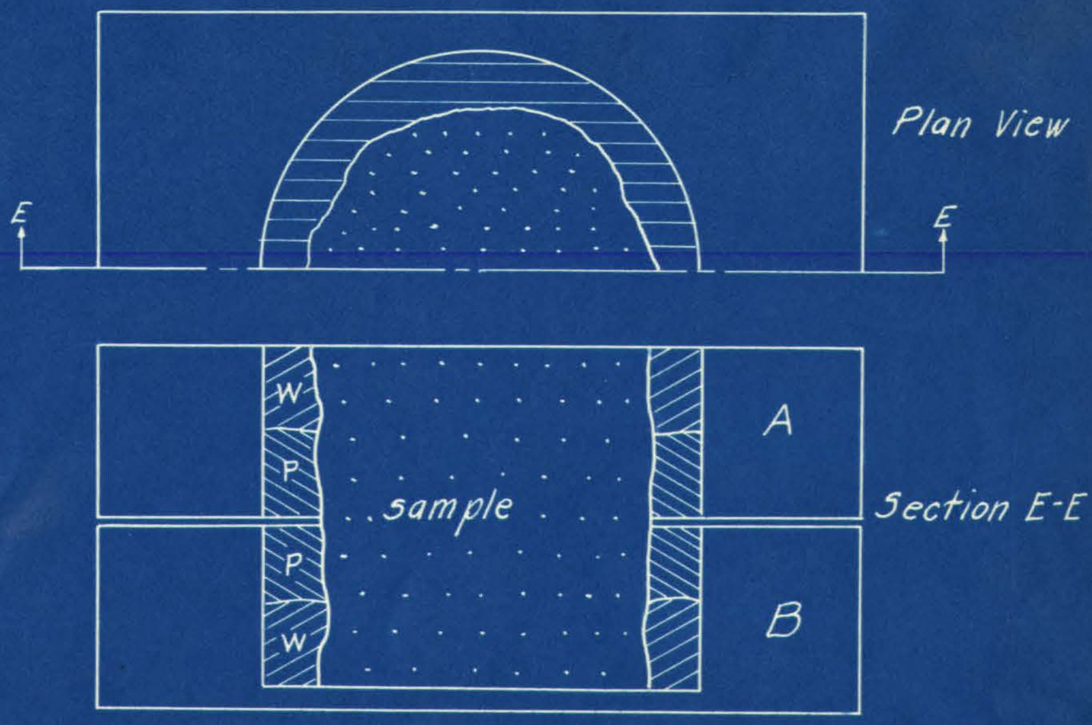


Fig. 8. Mold for preparing samples of undisturbed ground for shear test.

of sections A and B so that the shear stresses along this plane would be resisted only by the material making up the specimen. Section A was then fitted into place and plaster of paris was poured into it to form a ring around the sample. This was followed by Wood's metal. The specimen was then removed from the mold and the copper strips were extracted. It was then placed in the rings of the shearing apparatus where a snug but not too tight fit was accomplished by means of moist paper washers inserted between the sample and the rings.

After completion of a shearing experiment an outline of the sheared area was made on transparent paper. This area was measured with a planimeter and used to determine the stresses per unit of area exerted on the sample during the experiment.

Samples of undisturbed ground were collected from only two slide localities and treated in the manner described above. The average shearing strength of the samples thus measured was far greater than the average strength of the ground in which the slides occurred. Moreover the shearing strength of these specimens was so variable that an average value possessed little significance.

The shearing strength of ground depends on the combined forces of cohesion and internal friction. Of these two forces, cohesion is far more likely to vary for samples of similar material than is friction. The great variability in the shearing strength of the undeformed samples is attributable to variable values of cohesion. Since the cohesive strength of samples of undisturbed ground was far greater than that possessed by the ground mass taken as a whole, it was obvious that this experimental value of cohesion had no significance in calculations of ground stability. Experiments were continued therefore only for the purpose of measuring the angle of internal friction.

The experimental procedure used for measuring the angle of friction is described below. A sample was disintegrated with the hands or a rubber covered pestle until it passed a number 10 mesh screen. Water was added and a paste was made of the material. It was then pressed into the rings of the shearing apparatus with the hands and allowed to dry for one or two days. It was then sheared and a smooth surface was developed between the substance in the upper and lower rings. The rings containing the sample were placed together and contact between the contained material was made along the smooth surface. The sample was loaded normal to the slip-plane and the shearing force required to produce movement was measured. The results were plotted, normal loads as abscissas and shearing forces as ordinates. The points follow straight lines remarkably well.

The slope of the curve is the angle of friction. The ordinate for the extrapolated curve, where the abscissa is zero, is the value of the cohesive force for the material along the slip-plane.

In addition to measuring the angle of internal friction of the various samples of ground, several other characteristics were determined. These latter, although not used in calculating resistance to sliding, serve to describe and identify the ground material. A description of the ground in terms of its physical characteristics is valuable to investigators for purposes of comparison. Two such physical descriptions were proposed by Professor A. Atterberg of Kalmar, Sweden. They describe soil characteristics that have been termed the lower plastic and liquid limits.

The lower plastic limit is determined as follows: Material passing a number 10 mesh screen is mixed with water to a pasty consistency. The mass is then rolled on a glass plate into a thread 1/8 inch in diameter. The mass is rekneaded and rolled until the moisture content drops to the point where the thread can no longer be rolled without breaking into fragments. This moisture content expressed in per cent of the weight of dried material is called the lower limit of the plastic state or the plastic limit.

To determine the lower limit of the liquid state a flat porcelain cup is used. Fig. 9. Material passing the number 10 mesh screen is mixed with water to a soft

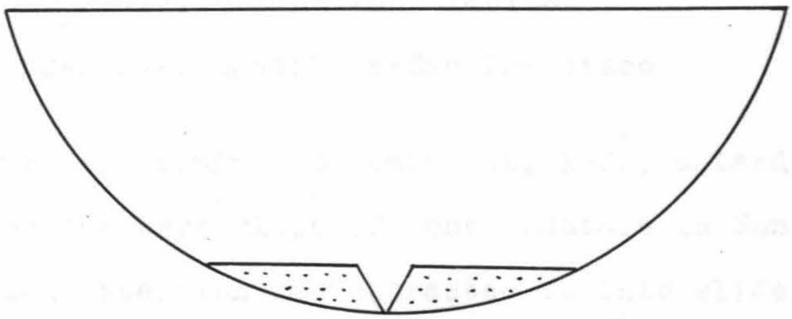


Figure 9. Determining liquid limit

consistency. It is spread on the bottom of the porcelain cup to a depth of about 1 cm. A standard V shaped notch is cut through the middle of the clay layer. The cup is subjected to taps on its under-surface by means of a standardized tapping device. The moisture content of the material expressed in per cent of its dry weight, when 25 taps with the standard instrument are required to bring the edges of the notch together, is called the lower limit of the liquid state or the liquid limit.

The difference in the moisture content between the plastic and liquid limits is called the plastic index.

A size analysis for each of the samples was also performed. For this purpose the writer used the hydrometer method of textural analysis for fine grained materials described by Wintermyer, Willis, and Thoreen (15), (16).

SPECIFIC LANDSLIDE STUDIES

Parker Ave. Landslide-San Francisco

On Thursday morning, December 26, 1935, a landslide occurred on the west slope of Lone Mountain in San Francisco. Much attention was attracted to this slide because of the attendant damage to streets and power lines and because the San Francisco College for Women, a new and costly structure, was left standing in an apparently unsafe position close to the edge of a steep thirty foot escarpment. Fig. 15.

The following newspaper account describes the movements of the earth mass prior to the major slip on Thursday:

"The unstable earth gave warning early Tuesday afternoon. A new six inch main to serve the subdivision gave way and spurted water. Workmen found an earth movement had shifted it about eight inches, according to E.G. Cahill, manager of utilities. When the water was turned off at 3 p.m. Parker Avenue was exhibiting a bulge.

"There was some movement later that day and some cracks showed in the hillside Wednesday. At 12:10 a.m.

yesterday (Thursday) there apparently was a prelude of sufficient jarring force to stop the clock in the Gothic tower of the College library, although no impact was noted by W.M. Putney, special police guard on the campus." (17).

In order to determine the cause of this landslide it is necessary to recognize all of the contributing factors. For this it is necessary to review the history of public and private improvements on and adjacent to Lone Mountain and to understand the geologic conditions at the site of the landslide.

Briefly, the history of improvements is as follows: In August of 1919 the pavement on Parker Avenue was completed. This improvement involved the removal of a prism of earth along the base of the west side of Lone Mountain. In January of 1933 construction of the college on the top of the hill was completed. Two years prior to this construction the hill top was graded and the excess rock was thrown on the slopes. On about October 15, 1935 grading for the new subdivision at the foot of Lone Mountain immediately to the west of Parker avenue was completed. The ground adjacent to the street was reduced to the same level as the edge of the pavement and from there it sloped gently downward toward the west and north. The landslide occurred on December 26, 1935, approximately two and one-half months after the ground for the subdivision had been graded.

A knowledge of the geology of the landslide locality is essential in order to complete the requisite information for determining the cause of the slide. Unfortunately no borings of the slide mass had been made up to March 27, 1936, the date on which the writer concluded his field investigations. The geology to be described is based on surface observations and all subsurface sketches and descriptions are inferential, based on these observations.

The core of the hill is of greenstone, a much altered basic rock composed in large proportion of serpentine. It is exposed in the street cuts bordering the base of the hill and was encountered in the excavations for the structure situated on the crown of the hill. It may be seen in many parts of San Francisco forming an important lithologic member of the Franciscan Series.

Overlying and depositional upon the greenstone is a sandstone, moderately indurated and of variable thickness. It is massive in character and grades upward from coarse angular particles of one and two inch dimensions to medium grained sand. The basal contact of this sandstone is very distinct and shows the irregularity of the old greenstone surface. This sandstone is well exposed along Parker and St Rose's avenues at the base of Lone Mountain.

The sandstone is overlain by windblown sand which is loosely compacted and appears to be held together chiefly by moisture. It is free running in many places where it is dry. It forms the major rock covering of the hill slopes and because of its loose character masks the contact with the underlying sandstone. On top of the dune sand and only present along the edges of the flat hill top is a comparatively thin layer of fragmental greenstone material dumped during the grading operations prior to construction of the college.

The stratigraphic succession of rocks forming Lone Mountain at the slide locality is comparatively simple with serpentinized greenstone at the base followed successively upward by massive sandstone, dune sand, and recently excavated fragmental material. The subsurface structural relations, because of the irregular surface of the greenstone, are more complex but may be inferred from exposures in the street cuts and from evidence exposed on the steep face of the escarpment made by the landslide at its upper edge. This escarpment is crescent shaped in plan and about thirty-five feet high. Almost the entire exposure is of dune material. However in the central section of the crescent about fifteen feet below the crest of the slope and underlying the wind-blown sand is a clay outcrop. This clay contains angular fragments of greenstone. As one moves laterally toward the ends of the crescent, the clay exposure passes into dune sand. This clay with its angular greenstone fragments appears to be a soil cover which formed as a weathering product on the formerly exposed slopes of greenstone and which has been subsequently covered by dune sand. Its thickness is not known but is probably of the order of ten feet or less. The writer believes that the greenstone lies within ten feet of the clay exposure although nowhere was it exposed on the face of the escarpment. The only exposure showing greenstone in actual contact with the slide-mass is in the Parker avenue cut where the trace of the northern edge of the landslide passes along the contact between greenstone and sandstone. From this geologic evidence the subsurface structural relations shown in figure 16 have been inferred.

It is very probably that the greenstone surface beneath the hill slope of Lone Mountain acted as a control on the shape and position of the slip-surface of the landslide. It should be understood however that geologic structure alone is not the direct cause of landslides. The ultimate cause of landslides is an unbalanced component of the force of gravity acting on rock masses.

The writer has made a study of the Lone Mountain landslide with a view to quantitatively evaluating the effects produced by the removal of rock material consequent upon the various public and private improvements on and adjacent to Lone Mountain.

The problem resolved itself into two parts: first,

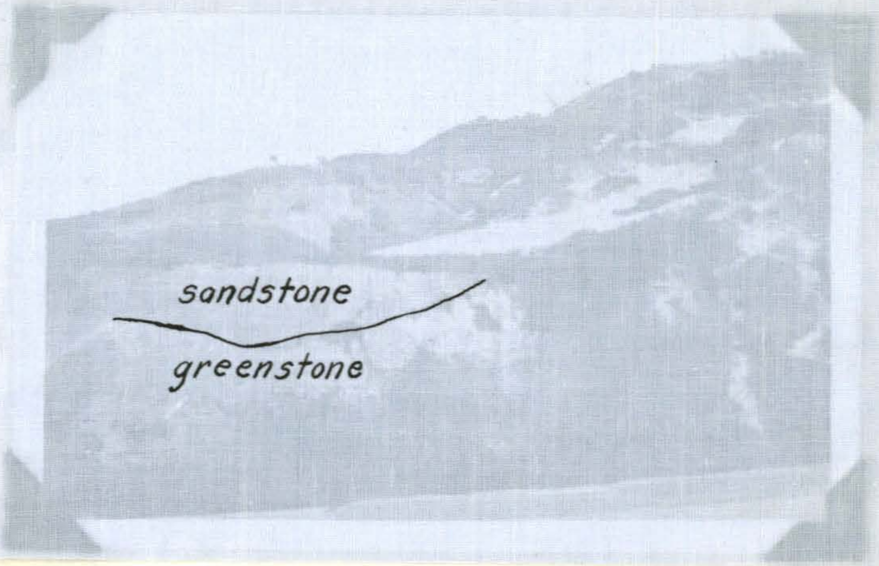


Fig. 10. Cut-crop along Parker Avenue showing contact between sandstone and greenstone.

Fig. 11. Escarpment formed by landslide at its upper edge. Parker Avenue slide.

Fig. 10. Cut-crop along Parker Avenue showing contact between sandstone and greenstone.

Fig. 11. Escarpment formed by landslide at its upper edge. Parker Avenue slide.

to determine the values for the coefficients of cohesion and friction that had existed along the slip-surface when motion impended; second, using these coefficients, to determine the stability of the hill slope for the various conditions which resulted from the civic improvements.

To determine the values for the coefficients, a section was taken through the hillside as it existed just prior to the occurrence of the landslide. Fig. 17. Slip-curves C_1, C_2, and C_3, were constructed. Curve C_2 represents the approximate slip-curve along which the landslide is presumed to have occurred. It is drawn to pass through the outer edge of the mound that was thrust up at the base of Lone Mountain. Its curvature is based on the slope of the escarpment. Moderate variations in curvature affect the calculated values of friction and cohesion only slightly.

Table 1 shows the values for the coefficients of cohesion that are required for maintaining equilibrium along each hypothetical slip-surface when various values are assumed for the angle of friction. Figure 17 shows the data of table 1 plotted to give a series of curves. The curve whose maximum falls on abscissa 0_2 shows a friction angle of 15° and a cohesion of 98 pounds per sq. ft. for slip-curve C_2 along which the slide is presumed to have occurred.

To check the value for the friction angle computed by the method described above, a second calculation was made, assuming a different set of conditions. A section of unit thickness having a post-landslide ground profile was taken and the values for the friction angles along the slip-surface C_1, C_2, C_3 were computed. Fig. 18. Curve C_1 is the curve along which sliding is presumed to have occurred. In this case it was assumed that once motion along the slip surface had started, cohesion vanished and frictional resistance alone finally brought the moving mass to rest. Owing to the momentum that the mass acquired during its descent, it must have moved beyond the position just necessary for static equilibrium and come to rest at a more stable position. Therefore the frictional resistance necessary to maintain the slipped-mass in its new position should be less than that which acted to bring the sliding mass to rest. This proves to be the case and serves as a check on the value computed by the first method. The coefficient of friction calculated for the mass in its new position is .211 compared to .268 which was required to maintain the block in its original position.

Knowing the values for the coefficients of friction and cohesion it is possible to determine the stability of hillsides with various shapes and angles of slope. Stability is defined as the ratio of the force which resists to the force which actuates movement along some

Fig. 12. View of Parker Avenue landslide looking eastward.

Fig. 13. Offset in pavement of Parker Avenue along northern edge of landslide.

slip-surface. A value of 1 indicates the lower limit of stability, that is when motion is impending. Slopes with stability ratios greater than 1 are stable; those with values less than 1 are unstable.

Figure 19 shows a section through the original hillside prior to the construction of Parker avenue. The table on this plate shows that the hillside possessed a stability of 1.5 against sliding.

Construction of Parker avenue, which entailed the removal of a sizable earth prism from the base of hill, caused the stability against sliding on a surface passing beneath the pavement to drop to 1.2. At the same time it created a danger zone for slip-surfaces passing through the toe of the slope in the Parker avenue cut. Fig. 20.

The final grading of the ground adjacent to and west of Parker avenue reduced the stability of the hillside for a slip-surface passing beneath Parker avenue to a value of 1, the lower limit of stability. Fig. 17. Failure occurred on December 26, 1935.

It is to be noted that construction of Parker avenue created a danger zone for slip-surfaces passing through the toe of the east slope of the cut. Fig. 20. Using the values for the coefficients based on the ground profile subsequent to the grading west of Parker avenue and prior to the landslide, the calculated stability against sliding on any of these slip-surfaces is less than 1. Sliding should therefore have occurred. The fact that slides did not occur is evidence to the effect that the active coefficients for the ground in this danger zone were greater than those which acted along the slip-surface of the Parker avenue slide.

Samples of the dune sand and of the clay taken from the slip-surface exposed at the upper edge of the landslide were tested in the laboratory. The experimental values for the angle of internal friction are 21° for the clay and 41° for the sand. Figs. 21, 22. The graphically determined value for the angle of friction that acted when motion impended is 15°. Apparently the full frictional properties of the ground as measured by experiment were not exerted at the instant of sliding.

It is believed that the slip occurred partly along the buried surface of the greenstone and that the full force of friction was prevented from acting because of a film of water along the buried greenstone surface. Fig. 16. This film supported a portion of the superincumbent load and consequently reduced the pressure between the solid matter along this contact. The effect of the decreased pressure was to reduce the frictional resistance but leave undiminished the tangential force acting to produce sliding along this slip-surface. See appendix for calculations showing the effect of hydrostatic lift.

Much of the water used to sprinkle and irrigate the ground surrounding the college on top of the hill sank into the ground and probably percolated downward along the buried greenstone surface. Although the sandstone overlying the greenstone is not an impervious rock, the passage of water through it was probably impeded sufficiently to cause a hydrostatic head of water to build up in the channel formed along the contact of the sandstone and greenstone.

The concept of a hydrostatic pressure acting along the greenstone is supported by the fact that the landslide did not occur along a surface passing through the toe of the Parker Avenue cut. As previously pointed out, a danger zone existed for slip-surfaces passing through the east toe of the Parker Avenue cut. The requisite angle of friction for stability was greater than that for the slip-surface along which the slide actually occurred. No slide occurred through the toe because the static frictional force inherent in the material was not reduced through hydrostatic action.

Weakness of the hillside cannot be attributed to natural precipitation. Reports from the U.S. Weather Bureau show a total precipitation of 1.24 inches for the month of November and of 0.89 inches for the first twelve days of December followed by an interval of thirteen

Fig. 14. Upthrust mound at the foot of Parker Avenue landslide. View looking westward.

days prior to the time of the slide during which no precipitation occurred. This amount of rainfall is no greater than the average for many years in the past and cannot logically be considered as the cause of the landslide.

The position of the mound at the toe of the landslide suggests that the ultimate cause of the slide was the removal of the knoll and the grading of the ground west of Parker Avenue. It should be observed that the undisturbed ground at the base of the hill slopes down toward the north. See figures 12 and 15. Moreover the mound was not thrust up along the entire front of the slide but only along the northern half where the graded ground was lower. From this it appears that as the ground surface sloped downward toward the north it passed below the horizon required for equilibrium and as a consequence when the hill mass slid a mound was thrust up where the ground lay below this horizon.

A chronologic summary of the factors contributing to cause the landslide is :

The construction of Parker Avenue in 1919 and the consequent removal of a sizable earth prism from the base of Lone Mountain reduced the stability of the hillside.

The continuous watering of the grounds atop the hill following construction of the college in 1933 and

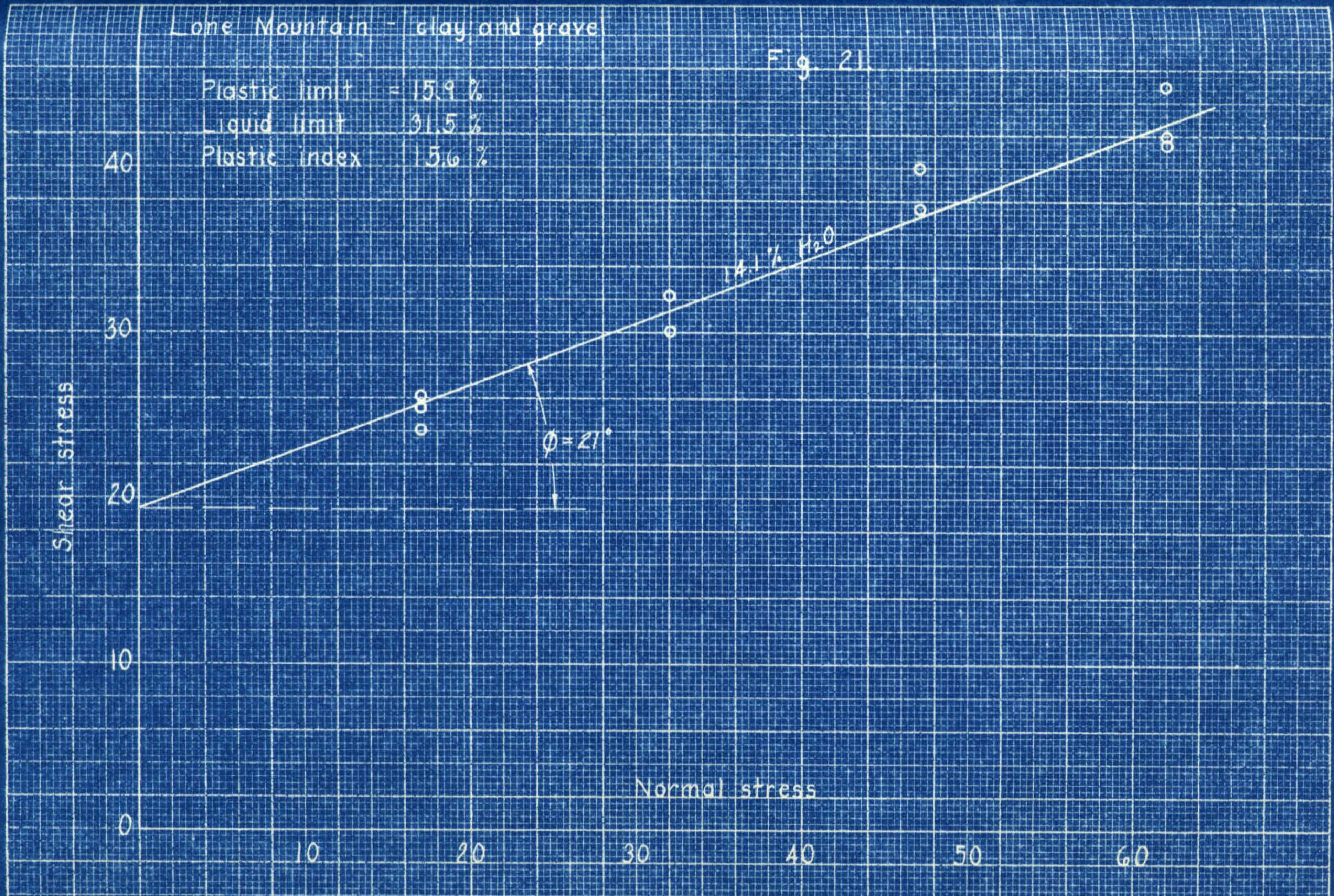
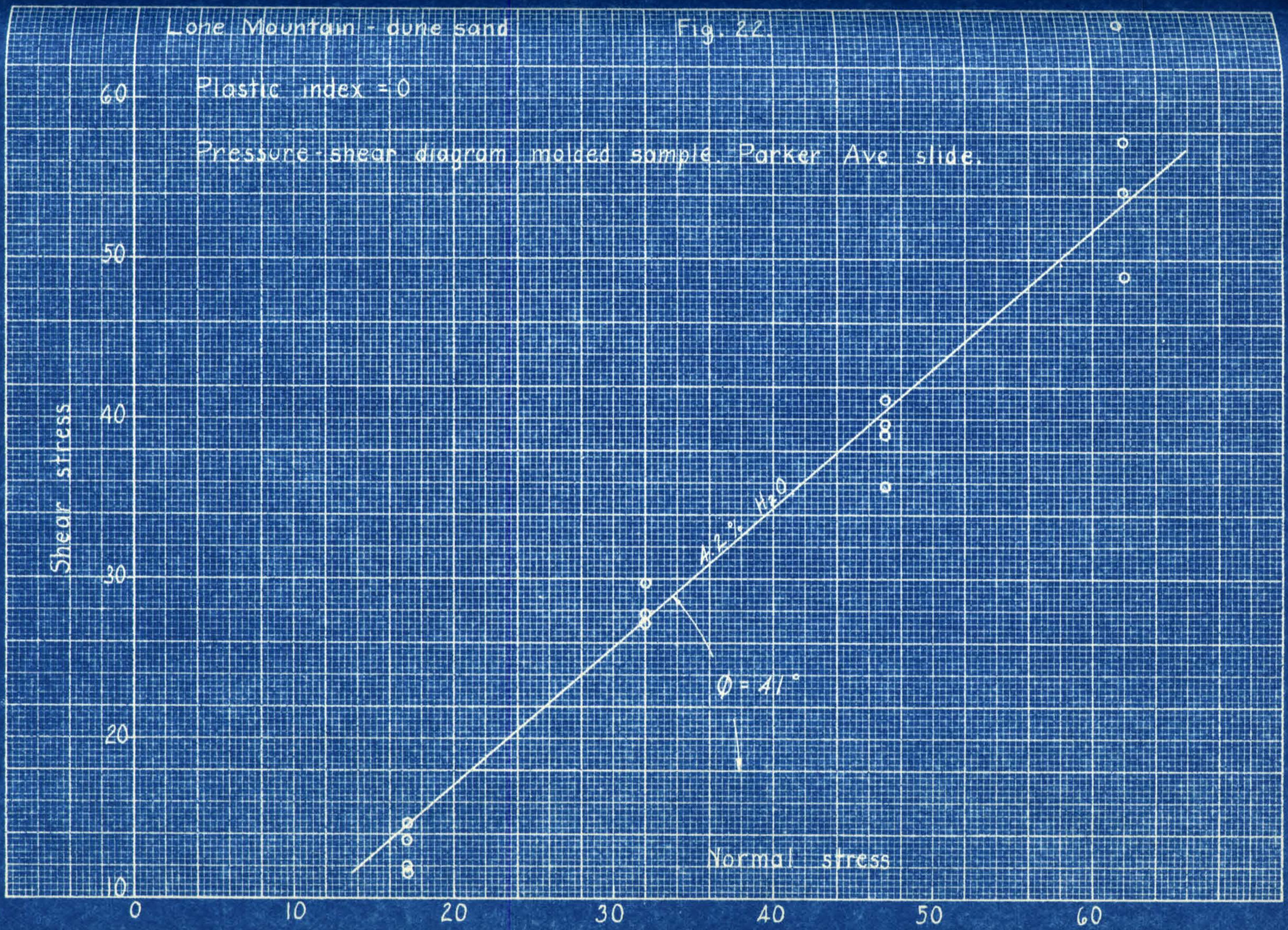
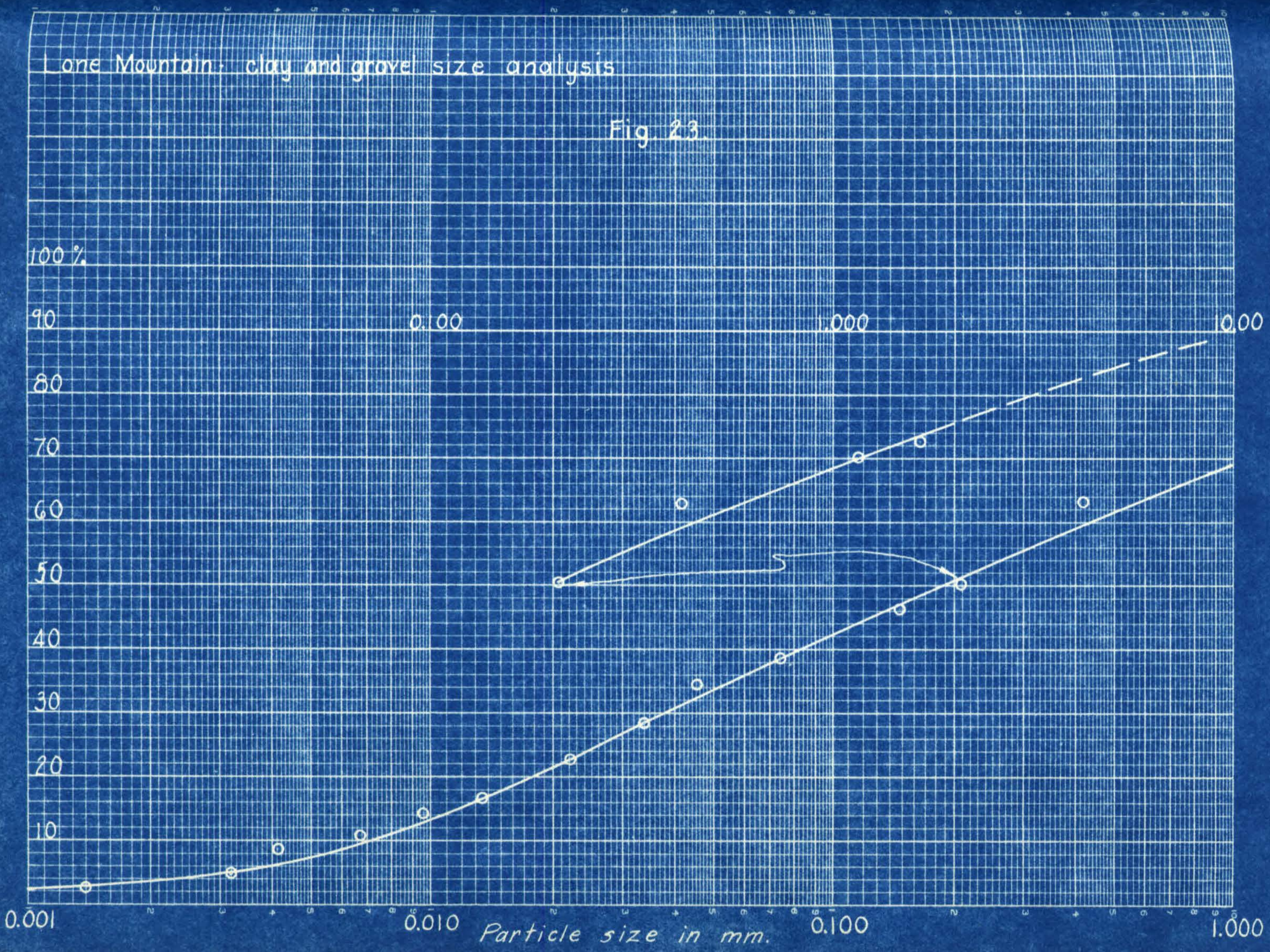


Fig. 21. Pressure-shear diagram, molded sample. Parker Ave. slide.

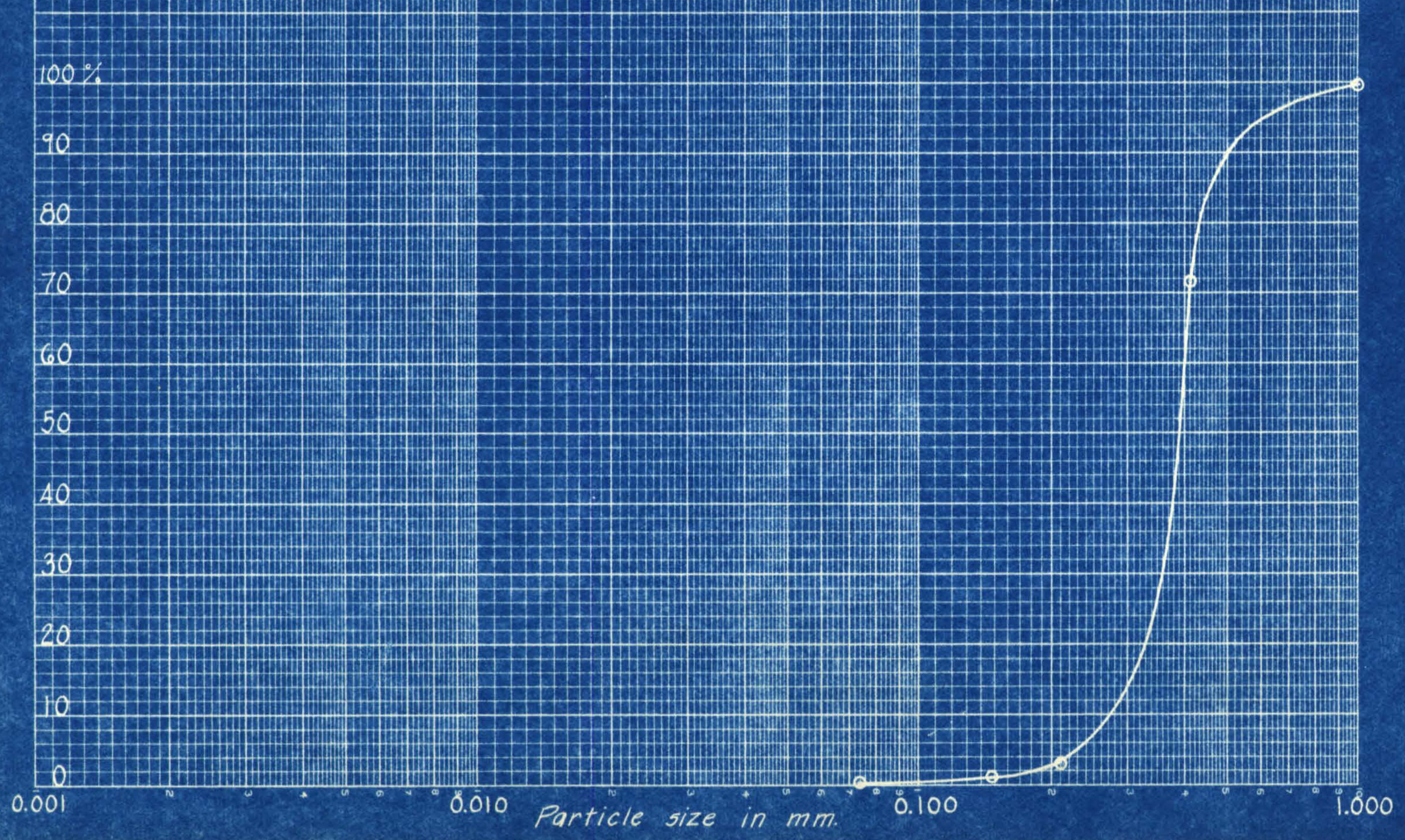


Lone Mountain - dune sand
Fig. 22.
Plastic index = 0
Pressure-shear diagram, molded sample. Parker Ave. slide.
4.2% H_2O
$\phi = 41°$
Shear stress
Normal stress
60
50
40
30
20
10
0
10
20
30
40
50
60


Lone Mountain: clay and gravel size analysis
Fig 23
100 %
90
80
70
60
50
40
30
20
10
0.100
1.000
10.00
0.001
0.010
0.100
1.000
Particle size in mm.

Lone Mountain - dune sand size analysis

Fig. 24.



the probable percolation of the water along the buried greenstone surface served to create a hydrostatic lift which resulted in the reduction of frictional resistance to sliding along this surface.

The grading to the west of Parker Avenue, which was completed in October 1935, reduced the stability of the hillside to the limit of equilibrium.

Failure occurred on December 26, 1935.

Turk Street Landslides-San Francisco

In addition to the large landslide along Parker Avenue several other slides have occurred on Lone Mountain. Two such slides studied by the writer occurred on the south side of Lone Mountain along Turk Street near its intersection with Parker Avenue. These have been designated in this report as landslides A and B. A is the westernmost of the two.

The ground in which these slides occurred is a dune sand, loosely packed and with practically no coherence save that caused by moisture. Figure 24 shows the range of particle size and the high degree of sorting for this material.

No greenstone was exposed in the immediate vicinity of these slides and the writer does not believe that these slips occurred along a buried greenstone surface.

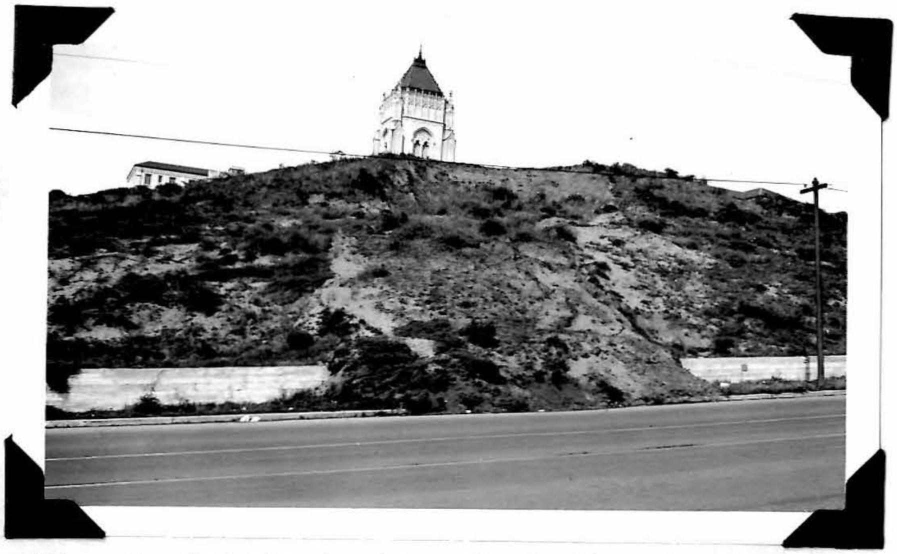


Fig. 27. Landslide A along Turk Street, San Francisco.

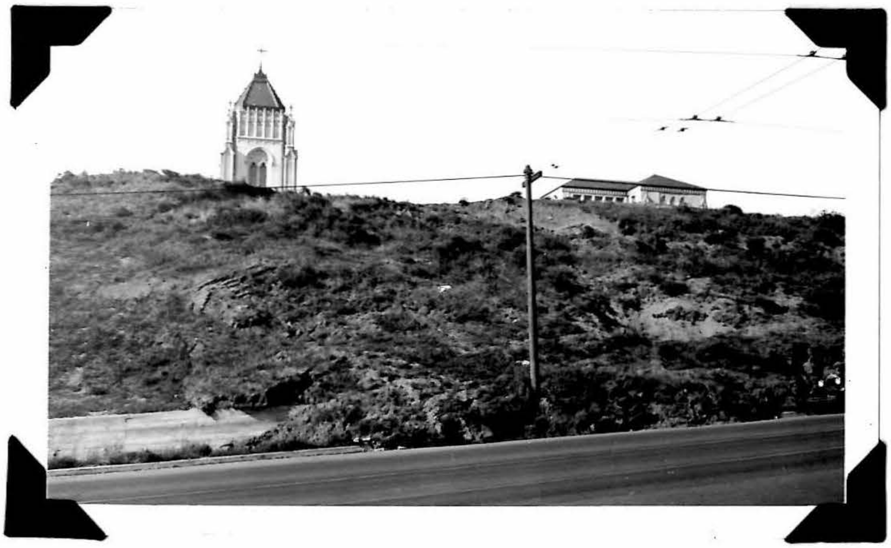


Fig. 28. Landslide B along Turk Street,

San Francisco.

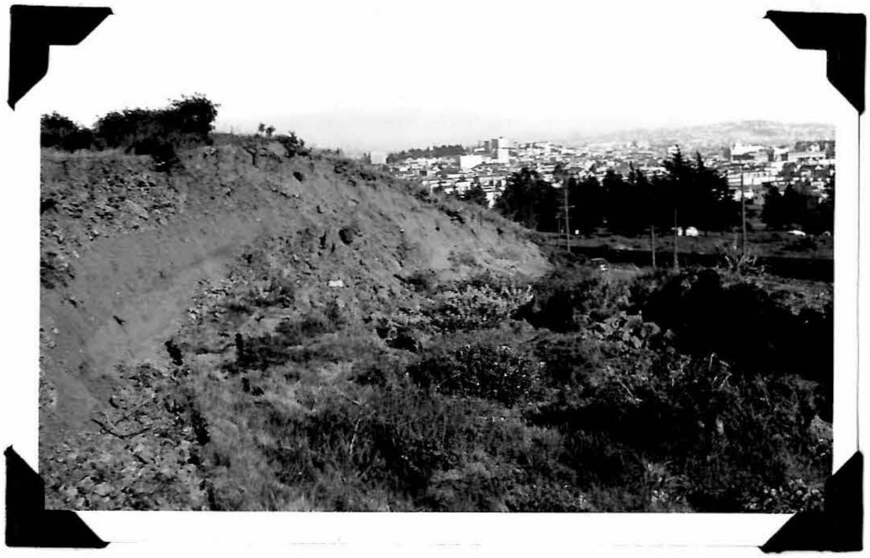


Fig. 29. Escarpment along upper edge of landslide B.

The graphical calculations, Figs. 25 and 26, give as the limiting values for equilibrium possessed by the ground of landslide A, 18° for the angle of internal friction and 50 lbs. per sq. foot for the coefficient of cohesion and for slide B, 12° and 220 lbs per sq. foot.

The experimental value for the angle of internal friction of the dune sand is 41°. Fig. 22. Even under the assumption that the dune sand possessed no cohesion, a friction angle of 41° should have been more than sufficient to give the slope a high degree of stability. Stated in another way, slopes up to a maximum steepness of 41° should have been stable. Yet two landslides occurred on slopes of approximately 25°.

It is obvious that these landslides occurred under conditions that nullified, at least in part, the frictional resistance inherent in the ground. The writer has not been able to secure verbal information concerning the conditions of the ground at the time of sliding. It is possible however to infer the conditions that must have existed. The effect of hydrodynamic stress upon frictional resistance was discussed earlier in this paper. The loose texture and lithology of the ground involved in these landslides are admirably suited for the development of a hydrodynamic stress and are therefore strongly suggestive of the phenomena that caused the slide.

The following description of the conditions leading to the landslides is inferred: The loosely packed dune sand became soaked with water as a consequence of rains. Street cars and motor trucks passing along Turk Street created vibrations that were transmitted to the adjacent hillside. Such vibrations occurred at some instant when the dune sand was saturated with water. They were strong enough to produce in the sand structure a tendency toward a denser state of packing. The voids in the mass of sand were completely filled with water. The reduction in volume during deformation was accompanied by an outflow of an equivalent amount of water. Because the water could not escape quickly enough, the volume decrease lagged noticeably behind that which would have taken place had no water been present. Consequently a hydrodynamic stress was induced and the pressure between the grains was transferred in part or entirely onto the water. This led to a consequent condition in the mass of little or no resistance to shear or a state of liquefaction and the landslides occurred. Figures 27 and 28 show the flow-like character of these landslides.

This phenomenon is admirably demonstrated by means of an experiment described by Casagrande (6).

"In the tank shown ----- is deposited a fine quartz sand in a loose, saturated state, with free water standing on its surface, and a weight is placed on the surface.

Then a stick is thrust into the sand, and suddenly the weight sinks below the surface. The slight but rapid deformation produced by the penetration of the stick results in a change of the sand structure and the formation of hydrodynamic stresses which quickly spread through the entire mass and so decrease the internal friction that the weight can no longer be supported.

"This experiment can be made even more striking by letting water percolate in an upward direction through the sand, which changes it into an exceedingly loose state, and then draining away all water from the surface, so that capillary forces will be mobilized. In this state the surface can support a heavy weight without noticeable subsidence. Upon driving the stick into the sand under these conditions the whole mass seems to liquefy suddenly and the weight disappears completely. In this case the liberation of water due to deformation produces, as a secondary effect, the disappearance of the capillary forces which helped materially in carrying the load."

Casagrande goes on farther to say, "Contrary to the belief of many engineers that a mass of sand will always be stable if the slope is less than the angle of repose, there are many examples on record of embankments, dikes, etc., consisting of fine, saturated sands, being destroyed by flowing out of the entire mass as if it had been suddenly liquefied."

Huntington Landslide

The Huntington slide is a comparatively small landslide along Huntington Drive near Minto Court in Los Angeles. It occurred on the side of a hill which had been trimmed and steepened to a 1:1 slope.

The ground of this locality consists of thin bedded, fine grained tuffaceous rock which, under the microscope, is seen to contain an abundance of fine glass shards. The rock is fractured to a moderate degree. Moreover, the entire surface of the hillside is covered with a network of cracks having openings which vary from a fraction of an inch up to two inches. These cracks indicate an exceptionally high shrinkage coefficient for the ground. During rains much water probably penetrates into the ground along these openings.

The slip surface of the slide intersects the bedding planes of the rock composing the hill. The upper break of the slide is characterized by a steep escarpment about eight feet high and crescent shaped in plan.

In the laboratory the samples from this locality were treated in two ways. 1. Samples of undisturbed material were collected, prepared as described earlier in this paper, and sheared. 2. Samples were disintegrated, moistened and molded into the testing apparatus and sheared.

A group of values for the shearing strengths of undisturbed material was secured. Fig. 30. The values are quite variable, even for material having approximately the same moisture content and subjected to equal normal pressures.

The significance of the tests on undeformed samples is that they prove definitely that no landslide would have occurred had the ground been unfractured and consequently capable of exerting the cohesion possessed by unfractured specimens.

From Fig. 31 the force acting to produce sliding is 6.75 K

$$K = 13,320 \text{ pounds}$$

$$6.75\ K = 90,000 \text{ pounds.}$$

The length of arc along which sliding is assumed to have occurred is 112 feet. The requisite shear resistance for
a
equilibrium along this arc is $\frac{90,000}{112} = 1800$ pounds per square foot. A shearing strength greater than this would insure stability of the slope. The smallest experimentally determined value for the shearing strength of undisturbed and unfractured specimens is about 2450 pounds per square foot which would have been ample for stability. However, the average strength of the hillside, because of prevailing fractures and opening, was much less than the strength of individual unfractured fragments.

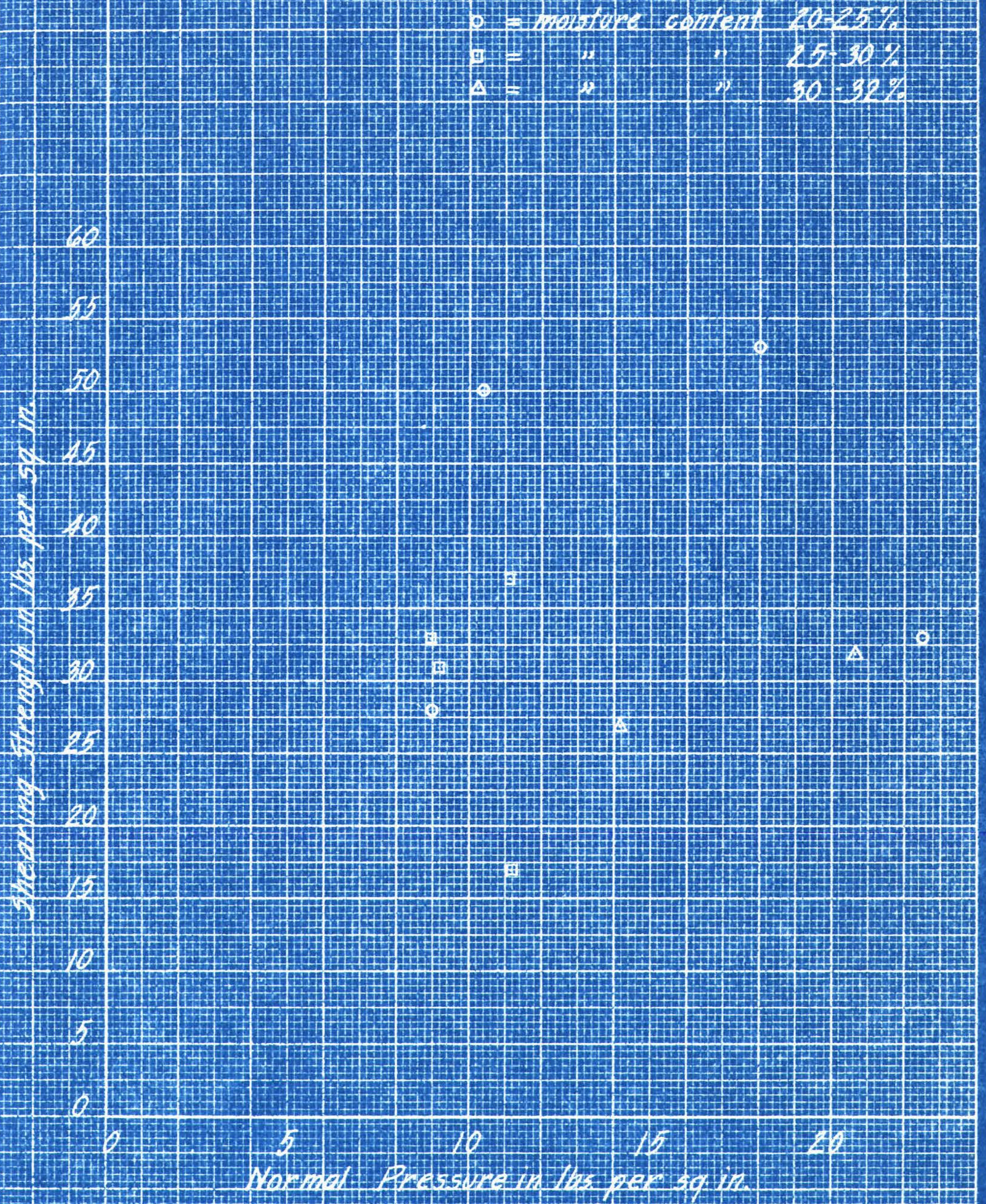


Fig. 30. Pressure-Shear diagram for undisturbed samples, Huntington Landslide.

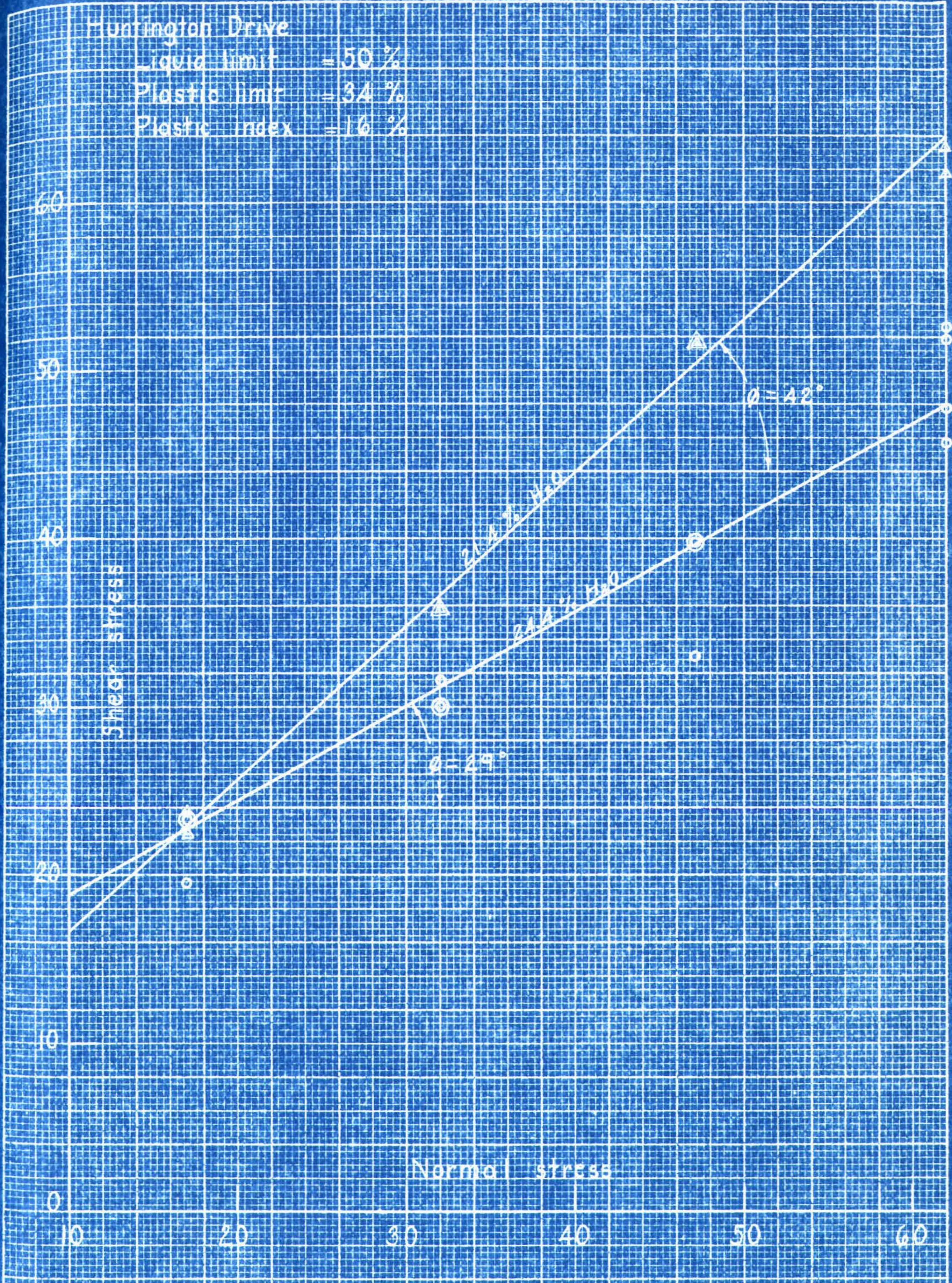
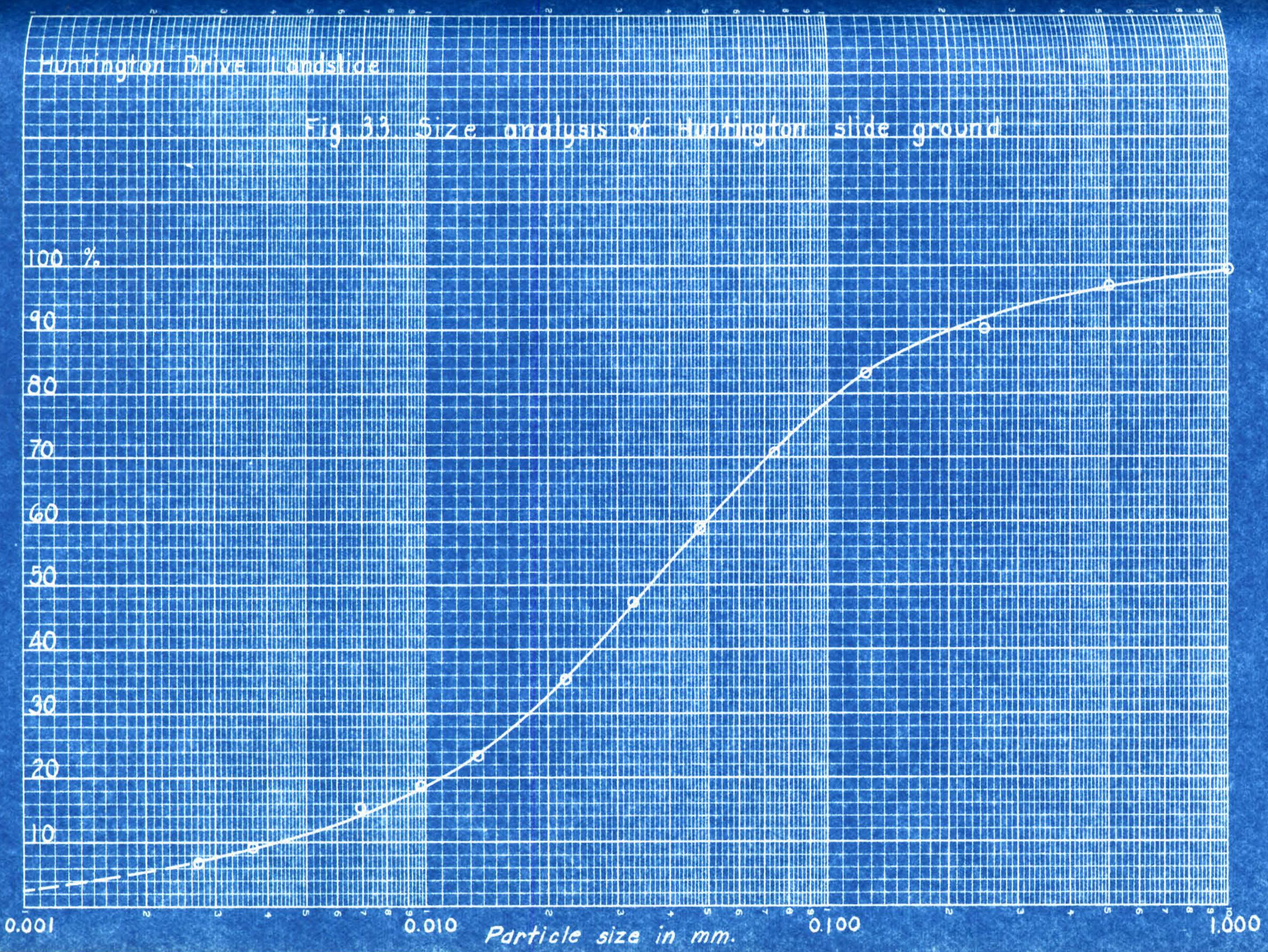


Fig. 32. Pressure-shear diagram - molded sample


Huntington Drive Landslide
Fig 33 Size analysis of Huntington slide ground
100 %
90
80
70
60
50
40
30
20
10
0.001
0.010
0.100
1.000
Particle size in mm.

Values for the angle of internal friction were determined experimentally in tests with molded material. Fig. 32. Angles of 42° and 29° were determined for the same material but with moisture contents of 21.4% and 24.4% respectively. It was difficult to determine the friction angle for samples with a moisture content greater than 24.4%. Internal hydrodynamic stresses were created in these samples when they were loaded with more than thirty pounds. Consequently friction was reduced and the measurements lost their significance.

The writer believes that the angle of friction may be reduced somewhat by the addition of a limited amount of moisture before a hydrodynamic stress begins to act. The reduction in friction is believed to be caused by thin films of water forming around the particles and acting as a lubricant between them. Such a lubricating action depends upon the shape and size of the particles and the accessibility of the water. It is much weaker for large, equidimensional, angular grains than for small, scaly particles.

Even if the ground is considered to have been entirely devoid of cohesion but to have possessed the experimental friction angle of 29°, it would have resisted sliding along the g_2 slip-surface, which is the one along which the slide is presumed to have occurred. Fig. 31. If it were devoid of cohesion however, it should have slipped along a plane surface inclined at an angle of 29° to the horizontal. The field evidence indicates however that the slip-surface is not plane but curved. This is indicated by the steep slope of the escarpment at the top of the landslide and by the topographic expression of the slipped-ground. Since the most dangerous slip-surface for material endowed with cohesion is curved, it must be concluded that the ground did possess cohesion.

It is of interest to note, despite the fact that the topographic expression of the landslide attests to a curved slip-surface, if one assumes the ground to be devoid of cohesion and to have broken along a plane, then the experimental ϕ value of 29° shows a good correlation with the angle of inclination of the plane slip-surface. The trace of the plane drawn in the section between the toe of the original slope and the upper edge of the slide is inclined about 32° to the horizontal. This value differs only a little from the lower of the two values determined experimentally for the angle of internal friction. Superfically it appears therefore that the Coulomb hypothesis of the maximum wedge sliding on a plane has more practical value than the hypothesis of curved slip-surfaces. However this correlation may be purely accidental.

Fig. 34. Huntington landslide.

Fig. 35. Upper edge of Huntington landslide.

Assuming a curved slip-surface the graphically determined values for the angle of friction and coefficient of cohesion which acted when motion impended are respectively 18° and 325 pounds per square foot. This value for the friction angle is considerably lower than that determined by experiment.

The reduction in friction was probably caused by the introduction of a hydrodynamic stress which may have been developed as described below. During a rain much water penetrated deeply into the hillside along cracks and fissures which had formed as a consequence of the high shrinkage coefficient of the ground. Ground that had been shrunken because of internal capillary tension expanded, especially in the free spaces along the cracks. Mud and water filled much of this space. The ground of the walls of the fissures became saturated. Conditions were suitable for the development of a hydrodynamic stress if a portion of the hillside weight could be shifted quickly to increase the load on the saturated ground. A passing motor truck probably set up enough vibration to jar the hill slightly and caused a transfer of load onto the saturated ground. A hydrodynamic stress was developed; frictional resistance to sliding was reduced and the landslide occurred.

Potrero Canyon Slide

This landslide occurred on the west wall of Potrero Canyon about 1200 feet from its mouth. Potrero Canyon debouches from the Santa Monica Mountains at a point along the Roosevelt Highway about three miles northwest of Santa Monica.

The canyon is very narrow, steep walled, and with an average depth of about 180 feet. It is incised in shaly rock belonging to the Modelo formation of Miocene age. In the vicinity of the landslide the beds stand nearly vertically and strike across the axis of the canyon. The Modelo formation has suffered much deformation in past geologic time and is intensely fractured. Scars of many old landslides can be recognized on the walls of the canyon.

The attention of the writer was directed to this landslide because it had caused property damage. A small house that had been built on the west brink of the canyon wall was carried down with the sliding ground and had to be moved and reconstructed. Fig. 36.

The history of the movements of this landslide as recounted to the writer by the owner of the damaged property is given below. For a long time prior to the occurrence of the landslide a crack existed parallel to the axis of the canyon and about eighty feet from the

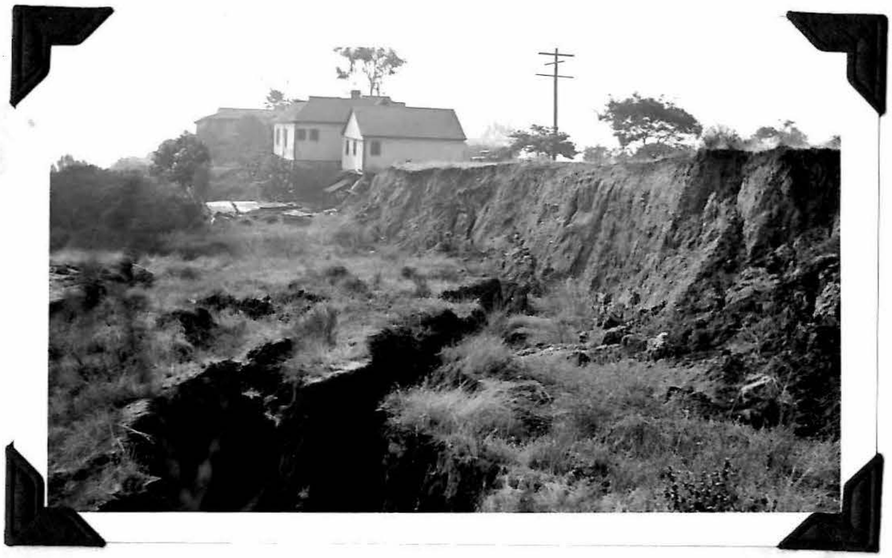


Fig. 36. Upper edge of Potrero landslide. View looking southward.

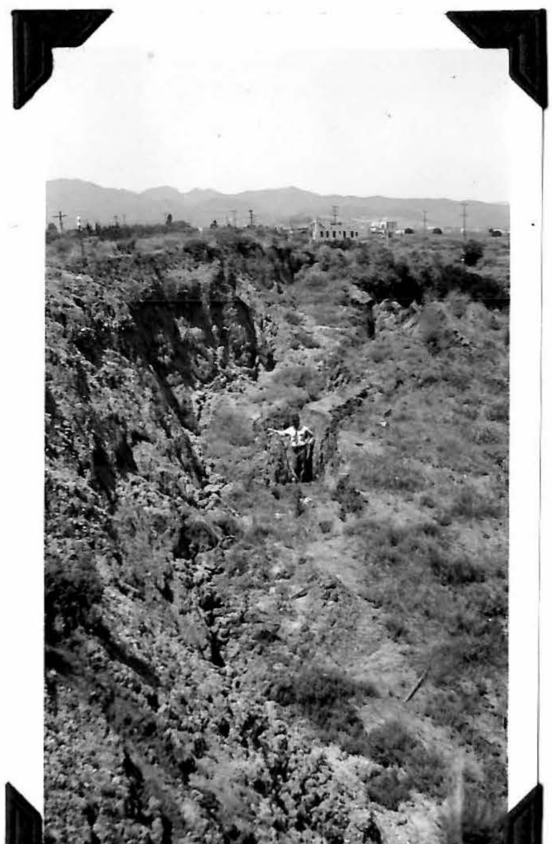


Fig. 37. Upper edge of Potrero landslide. View looking northward.

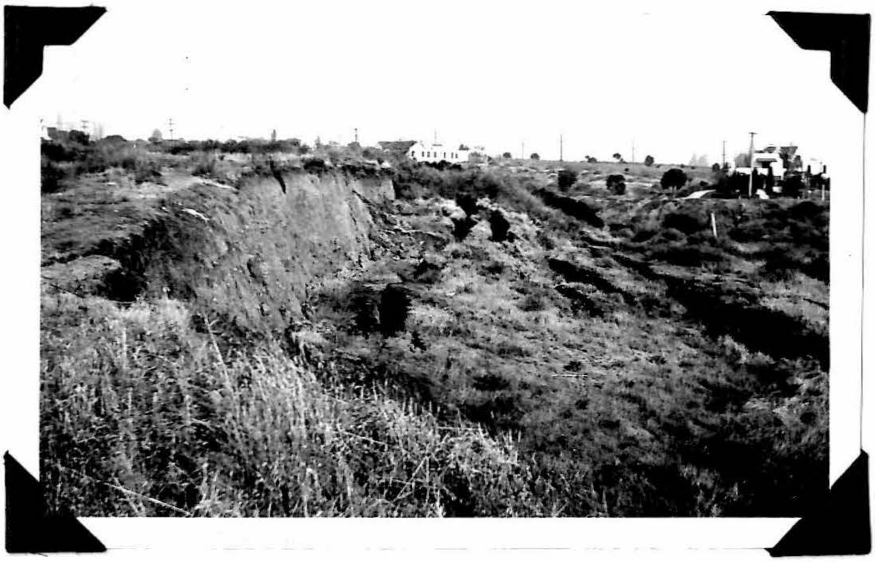


Fig. 38. Uper edge of Potrero slide showing open cracks. View looking northward.

brink. Immediately after the great rain storm of December 31, 1933 a downward displacement of several inches was observed to have occurred along this crack. From that date on movements continued to occur, usually following heavy rain storms. The recently measured vertical displacement of eighteen feet at the upper break of the slide represents the accumulated dislocations for a period of two and one-half years.

The graphically determined values for the friction angle and cohesion at the instant when the slide was imminent are respectively 10° and 2000 pounds per square foot. Fig. 39.

The experimentally determined value for the angle of internal friction is approximately 35°. Fig. 40.

Assuming that the ground is devoid of cohesion, if it had been able to develop a frictional resistance equivalent to the experimental ϕ value of 35°, sliding would not have occurred along curve g_3, figure 39, or any other hypothetical arc passing through the toe of the slope and the upper edge of the slide. Sliding would not have occurred even along a plane between these two points although the stability would have been much less than for the curved slip-surfaces. However, here as in the Huntington slide, using the hypothesis of a plane slip-surface and assuming cohesionless ground, a fair correlation is found to exist between the angle of inclination of the slip-plane (31°) and the experimental angle of internal friction (35°).

The field evidence, however, indicates that the slip-surface is not plane but curved. The steep escarpment which represents the upper portion of the actual slip-surface is very steep. Therefore this surface must curve to a more horizontal position with depth in order to intersect the open face of the slope.

There is an apparent if not real correlation between the experimentally determined angle of internal friction and the inclination of a plane passing between the toe of the slope and the upper edge of the slide, but this plane is most certainly not the slip-surface.

From the general history of the ground movement and from a comparative study of the graphically and experimentally determined values for the frictional resistance of the ground, it is apparent that water played an important part in reducing friction and causing the slide.

It is probable that during rains much water penetrated into the ground through the well developed surface crack. This water soaked into the ground and saturated the surface layer of material in the walls of fissures. Thus the ground was prepared to exert a hydrodynamic stress if loaded suddenly. During rains when

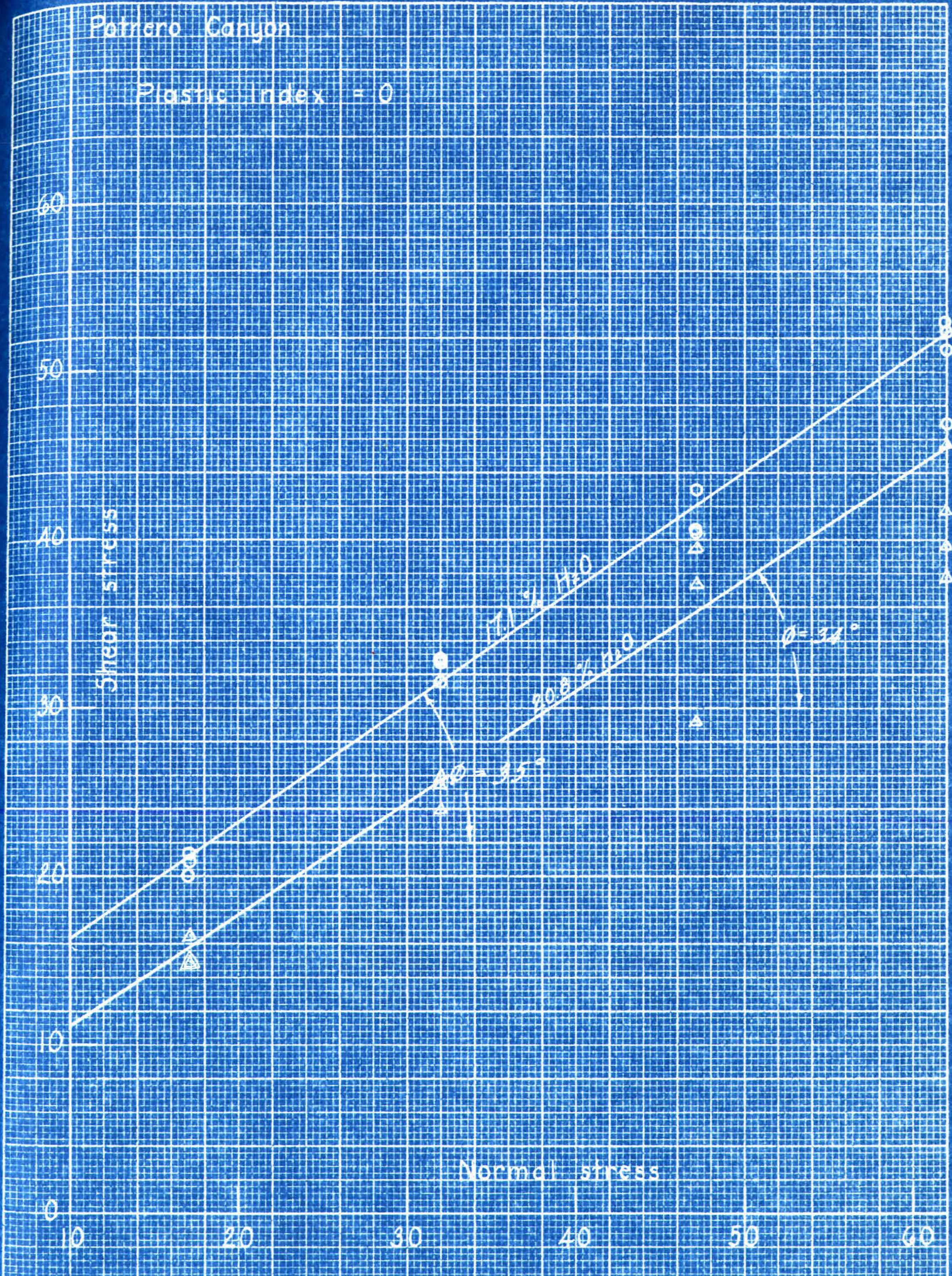


Fig. 40. Pressure - shear diagram - molded sample.

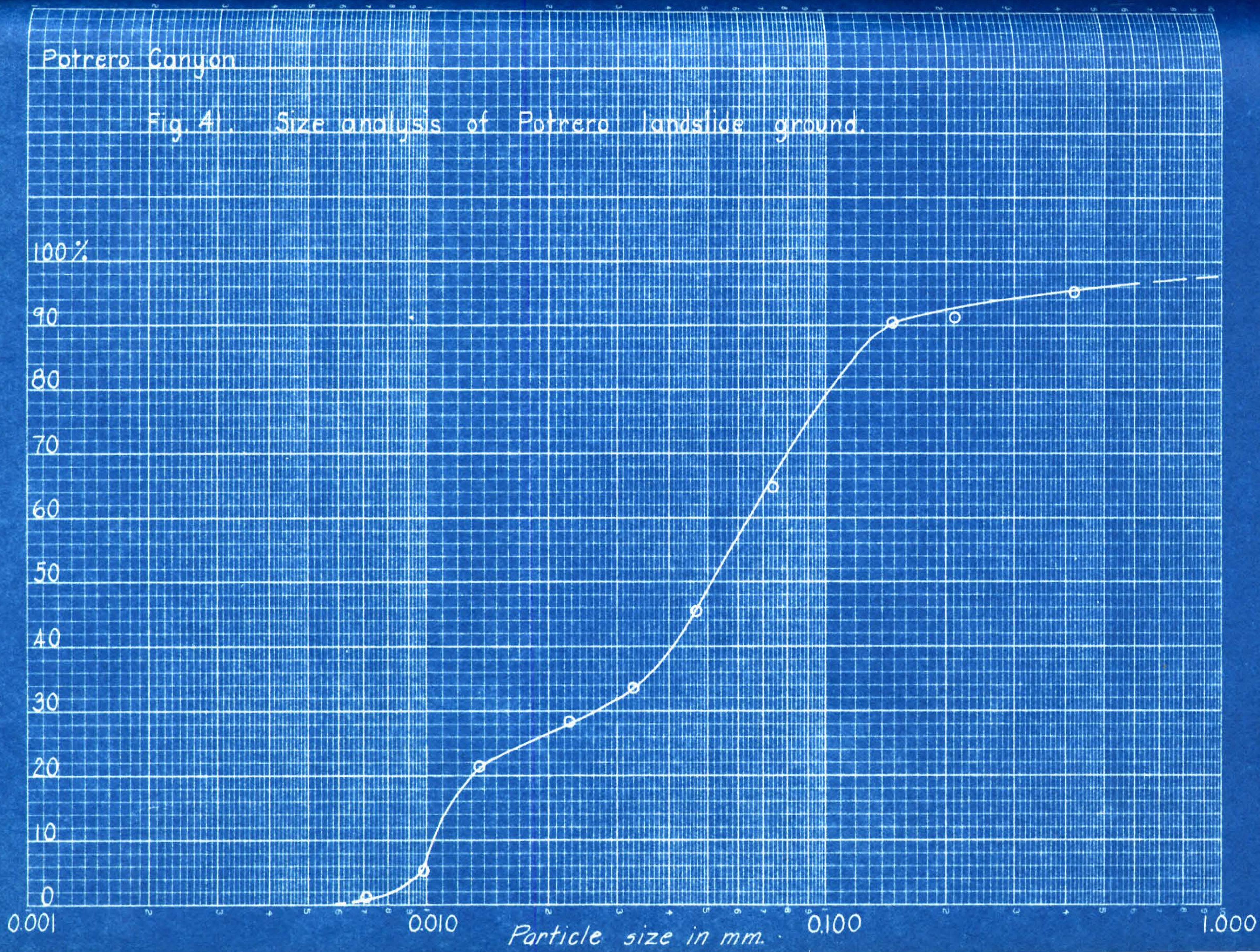


Fig. 4. Size analysis of Potrero landslide ground.

precipitation was heavy, water probably flowed into the fissures more quickly than it could drain away or be soaked up by the ground. Consequently a hydrostatic head of water formed in the fissures. This water, in the inclined fissures, exerted a lifting force upon the superincumbent load and reduced the pressure between the grains of solid matter. Consequently friction was diminished. In addition the water exerted a horizontal component of thrust which tended to push the mass into the canyon. With conditions prepared by a rain storm as described above, slipping of the mass is easy to understand.

The canyon is very narrow at its bottom. As the mass slipped downward a bulge of ground formed at the toe of the slope. This bulge filled the bottom of the canyon, buttressed itself against the opposite wall and tended to prevent further sliding. However, water draining down the floor of the canyon constantly eroded the fill away and allowed sliding to continue.

Bel-Air Landslide

The Bel-Air landslide is along the Roosevelt Highway a few miles northwest of Santa Monica, about 1000 feet west of Pulga Canyon and immediately east of the Bel-Air Beach Club.

It is a comparatively small slide recently initiated

Fig. 42. Oblique airplane photograph showing Bel-Air landslide and other slides along the coast highway.


landslide
landslide
Bel-Air
Slide

in a much older landslide mass as a result of slope steepening consequent upon highway development. The Bel-Air Beach Club is constructed on the more ancient landslide mass. The ground is composed of highly deformed and fractured shale of the Modelo formation (Miocene).

The graphically determined values for the friction angle and cohesion when the slip impended are respectively 10° and 800 pounds per square foot. Fig. 43. These values are for the assumed slip-curve g_3. As the proportion of cohesion to friction becomes smaller, the most dangerous curve for sliding becomes flatter. If ground were entirely devoid of cohesion, then the most dangerous slip-surface would be a plane.

The experimentally determined value for the angle of friction for a molded sample with a moisture content of 24 per cent is 18°. Fig. 44. This value lies between 26° required for a plane slip-surface devoid of cohesion and 10° for the curved surface g_3 which possesses cohesion. The most dangerous slip-curve for ground which possesses cohesion and a friction angle of 18° and which passes through points A and B is one whose curvature is between that of the straight line AB and the curve g_3. It follows therefore that on the basis of the experimentally determined friction value this landslide was to be expected along some curved surface between AB and g_3, provided the cohesion of the ground was not too great to prevent sliding.

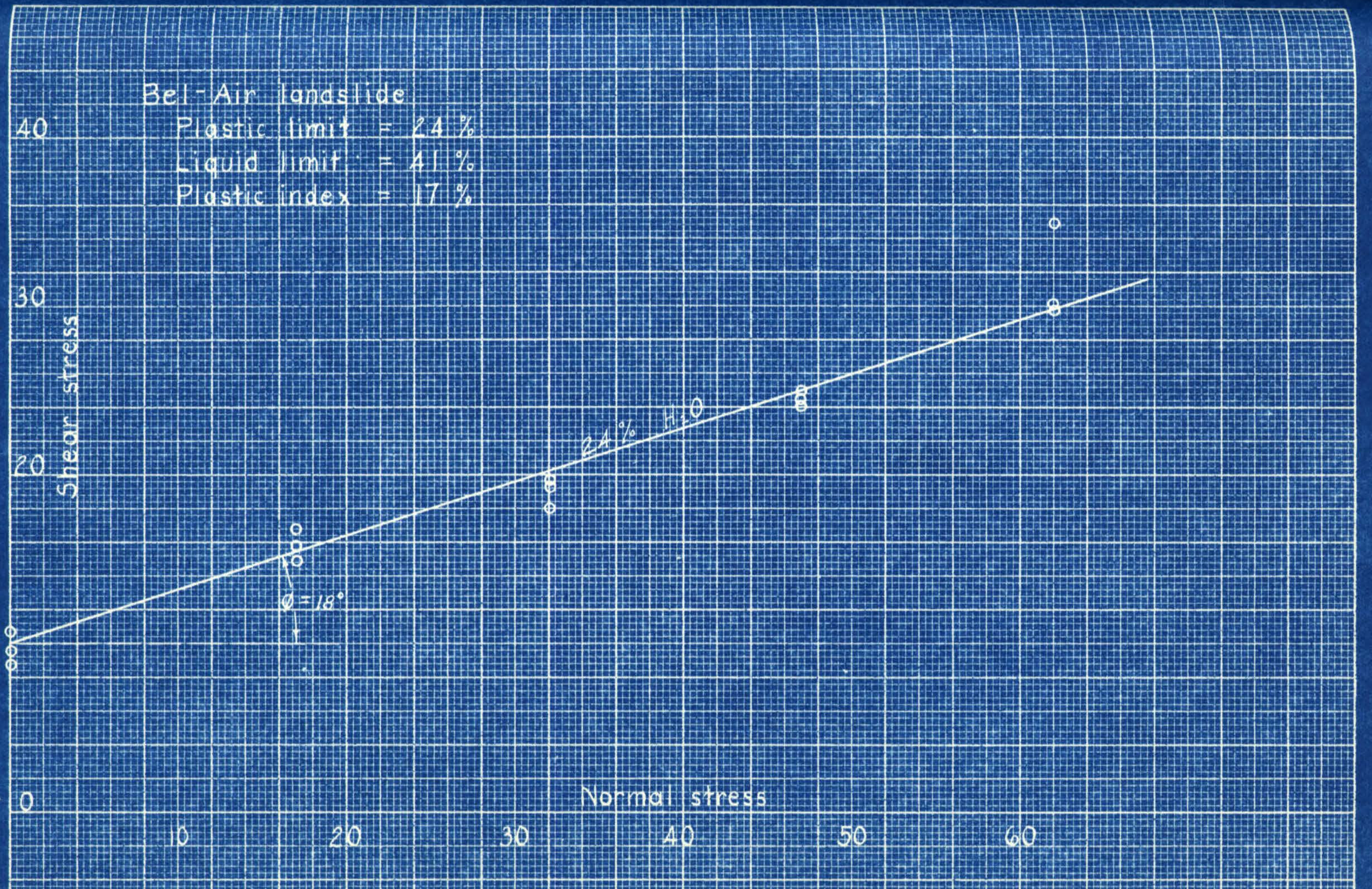


Fig. 44. Pressure-shear diagram - molded sample.

The small angle of friction (18°) which the ground of this locality possessed when wet is all that was necessary to cause landsliding. Although hydrodynamic stresses and hydrostatic thrusts may have acted, the danger of sliding existed without them.

Temescal Landslide

This slide is at the mouth of Temescal Canyon along the Roosevelt Highway about two and one-half miles northwest of Santa Monica. It occurred in the lower part of a trimmed slope, bordering the Highway. The acclivity had originally been cut steeper but the occurrence of several landslides led to remedial measures. These measures consisted in decreasing the steepness of slope and cutting terraces at regular intervals of elevation. Along these terraces channels were excavated and surfaced with asphalt to carry the water that drained down the acclivity during rains. Despite these precautions a landslide occurred. Figs. 47, 48.

The ground is composed of Modelo shale which is intensely broken, more so than in the cases described earlier in this paper. It is difficult to find a firm piece of shale larger than a few inches on a side.

Fig. 46. Oblique airplane photograph showing Temescal landslide.

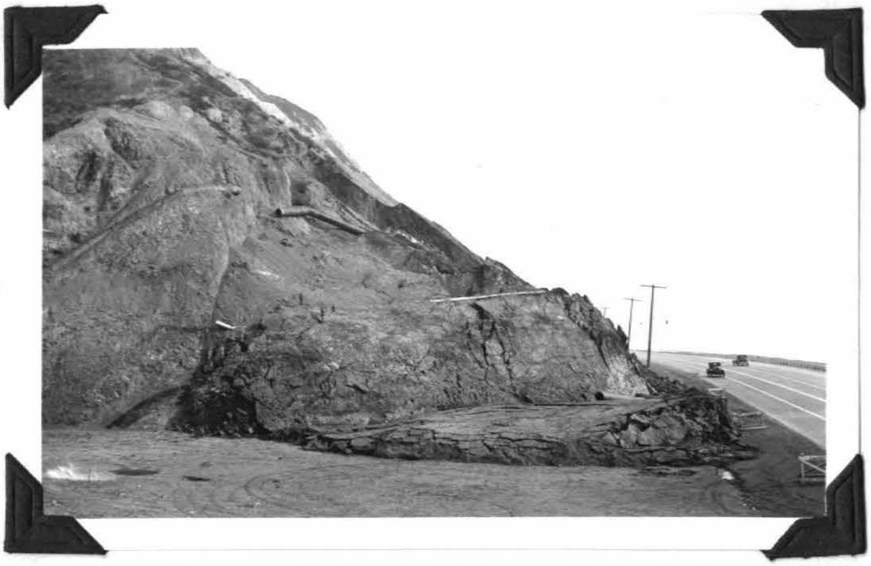


Fig. 47. Temescal landslide. Note bulged lower surface.

Fig. 48. Temescal landslide.

The graphically computed values for the friction angle and cohesion of the slid-ground are respectively 23° and 275 pounds per square foot. Fig. 49.

The experimentally determined friction angle for a molded sample with 35 per cent moisture content is 37°. Fig. 50.

The slope of the ground prior to the slide was approximately 36°. It is apparent therefore, even assuming cohesionless material, that no landslide would have occurred had the ground exerted its full static friction. For this slide as for the others described earlier in this paper, the cause must be attributed to hydrodynamic stresses initiated probably during a rain after the ground had become saturated with water. As a consequence of the hydrodynamic stress, frictional resistance against sliding was reduced below the limit of stability and the landslide occurred.

A moderately good correlation exists between the experimentally determined friction angle (37°) and the inclination (36°) of a plane between the upper and lower traces of the slip-surface. Working on the hypothesis of a plane slip-surface and cohesionless soil, it could have been said from the experimental friction value that the trimmed slope was close to the limit of stability. However this correlation, which may be accidental, is not between the slope of the actual slip-surface and the

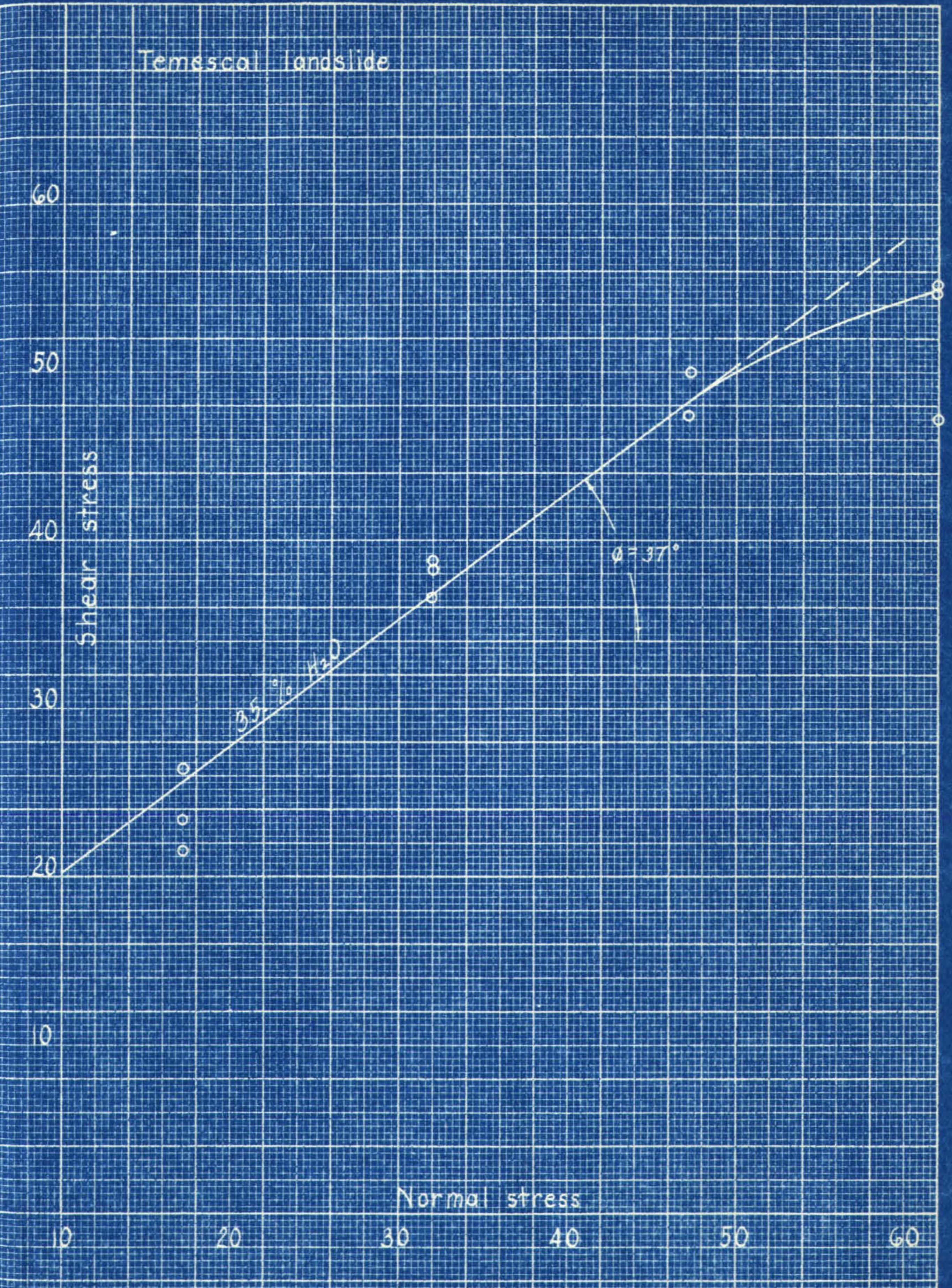


Fig. 50. Pressure-shear diagram - molded sample.

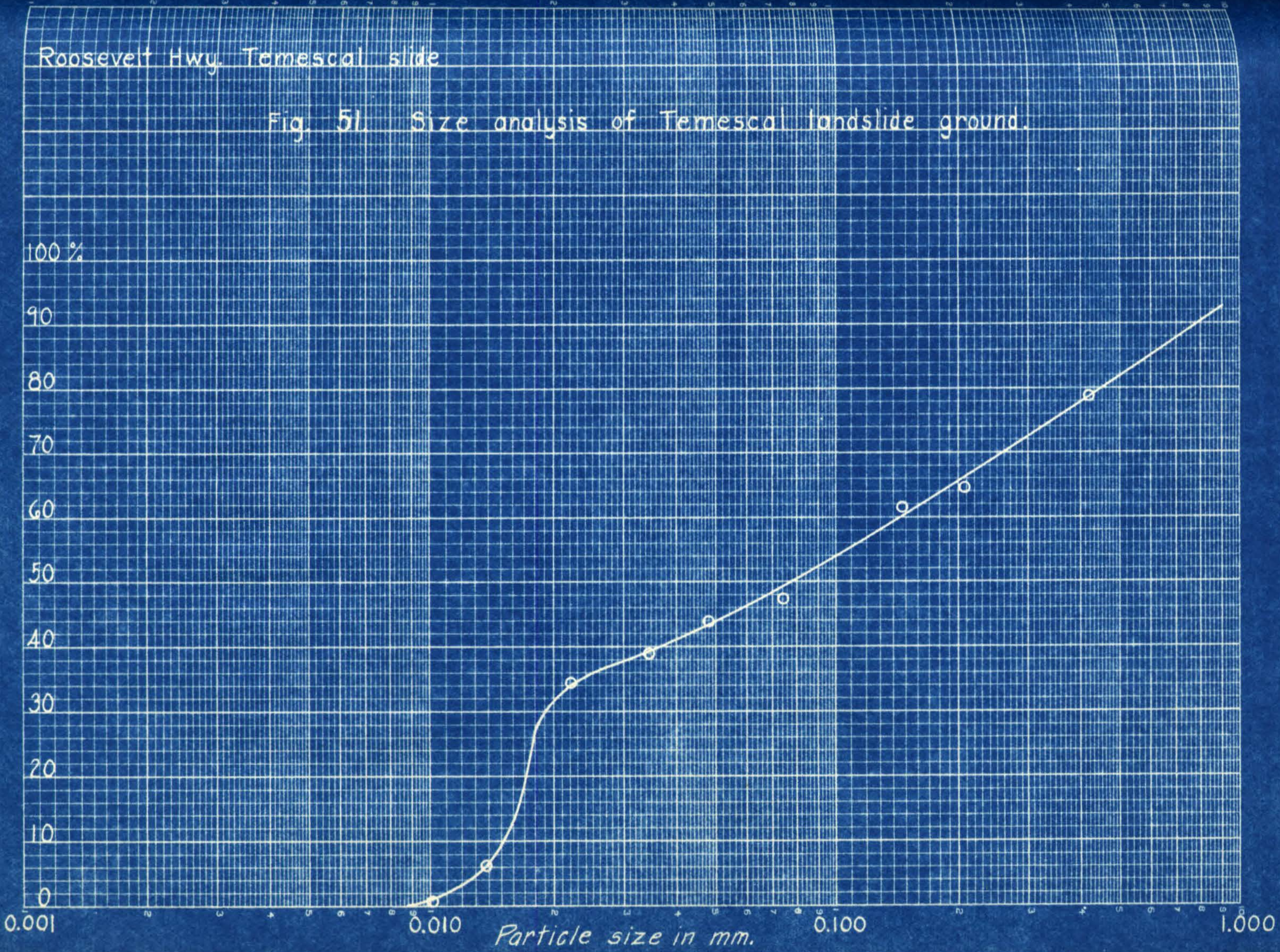


Fig. 51. Size analysis of Temescal landslide ground.

friction angle but between the inclination of a purely hypothetical plane and the friction angle.

Here as in several of the other slides already described, the field evidence attests to a curved slip-surface. The exposed portion of the slip-surface along the upper edge of the slide is very steep whereas a portion of the lower trace of the slip-surface is well out in front of the toe. It is obvious that the slip-surface exposed at the upper edge of the slide must curve with depth in order to intersect the ground at the lower trace. See figure 47.

CONCLUSIONS

The purpose of this investigation was to determine whether landslides could be predicted from experimentally determined values for the frictional resistance and cohesion of ground. Early in the investigation it became evident that the cohesive strength of undisturbed samples as measured in the laboratory was not the same as the average cohesive strength of a large mass of ground in the field, especially when the latter was thoroughly fractured. Measurements of cohesion for undisturbed samples had no significance therefore and were not continued.

The frictional resistance was measured for molded samples from seven landslides. On the assumptions of a cylinderical slip-surface and ground possessed of both cohesion and friction it would have been possible in only one case (Bel-Air slide) to have predicted a landslide with reasonable assurance on the basis of the experimentally determined friction angle. In the other six cases friction, as determined from experiment, should have been sufficient to prevent movement along the cylinderical slip-surfaces, even in ground devoid of cohesion.

Assuming Coulomb's hypothesis of a plane slip-surface and ground devoid of cohesion, two of the slides (Potrero and Huntington slides) could have been predicted from the laboratory determined friction data. That is to say, for these slides the measured angle of internal friction is approximately equal to the inclination of a plane drawn through the toe of the original slope and the upper edge of the landslide. This correlation is probably accidental, since the field evidence clearly indicates that the slip-surface is not a plane.

The writer believes that the hypothesis of a cylinderical slip-surface and the assumption of ground possessed of friction and cohesion leads to a truer understanding of the mechanics of landslides than Coulomb's hypothesis.

In order to properly estimate the probability of landslides the investigator should have information concerning the geology as well as the physical properties of the ground under consideration. The locality should first be given a thorough geologic examination which should yield information concerning the attitude of strata and of joints or fissures, the lithology of the beds, the magnitude and position of faults, the shape and position of unconformities, the character of the ground whether fractured or solid, and the position of the ground water level. Any of these geologic elements may play an important role in contributing to the cause of a landslide.

The attitude of all surfaces such as bedding planes, joints, fissures, faults, and unconformities already developed within a ground mass should be determined. The danger of a slide is markedly increased if any of these surfaces dip toward the open face of a cut. It is important to known the condition of the existing surface, whether it is highly irregular, plane, curved, rough, or smooth because the frictional resistance necessary to maintain stability varies with the shape and inclination of the slip-surface.

The lithology of the various rock units should be determined with a view to estimating their permeability to water and to gain some understanding of the drainage channels through the mass. Water moving slowly between two impervious units may create a sufficient hydraulic lift to reduce frictional resistance below the requisite value for stability. Material filling fissures or coating the existing surfaces, and gouge in fault zones should be examined. Clayey material, because of its low permeability will develope hydrodynamic stresses when saturated and subjected to rapid increases in load, As a consequence, friction will be reduced and slides may occur.

It is well to know the fluctuations of the ground water level. The buoyant effect of the water upon the ground mass may sufficiently reduce the pressure and consequently the friction between particles so that a slip will occur.

In ground that is more or less homogeneous, where definite continuous surfaces do not already exist or where there dip is into the hill, the elements controlling landslides are somewhat different.

If the ground is badly fractured it is best to assume it to be devoid of cohesion. In this type of ground the important factors that should be determined are the friction angle and permeability. The friction angle may be determined by experiment as described in this paper or by disintegrating material and measuring the angle of repose. The former method is probably the better since the friction angle may be determined for different moisture contents and the effect of cohesion may be excluded. The friction angle should be considered as the maximum angle of slope for which the ground will be stable when good drainage exists. In ground where no drainage is provided the permeability must be considered in addition to the friction angle. From assembled data it appears that material composed of particles smaller than fine sand is sufficiently impervious to cause the formation of hydrodynamic stresses when loads are suddenly applied to the saturated material. A size analysis or hand lense examination should therefore be sufficient to determine whether hydrodynamic stresses may be created. In ground where this possibility exists and where no provisions are made for water drainage a slide may occur even on slopes inclined much below the friction angle.

For homogeneous unfissured material possessing little cohesion such as dune sand the factor of structural density in addition to the friction angle and permeability must be considered in its effect upon landslides. If the natural structural density is less than the critical density, the material will assume a denser packing when subjected to shearing forces. Such ground when saturated with water and jarred will tend to assume a denser structural state. The water in the pores will try to escape as the porosity is diminished. If the water cannot escape quickly enough, hydrodynamic stresses will be created within the mass. The pressure between the grains will be reduced and the whole mass may liquefy and produce a flow slide.

The most ubiquitous agent that acts to reduce friction and cause sliding is water. No matter how high the normal frictional resistance of ground may be, if a hydrodynamic stress can be induced within it, sliding may occur. The hydrodynamic coefficient of friction which replaces the normal friction in such cases may drop as low as zero, in which case the superincumbent load is supported on a film of water.

Ground that is badly broken possesses dangerous possibilities for sliding because of the almost complete loss of cohesion and the ease with which water penetrates into the interior of the mass and prepares the ground for the initiation of a hydrodynamic stress. This statement is substantiated by field evidence along the coast highway for several miles northwest of Santa Monica in Southern California. The bluffs along the highway near Santa Monica, commonly known as the Palisades, stand very steeply and few landslides have occurred along them. They are composed of gravel bound by clay and are almost entirely without joints or fissures. They are able to stand so steeply because of their high cohesive strength which has not been weakened by fractures. Farther along the coast, about two miles from Santa Monica the Modelo

Fig. 52. Oblique airplane photograph showing Palisades along coast highway at Santa Monica.

shale outcrops. There is hardly a place where this shale is exposed for several miles along the highway that landslides either ancient or recent have not occurred. This shale is intensely fractured and deformed.

This study has shown that in addition to friction measurements a thorough field examination to consider every possibility for the penetration of water into the ground mass is necessary for a proper estimation of the probability of landslides.

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The writer wishes to express his gratefulness to all persons not specifically mentioned here who have in any way contributed to this study.

APPENDIX

Parker Avenue Landslide.

The experimentally determined angle on internal friction for the clay of the Parker avenue slide is 21° and for the dune sand is 41°. The graphically computed acting angle of friction when motion impended is 15°.

Let us assume that the material along the slip-surface inherently possessed an average friction angle of 25°. Let us then calculate the hydrostatic left necessary to reduce the intrinsic friction angle of 25° to an apparent one of 15°.

Assume:

1. The hydrostatic lift acts normal to the surface between points A and B. Figure 16.
2. The surface between A and B is plane.
3. The resultant of the weight of the section acts through the lower 1/3 point between A and B.
4. The component of the weight of the section normal to AB is 26.4K. See figure 17.

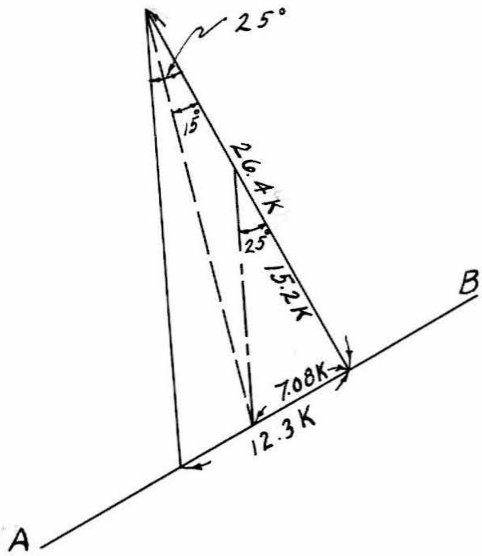

Figure 53

In figure 53 the frictional resistance induced by the normal force 26.4K is 12.3K when $\phi = 25^{\circ}$ and 7.08K when $\phi = 15^{\circ}$. The reduction in frictional resistance from 12.3K to 7.08K may be caused without introducing a smaller friction angle if the normal force acting to press the particles together is reduced from 26.4K to 15.2K. This condition occurs when a hydrostatic lift acts. The requisite hydrostatic lift to create an apparent friction angle of 15° is 11.2K.

$$K = 53{,}400 \text{ lbs.}$$

$$11.2K = 596{,}000 \text{ lbs.}$$

$$P = w \bar{z} A$$

Where P = hydrostatic force acting on surface

w = density of water

$\bar{z}$ = depth to the center of gravity of the surface below water

A = area of surface

Given $P = 596{,}000$ lbs.

$w = 62.4$ lbs. per cu. ft.

$A = 170$ sq. ft.

To find $\bar{z}$

$$\bar{z} = \frac{P}{w\,A}$$

$$\bar{z} = \frac{596{,}000}{(62.4)\,(170)}$$

$$\bar{z} = 56 \text{ ft.}$$

To produce the hydrostatic lift of 596,000 lbs. on surface AB a film of water would have to stand to point C, figure 16. Under the conditions stipulated this would result in an apparent friction angle of 15° along the slip-surface.

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Parker Avenue Landslide

$$R = L\,c + N \tan \phi \qquad \text{(see Fig. 3.)}$$

$$c = \frac{R - N \tan}{L}$$

R = total resistance along curve

N = total normal force acting at "g"

L = length of curve in feet

c = coefficient of cohesion

$\tan \phi$ = coefficient of friction

Table 1

Slip-curve	R	N	L	ϕ	$\tan \phi$	c
C_1	4.75K*	17.15K	412 ft.	10°	.1763	.00420K
				12°	.2126	.00267K
				15°	.2679	.000364K
C_2	7.80K	26.35K	434 ft.	10°	.1763	.00715K
				12°	.2126	.00506K
				15°	.2679	.00184K
C_3	10.9K	40.35K	467 ft.	10°	.1763	.00814K
				12°	.2126	.00493K
				15°	.2679	.000214K

* K = 53,400 lbs.

Turk Street Landslide A-San Francisco

Table 2.

Slip Curve	R	N	L	ϕ	tan ϕ	c
0_1	3.5K*	10.0K	146 ft.	12^o	.2126	.00956K
				15^o	.2679	.00548K
				17^o	.3057	.00302K
				18^o	.3249	.00171K
0_2	6.6K	18.5K	166 ft.	12^o		.0162K
				15^o		.0102K
				17^o		.00603K
				18^o		.00361K
0_3	10.2K	30.8K	185 ft.	12^o		.0195K
				15^o		.0103K
				17^o		.00433K
				18^o		.00108K

* K = 13,320 lbs.

See table I for explanation of symbols.

Turk Street Landslide B-San Francisco

Table 3.

Slip Curve	R	N	L	ϕ	tan ϕ	c
0_1	4.8K*	13.1K	133 ft.	10°	.1763	.0188K
				12°	.2126	.0150K
				14°	.2493	.0113K
				16°	.2867	.00752K
0_2	6.6K	19.7K	144 ft.	10°		.0215K
				12°		.0167K
				14°		.0118K
				16°		.00625K
0_3	8.5K	28.5K	160 ft.	10°		.0218K
				12°		.0150K
				14°		.00875K
				16°		.00188K
0_4	10.5K	40.2K	177 ft.	10°		.0192K
				12°		.0113K
				14°		.00282K
				16°		.00565K

* K = 13,320 lbs.

See table I for explanation of symbols.

Huntington Landslide

Table 4.

Slip Curve	R	N	L	ϕ	$\tan \phi$	c
O_1	4.8K*	7.8K	98 ft.	14°	.2493	.0292K
				16°	.2867	.0261K
				18°	.3249	.0230K
				20°	.3640	.0200K
O_2	6.8K	12.3K	112 ft.	14°	.	.0329 K
				16°		.0287K
				18°		.0246K
				20°		.0203K
O_3	9.4K	19.9K	133 ft.	14°		.0368K
				16°		.0278K
				18°		.0218K
				20°		.0165K
O_4	12.9K	33.6K	154 ft.	14°		.0292K
				16°		.0214K
				18°		.0130K
				20°		.00455K

* K = 13,320 lbs.

See table I for explanation of symbols.

Potrero Landslide

Table 5.

Slip Curve	R	N	L	Ø	tan Ø	c
O_1	13.5K*	20.4K	287 ft.	10°	.1763	.0345K
				14°	.2493	.0293K
				18°	.3249	.0240K
O_2	16.5K	28.1K	306 ft.	10°		.0376K
				14°		.0310K
				18°		.0242K
O_3	19.5K	38.4K	334 ft.	10°		.0380K
				14°		.0296K
				18°		.0210K
O_4	21.9K	50.9K	364 ft.	10°		.0355K
				14°		.0253K
				18°		.0148K

* K = 53,400 lbs.

See table I for explanation of symbols.

Bel-Air Landslide

Table 6.

Slip-Curve	R	N	L	ϕ	$\tan\phi$	c
O_1	9.6K*	22.9K	216 ft.	10°	.1763	.0259K
				12°	.2126	.0218K
				14°	.2493	.0181K
				16°	.2867	.0139K
O_2	11.9K	30.5K	240 ft.	10°	.	.0270K
				12°		.0225K
				14°		.0179K
				16°		.0133K
O_3	14.3K	40.8K	265 ft.	10°		.0268K
				12°		.0211K
				14°		.0155K
				16°		.0098K
O_4	16.8K	54.5K	290 ft.	10°		.0248K
				12°		.0179K
				14°		.0110K
				16°		.0041K

* K = 30,000 lbs.

See table I for explanation of symbols.

Temescal Landslide

Table 7.

Slip-Curve	R	N	L	ϕ	$\tan\phi$	c
O_1	8.1K*	12.7K	137 ft.	20°	.3640	.0255K
				23°	.4245	.0197K
				24°	.4452	.0175K
				25°	.4663	.0160K
O_2	12.4K	21.4K	161 ft.	20°		.0286K
				23°		.0205K
				24°		.0180K
				25°		.0149K
O_3	18.4K	34.4K	188 ft.	20°		.0314K
				23°		.0202K
				24°		.0165K
				25°		.0128K

* K = 13,320 lbs.

See table I for explanation of symbols.