27 March 1951

To the Dean and Heads of Departments
of the College of Engineering,
University of Maryland.

Gentlemen:

Having complied with all the other requirements for the professional degree of Civil Engineer prescribed by the regulations for granting such degrees by the University of Maryland, I now submit herewith, as further required, a thesis on the subject "The Application of Soil Mechanics to Highway Foundations."

Respectfully,

Edward S. Barker B.S.
in Civil Engineering
class of 1935

UNIVERSITY OF MARYLAND
COLLEGE OF ENGINEERING

THESIS

Submitted for the Professional Degree

of

Civil Engineer

by

Edward S. Barber

class of 1935

THE APPLICATION OF SOIL MECHANICS TO HIGHWAY ENGINEERING

27 March 1951

CONTENTS

Introduction

The highway design engineer is constantly faced with the necessity of making decisions dependent on a knowledge of the structural properties of soils. For instance, in the design of a bridge with its approaches, he must decide on a safe bearing value for the foundation soil, the pressure which will be exerted against the abutment by the adjacent fill, the safe slopes to use in cuts and on fills, the amount and rate of settlement of abutments and fills, and similar factors.

In the design of large structures, detailed investigations are usually made to provide specific data on the soils which will be utilized. In the design of smaller structures, it is common practice for the engineer to rely more heavily on the log of a few borings, past experience with soils in the same locality, and on rules of thumb. This is due to no lack of desire on the part of the engineer to utilize the results of soil tests but rather to the belief that they can be obtained only through laborious and costly tests and that involved and tedious computations are required to translate the results into information of practical value.

The purpose of this report is to present procedures for structural tests of soils which can be performed with a minimum of time and equipment, to show the application of the test results in a simple design problem, and to illustrate current practice by examples abstracted from the author's reports as consultant in private practice and with the U. S. Bureau of Public Roads.

For the sake of simplicity some approximations have been used and many considerations have been only mentioned or entirely neglected. The desired factor of safety is a matter of economics and will depend on the adequacy of sampling and testing and the improvement of analyses through correlation with observations during and after construction. It is believed, however, that the analysis presented is a reasonable approach to the utilization of soils investigations in the problems of the design engineer.

Field Exploration

The investigation of soil foundations for bridges and embankments may be divided into three parts, as follows:

1. Field exploration and sampling.
2. Laboratory testing of samples.
3. Analysis of field and laboratory data.

The first step in the foundation investigation of a given site is a gathering and digest of available data on subsurface conditions and the behavior of other structures in the vicinity of the proposed project. Valuable data on subsurface conditions are often obtained from maps, aerial photographs, and publications of Federal and State geological surveys. A preliminary geological survey based on outcroppings and other surface indications is generally made on large projects. From such a survey and the results of reconnaissance explorations, an engineer-geologist will often be able to sketch the general geological structure of the site, estimate the character and depth of the soils and indicate the location of possible buried channels, slide areas, faults and other subsurface irregularities. For example, at the Indian Head Road crossing of Piscataway Creek, Maryland, stiff clay was found by borings within a few feet of the 55 foot depth predicted from geologic maps.

Geophysical methods are used on some projects since they are rapid and relatively cheap and facilitate detection of subsurface irregularities which may be missed by individual borings. This method was used to advantage at the Washington National Airport to plot variations in depth of granular fill over soft silt.

Auger borings and wash borings are used most commonly to establish the soil profile and to obtain undisturbed samples for laboratory testing. Such borings are made at the locations indicated as critical in the preliminary survey, or at regularly spaced intervals, if especially critical areas do not exist.

In auger borings the hole is advanced by rotating a soil auger while pressing it into the soil and later withdrawing and emptying the soil-laden auger. By this method the soil profile and depths at which undisturbed samples should be taken can be determined with greater accuracy than with other methods. Since the bore hole is made without the addition of water, auger borings are particularly well-suited for advancing borings in materials above the ground water level, especially when undisturbed samples are to be obtained of these materials. The determination of the free ground-water level is also facilitated by auger borings. Due to these advantages and the development of light compact, motorized drilling rigs, power-operated augers are being used extensively in foundation explorations.

A wash boring is advanced partly by a chopping and twisting action of a light bit and partly by jetting with water which is pumped through the hollow drill rod and bit, Fig. 1. Cuttings are removed from the hole by circulating water. The drill rod and bit are moved up and down by pulling and slackening the rope and are at the same time rotated back and forth by means of the tiller. The water is pumped from a small sump or tub and the soil-laden water from the bore is discharged into the same reservoir where the coarse material settles. Changes in the character of the soil are determined partly by the feel of the tiller or resistance to penetration and rotation of the bit, and partly by examination of the cuttings in the wash water as it emerged from the casing. When a change in material is indicated, a split tube with heavy walls is driven into the bottom of the hole and pulled out, thus procuring a sample of soil with its natural composition. The number of blows of a drop-hammer of specified size required to drive the tube into the soil provides information as to the stiffness of fine-grained soils or the compactness of sand; see Fig. 1. In gravelly material, these penetration values are apt to be erratic and unreliable. The samples which are preserved in jars (Fig. 2) with tight tops provide a means of visual inspection of the foundation material. The characteristics of the disturbed samples and the resistance to penetration obtained from borings carefully made to adequate depths and covering the lateral extent of the proposed construction are sufficient for design purposes in many cases. Such data was used to check the founding elevations for the main supports of the Chesapeake Bay Bridge at Sandy Point, Maryland.

The following founding elevations were indicated

Pier Number	23	24	25	26	27	28
Negative Elevation	91	95	100	116	124	130

These elevations are well below the upper part of the sand, which is not consistently firm. A chart for estimating allowable soil pressures (for one inch settlement) on sand from penetration resistances is presented by Terzaghi and Peck in Soil Mechanics in Engineering Properties, p. 423. For example, for pier 26, the lowest penetration resistance, N, is 37. For an adjusted N of 15 plus 1/2 (37-15) equals 26 to allow for low permeability this chart gives an allowable pressure of 2.4 which multiplied by 2/3 to allow for submergence gives 1.6 tons per square foot as the allowable net pressure (gross pressure minus pressure of material displaced). The pressure of material displaced, assuming no water in the pier is 54x63 plus 10x77 plus 17x85 plus 35x125 equals 10,000 pounds per square foot or 5 tons per square foot. The gross allowable pressure is then 1.6 plus 5.0 equals 6.6 tons per square foot. Similar calculations give results as follows:

Pier Number	23	24	25	26	27	28
Minimum Penetration	55	55	55	37	47	47
Net Bearing, T/sq.ft	2.1	2.1	2.1	1.6	1.8	1.8
Displacement, T/sq.ft.	3.9	4.1	4.3	5.0	5.5	5.3
Gross bearing, T/sq.ft.	6.0	6.2	6.4	6.6	7.3	7.1

In conclusion, the elevations indicated are satisfactory for a gross pressure of 6 tons per square foot.

During boring operations undisturbed samples which preserve the structure and density of the material as well as its composition are taken from fine-grained plastic soils for the purpose of making laboratory tests to determine consolidation characteristics, shearing, and compressive strengths. Various types of samplers, as shown diagrammatically in Fig. 3, are used for this purpose, but the practice of taking samples 2 inches in diameter with thin-walled steel tubing (Shelby tubing) has become widespread during the past few years. Most authorities agree, however, that samples 3 to 5 inches in diameter, taken with the more elaborate samplers, are less disturbed by driving and, therefore, are more representative of the strata sampled.

The possibility of getting better samples with piston-type samplers is shown by the consolidation test results in Fig. 4, taken from records of the tunnel line of the Chesapeake Bay Crossing at Sandy Point. The sharp change in compressibility at the pre-consolidation load is evident for the piston sampler but was lost in samples taken in open tubing.

The proposed fill and abutment, shown in Fig. 5, will be used to illustrate the analysis of field and laboratory data in connection with the solution of various design problems.

The exploration by auger and wash borings disclosed the conditions for the site shown in Fig. 3 and thin-walled steel tubing was used to procure continuous samples through the silt in order not to miss any exceptionally weak strata. Samples were taken from four borings to determine the horizontal variations of the soil properties. Figure 6 shows a plat of the variations with depth of the properties of the soil from the typical boring. Such a plot aids in deriving average values and in selecting samples for further tests. For simplicity of illustration, the test results on individual samples, which represent average properties of the foundation materials and the fill material sampled from an adjacent cut will be considered hereafter.

Fill Compaction

The report of the classification test results is shown in table 1. The compaction test results for the clay fill material are shown in table 1 and plotted in Fig. 7 where they are compared with the zero air voids relation and the locus of optimum moisture contents for maximum density for average soils. It would be practicable to specify that the soil be compacted in the field to a dry density (weight of dry material for unit volume of wet soil) of 102 pounds per cubic foot at its natural moisture content of 22 percent of the dry weight of soil shown in Fig. 7. If the average density in the cut is 105 pounds per cubic foot, a volume increase of $\frac{105-102}{102} = .03$ from cut to fill is to be expected. A volume decrease is more often encountered, especially for shallow cuts. Figure 8 is useful in relating various measurements of moisture, voids and density.

The compaction of the slopes of the fill and the material behind the abutment is often difficult with rollers used for the main body of the fill. Due to the rigidity of the abutment, adjacent compaction is particularly necessary to control differential settlement. This and the similar problem of trench backfill are illustrated in Fig. 9. Use of selected fill material and special tampers (or vibrators for granular fill) have been used to advantage.

Fill Strength

The stability of the fill depends upon the strength of the soil in its weakest condition. This condition would occur during a wet season in which the soil would absorb water to its capacity under the restraint of the surrounding soil.

To determine this strength, disks of soil at 22 percent moisture were molded to 102 pounds per cubic foot between rough porous plates, loaded normally and inundated until the volume became constant, and then sheared by displacing one plate parallel to the other. See Appendix A.

A report of the direct shear test results on three soil disks under different normal loads is shown in Table 1. The difference between the initial thickness and the thickness before shearing indicates that the compacted soil will expand even against a pressure of 2 kips per square foot. The relation between the normal stress, n, and the maximum shear stress, s, is plotted in Fig. 10 and shows a curvilinear relation. A straight line is drawn in this figure to approximate the curvilinear relation for the range of pressures occurring in the clay fill shown in Fig. 5. This range of pressure is from zero to a maximum which is approximately the height times wet density of the fill = 20 (102 x 1.22) = 20 x 124 = 2,480 pounds per square foot = 2.5 kips per square foot. The two parameters which define this line are the intercept at n = 0 called the cohesion, c, and its slope, which is called the coefficient of friction, f, (the angle of internal friction is arc tan f). For a different range of pressures, different values of c and f would be obtained.

Critical Slope

Figure 11 may be used to calculate critical slopes of terraces of homogeneous material, which may fail by sliding along cylindrical surfaces. For the fill height and slope shown in Fig. 5 and a factor of safety of F = 2 applied to the shear strength from Fig. 10, enter Fig. 11 at a slope ratio of 2, go up to $\frac{f}{F} = \frac{.8}{2} = 0.4$ and find $\frac{c}{FwH} = 0.011$. Solving for c ~~gives a~~

gives a required value of .011 x 2 x 124 x 20 = 55 lb. per square foot or .06 kips per square foot. Assuming no cracks developed in the fill, the available cohesion is 0.7 kips per square foot, which is more than adequate. While the strength of the fill would permit a steeper slope, this may be precluded by consideration of safety, erosion, or landscaping. If the available strength were not so high, the effect of the cohesion being destroyed over part of the arc of sliding due to shrinkage during a dry season would have to be considered. The density of the soil after drying at 110 C (see Table 1) is 120 pounds per cubic foot as compared to the density of 102 pounds per cubic foot, at which the fill is to be placed. This difference indicates a possible volume decrease of $\frac{120-102}{120} = 0.15$, which would probably cause shrinkage cracks whose depth would depend on the climate and local moisture conditions.

Figure 11 may also be used to estimate the stability of cut slopes. Clay materials with negligible effective internal friction ($f = 0$) are the most troublesome. As shown in Fig. 11, for slopes flatter than 3/4:1 the most critical arc passes below the toe of the slope andthe value of $\frac{c}{FwH}$ depends upon the depth to a firm stratum or the width of a thru cut, whichever gives the lower value. In clay which is fissured from drying or previous overstress, the limited strength between the blocks must be used rather than the high strength of the unbroken pieces. Consideration must also be given to gradual loss of strength due to swelling after unloading which may cause sliding several years after the construction of a cut as shown in Fig. 12. For example, cut slopes along the Natchez Trace Parkway were composed of stiff fissured grey clay.

Figure 13 is a plot of the maximum shear stresses for different normal stresses. Because of the limited size of the shear testing apparatus, the shear plane is forced to pass thru some of the fragments. Neglecting the seepage forces and assuming this sample to be representative of a homogeneous mass, critical slopes may be derived from the figure. If the shear plane passes thru the fragments of soil, there is some effective cohesion and the critical slope depends on the normal stress as indicated by lines 1 and 2. Thus, for an average normal pressure of 4 kips per square foot, the critical slope as indicated by line 2 is $\frac{4}{2.8} = 1.4:1$. However, a large mass may fail by shearing between the fragments where there is no effective cohesion, in which case, the critical slope is given by line 3 as $\frac{16}{6.2} = 2.6:1$ for no lateral flow of water. For horizontal seepage, this slope must be multiplied by the ratio of the wet soil density (102) to its buoyed density (102-62) giving $\frac{102}{102-62} \times 2.6 = 6.6:1$ as the critical slope for this extreme condition.

Such weakening may be controlled by surface or subsurface drainage (see Fig. 14). Special equipment has been developed for drilling almost horizontal holes into the base of a slope for drainage installation.

As a further example, tests on undisturbed samples from station 445 on the Indian Head Road near Piscataway, Maryland, showed a minimum coefficient of friction of $\frac{1}{2.3}$ indicating that a saturated cut slope with no seepage would be just in equilibrium with slopes of 2.3:1. A 3:1 slope would give a safety factor of 1.3. However, field examination revealed ample evidence of horizontal seepage of water toward the cut face. For this condition, the slope required is $3 \times \frac{125}{125-63} = 6:1$. The advantage of drainage is apparent. A slide in a similar clay which threatened the foundations of Young School in Northeast Washington, D. C., was controlled by reducing the slope to 6:1.

Strength of Foundation Material

The foregoing analysis presumes that the foundation is not weaker than the fill but this remains to be determined. The results of triaxial compression tests (Appendix B) on the foundation materials are reported in Table 1. The triaxial test is an ordinary compression test plus provisions for applying a lateral pressure to the sample through an impervious membrane.

In order to simulate the field conditions, the tests are made quickly to minimize consolidation and the resulting increase in strength. The relation between stress difference, d, (vertical minus lateral pressure) and reduction in height for different lateral pressures, l, are shown in Fig. 15, which will be considered later. The relation between critical shear stress and normal stress is determined in Fig. 16 as the envelope of stress circles whose radii are half the maximum stress difference, from Table 1.

The sand has no cohesion so that the shear strength is directly proportional to the normal stress. The proportionality factor, f, which is the coefficient of friction, may be calculated without the construction shown in Fig. 16 by calculating a function $K_p = 1 + \frac{d}{l}$ from the test results and finding the corresponding value of f, in Table 2. Thus, from Fig. 16, for $l = 2$, $K_p = 1 + \frac{5.0}{2} = 3.5$, and for $l = 4$, $K_p = 1 + \frac{9.6}{4} = 3.4$. Using the average value of $K_p = 3.45$ in Table 2, gives $f = 0.66$.

As is indicated in the lower portion of Fig. 16, the coefficient of friction for the silt is negligible since the low permeability of the silt restricts its drainage and prevents the applied normal stress from appreciably increasing the strength by friction between the soil grains. Therefore, the shear strength of the silt is taken as a constant equal to half the average stress difference. Unconfined compression tests are sufficient for this type of material unless appreciable consolidation is anticipated before the foundation is fully loaded. While the construction of stress circles is not necessary in the present example, they would be used for more permeable cohesive materials and have been shown for the purpose of illustration.

Slide Through Foundation

Since several layers are involved in the problem indicated in Fig. 5, the possibility of rotation along an arc through the foundation with an assumed center and radius, as indicated in Fig. 17, will be analyzed. Such analysis is based on a balance between the moment tending to cause rotation due to the soil weight and the moment of the resisting forces due to the Cohesion and friction in the soil.

The moment tending to cause rotation is due to the fill alone since the foundation material balances itself. The weight of a vertical slice of the soil of unit thickness is calculated as the density of the material times the cross sectional area. The rotating moment is the weight of the slice times the horizontal distance from the center of gravity of the area to the center of rotation. The rotating moment of the fill within the arc of sliding may be calculated as the moment of the entire fill minus the moment of the approximately triangular section beyond the radius of the arc of sliding. The section is drawn to scale (preferably on cross section paper) so that required distances and areas may be determined graphically. For the assumed center of rotation, O, the moment of the entire fill in Fig. 17 is the weight 124 x 20 x 70 = 174 kips times the horizontal moment arm = 35 feet. As shown in Table 3, the moment of the fill outside the arc is subtracted to get the rotating moment, 5010 kip-feet.

The resisting moment is that which would result if the maximum friction and cohesion of the materials were developed. The component of W, the weight of any slice normal to the arc of sliding (as at point P for area KEF in Fig. 17) is $W\frac{V}{R}$ which multiplied by f gives the frictional resisting force, $\frac{fWV}{R}$. Multiplying by R gives the corresponding resisting moment, fWV.

In the computations shown in Table 3, the resistance on arc BC due to the weight of the fill and of the sand are computed separately.

The effective density of the sand is based on its dry density from figure 13 or $\frac{120}{1.21}$ = 99 pounds per cubic foot. With this density and zero air voids, Fig. 7 shows a moisture content of 26 percent giving a total wet density for the 5 feet of sand below the water table of 99 x 1.26 = 125 pounds per cubic foot or a buoyed density of 125-63 = 62 pounds per cubic foot. Assuming an average moisture content of 14 percent in the 15 feet of sand above the water table (see Fig. 6) gives a wet density of 99 x 1.14 = 113 pounds per cubic foot. The average density for the entire 20 feet of sand is then $\frac{113 \times 15 + 62 \times 5}{20}$ = 100 pounds per cubic foot. Altho not needed in the present analysis, the buoyed density of the silt is simply its wet density minus the density of water or 101-63 = 38 pounds per cubic foot.

The length of arc, L, required in computing the cohesive resistance, cL, is scaled directly or calculated from the central angle and the radius. The cohesive resisting moment is cLR. The total resisting moment divided by the rotating moment gives 1.48 as the factor of safety with respect to soil strength against rotation about point O. If cracks destroyed the cohesion on arc AB, the factor of safety would be reduced by $\frac{540}{5010}$ = 0.11 to 1.37.

Since point O was assumed and may not determine the most critical arc, the foregoing computation was repeated for other assumed centers of rotation. The resulting safety factors at the assumed centers are shown in Fig. 17. The minimum value, 1.42, is not high but may be considered as satisfactory.

A problem similar to the above was involved in sand island design for the Chesapeake Bay Bridge, as follows: Consider a sand fill at Station 217, 100' x 400' on top at Elevation +10 with 2:1 slopes down to El. -68 resting on the soft grey silt, about 14' of soft black silt having been removed or displaced.

For the sand fill, assume an angle of internal friction ϕ equals 35 degrees and a wet density of 123 pounds per cubic foot. For the silt, tests from boring 17T were used since they show the least disturbance and are consistent with a normally consolidated material as indicated by the geologic history. This gives somewhat greater compressibility and considerably more strength than indicated by other borings, such as 22D. The shear strength for samples in boring 17T varies from .29 to .39 times the buoyed overburden pressure, giving a minimum strength of .29 times the intergranular pressure.

For stability, Fig. 11 indicates that for H equals 78, D equals 42 (depth of silt), w H equals 123 x 10 plus 68 (123-63) equals 5310, F equals 1 and a slope ratio of 2; the average shearing strength, c, required is 0.154 x 5310 equals 820 pounds per square foot. For vertical shear in the sand fill, the average shearing resistance as derived in Fig. 18 is

$\frac{f}{1 + 2 f^2} \frac{wH}{2}$ = .35 x 1/2 x 5310 = 930 pounds per square foot.

Assuming 3/4 of the shear surface in the silt, it must have an average shear strength of 820 - 1/4 (930-820) = 790. The average pressure in the silt without the fill is 14 x (77-63) + 21 (85-63) = 660 pounds per square foot, giving a shearing strength of .29 x 660 = 191 pounds per square foot which is not nearly enough if the fill is placed quickly. If the load were applied slowly enough so that the intergranular pressure could increase, stability could be maintained. Assuming the average consolidating pressure over the sliding surface to be half wH, a degree of consolidation under the fill load of at least 0.78 is required. (.78 x .29 x 1/2 x 5310 plus 191 = 790, the required strength.)

Tendency for rotation can be reduced by reducing the depth of fill or by a surcharge to balance part of the rotating moment, as shown in Fig. 19. Slides need not always be circular as shown in Fig. ~~19~~ 20 when natural weak planes exist. These conditions are sometimes difficult to predict even with continuous cores.

While slopes are often designed with a low safety factor on the basis that repairs are easy, this is not the case when they are adjacent to structures as illustrated by a failure in a bridge over the Berlin Reservoir near Akron, Ohio.

The bridge is a continuous reinforced concrete structure, 408 feet long, supported by three reinforced concrete piers founded on rock and abutments supported by cast-in-place concrete piles. The height of the tallest pier above the foundation is 68 feet. The east approach, where the failure occurred, is a compacted silty clay fill with 2:1 side slopes and projects about 10 feet above the crest of the dam which controls the water in the reservoir. The piles for the east abutment of the bridge were driven through the fill which is 61 feet high at this point. The fill under the bridge was constructed with a $2\frac{1}{2}$:1 slope parallel to the center line and surrounded the lower half of the east pier. For widening, duplicate piers and abutments were constructed north of and adjoining the present bridge. The bridge and fill were constructed in the winter of 1943 as the reservoir was being filled so that the partially constructed fill was subjected to flooding.

At the time of inspection, the reservoir had been drained and the fill adjacent to the south side of the east abutment had failed by sliding. See Fig. 21. The abutment had been forced against the deck which, in turn, had moved $2\frac{1}{2}$ inches west. The east pier of the bridge had been displaced sufficiently to cause a construction joint at about midheight to open on the west face. The instability of the northwest corner of the fill was indicated by the westward tipping of the unbridged east pier. Based on shear tests on core samples taken from the fill after the failure, the State had calculated a factor of safety of the 2:1 fill slope of 1.05. They assumed a circular arc of sliding and used an average cohesion of 438.0 pounds per square foot and an average angle of internal friction of 27.8 degrees.

The bridge engineers favored placing a strut between the east and center piers to support the east pier. Since removal of fill under the bridge is not feasible without supporting the bridge independently, it was proposed to add rockfill to the existing slope.

The writer stated that the 3:1 rockfill slope which the State proposed to add around the fill should hold the unfailed slopes but that in the slide area at least a 4:1 rock slope should be placed unless the failed area is trimmed back to a 2:1 slope before the stone is placed. This opinion was based on an analysis of the forces required to prevent a failure along a circular arc of minimum factor of safety. The additional rockfill acts somewhat like a gravity retaining wall so that the support it offers depends upon its thickness. In order to get sufficient thickness to give a substantial increase in stability, a rockfill with 4:1 slope is recommended. The variation in thickness of the rock layer in the failed area for 3:1 and 4:1 slopes is shown in Fig. 21.

Bearing Capacity Under Abutment

The stability of the abutment, with the assumed dimensions shown in Fig. 5 is considered next. The total load due to the live load and the weight of the abutment per unit thickness is $6 + 31.8 = 37.8$ kips as shown in Fig. 22, and produces an average bearing pressure of $\frac{37.8}{12x1} = 3.15$ kips per square foot.

To calculate the stability of the abutment without the fill load, as in the case of a pier, the factors and nomenclature given in Table 4 are used. Thus $L = 30$ feet, $B = 12$ feet, $D = 6$ feet, $w = 100$ pounds per cubic foot, and $f = 0.66$. Since $c = 0$ for the sand, F_c is not used. Using a factor of safety of 2, enter Table 4 with $f = \frac{0.66}{2} = 0.33$ to find $F_D = 6.3$ and $F_B = 1.4$. Substitution of these values in the formula shown in Table 4 gives $q = 100 \times 6 \times 6.3 + (1 - .2\frac{12}{30})\ 100 \times 12 \times 1.4 = 3780 + .92 \times 1680 =$ 5330 pounds per square foot or 5.3 kips per square foot, which is more than required by the bearing pressure of 3.15 kips per square foot. A qualitative pictorial representation of Table 4 is given in Fig. 23.

Since the sand is underlain by soft silt, the possibility of a rotation along an arc passing through the silt could be analyzed as was done for the fill in Fig. 17. A rough analysis of such a possibility may be made by comparing the bearing capacity of the silt computed from Table 4 with the abutment pressure reduced by the pressure required to cause the abutment to punch thru the sand. This latter pressure is the shear strength in the sand multiplied by the ratio of the area of shear to the bearing area.

The shear strength of the sand is the coefficient of friction times the average lateral pressure which is uncertain. However, as a probable minimum, the shear strength may be taken as one-third the average vertical pressure, which is the average depth of the shear surface below the original ground times the density of material above the surface. From Fig. 22, with a factor of safety of 1, this shear strength is $1/3(6 + \frac{14}{2})100 = 433$ pounds per square foot or 0.43 kips per square foot. The ratio of areas is the perimeter-area ratio of the abutment base times the thickness of the layer or $\frac{2(30 + 12)}{12 \times 30} 14 = 3.27$. The pressure which the silt must support (in addition to 14 feet of sand) is then $3.15 - .43 \times 3.27 = 3.16 - 1.41 = 1.74$ kips per square foot. The bearing capacity of the silt, for a factor of safety of 2 applied to $c = .65$ and $f = 0$, is $q = (1 + .3\frac{12}{30}) \frac{.65}{2} \times 5.7 + \frac{100 \times 6}{1000} \times 1 = 2.07 + 0.6 = 2.7$ kips per square foot, which is more than required if, as assumed in Table 4, the silt is confined and cannot escape laterally. Thus the foundation has adequate bearing capacity to support the abutment.

Pressures on Abutment

If the clay fill of height H is placed against the abutment, the pressure which it exerts will depend on its moisture content. The minimum or active force, P_a, on the wall may be calculated by means of Table 2 and the formula

$$P_a = \frac{wH^2}{2} K_a - 2cH\sqrt{K_a} \quad \ldots\ldots\ldots\ldots\ldots\ldots(1)$$

assuming adhesion between the wall and the fill.

This condition cannot be relied upon and cracks from shrinkage may reduce the effective cohesion and increase this force. If cracks become filled with debris and the clay then absorbs water, it can exert pressures in excess of 2 kips per square foot, as shown by the swelling from the compacted stated reported in Table 1. The pressure in this case would be limited by the maximum or passive force which is roughly

$$P_p = \frac{wH^2}{2} K_p + 2cH\sqrt{K_p} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (2)$$

Because of this swelling and the difficulty of adequate compaction close to the wall, it would be well to borrow from the sand layer to make the fill behind the wall for a distance H times $\sqrt{K_p}$ for the sand from Table 2. Active and passive pressures for dry sand are shown in Fig. 24 while effects of submergence and drainage are shown in Figs. 25 and 26. Pressures for variable wall slopes and surcharges are given in Tables 5 and 6. Assuming that the sand as placed has a wet density of 120 pounds per cubic foot and is adequately drained, the horizontal force on the back of the abutment is calculated from equation 1. Adding one foot to the height of 26 feet to allow for live load and taking K_a from Table 2 for $f = 0.66$, formula 1 gives $\frac{120 \times 27^2}{2}$ .290 = 12, 685 pounds, or 12.7 kips. Since the abutment can move laterally, the pressure increases uniformly with depth and the point of application of the force is $\frac{H}{3} = \frac{27}{3} = 9.0$ feet above the base. Because of the friction between the wall and the sand, there is also a downward force which adds to the stability of the wall but which may be neglected for moderate heights.

As the abutment is pushed against the sand in front of it, a passive resistance is developed with a maximum horizontal component from equation 2 equal to $\frac{120 \times 6^2}{2}$ 3.45 = 7,452 pounds = 7.5 kips. If the base of the abutment is rough enough, it can develop a maximum frictional resistance to sliding of f times the vertical load or .66 x 37.8 = 24.9 kips.

The ratio of the resisting horizontal forces to the active force gives the factor of safety against sliding which is $\frac{7.5 + 24.9}{12.7} = 2.55$.

The increase, p_2, of the pressure at the toe over the average pressure, $p_1 = 3.15$ kips per square foot on the base of the abutment is computed from the moment per unit length, M_o, of the forces about the center of the base.

Thus $$p_2 = \frac{6M_o}{B^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots \quad (3)$$

From Fig. 22, $M_o = 9 \times 12.7 - 2 \times 6 - 1.5 \times 31.8 - 2 \times 7.5 = 39.6$ kip feet per foot and $p_2 = \frac{6 \times 39.6}{(12)^2} = 1.65$ kips per square foot.

The total pressure at the toe becomes 3.15 + 1.65 = 4.8 kips per square foot, which may be compared with the pressure required to cause failure at this point as computed from Table 4 for B = 0. Thus for c = 0 and f with a factor of safety of $2 = \frac{.66}{2} = .33$, $q = wDF_D = \frac{120 \times 6 \times 6.3}{1000} = 4.5$ kips per square foot, which is slightly less than the calculated pressure of 4.8. This may be permissible since the failure at a point rather than total failure is being considered.

Settlement from Shear Stresses

Altho the foregoing calculations indicate that the fill and abutment are safe against a shear failure in the soil, some displacement is inevitable because any change in the load on the soil causes strains therein. These strains are of two kinds—distortion or change in shape associated with normal stresses. While both strains may occur simultaneously, they can be computed separately using Poisson's ratio = 0.5 for distortion and Poisson's ratio = 0 for consolidation.

An important factor is illustrated in Fig. 27 which shows that while the intensity of stress reduces with depth as the influence of a surface load spreads, the total load at any depth is not decreased. This is applied in Fig. 28 which shows how a small surface load doesn't load a deep strata like a large load would even though the applied load per unit area is the same. It is also noted that a small settlement that would be overlooked in a loading test is important if it continues over a long period.

Figure 29 gives the settlement factors for Poisson's ratio = 0.5 under the corner of a loaded rectangular area at the surface of a mass with a constant ratio of strain to stress difference or shape modulus, m_s. The stress-strain relation as determined by the triaxial test is shown in figure 15. The modulus is the slope of the stress-strain curve.

Since the modulus for the sand is roughly proportional to the lateral pressure and is not constant, figure 29 cannot be used unless an effective average modulus is determined. Observations during full-scale construction correlated with bearing or penetration tests on soils in place can be used in determining an effective modulus and estimating displacements.

For stress differences in the silt well below the maximum, (see figure 15) the modulus is practically independent of the lateral pressure and can be taken as the slope of the secant thru points on the average curve representing the average initial and final stress differences. The stress difference in the silt before construction depends upon the geologic history and in the absence of other information is assumed to be zero.

The displacement in the silt layer below the corner of the toe of the abutment (shown in Fig. 5) due to the pressures shown in Fig. 22, may be calculated from the difference between the settlement factors at the top and bottom of the layer.

The pressure due to the excavated soil, $\frac{120 \times 6}{1000} = .72$ kips per square foot, is subtracted from the abutment pressure, 3.15 kips per square foot, to give an average increase in pressure, $p_1 = 3.15 - .72 = 2.43$ kips per square foot. The maximum difference from the average pressure remains, $p_2 = 1.65$ kips per square foot and its effect is added to that of p in the following computations using Fig. 29:

$\frac{z}{B}$ at bottom of silt $= \frac{44}{12} = 3.67$

$\frac{z}{B}$ at top of silt $= \frac{14}{12} = 1.17$

$\frac{L}{B} = \frac{30}{12} = 2.5$

F_1 at bottom $= 0.338$

F_1 at top $= \underline{0.128}$

Difference $= F_1 = 0.210$

F_2 at bottom $= 0.210$

F_2 at top $= \underline{0.089}$

Difference $= F_2 = 0.030$

$pF = p_1F_1 + p_2F_2 = 2.43 \times .210 + 1.65 \times .030 = 0.56$

Using the difference in z/B at the top and bottom of the silt, average stress difference, $d = \frac{pF}{z/B} = \frac{0.56}{3.67 - 1.17} = 0.224$ kips per square foot.

From Fig. 15, the slope of the secant from d = 0 to d = .224 is the strain per unit stress, $m_s = 0.023$.

Settlement, $S = m_spFB = .023 \times .36 \times 12 = 0.15$ feet.

The stress transmitted to the silt from the fill would cause additional settlement. However, unless the stresses approach the maximum strength, the settlement due to shear stresses occurs almost as fast as the load is applied and could occur unnoticed in the present case. It is important in the case of rigid structures and high live loads or when additions are made to existing structures. If the calculated settlement is excessive, the degree of disturbance of the sample should be considered since the stress-strain characteristics of some soils are very sensitive to even slight distortion.

Figure 29 is actually for a load at the surface, whereas the footing is below the surface. The effect of placing a load below the surface, as in Fig. 30, is indicated in Fig. 31, but for shallow footings this effect is often neglected. The relation between the stresses based on the theory of elasticity as used above and the building code device of assuming stress spread thru a cone with angles 60 degrees to the horizontal is shown in Fig. 32.

Settlement From Volume Change

The settlement due to volume change of the undersoil is computed by means of a settlement factor for Poisson's ratio = 0, taken from Fig. 33 and corresponding to given loads and dimensions and the soil characteristics from consolidation test results (Appendix C), as shown in Table 1.

Using the nomenclature of Fig. 33 and the dimensions in Fig. 17, the settlement under the center of the fill far from the abutment resulting from consolidation of the silt due to the weight of the fill is computed as follows:

$p_3 = 124 \times 20 = 2.5$ kips per sq. ft.

$a = 40$, $B = 55$, $\frac{L}{B} = \infty$

$z = 50$ and 20

$\frac{z}{B} = .910$ and $.364$

From Fig. 33, $F_1 = .222 - .096 = .126$

$F_2 = .120 - .071 = .049$

$F = F_1 + \frac{a}{B} F_2 = .126 + \frac{40}{55} .049 = .162$ for 1/4 of fill

$F = 4 \times .162 = .648$ for entire fill

The uniform pressure used to represent the trapezoidal distribution of the fill weight is $p = p_3(1 - \frac{a}{2B}) = 2.5(1 - \frac{40}{2x55}) = 1.59$ kips per square foot.

The average pressure increase in the silt due to the fill equals

$\frac{pF}{z/B} = \frac{1.59 \text{ x. } .648}{.910 - .364} = 1.9$ kips per square foot.

The average initial pressure in the silt layer (before the fill is constructed) = the pressure at the middle of the layer = 100 x 20 + 38 x 15 = 2570 pounds per square foot = 2.6 kips per square foot.

Total or final pressure = 2.6 kips per square foot.

From the data in Table 1, plotted in Fig. 34, the slope of the secant between the initial and final pressures is the strain per unit pressure increase = $m_v = .011$.

Then, $S = m_v pBF = .011 \times 1.59 \times 55 \times .648 = 0.62$ feet, is the ultimate settlement which may occur very slowly.

Time - Settlement

To calculate the settlement at various times, time-consolidation test data are plotted as in Fig. 35 (solid lines). The initial linear portion of the test relation is extended in both directions. If this straight line has a positive intercept at zero time, the intercept indicates consolidation which occurs as the load is applied. Through this intercept, a second line is drawn with abscissas 0.15 greater than the first line.

The intersection of this second line with the test relation curve determines a theoretical time, t_{90}. This is the time required for 90 percent of the consolidation to occur if all the time lag were due to resistance to the escape of water. For the thin layer used in the laboratory test, the time required for readjustment of the soil particles is appreciable (secondary consolidation), but it is generally assumed that this becomes relatively unimportant for thick layers. For highly organic soils the secondary consolidation must be considered.

Using the value of t_{90} = 11.7 minutes, from Fig. 35, with the initial thickness, H, of the sample (0.5 inches), the coefficient of consolidation, c_v, is computed as $c_v = \frac{.21\ H^2}{t_{90}} = \frac{21 \times (0.5)^2}{11.7} = .0045\ \text{in.}^2$ per minute = 0.045 ft.^{2} per day. The time, t, for consolidation of the layer 30 feet thick is $t = \frac{H^2}{c_v} T = \frac{(30)^2}{.045} T = 20{,}000\ T$ days where T is the time factor from Table 7.

If one face of the consolidating layer is impervious, as in the present case, the value of T depends on the initial pressure gradient which is approximately the ratio between the increase in pressure at the pervious face to the pressure at the impervious face. This ratio may be determined graphically, as indicated in Fig. 36, in which a portion of the curve from Fig. 33, showing the relation of F and $\frac{z}{B}$ ~~and~~ for $\frac{L}{B} = \infty$

has been reproduced to a larger scale. The pressure at any point is $p\frac{dF}{d(z/B)}$ which is the slope of a tangent to the curve at that point. Since only the ratio of the pressure at the top (z/B = .364) and bottom (z/B = .910) of the silt layer is required, lines 1 and 2, parallel to the tangents at points 1 and 2 at these depths in Fig. 36 are drawn through the original and the desired ratio determined as the ratio of the abscissas. This is

$\frac{x_1}{x_2} = 1.25$. For this value of the pressure ratio, the time factor from Table 7 for a degree of consolidation of 0.5 is by interpolation 0.18 and the time required is t = 20,000 T = 20,000 x 0.18 = 3600 days or approximately 10 years for a consolidation of 0.5 x 0.62 = 0.31 feet.

A complete time-consolidation relation may be calculated by using values of T from Table $\overset{7}{\&}$ for other degrees of consolidation. The load has been assumed to be applied instantaneously at zero time. If the load is applied at a uniform rate, zero time is taken as the middle of the construction period and the settlement curve during the construction period adjusted for the portion of the total load applied at a given time.

At the back of the abutment, the settlement due to the fill would be half that computed above but the settlement due to pressures transmitted from the abutment must be added. In calculating this additional settlement, the procedure outlined above is followed but it is not necessary to recompute m_v and the pressure ratio.

As indicated in Fig. 34, there would also be some volume change in the sand layer. However, the resulting settlement would be small and, as indicated in Fig. 35, it would occur almost as fast as the load is applied so that it would not affect a pavement placed on the completed fill.

If the coefficient of permeability as defined in Fig. 37 is determined, the coefficient of consolidation may be calculated as $c = \frac{k}{m_v w}$ where w is the density of water. The permeability test is shown in Fig. 38 To evaluate fissures or stratification the permeability should be determined in the field as shown in Fig. 39. Such a procedure was used to advantage in connection with the tunnel under the Elizabeth River at Norfolk, Virginia.

A 20-inch diameter hole punched 31 feet into the silt showed rather uniform seepage when the casing was pulled. The following measurements were recorded:

Time after pulling casing	Depth of hole	Water above bottom
hours	feet	feet
0	31	0
1	26	1
3	25	$2\frac{1}{2}$
20	9	4

For the initial flow into an uncased hole, the coefficient of permeability, $k = \frac{.1Q}{DH}$ where Q = quantity per unit time $= \pi \frac{D^2}{4} \times 1 \div \frac{1}{24}$

D = diameter $= \frac{20}{12}$

H = depth = 31 feet

This gives k = .1 feet per day compared to .0018 from laboratory tests. While the laboratory value was for vertical permeability and the field test is more lateral flow, the laboratory value was evidently too low, resulting in a gross overestimate of the time required for volume change to occur. Possibly some more permeable layers were missed in sampling.

Consolidation of several contiguous compressible layers is illustrated by the foundation of a 35 foot rolled fill of silty soil constructed over a tidal flat. See Fig. 40. Properties of these soils and others in this area are given in Table 8. While settlement was anticipated, it was decided to adjust the road at the bridges near the ends of the fill when necessary, rather than excavate the compressible soils in the foundation to eliminate settlement. See Fig. 41.

Using the data obtained from consolidation tests, an estimate of time-settlement relation was made before construction started. Since the permeability of the fill material was uncertain, calculations of settlement for the three layers were made on the basis of vertical drainage with both one and with two drainage faces. Table 9 indicates the method of calculation based on an average vertical permeability and average compressibility, using time factor, T, from Table 7. Thus for 50 percent consolidation with two drainage faces, T = .05 and

$$\text{time, } t = \frac{3830}{365} \times .05 = 0.52 \text{ years.}$$

Consideration of degree of consolidation in each layer would make considerable difference in results, but less difference than the uncertainty of boundary drainage.

Figure 41 shows a profile of the fill and the location of a settlement plate which was placed during construction. The settlement plate was composed of a steel plate 24 inches square to which was screwed a stem of one-inch pipe. The plate and first section of stem were placed 2 feet below the original ground surface and a two-inch guard pipe was placed around the stem. Additional sections of stem and guard were added as the height of the fill increased. After completion of the fill, the guard pipe was capped. Level readings referred to a permanent bench mark were taken on the stem at regular time intervals and the fill settlement calculated. The graph of the observed settlement over nine years is shown in Fig. 41.

A comparison of observed settlements with those calculated as outlined above indicates that the fill may have acted initially as a drain but that its resistance to flow of water from the foundation increased as it became saturated.

Calculations for two other locations on the same fill indicate ultimate settlements of 0.81 and3.62 feet, although the observed settlements were both approximately the same as shown in Fig. 41. This shows that the subsoil was more uniform with respect to support of a 35 foot fill than indicated by the three individual borings.

Level readings on temporary stakes and the pavement at the top of the fill showed the same settlement as the plate below the fill, indicating that there was no consolidation within the fill. A similar record of no movement within a rolled fill was previously reported in Public Roads based on readings of a liquid leveling device, one end of which was buried in the base of the fill as indicated in Fig. 42 along with other devices for making field measurements.

The time required for the consolidation of the silt layer shown in Fig. 5 can be decreased by constructing sand drains as shown in Fig. 43 right. The natural sand stratum (Fig. 5) will serve as the sand blanket between the relatively impervious fill and the compressible foundation. If sand drains 18 inches in diameter were constructed 10 feet center to center in a hexagonal array, the effective spacing, D, would be 10.5 feet. The ratio of drain or well diameter to effective spacing for the hexagonal array is $\frac{1.5}{10.5} = 0.14$ and interpolation in table 3 for 0.5 degree of consolidation gives $T = 0.11 = \frac{ct}{D^2}$ and $t = \frac{0.11D^2}{c} = \frac{0.11 \times 10.5^2}{.045} = 270$ days for 50 percent consolidation as compared to 10 years without the drains. If the silt were stratified, the horizontal permeability would be greater than the vertical and the time would be further reduced by the ratio of the vertical to the horizontal permeability.

Low permeability of homogeneous clay as at Game Creek on the New Jersey Turnpike may make sand drains not so satisfactory. For example, 13 feet of fill weighing 120 pounds per cubic foot superimposed on a 20 foot depth of the material represented by Sample 94-30L is considered, (c_v = .03 feet square per day, m_v = 5.5% per kips per square foot). A settlement of 5.3 feet would be obtained from consolidation including the weight of additional fill required to maintain the grade. For a factor of safety of 1.5 against failure by sliding, a shearing strength of 555 pounds per square foot is required in the silt. At its present moisture content its strength is only 120 pounds per square foot, but when consolidated under the weight of the fill its strength would be 1055 pounds per square foot; thus a consolidation of 52 per cent is required to maintain stability.

Without drains this would necessitate a four year construction period. This time could be reduced to one year by installing 18 inch drains 8 feet apart, but half the settlement would occur after the construction period. To eliminate post-construction settlement extra fill could be placed and later removed. For 11 months construction period practically complete consolidation could be obtained by placing 10 extra feet of fill and installing 18 inch drains 6 feet apart. The above calculations suggest that excavation and replacement of the silt may be cheaper than drainage.

Use of a surcharge was suggested at Morehead City, North Carolina where land was to be reclaimed for warehouse sites over 8 feet of silt. Thus, assuming 20 feet of fill at 120 pounds per cubic foot plus a building load of 400 pounds per square foot and using the maximum compressibility for an 8 foot layer of silt gives:

Settlement = compressibility x pressure x thickness

For 10' fill S = .09 x 1.2 x 96 = 10.4"

For building S = .04 x .4 x 96 = 1.5"

Using a coefficient of consolidation of 0.1 feet squared per day for an 8 foot layer with two drainage faces gives 134 days required for 90% consolidation or 90 days for 80% consolidation. Thus, no building settlement due to the silt could be achieved by maintaining a fill weighing 1.6 ÷ .8 equals 2 kips per square foot for 90 days which would give complete consolidation for a 1.6 kips per square foot load.

The above estimates are conservative since some of the material is definitely less compressible and consolidates more rapidly. In any event, observations in the field of progressive settlements on the fill should be made to indicate the time required and the settlement to be obtained.

Lateral Thrust

Figure 43 left shows another method of controlling settlement near abutments and of controlling the lateral thrust due to fill on soft soils as shown in Fig. 44 which also shows the erroneous assumption sometimes used as the basis of design.

In order to evaluate the lateral pressure transmitted by a surcharged soft silt, six Goldbeck pressure cells were installed on the west side of pier No. 2, bridge No. 6 of the Pentagon Road Network, Arlington, Virginia, early in 1946. The cells were attached to two 25-foot boards each spanning the inside of one section of the cofferdam piling which was used as a form for the concrete base of the pier below the silt line.

The piles were pulled in March 1946, allowing the soft silt to flow against the pressure cells. The original cross section is shown in Fig. 45 with the fill which was completed October 1946 to the west of the pier. The slope was riprapped and 25 inches of granular base placed on the fill in the spring of 1948. The readings of the pressure cells are shown in Table 10.

Core samples of the soft silt were taken in September 1948 with the following average test results:

Moisture content, % of dry weight	31
Wet density, pounds per cubic foot	119
Compressive strength, pounds per sq. foot	800
Consolidated shear strength -	
Cohesion, pounds per sq. ft.	400
Friction coefficient	0.35
Volume reduction for doubled load, %	1.6
Coefficient of consolidation, ft. sq. per day	0.2

The average net earth pressure in excess of full hydrostatic pressure was found to be

2.8 pounds per square inch at 8-foot depth
5.3 Do. 16 Do.
1.1 Do. 24 Do.

an overall average of 3 pounds per square inch.

Assuming a cohesionless fill with a coefficient of friction of .7 and a density of 130 pounds per dubic foot, an average net earth pressure with no time for consolidation was computed to be 10 pounds per square inch. Allowing 500 pounds per sq. ft., cohesion in the fill gives a computed pressure of 4 pounds per square inch. A maximum pressure is shown by the first reading taken (February 1947), after completion of the fill, indicating that time for consolidation reduces the pressure as would be expected from the test results.

Another example of lateral thrust is afforded by a bridge which was built in 1931 as part of the Memorial Highway to Mount Vernon over Boundary Channel, connecting the river bank to Columbia Island, newly formed by hydraulic fill. Figure 46 shows the deep layer of organic clay under the Boundary Channel Bridge and adjacent fill. The bridge, consisting of twin cantilevers with a small suspended span, was supported on piles to adequate bearing and did not settle. However, the bridge buckled due to the lateral pressure transmitted from the adjacent fill placed on the clay. Settlement of a bench mark set in the fill on 1 June 1934 is shown in Fig. 46, plotted against the square root of time, starting at the mean time of placement of the fill almost four years previous. A primary consolidation curve was fitted to this data and projected to zero time. A record of the fill settlement between 24 June 1932 and 1 June 1934 was subsequently found. This record, shown in Fig. 46, agreed with the fitted theoretical curve which indicates a total settlement before placing of the bench mark of 4.8 feet. This settlement had occurred due to consolidation in addition to any that occurred due to lateral displacement at the time of placing the fill.

The discrepancy between the fitted theoretical curve and the actual fill settlement after ten years is accountable to the 5 per cent additional load caused by approximately two feet of additional fill which was placed to keep the road reasonably smooth.

Table 11 shows the properties of the foundation clay based on samples from several bore holes which indicate some sandy strata, the continuity or extent of which was not determined. Considering a pressure of 2.7 kips per square foot on a 65 foot layer, the 7.0 foot settlement (4.8 plus 2.2) indicated in Fig. 4 would require a compressibility of 0.040 which compares well with the 0.043 average of the laboratory test results shown in Table 11. Assuming vertical flow, only the settlement record indicates a coefficient of consolidation (c) of 0.28 feet squared per day. This happens to agree with the average, c, shown in Table 11, but the weighted average as used in Table 9 for the Pentagon fill is only 0.10, suggesting that there was some lateral drainage which could not be evaluated from the data available before the recording of field settlements.

The similarity of the pier rotation and field settlement with time shown in Fig. 46 suggests that the lateral movement of the piers toward each other is controlled by the lateral consolidation of the clay between the pile groups. It may be noted that struts placed between the piers below water in August 1945 have shown no effect on the rotation of the piers.

If secondary consolidation, apparently independent of thickness, continued at the rate shown by the linear relation between thickness change and log of time between 1 and 24 hours in the laboratory, the secondary consolidation would be small compared to the primary consolidation as shown between 7 and 9 years in Fig. 41 and between 16 and 20 years in Fig. 46.

Pressures also occur laterally in a continuous fill as shwon in Fig. 47 with the problem and methods of displacing material under the center of the fill. Figure 48 shows method of preventing lateral thrust by a willow mattress while allowing slow settlement as practiced in Holland and the use of sand drains to accelerate settlement and accompanying strengthening as used in the United States. For example, to cross a bay of Lake Couer d'Alene in Idaho, a fill was proposed to be built to elevation 2143 and in water from elevation 2122' to 2050'. The foundation was soft silt to stratem 5 at elevation 1975. The following conclusions were based on detailed analysis carried out as previously outlined:

If a rock fill were placed rapidly, the subsoil would have insufficient strength to resist the side thrust and would flow laterally. The fill would tend to slip downward and outward away from the centerline and would trap compressible material below the center of the fill. The soft soils would intrude into the voids of the coarse rock fill and much of the fill material would be prevented from effectively supporting the material placed subsequently as shown in Fig. 49. For stability the fill would have to be extended on a slope of 1.5 to 1 to the top of stratum 5.

If material such as sand, which would be fine enough to prevent intrusion, were used for constructing the fill, it might be possible to displace the soft soils from below the center of the embankment by jetting or the use of explosives. These procedures have been used to settle fills as much as 60 feet in soft soils under shallow water but their effectiveness for the present case is less certain.

A method of construction which would shorten the construction period to a reasonable time, but which would require special materials and equipment and must be done with considerable care is outlined as follows:

1. Place 10 feet of material equivalent to a clean, fine concrete sand uniformly over the present bottom for the full length and width of 500 feet.
2. Construct 20-inch diameter vertical sand drains spaced 10 feet each way in triangular array. The bottom of the sand drains shall be at elevation 1975.

3. Place the rest of the fill uniformly to prevent overload over a minimum period of 2 years.
4. Install pore pressure cells at numerous locations between the sand drains to measure the developed pore pressure and the rate of consolidation, and thus prevent the rapid loading of the undersoil as the fill construction progresses.

Sand drains may also be useful to relieve water pressure under dams or pressure from pervious layers under excavations as shown in Fig. 50. The open cut problem occurred in connection with the approaches to the Elizabeth River tunnel at Norfolk, Virginia. Other methods of improving soil properties are outlined in Fig. 51.

Vertical Shear

When there is a tendency for differential settlement between two bodies, vertical shear stresses are set up which change the vertical pressures as shown in Fig. 52. To prevent the stresses and irregular surface caused by culverts placed on piles in weak foundations, it is sometimes suggested that the culvert be allowed to settle with the fill. While this is sometimes done as shown in Fig. 53, allowance must be made for loss in clearance, vertical sag in the pipe and the tendency for the pipe to spread with the fill.

Vertical shear is also important in piles in soft soil for, as shown in Fig. 54, the shear which tends to hold up the pile in driving or doing a loading test may be reversed by a fill which causes consolidation of the soft soil. A case in point is bridge 8 of the Pentagon network shown in Fig. 55. To support wing walls at elevation 25, piles were driven through 15 feet of rolled fill and 10 feet of dump fill into the clay. Due to the resistance to driving built up in the fill, the piles did not reach the sand and gravel below the clay. When fill was placed around the walls, settlements were observed as shown in Fig. 55. In analyzing the record the observed values were adjusted to eliminate the settlement due to the October 1942 fill leaving primarily the settlement due to the August 1942 fill. A curve for primary consolidation for simple vertical drainage was fitted to the curve. As shown in Fig. 55, the fit was very good up to 8 months or 90 percent of the indicated primary consolidation.

The thickness change of the laboratory samples of the peaty clay plotted against log of time was linear from 1 to 96 hours and showed a settlement per cycle of 20 percent of the total for each load increment. From 8 to 80 months this would account for an additional settlement of 20 percent beyond the indicated primary settlement or 0.2 x 0.71 = 0.14 feet. The observed difference is 0.25 feet. The excess may be due to the secondary consolidation from the fill placed in January 1942. It should be noted that the observed record is concave upward indicating that the linear relation shown up to four days in the laboratory is not maintained up to 80 months.

As a further example, the north abutment of the new 14th Street Bridge over the Potomac at Washington was supported on piles driven to good bearing according to pile driving formulas and short time loading tests. Despite the fact that borings showed soft organic clay below the piles, the design was approved because no trouble had been experienced with the old bridge which is situated nearby on a similar foundation. Subsequent investigation disclosed that the old abutment had settled 11 inches but without damaging the simply supported truss span. The presence of the settlement was obscured by the general settlement of the adjacent reclaimed marsh and the use of the abutment as a bench mark. An equal settlement could not be tolerated on the new bridge with continuous plate girder spans.

The abutment, as shown in Fig. 56, was built above the original ground and the rolled fill placed, the middle of the filling period being in February 1949. Four months later continuous records of settlement were started at the bridge seat and at the end of the wing wall. The fill and the wall settled together due to the compression of both the upper and lower layers. The settlement of the bridge seat is due primarily to the lower of the two compressible layers. Plotted against square root of time it is seen to be linear except for the rebound due to excavation for underpinning with piles to sand and gravel. It was calculated from laboratory tests that the total settlement in the lower layer would be 8 inches under the bridge seat and 14 inches under the wall. By August 1949, the bridge seat had also moved 3 inches toward the fill.

The settlement in the upper layer under the end of the wing wall was calculated by subtracting $\frac{14}{8}$ of the observed bridge seat settlement from the observed wall settlement. A primary consolidation for vertical consolidation was fitted to the derived curve and extrapolated as shown in Fig. 6. When underpinning, required to stop the movement of the abutment away from the bridge, was complete the movement of the wall stopped but the fill continued to settle. By adding $\frac{14}{8}$ of the observed bridge seat settlement to the extrapolated curve for the upper layer, a prediction of the fill settlement was derived. A check observation on the fill at 24 months shows excellent agreement as shown in Fig. 56.

Based on the 8 inch settlement of the bridge seat calculated from test results on the lower layer, 25 percent of primary consolidation occurred in six months, indicating a coefficient of consolidation of 0.11 feet squared per day based on vertical consolidation. As shown in Table 8, the average laboratory value is 0.04, showing that the sand lenses had appreciable effect in accelerating the settlement.

Scour ✓

Records of the Bureau of Public Roads indicated that the two chief troubles with bridges are settlement and displacement of abutments which could be analyzed as above and scour which has thus far not been adequately analyzed. One difficulty has been that observations are usually taken after floods when displaced material may have been replaced as shown in Fig. 57, or the bridge already lost. Fig. 57 also suggests the use of long piles well tied into the pier which have some chance of holding the pier, even though undermined. While not adequate,for design, the following considerations are of value in relation to foundations where scour may be a factor.

Flowing water can erode almost any material, the rate depending on the velocity, suspended load, and the river bed. A given stream approaches a dynamic equilibrium between erosion, suspended load, and deposition. However, flood flows may cause extreme variations, the effect of which may last for years, or a drought may have left deposits that are gradually removed with the renewal of normal flow. Flood intensities increase with deforestation or paving of large areas.

Scour is accentuated at bends in streams, by narrowed sections, confinement of steep banks and turbulence caused by obstructions. The danger of scour may be indicated by a study of the stream's history and present action. However, the maximum scour occurs at the height of floods, whereas most observations are made after floods, when deep holes may have been refilled and still deeper loosening of the soil may pass unobserved. Some observations have indicated a relation between the rise of water and the depth of scour. In several cases the depth of scour has been three or four times the rise of water. On the other hand, the greatest scour does not always accompany the highest water, so that the above rule cannot be general.

Changed conditions are a frequent cause of foundation failure. Reduction of suspended load, as by desilting reservoirs or storage dams, increase the capacity for scour. Abutments or piers may increase scour by increasing depth and/or turbulence. Levees or approach fills to bridges may increase scour by confinement of flood flows. Blocking of the channel by debris or ice is a frequent cause of increased scour. Greater scour may accompany increased gradients due to channel clearing, dredging or straightening.

On the positive side, scouring is very slow in solid rock, and intact clay has a high resistance. On the other hand, beds of boulders requiring blasting for excavation have been washed out by a single flood and several failures have been caused by placing piers on thin ledges, subject to undermining and by piles being stopped simply because of hard driving.

The danger of lateral shift of the channel is less in naturally straight reaches. Periodic inspection has prevented several failures by revealing the development of dangerous conditions. Some protection against local scour may be afforded by sheet piling, rip rap, or loaded mats around foundations. Other control devices include anchoring wire mesh to extend between piers and tight sheet pile walls across the channel just down stream from piers.

Despite all reasonable precautions, and although they cannot be predicted, the possibility of unprecedented floods must be reckoned with.

Closing

The utility of soil mechancis to the design of highway structures is well established. While not perfect, soil tests and analysis are far superior to professional opinions. Improvements are being made constantly which are reported in technical journals and by such organizations as the Highway Research Board and the American Society of Civil Engineers. The correlation of field observations with laboratory tests and analysis is particularly fruitful.

METHOD OF DIRECT SHEAR TEST OF SOIL

Scope

1. This method of test destermines the shearing resistance of a soil sample when subjected to a continuous shearing displacement. The sample can be subjected to various normal loads, tested in air or immersed, and the normal displacement can be measured.

Apparatus

2. The apparatus consists of the following:

   (a) Sample former - A cutter or mold for forming samples to fit the shear box. Samples can be compacted directly in the shear box.

   (b) Shear device - The sample is placed in a box made of two frames with an inside diameter of 1-15/16 inches. See figure 58. While shearing, the upper frame can move upward only and the lower frame rides on ball bearings with a minimum clearance of 0.01 inch between the frames. This clearance can be eliminated, except when shearing, by screwing together two wedges with the upper frame held down by four screws. The lower frame contains a porous stone whose height may be adjusted by brass fillers placed beneath. The normal load is applied with lead weights through a lever and a guided piston with a rough porous face. Two retaining pins prevent shearing displacement until removed when the shearing test is started.

   The shearing and normal displacements are measured by micrometer dials. The shearing displacements are measured by a hand-operated or motor driven gear system through a calibrated ring which deflects approximately 0.0002 inch per pound.

(c) Balance - A balance for weighing the soil sample to an accuracy of 0.1 percent.

(d) Oven - An oven for drying the soil samples at 110 C (230 F).

Procedure

3. A sample of known wet weight and approximately 0.5 inch thick is prepared to fit the shear box. With the shear box in place, the sample is carefully inserted, the piston and guide assembled over it, and the initial thickness determined from the vertical dial which is previously calibrated under various loads with a brass disk in place of the sample. If there is danger of water or soil running between the two frames of the shear box, the clearance is eliminated by tightening the wedges. The desired normal load is applied. The sample may be sheared immediately or allowed to come to equilibrium under the normal load either in air or under water poured through the piston guide. When ready to shear the sample, the vertical dial is read to determine the thickness change, the wedges loosened, if used, the shear displacement dial set to zero, and the retaining pins and four screws in the upper frame removed. The crank is turned 20 revolutions per minute which produces a shearing displacement of 0.04 inch per minute (minus the deflection of the calibrated ring). Sufficient dial readings are recorded to define the relation between shearing displacement, vertical displacement, and shearing load.

The sample is removed and weighed and its appearance noted. The sample is dried to constant weight at 110 C (230 F) and the dry weight determined.

4. Values computed from the following equations are tabulated:

$$\text{Moisture content, percent} = 100 \left(\frac{\text{wet weight}}{\text{dry weight}} - 1\right)$$

$$\text{Dry density} = \frac{\text{dry weight}}{\text{thickness x area of box}}$$

$$\text{Volume change, percent} = 100 \, \frac{\text{change in thickness}}{\text{initial thickness}}$$

$$\text{Shearing or normal stress} = \frac{\text{shearing or normal load}}{\text{area of box}}$$

The shearing stresses and volume changes as ordinates are plotted against the shearing displacements as abscissas. The maximum or otherwise designated shearing resistances as ordinates are plotted against the corresponding normal stresses as abscissas.

APPENDIX B ✓

METHOD OF TRIAXIAL COMPRESSION TEST OF SOIL

Scope

1. This method of test determines the relation between vertical stress and strain in a compressed soil sample under various constant lateral pressures.

Apparatus

2. The apparatus consists of the following:

(a) Sample Former - A cutter and trimming implements (figure 59) or a split mold with two pistons for forming a sample whose height is twice its diameter.

(b) Triaxial Device - As shown in figure 60, the sample is placed in a thin rubber sleeve between two disks (either porous or impervious) and subjected to a constant vertical and lateral air pressure which is measured by a gage and maintained by a pressure reduction valve connected to a pressure reservoir. A differential vertical load is applied through a spherical bearing and a lapped piston.

(c) Air Supply - Provision for a vacuum of half an atmosphere and a pressure of three atmospheres absolute.

(d) (d) Loading Device - A machine for displacing the piston one percent of the sample height per minute. The differential vertical load and displacement are measured by a calibrated ring and micrometer dial attached to the piston or by the loading device itself.

(e) Balance - A balance for weighing the samples to an accuracy of 0.1 percent.

(f) Oven. - An oven for drying the samples at 110 C (230 F).

Procedure

3(a). The prepared weighed sample, in a loose-fitting, thin metal sleeve, is placed on the lower disk; the rubber sleeve is pulled up around the metal sleeve which is then removed and the top disk is clamped in place. For cohesionless materials the sample is prepared on the lower disk in the rubber sleeve which is supported by a split metal sleeve which is removed after the top disk is in place and a vacuum temporarily applied through the lower disk.

(b) The lucite cylinder is put in place and the top of the device is bolted down on it, whereupon, the assembly is placed in the loading device, contact is made and zero displacement is recorded. Next the air pressure is applied and zero differential vertical load is recorded. The piston is displaced and the load and displacement recorded until the vertical stress becomes constant or the sample is shortened 20 percent of its height. The sample is removed, weighed, dried in the oven to constant weight, and weighed again.

(c) Generally three samples are tested under air pressures of 0, 2, and 4 kips per square foot.

Calculation and Plotting

4(a). Values calculated from the following equations are tabulated.

$$\text{Moisture content, percent of dry weight} = 100 \left(\frac{\text{wet weight}}{\text{dry weight}} - 1 \right)$$

$$\text{Initial density} = \frac{\text{initial weight}}{\text{volume of sample}}$$

$$\text{Strain} = \frac{\text{reduction in height}}{\text{initial height}}$$

$$\text{Stress difference} = \frac{\text{differential vertical load}}{\text{initial cross-sectional area}} \quad (1\text{-strain})^{*}$$

---

* To allow for increased area assuming constant volume.

(b) The stress differences as ordinates are plotted against the strains as abscissas.

The strain between two stress conditions within the range of the test values can be determined from these curves.

The maximum or otherwise designated stress difference derived from these curves can be used with the lateral pressures to determine a relation between normal and shear stresses.

METHOD OF TEST FOR CONSOLIDATION OF SOIL

Scope

1. This method of test is to determine the amount and rate of thickness change in a block of soil when loaded axially and confined laterally.

Apparatus

2. The apparatus shall consist of the following:

(a) Soil Cutter. -- A cylindrical ring, sharpened on the outside, of the same height and diameter as the consolidometer ring and a device for producing a controlled axial movement of the cutting ring, see figure 59.

(b) Trimmers. -- Instruments such as a piano wire saw, sharpened straight-edge, spatula, etc.

(c) Consolidometer.-- A cylindrical ring (depth not over one half the diameter) as shown in figure 61 to hold the soil sample with a porous stone above and below and appurtenances permitting immersion of the sample, transmission of axial load and measurement of thickness under various loads. A metal blank of known thickness is required for calibrating the dial gage.

(d) Loading Device. -- A device for applying constant static loads to the consolidometer.

(e) Balance. -- A balance for weighing the soil sample to an accuracy of 0.1 percent.

(f) Oven. -- An oven for drying the soil samples at 110C(230F).

Sample

3(a). The sample shall be cut from the block of soil as described in paragraph (b), and placed in the consolidometer ring so as to preserve its moisture content, density, and structure.

(b) In an atmosphere of 95 percent humidity, the block of soil shall be cut somewhat larger than the final sample with one plane face perpendicular to the direction in which the load is to be applied. The lightly greased cutting ring shall be placed on the opposite face and forced down gradually while the excess material is trimmed from the outside to minimize the pressure required on the ring. When the ring is full, the excess soil above shall be removed with a piano wire saw and the surface trued with a straightedge. The consolidometer ring shall be placed on a smooth surface, the cutting ring inverted over it and the sample forced into it with a uniform pressure. The second face shall be trimmed and the weight of the soil sample plus the ring determined. This total weight minus the weight of the ring previously determined equals the wet weight of the sample. The consolidometer shall be assembled aroung the consolidometer ring as shown in figure 2, and the dial gage read to determine the initial thickness of the sample (Note).

Note.--The gage shall be calibrated by taking readings with various loads on a metal blank of known thickness in place of the samples.

Procedure

4 (a). The loading shall be selected to include the stress changes anticipated in the material represented by the sample. A typical series of pressures to be applied consecutively to the sample shall be 500,1000, 2000, 4000, 8000, 16000, 4000, 2000, 1000, 500 lb. per sq. ft. Unless otherwise specified, the sample shall be immersed in water when the first pressure increment is applied. Each pressure shall be maintained until the change in thickness does not exceed 0.01 percent of the thickness per hour. The final thickness under each load shall be recorded. After the application of each load except the first, the gage shall be read at 0.09, 0.25, 0.49, 1, 4, 9, 25, and 64 minutes after half the pressure increment has been applied, the total increment being applied at a uniform rate.

(b) After the last gage reading, the sample shall be removed from the ring, dried to constant weight in the over at 110 C (230 F) and the weight determined.

Calculations

5 (a). Values shall be calculated and tabulated from the following equations:

$$\text{Initial moisture content, percent} = \left(\frac{\text{wet weight}}{\text{dry weight}} - 1\right) \times 100$$

$$\text{Initial oven-dry density, lb. per cu. ft.} = \frac{\text{oven-dry weight}}{\text{initial thickness x area of ring}}$$

Reduction in thickness, percent =

$$\left(1 - \frac{\text{final thickness under load}}{\text{initial thickness}}\right) \times 100$$

Consolidation, percent = $\frac{\text{thickness change for given time}}{\text{total thickness bhange for given increment}} \times 100$

(b) Average time-consolidation data shall be calculated by averaging the percentages of consolidation at a given time under the various pressures.

Plotting

6 (a). The stress-strain relationship may be shown by plotting the pressures as abscissas to a logarithmic scale and the percentage reduction in thickness as ordinates to an arithmetic scale positive downward.

(b) The time-consolidation relationship may be shown by plotting the square root of time as abscissas and the percentage consolidation as ordinates increasing downward.

Table 1

SOIL TEST RESULTS

Fill and Abutment for Field Engineer

Boring No.	Borrow pit			2		2	
Sample No.				2		5	
Depth, feet	2-8			48		36	
Per cent passing following:							
1½ in. sieve							
1 inch							
¾ inch							
⅜ inch							
No. 4 sieve							
No. 10	100			100		100	
No. 20							
No. 40	99			98		98	
No. 100							
No. 200	96			12		77	
0.005 mm.	79			7		24	
0.002 mm.							
0.001 mm.	49			3		6	
Ignition loss, per cent							
Liquid limit	78			NP		48	
Plastic limit	56			NP		19	
Plasticity index							
Shrunk density, pounds per cu. ft.	120						
Optimum moisture, % dry weight	21						
Maximum density, lb. per cu. ft. dry	103						
Loose density, lb. per cu. ft.							
	~~Compacted~~						
Consolidated shear strength, lb/sq. ft.	830	1630	2300				
Normal stress, lb/sq. ft.	350	1000	2000				
Thickness reduction before shear, %	-5.1	-2.2	-0.8				
Maximum stress difference in compression, lb/sq. ft.				5000	9600	1200	1400
Lateral pressure, lb/sq. ft.				2000	4000	0	2000
Reduction in height at failure, %				3.6	3.9	9	8
Initial density, lb/ cu. ft. wet	126	128	128	120	121	100	102
Initial moisture, % dry weight	23	22	22	21	22	61	58
~~Specific gravity~~ Final moisture	31	27	24				
Cumulative % reduction in thickness for following loads: 5"samples							
0.04 kips per sq. ft.				0		0	
~~0.5~~ .35 inundated				-		3.5	
1				0.4		6.8	
2				0.6		9.8	
4				0.8		12.5	
~~8~~ 6				0.9		14.0	
~~16~~ .04				0.6		8.5	
Coef. of consolidation, ft. sq./day						.045	
Corresponding pressure kips/sq. ft.						2-4	
Coef. of permeability, ft. per day							
	clay			sand		silt	

EDWARD S. BARBER
Soils Engineer

Table 2 --Soil pressure functions

f = friction coef.	K_a = $(\sqrt{1+f^2} - f)^2$	$\sqrt{K_a}$ = $\sqrt{1+f^2} - f$	K_p = $(\sqrt{1+f^2} + f)^2$	$\sqrt{K_p}$ = $\sqrt{1+f^2} + f$
0	1.000	1.000	1.000	1.000
.05	0.905	0.951	1.105	1.051
.10	.819	.905	1.221	1.105
.15	.742	.861	1.348	1.161
.20	.672	.820	1.488	1.221
.25	.610	.781	1.640	1.281
.30	.554	.744	1.806	1.344
.35	.503	.710	1.987	1.410
.40	.458	.677	2.182	1.477
.45	.418	.647	2.392	1.547
.50	.382	.618	2.618	1.618
.55	.350	.591	2.860	1.691
.60	.321	.566	3.117	1.766
.65	.294	.543	3.396	1.843
.70	.271	.521	3.689	1.921
.75	.250	.500	4.000	2.000
.80	.231	.481	4.329	2.081
.85	.214	.462	4.676	2.162
.90	.198	.445	5.042	2.245
.95	.184	.429	5.426	2.329
1.00	.172	.414	5.828	2.414

Table 3 --Calculation of safety factor against rotation about point O, figure 17.

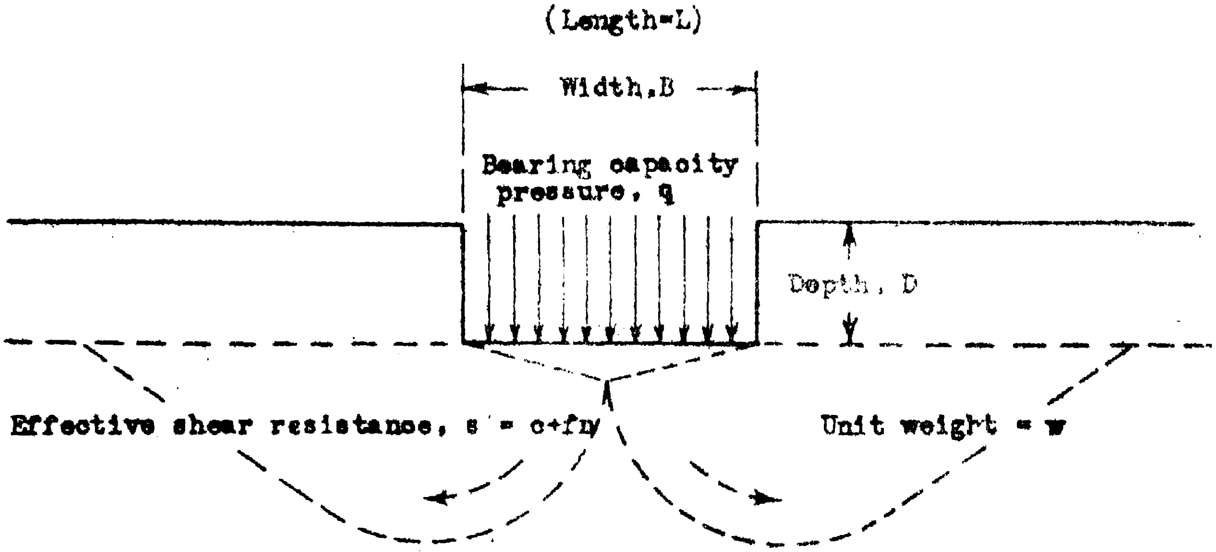
Radius of arc, R=70, maximum for assumed center
Rotating moment = 174 x 35 - 14.5 x 75 = 5010 kip-feet

L = length of arc
W = total weight of any slice
c = cohesion of soil
f = coefficient of friction

Arc	f x W x V	c x L x R
AB	.8x1x16 = 10	.7x11x70 = 540
BC	.66x17x32 = 360 .66x10x34 = 220	——
CDE	——	.65x134x70 = 6100
EF	.66x10x34 = 220	——
Total	810 plus	6640 = 7450 kip-feet

$$\text{Safety factor} = \frac{7450}{5010} = 1.48$$

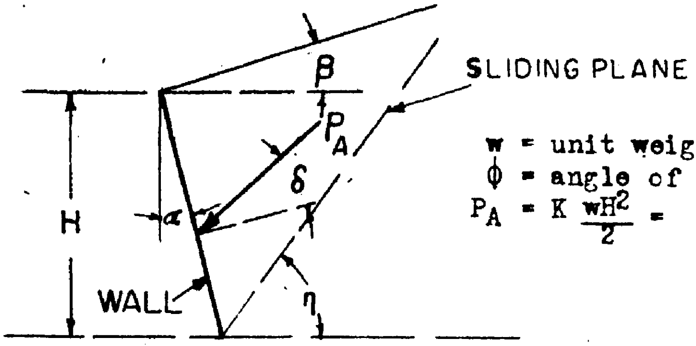
Table 4.--Bearing capacity factors for shallow rectangular footings



$$q = (1 + .3\frac{B}{L})\ cF_c + wDF_D + (1 - .2\frac{B}{L})\ wBF_B$$

Friction coef., f	Bearing capacity factors F_c	F_D	F_B
0	5.7	1.0	0.0
.05	6.6	1.3	0.0
.1	7.6	1.8	0.1
.15	8.9	2.3	0.2
.2	10.4	3.1	0.35
.25	12.1	4.0	0.6
.3	14	5.3	1.0
.35	17	7.0	1.6
.4	20	9.0	2.5
.45	24	11.7	3.7
.5	28	15.1	5.5
.55	34	20	8
.6	40	25	11
.65	48	32	16
.7	58	41	22
.75	69	53	31
.8	83	68	44
.85	100	86	62
.9	120	109	85
.95	143	137	115
1.00	172	173	160

Table 5 Active earth pressure factors for cohesionless material.



w = unit weight
φ = angle of internal friction
$P_A = K \frac{wH^2}{2}$ = Total load

δ =		-φ	0	φ	-φ	0	φ	-φ	0	φ	-φ	0	φ
φ =			15			25			35			45	
β =	α =	Values of K											
-10	-10	.93	.48	.40	.84	.32	.26	.73	.19	.16	.59	.10	.09
-10	0	.96	.63	.45	.90	.37	.32	.82	.25	.22	.70	.17	.15
0	-60	1.03	.23	.15	.35	.02	.01	-	-	-	-	-	-
0	-50	.68	.40	.26	.63	.09	.06	.21	.01	.01	-	-	-
0	-40	.98	.41	.30	.71	.17	.11	.44	.05	.04	.15	.004	.003
0	-30	.94	.46	.36	.76	.23	.17	.56	.10	.07	.35	.03	.02
0	-20	.87	.50	.40	.75	.28	.22	.61	.15	.12	.45	.07	.06
0	-10	.97	.54	.45	.91	.34	.28	.82	.21	.18	.71	.12	.11
0	0	.97	.59	.52	.91	.41	.36	.82	.27	.25	.71	.17	.18
0	10	1.03	.66	.60	.99	.48	.44	.93	.35	.34	.84	.24	.27
0	20	1.13	.74	.69	1.13	.57	.56	1.09	.43	.46	1.04	.32	.40
0	40	1.54	1.03	1.04	1.64	.86	.94	1.69	.73	.95	1.70	.59	.99
0	60	2.82	1.77	1.97	3.27	1.58	2.03	3.62	1.42	1.95	3.86	1.27	1.68
10	-10	.93	.64	.57	.85	.39	.33	.73	.23	.20	.65	.13	.12
10	0	.97	.71	.66	.91	.46	.42	.82	.30	.29	.71	.19	.19
30	-20	-	-	-	-	-	-	.65	.24	.20	.48	.09	.08
30	-10	-	-	-	-	-	-	.73	.34	.32	.59	.15	.15
30	0	-	-	-	-	-	-	.82	.43	.46	.71	.24	.26
30	10	-	-	-	-	-	-	.94	.57	.67	.85	.34	.42
30	20	-	-	-	-	-	-	1.10	.75	.99	1.03	.48	.69
		Values of Cot η											
-10	-10	.18	.78	.96	.18	.69	.83	.18	.59	.70	.18	.49	.57
-10	0	0	.62	.83	0	.56	.73	0	.48	.61	0	.40	.49
0	-60	1.73	2.41	2.48	1.73	1.92	1.93	-	-	-	-	-	-
0	-50	1.19	1.77	1.96	1.19	1.57	1.59	1.19	1.30	1.31	-	-	-
0	-40	.84	1.57	1.71	.84	1.30	1.36	.84	1.09	1.11	.84	.92	.92
0	-30	.58	1.30	1.47	.58	1.09	1.18	.58	.92	.96	.58	.76	.78
0	-20	.36	1.09	1.27	.36	.92	1.02	.36	.77	.83	.36	.64	.67
0	-10	.18	.92	1.13	.18	.76	.91	.18	.64	.72	.18	.52	.57
0	0	.00	.77	1.00	.00	.64	.80	.00	.52	.63	.00	.42	.50
0	10	-.18	.63	.90	-.18	.53	.73	-.18	.42	.58	-.18	.31	.45
0	20	-.36	.52	.82	-.36	.41	.67	-.36	.32	.57	-.36	.22	.41
0	40	-.84	.31	.76	-.84	.22	.66	-.84	.12	.57	-.84	.03	.48
0	60	-1.73	.11	.99	-1.73	.03	.98	-1.73	-.05	.74	-1.73	-.14	.33
10	-10	.18	1.25	1.55	.18	.87	1.00	.18	.68	.77	.18	.53	.60
10	0	.00	1.14	1.44	.00	.75	.92	.00	.57	.69	.00	.43	.52
30	-20	-	-	-	-	-	-	.36	.95	1.02	.36	.68	.72
30	-10	-	-	-	-	-	-	.18	.90	.98	.18	.60	.66
30	0	-	-	-	-	-	-	.00	.83	.94	.00	.52	.60
30	10	-	-	-	-	-	-	-.18	.78	.94	-.18	.46	.58
30	20	-	-	-	-	-	-	-.36	.74	.93	-.36	.40	.58

From Erddruck, by H. Krey

Table 6 - Earth pressure factors for cohesionless material - negative surcharge - vertical wall - zero wall friction.

ϕ β	15	20	25	30	35	40	45
				K for active pressure			
0	0.59	0.49	0.41	0.33	0.27	0.22	0.17
-5	.55	.46	.38	.32	.26	.21	.165
-10	.53	.44	.37	.30	.25	.20	.16
-15	.50	.42	.35	.29	.24	.20	.155
-20	-	.40	.34	.28	.23	.19	.15
-25	-	-	.32	.27	.22	.18	.145
-30	-	-	-	.26	.21	.18	.14
-35	-	-	-	-	.20	.17	.135
-40	-	-	-	-	-	.16	.13
-45	-	-	-	-	-	-	.125
				K for passive pressure			
0	1.70	2.04	2.46	3.00	3.69	4.60	5.83
-5	1.50	1.79	2.14	2.58	3.12	3.83	4.75
-10	1.30	1.55	1.85	2.20	2.64	3.19	3.90
-15	0.93	1.30	1.60	1.87	2.22	2.66	3.20
-20	-	0.88	1.28	1.55	1.85	2.20	2.63
-25	-	-	0.82	1.23	1.50	1.80	2.14
-30	-	-	-	0.75	1.16	1.43	1.91
-35	-	-	-	-	0.67	1.08	1.33
-40	-	-	-	-	-	0.59	0.97
-45	-	-	-	-	-	-	0.50

Table 7.--Effect of boundaries on time-consolidation.

Pressure at drainage face divided by pressure at impervious face	Degree of consolidation								
	0.1	0.2	0.3	0.4	0.5	0.6	0.7	0.8	0.9
	Time factor, $T = \frac{c_v t}{H^2}$								
0	0.049	0.100	0.154	0.217	0.29	0.38	0.50	0.66	0.95
0.2	.027	.073	.126	.186	.26	.35	.46	.63	.92
0.4	.016	.056	.106	.164	.24	.33	.44	.60	.90
0.6	.012	.042	.092	.148	.22	.31	.42	.58	.88
0.8	.010	.036	.079	.134	.20	.29	.41	.57	.86
1.0	.008	.031	.071	.126	.20	.29	.40	.56	.85
1.5	.006	.024	.058	.107	.17	.26	.38	.54	.83
2	.005	.019	.050	.095	.16	.24	.36	.52	.81
3	.004	.016	.041	.082	.14	.22	.34	.50	.79
5	.003	.013	.034	.069	.12	.20	.32	.48	.77
10	.003	.011	.028	.060	.11	.18	.30	.46	.75
Infinite	.002	.009	.024	.048	.09	.16	.28	.44	.73
Two drainage faces	.002	.008	.018	.031	.05	.07	.10	.14	.21
$\frac{\text{Well diameter}}{\text{Effective spacing}}$	Time factor, $T = \frac{ct}{D^2}$								
0.01	.046	.104	.167	.24	.38	.44	.58	.78	1.10
0.025	.032	.075	.124	.180	.25	.33	.44	.56	.86
0.1	.014	.037	.064	.096	.132	.178	.24	.32	.44
0.2	.006	.019	.035	.054	.077	.105	.14	.19	.29

Table 8. - Properties of Alluvial Clays

Project:	Pentagon Fill			Old Boundary Channel Bridge	Bridge 8 Pentagon Network		New 14th Street Bridge
Layer	1	2	3	Average	Upper	Lower	Lower
% Passing							
#10 sieve	100	100	100	100	74	99	100
#40 sieve	99	94	98	99	72	97	87
#200 sieve	88	70	75	85	59	65	71
.005 mm.	47	25	45	30	27	22	32
Liquid Limit	56	120	33	51	61	23	58
Plastic Limit	18	24	16	13	13	6	26
Coefficient of Consolidation, ft. sq./day	.14	.24	.10	.28	.17	.46	.04
Compressibility sq.ft./kip	.043	.090	.009	.043	.030	.006	.015

Table 9. - Time-Consolidation of Three Layer System

Layer	Thickness, H Feet	Compressibility, m. Sq.ft./kip	Coefficient of Consolidation, c Ft.^{2} per day	mH	$\frac{H}{mc}$	Settlement under 4.3 kips/sq.ft. (.43 mH) Feet
1	3	.0434	.14	.130	490	0.56
2	5	.0902	.24	.451	230	1.94
3	5	.0088	.105	.044	5410	0.19
Total	13			.625	6130	2.69

$$\text{Time, } t \approx T \sum mH \sum \frac{H}{mc} = T \times .625 \times 6130 = 3830 \ T \text{ days}$$

Table 10 - Readings of Pressure Cells on West Side of West Pier of Boundary Channel Bridge at North End Of Columbia Island, D. C.

Depth Feet	Pressure, pounds per square inch North set June'46	Feb.'47	Aug.'48	South set June'46	Feb.'47	Aug.'48
8	4.8	6.0	7.3	6.0	7.0	7.8
16	10.0	14.0	11.0	—	15.0	—
24	12.0	11.0	12.0	11.2	11.5	11.8

Table 11. – Consolidation Properties of Clay at Boundary Channel Bridge

Sample	Coefficient of Consolidation, c	Compressibility, m	$\frac{1}{cm}$
	Feet^2 /day	Sq. ft. per kip	
3A	.35	.049	58
3B	.03	.059	566
4	.28	.046	78
5	.07	.033	430
6	.14	.046	155
11	.08	.050	250
12	1	.020	50
Total 7	1.95	.303	1587
Average	.28	.043	229

$$\text{Weighted average vertical } c = \frac{1}{\text{Av. } m \times \text{av } \frac{1}{cm}} = \frac{1}{.043 \times 229} = 0.10 \text{ ft.}^2\text{ /day}$$

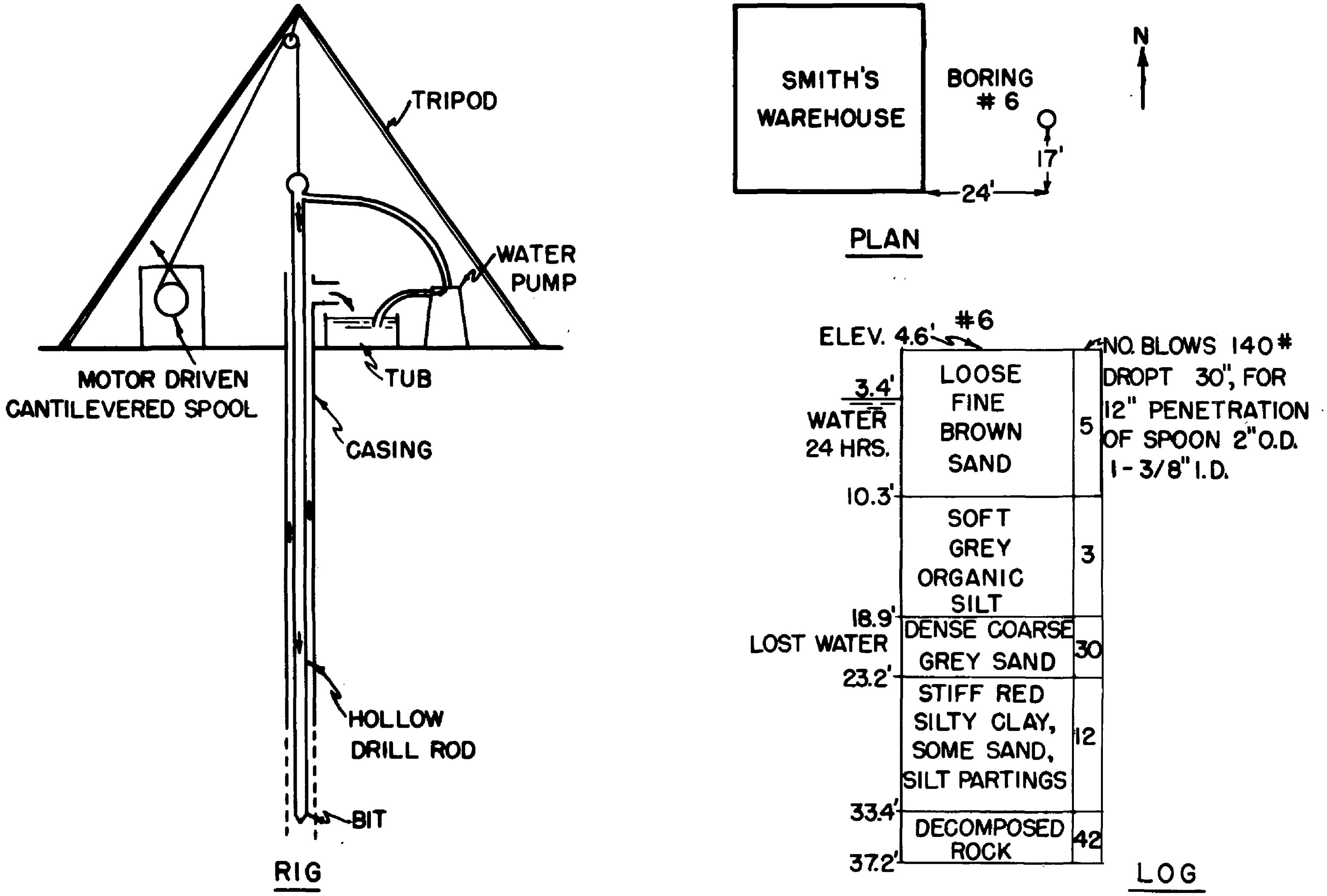
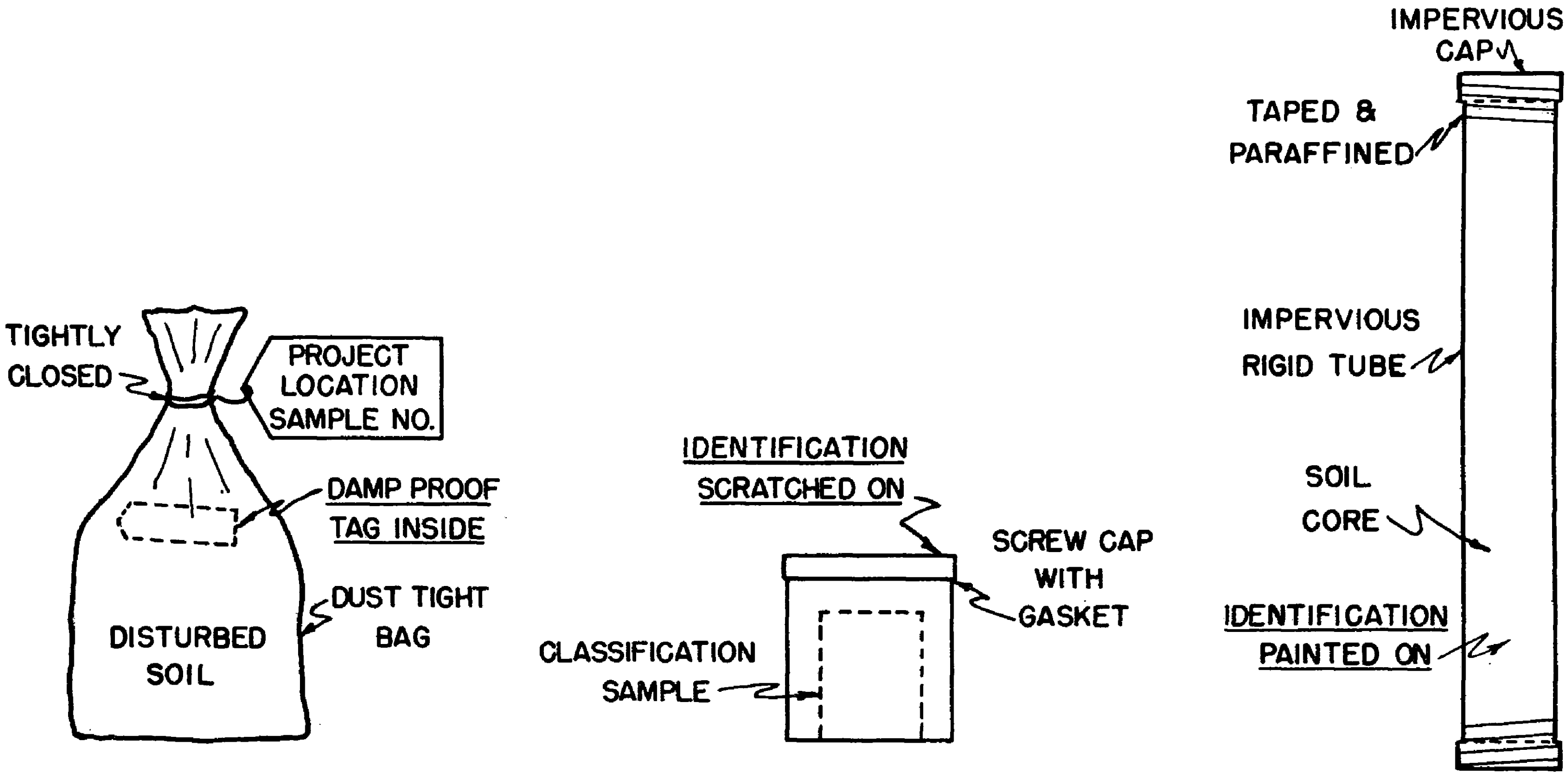


FIG. 1-TYPICAL BORING



BAG SAMPLE

JAR SAMPLE

TUBE SAMPLE

FIG.2-SOIL SAMPLES

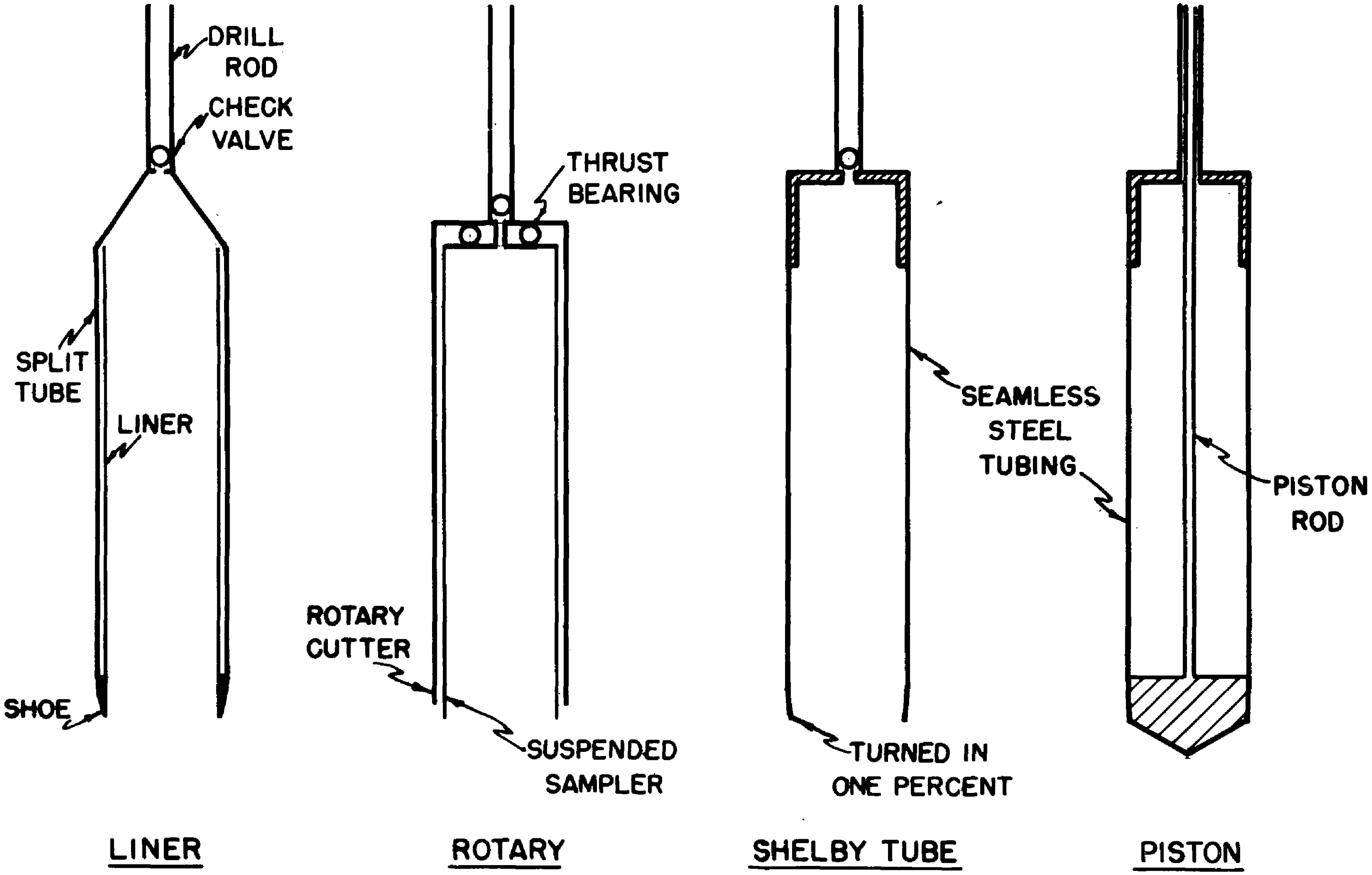


FIG.3-UNDISTURBED SOIL SAMPLERS

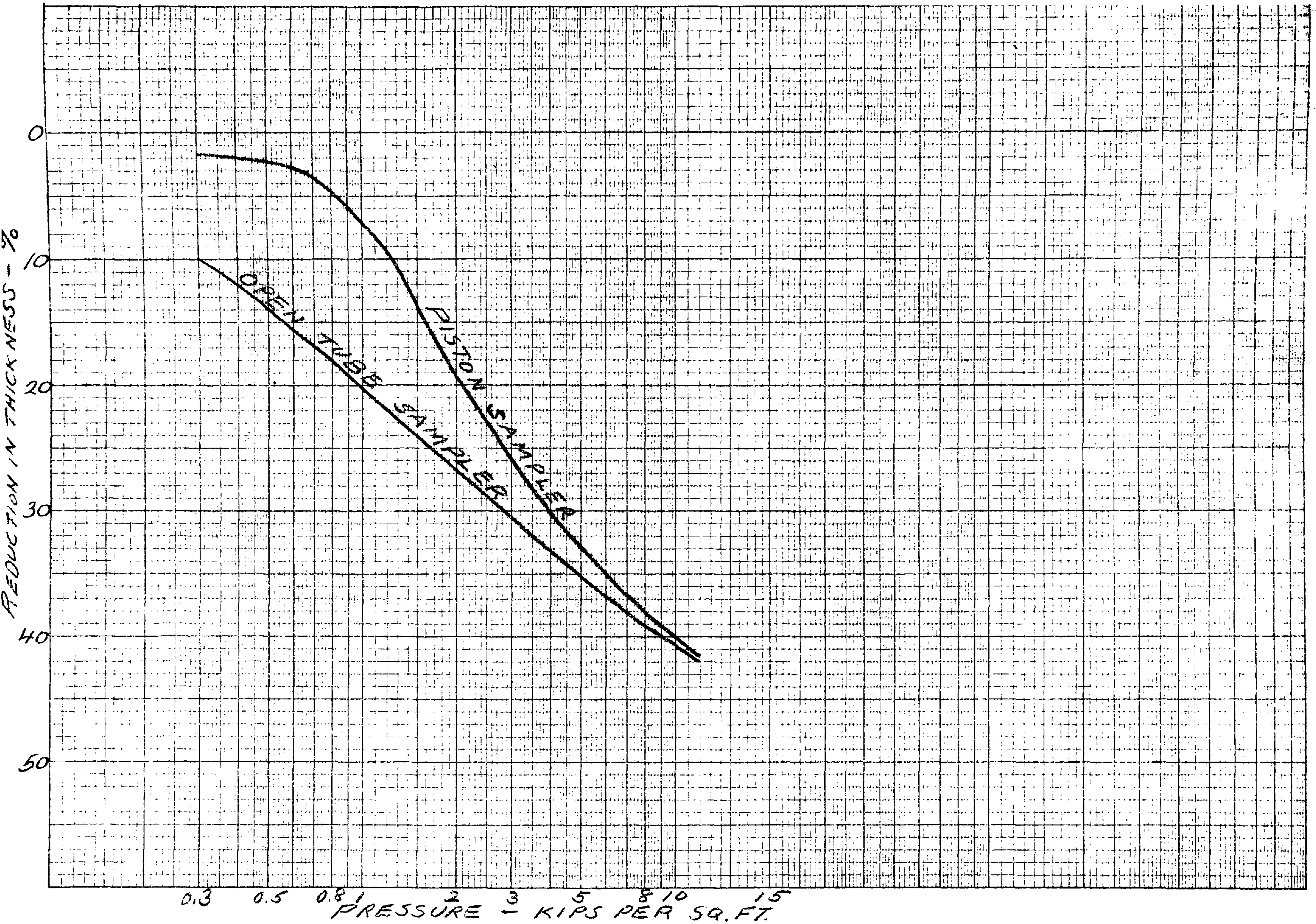


FIG. 4 - EFFECT OF SAMPLER ON CONSOLIDATION OF CHESAPEAKE BAY SILT

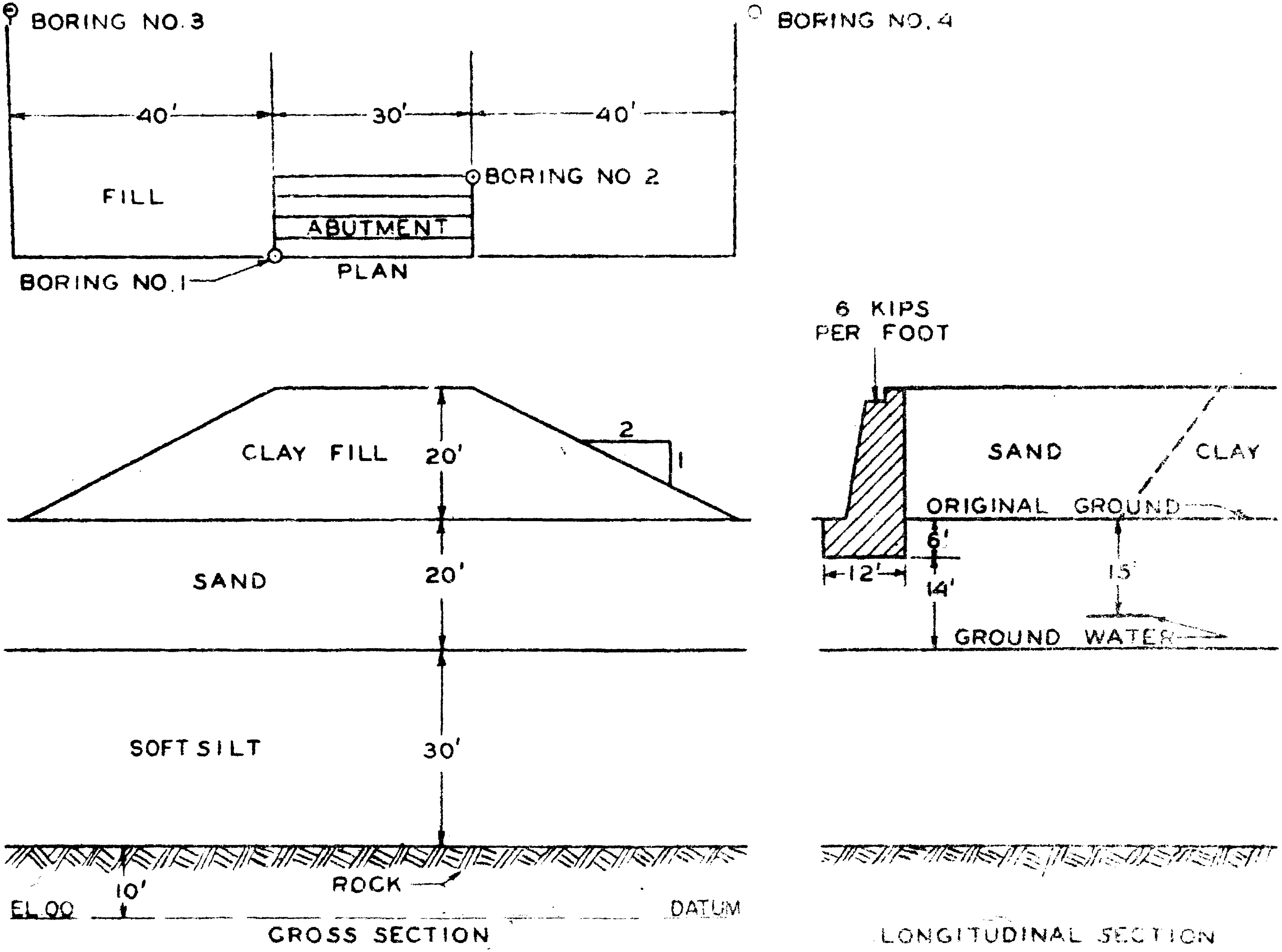


FIGURE 5 - FILL AND ABUTMENT STABILITY PROBLEM

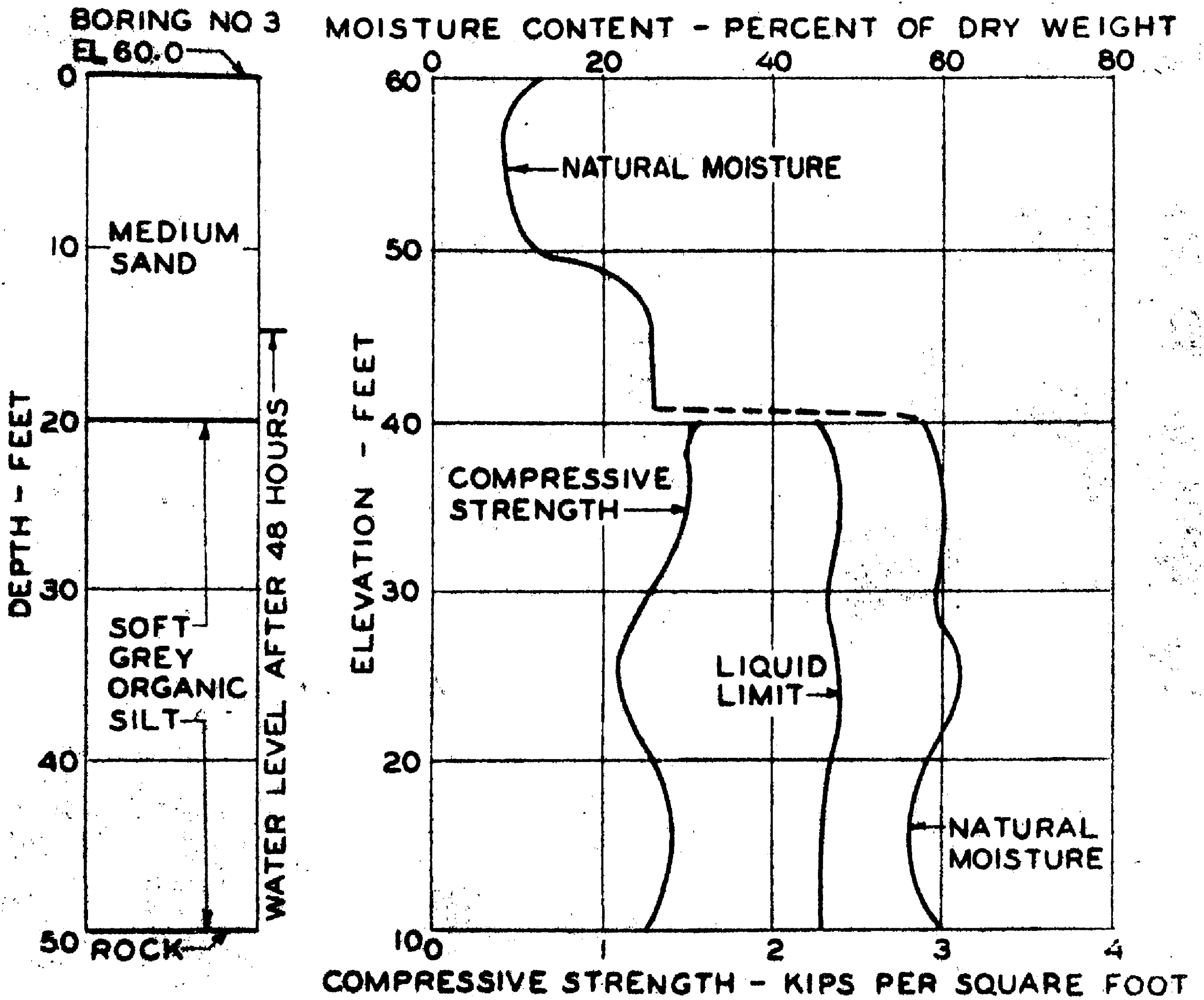


FIGURE 6 - TYPICAL BORING

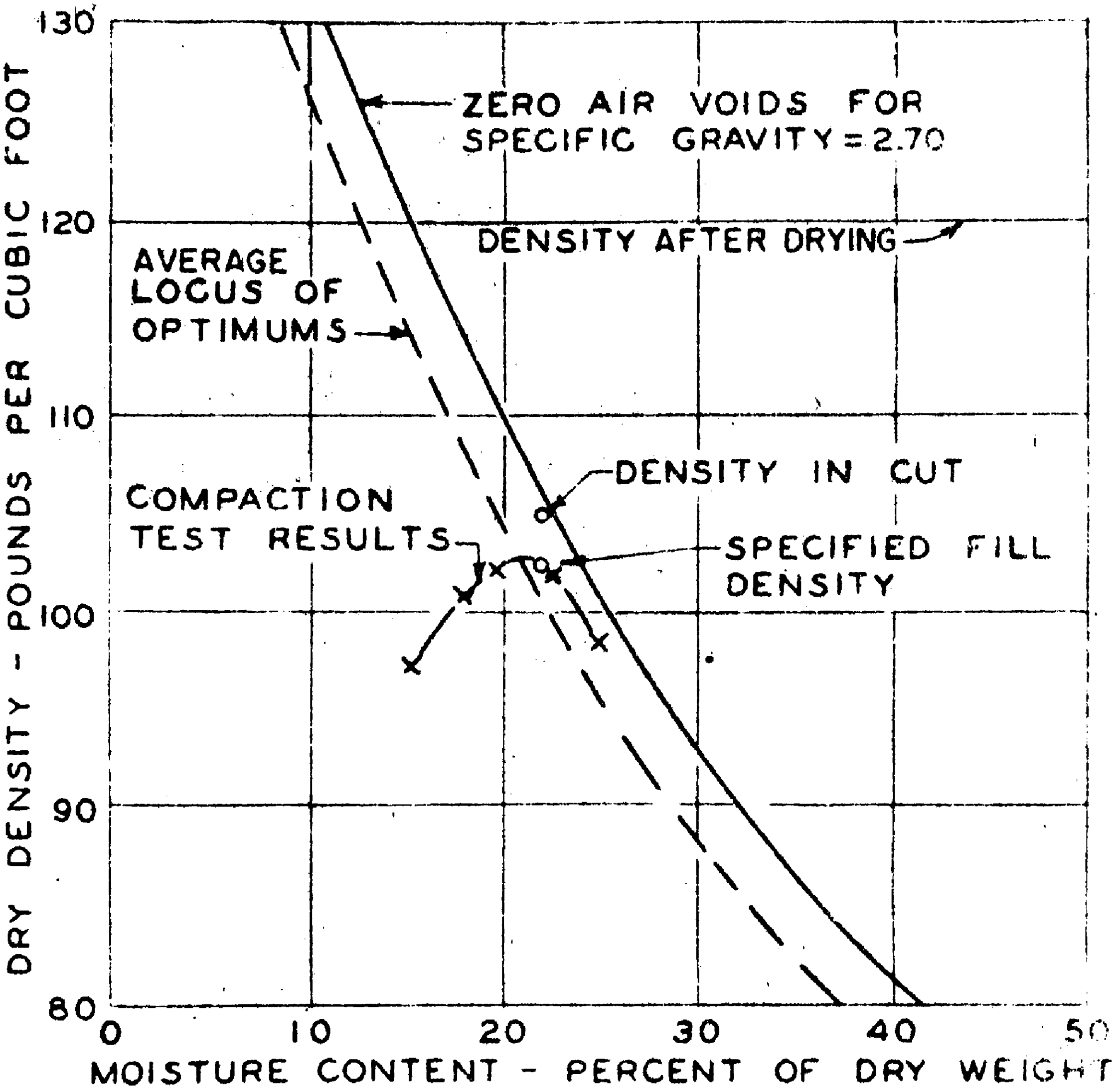
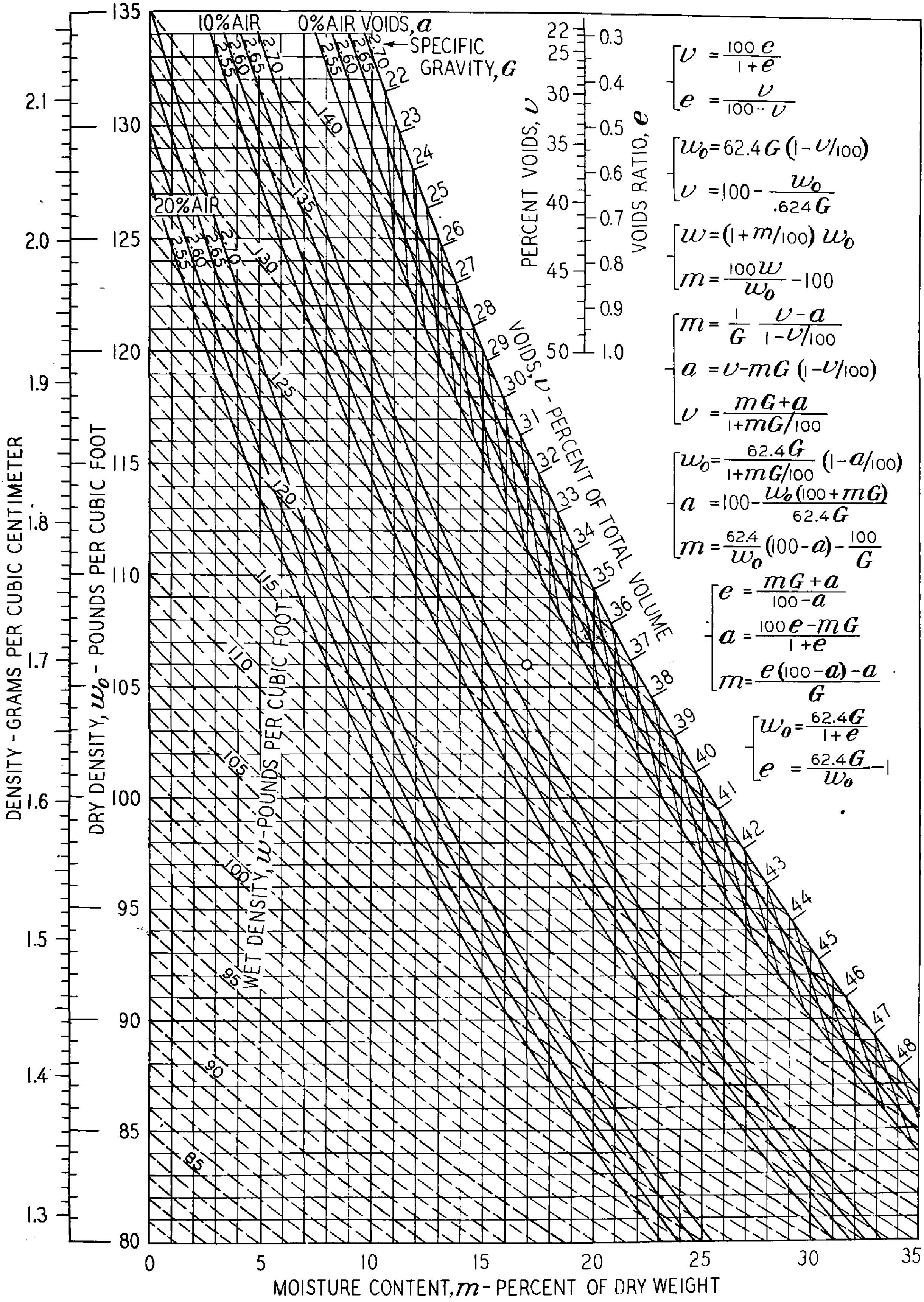


FIGURE 7 - MOISTURE - DENSITY RELATIONS



$$v = \frac{100\,e}{1+e} \qquad e = \frac{v}{100-v}$$

$$w_0 = 62.4\,G\,(1 - v/100) \qquad v = 100 - \frac{w_0}{.624\,G}$$

$$w = (1 + m/100)\,w_0 \qquad m = \frac{100\,w}{w_0} - 100$$

$$m = \frac{1}{G}\,\frac{v-a}{1-v/100} \qquad a = v - mG\,(1 - v/100) \qquad v = \frac{mG+a}{1+mG/100}$$

$$w_0 = \frac{62.4\,G}{1+mG/100}\,(1 - a/100) \qquad a = 100 - \frac{w_0(100+mG)}{62.4\,G} \qquad m = \frac{62.4}{w_0}(100-a) - \frac{100}{G}$$

$$e = \frac{mG+a}{100-a} \qquad a = \frac{100\,e - mG}{1+e} \qquad m = \frac{e(100-a)-a}{G}$$

$$w_0 = \frac{62.4\,G}{1+e} \qquad e = \frac{62.4\,G}{w_0} - 1$$

EXAMPLE :- GIVEN m=17, w_0=106, G=2.70 (POINT ○). THEN, w=124 AND BY INTERPOLATION BETWEEN 0 AND 10% AIR VOIDS CURVES, a = 8; 0% AIR VOIDS CURVE GIVES v=37.2, e=0.59 AND FOR SATURATION AT CONSTANT w_0, m=22, w=129; FOR SATURATION AT CONSTANT m, w_0=115.6, w=135, v=31.5, e=.46

FIGURE 8 - CHART OF SOLIDS-WATER-VOIDS RELATIONS OF SOIL MASSES.

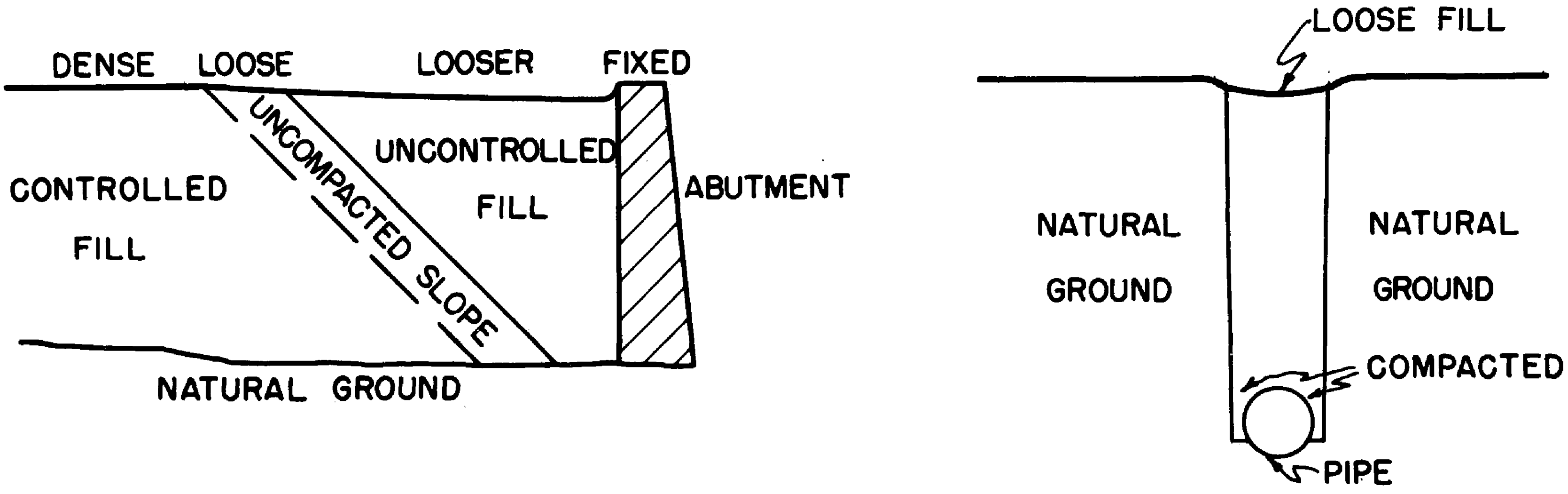


FIG.9-BACKFILL SETTLEMENT

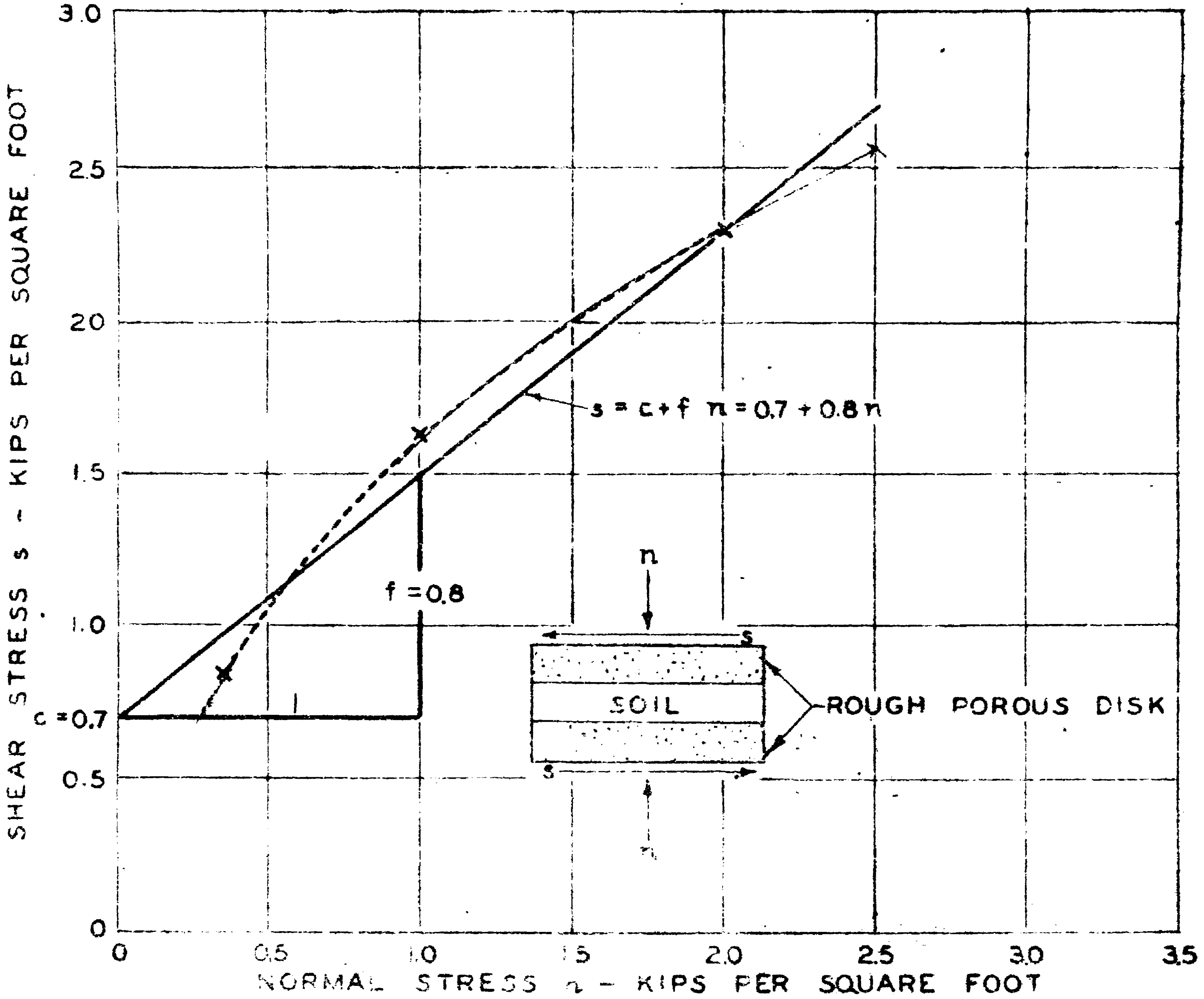


FIGURE 10- STRENGTH FROM DIRECT SHEAR TEST

P-3491

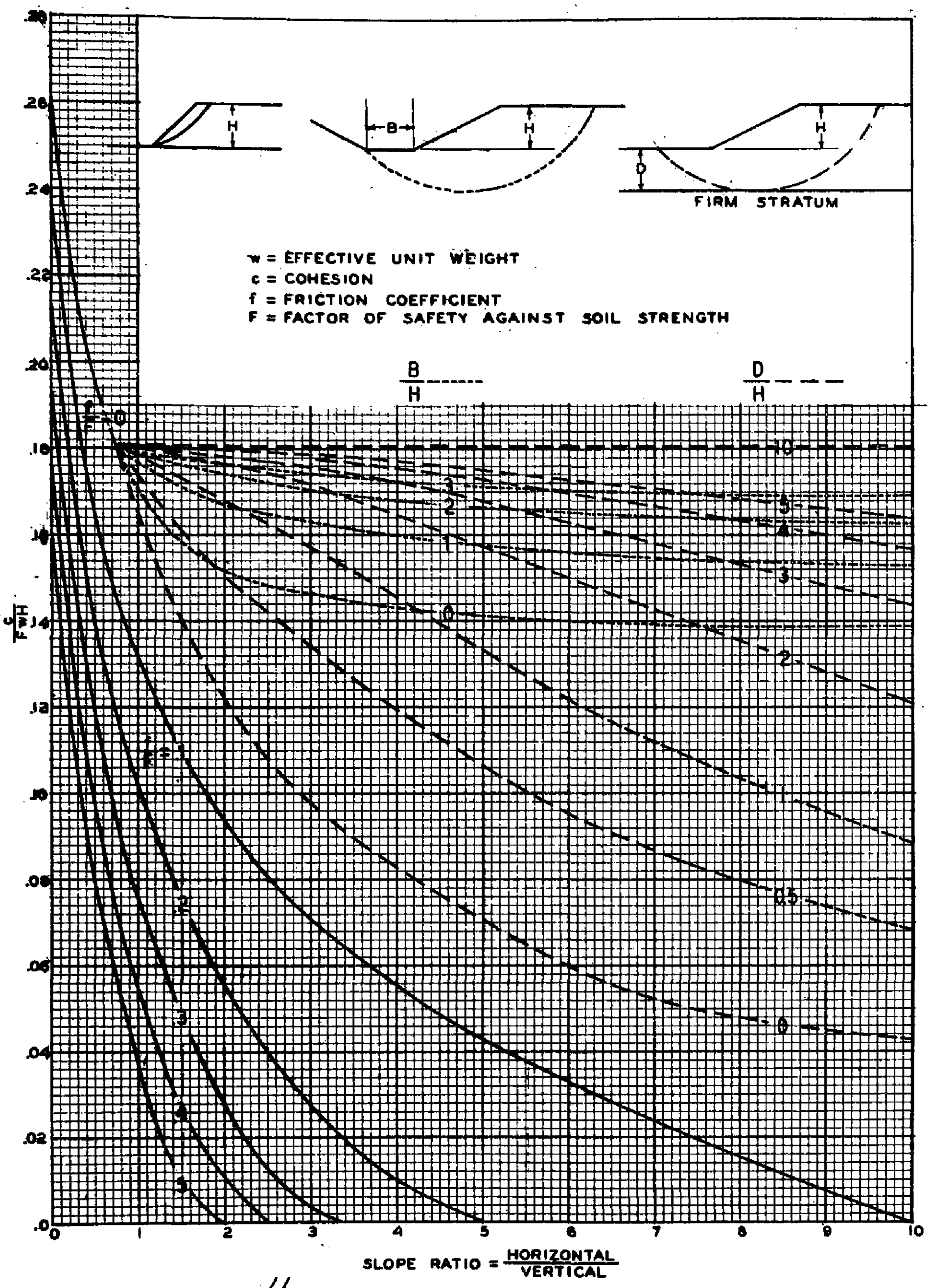


FIGURE ~~8~~ 11 - CHART FOR CALCULATING CRITICAL SLOPES

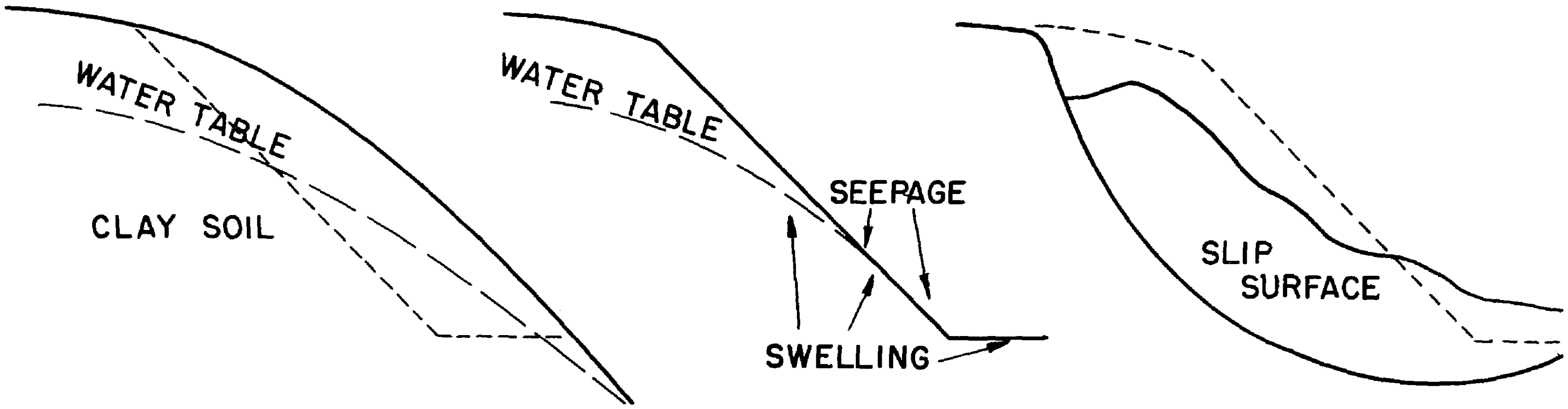


FIG.12-DEVELOPMENT OF SLIP IN CUT IN CLAY

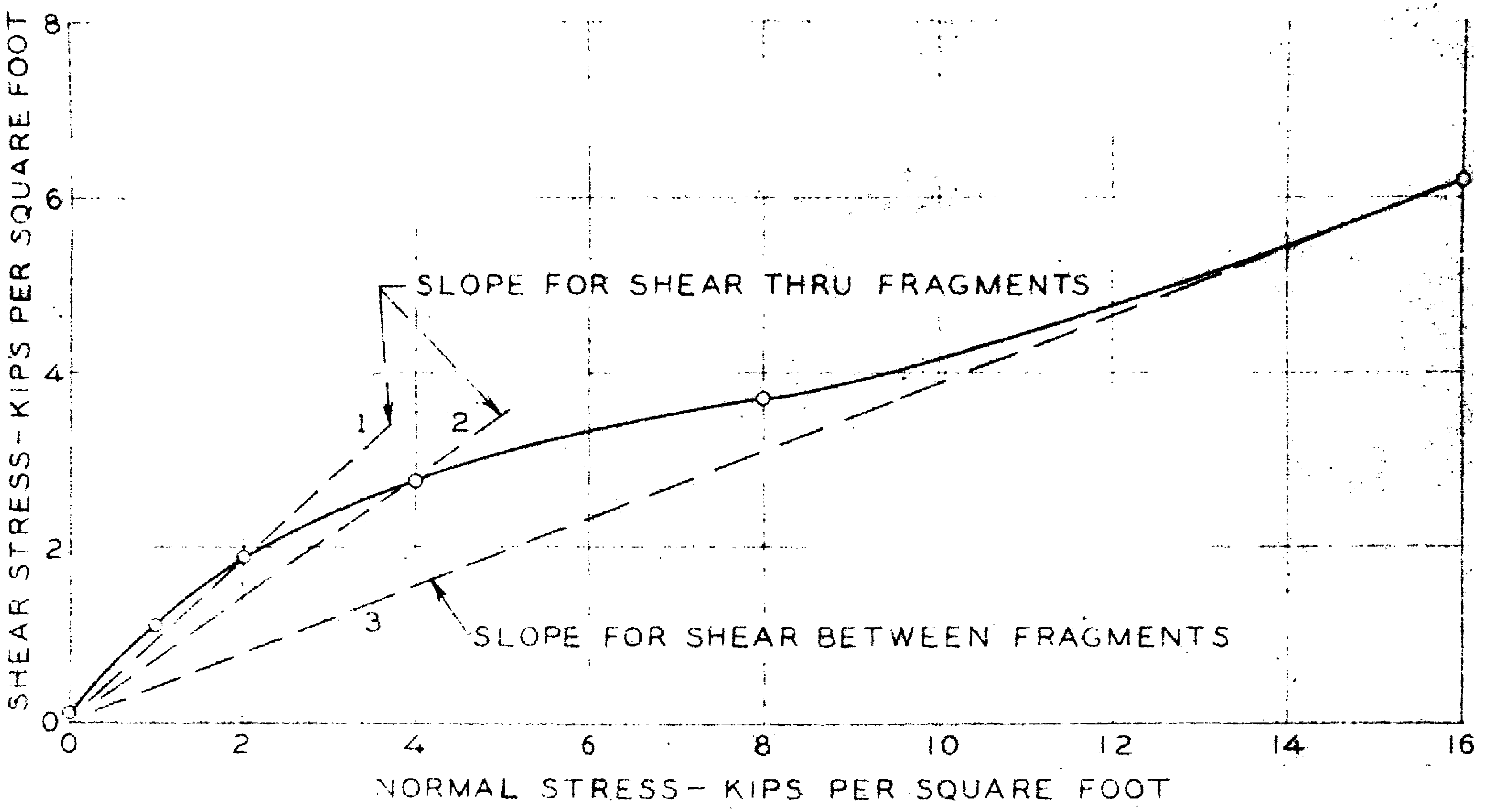
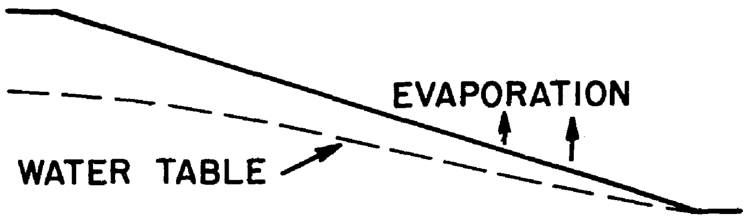


FIGURE 13 - DIRECT SHEAR TEST RESULTS (INUNDATED) ON SAMPLE 70814



FLAT SLOPE

CUT-OFF DRAIN

UNDER-DRAIN

FIG.14-CONTROL OF UNSTABLE SLOPES

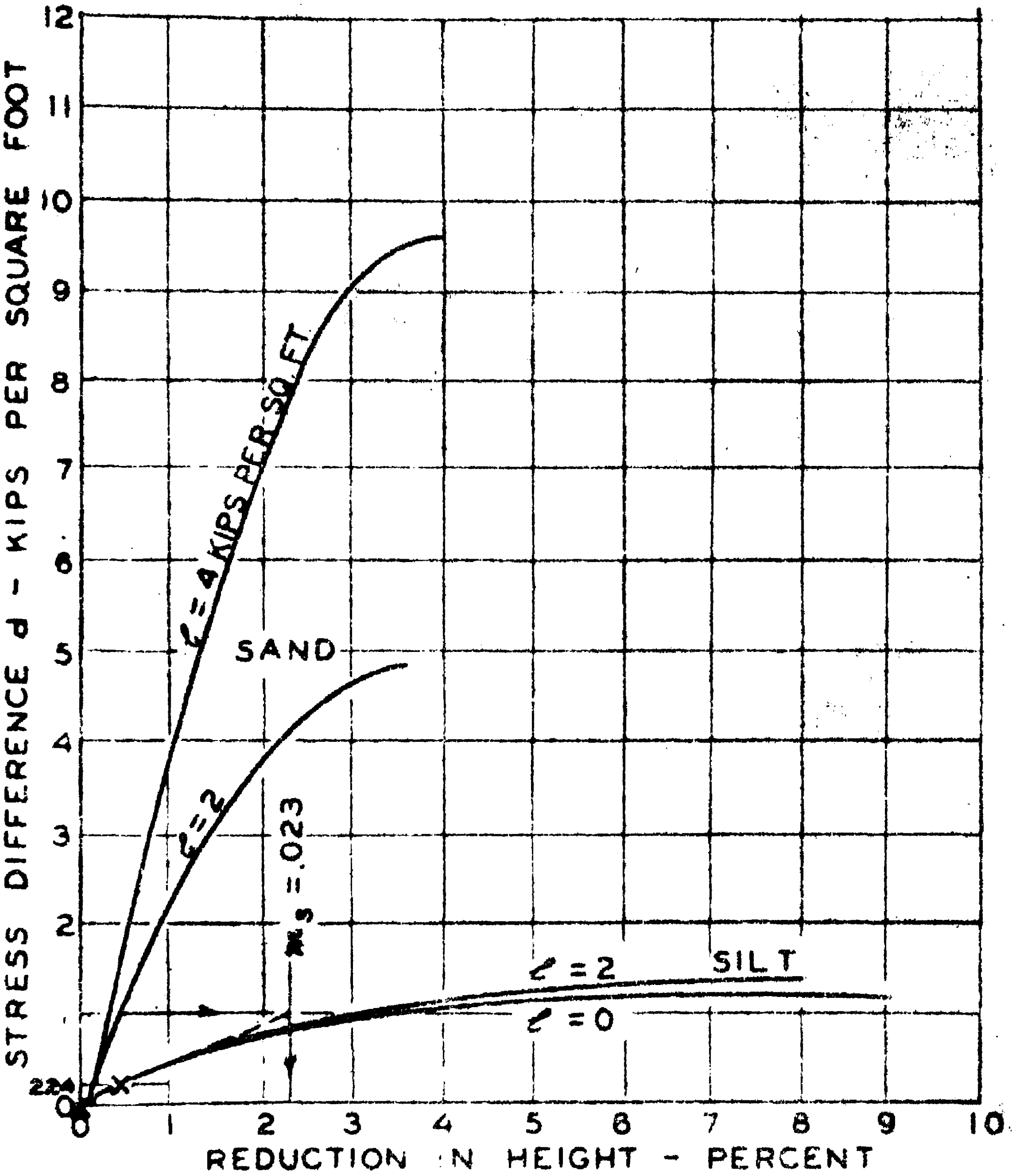


FIGURE 15 - DEFORMATION FROM TRIAXIAL COMPRESSION TEST

P-3491

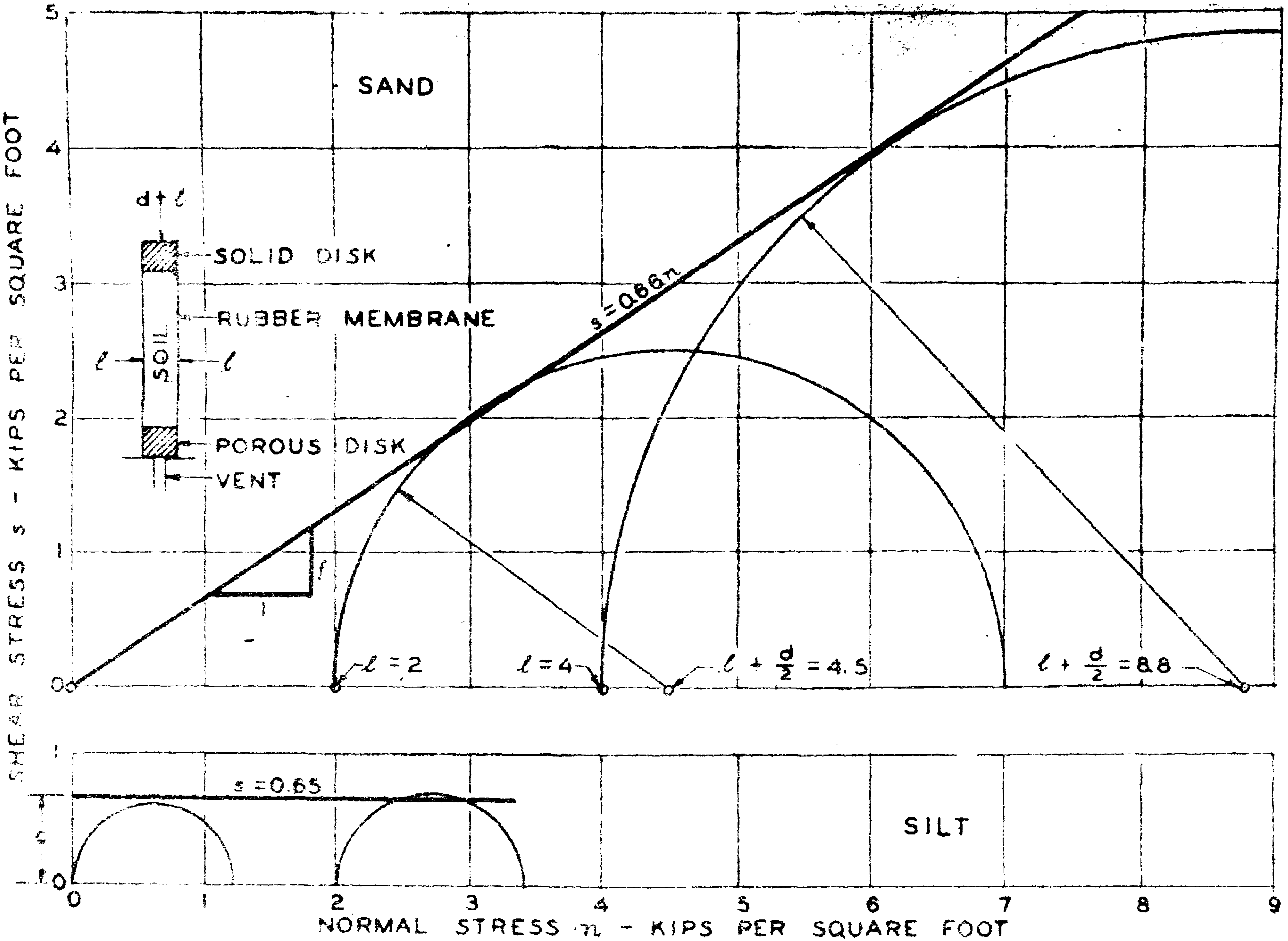


FIGURE 16 - STRENGTH FROM TRIAXIAL COMPRESSION TEST

P-5598

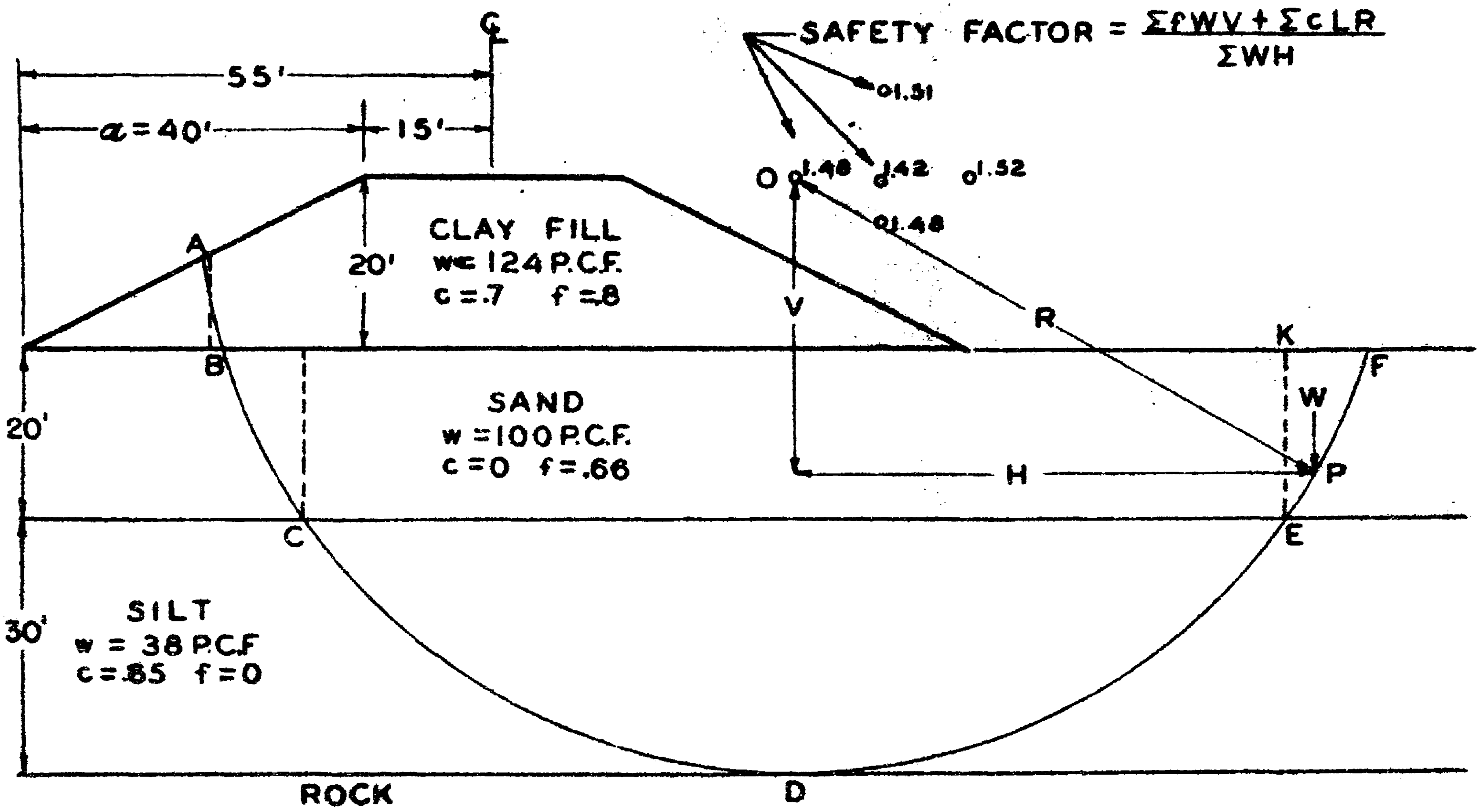


FIGURE 17 - CALCULATION OF RESISTANCE TO SLIDING ALONG CIRCULAR ARC

P J-71

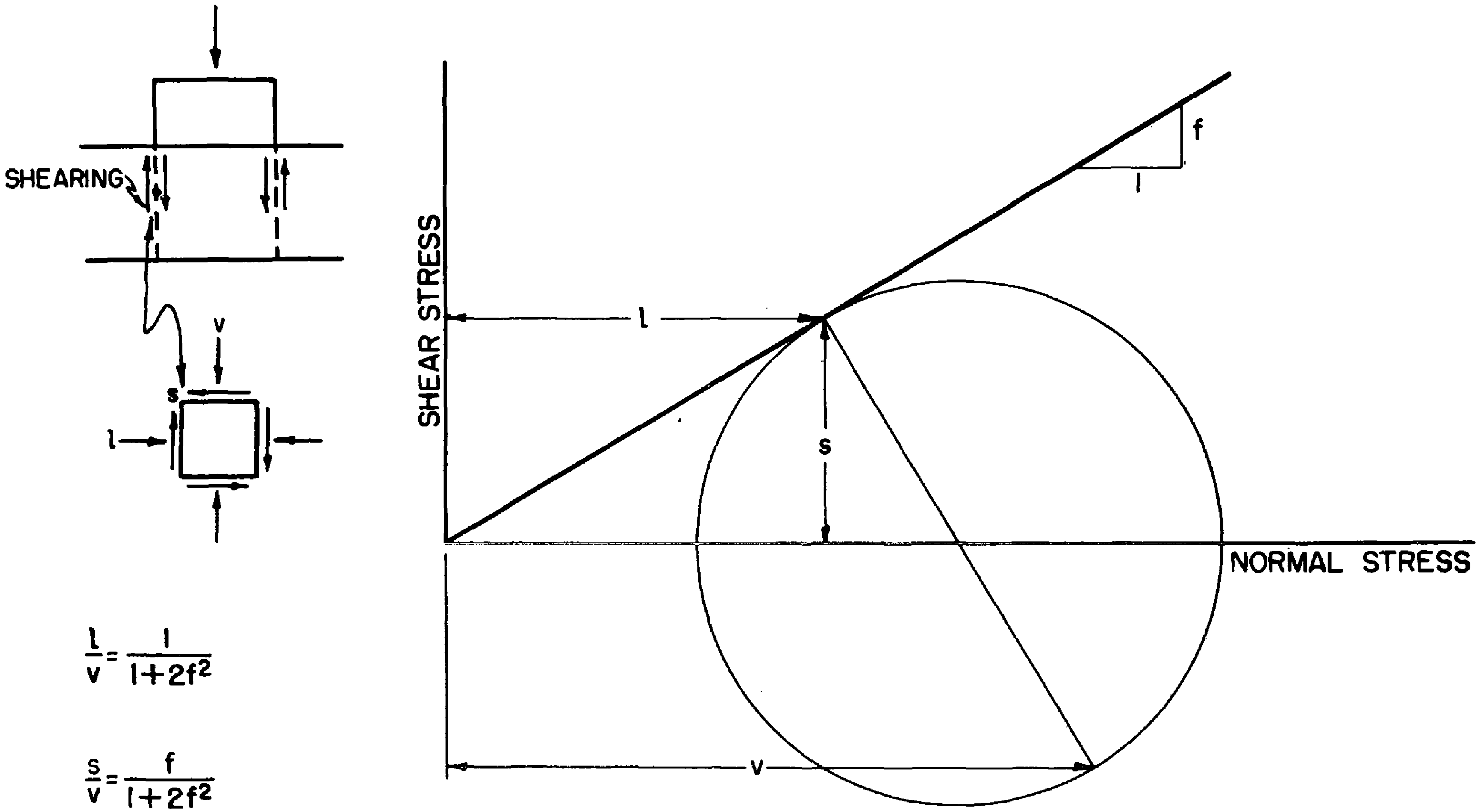
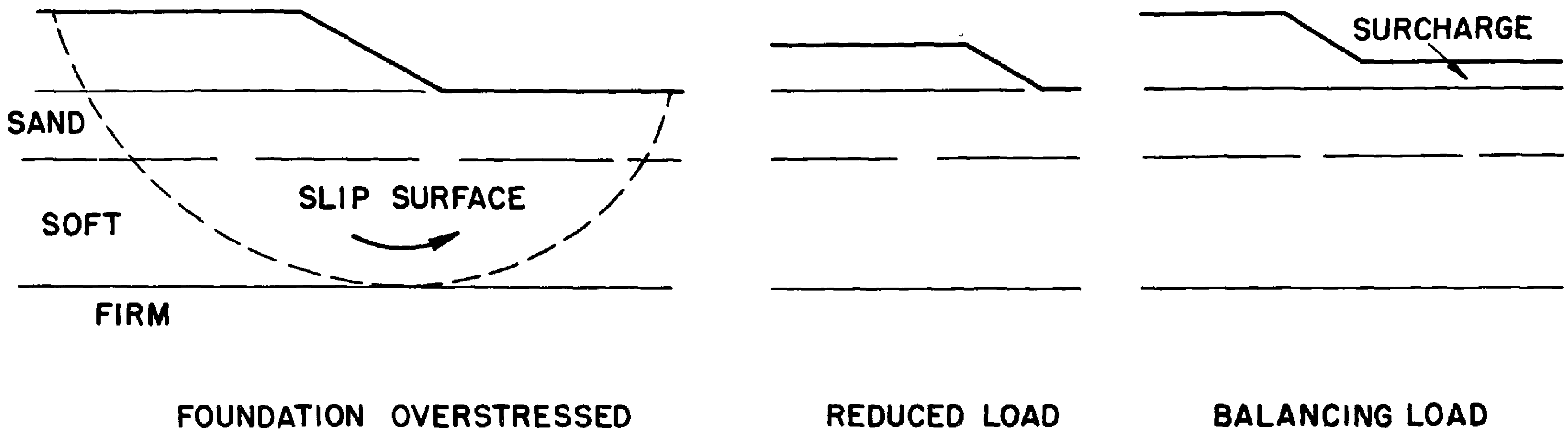


FIG.18-STRESS-RATIO FOR PLANE OF SHEAR



FOUNDATION OVERSTRESSED

REDUCED LOAD

BALANCING LOAD

FIG.19- REDUCING STRESS ON SOFT FOUNDATION

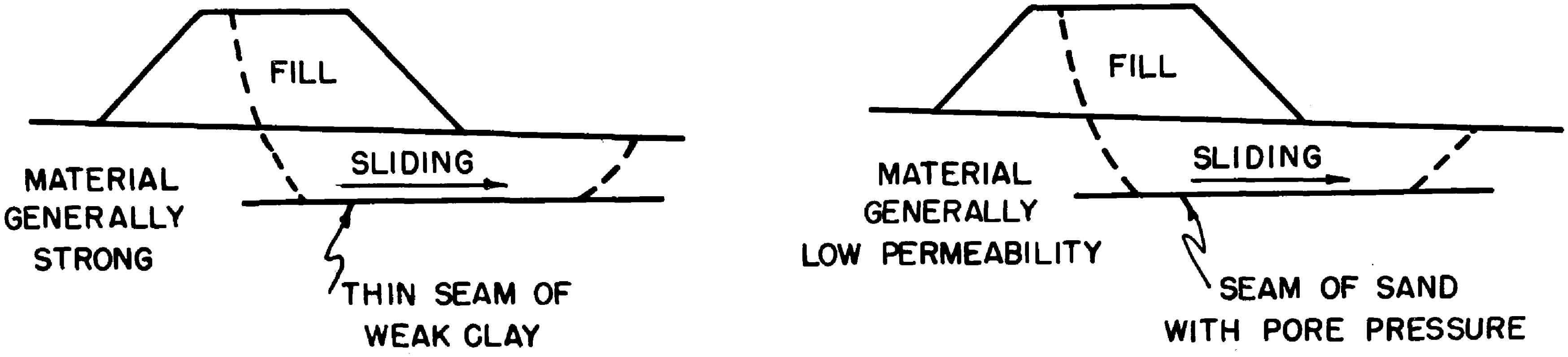
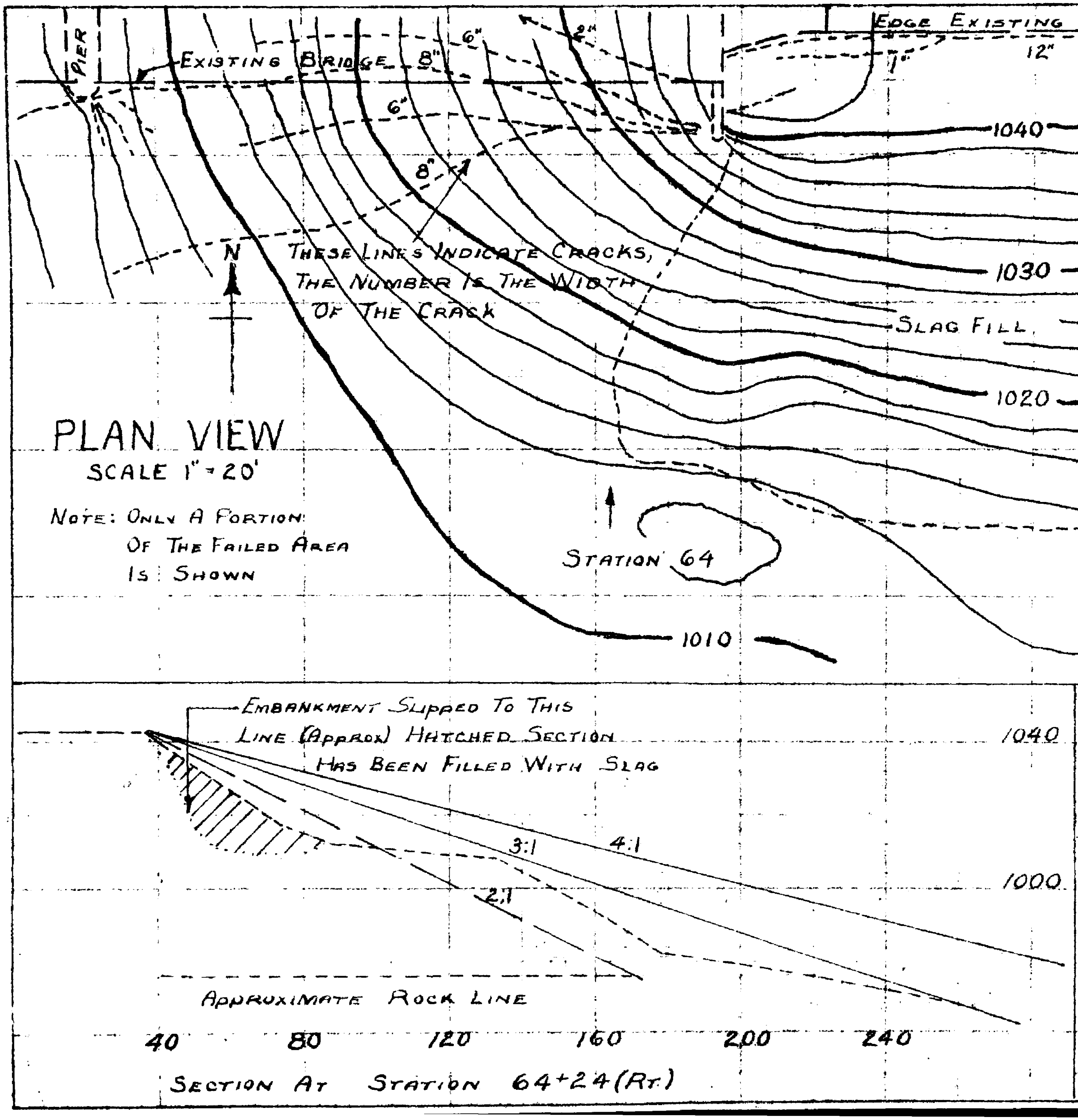


FIG. 20-DANGER OF SLIDING ON THIN SEAMS



SKETCH SHOWING PORTION OF EMBANKMENT FAILURE

PROJECT A1., F.A.P. 330 A (1)
MAHONING CO. OHIO

THIS SKETCH IS APPROXIMATE, IT WAS OBTAINED FROM PLANS FURNISHED BY DISTRICT NO 10, PUBLIC ROADS ADMINISTRATION

FIG. 21

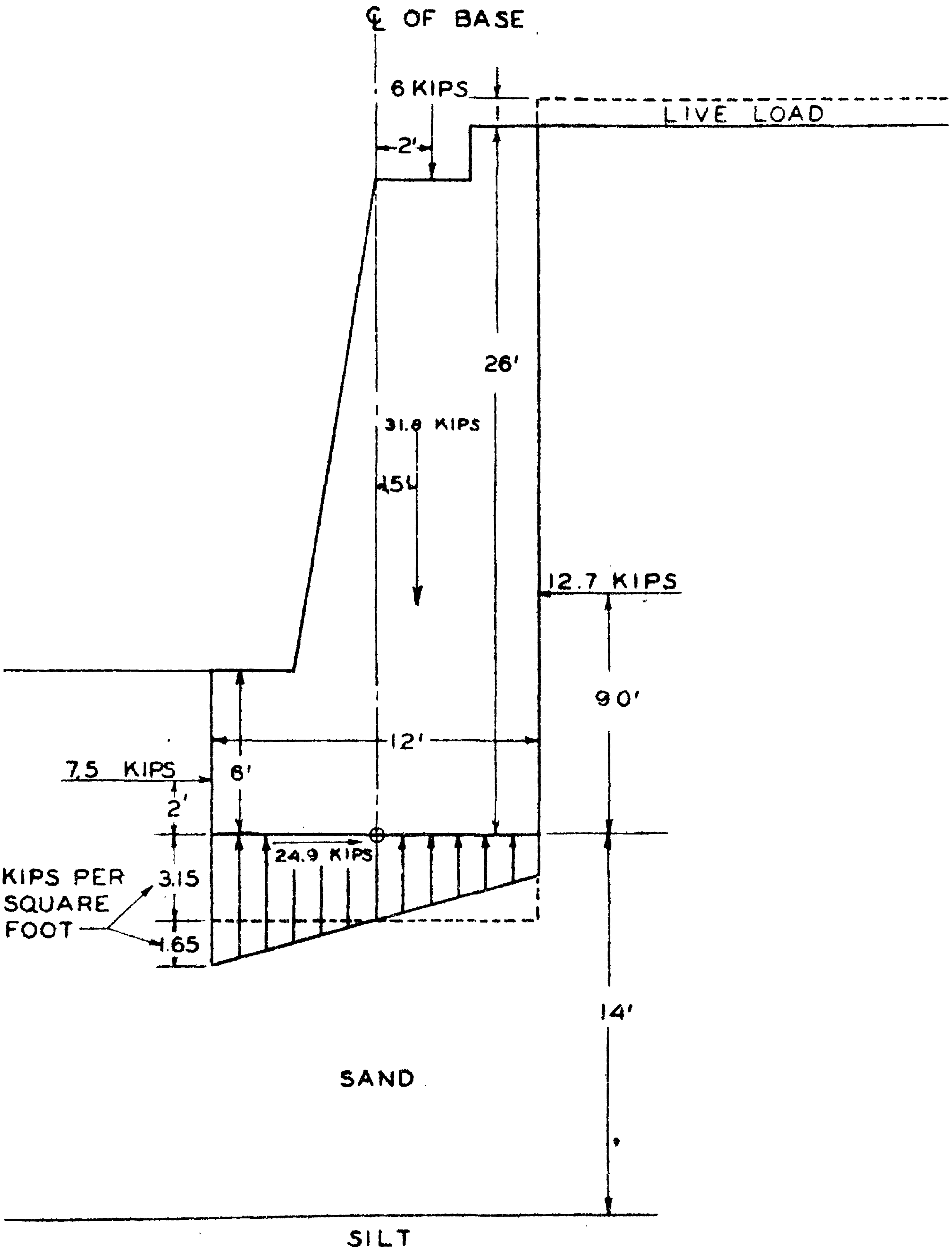


FIGURE 22 - FORCES ON ABUTMENT 30 FEET LONG

P-3491

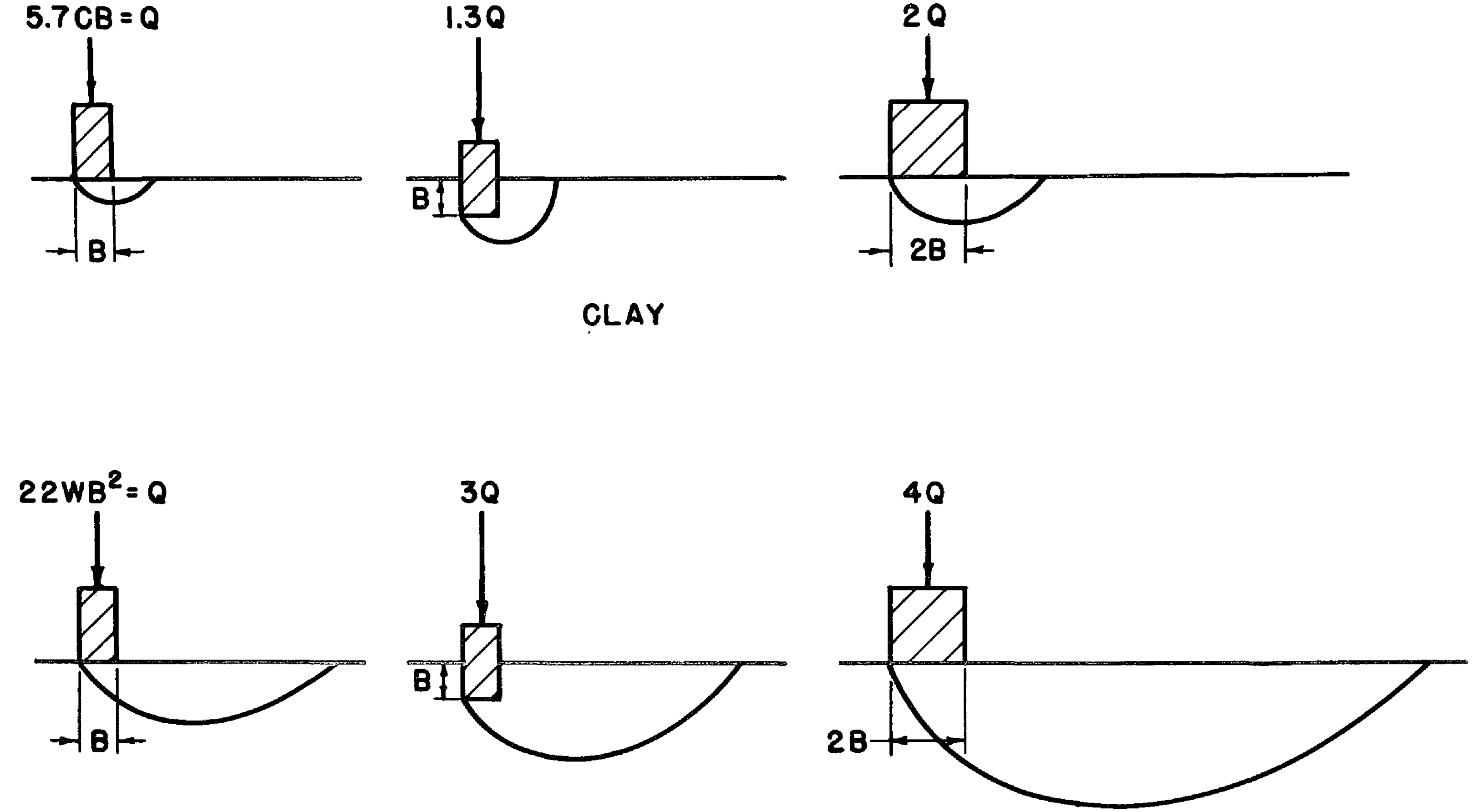
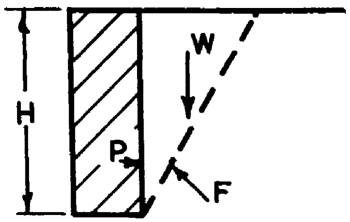


FIG.23 - ULTIMATE BEARING LOAD FOR SAND AND CLAY



$$K = \left(\sqrt{1+f^2} - f\right)^2$$

$$= \tan^2(45 - \phi/2)$$

$$= \frac{1-\sin\phi}{1+\sin\phi}$$

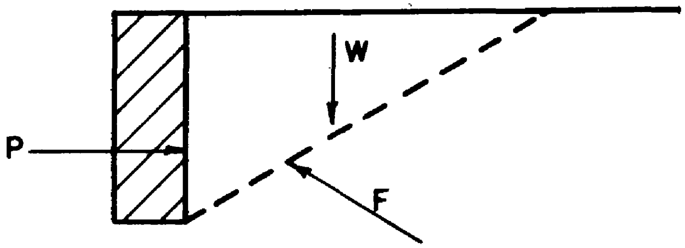
ACTIVE

w = UNIT WEIGHT

ϕ = ANGLE OF INTERNAL FRICTION

$f = \tan\phi$ = COEFFICIENT OF INTERNAL FRICTION

$$P = Kw\frac{H^2}{2}$$



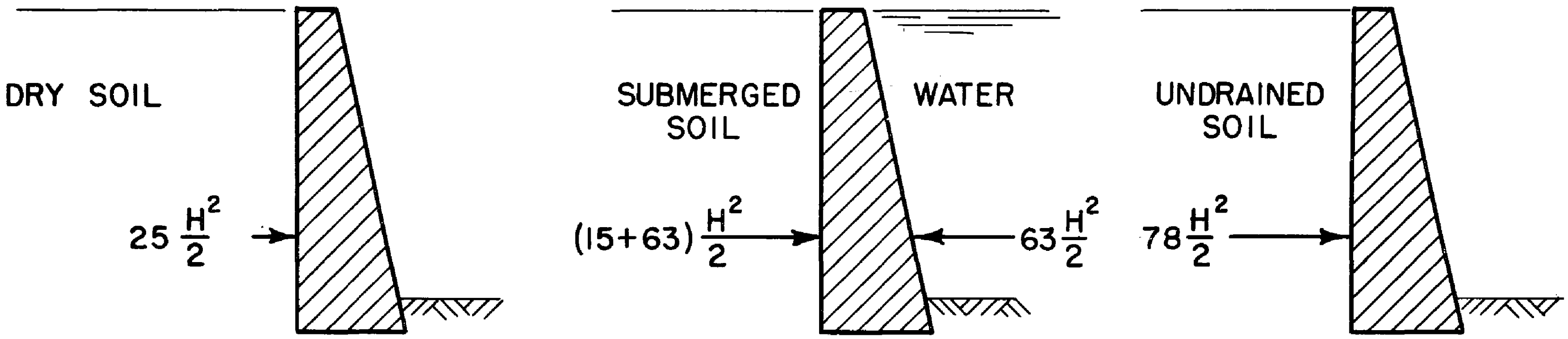
$$K = \left(\sqrt{1+f^2} + f\right)^2$$

$$= \tan^2(45 + \phi/2)$$

$$= \frac{1+\sin\phi}{1-\sin\phi}$$

PASSIVE

FIG. 24- EARTH PRESSURE ON SMOOTH WALL



CONDITIONS:

WET DENSITY	123 P.C.F.
DRY DENSITY	100 P.C.F.
BUOYED DENSITY	60 P.C.F.
PRESSURE COEFFICENT	0.25

FIG. 25- SOIL PLUS WATER PRESSURE ON WALL

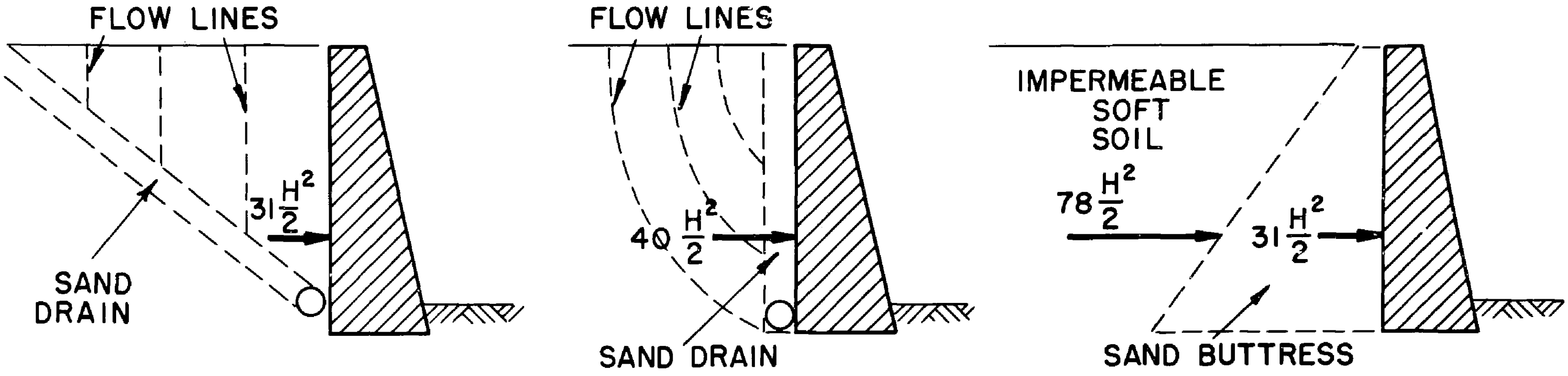


FIG. 26- CONTROL OF PRESSURE ON WALL

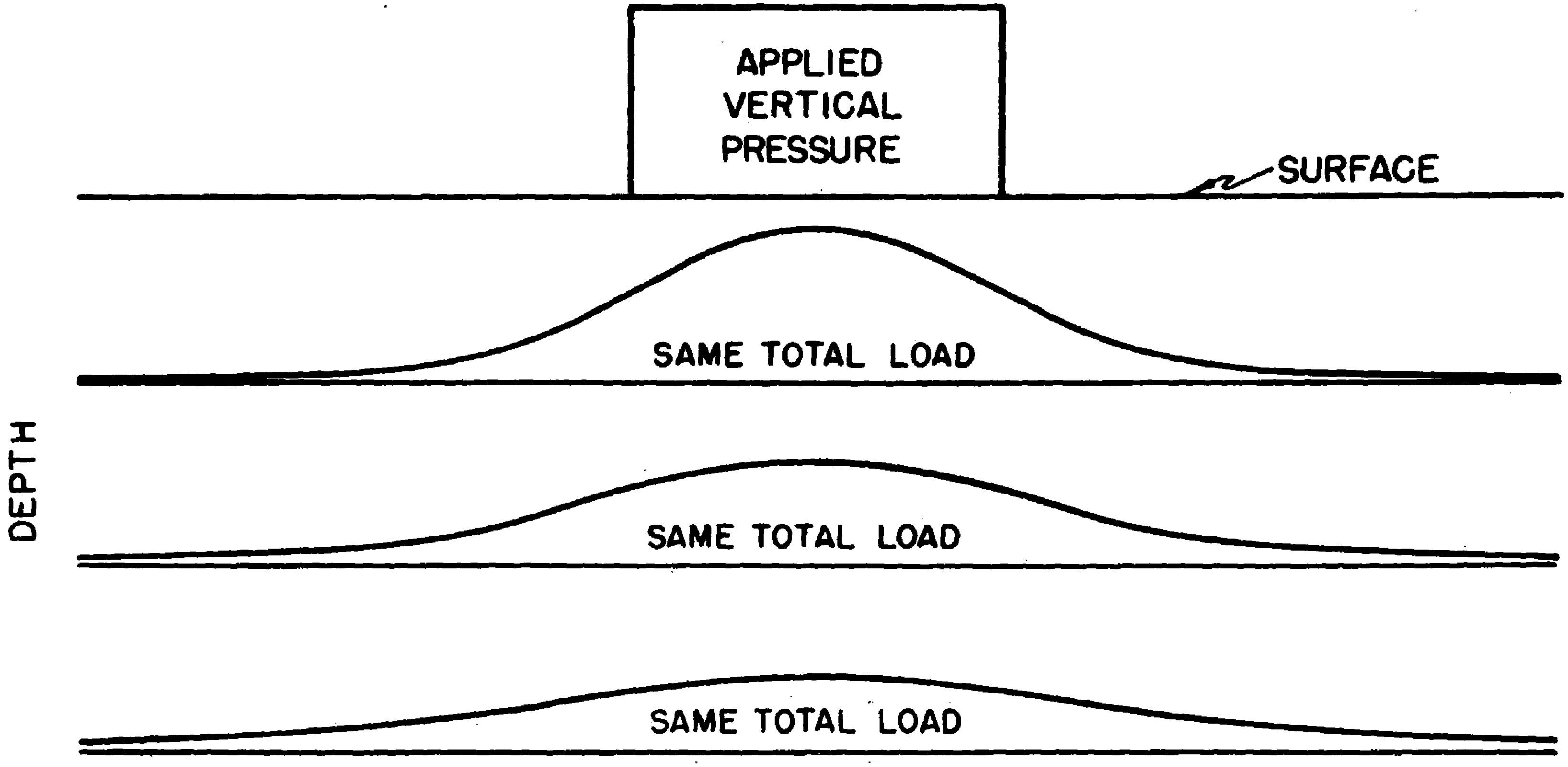
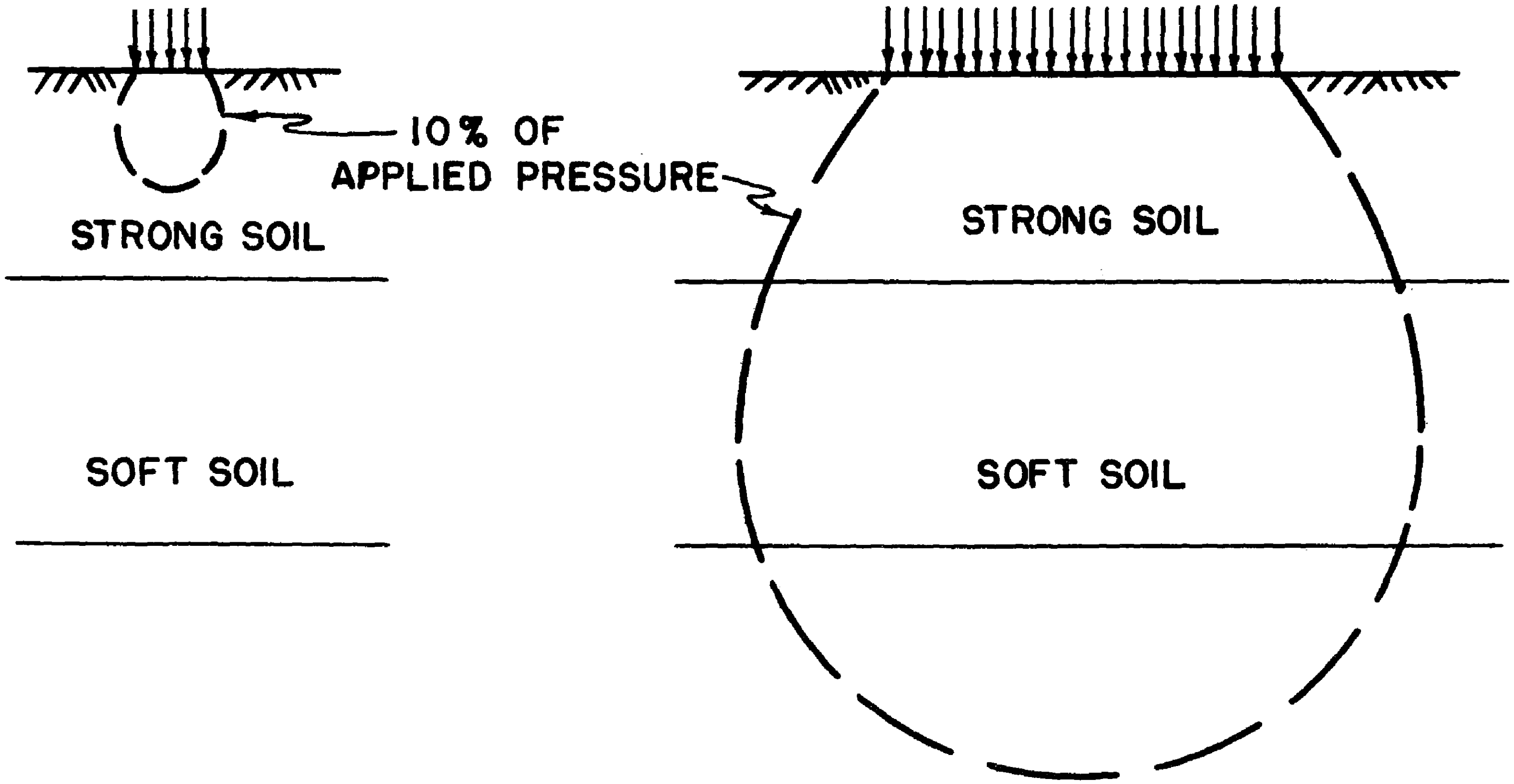
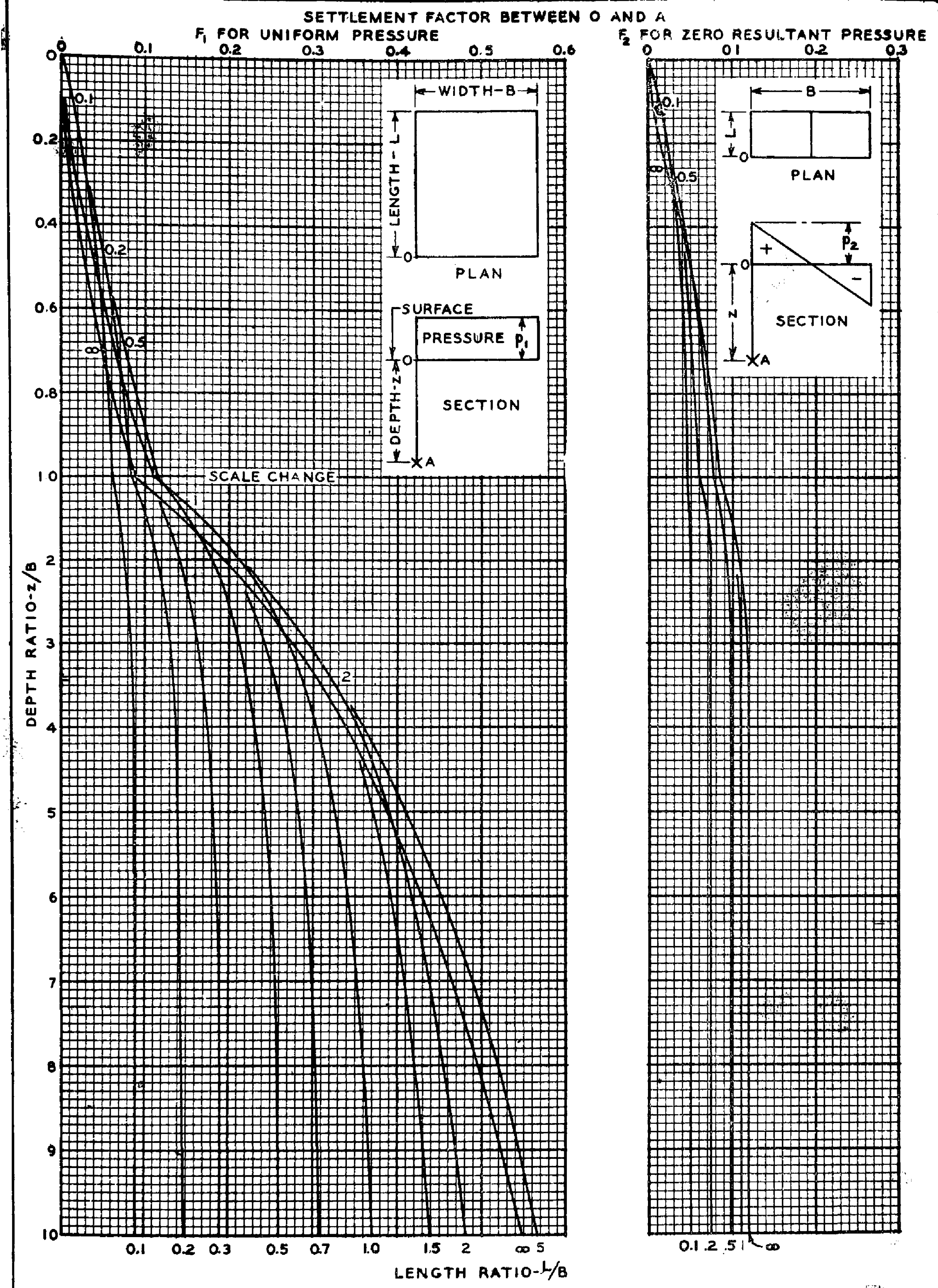


FIG. 27- DISTRIBUTION OF VERTICAL PRESSURE IN SEMI-INFINITE MASS



0.001" EACH DAY FOR 10 YEARS BECOMES 3.6"

FIG. 28-PROBLEM OF SMALL SCALE BEARING TEST



SETTLEMENT $S = m_s BF$ m_s = STRAIN PER UNIT STRESS DIFFERENCE FROM COMPRESSION TEST CORRESPONDING TO AVERAGE STRESS, $\frac{pF}{z/B}$.

FIGURE ~~14~~ 29 GRAPH OF SETTLEMENT UNDER CORNER OF LOAD ON RECTANGULAR AREA.
POISSON'S RATIO = 0.5

P-3491

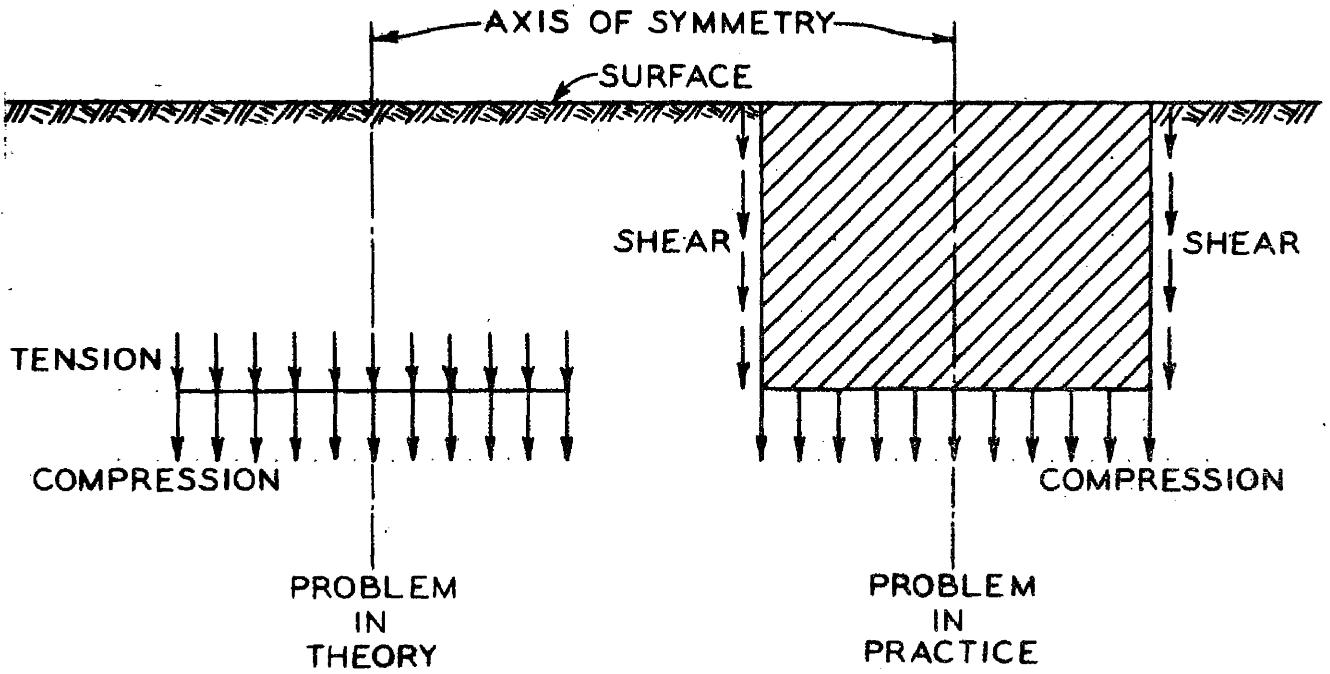


FIGURE 30- STRESS FROM LOADED AREA BELOW THE SURFACE.

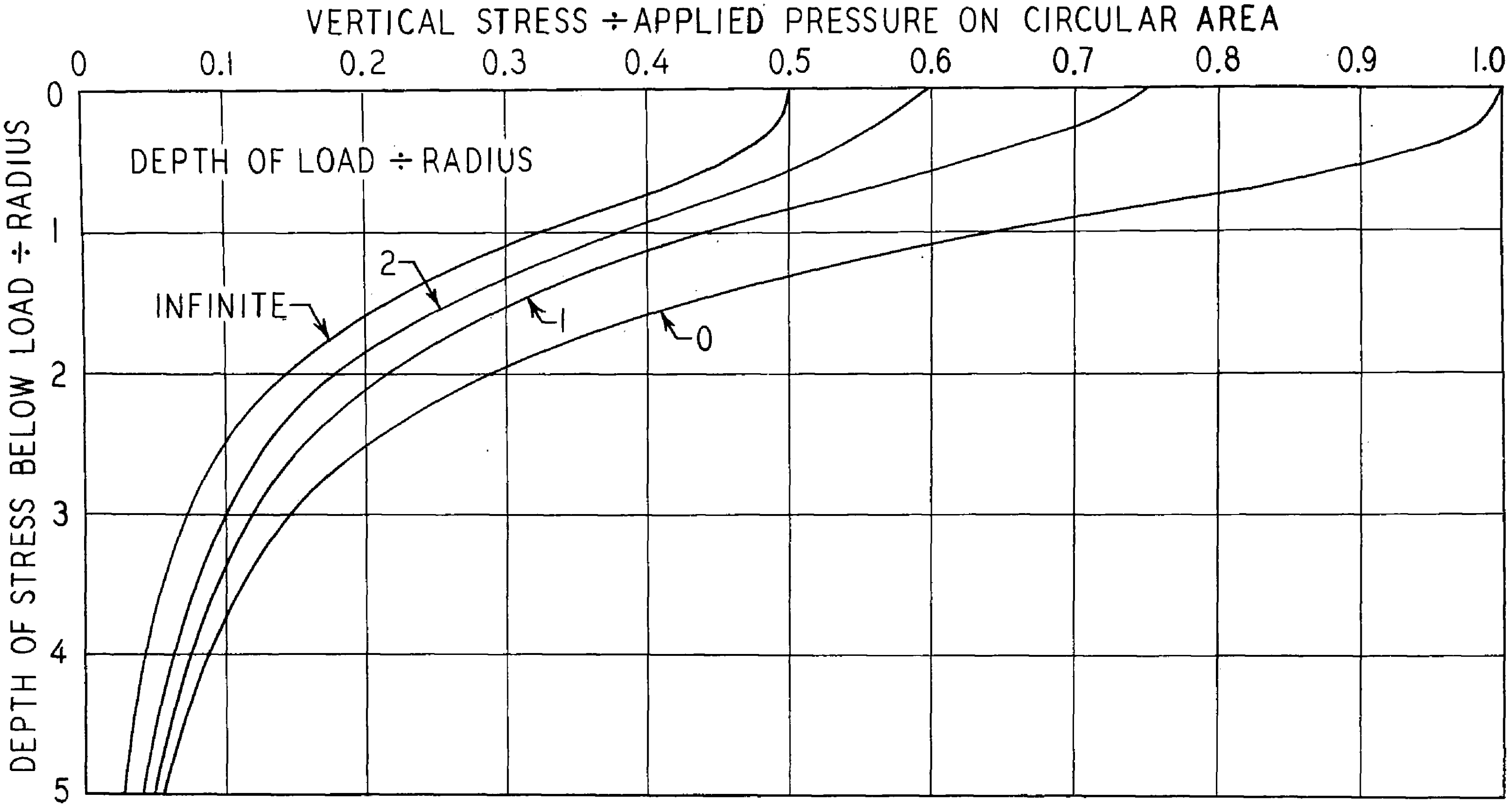


FIGURE ~~4~~ 3 - AXIAL VERTICAL STRESS FROM UNIFORM PRESSURE ON CIRCULAR AREA BELOW THE SURFACE (POISSON'S RATIO = 0.5)

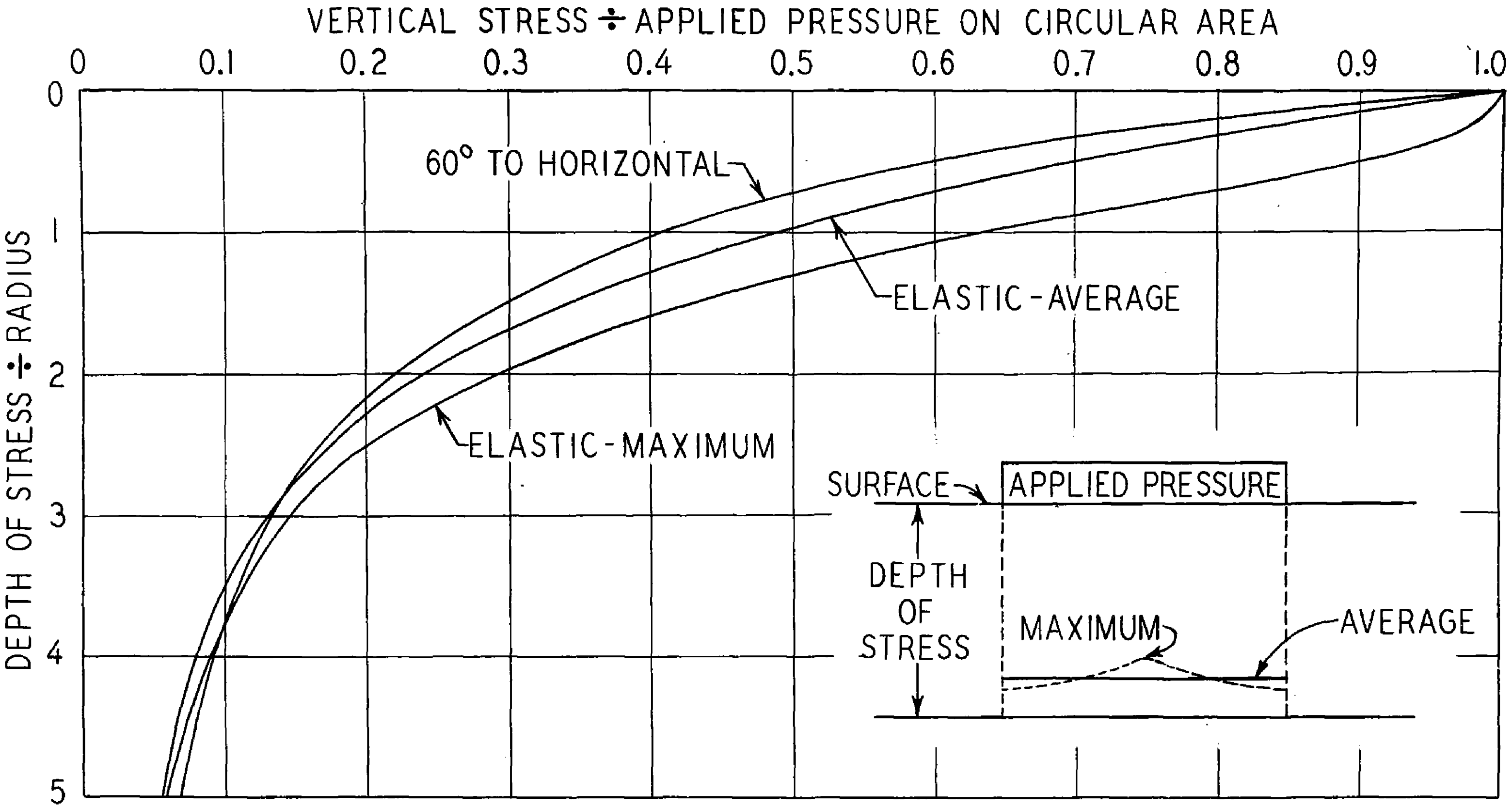
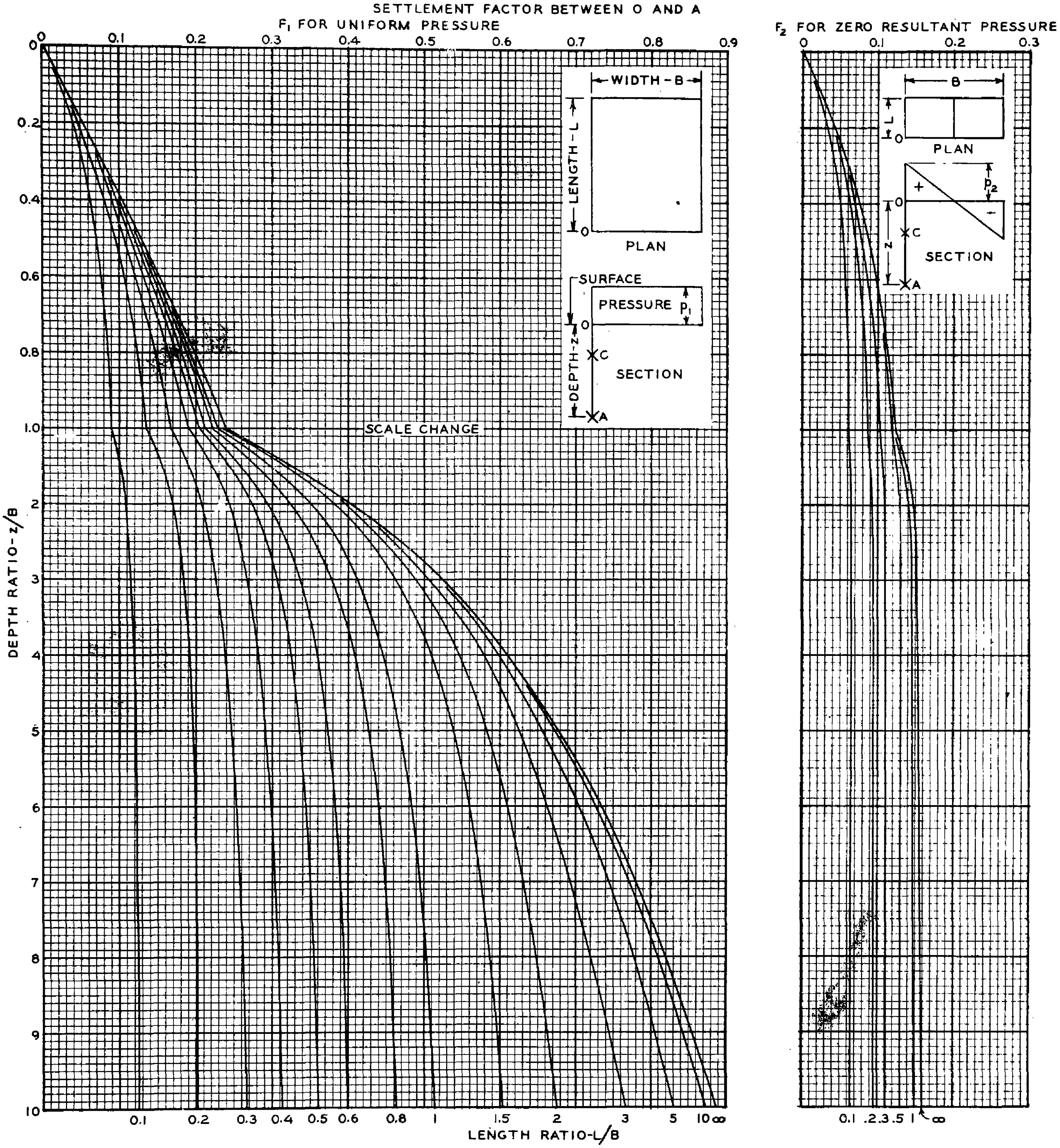


FIGURE 32 - VERTICAL STRESS BELOW UNIFORMLY LOADED CIRCULAR AREA.



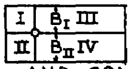
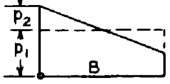
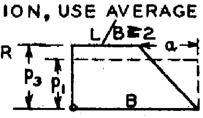
SETTLEMENT, $S = m_v p BF$. m_v-STRAIN PER UNIT PRESSURE INCREASE FROM CONSOLIDATION TEST CORRESPONDING TO AVERAGE PRESSURE, $\frac{pF}{z/B}$. VERTICAL PRESSURE AT A POINT IS $p \times$ SLOPE OF CURVE $= p\frac{dF}{d(z/B)}$. FOR LAYER BETWEEN C AND A, USE $F = F_A - F_C$. FOR A POINT NOT BELOW CORNER, USE COMBINED AREAS THUS: FOR , USE $BF = B_I(F_I + F_{III}) + B_{II}(F_{II} + F_{IV})$. FOR TRAPEZOIDAL PRESSURE DISTRIBUTION, USE AVERAGE PRESSURE, p, AND COMBINE F_1 AND F_2, THUS: FOR  USE $F = F_1 + p_2/p_1\, F_2$ AND $p = p_1$; FOR , $F = F_1 + \frac{a}{B}F_2$ AND $p = p_1 = p_3\left(1 - \frac{a}{2B}\right)$.

FIGURE 33 GRAPH OF SETTLEMENT UNDER CORNER OF LOAD ON RECTANGULAR AREA-POISSONS RATIO = 0

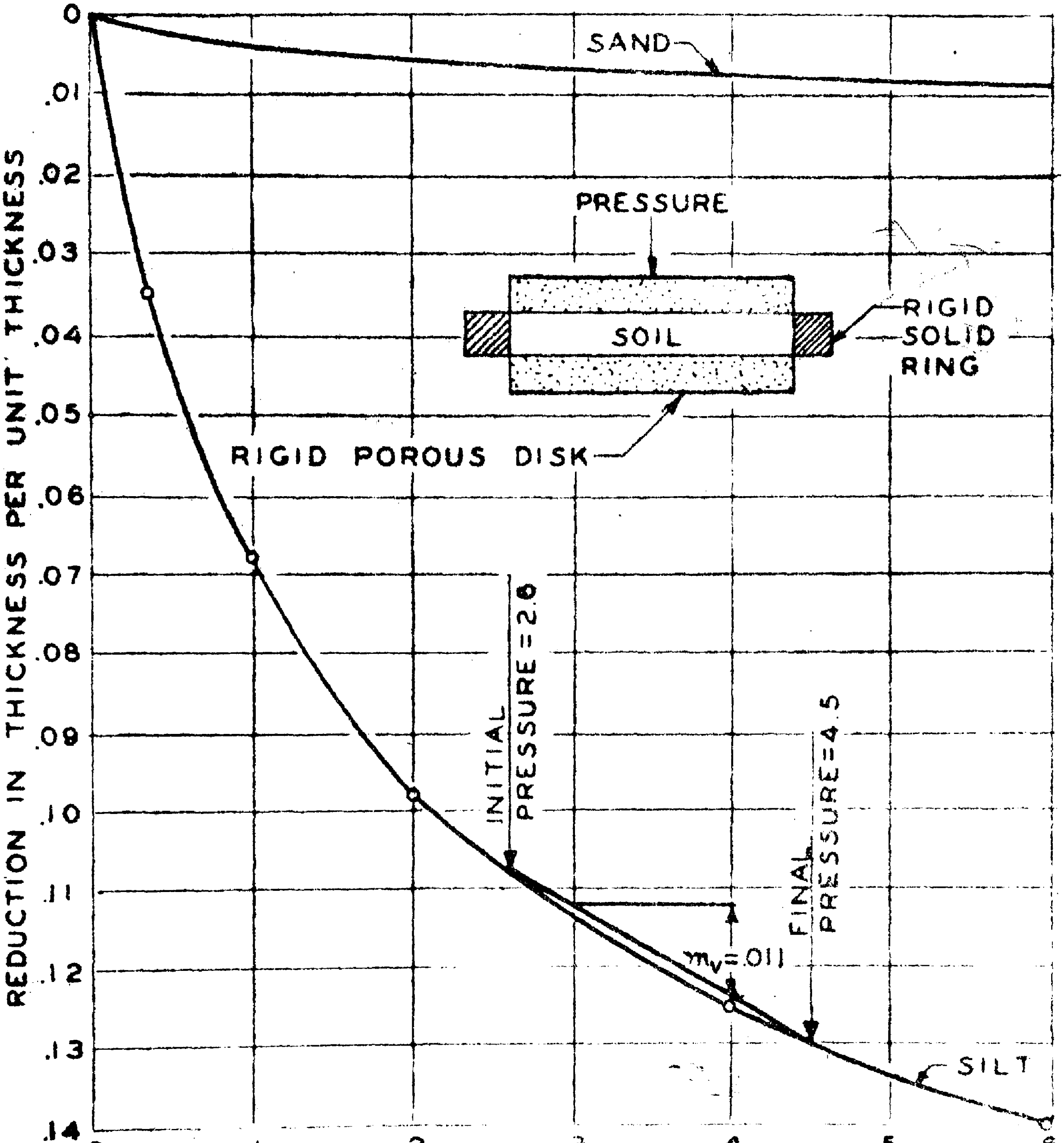


FIGURE 34 - VOLUME CHANGE FROM CONSOLIDATION TEST

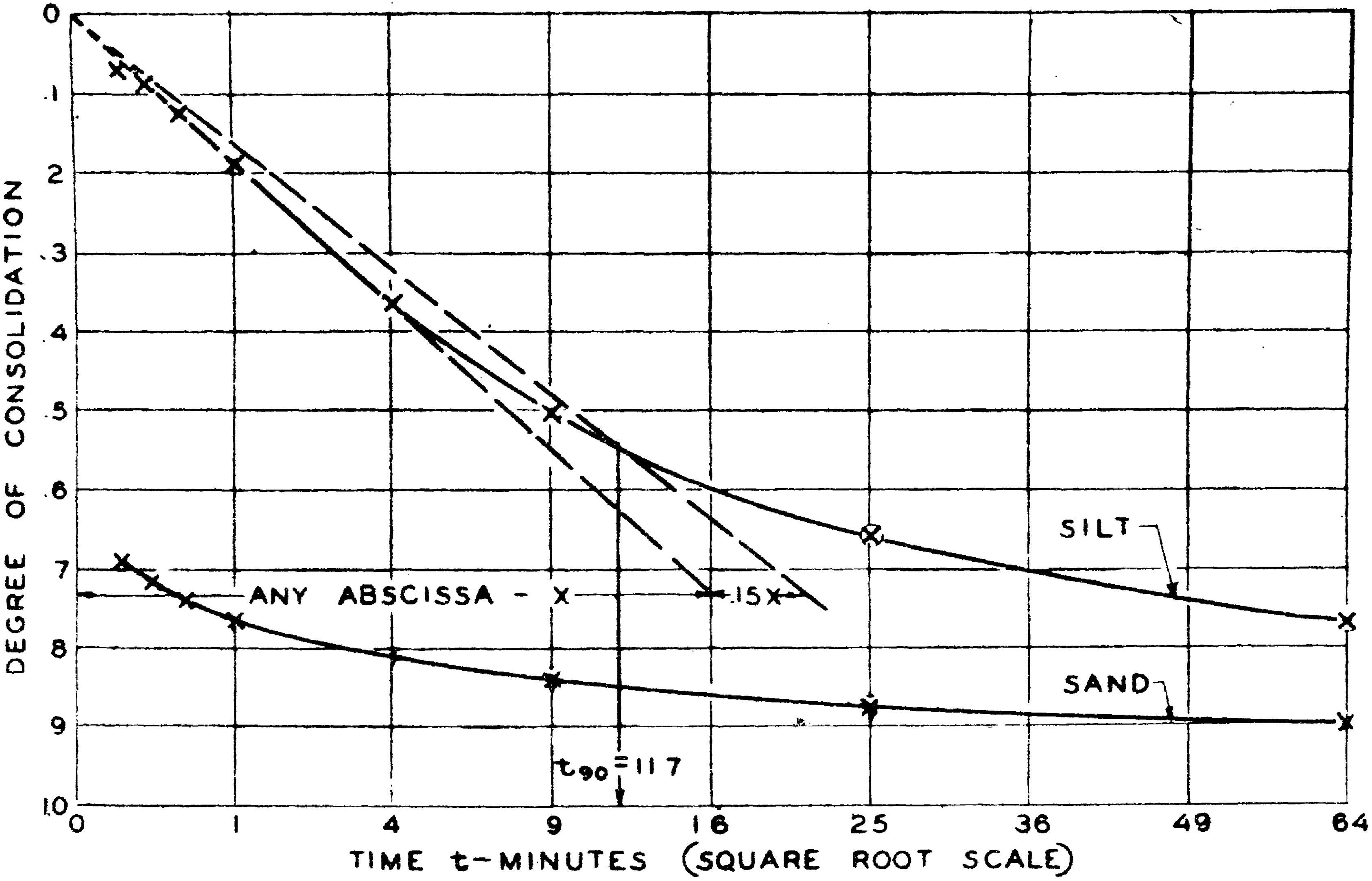


FIGURE 35 -RATE OF VOLUME CHANGE FROM CONSOLIDATION TEST

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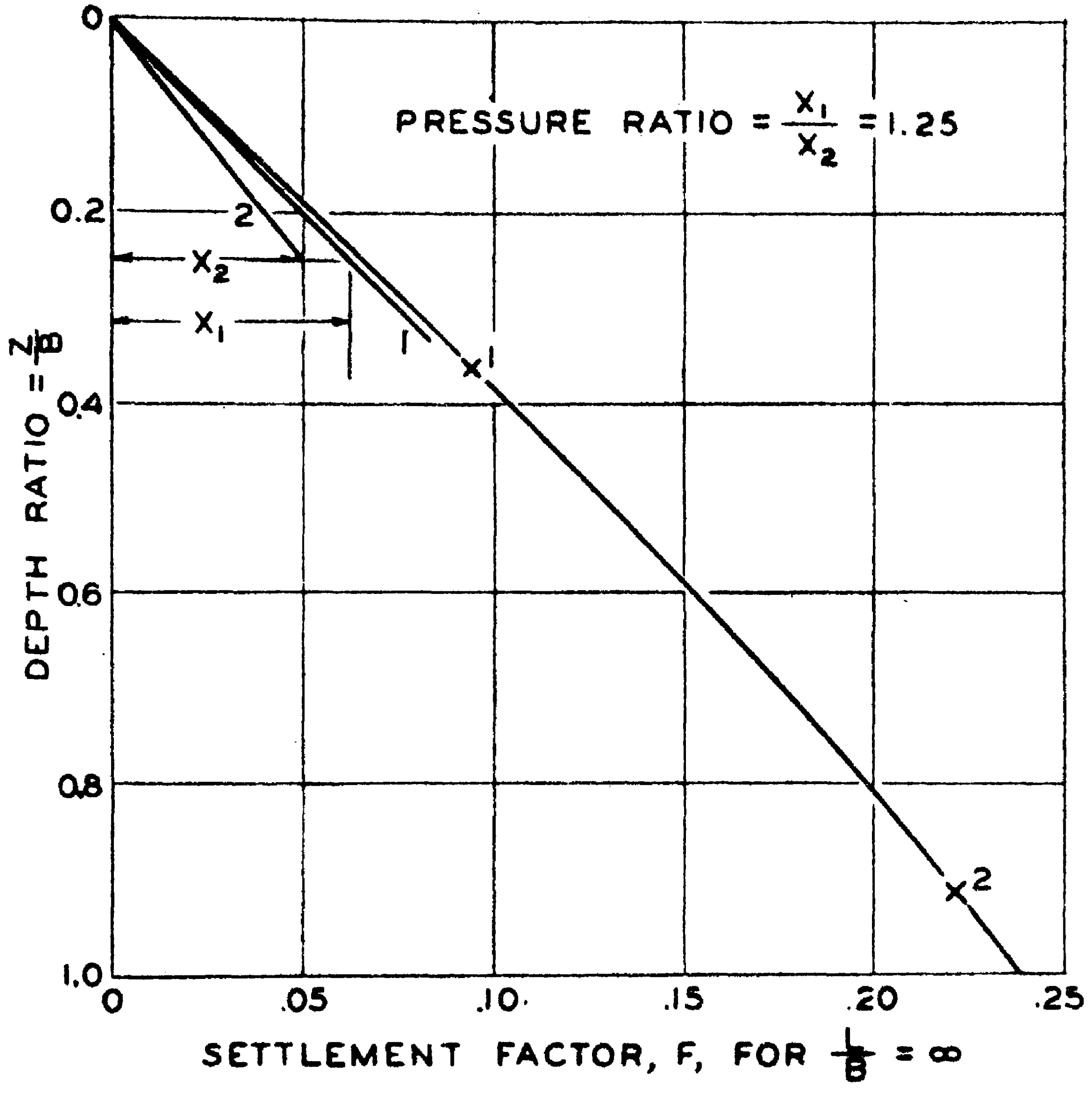
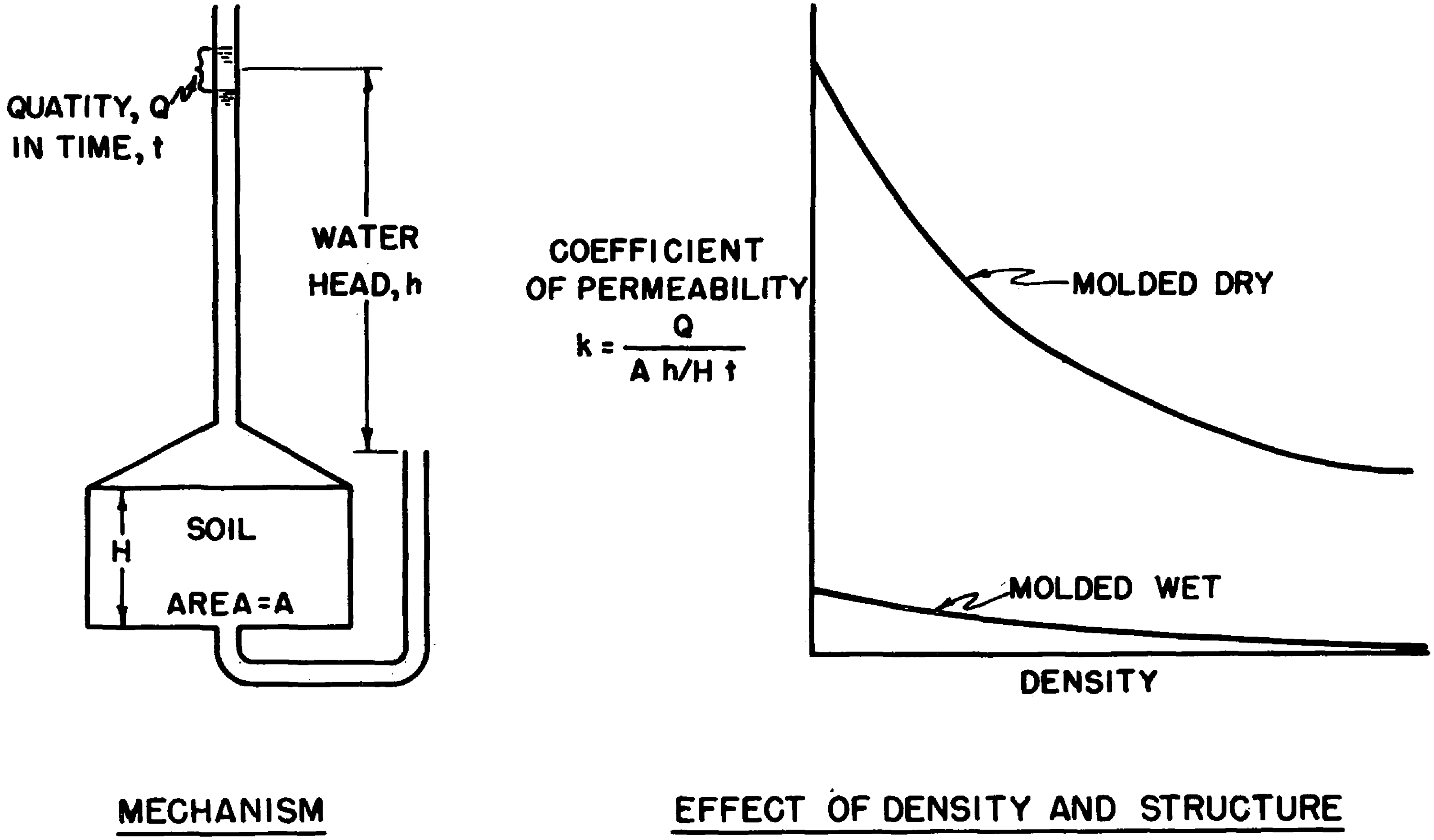


FIGURE 36 - CALCULATION OF PRESSURE RATIO FROM SETTLEMENT FACTOR CURVE

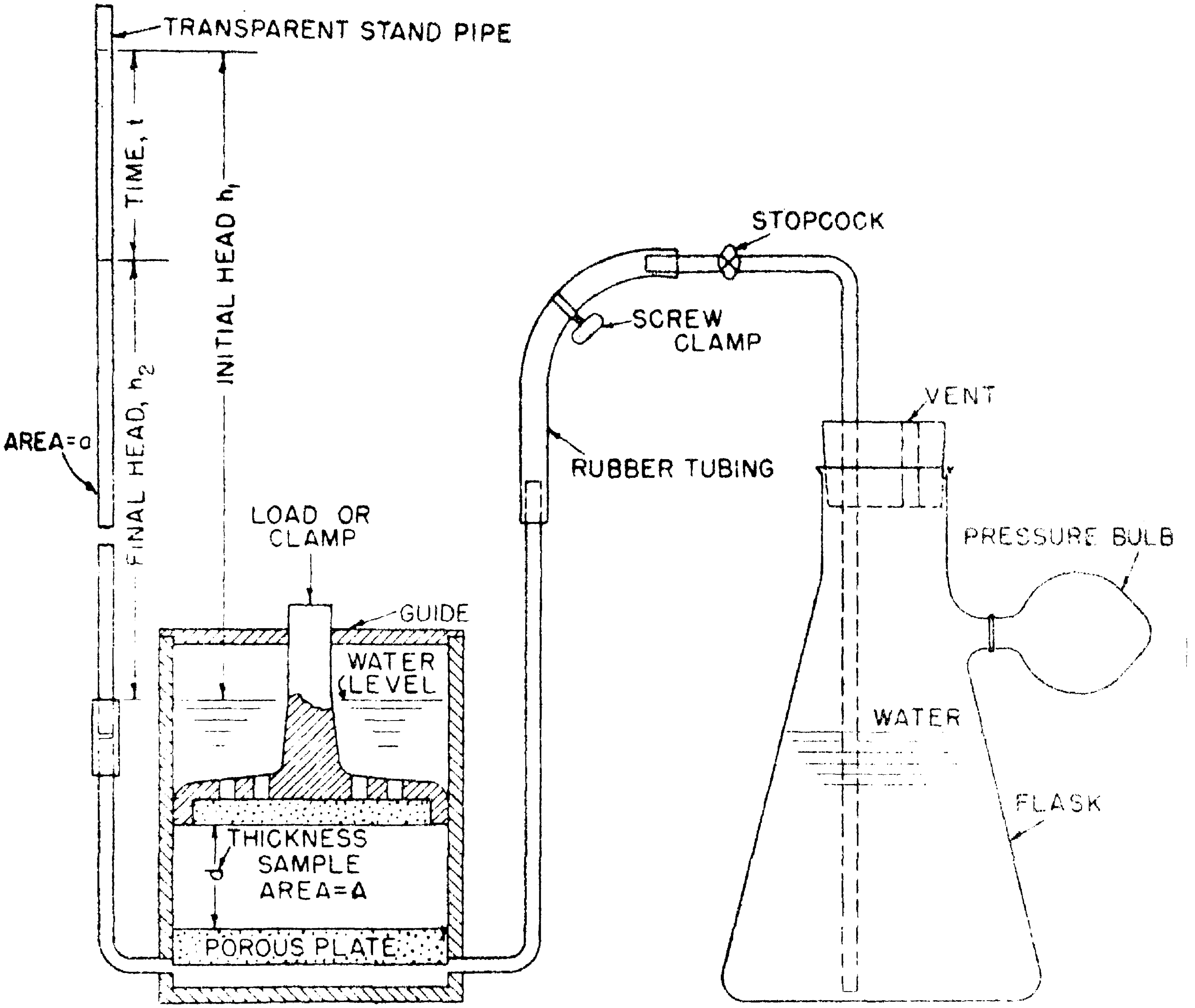
P-3491



MECHANISM

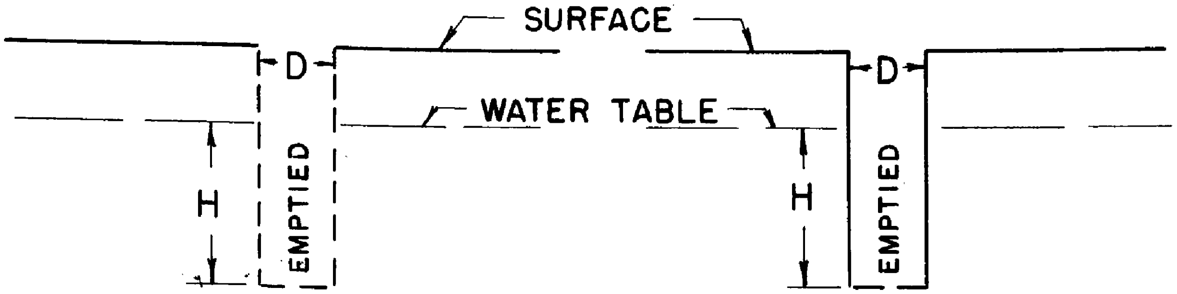
EFFECT OF DENSITY AND STRUCTURE

FIG.37-WATER PERMEABILITY TEST



COEFFICIENT OF PERMEABILITY, $k = \frac{2.3ad}{At} \text{LOG} \frac{h_1}{h_2}$

FIGURE 38 - APPARATUS FOR MEASURING PERMEABILITY BY MEANS OF A FALLING HEAD



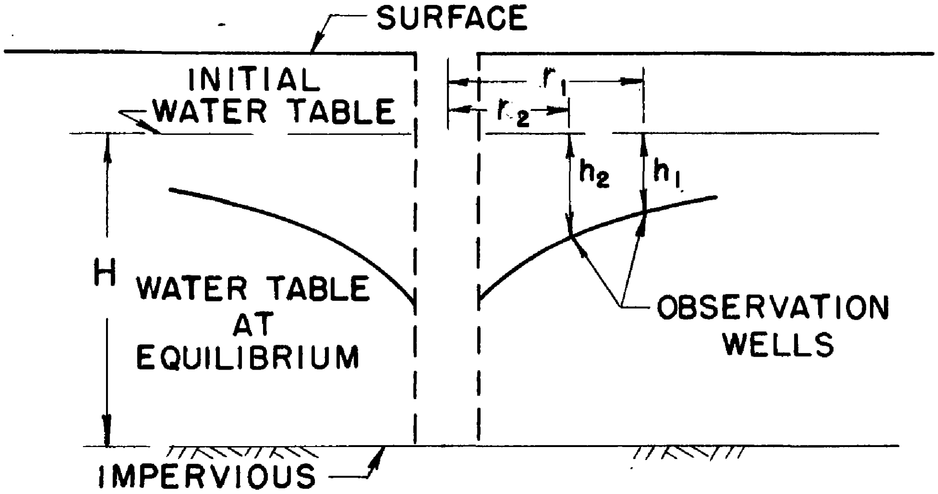
$$k = \frac{0.1Q}{DH}$$

$$k = \frac{.37Q}{DH}$$

Q = QUANTITY PER UNIT TIME
k = COEFFICIENT OF PERMEABILITY

OPEN HOLE

CASED HOLE



$$k = \frac{.36\ Q\ \mathrm{LOG}\ \frac{r_1}{r_2}}{H\ (h_2 - h_1)}$$

PUMPED WELL

FIGURE 39 – DETERMINATION OF COEFFICIENT OF PERMEABILITY IN FIELD BELOW WATER TABLE

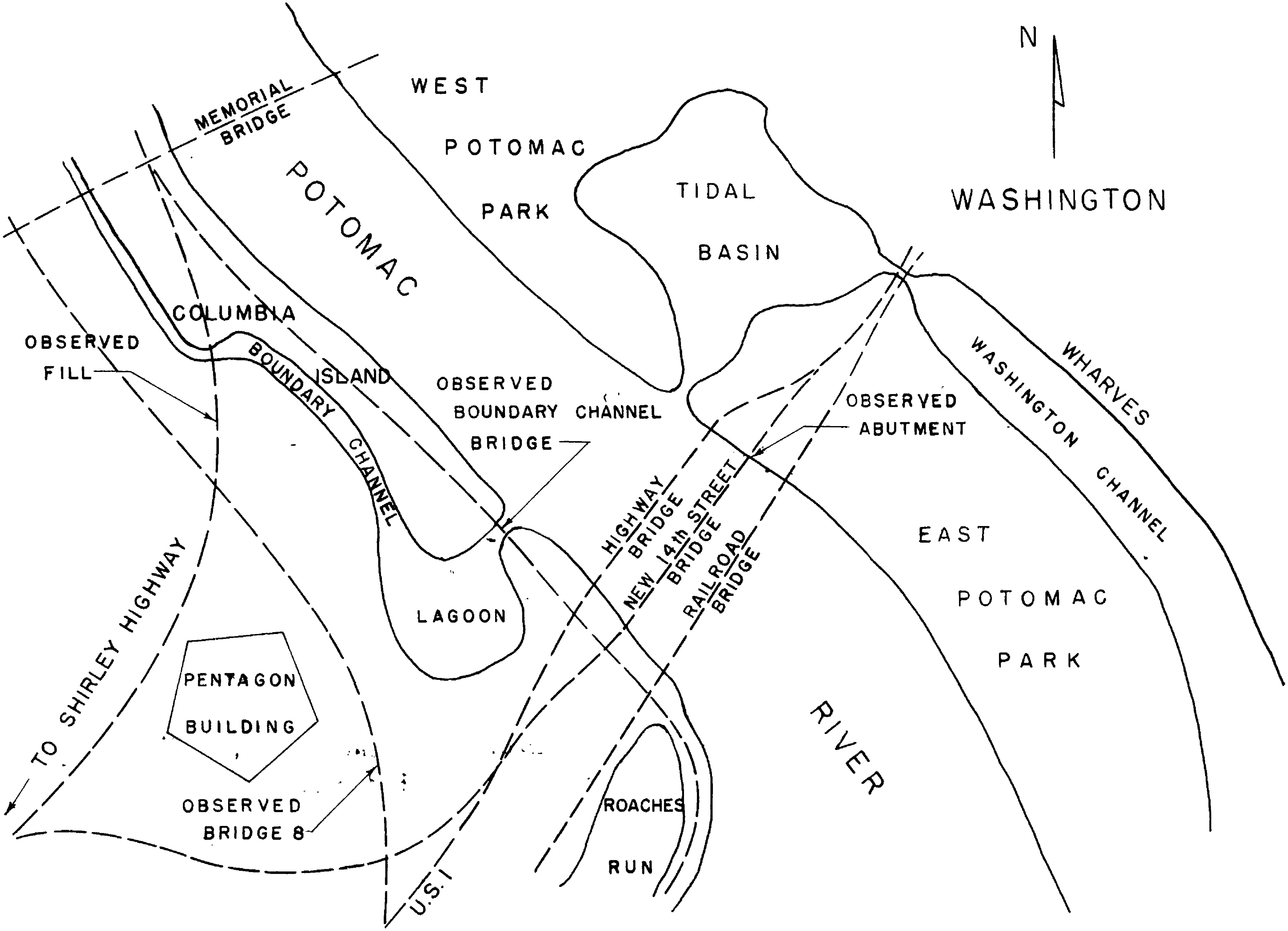


FIGURE 40— LOCATION OF OBSERVED SETTLEMENTS

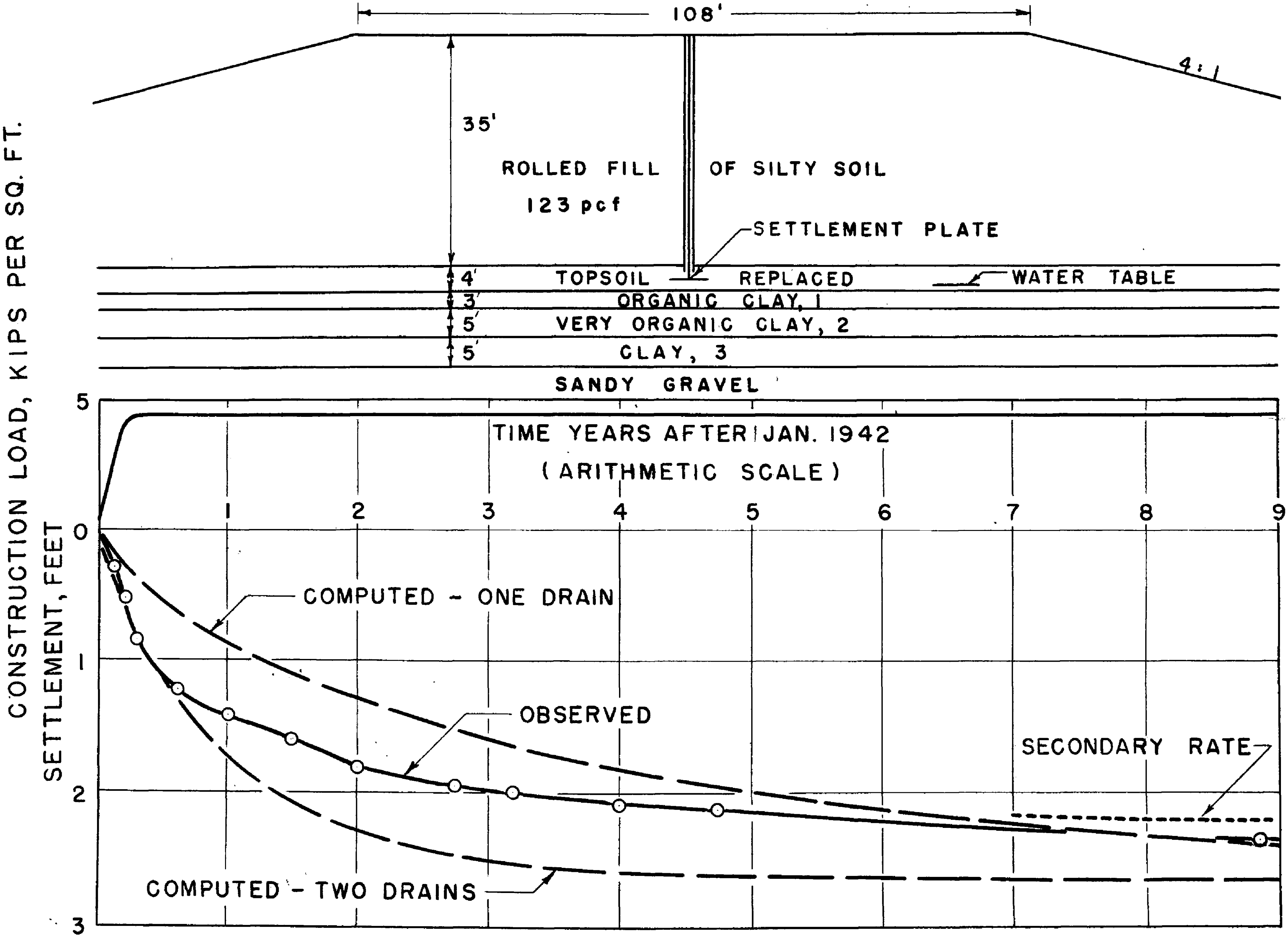


FIGURE 41 - SETTLEMENT OF FILL ON PENTAGON NETWORK

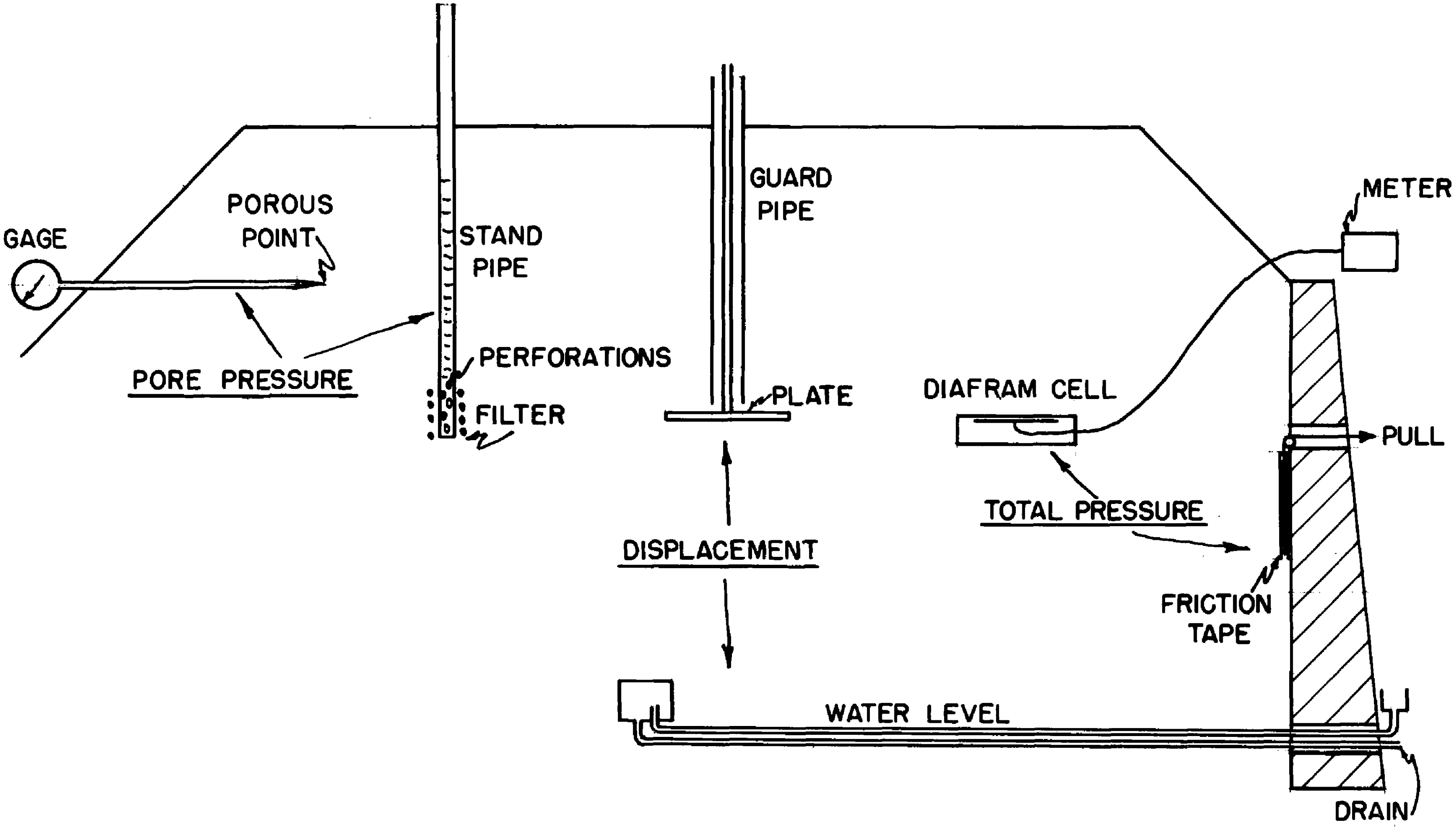


FIG. 42 - MEASUREMENT OF FIELD STRESSES AND DISPLACEMENTS

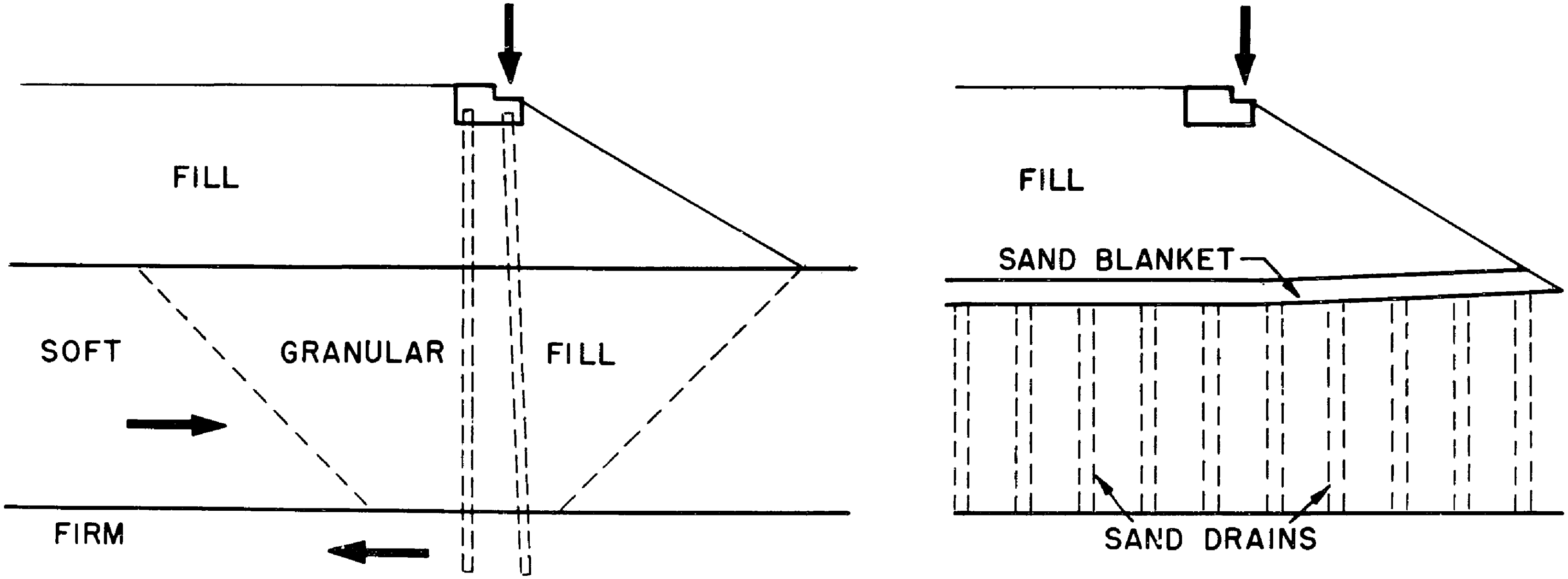
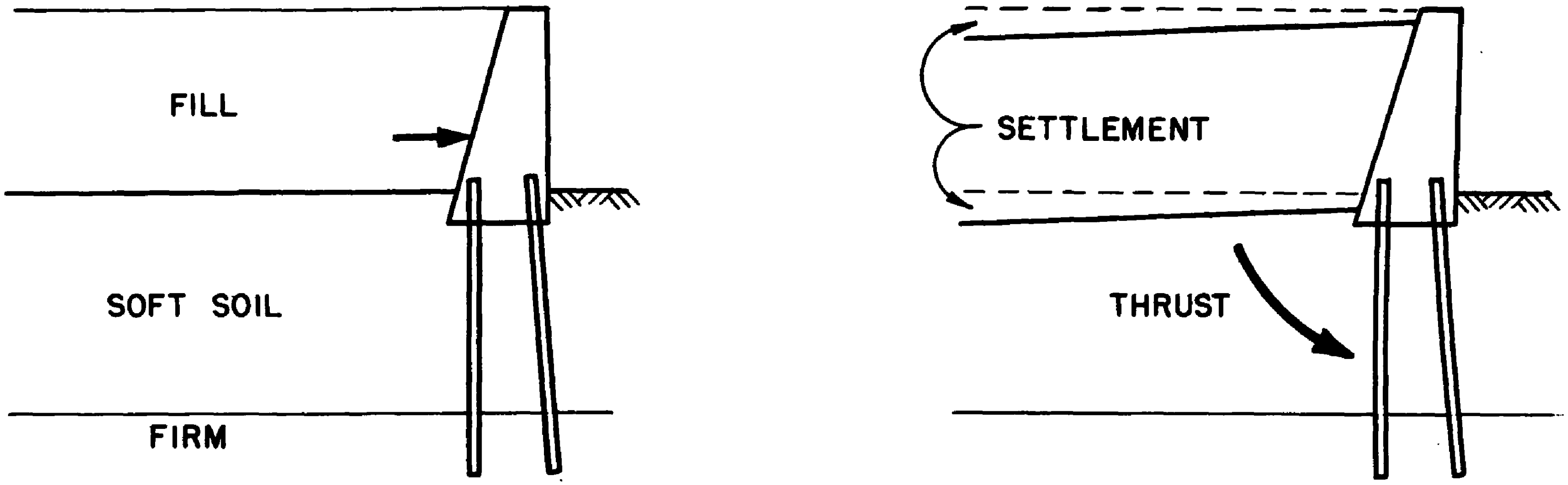


FIG.43–CONTROL OF LATERAL THRUST



ASSUMED

ACTUAL

FIG. 44-LATERAL THRUST OF SURCHARGED SOFT SOIL

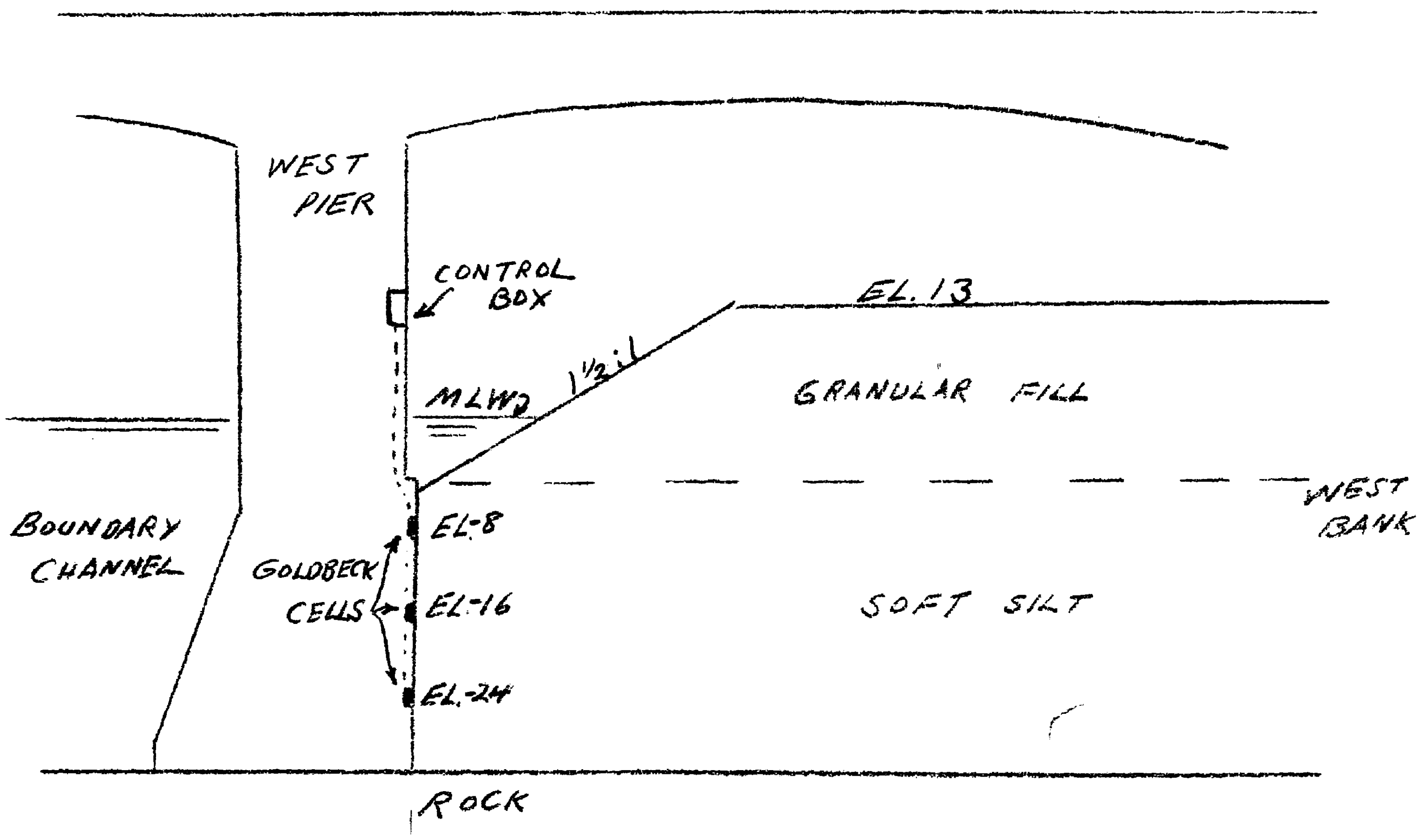


FIG. 45 – PRESSURE CELL INSTALLATION IN BOUNDARY CHANNEL BRIDGE AT NORTH END OF COLUMBIA ISLAND, D.C.

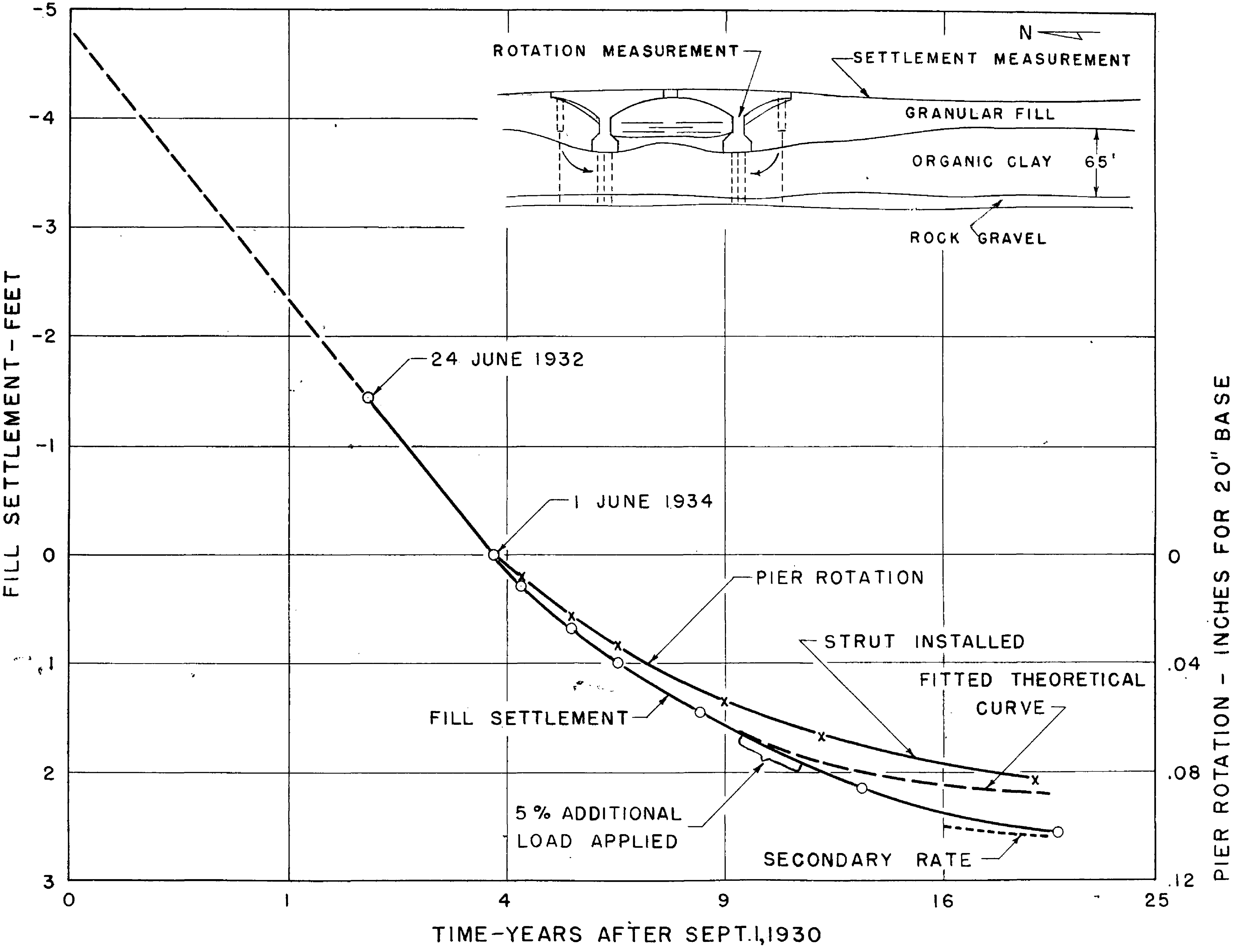


FIGURE 46 – FILL SETTLEMENT & PIER ROTATION AT BOUNDARY CHANNEL

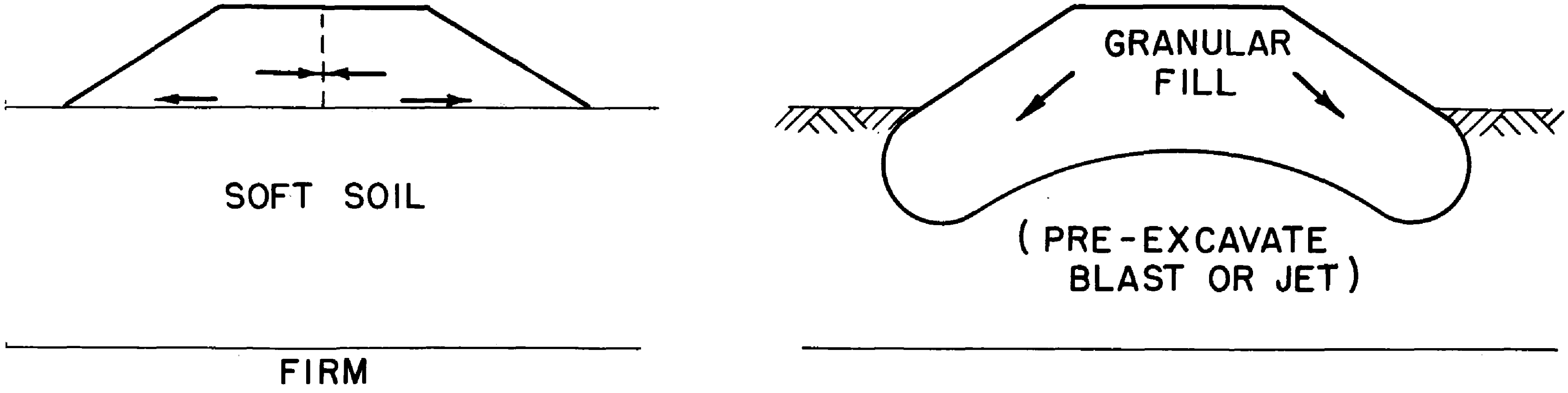


FIG.47- LATERAL THRUST OF FILL

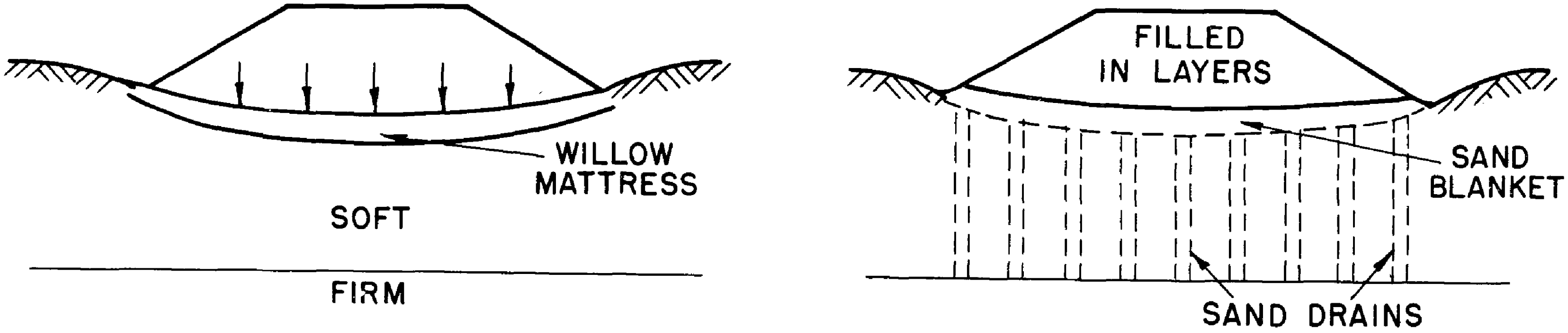


FIG. 48– CONTROL OF FILL DISPLACEMENT

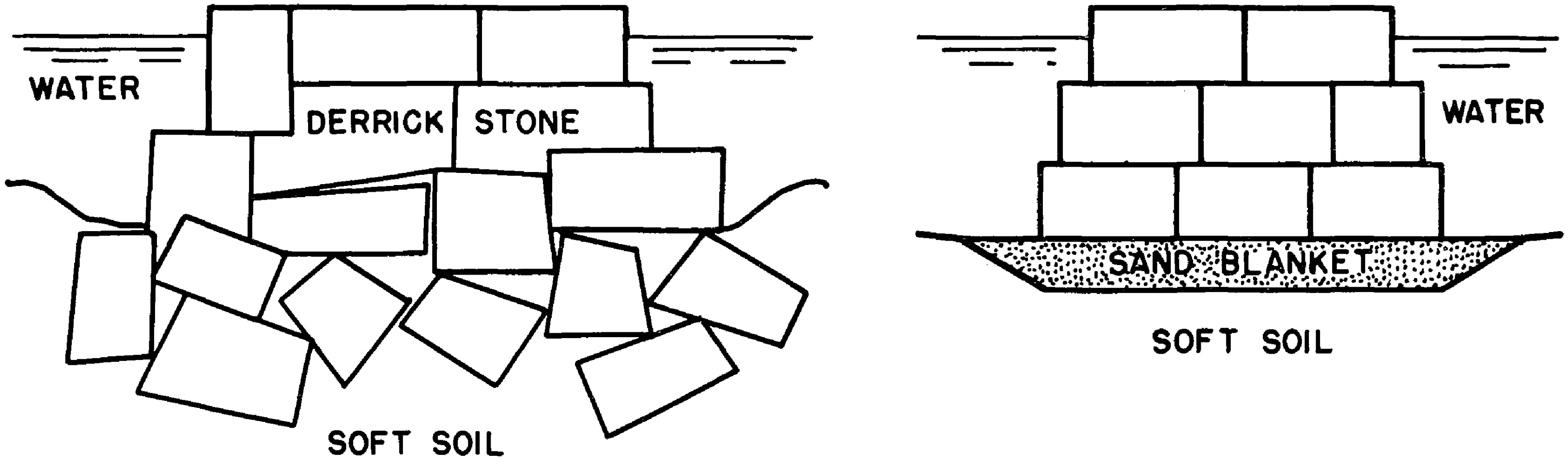


FIG. 49- BREAKWATER ON SOFT SOIL

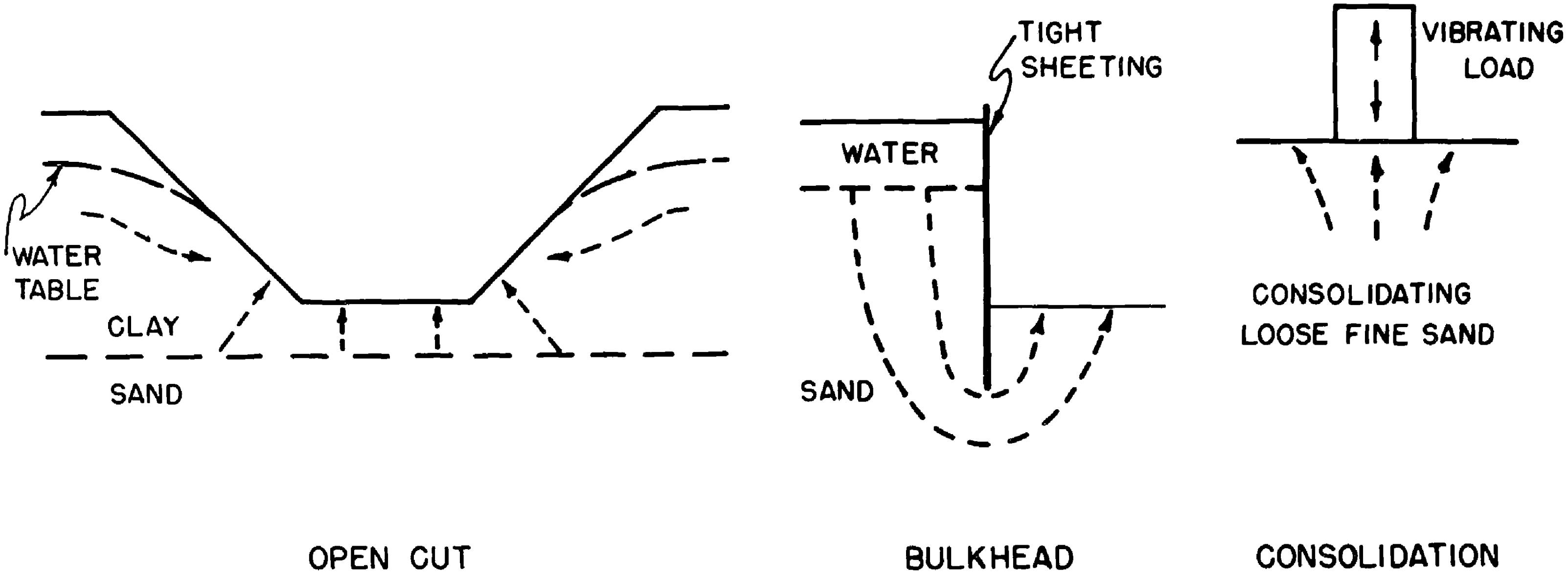


FIG.50-ILLUSTRATIONS OF UPWARD FLOW OF WATER

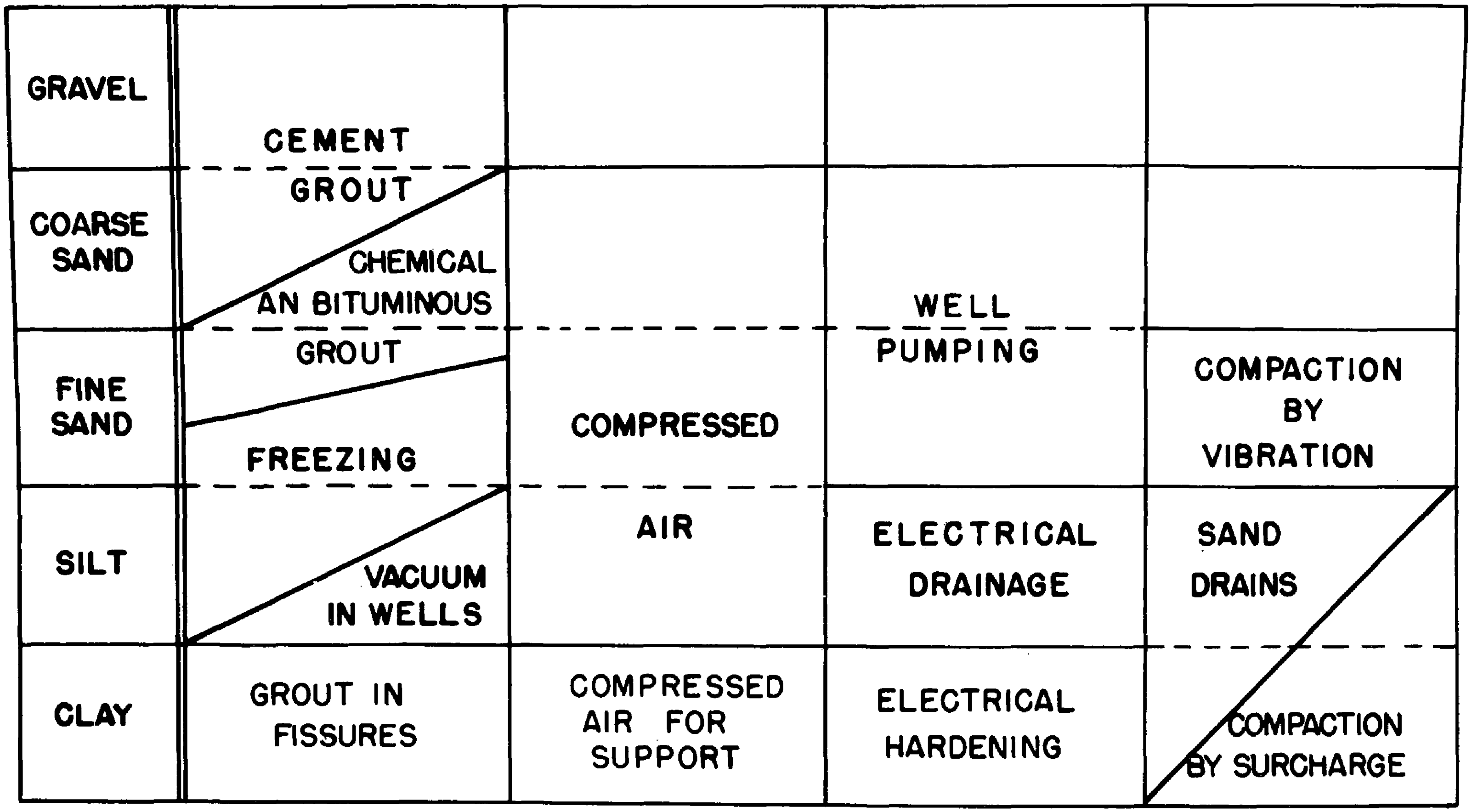
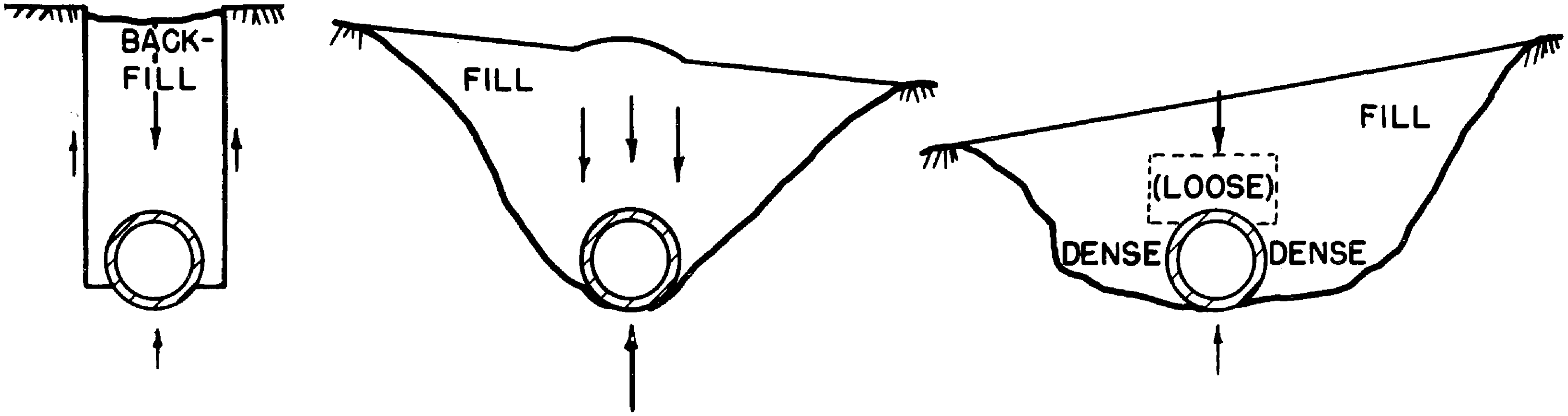
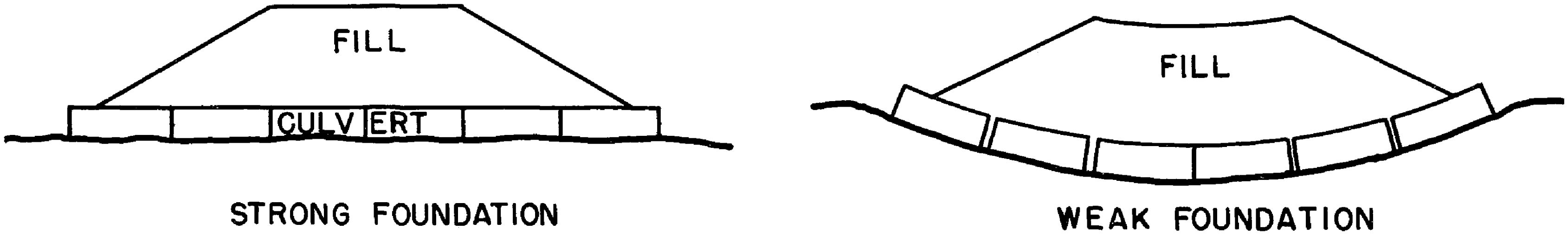


FIG.51- METHODS OF IMPROVING SOIL



USUAL USUAL CONTROLLED

FIG. 52 - VERTICAL PRESSURE ON PIPES



STRONG FOUNDATION

WEAK FOUNDATION

FIG.53-CULVERT DISPLACEMENT

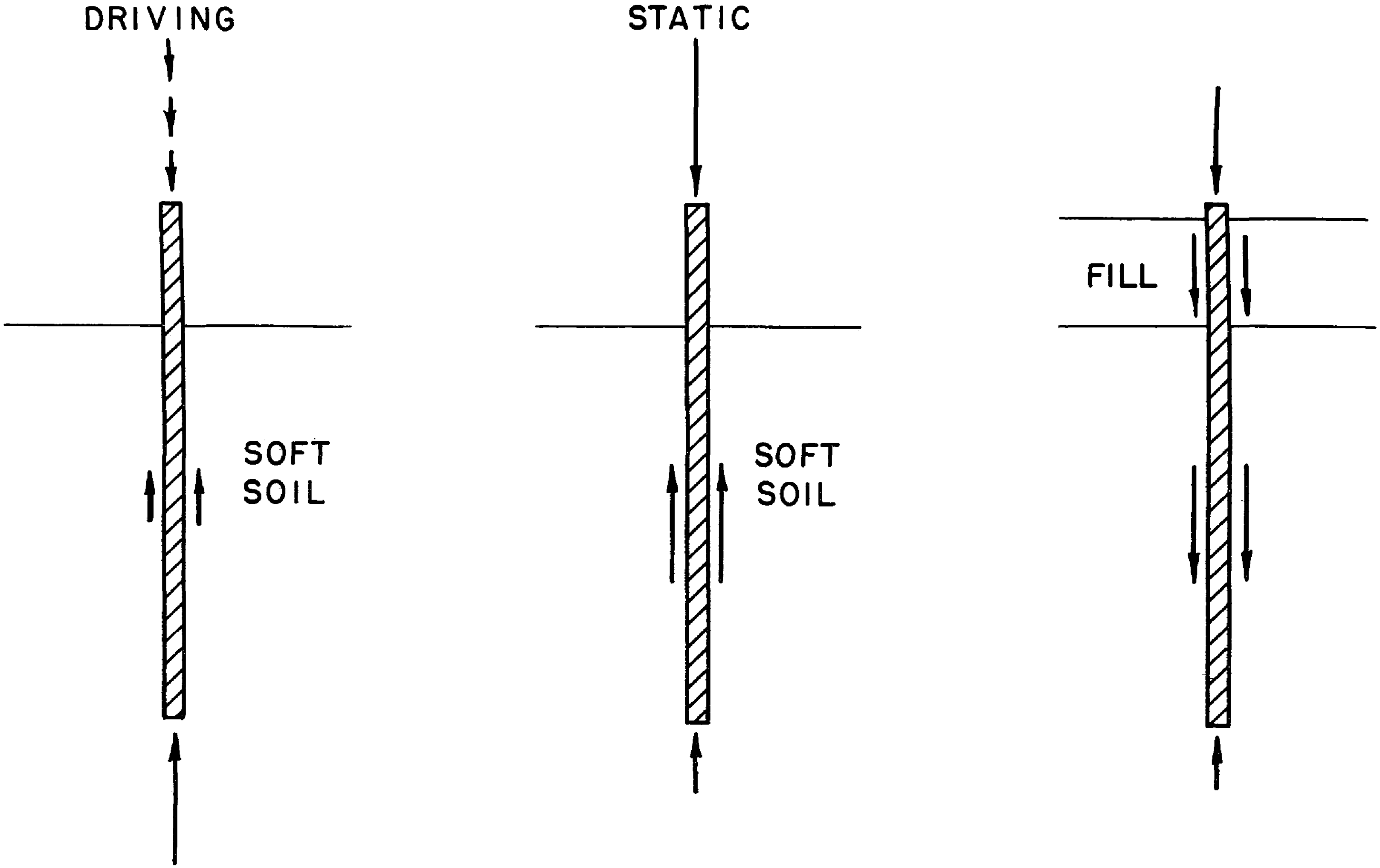


FIG.54- PILES THROUGH SOFT SOIL

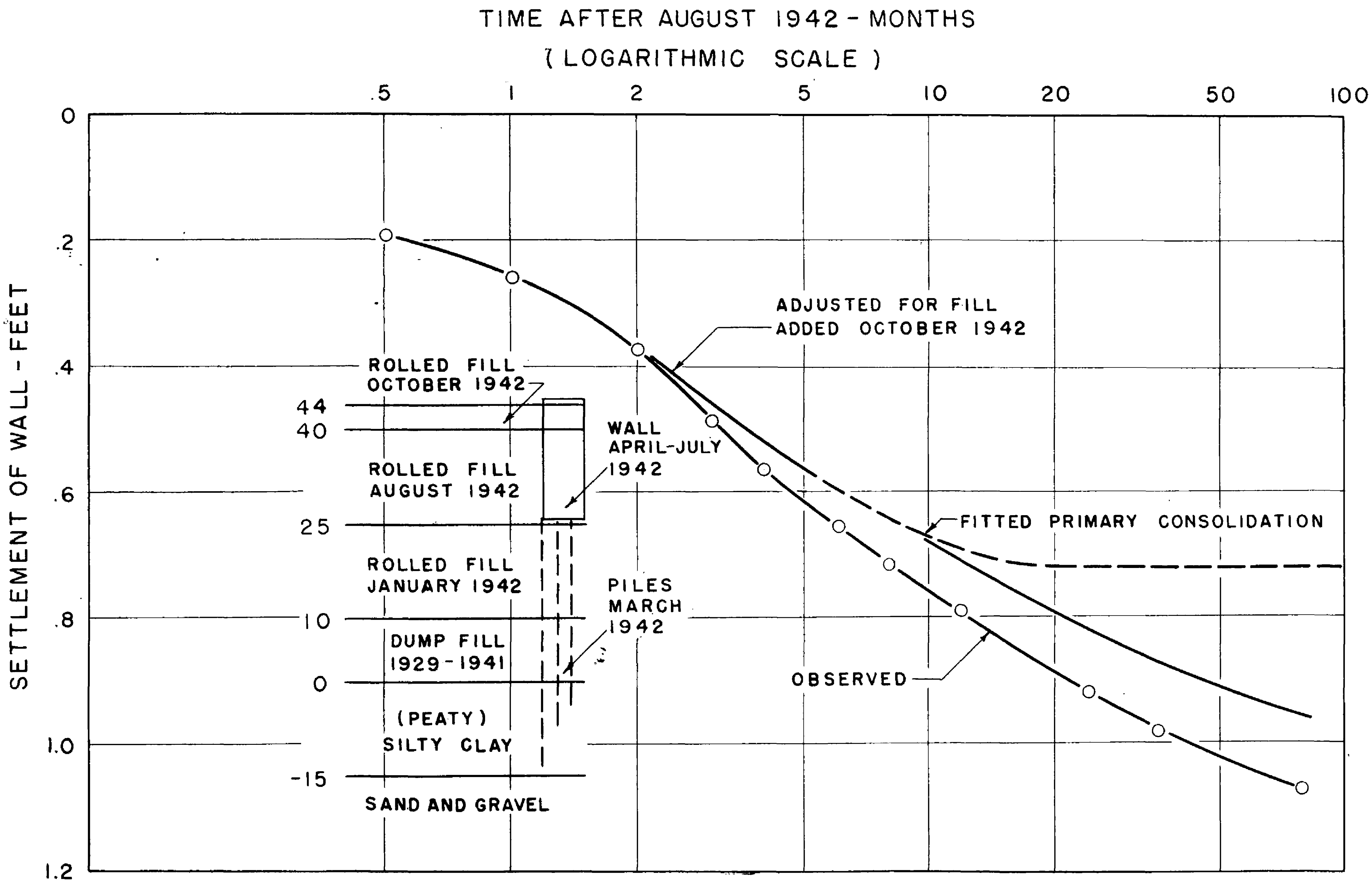


FIGURE 55– SETTLEMENT OF NORTHEAST WING-WALL, BRIDGE 8, PENTAGON NETWORK

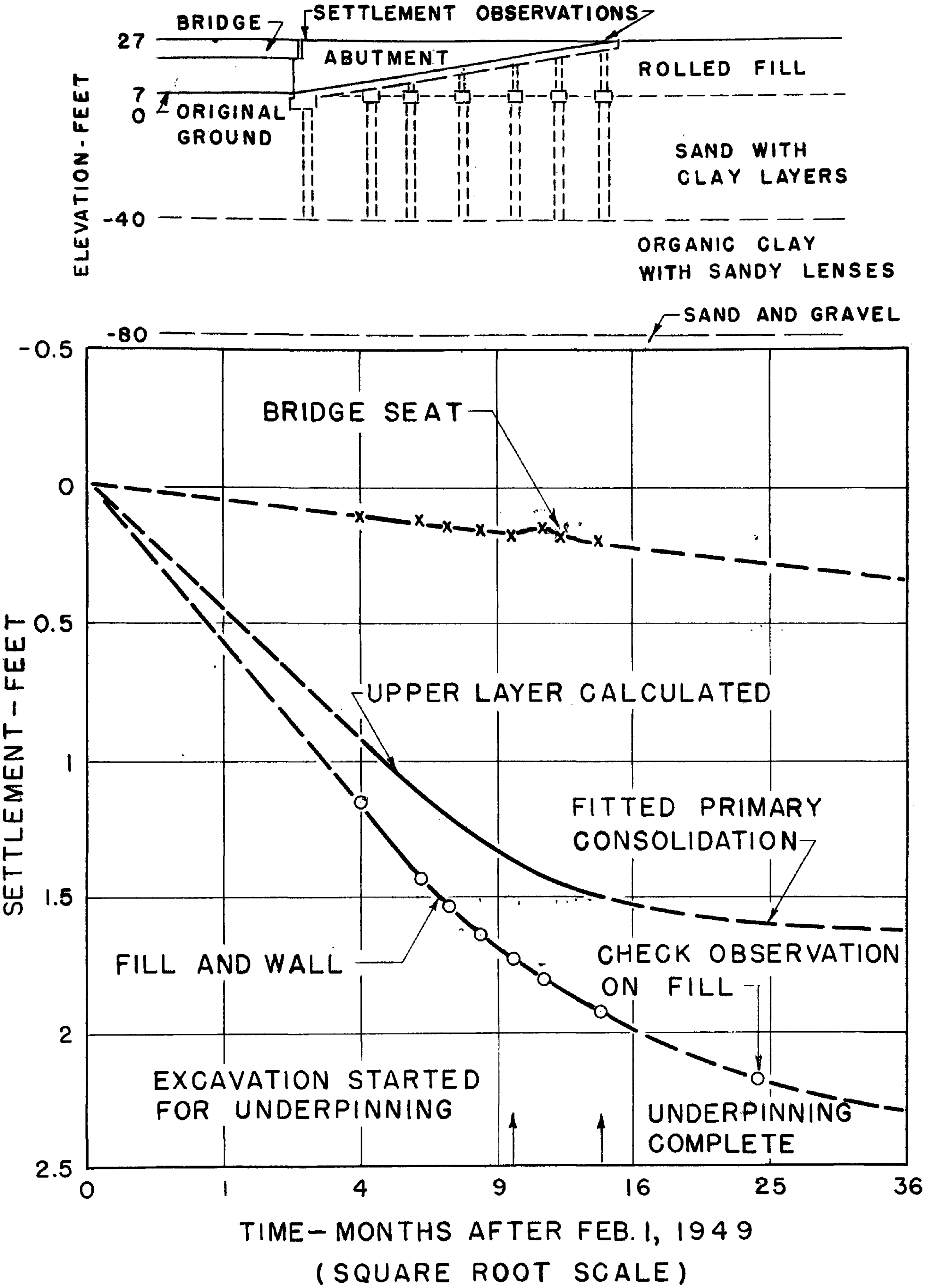
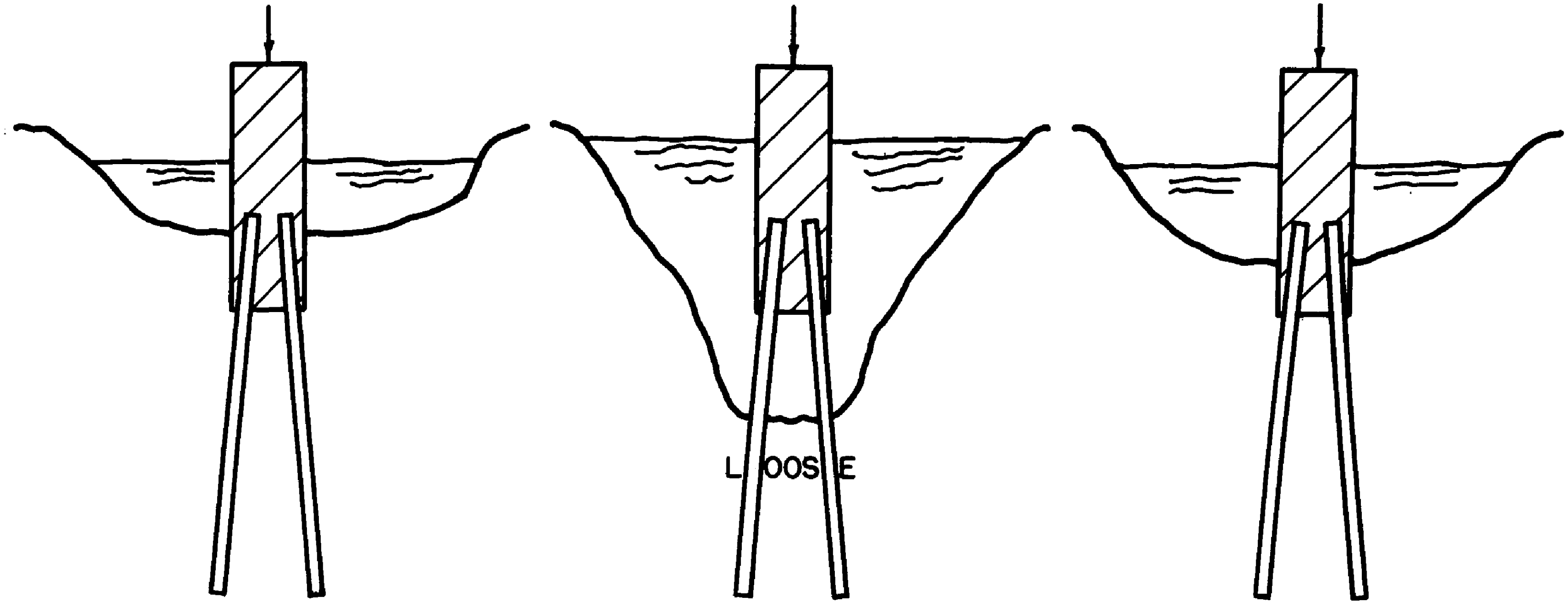


FIGURE 56 – SETTLEMENT AT NORTH ABUTMENT OF NEW FOURTEENTH STREET BRIDGE



NORMAL DURING FLOOD AFTER FLOOD

FIG. 57- SCOUR AROUND PIER IN STREAM

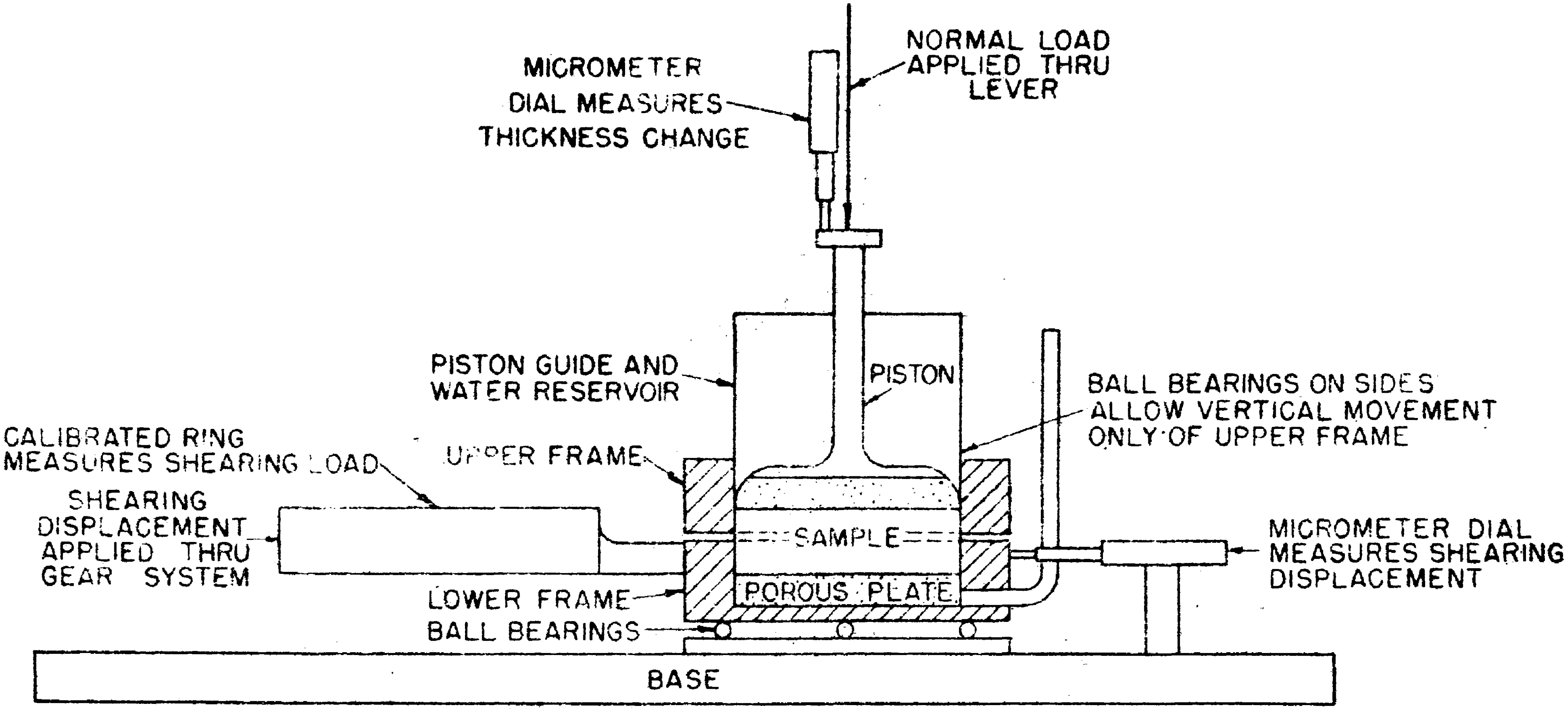


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FIGURE 58 — DIRECT SHEAR DEVICE

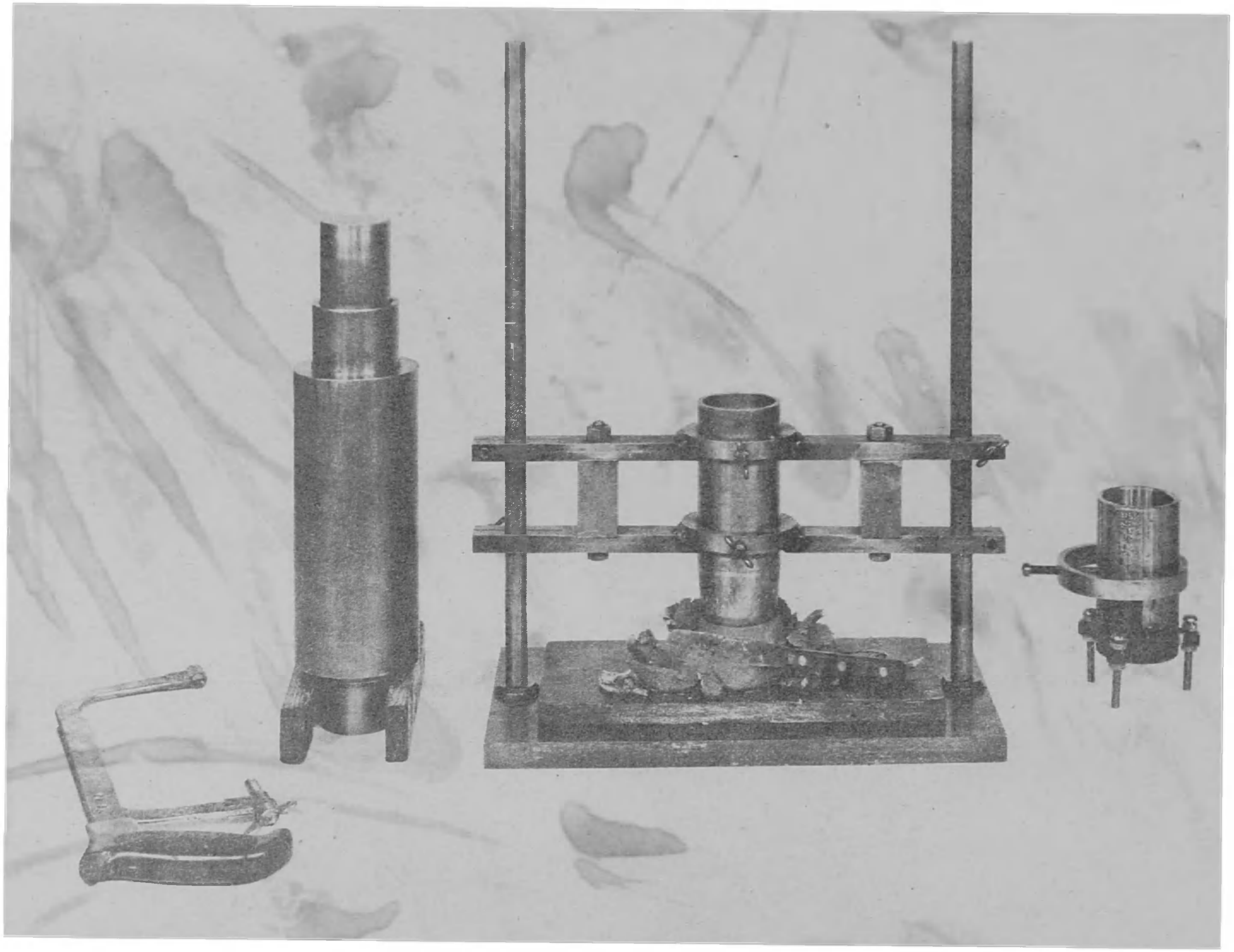

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FIGURE 59

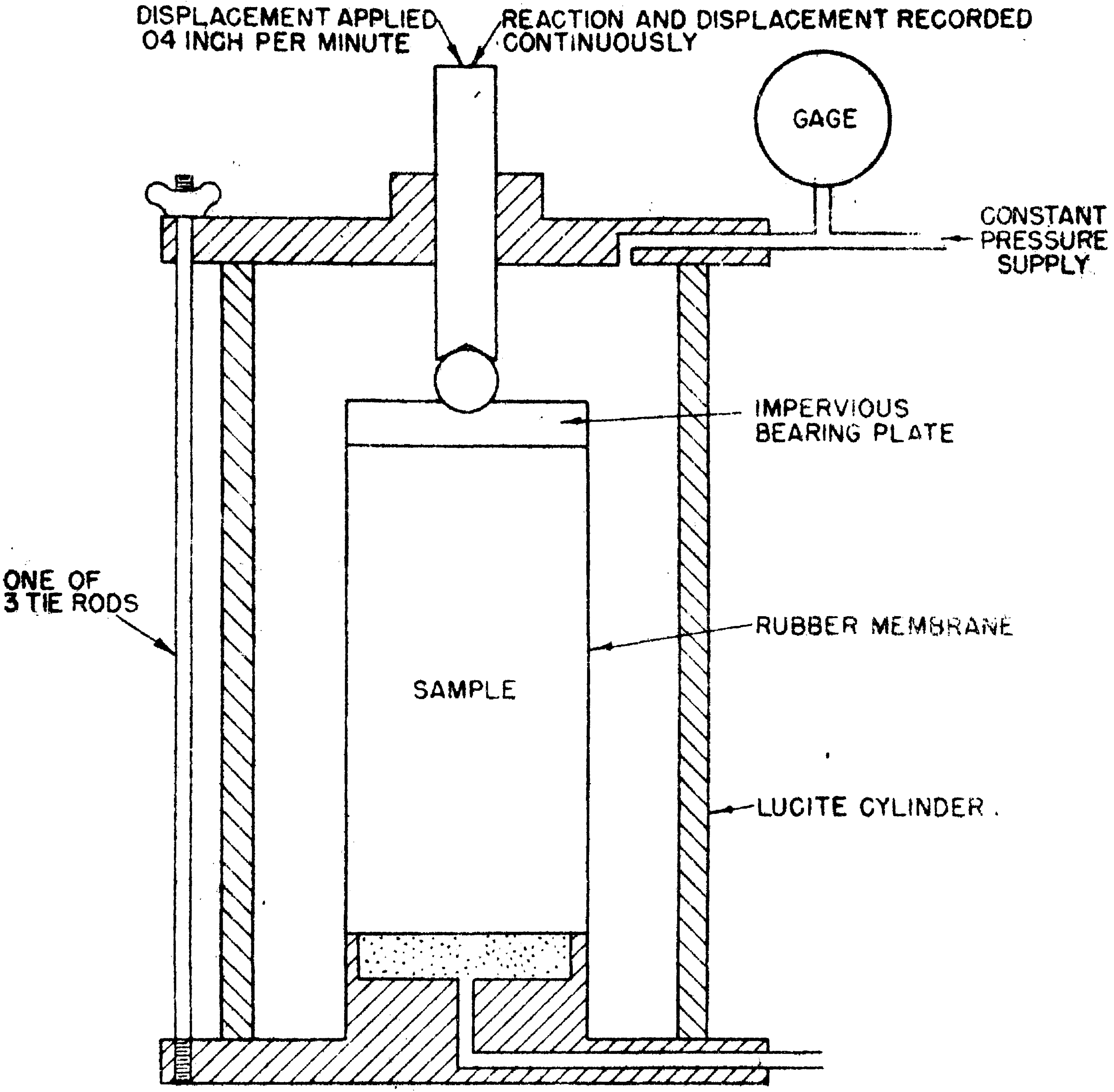

FIGURE 60— ESSENTIALS OF TRIAXIAL SHEAR TEST

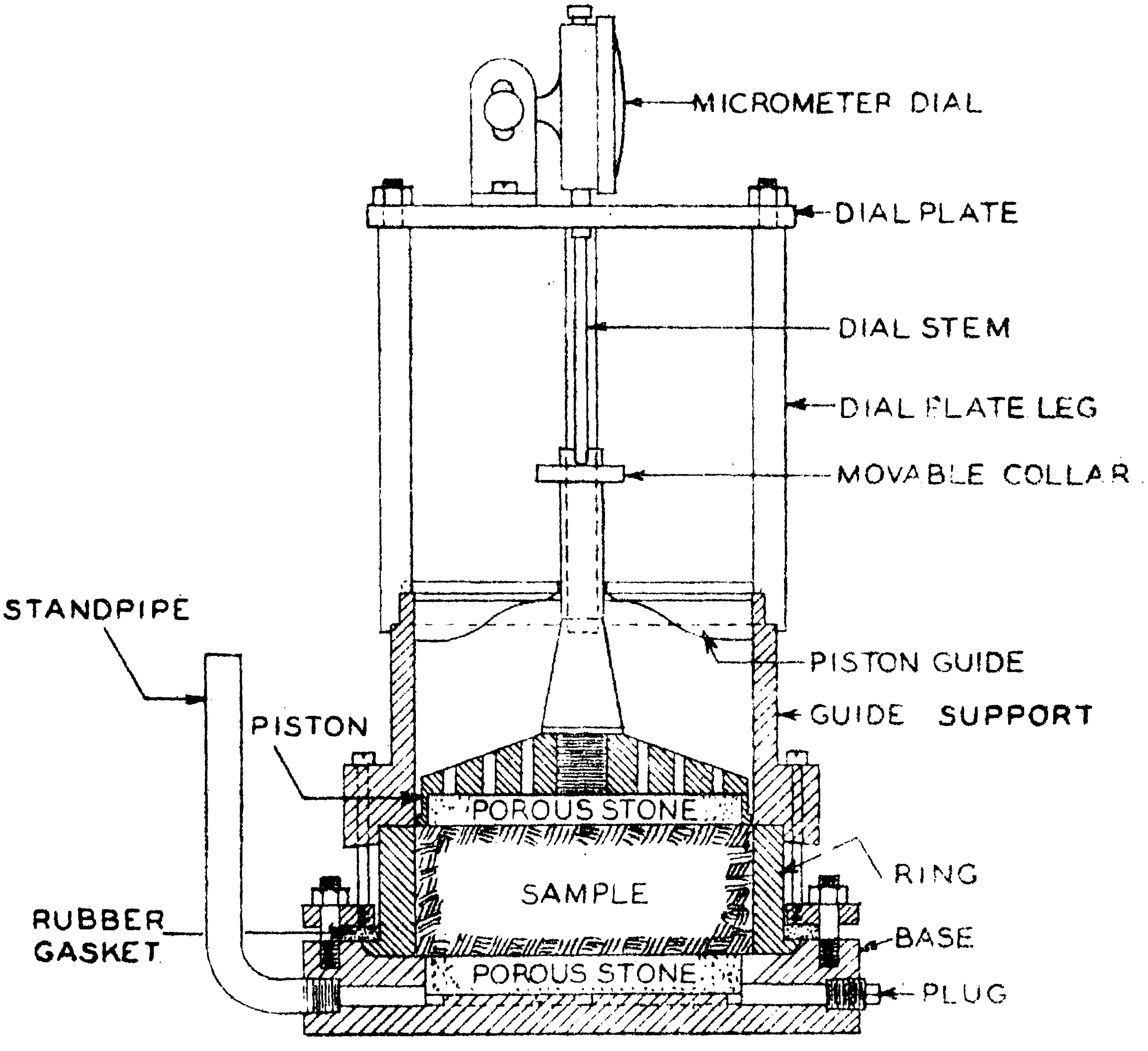

FIGURE6/ - CONSOLIDOMETER

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