# PRESENT TREND OF SUBGRADE RESEARCH

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## SYNOPSIS

Subgrade investigations have progressed to a point where the principal emphasis is now being placed upon research concerned with the more specific problems of test procedure, test apparatus and soil identification This report discusses briefly the essential features of the following problems of this character that have received special attention during the past year

(1) Modification of the hydrometer method of mechanical analysis to include the use of new apparatus and to make the method applicable for the analysis of portland cement as well as soils (2) Investigation of the liquid limit device (3) Improvement of the compression test device (4) Development of samplers for obtaining undisturbed soil cores (5) Development of cutting apparatus required in the preparation of undisturbed soil samples for swell tests (6) Design of the drainage indicator, a new apparatus to reveal quantitatively the capillary and drainage properties of soils (7) Development of the flocculation test as an indication of the presence of properties important with respect to subgrade performance (8) Development of a procedure for making a microscopic identification of crystals furnished by water soluble constituents of soils and portland cement concrete (9) Investigation of the causes of detrimental warping in concrete pavements (10) Design of an investigation for furnishing data on controlled outdoor frost heave phenomena and the effectiveness of suggested remedial measures (11) Investigation of the use of physical and chemical admixtures for stabilizing soils

### 1 MODIFICATIONS OF THE HYDROMETER METHOD OF MECHANICAL ANALYSIS

The hydrometer method for making the mechanical analysis of fine grained materials has been described in Public Roads,[1] Vol 12, No 8, October, 1931 It utilizes a sensitive hydrometer specially calibrated in grams per liter to disclose the density of the suspension at any stage of sedimentation, and Stokes' law to disclose the maximum grain size corresponding to a particular density Its use necessitates computations involving the use of variables which had to be determined by extended research of basic character

This work, which has just been completed, furnished a test procedure

[1] "Procedures for Testing Soils for the Determination of the Subgrade Soil Constants" by A M Wintermyer, E A Willis and R C Thoreen "Graphical Solution of the Data Furnished by the Hydrometer Method of Analysis" by E A Willis, F A Robeson and C M Johnston

utilizing any sensitive hydrometer calibrated in specific gravity, and the correction coefficients whereby both the grams per liter concentration and the maximum particle diameter corresponding to the hydrometer readings can be determined In addition, it disclosed that the results furnished by the hydrometer method check within practical limits those obtained by the pipette methods of the U S Bureau of Chemistry and Soils, and other methods As a further outgrowth of this work, a new effective deflocculating agent was discovered and a new type of hydrometer and a revised dispersing cup and stirring paddle were developed

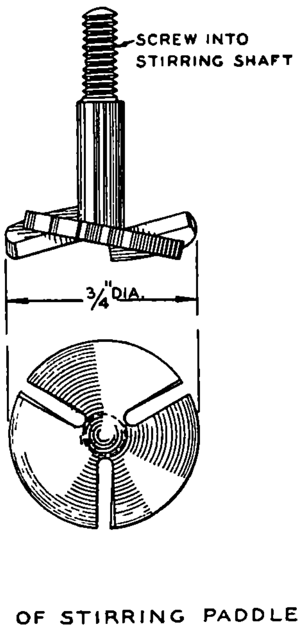



**Figure 1. Detachable Paddle for Soil Dispersing Device**

The new paddle shown in Figure 1 is made of wear-resisting material and consequently requires less frequent replacing than the softer paddles furnished with the dispersing apparatus

The paddle of the dispersing apparatus as purchased is an integral part of the stirring shaft The new one, which screws into the end of the stirring rod eliminates the necessity for changing the stirring rod with the paddle

The new cup with a semi-spherical bottom and detachable baffles, shown in Figure 2 right, eliminates the error due to the accumulation of soil particles in the bottom and around the baffles of the original flat bottom dispersing cup as shown in Figure 2 left

The general design of the new hydrometer, calibrated in specific gravities, is shown in Figure 3A as compared with the Bouyoucos hydrometer, Figure 3B. The shape of the new hydrometer is proposed to minimize the accumulation of soil particles on the bulb when the hydrometer is left immersed in the suspension, and the disturbance of the suspension when the hydrometer is inserted or removed

Extensive tests with various deflocculating agents showed sodium silicate to be the most effective for maintaining separation of the dispersed soil particles in suspension Table I illustrates the superiority of sodium silicate over other agents for dispersing colloids Here it is seen that the same soil, No 5853, for instance, may have an indicated

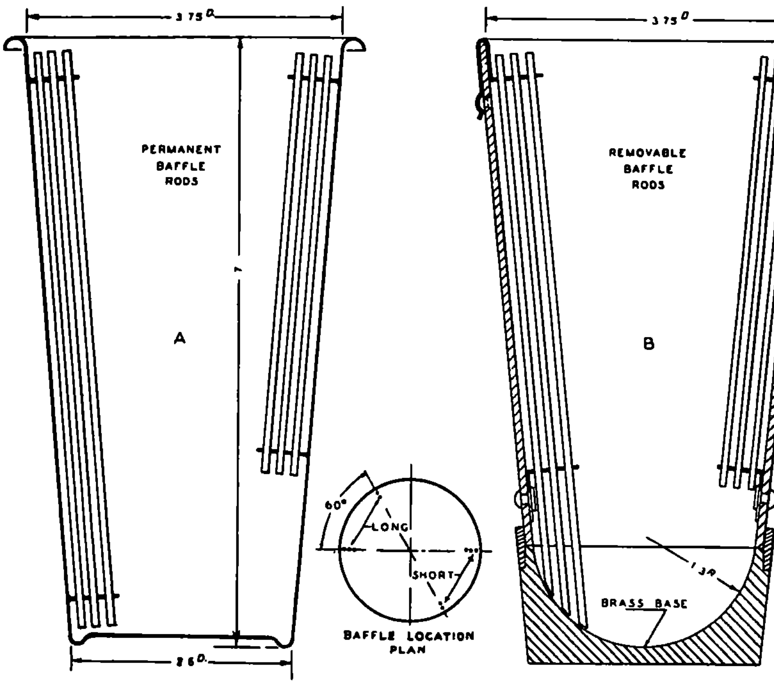



**Figure 2. Soil Dispersing Cups. Old Design, Left. New Design, Right**

colloidal content of 57 per cent, 50 per cent, 28 per cent, or 14 per cent, depending upon the type of deflocculating agent used in the tests

Table II shows the agreement of results furnished by the hydrometer and pipette methods of analysis Thus, for instance, in soil No 837, the material finer than 0 05 mm equals 94 per cent according to the hydrometer analysis and 95 per cent according to the pipette method of analysis Similarly, the clay content equals 75 per cent according to the former and 72 per cent according to the latter method of analysis

Such differences are much smaller than those due to lack of uniformity of samples of the same soil layer and consequently have no practical significance

In the adaptation of the hydrometer method of analysis to determine the grading of cements, a suspending medium had to be found which would prevent both the hydration and the flocculation of the cement particles.

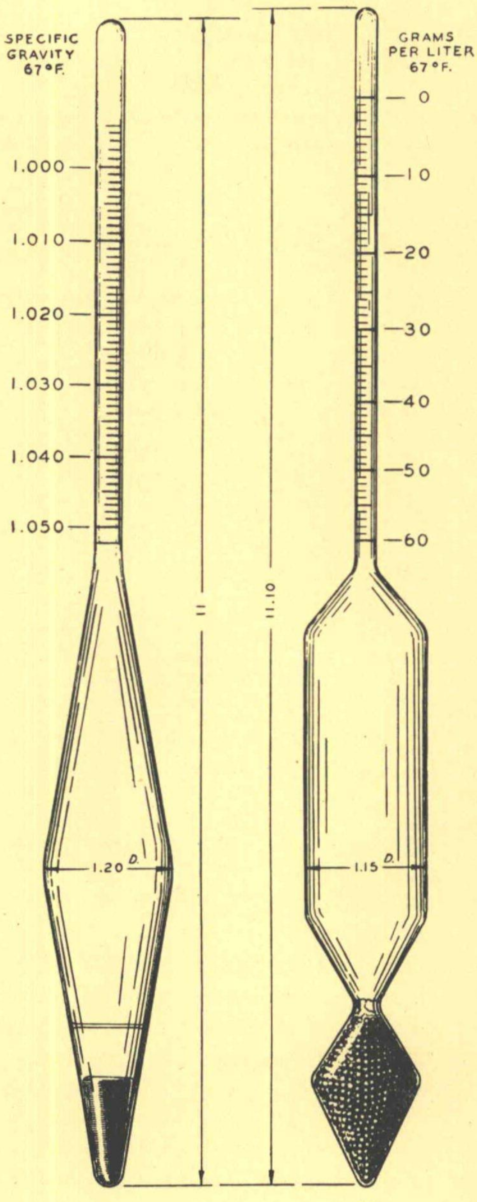



**Figure 3. Sensitive Hydrometers Used in Soil Analysis**

Carbon tetrachloride, pure ethyl alcohol and pure kerosene all gave promise of being satisfactory substances for this purpose.

Water free kerosene, however, with a small amount of oleic acid added as a deflocculant, seemed the most practical of the several possibilities With the added precaution that the cement be oven dried before being dispersed, the tendency to flocculate, which occurs more in some cements than in others, was completely eliminated

TABLE I

TOTAL PERCENTAGES OF PARTICLES SMALLER THAN 0 001 MM OBTAINED BY TREATMENT WITH EQUINORMAL SOLUTIONS OF DIFFERENT DISPERSING AGENTS

| Soil no | Source | No reagent | Potassium hydroxide 5 cc N/1 solution | Sodium carbonate 5 cc N/1 solution | Sodium oxalate 10 cc N/2 solution | Sodium silicate 5 cc N/1 solution |
|---|---|---|---|---|---|---|
| | | | Percentage of particles smaller than 0 001 mm | | | |
| 5792 | Iowa | * | 2† | 7† | 15 | 16 |
| 5853 | Miss | * | 14 | 28 | 50 | 57 |
| 6054 | N C | * | 3† | * | 3† | 23 |
| 6278 | Texas | * | * | 1† | * | 29 |

* Completely flocculated
† Partially flocculated

TABLE II

COMPARISON OF ANALYSES BY (1) HYDROMETER AND (2) PIPETTE METHOD

| Soil no | Percentage finer than | | | | | |
|---|---|---|---|---|---|---|
| | 0 050 mm | | 0 005 mm | | 0 002 mm | |
| | (1) | (2) | (1) | (2) | (1) | (2) |
| 4907 | 12 | 13 | 8 | 9 | 7 | 4 |
| 487 | 58 | 57 | 18 | 16 | 14 | 11 |
| 4913 | 59 | 56 | 24 | 27 | 14 | 19 |
| 763 | 69 | 67 | 36 | 38 | 26 | 30 |
| 630 | 81 | 83 | 40 | 41 | 27 | 26 |
| 695 | 83 | 85 | 47 | 48 | 36 | 38 |
| 837 | 94 | 95 | 75 | 72 | 64 | 62 |

Results of the mechanical analysis of cements by the combined sieve and hydrometer methods are shown in Figure 4

A complete description of the research on the mechanical analysis comprises a report entitled "Comments on the Hydrometer Method of Mechanical Analysis" prepared by R C Thoreen for publication in Public Roads

## 2 MECHANICAL DEVICE FOR MAKING THE LIQUID LIMIT TEST

The procedure for making the liquid limit test has been published in Public Roads,[2] Vol 12, No 8, October, 1931 This procedure has

[2] "Procedure for Testing Soils for the Determination of the Subgrade Soil Constants" by A M Wintermyer, E A Willis and R C Thoreen

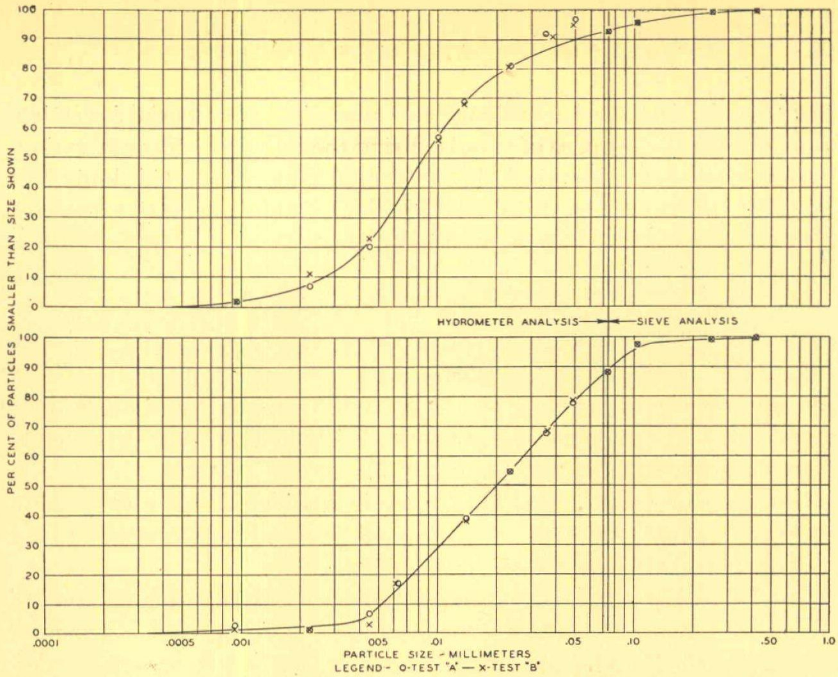



**Figure 4. Grain Size Accumulation Curves for Portland Cement Samples**

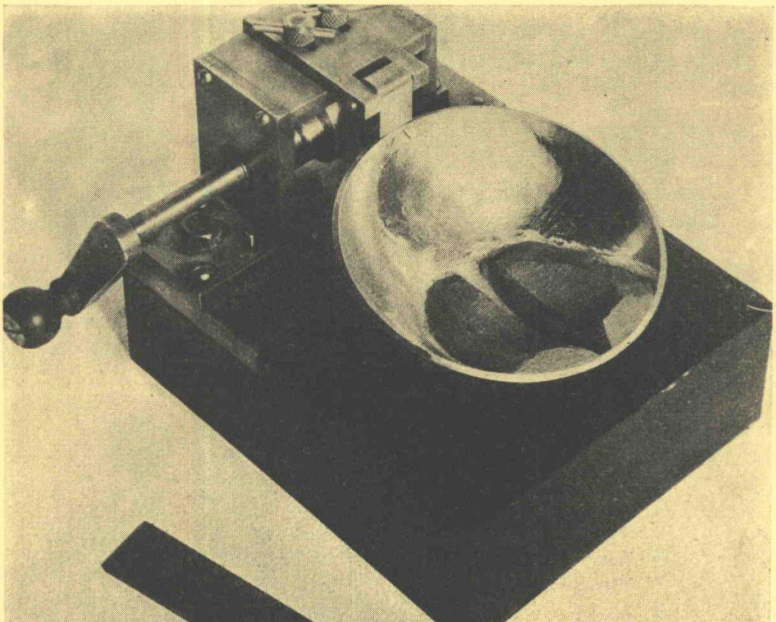


**Figure 5. Mechanical Device for Determining Flow Curves**

been found adequate for ordinary soil analysis. For special investigations and for the purpose of standardizing the test procedures in various laboratories the mechanical device for making the liquid limit test shown in Figure 5 was designed as a part of the cooperative research project between the Bureau of Public Roads and the Massachusetts Institute of Technology.

The significance of the Atterberg limits with respect to the consistency of soils is as follows The soil is very soft when containing moisture above the liquid limit, is plastic with moisture contents between the liquid and plastic limits, semi-solid with moisture contents between the plastic and shrinkage limits and solid with moisture contents smaller than the shrinkage limit

In this conception of plasticity there is considered only the range of moisture contents through which the soil is plastic with no regard for the relative stability of the soil in the plastic state

The liquid limit is the moisture content of a soil requiring a standard number of shocks of standard intensity to just close a groove of standard dimension in the soil cake In the routine test, water or soil powder, as determined by trial, is added to the test sample until the desired consistency is obtained

With the mechanical device, however, the number of shocks required to close the groove, with the soil at the different trial consistencies, is plotted against the corresponding moisture contents. The liquid limit is the moisture content corresponding to 25 shocks as shown by a curve connecting these points

Such curves, termed "flow curves" and illustrated in Figure 6, become straight lines when plotted on semi-logarithmic paper

The flow curves afford a means for determining the flow index and the toughness index, two constants which may be highly significant in the study of soils

The flow index is the slope of the flow curve and is expressed mathematically as follows:

$$F = \frac{C - W}{\log N}$$

in which $F$ = flow index.
$C$ = constant.
$W$ = moisture content (per cent of weight of dry soil).
$N$ = number of blows.

Since the flow curves shown in Figure 6 are semi-log plots, the flow index which is merely the slope of these plots becomes the range in moisture corresponding to the number of blows represented by one cycle on the logarithmic scale. Thus the flow indices, Table III, are the moisture content ranges corresponding to the cycle between 10 and 100 blows, Figure 6

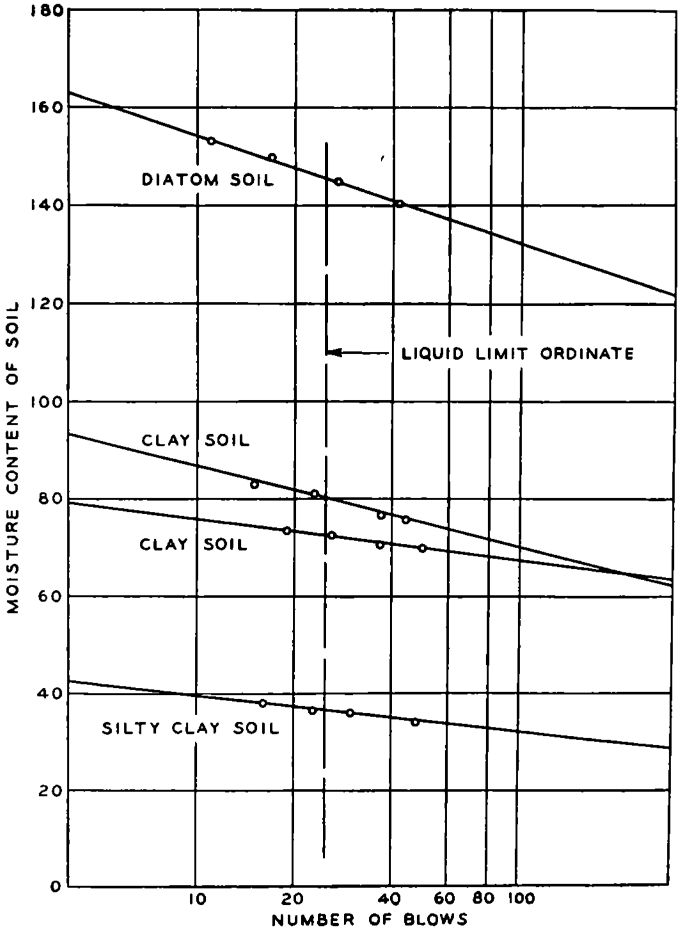



Figure 6. Typical Flow Curves

TABLE III

Constants of Soils Whose Flow Curves Are Shown in Figure 6

| Soil type | Liquid limit | Plasticity index | Flow index | Toughness index |
|---|---|---|---|---|
| Diatom | 166 | 36 | 21 8 | 1 7 |
| Clay—A | 80 | 54 | 16 5 | 3 3 |
| Clay—B | 72 | 47 | 8 5 | 5 5 |
| Silty clay | 37 | 16 | 7 2 | 2 2 |

The toughness index, $T$, is the quotient obtained by dividing the plasticity index, $I$, by the flow index Thus

$$T = \frac{I}{F}$$

This equation is based on two assumptions, to wit

(1) The number of blows required to close the groove of a soil paste at the *plastic* limit is a measure of the toughness of the soil at this moisture content

(2) In the whole range between the liquid and the plastic limit the relation between shearing resistance and water content, on a semi-logarithmic plot, is represented by a straight line

According to the foregoing, the following conclusions are warranted

a For soils having equal plasticity indices, the shearing resistance at the plastic limit is inversely proportional to the flow index

b For soils having equal flow indices, the shear resistance at the plastic limit is proportional to the plasticity index

c With respect to the relation between shear resistance and moisture content, soils having equal plasticity and also flow indices are alike

The relative effect of equal increases in moisture content for diminishing the stability is inversely proportional to the flow indices Of the soils in Table III, therefore, the effect of equal increases of moisture for softening the silt soil with a flow index of 7 2 would be about three times as great as for the diatom soil with a flow index of 21 8 At the plastic limit, however, the silt soil is tougher than the diatom soil in the ratio of 2 2 to 1 7

Thus the use of the flow curves utilizes stability in the measure of plasticity in addition to the mere range of moisture content through which the soil remains plastic, as originally conceived by Atterberg

By means of the toughness index formula the moisture contents at which the clay soils have equal toughness can be determined This is accomplished by substituting the respective plasticity indices for $I$ and the desired toughness for the index $T$ Thus for equal toughness, say that indicated by an index of 3 0, Clay B will have a moisture content of 30 5 while Clay A, with the higher plasticity index, will have a moisture content of 46 5 Similar information is furnished by the flow curves, Figure 6 Here for equal toughness or shear resistance, that indicated by 100 blows, the silt will have a moisture content of 32, the Clay A, 68 per cent, the Clay B, 70 per cent and the diatom soil 132 per cent

The report "Research on the Atterberg Limits for Soils" by Arthur Casagrande, Public Roads, Vol 13, No 8, October, 1932, contains a complete discussion of the flow curves, their determination and their significance

### 3 MODIFICATION OF THE COMPRESSION TEST APPARATUS

The bearing power of loaded soil depends upon three soil properties as follows

1 Stability of the resistance to lateral flow
2 Compressibility or the resistance to vertical consolidation

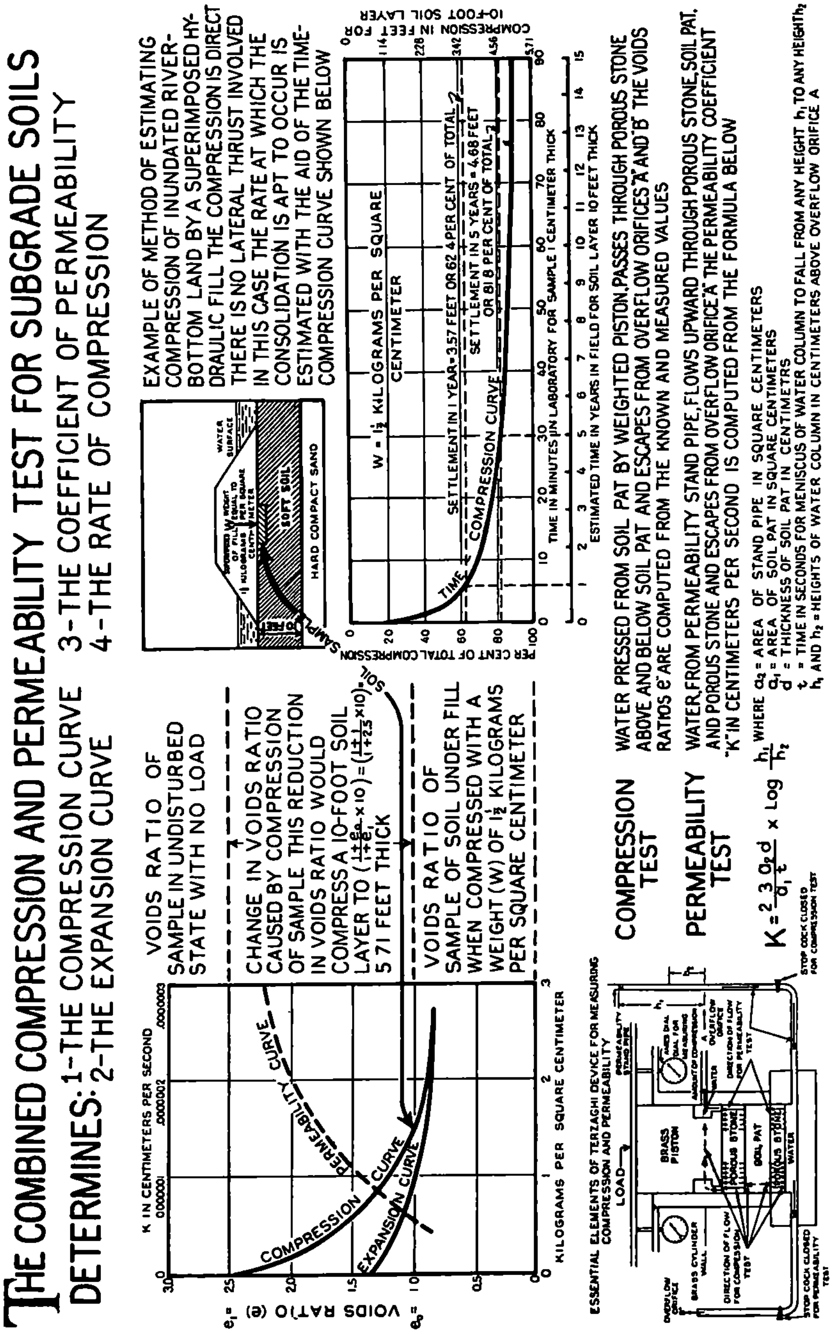

THE COMBINED COMPRESSION AND PERMEABILITY TEST FOR SUBGRADE SOILS
DETERMINES: 1-THE COMPRESSION CURVE 3-THE COEFFICIENT OF PERMEABILITY
2-THE EXPANSION CURVE 4-THE RATE OF COMPRESSION
K IN CENTIMETERS PER SECOND
VOIDS RATIO (e)
COMPRESSION CURVE
EXPANSION CURVE
PERMEABILITY CURVE
KILOGRAMS PER SQUARE CENTIMETER
VOIDS RATIO OF SAMPLE IN UNDISTURBED STATE WITH NO LOAD
CHANGE IN VOIDS RATIO CAUSED BY COMPRESSION OF SAMPLE THIS REDUCTION IN VOIDS RATIO WOULD COMPRESS A 10-FOOT SOIL LAYER TO 5.71 FEET THICK
VOIDS RATIO OF SAMPLE OF SOIL UNDER FILL WHEN COMPRESSED WITH A WEIGHT (W) OF 1½ KILOGRAMS PER SQUARE CENTIMETER
SOIL SAMPLE
HARD COMPACT SAND
SOFT SOIL
WATER SURFACE
EXAMPLE OF METHOD OF ESTIMATING COMPRESSION OF INUNDATED RIVER-BOTTOM LAND BY A SUPERIMPOSED HYDRAULIC FILL THE COMPRESSION IS DIRECT THERE IS NO LATERAL THRUST INVOLVED IN THIS CASE THE RATE AT WHICH THE CONSOLIDATION IS APT TO OCCUR IS ESTIMATED WITH THE AID OF THE TIME-COMPRESSION CURVE SHOWN BELOW
W = 1½ KILOGRAMS PER SQUARE CENTIMETER
SETTLEMENT IN 1 YEAR = 3.57 FEET OR 62.4 PER CENT OF TOTAL
SETTLEMENT IN 5 YEARS = 4.68 FEET OR 81.8 PER CENT OF TOTAL
TIME-COMPRESSION CURVE
PER CENT OF TOTAL COMPRESSION
COMPRESSION IN FEET FOR 10-FOOT SOIL LAYER
TIME IN MINUTES IN LABORATORY FOR SAMPLE 1 CENTIMETER THICK
ESTIMATED TIME IN YEARS IN FIELD FOR SOIL LAYER 10 FEET THICK
ESSENTIAL ELEMENTS OF TERZAGHI DEVICE FOR MEASURING COMPRESSION AND PERMEABILITY
LOAD
BRASS PISTON
BRASS CYLINDER WALL
POROUS STONE
SOIL PAT
WATER
OVERFLOW ORIFICE
PERMEABILITY STAND PIPE
STOP COCK CLOSED FOR PERMEABILITY TEST
STOP COCK CLOSED FOR COMPRESSION TEST
COMPRESSION TEST: WATER PRESSED FROM SOIL PAT BY WEIGHTED PISTON, PASSES THROUGH POROUS STONE ABOVE AND BELOW SOIL PAT AND ESCAPES FROM OVERFLOW ORIFICES A AND B. THE VOIDS RATIOS e ARE COMPUTED FROM THE KNOWN AND MEASURED VALUES
PERMEABILITY TEST: WATER, FROM PERMEABILITY STAND PIPE, FLOWS UPWARD THROUGH POROUS STONE, SOIL PAT, AND POROUS STONE AND ESCAPES FROM OVERFLOW ORIFICE A. THE PERMEABILITY COEFFICIENT "K" IN CENTIMETERS PER SECOND IS COMPUTED FROM THE FORMULA BELOW
K = 2.3 a₂d/a₁t × Log h₁/h₂
WHERE a₂ = AREA OF STAND PIPE IN SQUARE CENTIMETERS
a₁ = AREA OF SOIL PAT IN SQUARE CENTIMETERS
d = THICKNESS OF SOIL PAT IN CENTIMETERS
t = TIME IN SECONDS FOR MENISCUS OF WATER COLUMN TO FALL FROM ANY HEIGHT h₁ TO ANY HEIGHT h₂
h₁ AND h₂ = HEIGHTS OF WATER COLUMN IN CENTIMETERS ABOVE OVERFLOW ORIFICE A

Figure 7

3 Expansion or the tendency for the soil to take up moisture after being compressed.

The essential features of the compression test and its practical application is illustrated in Figure 7.

As originally designed by Terzaghi,[3] and shown diagrammatically at the left of Figure 8, the compression device furnished means for measuring only the compressibility and the expansion of soils Also, the soil sample was quite small, being approximately one centimeter thick and 39 square centimeters in area

In the practical use of the compression test data, the performance of foundation soil layers many feet thick is estimated on the basis of

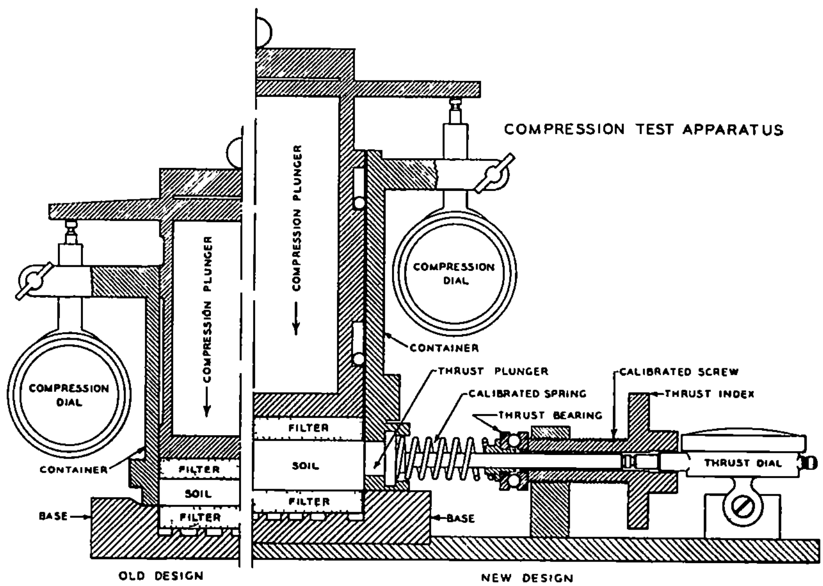



**Figure 8. Compression Device, Old and New Designs**

the movements of these very thin soil samples This requires measurement of the very minute compressions and expansions of the soil sample to an accuracy of about one ten-thousandth of an inch In order that measurements of this degree of refinement will be representative of soil movements and not seriously influenced by lag in the apparatus, the sliding parts of the apparatus have to be made so snug that experimental errors due to friction between the plunger and container are apt to occur

The new apparatus, as devised by the Bureau of Public Roads, is shown at the right of Figure 8 It differs essentially from the original design as follows

[3] See Public Roads, Vol 12, No 4, June, 1931, pages 11 and 12

1 It utilizes ball bearings to minimize the effects of friction between the plunger and the container wall

2 It provides space for a soil sample about twice as large (2 2. cm. thick by 73 sq cm. in area) as the one utilized in the original tests.

3 It is equipped to measure the lateral thrusts of the soil cakes at any stage of the compression test

This thrust is measured by reducing the compression on the calibrated spring which holds a small plunger against the side of the soil cake until the spring force just balances the lateral thrust of the soil cake The essential feature in this operation is that the measurement of the lateral thrust is accomplished without appreciable lateral movement or bulging of the soil cake Reducing the spring force by means of the slow motion

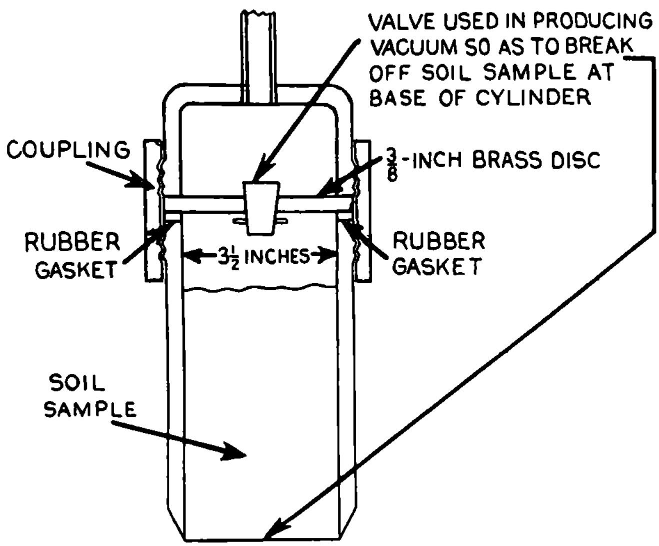



**Figure 9. Sampler for Obtaining Cores of Undisturbed Undersoil**

screw until the first tendency to move is registered by the dial, restricts the bulging of the soil cake to an infinitesimal amount

## 4 SOIL SAMPLERS

The density of completely saturated soil samples may be quickly computed from a knowledge of the moisture content of the soil. The density of soils containing both air and water, however, can be determined only from samples of soil in the undisturbed natural state. In addition to the cooperation of the Bureau with the Massachusetts Institute of Technology in the development of quite elaborate devices for securing samples of foundation soils from deep in the ground, several simple devices have been used by the Bureau for obtaining samples of the desired type

Figures 9 and 11 show the device used in an investigation of the settlement of a fill placed on a soft unstable undersoil It consists of a piece of four-inch pipe 18 inches long with a ground cutting edge as shown This pipe is fastened to a head by means of a union and the head is in turn fastened to the drill rod or pipe Inside the head is a metal diaphragm with a hole in the center, which acts as a seat for a rubber stopper forming a valve This valve allows air or water to escape as

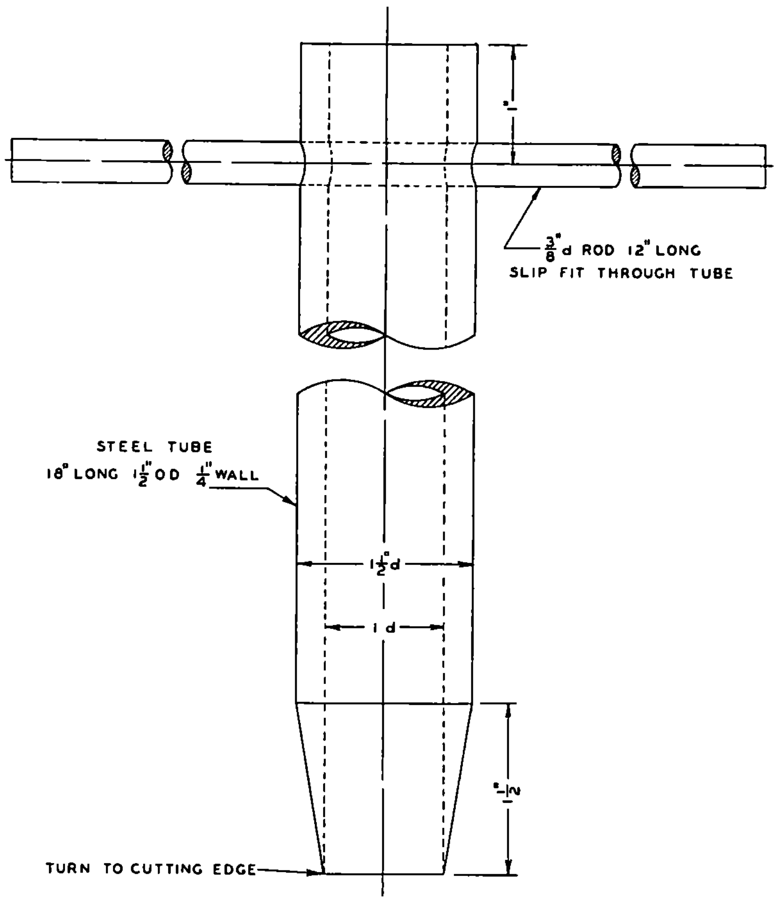



**Figure 10. Hollow Sample Tool for Subgrade Investigation**

the sampler is being forced into the soil but closes as soon as withdrawal is started It is used in connection with a wash boring outfit

Samples obtained with this device are shipped to the laboratory in the sampler bit after they have been sealed with paraffine and are there tested in the undisturbed state

Another type of sampler used in determining the degree of compaction obtained in subgrade or fill construction, consisting of a slightly tapered steel bit is shown in Figure 10 This is forced into the soil, removed and the sample is pushed out, weighed, dried and weighed again Knowing the volume of the sample and the specific gravity of the soil it is a simple matter to compute the relative amounts of soil, air and moisture in the natural soil mass

Figure 11 shows the details of another type of bit and holder,[4] and plunger (*H*)

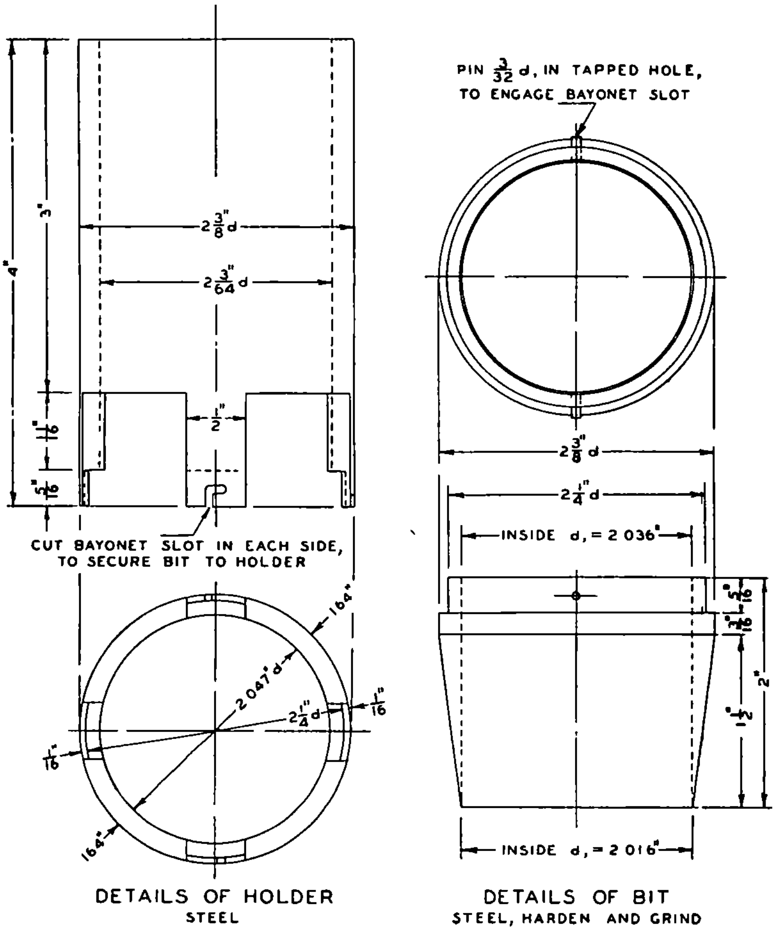



**Figure 11 Subgrade Sample Bit and Holder**

The bit attached to the holder is forced into the soil until the bit is entirely filled The soil is then removed from the bit, trimmed carefully to both rims of the cutter and then removed for the weighing and drying operations

The slight flare of the inside of the bit is for the purpose of reducing the effects of friction between the soil core and the bit.

[4] Design suggested by W I Watkins of the U S Bureau of Chemistry and Soils, cooperating in subgrade investigations of the Bureau of Public Roads

### 5. NEW SAMPLE CUTTING APPARATUS

Of special importance in connection with determining the bearing value of undisturbed soil samples is the manner in which the sample is transferred from the soil core as obtained in the field to the compression device to be tested.

Not only must the sample be cut to the exact diameter of the container but special precaution must be taken to make the depth of the disturbed soil at each face of the soil cake as small as possible. Furthermore, all work of this character must be done in a damp closet to prevent the moisture content of the soil sample from changing prior to test.

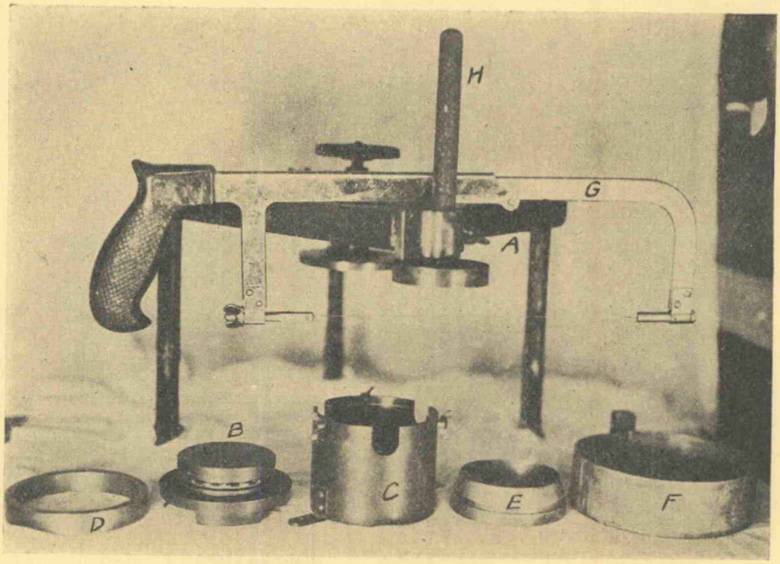



**Figure 12. Essential Parts of New Sample Cutting Apparatus**

To accomplish these purposes the apparatus shown in Figure 12 was devised. It consists of a stand (*A*) with a slow motion screw and thrust bearing (*B*), a cutter support (*C*), a guide ring (*D*), a special cutter (*E*), a thin metal band (*F*), a piano wire saw (*G*) and a plunger (*H*).

The cutting of the test sample and the placing of it in the testing device is accomplished by six distinct operations proceeding in the following order:

(1) A portion of the undisturbed soil is cut from the core as received from the field and placed in the metal band *F*, and the space between the band and the circumference of the soil cake is filled with melted paraffine.

(2) The soil sample thus prepared and the cutting apparatus are arranged as shown in Figure 13.

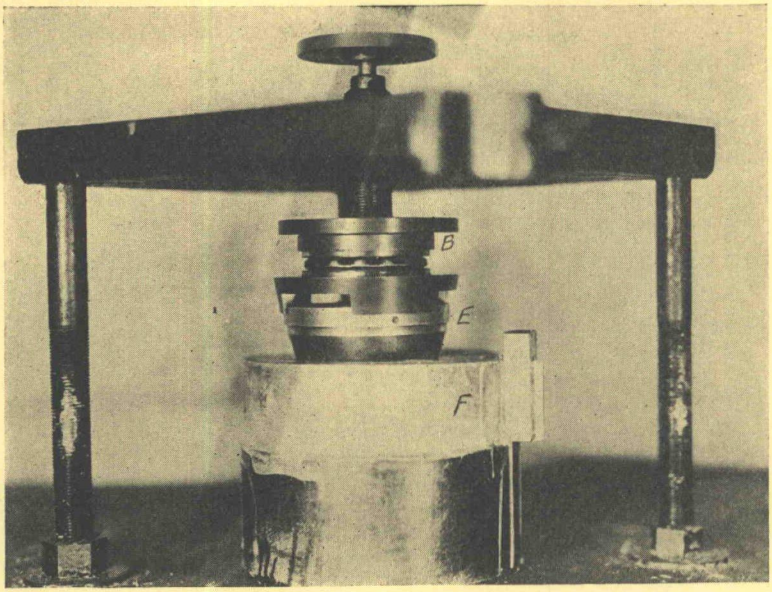



Figure 13. Assembly of Apparatus for Cutting the Samples from the Soil Core

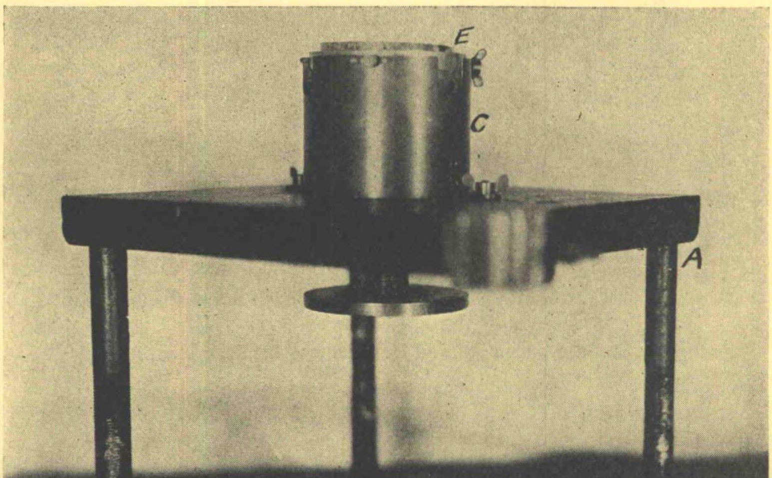



Figure 14. Apparatus Arranged for Trimming One Face of the Sample Flush with the Cutter Rim

(3) The cutter is forced carefully into the soil cake by means of the slow motion screw until the soil extends far enough above the top of the cutter to permit it to be smoothed off to the plane of the top of the cutter.

(4) The cutter containing the sample is now arranged in the holder as shown in Figure 14 and the portion of soil extending above the cutter is smoothed off carefully to the plane of the top of the cutter by means of the piano wire saw.

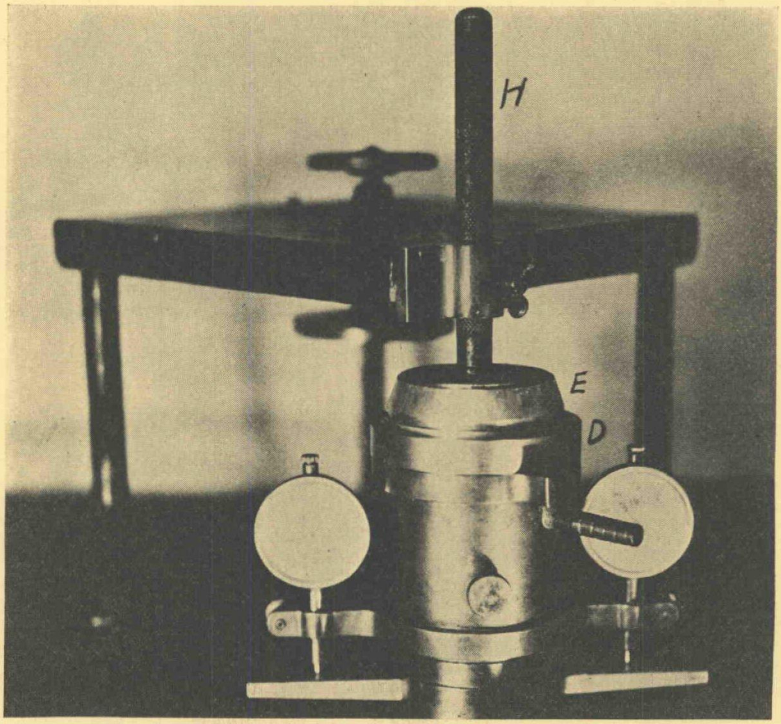



**Figure 15. Sample being pushed from Cutter into the Testing Apparatus**

(5) The sample, cutting apparatus and the testing device are then arranged as shown in Figure 15.

(6) The soil sample is then forced slowly into the testing container by means of the plunger (*H*), and trimmed off flush with the container rim, after which the testing apparatus containing the soil sample is ready to assemble for test as shown in Figure 8.

### 6. DRAINAGE INDICATOR

In contrast to the simplified soil tests which furnish qualitative information on important soil properties, the new device, termed the drainage indicator, is intended to furnish quickly and with minimum effort quantitative information on the following soil properties important with respect to drainage and frost heave

1 The relative amounts of air, gravitational moisture, capillary moisture and soil particles comprising the soil mass

2 The maximum capillary rise

3 The rate of capillary rise and the quantity of water furnished by capillarity at any distance above ground water elevation

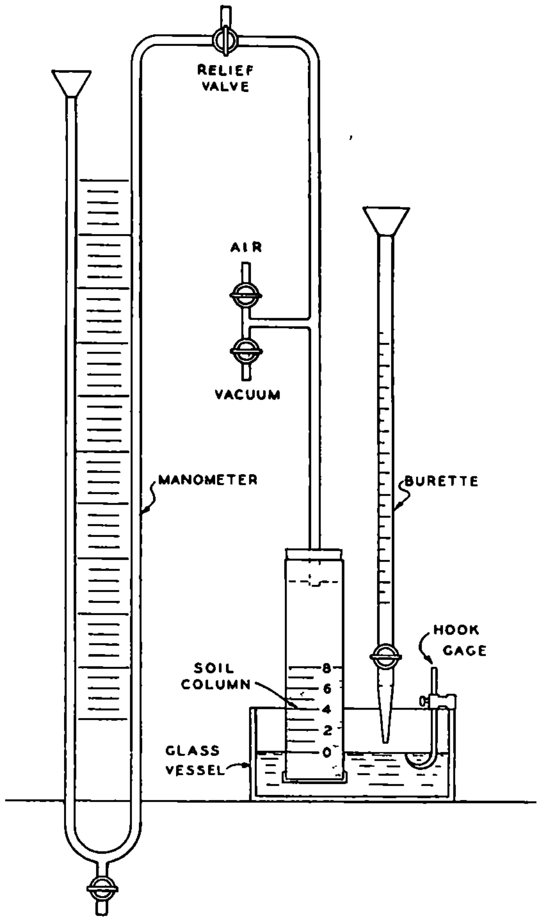
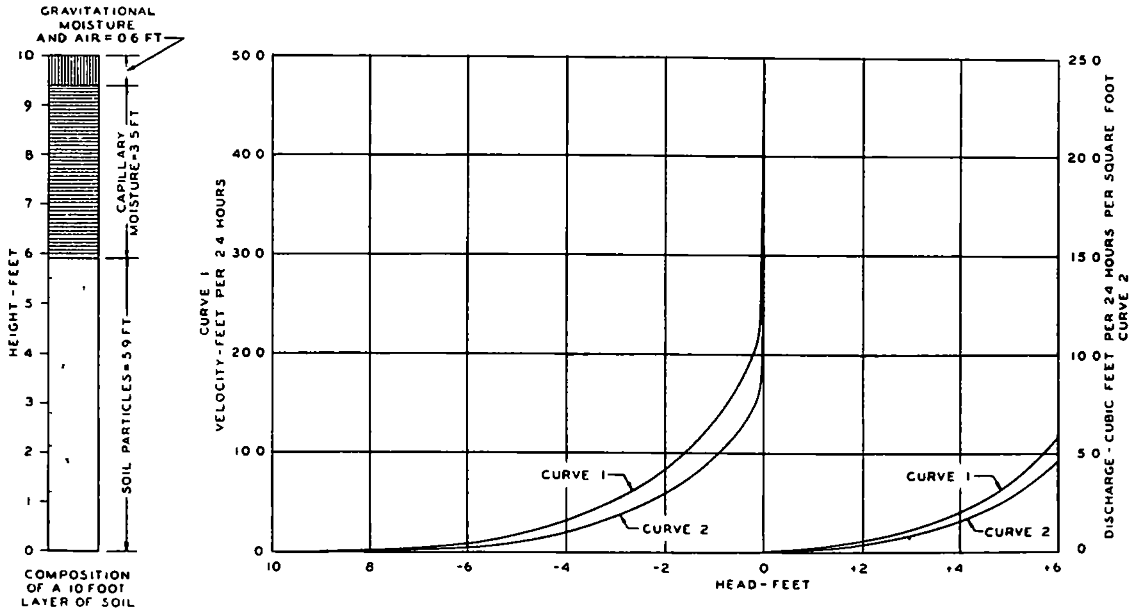



**Figure 16 Essential Features of the Drainage Indicator**

4 The velocity of gravitational water movement and the amount of water furnished under any head

This object is accomplished by observing the rise of water in the sample of soil under varying conditions of pressure in the apparatus shown schematically in Figure 16 The essential parts are a graduated glass container for the soil sample, a hook gage and a burette for measuring the quantity of water taken up by the sample and a mercury




DRAINAGE PROPERTIES - SOIL NO 6518

**Figure 17. Data Furnished by the Drainage Tests**
**Curve 1, left. Velocity of capillary moisture.**
**Curve 2, left. Discharge of capillary moisture.**
**Curve 1, right. Velocity of gravitational water.**
**Curve 2, right. Discharge of gravitational water.**

manometer for indicating the pressure resisting or assisting the entrance of water

Physically the test is based on the following three assumptions.

1 The pressure head required to prevent the entrance of moisture into the soil equals the maximum capillary rise

2 The pressure head against which the moisture penetrates the sample at a given observed rate equals the height above the ground water elevation at which capillary moisture would flow at a similar rate in a column of soil similar to that of the sample

TABLE IV

DRAINAGE PROPERTIES OF A SAND AND A SILT SOIL COMPARED

| Soil | | Beach sand | Silt soil |
|---|---|---|---|
| | | *feet* | *feet* |
| Composition of 10 feet of saturated soil | Soil particles | 6 5 | 4 9 |
| | Capillary moisture | 0 1 | 4 7 |
| | Gravitational moisture and air | 3 4 | 0 4 |
| Capillary rise per 24 hours | 1 5 feet above ground water elevation | 0 0 | 5 2 |
| | 3 0 feet above ground water elevation | 0 0 | 3 0 |
| | 6 0 feet above ground water elevation | 0 0 | 0 9 |
| Maximum capillary rise | | 0 4 | 12 0 |
| | | *cu ft* | *cu ft* |
| Capillary flow per 24 hours per sq ft | 1 5 feet above ground water elevation | | 2 7 |
| | 3 0 feet above ground water elevation | | 2 0 |
| | 6 0 feet above ground water elevation | | 1 0 |
| Gravitational flow per 24 hours per sq ft | 1 5 feet head | 22 5 | 0 1 |
| | 3 0 feet head | 39 7 | 0 4 |
| | 6 0 feet head | 65 7 | 2 4 |

3 The pressure head required to draw water through the sample at a given observed rate equals the distance below the gravitational water surface at which percolation would occur at a similar rate

The use of controlled pressures in the manner described eliminates the experimental errors introduced by the long tube formerly used in investigating the capillary properties of soils

The results furnished by this apparatus disclose very readily the monstrous amount of water which can be furnished by capillarity They show, for instance, that both the rate of capillary rise and the thickness of water raised in 24 hours can be measured in feet instead of inches

Figure 17 depicts the drainage test graphically It shows the makeup of a ten-foot layer of this soil to be 5 9 feet soil particles, 3 5 feet capillary moisture and 0 6 foot gravitational moisture and air The maximum capillary rise in this case is nine feet The rate of rise at a distance of three feet above the ground water elevation is 5 5 feet per 24 hours. At 1 5 feet above the ground water elevation the rate is 10 4 feet per 24 hours

The quantity of water furnished by capillarity is sufficient to form a layer 3 8 feet thick every 24 hours at a distance of 1 5 feet above the ground water elevation and a layer 1 9 feet thick in the same time at a distance of three feet above the ground water elevation

Table IV furnishes additional information of this character The difference between the drainable, non-frost-heaving beach sand and an unstable frost-heaving silt is quite evident Here it is indicated that in a ten-foot layer of soil the thickness occupied by the capillary moisture in the silt is 4 7 feet, while that in the sand equals but 0 1 of one foot This leaves in the sand an equivalent of 3 4 feet of space for the flow of gravitational water in contrast to but 0 4 of a foot in the silt As would be expected, this difference is reflected in the gravitational flow which in the sand varied from 22 5 to 65 7 feet in 24 hours where for equal heads the flow in the silt varied from but 0 1 to 2 4 feet in the same time Furthermore, the absence of capillary flow in the same is conspicuous

## 7 FLOCCULATION TEST

A soil mass consists of both soil particles and pores When soil swells or shrinks only the pore space is presumed to change Thus, when rain changes clay from the dry to the wet and muddy state, the clay particles do not necessarily change either in size or character but have merely separated due to entrance of water into the pores between the particles

The clay becomes muddy and unstable merely because the percentage of pores in the soil mass increases very appreciably While the voids ratios of dry clay samples may be but half those of stable sands, the voids ratios of muddy clays may be five times those of the wet sands The clay would become as stable as the sand if the pore space of the wet clay could be reduced to that of the wet sand and maintained there

Consequently, the maximum porosity which a soil is capable of acquiring in the presence of moisture becomes an index of the stability of the soil This, combined with the fact that the maximum water capacity of a soil is disclosed by the porosity of soil sediments, forms the basis of the flocculation test

In considering the sediments it must be remembered that soil particles may be of several kinds. bulky grains of any size, large size flat micaceous particles, spongy diatomaceous particles and the scale-like colloidal particles

Soils consisting of the bulky particles exist in nature in more or less compact masses as shown in Figure 18A Soils consisting of the platy or spongy particles are highly porous, either wet or dry.

The clay soils containing scale-like colloids having the electrolytic properties productive of flocculation may exist in nature in a dried state much denser than bulky grained soils, but in the presence of water may expand to an extended state, such as is illustrated in Figure 18B

When bulky grained soils settle out of suspensions, the grains act independently, the larger ones settling the fastest to form stratified sediments with the larger particles on the bottom, as shown in Figure 18C In flocculated soils, in contrast, the larger clay particles are trapped in the masses of flocks in such a manner that independent particle settlement is prevented with the result that in the sediments, the larger clay particles are distributed throughout the flocculated mass, as shown in Figure 18B

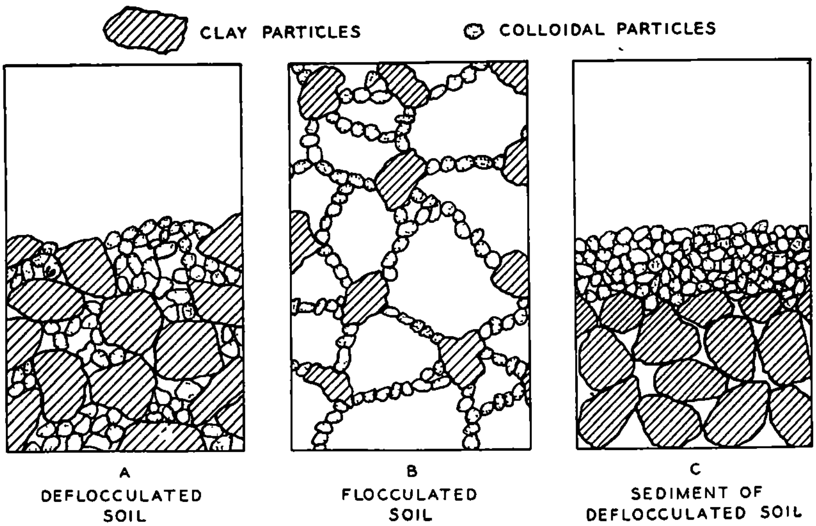



**Figure 18. Bulky Grained and Flocculated Soils**

Only when a deflocculating agent destroys the electrolytic bond, which causes the extended order of the particles, are the grains released to settle individually like the bulky grain soils to form the stratified sediments, Figure 18C

The porosity of the platy and spongy materials is due to the character of the grain instead of an electrolytic property. Consequently, deflocculating agents do not serve to reduce the porosity of such soils

The test (see Figure 19) consists essentially of determining the voids ratio of first the dry and powdered soil and, second, the sediment which occurs 24 hours after known volumes of soil particles have been thoroughly mixed with known volumes of (a) water and (b) water with the sodium silicate deflocculent

The voids ratio of the soil powder, Figure 19 left, is 1 0 that of the flocculated sediment, Figure 19 middle, is 3.5; and that of the deflocculated sediment, Figure 19 right, is 1.6.

These voids ratios are obtained by subtracting the absolute volume of the soil particles, five cubic centimeters in this instance, from the volumes of the soil masses shown by the graduates and dividing the remainders by the absolute volume of the soil particles. The soil in the middle graduate was mixed with 40 cc. of distilled water and that in the graduate on the right with 39 cc. of distilled water and one cubic centimeter of sodium silicate dispersing solution.

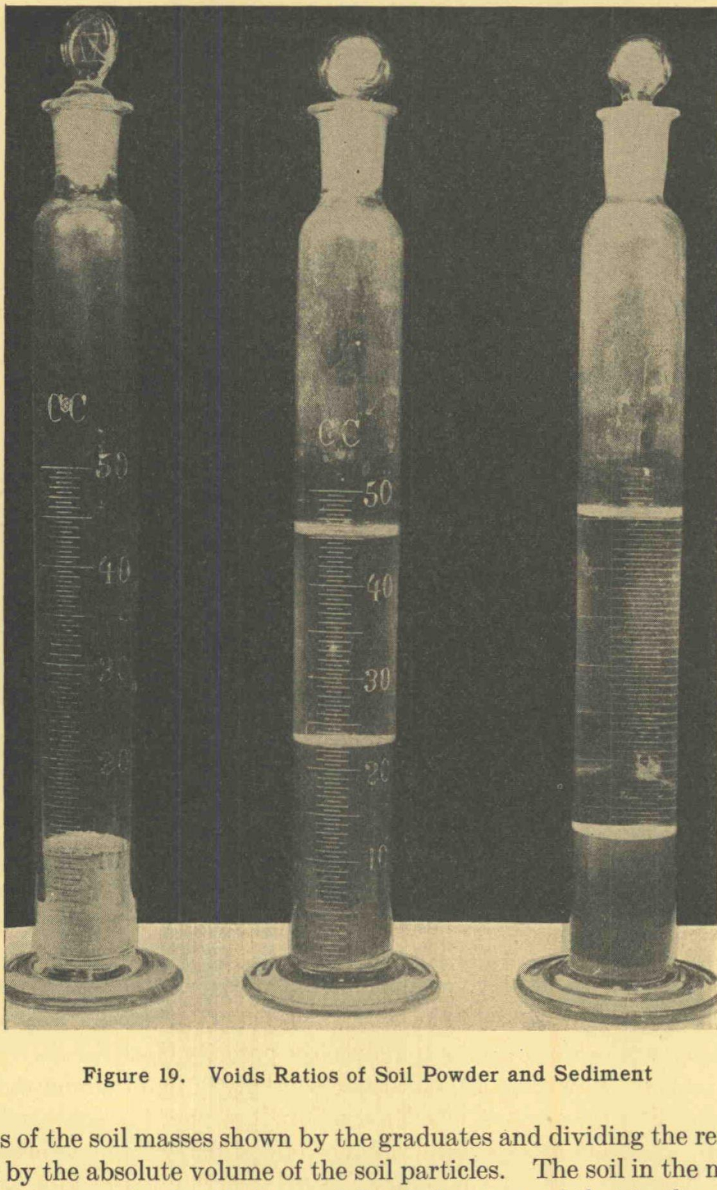



Figure 19. Voids Ratios of Soil Powder and Sediment

Similar to Figure 18C, the dispersed sediment, Figure 19 right, is stratified, whereas the flocculated sediment, Figure 19 middle, is of uniform appearance, like Figure 18B.

In connection with the above voids ratios it should be remembered that a voids ratio of about 0.35 represents the densest and that of about 0.90 represents the loosest possible arrangement of bulky spherical particles of equal size. These voids ratios for average soils correspond to moisture contents of 13.2 per cent and 34 per cent, respectively.

TABLE V

Results Furnished by the Flocculation Test

| Soil no. | Clay content | Voids ratio | | |
|---|---|---|---|---|
| | | Dry powder | Natural sediment | Deflocculated sediment |
| | *per cent* | | | |
| A | 48 | 1.0 | 5.5 | 0.8 |
| B | 54 | 1.4 | 1.2 | 1.2 |
| C | 42 | 0.9 | 3.4 | ..0.8 |

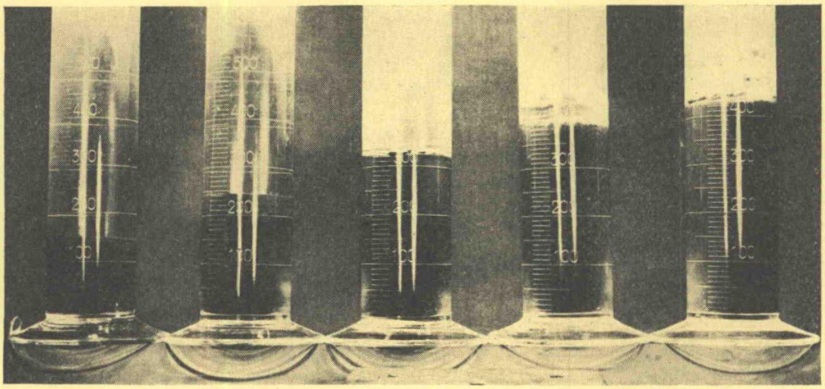


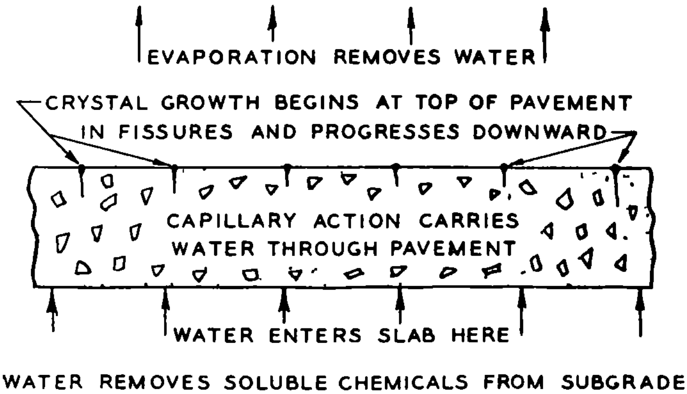
**Figure 20. Effect of Flocculation on the Density of Muck Sediments**

It is possible for spherical particles of graded sizes to have voids ratios smaller than 0.35. Usually, however, the presence of flat scale-like particles is indicated when wet undisturbed soil samples can be compressed to voids ratios smaller than 0.35, or by shrinkage limits of disturbed samples smaller than 13.

In the same manner the flocculated extended order illustrated in Figure 18B is indicated more and more as the voids ratios of sediments or undisturbed natural samples exceed 0.90 and liquid limits of disturbed soil samples exceed 34 per cent.

Generally the voids ratios of 24-hour sediments are as follows: Coarse sands, 0.6 or less; fine sands, 0.6 to 0.8; ground rock powders, bulky grained clays, 0 8 to 1 2, average silty soils 0 8 to 1 6, flocculated colloidal clays, 1 6 to 3 5, muck and peats, as high as 5 0 or 6 0 High voids ratios of sediments which are not reduced by deflocculating agents indicate the presence of spongy or elastic soil materials

Table V shows that the degree of consolidation which a soil will acquire upon settling is not controlled by the clay content For instance, in a total thickness of 10 feet, the natural sediment of soil B with 54 per cent of clay will consist of 4 5 feet of soil and 5 5 feet of water, whereas that of soil A with 48 per cent of clay will consist of 1 5 feet of soil and 8 5 feet of water Deflocculated, however, the layer of soil A becomes the more stable consisting of 5 6 feet of soil and 4 4 feet of water




GROWTH OF CRYSTALS
BY CHEMICALS THROWN OUT OF SOLUTION

**Figure 21. Water Movement Productive of Crystal Growth**

Figure 20 illustrates how flocculation may exert an influence upon the porosity of sedimentary soils in nature These sediments of the same muck soil which have been forming under observation for nine months have voids ratios ranging between 5 to 19, due to different degrees of flocculation produced by the use of different amounts of a chemical admixture

## 8 CRYSTAL GROWTH

A discussion of the disintegration of concrete, Proceedings Eleventh Annual Meeting of the Highway Research Board, page 299, suggests that certain soluble chemicals of the subgrade soil might be a contributing factor

These chemicals in solution are carried up through the concrete slabs by capillary action, after which they are deposited in crystalline form near the top of the slab when the carrier liquid evaporates

Especially productive of the movement of these solutions through the slabs are certain types of almost invisible fissures. And the first evidence of disintegrating action of this type is the appearance of crystalline

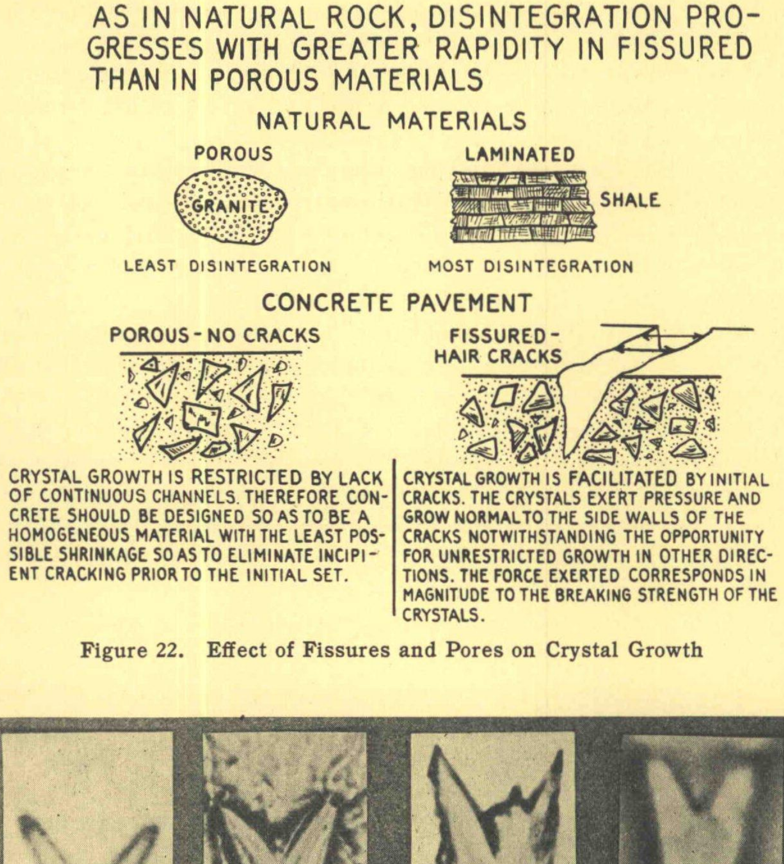



**Figure 22. Effect of Fissures and Pores on Crystal Growth**

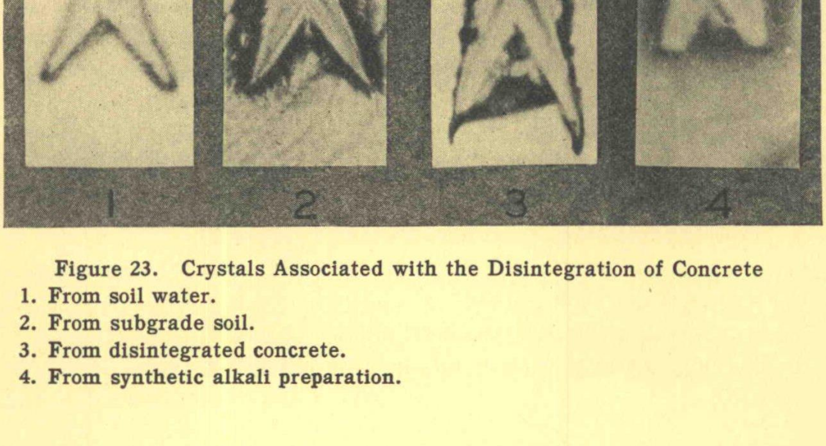



**Figure 23. Crystals Associated with the Disintegration of Concrete**

1. **From soil water.**
2. **From subgrade soil.**
3. **From disintegrated concrete.**
4. **From synthetic alkali preparation.**

deposits in the form or ridges directly over the fine, almost microscopic fissures

Continuation of this action, illustrated in Figures 21 and 22 A, causes the slab to expand until failure occurs due to the increasing compressive forces when joint space is limited and to the disruptive force of crystal growth otherwise

The microscopic method of attack was inaugurated because of its comparative simplicity as compared with an investigation requiring chemical analysis

Essentially it consists of mixing either powdered soil or the ground disintegrated cement mortar, with a small amount of distilled water, filtering the liquid, and allowing a small quantity of the filtrate to evaporate on a microscopic slide which is then examined for crystalline formations

These investigations are primarily in the formative stage Nevertheless, they have progressed far enough to disclose the fact that the presence of at least one type of crystal in the soil should be viewed with suspicion This type which has been found in the soil under certain disintegrated slabs, together with crystals from the ground water and also from the crystal deposits occurring over fissures in top of the same slabs is illustrated in Figure 23

The sawbuck character of these crystals and also that furnished by synthetic alkali mixtures is quite similar

### 9 DETRIMENTAL PAVEMENT WARPING

Request for a subgrade survey several years ago first called attention of the Bureau to an unusual type of permanent warping which had occurred in certain portions of a concrete pavement constructed in Mississippi

Since that time, high concrete pavement joints have been found to occur in parts of certain projects in other states with such monotonous regularity as to furnish the traveling public with a decidedly unpleasant impression that something has gone amiss in the construction of the road In a limited number of these projects the pavements have risen so perceptibly at the joints and also at transverse cracks as to be not only seriously disagreeable, but in extreme instances dangerous to fast moving traffic In such cases the drivers prefer to run with two wheels on the shoulder instead of with four wheels on the pavement

In the planning of the investigation possible causes of high joints were recognized as follows

a Improper finishing of the road surface

b Normal expansion of the concrete combined with "frozen dowels" across expansion joints

c Insufficient width of expansion joints

d Heaving of the pavement adjacent to cracks or joints e Settlement of portions of pavement not adjacent to cracks and joints

f Permanent shortening of the top, without equal shortening of the bottom of the pavement

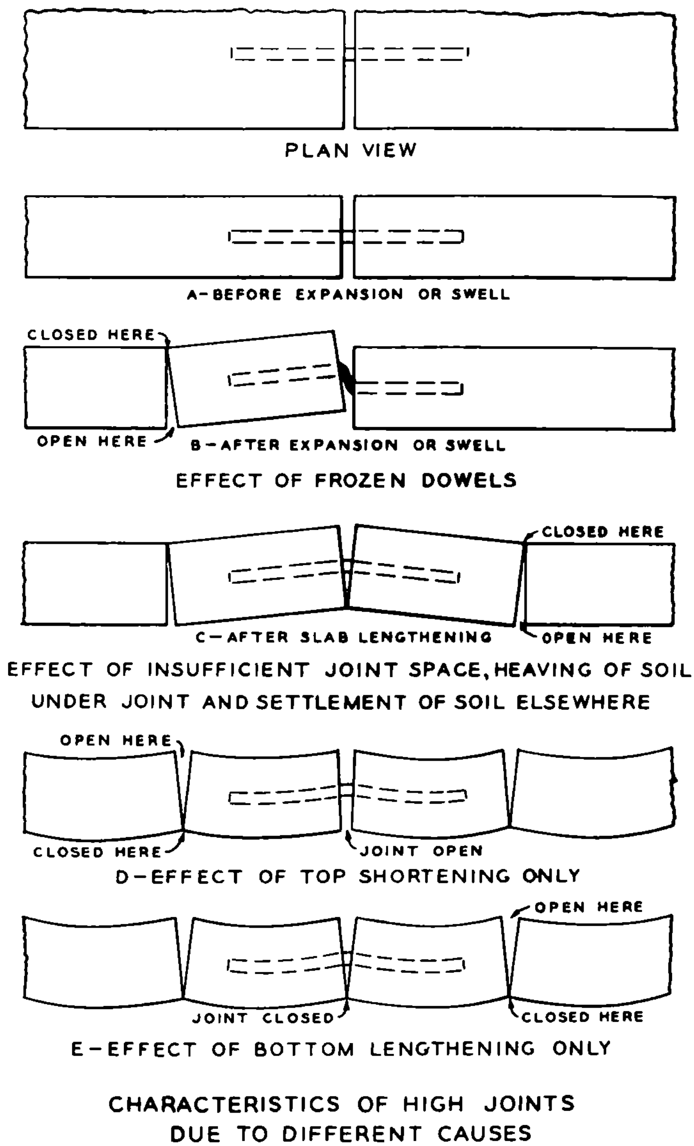



**Figure 24. Characteristics of High Joints in Concrete Pavements**

g Permanent lengthening of the bottom, without equal lengthening of the top of the pavement

Inspections of warped pavement, however, disclosed the following with respect to warped slabs

1 Warping does not occur until some time after construction
2 It causes the edges of slabs along cracks and sides to be elevated in the same manner as the edges along joints
3 It occurs primarily in slabs laid on certain soils of the A-7 subgrade group and has not yet been found in slabs laid on sand subgrades

Consequently, the first three possibilities listed above are eliminated as prime causes High joints, due to improper construction for instance, would be observed as soon as the pavement was open to traffic instead of sometime afterward Likewise, observations have not disclosed the presence of the essential features indicative of frozen dowels, shown in Figure 24B and listed as follows.

(a) Cracking or even some spalling occurring in the concrete located over the dowel
(b) Faulting at the joint, as shown in view 24B
(c) The occurrence of a crack several feet back of the joint, closed at top and open at bottom of slab
(d) No elevation of the slab at either transverse cracks or the side edges of the pavement

Likewise, the effect of the thrust, due to insufficient joint space, and illustrated in Figure 24C are absent Instead of the faulting, as noted in Figure 24B, the slab at both sides of the joint, in this case, is apt to rise in equal amount But similar to the case of the frozen dowels, there will be no warping of the side edges of the pavement when the joints are elevated due to thrust

This leaves then the difference in volume change in top and bottom of slabs, illustrated in Figures 24D and 24E, as the prime cause of the trouble. If the top fibers shortened, the joint would remain open, as shown in Figure 24D, and if instead the bottom fibers lengthened, the bottom of the joints would be closed, as shown in Figure 24E In all cases the cracks would be wider at the top than at the bottom, the elevations would occur at transverse cracks, as well as joints, and the side edges of the pavement would be elevated

Volume changes of different amounts in tops and bottoms could be due to

(a) Continuous wetting of the bottom, drying out of the top, or a combination of the two
(b) Crystal growth in the bottom of the slab, due to crystallization of chemicals carried in by the ground water
(c) Flow of the rigid concrete due to some combination of shrinkage and expansion
(d) Flow of concrete due to permanent strains produced in the slabs by some combination of load and subgrade support

The trend in this study is toward the construction of experimental road slabs in the field and miniature slabs in the laboratory, specially designed to reveal which of the possibilities previously referred to are responsible for the objectionable behavior and to indicate satisfactory methods for eliminating the detrimental effects produced by it

Essential features in these experimental designs concern

1 The waterproofing of the bottoms of the slabs
2 The incorporation of steel reinforcement to resist the flow of the concrete under strain
3 Stabilization of the elastic clay subgrade by means of bituminous treatments or the introduction of layers of granular materials

### 10 PROCEDURE FOR DETERMINING FROST HEAVE DATA

In the last analysis there are two essential requirements for frost heave They are (1) freezing temperatures in the soil and (2) the presence of moisture in amounts necessary for the formation of ice crystals

The preventive measures, therefore, are of two types Those proposed to prevent the detrimental cold temperatures from reaching the subgrade and those proposed to eliminate the offending moisture

Only by the use of insulating materials can there be any hope of preventing freezing temperatures from reaching the subgrade There are several possibilities for eliminating the offending moisture They are, change of soil character by treatment, change of soil character by substitution with another soil and drainage

An example of changing the character of the soil as regards its ability to freeze, is treatment by either common salt or calcium chloride brine The freezing temperatures of brines are as follows Common salt, 15 per cent, 12°F , common salt, 23 per cent, −6°F , calcium chloride, 15 per cent, 12°F , calcium chloride, 25 per cent, −25°F

Substitution of coarse granular material for cohesionless silts as a preventive measure is well known as is also the drainage of permeable soils

The use of insulating materials for preventing freezing temperatures from reaching the subgrade, a type of preventive measure practically untouched at present, is not only practical but very promising This is suggested by the fact that a covering of snow on the pavement very appreciably reduces the amount of frost heave in the subgrade In this connection it is interesting to note the great difference in the heat conductivity[5] of different materials This is illustrated by the relative thickness of various materials required to furnish the same resistance to heat transfer as follows

| | |
|---|---|
| Commercial insulators | About 1 0 |
| Dry peat | 1 3 |
| Lumber and dry soil | About 3 0 |
| Dry sand | 7 5 |
| Concrete | 19 0 |

[5] Average B T U passing per hour through a plate of material one square foot in area, one inch thick, per degree Fahrenheit difference in temperature of the two faces

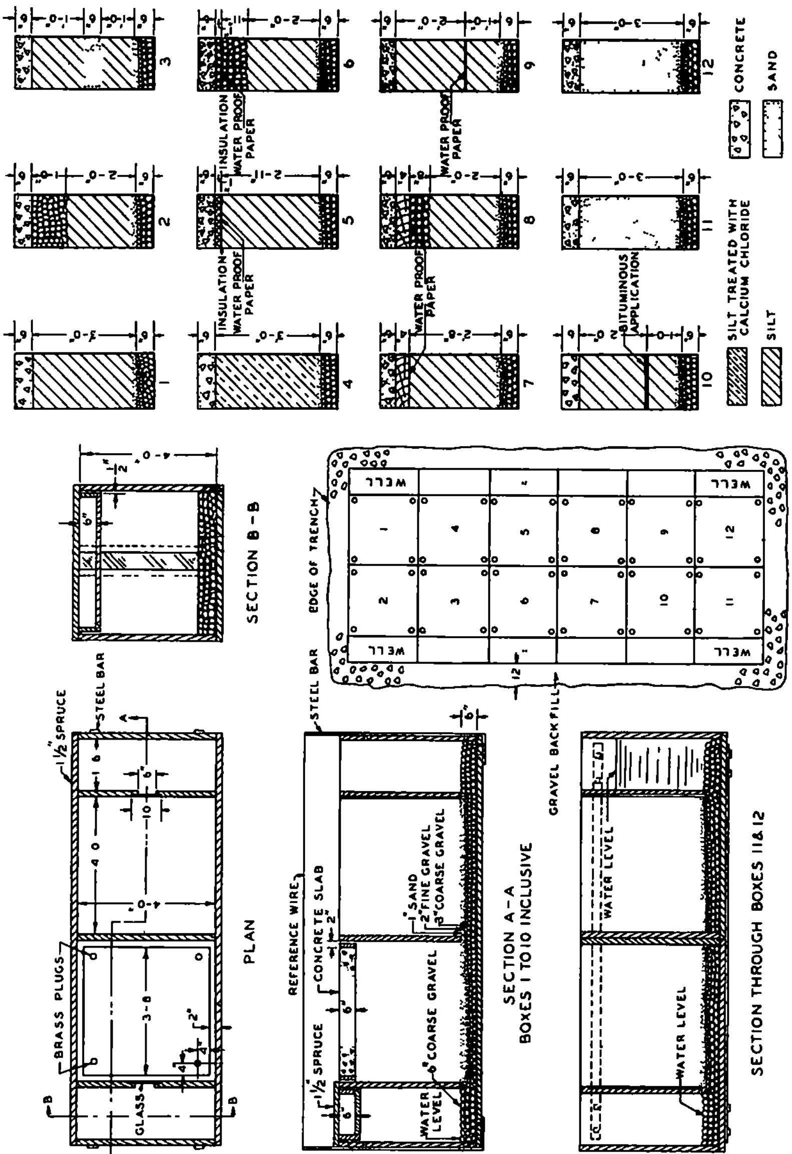



Figure 25 Details of Frost Heave Experiment

Among the commercial insulators is cork board with bituminous binder and this suggests that some combination of sawdust, peat or straw, etc , with bituminous binder might serve very efficiently as insulating material

Since peat is plentiful in many frost heave locations, thin layers of peat waterproofed by impregnation with bituminous material furnishes one of the best practical possibilities

Laboratory experiments have furnished valuable information on the mechanics of frost heave and the heaving characteristics of various soils They are not adapted, however, for furnishing reliable information on preventive measures

On the other hand, it is economically impossible to control such influencing variables as climate, amount of water available and soil profile in even a few full scale experimental pavement sections It is evident, therefore, that in order to secure quantitative information on the variables productive of frost heave and to determine the relative efficiency of preventive measures, the laboratory and field experiments must be supplemented by outdoor experiments extensive enough in character to eliminate errors due to the use of small size laboratory specimens and small enough to come within the range of practical cost considerations

Such a series of controlled outdoor experiments is illustrated in Figure 25 It is similar in character to the experiments performed at the Massachusetts Institute of Technology in cooperation with the U S Bureau of Public Roads, described in the Proceedings of the Eleventh Annual Meeting of the Highway Research Board, pages 168 to 172

The suggested experiment includes the use of four different ideas regarding preventive measures, as follows

1 Drainage of the permeable soils

2 Substitution of non-heaving coarse granular material for fine grained, cohesionless frost-heaving silt soil

3 Introduction of an impermeable under layer to cut off the flow of capillary moisture productive of frost heave

4 Incorporation of a chemical to reduce the freezing temperature of the natural soil

5 Introduction of an insulating layer to reduce the possibility of freezing temperatures reaching the subgrade soil

Figure 25 is intended to be suggestive of types of treatment. It is not supposed to contain all the possible types nor are all the treatments it suggests claimed to be practical from standpoint of cost in road construction

## 11 SOIL STABILIZATION

There are two general methods of improving the stability of a given soil The first consists of incorporating with the soil other inert soil materials in such proportions that the resulting mixture will have the properties of stable, well graded subgrades. This is the method which has been followed largely in the past and is discussed in Public Roads, Vol. 12, No. 5, July, 1931, pages 38 and 39.

The second method consists of treating the soil with materials designed to prevent the soil from acquiring a detrimentally large voids ratio in the presence of moisture.

Whereas the first method utilizes a change in the grading of the soil to effect a change in physical characteristics, the second method does not contemplate change in grading but instead proposes to reduce the water capacity of the soil by the incorporation of chemical or physical admixtures.

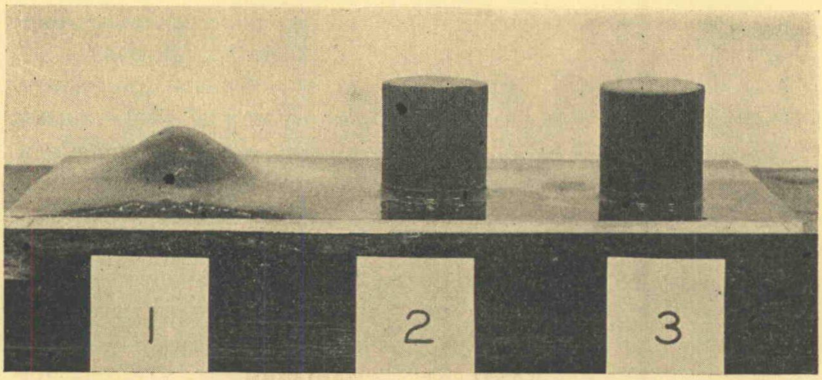



**Figure 26. Effect of Water on Treated and Untreated Silt Soil Samples after Equal Periods of Immersion**

1. **Untreated.**
2. **Portland cement admixture.**
3. **Bituminous material admixture.**

There is no intention to produce a hard rigid slab by means of the second method of treatment now being investigated but simply a firm semi-damp slab whose moisture content shall not exceed the plastic limit.

To date the efforts have been confined to determining the effect of introducing four different admixtures into the soil pores. They are light bituminous material, portland cement, ferric hydroxide and calcium silicate.

The general scheme is to carry the portland cement and emulsified bituminous material into the pores by mixing with amounts of carrier required to completely satisfy the water capacity of the untreated soil as indicated by its liquid limit. The ferric hydroxide is produced by treatment of ferric chloride with ammonia. The calcium silicate is produced by treating first with sodium silicate and then with calcium chloride.

Whereas a cubic foot of sand may have a surface area of 100 square feet, one of silt (0 02 mm ) may have 50,000 square feet, or one acre, and one of colloidal material 1,000,000 square feet, or 20 acres Consequently the same bituminous material, or other admixture which may serve to treat the sand, must be greatly diluted to be distributed over the silt and clay particles

Thus for a soil with a liquid limit of 50, the liquid container with the admixture would equal 50 per cent of the weight of the soil Furthermore, if it was desired to incorporate five per cent of the admixture in soil, this five per cent of material would have to be suspended in water or other carrier equalling 45 per cent of the soil

The carrier evaporating subsequently leaves 5 per cent admixture thoroughly distributed throughout the soil

Figure 26 shows some of the relative effects of treatments for increasing the resistance of silt soils to slaking

This problem requires not only the determination of the materials best suited for stabilizing purposes but also the development of the methods and equipment required in practice

In closing, attention is called to the fact that subgrade investigations have passed from the experimental into the practical stage According to a survey made by Division 1 of the Committee on Materials of the A A S H O , 13 states of the country and the District of Columbia, have laboratories equipped to perform all the routine tests employed by the Bureau of Public Roads in their soil investigations. An equal number of the remaining states rely to some extent on soil tests or otherwise recognize the importance of subgrade consideration in the design of their roads and pavements

The new apparatus described and the modified procedures suggested, cast no reflection on either the accuracy or efficiency of methods of soil identification utilized during the past five years Such improvements are the indications of the normal healthy growth of a branch of science rapidly progressing from the theoretical and experimental to the practical stage

## DISCUSSION

### ON

## SUBGRADE RESEARCH

Mr B E Gray, *Highway Engineer of The Asphalt Institute, New York* I would like to emphasize the importance of Mr Hogentogler's paper, and to urge that every highway engineer make a very careful study of it and other literature dealing with soil analysis It is a self evident fact that all loads placed upon a road surface are eventually distributed to the subgrade, and if it is practicable to introduce bitumens or other materials into a soil so that the optimum moisture content is maintained the year around, then a great advance will have been made in providing satisfactory riding surfaces at much reduced costs as compared with present methods The advances in the technique of constructing "low cost roads" have been possible because of recognition of varying soil behavior, and I venture the prediction that with this further information upon the subject still greater economies may be effected.