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16. ABSTRACT

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to develop their knowledge of clays, geologists and mineralogists have succeeded in mapping and and classifying the various formations and have given names to the numerous varieties of soil and rock.

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California Division of Highways

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CONTROL OF MATERIALS DURING CONSTRUCTION

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Since very ancient times, man's efforts to travel or to move his goods from one place to another has brought evidence that the surface of the earth in its natural state is rarely suitable for heavy or concentrated traffic. The entire history of highway construction from the days of the Roman roads down to the present largely represents the efforts to cover up or modify the earthy materials to provide a more suitable path for wheeled vehicles. In general, the Romans employed the expedient of covering up adverse soils with a heavy layer of carefully placed stone. McAdam and Telford both advocated somewhat similar methods and to a great extent these procedures are still relied upon.

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For the most part engineering treatises and texts have tried to establish the principles by which the load bearing capacity of soils could be computed as a basis for the design of bases or foundations for engineering structures which must rest upon the earth. The application of the mathematical technique invariably rests upon the concept that if the materials in question are very uniform, then the behavior can be predicted from certain assumed conditions.

It is not the purpose of this discussion to question the value of the so-called fundamental approach. It is obvious that if an engineer is to design a structure with reasonable assurance that it will be stable and permanent, he must be able to predict or calculate the load bearing capacities with considerable certainty. However, during construction, field engineers are faced with the day-by-day problems involved in the selection and control of materials for highways and air fields and are aware that the ideal soil of the mathematician is rarely encountered. It is generally necessary to construct pavements over the soils that exist and to utilize the granular materials economically available. The construction engineer may reach the conclusion that if the materials of the earth were created to serve a variety of purposes, the

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specialized needs of the highway engineer is one of the least of these. The problem facing the field engineer can be rather simply stated. By and large, it is a problem of identification and selection which means making sure that the right materials go into the right place. When faced with the necessity for rejecting or accepting a specific material, the engineer is forced to rely upon tests to determine whether or not a given soil, sand, gravel or crushed stone has the essential properties. He must deal with the real properties of the soil or the mineral aggregates and broad generalities are not of much help.

It was not many years ago when both design and construction engineers generally paid very little attention to differences in soil types. While it was frequent practice to classify excavation as a basis for pay quantities to the contractor, many attempts to distinguish between such markedly different materials as "dirt" and solid rock often led to arguments between the contractor and the engineer. In order to avoid such arguments, the California Division of Highways, for example, does not classify earth work for pay quantities.

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Nevertheless, in spite of the generally casual attitude certain highway engineers began to realize some 25 or 30 years ago that the native materials encountered on many highway routes were not always giving satisfactory performance and as subsidence and settlements were common, they began to specify rolling and compaction throughout the entire depth of embankments. One might hazard the surmise that the development of knowledge and understanding on the part of field and construction engineers has been retarded by the fact that these men do not ordinarily observe the materials in their worst condition. Most of the construction work is carried on in good weather when the soils are relatively dry and it is often difficult to visualize the conditions which will exist in the roadbed after the pavement has been in place for several years. The maintenance department and the more active materials laboratories are generally better acquainted with the adverse conditions that can and often do develop. It would be helpful if more engineers were impressed with the fact that the matter of chief concern is the behavior of <u>wet</u> soil.

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Engineers soon discovered that in order to secure maximum efficiency with field compaction equipment careful control of moisture content is necessary. However, in spite of increased attention and effort spent in compacting soils, the possibility of failure was not eliminated. Initial compaction alone will not prevent certain soils from becoming saturated and in the course of time losing the ability to support loads. For all practical purposes, failures in highway bases, subbases and basement soils are almost entirely due to water. This includes failures due to frost action as there is no reason to think that freezing would cause much trouble in dry soils if no water were available. Therefore, in order to select adequate materials for all layers or horizons in the roadbed, a field engineer must understand something of the essential properties of soil materials, especially those properties that affect the load supporting ability when the soil is wet. The field engineer must have at hand test methods for detecting or evaluating the essential properties.

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Dating from Coulomb's time, (1736-1806) most theoretical discussions on soil mechanics recognize that the entire structural strength of soils rests upon two distinct properties; namely, internal friction and cohesion. The stability of slopes and embankments, the pressures against retaining walls, and the ability to sustain vertical loads, all depend upon some combination of these two properties - friction (or resistance to sliding) between rock or soil particles, amplified by any cohesive forces which may exist. The laws governing the resistance to sliding between solid particles in contact do not appear to be too clearly understood even today and they are probably much more complex than has generally been recognized. However, for most practical purposes variations in frictional resistance may be well enough defined in the terms used by Amontons who was one of the first to investigate this particular phenomena. Amontons' law states rather simply that the resistance to sliding between adjacent solid surfaces in contact varies with the nature of the surfaces, directly with the pressure which holds the surfaces together and is largely independent of the speed and the apparent areas in contact.

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The definition of cohesion as used in soil mechanics texts does not correspond to the dictionary definition. For example, the soil mechanics usage generally states that the cohesive resistance in soils is that part of the resistance <u>which does not vary with the pressure</u>. This is a negative definition but this observation agrees with one of the characteristics of viscous liquids as the internal friction of liquids is virtually independent of the pressure and the resistance developed by a film of viscous liquid between two solid bodies varies directly with the area and directly with the speed of movement, but is <u>largely independent of the pressure</u>. It may be observed therefore that the behavior of dry solid bodies in contact is almost diametrically opposed to the behavior of viscous liquids in thin films. A bed of crushed stone or sand represents a case where the ability to sustain loads rests entirely upon interparticle friction and no cohesive forces are involved. On the other hand, metals are said to have little or no internal friction, therefore their resistance or strength depends entirely upon cohesive forces. Portland-cement concrete is a substance capable of sustaining substantial compressive loads due to moderate

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cohesive strength and the high internal friction characteristic of dry stone particles in contact. A well designed asphaltic pavement is an engineering material with relatively low tensile strength but in which the frictional resistance can be maintained at a high level so long as an excess of asphalt is not introduced. Asphalt is a viscous liquid and virtually all viscous liquids are lubricants. Under certain conditions water may also act as a lubricant. It is a matter of common observation that water alone will not produce a measurable lubricating effect on a layer of clean crushed stone as layers of sand, gravel or crushed stone have long been used for pathways and driveways and are generally satisfactory except for their inability to resist surface abrasion. But, water and clay mixtures have marked lubricating properties. Such mixtures are widely used in well drilling with rotary bits for several reasons which include the ability to lubricate the tools and to mobilize or float the cuttings and stone chips.

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The civil engineer considers that the term "soil" includes all materials of the earth's mantle, including gravel and sand as well as the various silts and loams, and virtually all such materials contain a greater or lesser percentage of extremely fine particles. These fine particles, when mixed with water, usually display the special properties of plasticity and mobility that are characteristic of the clay minerals. Apparently, there is no exact or simple definition to distinguish the clay fraction from finely ground rock particles but in the absence of clay, material finer than a No. 4 sieve is to all intent and purposes a sand even though some of the particles are very small. It is evident that many clays display properties that are not attributable to small particle size alone. It is not within the scope of this paper to attempt any discussion on the intimate shape, properties and special behavior of the clay minerals. In the December 1952 issue of the Public Roads magazine an article by Earl B. Kinter, Adolph M. Wintermyer and Max Swerdlow gives a very interesting discussion accompanied by numerous electron micrographs of the clay minerals. This article should be read by anyone interested in the properties of soils and behavior of clays. However, for the purposes of

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the average highway engineer who is attempting to evaluate the quality of soils, gravels or other granular materials, it is sufficient to recognize that when combined with sufficient water most clays are very effective lubricants and when a sufficient quantity of clay is intermixed with the coarser granular portion of the soil, lubrication will develop and the mixture becomes plastic whenever enough water is absorbed or accumulated. As the ability of soils to resist deformation depends very largely upon internal friction, wet clay has the effect of reducing or canceling out the frictional resistance and while the so-called cohesive resistance is almost entirely due to the clay, this gain in cohesive strength does not compensate for the loss due to reduction in friction. It must be emphasized, of course, that water is the essential factor causing loss in resistance as finely ground dry clay particles exhibit no cohesive properties and are not very effective lubricants. When water is added to a dry soil mixture the cohesive strength or resistance will normally increase with the addition of moisture and compaction and in most cases a certain optimum amount of water will be

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tolerated before the frictional resistance is greatly impaired. However, above a certain moisture content the resistance due to friction will diminish even though the cohesive strength or resistance may continue to increase up to some second point of higher moisture content. Finally, values of both friction and cohesion will diminish as the soil approaches a completely fluid state. If it were possible to develop a sufficiently high cohesive resistance we would not need to be concerned over the lubricating effects of clay but it is easily demonstrated that materials such as crushed stone and gravel develop a great deal more resistance due to interparticle friction than can be developed by cohesion from any of the natural clays when thoroughly saturated. Plastic soils, clay bearing gravels or rich asphaltic mixtures, all having high cohesive values but little internal friction are rarely adequate to sustain repeated vehicle loads. In other words, an excessive amount of clay is detrimental to highway bases and foundations. Soils or gravels containing high percentages of clay invariably become unstable and lack supporting power when wet.

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It is a matter of common knowledge among highway engineers that an excessive amount of asphalt in an otherwise normal mixture of sand and gravel will cause instability because the asphalt is also a lubricant and provides mobility or plasticity to the mass. It may be noted, however, that when compared on the basis of weight, apparently less asphalt will be required to produce instability than is the case with clay. For example, a dense graded aggregate will usually be unstable with 7 percent of asphalt by weight while over 10 percent of plastic clay soil might be required to produce the same effect. This difference exists, however, only in the weight relationships as the volume of asphalt corresponding to 7 percent of the dry weight of the aggregate is about equal to the bulk volume of wet clay corresponding to 11 percent by weight of the total aggregate. It should then be apparent that lubrication does not become serious until a certain volume of lubricant has been introduced, and, to carry the analogy further, if the aggregate were mixed with a plastic material such as red lead paint, instability would also be produced but in this case it would require approximately 23 percent by weight although the volume relationships would still remain about the same. It should not be necessary to dwell upon this relationship unduly but the

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conclusion is inescapable that the fundamental relationship in all design of aggregate mixtures whether for concrete, bituminous pavements, or gravel bases rests upon the relative volumes of each ingredient and not upon weight percentages. This fact is often overlooked.

The control of bituminous mixtures is simplified by the fact that paving asphalts do not vary widely in specific gravity but the mixtures of clay and water do vary in weight, and to illustrate the wide range of volume which can exist even with the same weight of dry material, Fig. 1 is included wherein a series of glass graduates are shown, each containing 100 grams of dry mineral powder or rock dust of the types often referred to as "filler dust" when used in asphaltic mixtures. It should be readily apparent to any engineer that a sieve analysis of mineral aggregates expressed as the weight percentages of each size does not necessarily give a true picture of the relative volume or mass of material represented by the size groups. As a further illustration, Fig. 2 shows a series of mineral aggregates of much coarser gradation, each representing an aggregate of different specific gravity and again illustrating that the total volume of material represented by a pound or a ton may vary widely. However, there are still more variables and it is even more important to realize that

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while it is troublesome to maintain proper proportions between aggregates and binders that vary in unit weight, the problem becomes more complex when the mineral aggregates consist of several types of stone particles, each size group having a different specific gravity. Figure 3 illustrates some of the contrast which may occur between mineral aggregates which are made up of identical gradations when expressed by weight but it will be observed that the volume proportions are vastly different. Troublesome as these variations can be it is nevertheless possible to determine the specific gravity of each size group and by calculation arrive at the true volume relationship. This is routine procedure in concrete mix design but is usually overlooked in other cases.

However, there is "more to come" as another factor is often involved when water is introduced into soils or untreated base materials. In these cases, certain clays may expand considerably and the effective volume of wet lubricating clay may be greatly increased or augmented. This condition is most marked in the montmorillonite class of clays of which bentonite is a well-known example. (Other clay types may expand but to a lesser degree.) In any event, an estimate of the effective amount of clay may be very misleading when based upon sieve analysis, hydrometer analysis, or upon any other means which leads to an expression in terms of weight percentages.

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Figure 4 shows two graduates, each containing an amount of sand and clay which, when thoroughly saturated with water and tested in the stabilometer, produced a Resistance value of 60. (Without the clay this sand has an R-value of 80.) It will be noted that the amount of sand is the same in each case but graduate (A) contains only 5 percent of bentonite where graduate (B) contains 21 percent of kaolinite by weight. In the weight proportions shown these materials have an equal effect in reducing stability or increasing the plasticity of the entire mass and therefore as illustrated in Fig. 6 bentonite is a much more effective lubricant and consequently can be far more troublesome than kaolinite for example. The widespread use of weight batching or proportioning devices and also the use of scales or balances in laboratory analyses has tended to obscure the important relationships that depend upon relative volumes.

Figure 5 is an analytical chart setting forth the factors affecting the design of pavements(1) wherein the problem is subdivided into secondary, tertiary and quaternary factors and the factors that are influenced by the clay content of the soil are indicated by circles near the right-hand margin of the chart. These serve to illustrate the properties or characteristics that may be affected by the amount and type of clay present in the soil.

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The average engineer is usually interested, however, only in the over-all effect of the particular clay when the soil becomes wet. The primary question is the effect on structural stability which, as stated above, really boils down to the question of whether or not there is any reduction in internal friction.

In order to measure the ability of soils and granular materials to sustain loads, many tests have been devised. One of the best known is the California Bearing Ratio which has recently been replaced by the stabilometer in the laboratory of the California Division of Highways. The testing procedure now followed requires the preparation of soil specimens by first introducing sufficient water to fill the expected void space, then compacting to produce a structural arrangement and a state of density comparable to that developed in most highway bases and basement soil layers. The next step is to determine the final equilibrium moisture content and then to measure the resistance to deformation. Basically, the stabilometer is a form of plastometer and primarily the test reflects the internal friction or degree of lubrication with cohesive resistance playing a minor part. When being tested

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in this instrument, a compacted sample is subjected to a vertical pressure which may be varied at will but for highway purposes is typically 160 psi for soils and granular base materials. The instrument makes it possible to measure the lateral pressure transmitted by the specimen while under load, Fig. 7. Figure 6 is a chart showing characteristic curves illustrating loss in stability or internal resistance plotted from stabilometer data on a crushed sandy gravel to which has been added increments of plastic clay. This test procedure is used as a basis for calculating the supporting value of the soil and by use of suitable formulas it is possible to compute the thickness of cover courses (i.e. bases and pavement) which will be necessary to support vehicle loads of a given magnitude and number of repetitions. These test procedures have been reported in considerable detail elsewhere and will not be discussed further here.(1) The stabilometer test and the elaborate compaction equipment required to produce specimens that are characteristic of soils in place in the roadbed means that this test procedure is virtually restricted to a fairly large, well equipped laboratory.

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It is essential, however, that the resident engineer or inspector in charge of construction should have some ready and convenient test for detecting the presence of excessive amounts of adverse clay or fine materials in base or subbase materials. As we have established that the lubricating effect of clay or of any other material tends to increase with the volume; i.e., the thickness of film between the particles, the most fundamental relationship is that reflected by the potential volume of wetted clay that exists in each mixture. In order to speed up the testing operation by avoiding the need for weighing the sample and drying out in an oven, a test has been developed called the "Sand Equivalent Determination." The test is applied to a sample of material passing a No. 4 sieve and the relationship between the quantity of clay present and the amount of coarser sand particles in the sample is developed on a volume basis and the test results indicate whether the volume of "sand" is either high or low - hence, the name "Sand Equivalent."

Essentially, the test is performed by shaking vigorously a sample of the fine aggregate in a transparent cylinder containing a special aqueous solution (Fig. 8) and noting the relative volumes of sand and of the partially sedimented clay

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after standing for 20 minutes. The entire operation can be carried through in less than 40 minutes. In order to speed up the sedimentation of the fine clays or colloidal particles, a flocculating agent was required and a solution of calcium chloride was selected on account of its relatively low cost, stability and non-irritating properties.[2] Since a small amount of bentonite is in lubricating effect equal to a much greater weight of kaolinite (Fig. 4), the strength of the $CaCl_2$ solution was adjusted to a point that would give emphasis or enlargement of volume with a bentonite clay while not exaggerating the volume of kaolinite. This relationship appeared to be best established by using a very weak $CaCl_2$ solution. However, the strength of the solution does not appear to be too critical for most natural soils therefore a working solution of .05N has been adopted and will be used until accumulated experience may warrant a change. After some experience with the calcium chloride solution, it was found that the addition of a small amount of glycerin produced a stabilizing effect and test results made on carefully quartered samples were more readily reproducible. Finally, it was noted that the calcium-chloride-glycerin solution was not sterile and certain moulds tended to grow. In order to sterilize the solution, formaldehyde was added.

For a complete description of test procedure and the method of calculating the sand equivalent value, see Appendix I.

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When the Sand Equivalent Test was first developed, it was hoped that it would furnish a good indication of the over-all resistance value of the soil. A correlation does exist but it is not sharply defined throughout the scale. The reasons for this are not difficult to understand if it is recognized that the ability of a mass of soil or granular material passing a No. 4 sieve to resist deformation will depend upon the following factors:

<u>Summary of Factors Affecting Resistance Value of Soil</u>

1. The volume of lubricant mixed with the sand fraction; i.e., asphalt, clay+water, etc.

2. The effectiveness or efficiency of the lubricating fraction. (Wet bentonite is a better lubricant than kaolinite, for example.)

3. The degree of roughness or irregularity of the sand grains or rock particles.

4. The amount of void space in the sand fraction of the soil.

5. The amount of intermingled coarse rock retained on a No. 4 sieve.

We readily perceive that of these five variables the Sand Equivalent Determination is primarily an indication of No. 1 (expanded clay volume). It attempts to compensate for No. 2 by means of the type of solution used.

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It cannot indicate the variation caused by Item 3, and as presently performed does not make allowance for No. 4 although it seems possible that means for making this correction may be worked out. Allowance for the effects of No. 5 need to be made if the coarse aggregate exceeds 25 or 30 percent of the total. Therefore, in order to evaluate the combined effect of all factors, some test such as the stabilometer is necessary. However, experience has shown that one of the principal variables is the amount of clay present as there is relatively little difference in sands and it may readily be determined that when the Sand Equivalent value is greater than 30 the clay fraction is not sufficiently large to have much influence on the resistance value of an untreated soil. See Fig. 14 - Resistance Value vs. Sand Equivalent.

Application and Tentative Sand Equivalent Limits

Very small amounts of clay may be detrimental to the performance of bituminous mixtures, especially when the clay exists as a coating on the surfaces of the sand grains. As the Sand Equivalent Determination furnishes a ready means for detecting the presence of such fine materials, a tentative scale of values has been set up to permit rapid testing

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and quick determination in the field. At the present time, SE values are being included in the specifications of the California Division of Highways. The following limits are proposed for each of the classes of aggregates listed:

	Sand Equivalent Minimum
Crusher run or gravel base materials	30
Aggregates and selected materials for road mix bituminous treatment	35
Aggregates for plant mix bituminous surface	45
Aggregates for asphaltic concrete or Class A plant mix	55
Concrete sand	80

By comparing SE test values to other test results, it was found that the majority of soils showing high expansion under soaking may also be identified by means of the sand equivalent. It has been the general practice in California in the past to consider that any soils showing an expansion greater than 5 percent on specimens prepared for the CBR will be unsuitable for placing in the upper levels of the roadbed. It appears that the same class of soils could be identified and segregated by stipulating that any soils

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having a SE less than 10 should not be placed in the upper layers of the roadbed as they are also likely to develop excessive expansion and certainly will have little supporting power when saturated.

Sand equivalent values are definitely influenced by the amount of fine dust in the sample and a plot of SE values versus either percentage of "sand" or percentage of dust will usually result in a smooth curve for each material. However, there is a marked difference in the relationship depending upon the type of dust or fine material present.

Figure 9 is a plot showing the relationship between SE values and the total percentage of dust added to a sample of standard Ottawa sand. It will be noted that only about 6 percent of clay is required to produce a SE value of 30 whereas it will require 33 percent of fine quartz dust and 44 percent of limestone or rock dust.

Figure 10 is a further illustration of the differences in type of dust wherein it is shown that the addition of a "natural" dust to a sample of fine aggregate reduces the SE more rapidly than would the addition of crusher dust. If one were to specify a minimum SE value of 55, not more than 9 percent of "natural dust" could be tolerated in the sand fraction whereas 13 percent of crusher dust would be acceptable.

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The effect of fineness alone is illustrated by Fig. 11 which shows a comparison between the proportion of dust and the SE value with two different dusts produced by the same method of artificial grinding. Dust (A) is more coarsely ground than dust (B) as illustrated by the comparative surface areas. A SE of 45 in this case would permit 34 percent of dust (A) but only 26 percent of dust (B) in the sand fraction.

The effect of fineness of grinding on the SE results is further illustrated by Fig. 12.

The relationship between the SE value and the resistance value of the granular base material with different percentages of two types of clay added is illustrated in Fig. 13. The test data were derived from a sample of crushed stone which, in its natural form, gave a resistance value of 80. The additions of small amounts of kaolinite have no effect on the R-value until a SE of about 30 is reached, after which the R-value drops off rapidly. The effects of bentonite are observed even with SE values above 40 but the drop in resistance does not become acute until a SE value below 30 is reached. Natural soil materials are generally intermediate between these two limits and rarely are as potent in reducing the R-value as are the typical bentonites.

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Figure 14 is an illustration of the values developed from a variety of materials which have been tested in the stabilometer and the SE values determined. There is evidence that mixtures of sand, silt and clay develop optimum resistance values when sand equivalents fall in the range of 30 to 70 while clean sands without dust tend to drop off somewhat in R-value. While certain materials do show an adequate resistance value with a SE as low as 20, many others fall off very markedly and it is believed that any material having a SE less than 30 under the methods of test described herein will be too critical and offers too small a margin of safety to be used in important layers of base or subbase intended to support a costly highway surface course. The symbols on Chart, Fig. 14, are intended to illustrate the approximate classification of the materials. It will be noted that clean cohesionless sands show very high SE values. Sands mixed with silt and some clay fall at intermediate points while the clays and silty clay mixtures are in general very low.

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From the foregoing, it will be apparent that the SE is useful in detecting the presence of adverse amounts of clay in granular base materials. It may also be used to limit the amount of fine dust or clay that may exist as a coating on particles of mineral aggregate intended for bituminous mixtures. It may be used as a quick field test to control the amount of adverse clays in concrete sands. The junior engineer or inspector who performs the test should in time become very much aware of the effects of the clay fraction in pavement and base materials.

In conclusion, it may be stated that the suitability of soils for engineering purposes depends largely upon their ability to remain in place and to support whatever loads may be placed upon them either by a permanent engineering structure or by transient vehicle loads. A study of the properties which distinguish the more satisfactory from the less satisfactory soils indicates that in the majority of cases clays are detrimental to stability and it is apparent that wet clay has the effect of a lubricant in diminishing the natural resistance due to friction that would otherwise exist. It is necessary that the civil engineer responsible for construction of any form of earth work should be informed not only concerning the quantity of clay minerals that are present but also should know something of their nature and their potential influence on the engineering properties of the soil.

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The development of the Sand Equivalent test procedure has been underway for over four years and many alternate devices have been tried and discarded in the process.

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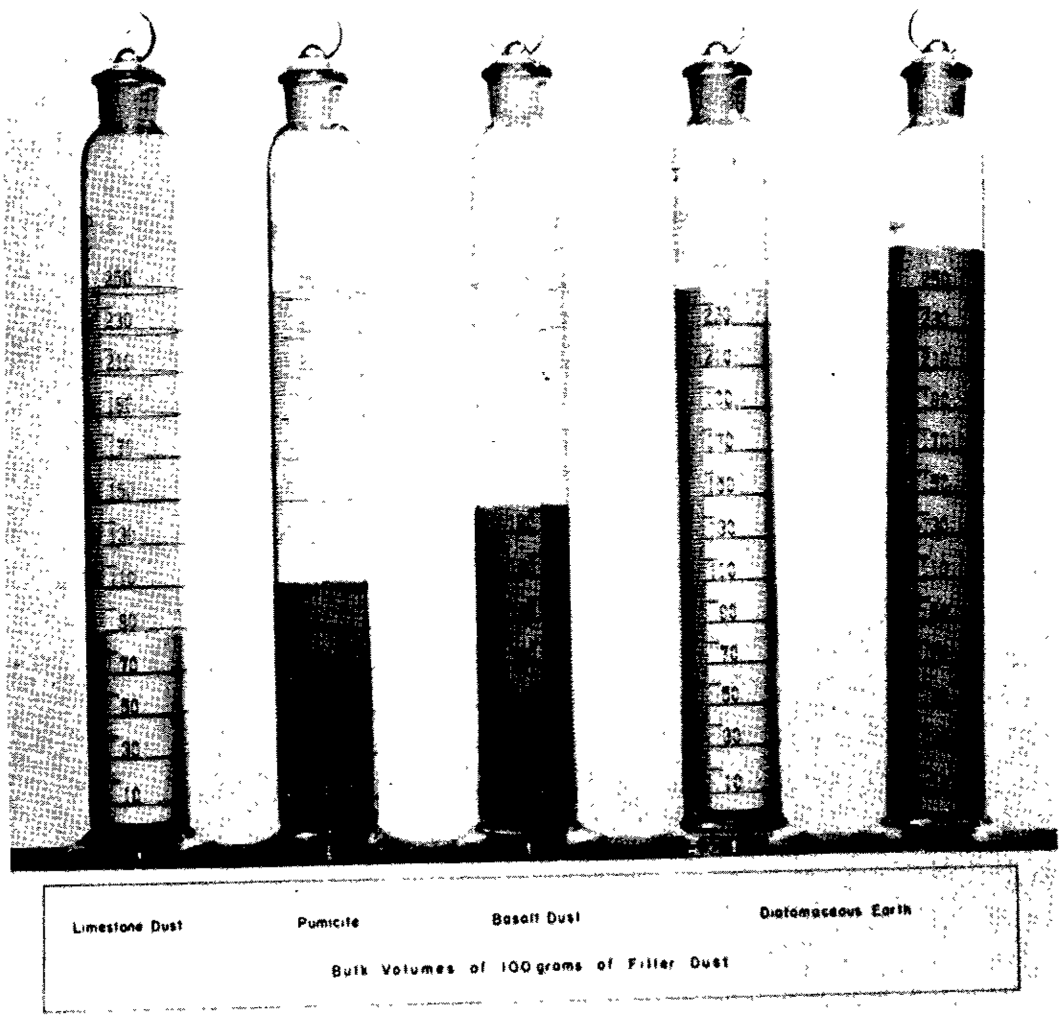

Fig. 1

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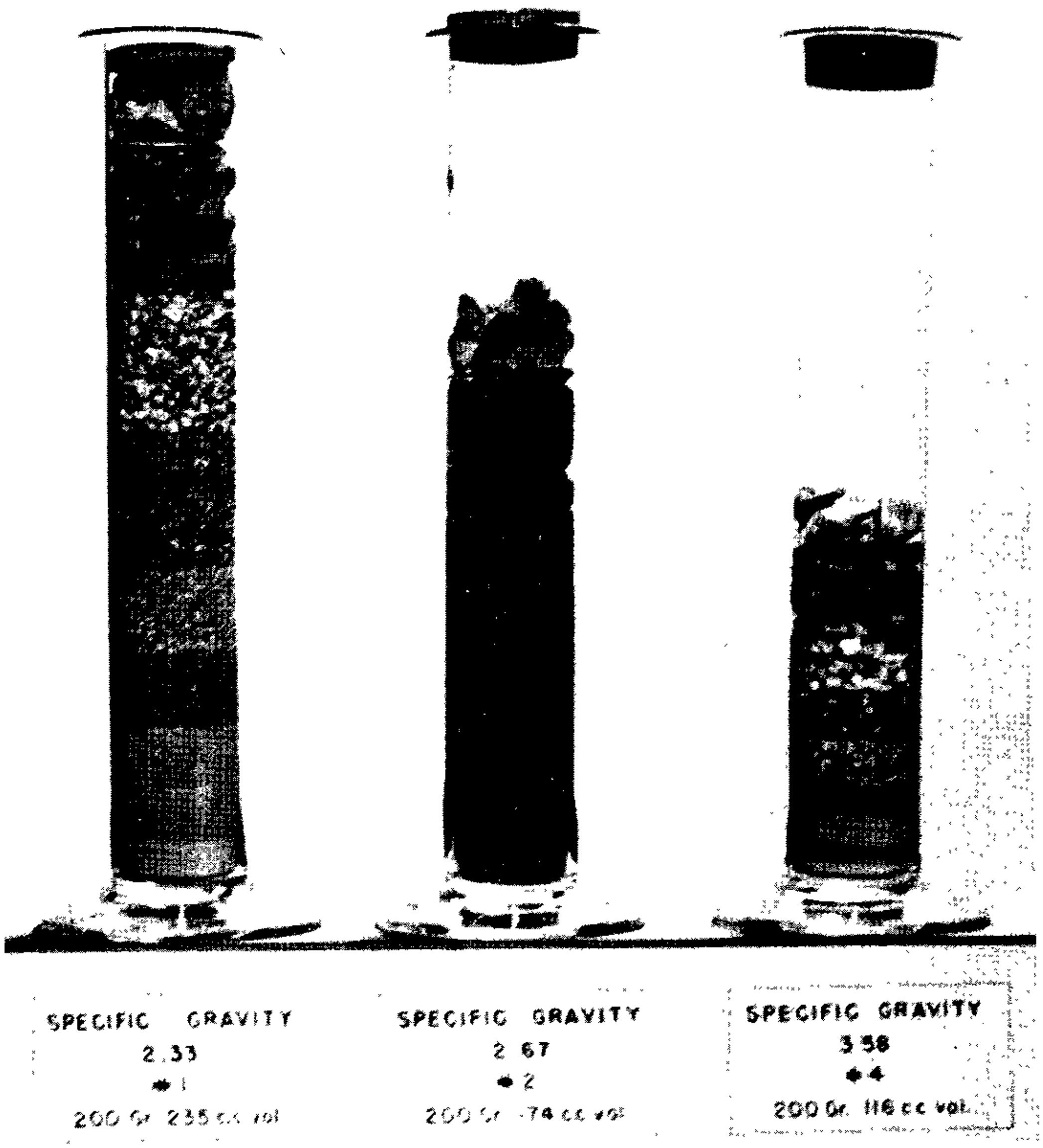

Fig. 2

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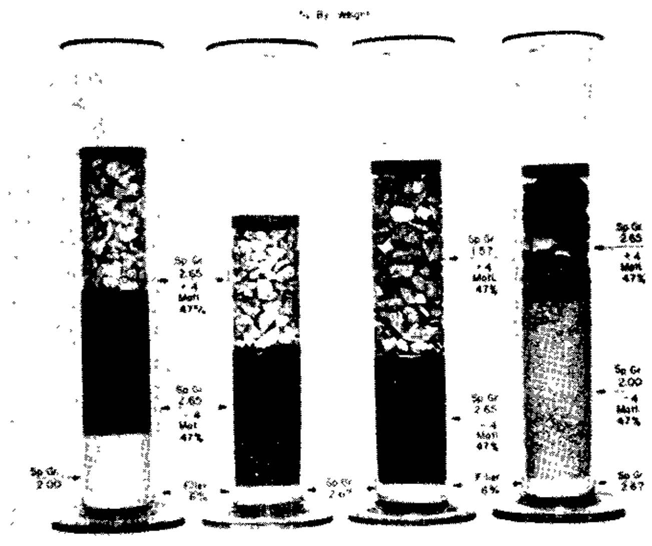

Fig. 3

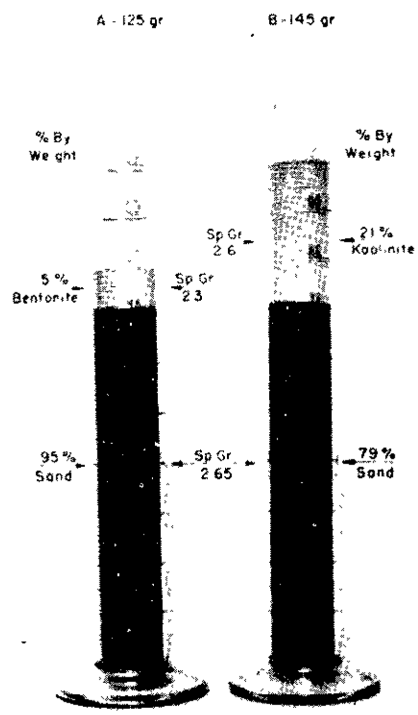

Fig. 4

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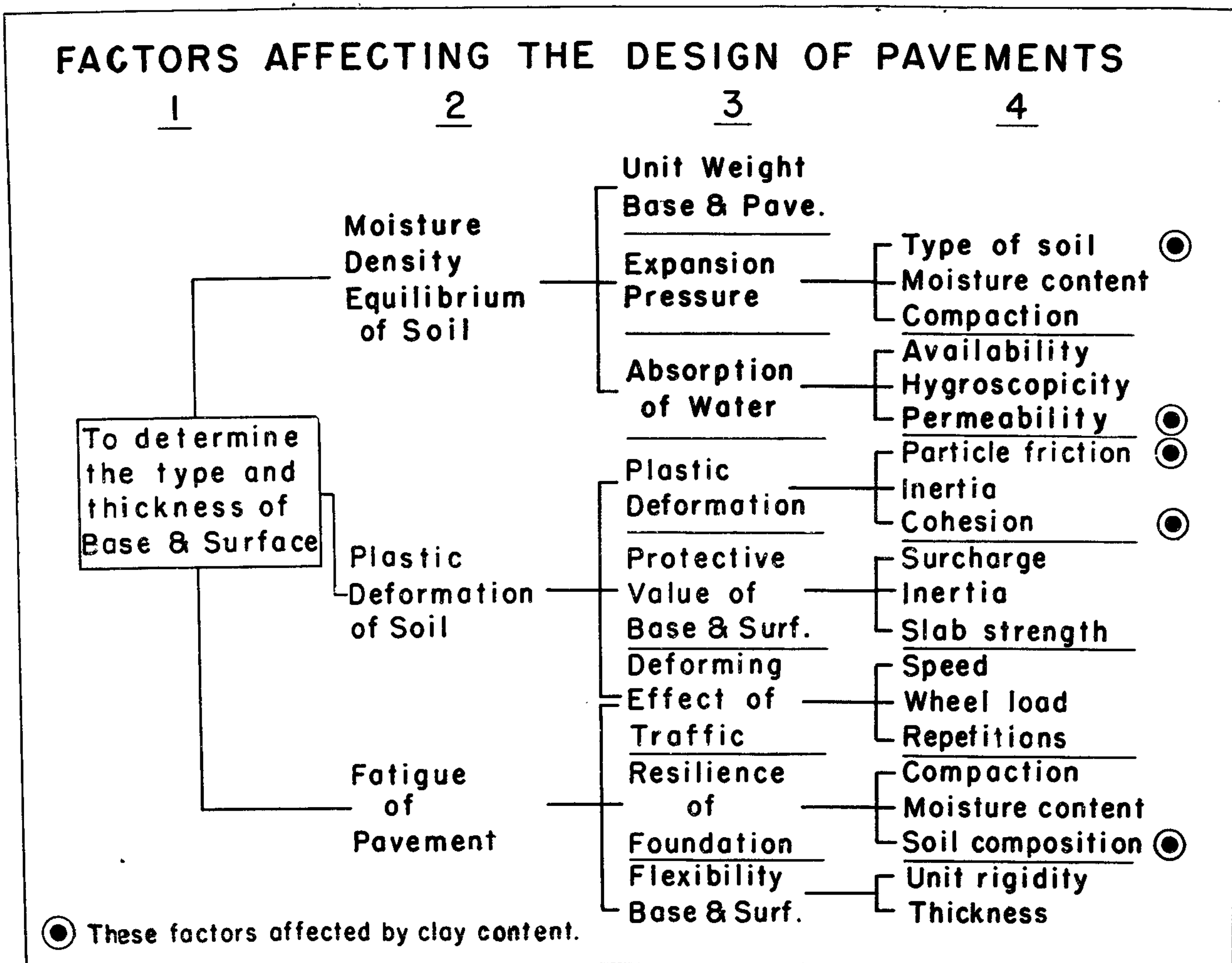

Fig. 5

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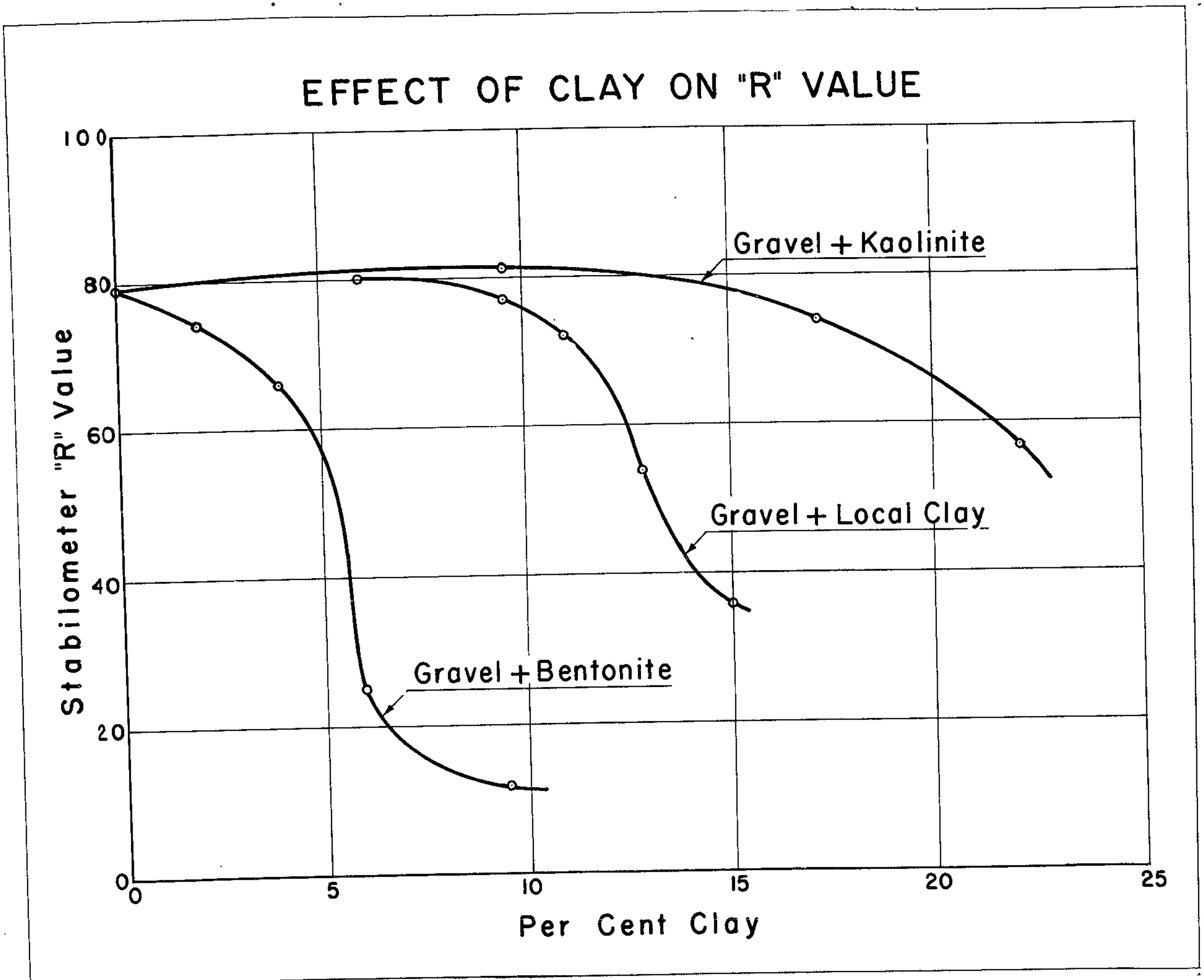

Fig. 6

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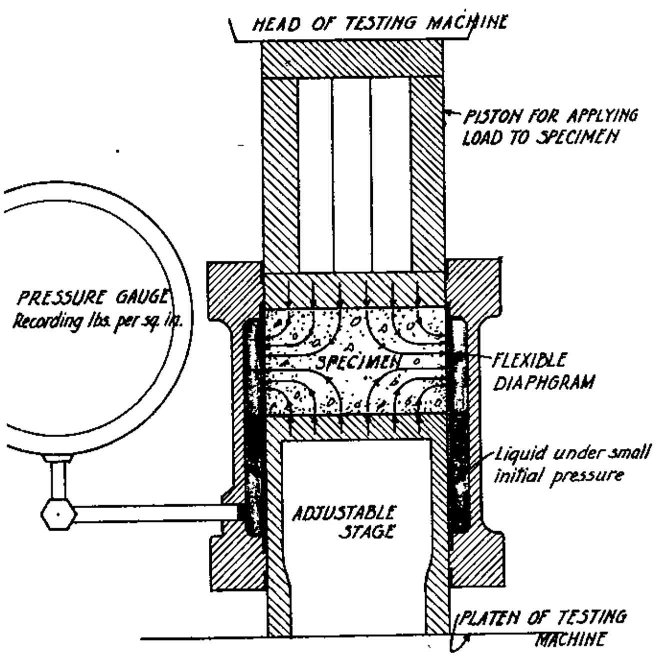

Fig. 7

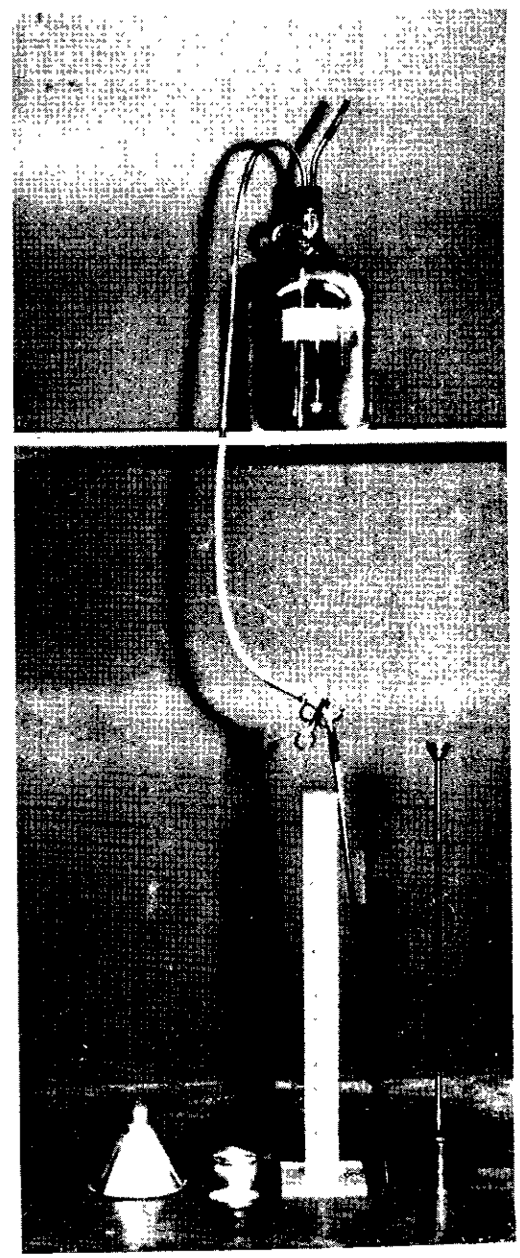

Fig. 8

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EFFECT OF VARIOUS FINE MATERIALS ON THE SAND EQUIVALENT

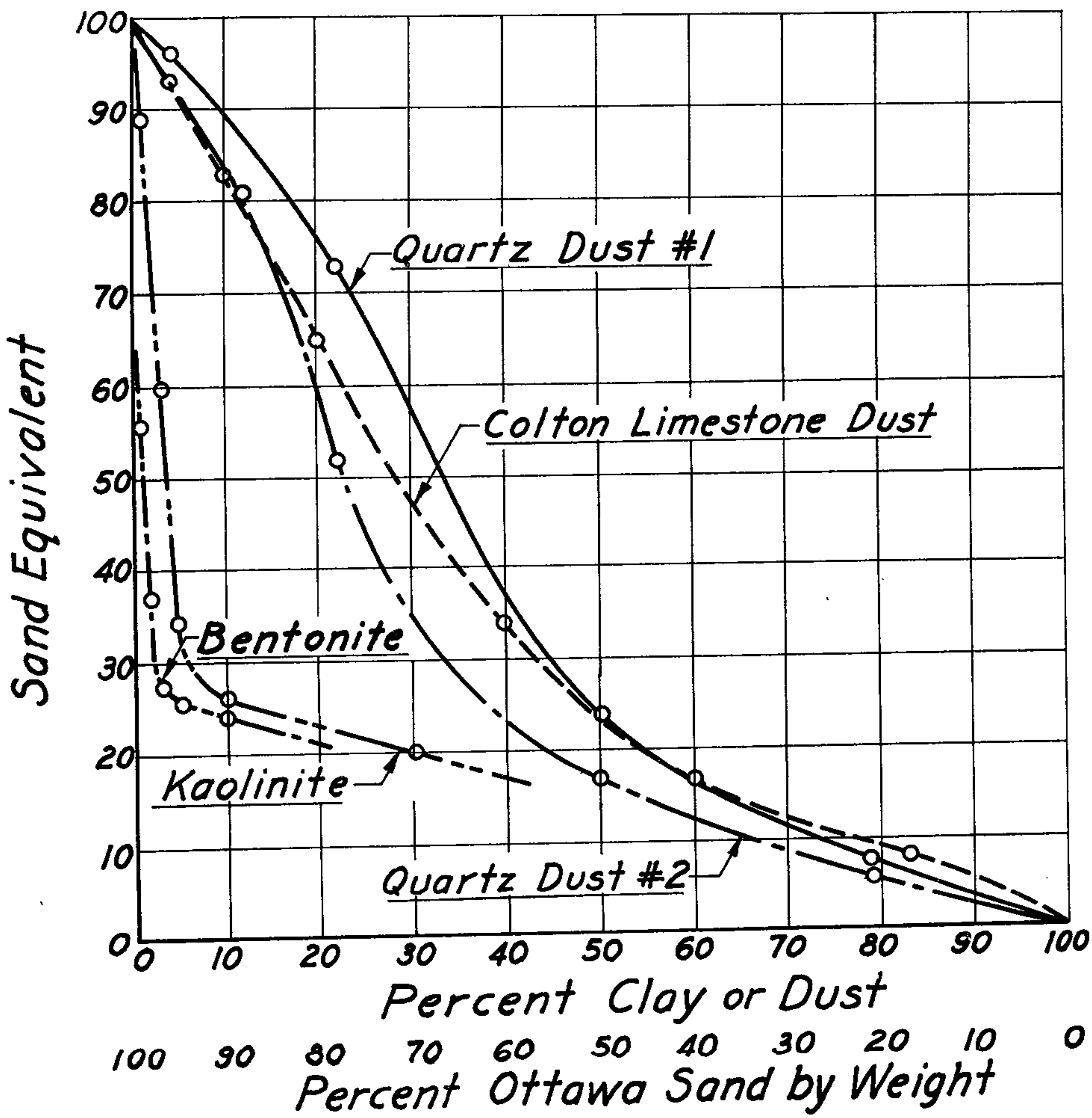

Fig. 9

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EFFECT OF DUST ON THE SAND EQUIVALENT
OF PLANT MIXED SURFACING FINE AGGREGATE
Sample No. 52-2745

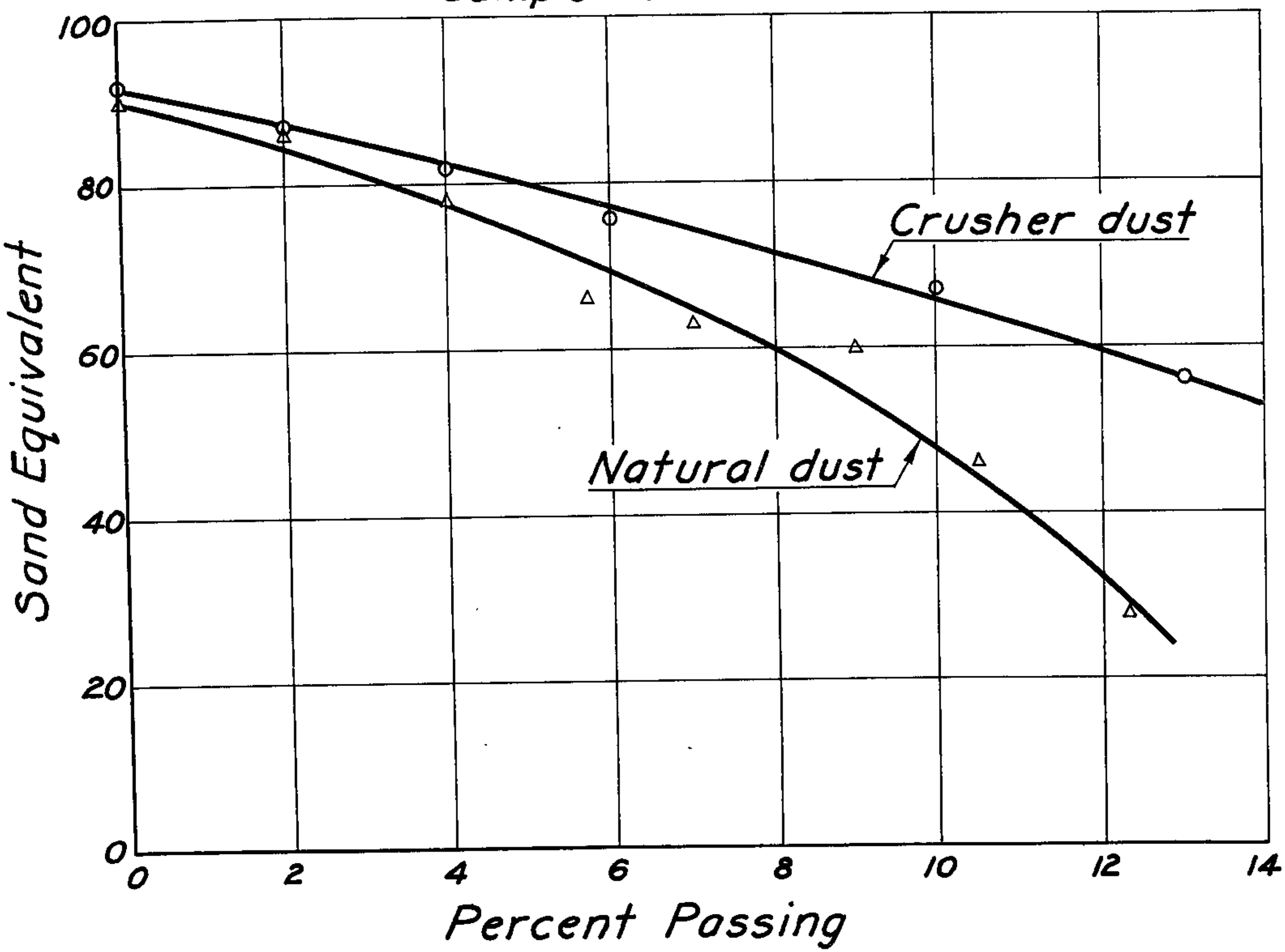

Fig. 10

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QUARTZ DUST & OTTAWA SAND

Effect of Particle Size on the Sand Equivalent

Note: Tests made with ground quartz dust.

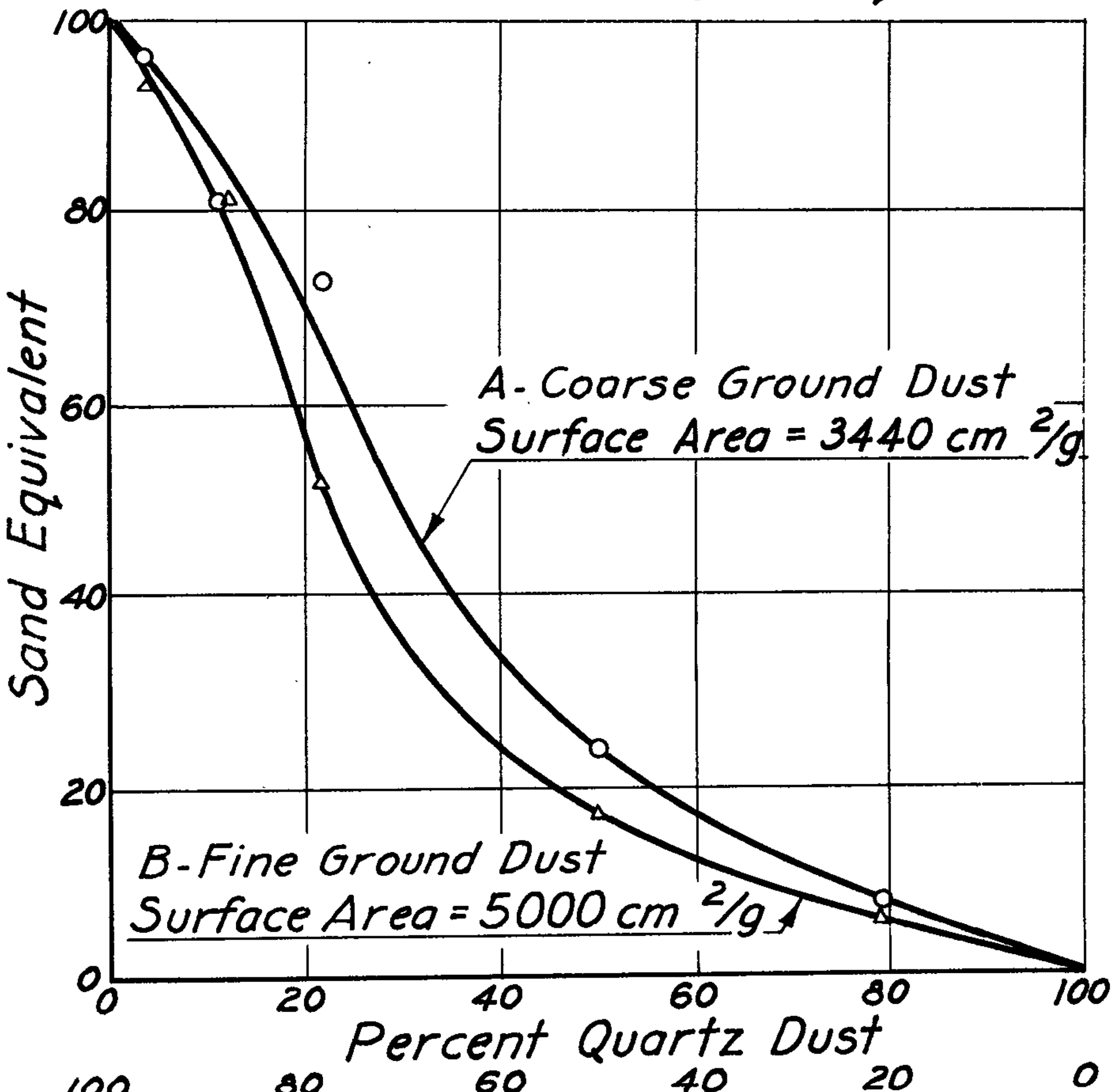

Fig. 11

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EFFECT OF DUST PARTICLE SIZE
11% Pulverized pea gravel and
89% Concrete sand

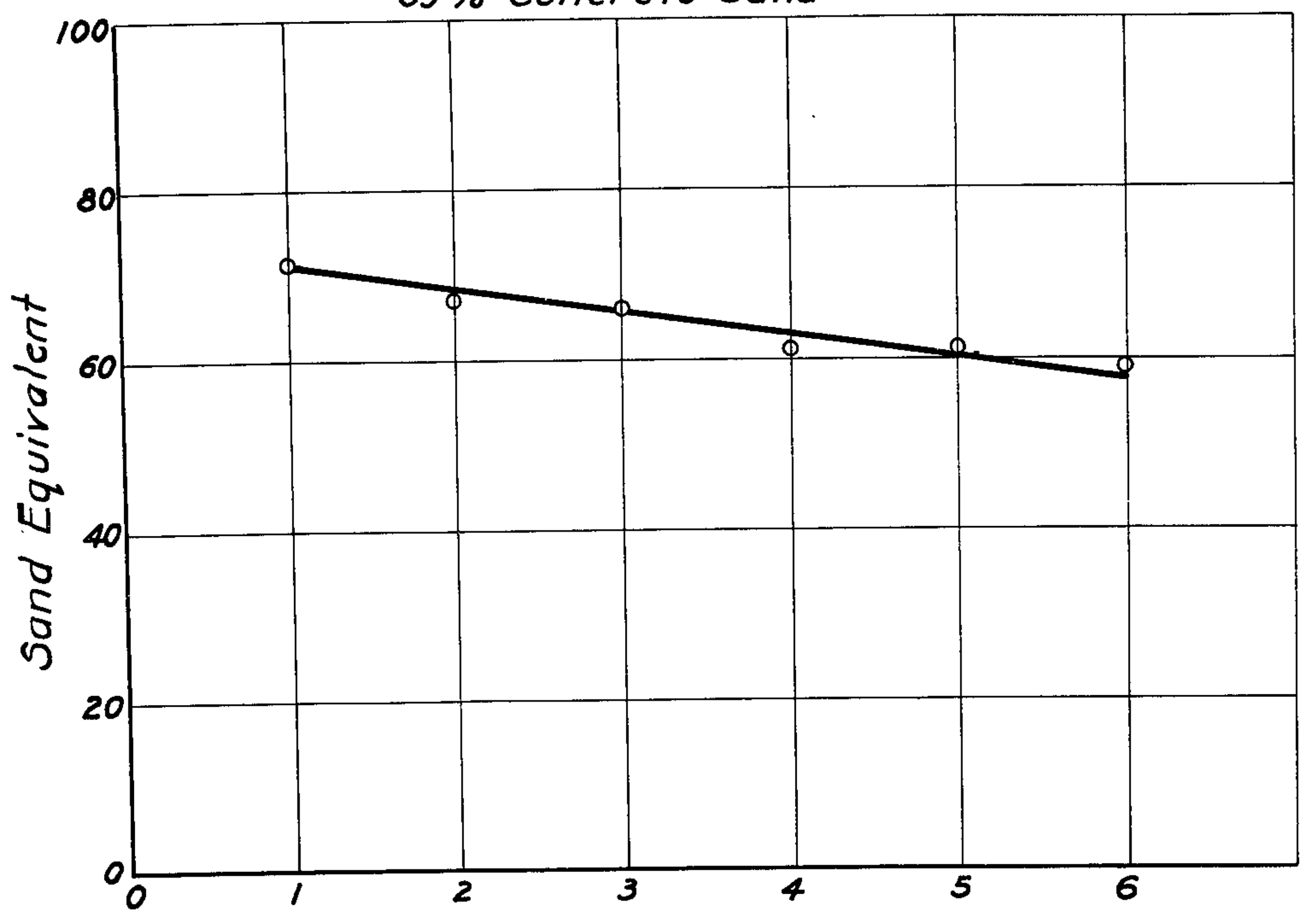

Fig. 12

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RESISTANCE VALUE VS SAND EQUIVALENT
FOR CRUSHED ROCK BASE
ADJUSTED TO VARYING PERCENTS OF CLAY

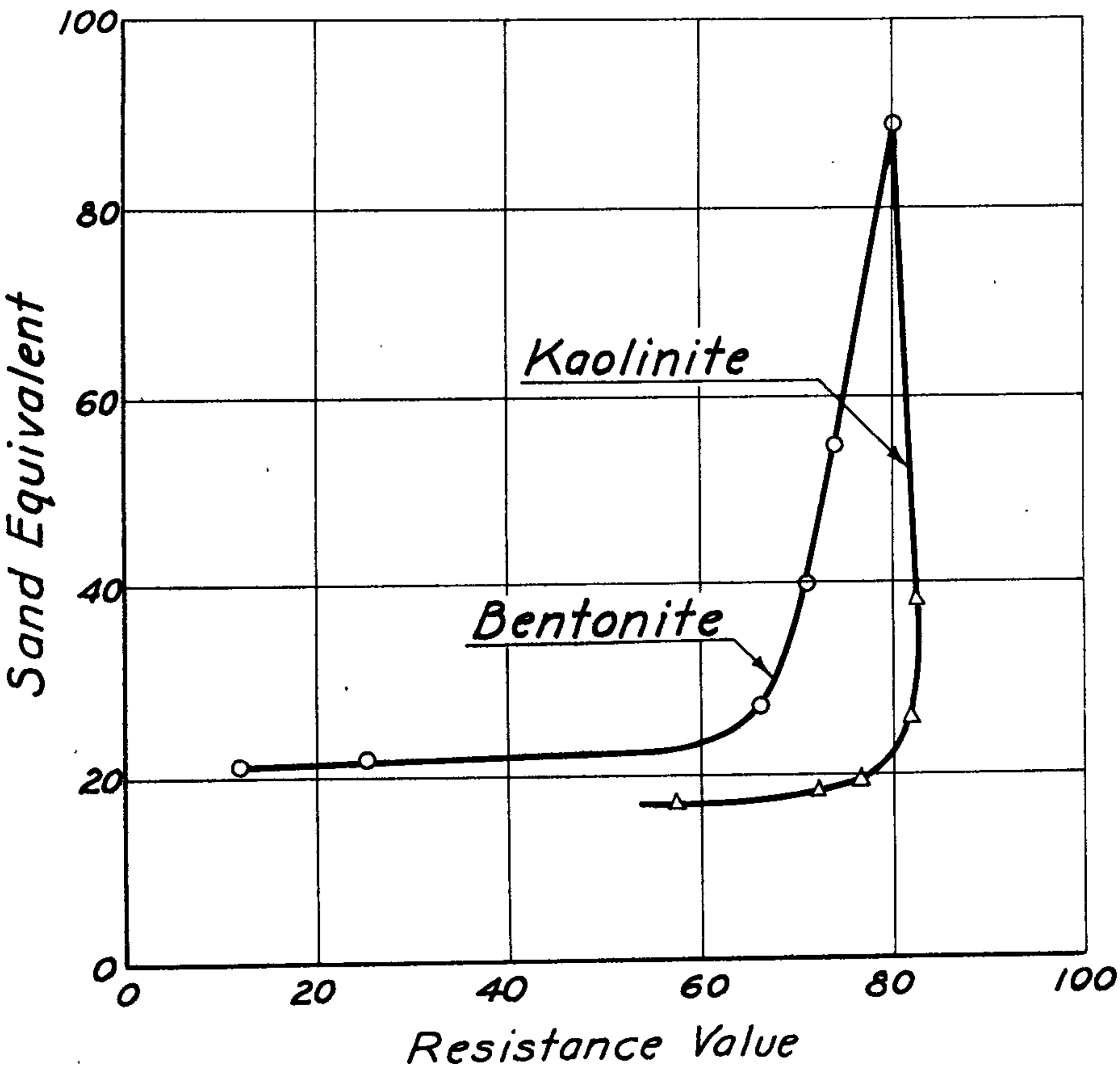

Fig. 13

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SAND EQUIVALENT VS RESISTANCE VALUE

LEGEND			
Minor Component	Major Component		
	Sand	Silt	Clay
Sandy	○	Φ	⊖
Silty	⦶	|	+
Clayey	⊖	+	—

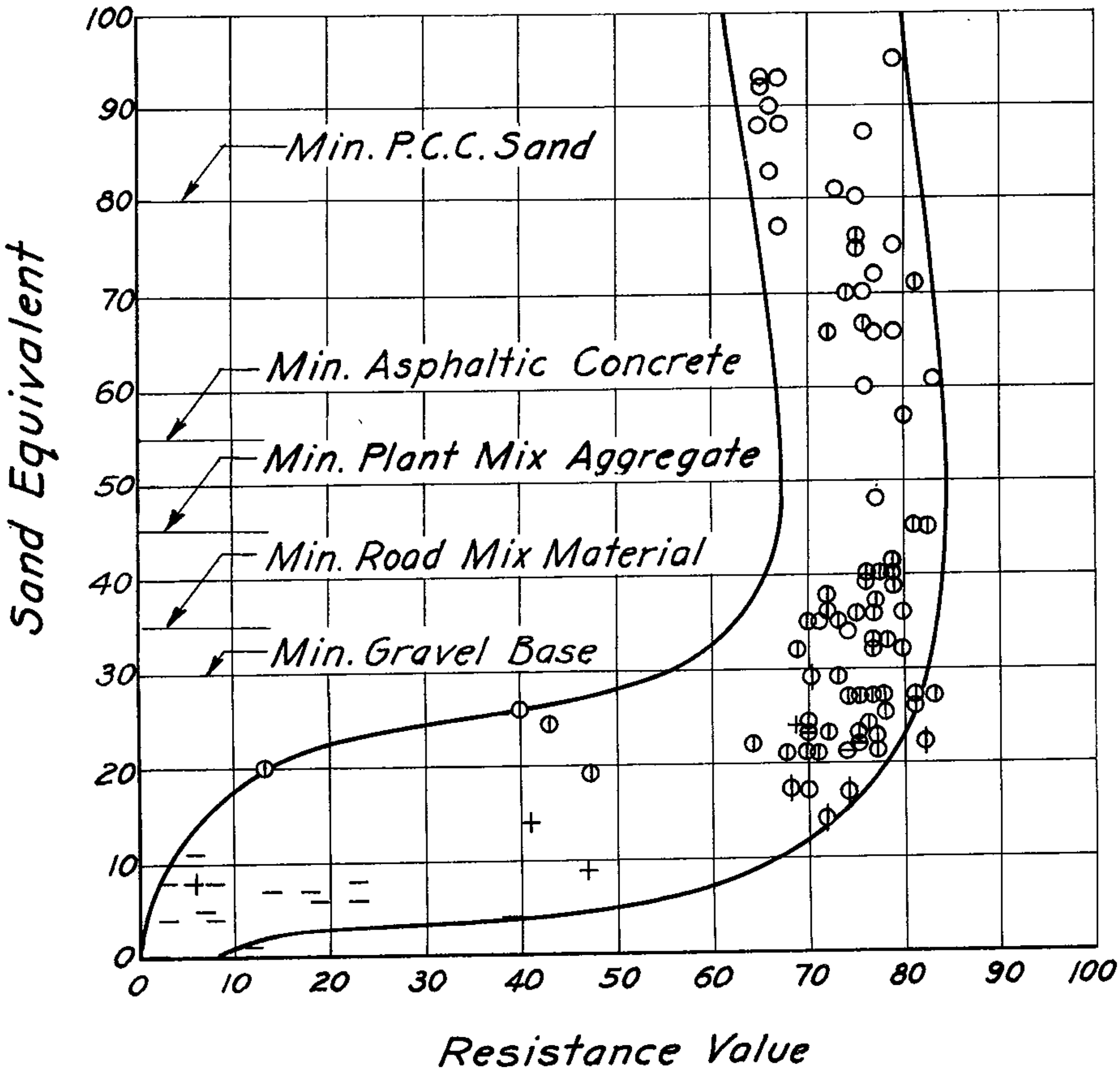

Fig. 14

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