THE EFFECT OF SKIP-GRADING ON STABILITY OF SOIL-AGGREGATE MIXTURES

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BY

A. FAIZ

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THE EFFECT OF SKIP-GRADING ON STABILITY OF SOIL-AGGREGATE MIXTURES

TO: J. F. McLaughlin, Director
Joint Highway Research Project

July 7, 1971

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FROM: Harold L. Michael, Associate Director
Joint Highway Research Project

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The attached Final Report titled "The Effect of Skip-Grading on Stability of Soil-Aggregate Mixtures" is submitted for acceptance as fulfilling the objectives of the Plan of Study of the same title approved by the Advisory Board on November 18, 1969. The report has been authored by the investigator on the research, Mr. Asif Faiz, Graduate Assistant in Research on our staff. Professor E. J. Yoder served as director of the research project.

The results of the investigation are presented in many charts and tables that show in detail the effect of the variation of the material between the No. 30 and No. 200 sieves on stability parameters. The conclusions suggest limiting values of dust ratio and percent passing the No. 30 sieve for soil aggregate mixtures with low plasticity fines and compacted to standard AASHTO density.

The project was totally JHRP financed. Comments on the report and reports of any application of the findings by the ISHC would be appreciated.

Respectfully submitted,

Harold L. Michael

Harold L. Michael
Associate Director

HLM:ms

cc: F. L. Ashbaucher
W. L. Dolch
W. H. Goetz
W. L. Grecco
M. J. Gutzwiller
G. K. Hallock

M. E. Harr
R. H. Harrell
M. L. Hayes
E. M. Mikhail
R. D. Miles
J. W. Miller

C. F. Scholer
M. B. Scott
W. T. Spencer
N. W. Steinkamp
H. R. J. Walsh
K. B. Woods
E. J. Yoder

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Final Report

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OF SOIL-AGGREGATE MIXTURES

by

Asif Faiz
Graduate Assistant in Research

Joint Highway Research Project

File: 6-18-9
Project: C-36-45J

Purdue University
Lafayette, Indiana
July 7, 1971

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"No man is an island, intire of it selfe; every man
is a peece of the continent, a part of the maine"

- John Donne

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ABSTRACT

Faiz, Asif. MSCE, Purdue University, June 1971. THE EFFECT OF SKIP-GRADING ON STABILITY OF SOIL-AGGREGATE MIXTURES. Major Professor: E. J. Yoder.

"What everybody echoes or in silence passes by as true today may turn out to be a falsehood tomorrow, mere smoke of opinion, which some had trusted for a cloud that would sprinkle fertilizing rain on their fields. What old people say you cannot do, you try and find that you can"

- Henry David Thoreau

Two main considerations govern the design of pavements. The first involves the design of a physical structure, that is capable of sustaining the applied loads. The second aspect of the problem is concerned with the design of materials that constitute the pavement structure. This investigation involves the latter aspect of the design.

Soil-aggregate mixtures are commonly used as base material under flexible and rigid pavements. This study evaluated the effect of varying the amount of material retained between the No. 30 and No. 200 sieves on the stability of soil-aggregate mixtures. For the extreme case, the test mixtures were completely devoid of material between the No. 30 and No. 200 sieves.

The study was confined to materials that conform essentially to Indiana specification No. 53 aggregate. The testing program included both gravel and crushed stone mixtures. A number of different soil

binders (material passing the No. 200 sieve) were used. These binders were classified as non-plastic, low plastic and high plastic fines.

To evaluate the effect of the material retained between the No. 30 and No. 200 sieves on the stability of various aggregate-soil binder combinations used in this study, the following tests were used:

1. Atterberg limits tests;
2. Compaction tests;
3. CBR test;
4. Texas triaxial compression test;
5. Permeability test.

The test data were analyzed statistically using analysis of variance techniques. It was determined that the variation of material between the No. 30 and No. 200 sieves had an important bearing on the stability of the soil-aggregate mixtures. This effect, however, was influenced by the quantity of fines present in the mixtures and the plasticity characteristics of the fines.

The results of this investigation are presented in the form of graphs and tables that show in detail the effect of the variation of the material between the No. 30 and No. 200 sieves on stability parameters. The conclusions suggest limiting values of dust ratio, and percent passing the No. 30 mesh sieve for the soil-aggregate mixtures with low plasticity fines and compacted to standard AASHO density.

INTRODUCTION

Paving engineers are concerned with two broad aspects of pavement design. The first is thickness design of a physical structure which is capable of sustaining the applied load. Design concepts for this are, for the most part, empirical; however, certain theoretical considerations can be used.

The second phase of the problem is concerned with the design of the materials with which the pavement will be built. Performance surveys have shown that type and quality of the base, for example, influence to a great extent the behavior of a pavement.

It is important to note that the above two design phases are interrelated and that it is difficult to separate these during the analysis of any particular problem. Also, it is necessary to consider many factors in the design of bases and subbases, including climate, position of the grade line, drainage characteristics of the subgrade, and many other factors.

In addition to the above, it is necessary to distinguish between the primary purposes for using bases and subbases under flexible pavements as compared to the use of the same materials under rigid pavements. For flexible pavements, the primary consideration is structural adequacy of the base and subbase layers whereas for rigid pavements, bases are used primarily for control of pumping and to provide a working platform on which to build the pavement.

For reason of economy and because of the growing scarcity of high quality aggregates, it is necessary to make as much use as possible of aggregates that do not require a great deal of processing. It is important to note, however, that for high type pavements it is false economy to permit deviation from specifications of base course materials since maintenance costs arising from the use of poor quality materials can be extremely high.

One approach to setting specifications for base aggregate would be to adopt a certain specification for bases under rigid pavements and another specification for bases under flexible pavements. Likewise, it would be possible to develop specifications for gravel bases and subbases, and another specification for crushed stone bases. From a practical point of view, however, this philosophy could cause complications from the point of view of production and control of the material. Hence, it is desirable, if at all possible, to develop specifications that can be applied to a variety of conditions.

Background for This Research

The standard specifications for the Indiana State Highway Commission (13)* set forth grading and plasticity requirements for bases and subbases used in the construction of pavements in Indiana. Particularly, reference is made here to the specification for size No. 53 aggregate, which establishes grading and plasticity requirements for granular materials irrespective of whether they be made from gravel or crushed stone. These specifications were established on the basis

*Numbers in parentheses refer to references listed in the bibliography.

of experience within the state of Indiana and have given satisfactory results.

Some aggregate producers in Indiana have found it difficult to meet the specifications for No. 53 aggregate in certain cases. For example, the specification requires that the amount of material passing the No. 30 sieve shall lie between 15 and 30 percent and the amount passing the No. 200 mesh sieve should lie between 5 and 10 percent. In addition to the above requirements, the specification states that the fraction passing the 200 sieve shall not exceed $\frac{2}{3}$ of the fraction passing the No. 40 sieve, the liquid limit of the fraction passing the No. 40 sieve shall not exceed 25 percent (except if slag is used it shall not exceed 35 percent) and the plasticity index shall not exceed 5 percent.

It has been suggested by some producers to deviate from the percentage requirements as specified for the No. 30 sieve. Specifically, it has been proposed to lower the minimum requirements of material passing the No. 30 sieve. In the extreme case, for example, if the lower limit of material passing the No. 30 sieve should be reduced to 10 percent and the maximum amount permitted passing the No. 200 sieve is kept at 10 percent, this would mean that the mix would have no material retained between the No. 30 and No. 200 sieves.

After an extensive review of the literature, it was determined that there is little factual information on the effect of the amount of material retained between the No. 30 and No. 200 sieves on the stability of soil-aggregate mixtures. Consequently, this research was developed to investigate the effect of skip-grading on the

stability of soil-aggregate mixtures with particular emphasis on the effect of the amount of material retained between the No. 30 and No. 200 sieves on the properties of the mix.

PURPOSE AND SCOPE

The primary objective of this study was to investigate the effect of varying the amount of material retained between the No. 30 and No. 200 sieves on stability of both gravel and crushed stone mixtures. Along with this, it was a further purpose to study the effect of plasticity and the interrelationship of plasticity and the material retained between the No. 30 and No. 200 sieves on stability.

The study was confined to materials that conform essentially to Indiana specification No. 53 aggregate.

The scope of this study included investigation of the following variables, using CBR tests for evaluation of stability:

1. gravel mixtures with low plasticity fines;
2. gravel mixtures with high plasticity fines;
3. crushed stone mixtures with low plasticity fines;
4. crushed stone mixtures with high plasticity fines.

In addition, triaxial compression tests were run on the following:

1. crushed stone mixtures with non-plastic fines;
2. crushed stone mixtures with low plasticity fines;
3. gravel mixtures with low plasticity fines.

Permeability tests were made on crushed stone and gravel mixtures having low plasticity fines.

REVIEW OF LITERATURE

General Properties of Soil-Aggregate Mixtures

Non-stabilized base courses are used under flexible pavements to increase the load-carrying capacity of the pavement by distributing the load through a finite thickness of pavement. This is the basic requirement for the base course, although it may provide drainage and give added protection against frost action. The strength properties of soil aggregate mixtures, when used in base courses, are influenced by their grain-size distribution, particle shape, plasticity, and permeability characteristics.

Under rigid pavements, base courses are used to prevent pumping, provide protection against frost action, prevent volume change of the subgrade, expedite construction, and provide drainage.

Effect of Fines

The stability of a soil-aggregate mix depends on particle-size distribution, particle shape, relative density, internal friction, and cohesion. Of these factors, the size distribution of the aggregate, particularly the proportion of fines to the coarse fraction is considered to be the most important factor. Figure 1 defines three idealized physical states of soil-aggregate mixes (19). Fines are defined as the proportion of the soil-aggregate mix passing the No. 200 sieve.

(a)

(b)

(c)

(a) Aggregate with no fines

Grain-to-grain contact

Variable density

Pervious

Nonfrost susceptible

High stability if confined,

low if unconfined

Not affected by adverse

water condition

Difficult to compact

(b) Aggregate with sufficient

fines for maximum density

Grain-to-grain contact with

increased resistance against

deformation

Increased density

Low permeability

Frost susceptible

Relatively high stability

in confined or unconfined

conditions

Not greatly affected by

adverse water condition

Moderately difficult to

compact

(c) Aggregate with great

amount of fines

Grain-to-grain contact

destroyed, aggregate "float-

ing" in soil

Decreased density

Low permeability

Frost susceptible

Low stability

Greatly affected by

adverse water condition

Not difficult to compact

FIG. 1. PHYSICAL STATES OF SOIL-AGGREGATE MIXTURES

(FROM YODER, PRINCIPLES OF PAVEMENT DESIGN, 1959)

Yoder and Woods (21) studied the variation of density and CBR with quantity of fines. The results of density and CBR tests made on soil-aggregate mixtures containing various percentages of fine material indicated that maximum densities for both well-graded crushed stone and gravel mixes resulted when the mixes contained between 8-10 percent fines. In contrast, maximum stability, as measured by the CBR test, resulted when gravel mixes contained 6-8 percent fines and crushed stone mixes contained 5-7 percent fines.

The results of work reported by Yoder and Woods indicated that:

1. For a given aggregate there is an optimum soil^{*} content at which maximum densities are attained. The soil content for maximum strength is less than the optimum as determined by density tests.
2. Optimum soil content decreases as compactive effort increases.
3. From the standpoint of density and strength, a small amount of soil is a desirable addition to base-course aggregates, but larger quantities are detrimental.

Gray (11) investigated the effect of quantity and plasticity of fines on gravels and crushed stone materials utilizing the triaxial tests. His results indicated that an excessive amount of fines not only lowers the load-carrying capacity of a base material, but may also make it frost susceptible. It is also possible that the optimum quantity of fines for maximum strength may not be the same as the optimum quantity of fines for good frost resistance.

* Soil as used herein refers to material passing a No. 200 mesh sieve.

A report by A & H Engineering and Testing Corporation (2) indicated that the maximum triaxial compressive strength (applied lateral pressure = 5 psi) for crushed stone mixes occurred when nine percent fines were present. In case of gravel mixtures, 11 percent fines resulted in maximum triaxial strength. A study of the variation of bearing ratios at 0.2 inch penetration with percent fines indicated that the maximum bearing ratios occurred when 9 and 11 percent fines were present in crushed stone and gravel mixes respectively.

To summarize, the quantity of fines in a soil-aggregate mix has a major influence on maximum density, strength, frost-resistance, and drainage.

Effect of Crushed Material

The strength characteristics of aggregate bases and subbases are derived principally from inter-particle friction, and the amount of friction which is mobilized is a function of unit weight, the gradation of the particles, particle shape, and the surface texture of the particles. The permeability properties are also influenced by particle shape and surface texture.

The shape of particles and the surface texture has been defined as the "shape factor" by Townsend and Madill (18). The "shape factor" serves as a measure of angularity. Studies conducted by Townsend and Madill indicated that as the angularity of the coarse fraction of a soil-aggregate mix increases there is:

1. a slight decrease in the unit weight for the same compactive effort;

2. a substantial increase in the stability and strength of the material;
3. a decrease in the permeability of the material, if the void ratio is kept constant.

Yoder and Lowrie (20) determined that crushed stone is better than round-grained gravel with respect to pavement thickness. The difference between the two materials varied directly with gradation from coarse to fine.

Yoder (19) found that relatively higher strength values (as indicated by CBR tests) are obtained for crushed stone than for gravel mixes, for the same compactive effort and proportion of fines in the soil-aggregate mix. However, lower values of density may be obtained for crushed stone as compared to gravel.

Effect of Plasticity

Soil-aggregate mixtures exhibit greatest strength when non-plastic fines are used. There appears to be a definite relationship between liquid limit, plasticity index, and the strength characteristics as measured by the CBR test. This relationship indicates that high plasticity is one of the most detrimental characteristics. There is a definite reduction in strength of the granular materials as plasticity index of the soil binder increases. A study made by the A & H Engineering and Testing Corporation (2) on the influence of plasticity on bearing ratio and triaxial compressive strength indicated that a substantial decrease in bearing ratio and triaxial strength occurs, in case of both gravel and crushed stone mixes, with increase in percentage of fines. This is due to the fact that an increase in

the quantity of plastic fines in a given soil-aggregate mix would result in an increase in the plasticity of the mix.

The results of a series of tests by Deklotz performed on granular mixtures with three types of binders established that the effect of soil-binder plasticity was most pronounced for high soil contents (10).

To conclude, the plasticity index of the fine fraction has a significant influence on the strength of granular materials. As the plasticity of the minus No. 40 material increases, the strength of the granular material is reduced. Although many specifications control the upper limit of PI at five or six percent, it should be recognized that the quantity of fines present in the soil-aggregate mixtures also influences the effect of plasticity.

As a rule, the lowest possible plasticity index should be strived for in order to achieve the best performance, if the percentage of soil present in the soil-aggregate mixtures is above the value as obtained by:

$$p = 100 \sqrt{\frac{d}{D}} \quad (1)$$

where

- d = the size of opening of No. 40 sieve,
- D = the maximum size of the aggregate,
- p = percent of weight finer than No. 40 sieve.

Permeability

The permeability characteristics of soil-aggregate mixtures are dependent upon:

1. grain-size distribution;
2. type of coarse aggregate;

3. type of binder;

4. density.

The permeability characteristics are related by:

$$v = ki \quad (2)$$

where

v = discharge velocity,

i = hydraulic gradient (loss in head/unit length),

k = coefficient of permeability.

The coefficient "k" has the units of velocity and depends on the properties of the permeable mass. It is also a function of density and viscosity of water. It is important to recognize that granular materials become less porous when soil binder is added. Crushed materials, in general, have higher rates of permeability, for a given gradation, than most common gravels (4, 18).

Intrusion

Soil-aggregate mixes used in base courses must be designed with a view to avoid intrusion of subgrade material into the base or removal of the fines from the mix by pumping under the action of repeated loads.

Yoder and Chamberlain (9) observed that an optimum range of grading exists within which a soil aggregate mix, used in a base course, will not experience significant subgrade intrusion into the base course. This range exists approximately between the grain-size distributions given by $p = 100 \left(\frac{d}{D} \right)^n$, where n varies from 0.7 to 1.2.

Effect of Skip-Grading

Little or no work has been done on the effect of skip-grading of granular materials on the behavior of soil-aggregate mixtures. However, the effects on properties of bituminous concrete have been evaluated by several individuals.

Instances have been reported by various agencies where aggregate compatible with their own specifications resulted in bituminous mixtures that were difficult to compact and remained tender for some time after rolling. Others have reported instances of splotchy pavements resulting as a consequence of using such aggregates. On plotting the percent passing a particular sieve vs. size of opening of the sieve raised to 0.45 power, the grading curves of these problem mixes indicated a well-defined hump at the No. 30 and No. 50 sieve sizes. It was noted that when the hump in the gradation curve at the No. 30 and No. 50 sieve sizes was absent, the aggregates produced satisfactory bituminous mixtures (5).

According to the work done by the Bureau of Public Roads (5), nearly all gradation curves of problem mixtures have been characterized by a hump above the max. density line* at or near the No. 30 sieve; such mixtures had an excess of fine sand in relation to total sand. This excess not only results in lower compacted densities, but tends to float the larger particles and destroys stability that

* Maximum density line is defined as the gradation curve obtained by using $n = 0.45$ in the relationship, $p = 100 \left(\frac{d}{D} \right)^{0.45}$

where p = percentage passing a particular sieve,
 D = maximum size of aggregate,
 d = size of opening of the sieve.

might otherwise result from coarse aggregate interlock. Also, fine sand is inherently less stable than coarse sand.

The report by the Bureau of Public Roads concluded that:

1. For gradations of the same type of aggregate which plot as identical curves except for the portion between No. 8 and No. 200 sieves, those that show appreciable humps above the maximum density line at about No. 30 sieve will have higher mineral voids and lower stabilities than those plotting with lesser humps. Gradation humps in the finer aggregate sizes have a detrimental effect.
2. For skip-gradations, low mineral voids are associated with curves that stay appreciably below the maximum density line in the coarse aggregate zone.

Specifications for Soil-Aggregate Mixtures

Most specifications for base-course materials require that the plasticity index of the minus No. 40 sieve fraction of the mix be no greater than six percent and the liquid limit should not exceed 25 percent. The first suggestions for a base-course specification, based on consistency limits, date back to 1936 when Hogentogler and Willis (12) first formulated upper limits for the plasticity index and liquid limit.

Carpenter and Willis (7,8) arrived at the same conclusion after conducting circular track tests on sand-clay, and gravel-sand-clay materials.

The above mentioned results were then adopted by AASHO and these specified limits of plasticity index and liquid limit together with limits on grain-size distribution became the basis of specifications for coarse aggregates. Such specifications are widely used by most highway departments. These specifications further require that the material be well-graded and place special emphasis on the amount of fines permitted in the soil-aggregate mix.

The limiting plasticity values stipulated in most specifications do not appear to be supported by published verification, and possibly have evolved from a purely empirical standpoint, dependent on performance in frost susceptible areas, as the criterion for evaluation.

Consequently, these limits are in certain cases restrictive. For example, when the amount of fines contained in the base-course is low, it may be possible to permit some leeway in plasticity requirements without causing great harm to the mix. This would particularly hold for areas not subject to frost action.

Routine specification tests suffer from a number of limitations. It is difficult to obtain consistent and reproducible results from plasticity tests. Also, the interpretations possible from the same grading and plasticity data may vary considerably. Hence, the use of index properties as selection criteria for soil-aggregate mixtures is at best debatable. This necessitates a critical approach to the design of specifications for pavement base materials and the investigation of a more rational method for selecting base-course materials.

Besides economic considerations, the stability of a pavement should be the guiding factor in establishing specifications to ensure

stable and workable pavement materials. This indicates the need for a direct method of material selection by means of a strength test rather than by use of empirical soil parameters. In the present specifications, dependent primarily on consistency limits, the stability of the material is implied rather than actually measured. Commonly, the CBR test is normally referred to for the valuation of strength properties of base materials. In practice, this test is rarely used for the selection of base course materials.

The Texas specifications (15) for flexible base materials offer another method for selection of adequate flexible base materials. Triaxial compressive strength together with consistency limits and grain-size distribution requirements form the criteria for evaluating base materials. Table 1 shows the Texas requirements for flexible base materials.

According to the Texas method, Grade 1 material would be used in cases where special tough hard particles, which test as Triaxial Class 1, are to be used. Use would be limited to local areas. Grade 2 would be more widely used than Grade 1 because requirements are less strict and this material is still capable of supporting traffic when covered with a relatively thin bituminous surface. Triaxial strength requirements are necessary in order to assure this. Generally, most final base courses should be equal to or better than Grade 2. Grade 3 generally represents satisfactory subbase materials. Grade 4 material is intended for use as stabilized bases; however, these are generally used for light traffic roads where a surfacing may need to be placed on such material. There are isolated cases

TABLE I. PHYSICAL REQUIREMENTS FOR FLEXIBLE BASE MATERIAL* (FROM TEXAS HIGHWAY DEPARTMENT).

TYPES	GRADES				GRADE 4: SHOW SPEC. LIMITS ON PLANS
	GRADE 1: TRIAXIAL CLASS 1 MIN. COMPRESSIVE STRENGTH PSI: 45 AT 0 LB LATERAL PRESSURE AND 175 AT 15 LB LATERAL PRESSURE	GRADE 2: (TRIAxIAL CLASS 1 TO 2.3) MIN COMPRESSIVE STRENGTH, PSI: 35 AT 0 LB LATERAL PRESSURE AND 175 AT 15 LB LATERAL PRESSURE	GRADE 3: (UNSPECIFIED TRIAXIAL CLASS RANGE PROBABLY FROM 2.3 TO 3.7)		
TYPE A CRUSHED OR BROKEN AGGREGATE	RETAINED ON SQ. SIEVE 1 3/4" 0 7/8 8-30 3/8 30-50 NO. 4 45-65 NO. 40 70-80 MAX. LL 35 MAX. PI 12 MAX WET BALL MILL 40 MAX. LA WEAR+4 40	RETAINED ON SQ. SIEVE 1 3/4" 0-10 NO. 4 45-75 NO. 40 60-85 MAX. LL 40 MAX. PI 12 MAX WET BALL MILL 50	RETAINED ON SQ. SIEVE 1 3/4" 0-10 NO. 40 60-85 MAX. LL 45 MAX. PI 15 WET BALL MILL 55		THIS GRADE AND TYPE OF FLEXIBLE BASE MATERIAL IS INTENDED FOR USE AS UNTREATED SUBBASE OR AS STABILIZED BASE OR SUBBASE AND SHOULD NOT BE USED AS A FINAL COURSE OR UNTREATED BASE WITHOUT SPECIAL APPROVAL.
TYPE B GRAVEL AGGREGATE		RETAINED ON SQ. SIEVE 1 3/4" 0-10 NO. 4 30-75 NO. 40 70-85 MAX LL 35 MAX. PI 12	RETAINED ON SQ. SIEVE 1 3/4" 0-5 NO. 4 30-75 NO. 40 65-85 MAX LL 35 MAX. PI 12		
TYPE C IRON ORE TOP SOIL		RETAINED ON SQ. SIEVE 2 1/2" 0 NO. 40 50-85 MAX LL 35 MAX. PI 12	RETAINED ON SQ. SIEVE NO. 40 45-85 MAX. LL 35 MAX. PI 12		
TYPE D SAND - SHELL		RETAINED ON SQ. SIEVE 1 3/4" 0-10 NO. 4 45-65 NO. 40 50-70 MAX. LL 35 MAX. PI 12	RETAINED ON SQ. SIEVE 1 3/4" 0-10 NO. 40 45-65 MAX LL 35 MAX. PI 12		
TYPE E SHELL WITH SAND AND CALICHE ADMIXTURE		RETAINED ON SQ. SIEVE 1 3/4" 0 NO. 40 45-65 MAX LL 35 MAX. PI 10	RETAINED ON SQ. SIEVE 1 3/4" 0 NO. 40 45-65 MAX. LL 35 MAX. PI 12		

*LIMIT TESTS MADE AFTER MATERIAL IS PREPARED BY WET METHOD

where very rocky materials with high PI may give satisfactory services and adequate triaxial results. Grade 4 would take care of these unusual, but explainable circumstances (15).

McDowell (16) compared the AASHO and Texas specifications for flexible base materials. As a result of this study, McDowell concluded that the performance of roads in Texas showed no relationship with the soil constants (liquid limit and plasticity index) and he further concluded that the results of his study did not substantiate the specifications set up by AASHO. Many of the materials tested by McDowell contained low percentages of fines. McDowell further suggested that the AASHO should reconsider the specifications for soil-aggregate mixtures and recommended that liquid limit values as high as 35 percent and plasticity index values as high as 12 percent may be permitted. However, suggestions by McDowell must be modified on the basis of the fact that the Texas Highway Department uses the "wet" method for preparing samples for the consistency limits tests. In general, "wet" preparation method results in higher indices than would be obtained using the dry preparation method.

The method of evaluating base course material by means of the Texas method offers a rational means of selecting base materials and exhibits obvious virtues as compared with the purely empirical methods that are generally in vogue at present.

EXPERIMENTAL PROGRAM

This study was confined to base course materials that essentially conform with Indiana specification size No. 53 aggregate. The amount of material between the No. 30 and No. 200 mesh sieves was varied to study the effect of this variation on stability characteristics of soil aggregate mixtures.

The specifications for size No. 53 aggregate are given in Table 2.

TABLE 2. SPECIFICATIONS FOR INDIANA SIZE NO. 53 AGGREGATE

Indiana Size No. 53	Total Percent Passing Sieve Having Square Opening							
	1½"	1"	¾"	½"	#4	#8	#30	#200
	100	80-100	70-90	55-80	35-60	25-50	15-30	5-10

The specifications further require that the gradation of material above the No. 8 sieve shall be uniform within the limits shown and shall not vary from the low limit on one sieve size to the high limit on the adjacent sieve size. The fraction passing the No. 200 sieve shall not exceed 2/3 the fraction passing the No. 40 sieve. These specifications further stipulate that the liquid limit of the fraction passing the No. 40 sieve shall not exceed 25 percent and the plasticity index shall not exceed 5 percent.

In Figure 2, the bold curves show the grading limits for the Size No. 53 aggregate. The curve between these limits indicates the grain-size distribution for a material conforming to the maximum density gradation equation,

$$p = 100 \left(\frac{d}{D} \right)^n \quad (3)$$

where

p = percent by weight finer than a given sieve,

d = size of given sieve,

D = maximum size of aggregate,

n = 0.5.

For this study D was held constant at 1". Hence,

$$p = 100 \sqrt{d} \quad (4)$$

where

d = size of sieve opening in inches.

After comparing the Indiana specifications for No. 53 aggregate and the grain-size distribution for maximum density as shown in Figure 2, the grain-size distributions for the materials to be tested were selected. These are shown in Figures 3 and 4. In all cases, the grain-size distributions through the No. 8 sieve remained the same.

In order to study the effect of skip-grading between the No. 30 and No. 200 mesh sieves, two specific cases were studied. In the first case, the quantity passing the No. 200 mesh sieve was held constant at five percent and the total percent passing the No. 30 sieve was varied from 25 percent to five percent at increments of

FIG. 2. INDIANA SIZE NO. 53 COARSE AGGREGATE COMPARED WITH MAXIMUM DENSITY GRADING FOR $n=0.5$ AND $D=1''$.

O—ORANGE OF GRAIN SIZE DISTRIBUTION FOR INDIANA SIZE NO. 53 SOIL-AGGREGATE MIXTURE
 Δ---ΔGRAIN SIZE DISTRIBUTION OF TEST MIXTURES WITH 5% PASSING NO. 200 SIEVE

FIG.3. GRAIN SIZE DISTRIBUTION OF TEST MIXTURES WITH 5 PERCENT
 PASSING NO. 200 SIEVE.

O RANGE OF GRAIN SIZE DISTRIBUTION FOR INDIANA SIZE NO. 53 SOIL-AGGREGATE MIXTURE
 Δ GRAIN SIZE DISTRIBUTION OF TEST MIXTURES WITH 10% PASSING NO. 200 SIEVE

FIG. 4. GRAIN SIZE DISTRIBUTION OF TEST MIXTURES WITH 10 PERCENT
 PASSING NO. 200 SIEVE.

ten percent (Figure 3). In the second case, the quantity passing the No. 200 sieve was held constant at ten percent while the total percent passing the No. 30 sieve was varied from 30 percent to ten percent at increments of ten percent (Figure 4). It may be of interest to note that in both cases the amount of material retained between the No. 30 and No. 200 sieves was varied from 20 percent to zero percent at increments of ten percent.

These distributions resulted in six distinct soil-aggregate mixtures. The gradations for these materials are shown in Table 3.

Materials

The materials used in this study consisted of gravel and crushed stone aggregates. These materials were obtained from local stock-piles. In order to encompass a relatively wide range of plasticity values, four different types of soil binders were used.

A soil binder in this study is defined as the material passing the No. 200 sieve. Henceforth, all such materials will be termed as fines. The plasticity characteristics of the different types of fines are summarized in Table 4. These fines will be referred to using the descriptive terms shown in the last column of Table 4.

To manufacture the test mixtures, No. 1 fines (CL, low PI) were combined with gravel, whereas No. 2 fines (CL-ML, low PI) were combined with crushed stone. The No. 3 fines (CH, high PI) were used with both gravel and crushed stone in high PI mixtures. The No. 4 fines (ML, NP) were combined only with crushed stone to prepare test mixtures for triaxial compression tests.

TABLE 3. GRAIN SIZE DISTRIBUTION OF TEST MIXTURES

PERCENT PASSING NO. 200 SIEVE	5			10		
PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES	0	10	20	0	10	20
TEST MIXTURE NO.	1	2	3	4	5	6
GRAIN SIZE	PERCENT FINER	PERCENT FINER	PERCENT FINER	PERCENT FINER	PERCENT FINER	PERCENT FINER
1"	100	100	100	100	100	100
3/4"	80	80	80	80	80	80
1/2"	67.5	67.5	67.5	67.5	67.5	67.5
3/8"	61.5	61.5	61.5	61.5	61.5	61.5
NO. 4	47.5	47.5	47.5	47.5	47.5	47.5
NO. 8	37.5	37.5	37.5	37.5	37.5	37.5
NO. 30	5	15	25	10	20	30
NO. 200	5	5	5	10	10	10

TABLE 4. PLASTICITY CHARACTERISTICS OF VARIOUS TYPES OF FINES.

NO.	CLASSIFICATION	LL,(PERCENT)	PL,(PERCENT)	PI,(PERCENT)	TYPE OF FINES
1	CL	30	21	9	LOW PI
2	CL-ML*	26	20	6	LOW PI
3	CH	77	31	46	HIGH PI
4	ML**	NP	NP	NP	NP

* RESIDUAL SOIL DERIVED FROM LIMESTONE.

** ROCK DUST, PRODUCED BY PULVERIZING LIMESTONE.

Evaluated Variables

To evaluate stability and strength characteristics, California Bearing Ratio (CBR) test and Texas triaxial test were used. Permeability tests were run on selected samples to evaluate drainage characteristics. Before these, a series of routine soil tests were run. These consisted of limits tests and compaction tests.

CBR tests were run on a relatively large variety of soil-aggregate mixes. Both gravel and crushed stone were used. Soil binders were confined to low PI and high PI fines. For any combination of aggregates and fines, six distinct mixes were used, as described earlier. As both gravel and crushed stone were used, each with two different types of fines, a total of 24 different soil-aggregate mixes resulted. Prior to running the CBR tests, limits tests and compaction tests were made on each of these soil-aggregate mixtures to evaluate plasticity index, maximum density, and optimum moisture content.

Texas triaxial tests were confined to the following materials:

1. gravel with low PI fines;
2. crushed stone with low PI fines;
3. crushed stone with NP fines.

This resulted in 18 distinct test mixtures. Prior to running the triaxial compression tests, compaction and limits tests were run on each test mixture.

Permeability tests were run only on crushed stone and gravel mixtures with low PI fines. This resulted in 12 test mixtures.

Design of Experiments

The design of an experiment involves planning an experiment so that information will be collected which is relevant to the problem under investigation. Thus, it is the complete sequence of steps taken ahead of time to insure that the appropriate data will be obtained in a way which permits an objective analysis leading to valid inference. The purpose of any experimental design is to provide a maximum amount of information relevant to the problem under investigation.

The three basic principles of experimental design are replication, randomization, and local control. Replication means the repetition of the basic experimental unit. This provides an estimate of experimental error which acts as a unit of measuring and assessing the significance of observed differences. Randomization refers to the random assignment of treatments to the experimental units so that the treatments will not be favored or handicapped systematically by extraneous sources of variation. Local control refers to controls, like balancing, blocking, or grouping, applied to the experimental units. The function of local control is to make any test of significance more sensitive or to make the test procedure more powerful.

Definitions (17):

1. Experimental Unit:

is that unit to which a single treatment (which may be a combination of many factors) is applied in one replication of the basic experiment.

2. Experimental Error:

describes the failure of two identically treated experimental units to yield identical results. This incorporates errors of experiment, errors of observation, errors of measurement, variation of experimental material, and extraneous factors which could affect the outcome of experiments, but which are not singled out.

3. Treatment:

this implies a particular set of experimental conditions which will be imposed on an experimental unit within the confines of the chosen design.

4. Factor:

is an independent variable.

Example: in CBR testing, a treatment might refer to a) molding moisture content; b) soaking or absence of it; c) surcharge weights; d) type of compaction; or d) a combination of the above. The last treatment would be called a treatment combination.

A brief description of the experimental designs used in this study is given below.

CBR Tests. A $2 \times 2 \times 2 \times 3 \times 2$ factorial design was used for the CBR tests. The five independent variables or factors involved in this experiment were:

1. material type;
2. plasticity of fines;
3. percent material passing the No. 200 sieve (percent fines);

4. percent material retained between the No. 30 and No. 200 sieves;

5. percent compactive effort used in molding the CBR specimens.

The dependent or the measured variable was the California Bearing Ratio (CBR). The experiment was completely replicated and all experimental units were randomized. This means that the CBR tests were performed in a random order to reduce systematic bias introduced by experimental error and other unquantified extraneous effects. Two CBR tests were run per experimental unit.

Texas Triaxial Tests. Two factorial designs were used for the Texas triaxial compression tests. Both designs were $2 \times 2 \times 3$ factorials. In the first design, the three independent variables or factors consisted of the following:

1. material type;
2. percent passing the No. 200 sieve (percent fines);
3. percent retained between the No. 30 and No. 200 sieves.

In the second design, the three independent variables or factors consisted of:

1. plasticity of fines;
2. percent passing the No. 200 sieve (percent fines);
3. percent retained between the No. 30 and No. 200 sieves.

In these designs, the measured variables were the triaxial compressive strength at 15 psi lateral pressure and the angle of internal friction.

Replication was not used in the triaxial tests, i.e., only one set of observations was taken per experimental unit. However, the triaxial tests were performed in a random order.

Permeability Tests. A study of published data did not justify a factorial design as the results of these tests are easy to analyze and the significance of various factors involved in the experimentation can be evaluated without taking recourse to statistical analysis.

TEST PROCEDURES

The experimental program involved the use of the following tests:

1. Atterberg limits tests;
2. Compaction tests;
3. CBR test;
4. Texas triaxial test;
5. Permeability test.

Preparation of Materials

In order to prepare the test mixtures, the gravel and crushed stone were separated into fractions and recombined as desired. Similarly, the fines were prepared separately and added to the aggregates in desired quantity. The bulk material was first dry sieved into the following eight fractions: 1"-3/4"; 3/4"-1/2"; 1/2"-3/8"; 3/8"-No. 4; No. 4-No. 8; No. 8-No. 30; No. 30-No. 200; and minus No. 200. Then the coarse fractions (retained No. 200 sieve) were wet sieved to remove all surface fines. These fractions were combined in accordance with the gradations outlined in Table 3 to give the six typical test mixtures for each combination of aggregates and fines.

Atterberg Limits Tests

The liquid limit and plastic limit were determined in accordance with ASTM Designations D 423-61T and D 424-59 respectively (3).

However, the method of preparing test samples was slightly modified.

For any soil-aggregate mix under investigation the ratio of percent retained between the No. 40 and No. 200 sieves to percent fines (passing No. 200 sieve) was determined from the grain-size distribution curves shown in Figures 3 and 4. Then the soil-aggregate fraction retained between the No. 40 and No. 200 sieves was combined with the passing No. 200 fraction, in accordance with this ratio, to prepare the test sample.

Compaction Tests

The compaction tests run prior to CBR testing were conducted in accordance with ASTM Designation D 698-64T, method-D (3). In all such tests, material retained on 3/4-in. sieve was replaced by an equal weight of material passing the 3/4-in. sieve and retained on No. 4 sieve.

In case of the preliminary compaction tests preceding the Texas triaxial tests, a modified method was used. The size of mold used for compaction was 6-in. diameter, and 8-in. in height. In order to obtain a compactive effort of 13.26 ft.-lb. per cu. in., the soil was compacted in four layers with 50 blows applied to each layer. A 10-lb. hammer with a free fall of 18-in. was used. The procedure for determining the optimum moisture content and maximum density remained the same as outlined in ASTM Designation D 698-64T. The material retained on 3/4-in. sieve was not replaced.

California Bearing Ratio Test

Soaked CBR tests were performed in accordance with ASTM Designation D1883-61T (3). The test specimens were soaked and tested under a surcharge of 10 lb. Each soil-aggregate mix was molded using two different compactive efforts. In the first instance the compactive effort was 56 blows, applied to three equal layers with a 5½ lb. hammer as specified in ASTM Designation D1883-61T. In the second case the compactive effort was reduced by 50 percent and the specimens were molded by 28 blows of a 5½ lb. hammer applied to each of the three equal layers.

In each case the test was replicated, i.e., for each soil-aggregate mix, four CBR tests were conducted. For two tests 56-blow compaction was utilized whereas for the other two, the compactive effort was reduced to 28 blows per layer.

Texas Triaxial Test

In essence, the Texas triaxial tests conformed to AASHO designation T 212-65 (1). However, slight modifications were effected in the standard method. The version of Texas triaxial test used in this study could be termed as an abbreviated (unsoaked) test. Three test specimens of each soil-aggregate type were molded at optimum moisture content, as determined by the compaction test described earlier. These specimens were tested immediately after molding in accordance with the test method outlined in AASHO designation T212-65. The confining pressure was varied from 5 psi to 15 psi with an intermediate value of 10 psi. Test results were reported in terms of the

Texas triaxial class, the triaxial compressive strength obtained at a confining pressure of 15 psi, and the angle of internal friction obtained from the Mohr's envelope.

Permeability Test

A constant head permeameter was used for this test. The permeameter was locally constructed and in principle the design conformed to the one outlined in the "Proposed Method of Test for Permeability of Granular Soils" under the jurisdiction of ASTM Committee D-18 (3). The testing procedure also conformed to the proposed method.

The permeameter cylinder had a 6-in. diameter and a 7-in. height. The test samples were molded dynamically by a 10-lb. sliding hammer with an 18-in. drop. The soil was placed in four layers and 44 blows were applied to each layer. This resulted in a compactive energy of 13.26 ft.-lb. per cu. in. It may be recalled here that the samples for the triaxial tests were molded using the same compactive energy. Hence, the test samples for both the permeability and Texas triaxial tests were molded using the same degree of compaction. This offered a common basis for comparing the permeability and triaxial strength characteristics of various soil-aggregate mixes.

The coefficient of permeability was computed using the formula,

$$k = \frac{Q L}{A t h} \quad (5)$$

where

k = coefficient of permeability (cm/sec),

Q = quantity of water discharged (cm³),

L = height of sample (cm),

A = cross-sectional area of sample (cm^2),

t = total time of discharge (sec),

h = total head lost (cm).

RESULTS

Test Data

Test data are presented in Appendix 1. Moisture content-density data for CBR tests are shown in Table A-1; CBR data are tabulated in Table A-2; Compaction test data for triaxial compressive tests are shown in Table A-3; and Table A-4 shows the Texas triaxial test results. The results of permeability tests are presented in Table A-5. The limits tests data are shown in Tables A-2 and A-4.

Statistical Analysis

Statistical analysis was used in analyzing the CBR and Texas triaxial test data. The statistical tool most often used in this study was analysis of variance. The permeability test data did not justify the use of statistical analysis as the test results could be analyzed in the light of similar studies in the past.

CBR Test Data

The following analysis of variance model was formulated for the CBR test results:

$$\begin{aligned}
 Y_{ijklmn} = & U + A_i + B_j + C_k + D_l + F_m + AB_{ij} + AC_{ik} \\
 & + AD_{il} + AF_{im} + BC_{jk} + ED_{jl} + BF_{jm} + CD_{kl} \\
 & + CF_{km} + DF_{lm} + ABC_{ijk} + ABD_{ijl} + ABF_{ijm} \\
 & + ACD_{ikl} + ACF_{ikm} + ADF_{ilm} + BCD_{jkl} + BCF_{jkm}
 \end{aligned}$$

$$\begin{aligned}
& + BDF_{jlm} + CDF_{klm} + ABCD_{ijkl} + ABCF_{ijkm} \\
& + ABDF_{ijlm} + ACDF_{iklm} + BCDF_{jklm} + ABCDF_{ijklm} \\
& + E_n(ijklm)
\end{aligned} \tag{6}$$

where

- Y_{ijklmn} = CBR value obtained from a given test,
 U = true mean value for the population,
 A_i = true differential effect of aggregate type
 (gravel vs. crushed stone),
 B_j = true differential effect of plasticity of
 fines (low vs. high),
 C_k = true differential effect of percent passing
 No. 200 sieve (5% vs. 10%),
 D_l = true differential effect of percent retained
 between the No. 30 and No. 200 sieves,
 F_m = true differential effect of compaction
 (56 blows vs. 28 blows),
 $E_n(ijklm)$ = true error effect of replicates.

The other terms denote interactions between the main effects A, B, C, D, and F. The subscripts assume the values:

- $i = 1, 2$
 $j = 1, 2$
 $k = 1, 2$
 $l = 1, 2, 3$
 $m = 1, 2$
 $n = 1, 2.$

In the ANOVA model, main effects A, B, C, D, and F are fixed while the effect E is random.

The fixed-effect model for the CBR data was formulated on the basis of the following assumptions:

1. homogeneity of variance;
2. normality;
3. additivity;
4. independence of the errors.

As there were only two replicates for each treatment combination, Bartlett's test and q-test could not be used for testing the assumption of homogeneity of variance. In order to accomplish this end, the range test was used.

Table 5 shows the CBR data layout for analysis of variance. It will be seen from this table that there are 48 cells consisting of two replicates each. Table 6 gives the range and mean of the CBR values within each of the 48 cells. In this discussion, $\bar{X}$ represents the cell mean value while R represents the range value. From Table 6,

$$\sum_{i=1}^{48} R_i = 659.2$$

$$\bar{R} = \frac{\sum R}{48} = \frac{659.2}{48} = 13.73$$

The control limits (6) are given by,

$$UCL_R = D_4 \bar{R} \quad (7)$$

$$LCL_R = D_3 \bar{R} \quad (8)$$

TABLE 5. SUMMARY OF CBR TEST DATA.

AGGREGATE TYPE	PLASTICITY OF FINES	PERCENT PASSING NO. 200 SIEVE	PERCENT RETAINED BETWEEN NO. 30-NO. 200 SIEVES	COMPACTION			
				56 BLOWS		28 BLOWS	
				1	2	1	2
GRAVEL	LOW	5	0	60.0	46.7	45.3	34.2
			10	88.8	96.7	38.3	49.3
			20	98.9	95.0	73.3	75.3
		10	0	184.9	182.6	76.7	77.8
			10	130.0	153.0	102.7	99.3
			20	61.4	53.3	49.3	70.7
	HIGH	5	0	97.0	52.0	46.7	31.3
			10	77.3	80.0	57.3	45.0
			20	113.3	85.3	44.0	83.3
		10	0	88.0	60.0	43.3	32.6
			10	30.0	26.7	32.0	23.3
			20	25.3	44.0	22.0	28.0
CRUSHED STONE	LOW	5	0	52.0	51.5	46.7	38.0
			10	110.7	81.4	43.3	60.0
			20	143.3	145.0	128.0	108.0
		10	0	101.4	100.0	46.7	64.7
			10	128.0	144.7	98.7	70.0
			20	123.3	163.3	125.3	96.7
	HIGH	5	0	80.0	51.6	37.3	24.7
			10	93.9	86.7	80.0	73.7
			20	118.7	120.0	53.3	61.3
		10	0	68.6	75.3	49.3	38.3
			10	55.0	49.3	31.3	38.7
			20	50.0	24.0	30.0	31.3

TABLE 6. TABULATION OF CBR MEANS AND RANGES.

AGGREGATE TYPE	PLASTICITY OF FINES	PERCENT PASSING NO. 200 SIEVE	PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES	COMPACTION			
				56 BLOWS		28 BLOWS	
				CELL MEAN $\bar{X}$	RANGE R	CELL MEAN $\bar{X}$	RANGE R
GRAVEL	LOW	5	0	53.4	13.3	39.8	11.1
			10	92.8	7.9	43.8	11.0
			20	47.0	3.9	74.3	2.0
		10	0	183.8	2.3	77.3	1.1
			10	141.5	23.0	101.0	3.4
			20	57.4	8.1	60.0	21.4
	HIGH	5	0	74.5	45.0	39.0	15.4
			10	78.7	2.7	51.1	12.3
			20	99.3	28.0	63.7	39.3
		10	0	74.0	28.0	38.0	10.7
			10	28.4	3.3	27.7	8.7
			20	34.7	18.7	25.0	6.0
CRUSHED STONE	LOW	5	0	51.8	0.5	42.4	8.7
			10	96.1	29.3	51.7	16.7
			20	144.2	1.7	118.0	20.0
		10	0	100.7	1.4	55.7	18.0
			10	136.4	16.7	84.4	28.7
			20	143.3	40.0	111.0	28.6
	HIGH	5	0	65.8	28.4	31.0	12.6
			10	90.3	7.2	76.9	6.7
			20	119.4	1.3	57.3	8.0
		10	0	72.0	6.7	43.8	11.0
			10	52.2	5.7	35.0	7.4
			20	37.0	26.0	30.7	1.3

where

UCL_R = upper control limit,

LCL_R = lower control limit,

D_4 = 3.267

D_3 = 0

$$\therefore UCL_R = 3.267 \times 13.73 = 44.89$$

$$LCL_R = 0 \times 13.73 = 0$$

Only one range value is above the UCL_R of 44.89. As all but one of the range values are below the critical upper control limit, the CBR data satisfies the requirement of homogeneity of variance.

Figure 5 shows a plot of range values, R vs. cell mean, $\bar{X}$, values. The scatter of the plotted data indicates that there is no apparent correlation. Therefore, the data was not transformed before ANOVA computations.

For calculating ANOVA, BMD2V, a computer program for computing analysis of variance for factorial designs was utilized. Before proceeding with an examination of the analysis of variance, it would be in order to define the following terms (17).

1. Main Effect. The main effect of a factor is a measure of the change in the response variable to changes in the level of the factor averaged over all levels of all the other factors.
2. Interaction. It is the differential response to one factor in combination with varying levels of a second factor applied

FIG.5. PLOT OF CELL MEANS VS. RANGES OF CBR VALUES WITHIN THE CELLS.

simultaneously. Interaction is an additional effect due to the combined influence of two or more factors.

3. Significance. This indicates that the effect (main effect or interaction) considered makes a major (non-zero) contribution to the variation in test results.

Table 7 summarizes the results of analysis of variance of the data presented in Table 5. Each main effect and interaction was tested for significance. This was accomplished by testing the ratio of the mean square for the effect under analysis and the experimental error mean square by means of the F-test. All effects were tested against experimental error. The tests were conducted at α -levels of 0.05 and 0.01. Table 7 shows the results of these tests. In this tabulation, significance is indicated by the α -level at which the test was significant. A test that did not show significance is indicated by the abbreviation 'NS'.

Level of significance indicates the probability of rejecting a hypothesis which is true. This would be best illustrated by an example. In the analysis of CBR data, consider the effect of aggregate type on the strength characteristics of soil-aggregate mixtures as measured by the CBR test. To do this it is necessary to first set up a hypothesis. The hypothesis in this case would be that the effect of aggregate type on the strength characteristics of soil-aggregate mixtures is not significant. This is known as the null hypothesis or the hypothesis of non-significance. Statistically, this would be shown as,

$$H_0 : A_i = 0 \quad (i = 1, 2) \quad \text{in equation 6.}$$

TABLE 7. SUMMARY OF ANALYSIS OF VARIANCE - CBR

SOURCE OF VARIANCE	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARE	F	F _{.05}	F _{.01}	SIGNIFICANCE
AGGREGATE TYPE (GRAVEL vs STONE), A	1	1518.85	1518.85	9.56	4.04	7.19	.01 LEVEL
PLASTICITY OF FINES (LOW vs HIGH), B	1	27481.13	27481.13	173.47	4.04	7.19	.01 LEVEL
PERCENT PASSING NO. 200 SIEVE, C	1	0.07	0.07	0.0004	4.04	7.19	NS
PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES, D	2	3363.84	1681.92	10.61	3.19	5.08	.01 LEVEL
COMPACTION (56 vs 28 BLOWS), F	1	23177.29	23177.29	146.30	4.04	7.19	.01 LEVEL
INTERACTIONS A x B	1	54.83	54.83	0.35	4.04	7.19	NS
A x C	1	293.48	293.48	1.85	4.04	7.19	NS
A x D	2	8380.77	4190.39	26.45	3.19	5.08	.01 LEVEL
A x F	1	0.49	0.49	0.003	4.04	7.19	NS
B x C	1	20188.35	20188.35	127.44	4.04	7.19	.01 LEVEL
B x D	2	2091.42	1045.71	6.60	3.19	5.05	.01 LEVEL
B x F	1	723.53	723.53	4.57	4.04	7.19	.05 LEVEL
C x D	2	17130.30	1865.15	54.07	3.19	5.08	.01 LEVEL
C x F	1	0.25	0.25	0.002	4.04	7.19	NS
D x F	2	856.27	428.14	2.70	3.19	5.08	NS
A x B x C	1	425.67	425.67	2.69	4.04	7.19	NS
A x B x D	2	7069.30	3534.65	22.31	3.19	5.08	.01 LEVEL
A x B x F	1	57.58	57.58	0.36	4.04	7.19	NS
A x C x D	2	1007.13	503.56	3.18	3.19	5.08	NS
A x C x F	1	10.90	10.90	0.07	4.04	7.19	NS
A x D x F	2	1173.99	587.00	3.71	3.19	5.08	.05 LEVEL
B x C x D	2	2143.53	1071.77	6.77	3.19	5.08	.01 LEVEL
B x C x F	1	2006.14	2006.14	12.66	4.04	7.19	.01 LEVEL
B x D x F	2	1648.71	824.36	5.20	3.19	5.08	.01 LEVEL
C x D x F	2	3215.67	1607.83	10.15	3.19	5.08	.01 LEVEL
A x B x C x D	2	1986.01	993.01	6.63	3.19	5.08	.01 LEVEL
A x B x C x F	1	0.49	0.49	0.003	4.04	7.19	NS
A x B x D x F	2	382.04	191.02	1.21	3.19	5.08	NS
A x C x D x F	2	780.67	390.33	2.46	3.19	5.08	NS
B x C x D x F	2	809.59	404.80	2.56	3.19	5.08	NS
A x B x C x D x F	2	806.87	403.43	2.55	3.19	5.08	NS
ERROR WITHIN REPLICATES, E	48	7604.10	158.42				
TOTAL	95	136389.24					

where

A_i = aggregate type.

The next step is to test this hypothesis. This is done by using the F-test as shown in Table 7. The F-test indicates that the hypothesis should be rejected at one percent significant level. If the hypothesis of non-significance were true it would be rejected, on the average, only one percent of the time. In other words, there is, on the average, one percent chance of rejecting the null hypothesis incorrectly. This test indicates, then, that there is strong evidence that aggregate type has a significant effect on the strength properties of soil-aggregate mixtures.

All other main effects and interactions were tested in a similar way. For each of these effects, a null hypothesis, similar to the one shown for the effect of aggregate type, was formulated. This approach holds true for all analyses of variance in this study. An examination of the results of the analysis of variance of CBR test data at one percent significant level reduces the model given by equation 6 to:

$$\begin{aligned}
 Y_{ijklmn} = & U + A_i + B_j + D_l + F_m + AD_{il} + BC_{jk} + BD_{jl} \\
 & + CD_{kl} + ABD_{ijl} + BCD_{jkl} + BCF_{jkm} + BDF_{jlm} \\
 & + CDF_{klm} + ABCD_{ijkl} + E_{n(ijklm)}
 \end{aligned} \quad (9)$$

A study of the main effects and interactions indicates the following:

1. The aggregate type had a significant effect on the strength characteristics of soil-aggregate mixtures as measured by the CBR test.

2. The CBR values were affected by the amount of compaction used in molding test specimens.
3. The effect of plasticity of fines on CBR was significant.
4. The amount of material retained between the No. 30 and No. 200 sieves had a significant effect on stability of soil-aggregate mixtures as measured by the CBR test. That is, the effect of skip grading on stability of soil aggregate mixtures was significant.
5. Though within the studied range the effect of percent material passing the No. 200 sieve on CBR values was not significant, the effect of the interaction of this factor with the percent material retained between the No. 30 and No. 200 sieves (CD_{kl}), was significant. This indicates that there is a definite interplay between stability characteristics and the dust ratio. In this study, dust ratio is defined as:

$$\text{Dust Ratio} = \frac{\text{Percent Material Passing No. 200 sieve}}{\text{Percent Material Passing No. 30 sieve.}}$$

6. The interactions of percent material retained between the No. 30 and No. 200 sieves with each of the factors, aggregate type and plasticity of fines, (AD_{il} , BD_{jl}), tested significant.
7. The interaction of plasticity of fines and the amount of fines (BC_{jk}) was significant in its effect on CBR values.
8. The three and higher factor interactions, shown to be significant, were mostly due to a relatively small error

mean square. This is not unusual when a large number of factors are used in a factorial analysis. The significance of such interactions is relatively difficult to analyze. However, the presence of such high order interactions dictates that the data be more closely studied in order to further clarify and substantiate the significance of various effects and interactions.

In order to evaluate the effect of varying the amount of material retained between the No. 30 and No. 200 sieves on stability of soil-aggregate mixtures, the following model was formulated:

$$Y_{ln} = U + D_1 + E_{n(1)} \quad (10)$$

where

Y_{ln} = CBR value obtained from a given test,

U = true mean value for the population,

D_1 = true differential effect of percent retained between the No. 30 and No. 200 sieves, with $l = 1, 2, 3$,

$E_{n(1)}$ = true error effect of replicates with $n = 1, 2$.

To use this model, the CBR data was divided into 16 groups, each group being a unique combination of one distinct level of each of the other four factors, i.e., aggregate type, plasticity of fines, percent passing No. 200 sieve, and compaction. Table 8 gives a summary of test data, one-way analysis of variance, and the result of the F-test at five percent significant level for each of these 16 data groups. An examination of the one-way analyses of variance indicates that the effect of varying the amount of material retained

TABLE 8. SUMMARY OF ONE-WAY ANALYSIS OF VARIANCE - CBR.

GRAVEL
 LOW PI FINES
 5% PASSING NO. 200 SIEVE
 56 BLOW COMPACTION

PERCENT RETAINED BETWEEN NO.30 AND NO. 200 SIEVES	C B R (PERCENT)
0	60.0 46.7
10	88.8 96.7
20	98.9 95.0

ONE-WAY ANOVA

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARE	F	F _{.05}	SIGNIFICANCE
PERCENT BETWEEN NO. 30 - NO. 200	2	2313.9733	1156.967	27.2756	9.55	SIGNIFICANT
ERROR	3	127.2550	42.4183			
TOTAL	5	2441.2283	1199.3853			

GRAVEL
 LOW PI FINES
 5% PASSING NO. 200 SIEVE
 28 BLOW COMPACTION

PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES	C B R (PERCENT)
0	45.3 34.2
10	38.3 49.3
20	73.3 74.3

ONE-WAY ANOVA

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARE	F	F _{.05}	SIGNIFICANCE
PERCENT BETWEEN NO. 30 - NO. 200	2	1426.9033	713.4517	17.2463	9.55	SIGNIFICANT
ERROR	3	124.1050	41.3683			
TOTAL	5	1551.0083	754.8200			

TABLE 8 , CONT.

GRAVEL

LOW PI FINES

10% PASSING NO. 200 SIEVE

56 BLOW COMPACTION

PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES	C B R (PERCENT)
0	184.9 182.6
10	130.0 153.0
20	61.4 53.3

ONE-WAY ANOVA

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARE	F	F ₀₅	SIGNIFICANCE
PERCENT BETWEEN NO. 30 - NO. 200	2	16562.1633	8281.0817	82.8246	9.55	SIGNIFICANT
ERROR	3	299.9500	99.9833			
TOTAL	5	16862.1133	8381.0650			

GRAVEL

LOW PI FINES

10% PASSING NO. 200 SIEVE

28 BLOW COMPACTION

PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES	C B R (PERCENT)
0	76.7 77.8
10	102.7 99.3
20	49.3 70.7

ONE-WAY ANOVA

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARE	F	F ₀₅	SIGNIFICANCE
PERCENT BETWEEN NO. 30 - NO. 200	2	1695.0833	847.5417	10.8029	9.55	SIGNIFICANT
ERROR	3	235.3650	78.4550			
TOTAL	5	1930.4483	925.9967			

TABLE 8 , CONT.

GRAVEL

HIGH PI FINES

5% PASSING NO. 200 SIEVE

56 BLOW COMPACTION

PERCENT RETAINED BETWEEN NO.30 AND NO. 200 SIEVES	C B R (PERCENT)	
0	97.0	52.0
10	77.3	80.0
20	113.3	85.3

ONE-WAY ANOVA

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARE	F	F _{.05}	SIGNIFICANCE
PERCENT BETWEEN NO. 30-NO. 200	2	705.7900	352.8950	.7518	9.55	N.S.
ERROR	3	1408.1450	469.3817			
TOTAL	5	2113.9350	822.2767			

GRAVEL

HIGH PI FINES

5% PASSING NO. 200 SIEVE

28 BLOW COMPACTION

PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES	C B R (PERCENT)	
0	46.7	31.3
10	57.25	45.0
20	44.0	83.3

ONE-WAY ANOVA

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARE	F	F _{.05}	SIGNIFICANCE
PERCENT BETWEEN NO. 30 - NO. 200	2	607.6758	303.8379	.9437	9.55	N.S.
ERROR	3	965.8563	321.9521			
TOTAL	5	1573.5321	625.7900			

TABLE 8 , CONT.

GRAVEL
HIGH PI FINES
10% PASSING NO.200 SIEVE
56 BLOW COMPACTION

PERCENT RETAINED BETWEEN NO.30 AND NO. 200 SIEVES	C B R (PERCENT)
0	88.0 60.0
10	30.0 26.7
20	25.3 44.0

ONE-WAY ANOVA

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARE	F	F _{.05}	SIGNIFICANCE
PERCENT BETWEEN NO. 30 - NO. 200	2	2448.0233	1224.0117	6.4167	9.55	N.S.
ERROR	3	572.2900	190.7633			
TOTAL	5	3020.3133	1414.7750			

GRAVEL
HIGH PI FINES
10% PASSING NO.200 SIEVE
28 BLOW COMPACTION

PERCENT RETAINED BETWEEN NO.30 AND NO. 200 SIEVES	C B R (PERCENT)
0	43.3 32.6
10	32.0 23.3
20	22.0 28.0

ONE-WAY ANOVA

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARE	F	F _{.05}	SIGNIFICANCE
PERCENT BETWEEN NO. 30 - NO. 200	2	187.2100	93.6050	2.4831	9.55	N.S.
ERROR	3	113.0900	37.6967			
TOTAL	5	300.3000	131.3017			

TABLE 8 , CONT.

CRUSHED STONE

LOW PI FINES

5 % PASSING NO. 200 SIEVE

56 BLOW COMPACTION

PERCENT RETAINED BETWEEN NO.30 AND NO. 200 SIEVES	C B R (PERCENT)	
0	52.0	51.5
10	110.7	81.4
20	143.3	145.0

ONE-WAY ANOVA

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARE	F	F _{.05}	SIGNIFICANCE
PERCENT BETWEEN NO. 30 - NO. 200	2	8542.5733	4271.2867	29.7433	9.55	SIGNIFICANT
ERROR	3	430.8150	143.6050			
TOTAL	5	8973.3883	4414.8917			

CRUSHED STONE

LOW PI FINES

5 % PASSING NO. 200 SIEVE

28 BLOW COMPACTION

PERCENT RETAINED BETWEEN NO.30 AND NO. 200 SIEVES	C B R (PERCENT)	
0	46.7	38.0
10	43.3	60.0
20	128.0	108.0

ONE-WAY ANOVA

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARE	F	F _{.05}	SIGNIFICANCE
PERCENT BETWEEN NO. 30 - NO. 200	2	6807.8233	3403.9117	27.0660	9.55	SIGNIFICANT
ERROR	3	377.2900	125.7633			
TOTAL	5	7185.1133	3529.6750			

TABLE 8 , CONT.

CRUSHED STONE
 LOW PI FINES
 10% PASSING NO. 200 SIEVE
 56 BLOW COMPACTION

PERCENT RETAINED BETWEEN NO.30 AND NO. 200 SIEVES	C B R (PERCENT)
0	101.4 100.0
10	128.0 144.7
20	123.3 163.3

ONE-WAY ANOVA

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARE	F	F ₀₅	SIGNIFICANCE
PERCENT BETWEEN NO. 30 - NO. 200	2	2089.3233	1044.6617	3.3325	9.55	N.S.
ERROR	3	940.4250	313.4750			
TOTAL	5	3029.7483	1358.1367			

CRUSHED STONE
 LOW PI FINES
 10% PASSING NO. 200 SIEVE
 28 BLOW COMPACTION

PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES	C B R (PERCENT)
0	46.7 64.7
10	98.7 70.0
20	125.3 96.7

ONE-WAY ANOVA

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARE	F	F ₀₅	SIGNIFICANCE
PERCENT BETWEEN NO. 30 - NO. 200	2	3059.4233	1529.7117	4.6693	9.55	N.S.
ERROR	3	982.8250	327.6083			
TOTAL	5	4042.2483	1857.3200			

TABLE 8 ,CONT.

CRUSHED STONE
HIGH PI FINES
5% PASSING NO. 200 SIEVE
56 BLOW COMPACTION

PERCENT RETAINED BETWEEN NO.30 AND NO. 200 SIEVES .	CBR (PERCENT)	
0	800	51.6
10	93.9	86.7
20	118.7	120.0

ONE-WAY ANOVA

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARE	F	F _{.05}	SIGNIFICANCE
PERCENT BETWEEN NO. 30 - NO. 200	2	2874.5033	1437.2517	10.0263	9.55	SIGNIFICANT
ERROR	3	430.0450	143.3483			
TOTAL	5	3304.5483	286.6000			

CRUSHED STONE
HIGH PI FINES
5 % PASSING NO. 200 SIEVE
28 BLOW COMPACTION

PERCENT RETAINED BETWEEN NO.30 AND NO. 200 SIEVES	CBR (PERCENT)	
0	37.3	24.7
10	80.0	73.7
20	53.3	61.3

ONE-WAY ANOVA

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARE	F	F _{.05}	SIGNIFICANCE
PERCENT BETWEEN NO. 30 - NO. 200	2	2117.4100	1058.7050	24.2036	9.55	SIGNIFICANT
ERROR	3	131.2250	43.7417			
TOTAL	5	2248.6350	1102.4467			

TABLE 8 ,CONT.

CRUSHED STONE

HIGH PI FINES

10 % PASSING NO.200 SIEVE

56 BLOW COMPACTION

PERCENT RETAINED BETWEEN NO.30 AND NO. 200 SIEVES	C B R (PERCENT)	
0	68.6	75.3
10	55.0	49.3
20	50.0	24.0

ONE-WAY ANOVA

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARE	F	F ₀₅	SIGNIFICANCE
PERCENT BETWEEN NO. 30-NO 200	2	1228.7100	614.3550	4.8928	9.55	N.S.
ERROR	3	376.6900	125.5633			
TOTAL	5	1605.4000	739.9183			

CRUSHED STONE

HIGH PI FINES

10 % PASSING NO. 200 SIEVE

28 BLOW COMPACTION

PERCENT RETAINED BETWEEN NO.30 AND NO. 200 SIEVES	C B R (PERCENT)	
0	49.3	38.3
10	31.3	38.7
20	30.0	31.3

ONE-WAY ANOVA

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARE	F	F ₀₅	SIGNIFICANCE
PERCENT BETWEEN NO. 30 - NO. 200	2	179.5233	89.7617	3.0351	9.55	N.S.
ERROR	3	88.7250	29.5750			
TOTAL	5	268.2483	119.3367			

between the No. 30 and No. 200 sieves was significant at the five percent level for the following conditions:

1. gravel, low PI fines, five percent passing No. 200 sieve, 56-blow compaction;
2. gravel, low PI fines, five percent passing No. 200 sieve, 28-blow compaction;
3. gravel, low PI fines, ten percent passing No. 200 sieve, 56-blow compaction;
4. gravel, low PI fines, ten percent passing No. 200 sieve, 28-blow compaction;
5. crushed stone, low PI fines, five percent passing No. 200 sieve, 56-blow compaction;
6. crushed stone, low PI fines, five percent passing No. 200 sieve, 28-blow compaction;
7. crushed stone, high PI fines, five percent passing No. 200 sieve, 56-blow compaction;
8. crushed stone, high PI fines, five percent passing No. 200 sieve, 28-blow compaction.

This analysis shows that the percent of material retained between the No. 30 and No. 200 sieves had a significant effect on strength characteristics of all soil-gravel mixes with low PI fines irrespective of the compaction level or the percent material passing the No. 200 sieve. In case of soil-crushed stone mixes, the effect of percent material retained between the No. 30 and No. 200 sieves was significant only in case of mixes with five percent material passing the No. 200 sieve, irrespective of the plasticity of fines or the compaction level.

Texas Triaxial Test Data

The analysis of variance models assumed for analyzing Texas triaxial test data are:

$$Y'_{ikl} = U + A_i + C_k + D_l + AC_{ik} + AD_{il} + CD_{kl} + E_{(ikl)} \quad (11)$$

$$Y'_{jkl} = U + B'_j + C_k + D_l + B'C_{jk} + B'D_{jl} + CD_{kl} + E_{(jkl)} \quad (12)$$

$$Y''_{jkl} = U + A_i + C_k + D_l + AC_{ik} + AD_{il} + CD_{kl} + E_{(ikl)} \quad (13)$$

$$Y''_{jkl} = U + B'_j + C_k + D_l + B'C_{jk} + B'D_{jl} + CD_{kl} + E_{(jkl)} \quad (14)$$

where

Y'_{ikl}, Y'_{jkl} = triaxial compressive strength value obtained from a Texas triaxial test,

Y''_{ikl}, Y''_{jkl} = value of angle of internal friction obtained from a Texas triaxial test,

U = true mean value for the population,

A_i = true differential effect of aggregate type (gravel vs. crushed stone, both with low PI fines),

B'_j = true differential effect of plasticity of fines (low PI vs. NP in crushed stone mixes),

C_k = true differential effect of percent passing No. 200 sieve (five percent vs. ten percent),

D_l = true differential effect of percent retained between No. 30 and No. 200 sieves,

$E_{(ikl)}, E_{(jkl)}$ = true error effect.

The other terms denote interactions between the main effects. The subscripts assume the values:

$$i = 1, 2$$

$$j = 1, 2$$

$$k = 1, 2$$

$$l = 1, 2, 3.$$

As equations 11, 12, 13, and 14 denote non-replicated fixed-effect models, the error and the three-factor interaction (ACD_{ikl}) or (BCD_{jkl}) are confounded, i.e., separate estimates of sum of squares error, E, and sum of squares ACD or BCD cannot be obtained.

The assumptions that underlie these models are homogeneity of variance, normality, additivity, and independence of errors. For unreplicated models, it is difficult to test the validity of these assumptions. However, analysis of variance is a fairly robust statistical method and is relatively insensitive to violations of the assumptions of normality and homogeneity of variance.

Table 9 shows the Texas triaxial test data layout for analyses of variance. The models assumed for these analyses are given by equations 11 and 13. Table 10 summarizes the results of analysis of variance of the triaxial compressive strength data. Equation 11 gives the assumed analysis of variance model for this case. The results of the F-tests indicated that only the effect of aggregate type on triaxial compressive strength was significant at the five percent level. Hence, equation 11 reduces to:

$$Y'_{ikl} = U + A_i + E_{(ikl)} \quad (15)$$

TABLE 9. SUMMARY OF TEXAS TRIAXIAL TEST DATA.

TYPE OF SOIL - AGGREGATE MIX	PERCENT PASSING NO. 200 SIEVE	PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES	ANGLE OF INTERNAL FRICTION ϕ (DEGREES)	TRIAXIAL COMPRESSIVE STRENGTH AT 15 PSI LATERAL PRESSURE (PSI)
CRUSHED STONE WITH LOW PI FINES	5	0	55°	170.8
		10	52°	179.4
		20	60°	248.0
	10	0	55°	179.3
		10	54°	199.4
		20	54°	210.4
GRAVEL WITH LOW PI FINES	5	0	51°	131.0
		10	52°	143.2
		20	47°	115.4
	10	0	49°	132.2
		10	43°	45.5
		20	37°	66.6

TABLE IO. SUMMARY OF ANALYSIS OF VARIANCE - TRIAXIAL COMPRESSIVE STRENGTH AT 15 PSI LATERAL PRESSURE.

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARE	F	F _{.05}	F _{.10}	SIGNI-FICANCE
AGGREGATE TYPE (STONE vs GRAVEL), A	1	216.7500	216.7500	38.8209	18.5	8.53	F _{.05}
PERCENT PASSING NO. 200 SIEVE, C	1	56.33333	56.33333	100.896	18.5	8.53	F _{.10}
PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES, D	2	16.16667	8.08333	14.478	19.0	9.92	N.S.
INTERACTIONS A x C	1	24.08333	24.08333	43.134	18.5	8.53	N.S.
A x D	2	63.5000	31.7500	56.866	19.0	9.92	N.S.
C x D	2	22.1667	11.08333	19.851	19.0	9.92	N.S.
ERROR, E	2	11.1667	5.58333				
TOTAL	11	410.16667					

TABLE II. SUMMARY OF ANALYSIS OF VARIANCE - ANGLE OF INTERNAL FRICTION.

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARE	F	F _{.05}	F _{.10}	SIGNI-FICANCE
AGGREGATE TYPE (STONE vs GRAVEL), A	1	22680.90750	22680.90750	103.7169	18.5	8.53	F _{.05}
PERCENT PASSING NO. 200 SIEVE, C	1	626.40750	626.40750	28.645	18.5	8.53	N.S.
PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES, D	2	68.32167	34.16083	15.62	19.0	9.92	N.S.
INTERACTIONS A x C	1	910.02083	910.02083	41.614	18.5	8.53	N.S.
A x D	2	4097.37500	2048.68750	93.684	19.0	9.92	N.S.
C x D	2	162.76550	81.382750	3.7215	19.0	9.92	N.S.
ERROR, E	2	437.36167	218.68083				
TOTAL	11	30448.04917					

All main effects and interactions not significant at the 25 percent level were pooled. Using $F_{(2,2;.25)}=3.00$ and $F_{(1,2;.25)}=2.57$ and comparing with computed F-values as shown in Table 10, these effects were comprised of the main effect D_1 and the interaction CD_{k1} . This resulted in an error term E'^* with an $MSE' = 8.25$ and d.f.= 6. Now:

$$\text{Main effect A: d.f.}=1; F_A = \frac{M S A}{M S E} = \frac{216.75}{8.25} = 26.27,$$

$$\text{Main effect C: d.f.}=1; F_C = \frac{M S C}{M S E} = \frac{56.33}{8.25} = 6.82,$$

$$\text{Interaction A x C: d.f.}= 1; F_{AC} = \frac{M S (AD)}{M S E} = \frac{24.08}{8.25} = 2.92,$$

$$\text{Interaction A x D: d.f.}= 2; F_{AD} = \frac{M S (AD)}{M S E} = \frac{31.75}{8.25} = 3.85.$$

Comparing these computed F-values with the statistics $F_{(1,6;.05)}=5.99$ and $F_{(2,6;.05)}=5.14$, only the main effects A and C are significant at the five percent significant level. That is, only aggregate type and percent passing the No. 200 sieve had any significant effect on triaxial compressive strength. Therefore, the model given by equation 15 is modified to:

$$Y'_{ikl} = U + A_i + C_k + E_{(ikl)} \quad (16)$$

Table 11 gives a summary of the analysis of variance of the angle of internal friction data. Equation 13 gives the assumed model of

$$\begin{aligned} * \text{SSE}' &= \text{SSD} + \text{SS}(\text{CD}) + \text{SSE} & \text{and} & \quad \text{MSE}' = \frac{49.5001}{6} = 8.25 \\ &= 16.16667 + 22.1667 + 11.1667 \\ &= 49.5001 \end{aligned}$$

analysis of variance for this case. The results of the F-test at the five percent significant level indicate that only the effect of aggregate type tested significant. This reduces equation 13 to:

$$Y''_{ikl} = U + A_i + E_{(ikl)} \quad (17)$$

Also, on testing at the 25 percent significant level, the effect D_1 did not show significance. On pooling this effect with the error term E, the pooled error term E' had an $MSE' = 126.42$ with d.f. = 4. Using the new error term, it was determined that besides the effect of aggregate type, the effect of interaction of aggregate type and the percent retained between the No. 30 and No. 200 sieves was also significant at the five percent level. Hence, equation 17 is replaced by:

$$Y''_{ikl} = U + A_i + AD_{kl} + E_{(ikl)} \quad (18)$$

This would be the model that would govern the variation in angle of internal friction data.

Table 12 shows the Texas triaxial test data layout for analyses of variance governed by the assumed models given in equations 12 and 14. Table 13 summarizes the results of analysis of variance of the triaxial compressive strength data. Equation 12 gives the assumed analysis of variance model for this case. The results of the analysis indicate that the effect of the interaction of percent material passing the No. 200 sieve and the percent material retained between the No. 30 and No. 200 sieves was significant at the ten percent level. The effect $B'C_{jk}$, not significant at the 25 percent level and the error term E, were pooled and the remaining effects were tested against the new error term E' for significance. The pooled error

TABLE 12. SUMMARY OF TEXAS TRIAXIAL TEST DATA.

TYPE OF SOIL - AGGREGATE MIX	PERCENT PASSING NO. 200 SIEVE	PERCENT RETAINED BETWEEN NO. 30 AND NO 200 SIEVES	ANGLE OF INTERNAL FRICTION ϕ (DEGREES)	TRIAxIAL COMPRESSIVE STRENGTH AT 15 PSI LATERAL PRESSURE (PSI)
CRUSHED STONE WITH NP FINES	5	0	42°	129.3
		10	55°	175.4
		20	50°	203.8
	10	0	55°	201.8
		10	53°	219.4
		20	50°	161.3
CRUSHED STONE WITH LOW PI FINES	5	0	55°	170.8
		10	52°	179.4
		20	60°	248.0
	10	0	55°	179.3
		10	54°	199.4
		20	54°	210.4

TABLE 13. SUMMARY OF ANALYSIS OF VARIANCE - TRIAXIAL COMPRESSIVE STRENGTH AT 15 PSI LATERAL PRESSURE.

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARE	F	F ₀₅	F ₁₀	SIGNIFICANCE
PLASTICITY OF FINES, B'	1	1089.27907	1089.27907	6.6891	18.5	8.53	N.S.
PERCENT PASSING NO. 200 SIEVE, C	1	573.11541	573.11541	3.5194	18.5	8.53	N.S.
PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES, D	2	1952.594471	976.29723	5.9953	19.0	9.92	N.S.
INTERACTIONS B' x C	1	353.49308	353.49308	2.1708	18.5	8.53	N.S.
B' x D	2	1495.45580	747.72790	4.5917	19.0	9.92	N.S.
C x D	2	4506.29470	2253.14703	13.8363	19.0	9.92	F ₁₀
ERROR, E	2	325.68540	162.84270				
TOTAL	11	10295.91729					

TABLE 14. SUMMARY OF ANALYSIS OF VARIANCE - ANGLE OF INTERNAL FRICTION.

SOURCE OF VARIATION	DEGREES OF FREEDOM	SUM OF SQUARES	MEAN SQUARE	F	F ₀₅	F ₁₀	SIGNIFICANCE
PLASTICITY OF FINES, B'	1	50.02083	50.02083	2.5956	18.5	8.53	N.S.
PERCENT PASSING NO. 200 SIEVE, C	1	3.52083	3.52083	1.827	18.5	8.53	N.S.
PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES, D	2	9.37500	4.68750	2.432	19.0	9.92	N.S.
INTERACTIONS B' x C	1	20.02083	20.02083	1.0389	18.5	8.53	N.S.
B' x D	2	39.04167	19.52083	1.0129	19.0	9.92	N.S.
C x D	2	44.54167	22.27083	1.1557	19.0	9.92	N.S.
ERROR, E	2	38.54167	19.27083				
TOTAL	11	205.06250					

term E' had $MSE' = 226.39$ and $d.f. = 3$. On testing at a five percent significant level and using the pooled error term, only the interaction between the percent passing the No. 200 sieve and the percent retained between the No. 30 and No. 200 sieves tested significant.

Hence, equation 12 reduces to:

$$Y'_{jkl} = U + CD_{kl} + E_{(jkl)} \quad (19)$$

Table 14 gives a summary of the analysis of variance of the angle of internal friction data. The governing model for this analysis is given by equation 14. On examining the F-test results, no effect or interaction was significant. Even on pooling the effects not significant at the 25 percent level and using the pooled error term as an estimate of the true error effect, the result still remained the same, i.e., no main effect or interaction was significant.

Therefore, equation 14 becomes:

$$Y''_{jkl} = U + E_{jkl} \quad (20)$$

Before deducing any conclusions, one aspect of the above analysis of variance must be kept in mind. A pure error term was not considered. It was confounded by the three-factor interaction. This makes the analysis less robust and to a degree less reliable. No analysis is better than the data on which it is based. Owing to lack of replicates, the results of the above analyses might have been based on a false premise. Hence, the conclusions derived from this analyses should be viewed with caution.

The analysis of Texas triaxial test data may be summed up as:

1. In case of soil-gravel vs. soil-crushed stone mixes, only the aggregate type and the percent fines had a significant effect on triaxial compressive strength. However, the variation in angle of internal friction was influenced not only by the aggregate type but also by the interaction of the aggregate type with the percent retained between No. 30 and No. 200 sieves.
2. In case of soil-crushed stone mixes with NP fines vs. soil-crushed stone mixes with low PI fines, the variation in triaxial compressive strength was explained by the interaction of percent fines with percent retained between the No. 30 and No. 200 sieves. This shows that dust ratio had an important influence on triaxial compressive strength. The variation in angle of internal friction data was mainly a result of experimental error.

INTERPRETATION OF RESULTS

The deduction of results required the study of the effect of the fraction retained between the No. 30 and No. 200 sieves on CBR, triaxial compressive strength, compactive density, plasticity, and permeability characteristics of the soil-aggregate test mixtures. The foregoing statistical analysis provided the guidelines for evaluating the results.

CBR Tests

For this study, a soil-aggregate test mixture testing at a CBR of 80 percent or above was considered to be a good base material relative to strength characteristics. Figures 6 and 7 show the variation of CBR with percent retained between the No. 30 and No. 200 sieves. In these illustrations, graphs were fitted only to the data of the mixtures that tested significant in the one-way analysis of variance.* The curves were plotted using a regression analysis as a means of fitting the curves. A summary of the results of the regression analysis is given in Table B-1. The discussion that follows applies to the CBR results obtained using the 56-blow compaction as required in the standard test procedure.

* The effect of percent retained between No. 30 and No. 200 sieves makes a significant contribution to the variation of CBR test results at the .05 level of significance.

NUMBERS IN PARENTHESES ARE
PI VALUES

FIG.6. VARIATION OF CALIFORNIA BEARING RATIO WITH PERCENT
RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES.
(CBR VALUES ARE THE AVERAGE OF TWO TESTS)

FIG. 7. VARIATION OF CALIFORNIA BEARING RATIO WITH PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES.
(CBR VALUES ARE THE AVERAGE OF TWO TESTS)

Figure 6 applies to mixes with five percent fines (material passing the No. 200 sieve). As the fraction retained between the No. 30 and No. 200 sieves was decreased, the gravel and crushed stone mixes with low plasticity fines suffered a decrease in CBR. This was also true for the crushed stone mixes with high plasticity fines. The variation in CBR of the gravel mixes with high plasticity fines was not influenced by the amount of material between the No. 30 and No. 200 sieves. For the completely skip-graded mixes, i.e., without any material retained between the No. 30 and No. 200 sieves, the CBR values were appreciably lower than those for well-graded mixes. This instability resulted from a relative lack of soil matrix in these mixes.* The physical state of these mixes corresponds to condition (a) in Figure 1.

The variation of CBR with percent retained between the No. 30 and No. 200 sieves, for mixes with ten percent fines, is shown in Figure 7. The variation in the strength of crushed stone mixes, irrespective of the type of fines, was not dependent on the amount of material retained between the No. 30 and No. 200 sieves. This was also true for the gravel mixes with high plasticity fines. However, this fraction had an important influence on the strength of gravel mixes with low plasticity fines. It can be observed that there was a substantial increase in the CBR of these mixes as the fraction between the No. 30 and No. 200 mesh sieves was

* In this study, soil is defined as the material passing the No. 30 sieve.

decreased. For the extreme case, where all material between the No. 30 and No. 200 sieves was removed, an average CBR of 184 percent was recorded.

Using a CBR of 80 percent as the strength criterion for an adequate base, the data indicate that:

1. For gravel mixes with five percent fines, irrespective of the plasticity of fines, the percent retained between the No. 30 and No. 200 sieves should be about ten percent or greater.
2. For crushed stone mixes with five percent fines, irrespective of the plasticity of fines, the percent retained between the No. 30 and No. 200 sieves should remain above six percent.
3. With only five percent fines in the mix, complete skip-grading of the fraction between the No. 30 and No. 200 mesh sieves was harmful to the CBR characteristics of both gravel and crushed stone mixes.
4. For gravel mixes with ten percent low plasticity fines, the percent retained between the No. 30 and No. 200 sieves should remain below 17 percent.
5. All crushed stone mixes with ten percent low plasticity fines showed adequate CBR irrespective of the amount of material between the No. 30 and No. 200 sieves.
6. The crushed stone and gravel mixes with ten percent high plasticity fines did not meet the minimum strength criterion

for adequate CBR, irrespective of the amount of material retained between the No. 30 and No. 200 sieves.

7. With ten percent fines, skip-grading of this fraction was not harmful to the CBR of either the gravel mixes or the crushed stone mixes with low PI fines. In fact, an increase in CBR resulted for the gravel mixes. Gravel and crushed stone mixes with high PI fines showed strength values below the stipulated criterion for all cases under investigation.

The above brings to light a very important characteristic. It appears that there is a strong interplay between the percent fines and the soil fraction between the No. 30 and No. 200 sieves in their effect on CBR characteristics of soil-aggregate mixes. This substantiates a similar deduction obtained from the statistical analysis.

To investigate this characteristic further, the variation of CBR with dust-ratio was studied. A dust ratio of 1.00 signifies a completely skip-graded mix. Examination of Tables 15 and 16 shows that in case of crushed stone and gravel mixes with five percent fines, an increase in the dust-ratio from 0.20 to 1.00 caused a decrease in CBR values. Gravel mixes, with ten percent low plasticity fines, showed a substantial increase in CBR with increase in dust ratio from 0.30 to 1.00. In essence, this was also true for crushed stone and gravel mixes with high plasticity fines. The trend did not hold for crushed stone mixes with low plasticity fines.

Using an 80 percent CBR to signify an adequate base material, Figures 6 and 7 suggest that:

TABLE 15. VARIATION OF CALIFORNIA BEARING RATIO WITH DUST RATIO FOR SOIL - GRAVEL MIXTURES (CBR VALUES ARE THE AVERAGE OF TWO TESTS).

5% PASSING NO. 200 SIEVE

PLASTICITY OF FINES	DUST $\frac{\% \text{PASSING NO. 200 SIEVE}}{\% \text{PASSING NO. 30 SIEVE}}$	CBR (PERCENT)	
		56 BLOW COMPACTION	28 BLOW COMPACTION
LOW PI FINES	0.20	97.0	74.3
	0.33	92.8	43.8
	1.00	53.4	39.8
HIGH PI FINES	0.20	99.3	63.7
	0.33	78.7	51.1
	1.00	74.5	39.0

10% PASSING NO. 200 SIEVE

PLASTICITY OF FINES	DUST $\frac{\% \text{PASSING NO. 200 SIEVE}}{\% \text{PASSING NO. 30 SIEVE}}$	CBR (PERCENT)	
		56 BLOW COMPACTION	28 BLOW COMPACTION
LOW PI FINES	0.30	57.4	60.0
	0.50	141.5	101.0
	1.00	183.8	77.3
HIGH PI FINES	0.30	34.7	25.0
	0.50	28.4	27.7
	1.00	74.0	38.0

TABLE 16. VARIATION OF CALIFORNIA BEARING RATIO WITH DUST RATIO FOR SOIL - CRUSHED STONE MIXTURES (CBR VALUES ARE THE AVERAGE OF TWO TESTS).

5 % PASSING NO. 200 SIEVE

PLASTICITY OF FINES	DUST - RATIO	% PASSING NO. 200 SIEVE % PASSING NO. 30 SIEVE	CBR (PERCENT)	
			56 BLOW COMPACTION	28 BLOW COMPACTION
LOW PI FINES		0.20	144.2	118.0
		0.33	96.1	51.7
		1.00	51.8	42.4
HIGH PI FINES		0.20	119.4	57.3
		0.33	90.3	76.9
		1.00	65.8	31.0

10 % PASSING NO. 200 SIEVE

PLASTICITY OF FINES	DUST - RATIO	% PASSING NO. 200 SIEVE % PASSING NO. 30 SIEVE	CBR (PERCENT)	
			56 BLOW COMPACTION	28 BLOW COMPACTION
LOW PI FINES		0.30	143.3	111.0
		0.50	136.4	84.4
		1.00	100.7	55.7
HIGH PI FINES		0.30	37.0	30.7
		0.50	52.2	35.0
		1.00	72.0	43.8

1. For a gravel mix with five percent low plasticity fines, the dust ratio should not be greater than $\frac{5}{5 + 10} = 0.33$.
2. For crushed stone mixes with five percent fines and irrespective of the plasticity of fines, dust ratio should not be greater than $\frac{5}{5 + 6} = 0.45$. This is also true for gravel mixes with high plasticity fines.
3. For gravel mixes with ten percent low PI fines, dust ratio should not be less than $\frac{10}{10 + 17} = 0.37$.
4. Dust ratio had no influence on CBR characteristics of crushed stone mixes with ten percent low plasticity fines.
5. Gravel and crushed stone mixes with ten percent high plasticity fines were not affected by dust ratio as the CBR of these mixes were below the governing criterion for an adequate base.

The effect of variable compaction on CBR test results is shown in Figures 6 and 7. There was no significant effect other than lower CBR values for 28-blow compaction as compared to mixes subjected to 56-blow compaction. With two exceptions, the trends regarding the effect of skip-grading on CBR remained basically the same as demonstrated by mixes subjected to 56-blow compaction. Gravel mixes with ten percent low plasticity fines showed higher CBR values when the percent retained between the No. 30 and No. 200 sieves was constrained to about eight percent. Crushed stone mixes with five percent high plasticity fines demonstrated a similar characteristic. This means that these soil-aggregate mixes, when subjected to a relatively lower compaction indicated an optimum

value for the percent material retained between the No. 30 and No. 200 sieves. A decrease below or increase above this value resulted in lower CBR values. This deviation from established trends can be explained by the fact that a decrease in compactive effort does not result in an equal decrease in compactive densities for all types of soil-aggregate mixes. A study of Table A-2 would indicate that a 50 percent decrease in compactive effort (28-blow compaction vs. 56-blow compaction) caused a decrease in compactive density anywhere from 93 percent to 99 percent for the soil-aggregate mixes under study.

Texas Triaxial Tests

The criterion used in evaluating test results required that an adequate base course material exhibit a triaxial compressive strength of 175 psi at a confining pressure of 15 psi. It was further required that the angle of internal friction, as measured from Mohr's envelope, should not be less than 50° .

The variation of triaxial compressive strength and angle of internal friction with the fraction retained between the No. 30 and No. 200 sieves is shown in Tables 17 and 18. The following results apply to mixes with five percent fines. (Refer to Tables 17 and A-3.)

1. Gravel mixes with low plasticity fines indicated the highest compressive strength and internal friction when the percent retained between the No. 30 and No. 200 sieves was confined to approximately ten percent and the dust ratio was about 0.33.

TABLE 17. VARIATION OF TRIAXIAL COMPRESSIVE STRENGTH ($\sigma_3 = 15$ PSI) AND ANGLE OF INTERNAL FRICTION WITH PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES (5% PASSING NO. 200 SIEVE).

GRAVEL WITH LOW PI FINES

PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES	DUST RATIO	PI	ANGLE OF INTERNAL FRICTION ϕ (DEGREES)	TRIAxIAL COMPRESSIVE STRENGTH AT 15 PSI LATERAL PRESSURE (PSI)
0	1.0	91	51	131.0
10	0.33	NP	52	143.2
20	0.20	NP	47	115.4

CRUSHED STONE WITH LOW PI FINES

PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES	DUST RATIO	PI	ANGLE OF INTERNAL FRICTION ϕ (DEGREES)	TRIAxIAL COMPRESSIVE STRENGTH AT 15 PSI LATERAL PRESSURE (PSI)
0	1.0	59	55	170.8
10	0.33	NP	52	179.4
20	0.20	NP	60	248.0

CRUSHED STONE WITH NP FINES

PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES	DUST RATIO	PI	ANGLE OF INTERNAL FRICTION ϕ (DEGREES)	TRIAxIAL COMPRESSIVE STRENGTH AT 15 PSI LATERAL PRESSURE (PSI)
0	1.0	NP	42	124.3
10	0.33	NP	55	175.7
20	0.20	NP	50	203.8

TABLE 18. VARIATION OF TRIAXIAL COMPRESSIVE STRENGTH ($\sigma_3 = 15$ PSI) AND ANGLE OF INTERNAL FRICTION WITH PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES (10 % PASSING NO. 200 SIEVE).

GRAVEL WITH LOW P I FINES

PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES	DUST RATIO	P I	ANGLE OF INTERNAL FRICTION ϕ (DEGREES)	TRIAxIAL COMPRESSIVE STRENGTH AT 15 PSI LATERAL PRESSURE (PSI)
0	1.00	9.1	49	132.2
10	0.50	3.0	43	95.5
20	0.30	NP	37	66.6

CRUSHED STONE WITH LOW P I FINES

PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES	DUST RATIO	P I	ANGLE OF INTERNAL FRICTION ϕ (DEGREES)	TRIAxIAL COMPRESSIVE STRENGTH AT 15 PSI LATERAL PRESSURE (PSI)
0	1.00	5.9	55	197.3
10	0.50	2.9	54	199.4
20	0.30	NP	54	210.9

CRUSHED STONE WITH NP FINES

PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES	DUST RATIO	P I	ANGLE OF INTERNAL FRICTION ϕ (DEGREES)	TRIAxIAL COMPRESSIVE STRENGTH AT 15 PSI LATERAL PRESSURE (PSI)
0	1.00	NP	55	201.8
10	0.50	NP	53	219.4
20	0.30	NP	50	161.3

2. Crushed stone mixes with both non-plastic and low PI fines showed an increase in triaxial compressive strength and angle of internal friction with an increase in percent retained between the No. 30 and No. 200 sieves. For these mixes, dust ratio should remain below 0.33.
3. Crushed stone mixes tested at a higher Texas triaxial class than gravel mixes.
4. Crushed stone mixes with low plasticity fines showed strength characteristics comparable to crushed stone mixes with non-plastic fines.

The results indicated below apply to mixes with ten percent fines. (Refer to Tables 18 and A-3.)

1. In case of gravel mixes with low plasticity fines, triaxial compressive strength and angle of internal friction decreased with increase in the fraction retained between the No. 30 and No. 200 sieves. For the extreme case, with a dust ratio of 1.0, highest values of compressive strength and internal friction were obtained.
2. The variation in triaxial compressive strength and the angle of internal friction was not influenced by the amount of material between the No. 30 and No. 200 sieves, in case of crushed stone mixes with low PI fines. The change in stability parameters was very small with variation in the fraction between the No. 30 and No. 200 sieves.
3. Crushed stone mixes with non-plastic fines indicated that better strength results were obtained as the material

between the No. 30 and No. 200 sieves was decreased.

4. Crushed stone mixes tested at a higher triaxial class than gravel mixes.
5. There was no distinct differences in the strength properties of crushed stone mixes with low plasticity fines as opposed to the ones with non-plastic fines.

In general, the triaxial compressive strength results showed the same trends as CBR test results. The high CBR values indicated for soil-gravel mixes were not corroborated by Texas triaxial test results. Invariably, the soil-gravel mixes tested at a lower Texas triaxial class as compared with soil-crushed stone mixes with similar gradation characteristics.

The critical values of dust ratio for various soil-aggregate mixes, as determined by the CBR test results, compared well with the values obtained from an analysis of Texas triaxial test results. Crushed stone mixes with low plasticity fines gave approximately the same strength values as crushed stone mixes with non-plastic fines.

Permeability Tests

Typical permeability values for soils (14) are given in Table 19. The permeability test results are graphically illustrated in Figure 8. An analysis of these results shows that: (a) permeability decreases with increased amounts of material between the No. 30 and No. 200 sieves; (b) mixes with five percent fines had higher permeability than mixes with ten percent fines; (c) the effect of material retained between the No. 30 and No. 200 sieves on permeability was

TABLE 19. TYPICAL PERMEABILITY VALUES FOR SOILS.

DEGREE OF PERMEABILITY	k IN CM/SEC
HIGH	OVER 10^{-1}
MEDIUM	10^{-1} to 10^{-3}
LOW	10^{-3} to 10^{-5}
VERY LOW	10^{-5} to 10^{-7}
IMPERMEABLE	LESS THAN 10^{-7}

FIG. 8. VARIATION OF PERMEABILITY WITH PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES.

more pronounced in case of mixes with ten percent fines as opposed to mixes with five percent fines; (d) where mixes were devoid of material between the No. 30 and No. 200 sieves, gravel mixes showed higher permeability than crushed stone mixes. However, in mixes which were more uniformly graded, crushed stone mixes were more permeable than gravel mixes. The above suggests that skip-grading can improve the permeability characteristics of both crushed stone and gravel mixes. This effect would be of greatest benefit to soil-gravel mixes which suffer from a substantial loss in permeability with increase in the amount of material between the No. 30 and No. 200 sieves; (e) using a k-value of 10^{-3} cm/sec as the lower limit of coefficient of permeability would ensure good drainage characteristics in base courses. It is seen from Figure 8 that for mixes with ten percent fines, the percent retained between the No. 30 and No. 200 sieves should not exceed four percent if gravel aggregates are used and it should not be over 2.5 percent if crushed stone aggregates are used. For mixes with five percent fines, the percent retained between the No. 30 and No. 200 sieves should not exceed 12 percent in soil-gravel mixes and it should not be over nine percent in case of crushed stone mixes. The above deductions are only valid when low plasticity fines are used as a binder.

The dust ratios that would provide adequate permeability characteristics are:

1. Gravel, five percent passing No. 200 sieve; dust ratio

$$> \frac{5}{12 + 5} = 0.29;$$

2. Gravel, ten percent passing No. 200 sieve, dust ratio

$$> \frac{10}{4 + 10} = 0.71;$$
3. Crushed stone, five percent passing No. 200 sieve, dust ratio

$$> \frac{5}{9 + 5} = 0.36;$$
4. Crushed stone, ten percent passing No. 200 sieve, dust ratio

$$> \frac{10}{2.5 + 10} = 0.8.$$

A Study of the Interaction of Plasticity and Skip-Grading on Strength Characteristics

Indiana specifications for soil-aggregate mixes require that liquid limit and plasticity index should not exceed 25 percent and five percent respectively.

This analysis indicates that the effect of plasticity on strength depends on the amount of fines (minus No. 200 fraction), the dust ratio, and the plasticity of minus No. 200 fraction. The effect of these factors is outlined as follows:

1. Figure 6 indicates that CBR increases with a decrease in PI when the fines in the soil-aggregate mix are confined to five percent.
2. Figure 7 shows that the effect of PI on CBR becomes less important as the amount of material between the No. 30 and No. 200 sieves is decreased. This deduction assumes that there are ten percent fines in the soil-aggregate mix.
3. Figures 6 and 7 reveal that irrespective of the percent fines in the mix, soil-aggregate mixtures with low

plasticity binders have higher strength values as opposed to mixes with high plasticity binders.

4. The effect of PI on strength is influenced by the interaction of the percent fines in a soil-aggregate mix and the material retained between the No. 30 and No. 200 sieves.

Referring to Figure 7 it is seen that a soil-gravel mix with a PI of 9.1 percent and an LL of 29.8 percent gave three times higher CBR values as the soil-gravel mix with an LL of 15.2 percent and no PI. The PI of minus 200 fraction was 9.1 percent in both cases. This was due to the fact that the soil-aggregate mix with a PI of 9.1 percent was skip-graded between the No. 30 and No. 200 sieves while the second mix had 20 percent material in this range. The effect of adding 20 percent sandy material was to make the second mix non-plastic. In the case of the first mix, even though the plasticity characteristics of the mix made it a substandard material, yet with skipping the material between the No. 30 and No. 200 sieves, it showed superior strength and drainage characteristics as compared to the material with plasticity parameters within the specifications.

In case of crushed stone and gravel mixes with ten percent high plasticity fines, the same phenomenon is observed. Skip-graded mixes, though having higher PI values (46.8), showed higher CBR values compared to well-graded mixes which had considerably lower PI values (12.8 and 11.9 percent for crushed stone and gravel mixes respectively).

Figure 6 indicates a soil-gravel mix with five percent high plasticity fines that tested non-plastic owing to the presence of 20 percent material between the No. 30 and No. 200 sieves. According to specifications this would be considered a standard material. However, it gave lower strength values as compared to a substandard soil-gravel mix with a PI of 9.1 percent as shown in Figure 7. The latter mix had ten percent low PI fines which gave it the better strength characteristics.

Figure 6 shows two crushed stone mixes with 20 percent material retained between the No. 30 and No. 200 sieves. Both tested non-plastic. However, one had high plasticity fines (PI = 46.8 percent, LL = 77.4 percent), while the other had low plasticity fines (PI = 5.9 percent, LL = 26.4 percent). According to specifications, both mixes satisfied the plasticity requirements. On testing for CBR, the mix with low plasticity fines gave 17 percent higher CBR as compared to the mix with high plasticity fines.

These examples demonstrate the effect of percent fines coupled with the plasticity of minus No. 200 sieve fraction on strength. A complete dependence on plasticity values (PI values of minus No. 40 fraction) may give misleading results. Hence, there is need for developing a criterion for designing soil-aggregate specifications that take into consideration the interplay of percent fines in the mix, the dust ratio, and the plasticity characteristics of the mix. It is suggested that plasticity becomes less important if the soil matrix does not fill the voids in the aggregate.

CONCLUSIONS

General

In this study, the stability characteristics of soil-aggregate mixtures were investigated. The factors considered to have an influence on stability were the type of aggregate, the amount of fines in the mix and their plasticity, the fraction retained between the No. 30 and No. 200 sieves, and variation in compaction. The parameters used as a measure of stability were the California Bearing Ratio (CBR), the Texas triaxial compressive strength obtained at a confining pressure of 15 psi, the angle of the internal friction measured from the Mohr's envelope, and permeability. The primary objective of this study was to determine the effect of skip-grading the material between the No. 30 and No. 200 sieves on the stability of soil-gravel and soil-crushed stone mixes. The conclusions that follow apply to the type of materials and test methods used in this study.

1. In general, the crushed stone mixtures resulted in higher test values than the gravel mixtures. The soil-gravel mixes indicated a Texas triaxial class between 3.4 and 4.0, whereas the soil-crushed stone mixtures showed a range of 1.0 to 2.6.
2. The soil-aggregate mixes with low plasticity fines tested at higher strength values, compared to mixes with high plasticity fines.

3. As expected, soil-aggregate mixes compacted at 50 percent of the standard compactive effort tested at a lower CBR than mixes compacted at 100 percent standard compactive effort (56-blow compaction). The densities for the lower compactive effort varied from 93 percent to 99 percent of the densities obtained with the 56-blow compaction. The percentage decrease in density observed by a given decrease in compactive effort was a function of the grading of the mix. In case of skip-graded and lean mixes, the amount of breakage affected the compactability of the mix, whereas in the richer and better graded mixes the compaction characteristics were controlled by the amount of fines in the mix.
4. Crushed stone mixes with non-plastic fines tested at approximately the same compressive strengths as mixes with low plasticity fines.
5. The effect of plasticity on strength characteristics of a soil-aggregate mix depended to a great degree on the amount of fines in the mix. When the fines in the mixes were restricted to a minimal amount, compatible with the stability of the mix, then the effect of plasticity became less significant. Similarly, with adequate fines and skip-grading of the material between the No. 30 and No. 200 sieves, the soil-gravel mixes showed adequate strength characteristics though the plasticity characteristics were

above the prescribed upper limits of plasticity index and liquid limit.

6. Crushed stone mixtures, in general, showed higher permeabilities than gravel mixtures. However, in the case of skip-graded mixes, devoid of material between the No. 30 and No. 200 sieves, the gravel mixes showed higher permeability than the crushed stone mixes.

Effect of Skip-Grading Between the No. 30 and No. 200 Sieves

Table 20 summarizes the results relating to the effect of skip-grading on stability and permeability of the soil-aggregate mixtures. This table shows the limiting values of dust ratio and percent passing the No. 30 mesh sieve for certain criteria given in the first column of the table. The values apply to the tested mixtures with low plasticity fines. A permeability of 10^{-3} cm/sec was considered to be the borderline between medium and low permeabilities. It was further assumed that soil-aggregate mixes exhibiting a permeability of less than 10^{-3} cm/sec would have poor drainage characteristics. A CBR of 80 percent was considered to be a measure of adequate strength characteristics. Triaxial criteria used stated that the material must have a minimum angle of internal friction of 50° and also a minimum strength of 175 psi at 15 psi lateral confining pressure. This is compatible with Texas Grade 1 base material.

TABLE 20. LIMITING VALUES OF DUST RATIO AND PERCENT PASSING THE NO. 30 MESH SIEVE FOR THE TESTED SOIL-AGGREGATE MIXES WITH LOW PI FINES AND COMPACTED TO STANDARD AASHO DENSITY.

LIMITING CRITERIA	GRAVEL				CRUSHED STONE			
	5 PERCENT PASSING NO. 200 SIEVE		10 PERCENT PASSING NO. 200 SIEVE		5 PERCENT PASSING NO. 200 SIEVE		10 PERCENT PASSING NO. 200 SIEVE	
	PERCENT PASSING NO. 30 SIEVE	DUST RATIO	PERCENT PASSING NO. 30 SIEVE	DUST RATIO	PERCENT PASSING NO. 30 SIEVE	DUST RATIO	PERCENT PASSING NO. 30 SIEVE	DUST RATIO
MINIMUM CBR = 80%	>15	< 0.33	< 27	> 0.37	>11	< 0.45	≥10	UP TO 1.00
MINIMUM PERMEABILITY k = 10 ⁻³ CM/SEC	<17	>0.29	<15	>0.67	<15	>0.33	<12	>0.83
MINIMUM ANGLE OF INTERNAL FRICTION = 50°; AND MINIMUM STRENGTH=175 PSI AT 15 PSI LATERAL PRESSURE	N.S.	—	N.S.	—	>15	< 0.33	≥10	UP TO 1.00

* N.S. INDICATES MATERIAL IS NOT SATISFACTORY ACCORDING TO ADOPTED LIMITING CRITERIA.

On the basis of tests reported herein the following statements can be made:

1. The fraction retained between the No. 30 and No. 200 sieves had a significant influence on both strength and permeability characteristics of soil-aggregate mixtures. However, the variation of strength and permeability parameters with the fraction retained between the No. 30 and No. 200 sieves was a function of the percent fines in the mix.
2. Skip-grading of the fraction between the No. 30 and No. 200 sieves was harmful in case of soil-aggregate mixes with only five percent fines in the mix.
3. In case of soil-gravel mixes with ten percent fines, skip-grading of the fraction between the No. 30 and No. 200 sieves improved the strength and permeability characteristics considerably. The crushed stone mixes with ten percent passing the No. 200 sieve showed excellent strengths irrespective of the amount of material retained between the No. 30 and No. 200 sieves.
4. In order to ensure adequate strength, (a) gravel mixes with five percent fines should have more than 15 percent material passing the No. 30 sieve (a dust ratio of less than 0.33); (b) gravel mixes with ten percent fines should have less than 27 percent material passing the No. 30 sieve (dust ratio greater than 0.37); (c) crushed stone mixes with five percent fines should have more than 11 percent passing

the No. 30 sieve (dust ratio less than 0.45); (d) crushed stone mixes with ten percent fines may have as little as ten percent passing the No. 30 sieve (dust ratio up to 1.00).

5. In order to ensure adequate drainage characteristics,
 - (a) gravel mixes with five percent fines should have less than 17 percent material passing the No. 30 sieve (dust ratio greater than 0.29);
 - (b) gravel mixes with ten percent fines should have less than 15 percent material passing the No. 30 sieve (dust ratio greater than 0.67);
 - (c) crushed stone mixes with five percent fines should have less than 15 percent material passing the No. 30 sieve (dust ratio greater than 0.33);
 - (d) crushed stone mixes with ten percent fines may have as little as ten percent material passing the No. 30 sieve (dust ratio up to 1.00).

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APPENDIX A

TEST DATA

TABLE A-1. COMPACTION TEST DATA FOR CBR TESTS.

TYPE OF AGGREGATE	PLASTICITY OF FINES	PERCENT PASSING NO. 200 SIEVE	PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES	MAXIMUM DRY DENSITY, γ_d (LB/FT ³)	OPTIMUM MOISTURE CONTENT, w_p (PERCENT)
GRAVEL	LOW	5	0	132.5	7.0
			10	136.0	5.0
			20	140.0	6.9
		10	0	140.0	5.8
			10	141.0	6.6
			20	139.5	7.6
	HIGH	5	0	133.0	8.8
			10	138.7	7.3
			20	137.5	7.8
		10	0	132.0	8.3
			10	133.7	8.1
			20	135.3	7.6
CRUSHED STONE	LOW	5	0	129.7	7.7
			10	134.3	5.6
			20	143.5	7.3
		10	0	138.0	7.5
			10	145.0	6.8
			20	147.5	6.5
	HIGH	5	0	130.0	7.5
			10	139.0	8.7
			20	147.5	6.5
		10	0	133.0	8.7
			10	135.0	9.0
			20	137.5	7.9

TABLE A-2. CALIFORNIA BEARING RATIO AND CONSISTENCY LIMITS TEST DATA.

TYPE OF AGGREGATE		GRAVEL										CRUSHED STONE											
PLASTICITY OF FINES		LOW					HIGH					LOW					HIGH						
PERCENT PASSING NO. 200 SIEVE		5		10			5		10			5		10			5		10				
PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES		0	10	20	0	10	20	0	10	20	0	10	20	0	10	20	0	10	20	0	10	20	
56 BLOW COMPACTION	CBR (PERCENT)	560	888	989	184.9	1300	614	97C	77.3	113.3	88.0	300	253	52.0	1107	143.3	101.4	128.0	123.3	800	939	1187	
		46.7	96.7	95.0	82.6	153.0	53.3	52.0	80C	85.3	60C	26.7	44.0	51.5	81.4	145.0	100C	144.7	163.3	51.6	86.7	120.0	
	DRY DENSITY γ_d (LB/FT ³)	1277	1333	136.7	1380	1387	140.1	1272	1331	136.2	131.6	133.7	133.4	127.4	136.2	140.2	135.4	143.9	146.4	127.1	135.6	138.4	
		1279	133.3	136.3	135.7	142.0	141.8	128.5	135.4	134.8	128.5	143.9	135.2	125.7	134.2	141.7	136.1	144.8	145.3	125.0	135.3	140.2	
	MOLDING MOISTURE CONTENT w (PERCENT)	65	54	68	59	56	67	56	70	75	80	87	83	62	60	63	51	56	60	59	65	62	84
56 BLOW COMPACTION	AVERAGE CBR (PERCENT)	57	51	65	58	60	67	71	72	79	93	94	75	72	57	65	59	58	57	53	70	63	85
		534	92.8	97.0	183.8	141.5	57.4	74.5	78.7	99.3	74.0	28.4	34.6	51.8	96.1	144.2	1007	136.4	143.3	658	90.3	119.4	
	AVERAGE DRY DENSITY γ_d (LB/FT ³)	1278	133.3	136.5	136.8	140.4	140.9	127.8	134.3	135.5	130C	138.8	134.5	126.6	135.1	141.0	135.8	144.3	145.8	126.1	135.6	139.3	
	AVERAGE MOLDING MOISTURE CONTENT w (PERCENT)	61	53	67	59	58	67	64	71	77	87	90	79	67	59	64	55	57	59	56	68	63	85
	AVERAGE SWELL (PERCENT)	-0.04	0.03	0.04	0.00	0.01	-0.06	-0.04	0.06	0.0	0.07	-0.18	0.12	-0.01	-0.05	-0.12	0.00	0.04	0.06	0.0	-0.03	0.05	-0.02
28 BLOW COMPACTION	CBR (PERCENT)	453	38.3	733	767	102.7	49.3	46.7	57.3	44.0	43.3	32.0	22.0	46.7	43.3	128.0	46.7	98.7	125.3	37.3	80C	533	
		342	49.3	753	778	99.3	70.7	31.3	45.0	83.3	32.6	23.3	280	380	60C	108.0	64.7	700	96.7	24.7	73.7	61.3	
	DRY DENSITY γ_d (LB/FT ³)	1256	1305	1362	1323	1397	1387	1251	1326	133.8	125.6	130.4	133.7	121.6	128.0	140.2	130.6	140.8	143.3	120.7	134.2	131.9	
		1278	130C	136.3	134.4	137.9	1382	123.8	128.2	135.1	127.6	129.5	131.5	122.1	125.3	140.9	132.1	139.1	138.2	119.2	132.8	133.6	
	MOLDING MOISTURE CONTENT w (PERCENT)	73	45	63	59	60	74	82	79	66	88	71	85	63	57	70	72	64	61	75	75	63	81
28 BLOW COMPACTION	AVERAGE CBR (PERCENT)	61	49	61	58	61	65	77	77	63	85	79	81	68	50	70	55	61	61	77	82	67	86
		398	438	743	773	101.0	60C	39.0	51.1	63.7	37.8	27.7	25.0	42.4	51.6	118.0	55.7	84.4	111.0	31.0	76.9	57.3	
	AVERAGE DRY DENSITY γ_d (LB/FT ³)	126.9	1301	136.3	133.4	138.8	138.5	124.4	130.4	134.4	126.6	123.0	132.6	121.9	126.6	140.6	131.4	140.0	140.8	1200	133.5	132.8	
	AVERAGE MOLDING MOISTURE CONTENT w (PERCENT)	67	49	62	59	61	70	78	78	65	86	75	83	66	53	70	64	63	61	76	79	65	84
	AVERAGE SWELL (PERCENT)	-0.02	-0.02	0.12	0.00	-0.05	-0.02	0.03	0.01	-0.06	0.01	0.08	0.04	-0.01	-0.03	-0.06	-0.04	-0.04	-0.04	-0.06	0.00	0.03	0.07
LIQUID LIMIT		29.8	15.2	12.7	29.8	20.2	15.2	77.4	28C	20.2	77.4	40.8	28.0	26.4	14.3	—	26.4	16.2	14.3	77.4	29.5	—	77.4
PLASTIC LIMIT		20.7	NP	NP	20.7	17.2	NP	30.6	16.1	NP	30.6	20.0	16.1	20.5	NP	NP	20.5	13.3	NP	30.6	16.7	NP	
PLASTICITY INDEX		9.1	NP	NP	NP	9.1	NP	46.8	11.5	NP	46.8	20.8	11.9	5.9	NP	NP	5.9	2.9	NP	46.8	12.8	NP	

TABLE A-3. COMPACTION TEST DATA FOR TEXAS TRIAXIAL TESTS.

TYPE OF AGGREGATE	PERCENT PASSING NO. 200 SIEVE	PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES	MAXIMUM DRY DENSITY γ_d , (LB/FT ³)	OPTIMUM MOISTURE CONTENT w_p (PERCENT)
GRAVEL WITH LOW PI FINES	5	0	134.5	5.9
		10	143.0	6.0
		20	141.5	7.0
	10	0	142.0	6.5
		10	143.0	6.5
		20	142.0	6.0
CRUSHED STONE WITH LOW PI FINES	5	0	130.8	6.3
		10	141.8	6.3
		20	143.8	6.5
	10	0	139.2	6.3
		10	147.2	6.2
		20	148.8	5.2
CRUSHED STONE WITH NP FINES	5	0	132.4	6.5
		10	144.0	6.5
		20	144.0	7.0
	10	0	141.8	6.0
		10	144.6	6.0
		20	141.0	6.0

TABLE A-4: TEXAS TRIAXIAL AND CONSISTENCY LIMITS TEST DATA.

TYPE OF AGGREGATE	GRAVEL WITH LOW PI FINES						CRUSHED STONE WITH LOW PI FINES						CRUSHED STONE WITH NP FINES					
	5			10			5			10			5			10		
	0	10	20	0	10	20	0	10	20	0	10	20	0	10	20	0	10	20
PERCENT PASSING NO. 200 SIEVE																		
PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES																		
ANGLE OF INTERNAL FRICTION ϕ (DEGREES)	51	52	47	48	43	37	55	52	60	54	54	54	42	55	50	55	53	50
TRIAXIAL COMPRESSIVE STRENGTH AT 15 PSI LATERAL PRESSURE (PSI)	131.0	143.2	115.4	132.2	95.5	66.6	170.8	179.4	248.0	197.3	199.4	210.4	129.3	175.4	203.8	201.8	219.4	161.3
TEXAS TRIAXIAL CLASS	3.8	3.7	3.4	3.5	3.7	4.0	2.6	2.1	2.2	2.5	2.2	2.2	2.6	3.2	1.0	2.6	1.0	2.6
LIQUID LIMIT	298	152	12.7	29.8	20.2	152	264	14.3	—	26.4	16.2	14.3	—	—	—	—	—	—
PLASTIC LIMIT	20.7	NP	NP	20.7	17.2	NP	20.5	NP	NP	20.5	13.3	NP	NP	NP	NP	NP	NP	NP
PLASTICITY INDEX	9.1	NP	NP	9.1	3.0	NP	5.9	NP	NP	5.9	2.9	NP	NP	NP	NP	NP	NP	NP

TABLE A-5. PERMEABILITY TEST DATA.

TYPE OF AGGREGATE	PERCENT PASSING NO. 200 SIEVE	PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES	COEFFICIENT OF PERMEABILITY, k * AT 15°C (CM/SEC X 10 ⁻⁴)
GRAVEL WITH LOW PI FINES	5	0	23.72
		10	11.76
		20	1.97
	10	0	18.01
		10	3.95
		20	0.78
CRUSHED STONE WITH LOW PI FINES	5	0	15.82
		10	9.76
		20	4.86
	10	0	14.48
		10	2.28
			1.75
* MEAN OF THREE READINGS			

APPENDIX B

EQUATIONS OF BEST FIT FOR CBR TEST DATA

TABLE B-1. EQUATIONS OF BEST FIT FOR CBR TEST DATA.

EQUATION OF BEST FIT TO OBSERVED DATA								
TYPE OF AGGREGATE	PLASTICITY OF FINES	PERCENT PASSING NO. 200 SIEVE	56 BLOW COMPACTION			28 BLOW COMPACTION		
			MODEL	R ²	STANDARD ERROR	MODEL	R ²	STANDARD ERROR
GRAVEL	LOW	5	Y = 59.22 + 2.18 X	.77	11.62	Y = 35.34 + 1.73 X	.77	9.45
		10	Y = 190.73 - 6.3 X	.94	14.88	Y = 77.25 + 561X - 0.32X ²	.88	8.86
	HIGH	5	*	—	—	*	—	—
		10	*	—	—	*	—	—
CRUSHED STONE	LOW	5	Y = 51.12 + 4.62 X	.98	10.44	Y = 32.84 + 3.78 X	.80	19.12
		10	*	—	—	*	—	—
	HIGH	5	Y = 65.04 + 2.68 X	.86	10.45	Y = 31.0 + 7.86 X - 0.33X ²	.95	11.21
		10	*	—	—	*	—	—
Y = CBR (PERCENT) X = PERCENT RETAINED BETWEEN NO. 30 AND NO. 200 SIEVES * REGRESSION NOT SIGNIFICANT AT α = 0.05 LEVEL								