Influence of Aggregate Shape on Base Behavior

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A re-examination and simplification of the original rut index concept for predicting rut susceptibility in aggregate bases is presented to eliminate some of the disadvantages of the original approach. The rut index can be determined from the results of a single cyclic load triaxial test performed at a confining stress of 6 psi, rather than at two confining stresses as originally proposed. The principal stress ratio to be used in the test varies from 2 to 6, depending on the structural strength of the pavement section. The resilient and permanent deformation characteristics of river gravel, granitic gneiss, shale, limestone, and quartzite aggregates were determined using the cyclic load triaxial test. Variables investigated included density, gradation, moisture content, and aggregate shape and surface characteristics. The revised rut index concept was used to evaluate and compare the relative permanent deformation behavior of these various unbound aggregates. The cubic-shaped, smooth rounded river gravel was found to be more than two times as susceptible to rutting as the crushed aggregates tested. The crushed aggregates were angular, blade, and disc shaped and had relatively rough surfaces. These aggregates generally performed similarly with respect to permanent deformation, although the visual appearance of the two blade-shaped aggregates was not as nice as the others. The use of a simple, slow triaxial shear test as a practical alternative to the conventional dynamic test was studied for evaluating the resilient and permanent characteristics of unbound base materials. The slow triaxial shear test was found to be suitable for evaluating the resilient modulus, but appeared not to be appropriate for evaluating permanent deformation characteristics.

The cyclic load triaxial test offers, at this time, probably the best available method for laboratory evaluation of the resilient and permanent deformation characteristics of unbound aggregate bases. Many material variables affect granular base performance, including aggregate shape, surface characteristics, gradation, density, and moisture conditions. Also, the state of stress to which the granular material is subjected has an important influence on performance. May and Witczak (*1*) and Witczak and Rada (*2*) have given excellent reviews of resilient properties. The measurement of permanent deformation characteristics of unbound aggregate bases has, however, received much less attention, although some notable contributions have been made (*3–7*).

The effect on performance of aggregate characteristics (such as shape, surface roughness, angularity, and roundness) has for the most part been neglected. The effect of aggregate characteristics on the resilient and permanent deformation behavior is investigated for five aggregates from different geologic sources. Other variables in the study include gradation, plasticity of fines, and degree of saturation. The possibility of using a static triaxial test as an expedient alternative to dynamic testing is considered. A re-examination and simplification is also made to the rut index concept.

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REEVALUATION OF RUT INDEX CONCEPT

The rut index concept was proposed by Barksdale (*5*) in 1972 for comparing the relative permanent deformation behavior of different unbound aggregates. Since then, many advances have been made in evaluating permanent deformation characteristics, and it is now appropriate to re-evaluate the rut index concept.

The rut index was proposed for comparing the relative permanent deformation behavior of aggregate bases placed within similar pavement structures. The basis for the rut index is that the permanent deformation developed in an aggregate base (or in other layers or sublayers) is proportional to the average permanent strain in the layer (or sublayer) times the thickness of the stratum (*5*). The rut index concept, as originally proposed, involves determining the average permanent strain in the top and bottom halves of the base using a cyclic load triaxial test. The two cyclic load tests are performed at different specified stress states that, as originally proposed, do not vary with the structural strength of the pavement. These stress states simulate the conditions in the top and bottom halves of the base. The original rut index was defined as the sum of the average strain from the two cyclic tests multiplied by 10,000. For a given structural section geometry, the rut index is approximately proportional to the permanent deformation that should occur in the base.

The rut index approach has proven quite useful for comparing the potential relative performance of different aggregate bases (*5*). The two primary disadvantages of the rut index concept are (a) two cyclic load tests are required to evaluate a material and (b) the appropriate stress states to use in testing are not constant, but actually vary with the strength of the structural section. A slightly modified approach is proposed which, for the most part, eliminates these two disadvantages. Also, this study takes advantage of the advances made since 1972 in analyzing the stress state in an aggregate base.

To predict the appropriate stress states to use for different pavements, the GAPPS7 nonlinear finite element computer program was used (*8–10*). The different pavement sections analyzed using a nonlinear, simplified contour model for the aggregate base are summarized in Table 1. The contour model probably offers the best available method for modeling unbound granular materials (*11*). Structural pavement sections studied included asphalt concrete surface thicknesses varying from 2 to 8 in. (50 to 200 mm) and unbound aggregate base thick-

TABLE 1 PAVEMENT SECTIONS USED TO DETERMINE STRESS STATES FOR LABORATORY TESTING

PAVEMENT SECTION	THICKNESS OF A.C. SURFACE (IN.)	THICKNESS OF BASE (IN.)	Teq (IN.)	SUBGRADE CONDITIONS	REMARKS
1	2	6	5	GOOD	LIGHT
2	2	6	5	POOR	LIGHT
3	2	10	7	GOOD	LIGHT
4	2	10	7	POOR	LIGHT
5	4	10	9	GOOD	MEDIUM
6	4	10	9	POOR	MEDIUM
7	8	6	11	GOOD	HEAVY
8	8	6	11	POOR	HEAVY
9	4	20	14	GOOD	HEAVY
10	4	20	14	POOR	HEAVY
11	6	18	15	GOOD	HEAVY
12	6	18	15	POOR	HEAVY

NOTES:

1. ASPHALT : Mr = 400,000 PSI, ν = 0.2
2. AGGREGATE BASE : SIMPLIFIED CONTOUR MODEL : K1 = 9400, G1 = 5300, γ = 0.14 (Ref. 27)
3. SUBGRADE-PIECEWISE LINEAR VARIATION OF Mr
   (1) POOR SUBGRADE : Mr = 16 KSI, σ_d = 0; Mr = 5 KSI, σ_d = 6 PSI; Mr = 5 KSI, σ_d = 25 PSI
   (2) GOOD SUBGRADE : Mr = 30 KSI, σ_d = 0; Mr = 15 KSI, σ_d = 5 PSI; Mr = 15.5 KSI, σ_d = 25 PSI

nesses varying from 6 to 20 in. (150 to 500 mm). Both poor and good subgrades were considered. The loading consisted of an 8,000 lb (35.6 kN) single-wheel load, having a uniform pressure of 120 psi (8.3 MN/m²) over a circular area. This loading approximates a dual-wheel loading.

Residual lateral stresses are developed in an aggregate base as a result of the compaction of the aggregate due to the application of large vertical stress from vibratory rollers or other compaction equipment. The results of a limited number of studies suggest that such residual lateral compaction stresses are important in defining the complete stress state that should be used in laboratory testing to simulate field conditions. In developing appropriate stress states for laboratory testing, lateral compaction stresses of 2 and 4 psi (14 and 28 kN/m²) were used based on work performed by Stewart et al. (*4*) and Uzan (*12*). Certainly a better definition of the actual residual stresses developed by compaction is needed, including field measurements.

In the finite element analysis, the unbound aggregate base was divided into three horizontal sublayers, each consisting of five horizontal elements. Figure 1 shows the average stress state obtained at the center of each of the three sublayers for light, medium, and heavy pavements, as defined in Table 2.

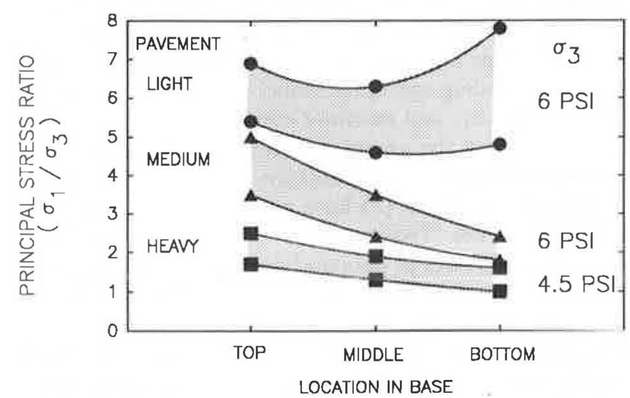

FIGURE 1 Average stress states for use in cyclic testing for light, medium, and heavy pavements.

The variation of confining pressures is from about 4 to 8 psi (28 to 55 kN/m²) and principal stress ratios σ_1/σ_3 from about 1.5 to 7.0. In general, the thinner pavements have larger confining pressures and greater principal stress ratios.

The results shown in Figure 1 indicate the use of a single stress state to characterize the layer should be sufficiently

TABLE 2 SUGGESTED STRESS STATES FOR LABORATORY TESTING TO DETERMINE RUT INDEX

STRUCTURAL STRENGTH	EQUIVALENT FULL DEPTH THICKNES (Teq) (INCHES)	STRESS STATES	
		σ_3 (psi)	σ_1/σ_3
LIGHT	Teq < 8	6	6
MEDIUM	8 < Teq < 11	6	4
HEAVY	Teq > 11	4.5 (1)	2

NOTE : 1. AN ALTERNATE WOULD BE TO USE A CONFINING PRESSURE OF 6 PSI FOR ALL PAVEMENT STRENGTHS.

accurate for most comparison purposes and reduces the required testing in half. This is particularly true when considering the current uncertainty concerning the magnitude of lateral compaction stresses, which can significantly influence the overall confining pressure.

Revised Rut Index

The relative behavior of different unbound base aggregates following the revised rut index approach is evaluated at only one stress state using the cyclic load test. Suggested stress states for light, medium, and heavy pavement sections are given in Table 2. The rut index is numerically equal to the measured permanent strain times 10,000, as originally proposed, so as to give an easy number to work with. Considerable experience indicates that 50,000 load repetitions is sufficient in performing the test; even fewer repetitions could be used.

Also, to account for preconditioning effects that occur during construction, the permanent deformation developed during the first 10 load applications should probably not be included in calculating the rut index. The effect of varying aggregate base thicknesses is not directly considered by the rut index concept. As an approximation, bases having different thicknesses can be compared by adjusting the rut index—assuming rutting would, for relatively small variations in base thickness, change proportionally to the base thickness.

The structural strength of the section for practical testing purposes can be defined as having an equivalent full-depth asphalt thickness using the classification system given in Table 2. This classification system requires converting the pavement to an equivalent full-depth asphalt concrete section. The equivalent thickness of full-depth asphalt T_{eq} can, as a simple approximation, be estimated by letting 1 in. (25 mm) of high-quality asphalt concrete replace 2 in. (50 mm) of high-quality crushed stone base compacted to 100 percent of AASHTO T-180 density.

Other methods for estimating base equivalencies can, of course, be used. For lower quality aggregate bases, the replacement ratio would typically vary from 2 in. (50 mm) to about 3 in. (75 mm) of aggregate to replace 1 in. (25 mm) of high-quality asphalt concrete. Pavements with thin asphalt surfacings having thick aggregate bases, such as pavement Sections 9 and 10 in Table 1, should be considered as lighter than indicated by the proposed equivalency method. This results because somewhat higher stresses are actually present than indicated by the equivalent thickness approach.

AGGREGATE SHAPE AND SURFACE CHARACTERISTICS

Particle Shape

The shape of the aggregate influences the gradation curve obtained by sieving (*13,14*). Many years ago, Rittenhouse (*14*) determined that flaky particles tended to pass sieves having square holes diagonally. For material retained on a given sieve size, Lees (*13*) showed that rod-shaped particles were about 2.5 times the size of disc-shaped particles retained on the same sieve. Rod-shaped particles could, for example, effectively be a complete size coarser in grading than disc-shaped particles retained on the same sieve size. These differences in size would affect both specific surface area and also the ability of the particles to fill voids of coarser-size aggregate properly.

The shape of aggregate particles can be divided into four general categories as discussed by Lees (*13*), Rosslein (*15*), and Zingg (*16*): cubic (equidimensional), disc, blade, and rod-shaped. The particle shape classification, as defined above, can be determined following the procedure given by British Standard BS 812:75 or the Army Corps of Engineers test method CRD-C119-53. These approaches, however, only separate the aggregate into four, rather broad categories of particle shape shown in Figure 2. Both methods employ simple-to-use gauges that can quickly separate aggregates into a shape classification.

Lees (*13*) has pointed out that the four broad categories defined by the British and Corps of Engineers standard tests permit quite a large range of particle shape characteristics within each classification. For research purposes, it is probably better to define particle shape more completely by determining the flatness and elongation ratios (*13,17*). The flatness ratio (p) is the shortest length, divided by the intermediate length. The elongation ratio (q) is the intermediate length, divided by the greatest length. By determining the actual flatness and elongation ratios, a continuously varying classification is developed (Figure 2).

An alternate way of describing aggregate shape, also shown on Figure 2, is by means of sphericity, ψ, and shape factor, F. The shape factor, F, is defined as the elongation ratio, divided by the flatness ratio. Sphericity is the ratio of surface area of a sphere of the same volume as the particle to the surface area of the particle (*17*). Following this method, each grain is approximated by a tetrakaidekahedron. The three mutually perpendicular particle dimensions measured to determine shape are used to calculate the ratio of surface area of the particle compared to that of an equivalent sphere (sphericity, ψ). Roundness (R) of a particle is a measure of curvature of the corners and edges and is expressed as a ratio of the average curvature of the particle as a whole, independent of its form (*13,17,18*). Angularity (A) describes the wear of edges and corners. For example, a heavily worn aggregate tends to be rounded.

Measured Aggregate Characteristics

For this study, 80 particles greater than the No. 8 sieve were randomly sampled, and the shape and other surface characteristics were carefully determined. The roundness and angularity of individual particles were evaluated visually using standard identification charts (*13*). The variation in particle shape for the five aggregates studied is shown in Figure 2; all aggregate surface characteristic data are summarized in Table 3.

The five aggregates studied and their shape classification are as follows: granitic gneiss (disc), limestone (blade), shale (blade), quartzite (blade), and river gravel (equidimensional). The shale and limestone aggregates were quite slabby appearing (i.e., blade-shaped), as compared to the granitic gneiss (disc-shaped) and the gravel (equidimensional). A rod-shaped aggregate was not included in the study, because no source for this material could be located.

To illustrate the influence of the combined effects of the important variables present in the study on permanent deformation and resilient modulus, an aggregate influence factor (AIF) was used. The AIF was taken to be a function of the particle sphericity, roundness, surface roughness, and angularity. Surface roughness of particles was examined visually where a roughness scale (zero for glassy particles and 1,000 for very rough particles) was used to assign a surface roughness value to each type aggregate tested. Surface area, as defined by sphericity, might be an important variable in defining material response for aggregates that are not rod-shaped and have shape factors that do not vary greatly. A sufficient variation in particle shape factor, F, was not present to determine its effect in this study.

Average values of the sphericity, roundness, angularity, and surface roughness were used to evaluate AIF, which is as follows:

$$\mathrm{AIF} = 2500 * (\psi + R) - (A + \mathrm{SR}) \tag{1}$$

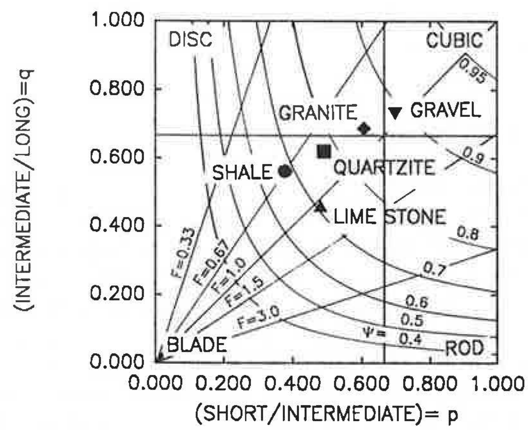

FIGURE 2 Shape classification of aggregates (*13*).

TABLE 3 SHAPE AND SURFACE CHARACTERISTICS OF THE FIVE AGGREGATES STUDIED

BASE TYPE	GRANITE GNEISS	GRAVEL	SHALES	QUARTZITE	LIME STONE
ELONGATION RATIO (q)	0.68	0.72	0.57	0.63	0.47
FLATNESS RATIO (p)	0.60	0.70	0.37	0.48	0.48
SPHERICITY	0.86	0.88	0.69	0.76	0.72
ANGULARITY	1350	150	750	1450	1450
ROUNDNESS	0.2	0.7	0.4	0.2	0.2
ROUGHNESS	800	100	300	800	800
AGGREGATE INFLUENCE FACTOR	500	3700	1675	150	50

where

AIF = aggregate influence factor,
A = average angularity value,
SR = surface roughness coefficient,
ψ = average sphericity value, and
R = average roundness value.

Several forms of the AIF were empirically tried before arriving at the expression given above. The main controlling criterion for the AIF was how good the fitted curve described the observed base behavior. Use of the AIF is considered a useful tool for presenting the results of this study; it is not intended to be a general aggregate characteristic. Certainly much more extensive research must be conducted before an AIF-type approach can be proposed for general use.

CYCLIC LOAD TESTS

Sample Preparation and Testing

Preparation

The resilient and permanent deformation characteristics of the five aggregate types were determined using specimens nominally 6 in. (150 mm) in diameter by 12 in. (300 mm) in height tested in a large triaxial cell. Sample preparation generally followed the procedure given by AASHTO T-274-82. All materials were mixed thoroughly with the required optimum quantity of water determined from the modified Proctor test (AASHTO T-180). The material was then placed in six 2-in. (50-mm) thick layers into a split steel mold and compacted with a vibratory compactor to obtain the required density.

Soaked specimens were prepared in a manner similar to that described above. Once the specimen was compacted and completely sealed, a vacuum was applied to the specimen from the top, while a de-aired water supply was introduced to the bottom of the specimen. Water was allowed to percolate slowly up through the specimen until the entrapped air was removed. The LVDT and load cell instrumentation used have been described elsewhere (*5*).

Test Procedures

The resilient modulus test procedure generally followed the recommendations of AASHTO T-274-82. Before testing, specimens were conditioned for 1,200 load repetitions at the stress states summarized in Table 4. After the conditioning phase, readings of both axial and radial resilient deformations were taken and recorded after 200 load repetitions for each state of stress tested.

To evaluate permanent deformation, one stress state was used for the entire cyclic test. A confining pressure of 6 psi (41 kN/m^2) was applied to the specimen, which was then subjected to 70,000 load repetitions using a principal stress ratio of either 4 or 6. Slow cyclic tests using a static type testing machine were also performed to determine the feasibility of using static methods for evaluating both resilient and permanent deformation properties of granular materials.

In the slow cyclic test, the specimen was subjected to a confining pressure of 6 psi (41 kN/m^2). The deviator stress was increased gradually at a slow loading rate of 0.03 in./min until reaching the full deviator stress; the load was then slowly removed. Five slow loading-unloading cycles were applied, and the complete deformation history for each cycle was recorded. All tests were performed in the drained condition.

Permanent Deformation Response

Influence of Aggregate Shape and Surface Characteristics

Figure 3 shows the relative tendency to undergo permanent deformation of the five aggregates. All specimens were compacted to 100 percent of AASHTO T-180 density. The medium gradation given in Table 5 was used for this comparison; this gradation had 4 percent fines unwashed. The specimens were subjected to 70,000 repetitions of loading at a confining pressure of 6 psi (41 kN/m^2) and principal stress ratios of σ_1/σ_3 = 4 and 6. These stress levels correspond to typical light and medium pavement sections.

The results showing relative rutting in Figure 3 indicate, for practical purposes, the permanent deformation charac-

TABLE 4 SPECIMEN CONDITIONING USED FOR RESILIENT MODULUS TESTING

CONDITIONING PHASE	NUMBER OF REPETITIONS	CONFINING STRESS (PSI)	DEVIATOR STRESS (PSI)
1	200	5	5
2	200	5	10
3	200	10	10
4	200	10	15
5	200	15	15
6	200	15	20

teristics of the disc-shaped granitic gneiss, blade-shaped limestone, and blade-shaped shale are all similar. The blade-shaped quartzite appeared to be about 30 percent more susceptible to rutting than the other crushed aggregates. Some of the differences in rutting of the quartzite might be due to scatter in the test data. Differences in the particle surface characteristics of these aggregates were not great as shown in Table 3.

These results indicate that blade- and disc-shaped aggregates having characteristics similar to those studied (although they may not appear to be as nice as equidimensional aggregates) can perform about as well with respect to permanent deformation. Of course, regardless of the aggregate shape, the gradation including the amount of fines and density has an important influence on performance. Had a greater variation in particle shape existed, more variation in permanent deformation performance might have been observed.

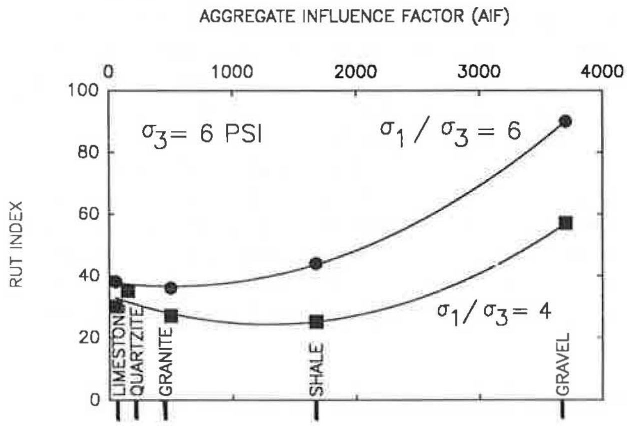

FIGURE 3 Effect of aggregate type on rut index.

The rounded river gravel tested was over two times more susceptible to rutting than the crushed aggregates. The uncrushed gravel was equidimensional (cubic) with a smooth surface, well-rounded, and had rounded corners—the worst possible characteristic for minimizing rutting. Also, the solid volume for 1 ft^3 of gravel was 0.77 ft^3 compared with 0.82 to 0.85 ft^3 for the other materials tested (Table 5). Hence, particle packing for the gravel was not as dense as for the other aggregates.

Influence of Finer Material

The tests described previously were performed on aggregates coming from different geologic formations and origins. As a result, these aggregates could have different plasticity characteristics of the finer material. To determine whether plasticity of the finer material influenced the rutting behavior of the aggregates studied, the shale and gravel were retested using specimens prepared with material passing the No. 40 sieve being replaced with the granitic gneiss. The results of this supplementary study are shown in Figure 4 together with results obtained from the earlier tests that used native finer

TABLE 5 AGGREGATE MATERIAL PROPERTIES AND GRADATIONS USED

MATERIAL PROPERTIES								
MATERIAL TYPE	SPECIFIC GRAVITY			ABSORP. (%)	WEAR	CLASS	γ_{max} (1) (PCF)	(2) Vs
	BULK	S.S.D.	APPARENT					
GRANITIC GNEISS	2.73	2.75	2.77	0.53	0.46	B	141	0.82
GRAVEL	2.61	2.62	2.65	0.69	0.464	A	126	0.77
SHALE	2.72	2.73	2.75	0.2	0.249	A	140	0.82
QUARTZITE	2.72	2.74	2.76	0.49	0.256	A	147	0.85
LIMESTONE	2.75	2.76	2.79	0.53	0.22	A	144	0.84

GRAIN SIZE DISTRIBUTION						
GRADATION	PERCENT PASSING					
	1.5"	3/4"	3/8"	NO. 4	NO. 40	NO. 200
MEDIUM	100	80	60	45	13	4
COARSE	100	65	43	27	7	0
FINE	100	85	70	58	25	10

NOTES : 1. MAXIMUM DRY DENSITY AS DETERMINED BY AASHTO T-180 METHOD.

2. VOLUME OF SOLIDS IN 1 FT.³ OF BASE MATERIAL COMPACTED TO 100 PERCENT OF MAXIMUM DRY DENSITY.

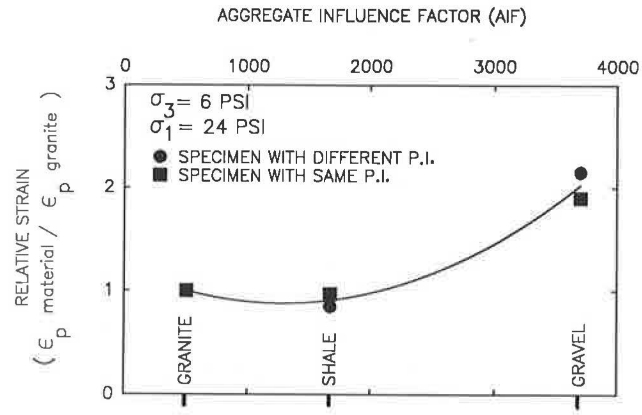

FIGURE 4 Influence of plasticity of fines on relative strain for the medium gradation.

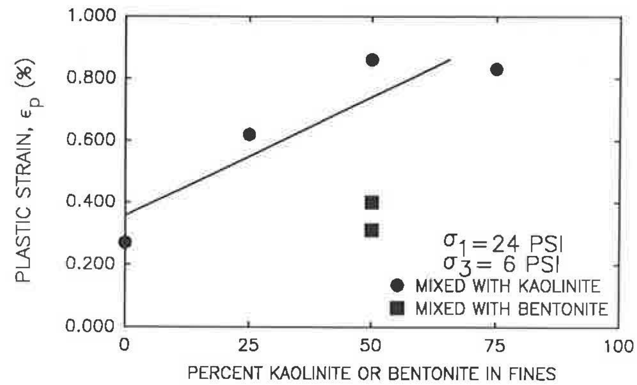

FIGURE 5 Influence of percent kaolinite or bentonite in the fines on plastic strain for the medium gradation of granitic gneiss.

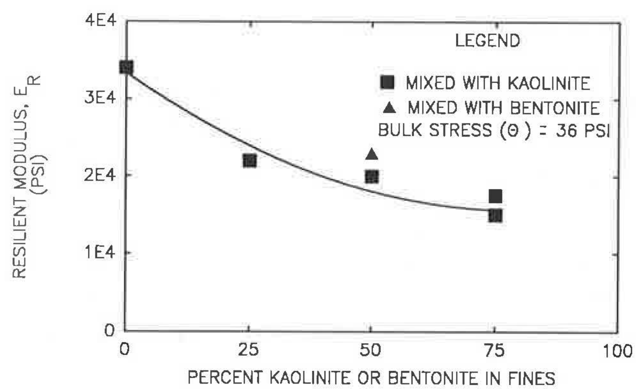

FIGURE 6 Influence of percent kaolinite or bentonite in fines on the resilient modulus of medium gradation of granitic gneiss.

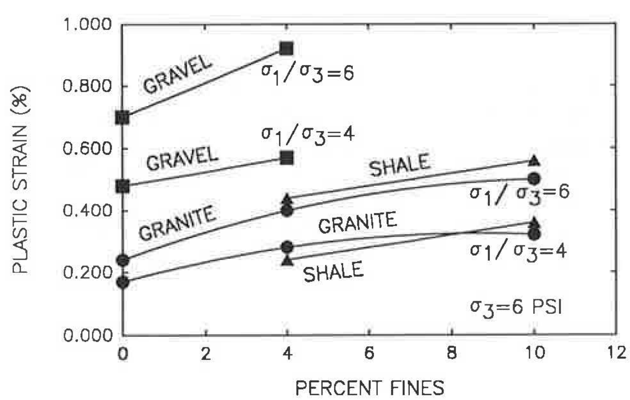

FIGURE 7 Influence of material type, stress level, and percent fines on plastic strain.

material associated with the aggregate. These results indicate that, for the aggregates tested, the material greater in size than the No. 40 sieve was apparently the dominant factor in determining the amount of rutting, and the effect of any variations in composition of the minus No. 40 particles was relatively small.

Influence of Plasticity of Fines

To study the effects of plasticity of the fines, tests were conducted using the granitic gneiss with kaolinite or bentonite substituted for a portion of the fines to determine their effect on resilient and permanent deformation characteristics. Specimens having 0, 25, 50, and 75 percent of kaolinite or bentonite fines were tested under a confining stress of 6 psi (41 kN/m^2) and a principal stress ratio of 4.

The permanent strain was found to increase by about a factor of 3 as the kaolinite in the fines increased from 0 to 75 percent (Figure 5). Hence, the presence of plastic fines can have a serious detrimental effect on the performance of an aggregate base course.

The effect of adding bentonite to the granitic gneiss is also shown on Figure 5. Adding bentonite had only a slight effect on permanent strain. The same water content, however, was used in sample preparation. The bentonite absorbs large quantities of water. If more water were added, the specimen prepared with bentonite fines would probably undergo more permanent deformation than if kaolonite were used.

Figure 6 shows the influence of percent kaolinite in the fines on the resilient response of the granitic gneiss.

Influence of Gradation

Figure 7 shows the influence of gradation (Table 5) on the permanent deformation characteristics of the granitic gneiss, shale, and gravel. As the gradation became finer (with more fines in the specimen), the tendency to undergo permanent deformation became greater; this trend has been observed elsewhere (*3,5*).

Resilient Moduli

The influence of various factors on the resilient moduli of aggregate bases has received considerable attention in the past (*1–5,12,19–21*). Therefore, only a relatively brief summary of the resilient moduli results is presented for three types of aggregates. At the present time, the resilient modulus is usually represented by the well-known k-θ model (*1,5*), which is used in this study to express test results. Improved alternate methods, however, now exist for representing resilient moduli of granular material such as the contour model (*11,19*).

Aggregate Type

Figure 8 shows the influence of material type and state of stress on the resilient modulus for the medium gradation of specimens compacted at 100 percent of T-180 density. The aggregate type had a significant influence on the resilient modulus when other factors were held constant. The resilient moduli of the rough, angular materials were higher than the rounded gravel by a factor of about 50 percent at low values of bulk stress. At high bulk stresses, the resilient modulus of the angular granite was higher than that of gravel by a factor of 25 percent.

Moisture

For granitic gneiss at a low bulk stress of 15 psi (103 kN/m^2), the resilient modulus decreased by a factor of about 40 percent upon soaking. At a high bulk stress of 100 psi (690 kN/m^2), the decrease in the resilient modulus was about 20 percent upon soaking. For the river gravel specimens, the resilient modulus decreased upon soaking by a factor of 50 percent at a bulk stress of 15 psi (103 kN/m^2); while at a bulk stress of 100 psi (690 kN/m^2), the decrease in the resilient modulus was about 25 percent upon soaking. These results are for tests conducted on drained specimens. Had the tests been performed on undrained specimens, the effect of moisture content on the resilient modulus would undoubtedly have been greater.

Gradation and Density

The influence of aggregate gradation on resilient moduli is shown in Figure 9 for the granitic gneiss. As the gradation became finer (with the amount of fines going from 0 to 10 percent), the resilient modulus decreased by about 60 percent. As the density of the granitic gneiss was increased from 95 to 100 percent of T-180 density, the resilient modulus increased by 50 to 160 percent at a low level of bulk stress corresponding to 10 psi (69 kN/m^2). At a high bulk stress of 100 psi (690 kN/m^2) the effect of increase in density was less pronounced, with the increase in resilient modulus being only about 15 to 25 percent.

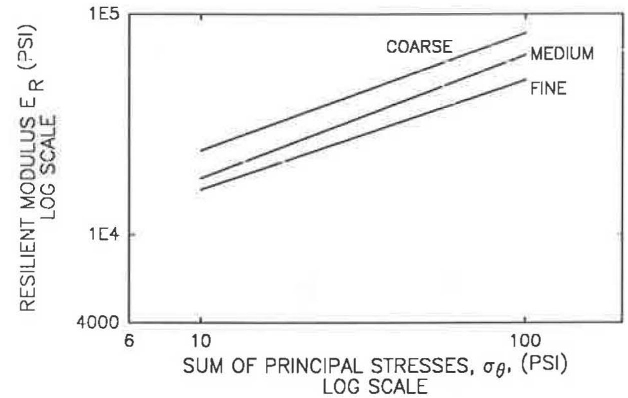

FIGURE 9 Influence of gradation and state of stress on resilient modulus: granitic gneiss.

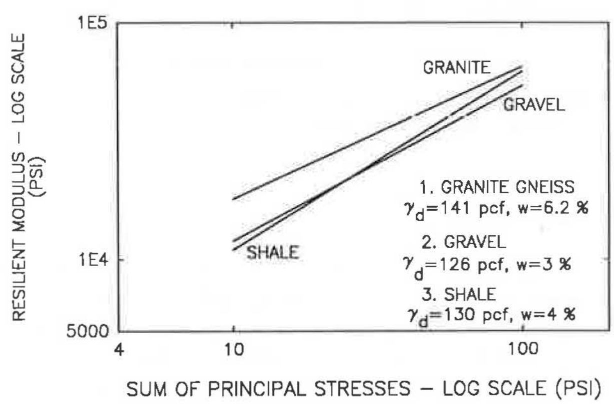

FIGURE 8 Influence of material type and state of stress on resilient modulus for medium gradation.

SLOW CYCLIC CONVENTIONAL TESTS

Many laboratories do not have the equipment and instrumentation necessary to perform conventional cyclic triaxial tests. Therefore, the potential use of a static triaxial test was investigated for determining both resilient and rutting properties of unbound aggregate bases. Development of simplified testing procedures is particularly desirable, because the new AASHTO flexible design method encourages the use of resilient modulus (*22*).

In the slow test used in this study, the applied load was slowly repeated using conventional triaxial test equipment for five cycles with the rate of loading equal to 0.03 in./min. Permanent deformation was the total recorded after the five cycles. The resilient modulus was the deviator stress, divided by the recoverable deformation observed upon unloading at the end of the fifth load cycle.

Permanent Deformation

Figure 10 shows the correlation for permanent deformation between the cyclic and static methods. These results show a much higher amount of scatter in permanent deformation than was found for the resilient modulus. The relative permanent strain between cyclic and static tests varied from 1.22 to 3.2 for all the specimens tested. The average ratio of permanent deformation obtained from the conventional dynamic test at 70,000 load repetitions over the permanent deformation obtained from the slow cyclic static test using five loading-unloading cycles was about 2.5 for the specimens tested under similar conditions (Figure 10).

These results indicate that a slow static test, as performed for this study, is probably not suitable for evaluating the cyclic load permanent deformation performance of aggregate bases.

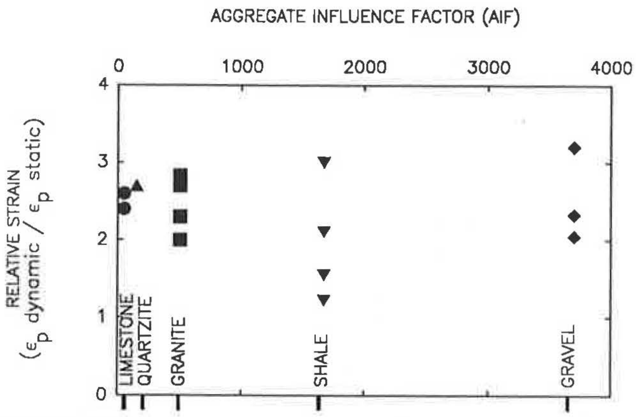

FIGURE 10 Relative permanent strain related to aggregate type.

Resilient Modulus

Figure 11 shows a comparison between the resilient modulus measured using the cyclic triaxial test and the static unload modulus measured after five cycles of slow loading. Excellent agreement is seen between the resilient and the slow cyclic unload modulus after five cycles. The average ratio of resilient modulus from the dynamic test to the resilient modulus from static test was about 1.08. Good correlations have also been obtained by Sweere and Galjaard (*23*) and Kalcheff and Hicks (*24*).

These findings, together with the results of the earlier studies, indicate that a slow cyclic test can be used to evaluate the resilient modulus of unbound aggregate bases for design purposes. The modulus obtained from a slow cyclic test could, if desired, be increased by 10 percent to give better results, which is in agreement with other studies (*23*).

CONCLUSIONS

The rut index concept proposed a number of years ago for comparing the relative permanent deformation characteristics of base course aggregates is re-evaluated and revised to give a simpler test procedure using a single test. The principal stress ratio σ_1/σ_3 to use in the test typically varies between 2 and 6 depending on the structural strength of the section.

Aggregate characteristics including shape, angularity, surface roughness, and roundness have an important influence on the resilient and permanent response of an unbound aggregate. Methods are presented for evaluating these aggregate properties. The permanent deformation characteristics of disc-shaped granitic gneiss, blade-shaped limestone, and blade-shaped shale aggregates were all similar for the same gradation and level of compaction. The general appearance of these aggregates was, however, quite different. A blade-shaped quartzite appeared to be slightly more susceptible to rutting than the other crushed aggregates. A cubic-shaped, rounded river gravel with smooth surfaces was over two times more susceptible to rutting than the crushed aggregates.

A conventional, slow triaxial shear test can be used for practical applications to evaluate the resilient moduli of an unbound aggregate. The slow triaxial test appears to be unsuitable for evaluating permanent deformation characteristics.

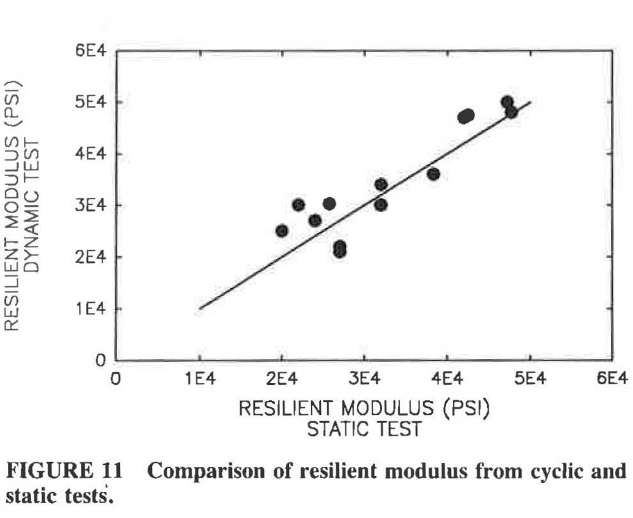

FIGURE 11 Comparison of resilient modulus from cyclic and static tests.

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