1972

Repetitive triaxial compression of granular base course material with variable fines content

Eldon Glen Ferguson
Iowa State University

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BASE COURSE MATERIAL WITH VARIABLE FINES
CONTENT.

Iowa State University, Ph.D., 1972
Engineering, civil

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Repetitive triaxial compression of granular base course material with variable fines content

by

Eldon Glen Ferguson

A Dissertation Submitted to the Graduate Faculty in Partial Fulfillment of The Requirements for the Degree of

DOCTOR OF PHILOSOPHY

Department: Civil Engineering
Major: Soil Engineering

Approved:

Signature was redacted for privacy.

In Charge of Major Work

Signature was redacted for privacy.

For the Major Department

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For the Graduate College

Iowa State University
Ames, Iowa
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INTRODUCTION

In recent years, there has been considerable interest in the response of highway materials to repeated loading, since design criteria based on material properties assessed by conventional strength tests have all too often been found to be inadequate. Field studies indicate that the repeated application of stresses caused by moving wheel loads often affects the strength and deformation characteristics of the pavement system. Design criteria may be modified to reflect this type of loading, but a better understanding of material properties must first be gained.

This study is one phase of an investigation carried out at Iowa State University on the properties of granular base course material. The study is centered around two crushed limestones that are representative of the material currently used in the state for granular base construction. Initial work has indicated that these have basically the same strength characteristics as determined by the conventional triaxial compression test, while performance records do not reflect this same similarity.

The objective of this study was to determine the response of granular material to repetitive stress conditions and the effect of varying amounts of fines (-200 sieve) present in the material. The method of testing selected was the triaxial compression test.
REVIEW OF LITERATURE

Methods of evaluating the physical characteristics of granular materials have changed little until the past decade, when there has been increased interest in effects of repeated stresses due to mounting evidence that the detrimental effects of wheel loads on the pavement surface cannot be satisfactorily predicted through use of conventional tests. Numerous investigations have been conducted with emphasis both on material variables and fluctuating state of stress of the material. The major stress parameters are the stress level, load duration and frequency. Material parameters include degree of saturation, dry density (or void ratio), and fines content.

Failure Criteria

The use of repeated loads requires the establishment of a criterion of failure since criteria used for static load tests are not directly applicable. The most commonly used criterion of failure is based upon the change in slope of the deformation vs. number of load applications curve for a material subjected to repeated loading:

Larew and Leonards (1962) concluded that for fine-grained soils subjected to repeated loading:

A critical level of repeated deviator stress, \( \sigma_{rc} \), exists at which the slope of the deformation vs. number of load repetitions curve is constant after the first few load applications. For levels of deviator stress in excess of this critical value, the deformation curves eventually turn concave upward, their slopes continue to increase until failure occurs either by sliding along a shear plane or by excessive
bulging. For levels of deviator stress less than the critical value, the deformation curves eventually approach a horizontal asymptote.

The critical deviator stress, $\sigma_{tc}$, therefore serves as a measure of the strength of a material below which a state of equilibrium will exist. Larew and Leonards (1962) further state that "the ratio of $\sigma_{tc}$ to the deviator stress at failure in a conventional triaxial test, $\sigma_{tc}/\sigma_s$, may be taken as a measure of the strength reduction due to the effects of repeated loading."

Ahmed and Larew (1963) found that the above failure criterion was also valid for coarser-grained clayey sands and residual limestone.

Marley (1969) did not observe a critical deviator stress for untreated granular base course material subjected to repetitive loading. All deformation vs. number of load application curves have decreasing slopes. Plots of axial strain versus cube root of deviator stress applications showed good linearity, with slopes increasing with increases in level of applied stresses.

Critical stress, though serving as a measure of material behavior, does not provide the complete picture of material response. Elastic or resilient rebound of materials stressed well below the critical stress may be excessive, resulting in pavement failure. Considerable work has been done on determining the resilient behavior of granular material subjected to cyclic loading both through use of triaxial compression

---

\(^{1}\)In soil engineering literature "deviator stress" denotes the difference in major and minor principal stresses, $\sigma_1 - \sigma_3$. 
and plate bearing tests. The resilient behavior of a material subjected to repeated loading may be expressed in terms of a resilient modulus, defined as the change in deviator stress applied to a specimen divided by the resilient axial strain. Modulus of resilience thereby provides a means of expressing the resilient behavior of a material to be incorporated in elastic theories for layered systems.

**Stress Level**

Studies of the resilient response of granular materials subjected to repeated loading have indicated that the modulus of resilience increases with confining pressure and is relatively unaffected by the magnitude of the applied deviator stress as long as it is not high enough to cause excessive plastic deformation.

Biarez (1962) found that the resilient modulus of a uniform sand could be expressed as follows:

\[ M_r = K'_1 (\theta)^{K'_2} \]

where \( K'_1 \) is a constant, \( \theta \) is the sum of the principal stresses and \( K'_2 \) is an exponent varying from 0.5 to 0.6.

Trollope et al. (1962) found that as long as a condition of failure was not reached, the modulus increased with increased confining pressure but was unaffected by the axial stress.

Hicks (1970) in summarizing the work of several investigators concluded that modulus of resilience could also be expressed as:

\[ M_r = K_1 \sigma_3^{K_2} \]
where $\sigma_3$ is lateral pressure and $K_1$ and $K_2$ are constants. He further observed that the similarity of $K_2$ or $K_2'$ for various granular materials suggests that the modulus increased with $\sigma_3$ or $\Theta$ in a similar manner for most granular materials. "The distinguishing characteristic is the coefficient $K_1$ or $K_1'$ which varies apparently with aggregate type, density and grading, and degree of saturation."

**Stress Duration and Frequency**

Seed and Chan (1961) in working with a silty sand observed that decreasing the stress duration from 20 minutes to $1/3$ second resulted in an increase in the resilient modulus of from 23,000 to 27,000. Considering the range of duration investigated, the resulting change in modulus was quite small.

Folque (1965) conducted dynamic triaxial tests of compacted unsaturated soils and concluded that different frequencies of load applications (0.5 to 5 cycles/sec) has little effect on the clayey soil. Minor variations in moisture content or density had a much greater influence than did frequency of applied forces.

**Degree of Saturation**

When comparisons are made on a total stress basis, generally the modulus of resilience decreases as the degree of saturation increases. Hicks (1970) found from comparisons on the basis of effective stress that the resilient modulus for 100% saturated samples differs only slightly from dry samples.
Haynes and Yoder (1963) reported the results for undrained repeated-load triaxial tests on gravel and crushed stone. They observed that the modulus of resilience for the crushed stone decreased only slightly as level of saturation increased. The modulus of the gravel was much more sensitive to changes in saturation, decreasing to half its original value at high degrees of saturation.

Density

Studies on the effect of density on the resilient response of granular material are limited. Hicks (1970) states that it is generally felt that dry density has a significant effect on the resilient response in that as density increases, resilient deformation tends to be reduced for the same level of stresses.

Trollope et al. (1962) in working with a poorly graded silty sand observed that the modulus of the sand increased with a decrease in void ratio. The difference in moduli between a loose and dense sand was as much as 50%.

Percentage of Fines

Studies of the effect of fines (percent passing No. 200 sieve) on the resilient response of granular materials indicate that the amount of fines present does have an effect but the effect is unclear.

White (1963) reported that resiliency of aggregate base materials did not appear to be influenced by the type of aggregate, gradation, moisture content and density of the original specimen, but was dependent
only upon the applied force and number of load applications. The type of test used was a confined compression test where the specimens were compacted and loaded while contained within a steel cylinder. This type of test would measure only the material compressive characteristics and not the shear failure.

Haynes and Yoder (1963) conducted repeated triaxial tests on granular base course materials used in the ASSHO Test Road. A lateral pressure of 15 psi and a deviator stress of 70 psi were selected for the tests on the basis of the approximate stresses that existed at the base course level in the road test pavement. Three levels of fines contents were used: 6.2%, 9.1%, and 11.5%. It was concluded that resilient modulus was affected only slightly by grading. Gravel exhibited a lower modulus at the intermediate fines content (9.1%) while the crushed stone was essentially the same regardless of grading.

Hicks (1970) reported that "In general aggregate grading (or fines content) was shown to have only a small effect on the resilient modulus, regardless of aggregate type." For the three fines contents used (3%, 5%, 10%) with crushed aggregate Hicks (1970) reported that the medium gradation yielded slightly higher values of modulus than did the fine and coarse gradations. These results were from repeated triaxial tests of dry well-graded angular crusher run base rock.

Colley and Nowlen (1958) investigated the performance of granular subbases under concrete slabs subjected to repeated plate bearing loading. It was observed that the densification of the subbase was related to the
permeability of the granular subbase material. The open graded, high permeability materials showed less densification than the dense-graded, low permeability materials. Also pumping, or forcible ejection of water and soil from under flexing slabs, occurred in some of the dense graded subbases but was not observed in any materials having less than 10% - #200 material.

It is generally felt that the effect of fines on the stability of granular bases is primarily a result of its influence on permeability. Barber and Steffens (1958) stated:

Critical pore pressures which affect the bearing capacity of a subgrade or base course can be controlled by changing the gradation and drainage so that the material when compacted has void spaces which are less than 80 percent saturated.

The author was primarily concerned with pore pressure resulting from temperature fluctuations within the base material and observed that the magnitude of dilation required to relieve temperature-induced pore pressures was within the range of expansion of a graded granular material.

Barber (1959) conducted tests to determine permeability of granular bases with varying amounts of fines. He observed that bases containing more than 5% fines may require excessive time to drain, causing a potential development of pore pressures under load. In effect this could reduce the strength of the base layer to a critical level.

Thompson (1969) observed similar results from load tests on crushed stone from the AASHO Road Test. Tests on soaked specimens with high fines contents experienced greater permanent deformation than specimens having lower fines contents. Increasing the fines content from 3% to 21% resulted in a 100% increase in permanent deformation.
If fines content is a contributing factor in the performance of a granular base we must be concerned not only with the amount of fines present in the material as it is brought to the construction site, but also the amount after construction and use for a period of time, since the fines may increase as a result of degradation. "Degradation" was defined by Erickson (1959) as "A breaking down and/or disintegration of particles of sand, gravel or stone, primarily due to the alteration and subsequent decomposition of their mineral components, accelerated by the action of mixers, mechanical equipment, traffic or the elements." Another cause of increased fines content could be the intrusion of the subgrade material. Day (1962) reported on a flexible pavement that had failed after several years service due to a concentration of fines and water just below the bituminous treatment and a lack of penetration or bond of the surface material to the base course. The original fines content of the base course material was 7%, within the sections that had failed the fines content had increased to an average of 12-13%, with a high of 16%.

Chamberlin and Yoder (1959) investigated the effect of base course gradation on fines migration during laboratory pumping tests. Of primary concern was the densification of the base material, pumping of the base, and intrusion of the subgrade material. They concluded that under the proper conditions of moisture and load, a dense-graded base course with a high fines content would have -#200 material removed by pumping, whereas a very open-graded base course material could be expected to be contaminated by intrusion of fines from the subgrade. A range of gradation
was found within which little or no intrusion or pumping occurred. The limits of this range were the grain size distribution curves expressed by the formulas \( p = 100 \left( \frac{d}{D} \right)^{0.7} \) and \( p = 100 \left( \frac{d}{D} \right)^{1.2} \) where \( p \) is the percent by weight passing a given sieve having opening of width \( d \) and \( D \) is the maximum particle size of the material. For a 3/4" granular base material the range in -#200 material within which little intrusion or pumping would occur would be from 2.0 to 0.13%.
EXPERIMENTAL PROGRAM

Materials

Two crushed limestones were used in this investigation. They were previously used in a program of standard triaxial testing, thus giving a good background on the basic properties of the stones.

The two stones are representative of a large portion of the material used for base course construction in Iowa and may be briefly described as follows:

1. A weathered, moderately hard limestone of the Pennsylvanian system obtained from near Bedford, Taylor County, Iowa, hereafter referred to as the Bedford sample. Rocks in this system outcrop in nearly one-half of the state, are generally soft and contain relatively large amounts of clay.

2. A hard dolomite obtained from near Garner, Hancock County, Iowa, hereafter referred to as the Garner sample. This material is from the Devonian System and has shown remarkable uniformity through several counties.

The two crushed limestones were initially tested in the same condition that they were received from the quarry stockpile. In the later stages of the testing program the fines content (% <200) was altered but no other changes were made.

Chemical and mineralogical properties of the stones as determined by X-ray diffraction and measurement of pH and cation exchange capacity (cec),
are shown in Tables 1, 2, and 3. Table 4 presents the engineering properties of each of the crushed stones.

Table 1. Mineral constituents of the whole material by X-ray diffraction

<table>
<thead>
<tr>
<th>Stone des.</th>
<th>Calcite</th>
<th>Dolomite</th>
<th>Quartz</th>
<th>Feldspars</th>
<th>Calcite/dolomite ratio</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bedford</td>
<td>Pred.</td>
<td>Small amount</td>
<td>Trace</td>
<td>Not ident.</td>
<td>25</td>
</tr>
<tr>
<td>Garner</td>
<td>Pred.</td>
<td>Second pred.</td>
<td>Trace</td>
<td>Not ident.</td>
<td>1.16</td>
</tr>
</tbody>
</table>

*Representative sample was ground to pass No. 100 sieve, here and in Tables 2 and 3 also.

Obtained from X-ray peak intensity.

Table 2. Non-HCl acid soluble clay mineral constituents of the whole material by X-ray diffraction

<table>
<thead>
<tr>
<th>Stone des.</th>
<th>Mont.</th>
<th>Vermiculite-chlorite</th>
<th>Micaceous material</th>
<th>Kaolinite</th>
<th>Quartz</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bedford</td>
<td>None</td>
<td>Not ident.</td>
<td>Pred.</td>
<td>Poorly crystalline</td>
<td>Large amt.</td>
</tr>
</tbody>
</table>

Table 3. Quantitative chemical analysis of whole material

<table>
<thead>
<tr>
<th>Stone des.</th>
<th>pH</th>
<th>CEC (me/100.0g)</th>
<th>Non-HCl soluble clay minerals</th>
<th>Non-clay mineral, Non-HCl Soluble material</th>
</tr>
</thead>
<tbody>
<tr>
<td>Bedford</td>
<td>9.40</td>
<td>10.88</td>
<td>10.92</td>
<td>Trace</td>
</tr>
<tr>
<td>Garner</td>
<td>9.25</td>
<td>10.60</td>
<td>5.70</td>
<td>1.03</td>
</tr>
</tbody>
</table>
Table 4. Representative engineering properties of crushed stone materials

<table>
<thead>
<tr>
<th></th>
<th>Bedford</th>
<th>Garner</th>
</tr>
</thead>
<tbody>
<tr>
<td><strong>Shear strength parameters</strong></td>
<td></td>
<td></td>
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<tr>
<td>Maximum effective stress ratio</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \phi' ), degrees</td>
<td>45.7</td>
<td>49.2</td>
</tr>
<tr>
<td>( C' ), psi</td>
<td>6.7</td>
<td>14.2</td>
</tr>
<tr>
<td>Minimum volume</td>
<td></td>
<td></td>
</tr>
<tr>
<td>( \phi' ), degrees</td>
<td>46.2</td>
<td>49.5</td>
</tr>
<tr>
<td>( C' ), psi</td>
<td>4.2</td>
<td>5.6</td>
</tr>
<tr>
<td><strong>Textural composition</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Gravel (2.00 mm)</td>
<td>73.2</td>
<td>61.6</td>
</tr>
<tr>
<td>Sand (2.00-0.074 mm)</td>
<td>12.9</td>
<td>26.0</td>
</tr>
<tr>
<td>Silt (0.074-0.005 mm)</td>
<td>8.4</td>
<td>10.2</td>
</tr>
<tr>
<td>Clay (0.0005 mm)</td>
<td>5.5</td>
<td>2.2</td>
</tr>
<tr>
<td>Colloids (0.001 mm)</td>
<td>1.7</td>
<td>1.4</td>
</tr>
<tr>
<td><strong>Atterberg limits</strong></td>
<td></td>
<td></td>
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<tr>
<td>Liquid limit, %</td>
<td>20.0</td>
<td>Non-plastic</td>
</tr>
<tr>
<td>Plastic limit, %</td>
<td>18.0</td>
<td></td>
</tr>
<tr>
<td>Plasticity index</td>
<td>2.0</td>
<td></td>
</tr>
<tr>
<td><strong>Standard AASHO-ASTM density</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Optimum moisture content, % dry soil weight</td>
<td>10.8</td>
<td>7.6</td>
</tr>
<tr>
<td>Dry density,pcf.</td>
<td>128.0</td>
<td>140.5</td>
</tr>
<tr>
<td><strong>Modified AASHO-ASTM density</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>Optimum moisture content, % dry soil weight</td>
<td>8.0</td>
<td>5.4</td>
</tr>
<tr>
<td>Dry density,pcf.</td>
<td>133.5</td>
<td>147.6</td>
</tr>
<tr>
<td><strong>Specific gravity of minus</strong></td>
<td></td>
<td></td>
</tr>
<tr>
<td>No. 10 sieve fraction</td>
<td>2.73</td>
<td>2.83</td>
</tr>
<tr>
<td><strong>Textural classification</strong></td>
<td>Gravelly sandy loam</td>
<td></td>
</tr>
<tr>
<td><strong>AASHO classification</strong></td>
<td>A-l-b</td>
<td>A-l-a</td>
</tr>
</tbody>
</table>
Specimen Preparation

The specimens used for this investigation were compacted by vibration. Previous work showed that the moisture-density relationship for vibratory compaction of these stones differs somewhat from that determined through use of standard Proctor compaction, in that vibratory compaction achieves standard Proctor density at a slightly lower moisture content (Hoover et al. 1970). Therefore, the specimens were compacted at optimum moisture content as determined by vibratory compaction studies.

Each specimen was compacted by vibration in a four-inch diameter by eight-inch high cylindrical mold attached to a Syntron Electric Vibrator table. The material was placed in the mold in four equal layers and rodded 25 times per layer with a 3/4 inch diameter, rounded tip rod. A constant frequency of 3600 cycles/min. and amplitude of 0.368 mm were used with a surcharge weight of 35 lb for a period of two minutes. Previous work showed that this method of compaction is capable of achieving standard Proctor density with a minimum amount of degradation and segregation of the specimen (Merrill and Hoover, 1968 and Hoover et al., 1970).

Following compaction, the height of each specimen was measured while in the mold. Specimens were then extruded, weighed, wrapped in two layers of Saran wrap and aluminum foil, and the ends sealed. The specimens were then stored a week or longer in an atmosphere of about 75°F and near 100% relative humidity.
Triaxial Compression Apparatus

The testing machine used in this study was fabricated by the I.S.U. Engineering Shop to specifications established by the Iowa State University Soil Research Laboratory. The pressure cell is a standard tri-axial cell capable of handling 4-inch diameter by 8-inch high specimens.

The axial load was applied by a hydraulic cylinder activated by an Enerpac program control center. The timer, counter and several pressure switches manufactured by Enerpac provided control over the magnitude of the applied load and length of time that the load was maintained. At the end of each load application the load rapidly decreased to zero and was immediately reapplied. No control of the rate of loading or unloading or the length of time at zero load was attempted but was dictated by the system used. The loading was essentially square or trapezoidal-wave as shown in Figure 1.

The loading phase consisted of a period of load build-up and a dwell period. The time required for load build up was dependent upon the load level and resiliency of the specimen being loaded since both affect the volume of fluid flowing to the actuating cylinder. The time required varied from 0.1 second to 0.2 second for almost all tests. The dwell period was controlled by a timer built into the system and was the only phase of the load cycle that could be varied. The timer allowed variation in dwell times of from 0.1 second to 1 second.
Figure 1. Sequence of repeated load.
The length of the unloading phase was controlled by the hydraulic system and could not be varied. The length of this phase remained constant at about 0.4 second. The unloading phase consisted of two components, the load release phase and a period of zero load. The length of the load release phase as for the loading phase was dependent upon the magnitude of applied load and resilience of the specimen since these again govern the volume of hydraulic fluid returning to the pump. The length of the period of zero load therefore is the difference between unloading phase and the time required for total load release. In all cases the load dropped to zero and remained there for a minimum of 0.1 second.

Measurement of the applied load was accomplished by use of a Dillon Series 200, 15,000 lb. capacity load cell and Dillon Type B meter readout.

Positive and negative pore pressures at the base of the specimen for the undrained tests were measured with a 0 to 100 psia pressure transducer manufactured by Consolidated Electrodynamics and read by a Daytronic Corporation Model 300D Amplifier-Indicator with Type 93 strain gage input module. The indicator was calibrated to read directly in pounds-per-square inch with an arbitrary zero reference taken at atmospheric pressure.

Volume change was measured by filling the cell with de-aired water and measuring the volume of water displaced from the cell. This volume was measured by the device shown in Figure 2. The float in the device was connected to the core of an LVDT (linear variable differential
Figure 2. Cross-section of volume change measurement apparatus.
porous end-stones were saturated.

The specimen was then placed between the end stones and set on the base, and 1½" wide rubber bands cut from a triaxial membrane were placed at both the top and bottom in such a manner that they covered the edge of the porous stones and extended about 1" onto the specimen. Although adding somewhat to end restraint, these strips greatly reduced the number of tests aborted due to rupture of the rubber triaxial membrane at the specimen-porous stone contact.

Following this step the specimen was encased in a 0.025 in. thick rubber triaxial membrane and was sealed by use of heavy rubber bands around the base and specimen cap. The cell was then assembled, bolted, and filled with de-aired water.

All tests were conducted after allowing complete consolidation under the confining pressure prior to axial loading. The volume change was monitored during the consolidation phase and in all cases consolidation was complete after 36 minutes. The time required for complete consolidation was quite short, attributed to the high permeability and low compressibility of the specimen and to the relatively low confining pressure used, 10 psi.

Axial loading of the specimen was begun at the end of the 36-minute consolidation period. Activation of the system resulted in the application of the preset load to the specimen cap for the desired length of time, followed by release and repetitive re-application. The applied load, axial deflection and volume change were continuously monitored.
Tests of short duration were monitored until failure, whereas for tests of long duration, only the first ten cycles out of each hundred were recorded after completion of the first 100 cycles of loading.

Loading was continued until a total axial strain of approximately 18% had been reached or until a state of apparent equilibrium had been achieved.

Upon completion of the loading phase, the specimens were removed from the cell and were divided into thirds. The material was then oven-dried and moisture contents determined for each segment. After drying, all material in each segment was used for a washed sieve analysis allowing determination of the gradation of the gravel and sand fractions and total fines content (≤200) for each segment and for the total sample.

Testing Program

All tests were conducted using a lateral confining pressure of 10 psi in order that the effect of cyclic loading and material properties could be observed at a lateral restraining condition closely approximating that in a granular base course. Selection of the level of lateral pressure for triaxial tests is very critical. Seed et al. (1967) concluded:

The behavior of granular materials comprising the pavement section should be measured under conditions of stress which are representative of the actual conditions existing in pavements, since the magnitude of the stress influences the resilient behavior of the material.

This is readily apparent when one recalls that the resilient modulus is a function of the lateral pressure and to a lesser extent, the sum of the
principal stresses. The lateral pressure used therefore must be representative of actual field conditions.

At the present time limited data exists as to the actual lateral pressures that are developed in granular bases. Repetitive triaxial tests of granular base course materials were conducted by Haynes and Yoder (1963) using a lateral pressure of 15 psi, selected on the basis of stresses believed to exist at the base course level in the AASHO Road Test pavement.

The range of lateral pressures that may be encountered within a granular base course may be estimated using an expression developed from the Boussinesq solution and presented by Ahlvin and Ulery (1962):

\[ \sigma_H = p[2\nu A + C + (1-2\nu) F] \]

where

\[ \sigma_H = \text{lateral pressure} \]
\[ p = \text{uniform pressure at the surface on a circular plate} \]
\[ \nu = \text{Poisson's ratio} \]
\[ A, C \text{ and } F = \text{functions of depth and offset distance from the center of the plate—determined from tables presented by Ahlvin and Ulery (1962).} \]

The stresses computed by this expression are for a uniform surface pressure (100 psi) applied to a circular area. A circular area having a radius of 5.5 inches was used, simulating a single wheel load of approximately 9,000 lb. The computed lateral pressures for various depths and offset distances from the center of the plate are given in Table 5, with the uniform plate pressure assumed to be acting at the surface of
Table 5. Computed lateral pressures within a granular base course subjected to a 9,000 lb single wheel load

<table>
<thead>
<tr>
<th>Depth</th>
<th>Offset distance from axis of plate</th>
<th>0 inches</th>
<th>5.5 inches</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>$\nu = 0.3$</td>
<td>$\nu = 0.5$</td>
<td>$\nu = 0.3$</td>
</tr>
<tr>
<td>0 inches</td>
<td>80 psi</td>
<td>50 psi</td>
<td>50 psi</td>
</tr>
<tr>
<td>4 inches</td>
<td>23</td>
<td>15</td>
<td>21</td>
</tr>
<tr>
<td>8 inches</td>
<td>4</td>
<td>0.6</td>
<td>8</td>
</tr>
</tbody>
</table>

an 8 inch granular base layer. Values of lateral pressure are given for Poisson's ratios of 0.3 and 0.5 since the actual ratio for granular materials is generally thought to lie between these two limits, Seed et al. (1967).

Since loads of this magnitude are seldom applied directly to a base course, the computed lateral pressures directly beneath the plate are not realistic. Generally, an asphaltic concrete surface course is placed over the base course. Since this surface course generally has a minimum thickness of 4 inches and would be more than equivalent to an equal thickness of crushed stone, the lateral pressures computed at a depth of 4 inches would probably represent the upper limit of pressures experienced within the base course. The lateral pressure at the midpoint of an 8 inch base course with a 4 inch surface course therefore would be less than the computed values for the 8 inch depth.

Based on these computed values and the work done by other investigators in this area, a lateral pressure of 10 psi was selected. It must
be recognized, however, that this is at best a rather poor simulation of field conditions implied in Table 5, first because the lateral pressure ideally should be much larger at the top than at the bottom of the specimen, a rather difficult requirement for a triaxial test. Secondly as the wheel moves, the lateral pressure increases in a complex manner while the vertical pressure increases to a maximum which is reached when the wheel is directly over the point of stress measurement. This is partly because of rotation of the principal stress axes as the wheel passes over, another factor that cannot be simulated in a triaxial test. A more precise simulation of the nonuniform stress conditions would be possible by a model test, but the lack of direct control on stresses and the difficulties in measurement of stresses make such a test undesirable for understanding the material behavior. The repetitive triaxial test therefore may be viewed as an idealized model which hopefully will produce the same types of material responses as occur in the field, and will lead to an understanding of these responses.

A total of 52 Garner specimens were tested. Four axial loads were used ranging from 1150 lb to 1700 lb resulting in applied deviator stresses of from 91 psi to 135 psi. This range of stress ratios was between the state of stress at minimum volume and maximum effective stress ratio as determined by the standard triaxial test.

The deviator stress at minimum volume is thought to be a "proportional limit" of the stress-strain curve for a granular material (Hoover, 1970) and has been suggested as a criterion for failure (Fish and Hoover, 1969, and Ferguson and Hoover, 1968). Therefore, stress levels above the
deviator stress at minimum volume would represent a state of incipient failure. Figure 3 shows a typical stress-strain curve for the Garner crushed stone. The deviator stresses selected for the repetitive loading are all between the two failure criteria for the standard tri-axial test.

Dwell times of 0.25, 0.50, 0.75 and 1.00 seconds were used at each of the stress levels. The majority of the tests, however, were run with a dwell time of 0.50 second.

As the testing on this material progressed, it became evident that variations in the amount of -#200 material had a pronounced effect on the failure rate of each specimen. To be better able to evaluate this dependence, the fines content was altered in increasing amounts to provide a range in -#200 material of from 2.11 to 17.8 per cent, on a total dry weight basis. The fines content was altered by removing part of the fines from the material for one specimen and adding these fines to the next specimen. This was done in increasing amounts thereby widening the range of fines contents. The amount of fines present in each specimen prior to compaction and testing was not measured; consequently the amount of degradation occurring during compaction and loading was not determined. It is possible that the fines content observed following testing was somewhat greater than that of the original material.

Previous studies of degradation resulting from vibratory compaction, (Hoover et al. 1970), indicate a maximum increase of fines of about 1 per cent by dry weight for the method of compaction used in this investigation. Degradation during compaction has no effect on the analysis of results since we are concerned only with the fines present within the specimen during the loading phase. Degradation during compaction does
Garner Crushed Stone

\[ \sigma_3 = 10 \text{ psi} \]

Maximum \( \sigma_1 - \sigma_3 = 180 \text{ psi} \)

\( \sigma_1 - \sigma_3 = 135 \text{ psi} \)
\( \sigma_1 - \sigma_3 = 123 \text{ psi} \)
\( \sigma_1 - \sigma_3 = 111 \text{ psi} \)
\( \sigma_1 - \sigma_3 = 91 \text{ psi} \)

\( \sigma_1 - \sigma_3 = 40 \text{ psi} \) at Minimum Volume

Minimum Volume

**Figure 3.** Typical stress-strain relationship for standard triaxial test of granular base course material.
however increase the difficulty of preparing duplicate specimen or specimens at specific fines contents.

The amount of aggregate degradation occurring during the loading phase has a direct bearing on the analysis. For this reason most tests were continued until an axial deflection of 1.25 inches was reached, in an attempt to standardize the amount of degradation. Samples reaching a state of equilibrium before 1.25 inches deflection were subjected to a minimum of 10,000 load applications.

Fifteen Bedford specimens were tested with a lateral pressure of 10 psi, dwell time of 0.50 seconds, and applied axial load of 700 lb, giving an applied deviator stress of approximately 55 psi. This deviator stress is slightly above the deviator stress at minimum volume as determined by the standard triaxial test. As with the Garner crushed stone, the fines content was altered in some of the specimens tested.
RESULTS

Axial Strain vs. Load Application

The response of both crushed stones to the cyclic loading varied widely even though all stress variables were held constant. Plots of axial strain vs. the number of deviator stress applications for the Garner material, all with an applied deviator stress of 135 psi, are presented in Figure 4. From this plot we may determine the rate of failure of the material when subjected to cyclic loading. The only variation between each of the tests shown is the amount of -#200 material present in each of the specimens, indicated by the numbers shown in parentheses.

The rate of failure of the specimens appears to be closely related to the fines content of the crushed stone. With less than about 9.5% fines this rate appears to be somewhat independent of the fines content, and the variation between specimens is less. As the fines content increases above this value the rate of deflection increases rapidly, higher rates coinciding with higher contents of fines.

The effect of fines content also follows a similar pattern for the Garner material subjected to lower deviator stresses. Figures 5 and 6 are for deviator stresses of 123 psi and 111 psi respectively. The total variation in fines content of the four specimens subjected to a deviator stress of 123 psi was only 1.1% but the rate of deformation varied considerably, apparently due to this slight variation in amount of fines present.
Figure 4. Axial strain versus deviator stress application curves for Garner crushed stone.

Garner Crushed Stone

\(\sigma_1 - \sigma_3 = 135\) psi

( ) % passing #200 sieve
Garner Crushed Stone

\[ \sigma_1 - \sigma_3 = 123 \text{ psi} \]

( ) % passing #200 sieve

Figure 5. Axial strain versus deviator stress application curves for Garner crushed stone.
Figure 6. Axial strain versus deviator stress application curves for Garner crushed stone.
Figure 6 gives the axial strain relationship for four specimens subjected to a cyclic deviator stress of 111 psi. Of the four, two (8.2% and 12.3% fines) were undrained tests and appear to be in close agreement with the drained tests. Discussion of role of pore pressures will be presented later.

The final series of tests was conducted with the Bedford crushed stone (Figure 7) at an applied deviator stress of 55.7 psi. This series incorporated the widest range of fines contents and consequently the widest variation in rate of axial strains. Tests on the three specimens having fines contents of 16% or less were not continued to failure. The test shown with 16.0% fines was terminated after 25,000 cycles after having undergone only 4.3% axial strain; the test with 10.4% was terminated after 30,000 cycles of load after having undergone 3.5% axial strain and the test with 5.3% fines was terminated after 14,000 cycles of load which resulted in less than 1% axial strain. It therefore appears that axial strain of the specimen subjected to a fixed number of load cycles may be directly related to the amount of #200 material present in the specimen. To test this hypothesis one should observe the amount of axial strain produced by a fixed number of load applications and determine if it is a function of the fines content.

Axial Strain vs. Percent Fines

Figure 8 shows that the axial strain produced by 100 load applications is closely dependent upon the fines content for the Garner stone
Figure 7. Axial strain versus deviator stress applications for Bedford crushed stone.
Figure 8. Effect of fine content on axial strain after 100 deviator stress applications for the Garner crushed stone.
subjected to a cyclic deviator stress of 135 psi. This relationship may be defined by two straight line segments with the intersection at about 9% fines indicating a maximum desirable fines content for this stress level. Below this amount, the axial strain is almost independent of fines content, whereas above, it is heavily dependent upon the amount of fines present, the slope of the best fit straight line being almost thirteen times greater. Two tests (2.7% and 6.6% fines) underwent much larger axial strains than the six other specimens having fines contents less than 9%. This was attributed to improper seating of the loading piston prior to application of the first cycle of load resulting in higher initial strains, and for this reason these two were not included in the regression analysis.

A similar relationship appears between axial strain and amount of fines present for the Bedford crushed stone (Figure 9). A cyclic deviator stress of 55.7 psi and lateral pressure of 10 psi were used for all fourteen tests, the only variable being the amount of fines present. As with the Garner series, the relationship can be defined by two straight-line segments, but the point of intersection is somewhat higher, 13.6% compared to 9% fines. Above the intercept, the amount of axial strain again is greatly dependent upon the fines content, the slope of the segment being thirteen times greater than for the segment below the intercept.

Other plots of axial strains produced by 100 applications of load vs. fines content of specimens for various stress levels are shown in
Figure 9. Effect of fine content on axial strain after 100 deviator stress applications for the Bedford crushed stone.
Figure 10. To avoid confusion, the data points from Figures 8 and 9 are not included but are represented by best fit straight line segments as determined by linear regression. A great deal of similarity is shown between the two stones, even though both segments for the Bedford crushed stone have somewhat greater slopes than for the Garner.

The three other series with the Garner crushed stone do not contain as many tests nor do they have the range in fines contents as the first two series discussed. The series having an applied deviator stress of 111 psi has the widest range of -200 material with two tests below, one at the point of inflection and the fourth above. The slopes of the segments are in agreement with those of the other series and indicate that the slopes of both segments decrease with increasing stress level. The four specimens tested at a deviator stress of 123 psi appear to have fines contents in excess of the maximum desirable fine content, and it is interesting to note that at this stress level, which is midway between the Garner series having deviator stresses of 135 psi and 111 psi, the slope of the axial strain vs. fines content relationship is intermediate between the two. No specimen in this series had fines contents less than 9.5%; thus the lower straight-line segment cannot be established, and an interpolation between the other two series is shown by a dashed line.

Only two specimens were tested with an applied deviator stress of 91.5 psi and would appear to be on either side of the maximum desirable fines content. Again by assuming a uniform increase of slope as stress ratio decreases, the slopes of these segments have been interpolated.
Figure 10. Effect of fine content on axial strain after 100 deviator stress applications for all stress levels.
from other tests to give dashed segments shown passing through the two test points.

From this figure it would appear that for a specimen of granular material the magnitude of axial strain that will be produced by a fixed number of repetitive load applications is closely dependent upon the amount of \( \# \leq 200 \) material present within the specimen and may be used to establish a maximum desirable fines content. The stress level appears to affect this maximum desirable fine content, which tends to decrease as the stress level increases. However, the slope of both segments tends to decrease as the applied stress is increased. Thus it would appear that the fines within the specimen are in some way altering the shearing resistance of the material.

Also of particular interest is the response of the Bedford crushed stone, which is in close agreement with the four series of tests using the Garner crushed stone, the slopes and intercept of the straight-line segments agreeing quite well with the other shown. This similarity becomes more significant if one refers to Tables 1-4 and notes the wide differences in properties of the fines present in the two crushed stones: The fines present in the Bedford crushed stone have a liquid limit of 20.0% and a plastic limit of 18.0%, while the Garner crushed stone has non-plastic fines. Bedford crushed stone contains 10.92% non-HCl soluble clay minerals whereas only 5.70% of the Garner crushed stone consists of non-HCl soluble clay minerals. It would therefore appear that within these limits the mere presence of fines rather than the type of fines is the controlling factor.
Maximum Desirable Fines vs. Stress Ratio

The relationship between maximum desirable fines content and applied deviator stress is shown in Figure 11, indicating a decrease in desirable fines at higher stress levels. Shown are the Bedford crushed stone and the four stress levels with the Garner material, even though the properties of the fines differ considerably. It is not possible to state that characteristics of the fines have no affect on the response of crushed stone subjected to cyclic loading without confirming tests for the maximum desirable fines content of the Bedford stone at other stress levels; however it does appear that the amount of fines present is the primary controlling factor. Subsequent analysis will be based on the premise that amount of fines rather than type is the dominant influence, and that for both crushed stones the maximum desirable fines content at any stress level would agree closely with those shown in Figure 11.

Maximum Desirable Fines vs. Load Applications

Up to this point the affect of fines has been limited to axial strain produced by 100 applications of load, this number of load applications being selected so as to incorporate all tests since several were terminated after 100 cycles because of excessive deformation (18% axial strain). The question now arises as to how the aforementioned relationship might relate to the number of load applications. Figures 12-16 show the relationship between fines content and axial strain produced by 10, 100, 200, 500, and 1000 applications of 55.7 psi deviator stress for the Bedford crushed stone.
Figure 11. Relationship between level of applied stress and maximum desirable fines content.
Bedford Crushed Stone

$\sigma_1 - \sigma_3 = 55.7$ psi

Figure 12. Axial strain after 10 load applications vs. fines content for Bedford crushed stone.
Bedford Crushed Stone

$\sigma_1 - \sigma_3 = 55.7$ psi

$\epsilon = 1.14 F - 14.15$

$r = 0.980$

$C = 0.097 F + 0.08$

$r = 0.931$

13.6%

Percent Passing #200 Sieve, F

Figure 13. Axial strain after 100 load applications vs. fines content for Bedford crushed stone.
Figure 14. Axial strain after 200 load applications vs. fines content for Bedford crushed stone.
Figure 15. Axial strain after 500 load applications vs. fines content for Bedford crushed stone.

Bedford Crushed Stone

\[ \sigma_1 - \sigma_3 = 55.7 \text{ psi} \]

\[ \epsilon = 3.76 F - 57.4 \]

\[ r = 0.934 \]

\[ \epsilon = 0.134 F - 0.03 \]

\[ r = 0.914 \]

15.9\%
Figure 16. Axial strain after 1000 load applications vs. fines content for Bedford crushed stone.
The dependency of axial strain on fines content becomes apparent even after only 10 applications of the deviator stress, and as was previously observed at 100 cycles, in each case the relationship may be defined by two linear segments, the intersection of which indicates a maximum desirable fines content. As the number of load applications increases there is a marked increase in the slope of the straight line segment at the higher fine contents. Between 10 cycles and 1000 cycles this slope changes from 0.57 to 4.23, indicating that above the maximum desirable fines content only slight variations in fines contents have a pronounced effect on the amount of axial strain produced. Below the maximum desirable fines content the slope of the linear segment increases only slightly (0.06 to 0.14) as the number of load applications is increased 10 to 1000. It is also interesting to note that the y-axis intercepts of these segments varies from 0.16 at 10 cycles to -0.03 at 500 cycles, indicating that this segment may be defined as a straight line passing through the origin with the slope increasing slightly with additional load applications.

The intercept of the two segments, i.e., maximum desirable fines content, increases from 13.6% fines at N = 100 to 15.5% at N = 200 and remains at this point with continued application of load. Thus it would appear that a fines content of about 15% is a point of transition from a relatively stable material to one that rapidly deforms with an increasing number of load applications. The specimen having 16% fines underwent axial strains much less than the other specimens in this range at all levels of load applications. Whether this may be attributed to sample
variation or that the sample is at an ideal gradation for this material cannot be concluded from the data available. The Garner series however did not exhibit this tendency at the inflection point, suggesting that it is a result of sample variation.

Figures 17-21 are for the Garner crushed stone with an applied deviator stress of 135 psi for the same number of load applications as Figures 12-16. It may be noted that there is somewhat more variation than there was with the Bedford material. This variation is undoubtedly due to the much higher stress level used for this series, possibly making the effect of other variables such as degree of saturation, density, etc., more pronounced.

It appears that at this stress level the Garner crushed stone follows much the same pattern as the Bedford. The slope of the straight line segment above the maximum desirable fines content increases considerably with additional applications of load, going from 0.201 at \( N = 10 \) to 7.47 at \( N = 1000 \), by which time six of the eleven tests having fines contents of 9.5% or greater had been terminated. Of the five having less than 15% axial strain after 1000 applications of load, four have fine contents ranging from 9.5% to 9.8%. As with the Bedford crushed stone, the point of intersection of the linear segments increased slightly with increased applications of load, going from 7.3% at 10 cycles to 8.8% at 100 cycles, and then remaining relatively constant as number of loadings increased. However, the segment for fines contents less than the intercept did not follow the pattern established by the Bedford
Figure 17. Axial strain after 10 load applications vs. fines content for Garner crushed stone.
Garner Crushed Stone

$\sigma_1 - \sigma_3 = 135$ psi

$\epsilon = 0.779 F - 4.63$
$\rho = 0.949$

$\epsilon = 0.059 F + 1.72$
$\rho = 0.883$ 8.8\%

Figure 18. Axial strain after 100 load applications vs. fines content for Garner crushed stone.
Garner Crushed Stone

\( \sigma_1 - \sigma_3 = 135 \text{ psi} \)

\[ \varepsilon = 1.28 F - 8.93 \]

\[ r = 0.948 \]

\[ \varepsilon = 0.044 F + 2.26 \]

\[ r = 0.919 \]

Figure 19. Axial strain after 200 load applications vs. fines content for Garner crushed stone.
Figure 20. Axial strain after 500 load applications vs. fines content for Garner crushed stone.
Garner Crushed Stone

\[ \sigma_1 - \sigma_3 = 135 \text{ psi} \]

\[ \epsilon = 7.47F - 61.5 \]

\[ r = \]

\[ \epsilon = 0.035F + 5.04 \]

\[ r = \]

Figure 21. Axial strain after 1000 load applications vs. fines content for Garner crushed stone.
crushed stone. The slope of this segment remained relatively constant while the intercept increased at a uniform rate. Perhaps due to the high level of stress application a condition of progressive failure developed, whereas the Bedford crushed stone at a lower level of stress tended to reach a condition of equilibrium. Since the slope is almost flat in this segment it would appear that any variations in amount of -200 material would have little affect on the rate of failure as long as the total fines content does not exceed approximately 8.5%.

The variation in the axial strain-percent fines relationship with increased application of the Garner crushed stone at a stress ratio of 12.3 is shown in Figure 22. Since all of the four tests in the series have fines contents in excess of the maximum desirable fines content, no inferences can be made as to the change of the lower segment. The linearity of the four tests is readily apparent as with the other series, the slope of the line increasing markedly as the number of load applications is increased.

A similar plot for the Garner series with an applied stress ratio of 11.1 is shown in Figure 23. With the exception of the relationship at N = 10 the intercept appears to remain relatively constant at about 10.6% fines for all levels of load applications. Above the intercept the slopes tend to increase with additional applications of load in much the same manner as the other series. Below the intercept, the slope, though somewhat larger than the series having an applied stress ratio of 13.5, remains relatively constant for all number of load applications. The intercept of these segments, i.e. axial strain at % -200
Garner Crushed Stone

\[ \sigma_1 - \sigma_3 = 123 \text{ psi} \]

( ) No. of load applications

\[ \epsilon = 5.67 F - 50.4 \]
\[ r = 0.997 \]

\[ \epsilon = 1.97 F - 15.0 \]
\[ r = 0.932 \]

\[ \epsilon = 1.65 F - 13.3 \]
\[ r = 0.992 \]

\[ \epsilon = 1.10 F - 8.4 \]
\[ r = 0.991 \]

\[ \epsilon = 0.38 F - 2.3 \]
\[ r = 0.991 \]

Figure 22. Axial strain vs. fines content for Garner crushed stone.
Garner Crushed Stone

\[ \sigma_1 - \sigma_3 = 123 \text{ psi} \]

( ) No. of load applications

Figure 23. Axial strain vs. fines content for Garner crushed stone.
equal 0, increases with continued loading.

The slope of each segment in Figures 12 to 23 serves as an indicator of the influence that variations in fine content will have on axial strain. It is readily apparent from all figures that above the maximum desirable fines content, the magnitude of the slope increases with additional applications of load. This means that the rate of failure increases rapidly as the amount of fines present is increased. Figure 24 shows the manner in which the slope changes with increased application of load for each of the stress levels.

A study of this relationship reveals several important facets. First, for the segments below the maximum desirable fines content the slope appears to be independent of the number of load applications, meaning that rate of axial strain in this region is independent of the amount of fines present. Total axial strain at any time is greater at the higher fine contents, but it would appear that this additional strain occurred during the first few load applications. This independency from fines content for rate of axial strain exists for all three stress levels.

Above the maximum desirable fines content an entirely different relationship exists. For all stress levels, the slope of the segments increases almost uniformly with increased number of stress applications, indicating that at the higher fines contents, the rate of failure, i.e. amount of axial strain per cycle, increased considerably with increased amount of fines. It is quite evident that for a given stress level, rate of axial strain is greatly dependent upon the amount of fines present. Also, of interest is the fact that the slopes of the segments are
Figure 24. Change in slope of linear segments of axial strain vs. fines content with increasing load applications.
considerably reduced at the higher stress levels. At 500 cycles of load the slope of axial strain vs. % - #200 for the Bedford crushed stone subjected to a cyclic deviator stress of 55.7 psi is almost twice that of the Garner crushed stone at a deviator stress of 135 psi. The Garner crushed stone specimen tested at intermediate levels of stress levels, i.e. deviator stresses of 111 psi and 123 psi had slopes between the above two. Reasons for this type of reaction will be discussed later.

Axial Strain vs. Stress Level

The pronounced influence of fines on the response of granular material subjected to cyclic loading causes difficulty in evaluating the effects of other variables. Preparation of uniform specimens, i.e. having equal amounts of fines, is almost impossible due to natural variation of fines of the material prior to compaction, and amount of degradation occurring during compaction. At higher fines contents, variations of a tenth of one percent may result in a totally different response, and control of the fines content to this degree is not possible.

In order to eliminate fines content as a variable and to study only the effect of level of applied deviator stress, it is possible to go to Figures 12 to 23 and for a fixed fines content determine the amount of axial strain produced by 10, 100, 200, 500 and 1000 load applications at each of the stress levels. By doing this at each stress level one may generate deformation curves for each stress level, simulating tests at identical fines contents. These deformation curves at fixed fines contents are shown in Figures 25 to 30.
Figure 25. Axial strain vs. number of load applications curves for crushed stone containing 4% fines.
Figure 26. Axial strain vs. number of load applications curves for crushed stone containing 8% fines.
Figure 27. Axial strain vs. number of load applications curves for crushed stone containing 10% fines.
Figure 28. Axial strain vs. number of load applications curves for crushed stone containing 12% fines.
Figure 29. Axial strain vs. number of load applications curves for crushed stone containing 14% fines.
Figure 30. Axial strain vs. number of load applications curves for crushed stone containing 18% fines.
The influence of fines present in a granular material appears to be so pronounced that slight variations in fines content tends to alter the response of the material to cyclic loading to a much greater extent than does application of different stress levels. For example at 1000 cycles of load with 10% fines present (Figure 27), an increase in applied deviator stress from 111 psi to 123 psi results in an increase in axial strain of 2.6% whereas for material subjected to a deviator stress of 111 psi only a 2% increase in fines from 10% to 12% results in an increase of axial strain of 7.6%, as shown in Figures 27 and 28.

Fines contents below the maximum desirable fines content may be seen to have deformation vs. number of load repetitions curves similar to those defined by Larew and Leonards (1962) for stress conditions below the critical deviator stress in that they have continually decreasing slopes. Stress levels above the maximum desirable fines content have deformation curves similar to those above the critical deviator stress in that curves tend to turn concave upward.
DISCUSSION

It has been shown in the previous section that the amount of minus No. 200 sieve material present in a crushed limestone base course mixture greatly influences its response to repeated loading. The manner of response appears to be independent of material type and stress level, and the degree of stability is dependent almost entirely on the amount of -#200 material present.

The profound effect of fines can reasonably be explained by two mechanisms. First, an increase in the amount of -#200 material greatly reduces the permeability of the material, which in turn causes a decreased ability to dissipate pore water pressures developed internally when the specimen is subjected to a deviatoric stress. This in turn reduces the shearing resistance of the material. Even though drainage was allowed during the test, the rate of change in stress may be so rapid that little dissipation of pore water pressure can occur.

A second hypothesis is that low contents of fines may be contained primarily in the voids between the coarse aggregate particles, such that good intergranular contact exists between the coarse particles. An increase in the amount of fines results in filling the voids to cause separation of coarse aggregate by the fines. An ultimate condition would be where the coarse aggregate are totally suspended in a matrix of -#200 material. Shearing then would occur within the matrix of fines.

The remainder of this section will be an examination of these two hypothetical mechanisms.
Influence of Pore Pressures on Rate of Failure

Even though drainage was allowed during shear, pore pressures do develop within the specimen when the load is applied, due to the insufficient time for complete dissipation. Lack of sufficient time for pore pressure dissipation may be shown by the time required for consolidation to occur under the 10 psi lateral pressure. Even at low fines contents, i.e., high permeability, a minimum of fifteen minutes was required for complete consolidation. Since pore pressure dissipation both under applied deviator stress and lateral confining pressure requires flow of the pore fluid, it is quite evident that pore pressures can exist within the specimen during cyclic loading even though drainage is allowed. Measurement of pore pressures within the specimen through use of pore pressure needles or similar devices is not practical with a granular material; thus no direct means of measurement of pore pressure within the specimen exists.

The relative magnitude of internal pore pressures may however be inferred by other test parameters. The magnitude of pore pressures produced in a soil compressed under load is directly related to the degree of saturation, the higher the degree of saturation the greater the magnitude of the pore pressures. Volume change of the material also serves as indicator of the relative magnitude of the internal pore pressures. These two parameters will therefore be used to evaluate the influence of pore pressures on rate of failure.

The Bedford specimen underwent a volume decrease during loading,
with the largest decreases in specimens having the lowest fines contents. That is, higher fines resulted in decreased amounts of volumetric strain until at the highest fine contents investigated, the test was almost a constant-volume test. The degree of saturation at the end of the loading phase was determined as follows for each specimen: The dry weight was calculated from the wet weight and moisture content at the time of molding, the volume of the specimen was taken at the time of the last load application, and moisture content was that determined after the test. The calculated degree of saturation will be referred to as the final degree of saturation, $S_f$.

A plot of $S_f$ vs. percent -#200 material for the Bedford crushed stone shows that the final degree of saturation increases to 0.90 with increased fines up to the maximum desirable fine content, 15%. Above this fines content the rate of change decreases, with $S_f$ varying from 0.9 to 1.0 as the fines content changes from 15% to 28%. This relationship is in good agreement because as $S_f$ approaches 1.0 the total shear strength of the material should decrease considerably, resulting in a more rapid rate of failure.

The Garner crushed stone at an applied stress ratio of 13.1 undergoes dilation during initial applications of load, which will cause a negative pore pressure. However, dilatancy is greatest at the lower fines contents and tends to decrease with increased fines contents, which will cause pore pressure to become more positive. Also after the initial cycles dilation ceases, and additional cycles cause very little change. As with the Bedford crushed stone, Garner specimens having fines contents
Figure 31. Degree of saturation vs. fines content.
Garner Crushed Stone

\[ \sigma_1 - \sigma_3 = 135 \text{ psi} \]

After consolidation

\[ S = 0.035 F + 0.32 \]
\[ r = 0.77 \]

After loading phase

\[ S = 0.0066 F + 0.50 \]
\[ r = 0.68 \]

Figure 32. Degree of saturation vs. fines content.
in excess of the maximum desirable fines content tend to fail at nearly constant volume.

The tendency for the Garner stone to dilate at this stress level is reflected in the degree of saturation existing at the time the loading phase is terminated. As shown in Figure 32, the degree of saturation at the end of the test is related to amount of fines present but the magnitude of $S_f$ is far less than for the Bedford material at the lower stress level. With the exception of two tests all have $S_f$ less than 0.85, the degree of saturation prior to loading for specimens having over 8% fines ranged from 0.90 to 1.0, with most having a degree of saturation of near 0.95. The effect of cyclic loading was therefore a decrease in the degree of saturation due to dilation of the material under stress.

Assessing the total effect of pore water pressure is quite difficult because of the existence of two phases or components of pore water pressure produced by repetitive loading. The first would be the pressure present at the time of zero deviatoric stress, i.e., the unloaded portion of the repetitive cycle. This may be referred to as the base pressure, and would be a result of the non-elastic volumetric strain produced by repeated applications of load. Ideally this base pressure should equal zero since drainage was allowed. Specimen having low fines contents probably had base pressure near zero due to the high permeability, whereas decreased permeability due to increased fines undoubtedly resulted in the development of some base pressure within the specimens having higher fines contents. The magnitude of the base pressure cannot be directly
ascertained from the test data, but the volumetric strain may be used to indicate whether this pressure is positive or negative.

The second component of pore pressure is that associated with resilient volumetric strain produced by application of the cyclic deviator stress. This transient pressure is undoubtedly more proportional to volumetric strain than the base pressure, due to the rate at which strain changes. Even with the higher permeabilities at low fines contents, the rate of change is so rapid that very little pressure dissipation will occur. Thus the elastic volumetric strain produced by each load application tends to indicate both the magnitude and type of transient pressure present within the specimen.

**Base Pore Pressure**

Plots of volumetric strain vs. axial strain for each of the four stress levels are shown in Figures 33 to 37. Average fines contents for each specimen are shown in parentheses. Positive values of volumetric strain denote dilation or volume increases, whereas negative volumetric strains signify total volume decreases. All curves begin with the tenth application of load.

A cyclic stress ratio of 13.5 resulted in a condition of dilation for the Garner crushed stone as shown in Figure 33. Higher fines contents tended to reduce dilatancy, with the specimens having fines contents in excess of 12% shearing at almost constant volume. A volume increase will result in negative pore pressure and therefore increase the stability of the material. This would imply that any decrease in permeability from
Garner Crushed Stone

$\sigma_1 - \sigma_3 = 135 \text{ psi}$

( ) Percent Passing
#200 Sieve

Figure 33. Volumetric strain vs. axial strain
Garner Crushed Stone

$\sigma_1 - \sigma_3 = 123$ psi

( ) Percent Passing #200 Sieve

Figure 34. Volumetric strain vs. axial strain.
Garner Crushed Stone

\[ \sigma_1 - \sigma_3 = 111 \text{ psi} \]

( ) Percent Passing #200 Sieve

Figure 35. Volumetric strain vs. axial strain.
increased fines should increase the negative base pore water pressure within the specimen, and thereby increase the resistance to shear. However, it does not appear that this is true because the rate of failure increases rapidly with increased fines.

The four Garner specimens subjected to a cyclic stress ratio of 12.3 had fines contents, all ranging from 9.7% to 10.9%, in excess of the maximum desirable fines content. Volumetric strain for these specimens is shown in Figure 34 and follows the same pattern established in the previous figure in that volume increases uniformly with additional axial strain and should therefore cause negative pore pressures within the specimen.

Reduction of the applied stress ratio to 11.1 causes a reduction in the magnitude of the volumetric strain, (Figure 35). This series has a wider range in fines contents than the previous series but tends to exhibit little difference as far as the volumetric strain. All four specimens dilated slightly during the first 10 cycles of load application and then maintained at nearly constant volume as loading continued.

The lowest stress ratio used for the Garner crushed stone was 9.15, volumetric strain vs. axial strain for this series being shown in Figure 36. This lower cyclic stress ratio resulted in a major change in the volumetric strain of the material in that the specimen having 12.3% fines reached a constant volume condition whereas the specimen having 10.4% fines content has continued volume decrease. This is in the same pattern as the Bedford crushed stone subjected to a cyclic stress ratio of 5.57 (Figure 37). Specimens having fines content below the maximum
Figure 36. Volumetric strain vs. axial strain.
Bedford Crushed Stone

\[ \sigma_1 - \sigma_3 = 55.7 \text{ psi} \]

( ) Percent Passing #200 Sieve

Figure 37. Volumetric strain vs. axial strain
desirable fines content have continually decreasing volumes or continued densification, whereas specimens having fines contents in excess of the maximum desirable fines content have little volume change occurring and could almost be considered to be constant volume tests.

As previously mentioned, the magnitude of the base pore pressure is a function of degree of saturation and volumetric strain which control the generation of pore water pressures, and permeability which governs the rate of dissipation. At the higher stress levels the degree of saturation decreases as the specimen is stressed due to dilation of the material, resulting in negative pore pressures. Increasing fines to decrease the rate of dissipation tends only to decrease the amount of volumetric strain. In general specimens having fines contents in excess of the maximum desirable fines content underwent little volume change as loading progressed. Dilation occurred initially at the higher stress levels but had little affect in reducing the rate of failure. Therefore, base pore pressure does not appear to be a contributing factor in the mechanism of failure for granular material subjected to cyclic loading.

Transient Pore Pressure

For this discussion the term transient pore pressure is intended to include that portion of the total pore pressure resulting from the resilient volumetric strain occurring during each load application. Due to the short time that this pressure exists it is doubtful that dissipation of pressure will occur even at lower fines contents. The magnitude of the pressure should therefore be proportional to the amount of
resilient volumetric strain produced by each load application.

The total volume change occurring during one load cycle ranged from 0.01 cu. in. to 0.23 cu. in., giving volumetric strains from 0.01\% to 0.22\%. Since the magnitude of resilient volumetric change is quite small it is not possible to quantitatively analyze the results; however several characteristics are consistent through all of the tests.

First, the resilient volumetric strain remains almost constant throughout each test for both crushed stones at all stress levels and fines contents. That is, although the magnitude of resilient volumetric strain varied with fines content and stress level, it remained relatively constant during a given test. A change in the rate of failure or number of load applications appeared to have no affect within the limits of precision of the volume change apparatus used. Thus the transient pore pressure resulting from the resilient volumetric strain is nearly constant throughout a given test, and instability of the material may not be attributed to a sudden change in the magnitude of transient pore pressures.

The second characteristic is that for all tests the resilient volume change was positive, i.e., a volume increase. As previously mentioned, positive volume change results in the development of negative pore pressures and hence an increase in the effective lateral pressure. Transient pore pressure should therefore act to increase rather than decrease the stability of the material, and should not be a contributing factor to failure of the material.
Thirdly, the magnitude of the resilient volume change and hence transient pore pressure depends on the amount of fines present within the specimen. Specimens having fines contents less than the maximum desirable fines content all had resilient volumetric strains of 0.07% or less, whereas specimens having fines contents in excess of the maximum desirable fines content experienced much greater volumetric strains and hence negative pore pressure. For the Bedford material at a stress ratio of 5.57, an increase in fines from 16.0% to 18.4% resulted in an increase in resilient volumetric strain from 0.07% to 0.22%. The Garner crushed stone had essentially the same response at all stress ratios.

In summary, transient pore pressure cannot be a contributing factor to the mechanism of failure for the two crushed stones used in this study, since the magnitude of the transient pressure remains essentially constant throughout the test even though the rate of failure changes, and in all cases the transient pore pressure was negative and was of greatest magnitude in those specimens having the highest fines contents, thereby tending to enhance rather than decrease their overall stability. The reverse of this was observed.

Shear Mechanism

We previously concluded that pore water pressures, though having some influence, are not the primary controlling factor in response of material to cyclic loading. The key to the problem therefore may be in the minus #200 material and role it plays within the specimen itself.
Even though the magnitude of permeability has little bearing on the stability of the material, its relationship to the fines content may indicate where the fines are located within the aggregate system.

Barber (1959) investigated the effects of various types and amounts of material passing the No. 200 sieve upon the permeability of graded aggregate. The aggregate used was a Potomac River sand and gravel, graded between the 3/4 in. and No. 200 sieve. Figure 38 is a plot of the tabulated data given by the authors for the change in coefficient of permeability of the graded aggregate as increased amounts of limestone dust were added. Addition of limestone dust up to 10% resulted in a marked decrease in the permeability of the aggregate, indicating that the fines were filling the voids between the coarse aggregate. Increasing the fines content above 10% caused little additional decrease in permeability, since voids in the coarse aggregate already had become filled with fines. This also would imply that additional fines in excess of about 10% began displacing the coarse aggregate and reducing the number of interparticle contacts of the coarse aggregate.

The following mechanism therefore appears to explain the response of the granular material to cyclic loading: At fines contents below the maximum desirable fines content, the -#200 material is present in the voids between the coarse aggregate; these voids are not completely filled and the fines are apparently in a loose state. Since sufficient
Permeability of Graded Aggregate with Admixtures of Various Amounts of Limestone Dust
Barber (1959)

Figure 38. Effect of fine content on permeability of granular base course material.
voids exist for the volume of fines present with the material there is good interparticle contact between the coarse aggregate. The fines, being in a relative loose state within the voids between the coarse aggregate, have no role in the overall shear response of the material. This explains why the response of specimen to cyclic loading was almost totally independent of the amount of fines present at the lower fines contents.

We may hypothesize that at the maximum desirable fines content the voids have been essentially filled by the fines, and additional fines result in the separation of the coarse aggregate and subsequent loss of interparticle contact. As the specimen is loaded, shear will therefore occur within the matrix of fines between the coarse aggregate. Increased amounts of fines result in a decreased number of contacts between the coarse aggregate, explaining the linear relationship between axial strain after a certain number of load applications and fines content, shown in Figures 8-23.

If this hypothesized shear mechanism is valid, the calculated void ratio of the coarse fraction should reflect the tendency for aggregate separation at a particular fines content. Figure 39 shows the void ratio of the +200 fraction vs. fine content for the Bedford and Garner samples. All specimens for both materials are included and the void ratios shown are for the specimen after complete consolidation in the triaxial cell but prior to the application of the cyclic deviator stress.

Below the maximum desirable fines content the void ratio for both materials remains relatively constant, although there is a tendency for
Figure 39. Void ratio of +#200 fraction of specimen tested vs.
% passing #200 sieve.
void ratio to decrease slightly with increased fines up to the maximum desirable fines content. This increase of fines must aid in densification of the coarse aggregate during compaction, but the fines are only present within the voids between the coarse aggregate, and a condition of intimate intergranular contact continues to exist. As the fines content increases above the maximum desirable fines content, the void ratio begins to increase with additional fines, indicating that an increasing number of the coarse particles are being separated by fines and confirming the hypothesis.

The curves through the points above the transition points are computed curves for an ideal granular material having all voids filled with fines having a dry density of 120 lb/ft$^3$. The slope of the curve for the ideal material is in close agreement with the actual materials. The difference in intercepts of the extended curves for the Bedford and Garner crushed stones is indicative of a difference in compactability of the two stones, the Bedford being less compactible.

The point where void ratio begins to deviate from the ideal slope should define the maximum desirable fines content since above this point separation of coarse aggregate exists, and below it the fines are present in the coarse aggregate voids. This transition point is well defined for the Garner crushed stone at a fines content of from 9 to 9.5%. Due to a lack of specimens in this range, the Bedford material is not as well defined but would appear to be within the range of from 12.5 to 14%. The ranges for both materials agree quite closely with the maximum desirable fines contents determined by repetitive triaxial compression.
Modulus of Resilience

The modulus of resilience, or deviator stress divided by resilient strain, is related to fines content for the Garner crushed stone and is shown in Figure 40. The measurement precision on the modulus is about ± 2000 psi. Since the modulus is generally thought to be independent of applied deviator stress, results from all three stress levels are included.

It is readily apparent that the modulus is directly related to the amount of fines present. Figure 41 shows that a similar relationship exists for the Bedford crushed stone. The relationship is similar for both materials with no inflection point at the maximum desirable fines content. The maximum desirable fines content and resilient modulus are dependent upon two different properties of granular material, i.e., plastic axial deformation and resilient axial deformation. As the level of fines content is increased from zero, resilient axial strain increases linearly whereas plastic axial strain remains nearly constant up to the maximum desirable fines content. Above this level the magnitude of plastic axial strain becomes dependent upon fines content.

The slope of the regression lines in Figures 40 and 41 are in very good agreement suggesting that the effect of fines is similar for both materials. Differences in intercepts may be attributed to differences in the physical characteristics of the aggregate, i.e., particle shape, texture and hardness, the Garner stone being more resilient than the Bedford.
Garner Crushed Stone

- $\sigma_1 - \sigma_3 = 135$ psi
- $\sigma_1 - \sigma_3 = 123$ psi
- $\sigma_1 - \sigma_3 = 111$ psi

Figure 40. Effect of fines content on the resilient modulus.

$$Mr = -2100F + 92,380$$
$$r = 0.662$$
Bedford Crushed Stone

\[ \sigma_1 - \sigma_3 = \text{55.7 psi} \]

\[ \text{Mr} = -2300 F + 79,000 \]

\[ r = 0.892 \]

Figure 41. Effect of fines content on the resilient modulus.
Total Shear Strength

The fines content of a granular base material has been shown to be a governing factor in response to the material to cyclic loading. It has been hypothesized that an increase of fines results in separation of the coarse aggregate thereby reducing the shearing resistance. Eight supplemental standard triaxial tests were run at a lateral pressure of 10 psi to test this hypothesis.

Four tests were run on each of the crushed stones, and the fines contents measured after the tests. Figure 42 shows the deviator stresses at minimum volume and at maximum stress ratio vs. the fines content of the specimen.

The effect of increased fines on the Garner crushed stone at maximum effective stress ratio is very pronounced, shown by line aa' in Figure 42, where an increase of fines from 9% to 13% resulted in a decrease of the maximum deviator stress of from 186 psi to 93 psi. The Bedford specimens had a rather narrow range of fines contents and give approximately the same stress ratios as the Garner specimens within the same range. Though the number of tests is quite limited, it would appear that an increase of fines results in a drastic reduction in the total shear resistance of a granular base course material.

The effect of fines content on the deviator stress occurring at the condition of minimum volume is much less pronounced, bb' in Figure 42, the deviator stress tending to decrease only slightly with an increase in fines. An explanation for this effect of fines content on static
Figure 42. Deviator stress vs. fines content for standard triaxial test.
shearing resistance is suggested by the work done by Kirkpatrick (1961) and Kirkpatrick (1965), who observed that the stress ratio at the condition of minimum volume was approximately equal to the stress ratio at the ultimate stage of the test where the volume tends to become constant. As a possible interpretation of these findings Kirkpatrick (1961) stated:

...the stresses at the point of volume increase result in the mobilization of the quasi frictional components of strength, as distinct from other components such as dilatancy which are developed after this stage.

It is further stated that the stress ratio at minimum volume defines the angle of internal friction of the material, and increases of stress ratio above this are primarily a result of dilatancy.

In tests of various grain size and gradings of sand and glass beads, Kirkpatrick (1965) concluded that frictional component of strength calculated from the stress ratio at minimum volume was independent of particle size and grading, and dependent primarily on the surface roughness and mineralogy of the granular material. In the present study, the deviator stresses for both materials at a state of minimum volume decreases only slightly with increased fines, indicating that the frictional component is independent of grading. Of primary interest is the close agreement between the two crushed stones, suggesting a similarity of the frictional component.

If the variation in fines content has little effect on the sliding frictional component of shearing strength, the effect must be due to changes in dilatancy. This is substantiated by the volume change characteristics of the material subjected to repeated loading in that increased
fines resulted in reduction of the magnitude of the dilatant volume change. This is reasonable, since dilation and the dilation component of strength for 3/4 inch aggregate when sheared will be quite large compared to values for -200 material. Based on this, the hypothesized shear mechanism appears to be valid in that as fines content increases, increasing amounts of the shear strain occur within the fines not affecting the frictional component by greatly reducing the dilatancy component and thereby reducing the total shearing resistance.

Due to the limited number of tests available it is not possible to ascertain the actual change in dilatancy component of strength over the complete range of fines contents used in the repetitive study. The change that occurs may however be hypothesized from the information available. It was shown in Figure 39 that below a fines content of 9%, the voids in the +200 of the Garner crushed stone were not completely filled with fines. As long as these voids were not filled it seems reasonable that a variation in fines would have little or no effect on the dilatancy component and that the total shear strength should remain fairly constant. Changes in the dilatancy component of strength for a range of fines content of from 9% to 13% have been fairly well defined by data in Figure 42. Increasing the fines content above 13% would undoubtedly result in a further decrease in the dilatancy component until the total strength was almost entirely attributable to the frictional component. This is consistent with the repetitive triaxial test, since above the maximum desirable fines content, the volume change during shear
decreased with increasing fines content until at the highest fines contents a condition of shear at almost constant volume occurred. This suggests that the dilatancy component of strength when the fines are in excess of 13% would be extremely small. The frictional component should be fairly independent of fines content and should remain constant over all ranges of fines contents.

Based on these observations a hypothesized curve showing the change in total strength of the Garner material with increasing fines content is presented in Figure 43. Such a relationship also implies that below the maximum desirable fines content, axial strain shows little dependency on fines content. Similarly, axial strain above the maximum desirable fines content is highly dependent on the fines content.
Figure 43. Hypothesized effect of fines content on total shear strength.
CONCLUSIONS

The effects of variations of fines content on the response of two granular base course materials subjected to repeated triaxial compression tests have been presented. It has been shown that for these materials, fines content is the controlling factor for response of the materials to cyclic loading, having more influence than any of the other material variables. The effects of fines may be summarized by the following observations:

1. There exists for a fixed state of stress a maximum desirable fines content below which a variation in fines content has little influence on rate of axial strain under cyclic loading. Variations in fines contents above this point greatly influence the rate of axial strain.

2. Below the maximum desirable fines content, fines within a granular material occur loosely packed within the coarse aggregate voids and thus have little effect on the response of the material. Increasing the fines content above this point fills the coarse aggregate voids, separating the coarse aggregate by fines. This separation reduces the number of contacts between the coarse aggregate, allowing shear planes to occur within the matrix of fines rather than at points of aggregate contact.

3. The maximum desirable fines content is inversely proportional to the state of stress, decreasing almost uniformly with increases in the applied stress ratio, \( \sigma_1/\sigma_3 \).

4. The manner of response appears to be independent of limestone type, the degree of stability being almost entirely dependent on the
the amount of -#200 material present.

5. The maximum desirable fines content is almost independent of the number of load applications.

6. Modulus of resilience decreases with increasing fines content, the rate of decrease being independent of type of limestone, although the magnitude of the resilient modulus does depend in part upon the limestone type.

7. The primary influence of increased fines in a granular base course material is to markedly reduce the dilatancy component of shear strength while having little effect on the frictional component.

8. Practical applications: Data in this thesis indicate that a limiting maximum desirable fines content applicable to any stress level does not require repetitive triaxial equipment, but may be quickly found from the fines content required to fill the aggregate voids with -#200 material as shown in Figure 39.

Compaction and determination of fines content of three specimens could easily define the two curve segments shown with the intercept being the maximum desirable fines content. From the specific gravity of the +#200 material and an assumed density for the -#200 material in the voids, the ideal coarse aggregate void ratio vs. percent fines may be generated for a given material. Compaction of one specimen having a fines content in excess of the anticipated intercept would then define the intercept of the ideal curve. Compaction of two specimens having fines contents below the anticipated maximum desirable fines content
would define the slope and intercept of the initial segment. The intersection of these segments would then give a close approximation of the maximum desirable fines content for that material. The intercept of the ideal curve indicates compactibility.
The role of fines in granular base course materials has been clarified to a considerable extent and the existence of a maximum desirable fines content has been shown rather conclusively for both crushed stones. Application of these findings to actual design procedure would be greatly benefitted by studies of the following factors:

1. Relation between desirable fines content and applied stress ratio for other crushed stones.

2. Change in total strength of a granular material with increasing fines.

3. Field determination of the amount of degradation occurring during construction and after service.

4. Sampling of base courses that have shown excessive deformation for correlation with lab data.

If fines content is the dominant variable, as has been suggested, a maximum desirable fines content-applied stress relationship could be derived that would be valid for all base course materials, and standards could be set to prevent the use of degradation susceptible stone.


ACKNOWLEDGMENTS

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Last and most of all I wish to thank my wife, Karolyn, whose love and devotion provided the incentive to complete this work.
Table 5. Summary of repetitive triaxial test data for Garner specimen

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<th>Test no.</th>
<th>% passing 200 sieve</th>
<th>Moisture content</th>
<th>Dry density lb/ft³</th>
<th>Axial strain; % cycles</th>
<th>Modulus of resilience</th>
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<td>Moisture content %</td>
<td>Dry density lb/ft³</td>
<td>Axial strain, % cycles</td>
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