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RESIDUAL CHARACTERISTICS OF UNTREATED GRANULAR BASE COURSE AND SUBGRADE SOILS.

THE OHIO STATE UNIVERSITY, PH.D., 1979
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RESIDUAL CHARACTERISTICS OF UNTREATED
GRANULAR BASE COURSE AND SUBGRADE SOILS

DISSERTATION
Presented in Partial Fulfillment of the
Requirements for the Degree Doctor of
Philosophy in the Graduate School of the
Ohio State University

By

Safwan A. Khedr, B.Sc, M.Sc, P.E.

* * * *
The Ohio State University 1979.

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Before all and after all thanks to GOD, for every thing.
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(2) "The Laboratory Verification of a Mechanistic Subgrade Rutting Model", Transportation Research Record # 616, 1976.

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(4) "Rutting in Asphalt Concrete", Paper Accepted for Publication by The Highway Research Board, 1979.

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I INTRODUCTION

1.1 Definition

The expression, "residual characteristics," describes the pattern of permanent deformation accumulation with dynamic load history (dynamic creep function), under various conditions, and for different pavement components. These characteristics depend mainly on the nature of the material under investigation — whether it behaves as elasto-plastic, viscoelastic, or otherwise. This phenomenon is called rutting when applied to a flexible pavement system.

1.2 Problem Statement

When a flexible pavement is subjected to wheel loads, it undergoes both resilient (recoverable) and residual (irrecoverable) deformations. The resilient deformation is directly related to the fatigue cracking of the pavement's surface layer. On the other hand, the accumulative residual deformation leads to pavement's rutting. Both criteria affect the pavement safety and serviceability. This is shown by the performance equation developed by Carey et al (10) to calculate present serviceability index (PSI) for flexible pavements. The equation, as developed in connection with the AASHO road test, is,

\[ \text{PSI} = 5.03 - 1.91 \log \left(1 + SV\right) - 1.38 RD^2 - 0.01 \sqrt{C + P} \quad (1.1) \]

Where:
Equation (1.1) states that rutting depth and surface cracking, which includes fatigue cracking, are two important factors that should be thoroughly considered when evaluating a pavement performance.

A pavement is considered to have failed when the deformation of its components is sufficiently large to cause an intolerably uneven riding surface or cracking of the surfacing material. These two phases are, in fact, interpretations of accumulative permanent deformation (rutting) and recoverable pavement deflection, which is the cause of fatigue cracking and failure of the surface resulting from repeated stress over a prolonged period of time. Modern methods for structural design of flexible pavements should take these two aspects of pavement behavior into consideration. Moreover, the recent approaches to pavement rehabilitation are based mainly on measuring the pavement deflection and correlating these measurements to the pavements' conditions.

Therefore, it is important to be able to analyze the pavement
response and predict the stress-strain patterns, and consequently deformations and deflections, throughout the structure. Several techniques can be employed to accomplish this prediction, e.g. elastic layered solutions, viscoelastic layered solution, finite element solution etc. However, when using any of these techniques, the proper parameters of the considered materials should be identified and quantified. Such a characterization should be carried out in testing conditions similar to those expected in the field.

It is also essential to establish a prediction model for permanent deformation in the pavement (rutting) to be incorporated in any modern pavement design or evaluation methodologies. Such a prediction model should be based on theoretical and experimental bases verified with actual field problems.

The flexible pavement is actually composed of bituminous asphalt concrete layer(s), unbound aggregate base and subbase layer(s) and compacted subgrade soil. Each of these constituent material must be investigated and characterized in accordance with the selected approach.

The influence of the aggregate base course and subgrade soil on the overall performance of the flexible pavement has been established (3), (14) & (27). These foundation layers have significant effect on the resilient deflection as well as residual deformation of the pavement system. The degree of significance depends on the pavement design structure and environmental conditions.
The response of these materials under traffic - simulated dynamic loading is different from that under static loads. Therefore in order to characterize them for the evaluation of pavement response under traffic loads, they should be tested using stresses of the magnitude expected in service, and these stresses should be applied dynamically. This fact had been recognized by investigators since 1958, (16). Several researches have reported the results obtained from such tests on pavement materials (9), (20), (22), (27), and others.

In these tests efforts were directed toward resilient and/or residual characterization of the materials. Three shortcomings were observed in these studies. Firstly, there have been no reported solid and complete constitutive equations that describe permanent deformation in untreated aggregate bases and subgrade soils during the pavement life. Secondly, to date the residual characteristics of untreated granular material have not been investigated in experimentation in which the confining pressure was varied simultaneously with the axial pressure on such a time scale as to accurately simulate the effect of moving wheel load. Previous investigations have approximated the maximum level of radial stresses to be found at various locations in a pavement system, but the effects of time dependent variations in these stresses as they reach these maxima on the residual properties of granular materials have not been explored. Lastly, there was no sound theoretical basis presented to describe the residual behavior in pavement materials with propagated wheel load.
1.3. Background to the study

A recent nationwide state questionnaire, conducted by AASHTO operations subcommittee has shown that rutting is the most frequently occurring type of distress, followed closely by cracking, on interstate and primary highways of the flexible pavement type in the U.S. (12). Table 1.1 shows a summary of combined flexible pavement distress types. The state of Ohio reported that longitudinal cracking is the most prevalent type of distress for flexible pavements. Rutting is one of the major factors responsible for longitudinal cracks in flexible pavements, since these cracks usually occur at the edges of the rutting ditches due to tensile stresses induced in the surface layer at these spots.

The effect of rutting on the pavement serviceability becomes more severe when the average total rut depth in the wheel path exceeds a critical depth of one inch. This is reflected in the performance equation (1.1). Considering the impact of rut depth on the serviceability of the road, it could be noted that many roads reach the critical value, PSI = 1.5, i.e., a virtual structure failure, when the mean rutting depth reaches 0.6 to 0.7 inches. Moreover, rut ditches along the wheel path can be a potential hazard to road safety.

In the United Kingdom, the performance of flexible pavements is judged primarily by the development of permanent surface deformation. Failure criteria defining the need for major structural repairs are based on either absolute levels of deformation or on the depth of rutting produced in the wheel tracks by traffic. A principal requirement,
<table>
<thead>
<tr>
<th>Distress Type</th>
<th>Interstate</th>
<th>Primary</th>
<th>Secondary</th>
<th>Farm-Market</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>No. a</td>
<td>%</td>
<td>No. a</td>
<td>%</td>
</tr>
<tr>
<td>Longitudinal Crack</td>
<td>13</td>
<td>14.9</td>
<td>10</td>
<td>11.6</td>
</tr>
<tr>
<td>Alligator Crack</td>
<td>3</td>
<td>3.4</td>
<td>7</td>
<td>8.1</td>
</tr>
<tr>
<td>Multiple Crack</td>
<td>15</td>
<td>17.2</td>
<td>13</td>
<td>15.2</td>
</tr>
<tr>
<td>Transverse Crack</td>
<td>12</td>
<td>13.8</td>
<td>14</td>
<td>16.2</td>
</tr>
<tr>
<td>Raveling</td>
<td>5</td>
<td>5.7</td>
<td>5</td>
<td>5.8</td>
</tr>
<tr>
<td>Rutting</td>
<td>20</td>
<td>23.2</td>
<td>23</td>
<td>26.7</td>
</tr>
<tr>
<td>Flushing</td>
<td>3</td>
<td>3.4</td>
<td>2</td>
<td>2.4</td>
</tr>
<tr>
<td>Roughness</td>
<td>3</td>
<td>3.4</td>
<td>4</td>
<td>4.6</td>
</tr>
<tr>
<td>Patching</td>
<td>10</td>
<td>11.5</td>
<td>4</td>
<td>4.6</td>
</tr>
<tr>
<td>Base Failure</td>
<td>0</td>
<td>0.0</td>
<td>2</td>
<td>2.4</td>
</tr>
<tr>
<td>Corrugations</td>
<td>0</td>
<td>0.0</td>
<td>0</td>
<td>0.0</td>
</tr>
<tr>
<td>Shrinkage</td>
<td>2</td>
<td>2.3</td>
<td>1</td>
<td>1.2</td>
</tr>
<tr>
<td>Pot Hole</td>
<td>1</td>
<td>1.2</td>
<td>1</td>
<td>1.4</td>
</tr>
</tbody>
</table>

| Totals               | 87 | 100.0 | 74 | 100.0 | 79 | 100.0 | 54 | 100.0 |

*Number of states naming the indicated distress type as either the most prevalent or second most prevalent.*
therefore, of any pavement design method for use with existing road materials is to predict, with reasonable accuracy, the surface behavior of roads under the expected traffic loading and climatological conditions (11).

While rutting and fatigue are two separate modes of distress, there is an interrelation between them. Fatigue in asphalt concrete will create fatigue cracking throughout its mass. This weakening process of the top relief layer will eventually cause higher stress in the underlying layers and stress concentration in the top layer itself. These higher stress levels will result in more permanent deformation in the pavement components. Meanwhile, rutting is usually accompanied by longitudinal cracks along the side of the depressions. This cracking is a potential location for initiation of fatigue crack propagation that will essentially lead to a shorter fatigue life. The two phenomena are, therefore, interrelated, so that at a certain critical point, they may lead to progressive failure of the pavement system.

Since the total pavement rutting is the sum of cumulative permanent deformations of the pavement's components, the subgrade and base course contribute to the total pavement deformation. The subgrade rutting is also environmentally-dependent. In the wet seasons, when the subgrade is at or close to saturation conditions, its permanent deformation amounts to a considerable portion of the total pavement rutting. However, it also depends on the thickness and properties of the top layer. Figures 1.1 and 1.2 present this finding as reported by Brown et al. (13)
Figure 1.1 Variation of Permanent Strain With Depth  ref. 13
Figure 1.2 Variation of Permanent Vertical Strain with Depth
ref. 13
when full scale tests were conducted on three different pavement systems. The contribution of the subgrade is higher for thinner bituminous layers, as is well-demonstrated by comparing pavements #1 and 2 in the above-mentioned figures. However, when comparing pavements #1 and 3, the effect of the material properties and pavement conditions become more of an influence.

Transverse profiles of three of the 1959 trenches at the AASHO test sites are shown in figure 1.3, two from loop 6 and one from loop 4. Those in loop 6 were cut when the serviceability of the pavements was about 1.5, while that in loop 4 was cut when its serviceability was about 0.5 and failure had reached a more advanced stage. These three sections show the effect of the top layer thicknesses on subgrade rutting, and considerable contribution of subgrade and base course to overall rutting.

Progressive accumulative permanent deformation due to cyclic dynamic loading is thought to occur due to the following mechanisms:

(a) Material densification due to the repeated loading. This mechanism is especially important in the case of poorly-compacted layers. In some instances, this mechanism was responsible for more than 50% of the total deformation. However, this phenomenon is less likely to be a significant influence in the case of well-compacted layers.

(b) Preliminary accumulative shear failure due to the fatigue action of shear and tensile stresses.

(c) Lateral dynamic creep distortion, which will result in lateral
Figure 1.3 Transverse profiles, 1959 trench study.
AASHO Road Test, Report 5
movement of the material away from the wheel path (Figure 1.3).

These three mechanisms play a significant role in the forming of rutting in pavements under normal working stresses well below the ultimate strength. The phenomenon considered in this study does not consider other pavement distress manifestations such as:

(a) Surface wear or attrition in the wheel paths which usually occur in heavily-traveled lanes and where studded tires are used.

(b) Plastic shear flow under a few exceptionally excessive loads. This loading will cause very high stresses in a pavement layer far beyond the material's elastic limit.

(c) Creep deformation under long-term or static loads.

(d) Deformation due to expansive (swelling or shrinking) soils, compressible soils (consolidation) and/or frost-susceptible materials.

Past pavement design methods, still being used by some designers, have been primarily involved with the traditional ultimate criteria of soil support parameters such as California Bearing Ratio (CBR), K, soil strength, group index and other indices for subgrade soils and correlation of empirical field and/or laboratory measured parameters of material behavior with field performance. On the other hand, the results of laboratory and field studies of soil and base course response have indicated that there are inherent limitations to such empirical measurements of materials characteristics.

First of all, these measured parameters are not considered to be fundamental material characteristics, and are highly sensitive to the
methods of testing and evaluation. Therefore, difficulties arise when the methods are extrapolated beyond the conditions for which they are originally developed.

Secondly, tests such as the CBR only reflect the ultimate characteristics of the material rather than the non-failure responses to load and deformation. For example, recent studies at Illinois (1) indicated that CBR values are not unique properties of any soil type. In fact, a soil with a known CBR may exhibit a wide range of moduli values depending on its inherent characteristics. This observation further supports the need to develop a modulus-based design strategy rather than using such empirical procedures. This design strategy should involve:

(1) Development of suitable techniques of stress and displacement analysis for pavement systems.
(2) Identification of space and time characteristics of loads and environmental conditions.
(3) Determination of suitable and representative properties of the pavement materials.
(4) Establishment of distress criterion.
(5) Verification of actual performance studies of pavement leading to decision criteria.

Points (3) and (4) above are the main concern of this study. If the elastic theory is to be used in step (1), however, and there is an increasing weight of evidence in the literature that this approach is practically valid, then the resilient characteristics have to be determined in terms
of elastic constants. If the elastic theory approach is adopted in the analysis, then the elastic parameters, in terms of the dynamic modulus \( E \) and dynamic Poisson's ratio, \( \nu \), have to be characterized.

1.4 Objectives and Scope:

The overall objective of this investigation is to develop prediction criteria for the permanent deformation of subgrade soils and untreated granular base course as part of a flexible pavement system. More specifically, the following are the objectives formulated to meet the overall objective:

(1) Derive a sound, theoretically-based approach to describe permanent deformation in these materials with pavement life.

(2) Describe and construct experimental techniques to evaluate the material characteristics under stress conditions that are comparable as much as possible with those expected in real pavements.

(3) Conduct an experimental program to investigate the phenomenon in subgrade soils and untreated granular materials.

(4) Develop constitutive relations that represent the experimental data and have the potential for application to predicting permanent deformations in the actual field.

(5) Present a rutting prediction model that could be applied under different environmental conditions during the pavement life.

(6) Verify the model obtained in (3) and (4) with practical field conditions if possible.
2.1. Literature Survey

The subject of pavement materials characterization is subdivided in the literature into various categories according to the type of material investigated. Asphalt concrete has been the subject of several extensive studies in the last fifteen years, e.g. (13), (28), (29) and others. However, this study is concerned with the evaluation of the untreated base course and subgrade layers. Therefore, the literature review will focus on the recent research conducted on these two materials.

2.1.1 Untreated granular material.

(A) Residual Characteristics

Two approaches have been advocated in considering rutting phenomena in design procedure. In one approach, the vertical elastic compressive strain, due to dynamic wheel loading, at the subgrade surface is to be limited to some tolerable amount associated with a specific number of load repetitions. Table 2.1 summarizes some of the criteria, suggested by different researchers, that can be used in this approach. By controlling the properties of the materials used in the pavement section and ensuring that it is of adequate stiffness and sufficient thickness so that the strain level is not exceeded, permanent deformation can be limited to a value equal to or less than the prescribed amount. This approach suffers serious disadvantages:
**TABLE 2.1**

Limiting Subgrade Strain Criteria (After Yoder and Witczak)

<table>
<thead>
<tr>
<th>Strain parameter*</th>
<th>Original Shell Oil Co.</th>
<th>Revised Shell Oil Co.</th>
<th>Asphalt Institute</th>
<th>Kentucky Highway</th>
</tr>
</thead>
<tbody>
<tr>
<td>ε vs</td>
<td>ε vs</td>
<td>ε vs</td>
<td>ε vs</td>
<td>ε vs</td>
</tr>
<tr>
<td>Type pavement</td>
<td>Highway</td>
<td>Airfield</td>
<td>Airfield</td>
<td>Highway</td>
</tr>
<tr>
<td>Allowable strain</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(N_j = 10)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(10^2)</td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>(10^3)</td>
<td>2700</td>
<td>4500</td>
<td>1904</td>
<td>790</td>
</tr>
<tr>
<td>(10^4)</td>
<td>1680</td>
<td>2700</td>
<td>1646</td>
<td>639</td>
</tr>
<tr>
<td>(10^5)</td>
<td>1050</td>
<td>1700</td>
<td>1508</td>
<td>502</td>
</tr>
<tr>
<td>(10^6)</td>
<td>650</td>
<td>1030</td>
<td>1423</td>
<td>364</td>
</tr>
<tr>
<td>(10^7)</td>
<td>420</td>
<td>650</td>
<td>--</td>
<td>227</td>
</tr>
<tr>
<td>(10^8)</td>
<td>260</td>
<td>400</td>
<td>--</td>
<td>89</td>
</tr>
<tr>
<td>(\infty)</td>
<td>--</td>
<td>--</td>
<td>1060</td>
<td>--</td>
</tr>
<tr>
<td>Effective (E_1), ksi</td>
<td>140 (thin AC)</td>
<td>150</td>
<td>100</td>
<td>480 (33 percent AC)</td>
</tr>
<tr>
<td></td>
<td>200 (thick AC)</td>
<td></td>
<td></td>
<td>300 (100 percent AC)</td>
</tr>
</tbody>
</table>

* \(\epsilon_{vs}\) is maximum compressive subgrade strain, \(10^{-6}\) in./in.
(a) These proposed limiting criteria have been developed for specific ranges of material characteristics, specific moduli and Poisson's ratios. Therefore, when strain limits are used, the same corresponding values for materials stiffness and Poisson's ratio should be used; otherwise, the resulting analyses will have little significance.

(b) These criteria are based on practical and field experience which limit any extrapolations for other cases.

(c) Pavements designed based on this procedure will have sufficient thickness to protect the subgrade soil but not to insure that permanent deformation in the upper pavement layers will not occur. Chou (35) conducted full scale testing on flexible airport pavements. He observed that surface rutting for pavement with thick asphalt concrete layers was much greater than that for the corresponding conventional flexible pavements. At a given performance level, the measured surface rutting of the thicker pavement needed for the heavier load was greater than that of the thinner pavement required for lighter load. The observations and discussions presented by Chou are in direct contrast with this approach. The basic assumption of the approach states that when two pavements are designed for the same performance level (same coverage at failure) the subgrade elastic strain will be the same and the subgrade rutting is very nearly the same. This assumption was found not to be strictly correct.

The second approach is concerned with the estimation of the actual amount of permanent deformation that may occur in the pavement structure. In this approach a constitutive equation is developed to describe the
phenomena through representative repeated triaxial load tests on a domain of samples representing the construction material used at the expected field conditions. The application of such a constitutive equation and pavement structure stress-strain analysis, using an appropriate theory, will lead to rutting estimations in each pavement sublayer. The overall rutting can then be calculated by simple summation of the permanent deformations of all pavement components,

\[ RD = \sum \epsilon_i \cdot h_i \]

where \( RD \) = rut depth
\( \epsilon_i \) = permanent strain in the \( i \)th sublayer
\( h_i \) = thickness of \( i \)th sublayer
\( n \) = number of sublayers

No effort has been made to investigate the residual characteristics of granular materials until very recently. Research in this area, presented below, was conducted through laboratory experimentation on aggregate samples tested in triaxial compression cells. They were subjected to constant confining cell pressure, at stress condition at which the sample was first consolidated. Then at that constant lateral pressure, they were subjected to dynamic axial loading. The permanent deformation was then recorded with the number of axial loading cycles. None of these studies considered the application of time dependent confining pressure that varies simultaneously with the vertical dynamic deviator stress. The testing data were then analyzed versus the influencing factors considered in each study.

Barksdale (30) was first to investigate the plastic deformation of a
variety of granular materials tested in repeated triaxial testing. In his testing, he applied a uniaxial dynamic stress of triangular loading function. Ten different materials and blends were investigated in that study. Each sample was tested to an average of 100,000 load repetitions at constant confining pressures of 3, 5 or 10 psi.

Barksdale noticed that the plastic strain accumulated approximately logarithmically with the number of load applications. However, he did not apply this observation. At a given confining pressure, for small values of deviator stress, plastic strain was almost proportional to the deviator stress and the rate of accumulation of plastic strain tends to decrease as the number of load applications increases. As the deviator stress increases, a critical value is reached beyond which the rate of strain development tends to increase with increasing number of load repetitions. Furthermore, after a relatively large number of load repetitions, the specimen may undergo an unexpected increase in the rate of plastic strain accumulation. Plastic strain was found to be strongly dependent on the confining pressure, undergoing a significant decrease as the confining pressure increases. Elastic strain was also strongly dependent upon the confining pressure, varying in the same pattern as the plastic strain.

When studying the effect of materials type, he found that the plastic strain increased significantly as the percent fines increased, with greater differences occurring at the larger deviator stress levels. The samples compacted at 95% of maximum compaction showed 185% increase in plastic
striae, while those compacted at 105% showed only about a 10% reduction in plastic strain. The soaked samples had 68% increase in its plastic strain over those tested in as-compacted condition, provided that the samples were permitted a free flow of water into and out of the specimen during testing to prevent any build-up of pore water pressure.

Figure (2.1) shows typical rutting curves of Barksdale's study. When plotting the plastic strain-dynamic stress curves at specific load 100,000 repetitions and confining pressure figure (2.2), it was found to be analogous to the stress-strain curves obtained from static tests performed at varying confining pressures. The plastic stress-strain curves exhibit a typical non-linear response similar to those described by Kondner (31) and Duncan et al (32). Barksdale suggested a hyperbolic expression similar to that developed by Duncan in the form,

\[ \varepsilon_a = \frac{[\sigma_1 - \sigma_3]}{k \sigma_3^n} \left( 1 - \left\{ \frac{[\sigma_1 - \sigma_3]}{2[C \cos \phi + \sigma_3 \sin \phi]} \right\} \right) \]  

(2.2)

Where,

\( \varepsilon_a \) = axial strain

\( k \sigma_3^n \) = relationship defining the initial tangent modulus as a function of confining pressure, \((k \& n \text{ are constants})\)

\( C \) = cohesion

\( \phi \) = angle of internal friction

\( R_f \) = a constant relating compressive strength to an asymptotic stress difference.
Figure (2.1) Influence of Number of Load Repetitions and Deviator Stress Ratio on Plastic Strain. (Ref. 30)

Figure (2.2) Influence of Deviator Stress and Confining Pressure on Plastic Strain. (Ref. 30)
He expected that this formula would hold for other types of materials and other numbers of load repetitions.

In applying equation (2.2) for practical estimation of rut depth with performance life, an extensive testing program should be performed to calculate the parameters of the equation at each number of load repetitions. The definition of the modulus as \( k \sigma_3^n \) is not accurate as it relates the dynamic deviator stress \((\sigma_1 - \sigma_3)\) to the permanent plastic strain, not the corresponding elastic strain. Also, equation (2.2) mainly relates the permanent strain (which is a non-failure stress condition) to the ultimate characteristics of the material, namely \( C, \), and \( Rf \).

Barksdale introduced the terms, rut index and rut potential; the former is defined as the sum of the plastic strains occurring at the center of the top and bottom half of the base multiplied by 10,000. The rut potential is a rut index calculated using data that have been extrapolated to a desired number of load applications.

Figure (2.3) shows the susceptibility of different materials and blends for plastic deformation, in the form of axial dynamic stress-plastic strain curves.

Allen (27) conducted a series of experiments on nine samples of granular materials in which he applied both time variables and constant confining pressures. Although his study was not intended to investigate permanent deformations, he made the following general comments. There is an increase in total plastic strain as the specimen density decreases.
Figure 2.3 Summary of plastic stress-strain characteristics at 100,000 cycles and confining pressure of 10 psi. (Ref. 30)
For each density level, the crushed stone specimens (with higher angles of internal friction) experienced the least plastic strains, while the gravel specimens accumulated the greatest plastic strain. Lastly, the non-recoverable deformation associated with the constant confining pressure portion of the tests exceeded that associated with the variable confining pressure portion for every specimen. It should be noted, however, that in Allen's testing each sample was subject to the two types of test at various stress conditions. Therefore, it is the opinion of the author that that comment should not be considered accurate.

A laboratory repeated triaxial test was conducted by the National Crushed Stone Association (NCSA) to study the characteristics of plastic deformation of graded aggregates. Kalcheff (33) reported that the plastic strains are greatly dependent on the degree of consolidation for the same gradation, the amount and type of fines in the gradation, the stress sequence and magnitude, and for some type of fines the moisture content.

Figure (2.4) is an illustration of how density affects plastic strains at the same stress level. Also, figure (2.5) shows the effects of different types of fines on the plastic response of two types of aggregates. The gravel mix shown in the figure with either type of dust had the same elastic properties. Kalcheff thus emphasized that all graded aggregates do not have the same plastic strain responses under the same loading conditions even though their elastic properties and the quantity of fines may be the same.

Similar to Barksdale's observation (30), Kalcheff also noted the
Figure 2.4 Effect of density on the plastic strain accumulations with load application (after Kalcheff 33)

Figure 2.5 Effect of type of fines on the plastic strains of two graded aggregate bases (after Kalcheff 33)
plastic strain accumulated approximately logarithmically with the number of load repetitions.

Brown (34) conducted dynamic testing on well-graded crushed material, applying continuous sinusoidal deviatoric stress (with no rest period between cycles) at constant confining (cell) pressure. He found that under drained conditions, both permanent and resilient strains reach equilibrium values after approximately 10,000 cycles of deviator stress. This finding contradicts Barksdale's and Chou's observations (30), (35). Brown also reported that the equilibrium permanent strain for the drained test was much less than that corresponding to the undrained conditions. He expressed the permanent strain at equilibrium $\varepsilon_p$ in the form

$$\varepsilon_p = 0.01 \left( \frac{q}{\sigma_3} \right)$$

where

$q$ = effective deviator stress

$\sigma_3$ = constant confining pressure

In a recent extension of this work at Nottingham, researchers investigated the influence of loading sequence and that of applying cyclic cell pressure to the same granular material. The limited study of loading sequence showed that the permanent strain was significantly affected. The permanent strain which built up after successive applications of about $10^5$ cycles of gradually increasing level was less than half the value resulting when the highest stress level was applied constantly.

Chou (35) tested sand and gravel subbase and crushed stone base
materials under uniaxial dynamic loading as well as full scale pavements. He suggested using octahedral stresses in the analysis to avoid dealing with tensile radial stresses resulting in granular material sublayer upon analyzing the pavement structure that include such sublayer, using elastic layered solution. However, difficulty was encountered since the octahedral stress ratios computed using linear layered elastic computer programs have values greater than three in most cases, while the ratios for specimens tested in the laboratory have magnitudes less than one. Specimens tested under higher stress ratios experienced excessive deformations leading to complete failure. He concluded that (a) the states of stress existing in the granular layers under aircraft loadings are extremely complicated, which cannot be simply described by constant values of vertical compressive stress $\sigma_1$ and horizontal stress $\sigma_3$; and (b) the response of the granular material to the repeated applications of aircraft loads cannot be simulated by the laboratory repeated load triaxial tests. When he used a nonlinear finite element program, the magnitude of the compressive radial stresses computed was very small, i.e., $\frac{1}{4}$ to 1 Psi.

He pointed out, however, that although the layered elastic program predicts radial stresses incorrectly, the presence of radial tensile stress in granular layers does not seem to affect the predictions of vertical stresses and deflections appreciably. Nevertheless, he applied high stresses that correspond to those experienced in flexible airport pavements. Some testing results are shown in figures (2.6 a and b). Chou did not use these results in permanent deformation computation, bec-
Figure 2.6a Accumulated plastic strains for crushed limestone specimens (ref.35)
<table>
<thead>
<tr>
<th>SPECIMEN</th>
<th>$\sigma_1$</th>
<th>$\sigma_3$</th>
<th>$\sigma_1 - \sigma_3$</th>
<th>$\sigma_{oct}$</th>
<th>$\sigma_{oct}$</th>
<th>$\sigma_{oct}$/$\sigma_{oct}$</th>
</tr>
</thead>
<tbody>
<tr>
<td>3</td>
<td>20</td>
<td>42.5</td>
<td>2.12</td>
<td>34.2</td>
<td>20.0</td>
<td>0.58</td>
</tr>
<tr>
<td>9</td>
<td>10</td>
<td>48.5</td>
<td>2.26</td>
<td>17.5</td>
<td>10.6</td>
<td>0.61</td>
</tr>
<tr>
<td>10</td>
<td>10</td>
<td>48.3</td>
<td>4.85</td>
<td>26.2</td>
<td>22.8</td>
<td>0.87</td>
</tr>
<tr>
<td>12</td>
<td>20</td>
<td>43.0</td>
<td>3.56</td>
<td>21.7</td>
<td>6.5</td>
<td>0.76</td>
</tr>
</tbody>
</table>

Figure 2.6b Accumulated plastic strains for gravelly sand and crushed limestone specimens (ref.35)
ause of the stress pattern limitations.

In this investigation (chapter VI), residual parameters will be related to resilient characteristics of the base course. Therefore, it may be appropriate here to review those studies which deal with the resilient characterization of the base course.

(B) Resilient Characteristics:

Generally, all materials involved in flexible pavement construction are inhomogeneous, anisotropic, non-linear and non-elastic and some of their properties are time dependent and affected by changes of environment such as temperature or moisture content. In order to compute stresses and strains accurately in structures composed of such complex materials, extremely involved and sophisticated theories are necessary. Several approaches and theories have been developed for that purpose. However, there is still limited information on the complete characterization of those materials be incorporated into these techniques of analysis. Nevertheless, linear elastic theory may be modified to deal with non-linear material properties by an iterative process, or non-linear elastic theory solutions seem to be promising (22). There is an increasing weight of evidence that this approach is reasonably valid (3), (13), (26). If this approach is adopted then the residual characteristics in terms of elastic constants such as modulus of elasticity (E) and Poisson's ratio (ν) have to be determined.

It is important that the method of measuring these properties; E and ν
of the materials should provide conditions comparable with those occurring on site. When a rolling wheel load passes over a point in a road structure the various layers are subjected to stress variation as a function of time. The stress pattern in all directions is closest to a haversine stress function of time. While the details of this function will differ between layers and between points with respect to geometrical configurations, the basic pattern is similar everywhere.

The applicability and desirability of using laboratory tests to characterize paving materials has been widely recognized. The aim of such tests should be to reproduce in-site conditions as closely as possible. This requires attention being given to the stress history and condition of the sample as well as to the stress pattern applied during a test. Although none of the various testing techniques presently available is capable of reproducing these stress patterns and combinations completely, the triaxial test has received increasing interest primarily because this apparatus can be programmed to apply repeated stresses of controlled duration, magnitude and function shape so as to simulate the field conditions.

Basically, the concept of resilient response seeks to formulate predictive equations for the resilient parameters—modulus and Poisson's ratio, through the use of repeated-load triaxial tests. By expressing these parameters as functions of the state of stress in the materials, it is possible to account for nonlinear material behavior. The derived functions could then be used to characterize the material in the numerical solution for transient pavement deflection and stress-strain distribution throughout its structure.
Because of the nature of the untreated granular material, i.e. the cohesionless properties, it cannot be tested without confinement. Except for a few researches (27), the confining pressure was kept constant during the dynamic deviatoric loading, which is considered a shortcoming in simulating practical loading conditions.

Dunlap (7) tested crushed limestone specimens at various constant confining pressures under repeated vertical stresses of varying magnitude. He proposed the relation to represent his data,

\[ M_z = k_2 + k_3 \left( \sigma_r + \sigma_\theta \right) \]  \hspace{1cm} (2.4)

Where

- \( M_z \) = modulus of deformation measured in the direction of applied stress, \( \sigma_z = \sigma_z \) / recoverable strain.
- \( k_2 \) = modulus of deformation in the unconfined condition and is analogous to Young's modulus in elastic theory.
- \( \sigma_r \) & \( \sigma_\theta \) = radial and tangential stresses.

For the triaxial test, (2.4) becomes:

\[ M_z = k_2 + 2 k_3 \sigma_r \]  \hspace{1cm} (2.5)

He reported that the deformation constants \( k_2 \) and \( k_3 \) were reduced by increasing the moisture content of the sample and by reducing the density. Also it was stated in that study that sample densification under greater numbers of load repetitions tended to increase the deformation constants. No sound explanation was found for this inverse trend.

Moore et al (36) used the relation,

\[ M_z = k_2 + k_3 \sigma_r \]  \hspace{1cm} (2.6)
to investigate the effects of number of load applications, stress history, and rate of loading on the modulus of deformation, \( M_z \). They found that:

1. the modulus increases as the number of repetitions increases;
2. it is not possible to predict the stiffness of a granular material unless its stress history is known;
3. the effect of rate of loading in the range 200 to 650 psi/sec is small.

Morgan (37) tested sands at constant confining pressures and dynamic axial loads. He concluded:

1. after a certain number of repetitions, permanent deformation remains constant, similar to Brown's conclusion (34);
2. confining pressure is the most significant factor affecting stiffness, while increasing the deviator stress reduces the modulus slightly;
3. Poisson's ratio appeared to be unrelated to any of those factors and ranged between 0.2 to 0.4.

Two models have been suggested to describe the resilient modulus,

\[
M_r = k_1 \sigma_3^{k_2} \tag{2.7}
\]

\[
M_r = k_1^* \theta^{k_2^*} \tag{2.8}
\]

Where

\( M_r \) = resilient modulus

\( k_1, k_1^*, k_2, k_2^* \) = constants from regression analysis

\( \sigma_3 \) = confining pressure, psi

\( \theta \) = sum of principal stresses, \( \sigma_1 + 2 \sigma_3 \)

Hicks (26) tested compacted samples of three types of granular materials using constant confining pressure and dynamic deviatoric stress. Figure (2.7) shows the axial stress-vertical strain-radial strain at specific confining pressure. Hicks defined secant and tangent dynamic
Axial Strain

\[ \varepsilon_n = \frac{f_{r_n}}{f_{v_n}} \]

Repeated Axial Stress

Radial Strain

\[ M_{r_n} = \frac{\sigma_n}{\varepsilon_{v_n}} \]

\[ \varepsilon_{r_n} = \frac{\varepsilon_{r_n}}{\varepsilon_{v_n}} \]

(a) Secant Values

(b) Tangent Values

Figure 7.7 Methods Employed for Computing Resilient Modulus and Poisson's Ratio. (Ref. 26)
modulus as shown in figure (2.7). The former definition can be used in an iterative type of pavement structure analysis, while the latter is applicable with incremental solutions. In the following conclusions by Hicks, the secant values are considered.

Resilient Poisson's ratio was correlated to the principal stress ratio in a third degree polynomial in the form:

\[ \nu = A_0 + A_1 \left( \frac{\sigma_1}{\sigma_3} \right) + A_2 \left( \frac{\sigma_1}{\sigma_3} \right)^2 + A_3 \left( \frac{\sigma_1}{\sigma_3} \right)^3 \]  \hspace{0.5cm} (2.9)

where \( A_0, A_1, A_2, A_3 \) are constants

\( \sigma_1 \) and \( \sigma_3 \) are the principal stresses.

In general, \( \nu \) increased with increasing principal stress ratio. It varied in the range 0.05 to 0.80 in Hicks' data, Figure (2.8).

Allen (27) tested nine samples of three different materials. In his testing, he applied simultaneous time-dependent chamber and deviatoric stresses (variable confining pressure (VCP)) as well as constant chamber and dynamic deviatoric stress (constant confining pressure (CCP)). He adopted the approach of investigating his data according to equations (2.7), (2.8) and (2.9).

In the following discussion, the dynamic (resilient) modulus is defined as follows,

\[ M_r = \frac{\sigma_1 - \sigma_3}{\varepsilon_a} \text{ for (CCP)} \]  \hspace{0.5cm} (2.10)

\[ M_r = \frac{\sigma_1 - 2 \nu \sigma_3}{\varepsilon_a} \text{ for (VCP)} \]  \hspace{0.5cm} (2.11)

where \( \sigma_1 - \sigma_3 \) = deviator stress
\[ \sigma_i = \text{vertical stress} \]
\[ \sigma_3 = \text{lateral pressure} \]
\[ \nu = \text{Resilient Poisson's ratio} \]
\[ \varepsilon_a = \text{axial recoverable strain} \]

* Both Hicks and Allen agreed on the conclusion that the resilient response of the materials tested is independent of stress pulse duration. The stress pulse duration range was between 0.1 and 0.2 seconds.

* As long as the applied stresses were representative of those found in an actual pavement system, Hicks and Allen also agreed that the resilient response of dry and partially saturated granular materials measured after 50 to 100 stress repetitions could be used to properly characterize the behavior of the granular material. However, Allen specified that the sample should be conditioned with several thousands of stress repetitions. They, separately, also stated that the stress sequence applied has insignificant effect on the sample behavior. Hicks found these conclusions applicable to saturated samples as well if conditioned with 1000 to 2000 stress repetitions in a drained condition to avoid any potential for building up the static pore pressure causing reduction in the effective confining pressure.

Nevertheless, Moore et al (36) and others found that the modulus does depend somewhat on stress history. They suggested that the reasons for such an effect are progressive sample densification, particle rearrangement and development and dissipation of pore
Figure (2.8) Secant Poisson's ratio as a function of principal stress ratio. (Ref. 26)

Figure (2.9) DCP test results—$E^* = f(\theta)$. (Ref. 27)
pressures.

Moreover, Hicks' and Allen's conclusions on the effect of material density coincide. They stated that more dense samples have a higher modulus, figure (2.9 a and b). Poisson's ratio did not change significantly with density with consistent variation. But Hicks observed diverse behavior in the case of coarse grading (low percentage of passing sieve # 200) and dry samples, when considering the effect of density.

The influence of material gradation on the resilient modulus was not well defined. However, over the range of $\sigma_3 = 0$ to 10 psi, Hicks reported that $M_r$ of partially crushed aggregate decreased as the fine content increased, while for the crushed aggregate, $M_r$ increased with increasing fine content. In most cases, he found that as the fine content increased, the main value of $\nu$ was reduced.

They (Hicks and Allen) concluded the same finding that the $M_r$ values were higher for material with higher particle angularity, figure (2.9 a&b). Poisson's ratio was slightly lower for such material. Allen, however, reported less sensitivity of this relation.

Similar to all previous and following researchers, Hicks as well as Allen have found that stress level and condition are the most influential factors on the resilient properties of granular materials. The non-linearities in the stress versus strain relationship were apparent to a higher degree in all cases with small values of confining pressure ($\sigma_3$). Increasing $\sigma_3$ is a stiffening factor, figures (2.10), (2.11). Hicks observed slight softening occurred at low axial stress levels, while at higher levels samples exhibited a stiffening type of response. He stated
Figure (2.10) Variation in Secant Modulus With confining Pressure. (Ref. 26)

Figure (2.11) Variation in Secant Poisson's Ratio with Stress Level. (Ref. 26)

Figure (2.12) Variation in Secant Poisson's Ratio with Stress Level (Ref. 26)
that the modulus generally increased with increasing axial stress (or principal stress ratio) for principal stress ratio greater than 2, which directly contradicts Brown's observation (29). Hicks and Allen promoted relations (2.7) and (2.8) to represent their data, figures (2.9), (2.10) & (2.12). However, Allen found that relation (2.8) had higher correlation coefficients in the regression analysis and believed it was more representative than relation (2.7).

In his study of the saturation effect, Hicks reported that increasing moisture content (degree of saturation) of the granular material has an adverse effect on its resilient modulus, figures (2.10), (2.11) & (2.12). It was believed that the development of excess pore water pressure with accompanying reduction in effective confining stress is the reason for such an effect. Poisson's ratio was reported to decrease as the degree of saturation increased.

Allen reported indications of anisotropic behavior that were observed for both the CCP and VCP tests. It appeared that specimens were less stiff in the lateral direction.

(1) CCP test greatly overestimated Poisson's ratio. Since the VCP test yielded values of \( v \) in the range of 0.35 to 0.40, it was felt that conditions of the CCP test are such as to impose greater amounts of volume change on the specimens, as indicated by the computed values of \( v \) consistently in excess of 0.50, see figures (2.13) and (2.14).
Figure 2.1. Poisson's ratio

Figure 2.2. Poisson's ratio
(2) Values of $M_r$ computed from CCP test data exceeded those computed from the VCP tests for most stress levels. The magnitude of the difference was a function of the state of stress and, thus, non-constant. However, this phenomenon was not observed for all specimens. He came to the conclusion that any differences in the predictive equations for $E_r$ derived from the two test procedures exert only minimal influence on such indicators of pavement response as surface deflection, maximum tensile strain in the asphalt concrete layer, and vertical stress and strain in subgrade. Therefore the continued use of the CCP triaxial test as a means of characterizing granular materials is justified, figures (2.15 a & b).

Brown (29) found, through a constant confining stress testing, that increasing the dynamic deviator stress caused a stress softening effect, while increasing the mean normal stress (defined as $q_{m} = \frac{1}{3} q_{m}$ where $q_{m}$ is the mean level of deviator stress $(q) = q/2$) made the samples stiffer. He did not rationalize this opposite effect caused by changes in the mean level $q_{m}$ and the amplitude of the applied deviator stress $q$. The real known stiffening effect of increasing confining pressure $a_{3}$ was observed.

The following relations were suggested in that study to represent the data of resilient behavior:

$$\varepsilon_{r} = 237 \left( \frac{q}{p + 0.25a_{3}} \right)^{1.5} \text{ microstrain} \quad (2.12)$$

$$M_{r} = 4,200 q \left\{ \left( \frac{p}{q} \right) + \frac{1}{4} \left( \frac{a_{3}^{2}}{q} \right) \right\}^{1.5} \quad (2.13)$$
Figure 2a,b: VCP and CCP test results—$E_r = f(\theta)$ model, specimen MD-1.

$a$

Figure 2c,d: VCP and CCP test results—$E_r = f(\theta)$ model, specimen HD-2.

$b$
where

\[ \varepsilon_r = \text{resilient strain} \]

& \[ M_r = \text{resilient modulus} \]

Relation (2.12) is represented in the figures (2.16 a & b). However, his data regressed well with the relation (2.8).

As mentioned previously, the elastic layered solution may have been shown as a satisfactory system of analyzing the pavement system which contain untreated granular base and/or subbase. However, because of the unrealistic tensile radial stresses calculated in these granular layers -- and also of those unrealistic low radial pressures calculated when using elasto-plastic theory in finite element method -- one can realize, in fact, that no model has been established which would describe and count for the behavior of such material as part of a pavement system under real traffic loading. All models in use characterize the granular layers as a continuum. Rather, it should be investigated as an assembly of oriented particles. When heavy wheel loads are applied on the pavement surface, radial tensile stresses will develop at the lower part of the granular layer. The granular material can resist a certain amount of such tensile radial stresses through the interparticle friction forces which are proportional to the normal stresses at the particles' interfaces. The development of this frictional resistance will cause the material to slip (35). Thence, passive pressure due to adjacent overburden will be mobilized and consequently the confining pressure will increase, causing higher strength of more confined material, i.e. higher modulus. The mechanism of this cohesionless material in developing its resistance to exerted stresses is
Figure (2.16a) Normalized Deviator Stress-Mean Normal Stress Relationship (Ref. 29)

Figure (2.16b) Correlation of All Resilient Strain Results (Ref. 29)
completely different from the cohesive materials idealized in the models as continuum. Although it is not in the scope of this study to and develop such a mechanistic analysis, this shortcoming in the present analytical models was raised to help in understanding its results.

2.1.2 Subgrade Soil

(A) Residual Characteristics:

The subgrade soils make considerable contribution to the total pavement rutting specially in the wet seasons and during thawing of frozen soils. The percentage of this contribution depends on pavement structure over the foundation soil, figures (1.1) and (1.2).

The investigation of subgrade soil behavior under dynamic loading was initiated in 1958 by Seed et al. (16) who conducted preliminary studies on clay subgrades. In this laboratory study, the significance of the effects of stress intensity, loading frequency, stress history, thixotropy, and saturation on the soil behavior were emphasized. However, during these tests, no distinction was made between residual and resilient deformation nor was any constitutive relationship proposed to describe the soil deformation.

In 1975, Monismith et al. (15) conducted a series of repeated load tests on fine-grained soils to study the effects of compaction conditions, stress magnitude and stress sequence. They observed that the following formula,
\[
\epsilon_p = A N^b
\]  

(2.14)

where

\[\epsilon_p = \text{permanent strain;} \]
\[N = \text{number of stress applications; and} \]
\[A, b = \text{experimentally determined coefficients} \]

could represent their experimental data. Also, using the concept of hyperbolic stress-strain relationships, they studied an equation of the form,

\[
\sigma_a = \frac{\sigma_p}{l + m \epsilon_p^a} \quad \text{or} \quad \frac{\sigma_p}{\Delta \sigma_a} = l + m \epsilon_p^a
\]  

(2.15)

where

\[\sigma_a = \text{repeated axial stress;} \]
\[\epsilon_p^a = \text{cumulative permanent axial strain at a specific number of stress applications;} \]
\[l, m = \text{experimentally determined coefficients.} \]

Their analysis of data showed good agreement with the proposed equation. However, it should be noted that Equation (2.15) describes stress-strain at a specific number of stress applications. Another equation with different values of the coefficients, \(l\) and \(m\), should be established at each required number of load repetitions throughout the pavement life.

If this equation holds true for all ranges of applied stress, it should be helpful in deducing the permanent deformation at low stresses, where difficulties arise in experimental deformation measurements, from measurements at higher stresses. At such higher stresses, the difficulties in measuring deformations could be minimized.
From their studies of cumulative permanent deformation with different stress sequences, Monismith et al. (15) concluded that when the smaller stresses are applied first, the specimen deforms less. However, their proposed time-hardening and strain-hardening procedures have failed to provide solutions that agree with the experimental results.

In 1976, Barker (38) analyzed data from dynamic triaxial tests as conducted by different researchers, and showed that relationships exist between permanent strain and resilient strain. He developed a predictive procedure using these findings to estimate rutting for practical cases. The results agreed reasonably well with the observed values; the differences between the observed and predicted values were thought to be due to the inability to compute resilient strains. This procedure is currently under review and improvement.

Studying rutting in flexible airfield pavements, Chou (18) conducted uniaxial dynamic tests on heavy clay (CH), with L.L. = 73% and P.I. = 48%, as part of an investigative program that included eleven full-scale test pavements. The test results showed that permanent strain increases greatly with increased load repetitions and also with increased load intensity. As the CBR of the soil increases, its resistance to permanent deformation increases rapidly. He noted that permanent strain increases with increasing elastic strain. However, he did not develop any constitutive equations to describe the phenomenon.

From the full-scale pavement tests, Chou found that two pavements having the same subgrade elastic strain will fail at the same coverage level but may experience different degrees of rutting in the subgrade.
Fig. 2.17 Influence of stress sequences on accumulation of permanent strain. (ref. 15)
Fig. 2.18 Procedures to predict cumulative loading from the results of simple loading tests. (ref. 15)
because of different subgrade conditions. This is contradictory to the generally-accepted limiting criteria discussed previously. According to those criteria, if the material and thickness of a pavement are properly selected and if proper compaction is applied so that the subgrade elastic strain is limited to certain levels, the subgrade rutting will be controlled and surface rutting equal to or less than some prescribed amount will be assured. According to Chou, this assumption is not strictly correct. He found that even when pavements are designed for the same performance level and the vertical elastic strain at the subgrade surface is nearly the same for both pavements, the surface rut depth and subgrade rutting will be greater for the thicker pavement (due either to heavier applied loads or to weaker subgrade support conditions).

At The Ohio State University, a series of researches have been carried out over the past seven years. Majidzadeh and others (3,9) had observed the relation(2.14) in the form:

\[
\frac{\varepsilon_p}{N} = A \cdot N^{-m} \quad \text{or} \quad \varepsilon_p = A \cdot N^{-m}
\]

where A and m are experimental and material constants. They proved this equation to hold for a wide range of materials and experimental conditions. Also, they showed that it holds for results obtained by other researchers conducting studies on asphalt concrete and untreated base courses.

It was claimed, however, that this equation was based on the rate process theory. Mitchell et al (39) investigated the creep phenomenon
in clays using the concepts of rate process theory. Using the analogy between the process of strain versus time in static creep and strain versus number of load repetitions in dynamic creep on the microscopic level, Mitchell's study was considered as a basis to the analysis of permanent deformation in the subgrade. However, Mitchell used the theory to investigate the effects of shear stress, strain rate, temperature and soil structure on the creep behavior of clay. He did not develop the strain-time relationship, analogous to equation \((2.16)\) from the rate process theory, but rather he found it through statistical analysis of experimental data. Mitchell and Campanella (39) stated "It has not yet been possible to account completely for this time-dependence of the creep rate in quantitative physical terms relating to the rate process model for behavior." Erol et al (41) stated that the rate process theory concepts imply; (a) at constant shear stress \((\tau)\) and temperature \((T)\) the flow volume, activation entropy and activation enthalpy of the system are constants, and (b) there is a linear relation between shear strain and time for constant clay structure, figure (2.19). To date, rate process theory could not be used to support relation \((2.16)\) or analogous equations.

Trials have been made, by the author to investigate the phenomenon using rheological models, (40), based on the rate process theory. Very complicated differential equations resulted due to the pattern of dynamic loading involved. Those differential equations did not have general solutions; mathematical consultants were involved in this decision. Numerical solutions were not visible because of the nature of coefficients involved that could not be measured or calculated prior to solution, through dynamic testing.
Fig (2.19) Strain vs. Time for Clay-Water System at constant Shear Stress (ref.41)
Using a test pit model and through dynamic test results and finite elements elastic analyses, it was shown (3) that for fine-grained subgrade soils, a sample tested under uniaxial deviatoric stress equal to \((\sigma_1 - \sigma_3)\) could approximate the behavior of soil elements under a two-dimensional stress state with \(\sigma_1\) vertical and its corresponding \(\sigma_3\) confining stress. Formula (2.16) holds for these cases as well.

They also suggested the formula (3):

\[
\frac{dy_p}{dN} = C (\varepsilon_p)^m
\]  

(2.17)

where

\(\varepsilon_p = \) permanent deformation

\(m = \) constant for soil failure under repeated loading

\(C = \) material constant, depending on density, moisture content, structure, etc.

Formula (2.17) does not appear to have any significance in a prediction model, since Equation (2.16) describes the phenomenon adequately.

Buranarom (17), in 1973, developed stress and moisture content shift functions to introduce their influence in Equation (2.16) in the form:

\[
\frac{\varepsilon_p}{N} = A N^m e^{B(\sigma_{\text{appl.}}/\sigma_{\text{ult.}})} e^{C(W - W_0)}
\]  

(2.18)

where

\(\sigma_{\text{appl.}} = \) the amplitude of the applied dynamic load

\(\sigma_{\text{ult.}} = \) the unconfined compressive strength
W = moisture content

\( W_0 \) = optimum moisture content

\( A, B, m, C \) = material constants to be determined from laboratory testing

It should be mentioned that these material constants have different values for the range of moisture contents on the dry side than those for the range on the wet side of the optimum. Although Equation (2.18) shows the influence of the moisture content and applied stress, it is not simple enough for practical application. Moreover, it describes the soil behavior in the original compaction condition; i.e., it does not take into account the environmental changes which occur primarily in the form of moisture content changes.

In 1975, Khedr (5) investigated the variations of parameters A and m in Equation (2.16). He concluded that "m" is almost constant for all practical non-failure cases of the silty clay and clayey soils investigated. Moreover, he correlated the parameter "A" with the resilient properties of the soils in the form:

\[ A = K(E^*)^{-S} \] (2.19)

where

\( E^* \) = soil dynamic modulus

\( K, S \) = coefficients dependent only on the soil structure and applied stress level

Equation (2.19) was found to be satisfactorily applicable to test data obtained for laboratory-prepared samples, as well as undisturbed field samples. Introducing the principle of resilient-residual characteris-
tics relationships into Equation (2.19) means that the dynamic modulus, \( E^* \), can characterize the soil material, accounting for the variations of moisture content, dry density and soil structure as far as rutting is concerned. Substituting Equation (2.19) into Equation (2.16) yields the following relation:

\[
\frac{\epsilon_p}{N} = K (E^*)^{-S} N^{-m}
\]  

(2.20)

A prediction model was suggested in the above study (5), which accounts for the environmental changes, in the form:

\[
\left( \frac{\epsilon_p}{N} \right)_n = A_{\text{max}} N^{-m} \prod_{i=1}^{n-1} N_i \left( \frac{-m_i + m_i+1}{m_i} \right)
\]  

(2.21a)

and if \( m_1 = m_2 = \ldots = m_i \), then Equation (5.9a) will be in the form:

\[
\left( \frac{\epsilon_p}{N} \right)_n = A_{\text{max}} N^{-m}
\]  

(2.21b)

where

- \( n \) = number of environmental changes at the end of which permanent deformation is to be predicted
- \( \left( \frac{\epsilon_p}{N} \right)_n \) = rate of permanent deformation at the end of the \( n^{th} \) environmental condition
- \( A_{\text{max}} \) = maximum value of the function "A" for the weakest condition of the soil through the \( n \) environmental changes
- \( N_i \) = total number of loading cycles through the \( i^{th} \) condition
- \( m_i \) = the value of the parameter "m" during the \( i^{th} \) environmental condition

This prediction model was verified by Bayomy (6) in 1977. Bayomy also introduced the stress shift function.
\[ \psi(\sigma) = \exp(\sigma_{\text{appl}} / \sigma_{\text{ult}}) \]

into Equation (2.19) to account for the effect of the applied stress level. The following equation resulted:

\[ A = R E^C \exp(\sigma_{\text{appl}} / \sigma_{\text{ult}}) \] (2.22)

where \( R \) and \( C \) are material constants independent of the applied stress, water content and dry density.

Equation (2.22) simplifies the rutting prediction model so that once the constants \( R \) and \( C \) are determined for specific soil materials in the laboratory, permanent deformation can be predicted upon knowledge of the dynamic modulus, either by field or laboratory means of testing (see Chapter V.) and using equations (2.21)

The prediction model for permanent deformation in subgrade soils, as developed at The Ohio State University, is unique. The purpose of this phase of study is to evaluate and refine this model and investigate its practical applicability.

(B) Resilient Characteristics:

From equations (2.20) and (2.22), the residual parameter "A" is related to the resilient properties of subgrade. In this section, the resilient characteristics are reviewed in the literature to establish the trend of its variations with different influential factors. Similar to base course characterization, the resilient modulus and Poisson's
ratio are the parameters that will be considered to characterize the subgrade soil. However, very few researchers have attempted to measure resilient Poisson's ratios, based on the belief that it does not change significantly. Also, all studies considered only unconfined uniaxial dynamic testing of soil samples, except as indicated.

In 1962, Seed, Chan and Lee (20) shed some light on the resilient characteristics of clay subgrades with respect to their influence in causing fatigue failures in asphalt pavements. They conducted a series of laboratory tests to study the variation of resilient modulus with the number of stress applications, the time interval between soil compaction and loading, the stress intensity, method of compaction, compaction density and water content, and changes in density and moisture content after compaction. They found that the influence of the number of stress applications is significant at the beginning of the test but less significant when occurring after 2000 or more cycles, depending on the soil's condition. However, Majidzadeh et al. (9) have noticed that the modulus variation with number of loading cycles is less significant than that observed by Seed et al. (20). For the clayey samples tested by Seed et al., the thixotropic phenomenon was observed, such that the longer the samples were stored after compaction, the higher the dynamic moduli they exhibited. However, this effect is only noticeable early during the tests; the deformations progressively induced by the repeated loading destroyed, in large measure, the thixotropic strength gained in aged samples. Practically, this occurs after the application of the first 1500 cycles or more.
It was noted that, at low stress levels, the resilient modulus decreases rapidly with increasing values of deviatoric stress. However, at stresses above 15 psi (approximately 25% of the ultimate strength), there is only a slight increase in resilient modulus with further increases in deviatoric stresses, figure (2.20).

Seed et al. (20) distinguished between flocculated and dispersed soil particle arrangement structures. They found that the first type of structure forms under full static compaction where no shear strain is induced during the compaction, and also when using compaction methods that induce shear strains (kneading or hammer compaction) only on the dry side of the optimum moisture content. The first structure type shows higher values of dynamic modulus. However, this influence disappears at high applied stresses, very high compaction moisture contents and upon saturation. Less dense samples with higher compaction moisture contents showed lower dynamic moduli. It was noted that the effect of the variation of these parameters was more pronounced on the wet side of optimum. Also, the saturation process increases the sample's deformation under dynamic loading.

In 1962, Ahmed and Larew (21) also presented a rather limited study reaching some of the conclusions discussed above. However, they compared the soils' moduli evaluated by repeated loading and gradually applied static loading tests. They observed that the values obtained in the first test were less than those for the second test and that the ratio of static modulus to repeated load modulus, $E_s/E_r$, varied from 1.05 to 2.0. These observations are in direct contradiction to those reported by Seed et al. (20).
Fig. 2. Effect of Stress Intensity on Resilience Characteristics—AASHTO Road Test Subgrade Soil.

(Ref. 20)
Pell and Brown (22), through their investigations, had observed similar dynamic modulus variations with deviatoric stress like those reported by Seed et al. (20) of a critical stress level. However, unlike Seed's findings, they noticed that the dynamic modulus continued to decrease with increasing stress levels but at a lower rate after the critical stress level. They proposed, as a means of simplifying the problem, a linear relationship between $E^*$ and $\sigma$ with steeper slope before a critical stress level, figure (2.21).

Pichumani (23) presented an investigative study for three computer codes (BISTRO, WIL67, and AFPAV) available for stress-strain analyses, and compared these results with field measurements obtained from full-scale flexible pavement test sections. He found that while surface deflection is not sensitive to the dynamic moduli of the asphalt concrete layer, base course, and subbase, the dynamic modulus of the subgrade has a significant influence on this parameter. The obvious reason for this is the enormous volume of subgrade subjected to stress, as compared to the upper pavement layers.

Thompson and Robnett (1) concluded a very similar, simplified relationship between $E^*$ and $\sigma$, like that adopted by Brown (22), although their test data were not consistent with that model. The deviatoric stress, $\sigma_d = 6$ psi, was considered the break point for the investigation of soils, and they proposed: (figure 2.21)

\[ E^* = k_3 + k_1(6 - \sigma) \quad \text{for } \sigma \leq 6 \text{ psi} \quad (2.23a) \]

\[ E^* = k_3 - k_2(\sigma - 6) \quad \text{for } \sigma \geq 6 \text{ psi} \quad (2.23b) \]
Fig (2.21) Typical Plot for Resilient Response Data for Subgrade Soil (ref.1)
However, they correlated the resilient modulus to the degree of saturation for the data obtained for a horizon of soils from the state of Illinois. They conducted linear regressions between $E^*$ and degree of saturation, $S_r$ (in percent), in the form:

$$E^* = 45.2 - 0.428 S_r$$

where $E^*$ is in ksi, with a correlation coefficient of $R = 0.706$.

Equation (2.24) was based on all of the soils included in their study. They also concluded that the resilient behavior of the various groups of soils in the classification systems are not, in general, significantly different. Accordingly, classifying the soil within the AASHTO, FAA or Unified system is not sufficient for the purpose of placing fine-grained soils into distinct resilient behavior groups. They also reported that CBR correlates positively with $E^*$, but for optimum and optimum plus two percent moisture contents, the soaked CBR was negatively correlated with $E^*$. The dynamic modulus was found to correlate linearly with the static modulus and unconfined compressive strength in that study.

Majidzadeh, Guirguis and Joseph (9) have conducted a comprehensive study for resilient characterization of soils. They measured the dynamic modulus ($E^*$) by means of transformation of creep test results in the frequency domain and resilient modulus ($M_R$) from direct dynamic testing. They found that $E^*$ and $M_R$ agree very closely and that both were higher than the initial tangent moduli determined from triaxial static compressive tests.
Unlike Seed's observations (20), Majidzadeh et al. (9) reported that the subgrade dynamic (resilient) modulus has an optimal (bell) type relationship with the compaction moisture content, and the optimum moisture content agrees very well with that for density, shear strength, and creep modulus. Also, at low stress levels where the material response can be considered in accordance with the linear viscoelastic theory, the dynamic modulus, $E^*$, is independent of applied stress level as well as the number of load applications. However, for deviatoric stresses varying from 22 to 126 psi (representing 12 to 68% of the ultimate strength) and unconfined testing, the $M_R$ increased rapidly until reaching an optimum value at a critical stress intensity (or stress level), after which it decreased rapidly with further increase in applied stresses, and then increased gradually and slightly with increasing stress. They suggested the following approximate relationships:

$$M_R = B \theta^{-m} \quad (2.25a)$$

or

$$M_R = B_1 \left( \frac{\sigma_{appl}}{\sigma_{ult}} \right)^{-m_1} \quad (2.25b)$$

where $B$, $B_1$, $m$ and $m_1$ are material constants and $\theta$ is the stress invariant equal to $\sigma_1 + \sigma_2 + \sigma_3$. In any case, it should be pointed out that the deviatoric stresses experienced in pavements in the field do not usually exceed 25 psi.

Furthermore, Majidzadeh et al. studied the variation of $M_R$ with the number of load applications, $N$. They defined the endurance limit
as the maximum stress at which a material can withstand an infinite number of cycles of stress without failure. Under stresses in excess of the endurance limit, the $M_R$ decreased with increasing $N$, while for stresses smaller than that limit, $M_R$ increased with increasing $N$. The first type of behavior was possibly due to work softening, while the latter may have been the result of work hardening. They proposed the general approximation:

$$M_R = A_1 N^{\alpha(s)}$$

where $A_1$ is a material constant. The parameter $\alpha$ depends on the stress level and is expected to be more than unity for stresses below the endurance limit and less than unity for the damaging stresses greater than the endurance limit. However, it was noticed from the studies conducted by Seed et al. (20) and Majidzadeh et al. (9) that after $N = 1000$ to $1500$ cycles, minor changes occur in the values of $E^*$, and the relation between $E^*$ and $N$ tends to level off after that limit. This observation was also reported by Buranarom (17) in his research on silty clay subgrade soils.

In a recent study by Edil and Motan (24) on silt loam soils, which applied the principle of energy status of soil-water in terms of soil suction, they concluded that the use of moisture content or degree of saturation alone to characterize the moisture regime of a soil is, in general, inadequate and that characteristic water retention curves are useful in reflecting the susceptibility of compacted soils to moisture changes. That is, the resilient modulus and post-repetitive loading strength are primarily related to soil suction. They also noted that
there was a trend between the dynamic and permanent deformations and the degree of saturation. Unlike the previously-mentioned studies, Edil and Motan reported that the number of loading cycles results in significant increases in the resilient modulus for samples tested at moisture equilibrium.

**Non-destructive Field Evaluation:**

All studies reviewed in previous paragraphs were concerned with laboratory resilient characterization of subgrade soils. Recently, methods were developed and equipments were constructed to conduct this characterization in the field. These non-destructive field testing methods are based on the concept of measuring the pavement deflection patterns under dynamic loading of specific intensity and frequency \( (3), (4), (42) \). Examples of such equipments are traveling deflectometer, L.C.P.C. deflectograph, dynaflect, road rater, WES 9-Kip Vibrator, WES 16-Kip Vibrator and Shell 4-Kip Bivibrator. Comparisons between the advantages and disadvantages of these methods are investigated elsewhere \( (42) \).

The pavement deflection data may be analysed through elastic multi-layer solution to calculate the resilient moduli of the pavement sublayers \( (4) \). The solution is computorized for 2-layer and 3-layer solutions. Also, statistical studies combined with field experience were performed to relate the pavement deflection patterns under the testing dynamic loading to reflect the performance of the pavement's components.
The adequacy of using such field instruments is investigated by Potter (43), Majidzadeh(4) and transportation research board (43). They concluded that the field testing results represent the pavement conditions to a satisfactory degree of accuracy, if analysed by an appropriate reliable methods. Nowadays, these methods are widely used for the design of rehabilitation plans in several states.

2.2 Research Approach

2.2.1 Theoretical Approach

To date, there has been no sound theoretical base to describe the dynamic creep phenomenon in pavement layers. This may be attributed to the complexity of the problem that involves micromechanistics investigation on the single particle characteristics level, macromechanical phenomena to investigate the behavior of groups of particles and the interaction among them and the integrated behavior of material body in a composite structure such as flexible pavements.

However, in this section the second phase of the problem will be investigated from the viewpoint of the mechanical energy concepts. The study of macromechanical phenomenological behavior of soil samples may project some light to help understand the rutting mechanism in such material. It will also provide the design engineer with methods that can be used to predict permanent deformation in pavement materials. It is best to deal directly with stress versus plastic strain when considering energy
involved in a hysteresis loop. Figure (2.22) shows schematically the relationship between stress ($\sigma$) and subgrade permanent strain ($\bar{\varepsilon}_p$) during a single cycle. The incremental work shown by the shaded section in the figure is $dW = \sigma d \bar{\varepsilon}_p$. The total area inside the loop is the hysteresis energy per cycle,

$$\Delta W_p = \int_{0}^{\Delta \varepsilon_p} \sigma d \bar{\varepsilon}_p \quad (2.27)$$

where

$\Delta \varepsilon_p$ = plastic deformation per cycle.

The nonlinear relationship of $\sigma - \bar{\varepsilon}_p$ during the loop can be expressed by a power function such as that for monotonic deformation as suggested by Sandor. (44),

$$\sigma = K \bar{\varepsilon}_p^n \quad (2.28)$$

Where

$n$ = cyclic strain-hardening exponent

$K$ = constant

It should be mentioned that in a condition of progressive sample failure under high stresses, the sample will undergo a strain-softening condition and equation (2.28) will not hold. Equation (2.28) will be determined and verified experimentally in chapter V.

Substituting equation (2.28) in (2.27) and integrating,
Fig (2.22) Analysis of Cyclic Energy

Fig (2.23) Through Thickness Crack in a Large Plate
According to figure (2.22) \( \sigma_a = \text{stress amplitude} = K (\Delta \varepsilon_p)^n \). Therefore, the plastic work per cycle is

\[
\Delta W_p = \frac{1}{n+1} \sigma_a (\Delta \varepsilon_p)^n
\]  

Equation (2.30) is a linear relationship even though the stress-strain curve itself is nonlinear. This is a property of all areas under curves expressed by power functions.

For this analysis equation (2.30) is assumed to represent the average plastic strain energy per cycle,

\[
\Delta \varepsilon_p = \frac{\varepsilon_p}{N}
\]  

Where \( N \) is the life span considered of number of loading cycles, and \( \varepsilon_p \) is the accumulated deformation in \( N \) cycles.

Energy is dissipated in fatigue because of plastic deformation. Most of this energy is converted into heat and is not recoverable as strain energy. The energy per cycle, \( \Delta W_p \), is measured by the areas of the hysteresis loop, equation (2.27), and the total energy expended during the life (\( N \)) is a summation of all loops areas. Thence, the total plastic strain energy, \( W_p \), is described by the equation

\[
W_p = \Delta W_p \cdot N
\]  

(2.32)
substituting in equation (2.30);

\[ W_P = N \frac{a^2}{n+1} (\Delta \varepsilon_p) \]  

(2.33)

Before continuing the derivation, consider the fracture criterion as studied by Griffith and illustrated by Hertzberg (45). Griffith noted that when a crack is introduced to a stressed plate of elastic material, a balance must be struck between the decrease in potential energy (related to the release of stored elastic energy).

and the increase in surface energy resulting from the presence of the crack. Likewise, an existing crack would grow by some increment if the necessary additional surface energy were supplied by the system. This "surface energy" arises from the fact that there is a non-equilibrium configuration of nearest neighbor particles at any surface in a media. For the configuration in figure (2.23), Griffith estimated the surface energy term to be the product of the total crack surface area \(2a_2t\), and the specific surface energy \(\gamma_s\), which has units of energy/unit area. Using stress analysis for the case of an infinitely large plate containing an elliptical crack, the decrease in stored elastic energy of the cracked plate is \(\pi \sigma^2 a^2 t / E\). Hence the change in potential energy of the plate associated with the introduction of a crack may be given by

\[ U - U_0 = - \frac{\pi \sigma^2 a^2 t}{E} + \frac{1}{4} a t \gamma_s \]  

(2.34)

where

- \(U\) = potential energy of body with crack
- \(U_0\) = potential energy of body without crack (independent of crack length).
$\sigma = \text{applied stress}$

$a = \text{one-half crack length}$

$t = \text{thickness}$

$E = \text{modulus of elasticity}$

$\gamma_s = \text{specific surface energy}$

To determine the condition of equilibrium, differentiate the potential energy $U$ with respect to the crack length and setting equal to zero,

$$\frac{\partial U}{\partial a} = 4t \gamma_s - \frac{2\pi \sigma^2 a t}{E} = 0 \quad (2.35)$$

Therefore,

$$\frac{\sigma^2 a}{2E} = \frac{\gamma_s}{\pi} \quad (2.36)$$

represents the equilibrium condition.

The nature of the equilibrium condition described by equation (2.36) is determined by the second derivative, $\frac{\partial^2 U}{\partial a^2}$. Since

$$\frac{\partial^2 U}{\partial a^2} = -\frac{2\pi \sigma^2 t}{E} \quad (2.37)$$

and is negative, the equilibrium condition is unstable, and the crack will continue to grow.

Griffith assumed elastic materials in which plastic deformation process is limited (zero) and the difference in surface and fracture energies is not expected to be great. However, this is not true for most materials. Recognizing this fact Irwin (illustrated by Hertzberg (45)), introduced the use of the energy source term (i.e., the elastic energy per unit crack length increment $\frac{\partial U}{\partial a} = g$), therefore,

$$\sigma = \sqrt{\frac{Eg}{\pi a}} \quad (2.38)$$
At the point of instability, the elastic energy release rate $g$ (also referred to as the crack driving force) reaches a critical value $g_c$, whereupon fracture occurs. The critical elastic energy release rate may be interpreted as a material parameter and can be measured in the laboratory.

Equation (2.38) is the fracture criterion both in fatigue and in monotonic loading, but there is an essential difference between the two cases in reaching the point of fracture. In monotonic loading, the stress is gradually increased by raising the external load until fracture occurs. In fatigue, the stresses increase because the gradually growing crack reduces the load bearing area and increases the local stress.

Thus for two different amplitudes, $\sigma_1$ and $\sigma_2$, the critical crack length $a_1$ and $a_2$ for fracture, at specific performance level, are given on the basis of equation (2.38) as

$$\frac{a_1}{a_2} = \left( \frac{\sigma_2}{\sigma_1} \right)^2 \quad (2.39)$$

A cracked material under load will deform plastically first at the notch tips. Localized plastic deformation occurs when the appropriate yield criterion is satisfied in the vicinity of the crack and a plastic zone is created near the crack tips. The size and shape of the plastic zone depend on the mode of deformation that acts on the crack and on the criterion for yielding. Two models are proposed to describe this plastic zone; extended zone and strip zone. The latter is proposed mainly for a sharp tipped crack. Describing the former type, Tetelman and McEvily (46) derived the expression,
\[ R = \left( \frac{\sigma}{\sigma_y} \right)^2 a \]  
\text{(2.40)}

where

\[ R = \text{extension of plastic zone beyond the crack tip} \]
\[ \sigma_y = \text{Yield stress} \]

Equation \( (2.40) \) implies, at specific stress level \( (\sigma/\sigma_y) \), the radius \( R \) is linearly proportional to the crack length \( 2a \). Although the shape of the plastic zone is not well defined, it may be appropriately assumed to be circular (or spherical) of radius \( R \).

The energy required to plastically deform the material near the crack is proportional to the volume of the plastic zone and also depends on the strain distribution within the zone. This volume is a function of the crack size, since \( R \) is linearly related to the crack length. Hence, the plastically deformed volume associated with a crack is proportional to the square of the crack length. Thus, the plastic strain energies required to propagate different sizes of crack are proportional in the form:

\[ \frac{\Delta W_{p1}}{\Delta W_{p2}} = \left( \frac{a_1}{a_2} \right)^2 \]  
\text{(2.41)}

Equations \( (2.39) \) and \( (2.41) \) relate the corresponding stresses and energies, for specific fatigue life \( N \), as:

\[ \frac{\sigma_2}{\sigma_1} = \left( \frac{W_{p2}}{W_{p1}} \right)^{-1/4} \]

or

\[ \sigma_1 \left( \frac{W_{p1}}{W_{p2}} \right)^{1/4} = \sigma_2 \left( \frac{W_{p2}}{W_{p1}} \right)^{1/4} = C \]  
\text{(2.42)}

where

\[ C \text{ is constant of proportionality.} \]

Now, substituting equation \( (2.42) \) in equation \( (2.33) \),

\[ N \left( \frac{1}{n+1} \right) \frac{a}{P_a} (\Delta \epsilon_p) = C^{1/4} \left( \frac{\sigma_a}{\sigma} \right)^{-1/4} \]  
\text{(2.43)}
From equations (2.28), (2.43):

\[
N \left( \frac{1}{n+1} \right) \sigma_a (\Delta \varepsilon_p) = (C) \left[ (K) \right]^{5n} (\Delta \varepsilon_p)^{(n+1)} N^{-1}
\]

\[
(\Delta \varepsilon_p)^{(n+1)} = (C) \left[ (K) \right]^{5n} (n+1) N^{-1}
\]

\[
\therefore \Delta \varepsilon_p = A \cdot N^{-m}
\]

where

\[
A = \left( \frac{C}{(1+5n)} \right) \left( \frac{K}{5n} \right)^{(n+1)}/(1+5n)
\]

and \(m = 1/(1+5n)\)

Equations (2.31) and (2.44) result,

\[
\frac{\varepsilon_p}{N} = A \cdot N^{-m}
\]

Equation (2.45) implies that at a specific stress level and certain material structure, the rate of permanent strain accumulation has a power relation with the number of load repetitions.

In equation (2.44), the parameter \(m\) is function of "n", i.e. it is dependent only on the applied stress-permanent strain pattern during a single loading cycle. Parameter \(A\) is a function of "C", "K" and "n", i.e. it depends on the stress-strain pattern and intensities, stress level and the dissipated plastic strain energy during the dynamic testing.

The plastic strain energy dissipated depends mainly on the material composition and structure and testing conditions such as temperature.

For subgrade soils, the material structure is generally defined as the mutual arrangement, orientation, and organization of the particles in the soil. The term is also used sometimes with reference to the geometry.
of the pore spaces. Since the arrangement of soil particles is generally too complex to permit any simple geometric characterization, there is no practical way to measure soil structure directly. Therefore, the concept of soil structure is used in a qualitative sense.

As mentioned in section (2.1.2A), equation (2.1.5) was observed experimentally in previous studies. It will be investigated further in this study. However, the theoretical derivation will help in understanding the nature and variation of the rutting parameters \( A \) and \( m \).

2.2.2 Experimental Approach:

To fulfill the primary objective of this study; i.e. to characterize the residual properties of untreated base course and subgrade soil, the rutting (dynamic creep) phenomenon in these materials will be investigated through traffic-simulated triaxial dynamic testing. The effects of various influencing factors are to be studied and descriptive constitute equations to represent the criteria are to be developed.

(A) Base Course

Principally, this material should be dealt with as graded assembly of oriented particles, rather than homogeneous continuum when considered in a pavement structure. However, no model has been found, which would adequately characterize the granular material according to that principle. Therefore, the present available theoretical and numerical solutions have been adopted to analyze the pavement structure and study the loading conditions of pavement sublayers.

When a wheel load rolls over a pavement structure, the element B in
figure (2.2b), is subjected to vertical and horizontal principal stresses when the wheel is directly over the element. However, the axes of the principal stresses are no longer vertical and horizontal when the wheel is no longer over the element. Shear stresses are induced in the second case. Since the triaxial test cannot apply shear stresses directly to the sample the equivalent in situ soil element B must be subjected to principal stresses only and hence "rotates" as the wheel passes (22), figure (2.24a). Since deformation of the sample is measured in the direction of the applied stress, the permanent deformation measured will not represent the vertical (or horizontal) deformation, but will in fact overestimate it. On the other hand experimental results for sands show that reorientation of principal stress axes during cyclic shear results in increases in density, (35). The density increase was related directly to the magnitude of the cyclic shear strain. The effect of stress re-orientation in granular materials would thus seem to produce larger permanent strains than those predicted by cyclic triaxial testing.

The sample should be tested ideally under conditions analogous to element B shown in figure (2.24b), where shear stresses are induced along with vertical and horizontal stresses. There has been no testing method introduced which is capable of applying such a complex stress pattern.

However, the effect of shear reversal is likely to be more marked on materials near the road surface than it is in the deeper layers (base, subbase and subgrade). Triaxial dynamic testing has been used in this study, as a presently possible test, to describe the behavior of base course
(a) Principal Stress - Element Rotates

(b) No Rotation - Shear Stress Reversal

Fig. (2.2h) In Situ Stresses Beneath a Rolling Wheel
In the previous studies of permanent deformation in base courses, constant confining pressure and axial dynamic stress were used. Such stress arrangement has the following disadvantages:

(1) It does not simulate the in-situ condition in which lateral pressure changes simultaneously with the vertical pressure as a function of time (13).

(2) The roll of the confining pressure in those studies is limited to being a conditioning stress not as direct reaction to vertical wheel loading.

(3) In the triaxial testing, there are two types of procedures, (a) either let the sample consolidate under confining pressure first before applying the dynamic load, thus neglecting its effect on permanent deformation; or,

(b) apply dynamic loading along with the constant confining pressure (σ3), thus overestimating the effect of σ3.

(4) In a triaxial cell, the confining pressure is applied on the sample in all directions which means inducing unrealistic static overburden pressure in the vertical direction.

In this study, the lateral pressure is applied dynamically to vary simultaneously with the vertical pressure to study the dynamic creep of base course material. It is not intended, however, to conduct a comparison between the two testing conditions; rather it is to establish a predictive model for this phenomenon under closest stress conditions possible to those
expected in the field.

Permanent deformation is mainly dependent on stress conditions, sequence and magnitude. It is also dependent on degree of consolidation, aggregate type, amount and type of fines, degree of saturation (moisture content) and degree of compaction. In this study, stress conditions and magnitude, moisture content and relative density will be incorporated as the influencing factors in the testing program.

(B) Subgrade Soils:

Two types of subgrade soils are tested dynamically in this study. These tests are the end of testing series conducted by Khedr (5), and Bayomy (6) to study the subgrade soil support conditions in the state of Ohio.

In the dynamic testing procedure conducted in this phase of the study, however, the specimens are tested with no lateral confining pressure, i.e. $\sigma_3 = 0$. The factors considered in deciding this test procedure are:

(i) Previous studies (3) on fine-grained soils (silty clay have proved that the behavior of samples tested under uniaxial dynamic loading with stress amplitude equal to $(\sigma_1 - \sigma_3)$ matches closely the soil behavior under three-dimensional dynamic loading with $\sigma_1$ vertical and $\sigma_3$ horizontal applied stresses. This was achieved by comparing the dynamic creep curves of soil masses tested dynamically in a test pit with those of representative samples tested under unconfined dynamic loads with the stress level equal to the average deviatoric stress in the soil mass (Figure 2.25);

(ii) In another study by Thompson and Robnett (1), it was indicated that small magnitudes of confining pressure (up to $\sigma_3 = 5$ psi) had no significant effect on the resilience behavior of the fine-
Figure 2.25 $\varepsilon_p$ vs N as Measured from Test Pit and Unconfined Dynamic Test Under Same Deviatoric Stress
grained soils examined. In most of the practical cases, the confining pressure in the subgrade does not exceed 5 psi.

(iii) Another factor was the simplicity and ease of testing, especially important when considering the large testing program employed in this investigation.

The influencing factors considered in this study are: stress intensity, compaction moisture content, dry density, saturation process and soil type. The dynamic experimentation program was conducted within practical ranges of these considered factors.

A field case was investigated for subgrade rutting in Pike County, State of Ohio. In that case the flexible pavement suffered complete failure due to excessive rutting experienced along about a three-mile section. Undisturbed field samples were obtained from different sections of the road. They were subjected to similar testing and analysis programs. This case will be discussed in detail in chapter VIII.

All testing data obtained from this experimental program, as well as those obtained by other researchers, were analyzed statistically in accordance with the theoretical approach introduced in the previous section with main concern for the principal goal of developing a rutting prediction model for the materials tested.
3.1 Experimental Program

Two types of materials were the subject of this testing program; untreated granular material and subgrade soil. Therefore, two principal series of testing were considered in this investigation; each phase is concerned with one material type. However, the main purpose of both phases is to conduct practically applicable laboratory dynamic testing that best simulates the in-situ conditions.

3.1.1 Preliminary tests:

Classification and identification tests were performed first on all materials investigated. For the base course, tests of sieve analysis, specific gravity, Atterberg limits for the fine portion, and compaction (moisture-content-dry density curve and extreme densities at each water content) were performed. Each subgrade soil was subjected to the basic tests of Atterberg limits, grain size analysis (mechanical and hydraulic), compaction and strength curves. It was, then, classified according to the known standard classifications of subgrade soils.

All these tests were performed in accordance with the standard procedures of the American Society for Testing Materials. The results of these tests are presented in chapter IV.

3.1.2 Dynamic Tests:

(A) Granular Material

A crushed limestone aggregate with maximum size of 3/4in. and limestone
fines, was subjected to a dynamic testing program. In these tests, dynamic lateral stresses that varied simultaneously with dynamic deviator stresses were applied to the specimens to simulate field conditions as closely as practically possible. All samples tested were almost of the same dry density. However, moisture content values of approximately 0.1, 4 and 6 percent were used for three groups of samples. Consequently, the relative density was varied.

The dynamic testing of the granular material was carried out in a triaxial cell. These tests were performed at dynamic deviator stress levels of 0.5, 10 and 20 psi with dynamic confining pressures of 5, 10 & 15 psi applied to each group of certain water content. The permanent and elastic deformations were monitored during the test for an average of 1,000 load repetitions.

Resilient modulus tests were conducted on some of the samples after concluding the rutting test. Such samples were subjected to different combinations of dynamic deviator and confining pressures. Also, the effect of overburden (static) deviator and confining pressures levels on resilient response was studied.

Although it was not intended to conduct comparison between constant confining pressure and variable confining pressure types of tests in this study, three samples were tested under the former testing procedure for brief qualitative comparison.

(B) Subgrade Soil:

Sandy silt and silty clay soils obtained from Hancock county (SR-12) and Jefferson county (I-7) in the state of Ohio respectively, were tested dynamically in the second series of laboratory program. Soil samples
were tested under uniaxial dynamic stress. The reasons for this testing procedure are illustrated in the previous chapter.

The samples tested for each soil type were of different moisture contents and dry densities. Moisture content was varied in the range of approximately ±4 percent above and below the optimum. The dry density was varied according to the compaction curve at modified standard compaction energy. The study also included testing saturated samples, some of which were tested prior and after saturation. Stresses of the intensity level 4, 8, 12 and 16 psi were applied to each group of specific water content and dry density.

Rutting tests were performed for about 4,000 loading cycles for the as-compact ed samples, and about 10,000 cycles for saturated samples. Permanent deformations were recorded along the duration of the test.

Modulus tests were run on some samples to study the modulus variation with stress levels, moisture content, dry density, material type and saturation effect. These occurred after the rutting test had been completed on the samples.

Undisturbed field samples obtained from the study case at Pike county (SR - 124), Ohio were subjected to a similar dynamic testing program. The samples were tested at as-obtained density and moisture content conditions. The uniaxial deviator stress levels were decided for each sample individually so that it will not exceed one third of its ultimate strength.

3.1.3 Ultimate Tests:

After dynamic testing, if the specimen had not failed during that test,
it was tested for ultimate strength. Base course specimens were tested at constant confining pressure equal to that applied as dynamic confining pressure during the dynamic testing. Gradually increasing deviator stress was applied until failure. Subgrade soil samples were tested for unconfined compressive strength.

3.2 Sample Preparation

3.2.1 Granular Material Samples:

Aggregates obtained from the American Aggregate Co. quarry in Columbus, Ohio was oven dried then sieved for particle size separation. Material was prepared for each sample by weighing and mixing calculated amounts of each aggregate size to compose the specified gradation considered. It was then mixed with a specified percentage of distilled water, sealed in plastic bags and kept in a humid room until sample compaction. A few samples were collected after dynamic and ultimate tests to check their gradations after compaction and testing. No significant changes have been found. Therefore, material could be used more than once for making other samples.

All specimens were 4 inches in diameter and 7.5 inches in height. The 2:1 height to diameter ratio was kept as constant as possible in order to minimize the end effects and buckling on the total deformation of the sample.

Each specimen was prepared in a 4 x 10 inch split mold between two end platens. A 4 x 12 x 0.014" rubber membrane was placed inside the mold and a vacuum was applied between the membrane and the mold so that the membrane was held tightly against the inside wall of the mold. The sample was compacted in four layers using a 10 pound hammer with a fall of 18 inches and
striking base plate of 3.9 inches diameter. Preweighed materials, to attain specified dry density, were divided into four equal amounts to be used in four compaction layers. Each layer was compacted to a height calculated to be one fourth of the sample's height. Exact moisture content was measured at compaction of the material patch used to make the sample, and after the test of the sample itself. The top and bottom loading platens were lubricated by use of silicone grease or oil between two sheet of cellophane cut into four inches diameter discs and placed between the platten and sample. This procedure was done to minimize the effect of friction between loading ends and sample aggregate.

After compaction the sample in the mold was placed on the triaxial cell base. The mold was then removed, a thin film of silicone oil was applied to the rubber membrane, and a second membrane was placed around the sample by means of a membrane jacket. The double membrane thickness was deemed necessary to prevent leakage and to ensure complete sample sealing during test. O-rings were then placed over the membranes onto the end platens and the triaxial base. The two lateral deformation measuring Bison soil strain gages (see details in the recording equipment) were placed exactly opposite on the perimeter of the sample and at its midheight. They were held to the samples by means of rubber bands. Figure (3.1) shows the compacted sample arrangement. Samples were tested after an average of 6 hours after preparation procedure.

3.2.2 Subgrade Soils:

From each soil type enough material to make a few samples was dried,
Fig. (3.1) Triaxial Testing Arrangement
then thoroughly mixed in a mechanical mixer with specific amounts of distilled water. Then each specimen, with a given moisture content, was sealed in a plastic bag and kept in a humid room for at least two days, to assure uniform distribution of moisture throughout the soil sample. The testing samples were compacted in three equal layers using drop hammer compaction equipment (Soil Test Corporation, Model AP - 172), with a hammer weight of 10 pounds and 1¼ inches drop and striking base plate of 2.7 inches diameter. The mold used was 2.8 inches inner diameter and 7 inches height to make soil samples of 2.8 inches diameter and 5.7 inches height. Exact moisture content of soil was measured immediately before specimen compaction. The samples were compacted with equal numbers of hammer blows for each compaction layer so that equal modified standard compaction energy was applied to each sample.

After compaction the sample was then extruded from the mold using a hydraulic jack, weighted to an accuracy of 0.5 gm., the height measured, and then was wrapped with saran wrap plastic sheet, placed in a plastic bag of appropriate size, sealed with wax and then kept in a humid room until ready for testing. The samples were kept in the humid room for a week to give sufficient time for the moisture to be redistributed so that the sample would have a uniform moisture content. Moisture content measurements were taken after testing for each sample.

The undisturbed field samples of subgrade soil were sampled from under the pavement by drilling cores in the asphalt concrete, then Shelby tubes of 2.9 inches diameter were used in dry sampling method. In
the laboratory, the ends of the brass tubes were cut by mechanical saw. The soil was then extruded from the tubes, cut into lengths of approximately 5.5 inches. The samples were capped with a brittle sulfur compound at both top and bottom ends to provide level ends so that uniform distribution of load applied to the sample can be assured when tested. Immediately, the capped samples were wrapped, sealed and kept in the humid room. Moisture content and dry density were measured after testing.

Some of the laboratory prepared samples were subjected to a saturation procedure. The sample was taken from the humid room, unwrapped and weighed. The sample was put in the saturation arrangement shown in figure (3.2). The porous stones were saturated before bringing them into contact with the sample. A total of 6 samples could be saturated simultaneously. A vertical pressure of 2 psi and a lateral pressure of 1.2 psi were applied to the sample during saturation process. The sample was reversed in position in the saturation device after 24 hours. After 48 hours, the sample was removed, weighed, wrapped and sealed and kept in the humid room for at least 7 days prior to testing.

3.3 Testing and Recording Equipments

3.3.1 Testing Equipment:

The laboratory investigation portion of this study was conducted at the Material Research Laboratory, Civil Engineering Department, the Ohio State University. The main aspect of testing the granular material in this study is that the confining pressure in the triaxial cell varies
Figure 3.2 Schematic Diagram of Saturation Device (Constant Stress Condition)
simultaneously with the axial load. This was accomplished by the use of two electro-hydraulic MTS (Material Testing System Corporation) systems connected to each other electronically through their function generators. One system was used to apply the deviator axial pressure, with load controlled electro-hydraulic panel, to the triaxial cell piston. The other applied the variable confining pressure, with a stroke-controlled electro-hydraulic panel through a hydraulic jack that was modified to work as a hydraulic piston whose output was connected to the triaxial chamber. The connection between the two MTS systems was designed so that a slight delay in applying the vertical deviator stress would take place to count for that in the confining pressure pulse. The delay of the confining pressure pulse was caused by compressibility of the chamber and lines fluid (which may include unavoidable air bubbles) and friction loss in the line connecting the triaxial chamber to the modified hydraulic jack. The detailed characteristics of MTS could be found elsewhere, e.g. (3) and (5). Figure (3.3) shows schematic diagram of the testing arrangement.

A haversine loading function was applied in both vertical and lateral direction. The deviatoric load duration was chosen to be 0.125 seconds with rest period of 0.875 Seconds between load pulses, i.e. the load was applied at frequency 1 cycle/sec, figure (3.5). The cell pressure was applied at the same frequency of 1 cycle/sec. However, its pulse duration was longer than 0.125 sec, due to the pulse transfer mechanism between the MTS and the chamber.

The tests were performed in a standard triaxial cell with 4 in. dia-
Fig. (3.3) Base Course Testing Arrangement
meter base. However, it was modified to assure a no-leakage testing condition to minimize the compliance of the system. Any leakage through the testing arrangement, including the triaxial cell, would result in unstable continuously diminishing dynamic cell pressure. The cell piston stroked in a bush ball bearing to minimize the piston friction, since it is very difficult to account for piston friction in dynamic loading. One difficulty encountered was preventing all liquid leakage through the piston bearing and at the same time minimize its friction. The chamber cylinder was made of \( \frac{1}{2} \)" thick plexiglass wrapped with steel belts to minimize its size expansion under testing confining pressure. Also, to minimize volume changes through connection, only brass tubing and hydraulic transmission reinforced rubber line were used. A pressure transducer was attached to the top cap of the triaxial cell.

Subgrade soil samples were tested using one MTS system to apply a uniaxial haversine loading function. The load duration was 0.125 sec. with rest period of 0.375 sec., i.e. the frequency of load applied was 2 cycles/sec.

3.3.2 Recording Instruments:

The axial loads applied to all types of samples were monitored by means of Lebow load cells of capacities 1,000 and 3,000 pounds. The load cells were connected directly to the MTS electronic bridge assembly. The chamber pressure, in granular material's testing, was measured using pressure transducer with a bleeding screw to bleed air bubbles out of the chamber.
A laboratory-made amplifier and bridge unit was constructed, to which the transducer was connected.

All deformations were recorded by means of Bison soil strain gage coils (1-inch and 2-inch diameter, disk-shaped coils of wire). The separation of any two gages of the same diameter affects the electromagnetic coupling between them. A change in the gage spacing results in a bridge unbalance; so an inductance bridge may be used to obtain output voltage as a function of displacement. This is done by a Bison unit. Moisture and temperature effects are negligible, provided the gage leads are waterproofed. Any nearby metal objects (at distance less than three time the diameter of the gage) should be avoided if possible. However, the effect of such metal could be counted for by proper calibration procedure.

A couple of these gages (1-inch diameter) were used to measure the stroke of the triaxial cell piston as to represent the total aggregate sample deformation. One gage was fixed to the cell cap and the other was attached before the load cell, figure (3.1). A similar arrangement was used to measure the deformation of the subgrade soil samples. However, another arrangement was also used, figure (3.4). A couple of 2 inch-diameter gages were used to measure lateral deformation of the aggregate samples. They were attached to the sample by means of wide rubber bands, figure (3.1).

Two Brush Recorders, Mark 280, were used to record the test results. Each recorder had two channels. In granular material testing, one recorder was used to monitor vertical load and confining pressure, figure (3.5).
Fig (3.4) Schematic Diagram for Subgrade Sample Testing Set-Up
Fig (3.1) Schematic Diagram for Subgrade Sample Testing Set-Up.
Fig (3.5) Deviatoric and Confining Stress Pulses Chart
while the other recorded vertical and lateral deformation, figure (3.6). However, because of the limitation of employing one Bison unit, these deformations were not recorded simultaneously but rather one at a time.

3.4 Testing Procedure

3.4.1 Base Course

1. The sample is carefully placed on the triaxial loading base, the triaxial chamber is tightly attached to the base and then the connections are secured to prevent any possible leakage.

2. Make sure that the line between A to F, figure (3.3), has no air bubbles. Then the triaxial chamber is filled with the liquid media (water, in this case) with valves F, D and E open and A, B and C closed.

3. The pressure transducer is to be attached to E and the bleeding screw is opened to bleed any air bubbles out.

4. The bleeding screw and then valve F are closed. The water supply tank is placed so that the filling differential to valve F is close to the assigned static confining pressure (simulating in-situ overburden pressure). The dynamic confining pressure will be superimposed on this static confining pressure to ensure that chamber pressure will not drop below zero during the test due to fluid bounce.

5. Vertical static pressure is then applied, and the sample is left to consolidate under these initial conditions until equilibrium. This static deviator represents the in-situ overburden pressure, it is essential during
Fig (3.6) Axial Deformations Measurements
the test to make sure the load cell, triaxial cell piston and loading balls are in complete contact at all times during the test. It should be of a minimum amount equal to confining dynamic and static pressure multiplied by the cross-section area of the triaxial piston (1.77 inch$^2$) for the same reason.

6. Start applying dynamic confining pressure; the stroke of the modified hydraulic jack under the first MTS system is adjusted to obtain the required confining pressure pulse. In some samples this pulsing was run for about 2,500 before applying the deviator stress to assure reasonable stable dynamic confining pressure. Deformations were recorded during this period. Then a very small increment of deviator stress is applied to adjust the required delay so that both types of pressures are applied simultaneously. Also, it helps to assure complete contact of the loading system with the sample.

7. The test is started by increasing the deviator stress quickly to the required level.

8. Deformations are monitored during the tests as well as the applied pressures which should be adjusted if necessary.

9. For some samples, and after step 8, the applied stresses were varied in practical ranges. At each condition, sufficient numbers of load repetitions are applied until the sample reaches a stable elastic condition, i.e. no significant changes in the elastic deformations occurs upon increasing loading cycles. Then, elastic deformations are recorded at each condition.
10. The sample is unloaded, and valve F is opened to partially empty the chamber. Disconnect the pressure transducer at E. Static confining pressure is then applied (that is, equal to the used dynamic confining pressure in step 6) through valve E by means of compressed air. The pressure is applied through a pressure regulator and monitored by pressure dial. This step was done using air pressure because the volume of the triaxial cell piston pushed, upon vertical loading, into the chamber filled with water increases the chamber pressure considerably.

11. Vertical load is applied gradually until failure. Deformations are also monitored during the strength test.

12. The water is discharged from the chamber by disconnecting E and open valves D and F.

13. The sample is collected for moisture content determination and reusage of its material if needed.

3.4.2 Subgrade Soils

A soil sample is taken from the humid room, unwrapped, and then loosely covered with cellophane wrap on the sides to minimize moisture loss during time of testing. A rigid loading platen is used to assure uniform load distribution on the cross-section of the sample. Two circular sheets of cellophane separated by a lubricating film of silicone oil are placed on each end of the sample to minimize end friction between the
sample and rigid loading ends.

Static deviator stress of about 0.8 psi is applied to the sample. It was left to consolidate first under this load. A preload of three cycles of stress equal to one fifth of the assigned dynamic loading, is applied to assure complete contact between sample and loading system. Then, the deviator dynamic load is applied to the sample and deformations are monitored.

After finishing the rutting test, some samples are subjected to different levels of applied dynamic load. At each load level, the sample should be given the chance to reach elastic equilibrium, i.e., no significant changes are observed in the elastic deformation. The elastic deformation is recorded at each stress level.

The unconfined compressive strength test is then performed on the sample with a strain rate of approximately 2.8% per minute. The MTS is used to conduct this test with a stroke-controlled ramp loading function.

Multiple - Step Test:

This is a test method to verify and determine the cyclic stress-strain curve represented by figure (2.22) and equation (2.28). The test is started at a small stress amplitude, and the amplitude increased in small increments. At each level of stress, 100 load repetitions are performed to produce a stable hysteresis loop (it is not too many to cause serious fatigue damage to the soil sample). Then the plastic deformation per cycle is to be recorded. Since such deformation is too small to be measured by empoiled recording methods, deformation is measur-
ed for 10 cycles (91st. to 100th cycle) and the average is considered. The result of this test is a dynamic stress versus plastic deformation curve for each sample which is thought to represent stress-plastic strain curve for the sample during a stable representative loading loop.
IV PRELIMINARY MATERIALS DATA

4.1 Granular Material

A crushed limestone aggregate obtained from the American Aggregate Co., Columbus, Ohio is the subject material tested in this phase of study. The gradation used is within the gradation limits of the Ohio specification for untreated granular subbase 310B, figure (4.1). All samples were prepared from this aggregate. Atterberg limits tests showed that the fines of this material has 13 percent liquid limit and non-plastic properties. The specific gravity was found to be 2.52 for the course portion (coarser than size #4) and 2.64 for the finer portion of the gradation.

Compaction control test was run using standard Proctor compaction procedures and modified compaction energy. This was considered to produce maximum density. The minimum density (maximum void ratio) test was run in accordance with AASHO T 19-70. The results are presented in figure (4.2).

4.2 Subgrade Soils

The soil testing involved in this investigation was the end of a test series conducted at the Ohio State University. Since the analysis will include test results obtained previously (5) & (6) as well as those obtained in this study, it is appropriate to present the classification data for all soil types investigated.
Fig.(4.1) Aggregate Gradation and Gradation Bands for Ohio 310B Subbase.
Modified Compaction Energy

Fig. (4.2) Compaction Curves for Tested Aggregates
All subgrade soils utilized in this investigation were obtained from construction sites in Ohio. They represent typical subgrades usually encountered in the state, consisting mostly of fine-grained silt and clay with sand. However, the dynamic behavior of each soil may depend on the percentage of each component present and the soil fabric (structure), which depends on the shape and arrangement of the soil particles. The particle shapes depend upon the geological history of the soil sedimentation. Particle arrangements in a subgrade soil mass may depend on the type of compaction, moisture content (with respect to the optimum) at compaction, moisture movement and history, and the history of applied stresses.

Although the microscopic study of soil particles is beyond the scope of this investigation, the evaluation of its effect on the macroscopic dynamic response is one of the research goals. Unfortunately, the well-known methods of subgrade soil classification do not consider this difference in soil structure. Consequently, one may find two soils with the same AASHTO classification, for example, but they display different dynamic responses. In other words, the AASHTO, FAA and Unified classification methods are not adequate to completely describe subgrade performance under traffic loading, as will be seen in the analyses presented in this study. Similar conclusions have also been reported by Thompson et al. (1).
Nevertheless, presenting the soils according to those classifications will help in the recognition of general trends observed in the soils under investigation. The soils were given designated numbers, as shown in Table 4.1, and will be identified throughout the remainder of this report by their designated numbers. There is no particular significance to these numbers; they are given for the purpose of brevity in subsequent discussions and analyses. Soils 1 to 5 were reported by Bayomy (6), soil 6 & 7 were tested in this study, and soils 8 & 9 were tested by Khedr (5).

A. Classification of Soils

All pertinent properties of the soils studied are tabulated in Table 4.1, along with their classifications. Soil numbers 1, 2, 3 and 6 are typical sandy silt and clay soils in the categories, A-4 and E-6. Soil numbers 5 and 7 are typical clayish soils in the A-6 and E-7 classifications. Soil #4 is the bottom layer of the subgrade detected at about three to four feet in depth below soil #3. It is a silty sand with some gravel and was obtained for the purpose of investigating a coarse type soil in the A-2-4 and E-2 categories. The reference soil, standard Kaolin clay, is classified as a fat clay in the A-7-6 and E-8 categories. The grain size distributions of these soils are shown in Figures 4.3 through 4.6.

B. Compaction Curves

Figures 4.7 and 4.8 show the compaction and strength curves, respectively, for soil numbers 1 through 7. In these compaction curves, the modified compaction energy was applied by 30 hammer blows per layer in three layers per sample.
<table>
<thead>
<tr>
<th>Soil No.</th>
<th>Project Location and Number</th>
<th>L.L. (%)</th>
<th>P.L. (%)</th>
<th>P.I. (%)</th>
<th>Specific Gravity</th>
<th>Unified Soil Classification</th>
<th>AASHTO Classification</th>
<th>FAA Classification</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>Auglaize Co., 39-75, Greenville Road</td>
<td>18.6</td>
<td>13.2</td>
<td>5.4</td>
<td>2.72</td>
<td>CL-ML</td>
<td>A-4</td>
<td>E-6</td>
</tr>
<tr>
<td>2</td>
<td>Carroll Co., 2-75, State Route 542</td>
<td>23.2</td>
<td>12.8</td>
<td>10.4</td>
<td>2.70</td>
<td>ML</td>
<td>A-4</td>
<td>E-6</td>
</tr>
<tr>
<td>3</td>
<td>Licking Co., 239-75, Top Layer, North 21st St.</td>
<td>23.9</td>
<td>17.7</td>
<td>6.2</td>
<td>2.63</td>
<td>ML-CL</td>
<td>A-4</td>
<td>E-6</td>
</tr>
<tr>
<td>4</td>
<td>Licking Co., 239-75, Bottom Layer, N. 21st St.</td>
<td>NP</td>
<td>NP</td>
<td>-</td>
<td>2.69</td>
<td>SM</td>
<td>A-2-4</td>
<td>E-2</td>
</tr>
<tr>
<td>5</td>
<td>Hamilton Co., 461-75, I-275</td>
<td>33.6</td>
<td>17.5</td>
<td>16.1</td>
<td>2.67</td>
<td>CL</td>
<td>A-6</td>
<td>E-7</td>
</tr>
<tr>
<td>6</td>
<td>Hancock Co., 7-75, State Route 12</td>
<td>18.4</td>
<td>9.3</td>
<td>9.1</td>
<td>2.69</td>
<td>CL-ML</td>
<td>A-4</td>
<td>E-6</td>
</tr>
<tr>
<td>7</td>
<td>Jefferson Co., 900-75, I-7</td>
<td>39.6</td>
<td>19.0</td>
<td>20.6</td>
<td>2.70</td>
<td>CL</td>
<td>A-6</td>
<td>E-7</td>
</tr>
<tr>
<td>8</td>
<td>Cuyahoga Co., --------, I-480</td>
<td>25.3</td>
<td>13.0</td>
<td>12.3</td>
<td>2.76</td>
<td>CL</td>
<td>A-6</td>
<td>E-5</td>
</tr>
<tr>
<td>9</td>
<td>Kaolin Clay</td>
<td>57.6</td>
<td>25.2</td>
<td>32.4</td>
<td>2.68</td>
<td>CH</td>
<td>A-7-6</td>
<td>E-8</td>
</tr>
</tbody>
</table>
Figure (4.3) Grain Size Distribution For Soils #1, 2, and 3
Figure 4.4 Grain Size Distribution for Soil #4, Licking Co., Bottom Layer
Figure (1.5) Grain Size Distribution For Soils # 5, 6, 7, and 8
Figure 4.6  Grain Size Distribution for Soil #9 (Kaolin Clay)
Figure 4.7 Compaction Curves
Figure 4.8  Strength Curves for Soils #1 through #7
For soil numbers 8 and 9, the compaction curves for different compaction energies (performed to obtain different values of dry density) are shown in Figures 4.2(a) and (b), & 4.10 (a) and (b) respectively. These compaction curves were compared to the typical set of "C" curves prepared by the Ohio Department of Transportation (see Table 4.2 below).

<table>
<thead>
<tr>
<th>Soil No.</th>
<th>Corresponding ODOT Curve (Set &quot;C&quot;)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>B</td>
</tr>
<tr>
<td>2</td>
<td>E</td>
</tr>
<tr>
<td>3</td>
<td>J</td>
</tr>
<tr>
<td>4</td>
<td>F</td>
</tr>
<tr>
<td>5</td>
<td>I-M</td>
</tr>
<tr>
<td>6</td>
<td>F</td>
</tr>
<tr>
<td>7</td>
<td>M</td>
</tr>
<tr>
<td>8*</td>
<td>H-K</td>
</tr>
<tr>
<td>9*</td>
<td>V</td>
</tr>
</tbody>
</table>

* Corresponds to a compaction energy of 25 blows/layer

It should be noted that the higher the soil rank (higher dry density with lower optimum moisture content), the lower the ultimate unconfined strength. This ultimate strength increases with increasing the percentage of clay in the soil up to a certain limit.
UNCONFINED COMPRESSIVE STRENGTH, psi

Figure 4.9(a) Compaction Curve for Soil #8

DRY DENSITY, pcf

Figure 4.9(b) Strength Curve for Soil #8
Figure 4.10(a) Compaction Curve for Soil #9 (Kaolin Clay)

Figure 4.10(b) Strength Curve for Soil #9 (Kaolin Clay)
5.1 SUBGRADE RESIDUAL CHARACTERISTICS

5.1.1 Cyclic Stress - Plastic Strain Relationship:

Practically, it is not possible to measure the stress - plastic strain relationship during a single loading cycle. However, it is thought that the multiple - step dynamic test, illustrated in section 3.4.2, could be representative of such a relationship (bh). This test, however, does not measure the stress and strain during a specific cycle; rather it measures these quantities over different cycles of different stress levels assuming that the sample does not undergo any structure variations during those cycles applied prior to a certain measurement. Nevertheless, the conditioning loading until stable cycling may account for that problem.

Figures (5.1a) and (5.1b) show the result of that test on two samples of each soil investigated (# 6 & 7). The samples have a fairly linear stress ($\sigma$) - plastic strain ($\varepsilon_p$) relationship for the stress range of 10 to 45 psi, preceded by initial concave upward curve.

For stress levels higher than 45 psi, and before failure conditions, the non-linearity becomes more apparent. At very high stress the samples undergo a strain softening process leading to excessive plastic deformation which may lead to fatigue failure of the sample. Such cases are
Fig (5.1a) Stress-Plastic Strain Curves
Fig (5.1b) Stress-Plastic Strain Curves
presented by the underlined points in figures (5.2a) and (5.2b). These strain softening conditions are beyond the scope of this investigation which is mainly concerned with the stress levels experienced under normal traffic loading in a flexible pavement structure. Regardless of these failure-leading conditions, figures (5.2a) and (5.2b) show fairly linear relationship between \( \log \sigma \) and \( \log \varepsilon_p \) which establish a power function between the stress and plastic strain during a loading cycle. This conclusion supports the expression presented by equation (2.28).

Figures (5.3a) and (5.3b) show the same relation for sample #656. For this sample the plastic deformations were measured by applying one loading cycle at each stress level. Although the non-linearity is more apparent in this curve, figure (5.3a), the power relationship (2.28) appears to be applicable to this sample as well, figure (5.3b).

5.1.2 Dynamic Creep

First, it should be pointed out that the test results in this research, as well as those of previous studies (3), (5) and (6), agreed to a high degree of accuracy with the equation (2.45) for all non-failure testing conditions. The correlation coefficient for the linear regression of \( \log \varepsilon_p /N \) vs. \( \log N \) relationship was always greater than 0.98. A failure condition (see Figure 5.4) shows an increasing rate of permanent deformation with loading cycles until the sample fails due to the excessive deformations. Such a case is not included in the analysis since it is not the working condition for the roadway being considered in this study.
Fig (5.2a) Stress-Plastic Strain Relationship

- O Sample # 657
- • Sample # 655

\[ \sigma \text{ (psi)} \]

\[ \varepsilon_p (\mu \text{in/in}) \]
Fig (5.2b) Stress-Plastic Strain Relationship
Fig (5.3a) Stress-Plastic Strain Curve with One Applied Loading Cycle (sample # 656)

\( \sigma_{\text{ult}} = 150.2 \text{ psi} \)
Fig (5.3b) Stress-Plastic Strain Relationship for Sample # 656
Figure 5.4 Typical log $\varepsilon_p/N$ vs. log $N$ Curve
From Equation (2.45), it is obvious that complete evaluation of the rutting parameters "A" and "m" is essential and sufficient to yield a complete predictive model for permanent deformation in subgrade soils. Consequently, in the following sections, the variations of these two parameters will be discussed. A summary of the test results which are analyzed in the following sections is tabulated in Appendix A.

1. **Parameter "m"**

This parameter is measured, for each dynamically tested sample, as the absolute value of the slope of $\log (\varepsilon/N)$ vs. $\log N$ linear relationship. In previous studies, Khedr [5] and Bayomy [6] concluded that parameter "m" is almost constant for each soil investigated. Furthermore, Bayomy found no significant difference in average "m" values when compared for different soils. The same observation is valid for the soils investigated in this study. The values of parameter "m" are plotted against dynamic modulus, $E^*$, for the soils investigated (Figures 5.5 and 5.5a).

It should be pointed out that these figures are plotted without regard to the stress level. It can be seen from these figures that all "m" values are in the range of 0.83 to 0.94 for all soils, except under those conditions in which the tested sample was very weak (high moisture content, low dry density and low dynamic modulus), and under high applied stress. These exceptional cases are those close-to-failure conditions in which the parameter "m" may be as low as 0.7.

The dynamic modulus which characterizes the soil sample has no significant influence on parameter "m" except for very low modulus values.
Fig (5.5) $m^n$ Variation for Saturated and As-Compacted Conditions
(Soil # 6)
Figure 5.5 Parameter "m" vs. E* for Soils #1 Through #7
To verify this observation statistically, an F-test was performed on each set of soil data to determine whether including the independent variable (dynamic modulus) would account for the variability in "m" in the linear model. To conduct the test, the following definitions were made:

\[ \text{hypothesis } C_1: \beta_1 = 0 \]
\[ \text{hypothesis } C_2: \beta_1 \neq 0 \]

where \( \beta_1 \) is the independent variable coefficient in a linear model and

\[ F^* = \frac{MSR}{MSE} \]

where

\( MSR = \text{Regression Mean Square} \)
\( MSE = \text{Error Mean Square} \)

The F-values were read from statistical tables according to the number of samples tested for each soil (n) and level of significance, \( \alpha = 0.05 \). Thence,

\[ \text{if } F^* < F(1 - \alpha ; 1, n-2), \text{ conclude } C_1 \]
\[ \text{if } F^* \geq F(1 - \alpha ; 1, n-2), \text{ conclude } C_2 \]

Table 5.1 shows a summary of the statistical data and supports the conclusion that for most soils tested, the dynamic modulus does not account for variation in "m". However, for those soils which show a trend of "m" variation with \( E^* \), the variability of "m" values occur within a narrow range as can be seen from the standard deviations. Generally, the
trend -- if it exists -- is that "m" increases slightly with increasing dynamic modulus. It should be noted that the value of "m" cannot exceed 1.0. Similar conclusions were obtained when considering all possible influencing variables, such as moisture content, dry density and stress level as the independent variable in the above statistical test.

Effects of saturation/drying processes on parameter "m" was investigated by Majidzadeh et al (25). They found that this procedure does not have significant effect on m - values. This observation was found valid when studying the test results of saturated samples of soil #6 (sandy silt from Hancock county), figure (5.5). However, as expected the weaker saturated samples with high applied stress level tend to have lower values of m.

Generally, it can be concluded that "m" is a constant and universal parameter for the domain of soils and variations of influencing factors investigated. This conclusion is considered sound from both the statistical and engineering points of view.

2. Parameter "A"

According to Equation (2.5) parameter "A" is defined as the antilog of the intersection of the linear relationship of log $\varepsilon_p/N$ vs. log N; i.e., it is equivalent to the permanent strain after the first applied loading cycle (N = 1). It is determined through linear regression of the data in accordance with that relationship. Parameter "A" is solely responsible for expressing the dynamic creep behavior, since parameter "m" has been proved to be almost constant. According to definition in equation (2.5a), A is believed to be functions of:
<table>
<thead>
<tr>
<th>Soil No. and County</th>
<th>No. of samples</th>
<th>Mean</th>
<th>Standard Deviation</th>
<th>F*</th>
<th>F</th>
</tr>
</thead>
<tbody>
<tr>
<td>#1, Auglaize Co.* (Unsat.)</td>
<td>36</td>
<td>0.884</td>
<td>0.0249</td>
<td>0.1479</td>
<td>4.17</td>
</tr>
<tr>
<td></td>
<td>(Saturated)</td>
<td>29</td>
<td>0.892</td>
<td>0.0255</td>
<td>4.012</td>
</tr>
<tr>
<td>#2, Carroll Co.* (Unsaturated)</td>
<td>19</td>
<td>0.883</td>
<td>0.0416</td>
<td>0.3813</td>
<td>4.26</td>
</tr>
<tr>
<td></td>
<td>(Saturated)</td>
<td>8</td>
<td>0.888</td>
<td>0.0231</td>
<td>0.0122</td>
</tr>
<tr>
<td>#3, Licking Co.*</td>
<td>19</td>
<td>0.895</td>
<td>0.0289</td>
<td>4.451</td>
<td>4.46</td>
</tr>
<tr>
<td>#4, Licking Co.*</td>
<td>18</td>
<td>0.859</td>
<td>0.0782</td>
<td>0.408</td>
<td>4.50</td>
</tr>
<tr>
<td>#5, Hamilton Co.* (Unsaturated)</td>
<td>35</td>
<td>0.839</td>
<td>0.0596</td>
<td>0.329</td>
<td>4.15</td>
</tr>
<tr>
<td></td>
<td>(Saturated)</td>
<td>4</td>
<td>0.769</td>
<td>0.0280</td>
<td>4.025</td>
</tr>
<tr>
<td>#6, Hancock Co., (Unsaturated)</td>
<td>42</td>
<td>0.842</td>
<td>0.0676</td>
<td>4.001</td>
<td>4.11</td>
</tr>
<tr>
<td></td>
<td>(Saturated)</td>
<td>13</td>
<td>0.822</td>
<td>0.0464</td>
<td>1.145</td>
</tr>
<tr>
<td>#7, Jefferson Co.</td>
<td>37</td>
<td>0.828</td>
<td>0.0661</td>
<td>3.967</td>
<td>4.14</td>
</tr>
<tr>
<td>#8, Cuyahoga Co.*</td>
<td>78</td>
<td>0.862</td>
<td>0.0455</td>
<td>17.797</td>
<td>3.98</td>
</tr>
<tr>
<td>#9, Kaolin Clay*</td>
<td>81</td>
<td>0.876</td>
<td>0.0580</td>
<td>17.390</td>
<td>3.98</td>
</tr>
<tr>
<td>Franklin Co.**</td>
<td>18</td>
<td>0.860</td>
<td>0.0420</td>
<td>7.114</td>
<td>4.50</td>
</tr>
<tr>
<td>Cuyahoga Co.**</td>
<td>27</td>
<td>0.865</td>
<td>0.0375</td>
<td>4.021</td>
<td>4.25</td>
</tr>
<tr>
<td>Pike Co.***</td>
<td>16</td>
<td>0.809</td>
<td>0.0412</td>
<td>3.188</td>
<td>4.61</td>
</tr>
</tbody>
</table>

**Undisturbed samples (After Reference (5))

***Undisturbed samples from field study case (see Chapter VII)

* Reference (5) and (6)
(a) the soil's moisture content (as compacted or saturated);
(b) the soil's dry density;
(c) soil structure;
(d) applied dynamic stress level (also, its frequency and function shape); and
(e) ultimate compressive strength

However, some of these factors are mutually intercorrelated. Statistical multiple regression and stepwise regression analyses were conducted to determine the most influencing factors and those which can be deleted due to high mutual intercorrelations. Table 5.2 shows a summary of the multiple regression analysis. The table shows the correlation coefficients as a measure of the effectiveness of different factors on the parameter "A".

The dynamic modulus, $E^*$, appears to be the most influential factor for most of the soils. The effect of the ultimate compressive strength ($\sigma_{\text{ult}}$) and water content (W/C) are also noticeable. However, the high intercorrelation coefficients between these two factors and dynamic modulus, $E$ (Table 5.2) may imply that the inclusion of $E^*$ in the regression model will take care of their effect. In other words, the dynamic modulus may be thought to account for the effect of these factors.

From the engineering point of view, the stress level, $\sigma_{\text{appl}}$, should be considered separately, although it may appear from Table 5.5 that it is not very effective.
TABLE 5.2
CORRELATION COEFFICIENTS OF INFLUENCING PARAMETERS
ON RUTTING PARAMETER "A"

<table>
<thead>
<tr>
<th>Soil #</th>
<th>$E^*$</th>
<th>$\sigma_{ult}$</th>
<th>$\sigma_{appl}$</th>
<th>$\sigma_{appl}/\sigma_{ult}$</th>
<th>Moisture Content</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>-0.719</td>
<td>-0.603</td>
<td>0.017</td>
<td>0.490</td>
<td>0.637</td>
</tr>
<tr>
<td>2</td>
<td>-0.792</td>
<td>-0.851</td>
<td>0.229</td>
<td>0.76</td>
<td>0.785</td>
</tr>
<tr>
<td>3</td>
<td>-0.878</td>
<td>-0.915</td>
<td>0.210</td>
<td>0.660</td>
<td>0.857</td>
</tr>
<tr>
<td>4</td>
<td>-0.644</td>
<td>-0.668</td>
<td>0.113</td>
<td>0.720</td>
<td>0.724</td>
</tr>
<tr>
<td>5</td>
<td>-0.608</td>
<td>-0.551</td>
<td>0.174</td>
<td>0.674</td>
<td>0.512</td>
</tr>
<tr>
<td>6</td>
<td>-0.658</td>
<td>-0.625</td>
<td>0.313</td>
<td>0.695</td>
<td>0.662</td>
</tr>
<tr>
<td>7</td>
<td>-0.792</td>
<td>-0.695</td>
<td>0.0644</td>
<td>0.779</td>
<td>0.720</td>
</tr>
<tr>
<td>8</td>
<td>-0.768</td>
<td>-0.389</td>
<td>0.173</td>
<td>0.301</td>
<td>0.731</td>
</tr>
<tr>
<td>9</td>
<td>-0.826</td>
<td>-0.352</td>
<td>0.276</td>
<td>0.282</td>
<td>0.704</td>
</tr>
</tbody>
</table>

Note: $A$ and $E^*$ are in the logarithm form.
TABLE 5.2 (cont'd)
INTERCORRELATION COEFFICIENTS OF E* AND OTHER FACTORS

<table>
<thead>
<tr>
<th>Soil #</th>
<th>$\sigma_{\text{ult}}$</th>
<th>$\sigma$</th>
<th>$\sigma/\sigma_{\text{ult}}$</th>
<th>Moisture Content</th>
</tr>
</thead>
<tbody>
<tr>
<td>1</td>
<td>0.523</td>
<td>0.404</td>
<td>-0.149</td>
<td>-0.633</td>
</tr>
<tr>
<td>2</td>
<td>0.856</td>
<td>0.220</td>
<td>0.470</td>
<td>-0.747</td>
</tr>
<tr>
<td>3</td>
<td>0.912</td>
<td>0.371</td>
<td>-0.474</td>
<td>-0.891</td>
</tr>
<tr>
<td>4</td>
<td>0.856</td>
<td>0.159</td>
<td>-0.590</td>
<td>-0.771</td>
</tr>
<tr>
<td>5</td>
<td>0.881</td>
<td>0.289</td>
<td>-0.631</td>
<td>-0.634</td>
</tr>
<tr>
<td>6</td>
<td>0.854</td>
<td>0.313</td>
<td>-0.305</td>
<td>-0.750</td>
</tr>
<tr>
<td>7</td>
<td>0.882</td>
<td>0.268</td>
<td>-0.633</td>
<td>-0.791</td>
</tr>
<tr>
<td>8</td>
<td>0.513</td>
<td>0.301</td>
<td>0.061</td>
<td>-0.871</td>
</tr>
<tr>
<td>9</td>
<td>0.419</td>
<td>0.116</td>
<td>0.0222</td>
<td>-0.800</td>
</tr>
</tbody>
</table>
It was found that parameter "A" decreases with increasing dry density, decreasing dynamic stress intensity, and lower moisture content. The effects of dry density and stress intensity are more significant at high moisture contents. The function "A" is more sensitive to stress variation at high stress levels. Also, the rate of its increase with higher moisture contents is significantly greater for moisture contents on the wet side of optimum. "A" may change from the value of $1 \times 10^{-5}$ for dry, dense samples under low applied stress to as high as $750 \times 10^{-5}$ for wet, less dense samples subjected to high stress intensities.

During the study of the effects of those factors, relative scattering of the points was observed. This scatter is attributed to structural variations in the samples tested. These variations are due to the micro-arrangements of soil particles throughout the body of the sample. To account for this factor, and from the statistical analysis (Table 5.2), parameter "A" is investigated against the dynamic modulus.

Khedr(5) suggested a power relationship between the parameter $A$ and dynamic modulus $E^*$ at constant applied stress to represent the testing data of soils number 8 and 9 and also of undisturbed field samples, equation (2.19). In a follow-up study, Bayomy (6) introduced the effect of stress level in exponential form, equation (2.22).

However, in this study a more comprehensive statistical study is to be conducted to establish the representative model of parameter $A$ in terms of the most influential factors. Therefore, in view of table 5.2, the following forms of multiple regression analyses are suggested:
A = S_1' (E*)S_2' \quad (a)

A = S_1'' (E*)S_2'' \exp (S_3''(\sigma_{\text{appl}}/\sigma_{\text{ult}})) \quad (b)

A = S_1''' (E*)S_2''' (\sigma_{\text{appl}})^{S_3'''} (w)^{S_4'''} \quad (c)

A = S_1'''' (E*)S_2'''' \exp (S_3''''(\sigma_{\text{appl}}/\sigma_{\text{ult}})) \times \exp (S_4'''' w) \quad (d)

where E*, \sigma_{\text{appl}}, and \sigma_{\text{ult}} are in pounds per square inch and w is in percentage. The equation form (a) was chosen as a reference form in which the data were analyzed regardless of applied stress level. The results of the analyses are summarized in Table 5.3.

The multiple correlation coefficients were used to assess the comparative effectiveness of these function forms to account for the variability of the parameter "A". From Table 5.3, it can be seen that the inclusion of the stress level, \sigma_{\text{appl}} or (\sigma_{\text{appl}}/\sigma_{\text{ult}}), form (a) to form (b), improves the correlation coefficients; i.e., form (b) is more representative than form (a). Upon introducing the water content (w), form (d), the correlation coefficients improve slightly. The improvement is small compared with that when including the stress level. However, form (c) is proved to be the most representative among all the tested forms. Soil #3 was the only soil which did not follow these analyses; rather, form (b) was the best representative form and including the moisture content factor did not improve the multiple correlation coefficient significantly. Table 5.3 (cont.) shows the values of the constant, S_1'.
<table>
<thead>
<tr>
<th>Model Form (see p. 98)</th>
<th>Soil Number</th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>1</td>
<td>2</td>
<td>3</td>
<td>4</td>
<td>5</td>
<td>6</td>
<td>7</td>
</tr>
<tr>
<td>a</td>
<td>0.719</td>
<td>0.792</td>
<td>0.878</td>
<td>0.644</td>
<td>0.608</td>
<td>0.658</td>
<td>0.782</td>
</tr>
<tr>
<td>b</td>
<td>0.772</td>
<td>0.906</td>
<td>0.921</td>
<td>0.772</td>
<td>0.714</td>
<td>0.838</td>
<td>0.864</td>
</tr>
<tr>
<td>c</td>
<td>0.832</td>
<td>0.922</td>
<td>0.899</td>
<td>0.797</td>
<td>0.736</td>
<td>0.871</td>
<td>0.848</td>
</tr>
<tr>
<td>d</td>
<td>0.806</td>
<td>0.915</td>
<td>0.927</td>
<td>0.789</td>
<td>0.718</td>
<td>0.839</td>
<td>0.866</td>
</tr>
</tbody>
</table>
Table (5.3) (cont'd) Values of Regression Coefficients, $S_i$, in the
Model Forms (a), (b), (c) and (d)

<table>
<thead>
<tr>
<th>Model Form</th>
<th>Soil Number</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td>$S_1'$</td>
<td>398.107</td>
<td>101.4</td>
<td>1238.797</td>
<td>1666.1</td>
<td>0.272</td>
<td>3.458</td>
<td>0.910</td>
<td>374.5</td>
<td>5976.4</td>
</tr>
<tr>
<td></td>
<td>$S_2''$</td>
<td>-1.843</td>
<td>-1.196</td>
<td>-1.884</td>
<td>-1.467</td>
<td>-0.688</td>
<td>-0.862</td>
<td>-0.838</td>
<td>-1.412</td>
<td>-1.710</td>
</tr>
<tr>
<td></td>
<td>Std. E#</td>
<td>0.240</td>
<td>0.184</td>
<td>0.235</td>
<td>0.436</td>
<td>0.148</td>
<td>0.132</td>
<td>0.115</td>
<td>0.127</td>
<td>0.124</td>
</tr>
<tr>
<td>b</td>
<td>$S_1''$</td>
<td>4242.9</td>
<td>1.076</td>
<td>2435.7</td>
<td>0.387</td>
<td>0.00392</td>
<td>0.134</td>
<td>0.0168</td>
<td>97.2</td>
<td>794.5</td>
</tr>
<tr>
<td></td>
<td>$S_2''$</td>
<td>-1.694</td>
<td>-0.843</td>
<td>-1.565</td>
<td>-0.757</td>
<td>-0.344</td>
<td>-0.644</td>
<td>-0.516</td>
<td>-1.371</td>
<td>-1.598</td>
</tr>
<tr>
<td></td>
<td>Std. E</td>
<td>0.226</td>
<td>0.147</td>
<td>0.224</td>
<td>0.463</td>
<td>0.170</td>
<td>0.111</td>
<td>0.121</td>
<td>0.110</td>
<td>0.107</td>
</tr>
<tr>
<td></td>
<td>Std. E</td>
<td>1.128</td>
<td>1.116</td>
<td>1.261</td>
<td>3.001</td>
<td>1.084</td>
<td>0.774</td>
<td>1.787</td>
<td>0.539</td>
<td>0.438</td>
</tr>
<tr>
<td>d</td>
<td>$S_1'''$</td>
<td>0.00415</td>
<td>0.00562</td>
<td>0.274</td>
<td>0.000120</td>
<td>0.00543</td>
<td>0.150</td>
<td>0.00229</td>
<td>11.43</td>
<td>0.00160</td>
</tr>
<tr>
<td></td>
<td>$S_2'''$</td>
<td>-1.538</td>
<td>1.082</td>
<td>-1.458</td>
<td>-0.771</td>
<td>-0.710</td>
<td>-1.103</td>
<td>-0.792</td>
<td>-1.558</td>
<td>-1.499</td>
</tr>
<tr>
<td></td>
<td>Std. E</td>
<td>0.275</td>
<td>0.193</td>
<td>0.480</td>
<td>0.549</td>
<td>0.167</td>
<td>0.153</td>
<td>0.158</td>
<td>0.173</td>
<td>0.140</td>
</tr>
<tr>
<td></td>
<td>$S_3'''$</td>
<td>-1.348</td>
<td>1.803</td>
<td>0.502</td>
<td>1.028</td>
<td>0.695</td>
<td>1.537</td>
<td>0.648</td>
<td>1.207</td>
<td>1.171</td>
</tr>
<tr>
<td></td>
<td>Std. E</td>
<td>0.349</td>
<td>0.381</td>
<td>0.414</td>
<td>0.504</td>
<td>0.208</td>
<td>0.199</td>
<td>0.213</td>
<td>0.136</td>
<td>0.121</td>
</tr>
<tr>
<td></td>
<td>$S_4'''$</td>
<td>4.635</td>
<td>1.723</td>
<td>2.95</td>
<td>3.224</td>
<td>0.793</td>
<td>0.673</td>
<td>1.723</td>
<td>0.474</td>
<td>2.930</td>
</tr>
<tr>
<td></td>
<td>Std. E</td>
<td>1.211</td>
<td>0.891</td>
<td>2.235</td>
<td>1.535</td>
<td>0.238</td>
<td>1.071</td>
<td>1.198</td>
<td>0.609</td>
<td>1.101</td>
</tr>
<tr>
<td></td>
<td>$S_1''''$</td>
<td>1.441</td>
<td>0.0349</td>
<td>3.226</td>
<td>0.00221</td>
<td>0.00141</td>
<td>0.395</td>
<td>0.00395</td>
<td>0.0681</td>
<td>0.000982</td>
</tr>
<tr>
<td></td>
<td>$S_2''''$</td>
<td>-1.216</td>
<td>-0.646</td>
<td>-1.133</td>
<td>-0.392</td>
<td>-0.289</td>
<td>-0.696</td>
<td>-0.450</td>
<td>-0.836</td>
<td>-0.897</td>
</tr>
<tr>
<td></td>
<td>Std. E</td>
<td>0.271</td>
<td>0.191</td>
<td>0.431</td>
<td>0.590</td>
<td>0.192</td>
<td>0.151</td>
<td>0.160</td>
<td>0.204</td>
<td>0.173</td>
</tr>
<tr>
<td></td>
<td>Std. E</td>
<td>1.06</td>
<td>1.173</td>
<td>1.279</td>
<td>3.436</td>
<td>1.126</td>
<td>0.932</td>
<td>1.894</td>
<td>0.520</td>
<td>0.429</td>
</tr>
<tr>
<td></td>
<td>$S_4''''$</td>
<td>0.430</td>
<td>0.169</td>
<td>0.195</td>
<td>0.207</td>
<td>0.035</td>
<td>-0.0525</td>
<td>0.0454</td>
<td>0.179</td>
<td>0.252</td>
</tr>
<tr>
<td></td>
<td>Std. E</td>
<td>0.152</td>
<td>0.108</td>
<td>0.167</td>
<td>0.207</td>
<td>0.054</td>
<td>0.111</td>
<td>0.0705</td>
<td>0.0587</td>
<td>0.052</td>
</tr>
</tbody>
</table>

# Standard error in determining the coefficient
Although forms other than (a), (b), (c) and (d) have been tested and found to be insignificant, it should be pointed out that conclusions of this analysis are not unique. This conclusion, however, supports the theoretical derivation of section 2.2.1, and the assumption established in equation (2.28) of the existence of a power relationship between applied stress and plastic strain during cyclic loading of specific soil structure. It should be pointed out here again that parameter $A$, in fact, represents permanent strain after the first cycle, although it is obtained through regression analysis of the testing data.

To simplify the problem for practical design purposes, and since the effect of water content can be incorporated in the dynamic modulus, the following constitutive equations may be suggested:

$$A = R_1 \left( E^* \right)^{-S_1} \exp \left( \frac{\sigma_{ap} \sigma \text{ult}}{\sigma} \right)$$ \hspace{1cm} (5.2)

or

$$A = R_2 \left( E^* \right)^{-S_2} (\sigma)$$ \hspace{1cm} (5.3)

where $R_1$, $S_1$, $R_2$ and $S_2$ are material constants independent of physical properties or applied stress level. That is, they are constant for specific soils under any testing conditions.

Equation (5.2) or (5.3) represents a master curve for each type of soil. Once the constants, $R$ and $S$, are determined for that soil, parameter "A" can be determined directly as a function of dynamic modulus and stress level.
To check the linearity of Equation (5.2) and (5.3), statistical F-tests were performed on linear and quadratic models between \( \log(\frac{A}{\exp(\sigma_{\text{appl}}/\sigma_{\text{ult}})}) \) and \( \log(\sigma^*) \) for the former model, and \( \log(\frac{A}{\sigma_{\text{appl}}}) \) versus \( \log(\sigma^*) \) for the latter model. This statistical test is to be done as follows:

1. Fit the full model (second degree) and obtain the error sum of squares. \( \text{SSE}(F) \)
2. Fit the reduced model (first degree) and calculate the error sum of squares \( \text{SSE}(R) \)
3. Test the hypotheses

\[ C_1 : B_2 = \text{coefficient of the second degree term} = 0 \]
\[ C_2 : B_2 = 0 \]

by comparing the calculated error sums of squares. This is attained by the statistic \( F^* \),

\[
F^* = \frac{\text{SSE}(R) - \text{SSE}(F)}{\frac{df_r - df_f}{df_r}} \cdot \frac{\text{SSE}(F)}{df_f}
\]

where \( df_r \) and \( df_f \) are the degrees of freedom of each model.

In our case:

\( df_f = n-2 \) and \( df_r = n-3 \)

(\( n \) is number of cases (samples) of each set (soil))

Then compare \( F^* \) with standard statistic \( F \) corresponding to a
confidence level $\alpha = 0.05$,

If $F_\alpha^* \leq F(1 - \alpha; 1, n-3)$, conclude $C_1$

$F^* > F(1 - \alpha; 1, n - 3)$, conclude $C_2$

Hypothesis $C_1$ implies that the additional higher degree term (second degree in our analysis) does not help significantly to reduce the variation of the tests observations around the fitted regression line.

Table 5.4 shows a summary of these tests as well as the typical values of constants $R$ and $S$ for the soils investigated in this study. The table shows that linearity is accepted to represent the test data. There is one exception of this conclusion; soil #6 for model (5.2).

By comparison between correlation coefficients in tables (5.3) and (5.4), it appears that (5.2) and (5.3) are adequate to represent the data compared to the full models (5.1a,b,c & d). It can be seen that considering (5.3) instead of (5.1c) will not significantly affect the accuracy of representation. In fact, in cases of soils number 2,3,5 and 3, the correlation coefficients implies improvement in data fitting. No explanation could be found for this observation. This conclusion supports the fact that the effect of moisture content is accounted for by considering the dynamic modulus. It was also noticed that placing $S_3 = 1$ in equation (5.1c) did not affect the correlation coefficients significantly.

Also comparing correlation coefficients of model (5.2) and models (5.1b and d) concludes similar findings. However, the correlation coefficients of (5.3) is usually higher than those of (5.2). That is, model (5.3) is considered more representative the testing data than
TABLE 5.4  STATISTICAL DATA FOR MODELS (5.2) AND (5.3) 
APPLIED TO THE SOILS UNDER INVESTIGATION

<table>
<thead>
<tr>
<th>Model</th>
<th>Soil #1</th>
<th>Soil #2</th>
<th>Soil #3</th>
<th>Soil #4</th>
<th>Soil #5</th>
<th>Soil #6</th>
<th>Soil #7</th>
<th>Soil #8</th>
<th>Soil #9</th>
</tr>
</thead>
<tbody>
<tr>
<td>(5.2)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( R_1 )</td>
<td>14,878</td>
<td>119.5</td>
<td>38,918</td>
<td>370.2</td>
<td>0.0476</td>
<td>0.626</td>
<td>0.533</td>
<td>3.717</td>
<td>174.93</td>
</tr>
<tr>
<td>( S_1 )</td>
<td>1.763</td>
<td>1.227</td>
<td>1.769</td>
<td>1.345</td>
<td>0.536</td>
<td>0.725</td>
<td>0.795</td>
<td>0.976</td>
<td>1.382</td>
</tr>
<tr>
<td>Correl. Coeff.</td>
<td>0.739</td>
<td>0.911</td>
<td>0.924</td>
<td>0.672</td>
<td>0.602</td>
<td>0.749</td>
<td>0.780</td>
<td>0.692</td>
<td>0.709</td>
</tr>
<tr>
<td>( F^* )</td>
<td>3.783</td>
<td>0.026</td>
<td>1.181</td>
<td>0.604</td>
<td>0.003</td>
<td>7.965</td>
<td>0.928</td>
<td>0.712</td>
<td>1.142</td>
</tr>
<tr>
<td>(5.3)</td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>( R_2 )</td>
<td>30,471</td>
<td>40.55</td>
<td>65,304</td>
<td>377.7</td>
<td>0.0782</td>
<td>0.890</td>
<td>0.382</td>
<td>1443.5</td>
<td>371.94</td>
</tr>
<tr>
<td>( S_2 )</td>
<td>2.093</td>
<td>1.370</td>
<td>2.085</td>
<td>1.585</td>
<td>0.856</td>
<td>0.992</td>
<td>0.973</td>
<td>1.883</td>
<td>1.739</td>
</tr>
<tr>
<td>Correl. Coeff.</td>
<td>0.816</td>
<td>0.952</td>
<td>0.937</td>
<td>0.745</td>
<td>0.763</td>
<td>0.882</td>
<td>0.852</td>
<td>0.925</td>
<td>0.904</td>
</tr>
<tr>
<td>( F^* )</td>
<td>0.630</td>
<td>0.036</td>
<td>0.081</td>
<td>0.050</td>
<td>1.472</td>
<td>1.456</td>
<td>1.300</td>
<td>0.319</td>
<td>1.524</td>
</tr>
<tr>
<td>( F (\alpha = 0.05) )</td>
<td>4.051</td>
<td>2.30</td>
<td>2.40</td>
<td>2.75</td>
<td>4.13</td>
<td>4.05</td>
<td>4.15</td>
<td>3.98</td>
<td>3.97</td>
</tr>
</tbody>
</table>

\( F (\alpha = 0.05) \) values are used to test the significance of the correlation coefficients.
Data obtained by Khedr (5) for undisturbed field samples were analyzed according to the same procedure and the proposed model was applicable to this data as well.

Figures 5.6 and 5.7 summarize the relations (5.2) and (5.3) for the soils investigated. It should be noted that the constants R and S are not direct indicators of the rutting potential for soils, since they are interrelated from the statistics point of view.

To study the effect of saturation process on parameter "A", samples of soil #6 were saturated and retested dynamically. The pertinent results are shown in appendix A. Conclusions found were similar to those suggested by Khedr (5) and verified by Bayomy (6). They can be summarized as: (1) after saturation, a sudden increase occurs in the rate of permanent deformation accumulation, (2) the saturated sample follows equation (2.45) in its dynamic creep behavior, and (3) the saturated condition governs over the as-compacted condition. It should be pointed out that all previous analyses of models (5.1), (5.2) and (5.3) included all as-compacted and saturated sample results. Therefore, the variation of parameter "A" due to the saturation process follows, and can be predicted by, equation (5.2) and (5.3).

3. Adjusted Parameter "A":

Both parameters "A" and "m" are calculated through linear regression of log \( \left[ \varepsilon_p/N \right] \) versus log N. The discussion of parameter "m" variation
Figure 5.6 Plots of Log E* vs. Log A/e^ult.
Figure 5.7 Plots of Log A/a versus Log E* for All Soils Studied
leads to the conclusion that it is a constant parameter. As expected of any laboratory-measured quantity, there are variations of "m" values even though it ranges within relatively narrow limits. However, the statistical regression is performed without restrictions, i.e. with \( (n - 2) \) degrees of freedom (\( n \) = number of test points measured during the dynamic creep of the sample). Accordingly, the calculated parameters "A" and "m" are intercorrelated.

If the regression procedure is restricted to \( (n - 1) \) degree of freedom by using constant "m_c" for each soil, and the least square regression method is employed, the adjusted "A_a" can be expressed as,

\[
A_a = \log^{-1} \left( \frac{\sum_{i=1}^{n} y_i - m_c \sum_{i=1}^{n} x_i}{n} \right)
\]

(5.4)

where

- \( m_c \) = average value of parameter "m" for the soil considered
- \( A_a \) = adjusted parameter A due to restricting parameter "m" to the value \( m_c \)
- \( x_i, y_i \) = horizontal and vertical coordinates of point \( (i) \) on the dynamic creep curve expressed as \( (\log (p/N) \) vs. \( (\log N) \)

\( n \) = number of points recorded on the dynamic creep curve.

Parameter \( A_a \) could be approximated using original values of \( A, m \) and \( m_c \).

Figure (5.3) shows the original regressed line, I, and the adjusted line when using \( m_c \), II. By integrating the area between those two lines
Figure (5.8) Adjusting Parameter "A" for the variation of Parameter "m"
over the life span considered and setting the integration equal to zero. "A_a" can be calculated for line II which will have the minimum differences from line I. A_a can be expressed as,

$$A_a = A \left(1 + N_1\right) \frac{(m_c - m)}{2} \quad (5.5)$$

where A and m are the originally calculated parameters, and N_1 is the considered life span.

The analyses considered for parameter A: models (5.1), (5.2) and (5.3), were applied to the variation of A_a. Table (5.5) presents the correlation coefficient of the multiple regression analysis of models (5.1a, b, c & d), applied to the adjusted parameter A_a as the dependent variable. Improvement of all correlation coefficients for all models and soils considered was observed upon comparing tables 5.2 and 5.5. That is, considering "A_a" instead of "A" improves the representation of the testing data by the proposed models. This fact may indirectly support the conclusion that "m" is constant, and considering "A_a" will count for any variation in "m" that may occur.

Furthermore, table (5.6) shows the statistical data for the testing results analyzed according to the simplified models (5.2) and (5.3) when considering parameter "A_a". Again, tables (5.6) and (5.4) show higher correlations for "A_a". The conclusions of linearity and comparison between the two models (5.2) and (5.3), derived from table (5.6), applies also to the analysis of "A_a", table (5.6). Figures (5.9) and (5.10) show, as examples, the computer output plots for the linear regression according
Table (5.5) Multiple Correlation Coefficients of Regression Models with \( A_a \) Dependent variable

<table>
<thead>
<tr>
<th>Model</th>
<th>Soil #</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
</tr>
</thead>
<tbody>
<tr>
<td>a</td>
<td></td>
<td>0.763</td>
<td>0.943</td>
<td>0.916</td>
<td>0.795</td>
<td>0.675</td>
<td>0.801</td>
<td>0.880</td>
<td>0.810</td>
<td>0.849</td>
</tr>
<tr>
<td>b</td>
<td></td>
<td>0.806</td>
<td>0.971</td>
<td>0.946</td>
<td>0.845</td>
<td>0.846</td>
<td>0.852</td>
<td>0.942</td>
<td>0.870</td>
<td>0.897</td>
</tr>
<tr>
<td>c</td>
<td></td>
<td>0.830</td>
<td>0.960</td>
<td>0.930</td>
<td>0.862</td>
<td>0.811</td>
<td>0.876</td>
<td>0.928</td>
<td>0.936</td>
<td>0.948</td>
</tr>
<tr>
<td>d</td>
<td></td>
<td>0.817</td>
<td>0.972</td>
<td>0.948</td>
<td>0.873</td>
<td>0.869</td>
<td>0.856</td>
<td>0.945</td>
<td>0.900</td>
<td>0.931</td>
</tr>
</tbody>
</table>
Table (5.6) Statistical Data for Models
(5.2) & (5.3) Using "A_a" instead of "A"

<table>
<thead>
<tr>
<th>Model</th>
<th>Soil #</th>
<th>1</th>
<th>2</th>
<th>3</th>
<th>4</th>
<th>5</th>
<th>6</th>
<th>7</th>
<th>8</th>
<th>9</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>5.2</td>
<td>R&lt;sub&gt;1&lt;/sub&gt;</td>
<td>13,855</td>
<td>2414</td>
<td>15,790</td>
<td>3,791</td>
<td>0.0502</td>
<td>1.611</td>
<td>3.446</td>
<td>479.1</td>
<td>10,505</td>
</tr>
<tr>
<td></td>
<td>S&lt;sub&gt;1&lt;/sub&gt;</td>
<td>1.757</td>
<td>1.282</td>
<td>1.680</td>
<td>1.588</td>
<td>0.543</td>
<td>0.827</td>
<td>0.986</td>
<td>1.480</td>
<td>1.823</td>
</tr>
<tr>
<td></td>
<td>Cor. Coef</td>
<td>0.771</td>
<td>0.949</td>
<td>0.925</td>
<td>0.791</td>
<td>0.658</td>
<td>0.809</td>
<td>0.885</td>
<td>0.828</td>
<td>0.877</td>
</tr>
</tbody>
</table>

| 5.3   | R<sub>1</sub> | 28,377 | 61.49 | 26,495 | 3,667 | 0.0832 | 1.944 | 2.466 | 387.5 | 2,385 |
|       | S<sub>1</sub> | 2.087 | 1.409 | 1.996 | 1.827 | 0.863 | 1.081 | 1.164 | 1.747 | 1.945 |
|       | Cor. Coef | 0.839 | 0.957 | 0.934 | 0.806 | 0.788 | 0.921 | 0.932 | 0.932 | 0.933 |
Fig. (5.9) $A_a/A_{appl}$ vs. $E^*$ on Log-Log scale (Soil # 7)
Fig. (5.10) $A_a / \delta_{\text{appl}}$ vs. $E^*$ on Log-Log scale

(Soil # 6)
to model (5.3) performed on the data obtained for soils # 6 & 7.

The practical significance and implementation of the found models will be discussed later in chapter VII.

5.2 Resilient characteristics

As mentioned previously, the characterization of the resilient parameter, modulus $E^r$ and Poisson's ratio $v$, is essential to use the theoretical and numerical methods of analyzing pavement structures. Moreover, it has been shown in the previous section that the residual and resilient properties of soils are intercorrelated. Therefore, it is appropriate to throw some light on these characteristics of the soils investigated and their variations with the influential factors of compaction conditions, saturation, soil type and applied stress level.

In this study only the resilient modulus will be considered to characterize the subgrade, since Poisson's ratio is known to have limited variations and it does not have significant effect on the final pavement analysis.

5.2.1 Effect of Compaction Conditions

The construction conditions experienced by the subgrade soil is of special importance since, by appropriate design and control of these conditions, the soil may be compacted so that it exhibits its best performance under dynamic traffic loading. The compaction condition forms the initial point of the soil's performance history. For a specific compaction energy, the dynamic modulus correlates with the moisture content at compaction, under constant applied dynamic stress, in an optimal curve
Dynamic Modulus, $E^*$, vs. Moisture Content at Constant Compaction Energy.

Figure (5.11) Dynamic Modulus vs. Moisture Content at Constant Compaction Energy.
However, the moisture content at the highest $E^*$ does not coincide with the optimum moisture (for maximum dry density); instead, it is on the dry side of the compaction curve. The effect of moisture content changes on the wet side of optimum is much greater than those on the dry side.

The sensitivity of the compaction water content differs from one soil to another. In some cases investigated in this study, the value of $E^*$ drops by a factor of 12 upon increasing the compaction moisture content by four percent over the optimum. Beyond four percent above optimum, however, the effect of increasing moisture content is rather insignificant. It should be mentioned, however, that this relation was not consistent for all the data. Some of the data did not show a specific trend within two percent (+) of the optimum.

In general, it can be concluded that the optimum dynamic performance of subgrade soils results at 1 to 1.5% less than the optimum moisture content; slight increases over that critical moisture content will result in substantial decreases in the moduli values.

Figure (5.12) illustrates the effect of dry density on the dynamic modulus of soils #8 and #9 for specific moisture contents and applied stress. The compaction energy was altered to obtain different dry densities; i.e., the dry density represents changes in the compaction energy in this figure. A linear relationship was observed between $E^*$ and $\gamma_d$ with other influencing factors kept constant. This relation can be expressed in the form:

$$E^* = S_1 + S_2 \gamma_d$$

(5.6)

where $S_1$ and $S_2$ are constants depending on stress level, moisture content and material type. The correlation coefficients obtained for five of the cases shown were between 0.85 and 0.99.
Figure (5.12) $E'$ vs. Dry Density (Different Compaction Energies)
It should be mentioned that some of the cases investigated did not comply well with this relation. The case of soil #9, for example, with $\sigma = 9.75$ psi and moisture content = 26.5%, shows a correlation coefficient of 0.47.

Relation (5.6) does not agree with an observation reported by Seed et al. (20) who studied the effect of dry density on resilient deformations for AASHTO road soil. They observed that if the soil was compacted at a certain moisture content, the resilient strains would decrease slightly as the dry density increases, up to a certain value; but further densification beyond that value would cause a marked increase in the deformation (i.e., a marked decrease in dynamic modulus).

However, Equation (5.6) is not expected to be of a practical design significance; rather, it is helpful in deciding the construction specifications, since it does not hold for conditions after compaction once the subgrade is subjected to moisture movement and/or saturation processes.

Figure (5.12) also shows that the dry density is more influential when samples are subjected to higher stress levels. For high compaction moisture contents (optimum + more than 2%), the dry density has no pronounced effect on $E^*$; i.e., there is no specific correlation between $Y_d$ and $E^*$ for such a condition.

5.2.2. Effect of Saturation

Saturation is a major cause of a deteriorating process through which the subgrade soil loses a considerable portion of its strength. Unfortunately, it is almost inevitable that every subgrade will
be subjected to such a process beneath the pavement structure. Susceptibility to the effects of saturation differs from one soil to another. Figure 5.13 is a plot of the dynamic modulus values before and after saturation. Soils #2 and #5 appear to be the most sensitive to saturation of all the soils investigated in this study. Soil #1 shows an average reduction of 74% in the modulus, while the moduli reductions are 89, 90 and 87% for soils #2, #5 and #6, respectively. The reduction of modulus due to saturation is more significant when the original soil compaction is performed on the dry side, as can be seen in Figure 5.14 for soil #6.

From the above-mentioned figures and Table 5.7, it is observed that saturation reduces the values of dynamic moduli by factors of five to fifteen. This assures the principle of considering saturation as the design condition, since the subgrade modulus will be reduced to very low moduli values as compared to the initial compacted condition, regardless of the compaction moisture content. However, it should be noted that high initial dry densities will help to avoid extremely weak subgrade upon saturation.

This observation also illustrates the necessity of providing adequate pavement drainage facilities to shorten the period of saturated subgrade conditions by reducing the continuous source of saturating water. A proposed method for considering the saturated conditions when dealing with permanent deformation prediction is to be discussed elsewhere in this report.
Figure (5.13) Effect of Saturation Process on Resilient Modulus, $E^*$
Figure 5.14 Effect of Compaction Moisture Content on Soil Susceptibility to Saturation, Soil #6
<table>
<thead>
<tr>
<th>Soil Number</th>
<th>Ave. $E^*$ (10^3 psi)</th>
<th>Std. Dev. (10^3 psi)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#1 Unsaturated Saturated</td>
<td>23.772</td>
<td>9.018</td>
</tr>
<tr>
<td>#1 Saturated</td>
<td>9.802</td>
<td>3.433</td>
</tr>
<tr>
<td>#2 Unsaturated Saturated</td>
<td>34.260</td>
<td>20.489</td>
</tr>
<tr>
<td>#2 Saturated</td>
<td>4.760</td>
<td>1.219</td>
</tr>
<tr>
<td>#3</td>
<td>23.144</td>
<td>18.500</td>
</tr>
<tr>
<td>#4</td>
<td>19.674</td>
<td>15.506</td>
</tr>
<tr>
<td>#5 Unsaturated Saturated</td>
<td>36.603</td>
<td>18.838</td>
</tr>
<tr>
<td>#5 Saturated</td>
<td>5.807</td>
<td>2.378</td>
</tr>
<tr>
<td>#6 Unsaturated Saturated</td>
<td>29.150</td>
<td>26.360</td>
</tr>
<tr>
<td>#6 Saturated</td>
<td>5.135</td>
<td>1.992</td>
</tr>
<tr>
<td>#7</td>
<td>29.274</td>
<td>20.187</td>
</tr>
<tr>
<td>#8</td>
<td>11.448</td>
<td>5.740</td>
</tr>
<tr>
<td>#9</td>
<td>9.822</td>
<td>5.312</td>
</tr>
<tr>
<td>Pike Co. SR-124*</td>
<td>4.654</td>
<td>2.948</td>
</tr>
</tbody>
</table>

* Field Study Case (see Chapter VI
5.2.3 Effect of Soil Type

The influence of this factor can be seen in Figure 5.11 and Table 5.7. Each soil structure has a different strength. No definite correlation could be obtained between moduli values and the soil's conventional classification (AASHTO, FAA, etc.), which means that in this study, these classification methods are not sufficient to describe the dynamic behavior of subgrade soils.

It is observed from Figures 5.11 and 5.13 that for high compaction moisture contents and saturation conditions, the variation in moduli values due to soil type is relatively insignificant.

5.2.4 Effect of Applied Stress Level

The resilient behavior of subgrade soils is nonlinear; i.e., the dynamic modulus is not constant under different stress conditions. This fact complicates the task of analyzing stress-strain relations, especially considering that there is no definite modulus-stress relationship. Figures 5.15 through 5.18 show the typical relation between $E^*$ and $\sigma_{appl}$. Each curve in these figures represents an individual sample tested under different stress intensities. The general trend is similar to that observed by Seed et al. (20). That is, $E^*$ decreases rapidly with increasing stresses at low stress levels until a certain stress level. Further increases in that limit results in a slight increase in the dynamic modulus. The critical stress level at which the dynamic moduli values shift from rapid decrease to slight increase is approximately equal to 10 to 15% of the ultimate compressive strength. However, this correlation did not hold strictly for all the samples tested.
Figure 5.15 Dynamic Modulus vs. Applied Stress (Soil #1)
<table>
<thead>
<tr>
<th>Sample #</th>
<th>W/C</th>
<th>$\gamma_d$, pcf</th>
<th>$\sigma_{ult}$, psi</th>
</tr>
</thead>
<tbody>
<tr>
<td>217</td>
<td>19.52</td>
<td>121.9</td>
<td>52.3</td>
</tr>
<tr>
<td>224</td>
<td>10.0</td>
<td>122.0</td>
<td>34.0</td>
</tr>
<tr>
<td>242</td>
<td>----</td>
<td>----</td>
<td>87.5</td>
</tr>
<tr>
<td>302</td>
<td>14.1</td>
<td>116.7</td>
<td>126.2</td>
</tr>
<tr>
<td>315</td>
<td>9.09</td>
<td>117.4</td>
<td>187.3</td>
</tr>
<tr>
<td>320</td>
<td>18.7</td>
<td>116.4</td>
<td>176.6</td>
</tr>
<tr>
<td>424</td>
<td>8.1</td>
<td>135.4</td>
<td>144.2</td>
</tr>
<tr>
<td>410</td>
<td>7.8</td>
<td>136.5</td>
<td>69.2</td>
</tr>
<tr>
<td>405</td>
<td>13.6</td>
<td>123.4</td>
<td>----</td>
</tr>
</tbody>
</table>

Figure 5.16 Dynamic Modulus vs. Applied Stress (Soils #2, 3, 4)
Figure 5.17 Dynamic Modulus vs. Applied Stress (Soils #5, 6, 7)
Fig (5.18) Dynamic Modulus vs. Applied Stress (Soil #7)
Figure 5.19 shows the same relationship, but each point in the figure represents a tested sample. From that figure, it is observed that beyond applied stresses of approximately 10 psi, there is no significant change in $E^*$. For design purposes, it is most likely that the subgrade under a pavement will be subjected to low-level stresses at which rapid variation of $E^*$ occurs with stress changes. The stress in the soil decreases with depth under wheel loads; consequently, the soil's modulus will increase with depth for a uniform, homogeneous subgrade. This indicates that selecting a single modulus to represent the entire subgrade mass is an approximation that may be, in some cases, unreasonable since the modulus may differ by a factor of four due to changes in stress levels. Immediately after compaction, when the top soil layer is more compacted than the deeper soil and before any environmental effects occur, this approximation may be closer to the real behavior. Nevertheless, when moisture movements begin to occur, the soil's modulus will change. As a result of moisture, whether due to ground or surface water, the soil strength and its distribution with depth would be affected. An example for these considerations will be illustrated in Chapter VII of this report.

The variation of dynamic modulus with stress level is actually a key factor when considering non-destructive field measurement methods which use loads that are substantially lower than standard wheel loads. The stresses in the subgrade due to these devices (such as a Dynaflect, for example) will be in the order of 10 percent of the expected corresponding stresses under actual traffic. Therefore, correction factors should be developed according to the $E^* - \sigma_{appl}$ relation, for each individual condition to account for this difference in stress level.
Figure (5-19) Effect of Dynamic Applied Stress on Dynamic Modulus (Ref. 6)
VI PRESENTATION AND ANALYSIS OF
RESULTS (BASE COURSE)

6.1 Residual Characteristic

6.1.1 Dynamic Creep Curve:

The argument presented in section 2.2.1 may not be applicable to untreated granular material because of its cohesionless nature. Nevertheless, the applicability of equation (2.4.5) was checked for the tested aggregate samples. It was found to be applicable to a very high degree of accuracy, within the life spans applied, for all samples tested. Figure (6.1) and (6.2) show two typical examples of output plots of the computer program employed to conduct linear regression analysis between the logarithm of rate of permanent strain accumulation, \( \log \left( \frac{\varepsilon_p}{N} \right) \), versus the logarithm of number of load repetitions, \( \log (N) \), for each sample tested. It should be pointed out that while figure (6.1) shows the relation for sample 121 tested under dynamic confining pressure (DCP), figure (6.2) shows the same relation for sample 51 which was tested under constant confining pressure (CCP). The rest of the figures for this relationship are presented in appendix B.

The correlation coefficients which are very close to unity and the extremely high values of the statistic \( F^{w} \) indicate strongly that equation (2.4.5) can represent the testing results. However, closer look at the
Sample # 121
Correl. Coeff. = 0.99983

Log(εp/N)

Fig. (6.1) Rate of Permanent Strain Accumulation
Versus Number of Load Repetitions (DCP)
Fig. (6.2) Rate of Permanent Strain Accumulation Versus Number of Load Repetition (CCP)
regression residuals (errors) suggests that the data points on the log $\varepsilon_p/N$ vs. log $N$ graph tends to form a very flat convex curve that can be approximated by a straight line. A schematic plot is shown in figure (6.3) to magnify this trend. Nevertheless, this observation will not reduce the significance of the applicability of equation (2.45) to samples dynamic creep curves.

Results obtained by Chou (35) were analyzed in the same manner. Curves reported in figure (2.6a) are presented in figure (6.4). The same trend of the data and equation (2.45) were found applicable to Chou's results as well. Barksdale's results (30) were also found to be consistent with this analysis.

In general, permanent deformation was found to increase with increasing deviatoric stress and decreasing any of confining pressure, dynamic modulus, water content, static initial modulus, angle of internal friction, and relative density, within the ranges considered in this study. It should be mentioned that the dry density was kept almost constant for all samples. Therefore, water content and relative density are also reflections of the degree of compaction. Resilient Poisson's ratio has no apparent effect on the permanent deformation. However, dynamic modulus may be considered to count for some of the samples' physical and mechanical properties. This approach will be discussed in the proceeding section.

The same trends were observed for the samples tested under zero deviator stress; i.e., dynamic consolidation, except for the fact that higher dynamic confining pressure results in more sample consolidation. The
Fig. (6.3) Data Trend and Model Representation

Equation (2.45)
Fig (6.4) Data Obtained by Chou (35) Analyzed According to Equation (2.45)
influence of the dynamic modulus was more noticeable in this type of testing. Accumulation of permanent deformation was measured even for those samples whose resilient Poisson's ratios were larger than 0.50, i.e. negative vertical recoverable deformations.

The main purpose of the dynamic consolidation tests was to check the effect of applying dynamic confining pressure. The dynamic consolidation curves followed equation (2.45) as well, see appendix B. However, the mechanism of dynamic creep - mainly due to shear straining - is different from that consolidation which is attributed to volume changes. These tests are, in fact, of no practical significance since they do not represent an actual in-situ condition.

Statistical analysis approach, similar to that adopted for subgrade soils analysis (section 5.1.3), will be applied to the base course testing results. From equation (2.45), a complete evaluation of parameters "m" and "A" will be essential and sufficient to characterize the residual behavior of the base course material. In the following sections a detailed analysis and discussion will be presented regarding the variation of those parameters.

Table (6.1) presents the physical and ultimate properties of the tested samples. Table (6.2) shows the stresses at which each sample was tested for dynamic creep (rutting) as well as the calculated rutting parameters "A" and "m". It should be mentioned that samples # 11, 41 & 51 tested at constant confining pressures - were not included in analysis discussed in the following section.
Table (6.1) Physical and Ultimate Properties of Aggregate Samples.

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Table 6.2 Stress State and Mechanical Properties of Aggregate Samples.

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</tr>
<tr>
<td>242</td>
<td>2.12</td>
<td>10.21</td>
<td>2.79</td>
<td>8.96</td>
<td>13.20</td>
<td>4.22</td>
<td>0.452</td>
<td>30,500</td>
</tr>
</tbody>
</table>

181
6.1.2 Analysis of Rutting Parameters Variations:

(a) Parameter "m"

This parameter is the slope of the linear relationship between log $p/N$ vs. log $N$, calculated through linear regression of the testing data. Parameter "m" was found to vary within the general range of 0.7 to 0.9 with very few exceptional samples, figure (6.5). The overall average value of "m" is 0.804 with standard deviation of 0.0671.

In the following analysis, the following expressions of the octahedral stresses are adopted:

$$T_{oct} = \frac{1}{3} (\sigma_1 - \sigma_2)^2 + (\sigma_2 - \sigma_3)^2 + (\sigma_1 - \sigma_3)^2 \quad (6.1a)$$
$$\sigma_{oct} = \frac{1}{3} (\sigma_1 + \sigma_2 + \sigma_3) \quad (6.1b)$$

In case of triaxial testing, equations (6.1) will be,

$$oct = \frac{2}{3} (\sigma_1 - \sigma_3) \quad (6.2a)$$
$$\sigma_{oct} = \frac{1}{3} (\sigma_1 + 2 \sigma_3) \quad (6.2b)$$

where

$T_{oct}$ = octahedral shear stress
$\sigma_{oct}$ = octahedral normal stress
$\sigma_1, \sigma_3$ = principal stresses (vertical and lateral in triaxial cell)
The octahedral stresses were checked against the principal stresses' influence in the analysis of rutting parameters variations.

Linear regression analyses were performed on $m$-values versus the samples' properties and testing conditions. As noted from table (6.2), ten samples were tested under dynamic confining pressure with zero dynamic deviator stress. Therefore, the analyses were carried out for; data set with $(\sigma_1 - \sigma_3) \neq 0$, data set with $(\sigma_1 - \sigma_3) = 0$, and all obtained testing results together. Tables (6.3a and b) show the correlation coefficients and the values of the statistic $F^*$ to be compared with standard $F$ obtained for confidence level $\alpha = 0.05$.

From table (6.3a), the first set of data shows that none of the individual factors is considered effective in accounting for the variation in "$m$", from the statistical point of view ($F^* < F$), except the deviator stress (and octahedral shear). Consequently $m$-values were plot versus $(\sigma_1 - \sigma_3)$ in figure (6.5). From the engineering point of view, the plot does not show appreciable trend between the two variables. The same was observed for the second set of data when plotting $m$-values versus water content, figure (6.5). Note that the extreme values of "$m$" occurs in this set of results. It is noticed, however, from table (6.3) that $m$ for the first set may be presented by the expression

$$m = C_1 \left( \frac{E^*}{E^*} \right)^{C_2} \left( \frac{\sigma_1 - \sigma_3}{\sigma_3} \right)^{C_3} e^{C_4 (w/c)} \quad (6.3)$$

which counts for about 49 percent of the variation of $m$. $C_1, C_2, C_3, \text{ and } C_4$ are material's constants. When considering all testing data, no specific
Table (6.3) Statistical Linear Analysis of "m" Variation

<table>
<thead>
<tr>
<th>Independent Influencing Factor</th>
<th>Data Set For ((\sigma_1 - \sigma_3) \neq 0)</th>
<th>Data Set With ((\sigma_1 - \sigma_3) = 0)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Correl. Coeff.</td>
<td>(F^*)</td>
</tr>
<tr>
<td>(E^s) (Dynamic Modulus) .....................</td>
<td>0.166</td>
<td>0.568</td>
</tr>
<tr>
<td>((\sigma_1 - \sigma_3)) (Deviator Stress)</td>
<td>0.517</td>
<td>7.279</td>
</tr>
<tr>
<td>(\sigma_3) (Confining Stress) .............</td>
<td>-0.114</td>
<td>0.263</td>
</tr>
<tr>
<td>(f^{oct}) (Octahedral Shear) ..............</td>
<td>0.516</td>
<td>7.265</td>
</tr>
<tr>
<td>(\sigma^{oct}) (Octahedral Normal) ........</td>
<td>0.173</td>
<td>0.613</td>
</tr>
<tr>
<td>(W/c) (Water Content) .....................</td>
<td>0.179</td>
<td>0.661</td>
</tr>
<tr>
<td>(\gamma_r) (Relative Density) ............</td>
<td>-0.133</td>
<td>0.361</td>
</tr>
<tr>
<td>Tan(\phi) ((\phi) = angle of Internal Friction)</td>
<td>0.255</td>
<td>0.613</td>
</tr>
<tr>
<td>(E^s), oct, oct, (W/c) ..................</td>
<td>0.699</td>
<td>4.063</td>
</tr>
<tr>
<td>(E^s), ((\sigma_1 - \sigma_3)), (\sigma_3), (W/c)</td>
<td>0.658</td>
<td>3.242</td>
</tr>
</tbody>
</table>

Note: All variables are in logarithm form except \(W/C\)
Table (6.4) Statistical Linear Analysis of "m" variation
(All Testing Data)

<table>
<thead>
<tr>
<th>Independent Influencing Factor</th>
<th>Correl. Coef.</th>
<th>F*</th>
<th>F</th>
</tr>
</thead>
<tbody>
<tr>
<td>$E^*$</td>
<td>-0.0002</td>
<td>0.000</td>
<td>4.17</td>
</tr>
<tr>
<td>$(\sigma_1 - \sigma_3)$</td>
<td>0.230</td>
<td>1.672</td>
<td>4.17</td>
</tr>
<tr>
<td>$\sigma_3$</td>
<td>0.054</td>
<td>0.086</td>
<td>4.17</td>
</tr>
<tr>
<td>$\tau_{oct}$</td>
<td>0.230</td>
<td>1.670</td>
<td>4.17</td>
</tr>
<tr>
<td>$\sigma_{oct}$</td>
<td>0.258</td>
<td>2.145</td>
<td>4.17</td>
</tr>
<tr>
<td>w/c</td>
<td>-0.0818</td>
<td>0.196</td>
<td>4.17</td>
</tr>
<tr>
<td>$\gamma_r$</td>
<td>-0.071</td>
<td>0.153</td>
<td>4.17</td>
</tr>
<tr>
<td>tan $\phi$</td>
<td>-0.170</td>
<td>0.897</td>
<td>4.17</td>
</tr>
<tr>
<td>$E^*$, $\tau_{oct}$, $\sigma_{oct}$, w/c</td>
<td>0.331</td>
<td>0.833</td>
<td>2.74</td>
</tr>
<tr>
<td>$E^*$, $(\sigma_1 - \sigma_3)$, $\sigma_3$, w/c</td>
<td>0.272</td>
<td>0.541</td>
<td>2.74</td>
</tr>
</tbody>
</table>

Note: All variables are in logarithm form except w/c, $(\sigma_1 - \sigma_3)$, $\tau_{oct}$ and tan $\phi$.
Fig(6.5) "m"-Values vs. \((\sigma_1 - \sigma_3)\) for the First Set of Data, and w/c for the Second Set.
trend was observed for $m$ - variation, table (6.4).

It should be pointed out that parameter "$m$" varies within a narrow range, figure (6.5). Therefore, for practical design purposes, it could be considered a constant parameter. Using the mean and standard deviation of the testing data, a statistical interval estimate could be obtained for the population (base course material), $(\mu)$. The mean of the data set $\bar{m}$ is approximately normally distributed, with a mean $\mu$ and a standard deviation $\sigma_y$, since the number of cases in the set is larger than 30. Then, the probability that

$$\bar{m} - Z(\frac{a}{2}) \sigma_y < \mu < \bar{m} + Z(\frac{a}{2}) \sigma_y \quad (6.4)$$

is $(1-a)$, where $a$ = width of certainty

- $\sigma_y$ = standard deviation of data set \(\sqrt{n}\)
- $n$ = number of cases (samples) in the set
- $Z(\frac{a}{2})$ = area under normal - distribution curve beyond $a/2$ limits

Figure (6.6) shows these limits predicted statistically versus confidence level $(1-a)$. The variations in the predicted values of "$m$" of the base course material, based on laboratory results, are not significant, which justify the conclusion the "$m$" is constant for the material.

(b) Parameter "A"

Parameter "A" is the plastic strain after the first cycle applied to
Fig (6.6) Predicted Statistical Range for Parameter "m"
the sample. It is calculated through linear regression of \( \log \frac{\ell p}{N} \) versus \( \log N \) for the dynamic creep curve. From the previous discussion, \( m \) was concluded to be almost constant. Consequently, parameter "A" is expected to reflect the residual behavior of the tested samples.

In base course material, the factors expected to influence the residual characteristics are stress conditions and material's properties. Stress conditions represented in dynamic triaxial testing are the principal stress; deviatoric stress \( \sigma_1 - \sigma_3 \) and confining stress \( \sigma_3 \). The material's properties considered are physical; e.g. water (w/c) content and relative density \( \gamma_r \), and mechanical, e.g. dynamic modulus \( E^* \), angle of internal friction \( \phi \) resilient, Poisson's ratio \( \nu \) and static intial modulus \( E_s \). Because of the complexity of the problem, multiple regression analysis in the power form was conducted between "A" and those factors. Only the data set with \( \sigma_1 - \sigma_3 \neq 0 \) was analyzed, i.e. dynamic creep curve. Octahedral stresses, defined in equations (6.1) and (6.2), were considered to study its effect as compared with that of principal stresses.

In general, parameter "A" has positive proportionality with \( \sigma_1 - \sigma_3 \), \( T_0, \sigma_0 \), principal stress ratio \( \frac{\sigma_1 - \sigma_3}{\sigma_3} \) and octahedral stress ratio \( \frac{T_0}{\sigma_0} \). On the other hand, it has negative proportionality with \( w/c, \gamma_r, E_s, E^*, \phi \) and \( \sigma_3 \), table 6.5.

While in subgrade soils the dynamic modulus was the most influential factor, deviator (shear) stress is found to be more effective on rutting in aggregate materials. Increasing lateral pressure decreases the permanent
deformation. However, its effect is not significant as noticed from table 6.5. The principal stress ration \((\sigma_1 - \sigma_3) / \sigma_3\) (and octahedral stress ratio \(\tau_0 / \sigma_0\)) is found to have higher correlation to the parameter "A".

Table 6.5 shows that dynamic modulus is not as influential as was noticed for subgrade soils. It has inverse effect on parameter "A" with relatively low coefficient of correlation. The effect of relative density reflects the influence of the degree of compaction. Higher relative density results in lower permanent deformation. However, the relative density is intercorrelated with the water content. Therefore, including the moisture content in the multiple regression analysis will account for the effect of the relative density.

The variation of the angle of internal friction \(\phi\) has little inverse effect on parameter "A". It is thought that such an effect will be incorporated in the analysis when considering the dynamic modulus \(E^*\).

Multiple and stepwise regression analyses were performed on the testing data. The independent variables of principal stresses, octahedral stresses, dynamic modulus and water content were considered in several forms. Exponential forms of the stresses and stress ratios were among these forms. However, it did not lead to adequate representation of the data. Therefore the following models were considered for the analysis:

\[
A = C_{11}(\sigma_1 - \sigma_3)^{C_{21}} (\sigma_3)^{C_{31}}, \quad (6.5a)
\]

\[
A = C_{12} (\sigma_1 - \sigma_3)^{C_{22}} (\sigma_3)^{C_{32}} (E^*)^{C_{42}}, \quad (6.5b)
\]
Table (6.5) Correlation Coefficients for Factors Affecting Parameter "A" and Dynamic Modulus (E*)

<table>
<thead>
<tr>
<th>Factor</th>
<th>Correlation Coefficients</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>&quot;A&quot;</td>
</tr>
<tr>
<td>(\sigma_1 - \sigma_3)</td>
<td>0.637</td>
</tr>
<tr>
<td>(\tau_0)</td>
<td>0.637</td>
</tr>
<tr>
<td>(\sigma_3)</td>
<td>-0.149</td>
</tr>
<tr>
<td>(\sigma_0)</td>
<td>0.241</td>
</tr>
<tr>
<td>((\sigma_1 - \sigma_3)/3)</td>
<td>0.777</td>
</tr>
<tr>
<td>(\tau_0/\sigma_0)</td>
<td>0.798</td>
</tr>
<tr>
<td>(E^*)</td>
<td>-0.215</td>
</tr>
<tr>
<td>(E_s)</td>
<td>-0.198</td>
</tr>
<tr>
<td>(\tan \phi)</td>
<td>-0.049</td>
</tr>
<tr>
<td>w/c</td>
<td>-0.274</td>
</tr>
<tr>
<td>(\gamma_r)</td>
<td>-0.414</td>
</tr>
</tbody>
</table>

Note: All variables are in logarithm form except w/c.
The analyses of these models were performed in three phases: using principal stresses (as shown in equation (6.5a - e)), substituting octahedral stresses for principal stresses (\(\sigma_0\) instead of \(\sigma_1 - \sigma_3\) and \(\sigma_0\) instead of \(\sigma_3\)), and finally incorporating relative density to replace moisture content. It was found that the last phase did not have a significant effect on the results of the analyses. That is including either water content or relative density in the analysis will lead to very close results.

The results of the first two phases are shown in table (6.6). From this table it appears that parameter "A" is correlated to the octahedral stresses to a higher degree than that associated with the principal stresses for all models considered. Equation (6.5a) expresses the effect of the applied stresses. The influence of \(E^*\) variation could be detected when comparing this equation with equation (6.5b). The improvement in the correlation coefficient between models (6.5a) and (6.5b) is higher than that between models (6.5b) and (6.5c). That is the impact of including the dynamic in the analysis is more noticeable.

While the difference in correlation coefficients between models
Table (6.6) Result of Multiple Regression
Analysis of Parameter "A"

<table>
<thead>
<tr>
<th>Model</th>
<th>Parameter &quot;A&quot;</th>
<th>Parameter &quot;Aa&quot;</th>
<th>F</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Correl. Ccoeff.</td>
<td>F* Correl. Ccoeff.</td>
<td>F*</td>
</tr>
<tr>
<td>Phase I using Principal Stresses</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.5a</td>
<td>0.777</td>
<td>13.72</td>
<td>8.96</td>
</tr>
<tr>
<td>6.5b</td>
<td>0.836</td>
<td>13.10</td>
<td>11.15</td>
</tr>
<tr>
<td>6.5c</td>
<td>0.857</td>
<td>11.02</td>
<td>13.87</td>
</tr>
<tr>
<td>6.5d</td>
<td>0.800</td>
<td>15.99</td>
<td>11.04</td>
</tr>
<tr>
<td>6.5e</td>
<td>0.844</td>
<td>14.00</td>
<td>16.56</td>
</tr>
<tr>
<td>Phase II using octahedral Stresses</td>
<td></td>
<td></td>
<td></td>
</tr>
<tr>
<td>6.5a</td>
<td>0.801</td>
<td>16.15</td>
<td>9.41</td>
</tr>
<tr>
<td>6.5b</td>
<td>0.844</td>
<td>14.06</td>
<td>11.16</td>
</tr>
<tr>
<td>6.5c</td>
<td>0.867</td>
<td>12.14</td>
<td>13.74</td>
</tr>
<tr>
<td>6.5d</td>
<td>0.830</td>
<td>20.00</td>
<td>12.55</td>
</tr>
<tr>
<td>6.5e</td>
<td>0.865</td>
<td>16.81</td>
<td>17.55</td>
</tr>
</tbody>
</table>
(6.5c) and (6.5e) is not significant, it should be pointed out that only one parameter is used to express the stress conditions, stress ratio, which will decrease the constants, required to describe the correlation, from five to four constants. That is also detected from the higher F-values, Table (6.6). Therefore, the stress ratios can be used effectively to describe the stress conditions.

The multiple correlation coefficients of equations (6.5d) and (6.5e) show the effect of including the factor of water content - for practical design purposes equation (6.5d) may be used to express the rutting parameter "A" in terms of stress ratio and dynamic modulus.

The high effectiveness of the stress conditions with respect to that of the other parameters including $E^*$, w/c, $\phi$ and $\gamma_r$, suggests that the material type and conditions are rather insignificant when compared to stress level. This conclusion can not be generalized until further studies are made to investigate other aggregate types and gradations.

Figure (6.6) shows the relation between parameter A and stress ratio (octahedral and principal) for every water content on log-log graph. Dynamic modulus $E^*$ could be used to categorize the curves of the difficulty to group the data according to specific values of $E^*$. The linearity of the relation between log (A) and log (stress ratio) can be noticed from figure (6.6) which supports the power relation suggested in equations (6.5d) and (6.5e).

Substituting equation (6.5d), using octahedral stress ratio, into
Fig (6.6) Parameter "A" vs. Principal and Octahedral Stress Ratios.
equation (2.145) results,

\[ \frac{\varepsilon_P}{N} = C_{14} \left( \frac{T_0}{\sigma_0} \right) C_{24} \left( E^* \right) C_{44} N^{-m} \]  \hspace{1cm} (6.6a)

or

\[ \varepsilon_P = C_{14} \left( \frac{T_0}{\sigma_0} \right) C_{24} \left( E^* \right) C_{44} N^{1-m} \]  \hspace{1cm} (6.6b)

Equation (6.6b) is a constitutive equation which expresses the permanent strain in terms of octahedral stress ratio and dynamic modulus over the life span \( N \) of the base course. The design implementation of this equation will be discussed in chapter VII.

(C) Adjusted Parameter "Aa"

The idea of parameter "Aa" is similar to that explained for subgrade soils, section 5.1.3. This parameter was calculated according to equation (5.14). Its values are shown in table (6.2). Parameter "Aa" was analyzed through the same procedure performed on parameter "A". All trends discussed for "A" variations were found applicable to Aa. Table (6.6) shows the results of the analyses applied to this parameter. It is observed, from this table, that considering "Aa" instead of "A" improves the correlation coefficients only when the moisture content is involved in the analysis. That is, it is preferred to use "Aa" if equation (6.5e) is employed to represent the residual behavior of the aggregate.

Parameter "Aa" was not found very effective, in improving the corre-
lation between "A" and independent variables, like that observed for subgrade soils. However, it is believed to be a promising approach to account for variations that may occur in parameter "m".

6.2 Resilient Characteristics:

Although it is not the main objective of this study to investigate the resilient characteristics of the base course material, a few observations and comments are discussed in the proceeding paragraphs:

(1) Allen (27) had presented resilient Poisson's ratio (ν) as a third-degree polynomial function of the principal stress ratio (σ₁/σ₃) for dynamic confining pressure (DCP) test equation (2.9) and figure (2.13). The range of variation of ν observed by Allen was a rather narrow range (0.35 - 0.44), for practical range of stress ratio (1.5 - 7.5). Consequently, no further attention was given in this study to investigate ν versus stress ratio, and was considered almost constant for each sample tested.

Allen (27), however, limited the anisotropic behavior of the untreated aggregate sample to the constant confining pressure (CCP) tests. In this study, cases of anisotropy were observed for samples tested under DCP conditions. Resilient Poisson's ratio did not exceed 0.60 in most of those cases. The anisotropy did not depend only on the stress type (DCP or CCP), but also on the degree of compaction and water content so that the sample may undergo volumetric changes during the dynamic testing.
(2) The resilient dynamic modulus was calculated through the equation

\[ e_a = \frac{1}{E''} \left( \sigma_1 - 2\nu\sigma_3 \right) \]  

(6.7)

where \( e_a \) = recoverable axial strain which describes the behavior of elastic material.

The modulus \( E'' \) did not vary significantly with number of loading cycles \( N \), especially after 1000 cycles. Figure (6.7) and (6.8) show \( E'' \) vs \( N \) for some of the tested samples. Therefore, dynamic modulus used to characterize the samples at certain condition was measured after 1000 cycles or whenever a stable recoverable strains is attained. This observation agrees with the studies made by Hicks (26) and Allen (27).

(3) The dynamic modulus \( E'' \) is most sensitive to stress state during testing. It increases with dynamic deviator stress (for non zero values) as well as dynamic confining pressure, figures (6.9) thru (6.12). In some cases, the relation starts with \( E'' \) decreasing with increasing \( \sigma_1 - \sigma_3 \) untill certain stress level. CCP tests has the same trend as that for DCP as far as \( E'' \) is concerned. Byond \( (\sigma_1 - \sigma_3) = 35 \) psi the change in \( E'' \) upon changing \( (\sigma_1 - \sigma_3) \) is minimal.

Figure (6.12) shows these relations for CCP and DCP performed on sample #82. It is noticed that DCP yields higher \( E'' \) for deviator stresses higher than 8 psi. At stress levels less than 8psi, \( E'' \) measured from CCP were higher than those measured from DCP tests. Allen (27) reported similar trends of \( E'' \) variations, and concluded that these differences
Fig (6.7) Dynamic Modulus vs. Number of Load Repetitions
Fig (6.8) Dynamic Modulus vs. Number of Load Repetitions.
Fig (6.9) Dynamic Modulus vs. Stress State, DCP Tests
Fig (6.10) Dynamic Modulus vs. Stress State, CCP Tests
Fig (6.11) Dynamic Modulus vs. Stress State, DCP Tests
Fig (6.12) Dynamic Modulus vs. Stress State, Sample # 82
does not have a significant effect on the results of pavement structure analysis. However, the differences observed in this study were the order of 20%. These differences increase for higher confining pressures.

These trends were observed when a test was run on one sample, figures (6.9) thru. (6.12), as well as for tests run on different samples of the same material, see table (6.5). The relatively higher positive inter-correlation coefficients between \( E^k \) and shear and normal stresses reflect such a trend.

(1) The relation (2.8) suggested by Hicks (26) and supported by Allen (27), to relate dynamic modulus \( E^k \) to \( \theta = \sigma_1 + 2 \sigma_3 \) in power form, was investigated in this study. Samples # 82, 22 and 2h0 were in accordance with that relation. The dynamic modulus of sample # 82 was found to follow equation (2.8) for stress level \( (\sigma_1 - \sigma_3) > 10 \) psi, figure (6.13). The underlined points in that figure are those which have \( (\sigma_1 - \sigma_3) < 10 \) psi. However, samples 22 and 2h0 followed the relation satisfactorily, figure (6.14). Table (6.7) shows the correlation coefficients and constants \( K_1, K_2 \) of equation (2.8) when applied to these samples (only those points of \( (\sigma_1 - \sigma_3) > 10 \) for sample # 8 were subjected to the regression procedure).

It is interesting to note that there is a logarithmic relationship between the constants \( K_1 \) and \( K_2 \). The data obtained by Allen (27) for DCP and CCP tests and that obtained by Hicks (26) for dry and wet samples were plot in figure (6.15). The trend for linearity on the semi-log graph bet-
Fig (6.13) $E^*$ vs. $\theta$ for Sample # 82
Fig (6.1h) $E^*$ vs. $\theta$ (DCP Tests) Samples # 22 and 240
Table 6.7 Regression Results
of Equation (2.8)

<table>
<thead>
<tr>
<th>Sample # and Test Type</th>
<th>K₁</th>
<th>K₂</th>
<th>Correl. Coeff.</th>
</tr>
</thead>
<tbody>
<tr>
<td>Samples # 8* CCP</td>
<td>1.70 x 10³</td>
<td>0.802</td>
<td>0.949</td>
</tr>
<tr>
<td></td>
<td>2.79 x 10³</td>
<td>0.703</td>
<td>0.953</td>
</tr>
<tr>
<td>Sample # 22 DCP</td>
<td>5.45 x 10³</td>
<td>0.526</td>
<td>0.860</td>
</tr>
<tr>
<td>Sample # 240 DCP</td>
<td>1.76 x 10³</td>
<td>0.749</td>
<td>0.953</td>
</tr>
</tbody>
</table>

* Only points with (σ₁ - σ₃) > 0 were included.

A trend could be seen in this figure. This trend could be expressed mathematically in the form

\[ K_2 = S_1 - S_2 \log K_1 \]  
(6.8)

where S₁ and S₂ are constants.

It should be mentioned that the plotted data from references (26) in figure (6.15) is for different materials, gradations and fine contents. In other words, relation (6.8) holds regardless of these factors. Therefore, the constants S₁ and S₂ are dependent only on the type of testing (CCP or DCP) and water condition of the aggregate sample. The difference between those curves obtained by the two different references for CCP may
Fig (6.15) Interrelationship Between $K_1'$ and $K_2'$
be due to different experimentation procedure. Nevertheless, equation (6.8) is found applicable to each set of data separately. Samples # 82, 22 and 240 are designated in figure (6.15) by squares. The three samples form a linear trend also, although three points may not be adequate to designate a linear relationship of testing results.

(5) During experimentation the dynamic stresses (deviatoric and confining) are superimposed on small static stress in the same direction. These static stresses represent the overburden stresses of the in-situ conditions. From the experimentation view, it is necessary to apply these static pressures. It is to prevent any negative pressures applied on the sample during the testing; that is, to ensure complete contact between the loading assembly with sample in the vertical direction, and eliminate the possibility of having triaxial chamber negative confining pressure due to liquid bouncing. Such a negative chamber pressure will result in immediate destruction of the sample.

However, the effect of the variations of these static pressure on the behavior of the sample should be investigated. It was found that variations of static confining pressure within 5 psi did not affect the resilient behavior significantly. On the other hand, the vertical static pressure has considerable effect on the recoverable strains. Figure (6.16) and (6.17) show this effect on the dynamic modulus. It can be seen that the deviator static stress has considerable effect on the measured E*. Therefore, the dynamic modulus used in this study to characterize a
Fig (6.16) Dynamic Modulus vs. Static Deviator Stress

- Sample # 203
- 214
- 223
- □ 234 \((\sigma_1 - \sigma_3) = 19.7 \text{ psi}\)
- ▼ 234 \((\sigma_1 - \sigma_3) = 4.7 \text{ psi}\)
- △ 243
- ◇ 183
Fig (6.17) Dynamic Modulus vs. Static Deviator Stress
sample was corrected - if necessary - to account for such variation and calculate the modulus at constant common static deviator stress.

Consequently, in any future standardization of the dynamic testing of granular materials this factor should be taken into account in order to attain comparable properties of the material. This factor my have an influence when comparing the results obtained by Hick (26) and Allen (27) in figure (6.15).

The influence of the static stresses were not studied for characterizing the residual properties because the samples were given the chance to deform first under those pressures before applying the super imposed dynamic vertical and confining stresses.
7.1 Rut Design Guidelines

To translate these research findings into practical engineering use, in this chapter an attempt has been made to present a rutting prediction model for the pavement foundation.

It is already established that the creep curve in both subgrade soils and base course aggregate can be described by equation (2.45) in the form

\[ \frac{\epsilon_p}{N} = A N^{-m} \]  

(2.45)

As it was indicated previously, in the proposed rut design model, the evaluation of the variation of parameters "A" and "m" are the key to practical estimation of permanent deformation. The parameter "m" was shown to be almost constant except for very few cases. However, parameter "Aa" was suggested to replace parameter "A", and account for "m" variations that may occur. This parameter "Aa" was found to be effective in improving its correlation, when replacing "A", with the influencing factors, specially for subgrade soils.

In the preceding chapters, equations (5.3) and (6.5d) were presented as the most simplified relations to describe the function "Aa".

\[ Aa = R \left( \frac{E_s}{\sigma} \right)^{-s} \left( \sigma_{appl} \right) \quad \text{for subgrade soils} \quad (5.3) \]

\[ Aa = C_1 \left( \frac{T_o}{\sigma_o} \right)^{C_2} \left( \frac{E_s}{\sigma} \right)^{C_3} \quad \text{for base aggregate} \quad (6.5d) \]
where $E_s$ & $E_b$ are the dynamic moduli for subgrade soil and base aggregate respectively, and $R$, $S$, $C_1$, $C_2$, and $C_3$ are material constants independent of physical properties or applied stress level. That is, they are constant for specific soil or base aggregate respectively under any testing conditions. However, from an engineering point of view, field data are needed to verify the applicability of these relations.

Those constants in equations (5.3) and (6.5d) can be determined experimentally. For subgrade soils, careful selection of representative samples will allow the use of very few test samples, since the investigated equation will be linear on log-log graphs. More samples are needed for base course material since there are more variables involved.

Direct substitution of equations (5.3) and (6.5d) into equation (2.45) yields:

$$f_p = R \left( \frac{E_s}{E_b} \right)^{-S} (\sigma) N^{1-m} \quad (7.1)$$

and

$$f_p = C_1 \left( \frac{T_0}{\sigma_0} \right)^{C_2} \left( \frac{E_s}{E_b} \right)^{C_3} \quad \text{respectively} \quad (7.2)$$

Consequently, the knowledge of $E_s$ and $\sigma$ for any subgrade soil and/or $E_b$ and $\frac{T_0}{\sigma_0}$ for any specific condition will be adequate to estimate the dynamic creep curve ($f_p$, vs. $N$) by substituting into equations (7.1) and/or (7.2) respectively. This procedure could be easily included in permanent deformation prediction nomographs or computer programs.

The procedure for estimating the contribution of the foundation layers to the total pavement system rutting, as developed in this model is as follows:
(1) Conduct stress analyses throughout the pavement system under the expected wheel load on the roadway. Several methods for such analyses are proposed in the literature, e.g., elastic multi-layer system analysis, viscoelastic approach, etc. The dynamic moduli foundation is required as input data in these methods. It can be preliminarily assumed or obtained from field measurements. It is appropriate to use computer-programmed numerical methods in applying these methods of analysis. Using an appropriate method and technique of analysis, determine the stress distribution under the wheel path and along the depth under the pavement surface.

(2) Prepare field samples or representative laboratory-prepared samples (some of which may be subjected to saturation and/or freeze-thaw procedures). Perform dynamic loading tests on these samples under a stress state in accordance with the distribution determined in step (1).

(3) According to the laboratory results, establish the variation of dynamic modulus with such factors as moisture content, dry density and stress state. According to these results, check or re-perform the analysis in step (1) to find the stress distribution closest to the practical conditions. Note that the same samples can be used for dynamic creep curves and variations of E*

(4) Using the results of dynamic testing, find the constants R and S equation (5.3) and constants C_1, C_2, and C_3 in equation (6.5d). However, these constants could be obtained from previous testing on the same or similar layer types since they are constant for each material.
(5) Divide the foundation layers into imaginary sublayers. The number of these sublayers depends on the total foundation thickness and stress distribution along its depth.

(6) For each sublayer, find the stress state, and the corresponding dynamic modulus, $E^*$. Using Equation (5.3) and (6.53) and the results of item (4) above, find the parameters "Aa" and "m". Repeat the same step for different expected seasonal conditions in the sublayer throughout the pavement life. The effects of seasonal variations have been suggested by Khedr (5) and supported by Majidzadeh et al (47).

(7) Find the traffic data over these seasons. Then use Equation (6.3) to predict the permanent strain for each sublayer at the required number of cycles, $N$, Khedr (5),

$$\left( \frac{c_p}{N} \right)_n = \frac{A_a \times m}{m}$$

where

$n$ = number of environmental changes at end of which permanent deformation is to be predicted;

$\left( \frac{c_p}{N} \right)_n$ = rate of permanent deformation at the end of the $n^{th}$ environmental condition

$A_a_{max}$ = maximum value of parameter "A" for the weakest condition of the soil through $n$ environmental changes

(8) The total contribution of the subgrade to pavement rutting is the sum of accumulative permanent deformations in the sublayers, computed as follows:
\[ \gamma_p = \sum_{i=1}^{m} \varepsilon_{pi} h_i \]  

(7.4)

where

- \( \gamma_p \) = subgrade permanent deformation
- \( m \) = number of sublayers
- \( \varepsilon_{pi} \) = permanent strain in the \( i^{th} \) sublayer, from step (7) above
- \( h_i \) = thickness of the \( i^{th} \) sublayer

**B. Case Study**

This field study case is located on State Route 124 in Pike County, Ohio, between stations 958+00 and 1098+00, a two-lane, one-way in the eastbound direction. This section of the road had failed due to excessive pavement rutting and was closed to traffic during the summer of 1978.

The pavement structure consists of two inches of Item 404 asphalt concrete, eight inches of Item 301 bituminous aggregate base (local tar was used as the binding material) over a silty clay subgrade soil. The subgrade soil is classified as silty clay with some gravel. There is no drainage system for the pavement or the subgrade. The section is located mostly in a cut area, although part of it consists of a constructed embankment.

The road was constructed in 1971 and was one of the objects of research conducted at The Ohio State University by Majidzadeh et al. (3, 19). The first research project was mainly concerned with evaluation of the pavement mixes, while the second project focused on subgrade perfor-
mance. Dynaflect measurements were taken over a period of about 2\(\frac{1}{2}\) years.

To investigate the problems occurring on this roadway, the following steps were considered in cooperation with Ohio Department of Transportation engineers:

(1) Inspection trenches were cut across the two lanes closed to traffic, and the rutting profile was inspected. In some areas, permanent deformation was more pronounced in the bituminous base, while it was more severe in the subgrade in other spots. However, the total rut depth ranged between two and six inches. The subgrade contributed between 0.5 and 3.0 inches of permanent deformation. It was also concluded, at a few stations, that the subgrade had suffered considerable rutting during construction due to heavy equipment hauling and rolling on four inches of 301 base prior to completion of the pavement construction.

It should also be mentioned that the subgrade was found to be saturated and in some cases, in a soaked condition, and free water was observed running out of the pavement for several hours at a time. Longitudinal cracks were observed along the outside of the wheel path. Also, the tar-bound base was found to be deteriorating under the effect of high pore water pressure building up under heavy traffic in the poorly-drained pavement, and due to the average pavement temperature of 100°F during summer days.

(2) Preliminary stress analysis of the pavement system was carried out using the Chevron elastic layer program. The distribution of the deviatoric stress along the pavement depth is shown in Figure 7.1.
Figure (7.1) Deviatoric Stress Distribution Within The Pavement System (Pike Co. SR-12h)
The values of the dynamic moduli are preliminarily assumed to be 200,000, 60,000 and 5,000 psi for the top asphalt concrete, tar-bound base and subgrade soil respectively. These assumptions were based on a 90°F average pavement temperature, deteriorating tar-bound base and saturated subgrade. The Poisson's ratios were assumed at 0.38, 0.40 and 0.45 respectively. The wheel load was taken as 10,000 lbs. tire pressure equal to 75 psi. From this figure, the deviatoric stress is 8.3 psi at the top of the subgrade, decreasing to 1.8 psi at a depth of 40 inches.

(3) Undisturbed soil samples were obtained from beneath the pavement at sites where complete failure had not occurred. The samples were prepared and tested in the laboratory under dynamic loading. A summary of the test results is shown in Appendix A, Table A-10.

The variations of $E^*$ with applied stress amplitude are shown in Figures 7.2 and 7.3. The general trend for this relation is that $E^*$ decreases rapidly with increasing $\sigma_{\text{appl}}$ until a critical value of $\sigma_{\text{appl}}$ is reached (equal to 10 to 20% of $\sigma_{\text{ult}}$). The dynamic modulus increases slightly with the applied stress beyond this critical stress level. The different behaviors observed in Figure 7.3 are mainly due to the inconsistency of the field samples (some may have a higher percent of aggregates than others). It should be pointed out that the samples are subjected to incrementally increased applied stress until they experience excessive deformation that will not allow accurate measurement of elastic deformation.

The stress analysis in step (2) should be repeated with the new measured subgrade dynamic modulus if it differs from the assumed
Figure (7.2) $E^*$ vs. Applied Stresses For Undisturbed Field Samples (Pike-12h).
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<td>985+00</td>
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<tr>
<td>1058+00</td>
<td>(2)</td>
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</tbody>
</table>

**Figure (7.3) $E^*$ vs. Applied Stress For Undisturbed Field Samples (Pike-12h)**
value. These values will differ from one station to another and according to the depth at which the sample was obtained. If precise estimation is required, the analysis should be performed at each station and the rutting prediction should be investigated statistically along the road section. However, one can consider an average station as an example for the purposes of this discussion.

Three samples were obtained at station 983+00: #1 from the top of the subgrade (1.3 to 1.8 ft. depth); #2 from 1.9 to 2.4 ft. depth; and #3 from 2.4 to 2.9 ft. depth. The top five inches were wet gravel with some sand and silt. From Figures 7.2 and 7.3, it is seen that the top sample was weaker than sample #2, which was weaker than #3, considering the dynamic modulus as a measure. This is believed to be due to the soaking effect of water in the pavement. The top sample, #1, had $E^* = 2.90 \times 10^3$ psi at average $\sigma_{appl} = 5$ psi; #2 had $E^* = 5.0 \times 10^3$ psi at an average $\sigma_{appl} = 3.5$ psi; and #3 had $E^* = 8.3 \times 10^3$ psi at an average $\sigma_{appl} = 3.0$ psi. These values were used to repeat the stress analysis which came out as shown in Figure 7.1. Accordingly, it was decided to divide the subgrade into three sublayers of average dynamic moduli and stress levels as illustrated in Figure 7.1. This type of modulus variation may not be the case at another station. For example, at station 1002 + 00, we obtained a constant dynamic modulus with depth since $E^*$ for sample #1 at $\sigma_{appl} = 8$ psi was equal to that of sample #2 at $\sigma_{appl} = 5.4$ psi, a modulus value of 8500 psi.

The residual characterizations of the tested samples are summarized in Appendix A, Table A-10. This data was analyzed in the
same manner as the procedure followed for laboratory-prepared samples. It was fitted to the relations (5.2) and (5.3) and the resulting equations are:

\[ A = 149.61 \left( \frac{E^*}{E_n} \right)^{-1.429} \exp \left( \frac{\sigma}{\sigma_{ult}} \right) \] (7.5)

\[ A = 104.89 \left( \frac{E^*}{E_n} \right)^{-1.562} \] (7.6)

The correlation coefficient for equation (6.5) is 0.643 and that for equation (6.6) is 0.678. However, when considering parameter "Aa" instead of "A", equations (7.5) and (7.6) become,

\[ A_a = 6.8314 \left( \frac{E^*}{E_n} \right)^{-1.866} \exp \left( \frac{\sigma}{\sigma_{ult}} \right) \] (7.7)

and

\[ A_a = 4.792 \left( \frac{E^*}{E_n} \right)^{-2.000} \] (7.8)

with correlation coefficients of 0.731 and 0.753 respectively.

Consequently, it was decided to use equation (7.8) as the prediction model. The corresponding values of parameter "Aa" for the selected sublayers are shown in figure 7.1. The average value of parameter "m" is 0.77. It may be appropriate, in this case, to assume that the values of parameter "Aa" are close to \( A_{max} \) since the values of \( E^* \) were measured for samples taken from a subgrade in weakened condition. If we assume that "m" is constant, then using equation (7.3) yields

\[ Y_{pi} = A_{max} N^{1-m} \] (7.9)

where \( i \) denotes the considered sublayer.
Then, the total permanent deformation in the subgrade under this pavement can be calculated as:

\[ \chi_p = \sum_{i=1}^{3} \frac{\varepsilon_i}{p_i} h_i \]

\[ = A_{\text{max}_1} N^{1-m}(10.0) + A_{\text{max}_2} N^{1-m}(8.0) \]

\[ + A_{\text{max}_3} N^{1-m}(12.0) \]

\[ = N^{1-m} \left( 3 \times 843 \times 10^{-2} \right) \]

\[ = 0.0381 \ N^{(-0.23)} \]

The estimation of the accumulative total contribution of the subgrade soil to the total pavement rutting is shown in Figure 7.4 as a function of the number of equivalent axle wheel loadings. It can be seen that the subgrade makes a considerable contribution to the total rutting in the pavement system. The rutting is predicted to be about 0.5 inches at a pavement life of 100,000 load repetitions at the assumed equivalent loading and tire pressure. Also, a similar procedure could be performed on the asphaltic concrete layer to estimate the rutting in that layer. A detailed description of such a procedure will be discussed in the final report of the research project, EES 560 (under preparation).

Carrying out the same procedure at selected stations, one can estimate the subgrade rutting profile along the road section. Statistical analysis could show the trend of such a profile with the life of the pavement. This procedure can easily be computerized for efficient use.
Figure (7.4) Rutting Prediction For Pike SR-124 (Field Study Case)
8.1 Summary:

The purpose of this theoretical and laboratory investigation was to develop data and correlation on pavement foundation support conditions for use in a rational design scheme for flexible pavements. Residual as well as resilient characterization of the supporting foundation materials are considered key inputs to such a design scheme.

The primary objectives of this study were to develop a sound theoretical-based approach to describe the residual behavior of number of silty and clayey subgrade soils and base course aggregate, and to develop correlations that identify resilient properties, and finally, to establish a rutting model to predict the accumulative permanent deformation susceptibility for the considered materials.

To achieve these objectives, the investigation included three general phases: a theoretical investigation to describe the dynamic creep curve, an experimental program performed on subgrade soils, and an experimental program conducted on base course untreated aggregate.

The theoretical approach was based on the energy concepts. It described the dynamic creep curve in the form of a power relationship between rate of permanent strain accumulation and number of load repetitions.

The soils and aggregate considered were first evaluated for basic
material characteristics such as Atterbeg limits, gradation, compaction, etc.

To investigate the applicability of the theoretical approach, laboratory-prepared samples were prepared for each soil type at various compaction moisture contents and dry densities. Some of them were subjected to a saturation procedure. These samples were tested under dynamic uniaxial loading in which the dynamic stress intensities were varied within a certain range to simulate the variations of stresses in field subgrades. Rutting and dynamic modulus testing procedures were applied to study the variation of rutting parameters and dynamic modulus.

Samples of base course aggregate were subjected to field simulated dynamic vertical and lateral pressures. A triaxial cell in a testing arrangement was used to apply such a stress pattern. At constant dry density the base course samples were prepared at three different compaction moisture contents. The stresses intensities were varied in a practical range similar to that expected in the field.

A complete statistical analysis program was conducted to investigate the variation of rutting parameter - in accordance with the theoretical approach - with the influencing factors of material properties and stress state. Data obtained by other researchers were also considered and included in these analyses.

A field study case was also selected in Pike County (SR 124), in the State of Ohio, to set a practical example and verify the use of the findings of the study. Undisturbed field samples were taken from this project site and subjected to laboratory testing and analysis procedure similar to the
approach used for the laboratory - prepared samples.

8.2 Conclusions:

Based upon the results and laboratory and theoretical analyses, the following conclusions were drawn:

(1) A theoretical approach was established based on an energy concept to develop the equation,

\[ \frac{\varepsilon_p}{N} = A N^{-m} \]  

(2.45)

to describe the dynamic creep curve.

where \( \varepsilon_p \) = plastic strain

\( N \) = number of load repetitions

and \( A \) & \( m \) are rutting constants for specific stress and material condition.

Although this equation had been observed experimentally before for subgrade soils, the theoretical approach establishes a better understanding of the rutting parameters "A" and "m".

(2) The equation (2.45) in conclusion (1) was found applicable to all subgrade soils and base course aggregate considered in this study to a high degree of accuracy. The only exception to this conclusion are those cases where weak samples suffered progressive failure under applied stresses. These conditions are not included in the scope
of this study.

(3) Parameter "m" found to be almost constant. However, parameter "A" was suggested to be adjusted to parameter "Aa" which accounts for m variation if it occurs. Parameter "Aa" is calculated through restricted linear regression of dynamic creep curve, in accordance with equation (2.45), with m assumed constant and equal to m_0 (average value of m for specific material).

(4) Parameter Aa was correlated to soils dynamic modulus in a power form and linearly to the dynamic deviatoric stress level, equation (5.3).

(5) Parameter Aa was also correlated, for base course aggregate, to octahedral stress ratio (octahedral shear stress/octahedral normal stress) and dynamic modulus in a power form, equation (6.5d).

(6) Combining equations (2.45), (5.3) and/or (6.5d) a rut depth prediction model is developed for the flexible pavement foundation layers. The model states that rutting can be expressed in terms of dynamic modulus and stress state for each material type.

(7) The resilient modulus exhibited a considerable dependency on the stress state. However, the influence of the stress varies according to material type. For subgrade
soil, the main trend was that the modulus decreases rapidly with low stress level. At higher stress levels the modulus increases with stress level but at a slower rate. This trend was not observed strictly for all samples. Other trends were reported which suggest that there is no standard trend for all soils.

The modulus of the base course aggregate was found, in general, to increase with both deviatoric and lateral stresses. The power relation between the modulus and stress invariant, $\theta = \sigma_1 + 2 \sigma_3$, suggested by other researchers, was found applicable in most of the cases in the form

$$E^* = K_1 \theta K_2$$

Equation (2.8) was found not applicable in some cases for low levels of deviator stresses.

(8) An interrelationship between the constants $K_1$ and $K_2$ in equation (2.8) was found in the form

$$K_1' = S_1 - S_2 \log K_2'$$

where $S_1$ and $S_2$ are constants dependent on the type of testing procedure.

(9) Dynamic modulus for subgrade soils was found to be greatly dependent on moisture condition of the soil; as - compact-
ed and saturation process. It has a bell-shaped curve with the compaction water content with the optimum \( w/c \) less than that of the compaction curve. The effect of saturation was found to be significant. It may reduce the modulus by about 3 to 10 times, depending on the material's susceptibility for such a process.

Furthermore, the modulus was found to have a linear relationship with the dry density at compaction.

(10) For base course aggregate, the resilient modulus was found sensitive to the initial static deviator stress on which the dynamic deviator stress superimposed. The modulus is not, however, dependent on the initial static lateral pressure within the range of 5psi.

8.3. Recommendations:

In view of the research procedure and findings the following recommendations are made:

(1) Further investigations are needed, in the area of residual characteristics study, to be performed on sandy subgrade soils and different base untreated aggregate materials to check the generalization of the findings of this study. Also, different environmental conditions, e.g. saturation, should be considered in these studies.

(2) Full scale tests that simulate actual conditions in a test pit
are also required to study the applicability of the presented model. Also, field information is needed to support such an investigation. These studies are recommended more for the cohesionless materials involved in laboratory investigations.

(3) DCP (dynamic confining pressure) tests require unusual equipment arrangements that are not available in every materials testing laboratory. Research is required, therefore, to determine if such testing could be substituted by CCP (constant confining pressure) tests as far as residual characteristics are concerned.

(4) It was found during this study that it was difficult to establish generalized resilient behavior trends for both subgrade soils and base course untreated aggregate. Consequently, it is recommended to investigate each material, to be used in pavement construction, individually.

(5) The effect of setting static pressures, on which the dynamic pressures are superimposed, should not be neglected, especially that of the deviatoric stress. In order to attain repeatable comparable results, these factors should be fixed.

(6) Rutting phenomena should be included in any modern design scheme. The model presented in this study can be computerized in further studies.
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APPENDIX A

PERTINENT DATA FROM DYNAMIC TESTS
ON SATURATED AND UNSATURATED LABORATORY SAMPLES (SOILS # 6 & 7)
AND
UNDISTURBED FIELD SAMPLES (SR124, PIKE CO.)
Table A.1 (a) Pertinent Data from Unsaturated Laboratory Compacted Samples
(Soil #6)

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*Most ultimate strength values were computed from strength curves.
### Table A.1(a) Continued Data from Unsaturated Laboratory Samples (Soil #6)

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Table A. 1(b) Pertinent Data from Saturated Laboratory Compacted Samples (Soil #6)

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*Most ultimate strength values computed from strength curves
Table A.2 Pertinent Data from Unsaturated Laboratory Compacted Samples (Soil #7)

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Table A.2 (continued) Data from Unsaturated Laboratory Samples (Soil #7)

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Table A.3 Test Results for Undisturbed Samples (Pike County, SR124)

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<th>Height (in)</th>
<th>W/C (%)</th>
<th>Dry Density (pcf)</th>
<th>Applied Stress (psi)</th>
<th>Ultimate Strength (psi)</th>
<th>Dynamic Modulus (psi)</th>
<th>Rutting Parameters A m</th>
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* The sample number 1 indicates top of sampling tube
  2 indicates middle of sampling tube
  3 indicates bottom of sampling tube
** Bad Seal
APPENDIX B

REGRESSION ANALYSES OF THE AGGREGATE SAMPLES' TESTING DATA
Figure B.1 Rate of Permanent Strain Accumulation
vs. Number of Load Repetitions
Figure B.2 Rate of Permanent Strain Accumulation

vs. Number of Load Repetitions
Figure B.3 Rate of Permanent Strain Accumulation
vs. Number of Load Repetitions
Figure B.4 Rate of Permanent Strain Accumulation vs. Number of Load Repetitions
Figure B.5 Rate of Permanent Strain Accumulation
vs. Number of Load Repetitions
Figure B.6 Rate of Permanent Strain Accumulation vs. Number of Load Repetitions
Figure B. 6 Rate of Permanent Strain Accumulation vs. Number of Load Repetitions
Figure B.7 Rate of Permanent Strain Accumulation vs. Number of Load Repetitions
Figure B.8 Rate of Permanent Strain Accumulation vs. Number of Load Repetitions
Figure B.9 Rate of Permanent Strain Accumulation vs. Number of Load Repetitions.
Figure B.10 Rate of Permanent Strain Accumulation vs. Number of Load Repetitions.
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**Figure B.11 Rate of Permanent Strain Accumulation vs. Number of Load Repetitions.**
Figure B.12 Rate of Permanent Strain Accumulation vs. Number of Load Repetitions.
Figure B.13 Rate of Permanent Strain Accumulation vs. Number of load Repetitions.
Figure B.14 Rate of Permanent Strain Accumulation

vs. Number of Load Repetitions.
Figure B.15 Rate of Permanent Strain Accumulation vs. Number of Load Repetitions.

Sample # 151

Correlation Coeff. 0.9861
Figure B.16 Rate of Permanent Strain Accumulation
vs. Number of Load Repetitions.
Figure B.17 Rate of Permanent Strain Accumulation
vs. Number of Load Repetitions
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Figure B. 18 Rate of Permanent Strain Accumulation vs. Number of Load Repetitions.
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Figure B.19 Rate of Permanent Strain Accumulation

vs. Number of Load Repetitions.
Figure B. 20 Rate of Permanent Strain Accumulation vs. Number of Load Repetitions.
Figure B. 21 Rate of Permanent Strain Accumulation vs. Number of Load Repetitions.
Figure B. 22 Rate of Permanent Strain Accumulation

vs. Number of Load Repetitions.
Figure B. 23 Rate of Permanent Strain Accumulation

vs. Number of Load Repetitions.

log (εp / N) vs. log N

Sample # 192

Correl. Coeff. 0.99978

Predicted and Observed

1.25 1.50 1.75 2.00 2.25 2.50 2.75 3.00 3.25 3.50 3.75

1.25 1.50 1.75 2.00 2.25 2.50 2.75 3.00 3.25 3.50 3.75
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*Correl. Coeff. 0.99976

Figure B.24 Rate of Permanent Accumulation vs. Number of Load Repetitions.
Figure B. 25 Rate of Permanent Strain Accumulation

vs. Number of Load Repetitions.
Figure B.26 Rate of Permanent strain Accumulation vs. Number of Load Repetitions.
Figure B. 27 Rate of Permanent Strain Accumulation vs. Number of Load Repetitions.
Figure B.28 Rate of Permanent Strain Accumulation
vs. Number of Load Repetitions.
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Sample # 222

Corr. Coeff. 0.99943

Figure B. 29 Rate of Permanent Strain Accumulation vs. Number of Load Repetitions.
Figure B. 31 Rate of Permanent Strain Accumulation vs. Number of Load Repetitions.
Figure B. 30 Rate of Permanent Strain Accumulation

vs. Number of Load Repetitions.
Figure B. 32 Rate of Permanent Strain Accumulation vs. Number of Load Repetitions.
Figure B. 33 Rate of Permanent Strain Accumulation vs. Number of Load Repetitions.
Figure B.34 Rate of Permanent Strain Accumulation

vs. Number of Load Repetitions.