CHAPTER II

REVIEW OF LITERATURE

2.1 General

The principal objective of any pavement design procedure is to provide a structural system which will be suitable in a specific regional area and be able to sustain the anticipated traffic loading and frequency [13, 8, 14]. It is generally accepted that pavement deteriorates or loses serviceability with time due to load repetitions and environmental conditions. Existing design methods attempt to control or limit this loss in serviceability by minimizing the factors contributing to the different distress modes such as fatigue, rutting, excessive deflection, temporary excessive rebound in the subgrade and base materials and lack of stability in the wearing course [20, 24]. Thus, the design of a pavement-section is not simply a matter of guessing or estimating the thickness of the surface, base, subbase and subgrade of the pavement structure. Rather it embraces a more detailed study of each pavement component through the investigation of their physical properties and interaction mechanism. These properties are looked at, in general, through three different aspects. The first of these is the stress-strain characteristics (mechanistic model) of the different materials used in the various layers of the pavement structure. The second, is the most likely failure mode of the various pavement components. Finally the third aspect is the interaction between the different materials and their integrated behavior under traffic loadings and environmental conditions. Current pavement-design procedures use different design criteria and call for different
material characterization techniques using one or more of these three aspects. Consequently, it may be beneficial at this time to look briefly at several different design methods.

2.2 Design Methodologies

The strength of a flexible pavement is the result of building up thick layers and thereby distributing the load over the subgrade rather than by the bending action of the slab [6]. Historically, pavement design has been approached from two broad differing points of view. First, the practicing engineer often approaches the problem solely from the standpoint of pavement performance. In contrast researchers and educators approach the problem largely from theoretical concepts. Neither of the above approaches is satisfactory within itself. Complete reliance upon pavement performance represents a lengthy process. Thus, one must wait a relatively long period of time before new concepts can be proven. On the other hand, theoretical equations are generally based upon simplified assumptions and many times do not apply to conditions as they exist in the field. For comprehensive and ideal pavement design, both approaches must be integrated and used properly [19]. For any pavement design procedure to be completely rational, total consideration must be given to three elements. These elements are:

01. the theory used to predict the failure or distress parameter,
02. the determination of the relationship between the magnitude of the parameter in question to the failure or performance level desired, and
03. the evaluation of the pertinent material properties necessary for the theory selected.

A great deal of research and analysis has been devoted toward development of a fundamental rational design system for flexible pavement based on the above stated elements. Even though all of the design elements have been
recognized by many pavement engineers, differences exist among them in adapting these design factors. Therefore, the design methods that they adopt for a given set of conditions are also different.

The design of flexible pavement has changed rather significantly in the past several years. Generally speaking, flexible pavements were classified as pavement structures having a relatively thin asphalt-wearing course with layers of granular base and subbase used to protect the subgrade from being overstressed. This type of pavement design was primarily based upon empiricism or experience, with theory playing only a subordinate role in the procedure. Presently, highway engineers are faced with the need to provide remedial measures to upgrade existing pavements to meet today's traffic loadings and frequencies. Also, they have recognized that various independent distress modes, such as rutting, shoving, cracking, etc., contribute to pavement structural and/or functional failure. These needs and knowledge have brought about several changes in pavement design and have led many investigators to search for a more comprehensive pavement design analyses based on theoretical and experimental considerations. Today, there is no one fundamental or rational design procedure that is widely accepted in the pavement design industry. Yoder and Witczak [13,19] described two broad categorical approaches to the problem of pavement design based upon the limitation of subgrade overstress. The first category is based on empirical correlations of excessive deformations related to some predefined failure condition of the pavement. The second category is based on the prediction of the cumulative deformations (cumulative damage) of the pavement system under consideration. These two categories will be discussed further below.

2.2.1 Deformation-failure approach:

This category is further subdivided into two procedures:
2.2.1.a Laboratory or field index test procedure:

In this design procedure, laboratory or field index tests (CBR, stabilometer...) are used to categorize the strength of the subgrade materials. It is one of the most widely accepted design procedures for control of repetitive shear deformations used to date [19,18,22]. Generally the fundamental approach is to control pavement layer thickness and material quality based upon some of the above mentioned index tests. It is inherently assumed that the primary source of deformation occurs in the subgrade provided that the thickness and material quality controls are met [19,3,14]. Consequently, allowable deformations are controlled by adjustment of the pavement thickness to reduce the stresses on the subgrade to a level such that actual deformation will not exceed the allowable deformation within the design life of the pavement [19,8]. One such design method is presented in the AASHO interim guide for design of pavement structures [14]. A brief review of this method is presented below.

In the early 1950's, the highway engineers were confronted with the need to predict the performance of pavement systems subjected to greater wheel loads and frequencies than they had ever before experienced [19] and to establish an equitable policy for vehicle sizes and weights. This need has led the American Association of State Highway and Transportation officials (AASHTO) to develop the AASHO-Interim Guide design procedure to alleviate the above-mentioned problem. The procedure is based on an extensive road test that was conducted in Ottawa, Illinois. The test site consisted of six loops (two small loops and four large ones). The first AASHO Interim Guide [14] was published in 1961 and all recommendations for the design procedure were based on the result obtained through a period of 25 months of testing. The primary objectives of the AASHO Road tests were:
a. To establish relationships between the number of load repetitions and the performance of different pavement systems of known subgrade soil characteristics.

b. To determine the effect of different loadings, represented by the magnitude and frequency of axle loads.

c. To establish instrumentation, test procedures, data charts, graphs and formulas which would be helpful in future highway design, for both rigid and flexible pavements of conventional design.

In general, the AASHO interim guide is used to determine the total thickness of the pavement structure, as well as the thickness of the individual pavement components. It should be noted that the main assumption of the procedure is that most subgrade soils can be adequately represented, for pavement design purposes, by means of their soil support value (SSV) for flexible pavements or by their modulus of subgrade reaction (K) for rigid pavements. In special cases when poorer soils (frost susceptible, highly organic, etc.) are encountered, adequate pavement performance is achieved by increasing the thickness of the pavement structure, or by using special precautions. The term "pavement performance" is defined in the AASHO interim guide as follows: "a pavement which maintains a high level of ability to serve traffic over a period of time is superior in performance to one whose riding quality and general conditions deteriorate at a more rapid rate under the same traffic conditions." The term pavement serviceability was adopted to represent the ability of a pavement to perform under the given traffic. Thus, pavement performance is assigned a value from zero to five and it is called pavement serviceability index. Prediction of the present serviceability index of a pavement system can be achieved by using a combination of different physical measurements and is given by the following relationships (14).

\[
\text{PSI} = 5.03 - 1.91 \log (1+SV) - 1.38 \overline{RD}^2 - 0.01 (C+P)^{1/2}
\]  

(2.1)

where

\[ SV = \text{slope variance, a measure of longitudinal roughness} \]
RD = average rut depth

C+P = area of class 2 and 3 cracking plus patching per 1000 ft² (92.9 m²)

This serviceability-performance concept is the basic philosophy of the AASHO interim guide. Thus, a pavement section may be designed for the level of serviceability desired at the end of the selected traffic analysis or after exposure to a specific total traffic volume. The basic flexible pavement design equation, developed from the results of the AASHO Road test, uses a traffic equivalency criterion which convert mixed-traffic to 18-kip equivalent single-axle load.

\[
\log W_{18} = 9.36 \log (SN+1) - 0.20 + \frac{\log[4.2-P_t]/(4.2-1.5)]}{0.40 + [1094/(SN+1)]^{5.19}}
\]

\[+ \log \left(\frac{1}{R}\right) + 0.372 (SSV-3.0) \]  

(2.2)

where

\( W_{18} \) = number of equivalent 18-log single axle loads expected in time \( t \)

\( SN \) = structural number of the pavement system

\( P_t \) = the terminal serviceability index or the serviceability index at time \( t \)

\( R \) = regional factor

\( SSV \) = soil support value

The soil support value (SSV) of any given soil ranged from 3.0 for A-6 materials to 10.0 for A-1 materials. The objectives of this research project include a study of the (SSV) scale as related to some physical characteristics of the subgrade soil in question.

2.2.1.b Limiting subgrade strain procedure

This design approach as described by Yoder and Witczak [13] uses the elastic layered theory to limit the vertical subgrade strain. Concepts for designing flexible
pavements using multilayer elastic analysis were presented by Dorman and Metcalf in 1965 [9]. The principles outlined by these investigators were based upon limiting strains in the asphalt surface and permanent deformation in the subgrade. The use of multilayered elastic theory in conjunction with a limiting strain criteria for design involves the consideration of three factors: the theory used, the material characterization technique, and the development of failure criterion for each mode of distress. In the development of the procedure, use was made of computer solutions to solve stresses, strains and displacements within a multilayered (elastic) pavement system [24,25,26]. Most elastic layered design procedures, considers both permanent deformation (rutting) of subgrade as well as fatigue cracking of the asphalt-bound layer as the two most significant failure mechanisms.

Dissatisfaction has been expressed by many highway agencies concerning the use of these conventional procedures, because both design procedures are based on empirical relationships derived from experience and observations. Furthermore they are applicable only to a defined range of pavement materials, traffic loads and environmental conditions for which experience is available [19,8,18,27,16]. Also both procedures failed to predict the amount of anticipated deformation after a given number of load applications.

2.2.2 Prediction of cumulative deformation approach

Yoder and Witczak [19] described this category as representative of procedures that are based upon the prediction of accumulated deformations in pavement systems using quasi-elastic or viscoelastic approaches. These approaches, however, are not presently refined to the point where this can be accomplished with a level of confidence needed for adequate design methods [19,8,28,29]. Despite this disadvantage, the methodology is the most preferred for use in a more advanced or rational design
method due to its capability of obtaining cumulative deformations of any pavement system [19,28,29,18,27,30,31,3]. Many investigators have suggested that research should be directed towards developing better material characterization techniques for use in such rational design methods [19,8,18,27,30,3,32,33]. A comprehensive literature review of the quasi-elastic and viscoelastic approaches may be found in reference [23], a part of which is repeated here for the benefit of the reader.

2.2.2.a Quasi-elastic approach

The quasi-elastic approach as described by Yoder and Witczak [19] is based upon the use of elastic theory and the results of plastic strains determined by repeated load laboratory tests on pavement materials. This approach was initially introduced by Heuklom and Klomp [34]. Since then, research has been conducted by others such as Monismith [35] and Barksdale [29] for soils, granular materials and asphalt concrete. The fundamental concept of the analysis is the assumption that the plastic strain \( \varepsilon_p \) is functionally proportional to the elastic state of stress (or strain) and number of load repetitions. This constitutive deformation law is considered applicable for any material type and at any point within the pavement system. The response of any material must be experimentally determined from laboratory tests for conditions (time, temperature, stress state, density, moisture, etc.) expected to occur in situ. The elastic theory (either linear or nonlinear) is then used to determine the expected stress state within the pavement provided that the plastic deformation is known. Subdividing each layer into convenient thicknesses \( \Delta Z_j \) and determining the average stress state at each layer increment, the permanent deformation of the pavement may be computed using [13,10,14]

\[
\Delta t = \sum_{j=1}^{n} \varepsilon_p \Delta Z_j
\]  

(2.3)
where
\[ \Delta t = \text{total deformation} \]
\[ n = \text{number of layers} \]
\[ \varepsilon_p = \text{permanent strain} \]
\[ \Delta z = \text{thickness} \]
\[ j = \text{the layer in question} \]

2.2.2.b Viscoelastic Approaches

A pavement design method employing the viscoelastic approach has been developed under the direction of the Office of Research, Federal Highway Administration (FHWA) [18]. The procedure is based on a mechanistic structural subsystem known as VESYS IIM computer program. This computer program predicts the performance of a pavement in terms of its present serviceability index, PSI, derived from the American Association of State Highway Officials (AASHO) Road Test analysis [19,18]. Inputs to the program must be in the form of statistical distributions describing material properties, geometry of the pavement section being analyzed, traffic and environment. Program outputs are presented in terms of means and variances of the damage indicators - cracking, rutting, roughness and serviceability. The VESYS IIM computer program consists of three models shown diagramatically in Figure 2.1. These models are:

2.2.2.b.1 Primary Response Model

The Primary Response Model represents the pavement system by a three layer semi-infinite continuum such that the upper two layers are finite in thickness while the third layer is infinite in extent. Each layer is infinite in the horizontal directions and may have elastic or viscoelastic behavior. The model constitutes a closed form probabilistic solution to the three layers linear viscoelastic boundary value problem. It is valid for a
FIGURE 2.1 Modular Structure of VESYS IIM (18).
single stationary circular loading at the pavement surface. Stochastic inputs to the model are in terms of the means and variances of the creep compliances for viscoelastic materials, and elastic or resilient moduli for elastic materials. The output from the Primary Response Model, in the form of statistical estimates of stresses, strains and deflections, is used as input to the Damage Model.

2.2.2.b.2 Damage Model

The Damage Model consists of three separate models each designed to predict distress accumulation in the pavement.

01. The Rut Depth Model uses the results from the Primary Response model along with laboratory determined permanent deformation characteristics of the pavement and subgrade materials to compute the mean and variance of the rut depth accumulated over any incremental analysis period.

02. The Roughness Model uses the rut depth output from the Rut Depth Model, along with the assumption that rut depth at any time along the wheel path will vary due to material variability and non-uniform construction practices, to compute the roughness in terms of slope variance as defined by AASHO [14].

03. The Fatigue Cracking Model is a phenomenological model which predicts the extent of cracking of the asphalt layer based on Miner's hypothesis. This cracking is due to fatigue resulting from tensile strain at the bottom of the asphalt concrete layer. A crack index is computed after any number of load applications using the viscoelastic radial strain amplitude at the bottom of the asphalt concrete layer along with laboratory determined fatigue properties of the asphalt concrete. The radial strain amplitude is found at the peak of a haversine load pulse of specified duration applied to the pavement surface. From this crack index the expected area of cracking is computed in square yards per 1000 square yards.

The output from the above three parts of the Damage Model, i.e., rut depth, slope variance, and crack index, is used as input to the Performance Model.
2.2.2.b.3 Performance Model

The Performance Model computes a Serviceability Index, Pavement Reliability and Expected Life of the Pavement. The serviceability index, PSI, is defined according to the AASHO Interim Guide 1972 [14] as

\[ \text{PSI} = a + b \log_{10}(1 + SV) + c\sqrt{C} + P + dR^2 \] (2.4)

where

- \( a \approx 5.03 \), \( b \approx 0.01 \), \( c \approx 1.91 \), \( d \approx 1.38 \) are multiple regressions constants
- \( SV \) = Slope Variance (Roughness)
- \( C \) = Crack Index
- \( R \) = Rut depth
- \( P \) = Patched area

The expected value and variance of the PSI is then calculated at any time. The reliability of the serviceability index at any time is defined as the probability that the PSI is above some unacceptable level, \( \text{PSI}_f \), which has been established beforehand. The distribution of PSI's is obtained assuming a Gaussian distribution. The expected life of the pavement is the time for the Serviceability Index to reach the unacceptable level, \( \text{PSI}_f \).

Two categories of mechanical properties are required for the VESYS IIM structural analysis, primary response properties, and distress properties. The primary response properties define the response of layer materials to the given loads and environments. These properties are in the form of elastic or viscoelastic characteristics which may exhibit non-linear behavior because of previous load histories, plastic effects, and stress dependencies. The distress properties are those properties defining the capability of the materials to withstand the imposed loads. The Rut Depth Model in the current version of VESYS IIM [18] assumes a permanent deformation accumulative
damage law of the form

\[ F(N) = \mu_i N^{\alpha_i} \]  \hspace{1cm} (2.5)

where

- \( N \) = Number of axle load repetitions
- \( \alpha_i \) and \( \mu_i \) = Permanent deformation response parameters for material in layer \( i \).

One method for determining \( \alpha_i \) and \( \mu_i \) for equation 2.5 is to use the results of the Dynamic Series of an "Incremental Static-Dynamic" test described by the load program shown in Figure 2.2. For more detailed information the reader is referred to reference [11].

A sensitivity analysis of the VESYS IIM structural model [29] determined that calculated responses of the system were: a) insensitive to variations of the parameter \( \mu \) for base and subgrade; b) insensitive to variations of parameter \( \alpha \) for base materials; c) sensitive to variations of \( \alpha \) for subgrade material.

Researchers have indicated that one of the most urgent research needs in material characterization is the development of simplified tests which decrease the total number of tests, shorten the amount of time required for each test, and simplify the test methods and instrumentation requirements [30,27,18,3,32].

2.3 Cyclic Loadings

Timoshenko [36] credits Poncelet as being the first to consider the strength of materials under repeated loadings and to introduce the term "fatigue" to describe the resulting strength-deterioration characteristics. Timoshenko also credits Wohter for conducting the most extensive and the earliest experimental repeated load tests, Wohter found that the number of load cycles to failure increased as the cyclic stress intensity increased. Other investigators [37,38,39] concerned themselves with fundamental aspects of fatigue and developed hypotheses to
FIGURE 2.2 Load History Used in "Incremental Static-Dynamic" Test (18).
explain their experimental data. They postulated the formation of crystalline or intergranular structure during cyclic loading. These studies are still continuing with many theories proposed each satisfying one or more aspects of the fatigue phenomenons and yet none being adequate for all cases. In general, all materials including soils lose strength or stiffness, or both, with increasing number of cyclic stress [40] as shown in Figure 2.3. Most of the early studies, and indeed most of the more recent studies have used uniform repeated load intensity rather than irregular one to study the effects of traffic loading on the pavement system in question. This is so because a uniform repeated load intensity test machine is easier and cheaper to build and operate than an irregular loading apparatus. Generally, the loading patterns are likely to vary from vehicle to vehicle or from case to case even within the same problem area. Thus, irregular or variable cyclic loading tests will better simulate the traffic action. However, this requires the evaluation of each possible load pattern to be expected throughout the lifetime of the pavement structure [41,42]. A review of literature concerning the behavior of cohesive soils subjected to cyclic loading is presented in the next paragraph. The background information for cohesionless soils, on the other hand, may be found in Reference [1].

2.3.1 Behavior of Cohesive Soils Subjected to Cyclic Loadings

A qualitative measure of the behavior of soils subjected to cyclic loadings, such as that induced by earthquakes, has been widely recognized since they were examined by Casagrande in 1936 [43]. Over the past several years, considerable advances have been made in our understanding of soil behavior during cyclic loading and in our ability to reasonably predict this behavior. According to Idriss and Ricardo [27], the stress-strain characteristics of soils subjected to cyclic loadings is nonlinear and
<table>
<thead>
<tr>
<th>Material</th>
<th>Reference Stress Conditions (2)</th>
<th>Cyclic Stress Conditions (3)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Metals</td>
<td>Static tensile strength</td>
<td>Reversing compression/extension</td>
</tr>
<tr>
<td>Clays</td>
<td>Static Compressive strength</td>
<td>Reversing compression/extension</td>
</tr>
<tr>
<td>Sands</td>
<td>Cyclic stress to fail in N = 1 cycle</td>
<td>Reversing compression/extension</td>
</tr>
<tr>
<td>Asphalt and Treated Soil</td>
<td>Cyclic stress to fail in N = 1 cycle</td>
<td>One-directional beam bending</td>
</tr>
</tbody>
</table>

FIGURE 2.3 No. of Load Applications versus Ratio of Cyclic Stress to Static Strength (40).

1) 5% Cr-Mo V Steel
2) Cement and Lime Treated Soils
3) Ferrous and Non-Ferrous Materials
4) Saturated Sands and Clays
5) Asphalt
hysteretic in nature. Figure 2.4 shows an idealized stress-strain loop obtained for a soil specimen subjected to a symmetrical cyclic shear load along a plane free of initial shear stress [44,45]. Seed [46] reported that the method of cyclic load application to a soil has an important effect on the magnitude of soil deformation. For example, a specimen subjected to repeated loading has been found to deform many times more than an identical specimen subjected to a sustained load of equal magnitude. This difference in soil behavior under different types of loading raises the question whether tests performed under conditions of slowly increasing stresses can satisfactorily indicate the performance of a soil under the repetitive type loading to which it is subjected to under a pavement structure [46,47]. Further, a pavement may be considered to have failed when the deformation of the soil below the wearing surface is of such a magnitude as to cause an uneven riding surface or to cause cracking of the surfacing material. One of the objectives of pavement design procedures is to determine the thickness of pavement and base which must be placed over a subgrade in order that the deformation of the subgrade will not be excessive. Thus, for a satisfactory method of pavement design, it is necessary to devise some means of evaluating the resistance to deformation of the subgrade soils when it is subjected to a series of repeated loads of different magnitudes, durations and frequencies [8,32,47]. Recent research [31], however, has shown that it is not sufficient to evaluate only the resistance to permanent or plastic deformation of the subgrade, but also the elastic or resilient properties of the subgrade soils. A number of investigations conducted by the California Division of Highways have shown that there is a close correlation between observations of cracking and fatigue-type failures in bituminous pavements and the measured deflections of these pavements due to passing wheel loads. It appears, therefore, that large elastic deformations in
FIGURE 2.4 Shear Stress vs Shear Strain for Cyclic Loading (27).

FIGURE 2.5 Permanent Strain vs Number of Load Repetitions for Silt Clay (46).
a soil are a primary cause of pavement failure. Several cases were reported where soils having low resistance to plastic deformation exhibited high resilient deformations. Also, it is likely that some soils may exhibit extremely small plastic deformations and yet show high elastic deformations. Such soils would probably cause more fatigue failure in the surfacing materials than would a soil exhibiting a larger plastic deformation and much smaller elastic movement. Therefore, it is necessary to evaluate the soil resistance to elastic and plastic deformations separately prior to the design of pavement structure [48,49].

Soils are often subjected to vibratory loadings as a result of natural forces (earthquakes, wind, waves) or human activities (trains, pile driving, blasting, traffic, etc.). Variations in the soil responses due to these forces are to be expected since the response depends on the load and soil parameters. These parameters include: 1) number of load applications (N), 2) frequency, 3) magnitude of loadings, 4) load duration, 5) relaxation period, 6) density and moisture content of the soils, 7) thixotropy and 8) stress history (2,3,7,56). The effects of some of these factors on the plastic and elastic response of cohesive soils will be reviewed next.

2.3.1.1 Factors Affecting the Plastic Deformations of Cohesive Soils

2.3.1.1.a Number of Load Applications

Several investigators [50,51,52] stated that, in general, silt and clay subgrade materials exhibit a stiffening behavior with an increasing number of stress applications (N). The permanent deformation of cohesive soils subjected to repeated load applications is large during the first few cycles. Each subsequent load application results in a smaller increment of permanent deformation. After a large number of load applications the rate of change in permanent deformation becomes very small.
The total permanent deformation of test specimens, however, increases with increasing number of stress applications \([19,53,54,55,56]\). Seed [55] studied the effects of the number of load applications on the plastic behavior of soils by testing several samples up to 100,000 load repetitions using triaxial cyclic apparatus. He reported that the cumulative plastic strain increased with increasing number of stress applications. Seed concluded that the relationship between the total permanent strain and the logarithm of the number of load cycles could be expressed by a linear function as shown in Figure 2.5. Yoder [13] on the other hand reported a linear relationship between the logarithm of the accumulated plastic strain and the logarithm of the number of load applications.

2.3.1.1.b Magnitude of Loadings

Most researchers agree that the loading magnitude (confining pressure and cyclic principal stress difference) is the most important test parameter controlling the plastic soil behavior. However, the magnitude of this load in the highway subgrade materials is very difficult to determine [57]. This is so because the locked in radial stresses during compaction are highly variable and may be as high as 50 or 100 psi (345 or 689 KN/m\(^2\)). Hicks and Monismith [58] reported that the range of radial stress encountered in the subgrade materials as a consequence of the passage of a load vehicle varied from zero to ten psi (0 to 68.9 KN/m\(^2\)). Thus, when evaluating the resilient and permanent characteristics of subgrade materials it is desirable to do so under wide range of confining pressure and cyclic stress difference. Researchers unanimously agree that for the same number of load applications and for the same confining pressure, the higher the stress ratio (principal stress difference to confining pressure) the higher the permanent strain, as shown in Figure 2.6 [22,59]. Also, for the same stress
FIGURE 2.6  Effect of Deviatoric Stress on Deformation of Silty Clay under Repeated Loading (22).
FIGURE 2.7  Strength Regain in a Thixotropic Material (24).

FIGURE 2.8  Effect of Thixotropic in Three Clay Minerals (24).
ratio, the higher the confining pressure the higher the permanent strain.

2.3.1.1.c Effect of Thixotropy

The response of cohesive soils to cyclic loadings is greatly influenced by the length of time between sample preparation and testing. Generally, the sample strength increases as the time between preparation and testing (storage time) increases. However, this effect tends to diminish as the number of load applications increases [59]. Several investigations have been conducted to determine the extent to which the sensitivity of natural deposits of saturated clays is attributable to thixotropy [60,61]. The properties of a purely thixotropic material have been illustrated by Skempton and Northey [24] as shown in Figure 2.7. The shear strength of the material assumes a value of $C$ in the undisturbed state as shown in the figure. This value drops to $C_r$ immediately after remolding. If the material is then allowed to remain under constant external conditions and without any change in composition, the strength will gradually increase and after a sufficient length of time the original strength $C$ will be regained. Figure 2.8 shows the thixotropic strength increase for three clay minerals as measured by Skempton and Northey. They reported that Kaolin shows almost no thixotropy and illite shows only a small effect. In contrast, the bentonite shows a remarkable strength regain at very short time interval.

2.3.1.1.d Effect of Stress History

Stress history has a significant effect on permanent strain of soils [55,56,62,63]. It has long been recognized that stress history has an important effect in determining the consolidation and strength characteristics of saturated clays. Recently, it has been shown that changes in the sequence of pressure application can
also affect the swelling characteristic of clays [55,63]. Lentz [23,8] concluded that subjecting soil samples to a low stress level increases their resistance to permanent strain under subsequent higher loads.

2.3.1.1.e Effect of Frequency and Duration

The duration of the stress pulse applied to a subgrade soil by a moving wheel load lasts about 0.01 to 0.1 second under actual field conditions [64]. This duration time is primarily dependent upon the speed of the vehicle and the position of the element under consideration within the pavement structure. Hence, the vehicle speed is inversely related to the load duration. As vehicle speed increases, the duration of loading decreases and visa versa [43]. Barksdale [64] found that the load duration time increases with depth by a factor of about 2.7 from the pavement surface to the subgrade. This is shown in Figures 2.9 and 2.10. Barksdale recommended the use of the appropriate magnitude of the principal stress and its time pulse for investigation of the resilient and permanent characteristics of the soil materials in question.

2.3.1.2 Factors Affecting the Resilient or Elastic Characteristics of Cohesive Soils

Unlike cohesionless soils, cohesive subgrade materials cannot be accurately characterized without great attention being given to the sample preparation. In determining the resilient parameters for clay, the laboratory samples should be identical in composition to the field. This means that water content, density and the structural arrangement of the particles (which is controlled by the method of compaction used in preparing the sample) must be identical. The importance of this may be recognized by knowing that the resilient deformation of a flexible pavement structure is a major contributor to fatigue failure in the asphaltic concrete surface course. Recognition of the importance of the resilient behavior of
FIGURE 2.9 Variation of Equivalent Vertical Stress Pulse Time with Vehicle Velocity and Depth (64).
FIGURE 2.10 Variation of Equivalent Principle Stress Pulse Time with Vehicle Velocity and Depth (64).
flexible pavements is reflected by the fact that many current flexible pavement thickness design philosophies incorporate limiting deflection criterion [65,66]. Generally, the factors that influence the resilient characteristics of cohesive soils include:

2.3.1.2.a Number of Load Applications

Resilient deformation generally decreases as the number of load repetition increases. Thus, deformations that determined under a relatively small number of stress applications may present a misleading picture of the resilient characteristics of the subgrade soil [59,67]. In tests on stiff clays, Dehlen [68] found that 1000 stress repetitions were sufficient to condition the sample for testing without significantly altering the specimen response. He found that once the sample was conditioned, the response obtained at a relatively low number of stress applications was representative. Tanimoto and Nishi [69] also emphasized the importance of selecting the proper number of stress applications to determine the resilient properties. Seed et al. [50] found that the response of clay samples was dependent on the number of stress applications (N). In general, they reported that compacted clays develop their greatest resilient deformation when N is less than 5000.

2.3.1.2.b Confining Pressure

The resilient response of cohesive soils is relatively unaffected by changes in cell pressure during the repeated load triaxial test [43,52,53,54].

2.3.1.2.c Stress-Level

In all investigations, the relationship between the resilient modulus and the principal stress difference is similar. At low stress levels, the resilient modulus decreases and the principal stress difference increases. This is true up to a value of about 10 psi where the
resilient modulus is found to be unaffected or increases only slightly with further increase in principal stress difference. Because of this dependence on the principal stress difference, it is important that laboratory tests be conducted at stresses which are expected in the field. Figure 2.11 shows the decrease in the resilient modulus $M_R$ as the principal stress difference increases from 2 to 10 psi (0.1406 to 0.703 Kg/cm$^2$) under a constant radial pressure. It also shows that Poisson's ratio is only slightly affected by changes in the applied stress. For tests on silty clays Mitchell et al. (58), using 24,000 load applications, found that the resilient modulus decreased with increasing applied stress up to 25 psi (0.176 Kg/cm$^2$), above which the resilient modulus increased slightly. Seed et al. [50] had also found that the resilient modulus decreased rapidly with a variation of 300 to 400 percent as the principal stress difference increased from 3 to 15 psi (0.21 to 1.05 Kg/cm$^2$). Above this range the resilient modulus was observed to increase slightly, as shown in Figure 2.12.

2.3.1.2.d Load Duration and Frequency

Most researchers agree that the effect of stress duration on the resilient response of cohesive soils is negligible. In general, the resilient modulus tends to increase slightly as the time of load duration decreases, this effect is considered insignificant for the range of load durations encountered in pavement structures [59].

Conflicting findings concerning the effects of frequency on the resilient response are reported in the literature. Coffman [71] stated that the resilient modulus increases as the load frequency increases. This increase was on the order of 50 to 400 percent depending on the water content and density of the sample. Tanimoto and Nishi [69], on the other hand, reported a decrease in resilient modulus with an increase in load frequency.
FIGURE 2.11 Secant Modulus and Poisson's Ratio of Clay Subgrade as a Function of Repeated Axial Stress and Depth Beneath Pavement Surface (58)
FIGURE 2.12 Effect of Stress Intensity on Resilient Characteristics for AASHO Road Test Subgrade Soil (50).
Further, Kalcheff and Hicks [67] found that frequency changes had no effect on the resilient modulus.

2.3.1.2.e Compaction Density and Water Content

All investigators have found that increasing water content at compaction leads to an increase in resilient deformation, and a decrease in strength and resilient modulus. For a given compactive effort, the resilient deformation is relatively low at water contents dry of optimum, but it increases rapidly as the water content at compaction exceeds the optimum. Several researchers [70,69,72] found that for a given dry density, the resilient modulus decreased as the water content at compaction increased. Consequently, the resilient deformations increased with the water content. Seed et al. [50] and Tanimoto and Nishi [61] reported similar results. Figure 2.13 from Finn et al. [73], relates the resilient modulus to water content and dry density. It shows the decrease of $M_R$ with increasing water content. It also shows that for a given water content at compaction, as the dry density increases, the resilient modulus also increases, until it levels off at the optimum condition, then $M_R$ begins to decrease slightly.

At high degrees of saturation, minor changes in dry density or water content have significant effects on the resilient behavior. Seed [50] suggested that this is attributable to the marked change which can take place in the soil structure at this range. He feels that it is desirable to compact samples at 80 percent saturation to avoid this and minimize the effects of resilient deformation. One further caution is also made that under field conditions, traffic loading of the subgrade soil may tend to densify it and reduce the water content. Both of these conditions, along with the large number of repeated loadings, will lead to higher strength and resilient modulus than expected. This is an important consideration in pavement deflection predictions.
Water Content

Deviatoric Stress = 2 psi
Confining Pressure = 2 psi
After 1000 load repetitions
Frequency = 20 cycles/min

(1 psi = 0.07 kg/cm²)
(1 pcf = 158 N/m³)

FIGURE 2.13 Water Content - Dry Density - Resilient Modulus Relationship for Subgrade Soil (73).