MECHANISTIC BEHAVIOR OF SATURATED COHESIONLESS LAYERS IN PAVEMENTS

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ABSTRACT

Poor drainage of structural sections on airports and roads leads to weakening of unbound layers and accelerated damage to pavements. This paper presents results from investigations into the effects of saturation on the mechanistic behavior of cohesionless materials. Repeated load testing of poorly drained sites with a falling weight deflectometer was performed. Falling weight deflectometers were not found to simulate traffic wheel loading in terms of shear stress reversal. Laboratory cyclic triaxial testing of several gradations of crushed aggregate base was performed. Pore pressure buildup, variation in cyclic strain (Young’s) modulus, shear modulus, resilient modulus and damping were investigated. Dense graded bases, of low permeability were found to rapidly generate pore pressure to the point of initial liquefaction in strain controlled triaxial tests. Open-graded, permeable bases were found to generate pore pressure much slower and retain greater strength, when subjected to cyclic loading, in the saturated condition.
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CHAPTER 1
INTRODUCTION

1.1 Problem Definition

Excess moisture in pavement layers results in severe damaging actions when heavy loads are applied, Cedegren (1989). Excess moisture in untreated granular pavement sublayers and thawed subgrade can lead to pore pressure buildup under traffic loading. Positive pore pressures tend to overcome confining stresses and frictional forces between soil particles lowering resistance to shear, T.A.I. (1984). Buildup of pore pressure leads to pumping, channeling, erosion, stripping of asphalt coatings of bituminous treated layers, disintegration of cement treated layers, and over stressing of weakened subgrades, Black (1949), Cedegren (1974). These effects may result in loss of foundation support, Raad (1982) and premature pavement failures.
Based on field studies, Cedegren (1974), found that the relative damage due to a single axle load for wet sections varies from 5 to 70,000 times compared to drained sections. It has been estimated that billions of dollars in maintenance and rehabilitation could be saved in the United States if pavements were designed and constructed to drain from the subsurface as well as the surface, Hughes and Allen (1986), Cedegren (1978 & 1989). Alaska is the state with the harshest weather subjecting pavements to extreme conditions of temperature and moisture. Alaska can benefit greatly from improved drainage.

Excess pore pressures may cause erosion as water and fines are moved under the pavement (channeling), or are ejected through surface cracks and joints (pumping), Dempsey (1982). On gravel roads, water and fines work their way up through the gravel on to the driving surface. These mechanisms can reduce the amount of material supporting the surface, lower the density of the supporting layers and increase the compressibility of the material. Settlement and failure of the pavement may result.

Inadequate control of moisture severely affects pavements as a result of frost action. Frost action requires a readily available supply of subsurface water, frost susceptible soil and sustained periods of subfreezing temperatures.
Water moves through the soil by capillary action to a freezing zone, where large ice masses may form. Frost heaves during winter are damaging and especially noticeable if differential frost action occurs resulting in increased roughness and cracking of pavement surfaces.

When the ice lenses thaw, support in the vicinity is drastically reduced. Alligator cracking, pavement disintegration, and potholes may occur in the spring thaw, McHattie, Connor, & Esch (1980). The frozen ground beneath the thaw line remains essentially impermeable. Snow piled at the road sides keep these areas frozen and impermeable to drainage. Then the base is left saturated in a trough or bathtub effect, MacMaster, Wrong, & Phang (1982). The situation is somewhat similar to an undrained triaxial specimen. Provision should be made for adequate drainage and/or base materials that are not susceptible to weakening under these conditions.

Other damaging effects due to excessive moisture in unbound layers may be buoyancy and expansion, The Asphalt Institute (1984). The buoyant effect of water reduces the weight of particles and correspondingly reduces the friction between them.
The volume of some soils is greatly affected by the addition of water, causing differential heaving and weakening of the structure. Expansive soils are not studied here. They are mentioned as one of the problems that may arise in poorly drained sections.

Alaska uses dense graded base course materials in standard construction practice, AKDOT&Pf (1972, 1981, 1988). However the trend in recent years has been towards use of slightly less material passing the No. 200 (0.075 mm) sieve. Still, up to 6% passing the No. 200 sieve is allowed in crushed aggregate base course. Dense graded materials are believed easier to use in construction than open graded materials since the fines lend the material to greater stability and ease of compaction near optimum moisture content.

An essential property of asphalt pavements is that "It must be waterproof and sloped to shed surface water to the roadside, and thus protect the entire pavement structure and the subgrade from the weakening effects of moisture.", T.A.I. (1989). This property can rarely be met in cold regions where thermal cracking and frost action are prevalent. Perhaps the dense graded bases are felt to impede the infiltration of water.
Dense graded materials when compacted have very low permeabilities. However, the low permeabilities, while impeding infiltration, also impede drainage. The presence of silt may also enhance frost action.

Materials of high permeability (open graded, containing little to no fines) will drain rapidly when moisture infiltrates and are not subject to frost heaving. When properly confined open graded materials have been found to be more resistant to cyclic loads than dense graded materials even in the saturated condition, Wong, Seed & Chan (1975).

A study of construction using open graded bases in Pennsylvania concluded that adequate stability to support construction equipment was provided by these materials, Hoffman (1982). That study also concluded that porous materials used for drainage should have a minimal amount passing the No. 10 (2 mm) sieve. It was found that even 5% of this fine material could significantly alter the permeability.

The falling weight deflectometer (FWD) is used in Alaska for the purpose of determining load restrictions and for design of pavement rehabilitation projects. This equipment drops a weight onto a round plate placed on the surface which simulates a design wheel load in terms of vertical stress.
However, the FWD does not impart a shear stress reversal on the structural section similar to the passing of a wheel load. See Figure 1.1. Shear stress reversal is necessary for the increase of pore pressure within material of high moisture content. Therefore, the FWD, while being useful for determining layer moduli of well drained section, generally does not measure the true pavement reaction to repeated loads. Since it was the only dynamic pavement testing equipment available at the time, the FWD has been used in this study.

This study presents results of testing regarding the mechanistic effects of moisture on cohesionless soils (sand and gravel). The primary variables are moisture content, gradation and density. Raad (1982), presented results of field testing and analysis of pumping mechanisms under rigid (concrete) pavements. It was found that saturated sublayers would develop pore pressures to the level of their initial confining stress (initial liquefaction) under repeated loading. It was also reported that the tendencies of granular bases to reach the point of initial liquefaction was a function of permeability, compressibility and number of load repetitions. Raad’s pilot study is the foundation of this thesis.
Figure 1.1  Typical Shear Stress Resulting from a Truck Wheel Load
1.2 Research Objectives and Scope

In this study the effects of static and dynamic loading on granular materials of high moisture content were evaluated. Specifically, the main objectives are:

1. To determine the extent of drainage problems in the area of Fairbanks, Alaska. Several other areas outside Fairbanks, including Anchorage, Nome, Kodiak, and the Alaska Highway near the Canadian border were considered.

2. To field test using the FWD at selected sites exhibiting drainage problems. The intent is to assess loss of pavement support associated with poor drainage. Additional tests to determine moisture content and density at these sites have been performed.

3. To investigate experimentally the behavior of typical saturated base course materials under triaxial conditions that simulate traffic loads. Stress-strain characteristics, pore pressure generation, damping, permeability, and compressibility were determined from specimens.
4. To compare analysis of the performance of saturated granular bases with different gradations in terms of excess pore pressure generation under dynamic loads and potential loss of foundation support.

5. To present procedures for determining limiting stress and strain values of a base course in order to minimize pavement damage associated with excess moisture conditions.

1.3 Research Methodology

Correlations between poorly drained sublayers and pavement performance are investigated. Alaska Department of Transportation and Public Facilities (AKDOT&PF) personnel assisted in identifying test locations. Their recommendations helped locate problem areas. Poorly drained sections of three airports and ten roads were investigated.

Literature searches were performed through the University of Alaska Fairbanks Library, the National Transportation Research Service in Washington, D.C. and the AKDOT&PF Transportation Technology Transfer Section. Subjects for which all available information on publications were ordered include:
1. Pavement design and drainage
2. Liquefaction of soils
3. Pumping, channeling, and erosion in pavements
4. Soils/pavement damping
5. Moisture content and modulus of soils

Field investigations were performed between the Spring of 1990 and the Summer of 1991. These investigations involved examination of flexible pavements using a Dynatest FWD. This equipment is designed to impact the pavement surface with a dynamic load and measure deflections to the nearest 0.001 inch at the load center and at six radial locations. An internal load cell measures the drop load (approximately 6000-18000 lbs.) and the peak deflections are recorded with velocity transducers. Repeated load tests were performed at each test site. Sequences of up to sixty-four drop tests were conducted to measure the pavement's reaction. Data are copied to a computer disk and used in backcalculation of layer moduli. Backcalculation was performed using the Dynatest computer program ELMOD, Version 3.2.

Subgrade soil and base samples were obtained at the sites for the purpose of determining moisture content and gradations. A limited number of density tests were performed on the base course using a Washington Densimeter.
Triaxial testing was chosen as the best means of simulating field conditions. First a series of static and dynamic tests were performed using Ottawa sand. The purpose of testing Ottawa sand is to insure results are consistent with similar tests presented in the literature, e.g., Seed and Lee (1966).

Strain controlled cyclic tests were run on three gradations of crushed aggregate base course. One dense graded base and two open graded bases were tested. The base courses were tested first at low moisture content condition and then when saturated. Subsequent comparisons are then made regarding variation of strength (cyclic strain modulus, shear modulus and resilient modulus), pore pressure buildup and damping between the two test series.
CHAPTER 2
LITERATURE REVIEW

2.1 Introduction

It is the purpose of this chapter to provide the reader with a state-of-the-art summary of the following four subject areas. They are: 1) drainage; 2) liquefaction of soils; 3) pumping, channeling, erosion and; 4) soil damping.

2.2 Drainage

It has been said that the most important criteria regarding pavement design is drainage. Probably one of the most outspoken proponents of pavement drainage in modern times is Harry R. Cedergren. Cedergren has written and/or co-authored several books, research reports, and journal articles (1962, 1973, 1974, 1978, 1979, 1989) pertaining to the subject of subsurface drainage. Cedergren (1973), estimated that "more than 90 percent of the major pavements in the United States may be periodically exposed to surface water inflows in sufficient quantities to cause significant saturation and flooding of pavement structural sections".
Cedergren (1989), said that few designers even consider pavement drainage as a viable design concept to increase pavement life. This statement is true, referencing the author's 8 years of engineering design practice in Alaska.

Surprisingly large volumes of water may enter pavement sections and yet the drainage rates of "standard" dense graded bases and subbases are very slow. Normal practice is to construct paved sections in layers with increasing amounts of fine materials, AKDOT&PF (1982). Since permeability decreases with increasing fines, Cedergren (1989), water entering at the surface may be effectively trapped and until critical levels when the material weakens. If pore pressures increase, the compressibility of the material will increases, Seed, et al. (1975). When this happens under traffic loading, eventually the support for the pavement is lost.

Evidence that many pavements are slow draining systems can be seen by the bleeding and pumping of water out of joints and cracks, even several days after rain has stopped. Cedergren (1974), observed this in almost every part of the country. Similar observations have been made in Alaska in the spring of the year when of snow adjacent to pavements and ice thaws in frost susceptible layers. The resulting moisture levels can actually exceed saturation limits, McHattie, Connor & Esch (1980).
The influence of base permeability on the drainage of granular layers in pavement structures has been recognized by a number of investigators including: Cedergren (1974); Markow (1982); Moulton (1980); and Mathis (1989). Increasing the base permeability can significantly decrease drainage time. According to Darcy's Law, Lambe & Whitman (1969), the rate of flow is directly proportional to the permeability of the material. Open graded bases may have permeability coefficients from 2 to 4 orders of magnitude greater than dense graded bases, Hoffman (1982). Open graded bases therefore reduce the time that water is contained thereby reducing damage to the pavement during periods of base soaking. Bases of higher permeability should be used in areas subject to soaking.

The Federal Highway Administration surveyed ten States known to have constructed permeable base pavements, Mathis (1989). Most of the States use permeable bases under Portland Cement Concrete (PCC) pavement, but several are using them under asphalt concrete pavement. The grading specifications Mathis reported as used by several states are shown in Tables 2.1 and 2.2.
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| Coefficient of Permeability (feet/day) | 20,000 | 1,000 | 200 | 2,000 | 1,000 | 18,000 |
Since the first literature reference to asphalt treated permeable base, Lovering & Cedegren (1962), its use has proliferated. The addition of 2 to 3% asphalt to the open graded aggregate has been reported to compact readily to form firm, nonshifting drainage layers with a high degree of permeability, Cedegren (1989). Munn (1990), listed more than 10 other states which had plans to build pavement sections with permeable bases. Studies of pavements constructed on permeable bases in California suggest a minimum service life increase of 33% for asphalt concrete pavements and 50% for Portland Cement Concrete pavements, Mathis (1989).
The effectiveness of sub-surface drains in dissipating dynamic pore pressure depends on permeability (k) and volume compressibility (\( m_v \)) of the drainage material. Assuming one dimensional flow and applying Darcy’s law, the basic differential equation for simultaneous generation and dissipation of pore water pressure can be written.

\[
\frac{\partial u_r}{\partial t} = \left( \frac{k}{m_v \gamma_w} \right) \frac{\partial^2 u_r}{\partial z^2} + \left( \frac{\partial u_i}{\partial N} \right) \left( \frac{dN}{dt} \right) \tag{2.1}
\]

where: \( m_v = \epsilon_v / \Delta u \), volumetric strain (\( \epsilon_v \)) per change in pore pressure.

\( \gamma_w \) = Density of water

\( \frac{\partial^2 u_r}{\partial z^2} \) = Change in rate of residual pore pressure (\( u_r \)) in terms of depth \( z \)

\( \frac{\partial u_i}{\partial N} \) = Rate of residual pore pressure generation under undrained loading condition (\( u_i \)) with number of load repetitions, \( N \)

\( \frac{dN}{dt} \) = Frequency of load applications in terms to time (\( t \)).
From Eq. 2.1 it is seen that the greater the permeability, the faster pore pressure will be dissipated in the stratum, Raad (1982). Conversely, the less the compressibility of the material the faster pore pressure can be dissipated.

2.3 Liquefaction of Soils

Engineering research in liquefaction of soils accelerated after several earthquakes caused widespread damage in the late 1950’s and early 1960’s. Liquefaction of saturated sands during these earthquakes caused severe building settlements, tilting of buildings and numerous slope failures, H. Seed & Lee (1966). Up to this time, liquefaction under dynamic loading, was only understood qualitatively.

Casagrande (1936), found that under high shear stress dense sands tend to expand (dilate) whereas loose sands tend to decrease in volume (compress). For any sand there was claimed to be some critical void ratio, for which no volume change during drained shear, and no pore pressure changes during undrained shear will occur. Casagrande reasoned that sands with void ratios above the critical value and therefore tending to contract during shear, would, under undrained conditions develop positive pore water pressure that might
possibly become large enough to produce liquefaction. On the other hand, sands having void ratios below the critical value would tend to dilate during shear, producing a decrease in pore water pressure and a corresponding increase in effective confining stress under undrained conditions.

"Effective confining stress," as used here refers to the idealized situation where the soil is saturated in which all the voids are filled with water, where it is equal to the confining stress minus the pore pressure. This method of determining effective stress has been found to work well in soils when applied for engineering purposes. When the material in question is partially saturated and/or is made up of rock formations other methods for estimating the effective confining stress should be used, Skempton (1960).

Casagrande (1938), noted that the critical void ratio was not a constant value for a given sand, but depends on the confining pressure to which the sand is subjected. Because dilation tendencies are smaller at high confining pressures, the critical void ratio decreases as the confining pressure increases. Thus it has sometimes been concluded that a saturated sand under a high confining pressure is potentially less stable (producing compressive characteristics) than under low confining pressures (producing dilettante characteristics).
The critical void ratio approach is applicable only for static stress applications. Casagrande (1936), said that volume changes under cyclic load loading conditions are quite different than from those occurring under one-dimensional static loading and that his concept could hardly be expected to be applicable to dynamic loading.

Seed & Lee (1966), presented a report pioneering work in analysis and testing of liquefaction of sands during cyclic loading. In their study, undrained triaxial tests were performed on saturated sands applying a cyclic deviator stress maintaining the mean of the minor and major principal stresses constant at all times. This condition was shown to simulate cyclic shear stresses similar to ground shaking during earthquakes. Cyclic strain tests were also performed in this study. Samples were subjected to cyclic tests up to the point of initial liquefaction (pore water pressure equal to initial confining stress). Static tests were performed following liquefaction.

Variables considered were density, confining pressure, and magnitude of deviator stress. The number of cycles required to produce liquefaction under comparative conditions were found so that hypotheses regarding relative effects were determined. Important fundamental findings of the study were:
1. Lower density sands liquefy more rapidly than high density sands.

2. Higher confining pressures decrease the likelihood of liquefaction of sands to occur.

3. Higher deviator stresses caused liquefaction to occur more rapidly.

4. Static tests following liquefaction gave large strains initially, then the sand strengthened as dilation occurred and pore pressures decreased.

This paper paved the way for many research projects that have the intent of determining liquefaction potential of sands for application to earthquake engineering problems. Further studies in liquefaction involved refining testing, design, and analytical procedures. Other test methods that have developed (besides triaxial testing) include Simple Shear test equipment (Peacock & Seed, 1968), Shake tables (De Alba, 1975), and Centrifuges. (Committee on Earthquake Engineering, 1985). H.B. Seed’s research dominated liquefaction studies until his recent death.

Ladd et.al. (1989), reported data from strain controlled testing that for low shear strains (above the threshold where pore pressure build-up occurs) that pore pressure buildup is basically independent of relative density. The remaining three findings of Seed and Lee still seem to be true.
Cyclic stresses are also imparted to pavement structural layers which are also subject to flooding and saturation. Therefore the understanding of liquefaction is important to pavement engineering.

In order to model what is occurring under traffic loads, cyclic loading resulting in shear stress reversals should be imparted to test samples. This is most easily accomplished with the equipment available using strain-controlled testing where the sample is cycled between zero strain and a fixed amount of compressive strain. With this type of testing when any permanent deformation occurs the rebound to zero creates a slight tensile stress on the sample thereby causing the full cycle to go through shear stress reversal similar to the field. Shear stress is defined as one half the deviator stress, as in elastic theory, Timoshenko & Goodier (1987).

Using triaxial test apparatus, cylindrical samples were contained in a rubber membrane and confined either by air or water pressure. Drainage lines were connected to the top and bottom of the sample. The cell pressure (confining stress) and pore pressure were monitored with transducers. The load on the sample was monitored with a load cell inside the triaxial cell. Deformations were monitored with LVDT’s. Tests could be run using either stress or strain control by using the load cell or LVDT’s respectively as controlling devices.
Finn, Yong, and Lee (1978) concluded that generally only sands and silts are prone to liquefaction. Free draining gravels are not considered likely to liquefy because of rapid dissipation of any excess pore water pressure generated by a dynamic force. However, coarse soils, such as gravels, have been observed to liquefy in field conditions by several researchers, e.g., Black (1949); Coulter and Migliaccio (1966); Chang (1978); Raad (1982); Dempsey (1982); Ishihara (1984); and Youd et. al. (1984).

Even though liquefaction has been observed to occur in gravelly soils in the field, it has evidently not often accomplished in the laboratory. Some extensive research performed for the U.S. Army Corps of Engineers by Seed and Anwar (1987), states: "In fact, it is virtually impossible to demonstrate the occurrence of liquefaction on even very loose coarse gravelly soils because of membrane compliance effects...."

Membrane compliance refers to changes in sample volume during undrained testing due to changes in membrane penetration around voids on the surface of coarse grained samples. It is brought about by changes in pore pressure or cell pressure. If the membrane moves during testing, pore pressure measurements will be inaccurate. This phenomena could lead to overestimation of a sample's degree of saturation when testing B-values, Lade & Hernandez
(1977), as well as overestimation of the resistance to liquefaction.

A fundamental assumption in undrained triaxial testing is that no sample volume changes occur except for a negligible compression of pore water as a result of increased pore pressure. It was first observed by Newland and Allely (1959), that penetration of rubber membranes used to confine the sample into surficial voids on coarse grained specimens varies as a function of the effective confining pressure. This study noted significant volume change for membrane penetration in changing cell pressures from 7.5 to 80 psi.

The study of Seed and Anwar (1987), determined an empirical equation for estimating the volume of membrane penetration as a function of grain size and cycle change in effective confining stresses of greater than 1 psi. Unbound layers beneath pavements are expected to have effective confining pressure of less than 5 psi. Dynamic testing performed in this study commenced at an effective confining pressure of 5 psi. The maximum cyclic change in effective confining stress could only be 5 psi in the case where a sample went to the point of initial liquefaction in one cycle. This was never the case. Derivation and application of a method for estimating membrane compliance effects based on Seed and Anwar’s findings is in Appendix A.
2.4 Predictive Models

Several predictive models for pumping have been developed over the past 3 decades with very limited success. All are intended to model actions under PCC pavements, therefore they are generally not applicable to Alaskan AC pavement systems.

The only two which may be applicable to general conditions due to their theoretical, rather than empirical nature are Equation 2.1 (discussed earlier) and one by Dempsey (1982).

Dempsey used the Bernoulli energy equation and states that when water velocity exceeds about 16 fps in unbound base courses, erosion and pumping can be expected. Thus, ejected water velocity could be predicted from the one dimensional energy equation:

\[ Z_b + \left( \frac{V_b^2}{2g} \right) + \left( \frac{P_b}{\gamma_w} \right) = Z_o + \left( \frac{V_o^2}{2g} \right) + \left( \frac{P_o}{\gamma_w} \right) + h_f \quad (2.2) \]

where

- \( Z_b \) = Vertical distance from datum to location in base course
- \( V_b \) = Water velocity in base (assumed to be zero)
- \( P_b \) = Pore water pressure in base
- \( Z_o \) = Vertical distance from datum to pavement surface (zero)
\[ V_o = \text{Velocity of water at pavement surface} \]
\[ P_o = \text{Pore water pressure at pavement surface} \]
\[ h_f = \text{Friction head loss} \]
\[ g = \text{Acceleration of gravity} \]
\[ \gamma_w = \text{Density of water} \]

If the datum and water level are assumed to be at the pavement surface and that friction losses are negligible (e.g. in a crack), the solution of equation (2.2) for a depth 12 inches below the pavement surface and an excess pore pressure of 3 psi would indicate an ejected water velocity of about 21.1 feet per second. This exceeds the 16 feet per second criteria.

The velocity limitation would have to be a function of a particular gradation. These limits would have to be determined in the laboratory. Dempsey stated that only 3/8" or smaller, material will erode and pump. This is roughly consistent with field observations of Havers and Yoder (1957). It may not be accurate to assume zero head loss and water velocity in the base. However, this model may be useful in analyzing field instrumented sections.
Based on a limited test track study and laboratory experiments, Dempsey hypothesized that loads on dense graded base course materials first cause residual pore pressure build up and liquefaction. This is followed by pore pressure release through channels and pumping once an outlet is formed. He found that two material properties of great importance in pumping analysis are compressibility and permeability.

When significant amounts of minus no. 4 sieve (4.75 mm) materials are present in a base, it is this fraction that can be expected to pump, channel and erode. From a materials standpoint, the minus no. 4 material is only included to increase density, fill voids and add stability. Material passing the No. 200 sieve helps provide cohesion to a gravel by blocking pores to maintain negative pore pressures caused by dilation of the gravel during the compaction process. So, minus no. 4 material contents of structural embankments have their benefits and liabilities. The benefits being that they provide a more workable material, which may be compacted with greater ease. The liabilities are that their low permeabilities cause the material to hold moisture and if excess pore pressures are then generated they will erode.
2.5 Soil Damping

A pavement with its sublayers (e.g., base course, subbase, embankment, and subgrade) is a structural system subject to static and dynamic loading. The action of traffic loading results in forced vibrations. The system may be analyzed as a soil-structure interaction problem at the interface between the pavement and unbound layers.

The loading magnitudes and frequencies are never constant and are influenced by variables such as tire pressure, tread design, surface condition, and vehicular suspension. The pavement is a multi-degree of freedom, layered system with each of the layers' material properties varying with the applied load, time, temperature, moisture content, and thickness. Variations are often non-linear, Ullidtz (1987).

Damping is not normally considered in pavement design. However, damping plays an important part in the pavement's reaction dynamic loading, Thompson (1963). As will be seen, the moisture content and pore pressure greatly effect the damping of cohesionless materials used in pavements.
There are many possible ways of modeling damping effects. A few of these methods are listed below.

A. Linear damping:

1. Damped Oscillator Methods
2. Voight Model
3. Maxwell Model
4. Stress-Strain Energy Methods

B. Non-linear damping:

1. The Bilinear Model
2. Non-linear Energy Methods
3. Ramberg-Osgood Model
4. Davidenkov Model

An excellent literature search including development of the above, with variations, is found in Martin (1976). More investigations with applications of damping may be found in Newmark and Rosenbleuth, (1971); Lazon, (1968); Jacobsen, (1930 & 1960); Thompson, (1963); Lysmer & Richart, (1966); Ehrler, (1968); Komamura and Huang, (1974); Dobry et. al., (1986(2)); Hardin and Drnevich, (1972) and Hunt, (1986), to name a few.
Damping of pavement layers in the field is found to decrease stress response (as will be shown in Chapter 3). In poorly drained pavements this damping is not beneficial, but rather, indicative of impending failure.

For the purposes of this study, where it is interesting to be able to monitor damping characteristics, a hysteretic stress-strain model resulting in a damping ratio seems best.

The damping analysis used in this study applies the equivalent linear damping ratio ($\beta_{eq}$) for a non-linear material, Martin (1975), which is defined as:

$$\beta_{eq} = \frac{1}{4\pi} \frac{\Delta W}{W} \quad (2.3)$$

where: $\Delta W =$ the area inside of a shear stress-strain hysteresis loop for a test cycle (damping energy).

$W =$ the area of the triangle similar to as shown in Figure 2.1 (strain energy)
Figure 2.1 shows the typical reaction of a linear material. Cyclic tests of non-linear materials, such as soils, display hysteresis loops having sharp corners at their extreme points, Martin (1976). Therefore, the damping energy and strain energy found in determination of $\beta_{eq}$ in this study uses equivalent areas for non-linear materials as shown in Figure 2.2. The background and derivation is explained in Appendix B.
Voight Model
\[ \gamma = \sin pt \]
\[ \tau = G \sin pt + \eta_p \cos pt \]

Damping Energy
\[ \Delta W = \int \tau d\gamma \]
\[ \Delta W = \pi \eta_p \gamma_a^2 \]

Strain Energy
\[ W = \int_0^{\gamma_a} C \gamma d\gamma \]
\[ W = \frac{1}{2} C \gamma_a^2 \]

Figure 2.1 Dynamic Characteristics of Voight Model
Figure 2.2  Equivalent Linearization of Damping
CHAPTER 3
FIELD TESTING

3.1 Introduction

This Chapter describes field testing, procedures and results. Between April, 1990 and August, 1991 numerous pavement locations identified as poor drainage locations were tested. The field testing procedure involved use of either a Dynatest Model 800 or Model 8000 Falling Weight Deflectometer (FWD). When practicable, gradation, density and moisture contents of all sublayers were determined.

Under repeated load testing, the FWD data can be used to illustrate or determine:

1) Variation of surface deflections with repetitions.

2) Variation of pavement layer moduli with repetitions.

3) Variation of deflection with distance from load.

4) Depth of theoretical stiff layer (est. of thaw depth).
These variations in FWD data and backcalculation results are presented graphically in this Chapter. Background regarding the FWD and backcalculation procedures use in analysis are presented in Appendix C.

The purpose of repeated load testing of poor drainage sites is to find indications of pore pressure generation. Indications of pore pressure increases during repeated load testing would be:

1) Pumping of water on to the surface through cracks adjacent to the FWD loading plate
2) Deflections increasing with repeated loads
3) Backcalculated layer moduli decreasing with repeated loading
4) Failure of the pavement

Each of these phenomena did occur, to some extent, during FWD testing of poorly drained sites. But the limitations of the equipment, e.g., deflection readings greater than 80 mils, have made test result from particularly poor sites unreportable. Test sites and results for which data was obtained within the accurate range of the FWD are described in the following sections. Some description of out-of-range tests are included for information.
3.2 Paved Road

Holmes Road is a 2 lane minor collector paved in 1985. It is located approximately 5 miles east of Fairbanks, Alaska, and is 3 miles in length connecting two arterial roads. The traffic is generally automobiles and light trucks. The 1989 average annual daily traffic (AADT) was estimated at 700 (Fantazzi, et. al, 1989).

Holmes Road has poor drainage conditions including: a relatively low embankment (36" or less); many approaches (hampering drainage); flat terrain; and brush growing in the ditches. Standing water was the edges edge elevation in many locations at the time of testing (September, 1990). The road was also alligator cracked and potholed in several locations.

Subgrade materials are silty sands. The material is non-plastic, but a liquid limit of 40.5 was determined from a sample taken at one test site. The base course and embankment are made from a well graded gravel with grain sizes ranging from 2" and down (See Figure 3.1).
Figure 3.1 Holmes Road Gradations
With a Plasticity Index (PI) of 16.3 and 21% passing the No. 200 sieve, this material meets only the lowest grade Alaska material specifications of Selected Material, Type C, AKDOT&PF (1988). It could be that the originally placed material has degraded and/or fines from the subgrade have pumped into the structural fill.

Tests performed at 0.2 miles on center were conducted and the backcalculation results for the second nominal 9000 lb. drop are shown in Figure 3.2. Locations showing higher moduli were in areas of better drainage, as indicated by clearing at the roadside, and/or deeper ditches.

The location of the repeated load test was a mile 0.9. This location was not the worst on the road, but the pavement was more continuous there than others where alligator cracking predominated. Tests using sixty drops on the FWD plate were conducted at this location. The testing procedure involved: first using a nominal 6000 lb. drop force (drop ht. 1); then using a nominal 9000 lb. (drop ht. 2) drop force, at approximately 4 second intervals.
Figure 3.2 Holmes Road - Full Length Backcalculation Results
Figure 3.3 shows the deflection readings from the first two tests. Deflections generally diminished and stabilized after about 10 drops. However, the readings of over 30 thousands of an inch (mils) at the 9000 lb. drop force are indicative of a weak section. Similarly, readings greater than 20 mils on the lower drop force are higher than desirable.

The points on Figure 3.4 show backcalculated moduli for the first 6000 lb. drop force sequence. The moduli remained fairly constant throughout the test with slight strengthening. The average moduli determined here are 7 ksi for the subgrade and 28.5 ksi for the fill. The fill is remarkably strong considering its high fines content and damp nature. With the maximum aggregate size being over 1-1/2", some extra resistance to shear may be afforded. The computed depth to a stiff layer, by Eq. C8 in Appendix C, is 140 inches.

At the 9000 lb. drop force the moduli decreased (See Figure 3.5). Average for the subgrade and fill were determined at 27 ksi and 6 ksi respectively. This type of behavior indicates that the effective confining stress might be decreasing with increasing vertical stress due to increases pore water pressure.
Figure 3.3 Holmes Road - FWD Deflection Measurements
Figure 3.4  Holmes Road - Backcalculated Moduli at 6000 lb. Drop Force
Figure 3.5  Holmes Road - Backcalculated Moduli at 9000 lb. Drop Force
The resilient modulus is often considered to be dependent on the bulk stress at a point, Chou (1977). Confining stress has a primary influence on the modulus. With pore pressures increasing, the effective confining stress decreases and the modulus can be expected to decrease.

3.3 Gravel Road

A gravel road Section in south Fairbanks was reported by AKDOT&PF Maintenance personnel to be pumping water onto the surface under traffic loading. Upon visiting the site, the pumping section was found to consist of 8" of gravel surfacing over an organic silt subgrade. The pumping of water to the surface caused standing water to remain there.

A dryer section on the same road was also investigated. There, the gravel surfacing was 36 inches in thickness overlying similar subgrade materials. Figure 3.6 shows gradations that were found at each location.
0.45 POWER GRADATION CHART

- ◊ WET SECTION
- △ SUBGRADE
- ○ DRY SECTION

SIEVE SIZES

Figure 3.6 Gravel Road Gradations
3.3.1 *Spring Testing*

Falling weight deflectometer testing at the site was first attempted on May 12, 1990. Even at the lowest possible drop force (impacting less than 30 psi at approximately 3000 lb.), all test readings were off range of the equipment. Backcalculating moduli with this data gave a surface moduli ranging from 4 to 11 ksi and subgrade moduli ranging from 3 to 0 (which is impossible). The deflections appear to be increasing with repetitions on the sixty-drop tests, but it was hard to tell. Pore pressures were likely increasing in the loading plate vicinity as evidenced by water coming to the surface adjacent to it and indentation of loading plate into the surface.

The dryer section was tested on the same date using the same sixty drop test at the lowest drop height. Here the embankment thickness has increased to 36" over similar subgrade of lower moisture content. Maximum deflection readings at this site averaged only slightly greater than 20 mils, starting at 23 mils and slowly diminishing to 21 mils.
3.3.2 Summer Testing

On June 6, 1992 the gravel road sites were again visited. It was hoped that the elapsed time between the last test would have allowed the "wet section" to harden up so that measurements within the range of the FWD can be obtained. The weather had been dry, but was cloudy at the time of testing.

The dry section was initially tested, using drop heights 1, 2, and 3 (6000, 9000, 12000 lb. nominal drop forces). Just at the completion of this testing sequence, it began to rain. The rainfall was heavy, lasting about one hour.

As a result of the rainfall, the moisture content of the surface (0 - 6") increased from 2% to 6.8%. Duplicate tests were at the same location were run following the rainfall. Comparisons of the centerline deflection readings obtained are shown in Figures 3.7 to 3.9.

Mean deflection increases between before and after rainfall were found to be 38.4%, 40.6% and 32.2%, for drop heights 1, 2, and 3 respectively. It is interesting to notice when comparing these figures that each stress level has a different response.
Figure 3.7  Gravel Road Deflections Before and After Rainfall - 6000 lb. Force
Figure 3.8  Gravel Road Deflections Before and After Rainfall-9000 lb. Force
Figure 3.9  Gravel Road Deflections Before and After Rainfall—12000 lb. Force
At the lowest level (shown in Figure 3.7) the deflections attenuate for the first 3 drops on both tests. The deflection measurements taken before rainfall tend to stabilize, while those measured after rainfall gradually decrease.

The load cell measurements show decreased stress after the rainfall for the same drop height. An approximately 15 psi decrease was found at drop height 1. This is on the order of 25%. Evidently the higher moisture content dampens the load. This was also found to occur in the laboratory.

At drop height 2 (initially approximately 88 psi), the deflections increase with repetitions. Before the rainfall, the deflections stabilize after about 20 repetitions. The tests run after the rainfall show the deflections generally increase for about 50 repetitions. The damping due to the moisture content for this test amounted to about 14 psi or 16%.

The drop height 3 tests (Figure 3.9) are probably at stress levels beyond what this type of road should vary (initially approximately 120 psi). Tests before rainfall show variable trends and the apparent stabilization at about 50 repetitions. Deflection levels measured after rainfall are nearly to the limit of accurate measurement with the FWD (80 mils).
Deflections increase for the first 10 drops then gradually decrease up to drop 40, then increase again. A combination of pore pressure buildup and drainage or dilation of the material may be occurring.

The backcalculation results show the moduli to be both stress and moisture dependent. Under both conditions, the moduli are seen to drop under increased stress (See Figures 3.10 to 3.12).

The average modulus of the embankment material is seen to drop from 32 to 23 ksi for an average stress increase of from 64 to 119 psi (drop height 1 to 3). Moisture appears to have the greater effect (See Figure 3.13) decreasing the moduli by an average of 29%. The subgrade was little affected by the brief rain shower.

In the dry condition (prior to rainfall) the embankment modulus is nearly linearly related to the drop stress at the surface. After the rainfall the relationship becomes nonlinear. The moduli of granular materials are dependent on the confining stress as well as deviator stress.
Figure 3.10 Gravel Road Backcalculated Modulus Comparison-6000 lb. Force
Figure 3.11 Gravel Road Backcalculated Modulus Comparison - 9000 lb. Force
Figure 3.12 Gravel Road Backcalculated Modulus Comparison-12000 lb. Force
Figure 3.13  Gravel Road Embankment - Backcalculation Summary
The backcalculation summary graph (Figure 3.13) shows the confinement being roughly linearly related to the vertical stress in the dryer state. Bonds created by the fines are broken by high moisture contents and the effective confining stress decreases. The FWD drop heights proceed from left to right on this graph. With the loading plate radius at 5.9 inches, the stresses on a firm surface should be approximately 55 psi, 82 psi, 110 psi at drop heights 1 through 3, respectively. Material properties and equipment variation may change these between tests. When comparing the measured stresses at the same drop height before and after rainfall, a decrease of over 10 psi is observed. This demonstrates the greater damping of moist materials. Figure 3.14 shows the variation in the subgrade moduli. The subgrade has weakened slightly, but in the short time between tests there was be little chance for moisture infiltration to 36 inches below the surface.

Another attempt was made to test the wet section, after this rainfall. But even at the lowest drop height, with measured stresses in the 40 psi range, the deflection readings were off range. As mentioned earlier, the FWD is not suitable to measure very poor sections.
Figure 3.14  Gravel Road Subgrade - Backcalculation Summary
3.3.3 Fall Testing

By September 4, 1991, the moisture content of the gravel fill in the "wet section" of this gravel road had decreased to about 6%, and the area could barely be tested with the FWD. Still, measured deflections at drop height 1 were borderline to the off-range level ( > 80 mils).

The deflection readings are even higher when normalized to the 6000 lb. drop force which should result from this level of impact. The average measured drop force for this sixty drop sequence was 4900 lbs. (45 psi). The normalized centerline deflections were gradually increasing (See Figure 3.15) after decreasing on the first few drops. It is easily conceivable that pore pressures were building slightly in the system. The actual measured deflections were around 80 mils.

Upon backcalculation of moduli, the subgrade modulus was found to remain fairly constant at a low 2 ksi. The surface layer modulus varied, and averaged 14.8 ksi. See Figure 3.16 for results. The computed depth to stiff layer for this test averaged about 75 inches. This is consistent with typical measured thaw depths on this type of ground.
FIGURE 3.15  Gravel Road Wet Section Deflections at 6000 lb. Force
Figure 3.16 Gravel Road Backcalculated Moduli - Wet Section
3.4 **Airport Taxiway**

A particular area of a general aviation taxiway at Fairbanks International Airport is prone to drainage problems, settlement, and pavement cracking. A paved apron abuts one side of the taxiway, while the other side is a grassy area of about 1 foot lower elevation. The grassy area had formerly been brush covered. According to the Airport Maintenance Foreman pavement drainage problems were worse when the side was covered with brush.

Some success has been accomplished by cutting brush adjacent to paved areas, plowing the ground, then planting grass. Evidently, since grass provides little shade, evaporation and transpiration may take place more rapidly. This helps to remove water from subgrade soils.

The subgrade in this area is a sandy gravel. The embankment materials are slightly coarser. Figure 3.17 shows gradations of the unbound materials.

The structural section is 2" of asphalt concrete pavement over 30" of uncrushed embankment on subgrade. Prior to paving this was a gravel taxiway. No particular base layer was discernable upon excavation.
Figure 3.17  Airport Taxiway Gradations
This pavement was first FWD tested after an approximately 24 hour rainfall on September 5, 1990. It was raining at the time of testing also. The run-off from the apron covered the taxiway with a sheet of water, making excavation for the purpose of obtaining moisture content samples impractical.

The first testing run involved hitting the pavement with four drop heights (6000, 9000, 12000, 16000 lbs.), at 100 foot centers to compare results. Figures 3.18 and 3.19 show the results of moduli backcalculations for the embankment and subgrade respectively.

The "poor drainage" location at a distance of 127 feet from the intersection with another taxiway. This intersection is the reference point for locating test points. A section at distance 230 feet also exhibited some differential settlement. Notice that the backcalculated moduli here generally increased with increasing drop force. This is different than other poor drainage areas, such as the gravel road, where decreases in moduli were computed with increased drop stresses.
Figure 3.18  Airport Taxiway Embankment - Full Length Backcalculation Results
Figure 3.19 Airport Taxiway Subgrade - Full Length Backcalculation Results
When moduli increase with surface stress (deviator stress), it is an indication that the effective confinement is not diminished in excess of the vertical stress. Also intergranular friction must not be diminishing greatly. The moduli of the poorly drained site are relatively low, but not to a point indicating immanent structural failure.

The computed strain at the base of the asphalt concrete using the minimum moduli found at the poor drainage location and the surface modulus of 900 ksi (at 55 deg.F), is 388 microstrains. This strain would result from a pass of a twin otter which is the design vehicle for this type of taxiway. The predicted number of repetitions of this type of traffic to fatigue failure in the pavement is over 100,000, using the NCHRP 1-10B Equation, T.A.I. (1982).

For the purpose of comparison the moduli of the embankment at a better test point is 65% higher with subgrade moduli 113% higher. Using these moduli supporting the same pavement, the maximum strain at the base of the asphalt is 296 microstrains, for a 23% reduction. The allowable traffic for fatigue computed as before is over 260,000 repetitions. The improved strength of better drained material will theoretically improve the pavement life by over a factor of 2.5. This is good reason to promote better drainage of pavements that are desired to last.
Two days after the first testing (during rainfall), the taxiway was retested. The weather had cleared and become windy. Pavement temperatures during both tests were the same. The comparative results show something of the mechanistic effects of moist conditions on a paved surfaced embankment.

The pavement is continuous in the test area. Pavement cracks have been patched. During the rain, the sideslope of the embankment has been covered with water. Infiltration points were not evident. Tests were conducted approximately 15' from the pavement edge.

Figure 3.20 shows the centerline deflections measured on the two dates. Initially the deflection is 12% higher during the rainfall (September 5, 1990). After approximately 45 repetitions the deflections measured on the two dates match. Notice that the deflections measured on September 7, 1990 stabilize much faster than those in the earlier test.

A comparison of the backcalculated moduli is shown on Figure 3.21. Initially the same embankment material tested during the rain has a modulus approximately 26% higher than they were 2 days earlier. After about 53 repetitions the two are convergent.
Figure 3.20  Airport Taxiway Deflection Comparison at Poor Drainage Location
Figure 3.21 Airport Taxiway Backcalculated Modulus Comparison

6000 lb. Force
The subgrade moduli are effectively the same on the two dates. It appears that some moisture had infiltrated the embankment causing weakening during the rainy period.

Samples taken on September 7, 1990 indicate that the moisture content at the top of the embankment is only 3.5%. This may be below optimum. The subgrade was found to contain 6.4% moisture. Some organic material was found in the embankment, which may be the actual cause of settlement here.

Variable drop height repeated load tests were also conducted at this location on the two dates. Figure 3.22 shows a comparison of the deflections measured. In almost every case, the deflections measured on September 5, 1990 are higher. In both cases the levels are decreasing indicating that the granular material is strain hardening.
Figure 3.22 Airport Taxiway Variable Drop Height Deflection Comparison
The embankment material on this taxiway has a substantial amount of sand-sized particles which could add to its propensity to generate pore water pressure under dynamic loading. However since it does not appear to be dense graded, as would be indicated by a straight line on the gradation chart, the permeability may be high enough to allow drainage. Comparing the gradations of this with the previous test sites, shows this to be the farthest from being dense graded of them all.

Figures 3.23 and 3.24 show the backcalculated moduli resulting from the deflection testing on the two test dates. The moduli tested after the rainfall (Figure 3.24 - 9/7/90) are nearly constant. Moduli determined during the wet period gradually increase to the dry period levels. Except in the subgrade where they are slightly lower.

In summary, it appears that rainfall affects the sublayers of even "impermeably" surfaced (paved) sections. There appears to be a relationship between moisture content and/or gradation with a materials mechanistic reaction to applied stress. Actually this is obvious. The definitions of limiting moisture content, gradation, and stress is the complicated part. This will have to be investigated in the laboratory.
Figure 3.23 Taxiway Backcalculated Moduli at Variable Drop Force-Raining
Figure 3.24 Taxiway Backcalculated Moduli at Variable Drop Force-Dry
3.5 **Highway Overpass**

During the summer and fall of 1990, an overpass was constructed on Minnesota Drive at Raspberry Road in Anchorage, Alaska. The southbound three lanes consisted of the following typical structural section, AKDOT&PF (1988):

1. 2 inches Asphalt Concrete, Type II - (minus 3/4")
2. 6" Crushed Aggregate Base Course, Grading D-1 - (minus 3/4")
3. 16" Select Material, Type A - (0-6% p200)
4. 20" Select Material, Type B - (0-10% p200)
5. +5' Select Material, Type C - any inorganic soil

The final pavement thickness is to be 3.5"-4", depending on location. For simplicity, Selected Material is often referred to as "Borrow".

Problems arose in the fall of 1990, when heavy rains hampered the construction efforts in the southbound lanes. Moisture contents became higher than optimum, making compaction difficult. According to construction records, the specified compaction for each layer was finally obtained and the first lift of asphalt concrete (2") was placed.
The pavement subsequently failed under construction traffic loading. Alligator cracking with moisture bleeding between the cracks was the failure mode. Efforts to patch the failed areas continued into the fall, when the project was stopped. Traffic was not allowed on the road all winter.

During the spring of 1991, the pavement was tested with the FWD, to determine whether it was strong enough to pave. Deflections ranging from 20-37 mils (averaging at 28 mils) were measured at a 9000 lb. drop force. These compare unfavorably with deflections measured in the northbound lane, which was constructed earlier in 1990 and trafficked all winter. There the deflectors ranged from 9-14 mils, averaging at 11 mils.

The northbound lanes have already been paved to full depth (3-1/2"). However, backcalculations of tests taken at both places on the same day showed the sublayer moduli over 50% lower on the southbound lane areas tested. Upon consideration it was determined that the road would not carry its design traffic - even with the planned overlay.
A comparison between the FWD measured deflection bowls at the worst ("critical") and best locations is shown on Figure 3.25. Notice that on the critical section tests, the deflections are not only higher at the center, but taper off to very low values on the last two sensors. This type of behavior indicates that the subgrade (here Borrow C) was stiffer than the upper layers.

The best sections show moderate deflections across the sensors. This type of behavior is less likely to be associated with the development of high tensile strains at the bottom of the asphalt concrete pavement which lead to fatigue cracking.

Computer depths to stiff layer from the backcalculation results show the critical section with stiff layers (thaw) just beneath the Borrow C layer. The best section indicated rigid layer at over 10’ depth.

On two other occasions in succeeding weeks the southbound lanes were again tested. Deflection readings on the average were decreasing slightly indicating that the structural section was improving.
Figure 3.25 Highway Overpass Deflection Bowl Comparison
A particular test site, on the paved shoulder, was FWD tested at two locations separated by 45 inches. The testing sequence at each location was using drop heights 1, 2, 2 and 3. The first place tested was 75" from the pavement edge. The second was 30" from the edge.

Figure 3.26 shows the deflection bowls of the 3rd drop. They are very similar to each other and the critical one shown in Figure 3.25.

The pavement mix used AC-5 viscosity graded asphalt. The pavement temperature at the time of testing was 43 deg. F. The pavement modulus was estimated using the Asphalt Institute equation, T.A.I. (1982).

The backcalculated moduli are shown in Table 3.1. The moduli show a poor section likely to fail by cracking and rutting. The weakness of the material leads to high compressibility and possible pore pressure build up when wet.
Figure 3.26  Highway Overpass Deflection Bowls at Test Sites
<table>
<thead>
<tr>
<th>Test</th>
<th>Drop Force (lb.)</th>
<th>AC Modulus (ksi)</th>
<th>Base Modulus (ksi)</th>
<th>Borrow A &amp; B Modulus (ksi)</th>
<th>Borrow C/Sub-grade Modulus (ksi)</th>
<th>Depth to Stiff Layer (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>#1</td>
<td>6000</td>
<td>1335</td>
<td>7</td>
<td>5</td>
<td>8</td>
<td>155</td>
</tr>
<tr>
<td>#1</td>
<td>9000</td>
<td>1335</td>
<td>10</td>
<td>6</td>
<td>8</td>
<td>None</td>
</tr>
<tr>
<td>#1</td>
<td>9000</td>
<td>1335</td>
<td>13</td>
<td>8</td>
<td>10</td>
<td>None</td>
</tr>
<tr>
<td>#1</td>
<td>15000</td>
<td>1335</td>
<td>13</td>
<td>8</td>
<td>4</td>
<td>60</td>
</tr>
<tr>
<td>#2</td>
<td>6000</td>
<td>1335</td>
<td>7</td>
<td>5</td>
<td>8</td>
<td>None</td>
</tr>
<tr>
<td>#2</td>
<td>9000</td>
<td>1335</td>
<td>8</td>
<td>5</td>
<td>3</td>
<td>60</td>
</tr>
<tr>
<td>#2</td>
<td>9000</td>
<td>1335</td>
<td>11</td>
<td>7</td>
<td>4</td>
<td>None</td>
</tr>
<tr>
<td>#2</td>
<td>15000</td>
<td>1335</td>
<td>14</td>
<td>9</td>
<td>8</td>
<td>63</td>
</tr>
</tbody>
</table>

Following the FWD tests the pavement was cut and removed in an 8’ x 10’ square. A rough density check, using a nuclear gauge, showed the base course density near 140 pcf. Gradation and moisture content samples were obtained at this time. The appearance of the materials was of fine-sandy gradation. Sieve analysis results are on Figure 3.27.

All of the computed depths to a stiff layer are greater than the distance to the outer sensor used here, which was 47.2 inches. Four out of the eight tests computed no stiff layer depth. At a depth of about 72 inches, the temperature was 35° F. There was no stiff layer at this depth.
Figure 3.27  Highway Overpass Embankment Gradations
The Table 3.2 shows moisture contents ($M_c$) that were determined. The degree of saturation ($S_s$), dry densities ($\gamma_d$), specific gravities of the blend ($G_b$) and optimum moisture content shown are from construction records.

<table>
<thead>
<tr>
<th>LAYER</th>
<th>$M_c$(%)</th>
<th>$S_s$</th>
<th>$\gamma_d$(pcf)</th>
<th>$G_b$</th>
<th>OPTIMUM $M_c$(%)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Base</td>
<td>4.6</td>
<td>.612</td>
<td>141.2</td>
<td>2.725</td>
<td>5.2</td>
</tr>
<tr>
<td>Borrow A</td>
<td>5.6</td>
<td>.513</td>
<td>132.0</td>
<td>2.748</td>
<td>6.1</td>
</tr>
<tr>
<td>Borrow B</td>
<td>8.1</td>
<td>.664</td>
<td>129.0</td>
<td>2.763</td>
<td>7.2</td>
</tr>
<tr>
<td>Borrow C</td>
<td>7.8</td>
<td>.883</td>
<td>136.2</td>
<td>2.703</td>
<td>7.2</td>
</tr>
</tbody>
</table>

Here the Borrow B and C have been found to be above optimum moisture content. The backcalculation results found essentially no stiff layer within the structural section. When a stiff layer was used, it was deeper than the outer FWD sensor (47.2 inches). The temperature Borrow C at approximately 6 foot depth was measured at 35° F, just after digging it out with a backhoe. Therefore the high moisture content lower layers were likely contributing greatly to the high deflections.

The base course gradation was found to be slightly out of specification on the #200 sieve at 7%. The other sieves show the base is on the fine side of the specifications, but acceptable.
It was decided to perform laboratory tests on this base and another comparative "typical" sample from a local gravel company. The laboratory tests were to be used to check the FWD backcalculated moduli - which were low. The highway here had been designed assuming base moduli in the 70 ksi range.

According to the project plans, the design traffic for this section is 3,806,000 EALS with a mid-design AADT of 28,550. According to elastic theory analysis using the moduli backcalculated from the tests on the entire section, the highway was estimated to not last over 2 of the 20 year design life.

Laboratory resilient moduli of the base over a range of confining and deviator stresses were determined by triaxial testing of this material. The material tested had a density of 138 pcf at 4.5% moisture content. Fifteen test sequences of 100 repetitions were run. Confining stresses ranged from 3 to 20 psi while deviator stresses were varied from 3 to 40 psi.

Mean resilient moduli (M,) were found for each testing sequence and are plotted as a function of the bulk stress (θ). A curve was then fitted through the data to obtain the non-linear modulus. Figure 3.28 shows the results. The computed resilient modulus function is given in Equation 3.1.
Figure 3.28 Highway Overpass Crushed Aggregate Base Laboratory Resilient Moduli
\[ Mr = 1348.2 e^{0.7291} \tag{3.1} \]

The regression coefficient (R^2) came out to be 0.990 for the data. This indicated good agreement with the data.

At least 2 other tests were run similar to the above varying gradation and density. When performing the analysis and curve fitting to find the functional expressions for moduli, it was found that improved regression coefficients were obtained in every case by using linear regression. For the test above the linear regression equation is:

\[ Mr = 5315.97 + 1187.43 \sigma_e + 250.04 \sigma_d \tag{3.2} \]

Equation 3.10 has an R^2 value of 0.992. The improvement here is rather insignificant. However, it is better than the traditional means of modelling. The linear function also shows the high dependence or confinement of the moduli.
The low moduli backcalculated for this base indicate low confining stress. The relatively low degree of saturation of the material likely precludes pore pressure build up. Since less than 50% of the base course material is greater than the No. 4 sieve, there is less strength derived from inter-granular friction between larger stones than might be available by using coarser gradations.

The moduli computed from the laboratory data (Eq. 3.1), set equal backcalculated base course moduli, indicate similarity at a confining stress of around 5 psi and deviator stress at 10 psi, that is, bulk stress of around 25 psi. This type of loading will be investigated in the following chapter.

Table 3.3 shows mean backcalculation results for testing performed on the center traffic lane of the overpass on four dates. Based on the increasing unbound layer moduli and increasing thaw depth, in the first three testing cycles, it was decided to complete the paving of the project in mid-May. The results for August 8, 1991 are for the completed pavement having a nominal thickness of 4 inches.

The unbound layer moduli effectively doubled between spring and summer. The completed pavement is now performing well.
TABLE 3.3 Mean Backcalculation Results for Overpass

<table>
<thead>
<tr>
<th>TESTING DATE</th>
<th>AC MOD. (ksi)</th>
<th>BASE MOD. (ksi)</th>
<th>BOR. A&amp;B MOD. (ksi)</th>
<th>BOR. C/S.G. MOD. (ksi)</th>
<th>DEPTH TO STIFF LAYER (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td>4/21/91</td>
<td>1126</td>
<td>23</td>
<td>15</td>
<td>8</td>
<td>49</td>
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<tr>
<td>4/23/91</td>
<td>750</td>
<td>24</td>
<td>15</td>
<td>8</td>
<td>41</td>
</tr>
<tr>
<td>5/2/91</td>
<td>1100</td>
<td>29</td>
<td>19</td>
<td>13</td>
<td>124</td>
</tr>
<tr>
<td>8/8/91</td>
<td>1323</td>
<td>40</td>
<td>25</td>
<td>15</td>
<td>146</td>
</tr>
</tbody>
</table>

The unbound layer moduli effectively doubled between spring and summer.

The completed pavement is now performing well.
3.6 Summary

Falling weight deflectometers do well in simulating the deflection, vertical stress, and vertical strain of a design wheel load. However, the test, involving an impact load on a plate placed on the surface does not simulate the shear stress reversal of a moving wheel load. Dynamic pore pressure increases in wet soils are the result of cyclic shear stresses such as imparted by moving wheel loads. Therefore, falling weight deflectometers are not ideal for field testing for pore pressure generation.

Increases in moisture content of unbound layers, between falling weight deflectometer tests, result in: 1) increased surface deflections; 2) decreased backcalculated moduli and 3) damping of the measured surface stresses at a given drop height. The increased deflections and damping are a result of the loss of stiffness and weakening of the system. These reactions may result in overstressing the unbound layers and permanent deformations which shorten the life of a pavement.
Gradations away from maximum density, as indicated by a straight line on a 0.45 power gradation chart, appear to perform better under high moisture conditions. This may be because less dense graded materials are more permeable. Aggregate gradations containing higher percentages of coarse material also appear to be superior in maintaining strength in wet conditions.
CHAPTER 4

EXPERIMENTAL WORK

4.1 Introduction

This chapter presents results of laboratory experiments performed on cohesionless soils. A series of "static" triaxial tests were performed at a constant compressive strain rate. Cyclic repeated load testing of typical base courses followed. Pavement drainage systems are described in section 4.7 as an introduction to the testing of open graded permeable bases. Testing equipment used, sample preparation procedures and, methods for computation of mechanistic properties are described in Appendix D.

Static tests were all performed on saturated samples of Ottawa Sand. The effective confining stress was varied. Both drained and undrained tests were conducted on similar samples. The effects of drainage and confinement on the material properties was determined.
Dynamic tests were run using Ottawa Sand as an initial measure of the repeatability of the test set up. Then, three crushed aggregate base course gradations, ranging from very dense to open-graded, were tested. Testing was conducted when the bases were damp, near optimum moisture content, and on the same samples after saturation.

4.2 Testing Procedure - Static Tests

The series of tests presented here involve samples prepared identically, saturated, and tested under the same loading conditions (1 mm/min strain rate). The variables used in the testing are confining stress and drainage. Tests were performed at the 4 effective confining stress levels (5, 10, 15, 25 psi) in both the drained and undrained condition. Targeted relative densities for the test samples is 40 to 50 percent.

The engineering parameters that were monitored includes: variation in shear stress; pore-water pressure (undrained tests); effective stress ratio; volume change (drained tests); with axial strain. If no other apparent peak shear stress is found in the tests, failure is assumed at 20% axial strain.
4.3 Static Testing Results

This section will present the results of the testing and discuss their significance to pavement loading conditions. For the loading conditions applied, the reaction of the samples are greatly affected by the drainage condition. These undrained samples had higher peak strength than the drained samples.

With the apparatus used to determine the coefficient of permeability for the Ottawa Sand, the average permeability was determined to be 0.006 cm/sec (17 ft./day), ranging between 0.001 cm/sec and 0.009 cm/sec (3 - 26 ft./day). These numbers are typical for a fine sand as this (Cedergren, 1989). Qualitatively, this material is of average drainability, with permeability similar to a sand and gravel mixture.

The shear strength of the sand in the drained tests is generally less than half of the undrained tests (compare Figures 4.1 & 4.2). The drained tests show the marked influence of confining stress on the strength, see Figure 4.2). The ultimate shear strength of this sand is almost linearly proportional to the confining stress. For example, doubling the confining stress from 5 to 10 psi, raised the peak shear stress from 11 to 22 psi.
Figure 4.1  Ottawa Sand Drained Shear Strength
Figure 4.2 Ottawa Sand Undrained Shear Strength
The undrained tests do not show as marked differences between shear strength as effected by the initial confining stress. Unlike in the drained tests where the confining stresses remain constant, here the effective confining stresses actually increase as a function of the vertical strain (see Figure 4.3).

Initially, the effective confining stresses decrease as a result of compression of the soil gains causing an increase in pore pressure. As the strain increases the particles begin sliding over each other and through the process called "dilation" the void space increases. The tendency volume of voids to increase would, in the undrained case, result in a corresponding decrease in pore-water pressure, which in turn increases the effective stress on the sample.

As seen on the drained test series, increasing confining stress produces higher shear stress resistance. Notice how closely the shapes of the effective confining stress curves approximate the shear stress curves for the undrained cases at the higher strain levels. The test starting at 25 psi effective confining stress did not decrease in pore pressure as much as the others (see Figure 4.4). Therefore, the effective confining stress did not increase as much as the 10 and 15 psi tests and the shear strength of the 25 psi sample is less.
Figure 4.3  Ottawa Sand Undrained Effective Confining Stress
Figure 4.4  Ottawa Sand Undrained Pore Pressure
Evidently, the higher initial confinement reduces the dilation within the sample which causes a higher pore pressure increase at lower strain and less decrease in pore pressure at higher strain. This is consistent with Casagrande (1936 & 1938), as explained in Chapter 2. A most important thing to note about the pore pressure change is that increases are always at strain levels of less than 2%.

The volumetric strain within the samples, as measured in the drained tests, initially is negative, indicating compression of the sand particles. This is the reason for the pore pressure increases. Figure 4.5 shows the volumetric strain measurements. Generally, volumetric strains are higher (greater dilation) for lower confining stresses. For the drained case this means that water initially flows out of the soil then at higher strain levels the water flows in. The increase of water in the void space of the soil decreases intergranular friction and the samples reached peak shear stresses at volumetric strain levels around 2%.
Figure 4.5   Ottawa Sand Drained Volumetric Strain
Thus, the static shear strength of the drained sands tested here were greatly influenced by the strain level and confining stress. Increases in effective confining stress tend to increase the shear strength. Higher strain levels result in increasing the volume. These basic findings are consistent with the findings of others, e.g., Kitamura & Haruyama (1988).

In the undrained condition, the effective confining stress of this sand becomes a variable. This is as was shown by Seed & Lee (1966). Higher initial confining stresses of a soil actually tend to weaken the sand in the undrained static loading condition. This is due to the higher initial stress resisting deformation and dilation of the samples, which leads to smaller increases in effective confining stress, Casagrande (1936).

Soils of low permeability will not drain easily when saturated making them behave as the undrained samples here. Once drainage is provided the soil may maintain constant or predictable confining stress and the loss of internal friction between particles may be avoided.

Under the stimulus of repeated loading the situation is quite different. Without enough time to dissipate the pore pressure, it may continue to increase. This process and reaction is investigated in the following section.
4.4 Dynamic Testing Procedure

A cell capable of testing 4 inch diameter by eight inch height samples was fitted and rigged for dynamic testing. A new 1000 pound capacity internal load cell was purchased and connected to the load piston. The triaxial testing apparatus was modified, enabling connection of the load frame to the loading piston, to allow testing involving stress reversal.

The same MTS load frame, controller, function generator, water system, and data logging computer were used in this testing as in the static tests. The new cell was fitted with the same cell pressure and pore pressure transducers as used previously.

Two wave forms, each of 1 Hz duration were used in this testing. The first was sinusoidal continuous and the second was a haversine (w/ 0.1 second pulse and a 0.9 second rest). The sinusoidal wave form was used only on the first few tests at 1% cyclic strain rates. Later the 0.1 second pulse was used exclusively. Data sampling rates were at 120 to 150 Hz, which was the limit if the data logging equipment. One problem with the 0.1 second pulse tests was that at the maximum sampling rate only 12 to 15 data points could be obtained during the strain cycle.
With this rather coarse sampling rate it was not always possible to record peak values. This is likely the explanation for why some of the test data does not always follow smooth curves. Figure 4.6 shows typical stress-strain data from which moduli are computed.

Permeability coefficients were measured following saturation of test specimens. Compressibility was determined at the end of the test by opening the top drain valve and measuring the volume of water released in changing the confining stress back to the initial value.
4.5 Saturated Ottawa Sand Tests

Two samples of Ottawa Sand were prepared similarly and tested likewise. The purpose of this testing is twofold in that it is intended to determine the dynamic characteristics of the sand and to ensure that the experiments are repeatable. A sinusoidal strain-controlled waveform of 1% strain amplitude was applied to the samples. The samples were saturated and tested undrained.

Both samples reached initial liquefaction within 6 strain cycles as indicated by a measured pore pressure ratio of unity. Figure 4.7 shows the change in this ratio as a function of cycle number. The test results are nearly identical, demonstrating the repeatability of the test procedure.

Test #1 sample had a density of 90 pcf and measured permeability of $4 \times 10^{-3}$ cm/sec. The second test sample was 91 pcf in density, with permeability of $2 \times 10^{-3}$ cm/sec, for an average of $2 \times 10^{-3}$ cm.sec.
Figure 4.7  Ottawa Sand Pore Pressure Generation Under Cyclic Strain
The cyclic strain modulus (E) shear modulus (G) diminishes similarly for both tests. At the points of liquefaction the material silt has some strength left. This, however, goes to near zero within 12 cycles. See Figures 4.8 and 4.9. The curves are truncated at the point beyond which the samples gave no compressive reaction. These results are similar to those obtained by Seed & Lee (1966).

Resilient moduli determined on a cyclic basis for the two tests are shown in Figure 4.10. It has been found that resilient moduli determined above a pore pressure ratio of around 0.8 are not likely indicative of material behavior. This is due to the fact that recoverable strains become very small. Recoverable strain, being in the denominator of the equation, tend to show high values. In the Ottawa Sand test series it is seen that the resilient moduli do not degenerate significantly after the 4th cycle even though liquefaction has occurred. The deviator stress diminishes to near zero following the fourth cycle, but the recoverable strain decreases more rapidly.
Figure 4.8  Ottawa Sand Moduli - Cyclic Test #1
Figure 4.9  Ottawa Sand Moduli - Cyclic Test #2
Figure 4.10 Ottawa Sand Resilient Moduli
A similar phenomena is observed with the damping ratios. The test results are shown in Figure 4.11. The damping ratios appear rather constant up to the point of initial liquefaction (Cycle 5 for test #1 and 6 for test #2), but increase after this point. Here the stress energy (loop) is decreasing at a slower rate than the strain energy (triangle). Consequently, the damping energy is shown to increase. It is not felt that damping ratio computations beyond the point of liquefaction are of use, since the stress energy is largely taking place in the negative (tensile) shear stress zone.

The compressibility determined for the liquefied Ottawa Sand is $1.67 \times 10^5$ sf/lb.
Figure 4.11 Ottawa Sand Damping Ratios
4.6 Dense Graded Base Testing

Two representative tests on dense graded base course are described herein. The first is at a 1% cyclic strain level, as with the Ottawa Sand, in saturated condition. The second is using a 0.25% cyclic strain level tested first at approximately 4% moisture content then saturated.

The 1% strain level is at a deflection of around 0.08 inches. This is similar to the maximum deflection measurable by the Falling Weight Deflectometer. The 0.25% strain level is at a deflection of around 0.020 inches which may be considered typical for Falling Weight Deflectometer tests on fair materials. The 0.25% strain level may also be considered typical to occur in a relatively weak base (15 ksi modulus) under truck traffic loading on a pavement during the spring as determined using layered elastic theory (ELSYM5 program).

The aggregate used is a crushed alluvial gravel. Current Alaska Crushed Aggregate Base Course, Grading D-1, standard specifications and the gradation of material tested in this study are shown in Table 4.1.
### TABLE 4.1 Dense GradeD Base Specifications

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Alaska Specifications (% Passing)</th>
<th>Test Gradation (% Passing)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1&quot;</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>3/4&quot;</td>
<td>70 - 100</td>
<td>100</td>
</tr>
<tr>
<td>3/8&quot;</td>
<td>50 - 80</td>
<td>65</td>
</tr>
<tr>
<td>No. 4</td>
<td>35 - 65</td>
<td>50</td>
</tr>
<tr>
<td>No. 8</td>
<td>20 - 50</td>
<td>35</td>
</tr>
<tr>
<td>No. 40</td>
<td>8 - 30</td>
<td>20</td>
</tr>
<tr>
<td>No. 200</td>
<td>0 - 6</td>
<td>10</td>
</tr>
</tbody>
</table>

The test gradation is out of specification on the No. 200 sieve, but Alaska specifications prior to 1981 allowed up to 10% passing this sieve. So, this gradation is used a test what may occur on older roads which have had the years required to become soaked.

Measured permeabilities of the dense graded base course were both near $5 \times 10^{-4} \text{ cm/sec}$. That is the permeability of this material is about 1/6th of Ottawa Sand.

Figure 4.12 shows the pore pressure ratio as a function of cycle number for the 1% cyclic strain level test. Here the sample liquefied in 5 cycles, as did the Ottawa Sand.
Figure 4.12 Dense Graded Base Pore Pressure Generation at 1% Cyclic Strain
The strength of this material, as tested, was very poor. The cyclic strain moduli and shear moduli are shown in Figure 4.13. The maximum stress measured on the sample became less than zero on the 9th cycle. This means the material has no remaining bearing capacity.

Resilient moduli remain between 3-5 ksi for the pre-liquefaction cycles (see Figure 4.14). However, the resilient moduli are increasing after the 3rd cycle (.92 of liquefaction). On the 8th cycle the resilient modulus is computed to be over 200 ksi. Which is definitely not indicative of its stiffness as shown in Figure 4.13.

The damping ratios computed and shown in Figure 4.15 diverge after the 4th cycle as do the resilient moduli. Strain energies here are low and becoming relatively smaller beyond pore pressure ratios above 0.92. The damping ratio exceed unity at the point of liquefaction.

A second sample was tested at a 0.25% cyclic strain level. At a moisture content (MC) of approximately 4% the moduli were almost tenfold higher than the previous test in saturated condition and higher strain level. Figure 4.16 shows the various moduli for 100 repetitions.
Figure 4.13 Dense Graded Base Saturated Modulus Variation at 1% Cyclic Strain.
Figure 4.14  Dense Graded Base Saturated Resilient Modulus Variation at 1% Cyclic Strain
Figure 4.15 Dense Graded Base Saturated Damping Ratio Variation at 1% Cyclic Strain
Figure 4.16 Dense Graded Base Low MC Modulus Variation at 0.25% Cyclic Strain
The resilient moduli are larger since they are based on the smaller recoverable strains and peak stress, rather than stress at maximum strain, as the E and G moduli are. The slight increase in resilient modulus with repetitions is indicative of a small loss in recoverable strain.

Figure 4.17 shows damping ratios computed at low moisture content (4%). These values are much less than when the material is saturated since the strain energy is less when the material is soaked. After the initial few cycles the damping ratios show little variation and remain at less than 0.2.

Pore pressure ratios run near zero to slightly negative in the low moisture content tests. Following saturation of the material, the sample is found to liquefy within 35 repetitions at the 0.25% cyclic strain level. Two tests were performed samples obtaining these same results. The variation in pore pressure ratios in one test are shown in Figure 4.18.

By the 22nd cycle the stress at maximum strain becomes less than zero. By the 25th cycle the peak stress reaction on the sample is less than zero. These cycles occur at 0.94 and 0.96 pore pressure ratios.
Figure 4.17 Dense Graded Base Low MC Damping Ratio Variation at 0.25%

Cyclic Strain
Figure 4.18 Dense Graded Base Saturated Pore Pressure Variation at 0.25%

Cyclic Strain
Cyclic strain and shear moduli for the saturated sample using 0.25% cyclic strain are shown on Figure 4.19. These results are approximately 4 times higher than at the 1% strain level as shown in Figure 4.13. Here the strain level is 1/4 of the 1% strain and since the strain is in the denominator of the modulus computation, the result could be expected for linear materials.

A comparison of the computed resilient moduli for both the saturated and low moisture content (Low MC) is shown in Figure 4.20. In the low moisture content condition the resilient modulus is rather constant around the 16 ksi level. When saturated the resilient modulus rises, untypical of the material behavior, beyond about the 15th cycle. The 15th cycle is near a pore pressure ratio of 0.9. Actually the saturated resilient moduli start to increase with the 6th cycle which is at a pore pressure ratio of 0.65. It could be argued that resilient moduli tests for saturated materials as this are not indicative of material behavior at pore pressure ratios above 0.6.
Figure 4.19  Dense Graded Base Saturated Modulus Variation at 0.25% Cyclic Strain
Figure 4.20 Dense Graded Base Saturated Resilient Modulus Variation at 0.25% Cyclic Strain
Damping ratios for the saturated dense base are at least twice that of the low moisture content tests. In fact, the strain energy becomes negligible beyond the 18th cycle which is at a pore pressure ratio just above 0.9. Figure 4.21 shows the damping ratios computed for the saturated sample. Notice that the values are increasing over almost the entire range of the testing. Unity is exceeded on the 11th cycle when the pore pressure ratio is slightly above 0.8.

Compressibility for this material, determined at the end of the tests averaged at $3.6 \times 10^6$ sf/lb. That is about twice that of the Ottawa Sand.
Figure 4.21 Dense Graded Base Saturated Damping Ratio Variation at 0.25%

Cyclic Strain
4.7 **Pavement Drainage Systems Using Permeable Base**

Open-graded or permeable bases are used in pavement drainage systems. These systems are designed such that any excess moisture that gets into the layer beneath the pavement is carried away to the sides. Figure 4.22 shows an edge detail of a typical pavement drainage system (see also Mathis (1989)). In this Figure a filter and filter fabric are placed beneath the permeable base.

The filter may be constructed of granular material, designed considering piping ratios, to allow moisture movement but avoid contamination of the base. A drainage fabric could also be used here to serve the same purpose, plus facilitate drainage. A filter fabric is placed around the permeable base holding the perforated collector pipe. Its purpose is to avoid contamination and loss of permeability in this area.

There are several good publications written, e.g., Cedergren (1989), which describe design procedures for these systems. Flow nets and Darcy’s Laws are utilized in design. Outfall pipes are installed at spacings to avoid over-filling of the collector pipes.
The gradations of tested in this research are shown in Figure 4.23. It is generally accepted that dense gradations plot as straight lines on the 0.45 power exponential gradation charts. The dense graded base approximates this. The Minnesota Specification and Wisconsin Specification permeable base gradations are coarser as indicated by being lower on the chart. Permeable materials would not be expected to plot as straight lines on the chart which would indicate low void ratios. Notice that the two permeable bases are below the dense graded base and do not approach a straight line on the chart. The Wisconsin Specification material is the most open graded.
0.45 POWER GRADATION CHART

Figure 4.23 Base Course Gradations
4.8 **Minnesota Specification Permeable Base**

The gradation specification for permeable base, developed in Minnesota (Mathis, 1989) and the gradation used in this testing is shown in Table 4.2. Crushed aggregate of this gradation may be considered of medium permeability. It is not a dense graded material, which would be indicated by a straight line when plotted on a 0.45 power gradation chart. Yet it is not extremely open graded, since it may contain up to 40% passing the No. 4 (4.75 mm) sieve.

<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>Specification (% Passing)</th>
<th>Test (% Passing)</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Inch</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>3/4 Inch</td>
<td>65 - 100</td>
<td>100</td>
</tr>
<tr>
<td>1/2 Inch</td>
<td>45 - 80</td>
<td>80</td>
</tr>
<tr>
<td>3/8 Inch</td>
<td>35 - 70</td>
<td>70</td>
</tr>
<tr>
<td>No. 4</td>
<td>20 - 45</td>
<td>40</td>
</tr>
<tr>
<td>No. 8</td>
<td>13 - 30</td>
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<td>No. 40</td>
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<td>10</td>
</tr>
<tr>
<td>No. 100</td>
<td>1 - 5</td>
<td>4</td>
</tr>
<tr>
<td>No. 200</td>
<td>0 - 3</td>
<td>0</td>
</tr>
</tbody>
</table>

The published permeability, Mathis (1989), for this gradation is 200 feet per day. Permeabilities measured by this test system of two samples are $1.8 \times 10^3$ cm/sec. and $3.5 \times 10^3$ cm/sec. That is 5.1 Ft/day and 9.9 Ft/day respectively.
Obviously, the permeability of a material is dependent on many factors, such as, gradation, density, type of material, particle shape and testing method. However, for comparison, the permeability of this material is similar to Ottawa Sand, being 3.5 to 7 times as permeable as the dense graded bases tested here.

Two samples were tested at the 0.25% cyclic strain level, first at a low moisture content of approximately 4%, then when saturated. Both tests series are presented here.

In the low moisture content condition, the material reacted similar to the dense graded base shown in Section 4.6. The test of the material with the lower permeability (#1) is slightly lower than the dense graded base. The second test, with permeability higher, was slightly stronger. Figures 4.24 and 4.25 show the variation in moduli with cycle number for these two test samples.

Saturated testing shows the material with lower permeability (#1) reaching the point of initial liquefaction prior to the other. However, in both cases, there specimens lasted much longer than the dense graded base. The test #1 material liquefied in 240 cycles and the test #2 material reached this point in 400 cycles. Figure 4.26 shows how the pore pressures built up in 500 cycles.
Figure 4.24 Minnesota Specification Permeable Base Low MC Modulus Variation - Test #1
Figure 4.25 Minnesota Specification Permeable Base Low MC Modulus Variation - Test #2
Figure 4.26  Minnesota Specification Permeable Base Saturated Pore Pressure Variation
The saturated strain moduli and shear moduli degenerations of the two tests are shown in Figures 4.27 and 4.28. The strengths had diminished by over one half in less than 25 repetitions or a pore pressure ratio of approximately 0.4. However, this material still gave a positive stress reaction, even beyond the point of liquefaction. It is easily concluded that this material is superior to dense graded base under these conditions.

Resilient moduli comparisons for the first 100 cycles in low moisture content and saturated condition for both tests are shown in Figures 4.29 and 4.30. In these figures the post-saturation resilient moduli go to 1/2 or less of the low moisture content moduli.

A comparison of the resilient moduli determined from the two saturated tests is shown on Figure 4.31. This data indicates some instability in the resilient moduli for test #1 beyond the 80th cycle or a pore pressure ratio of 0.8. Test #2 resilient modulus data commences instability around the 120th cycle. This is also at a pore pressure ratio of 0.8.

Damping ratio variation with cycle number for the low moisture content tests are shown in Figure 4.32. The damping characteristics of this material are similar to the dense graded base as shown in Figure 4.17.
Figure 4.27 Minnesota Specification Permeable Base Saturated Modulus Variation - Test #1
Figure 4.28 Minnesota Specification Permeable Base Saturated Modulus Variation - Test #2
Figure 4.29 Minnesota Specification Permeable Base Resilient Modulus Variation - Test #1
Figure 4.30 Minnesota Specification Permeable Base Resilient Modulus Variation - Test #2
Figure 4.31 Minnesota Specification Permeable Base Saturated Resilient Modulus Variation
Figure 4.32  Minnesota Specification Permeable Base Low MC Damping Ratio Variation
A similar comparison of the saturated sample tests are in Figure 4.33. This data shows damping ratios to be increasing with pore pressure built up. The peaks have been determined to be points where the maximum stress and strain become out of phase. The damping ratio strain energy is determined from the stress at maximum strain. At these peaks the peak stress measured as the opposite side of the strain energy triangle is less than the maximum stress, and the triangle is smaller than when the maximum stress is in phase with the maximum strain. With the strain energy in the denominator being smaller at certain points, a higher damping ratio is computed.

Notice that here the sample that liquefied first (test #2) has higher damping ratios. The point of initial liquefaction occurs in each just prior to the damping ratios exceeding units.

The mean compressibility measured for these specimens is $1.75 \times 10^{-5}$ sf/lb. This is only slightly higher than the Ottawa Sand.
Figure 4.33  Minnesota Specification Permeable Base Saturated Damping Ratio Variation
4.9 Wisconsin Specification Permeable Base

The Wisconsin specification permeable base is a very open-graded, highly permeable material. Its gradation band is far from a straight line on the 0.45 power exponential gradation chart. It is believed by some that the strength of a soil is maximum at the maximum density indicated on the 0.45 power gradation chart. If this is true, the Wisconsin specification permeable base should be much weaker than the denser gradations tested previously.

This gradation is designed specifically to avoid erosional pumping. Very little fine material (passing the No. 4 sieve) is available for erosion. The specification which may be used for a base of this type (Mathis, 1989) and the gradation used to test is shown in Table 4.3:
<table>
<thead>
<tr>
<th>Sieve Size</th>
<th>% Passing Specification</th>
<th>% Passing Test</th>
</tr>
</thead>
<tbody>
<tr>
<td>1 Inch</td>
<td>100</td>
<td>100</td>
</tr>
<tr>
<td>3/4 Inch</td>
<td>90 - 100</td>
<td>100</td>
</tr>
<tr>
<td>1/2 Inch</td>
<td>55 - 70</td>
<td>67</td>
</tr>
<tr>
<td>3/8 Inch</td>
<td>20 - 55</td>
<td>50</td>
</tr>
<tr>
<td>No. 4</td>
<td>0 - 10</td>
<td>8</td>
</tr>
<tr>
<td>No. 8</td>
<td>0 - 5</td>
<td>5</td>
</tr>
<tr>
<td>No. 40</td>
<td>-</td>
<td>0</td>
</tr>
</tbody>
</table>

The published permeability for this base is 18,000 ft/day. The permeabilities measured for the two samples tested in this study are $6.7 \times 10^{-3}$ cm/sec. and $4.9 \times 10^{-3}$ cm/sec. These convert to be 19 ft/day and 13.9 ft/day for test numbers 1 and 2 respectfully. Here again the measured values are much lower than as published due to the difference in gradation, density, etc. Relative comparisons to previous tests show this material to be on the average:

1.9 times more permeable than Ottawa Sand.
11.6 times more permeable than the dense graded base.
2.2 times more permeable than the Minnesota specification base.
The two samples were tested in the low moisture content (4% Mc) condition and saturated at the 0.25% strain level. One of the saturated samples was retested, at a 1% cyclic strain level, since only negligible damage had occurred in the previous 0.25% cyclic strain level.

A series of charts follow (Figures 4.34 - 4.37) which show variation in moduli and damping for the materials in the low moisture content condition of the two tests. Here the material with slightly greater permeability is slightly weaker than the other. However, the material strength is greater than the other bases tested in terms of all moduli. Damping is very low for this material likely due to the increased rock to rock contact and less buffering by fines.

Upon saturation and retesting, the pore pressure ratios increased very slowly at the 0.25% strain level (see Figure 4.38). The curve termed "Test #3" in this Figure is sample #2 retested at 1% cyclic strain.

At the lower strain level both samples reached a maximum pore pressure ratio of only 0.42 in 600 repetitions, which was the test duration. Extrapolating the average slope of the last 50 cycles for these tests indicate that they would not pass a pore pressure ratio of 1.0 until at least 1,500 cycles.
Figure 4.34 Wisconsin Specification Permeable Base Low MC Modulus Variation - Test #1
Figure 4.35 Wisconsin Specification Permeable Base Low MC Modulus Variation - Test #2
Figure 4.36 Wisconsin Specification Permeable Base Low MC Resilient Modulus Variation
Figure 4.37  Wisconsin Specification Permeable Base Low MC Damping Ratio Variation
Figure 4.38 Wisconsin Specification Permeable Base Saturated Pore Pressure Variation
The 1% cyclic strain test material failed in 280 cycles. A pore pressure ratio of only 0.93 had been reached, but the material had plastic strains equal or greater than the cyclic strains at this point. This still compares very favorably with the dense graded material which only sustained 8 cycles at this strain level before failing.

Variations in cyclic strain moduli, total strain moduli and shear moduli for the three tests (saturated) are shown in Figures 4.39 - 4.41. The moduli are found to diminish approximately in half after about 150 repetitions in tests #1 and #2 (0.25 cyclic strain). After that the moduli decline at a much slower rate.

Test #3 moduli diminish rapidly and the strength is much lower at this high strain level. By the 50th cycle (pore pressure ratio about 0.70) the stiffness has diminished to approximately 1/4th of the initial value. Beyond about 280 cycles no positive stress can be measured at the peak cyclic strain, so the moduli are meaningless.

Variations in resilient moduli for samples #1 and #2 for the first 100 cycles at a 0.25% strain level both at low moisture content and saturated are compared in the next two charts (Figures 4.42 and 4.43). Saturation does not greatly decrease the resilient moduli, in this range, as it does with the other gradations.
Figure 4.39 Wisconsin Specification Permeable Base Saturated Modulus Variation - Test #1
Figure 4.40 Wisconsin Specification Permeable Base Saturated Modulus Variation - Test #2
Figure 4.41 Wisconsin Specification Permeable Base Saturated Modulus Variation - Test #3
Figure 4.42 Wisconsin Specification Permeable Base Resilient Modulus Variation - Test #1
Figure 4.43 Wisconsin Specification Permeable Base Resilient Modulus Variation - Test #2
Resilient moduli over the entire range of cycles (up to 600) for all three saturated tests are shown in Figure 4.44. These values are fairly well behaved compared to the other gradations. Even test #3 did not diverge near the point of liquefaction. This means the recoverable strain did not generally decrease faster than the deviator.

Damping ratios computed as a function of cycle number for the three saturated tests are plotted for comparison on Figure 4.45. The ratios are seen to increase with the increasing repetition (and increasing pore pressure ratios), but are very low compared to previous tests. Even at the 1% cyclic strain level, the damping ratios do not exceed 1.0 until just prior to failure. This is indicative that the stress energy remains large throughout these tests. Physically, the rock to rock contact inherent in this coarse gradation gives the material low compressibility and little buffering of the cyclic loads.

The compressibility measured after the 0.25% cyclic strain test on this material computes to $1.53 \times 10^{-5}$ sf/lb. That measured following the 1% cyclic strain test is $3.1 \times 10^{-5}$ sf/lb. Using 1% cyclic strain on the specimen caused the volumetric strain to increase substantially. This result indicates that the compressibility is dependent on the vertical strain.
Figure 4.44 Wisconsin Specification Permeable Base Comparison of Saturated Resilient Modulus Variations
Figure 4.45  Wisconsin Specification Permeable Base Comparison of Saturated Damping Ratio Variations
4.10 Summary

The widely differing response between saturated, static (constant strain rate) and, dynamic (cyclic) testing is shown using Ottawa sand. The static loading tests on these samples show the sand to maintain shear resistance even to 20% vertical strain. In dynamic tests, using only 1% cyclic strain, the sand reached the point of initial liquefaction in less than 10 cycles. Complete loss of shear strength soon followed. A dynamic test on dense graded base course sample showed response similar to Ottawa sand.

Cyclic tests performed in this study, on saturated specimens, found the open graded permeable bases to be superior to dense graded base in terms of pore pressure generation, stiffness (cyclic strain and shear modulus), and damping. The stiffness of unsaturated open graded materials were generally equivalent or better than dense graded bases.

The resilient modulus, being a function of recoverable strain on the sample, was found not to be a good indicator of material strength when pore pressure ratios exceed around 0.6. At higher pore pressure ratios the recoverable strain approaches zero, making the computed resilient modulus appear large. In actuality, the specimen may have lost all compressive and shear strength.
Damping ratios determined for these tests show that damping increases with saturation and also with pore pressure buildup. As the pore pressure increases, under cyclic loading, the strain energy decreases. The computation results for damping ratios at higher pore pressure ratios become unstable, similar to the resilient moduli, at higher pore pressure ratios.

The results obtained in this chapter are comparatively analyzed and summarized in the following two chapters.
CHAPTER 5
IMPLEMENTATION OF TEST RESULTS

5.1 Introduction

The purpose of this chapter is to discuss the application of testing results to the mechanistic design and construction of permeable pavement structures. This study has given results that point towards the development of limiting criteria in terms of vertical stress, vertical strain and gradations of base course to minimize pore pressure development of crushed base coarse subjected to repeated loads. Applications in terms of these results are illustrated and discussed.

5.2 Saturated vs. Low Moisture Strength

Sands and gravels containing material passing the No. 200 (0.075 mm) sieve are subject to rapid pore pressure generation when saturated and subjected to cyclic loading. Pore pressure generation results in decreased effective confining stress and thus decreased strength. The rate of residual pore pressure generation is affected by increasing vertical strains.
As was demonstrated in Chapter 4, the resilient modulus is not an appropriate measure of soil strength of saturated materials with pore pressure ratios above approximately 0.6. Recoverable strains become very small at pore pressure ratios above 0.6 or so resulting in high resilient modulus computations. This occurs even when the stress reactions are decreasing as in strain-controlled testing.

A better means of analysis for these tests is the cyclic strain (Young’s) modulus. Cyclic strain moduli give an indication of the decrease in the material’s resistance vertical stress.

Figure 5.1 shows the modulus ratios of the cyclic strain moduli determined in the saturated tests divided by the low moisture content tests of the same material ($E_{sat}/E_{dry}$), as a function of the first 100 cycles. All tests shown in this figure were run at approximately 0.25% cyclic strain. Damage to the material may be estimated as one minus the modulus ratio.

The comparison in Figure 5.1 shows the superiority of the permeable bases. The dense graded base (DENSE), when saturated initially, only had 0.44 (44%) of its low moisture content strength and diminishes rapidly from there.
Figure 5.1  Saturated Base Cyclic Strain Modulus Comparison
Permeable bases were in every case initially stronger than when at low moisture content. This is caused by slight dilation which decreases the pore pressure yielding greater effective confining pressure at the beginning of the test. The terminal (residual) values and slopes of decay are the most useful information here.

Residual strength, as indicated by the modulus ratio, of the Wisconsin specification material (Wis #1 & Wis #2) is at greater than 80% of the low moisture content strength even after 100 repetitions. The Minnesota Specification base (MN #1 & MN #2) retained less than 40% of its strength.

Similar laboratory testing of gradations expected to be used on a project subject to high moisture conditions is recommended. Tests should be performed at each end of the expected specification band. First at near optimum moisture content and then when saturated. Testing at several strain levels such as 0.1, 0.25, and 1% will give indication of strain and stress limitations that should be in effect during spring thaw.

Mechanistic analysis of existing or similar paved sections under loading of the design vehicle will give prediction of stress and strain levels to be expected.
In northern regions the critical stress and strain occur at the point where thawing has proceeded only slightly into the unbound layers. At this point the base is bounded by pavement at the top and a semi-rigid frozen layer beneath.

5.3 **Effect of Shear Modulus on Pore Pressure Build Up**

It has been shown that as pore pressures build up, the shear modulus decreases. The shear modulus is a function of the shear strain. We have also seen that pore pressure build-up rate is a function of the magnitude of cyclic strain.

This is consistent with Ladd, et. al. (1989). In fact, Ladd’s report concluded that pore pressure build up during cyclic shear loading of sand is controlled mainly by magnitude of cyclic shear strain which leads to the conclusion that shear modulus rather than relative density, is the main parameter controlling pore pressure buildup in the field for a given cyclic loading.

Sample preparation methods have also been shown to affect pore pressure generation, Mulilis, Chan & Seed (1975). Here the samples have all been prepared in like manner to eliminate this variable.
The graphs shown as Figure 5.2 are the summary plots of pore pressure ratio as a function of the Shear Modulus. The legend gives the cyclic strain level parenthetically.

Using this type of graphical analysis one may determine several parameters regarding test soils. Generally pore pressure ratios increase as the shear moduli diminish for all of the bases. This relationship appears to be nearly linear. The limits of shear modulus below which liquefaction will occur may be found. For the materials shown here, it is found that liquefaction only occurs when the shear modulus falls below 500 psi (or $E = 1350$ psi). A pore pressure ratio of 0.4 (40% liquefaction) occurs only when the shear moduli have degenerated to 1,500 psi ($E = 4050$ psi) or less. Wet unbound layers with moduli approaching these levels could be subject to rapid pore pressure generation under traffic loading.
Figure 5.2 Shear Modulus vs Pore Pressure Relationships for Saturated Bases
5.4 Limiting Vertical Stress

The testing performed in this study have not been directed at obtaining general limiting vertical deviator stress criteria for all situations. However, the test data may be analyzed to show relationships with the materials used here as an example.

Figures 5.3 and 5.4 show the cyclic change in pore pressure of these materials as a function of the vertical stress applied for 0.25% and 1% cyclic strain, respectively. With knowledge of the expected stress upon the base by a design vehicle, using graphs as these near the expected strain level, one may choose limiting stress criteria. The confining stress of an unbound base is low, being a shallow depth. Therefore, acceptable limits for pore pressure change would be small.
Figure 5.3  Cyclic Pore Pressure Change for Saturated Bases at 0.25% Cyclic Strain
Figure 5.4  Cyclic Pore Pressure Change for Saturated Bases at 1%

Cyclic Strain
Determination of a limiting value for pore pressure change may have to come from field instrumentation and/or using predictive models such as Eq. 2.2. As an example, assume 1.0 psi is the maximum allowable change in pore pressure beyond which excess damage is expected. Then at a 0.25% cyclic strain level (Figure 5.3), dense base must be limited to less than around 4 psi vertical stress. The Minnesota and Wisconsin gradations would not require limitations for this strain level.

If the pavement was expected to sustain 1% cyclic strains (Figure 5.4), then dense graded base would necessarily be limited to 3 psi and the Wisconsin specification base to 12 psi. Should Ottawa sand type materials be used, they would have to be limited to 5 psi deviator stress.
5.5 Limiting Vertical Strain

Here again conclusive limits are not determined in this study, but a possible means of analysis is. Samples may be tested over a range of vertical strains. First, the range should be tested at low moisture content. Then the specimen may be saturated and retested for a pre-determined number of repetitions (say 500), starting at the lowest level, then increasing until liquefaction occurs.

Plotting the modulus ratio (as defined in Section 5.2) as a function of the cyclic vertical strain, one may interpolate (or, with caution, extrapolate) limiting strain levels to minimize strength loss and pore pressure increases. Figure 5.5 shows these results for the first 8 cycles of the dense graded base. Recall that this material liquefied and lost all strength after 8 cycles at the nominal 1% cyclic strain level. There is no point within these test limits where the material is at full strength.
Figure 5.5  Limiting Strain Analysis for Dense Graded Base
Often the spring strengths of unbound pavement layers may be half or less of the summer strengths. Alaskan pavements are often designed for half strength in the spring. Extrapolation of the lines in Figure 5.5 shows that at 0.15% strain the material will be at 50% strength for one cycle. Testing at lower strain, will give better limiting values, but then this strain level may be so small that the amount of pavement covering required to attain this limiting strain is economically impractical.

Tests by Ladd, et. al. (1989), on sands showed that at cyclic shear strain levels of $1 \times 10^{-3}\%$ or less that pore pressure build up will not occur. In the context of these tests a $1 \times 10^{-3}\%$ shear strain converts to $6.7 \times 10^{-4}\%$ vertical strain, that is 6.7 microstrains. This is very small and much less than may be expected to occur in a pavement base or subbase under design vehicle traffic.

If a gradation is found to be unsuitable, another should be tried and the data plotted similarly. Figure 5.6 shows results of Wisconsin specification base. Here it is found that to maintain at least 50% strength the vertical strain should be limited to approximately 0.85% (8,500 microstrains) for single repetitions. The strain limit decreases with repetitions. After 8 cycles the limiting strain becomes 0.68%.
Figure 5.6  Limiting Strain Analysis for Wisconsin Specification

Permeable Base
Another way to consider the situation is as shown in Figures 5.7 and 5.8. Here the peak change in pore pressure for various test cycles are plotted as a function of the cyclic vertical strain.

These two graphs are very interesting. They show for these tests that after a given number of cycles the lower strain produces as much or more pore pressure change than the higher strain. In fact, there was a point in which pore pressure generation is independent of cyclic strain level. The cause of this occurrence is probably due to permanent deformation of the sample, decreasing its reaction.

For the dense graded base (Figure 5.7) the cyclic strain independence is at the 3rd cycle. For the Wisconsin specification permeable base (Figure 5.8) this is at around the 80th cycle. The dense graded base has its maximum change in pore pressure on the first cycle, at either strain level, then the changes diminish with continuing repetitions. With the Wisconsin specification base, at the lower strain level, the pore pressure change is minimum on the first cycle, then increases to approximately the 50th cycle and decreases slightly from there. At the higher strain level the open graded base behaves similar to the dense graded base except it generates pore pressure at much slower rates.
Figure 5.7  Cyclic Stain - Pore Pressure Relationships for Dense Graded Base
Figure 5.8 Cyclic Stain - Pore Pressure Relationships for Wisconsin Specification Permeable Base
For a given saturated aggregate, pore pressure generation is dependent on the cyclic stress level, as shown earlier. From a strain standpoint, it is dependent on the number of cycles and the initial strain level. Following a certain number of repetitions the pore pressure generation becomes independent of the cyclic strain level. Considering this, it is appears that limiting vertical deviator stress is a better method for use in design.

5.6 Construction of Permeable Bases

This section discusses specifications, construction techniques, special considerations, and typical materials used with permeable bases. Although there is often much apprehension regarding the use of open graded materials, reports indicate that once construction begins reservations vanish, Hoffman (1982), Mathis (1989).

Minnesota and Pennsylvania require a coefficient of uniformity \( \frac{D_{60}}{D_{10}} \) of at least 4 to assure stability. Applying the coefficient of uniformity to the gradations tested in this study, the dense graded material calculates to be 107.
The coefficient of uniformities for the Minnesota and Wisconsin specification material are 19 and 2.2, respectively. Evidently the Wisconsin gradation would not meet the Minnesota requirements.

The Wisconsin specification permeable base gradation used performed outstandingly. It is a very coarse gradation. The compressibility of this material is low due to its high percentage of rock. Since the rocks are crushed, interlocking is increased, and the sliding at contacts between particles which is thought to lead to pore pressure build-up is minimized. Wong, Seed & Chan (1975), reported that in laboratory tests of saturated gravels that the larger and more uniformly graded the aggregates were, the greater was their strength. Therefore the coarser gradations are most desirable for base course. However, some difficulties may arise during construction, especially with very uniformly graded and rounded aggregate.

Limiting traffic prior to paving open graded bases may be required in order to maintain lines, grades, and avoid loss, degradation and/or segregation of material. Sometimes it is specified that the permeable material be placed at a certain percent moisture content in order to reduce segregation.
Rubber tired pavers may rut and displace the permeable bases, so tracked pavers, which distribute loads better are usually specified. A roller may be used to "dress up" the permeable material immediately in front of the paver. Requiring the permeable base to be constructed with at least 85% crushed material adds greatly to the stability of the material. No stability problems with permeable bases were reported by Mathis (1989), when crushing is required.

Some means of horizontal confinement should be provided. Often a separation geotextile fabric is placed beneath and around the edges of a permeable base. Perhaps using a stabilization grid would also make construction easier.

The open graded nature of permeable bases provides for less density. This leads to several ramifications. Less material is required to fill the same volume, which can lead to saving material and cost. Hoffman (1982), reported that permeable base used on a project in Pennsylvania only cost 5% more than the standard dense graded base. Thermal conductivity is reduced by lower density, so open graded bases could decrease frost and thawing penetration. This property may be advantageous in cold regions.
A possible negative factor to open graded bases is that the if free passage of ambient air is supplied to the base of the pavement, that the asphalt concrete may cool more rapidly, leading to thermal cracking and more rapid aging of the asphalt. Placing perforated pipe in the base, directly under the pavement, may not be advisable in northern climates.

Cedergren (1990), found that the primary sieve effecting permeability is the No. 100 (.150 mm) sieve. In fact, he has tabulated permeability estimates as a function of the percent passing this sieve. The less that passes, the higher the permeability. With high permeability desirable it is better to allow no material passing the No. 100 sieve. As was mentioned earlier, Hoffman (1982), noticed during construction of permeable layers that when the material had as much as 5% passing the No. 10 (2 mm) sieve, there were problems with permeability testing.

There are procedures for measuring the in-place permeability of base and subbase courses, Moulton & Seals (1979). Sometimes there may be field permeability specifications for open graded materials subject to testing with these special devices.
5.7 **Summary**

The open graded bases tested in this study retained a much higher percentage of their dry strength when saturated. The shear modulus of saturated samples tested diminished almost linearly with pore pressure increases.

Limiting stress and strain criteria for particular gradations can be determined from this type of testing. Gradations could be found that will withstand the design loading in the worst condition. Limiting vertical deviator stress seems to be the most promising.

Construction of permeable bases requires extra considerations, none of which pose real drawbacks. It appears that the benefits far exceed the liabilities.
CHAPTER 6

SUMMARY, CONCLUSIONS AND RESEARCH NEEDS

6.1 Summary

This Chapter comparatively summarizes the important findings of this research study. Conclusions regarding the testing that has been done are presented and recommendations are made for further study.

Pore pressure may increase rapidly to the point of liquefaction in saturated cohesionless soils when the material is subjected to repeated shear stress reversals. Shear stress reversals occur within pavement structural layers under traffic loading. Pore pressure increases lead to less effective confinement of unbound materials which deceases its strength. Increased pore pressures and loss of support beneath a pavement can lead to pumping and premature failure of the pavement structural section.
Laboratory tests conducted here under controlled conditions have allowed shear stress reversal. Several gradations of crushed aggregate base course as shown in Figure 6.1 have been tested to measure their reaction to this type of loading at both low moisture content and when saturated. Permeable base gradations tested (Minnesota and Wisconsin spec.) demonstrated superior characteristics.

Falling weight deflectometer (FWD) testing of poorly drained sections demonstrated weakening of the supporting layers due to excess moisture. However, the FWD does not instigate shear stress reversal, as occurs under traffic loading, when testing. This device is useful for comparative studies but, it does not allow true analysis of poorly drained pavements. Base course gradations of poorly drained pavements investigated using the FWD are shown in Figure 6.2.
0.45 POWER GRADATION CHART

DENSE GRAD.  MINN. SPEC.  WIS. SPEC.

PERCENT PASSING

SIEVE SIZES

Figure 6.1 Laboratory Tested Base Course Gradations
Figure 6.2  Base Course Gradations at FWD Test Sites
6.2 Conclusions

According to what has been seen in this study it appears that gradations deviating from a straight line on the 0.45 power gradation chart perform better when wet. This makes sense because a less dense graded material will have higher permeability and therefore greater ability to dissipate pore pressures.

The mean permeabilities (k) measured on the various laboratory samples are given in Table 6.1.

<table>
<thead>
<tr>
<th>SAMPLE TYPE</th>
<th>k(cm/sec)</th>
<th>k(Ft/day)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ottawa Sand</td>
<td>3.0 x 10^{-3}</td>
<td>8.5</td>
</tr>
<tr>
<td>Dense Graded Base</td>
<td>4.1 x 10^{-4}</td>
<td>1.2</td>
</tr>
<tr>
<td>Minnesota Specification</td>
<td>2.7 x 10^{-3}</td>
<td>7.7</td>
</tr>
<tr>
<td>Permeable Base</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Wisconsin Specification</td>
<td></td>
<td></td>
</tr>
<tr>
<td>Permeable Base</td>
<td>5.8 x 10^{-3}</td>
<td>16.4</td>
</tr>
</tbody>
</table>
When saturated, the propensity to weaken and build pore pressures under cyclic loading appears to be somewhat dependant on the permeability. The permeable bases retained strength longer, and went to the point of initial liquefaction at much slower rates, than the dense graded base under these conditions. Higher permeability evidently allows for dissipation of pore pressure.

All of the saturated tests indicate some weakening or softening of the specimens as a function of load repetitions. Figure 6.3 summarizes test results of the saturated bases at .25 percent cyclic strain levels. Figure 6.4 shows the dense graded and Wisconsin specification bases tested at 1 percent cyclic strain. These figures indicate the superiority of the more open graded bases under repeated load conditions.

The decrease in stiffness in saturated soils tested here was directly related to pore pressure buildup (see Chapter 5). Since pore pressure generation decreases the effective confining stress, this shows the great dependance of these cohesionless materials upon confinement. Summaries of pore pressure ratios within the various saturated based course tests are shown in Figures 6.5 and 6.6.
Figure 6.3  Saturated Base Modulus Variation Summary at 0.25% Cyclic Strain
Figure 6.4  Saturated Base Modulus Variation Summary at 1% Cyclic Strain
Figure 6.5  Saturated Base Pore Pressure Variation Summary at 0.25% Cyclic Strain
Figure 6.6  Saturated Base Pore Pressure Variation Summary at 1% Cyclic Strain
Table 6.2 summarizes the liquefaction testing results.

<table>
<thead>
<tr>
<th>SAMPLE</th>
<th>Nominal Cyclic Strain (%)</th>
<th>Number of Cycles to Initial Liquefaction</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ottawa Sand, #1</td>
<td>1.0</td>
<td>5</td>
</tr>
<tr>
<td>Ottawa Sand, #2</td>
<td>1.0</td>
<td>6</td>
</tr>
<tr>
<td>Dense Graded Base, #1</td>
<td>1.0</td>
<td>5</td>
</tr>
<tr>
<td>Dense Graded Base, #2</td>
<td>0.25</td>
<td>35</td>
</tr>
<tr>
<td>Dense Graded Base, #3</td>
<td>0.25</td>
<td>32</td>
</tr>
<tr>
<td>Minnesota Permeable, Base #1</td>
<td>0.25</td>
<td>400</td>
</tr>
<tr>
<td>Minnesota Permeable Base, #2</td>
<td>0.25</td>
<td>242</td>
</tr>
<tr>
<td>Wisconsin Permeable Base, #1</td>
<td>0.25</td>
<td>&gt;600</td>
</tr>
<tr>
<td>Wisconsin Permeable Base, #2</td>
<td>0.25</td>
<td>&gt;600</td>
</tr>
<tr>
<td>Wisconsin Permeable Base, #3</td>
<td>1.0</td>
<td>280</td>
</tr>
</tbody>
</table>

It was not the intent of this study to liquefy gravels. The intent was to measure roadbed soils mechanistic reactions to repeated loads under high moisture conditions. The fact that initial liquefaction was obtained in the laboratory was surprising.

Membrane compliance effects can influence the measured pore pressure buildup. The affect of membrane compliance should make these results conservative because it actually buffers pore pressure buildup.
For this reason it may be more difficult to liquefy gravelly soils in the laboratory. However, in the field, gravelly soils are generally known to perform better (dissipating pore pressures) than sands under high moisture conditions, Wong, Seed & Chan (1975), etc.; therefore, they should also perform better in the laboratory. Perhaps membrane compliance problems actually give a better representation of field behavior. The goal of laboratory testing should be to simulate field conditions. A discussion and analysis of membrane compliance affects is included in Appendix A.

Resilient modulus, defined as the deviator stress divided by the recoverable strain, has not been found to be an effective way to measure the strength of saturated cohesionless soils. Recoverable strains may become very small as the pore pressure ratio exceeds 0.6. Figures 6.7 and 6.8 summarize testing results.
Figure 6.7  Saturated Base Resilient Modulus Variation Summary at 0.25% Cyclic Strain
Figure 6.8  Saturated Base Resilient Modulus Variation Summary at 1% Cyclic Strain
Damping ratios of saturated materials were in every case larger than when at low moisture content. The damping ratios also increase with pore pressure increases in the saturated tests (see Figures 6.9 and 6.10). Damping ratios above unity often occurred when serious weakening or initial liquefaction had taken place. A problem occurs with the cyclic computation of the damping ratios similar to as with the resilient moduli.

As the tested soils weakened, the strain energy becomes very small. Strain energy is in the denominator of the damping ratio definition; therefore, the damping ratios become larger as it decreases.

The limited number of tests performed in this study do not provide enough data in order to substantiate proof of the trends shown here. Yet, further testing performed by Raad, Minassian and Gartin (1991), gave similar results. Due to the great diversity of soils, it is hard to say how many tests are enough. It is believed that the general trends shown here give reasonable comparative results.
Figure 6.9  Saturated Base Damping Ratio Variation Summary at 0.25% Cyclic Strain
Figure 6.10 Saturated Base Damping Ratio Variation Summary at 1% Cyclic Strain
6.3 Research Needs

Permeable bases should be tried in Alaska. Test sites should be monitored for performance and the results published. There is little to lose.

Field measuring of pore pressure generation in conjunction with traffic loading has not been done in Alaska. Pore pressures are known to increase under traffic loading, but the magnitudes, required stimulus, and soil conditions are not well understood.

Further laboratory testing should be performed to determine influences of density and moisture content on the mechanistic behavior of gravels. Testing of asphalt treated permeable bases is also recommended. Limiting criteria should be determined for the use of designers and to minimize the need for load restrictions in the spring.

For the purpose of pavement testing a moving wheel load would be superior to falling weights. Test vehicles fitted with standard design axles and loads travelling at design speeds would be ideal.
Better means of determining springtime load restrictions need to be developed. Currently Alaska relies upon FWD testing for this purpose. Since FWD’s do not perform well at temperatures below 40 degrees F, the pavement cannot be tested until much of the damage is already done.
REFERENCES


Kitamura, R. and Huruyama, M., *Compression and Shear Deformation of Soil
Under Wide Ranging Confining Pressure*, from: *Advanced Triaxial Testing

Komamura, F. and Huang, R.J., *New Rheological Model for Soil Behavior.*
GT7, July 1974.

Pressure Buildup in Clean Sands Because of Cyclic Straining.*

Lade, P.V. and Hernandez, S.B., *Membrane Penetration Effects in Undrained
Tests*. ASCE Journal of the Geotechnical Engineering Division, Vol. 103,
Gt2, February 1977.

York, 1969.

Lazon, B.J., *Damping of Materials and Members in Structural Mechanics.*

Lee, F.H., Schofield, A.N., *Centrifuge Modeling of Sand Embankments and


APPENDIX A

ANALYSIS OF MEMBRANE COMPLIANCE

AFFECTS

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Membrane Penetration Affects

Pore pressures measured in undrained triaxial tests on saturated soils are affected by the system employed for pore pressure measurements. The interference of the measuring system with the pore pressures is caused by the flexibility of the pore pressure measuring device, the tubing, fittings, valves, and by the compressibility of water filling the measuring system. Triaxial cells designed for undrained testing are designed to minimize flexibility affects. However, penetration of the rubber membrane into specimens of granular soil may have considerable effects on the soil and pore pressure behavior in undrained triaxial tests, Lade & Hernandez (1977).

Changes in sample volume caused by membrane penetration during undrained testing are called "membrane compliance". Membrane compliance can cause erroneous pore pressure reading during the saturation phase and during testing. When samples are being tested for B-values, the cell pressure is changed, the pore pressure change noted. Lade & Hernandez (1977), found that for changes in cell pressure of 10 psi, the computed B-value can increase if 5 to 10 minutes elapse between the change and the reading of the pore pressure. Using readings taken after too long a time can result in over estimation of the degree of sample saturation.
When performing B-value tests it is therefore recommended to change the cell pressure less than 10 psi and wait a minimum amount of time before checking the pore pressure. This procedure was followed during this study.

During undrained testing membrane compliance effects may result in an increase in sample volume and a corresponding decrease in the pore pressure measurement. This can lead to a serious overestimation of the resistance of a sample to liquefaction.

**Predictive Model**

Membrane compliance is primarily a function of: changes in effective confining stress, grain size, sample size, Seed and Anwar (1987). As a result of research performed by various investigators an empirical equation was developed for the estimation of normalized unit membrane penetration ($S$), Seed and Anwar (1987). The reported equation is:

$$ S = [(0.0009) \log_{10}(D_{20} + 2.0)]^{3.8} + 0.0005 \times F_1 \times F_2 \quad (A1) $$

where:

$$ S = \text{cc/cm}^2 \text{ per log cycle change of } \sigma'_3 $$

($\sigma'_3$ in psi)
\[ D_{10}, D_{20} = \text{soil grain sizes (mm)} \]

\[ F_1 = \left( \frac{D_{10}}{D_{20}} \right)^{0.33} \]

\[ F_2 = \left( \frac{D_{20}}{\sqrt{D_{10}}} \right)^{-0.25} \]

Knowing the area of the membrane, the gradations and the volume of the specimens, this equation can be used to predict the membrane compliance in terms of volumetric strain, "S(vol)", as a function of log cycle changes in effective confining stresses of greater than 1 psi. The developers of this equation felt that cyclic changes in effective confining stress of 1 psi or less were not of interest.
Application of Predictive Model

The following table gives the mean values determined from each gradation tested and the resulting S(vol) constants:

<table>
<thead>
<tr>
<th>Grad.</th>
<th>D₁₀ (mm)</th>
<th>D₂₀ (mm)</th>
<th>Area (sq. cm)</th>
<th>Volume (cc)</th>
<th>S(vol) (*10^-5)</th>
</tr>
</thead>
<tbody>
<tr>
<td>Ottawa sand</td>
<td>0.13</td>
<td>0.189</td>
<td>676.1</td>
<td>1699.2</td>
<td>1.224</td>
</tr>
<tr>
<td>Dense Graded</td>
<td>0.075</td>
<td>0.425</td>
<td>682.0</td>
<td>1692.5</td>
<td>.6919</td>
</tr>
<tr>
<td>Mn Spec.</td>
<td>0.425</td>
<td>1.72</td>
<td>683.2</td>
<td>1706.0</td>
<td>.9937</td>
</tr>
<tr>
<td>Wis. Spec.</td>
<td>4.98</td>
<td>6.11</td>
<td>686.6</td>
<td>1684.6</td>
<td>2.354</td>
</tr>
</tbody>
</table>

Figure A1 shows the predicted volumetric strains as a function of cyclic change in effective confining stress. Only one cycle of any of the 0.25% cyclic strain tests produced more than 1 psi of cyclic change in effective confining stress (see Fig. 5.3).
Figure A1  Predicted Membrane Compliance Effects
That was the first cycle of the Dense graded specimen, which liquefied in 35 cycles. The remaining 34 cycles of the test resulted in changes of less than 1 psi, necessitating no correction.

The 1% cyclic strain tests did have more cycles in which the change in effective stress was greater than 1 psi. Still, these changes degenerated rapidly. Figure 5.4 shows the results for all the 1% cyclic strain tests. The major offenders are the Ottawa sand, which are not of particular analytic interest in this study. The dense graded base again only had one cycle with excess change in pore pressure (effective confining stress) and liquefied in 5 cycles anyway, so there is no need to correct this test. The first 9 cycles on the Wisconsin specification base had changes of slightly over 1 psi.

A proposed method of correcting pore pressures for membrane is to do the following:

1. Compute the \( S(\text{vol}) \) constant for the test sample
2. Apply the following correction to pore pressure for test cycles which had changes in effective confining stress greater than 1 psi:

\[ \Delta u = S(\text{vol}) \times [\log_{10}(\Delta \sigma'_3)] \times K_w \]

(A2)

where: \( K_w \) is the bulk modulus of water (312,500 psi, Martin, Finn & Seed (1978))

This equation may be applied under the most conservative assumption that all of the volume change resulting from membrane compliance results in the decompression of water in the sample.

3. The change in pore pressure determined from Eq. A2 may then be added to the tested value and the "corrected" pore pressure ratio recomputed. Figure A2 shows the results of corrections to the 1% strain Wisconsin specification permeable base test. The changes determined thusly show slight increases in pore pressure ratios for the first few cycles, but then revert to the measured values when the cyclic changes in pore pressure fall below 1 psi. Since the corrected pore pressure ratios have no effect on the final test results they are not reported in the Thesis.
Figure A2  Membrane Compliance Corrections for Wisconsin Specification
Permeable Base
This Appendix describes background for the derivation of Eq. 2.3, summarizing the work of Martin (1976). The development for using damping ratios follows damped oscillator modeling. Equations for and methods of solution of vibration problems using damped oscillators may be found in most any differential equations and or vibration analysis text book. The following sections briefly describe damped oscillator and stress-strain damping analysis.

**Damped Oscillator Methods**

Using dynamic analysis, the system could be analyzed as a vibration problem using:

\[
[M] \ddot{u} + [C] \dot{u} + [K] u = [F(t)] 
\]  
(B1)

Where:

- \([M]\) is the Mass Matrix
- \([C]\) is the damping characteristic matrix
- \([K]\) is the stiffness matrix
- Each matrix row characterizes a layer
- \(\ddot{u}, \dot{u}, u\) are the vertical acceleration, velocity and displacement
- \(F(t)\) is the forcing function matrix of traffic loading.
The left side of this equation should model the pavement’s reaction to the motion instigated by the forcing function (traffic). Conceptually the physical system is composed of an assembly of structural components (i.e., Springs and dashpots) and inertial components (lumped or distributed masses). The damping of the system may be determined by observing and characterizing the response of the system under various loading conditions. Using the theory of visco-linear elasticity, the dynamic response of the system lends itself to complete analytical treatment. The damping in the system is thus represented by equivalent linear viscous damping.

Taking Eq. B1 and dividing through by the mass and digressing to a single-degree of freedom system gives:

\[ \ddot{u} + 2 \beta \omega \dot{u} + \omega^2 u = \frac{1}{M} F(t) \]  

(B2)

Where:  
\[ \beta = \frac{C}{2 \sqrt{KM}}, \text{ the damping ratio} \]  
(B2.1)

\[ \omega = \sqrt{K/M}, \text{ the undamped natural frequency} \]  
(B2.2)
The complete solution for the problem involves solving first for the homogeneous equation (equal to zero). This complimentary solution is called the "transient" term in the full solution. The transient term solves for the system's response to release from an initial deflection and goes to zero as time goes on. Its behavior depends on the value of $\beta$.

When $\beta$ is greater than unity the system is said to be "over-damped". The motion of over-damped systems is described by smooth non-oscillatory curves.

If $\beta$ is less than unity, the motion is oscillating and the system is said to be "under-damped". Solutions for under-damped systems are in the form of dampened sinusoids, that is, sinusoidal oscillation with decreasing amplitude as a function of time.

In the case where $\beta$ is equal to one, the system is "critically damped". This is motion right on the interface between oscillatory and non-oscillatory motion. When considering a point on a pavement, the transient term for the motion of this point would be the resulting motion following the passing of a single traffic load. The particular solution to the equation would model the response to cyclic traffic loading.
If you are considering the motion of a pavement moving along with the wheel, the solution and equation becomes much more complex. Thompson (1963), performed damping analysis relating moving wheel loads on pavements and determined that the pavement in front of a moving wheel load is oscillatory. Behind a moving wheel load he found over-damped curves. His solution was actually even more complicated, involving the solution of a fourth-order differential equation.

**Damping Using Stress - Strain Characteristics**

The equivalent linear damping ($\beta_{eq}$) for a non-linear material ratio is defined as:

\[
\beta_{eq} = \frac{1}{4\pi} \frac{\Delta W}{W} \tag{B3}
\]

where: $\Delta W$ is the area inside of a shear stress-strain hysteresis loop for a test cycle (damping energy).

$W$ is the area of the triangle as shown in Figure 2.2 (strain energy) is used for damping analysis in this study.
Soils exhibit non-linear behavior under dynamic loading. For such materials the study of its load-deformation characteristics is the only tool available to assess the energy dissipation properties.

Using this approach it is assumed that the energy dissipated in damping originates in the material of the structural components of the physical system. The damping characteristics may then be determined by studying the stress-strain nature of the material or the load-deformation curves of the structural system under different loading conditions.

Consider a single layer of a continuous system of thickness, $H$, and density $\rho$, subjected to cyclic loading. Assume that the constitutive material may be described by a system similar to a Voight model, with shear modulus $G$ and viscosity $\eta$. The stress-strain law of the material is written as:

$$\tau = G\gamma + \eta \dot{\gamma} \quad \text{(B4)}$$

Where $\tau$ is the shear stress and $\gamma$ is linear shear strain.
The continuous system may be represented by a damped oscillator by setting:

\[ u = \gamma H \]
\[ K = G/H \]
\[ C = \eta/H \]
\[ M = \rho H \]  \hspace{1cm} \text{(B5)}

Substituting Eq. B5 into a single degree of freedom version of Eq. B1. The results in giving the right side of Eq. B4 so:

\[ F = \tau \]  \hspace{1cm} \text{(B6)}

Therefore, in the Voight model there is identity between structural and mechanical behavior. That is, the shear stress variation is identical to the forcing function.
**Damping Energy.** By definition the damping energy of the system, $\Delta W_s$, is the total energy absorbed per cycle of loading by the entire specimen. In other words it is measured by the area of the load deformation loop, which may be expressed mathematically by:

$$\Delta W_s = -\int F du$$

(B7)

Similarly, the damping energy per unit volume of the material, $\Delta W$, is the total energy absorbed per cycle of loading per unit volume of material. Its mathematical expression is:

$$\Delta W = -\int \tau \, d\gamma$$

(B8)

As a consequence of Eq. B6:

$$\Delta W_s = H \Delta W$$

(B9)

The energy loss per cycle is defined as the area inside the hysteresis loop of stress versus strain. Damping energy is a material property.
Normally in analysis of linear behavior at steady state oscillation the hysteresis loop is considered as an ellipse and the area is found to be:

\[ \Delta W = \pi \eta \Omega \gamma_a^2 \]  \hspace{1cm} (B10)

where: \( \Omega \) is the excitation frequency
\( \gamma_a \) is the shear strain amplitude

Similarly, the damping energy of the system has the expression:

\[ \Delta W_s = \pi C\Omega u_a^2 \]  \hspace{1cm} (B11)

where: \( u_a \) is the amplitude of displacement

By setting Eq. B10 and B11 equal according to equation B9 gives:

\[ Cu_a^2 = \eta H \gamma_a^2 \]  \hspace{1cm} (B12)
Eq. B12 can be modified by using the Eq. B5 definition for \( u \), in solving for \( C \) in Equation B2.1 and noting that \( \sqrt{KM} \) is equal to \( \frac{K}{\omega} \) per Eq. B2.2. The result is:

\[
2\beta \frac{K}{\omega} H = \eta \quad (B13)
\]

According to Eq. B5, \( G = KH \), inserting this into Eq. B13 gives:

\[
\frac{2\beta G}{\omega} = \eta \quad (B14)
\]

Inserting Eq. B14 into B10 gives:

\[
\Delta W = 2\pi \frac{\Omega}{\omega} \beta G \gamma^2_a \quad (B15)
\]

This is the definition for damping energy of an elliptical stress-strain loop.
Strain Energy. The strain energy for the Voight model as shown in Figure 2.1 is the elastic energy stored in the spring of shear modulus (stiffness). In other words the area under the shear modulus line above the shear strain axis. Mathematically the linear strain energy, $W$, is:

$$W = \frac{1}{2} G \gamma^2_a$$  \hspace{1cm} (B16)

By dividing B16 into B15, one derives a fundamental relationship for the fraction of critical damping to stress-strain characteristics:

$$\frac{\Omega}{\omega} \beta = \frac{1}{4\pi} \frac{\Delta W}{W}$$  \hspace{1cm} (B17)

At resonance $\Omega = \omega$, then we have the linear damping ratio:

$$\beta = \frac{1}{4\pi} \frac{\Delta W}{W}$$  \hspace{1cm} (B18)
Rate Independent, Non-Linear Damping

Some conclusions important to the construction of an appropriate model for soil behavior are:

1. Soil dynamic response is highly non-linear. The Secant modulus decreases while the damping increases with strain amplitude.

2. The damping is rate-independent in the range of frequencies of interest.

For some materials the damping ratio is not a function of the frequency. This type of damping has been called "hysteretic damping" (Lazon, 1968), but since all damping phenomena are associated with hysteresis loop effects, it may be more appropriately called "rate-independent damping" (Martin, 1976).

For materials of this type, the shape of the stress-strain loop is not influenced by the excitation frequency. Such damping may be linear, viscous-type dissipation force (not effected by loading rate) or non-linear.

Generally, rate independent non-linear materials such as soils, display hysteresis loops having sharp corners at their extreme points.
Equivalent Linearization Techniques

A logical method of solving the dynamic response of actual non-linear systems is to replace them conceptually with equivalent visco-linear systems, which solutions are available. Attention was first drawn to this approach by Jacobsen (1930). He concluded that the most useful criterion was the equivalence of energy dissipated (absorbed) per cycle, $\Delta W$. The basis for judging the appropriateness of this criterion was a comparison of the amplitudes of steady state resonant vibrations of an equivalent linear system, and a prototype non-linear system.

Accordingly, Eq. B18 becomes Eq. B3 (Eq. 2.3, in the Thesis):

$$\beta_{eq} = \frac{1}{4\pi} \frac{\Delta W}{W}$$

where: $\Delta W$ is the area in that non-linear shear stress-strain (hysteresis) loop, determined by numeric integration.
W is the area of the triangle under the \( G_{eq} \) line defined as the slope of the line passing through the points of maximum and minimum shear strain of a given hysteresis loop. The base of the triangle is the compressive shear strain of the cycle, and the height is the compressive shear stress at maximum strain.

For the type of tests performed in this study, W is mathematically defined as:

\[
W = \frac{1}{2} \frac{\tau_{\text{max}}^2}{G_{eq}}
\]

(B19)

The shear modulus and shear stress are determined from Young's modulus and measured deviator stresses using definitions from elastic theory, Timoshenko & Goodier (1987), with an assumed Poisson's Ratio of 0.35 for all materials tested.
APPENDIX C

FALLING WEIGHT DEFLECTOMETERS

AND

BACKCALCULATION
Falling Weight Deflectometer - Background

The falling weight deflectometer (FWD) was originally built in Denmark and is a development of a French device, Ullidtz (1987). The force pulse is obtained by dropping a weight on a specially designed spring system. Using Dynatest Model 800 or 8000 FWD's, this produces an impact load of peak force up to 18,000 or 27,000 pounds, respectively, with a duration of 25-30 microseconds. Drop heights may be varied with the FWD to create lesser impact loads which are appropriate for testing unbound layers. Figure C1 shows a Model 8000 FWD.

The falling weights are dropped on a loading plate of 11.8 inch diameter. The duration of the load over this distance corresponds to an approximately 25 mph wheel velocity. Ullidtz instrumented a test road and compared the response of a moving truck wheel load to an FWD impact load. Excellent correlations were found between the two in terms of deflection, vertical stress and vertical strain.
Figure C1  Dynatest Model 8000 Falling Weight Deflectometer
Deflections are measured with geophones at seven different distances from the loading plate. Measurement of deflection are recorded to the nearest 0.01 mils, with typical accuracy of 0.5% ± 0.04 mils. This accuracy is necessary because the subgrade modulus is determined from the outer sensors which often record deflections of only 0.8 to 1.2 mils, Ullidtz (1987).

The total test sequence is controlled from the drivers seat of the towing vehicle. The results are automatically stored on a computer floppy disk, for later uploading and processing. Figure C2, Gartin (1991), shows graphs of typical FWD data. The data is used for backcalculation of layer moduli which are used in the prediction of stresses and strains at critical locations, using elastic theory, for the purpose of pavement design.

However, as was explained previously, the FWD impact load does not result in shear stress reversal. In this way it does not represent a moving wheel load or tend to generate much pore pressure. Also, the standard FWD sensors can only measure accurately deflections up to 0.080 inch (2 mm). Testing of poorly drained sites often results in readings beyond this range, rendering the data useless.
Figure C2  Typical Falling Weight Deflectometer Time History

A) Surface Stress vs. Distance from Load
B) Time vs. Load and Sensor Deflections
C) Time vs. Deflection vs. Distance from Load

NOTE: Sensor 1 is at the Load center, Sensor 7 is offset 47.2" The others in between.
In poor sections often the post impact response would be non-typical (e.g. deflections not decreasing with distance from the load center). The reason for this type of response is due sometimes to shear failure of the material tested causing upward deflections to be measured adjacent to the loading plate. Data, as this, is of negligible value.

The FWD deflection transducers are unable to measure permanent deformations. They only measure deflections relative to the particular drop. As far as the backcalculation is concerned all deformations are then considered to be recoverable, which could lead to erroneous results should data taken where permanent deformations are occurring be relied upon for design.

**Modulus Backcalculation - Background**

The computer program, ELMOD, Versions 3.1 and 3.2, were used to backcalculate layer moduli and equivalent depth to stiff layer at field test sites. This program (by Dynatest Engineering A/S) uses the drop stress and measured deflections from the FWD computer disks. ELMOD is an acronym standing for Evaluation of Layer Moduli and Overlay Design.
Moduli are determined using Boussinesq’s Equations and Odemark’s Method. The principals used in this method are briefly described in the following pages of this section.

In 1885, Boussinesq published a report presenting equations for calculating stresses, strains, and deflections of a homogeneous, isotropic, linear elastic semi-infinite space under a point load. At the centerline under the load the equations for vertical stress ($\sigma_z$), vertical strain ($\varepsilon_z$) and displacement ($d_z$) at depth $z$ are:

$$\sigma_z = \frac{3P}{(2\pi z^2)}$$  \hspace{1cm} (C1)

$$\varepsilon_z = \frac{(1 + \mu(3 - 2\mu))P}{(2\pi z^2E)}$$  \hspace{1cm} (C2)

$$d_z = \frac{(1 + \mu)(3 - 2\mu)P}{(2\pi zE)}$$  \hspace{1cm} (C3)

Where:  

$P$ is the point load  

$\mu$ is the Poisson’s ratio  

$E$ is Young’s Modulus
These equations reveal some interesting facts. Vertical stress is independent of the elastic parameters ($\mu$, $E$). Stress and strain decrease proportionally to the square of the depth, whereas deflections only decrease linearly with depth. It is also interesting to notice that the strain at a given depth is equal to the deflection divided by the depth.

By integrating the Boussinesq Equations over the loaded area at the surface, stresses and strains can be determined in terms of the surface load stress. In most cases this must be done by numeric integration.

Boussinesq Equations are useful for interpreting results of plate loading tests on subgrade materials. Unfortunately the actual load distribution on soils is neither uniform nor a stiff plate distribution.

According to Ullitz (1987), the problem arising from unknown stress distribution may be solved by measuring the deflections at different distances from the center of the load. It was further reported that comparing the deflections obtained for a distributed load to those obtained for a point load, that for distances greater than twice the radius from the load center, a distributed load may be treated as a point load.
The modulus can then be found from:

\[ E = P \cdot \frac{(1 - \mu^2)}{[\pi r^* d_a(r)]} \]  

(C4)

Where: \( d_a(r) \) is the surface deflection at distance \( r \) from the center of load.

If the medium tested was a true linear elastic space, the moduli computed at different distances must be identical with FWD testing the modulus values computed measuring the load and deflections at six radial points are computed and graphed. The results give the user an idea of the non-linearity or variation in layers of the system.

Odemark's (1949) Method, is used to transform a system consisting of layers, with different moduli into an equivalent system where all layers have the same modulus, on which Boussinesq equations can be used.
For the stiffness to remain the same in a two layered system:

\[
\frac{E_2 I}{(1 - \mu_2^2)} = \frac{E_1 I}{(1 - \mu_1^2)} \quad (C5)
\]

Where: \( I \) is the moment of inertia, which must remain constant.

This leads to:

\[
\frac{h^3_0 E_2}{(1 - \mu_2^2)} = \frac{h^3_1 E_1}{(1 - \mu_1^2)} \quad (OR) \quad h_0 = h_1 \left[ \frac{E_1}{E_2} \left( \frac{1 - \mu_2^2}{1 - \mu_1^2} \right) \right]^{1/3} \quad (C6)
\]

Where: \( h_0 \) is the equivalent thickness of layer 2 in terms of layer 1

The ELMOD program assumes that all Poisson’s Ratio’s (\( \mu \)) are equal to 0.35 so Equation 3.6 reduces to:

\[
h_0 = h_1 \left( \frac{E_1}{E_2} \right)^{1/3} \quad (C7)
\]

Correction factors may be applied depending on layer thicknesses in relation to the radius of the loaded area and modular ratios. The correction factors are used to improve the agreement with elastic theory.
Moduli are backcalculated, using FWD data, by an iterative process to best match the measured peak deflections obtained in the FWD data. Numeric integrations are performed to determine deflections as a function of load, and modulus at given depths. The FWD data provides 7 deflection readings, thus, the process can be worked at each location and run until the solution is optimized.

Pavement temperatures are recorded for each testing sequence. Pavement moduli for thin (less than 3 inches) asphalt concrete pavements may be computed using an interpolating table which defines the temperature-modulus relationship in an ELMOD parameter file. For this study the table was generated using the Asphalt Institute equation, T.A.I. (1982). This Asphalt Institute equation is used to predict the dynamic modulus of an asphalt concrete mix based on P200 of the mix aggregate, load frequency, % air voids in the mix, penetration of the asphalt cement at 77 deg.F, asphalt content of the mix, and temperature. Once estimates of the preceding values are known, the equation defines a temperature-modulus relationship for the mix.
Estimation of moduli of cracked, thin pavement is done by dividing the otherwise backcalculated modulus by constant. The ELMOD User's Manual, Dynatest (1990), recommends using a constants of 1.5 on pavements with some cracking, and 3.0 with severe cracking. These constants were used in this study.

The presence of rock, or a stiff layer, at a shallow depth will greatly influence the deflection basin. When the stiff layer option is chosen the ELMOD program computes the equivalent depth to a stiff layer \( (h_s) \), in terms of the subgrade modulus \( (E_{sg}) \), for each test drop. Choosing this program option often results in improved deflection basin matching. The "actual" computed depth to the stiff layer \( (d_{sl}) \) may be determined using:

\[
d_{sl} = \sum_{i=1}^{n-1} h_i + \left[ h_s - \sum_{i=1}^{n-1} f_i h_i \sqrt[3]{\frac{E_i}{E_{sg}}} \right] \quad (C8)
\]

Where: \( f_i \) is a correction factor equal to 0.8, except for the first layer where it is 0.9 for a two layer system and 1.0 for a multi layer structure.

\( h_i \) is the thickness of layer i (layer 1 is the surfacing).

\( E_i \) is the Modulus of layer i.
Once the layer thicknesses and moduli are found, equation C8, Dynatest (1990), can be used to obtain a rough estimate of the thaw depth, at the time of testing, in northern climates. This is assuming that the frozen material is behaving as a stiff layer.

Equation C8 is highly dependent upon the surface layer modulus. When the surface modulus is determined on the basis of an estimated temperature-modulus relationship, the computed $d_{sl}$ found using this appears to be more variable, in the author’s experience.
APPENDIX D

PREPARATION OF LABORATORY SAMPLES

AND

COMPUTATION OF MECHANISTIC PROPERTIES
Testing Equipment

A triaxial cell using water as the confining medium, with nominal 2.8 inch diameter by 6 inch height samples, was used for static testing. The cell used a 1000 pound internal load cell. The triax cell loading piston and load cell are as manufactured by ELE International, Limited.

The cell pressure and sample pore water pressure gages are 100 psig capacity transducers as manufactured by Sensotec, Inc. The cell pressure transducer is attached to the base of the triaxial cell. Measurement of the pore water pressure was from the base of the sample. Earlier researchers, such as Mulilis, Chan & Seed (1976), placed pressure transducers at the top and base of samples, but found negligible differences between measured pore pressures.

The load frame, controller and function generator used in testing was manufactured by MTS Systems corporation. The external load cell on the system has a 50 kip capacity. The controller is capable of controlling and/or monitoring four channels. These first four channels used in the following experiments were 1) the external load cell 2) the internal load cell, 3) displacement, and 4) pore pressure. The cell pressure and volume change (when used) were monitored using external digital readout devices.
The Micro-profiler card in the MTS controller is capable of generating control functions of up to 99 segments. The programmed functions may be continuous or timed to stop. The program used in this testing was a linear displacement control function (ramp) compressing the samples at a rate of 1 mm per minute. At this strain rate the samples were brought to 20% strain, which was assumed as failure, in approximately 1/2 hour.

Data logging was done using an MTS Model 459.16 Testlink Interface package. The package includes a board and software that is installed on an IBM AT computer. This system is capable of monitoring and logging data, directly to the computer hard disk, on up to 8 channels simultaneously. Programming to call the interface routines and set up the data acquisition for this testing was done using Microsoft QuickBASIC, Version 4.5.

The sampling rate used in these experiments was 1 Hertz. At this relatively slow rate, the computer-read voltages from the transducers may be converted to engineering units (stress strain, etc.) and saved in ASCII format.
The volume change device, manufactured by ELE International, Limited, reads in milliliters (ml). The capacity of this device is 110 ml. It was used in the drained tests and for measuring volume compressibility following undrained tests.

A water board, with holding tanks and a de-airing device, was used to move water in and out of the triaxial cell. Air pressure and vacuum were used to control movement of water. Distilled water was used in all testing. Cell pressure and back pressure is controlled from the water board.

**Sample Preparation - Static Tests**

This section primarily pertains to sample preparation for the static testing. Each step is explained in detail. It is intended as a reference for those interested in background regarding how to perform triaxial tests on saturated soil samples. For more general information, consult the reference by Mulilis, Chan & Seed (1976). The procedures for obtaining saturation of the sample and measuring permeability worked quite well.
The mean gradation of the Ottawa Sand used is shown in Table D1.

<table>
<thead>
<tr>
<th>TABLE D1</th>
<th>OTTAWA SAND GRADATION</th>
</tr>
</thead>
<tbody>
<tr>
<td>Sieve Size</td>
<td>% Passing By Weight</td>
</tr>
<tr>
<td>No. 16</td>
<td>100</td>
</tr>
<tr>
<td>No. 20</td>
<td>99</td>
</tr>
<tr>
<td>No. 30</td>
<td>94</td>
</tr>
<tr>
<td>No. 40</td>
<td>77</td>
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<tr>
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<tr>
<td>No. 80</td>
<td>18</td>
</tr>
<tr>
<td>No. 100</td>
<td>13</td>
</tr>
<tr>
<td>No. 200</td>
<td>2</td>
</tr>
</tbody>
</table>

The apparent specific gravity of the sand is 2.63. The maximum dry density of the sand was determined to be 105.6 pcf (1.693 gr/cc). Minimum dry density was found at 83.2 pcf (1.334 gr/cc). Determination of the above parameters is according to applicable ASTM standards.

Samples were prepared by pouring dry sand in 5 equal lifts into a membrane jacket placed on the triax cell base. Each was tamped with a tamping rod to level lifts. Membranes were 0.014 inches in thickness. Porous stones used at the top and base of each specimen were 1/4 inch thick and made of corundum. The membranes were held taunt to the membrane jacket with approximately 3 inches of mercury vacuum.
Prior to filling the membrane jacket mold with sand, carbon dioxide gas was bled into the chamber. Following completion of the initial sample preparation and after confining stress was applied, carbon dioxide gas was again bled through the sample at a pressure of about 0.5 psi for 15 minutes.

The purpose of using carbon dioxide gas in sample preparation is to facilitate saturation of the sample. The ability of a gas to be dissolved in water is dependent in part on Henry’s Constant for the gas, Black (1973). Since Henry’s Constant for carbon dioxide is much larger than for air, a much larger amount of carbon dioxide (relative to air) can be dissolved in a given amount of water, at much lower pressures. Consequently, by forming samples in an atmosphere of carbon dioxide, saturation can be achieved at comparatively low back pressures. The maximum back pressure attainable with the equipment on hand was only 76 psi.

Once the mold is filled with sand and leveled with the tamping rod. The top porous stone is placed on the sample. Then the top cap is placed on the porous stone, the membrane rolled up and O-rings put in place. Two O-rings are used to seal the top and bottom of the membrane.
A vacuum of approximately 5 inches of mercury is then applied to the base of the sample with the top water line shut off. The vacuum on the membrane jacket is released and the apparatus removed. The test specimen is now ready for volume measurement.

The circumference (C) is measured with tape at the top (T), middle (M), and base (B). The mean circumference is determined from:

$$C_{\text{mean}} = \frac{1}{4} \left( C_T + 2C_M + C_B \right)$$  \hspace{1cm} (D1)

The initial sample diameter ($D_o$) is determined from:

$$D_o = \frac{C_{\text{mean}}}{\pi} - 2t$$  \hspace{1cm} (D2)

where:  \hspace{0.5cm} t = \hspace{0.5cm} \text{the membrane thickness}

The sample length is measured by the MTS system (to .001 inch). A gauge block was machined and measured with a micrometer and placed in the cell for calibration. As a check, the samples are measured for length on 4 sides and the mean determined.
Once the sample height and diameter is known the initial area and volume are computed and recorded. The sample weight is determined by the difference between the starting and ending weights of the material in a pan. Knowing the weight and volume of the sample the density (\( \gamma_d \)) and relative density are then computed. Relative density \( (D_r) \) is found from:

\[
D_r = 100 \left( \frac{\gamma_{\text{max}}}{\gamma_d} \times \frac{\gamma_d - \gamma_{\text{min}}}{\gamma_{\text{max}} - \gamma_{\text{min}}} \right)
\]  

(D3)

Maintaining the slight vacuum on the sample, the cover is placed and secured on the triax cell. The cell is placed into the load frame and raised until approximately a one pound load is on the top of the sample, as measured by the internal load cell.

At this point the initial sample length \( (L_o) \) is measured, electrical connections to transducers attached, and water lines connected. The cell is then filled with water for confinement. Once the cell is filled with water, the valves are closed and the vacuum removed gradually as a cell pressure of 3 psi is applied.
Carbon dioxide is subsequently bled through the sample until the voids are filled primarily with carbon dioxide. Once this is completed, saturation with deaired water begins. Deaired water is applied through the base, initially at about 3 psi back pressure, while the cell pressure is increased to maintain a 3 psi effective stress on the sample.

Initially the deaired water is allowed to flow through the specimen until no more bubbles are running out. Then the outlet is shut off and the backpressure into the base allowed to stabilize. Once the backpressure is stable, the inlet value is closed and the B-valve is checked.

The procedure for checking the B-valve is as follows:

1. Change (usually increase) the cell pressure by 3-6 psi
2. Measure the resulting change in pore pressure
3. The B-valve is computed as the ratio of the change in pore pressure divided by the change in cell pressure

When the B-valve is greater than 0.96 the sample is assumed to be saturated. On the initial check, the B-valve is usually around 0.5.
After the initial check, the back pressure is adjusted to yield the desired effective confining stress for the test, then the inlet valve opened. Once the back pressure stabilizes, the outlet valve is briefly opened to release any bubbles that may accumulate near the top, then shut off. Back pressure is stabilized to the desired level and the initial shut off and the B-valve is checked again.

The process works well, allowing saturation to be obtained usually within one hour on Ottawa Sands. The maximum air pressure supply in the lab is 80 psi. So the process is jumped up a step at a time (about 5 psi) until the desired B-valve is obtained or maximum pressure is reached in confinement. Care must be taken so as not to allow the internal pore pressure exceed the cell pressure, in which case the sample will fail.

B-parameters may also be checked by decreasing the cell pressure with the valves closed. The computations and results are similar. If the maximum cell pressure in the laboratory is reached before an acceptable B-valve is obtained then decreasing cell pressure is required. However, since gas tends to dissolve in water easier at higher pressures, one should not decrease the confinement by over 10-15 psi.
For this same reason, the length of time that bubbles are allowed to flow out of the outlet should be minimized. If the pressure inside the sample decreases substantially, it was noted that some of the dissolved gas may go back into vapor form.

A computer program was written calling the Testlink analog to digital routine for the purpose of this testing and monitoring the various channels (load, stroke, pressure, etc.) during sample preparation. This program is also used to compute B-valves and measure the permeability of the specimens once saturation is obtained.

The standard equation used to determine the coefficient of permeability ($k$) of a specimen in terms of constant head is:

$$ k = \frac{QL}{AH_c} $$  \hspace{1cm} (D4)

where:

$Q$ = flow (e.g., cc/sec)

$L$ = length of sample

$A$ = area of sample

$H_c$ = constant piezometric head
The volume change device is used to measure volumetric flow and the computer clock to log time from which flow (Q) may be found. The length of the sample is measured by the computer, as discussed earlier. The area of the area of the sample is given upon starting the computer program.

Constant piezometric head is determined from the pressure head at the inlet pore pressure transducer. That is, the pressure divided by the unit weight of water at the laboratory temperature. For ultimate precision the head losses in the system including the pipes, valves and porous stones should be deducted from the pressure head. These losses are, in practice, assumed to be negligible compared to the soil, so are neglected here.

Permeability testing using this equation is based on Darcy's law. The coefficient of permeability is an empirical constant of proportionality to relate flow to the head gradient and area of testing. Darcy's law may not be valid when the flow through the sample voids becomes turbulent. For medium to fines soils where the velocities are low, as they are here, Darcy's law is assumed to be valid.
To perform a permeability test with this system, the inlet back pressure is stabilized and operated through the volume change device. The outlet is opened and the pressure allowed to stabilize momentarily. Pressing the F3 button on the keyboard of the computer logs initial time \( t_1 \), volume change device reading \( V_1 \) and pore pressure \( P_1 \). Once the capacity of the volume change device (110 ml) is nearly all gone through the sample, pressing the F4 key logs the end time \( t_2 \), volume change device reading \( V_2 \), and pore pressure \( P_2 \). The coefficient of permeability is then computed, averaging the pressures, and displayed on the screen using:

\[
K = \frac{2(V_2 - V_1) L \gamma_w}{A (t_2 - t_1) (P_1 + P_2)}
\]

(D5)

Using the triaxial set up and computer monitoring good estimates of the permeability coefficients for various materials were obtained. The system output was checked by standard constant head permeability tests.
Sample Preparation - Cyclic Tests

Ottawa Sand was used in the first two dynamic tests presented in this study. The procedure used for preparing the samples is identical to that in preparing samples for static tests (dry pulveration). B-values of 0.94 or greater were taken as acceptable saturation levels on these larger samples.

With base course samples the plus No. 8 (2.36 mm) sieve material is soaked overnight, then drained and blended with the minus No. 8 sieve for placement in the cell. Base course materials were placed in four equal lifts, then vibrated under a 10 pound weight equal to the sample diameter for thirty seconds per lift. This results in dry densities in the 110 to 120 pcf range.

Membranes used with base course testing were of 0.03 inch thickness. The 0.014 inch thick membranes used in testing Ottawa Sand were found to be insufficient for the larger angular particles in crushed aggregates. Small air leaks result when using the thinner membranes making the samples unusable.

The dense graded base took about 4 to 5 hours to obtain B-values of greater than 0.94. More permeable materials may take as little as 1 to 2 hours to saturate.
The larger triaxial cell used in this testing had a pop-off valve at 35 psi cell pressure. All tests were run at initial effective confining stress of 5 psi unless otherwise noted. Therefore the maximum back water pressure was around 30 psi.

**Computation of Mechanistic Properties**

The shear stress calculated is:

\[ \tau = \frac{\sigma_1' - \sigma_3'}{2} = \frac{\sigma_d}{2} \]  \hspace{1cm} (D6)

The pore pressure ratio is defined as the pore pressure generated in the sample divided by the initial effective confining stress. When the pore pressure ratio becomes equal to or greater than unity, initial liquefaction has been reached. With this study, the pore pressure generated is computed as the average of all readings in a given cycle.

The mathematical dimensions used in computing the various moduli are shown in Figure D1 as line segments. The figure shows actual data from an initial test cycle, with vertical stress starting at zero.
Succeeding cycles started with the stress at point "J" (or "T"). The slopes indicating the cyclic strain and shear moduli were chosen to traverse the full range of material response. Definitions used are:

\[ Cyclic\ Strain\ (Young's)\ Modulus\ (E) = \frac{\sigma_{dc}}{\varepsilon_c} = \frac{A J}{OH} \]  \hspace{1cm} (D7)

\[ Shear\ Modulus\ (G) = \frac{\tau}{\gamma_c} = \frac{\sigma_{dc}}{2(1 + v)\varepsilon_c} = \frac{A J}{2.7\ OH} \]  \hspace{1cm} (D8)

\[ Resilient\ Modulus\ (MR) = \frac{\sigma_d}{\varepsilon_r} = \frac{B J}{F H} \]  \hspace{1cm} (D9)

\[ Compressibility = \frac{\varepsilon_{vol}}{\Delta u} \]  \hspace{1cm} (D10)

Where:
- \( \sigma_{dc} \) = deviator stress at maximum strain
- \( \varepsilon_c \) = cyclic strain on sample
- \( \gamma_c \) = cyclic shear strain on sample
- \( v \) = Poisson’s Ratio
- \( \sigma_d \) = deviator stress
- \( \varepsilon_r \) = recoverable strain per cycle
- \( \varepsilon_{vol} \) = volumetric strain
- \( \Delta u \) = change in pore pressure
An estimate of the damping ratio has been found by computing the area in the shear stress-strain loop, and the area of the triangle. These areas are shown in Figure D2. The damping ratio is then computed using:

\[
Damping\ Ratio\ (DR) = \frac{\text{AREA OPT}}{4\pi \text{AREA RSP}}
\]  \hspace{1cm} (D11)

The area of the loop OPT is determined by numeric integration and the area inside the triangle is found using:

\[
\text{AREA RSP} = \frac{1}{2} \times SR \times FR = \frac{x_{\text{max}}^{2}}{2G}
\]  \hspace{1cm} (D12)

This method of determining the damping ratio is consistent with other procedures, Martin (1975) and Hunt (1986), even though the stress-strain loops are not centered on the axis. This method for determining damping ratios is recommended for comparative studies. If material damping is the emphasis of the research, other methods may be employed, as referred to in Appendix B.
Figure D2  Damping Ratio Determination