Measurement Concepts and Technical Specifications of Surface Analyzer Device
(Project H-104B, Phase I: Tasks 1 and 2)

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Abstract

An instrument for monitoring conditions associated with pavement deterioration has been designed. The conditions being measured are voids or loss of support under a pavement, moisture infiltrating in AC pavement, fine cracking in pavements, delamination of overlays, and aging of asphalt.

These conditions of pavement deterioration are determined by estimating Young’s and shear moduli in the pavement, base, and subgrade from the following wave propagation measurements.

1) Impact Echo
2) Impulse Response
3) SASW
4) Ultrasonic Surface Wave
5) Ultrasonic body wave velocity

The instrument has been designed to make these measurements records high and low-frequency pneumatic hammers on five accelerometers and three geophones over a wide range of distances. Data acquisition, instrument control, and interpretation are computer controlled with measurements and interpretations reported in both screen and database formats. The target hardware costs for duplication of the prototype are anticipated to be approximately $10,000.

In this report, the mechanisms involved in the propagation of different types of distress suitable for maintenance activities are described. The qualitative and quantitative specifications for each distress precursor are developed. The operation of the surface analyzer to determine different types of distress precursors is illustrated. The theoretical aspects and experimental methodology of each testing technique are detailed. The specifications and design of different components of the device are presented.
Executive Summary

This document describes the design of a new project-level measurement device called the Surface Analyzer. The equipment has been designed and will be built considering the needs of a maintenance engineer. The surface analyzer is an effective tool for determining distress precursors in early stages.

Five distress precursors are addressed. These are:

1. moisture in base layer (flexible pavement),
2. voids or loss of support under joints (rigid pavement),
3. overlay delamination,
4. fine cracking, and
5. pavement aging.

To effectively diagnose the specific distress precursors identified, an equivalent number of independent pavement parameters are required. This large number of parameters are measured using an equipment similar to a falling weight deflectometer, but more sophisticated computer processing and interpretation algorithms are used.

The potential savings realized will be tremendous. First, as the precursor of distress is measured, the potential problem can be resolved with preventive maintenance at a fraction of the cost of general maintenance. Second, the device will enable the maintenance engineer to distinguish between maintainable sections and those which require rehabilitation. As such, the available maintenance funds can be directed towards maintainable projects.

The equipment can be used in several modes. The first mode will be to perform more detailed analyses of pavement conditions identified in the network level surveys. The second mode will be
to diagnose specific distress precursors to aid in selection of the maintenance treatment. The third mode will be to monitor pavement conditions after maintenance to determine its effectiveness.

The advantages of the device are several. The surface analyzer provides high accuracy and precision in determining the state of the pavement. All methodologies utilized are based upon sound theoretical background. Field testing and data reduction methodologies utilized are compatible with the theoretical assumptions. The hardware associated with the device is relatively inexpensive. Further modifications of the device are cheap because only the software has to be updated and replaced.

The principle of operation of the device is based upon the generation and detection of stress waves in a layered medium. Five testing techniques are combined. Each test and its strengths are summarized in Table 1. The design and construction of the surface analyzer is based upon two general principles. First, the strength of each method should be fully utilized; and second, enough redundancy should be provided so that the layer which will potentially contribute to the distress of pavement can be identified.

<table>
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<th>Testing Technique</th>
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<td>Young's Modulus of top paving layer</td>
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<td>Ultrasonic Surface Wave</td>
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<tr>
<td>Impulse Response</td>
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<tr>
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<td>Modulus of each layer; Thickness of each layer; and Variation in modulus within each layer</td>
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<tr>
<td>Impact Echo</td>
<td>Thickness of paving layer or depth to delaminated layer</td>
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Through an analytical study, the levels of accuracy required for measuring different distress precursors were established. A sensitivity analysis and a perturbational study were carried out to determine realistic levels of measurement accuracy and precision of different testing techniques. It has been shown that in all cases, the proposed techniques have adequate levels of accuracy to identify precursors of distress.
The surface analyzer can be effectively used to enhance the results of many other SHRP projects. As the device yields information that is inherently more accurate than the falling weight deflectometer, many of the LTPP projects can utilize the equipment for better diagnosis and more effective evaluation purposes.
1

Introduction

Problem Statement

In recent years, the focus of pavement engineering has shifted from design and construction of new highways to preventive maintenance and rehabilitation. Maintenance schedule is usually based upon visual condition survey and to a lesser extent based on appropriate in situ tests. When the manifestation of deterioration is visible, major rehabilitation or reconstruction is required. If the onset of deterioration can be measured accurately in the early stages, the problem can be resolved or stabilized through a preventive maintenance schedule.

SHRP has identified six broad elements which are the cause of, and contribute to, the initiation of pavement deterioration. These six parameters are: 1) pavement moisture, 2) fine cracking, 3) subsurface problems or discontinuities, 4) voids or loss of support under rigid pavements, 5) overlay delamination, and 6) asphalt aging.

The University of Texas at El Paso (UTEP), and its consultants, have proposed the development of an inexpensive and precise device for project level measurements. The device will address all the six items above, except subsurface problems and discontinuities and the potential for stripping of the AC layer.
Objectives

The main objective of this project is to develop an inexpensive and precise device for project level studies. The device has been named the Surface Analyzer. To be effective in maintenance measurements, the device should have four major features. First, the device should be sensitive enough so that the contributing factor to a potential distress can be measured "soon enough". Second, the measurements should be accurate and comprehensive enough so that the layer contributing to a potential distress can be identified. Third, the device should be precise enough so that the effectiveness of maintenance processes can be verified. Finally, much time and effort can be saved if the device can discriminate between a rehabilitation activity and a maintenance activity.

The Surface Analyzer is a non-destructive testing (NDT) device, six to ten feet long. The principle of operation of the device is based upon the generation and detection of stress waves in a layered medium. Several seismic testing techniques are combined. These seismic methods are: 1) Ultrasonic Body Wave, 2) Ultrasonic Surface Wave, 3) Impulse Response, 4) Spectral Analysis of Surface Waves (SASW), and 5) Impact Echo.

The design and construction of the surface analyzer are based upon two general principles. First, the strengths of each of the five methods should be fully utilized. Second, enough redundancy should be provided so that the layer which will potentially contribute to the distress of pavement can be identified (see Chapter 4).

Organization

This report contains three major sections. In the first section, Chapters 2 and 3, the pavement conditions to be addressed and their relevant quantitative descriptions in terms of measurement ranges, accuracy, and precision are introduced. In the second section, Chapters 4 through 6, the conceptual approach to detecting the pavement conditions described in the previous sections is addressed. Finally, the design of mechanical, electronic, and software components are detailed in Chapters 7 through 9. Several appendices provide theoretical background on major topics described in the report.
Description of Pavement Conditions

The definition of maintenance as used in this discussion is an activity used to correct a localized area of deterioration, to preserve the existing pavement, and to reduce the rate of deterioration. In general, these treatments do not increase the structural or traffic handling capacity of the roadway. Treatments that generally are considered maintenance are included in Table 2.1.

Each of these activities is used to address specific problems in the pavement structure. To determine when to apply a maintenance treatment and which treatment is appropriate, the maintenance engineer basically tries to address several questions. Does this section of pavement need a treatment now? If not, will it need one in the near future (less than three years)? Is the problem being addressed localized, or does it cover a large area? Which treatment should be applied? Will the treatment provide an effective return on the funds expended?

Each pavement condition, the basic life extension mechanism, and the methods the maintenance engineer normally uses to determine whether and when to apply them are addressed in this chapter.

Moisture in Foundation

Types of distress to be expected from moisture related problems in the foundation layers (in advance stages) are summarized by Carpenter, et al (1981). Typically, the softening of one
or more of the foundation layers, and degradation of material quality in terms of stiffness and strength are the initial manifestation of excess moisture within the pavement system. Field studies (Cedergren, 1974) have shown that wheel loads on saturated sections are many times more damaging than those on dry sections.

Table 2.1 - Treatments that are considered maintenance

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<td>Patching</td>
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<td>Joint Repair</td>
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<td>Surface Sealing</td>
<td>Crack and Joint Sealing</td>
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<tr>
<td>(all types)</td>
<td>Undersealing</td>
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</table>

**Moisture under Joints**

The mechanism of exposure of foundation layers to moisture in a rigid pavement is similar to that of a flexible pavement. Types of moisture-related distress to be expected for rigid pavements are also summarized by Carpenter, et al (1981). Due to a rise in the water table under a slab, foundation materials will deteriorate. As a result, the foundation layer will become softer or in more severe cases, a void will develop under the concrete slab. Slab curling (a.k.a. bending or warping) will contribute to the progress of this problem.

**Voids or Loss of Support**

The presence of voids or loss of support underneath a slab is detrimental because of increase in stresses and the subsequent reduction in the fatigue life of the pavement. Factors of importance in this process are discussed by Torres and McCullough (1983). The larger the size of the void or the thinner the slab, the lower the support and the lesser is the life of the pavement.

**Overlay Delamination**

The process and significance of overlay delamination is well-known. The degree of interfacial bonding influences the state of stress within the overlay. Interfacial bonding has
been singled out as the most significant factor that substantially affects overlay performance (Ameri-Gaznon and Little, 1988). In the case of delamination, the overlay acts independently of the rest of the pavement system. This allows excessive movement at the bottom of the overlay relative to the top where the wheel load is in contact with the pavement.

**Fine Cracking**

Cracks often begin as hairline cracks which allow little water into the structure. Although they are a discontinuity in the pavement structure, they are not generally a problem until they become wide enough to allow water to enter into the structure. If they are allowed to deteriorate, they become wider and develop spalling at the surface allowing much more water to enter. The intrusion of incompressibles during the cold period creates high compressive forces on the crack or joint face creating spalling during warmer periods. Water in the crack will carry dissolved oxygen into the asphalt at the crack face crating an aged asphalt on the crack face accelerating crack deterioration.

**Pavement Aging**

The aging process in the field is complex. Several independent investigations (Tia et al, 1988; Von Quintas et al, 1988; Goodrich, 1988) indicate that the modulus of the asphalt increases with time (aging). Aging should be considered in two stages: 1) short term and 2) long term (Bell, 1989). The short term aging occurs during construction, while the mix is hot. The long-term aging occurs while the mixture is in place. The short term aging is mainly due to loss of volatile components in asphalt. The long term aging is mostly attributed to progressive oxidation of the in-place material in the field.

**Maintenance Activities**

**Patching**

Every type of possible deterioration is addressed with patching if the damage is localized. The basic life extension mechanism provided by patching is the repair of localized areas of low strength or other types of deterioration. This can be in any pavement layer which will lead to a localized failure observable on the surface. The loss of strength can be from a change in the material properties or it can be due to localized differences in construction and original materials properties. It can also be due to loss of support from supporting layers due to erosion or degradation.

Maintenance engineers generally do not test to determine if patching is required. They begin
applying patches when the deterioration affects the pavement surface to the point that some
repair is needed. The key question that a maintenance engineer needs to address with testing
is whether the patching will effectively address the problem. This generally is determined
based on whether the deterioration is localized or whether the problem is widespread
requiring a more comprehensive rehabilitation treatment to give an effective return on the
funds spent.

**Crack and Joint Sealing**

Many of the materials used in pavement construction have moisture sensitive stiffnesses. As
the moisture content of unbound granular materials and soils increases, the stiffness
decreases. Moisture leads to the degradation of asphalt concrete due to stripping, aging,
weathering, and ravelling. Free water under portland cement slabs can develop very high
pressures resulting in the erosion of base and subbase materials or pumping and loss of
support. Crack and joint sealing are applied to reduce the influx of moisture into the
pavement structure from the surface.

The maintenance engineer generally will use the observable condition of the crack and joints
to determine if crack and joint sealing is appropriate. Many maintenance engineers will not
seal a crack until it is greater than one-eighth to one-quarter inch wide. If the amount of
weakening due to presence of moisture at joints and cracks could be determined, that
information could be used to determine when crack and joint sealing is needed to reduce the
infiltration of moisture.

**Surface Seals**

Surface seals are generally used to extend the life of pavements by improving the surface
friction of the pavement; reducing weathering and ravelling; or reducing the infiltration
of moisture into the pavement structure. Surface friction is not considered in this project.
Weathering and ravelling are basically the result of asphalt concrete aging.
The maintenance engineer normally looks for signs of weathering and ravelling or the
presence of a network of fine cracks which can be sealed with the surface seal. If the
presence and level of aging could be determined, the degradation of asphalt due to aging
could be prevented or reduced. If the degradation of paving materials due to presence of
abnormal moisture levels in the asphalt and supporting layers or the presence of fine cracking
could be determined, the need to place a seal to reduce infiltration of water into the structure
could be evaluated.
Undersealing

Undersealing is the process of filling voids under portland cement concrete pavements by reestablishing support under the slab. The movement and loss of fines creates voids, normally on the leave side of the joint or crack. This leads to faulting of the joint or crack. The loss of support also increases the stress in the portland cement concrete pavement near the corners leading to corner breaks.

In some cases, the base material is degraded but not ejected. This can create a thin layer of very soft material under the joint. A loss of support is then present without a true void. In such a case, the undersealing generally cannot displace the deteriorated materials sufficiently to reestablish full support.

The maintenance engineer normally looks for the presence of pumping, faulting, and corner breaks to determine that voids are present and to determine if undersealing is appropriate. Measurements to determine the loss of support, the presence of voids, and the size of voids are needed to adequately determine if the voids are developing and if undersealing should be considered to reestablish support.
Development of Specifications for Measurement

Currently, maintenance engineers decide when to apply maintenance and what type of maintenance to apply based on the surface condition and their experience. Corrective maintenance is applied to repair damage that has occurred. Preventive maintenance is applied to retard the development of damage or to reduce the rate of damage development. Some damage will have developed, but the amount of damage must be small enough that the type of treatments used in preventive maintenance can be effective in retarding further deterioration or reducing the rate of deterioration. Corrective maintenance may be used in a few localized locations as a part of the preventive maintenance treatment. The purpose of this project is to develop equipment which can identify damage developing in the pavement at a stage. This should assist the maintenance engineer in determining the level, amount and extent of damage present. This should also assist him/her in effectively timing and selecting the preventive maintenance techniques.

The precursors of damage which this project plans to measure have not been routinely employed by maintenance engineers in selecting the treatment or the timing of the treatment. Maintenance engineers have been forced to use the visible distress along with pavement age and traffic levels to determine the type and timing of preventive maintenance. Information on the moisture in the pavement layers, at joints, etc. has not been available, in part due to the difficulty of measuring the damage that this project proposes to measure.

This chapter shows how early changes in pavement properties, before significant damage is readily apparent on the pavement surface, will affect the rate of deterioration and the
pavement life. This information is then used to determine the level of early change in selected pavement properties, those this project proposes to measure, which will have a significant impact of pavement performance, such as fatigue cracking. This defines the level of change in pavement properties which produces a signal that damage is beginning to occur. The cracking, change in ultimate life, and rate of deterioration are the measure of damage that is of importance to those who fund pavement maintenance. These changes occur early in the life of the pavement, and they by themselves would not generally indicate a need for maintenance. However, if these early changes lead to an increased rate of deterioration, distress will develop more quickly and the ultimate life of the pavement will be decreased. Changes explained in terms of decreased life or increased damage are necessary to justify allocation of funds for preventive maintenance. The importance of early distress precursors is the increase in cracking, change in pavement life or the rate of deterioration that they create.

Pavement conditions change by location and over time. Some of these changes are only transient in nature. The equipment proposed in this project will operate in a fashion similar to that used currently for a falling weight deflectometer and the operation costs will also be similar. The equipment may be used in routine surveys; however, it is more likely to be used in project-level surveys. These projects will have been selected for survey because a radar or other similar equipment has indicated a problem, or because a maintenance supervisor believes that maintenance may be needed. The device will then be used to determine what kind of early changes are occurring in the pavement characteristics so that decisions can be made concerning what kind of maintenance is appropriate and the time to apply it. The type of information the maintenance engineer will be seeking will include the following:

1. is the stiffness less (moisture content higher) at cracks and/or joints - which would lead to a decisions about sealing the cracks and joints,

2. is the stiffness (moisture content) significantly different along the pavement, or has it significantly changed since the last measurement (adjusted for seasonal variation) - which would assist in decisions to apply surface seals, and

3. is the damage (e.g. delamination) localized or widespread and has the amount changed significantly since the last measure - which would lead to decisions to continue repair by patching or to program major rehabilitation.

These are the type of information that this project seeks to provide the maintenance engineer. It will provide much more accurate information on the level of damage and the types of change occurring in the pavement properties at much earlier time that previously available. However, there will still be a need for the maintenance engineer to interpret how these
indicators are expected to affect pavement performance and to make decisions about what maintenance activities to apply and when. This will give the maintenance engineer a tool to assist him/her in better diagnosing the cause of the deterioration. However, he/she will not spend limited resources on the pavement until he/she sees some level surface deterioration. This equipment will help him/her to determine the cause of the observed deterioration and select better preventive maintenance programs.

The remaining life of a pavement is controlled by the complex interaction among several factors such as traffic, pavement structure, drainage, road geometry, climate, economy, etc. In recent years several algorithms have been developed which can predict the type and rate of deterioration, and which can suggest alternative maintenance strategies at appropriate time intervals. A good example is the Texas Flexible Pavement System (TFPS) (Uzan and Smith, 1988 and Rhode, et al, 1990). In ideal conditions, one can strictly adhere to these "theoretical" maintenance schedules. However, in many cases, pavements experience distress prematurely or the maintenance activities are not effective. A process is suggested here to define the levels of accuracy and precision required for a maintenance measuring equipment.

Moisture in Foundation Layers (ACP)

Moisture in the foundation and cracking are considered together here because of the strong interaction that exist between them. In the initial stages, a new pavement is in a desirable condition. The paving layer is usually impervious and cracks are scarce or nonexistent. In this stage, most of the damage to the pavement is due to traffic or environment, and moisture infiltrates either from the shoulders or from the water table. As soon as cracks are developed, moisture may penetrate from the surface. If the surface layer is primarily a wearing course and the majority of structure is in the base, the infiltration of moisture or the existence of cracks are not of great concern if the base and subgrade materials do not lose their integrity due to exposure to moisture.

Factors that should be considered here (excluding political and economic factors) are the climatic parameters, such as the amount and seasonal distribution of rainfall, the drainage properties of the base and subgrade materials, structural properties of the AC, base, subgrade and their seasonal variations, and the nature and seasonal distribution of traffic (Markow, 1982). TFPS considers the effects of these parameters as well as their interaction in a comprehensive fashion. The amount and seasonal distribution of the rainfall are modelled utilizing the historical data from each county in the State of Texas. Based upon these climatic models, the properties of each layer are modified and updated with time.

The parameters that are considered include moisture, temperature and distress type. The program does not consider the transient and dynamic nature of change in moisture or
modulus with time. Considering the most basic principles of geotechnical engineering, the transient and dynamic nature of change in moisture is of little practical use in predicting maintenance life. A material that becomes saturated and unsaturated over a short time period is a well-drained material and has a high permeability. The strength and stiffness of such a material (and as a result the remaining life of a pavement constructed with or over such a material) is not significantly affected by change in moisture. On the other hand, for a material which does not exhibit large fluctuation in moisture over short periods (i.e. a material with a low permeability) the change in equilibrium base moisture may significantly affect its modulus and remaining life.

A shortcoming of TFPS, as applied to maintenance problems, is that it does not model accumulation of damage due to change in equilibrium base moisture/modulus. To implicitly model the accumulation of damage with time, a simplified procedure was utilized. The process is described below.

**Conceptual Approach**

A conceptual model depicting the variation in cracking with traffic is shown in Figure 3.1. Depending on the severity of cracks, four levels of maintenance or rehabilitation activities are envisioned. Within Level 1, the pavement is in satisfactory condition and no maintenance is needed other than occasional localized maintenance. In Level 2, the pavement section is in the condition in which preventive maintenance is appropriate (i.e. surface sealing and crack sealing). In more severe cases, Level 3, overall or general maintenance or rehabilitation activity should be carried out (i.e. thin overlay or localized repair with surface sealing). Finally, in Level 4, the pavement has to be rehabilitated or reconstructed.

For effective preventive maintenance, the damage must be identified early. An appropriate treatment should then be selected which will arrest the deterioration. Measurements made with the surface analyzer will be used to determine the present level of deterioration of the pavement and the time when the pavement would be expected to advance to the next level. In this section, the impact of early changes in pavement properties which can lead to a significant change in the life of a pavement, and which would justify maintenance expenditures are discussed. It is assumed that changes occur in the "no maintenance" level. The change in the development of damage, rate of deterioration and the life of pavement can then be determined. The same exercise can be carried out at other levels of deterioration. However, the degrees of accuracy obtained will be larger.

Computer models such as TFPS consider the long-term trend of seasonal variation of modulus and moisture content on the propagation of cracks. However, these models fail to consider the effects of reduction in moduli above and beyond those anticipated due to
Figure 3.1 - Conceptual variation in distress with traffic
seasonal variations. One way of resolving this matter is to develop a sophisticated conceptual model considering to the fullest detail the interaction of parameters considered above. Markow (1982) and Liu and Lytton (1985) have developed such models. However, such models require many assumptions describing the inter-relation between different values. We approached the problem from a different aspect.

Shown in Figure 3.2 are the relationships between cracking and the number of ESAL for two different moduli (\(M_1\) and \(M_2\)) of base. Let us assume that at a given number of ESAL, \(n_i\), the modulus decreases from \(M_1\) to \(M_2\). As indicated before, \(n_i\) is located in the "no maintenance" zone of Figure 3.1. From point \(n_i\) on, the remaining life curve will follow a conceptual curve \(M_{21}\). With the TFPS, this curve cannot be developed. However, the upper and lower bounds can be reasonably defined. First, it is intuitive that the curve has to be bound between the \(M_1\) and \(M_2\) curves. Secondly, the slope of the curve at any given time should be bound between the slope of curves \(M_{2u}\) and \(M_{2l}\). Curve \(M_{2u}\) is determined by a linear transformation of curve \(M_2\) in the Y-direction equal to \(\Delta M [M_2(n_i) - M_1(n_i)]\). This implies that the progression of distress is more influenced by time (traffic). Curve \(M_{2l}\) corresponds to a linear transformation of curve \(M_2\) in the X-direction equal to \(\Delta n (n_1' - n_1)\) which can be interpreted as the progress of distress being more influenced by the extent of cracks. This procedure is adequate for small levels of distress (such as those used in this study). For extensive cracking, the model may yield erroneous results. We will assume that curve \(M_{21}\) is located half-way between Curves \(M_{2u}\) and \(M_{2l}\).

Two types of measurement should be considered -- relative and absolute. In relative tests, the pavement is tested shortly after the completion of the project and then periodically retested. Let us assume that at a given time a reduction in modulus from \(M_1\) to \(M_2\) is detected. If the differences between Curves \(M_1\) and \(M_{21}\) (see Figure 3.2) are not significant, one can conclude that the change in modulus from \(M_1\) to \(M_2\) is not of concern. With such a small variation in modulus, if the precursor of stress is not detected in a timely manner, the remaining life of pavement is only slightly affected. Conversely, if the change in modulus from \(M_1\) to \(M_2\) results in a large deviation between curves \(M_1\) and \(M_{21}\), the process of maintenance has to be scheduled earlier. This is the preferred approach for monitoring new pavements. The same line of reasoning can be also used to determine the effectiveness of a maintenance activity. The only difference is that significant increase in or stabilization of modulus values with time is of interest.

For absolute measurements, the pavement is tested at a given point for the first time. The only information available is probably the traffic history and the pavement structure. In this case, the information should be utilized to approximately determine the level of integrity of the pavement. The disadvantage of this method at this time is that a precise model may not be available.
Figure 3.2 - Conceptual development of accumulative damage model for softened base layer
Practical Approach

In this section, we attempt to establish the level of accuracy necessary for measuring moduli of base for maintenance purposes. The modulus of the base and its change with time can be measured with a finite accuracy. Therefore, it is given that the change in the distress precursor below a certain level cannot be detected. We will investigate the impact that such a level will have on the remaining life of pavement.

Given the conceptual approach described above, many qualitative levels have to be defined. In the following section, the upper limits of the "no maintenance", "preventive maintenance", and "overall maintenance" (shown in Figure 3.1) are arbitrarily set at fatigue cracking levels of 2 percent, 5 percent and 20 percent. In practice, for a highway agency, these levels should be selected by a panel of its experienced maintenance engineers in charge of developing maintenance policies. The factors to be considered are the overall state of deterioration of the highways, and available funds. In this manner, implementable standards can be developed and periodically updated. This matter will be further discussed later in this chapter.

The percent cracking as a function of number of ESAL determined by program TFPS is shown in Figure 3.3 for a typical pavement. The pavement consists of five in. of AC layer with a modulus of 500 ksi (at 70°F), twelve in. of base with a modulus of 30 ksi over a granular subgrade with a modulus of 10 ksi. This pavement section will be called the control pavement in the rest of this section. The pavement is assumed to be in Beaumont, TX, corresponding to a wet climate with high water table. The 20-year design traffic was assumed to be 5 million ESAL. The same curve is shown for identical pavement in an extremely different climatic condition -- El Paso, TX (water table over 400 ft deep, average annual rainfall of 9 in.). The pavement in Beaumont experiences distress faster than that in El Paso. This can be attributed to damaging factors associated with the interaction between the extent of cracks and the level of precipitation. The traffic and pavement structure were identical for both climatic regions.

Variation in fatigue cracking as a function of number of ESAL for the control section in Beaumont is shown again in Figure 3.4. Also shown in this figure are curves for the cases when the modulus of base is initially 10 to 50 percent less than the control section. Using the procedure depicted in Figure 3.2, Figure 3.4 can be modified to develop damage curves for the situation when the modulus of base is reduced by 10 to 40 percent due to change in the equilibrium moisture at a given time. We arbitrarily and conservatively assume that the change in equilibrium moisture (and as a result the modulus) occurs as soon as the percent cracking of the control pavement section deviates from zero. This point is clearly marked on the figure. In practice, this is set by regular testing of pavement.
Figure 3.3 - Variation in fatigue cracking with traffic for a typical AC pavement
Figure 3.4 - Accumulative damage model for a typical AC pavement with softened base layer
In the next step, the degree of accuracy at which the change in modulus has to be measured is determined. We assume that the maintenance engineer would like to know that within the following three years:

1. If any type of maintenance is necessary or not?
2. If maintenance necessary, is it localized or overall?

Shown in Table 3.1 is the degree of accuracy with which modulus of base has to be measured given an acceptable level of cracking and number of years from the date measurements have been carried out. These values are obtained from Figure 3.4. Shown on the figure is the point when a reduction in modulus occurs. Also shown are three other vertical lines marked Year 1, Year 2 and Year 3. These three lines correspond to the three years of scheduling mentioned above. To obtain the values shown in Table 3.1, the intersection of each of these time lines with the appropriate percent cracking was determined. As an example, the point corresponding to an acceptable level of cracking of 10 percent and at year 2 is marked on Figure 3.4. This value is roughly equal to 45 percent.

Table 3.1 - Degree of accuracy necessary for detecting moisture in base

<table>
<thead>
<tr>
<th>Acceptable Fatigue Cracking (Percent)</th>
<th>Required Accuracy in Measuring Base Modulus (Percent)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Year 1</td>
</tr>
<tr>
<td>5</td>
<td>&gt;50</td>
</tr>
<tr>
<td>10</td>
<td>&gt;50</td>
</tr>
<tr>
<td>15</td>
<td>&gt;50</td>
</tr>
<tr>
<td>20</td>
<td>&gt;50</td>
</tr>
</tbody>
</table>

Practically speaking, it can be seen that for a typical pavement an accuracy better than 25 to 30 percent is quite sufficient. It will be shown later that this accuracy is achievable with the Surface Analyzer. The same procedure was followed for the El Paso data. Even though not shown, it was found that the levels of accuracy shown above are applicable.

As indicated above, in practice, the three levels of maintenance as a function of percent acceptable distress should be set. Due to wide ranges of pavement structures, materials, climatic regions, etc., the development of rigid guidelines would be impractical. The availability of funds may be the most important factor in determining the percentages of roads to be maintained. A simple algorithm for determining the optimum levels of accuracy and the frequency of measurement was proposed here. The advantage of this algorithm is
that for each climatic region, for each acceptable level of maintenance, for each pavement structure and for each economically feasible level of deferred maintenance, the required level of measurement accuracy and measurement frequency can be determined. Once again, the surface analyzer will provide much more accurate information on the level of damage and the types of change occurring in the pavement properties at much earlier time that previously available. However, there will still be a need for the maintenance engineer to interpret how these indicators are expected to affect pavement performance and to make decisions about what maintenance activities to apply and when.

**Void or Loss of Support (PCC)**

The mechanism involved in loss of support at the edge of a PCC pavement is experimentally described by Dempsey (1982) and detailed theoretically by Raad (1982). The LCPC (1979) recommend that the interaction among traffic, climate, subbase materials and joint load transfer should be considered. They represented these values in a three-dimensional space to provide a qualitative model for determining the interactive parameters that should be considered. The loss of support in a PCC pavement is initiated by the penetration of water to the interface of the slab and an erodible subgrade. The accumulated moisture along with the dynamic nature of the traffic generates high pore water pressures. Excess pore pressure initiates the collapse of the original soil matrix, analogous to liquefaction in soils. The collapse and softening of the subgrade initiates excessive deflections at the edge of the slab. Excessive deflections along with presence of water contribute to the ejection of water at high velocities. Excessive water velocities result in pumping which can be translated to progressive loss of support. As the loss of support progresses, tensile stresses and strains at the bottom of the slab increases. The increase in the stresses contributes to the increase in cracking of slab and eventually faulting.

Based on this model, the critical parameter is the horizontal extent of loss of support. To model this progressive damage model, the increase in tensile stresses as a function of loss of support was determined. These stresses were adapted from the work of Darter (1977). He utilized a finite element program to determine tensile stresses as a function of extent of loss of support and the thermal gradient. In the next step, the fatigue life of the pavement was estimated. Smith, et al (1990) suggest that the fatigue life, N, can be determined from:

$$\log_{10} N = 2.13 \left(1/\text{SR}\right)^{1.2}, \tag{3.1}$$

Parameter SR is the stress ratio and is defined as the ratio of the tensile stress measured to 28-day modulus of rupture. Furthermore, they proposed that the percent cracking, P, as a function of traffic, n, can be conservatively determined from:
\[ P = \frac{1}{0.01 + 0.03 \times [20^{-\log (\frac{1}{V})}]} \]  

(3.2)

As an example, the variation in tensile stress at the edge of a slab with the extent of loss of support is shown in Figure 3.5. The slab is a 15 ft X 12 ft X 10 in. overlying a soil with a modulus of subgrade reaction of 200 pci. A thermal gradient of 1°F/in. is also assumed. Based on these stresses, the percent cracking of the slab as a function of traffic and extent of loss of support is determined and shown in Figure 3.6. From Equation 3.1, a 20-year design ESAL of 5 million was determined.

Once again, the simplified damage accumulation methodology described above was utilized to develop deviation in behavior as a function of cracking. As a significant change in the rate of cracking for the control section exists, it was arbitrarily assumed that the transition between good contact and loss of support occurs at a two percent cracking level. However, it is understood that the transition is rather gradual in the field. Such a curve is shown in Figure 3.7.

From this figure, the levels of accuracy were determined again. A level of accuracy corresponding to 24 in. is selected herein.

Table 3.2 - Degree of accuracy necessary for detecting loss of support

<table>
<thead>
<tr>
<th>Acceptable Cracking (Percent)</th>
<th>Required Accuracy in Measuring Loss of Support (in.)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Year 1</td>
</tr>
<tr>
<td>------------------------------</td>
<td>--------</td>
</tr>
<tr>
<td>5</td>
<td>24</td>
</tr>
<tr>
<td>10</td>
<td>&gt;24</td>
</tr>
<tr>
<td>15</td>
<td>&gt;24</td>
</tr>
<tr>
<td>20</td>
<td>&gt;24</td>
</tr>
</tbody>
</table>

Foundation Softening (PCC)

Another possible precursor of distress for the concrete slabs is the softening of the subgrade layer without any actual void development. The same methodology used to model the loss of support was used to model this problem as well. The only difference is that the increase in tensile stress as a function of reduction in the modulus of subgrade reaction was determined. Once again, it was assumed that the reduction in modulus will occur when two percent of the slab is cracked. This level can be reduced or increased without affecting the generality of the conclusions. A graph of percent cracking versus traffic using the simple accumulative
Figure 3.5 - Variation in tensile stress with loss of support for a typical PCC pavement (after Darter, 1977)
Figure 3.6 - Variation in cracking with traffic for a typical PCC pavement experiencing loss of support
Figure 3.7 - Accumulative damage model for loss of support under a typical PCC slab
damage model is presented in Figure 3.8. Based on this relationship, acceptable levels of accuracy were once again developed. This relationship is shown in Table 3.3. Basically, in most cases a level of accuracy of better than 30 to 40 percent is required.

Table 3.3 - Degree of accuracy necessary for detecting foundation softening

<table>
<thead>
<tr>
<th>Acceptable Cracking (Percent)</th>
<th>Required Accuracy in Measuring Modulus of Subgrade (percent)</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Year 1</td>
</tr>
<tr>
<td>5</td>
<td>40</td>
</tr>
<tr>
<td>10</td>
<td>&gt;40</td>
</tr>
<tr>
<td>15</td>
<td>&gt;40</td>
</tr>
<tr>
<td>20</td>
<td>&gt;40</td>
</tr>
</tbody>
</table>

Delamination of Overlays

In this case, the actual damage is detected. Currently-used maintenance activities are not appropriate for correcting these defects, unless the delamination occurs over very limited areas. Even then, only those areas which breaks loose are repaired with patching. Therefore, from a maintenance point of view, the value of detecting this type of distress precursor is limited. The primary concern is knowing the extent of delamination. If the delamination is extensive, the maintenance personnel will program rehabilitation rather than trying to address the problem with patching. This will assist the engineer in charge of maintenance to more effectively utilize his budget.

Fine Cracking

The reduction in remaining life of pavement as function of reduction in the effective stiffness of a cracked paving layer is of little concern. The fact that the cracks would allow water to penetrate more readily into other layers and accelerate the rate of damage is quite important. This aspect of damage due to cracks was discussed in the previous section.

Aging of Asphalt Layer

As indicated before, the process of aging is the early stages is not well understood. Several research studies are presently being carried out to develop this process. Most notable of these projects is the SHRP Project A-003 pursued at the Oregon State University.
Figure 3.8 - Accumulative damage model for subgrade softening under a typical PCC slab
Surface Analyzer Overview

In order to effectively measure pavement conditions that serve as precursors to distress described in previous chapters, a large number of pavement properties must be measured. As the distress precursors identified in previous sections are not directly measurable physical properties, several physical property measurements must be made to effectively diagnose the precursors.

This chapter gives an overview of the measurements and their interpretation for diagnosing distress precursors. Three subsequent sections deal with 1) measurement procedures, 2) data analysis techniques, and 3) data interpretation techniques that serve as an introductory descriptions of the three chapters that follow.

Measurement Procedure

Measurements for diagnosis of distress precursors are based on measuring mechanical properties and thicknesses for each of the pavement system layers. This is accomplished by lowering a frame carrying acoustic transducers and sources to the pavement and digitally recording surface deformations induced by a large pneumatic hammer, generating low-frequency vibrations, and a smaller pneumatic hammer, generating high frequency vibrations.

This transducer mounting frame will be mounted on a trailer that may be towed behind a vehicle, and is similar in size and concept to the Falling Weight Deflectometer. The Surface Analyzer differs from the FWD in that more and higher frequency transducers are used, and more sophisticated interpretation techniques are applied to the higher mode vibrations induced in the pavement.
The Surface Analyzer will be controlled, by the operator, with a portable computer connected to the Surface Analyzer by a cable. This computer may thus be run from the cab of the truck towing the Surface Analyzer, or from various locations around the Surface Analyzer. The type of computer depends on the type of data collection: routine data collection requires a minimal computer, interactive collection and analysis will require a more capable computer.

All measurements are spot measurements; that is, the device has to be towed and situated at a specific point before measurements can be made. Non-technical factors affecting the performance of the surface analyzer are summarized in Table 4.1. Basically, a complete testing cycle at one point will take about one minute. A complete testing cycle includes situating at the site, lowering the source/receiver assembly, making measurements and withdrawing the equipment. Measurements with the surface analyzer should be faster than those with a falling weight deflectometer (FWD). With the Surface Analyzer, less time is required to impart loads to the pavement surface.

To operate the device safely, traffic control is required. The level of traffic control necessary is equivalent to that provided for an FWD. The level of skill of the operator depends on the mode of operation of the device. Two major levels of operation will be provided with the surface analyzer, operation mode and research mode. For the operation mode, a conscientious technician with a degree from high school or a two-year technical college is needed. It is estimated that one or two weeks of training through video tape and assistance of a maintenance engineer is also necessary. In the research mode, a research engineer with background in pavements and wave propagation is desirable.

Effective frequency of measurement with the device is dependent on the intended use. For routine maintenance, a procedure similar to that of the FWD can be used. However, for high-precision diagnostics, tests should be carried out every 1 ft to 10 ft depending upon the nature of distress. Should the device be used for research purposes, the frequency of measurement should be judged by the operator.

An extensive field study has been carried out to determine the effects of temperature on the results of different tests. Based upon this study it was concluded that testing of rigid pavements at ambient temperature in excess of 95 degrees is not feasible. For flexible pavements, temperature should not exceed 120 degrees. At such high temperature, the AC layer is too viscous and coupling of energy to it is difficult. To minimize the effects of fluctuation in the moisture level due to precipitation, it is recommended that the equipment not be utilized two days after a significant precipitation. As discussed before, the short-term transient change in moisture is not of any practical interest in maintenance activities.
The cost of operation of the device is estimated as 20 cents per point plus $10 per hour. These numbers were estimated based upon the cost of operation of FWD reported by one of the states.

Table 4.1 - Non-technical factors affecting the performance of surface analyzer

<table>
<thead>
<tr>
<th>Item</th>
<th>Remarks</th>
</tr>
</thead>
<tbody>
<tr>
<td>Measurement Speed</td>
<td>one minute per point</td>
</tr>
<tr>
<td>Traffic Control Required</td>
<td>yes, similar to that used for FWD testing</td>
</tr>
<tr>
<td>Skill Level of Operator</td>
<td>Operation Mode: A conscientious technician</td>
</tr>
<tr>
<td></td>
<td>Research Mode: A research engineer</td>
</tr>
<tr>
<td>Frequency of Measurement</td>
<td>Routine Maintenance: Similar to the procedure used with FWD</td>
</tr>
<tr>
<td></td>
<td>Diagnostics: Every 1 to 6 ft depending on the project</td>
</tr>
<tr>
<td></td>
<td>Research: Determined case by case</td>
</tr>
<tr>
<td>Necessary Ambient Condition</td>
<td>Concrete: Ambient Temperature not to exceed 85°F</td>
</tr>
<tr>
<td></td>
<td>Asphalt: Ambient Temperature not to exceed 120°F</td>
</tr>
<tr>
<td>Initial Cost per Device</td>
<td>Phase II: About $15,000</td>
</tr>
<tr>
<td></td>
<td>Phase III: About $10,000</td>
</tr>
<tr>
<td>Operating Cost per Measurement</td>
<td>20 cents per point Plus $10 per hour</td>
</tr>
</tbody>
</table>

Data Analysis

Three types or levels of data are collected by the Surface Analyzer. The first level is raw data: these are the waveforms collected from hammer hits. The second level is processed data: these are pavement layer properties derived from the raw data through established theoretical models. The third level is interpreted data: these are diagnoses of pavement distress precursors from processed data through unproven and (likely) inadequate models.
These models will be improved and upgraded in Phase II and Phase III. Processed data will be archived, in addition to the interpretations, so that the interpretation models may be tested and upgraded with experience.

The raw data (waveforms) are collected from the hammer hits, are immediately processed, and are not saved for archival. Each of eight vibration sensors records three hammer hits. The storage requirements for saving these raw data are large (up to 0.4 megabytes per sample). The capability to save these data for troubleshooting or research on enhanced processing techniques will be included, but not readily available.

Processed data are the result of calculations performed on the raw data and are independent estimates of physical properties of the pavement system. These calculated properties will be archived for all measurements. Table 4.2 lists the pavement properties we are estimating from the raw data. Young's modulus estimates are made from compressional velocity measurements in the asphalt and pavement, and from mechanical impedance in the base. Shear modulus estimates are made from surface wave velocity dispersion. Thicknesses are estimated with the impact echo in the paving layer, and with surface wave dispersion in the pavement and base. Damping is estimated from the impact echo in asphalt, and from mechanical impedance in the base.

<table>
<thead>
<tr>
<th>Pavement Component</th>
<th>Parameter Measured</th>
</tr>
</thead>
<tbody>
<tr>
<td></td>
<td>Young's Modulus</td>
</tr>
<tr>
<td>Paving Layer</td>
<td>yes</td>
</tr>
<tr>
<td>Base</td>
<td>yes</td>
</tr>
<tr>
<td>Subgrade</td>
<td>no</td>
</tr>
</tbody>
</table>

*Thickness estimate of base depends on shear modulus contrast with subgrade.

Interpreted data will be diagnoses of distress precursors with a numerical indication of the reliability. Table 4.3 lists the seven distress precursors to be diagnosed from the physical property measurements (at least eight) listed in Table 4.2. In the pavement and base layers the "unmaintainable" category is included for conditions where failure appears to be imminent and probably should have already occurred.
Data Interpretation

For the prototype, the interpretation technique will assume that the distress precursors that need to be identified are specific to a given pavement layer, or are essentially independent of the presence of other distress precursors. Aging, fine cracking, overlay delamination, and voids are directly observable physical conditions unrelated to a failure model. Moisture in the base and under joints is strongly related to a failure model, but infiltration paths may be highly localized and not seen in the measurements.

Table 4.3 - Distress precursor interpretations from pavement properties.

<table>
<thead>
<tr>
<th>Paving Layer Distress Precursors</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Fine cracking</td>
<td></td>
</tr>
<tr>
<td>Aging</td>
<td></td>
</tr>
<tr>
<td>Delamination</td>
<td></td>
</tr>
<tr>
<td>Unmaintainable</td>
<td></td>
</tr>
</tbody>
</table>

<table>
<thead>
<tr>
<th>Base Distress Precursors</th>
<th></th>
</tr>
</thead>
<tbody>
<tr>
<td>Void under pavement</td>
<td></td>
</tr>
<tr>
<td>Softening of slab under pavement</td>
<td></td>
</tr>
<tr>
<td>Moisture change in base</td>
<td></td>
</tr>
<tr>
<td>Unmaintainable</td>
<td></td>
</tr>
</tbody>
</table>

The assumption of independence of distress precursors permits a layer by layer diagnosis since the Surface Analyzer measurements (Table 4.1) are also layer specific. In the asphalt, aging is diagnosed through an increase in brittleness apparent in Young’s and shear moduli and delamination is apparent in thickness and damping. In pavement, fine cracking is diagnosed through a strong reduction in the vertical shear modulus relative to a reduction in Young’s modulus. In the base, voids and moisture both reduce moduli while voids present a relatively undamped system compared to the presence of moisture.

Interpretation of distress precursors will be performed using a hypothesis testing approach relative to the design parameters for the pavement system. For instance, if fine cracking exists, then Young’s and shear moduli in the pavement will fall within a specified range of values that are a fraction of the ideal stiffness for the concrete type used. If measurement points fall outside these ranges for known distress types, distances from the measurement points to the region will be used to weight probability that the hypothesis is true.

The assumption that distress precursors are independent and layer specific is a "weak" or cautious interpretation model. This classification can be viewed through the perspective that
The assumption that distress precursors are independent and layer specific is a "weak" or cautious interpretation model. This classification can be viewed through the perspective that we have at least eight knowns or equations (pavement properties) and seven unknowns (distress precursors). Nine equations with seven unknowns is an overdetermined system. The addition of a rule, numerical correlation, or condition relating the distress precursors adds an additional equation to the system, making it even more overdetermined and "stronger".
Description of Measurement Technologies

In this chapter, the theoretical aspects of measurement technologies are described. For the convenience of the reader, a background on wave propagation theory is included in Appendix A.

Impulse Response (IR) Method

Two parameters are obtained with the IR method -- modulus of subgrade and damping ratio of the system. These two parameters are then utilized to characterize the existence of several distress precursors (see Chapter 4 and Chapter 7). In general, modulus of subgrade can be used to delineate between good and bad support; whereas, the damping ratio can be used to distinguish between the loss of support or weak support. The two parameters are extracted from the flexibility spectrum measured in the field.

A simplified layout of the source-receiver configuration for the surface analyzer is shown in Figure 5.1. In the IR tests, the low-frequency source and Geophone G1 are utilized. The pavement is impacted to couple stress wave energy in the surface layer. At the interface of the surface layer and base layer, a portion of this energy is transmitted to the bottom layers and the remainder is reflected back into the surface layer. The imparted energy is measured with a load cell. The response of the pavement, in terms of particle velocity, are monitored with the geophone. The load and velocity time histories are simultaneously recorded, which then, are transformed to the frequency domain utilizing a fast-Fourier transform algorithm. The ratio of particle velocity and load (termed mobility) at each frequency is then
Figure 5.1 - Schematic of sensor configuration of surface analyzer.
determined. A plot of mobility versus frequency, called the mobility spectrum, is developed. To determine the flexibility spectrum, the mobility spectrum should be integrated. A mobility and corresponding flexibility spectra are shown in Figure 5.2. The flexibility spectrum demonstrates the characteristics of a single-degree-of-freedom (SDOF) system. The coherence function is also measured. This function is analogous to signal-to-noise ratio and is an indication of the quality of the data collected.

For analysis purposes, the pavement is also modelled as a SDOF system. The key characteristics of a SDOF are shown in Figure 5.3. Three parameters are required to describe such a system -- natural frequency, damping ratio and gain factor. The last two can be replaced by the static amplitude and the peak amplitude. These three parameters are collectively called the modal parameters of the system. The natural frequency and gain factor are used to determine the modulus of subgrade. The damping ratio is directly used.

To determine the modal parameters, a curve is fitted to the flexibility spectrum. An elaborate curve-fitting algorithm which uses the coherence function as a weighing function is used Richardson and Formenti (1982). For a SDOF, the curve can be theoretically described as:

\[
A(f) = B \sum_{i=1}^{n} \frac{(s-Z_i)}{\sum_{i=1}^{n} (s-P_i)}
\]  

(5.1)

where:

\begin{align*}
A(f) & = \text{flexibility at a frequency } f, \\
\mathbf{s} & = \text{Laplace operator } = j(2\pi f), \\
\mathbf{z}_i & = \text{ith zero,} \\
\mathbf{p}_i & = \text{ith pole,} \\
\mathbf{n, \ m} & = \text{number of poles and zeros (2 for a SDOF), and} \\
\mathbf{B} & = \text{gain factor.}
\end{align*}

The poles, zeros and gain factor obtained from the curve-fitting are easily converted to modal parameters (Lathi, 1974). From these parameters, the modulus of subgrade is determined.
Figure 5.2 - Typical spectral functions for impulse response testing.
Figure 5.3 - Frequency response of an ideal single-degree-of-freedom system
The modulus of subgrade, $G$, is calculated from (Dobry and Gazetas, 1986):

$$G = (1 - \nu) / [2L \, A_o \, S_z]$$  \hspace{1cm} (5.2)

where:
- $\nu$ = Poisson's ratio of subgrade
- $L$ = length of slab, and
- $A_o$ = static flexibility of the slab (flexibility at $f = 0$).

The shape factor has been developed by Dobry and Gazetas (1986) and can be determined from:

$$S_z = 0.73 + 1.54 \left( \frac{A}{4L^2} \right)^{0.75}$$  \hspace{1cm} (5.3)

where $A$ is the area of the slab. The value of $S_z$ is equal to 0.80 for a long flexible pavement.

Parameter $I$, (Timoshenko and Goodyear, 1951) is a parameter which considers the effect of increase in flexibility near the edges and corners of a slab. Parameter $I'$ is a function of the length and width of the slab as well as the coordinates of the impact point relative to one corner. Depending on the size of the slab and point of impact, the value of $I'$ can be as high as 3.

The damping ratio, which typically varies between 0 to 100 percent, is an indicator of the degree of resistance of the slab to movement. A slab which is in contact with the subgrade demonstrates a highly damped behavior and has a damping ratio of greater than 70 percent. In this case, the static and peak amplitudes (see Figure 5.3) coincide and occur at a frequency of zero Hz. A slab containing an edge void would demonstrate a damping ratio in the order of 10 to 40 percent. A loss of support located in the middle of the slab will have a damping of 30 to 60 percent.

A computer program has been coded which contains this simplified algorithm. The algorithm described above is a significant step towards automated implementation of the IR method. To our knowledge, the state-of-the-art in interpreting the field data is quite subjective and is based upon empirical correlations; whereas in the developed algorithm, physical parameters of the pavement are measured. So far, the algorithm has been successfully utilized in several laboratory and field studies.

To realize fully the possible limitations of the method, an elasto-dynamic finite element program has been also developed. The debugging of the program has just been completed.
SASW Method

The Spectral-Analysis-of Surface Waves (SASW) method has been mainly developed by Nazarian and Stokoe (1985, 1986, and 1989). The SASW method is a seismic method that can be used to determine elastic modulus profiles of pavement sections nondestructively.

The key points in the SASW method are the generation and detection of surface waves. In a layered system, such as a pavement, the velocity of surface waves varies with wavelength. This wavelength dependency of surface waves is termed dispersion. A plot of wave velocity against wavelength is called a dispersion curve. A complete investigation of a site with the SASW method consists of three phases: 1) field testing, 2) determination of experimental dispersion curve, and 3) determination of stiffness profile (a.k.a. inversion process).

The set-up used for the SASW tests is depicted in Figure 5.1. All accelerometers and geophones are utilized. A disturbance is applied to the ground surface to generate stress waves which propagate mostly as surface waves of various wavelengths. The waves are monitored and captured with a data acquisition system (through the receivers). Signal and spectral analyses are then utilized to determine the phase information of the cross power spectra (CPS) and the coherence functions amongst consecutive adjacent receivers. An example of these two spectral functions are shown in Figure 5.4.

To obtain the dispersion curve, the velocity of propagation, $V_{ph}$, and wavelength, $L_{ph}$, are determined from the phase of the CPS, $\phi$, at any frequency, $f$. This relationship can be written as:

$$V_{ph} = D/[(\phi/360)f] \quad \text{(5.4)}$$

$$L_{ph} = D/[(\phi/360)] \quad \text{(5.5)}$$

where $D$ is the distance between the receivers. The dispersion curve obtained from the phase information of CPS shown in Figure 5.4 is included in Figure 5.5.

The last step is to determine the elastic modulus of different layers given the dispersion curve. Several alternatives are available (Nazarian, 1989). A recently-developed automated inversion process (Yuan and Nazarian, 1990) will be utilized. The process is theoretically complex and as such is not discussed here. An actual dispersion curve and corresponding modulus profile after inversion are shown in Figure 5.6.
Figure 5.4 - Typical spectral functions for SASW tests

Figure 5.5 - Dispersion curve obtained from spectral functions
a) Dispersion curve

b) Composite profile

Figure 5.6 - Typical results from SASW tests
Ultrasonic Surface Wave Method

The ultrasonic surface wave is an offshoot of the SASW method. The major distinction between these two methods is that the properties of the top paving layer can be easily and directly determined without a need for a complex inversion algorithm. To implement the method, the high-frequency source and Accelerometers A1 and A2 (see Figure 5.1) are utilized.

A theoretical dispersion curve for a two layer system is shown in Figure 5.7. Two distinct branches are obvious. First, up to a wavelength approximately equal to h (thickness of the uppermost layer) the velocity of propagation is independent of wavelength. At wavelengths greater than h, the dispersive characteristic of surface waves (i.e. variation of velocity with wavelength) is strongly evident. Therefore, if one simply generates high-frequency (short-wavelength) waves, and if one assumes that the properties of the uppermost layer are uniform, the modulus of the top layer, G, can be very easily determined from:

$$ E = \rho \left[ K \ V_{ph} \right]^2 \quad (5.6) $$

where,

$$ K = 1.13 - 0.16 \nu \quad (5.7) $$

In the above equations, $V_{ph}$, $\rho$ and $\nu$ are velocity of surface waves, mass density and Poisson's ratio, respectively. An estimate of the thickness of the surface layer can be made by determining the wavelength above which the surface wave velocity is constant.

The methodology can be simplified even further. Assuming that the stiffness of the top paving layer is constant, Equation 5.4 can be written as:

$$ \phi = \left[ 360D / V_{ph} \right] f = m f \quad (5.8) $$

Equation 5.8 represents a linear relationship between the phase of the cross power spectrum and frequency provided the phase velocity is constant. So one can easily determine $V_{ph}$, by performing a least square linear regression over the high-frequency region of cross power spectrum and to obtain the slope of the best-fit line. An example is shown in Figure 5.8.

Ultrasonic Compression Wave Velocity Measurement

Once the compression wave velocity of a material is determined, its Young's modulus can be readily determined. The same set-up used to perform the SASW tests, can be readily used to determine compression wave velocity of the upper layer of pavement. Miller and Pursey (1955) found that when the surface of a medium is disturbed, the generated stress waves
Figure 5.7 - Theoretical dispersion curve from a two-layer pavement.

Figure 5.8 - Procedure for determining Young's modulus of paving layer from ultrasonic surface wave method.
propagate mostly with the Rayleigh wave energy and to a lesser extent with the shear and compression wave energy. As such, body wave energy present in a seismic record generated using the set-up shown in Figure 5.1 is very small and for all practical purposes does not contaminate the SASW results. However, compression waves travel faster that any other type of seismic waves, and are detected first on seismic records.

Several automated techniques for determining the arrival of compression waves are available. Times of first arrival of compression waves are typically measured by triggering on an amplitude range within a time window (Willis and Toksoz, 1983).

Impact Echo

The impact echo method can be effectively used to locate defects, voids, cracks and zones of deterioration within concrete. The method has been thoroughly studied and effectively used on many projects by researchers at the National Bureau of Standards. In a comprehensive theoretical and experimental study, Sansalone and Carino (1986) considered the effects of: type of impact source; distance from the impact point to receiver; type of receiving transducers; depth of reflecting interfaces.

The high-frequency source and the accelerometer A1 and possibly A2 are used (see Figure 5.1). Once the compression wave velocity of concrete, $V_p$, is known, the depth to reflector, T, can be determined readily from (Sansalone and Carino, 1986):

$$ T = \frac{V_p}{2f} $$

(5.9)

where f is the resonant(return) frequency obtained by transforming the deformation record into the frequency domain.

As an example, the frequency domain representation corresponding to impact echo test on a nominally 7.5-in.-thick concrete is shown in Figure 5.9. The compression wave velocity of the concrete was measured to be 13500 fps. The resonant(return) frequency is 10.5 kHZ. Utilizing Equation 5.9, the thickness of the concrete of 7.7 in. is determined.
Figure 5.9 - Typical impact echo result in frequency domain
Principles of Pavement Condition Measurement

The quantity or levels of measurements, the associated pavement component and the testing spatial distribution are summarized in Table 6.1. As reflected in the table, for the moisture related distress precursors, the change in modulus is measured and then converted to variation in moisture using empirical relationships. As the surface analyzer is a project-level device, the value of this conversion is questionable. The variation in modulus may be of greater significance than the variation in moisture.

The factors that affect the modulus of paving materials are defined in Appendix B. Basically, the strain amplitude, the state of stress, void ratio, grain size distribution are the main factors. However, for this study dealing with in situ tests for shallow depths, the effects of strain amplitude and state of stress are not of significance. The main two parameters are the void ratio and the grain size distribution. These are the exact two parameters that affect the infiltration of moisture in the base and subgrade materials. The higher the void ratio or the coarser the material the less its stiffness is affected by moisture. A material with such properties does not retain moisture well. This proves that the modulus is a suitable parameter to be used for distress precursors related to moisture. For non-moisture related distress precursors, stiffness is a direct and important parameter which can be directly utilized in almost all predictive models.

The effects of change in moisture content on modulus are discussed in Appendix C. A recently-developed empirical relationship between change in moisture and change in modulus from dry state to saturated state is also given in Appendix C (Qian, 1990). Basically, the
grain size analysis is required to develop adequate relationships. Typically, this information is available for most base materials.

Table 6.1 - Levels and nature of measurements for each distress precursor.

<table>
<thead>
<tr>
<th>Distress Precursor</th>
<th>Test</th>
<th>Quantity</th>
<th>Pavement Component</th>
<th>Spatial Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Moisture in Foundation</td>
<td>Impulse Response</td>
<td>Change in Flexibility Due to Change in Moisture Content (or Degree of Saturation)</td>
<td>Overall Change in Flexibility of Pavement System</td>
<td>Spot Measurement Over a Distance of About 6 in.</td>
</tr>
<tr>
<td></td>
<td>SASW</td>
<td>Change in Young’s Modulus Due to Change in Moisture Content (or Degree of Saturation)</td>
<td>Change in modulus of each layer as well as variation in modulus within each layer (base, subbase and subgrade)</td>
<td>Spot Measurement Over a Distance of 6 ft.</td>
</tr>
<tr>
<td>Moisture Under Joints</td>
<td>Impulse Response</td>
<td>Change in Flexibility Due to Change in Moisture Regime Under Joints</td>
<td>Overall Change in Foundation Support Due to Change in Moisture in Foundation Layers (Base and/or Subgrade)</td>
<td>Spot Measurement Over a Distance of About 6 in.</td>
</tr>
</tbody>
</table>
Table 6.1, cont. - Levels and nature of measurements for each distress precursor.

<table>
<thead>
<tr>
<th>Distress Precursor</th>
<th>Test</th>
<th>Quantity</th>
<th>Pavement Component</th>
<th>Spatial Distribution</th>
</tr>
</thead>
<tbody>
<tr>
<td>Fine Cracking</td>
<td>Impulse Response</td>
<td>Reduction in Rigidity of the Paving Layer Due to Cracks</td>
<td>Overall Change in the Rigidity of the Paving Layer Only</td>
<td>Spot measurement over a distance of about 6 in.</td>
</tr>
<tr>
<td></td>
<td>Body Wave Velocity</td>
<td>Delay in Travel Time of Compressional Wave Because of Longer Travel Path and Lower Rigidity</td>
<td>Approximate Depth and Width of Cracks in the Paving Layer</td>
<td></td>
</tr>
<tr>
<td>Voids or Loss of Support</td>
<td>Impulse Response</td>
<td>Significant Increase in Flexibility of Slab Due to Lack of Support Under the Slab</td>
<td>Overall Change in the Rigidity of Paving Layer Only</td>
<td>Spot measurement over a distance of about 6 in.</td>
</tr>
<tr>
<td></td>
<td>Impact Echo</td>
<td>Return (Resonant Frequency) Associated With the Thickness of Slab</td>
<td>Overall Thickness of Slab to Distinguish Between Delamination and Voids or Loss of Support</td>
<td></td>
</tr>
<tr>
<td>Distress Precursor</td>
<td>Test</td>
<td>Quantity</td>
<td>Pavement Component</td>
<td>Spatial Distribution</td>
</tr>
<tr>
<td>-------------------</td>
<td>-----------------</td>
<td>------------------------------------------------------</td>
<td>--------------------------------------</td>
<td>------------------------------------------</td>
</tr>
<tr>
<td>Overlay Delamination</td>
<td>Impulse Response</td>
<td>Significant increase in flexibility of overlay due to lack of support under the overlay</td>
<td>Overall change in the rigidity of overlay only</td>
<td>Spot measurement Over a Distance of About 6 in.</td>
</tr>
<tr>
<td></td>
<td>Impact Echo</td>
<td>Return (resonant) frequency associated with the thickness of overlay</td>
<td>Overall thickness of slab to distinguish between delamination and voids or loss of support</td>
<td></td>
</tr>
<tr>
<td>Aging</td>
<td>Ultrasonic SASW</td>
<td>Shear wave Velocity of AC layer</td>
<td>Variation in shear wave velocity (stiffness) of the AC layer only</td>
<td>Spot measurement Over a Distance of About 6 in.</td>
</tr>
<tr>
<td></td>
<td>Body Wave Velocity</td>
<td>Compression wave Velocity of AC layer</td>
<td>Average compressional wave velocity of AC layer, which when combined with the shear wave velocity, will be used to determine the Poisson's ratio of the AC layer</td>
<td></td>
</tr>
</tbody>
</table>
Moisture in Foundation

Two different tests are proposed for detecting moisture in foundation layers. These two tests are: 1) impulse response and 2) SASW. A third test, ultrasonic body waver velocity, is used to assess the quality of AC layer.

The utilization of the impulse response method is based upon the philosophy that the material becomes less rigid (more flexible) as the water content increases. This is true for most paving materials except concrete and some stabilized foundation layers. High flexibility indicates the need for maintenance.

Practically speaking, the precision of the IR method is about 5 percent due to strong signal-to-noise ratio associated with the output of the load cell and geophone. In addition, based upon years of experience, a 25 percent accuracy is reasonable. A series of perturbational studies was carried out to study the theoretical accuracy of the method. An elasto-static finite element program, based upon the principles of elastic beam on a Winkler foundation, was coded to model the IR tests.

To model the softening of the foundation layer, the modulus of subgrade reaction was perturbed by ten percent and the associated change in measured static stiffness was determined. The thickness of the paving layer was assumed to be 6 in. with a modulus varying from 7,000 ksi to 350 ksi. The modulus of subgrade reaction was assumed to be equal to 170 pci (medium stiff). The results are shown in Figure 6.1. The changes in moduli at the edge and middle of the pavement were determined. The results from a corner point is also shown. The wide range of change in modulus selected makes these results applicable to a rigid pavement also. The same figure will be utilized later for this purpose. The change in stiffness is virtually independent of the position of the test, and the modulus of slab. A ten percent change in modulus will result in a six percent change in stiffness. Therefore, if a 15 percent change in stiffness is conservatively selected as a target measurement, the accuracy associated with the change in modulus of 25 percent should be readily achievable.

Similarly, as the wet front rises in the foundation layers the materials become less stiff. By measuring the shear wave velocity profile of a given site using the SASW method, one can readily determine the location and the amount of decrease in moduli of different layers in the pavement.

The major advantages of the SASW measurements are that 1) the layer that would potentially contribute to distress can be located; and 2) modulus values are determined which can be directly used in maintenance strategies which are based upon mechanistic analyses.
Figure 6.1 - Perturbational study on softening of base in impulse response tests
The precision associated with the close receiver spacings (utilized to determine modulus of top layer) is about 5 percent. For farther receiver spacings, the precision is about 10 percent. These values are established based upon experience with the method.

The layer thickness is determined with an accuracy of about 10 percent. This value has been established based upon blind tests. As indicated above, modulus of the top layer can be determined with an accuracy of about 10 percent. Based upon extensive field testing, and independent comparison with other seismic methods, the modulus of the base can be determined with an accuracy of 20 to 25 percent. Similarly, modulus of the subgrade layer can be determined with an accuracy of 15 to 20 percent. The modulus of subgrade can be predicted more accurately because of the limitations of the inversion program.

A perturbational study to establish analytically the sensitivity of the method follows. A typical pavement section was utilized. The pavement consisted of 5 in. of AC layer over 12 in. of base and a subgrade. The shear wave velocities of these three layers were assumed to be 2500 fps (modulus of about 500 ksi), 1000 fps (modulus of about 60 ksi) and 500 fps (modulus of about 15 ksi). Two dispersion curves are shown in Figure 6.2a. The solid line correspond to the control section. The dashed line is obtained by reducing the modulus of the base by 10 percent. Between the wavelengths of 2 ft to 5 ft, the difference between the phase velocities from the two profiles is about 7.5 percent. As such, the dispersion curve is sensitive to the change in properties of the base. To maintain a 10 percent difference between the two dispersion curves, a change in shear wave velocity of about 15 percent is recommended.

The two phase information of the cross power spectra are compared in Figure 6.2b. A distance between the receivers of 24 in. is selected. No significant difference is apparent at high frequencies. At lower frequencies (between 500 Hz and 1500 Hz) some difference (about 10 percent) is apparent. Even though visible, such a small change may not be adequate. Therefore, a variation of 15 percent in shear wave velocity may better reflect the accuracy for this case. It should be mentioned that for a receiver spacing of 48 in., the difference between the two phase spectra are better depicted.

**Moisture under Joints**

The impulse response technique is well-suited for detecting the initiation of voids or the existence of loose material under a joint. The method is used to measure the flexibility of the pavement system. In an intact pavement section, the system will demonstrate high rigidity (low flexibility). As soon as a void is initiated, the rigidity will decrease and the slab will act quite flexibly.
Figure 6.2 - Perturbational study on softening of base in SASW tests
One advantage of the impulse response method over some other methodologies is that a very small separation (void) will result in a large increase in flexibility. Therefore, the method is well-suited for preventive maintenance. The precision of the IR method is about 5 percent as discussed in the previous section. For moisture under joints, a 25 percent accuracy is reasonable. A series of perturbational studies, similar to those described for the moisture in foundation, was carried. A 15-ft square slab was considered. The thickness of the slab was assumed as 12 in. The modulus of the slab was assumed to vary from 7,000 ksi to 350 ksi. The modulus of subgrade reaction was assumed to be equal to 170 pci (medium stiff). The position of testing in terms of center, edge and corner was also considered.

To model the softening of the foundation layer, the modulus of subgrade reaction was perturbed by ten percent and the associated change in measured static stiffness was determined. The results are shown in Figure 6.3. The change in stiffness is virtually independent of the position of the test, and the modulus of slab. A ten percent change in modulus will result in a six percent change in stiffness. Therefore, if a 15 percent change in stiffness is conservatively selected as a target measurement, the accuracy associated with the change in modulus of 25 percent should be readily achievable.

The results shown in Figure 6.1 also correspond to a 6-in.-thick slab. Once again, a variation of 10 percent in modulus results in a reduction in stiffness of about 6 percent. Therefore, the conclusions mentioned above holds for the thinner slab as well.

**Fine Cracking**

Two methodologies are used for detecting fine cracks. These two are the impulse response and the ultrasonic body wave.

The philosophy behind the use of the impulse response method is that a cracked section is less rigid than an intact one. The limitation of this method is that the crack has to propagate through the thickness of the paving layer.

The ultrasonic body wave method can be used to determine the existence of cracks even if they are not extended through the thickness of the layer. In this method, stress wave energy is generated at one point and detected at several other points. Any cracks in the material located between the source and any receiver will delay the direct propagation of waves. With cracks, the travel time will increase. Also, the amplitude associated with the wave will decrease. It should be mentioned that differentiation between a strong material containing cracks and a weak material is not possible at this time.
Figure 6.3 - Perturbational study on softening of subgrade under a slab in impulse response tests.
The process of determining the compression wave velocity is straightforward. The amount of data manipulation and interpretation is minimal. Based upon field experience, the travel time records can be measured with a precision of better than five percent. The accuracy of the modulus determined for the top layer is typically within 10 percent.

The method is primary utilized for determining the deterioration within the paving layer (i.e. AC or PCC). Compression wave velocities are controlled by porosity, water saturation, and most strongly by pore geometry (parameter Beta). Theoretically, Beta is the ratio of modulus of the drained porous solid to the modulus of the nonporous solid and can range from zero (vertical fracture) to one minus the porosity (horizontal fracture). Figure 6.4 shows the variation in compression and shear wave velocities as a function of Beta and water saturation for a constant porosity of 0.5 percent. As reflected in the figure, compression wave velocity decreases faster when the voids and cracks are filled with air (compared to water). The propagation velocity of water is higher than that of air.

The ten percent level of accuracy associated with the measurement of wave velocity is marked on Figure 6.4. With this level of accuracy, one can detect the manifestation of voids and cracks (in terms of reduction in modulus) when 35 percent of the thickness of the concrete has air-saturated cracks. Similarly, if about 45 percent of the concrete thickness is cracked with water, one can detect its weakening affects. Similar curves for a high-quality and poor quality AC layers are presented in Figure 6.5. These curves yield similar results.

**Voids or Loss of Support**

The existence of a void or loss of support beneath a slab can be detected using the impulse response method, and it can be verified using the impact echo method.

Similar to the detection of moisture under a joint, a void beneath or within a slab will result in a large increase in flexibility of the slab. Therefore, by measuring the flexibility of the slab at different locations, one can pinpoint the voids or loss of support.

For voids and loss of support beneath a slab and overlay delamination a 20 percent level of accuracy can be conservatively selected. To model a void or delamination, the modulus of subgrade reaction was reduced to zero in the elasto-static finite element program indicated above. Voids were assumed to be square and extended horizontally 12 in., 24 in. or 48 in. As before, slab thickness of 6 in. and 12 in. were used. The modulus of the slab was assumed to be 7,000 ksi, 3,500 ksi or 350 ksi. The voids were positioned in the center, at the edge or at the corner of the slab.
Figure 6.4 - Variation in velocity with orientation of voids for a PCC slab

Figure 6.5 - Variation in velocity with orientation of voids for an AC pavements
The ratio of the stiffness of the control section and same section but with a void as a function of void size is shown in Figure 6.6. In this case, the thickness of the slab is 12 in. and its modulus is 7,000 ksi. This represents a very stiff PCC section. It is conservatively assumed that a change in stiffness of about 20 percent is required before the void can be detected. Based upon this assumption, an interior or an edge void should be more than 48 in. in dimensions before detected. For a corner slab dimensions of voids of greater than 24 in. can be readily detected.

To verify the existence and possibly determine the location of voids beneath or within the slab, one can utilize the impact echo method. The method, a special case of ultrasonic body wave method, can be utilized to determine the depth to the reflector.

The process is straightforward. The return (resonance) frequency of the reflected waves from the boundary should be determined. Practically speaking, if good contact between the transducer and the pavement is achieved, a precision of less than 5 percent and an accuracy of less than 10 percent can be expected.

Theoretical variations in the return frequency with the depth of reflective boundary for three different material properties are shown in Figure 6.7. Compression wave velocities of 14000 fps, 7000 fps, and 3500 fps correspond to a high-quality PCC, a low-quality PCC (or a high-quality AC), and a poor-quality AC, respectively. From the figure, the higher the compression wave velocity (modulus) and the thinner the paving layer, the higher the return frequency of the layer. Based upon our set-up, it is anticipated that return frequencies up to 15 kHz can be determined. That will translate to a thickness of about 5 in. to 6 in. for PCC and 2 in. to 4 in. for AC.

**Overlay Delamination**

The impulse response and the impact echo are the prime candidate methods for determining the location and existence of delamination. The theoretical and experimental aspects of these two tests as used for detecting overlay delamination are identical to those used for locating voids and loss of support. The only differences are the nature and location of the interface. For delamination, the void occurs at the interface of two layers and the delaminated layer is located closer to surface.

**Pavement Aging**

The ultrasonic SASW method can be utilized to determine the shear wave velocity of the asphalt layer. The direct compression wave propagation can be employed to determine the compression wave velocity. Knowing the shear and compression wave velocities one can
Figure 6.6 - Perturbational study on loss of support beneath a slab in impulse response tests

Figure 6.7 - Sensitivity of impact echo method in detecting depth of delamination
determine Poisson's ratio of the material. Due to aging, the asphalt layer becomes stiffer and more brittle. By measuring shear wave velocity and Poisson's ratio, one can follow the effect of aging process on the behavior of asphalt layer. The development of the methodology for characterization of aging is not done in this project. Two pioneering projects [(i.e. SHRP A-003, (Bell, 1989), and AAMAS (Van Quintas, et al, 1988)] are studying this phenomenon.

The precision and accuracy of the ultrasonic body wave method were discussed before. Similarly, the ultrasonic SASW method is as precise. Based upon many years of experience, a precision of 5 percent is assigned to this technique. The modulus and thickness can be calculated with an accuracy of ten percent. These levels of accuracy are determined based upon years of field testing and several blind tests.

The dispersion curve associated with this profile is shown in Figure 6.8a. In actual field tests, the dispersion curve is utilized to determine the shear wave velocity profile. In Figure 6.8b, the raw data, the phase information of cross power spectrum, are shown. The phase information of the cross power spectrum is the raw data measured in the field.

The results of a study to establish theoretically the sensitivity of the method are shown in Figure 6.8. The profile consists of a 5-in.-thick layer of asphalt with a shear wave velocity of 2500 fps (Young's modulus of about 500 ksi), a 12-in.-thick base layer with a shear wave velocity of 1000 fps (Young's modulus of about 60 ksi) and a subgrade layer with a shear wave velocity of 500 fps (Young's modulus of about 15 ksi).

The shear wave velocity of the first inch of the AC layer is perturbed by 10 percent. The resulting dispersion curve is compared with the control section in Figure 6.8b. A ten percent change in the velocity of the AC layer results in a ten percent change in the dispersion curve at short wavelengths. The two dispersion curves converge at longer wavelengths. There is a one-to-one relationship between the change in the input and output; i.e. the method is very sensitive to change in properties of the AC layer.

The difference between the phase information of cross spectra of the control and perturbed profiles is reflected in Figure 6.8b. From this figure, a small change in the modulus of the top layer will result in large variation in the raw data measured.
Figure 6.8 - Perturbational study on effect of pavement aging in SASW tests
Conceptual Design and Data Interpretation

This chapter deals with three diverse topics that are tied together by their abstractness, and by their longer term contribution to a successful and useful implementation of the Surface Analyzer concept. The first topic deals with aspects of the surface analyzer design that make it easily and reliably learned and operated, maintained, and manufactured. The second topic describes the design considerations in computer architecture considered important to successful implementation and modification. The third topic describes the proposed data interpretation technique and its implementation.

Design Considerations

In implementing the measurements, described in previous chapters, in the Surface Analyzer, several additional criteria were considered in the design. These criteria are broken down into three subsections that deal with (1) how the user interacts with the analyzer, (2) design of reliable, maintainable hardware, and (3) manufacture and modification of the device.

User Considerations

For the sake of maximizing efficiency and reliability of data collection the user should be able to operate the Surface Analyzer using a console (portable laptop computer) in the driver's seat of the vehicle. Adequate quality indices and diagnostics would be available to the user so that the vehicle need only be left to perform special hardware adjustments.
Since the user interacts primarily with software being run on a computer, the software must accommodate a broad range of user's skills. The software was designed to accommodate three specific users, representing the most likely modes of use of the equipment. The first user is the technician performing routine data collection in the field or routine data analysis in the office. Conceptually simple, repetitive sequences requiring minimal training are performed with the results being evaluated by a supervisor. An appropriate level of warning about failure or misuse is provided with suggested actions to be taken. The second user is the engineer or experienced technician configuring the software for routine collection or analysis by the inexperienced technician. This person requires a more sophisticated knowledge of the measurements and software functions. The third user is an engineer performing extensive diagnostics and calibration, or detailed interactive collection and analysis of data. This person may operate outside the confines of the user interface software for special functions.

Maintenance Considerations

The equipment has been designed to be as simple as possible with few moving parts. As many components as possible will be available from local suppliers and replaceable with simple tool kits. For instance, pneumatic control was chosen over hydraulics as parts are readily available in hardware stores, leaks are easy to find, and repairs may be made without special tools.

Calibration of measurements is performed in software in the surface analyzer. This minimizes adjustments that must be made in electronics hardware with less reliable and less rugged components. It also permits a nonspecialist to perform calibrations and diagnostics before calling an electronics specialist for repair.

Construction Considerations

To meet target replication costs of $10,000 per machine with the expected small number of machines produced, extensive design costs of specialized hardware cannot be justified. Consequently, as many subsystems will be bought from existing vendors as is possible.

The Surface Analyzer is expected to be mounted on a lightweight trailer that could be towed behind a car or truck. The design could be mounted beneath a dedicated truck but we do not expect to do so in this project due to the difficulties with modification and maintenance.
Computer Configuration

Computer control plays a very large part in operation of the Surface Analyzer. Criteria for efficient concept testing differ from those for efficient production. Hence, this section lays out the differences between the design philosophy of the prototype and the design philosophy of the production machine.

The components of the Surface Analyzer computer system are broken into the IBM PC/AT equivalent computer, the data acquisition software, and the user interface software. The data acquisition software is treated as hardware and is not accessible by the user, is optimized for speed, may be put into Read Only Memory (ROM) if necessary, and is concerned with controlling data acquisition, digitization, reduction, and storage on the computer. The user interface software is more conventionally disk based, modifiable, and easily extensible, and serves to translate the user's intentions into commands that the data acquisition and processing software needs.

The IBM PC/AT equivalent computer was chosen as the development and production platform due to its low cost, the wide availability of vendor supplied hardware for data acquisition, and for the wide range of software compatible hardware platforms. An additional consideration was that PC's are widely used in state highway departments for data management.

The user software has been designed to communicate with the data acquisition software through the concept of message passing. This means that a specific language has been defined that is independent of user or data acquisition software structure. The use of message passing introduces a great deal of flexibility in computer hardware configuration, modification, extension, and makes maintenance of the software much easier. The disadvantage is that speed is not optimized, but given trends in hardware development that disadvantage is strongly outweighed by the ease of moving the software to a faster computer.

Prototype Philosophy

Development of the prototype Surface Analyzer follows a slightly different path than will be used for the production version. In developing a prototype, ease of modification and testing speed dominate design criteria. Cost of production, ease of maintenance, and reliability are more important in the production instrument, and are addressed in the subsequent section.

The prototype Surface Analyzer will have the user interface software and data acquisition hardware and software on the same computer (Figure 7.1a). Message passing will be accomplished storing the messages on a dedicated disk partition. Messages are readily
Figure 7.1a - Schematic of computer structure for the prototype surface analyzer

Figure 7.1b - Schematic of computer structure for the production surface analyzer
available for debugging of individual software components. Software components may also be repeatedly run on the same control message for testing and debugging.

Speed of this prototype configuration is also a significant consideration. During the development phase, software is not optimized and frequently needs to be recompiled. Modifications to add unexpected functions also needs to be accommodated. It is always easier to optimize and configure software onto a smaller machine after the design has stabilized, than it is to develop and add new functionality into limited hardware. Consequently, a much more powerful computer is required during the prototype phase.

**Production Philosophy**

In the production machine, cost of production, ease of maintenance, and reliability are the primary considerations. To satisfy these conditions, the Surface Analyzer will be split onto two separate computers. A dedicated computer will be used for data acquisition and a separate machine used for the user interface and data archiving. Communication between the two machines will be via a parallel printer port (see Figure 7.1b).

The dedicated data acquisition computer travels with the trailer and must be rugged, environmentally sealed, and reliable. Its functions are fixed and are used in only one way. As conventional floppy and hard disk drives are not very reliable under the conditions to be found in hard highway use, program storage will be in static semiconductor memory. No monitor is necessary for its operation. Keyboard input is also not required for operation, so all of the low reliability parts for dirty environments have been eliminated.

The computer that serves as the user interface could be a wide variety of machines, depending on which functions are likely to be used frequently. If routine data acquisition is being performed, an inexpensive portable laptop with little processing power and disk storage for collected data will function well. If a great deal of computationally intensive analysis is being done along with data collection, a more powerful computer would be useful.

**Data Interpretation**

The interpretation of measurements made by the Surface Analyzer for distress precursors must be based on assumptions about the nature of pavement aging and failure as an adequate database of observations of pavement deterioration does not exist. This section summarizes our proposed interpretation technique and describes how the technique can be improved as this knowledge base is built. First, the structure of measurements and diagnoses is analyzed to set the foundation for the assumptions we make. Second, the interpretation technique will
be presented and its relation to other potential interpretation techniques described. Third, the theoretical predictions for interpretation of the measurements will be presented. Finally, the approach for improving the interpretation technique with experience will be described.

Measurement and Diagnosis Structure

Selection of an appropriate, extensible interpretation technique for diagnosing distress precursors requires an understanding of the structure of measurements, unknowns, and equations or rules governing the system. This section describes the nature of the distress precursors (unknowns), the measurements (Surface Analyzer measurements), the equations, and the considerations in choosing the appropriate interpretation philosophy.

Pavement distress precursors to be identified by the Surface Analyzer are listed in Table 4.2. These seven distress precursors are contained in a single layer of the pavement system, or at a boundary between two layers. Two or more of these distress precursors may occur together and may be causally linked, but also may occur independently. Several of the distress precursors are based on logical distinctions: for example, is a void present or has delamination occurred? Other distress precursors are of a continuous nature: for example, what is the change in moisture in the base?

Since the distress precursors are specific to layers, measurements from processing of Surface Analyzer raw data have been designed to also be specific to the pavement system layers (Table 4.1). The Surface Analyzer reports at least nine properties for interpretation of distress precursors. All of these properties are smoothly varying, continuous variables.

Given the distinction we have made between pavement property measurements as knowns and distress precursors as unknowns, three types of equations or rules are possible to assist in interpretations. In the rest of this discussion a rule will be considered a form of equation, as logical rules may be expressed as equations. The first type of equation relates measurements to measurements. These are applied as consistency checks during the raw data interpretation process and are not considered further here. The second type of equation relates pavement properties to distress precursors. These form the heart of the interpretation process and are derived from theoretical calculations of pavement property changes due to deterioration. The third type of equation describes the interrelation of distress precursors. These equations are derived from a conceptual or numerical model for pavement deterioration.

An interpretation philosophy may take either of two approaches depending on the complexity of measurements, unknowns, and equations. The first is an inductive or hypothesis testing approach that hypothesizes a distress precursor exists (or does not exist) and tests this hypothesis for consistency with the measurements. The second approach is a deductive or
inference based approach that applies the equations to the measurements and reaches a conclusion that the distress precursor either exists or does not exist.

The inductive interpretation approach is suitable for highly nonlinear (multi-valued), noncausal, empirical, or conditional equations with uncertain measurement quality. Inductive reasoning is difficult to implement, is suitable for research operations, and does not always satisfy our need for clear-cut conclusions. The deductive approach is appropriate where measurement quality is high, equations are single-valued and mildly nonlinear, and conclusions are not strongly interrelated. The deductive approach is simple to implement, gives clear-cut conclusions appropriate to field operations, but is likely to be wrong in conditions where the inductive approach is ambiguous.

**Proposed Interpretation Technique**

The data interpretation approach for the prototype Surface Analyzer will be based on an inductive approach to diagnosing distress precursors in specific layers. Distress precursor hypotheses will be tested using the two most diagnostic measurements available. The interpretation equations will rely on the theoretical relations between measurements and distress precursors.

The more complex inductive interpretation approach will be used because of its generality and robustness. The inductive approach provides an upgrade path to include highly nonlinear interpretation equations from models of pavement deterioration. It also seems preferable that the Surface Analyzer tell the maintenance engineer when results are ambiguous instead of presenting shaky conclusions as truth.

Testing distress precursor hypotheses on only two diagnostic measurements is based on design simplicity in the prototype and the current capability of our predictive models. Theoretically, diagnosis of a distress precursor could require a model spanning a space containing all 8 measurements and the other 6 distress precursors. Due to layer specificity of measurements and distress precursors and assumed independence of distress precursors, this reduces to a model spanning a space containing four measurements. In practice, two of the four measurements are frequently unrelated to the distress precursor. For instance, moisture content in the base is shown in Young’s and shear moduli but does not significantly affect damping and is independent of thickness. Likewise, the presence of voids has a weak influence on modulus but is distinguished by the damping.

The hypothesis testing can be visualized in terms of polygonal regions on a cross-plot of two measurements, such as that of Figure 7.2 for moisture content in the base. If the measured moduli values fall within the polygonal field for high moisture content, then the measurements are consistent with high moisture content. If the moduli values fall between the
Figure 7.2 - Moisture control on modulus

Figure 7.3 - Base distress precursors
polygonal fields for high moisture and voids, then the distance of the measurement from each polygon gives a measure of which distress precursor is more likely.

The resolution procedure for hypothesis testing begins by testing all distress precursor hypotheses in a pavement layer. If the distress precursor exists, then the measurement should fall inside the table-specified polygon, and the hypothesis is confirmed. If the measurement point falls outside the polygon, the distance to the polygon surface is measured and saved for comparison with other hypotheses. Each distress precursor is tested in turn in this manner. If measurements are exterior to all polygons, relative distances to polygons will be normalized into probabilities for each hypothesis.

**Theoretical Interpretation Model**

Theoretical bases relating the Surface Analyzer measurements to distress precursors are summarized here for moisture in the base, voids or weakening in the base, fine cracking in pavement, and aging in asphalts. Sample graphs for specific cases are shown with a description of the calculation technique.

A sample of variation in Young’s and shear moduli with changes in base conditions is shown in Figure 7.2. This example is for silt-sized grains at 3.6 psi confining pressure for a range of moisture contents from 0% to 70%. Shear modulus variations from design stiffness, with respect to moisture content and confining pressure due to capillarity, are well established. Young’s modulus variations are calculated using equations published by Domenico (1976) that relate solid and fluid modulus variations through a common pore geometry, and uses the Reuss bound to calculate the fluid modulus for air-water mixtures. Additional regions show the effects of voids and reduced stiffness in the base from finite element calculations we performed.

Figure 7.3 shows a cross-plot of damping and modulus normalized to the design value for the base material. This plot shows polygonal regions delineating weak base and voids based on finite element modeling. Plotted data points are from the Gainesville test slab.

Figure 7.4 shows a cross-plot of Young’s and shear modulus for a PCC pavement layer relative to design moduli for the concrete. Values on vertical lines, ranging from 0% to 100%, indicate the vertical fractional extent of cracking from the base of pavement layer, if all porosity is in the crack. Porosity of the cracks ranges from zero to one percent. A crack water saturation of zero falls below the normalized modulus of one, while 100% water saturation falls in the narrow region above the normalized modulus of one. These computations are based on equations published by Geertsma (1961), using parameters for a vertical crack pore geometry.
Figure 7.4 - Pavement distress types

Figure 7.5 - Asphalitic concrete aging
Figure 7.5 shows a cross-plot relating Young's and shear moduli to aging. Ductile to brittle calculations are based on varying Poisson's ratio from 0.45 to 0.15. Young's modulus as a function of temperature was described by Van der Poel (1984). Temperature variation covers a much larger range of modulus variation than aging, but modulus trends due to aging are resolvable.

**Interpretation Upgrade Path**

Models of controls on pavement deterioration, additional modelling of dynamic pavement system responses, coupled with additional field data collection with the prototype Surface Analyzer is likely to bring about rapid evolution in the interpretation models for distress precursors. This evolution will come about through refinement in the equations relating measurements to distress precursors described above, and through development of conditional relations between distress precursors and external climatic and traffic controls.

Refinement in equations relating measurements to distress precursors can be accommodated by modifying tables that contain the polygonal bounds for the distress precursors. Additional refinement to numerical uncertainties will come with field verification experience.

The development of conditional relations between distress precursors, and an improved understanding of dynamic pavement behaviour will require an increase in the number of parameters used to characterize the polygons. An unpublished algorithm to determine the interior points and exterior points, and to determine the distance from an exterior point to the surface of an n-dimensional polygon has frequently used in geophysical processing techniques up to n = 10. Less accurate algorithms released by IBM research (Inselberg, pers. comm.) are useful at higher dimensionalities than we could ever use. The polygon specification permits straightforward handling of nonlinear, conditional, or multivalued functions that may arise as our knowledge of interrelations between distress precursors improves. The overall logical structure of hypothesis testing does not change as dimensionality increases, nor does the appearance to the user.
Subsystem Design

Transducers, Sources, and Mounting

The major mechanical components of the Surface Analyzer are schematically depicted in Figure 8.1. These include the transducer mounting member, individual transducer holders, two pneumatic hammers, a pneumatic scissors jack to raise and lower the transducers, a pressurized air supply, and a fifth wheel to record distance. These are mounted on a light trailer for towing behind a car or truck. Subsections of this chapter describe the transducer mounting member, geophone and accelerometer mounts, source mounting, and the hammer load cell construction.

Transducer Mounting

The transducer mounting member of the Surface Analyzer is a 2 in. by 6 in. U-channel that is 6 ft. long. Individual geophone and accelerometer holders are bolted on below this U-channel and electrical cables and pneumatic lines are protected by the U-channel.

The transducer mounting member is lowered by pneumatic cylinders acting as a jack. The weight of the member is counterbalanced by three prestressed, constant-force springs.
Figure 8.1 - Surface analyzer schematic
Fully retracted, the transducers will have a clearance of 12 in. for high speed travel. A partially retracted position may be included for use between closely spaced measurements that reduces raise/lower times.

Positive air pressure is required to lower the transducer mounting member so the normal position of the member is raised. In the event of electrical or pneumatic failure, the member raises so that the trailer may be safely towed to a safe repair facility.

**Geophone Mounting**

Geophones are mounted in a 2 in. wide piece of rectangular steel channel with a 4 in. by 6 in. face. Springs are used to force the geophone onto the pavement as the mounting is lowered. A cone mounted on the bottom of the geophone provides the contact point, and self-centers and levels the geophone in the mounting.

The geophone is housed in a standard waterproof case used in oil exploration. The spike typically used for planting in soils is replaced by the centering cone.

The geophone holder is bolted to the transducer mounting member with two bolts. A foam sheet is placed between the holder and mounting member to isolate vibrations. Vibration isolation tests indicate that greater than 50 db signal reduction is achieved between the geophone and the frame.

**Accelerometer Mounting**

Accelerometers are mounted in a 2 in. wide piece of rectangular steel channel with a 4 in. by 6 in. face. A stiff spring above the accelerometer forces the accelerometer onto the pavement as the mounting is lowered. A cone mounted on the bottom of the geophone provides the contact point, and self-centers and levels the accelerometer in the mounting.

The accelerometer holder is bolted to the transducer mounting member with two bolts. A foam sheet is placed between the holder and mounting member to isolate vibrations. The outside dimensions of the mounting frame and bolt spacing are identical to the geophone holder and are interchangeable.

**Source Mounting**

The high and low-frequency pneumatic hammers are mounted to vertical member attached to the transducer mounting member. The sources are raised and lowered with the transducers to
insure adequate clearance during travel. The sources are isolated from the transducer mounting member by a bonded steel-rubber U channel. The mounting permits adjustment of the stroke of the hammer which may be required to control hammer force in extreme variations of pavement conditions. The source mounting assembly has not yet been built. Consequently the level of vibration isolation has not yet been measured.

**Load Cell Construction**

Load cells are included in the high and low-frequency hammer heads to measure the applied force of the hammer hits. The load cells measure force in both extension and compression.

The current design is being reconsidered for two reasons. The first, stated in the chapter on key unresolved design issues, is that the mass of the high-frequency hammer head ideally should be reduced to enhance S/N at higher frequencies on asphalts. The second is that the stroke of the hammer requires a flexible electrical connection whose reliability could be improved by minimizing its movement.

**Pneumatic Control**

Air pressure is used in the Surface Analyzer to raise and lower the transducer mounting member, to impact the low and high-frequency hammers, and to clear loose sand from below the transducer contact points. A 12 volt compressor is used to charge an air tank (Figure 8.1) that provides air power during operation.

Tank size will be chosen so that extensive measurements may be taken without running the compressor. This permits "quiet" operation so that compressor vibrations and electrical interference do not influence pavement measurements.

The air compressor could possibly be powered by belt drive from the trailer wheel for the production model of the Surface Analyzer. The air tank would then be charged during routine measurements by the trailer motion between stations. This option is not included in the prototype design as it is easily added and is not likely to be used in the first stages of the prototype testing.

The general schematic design of the pneumatic control system is shown in Figure 8.2. The following three subsections describe the design considerations for the raising/lowering mechanism, the physical pneumatic hammers, and the feedback control of hammer characteristics.
Figure 8.2 - Schematic of pneumatic control
Raising/Lowering Mechanism

The raising and lowering of the transducer mounting member is controlled by a scissors jack activated by a pneumatic cylinder. Prestressed springs counterbalance the transducer mounting mechanism weight so that active pressure is required only to lower the mechanism. The cylinder has an electrically controlled solenoid valve that is software controlled, with a manual override switch for testing. This valve connects the cylinder to outside air when turned off, and to the high pressure line when turned on.

Pressure to the transducer lowering mechanism is provided by a mechanical regulator and should require adjustment only after major modifications or repairs.

Sources

Each pneumatic hammer consists of a computer controlled pressure regulator, an accumulator chamber, a computer controlled firing solenoid, and a spring return air cylinder (Figure 8.3). Air from the supply tank fills the accumulator through the regulator. Upon receiving a signal from the computer, the solenoid turns on and allows the air from the accumulator into the hammer cylinder. When the solenoid turns off the cylinder is connected to outside air pressure, the spring retracts the hammer, and the accumulator refills.

The solenoid valve is mounted directly on the cylinder. The accumulator is mounted as close to the valve as physically possible to minimize time delays associated with propagation of air pressure transients.

The accumulator provides a high volume (several cylinder volumes) of pressurized air that may be quickly moved into the cylinder to provide rapid hammer acceleration. This isolates the hammer movement from regulator flow restrictions and distance from the air supply.

The cycle time for the hammer stroke and preparation for the next stroke is controlled by pneumatic line size, accumulator volume, and regulator flow rate. Using conventional 1/4 inch connections and pneumatic lines cycle times are less than two seconds and are longer than the data acquisition/processing phase.

Source Feedback Control

Two factors are important in controlling the hammer hit, the force of the impact and the duration of the impact. Ideally, we would like the shortest impact time at a controlled force level. These are influenced by external conditions such as pavement or asphalt stiffness, surface condition, and temperature.
Figure 8.3 - Schematic of source control
Two controls are available over the hammer behaviour. The first is regulator pressure that controls the maximum applied force. The second is the duration of the solenoid opening that determines both applied force and pulse duration.

Figure 8.3 shows the computer feedback loop that will be used to control the hammer hit characteristics. The load cell signal is already digitized and used in calculating pavement conditions and quality control indicators. When the load cell signal quality falls outside limits the computer will adjust the solenoid opening duration, for the subsequent hit, first to increase force, decrease force, or decrease duration of the hit. If these adjustments are not adequate then the pressure is adjusted by driving a stepper motor attached to the pressure regulator. The load cell quality limits that trigger these adjustments is smaller than limits that indicate degraded signal quality.

Electronic Components

This section on the electronics of the Surface Analyzer gives a general description of the circuits that are not purchased as systems from other vendors and must be custom built. The circuits will be built on plug-in cards for the PC bus that are consistent with boards purchased from external vendors.

A general schematic of the total electronic system, including interface with the computer, is shown in Figure 8.4. Transducer signals are indicated on the left and the first level of boxes to the right indicate the first stage of analog signal conditioning. These analog signals then go into the analog multiplexer, which routes a subset of four of the twelve signals through to the programmable gain stage and into the A/D board. The distance measurement is purely digital and is input to the computer through a standard parallel printer port. The internal diagnostics circuits inject a known signal into the signal conditioning circuits so that overall circuit functions may be tested and compared with ideal responses.

Subsequent subsections of this chapter will deal with the signal conditioning functions (including temperature, accelerometer, load cell, and geophone conditioning), multiplexer and gain control circuits, distance measuring, and internal diagnostic circuits.

Signal Conditioning

Signal conditioning circuits include geophone conditioning, accelerometer and load cell conditioning, and temperature corrections. This classification is based on the level of signal the transducer produces, the output impedance of the transducer, and the type of circuit required for conditioning.
Figure 8.4 - Schematic of surface analyzer electronics
The geophone signal conditioning consists of a simple operational amplifier. This amplifier has a hardware controllable gain to account for different geophones that might be used in the system. It also converts the moderate geophone impedance to a low-impedance output signal that will not influence that analog multiplexer, and includes a low-pass filter to eliminate any high-frequency interference.

The accelerometer and load cell signal conditioning circuits consist of a single operational amplifier. This amplifier has a hardware controllable gain to account for different accelerometer sensitivities that might be used in the system. It also converts the high accelerometer and load cell impedances to a low-impedance signal that will not influence the analog multiplexer. It also includes a low-pass filter to eliminate high frequency interference and aliasing in digitization.

The temperature measurement circuit consists of two thermocouples and cold junction compensation. This circuit serves as a reference potential and a very high input impedance amplifier so that the analog multiplexer does not load the measurement. This signal conditioning circuit is shown in Figure C.3. Temperature sensitive resistors are also being considered for the temperature measurement for their lower cost.

**Multiplexer and Gain Control**

As the A/D converter used in the Surface Analyzer handles only four channels, it is necessary to route subsets of the twelve input signals under computer control. The analog multiplexer takes a two bit control signal (permitting selection of up to four banks) and routes one bank of four input signals to the A/D. The multiplexer is basically an electronically controlled set of switches that are controlled by the parallel I/O board in the computer.

Since the high-frequency load cell and one accelerometer are used in multiple measurements these signals are wired to be chosen in two banks. The temperature measurements occupy their own bank. The low-frequency measurements occupy one bank.

Signal strengths from the transducers may vary widely with pavement conditions so computer controlled programmable gain is included between the multiplexer and the A/D board. In order to take advantage of the full precision of the A/D board the gain will be recursively adjusted based on previous signal strengths to give the best possible S/N ratio.

**Distance**

Distance measurement is accomplished by counting revolutions of a fifth wheel. An optical
or magnetic encoder is connected to the wheel and transmits two pulsed signals with rotation of the wheel. The two signals are phased differently so that electronic circuits can determine the direction of rotation. The pulses and rotation direction are fed into an up/down counter which can be read into the computer giving an uncalibrated distance.

Calibration of the distance will be performed in software so that it is easily performed and arbitrary units may be used. In addition, setting an initial distance value and resetting of distances are done in software.

**Internal Diagnostics**

To assist in interchannel calibration and troubleshooting malfunctions the capability to inject a known signal into the input of the signal conditioning circuitry is included. This function tests the signal conditioning, analog multiplexer, programmable gain, and A/D boards for proper functioning. Implicitly, this also tests for failure of a transducer.

The data acquisition software has the option to switch signal conditioning inputs to a step function generator if there is a need to test circuit operation. The resulting digitized waveform may be compared with ideal responses for each channel to help diagnose failures.

**Computer Specifications**

The user interface software of the Surface Analyzer (production version) will run on an IBM-PC XT or AT equivalent computer with Hercules, CGA, or EGA/VGA graphics, 640Kb ram, two 720Kb floppy disk drives, with one parallel port and one serial port. An AT with hard disk and a floating point chip would be highly desireable, but not necessary, to increase speed of operation in analysis intensive operations.

The data acquisition software and hardware will run in an IBM-PC AT equivalent machine with 2Mb ram, two parallel ports, and four expansion slots. As described in Chapter 9 on key design issues, marginal performance/cost ratio improvements are not yet apparent. The data collection and acquisition speed will be directly related to hardware cost.

**Data Acquisition Hardware**

Data acquisition functions are concerned with selecting measurement channels, digitizing waveforms, digital signal processing, and analyzing the collected data. Selecting channels and digitizing data require both hardware and software while signal processing and analysis are software functions.
Data acquisition for the Surface Analyzer requires digitization of six high-frequency signals at a 200 KHz sample rate, four low-frequency signals at 10 KHz, and two temperature signals at a low frequency. Since data transfer rates on the PC bus are too low to directly record more than one high-frequency signal, signals must be digitized into local memory on the A/D board which is then transferred to the PC after collection.

The best performance/cost performance A/D board we could find meeting the requirements is the Metrabyte DAS-50 model. This digitizes four channels at 250 KHz into onboard memory. The four channel limit requires that transducer signals be multiplexed into the A/D board. A Metrabyte PIO-12 board with three parallel I/O channels was added to control the multiplexer, gain ranging, and other pneumatic control functions.

Data Acquisition Software

Software for data acquisition is designed around the concept of message passing with the user interface software. The function of the data acquisition software is to interpret these messages, acquire the requested data, process it, and return messages to the user interface containing the results. In addition, the data acquisition software also must handle calibration of the hardware, quality control tests on the collected data, and diagnostics of hardware functions.

Figure 8.5 shows the flow chart for the highest abstract level of the data acquisition software. Upon boot-up of the computer the loop will be entered at the I/O and A/D initialization functions. Following initialization, the software will wait for a start of message from the user interface and acknowledge receipt of the start of message. The acquisition software then reads in subsequent data collection messages from the user interface, sorts, and interprets the messages. It acknowledges receipt of the end of messages by sending busy messages to the user indicating the tasks being performed. Following completion of collection and processing the acquisition software returns messages to the user with the requested results. Initialization of I/O and A/D hardware is made again and the wait for messages begun again. Subsequent paragraphs and flow charts expand on the functions shown in Figure 8.5.

Figure 8.6 shows a flow chart for message reading and sorting functions. After reading a message its type is determined. Messages may be either data collection, hardware setup, or an end of message. Data collection messages are sorted into bins that determine the time sequence of functions and the multiplexer bank and source to be used. Setup messages configure tables used in collection and processing and override default values in the software.
Figure 8.5 - Data acquisition software structure
Figure 8.6 - Read and sort data collection option
Key Unresolved Design Issues

In design of the Surface Analyzer, some testing of individual hardware and software components has been made. This chapter describes the potentially weak areas that we have identified in the components tested to date, and in what we expect to find in the complete system when constructed.

We have identified four areas of uncertainty that are discussed in subsequent sections. The first area is the coupling of accelerometers to the pavement surface under a wide variety of surface conditions. The second is in the performance of the high frequency source under varying surface conditions. The third is the tradeoffs between cost and performance of the complete system.

Accelerometer Mounting and Coupling

We have constructed and tested several accelerometer mountings and have found a design held down by spring force that gives useable signal-to-noise (S/N) levels under our test conditions. We do not consider our safety margin to be as large as will be necessary to handle the wide variety of pavement conditions to be found outside the laboratory.

The key aspect of accelerometer coupling is the frequency of resonance of the mounting, and may be viewed as a spring-mass-damper system. The contact point of the accelerometer with the pavement is viewed as having a certain spring constant and damping, while the mass of the system is determined by the accelerometer and mounting mass. The ideal case is to have
the contact resonance at a frequency higher than that to be measured (50 KHz in the Surface Analyzer) but this is well above the resonant frequency of the accelerometer (10 KHz) and is not achievable.

Our current design uses a very stiff spring to force a cone onto the surface, and has a resonance at about 4 KHz. The presence of loose sand on the pavement surface is our current major concern with this design. The loose sand creates a low contact stiffness and lowers the resonant frequency. Using a more pointed tip on the accelerometer to move sand away also gives a lower mounting stiffness and lowers the resonant frequency.

The measurements made by the Surface Analyzer are quite insensitive to the coupling resonance as long as the S/N ratio remains high. The amplitude of signals arriving higher than the resonant frequency are decreased, and a phase delay is added to that frequency. Approximately one octave above the resonant frequency the phase delay becomes a constant. The major effect is on the SASW where phase delays around the coupling resonance are not consistent between adjacent transducers, but the interpretation algorithm we use does not use these unreliable data points.

We are currently testing a new version of the accelerometer coupling that greatly reduces the accelerometer mass. We are also planning to use air jets to move loose debris from under the accelerometer as it is lowered. Within the current design, the modified accelerometer mounting is replaceable with two bolts and adds no cost.

Source Tests

At our current level of hardware testing, the high-frequency hammer described in Section 4 appears to give adequate energy to 50 KHz. We have not yet tested this hammer on soft asphalt where high frequencies are attenuated rapidly.

The frequency content of the hammer hit depends strongly on the stiffness of the pavement and the mass of the hammer head. On stiff pavements and/or with a low hammer mass, high frequencies are preferentially generated. We are currently evaluating several designs to reduce the hammer mass to guarantee adequate S/N ratios at high frequencies should the existing design be inadequate.

Speed/Cost Tradeoffs

The current design is strongly motivated by attempting to meet the target of a prototype replication cost of $10,000. Additional criteria are that the time of one measurement be less than one minute and that all data reduction and quality assurance parameters be completed.
within that one minute time period.

The time period required for a measurement is one of the major criteria for determining the operating cost of the Surface Analyzer. Consideration of the assumptions behind that time estimate, and of the margin of time reduction with expenditure on capital equipment are important aspects of understanding the overall costs to be associated with the Surface Analyzer.

Meeting the target cost of $10,000 has required design decisions that increase acquisition time. The important bottleneck in the current design lies in data processing speed, where an increase in $3,000 applied to computer processing would reduce data collection time to about 40 seconds. An increase of $5,000 applied to data acquisition hardware and computer processing would reduce acquisition time to approximately 30 seconds.

The assumption has also been made that the Surface Analyzer data collection will not be considered complete until all five measurements, with quality assurance estimates, have been completed. Should a lower standard of reliability be acceptable, or should fewer measurements be taken, time per station will be proportionately lower.
Appendix A

Wave Propagation Theory

For engineering purposes, most pavement sections can be approximated by a layered half-space with reasonable accuracy. With this approximation, the profiles are assumed to be homogeneous and to extend to infinity in two horizontal directions; while being heterogenous in the vertical direction. This heterogeneity is often modelled by a number of layers with constant properties within each layer. In addition, it is assumed that the material in each layer is elastic and isotropic.

In this Appendix, the principals of wave propagation are briefly introduced. Relationships between wave velocities and moduli are clarified.

Seismic Body Waves

Wave motion created by a disturbance within an ideal whole-space can be described by two kinds of waves: compression waves and shear waves. These waves are collectively called body waves as they travel within the body of the medium. Compression and shear waves can be distinguished by the direction of particle motion relative to the direction of wave propagation.

Compression waves (also called dilatational waves, primary waves, or P-waves) exhibit a push-pull motion. As a result, wave propagation and particle motion are in the same direction, as shown in Figure A.1a. Compression waves travel faster than the other types of waves; hence, appear first in a direct travel time record.

Shear waves (also called distortional waves, secondary waves or S-waves) generate a shearing motion, which causes particle motion to occur perpendicular to the direction of wave propagation as shown in Figure A.1b. Shear waves can be polarized. If the directions
Figure A.1 - Characteristic motion of seismic waves (from Bolt, 1976)
of propagation and particle motion are contained in a vertical plane, the wave is said to be vertically polarized. This wave is called an SV-wave. However, if the direction of particle motion is perpendicular to a vertical plane containing the direction of propagation, the wave is said to be horizontally polarized. This wave is termed an SH-wave. Shear waves travel slower than P-waves and thus appear as the second major wave type in a direct travel time record.

Seismic Surface Waves

In a half-space, other types of waves occur in addition to body waves. These waves are called surface waves. Many different types of surface waves have been identified and described. The two major types are Rayleigh waves and Love waves.

Surface waves propagate near the surface of the half-space. Rayleigh waves (R-waves) propagate at a speed of approximately 90 percent of S-waves. Particle motion associated with R-waves is composed of both vertical and horizontal components, which, when combined, form a retrograde ellipse close to the surface (see Figure A.1d). However, with depth R-wave particle motion changes to pure vertical and finally, to a prograde ellipse as illustrated in Figure A.2. The amplitude of motion attenuates quite rapidly with depth. At a depth equal to about 1.5 times the wavelength, the vertical component of the amplitude at the ground surface.

Particle motion associated with Love waves is confined to a horizontal plane and is perpendicular to the direction of wave propagation as shown in Figure A.1c. This type of surface wave can only exist when low-velocity layers are underlain by higher-velocity layers because the waves are generated by total multiple reflections between the top and bottom surfaces of the low-velocity layer. As such, Love Waves are not generated in pavement sections.

The propagation of body waves (shear and compression waves) and surface waves (Rayleigh waves) away from a vertically vibrating circular source at the surface of a homogeneous, isotropic, elastic half-space is shown in Figure A.3. Miller and Pursey (1955) found that for the situation shown in Figure A.3, approximately 67 percent of the input energy propagates in the form of R-waves. Shear and compression waves carry 26 and 7 percent of the energy, respectively. Compression and shear waves propagate radially outward from the source. R-waves propagate along a cylindrical wavefront near the surface. Although, body waves travel faster than surface waves, body waves attenuate in proportion to $1/r^2$, where $r$ is the distance from the source. Surface wave amplitude decreases in proportion to $1/r^{0.5}$. 

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Figure A.2 - Amplitude and particle motion distribution with depth for Rayleigh waves
Figure A.3 - Distribution of Rayleigh, shear and compression wave displacement from a circular footing on a homogeneous, isotropic, elastic half-space (from Richart et al, 1970)
Seismic Wave Velocities

Seismic wave velocity is defined as the speed that a wave advances in the medium. Wave velocity is a direct indication of the stiffness of the material; higher wave velocities are associated with higher stiffness. By employing elastic theory, compression wave velocity can be defined as:

$$V_p = \sqrt{\frac{\lambda + 2G}{\rho}} \quad (A.1)$$

where,
- $V_p$ = compression wave velocity,
- $\lambda$ = Lame’s constant,
- $G$ = shear modulus, and
- $\rho$ = mass density.

Shear wave velocity, $V_s$, is equal to:

$$V_s = \sqrt{\frac{G}{\rho}} \quad (A.2)$$

Compression and shear wave velocities are theoretically inter-related by Poisson’s ratio.

$$\frac{V_p}{V_s} = \left[ \frac{1 - \nu}{0.5 - \nu} \right]^{0.5} \quad (A.3)$$

where $\nu$ is the Poisson’s ratio. A graphic illustration of Eq. A.3 is shown in Figure A.4. From this figure it can be seen that, for a constant shear wave velocity, compression wave velocity increases with an increase in Poisson’s ratio. For a $\nu$ of zero, the ratio is $V_p$ to $V_s$ is equal to $\sqrt{2}$, and for a $\nu$ of 0.5 (an incompressible material), this ratio is to infinity.

For a layer with constant properties, R-wave velocity and shear wave velocity are related by Poisson’s ratio as well. Although, the ratio of R-wave to S-wave velocities increases as Poisson’s ratio increases, the change in this ratio is not significant as shown in Figure A.5. For Poisson’s ratio of zero and 0.5, this ratio changes from approximately 0.86 to 0.95,
Figure A.4 - Theoretical elastic relationship between Poisson's ratio and the ratio of compression to shear wave velocity
Figure A.5 - Theoretical elastic relationship between Poisson's ratio and the ratio of Rayleigh to shear wave velocity
respectively. Therefore, it can be assumed that the ratio is equal to 0.90 without introducing an error larger than about five percent.

Equation A.3 can be rewritten as:

\[ v = \frac{0.5 \left( \frac{V_p}{V_s} \right)^2 - 1}{\left( \frac{V_p}{V_s} \right)^2 - 1} \]  \hspace{1cm} (A.4)

This equation can be used in the calculation of Poisson’s ratio once \( V_s \) and \( V_p \) are known.

**Elastic Constants**

Propagation velocities per se have limited use in engineering applications. In pavement engineering, Young’s moduli of the different layers should be measured. Calculation of elastic moduli from propagation velocities is, thus, important.

Shear wave velocity, \( V_s \), is used to calculate shear modulus, \( G \), by:

\[ G = \rho \cdot V_s^2 \]  \hspace{1cm} (A.5)

in which \( \rho \) is the mass density. Mass density is equal to \( T_i/g \), where \( T_i \) is total unit weight of the material and \( g \) is gravitational acceleration. If Poisson’s ratio ( or compression wave velocity) is known, other moduli can be calculated for a given \( V_s \). Young’s and shear moduli are related by:

\[ E = 2G(1+v) \]  \hspace{1cm} (A.6)

or,

\[ E = 2\rho V_s^2(1+v) \]  \hspace{1cm} (A.7)

In a medium where the material is restricted from deformation in two lateral directions, the
ratio of axial stress to axial strain is called constrained modulus. Constrained modulus, $M$, is defined as:

$$M = \rho v_p^2$$  \hspace{1cm} (A.8)

or in terms of Young's modulus and Poisson's ratio:

$$M = \frac{(1-v)E}{(1+v)(1-2v)}$$  \hspace{1cm} (A.9)

Bulk modulus, $B$, is the ratio of hydrostatic stress to volumetric strain and can be determined by:

$$B = M - 4/3G$$  \hspace{1cm} (A.10)
Appendix B

Factors Affecting Elastic Moduli

Soil (or Subgrade)

Based upon numerous laboratory tests, Hardin and Drnevich (1972) proposed many parameters that affect the moduli of soils. These factors, along with their degree of importance in affecting moduli, are tabulated in Table B.1. They suggested that state of stress, void ratio, and strain amplitude are the main parameters affecting moduli measured in the laboratory. However, for this study dealing with measurement of moduli in situ, the main factors affecting the elastic moduli and wave velocities are void ratio and state of stress (confining stress).

Strain amplitude has essentially no effect on the in situ tests because the measurements are performed at very low strains. Up to a strain amplitude of about 0.01 percent, moduli are nearly constant, with a slight decrease in the range from 0.001 to 0.01 percent. This constant modulus is called the elastic modulus, or maximum modulus. Above a strain level of 0.01 percent, moduli decrease significantly. A typical example of the variation in Young’s modulus, E, with normal strain, ε, for a stiff clay is shown in Figures B.1 and B.2. An undisturbed sample of stiff clay from San Antonio, Texas was tested using the resonant column method (Richard, et al, 1970). The variation of E with log ε at several confining pressures is shown in Figure B.1. As the confining stress increases, the low-amplitude modulus increases, as shown in this figure. Also, it is evident that below strain levels of 0.001, E is constant and independent of strain at each pressure.

The effect of strain on modulus is easily seen by plotting the variation of normalized modulus, $E/E_{\text{max}}$, versus log ε as shown in Figure B.2. In this figure, $E_{\text{max}}$ is taken as the maximum value of Young’s modulus at each confining pressure. It can be seen that normalized modulus is constant below a strain of about 0.001 percent and is equal to $E_{\text{max}}$. Also all the modulus-strain curves are nearly independent of confining pressure once they are
Figure B.1 - Variation in Young's modulus with strain amplitude at different confining pressures of an unsaturated clay subgrade.

Figure B.2 - Variation in normalized Young's modulus with strain amplitude of an unsaturated clay subgrade.
normalized. If a normalized modulus-strain curve such as that shown in Figure B.2 is available for the material, then moduli at higher strains can be determined once $E_{\text{max}}$ has been measured. Similar trends also occur with shear moduli as shown in Figure B.3 for a soft clay from the San Francisco bay area.

Table B.1 - Parameters Affecting Shear Modulus (from Hardin and Drnevich, 1972)

<table>
<thead>
<tr>
<th>Parameter</th>
<th>Importance$^a$</th>
<th>Clean Sands</th>
<th>Cohesive Soils</th>
</tr>
</thead>
<tbody>
<tr>
<td>Strain Amplitude</td>
<td>V</td>
<td>V</td>
<td></td>
</tr>
<tr>
<td>Effective Mean Principal Stress</td>
<td>V</td>
<td>V</td>
<td></td>
</tr>
<tr>
<td>Void Ratio</td>
<td>V</td>
<td>V</td>
<td></td>
</tr>
<tr>
<td>Number of Cycles of Loading</td>
<td>R$^b$</td>
<td>R</td>
<td></td>
</tr>
<tr>
<td>Degree of Saturation</td>
<td>R</td>
<td>V</td>
<td></td>
</tr>
<tr>
<td>Overconsolidation Ratio</td>
<td>R</td>
<td>L</td>
<td></td>
</tr>
<tr>
<td>Effective Strength Envelope</td>
<td>L</td>
<td>L</td>
<td></td>
</tr>
<tr>
<td>Octahedral Shear Stress</td>
<td>L</td>
<td>L</td>
<td></td>
</tr>
<tr>
<td>Frequency of Loading</td>
<td>R</td>
<td>R</td>
<td></td>
</tr>
<tr>
<td>Other Time Effects (Thixotropy)</td>
<td>R</td>
<td>L</td>
<td></td>
</tr>
<tr>
<td>Grain Characteristics</td>
<td>R</td>
<td>R</td>
<td></td>
</tr>
<tr>
<td>Soil Structure</td>
<td>R</td>
<td>R</td>
<td></td>
</tr>
<tr>
<td>Volume Change Due to Shear Strain</td>
<td>V</td>
<td>R</td>
<td></td>
</tr>
</tbody>
</table>

a) V means Very Important, L means Less Important, R means Relatively Unimportant, and U means Relative Importance is not known at the time.

b) Except for saturated clean sand where the number of cycles of loading is a less Important Parameter.
Figure B.3 - Variation in normalized shear modulus with shearing strain and confining pressure (from Stokoe and Lodde, 1978)
Several studies performed on Poisson’s ratios of different soils show that they are strain dependent. Chen (1948) reported Poisson’s ratio as low as 0.1 for small-strain measurements on sand. Kriek (1977) reported the variation of Poisson’s ratio with strain based upon measurements in unconfined compression tests. His data are shown in Figure B.4. The range of Poisson’s ratio in this figure is from 0.10 for near zero strain to 0.50 for 10 percent strain. Hardin (1978) recommends using values for Poisson’s ratio between zero and 0.20 for low-strain tests, with a mean value of 0.12.

Base and Subbase

Two types of base and subbase materials are usually used, granular or treated materials. Granular base and subbase materials demonstrate the same characteristic as natural soil deposits. The base or subbase material are sometimes treated by additives such as cement, bitumen, or lime. In this situation, the moduli of the layer depends also on factors such as type of aggregate, percentage of additive as well. Typical values of Young’s moduli for granular and treated layers are in the range of 15 to 110 ksi and 50 to 2000 ksi, respectively. Poisson’s ratio of these materials are in the order of 0.20 to 0.45 (Yoder and Witczak, 1975).

Asphaltic Concrete

The main factor that affects the moduli of asphaltic materials, besides the mixture properties, is temperature. Typical variation of elastic moduli with temperature is shown in Figure B.5 for a bituminous sample. As the temperature increases the material behaves more viscously resulting in a decrease in the modulus (Van der Poel, 1954). The age of the material affects the modulus; with time, asphaltic materials become stiffer. The other factor that has some effect on the asphaltic material is the level of strain (or stress). The variation of modulus with strain level is similar to the effect of temperature; that is, the modulus decreases with increase in strain level. The asphaltic material should behave somewhat like the soil samples shown in Figures B.2 and B.3. Typical values of Young’s modulus and Poisson’s ratio of asphaltic material are in the range of 200 to 1100 ksi and 0.25 to 0.50, respectively. Total unit weight of this material is on the order of 125 to 145 pcf.

Concrete Material

The factor that affects the modulus of concrete, ignoring the method of preparation and curing, may be the strain level. However, the concrete used in overlay of roads and runways are of high quality and are very stiff, and it is expected that it behave elastically under most of the loads imposed by vehicular traffic. The elastic modulus of concrete as
Figure B.4 - Variation of Poisson's ratio with strain for sedimented kaolinite tested in unconfined compression
Figure B.5 - Effect of temperature on Young's modulus of asphaltic concrete material (inferred from Van der Poel, 1984)
reported by Yoder and Witczak (1975) ranges from 3000 to 6000 ksi and Poisson’s ratio varies from 0.10 to 0.25. The unit weight of concrete is typically 140 to 150 pcf.
Appendix C

Effects of Moisture Content on Modulus

It has been recognized for many years that the major effect of moisture in soil on the modulus is in the contribution of that moisture to the mass density of the sand (Richart, Hall & Woods, 1970). The work leading up to that conclusion, however, did not examine in detail the ranges of water content in which capillarity is operative, i.e. relatively low water contents. Recently two studies at the University of Michigan (Wu, et al, and Qian, 1990) addressed this matter. The results from these studies are summarized herein.

Basic Effects of Capillarity

It is well known that small amounts of water in soil has the affect of creating pressure in the soil skeleton. The capillary rise of moisture above the phreatic line develops stress in the soil skeleton. This stress is in addition to that caused by the self weight of the soil (geostatic stress).

Wu et al (1984) first showed that there is an "optimum water content or degree of saturation" for maximum capillary effects for each soil as determined from shear modulus, (see Figure C.1). The increase in shear modulus shown in Figure C.1 in the 10 % to 20% saturation range cannot be attributed exclusively to changes in mass density.

The basic equation relating shear wave velocity and shear modulus is as follows:

\[ G = V_s^2 \times \rho \]  \hspace{1cm} (C.1)

where \( V_s \) is the shear wave velocity
\( G \) is the shear modulus, and
\( \rho \) is the mass density.
Figure C.1 - Low-amplitude shear modulus versus degree of saturation for glacier way silt (from Wu et al, 1984)

Figure C.2 - $G/G_{o(dry)}$ versus degree of saturation for glacier way silt (from Wu et al, 1984)
If mass density were the dominant parameter, the shear modulus would increase smoothly as the degree of saturation increased; but in Figure C.1, a sharp peak is noted in the curve relating shear modulus ($G_o$) and degree of saturations ($S_o$).

It is very helpful to normalize that increase in $G_o$ by dividing by the shear modulus of the dry sand. This relationship is shown in Figure C.2. A maximum shear modulus ratio up to 2.0 is shown for the Glacier Way Silt at a confining pressure of 3.6 psi (518 psf). This represents a rather shallow depth in most soils. As the confining pressure increases, the shear modulus ratio decreases, as also shown in Figure C.2. At a confining pressure of 14.2 psi the maximum modulus ratio is 1.6. According to this trend, there is a confining pressure or depth at which the capillary affect is overshadowed by the geostatic stress and no longer has an important influence on shear modulus.

Another way of looking at the effects of capillarity is by plotting the increase of confining pressure over and above the ambient value required to produce the increase in shear modulus. An example of this is shown in Figure C.3. It was further learned that the maximum modulus ratio is a function of the $D_{50}$ particle size. Figure C.4 shows that the finer soils have lesser maximum modulus ratios. It is also clear in Figure C.4 that the maximum modulus ratio decreases with confining pressure.

The optimum degree of saturation for maximum modulus ratio can be predicted for the soils presented in Figure C.4. Figure C.5 shows the relationship between the $D_{50}$ size of the soil and the optimum degree of saturation as determined from the five soils tested. The soils tested represent a variety of grain size distributions, as shown in Figure C.6, but do not represent all possible granular soils which can be influence by capillarity.

A method of estimating both the optimum degree of saturation and the maximum modulus ratio for a sand with any particular grain size distribution is required. Qian (1990) show that the grain shape plays an important role in determining the capillary effects. One the characteristics of each size fraction of particles with a common shape are known, the characteristics of a mixture of those particles can be predicted. For example, the modulus ratio vs degree of saturation for various grain fractions of an angular and are shown in Figure C.7.

It was also found that the optimum degree of saturation for maximum modulus ratio depended principally on void ratio and on the minus #400 sieve fraction, $C_s$, for example, Figure C.8. It is quite striking how the minus #400 sieve fraction influences the optimum degree of saturation. The soils used in the studies on which the conclusions are based are basically quartz sands and silts for which the fines were non-plastic. Another way of presenting the data from Figure C.8 is shown in Figure C.9. Again the importance of the minus #400 size is shown.
Figure C.3 - Additional effective stress due to capillarity versus degree of saturation for (from Wu et al, 1984)

Figure C.4 - Relationship between maximum value of $G_v/G_{v(\infty)}$ and $D_{10}$ for five test materials (from Wu et al, 1984)
Figure C.5 - Relationship of optimum degree of saturation versus $D_{10}$ for five test materials (from Wu et al., 1984)

Figure C.6 - Grain size distribution of soils tested
Figure C.7 - \( G/G_{(dry)} \) versus degree of saturation for angular sand with \( Cs = 0.15 \) (from Qian, 1990)

Figure C.8 - Optimum degree of saturation versus content of minus #400 sieve size fraction at various void ratio for angular sand (from Qian, 1990)
Figure C.9 - Optimum degree of saturation versus void ratio for angular sand with various Cs (from Qian, 1990)

Figure C.10 - $G_{max}/G_{dry}$ versus grain diameter for angular and subrounded sands (from Qian, 1990)
An empirical equation relating the optimum degree of saturation to the void ratio and $C_s$ has been derived as follows (Qian, 1990):

$$(S_e)_{opt} = a^e + b$$  \hspace{1cm} (C.2)

where, $\ e = \ \text{void ratio, and} \ a \ \& \ b \ \text{are empirical constants shown in Table C.1.}$

A comparison of the maximum modulus ratio vs. grain size, $C_s$, and grain shape is shown in Figure C.10 for one void ratio and confining pressure. This demonstrates the difference in behavior due to grain shape.

An empirical relation for the maximum modulus ratio also has been derived (Qian, 1990):

$$\frac{G_{max}}{G_{dry}} = (\frac{G_{max}}{G_{dry}})_{0.78} \cdot k \cdot (e - 0.78)$$  \hspace{1cm} (C.3)

where, $k$ is a constant depending on grain shape, and $G_{dry}$ can be estimated from Hardin's equations.

**Table C.2 - Summary of empirical constants**

<table>
<thead>
<tr>
<th></th>
<th>Angular Sand</th>
<th>Subrounded Sand</th>
</tr>
</thead>
<tbody>
<tr>
<td>$a$</td>
<td>81.24 $\sigma_o^{0.331} \cdot D^{0.185}$</td>
<td>53.35 $\sigma_o^{0.343} \cdot D^{0.118}$</td>
</tr>
<tr>
<td>$b$</td>
<td>3.80$\sigma_o^{-0.209} \cdot D^{0.288} \cdot e^{-0.319}$</td>
<td>2.56$\sigma_o^{-0.154} \cdot D^{0.165} \sigma_o^{0.217}$</td>
</tr>
<tr>
<td>$k$</td>
<td>3.09$\sigma_o^{-0.244}$</td>
<td>2.79$\sigma_o^{-0.130}$</td>
</tr>
<tr>
<td>$a$</td>
<td>1.025</td>
<td>0.450</td>
</tr>
<tr>
<td>$b$</td>
<td>0.0211 (0.00391$e^{+1.1}$ + 4.80)</td>
<td>0.0878 (0.00177$e^{+1.1}$ + 4.45)</td>
</tr>
<tr>
<td></td>
<td>12.3 - 0.106 (1291$e^{+1.1}$)</td>
<td>11.7 - 0.344 (42.6$e^{+1.1}$)</td>
</tr>
</tbody>
</table>

The void ratio 0.78 was arbitrarily chosen because of the ease of preparing samples in the laboratory at that value.
Any given soil may be composed of many particle sizes which can be separated into segments of sample as shown in Figure C.11. Each segment of that sample resists torque in proportion to the shear modulus of that segment, represented schematically in Figure C.12. By summing up the contributions of each segment in proportion to its size in the sample ($\Theta_i$), the shear modulus of the composite sample can be obtained from Equation C.3 as follows:

$$\left(\frac{G_{\max}}{G_{\text{dry}}}ight)_{0.78} = \Sigma \alpha_i Y_i$$  \hspace{1cm} (C.4)

where,

$$\alpha_i = \left[\frac{(A^{1-c_s}-1)/(A-1)}{b_c} + \left[1 + \frac{(A^{1-c_s}-1)/(A-1)}{b_1}\right] \right]$$

$$Y_i = \Theta_i/360,$$

and,

A, $b_c$, $b_1$, and k used in Equation C.2, and a and b used in Equation C.1 are empirical constants defined in Table C.1. Qian (1990) recommends a relative simple step-by-step procedure for determining these empirical factors.
Figure C.11 - Schematic of soil sample added some minus #400 sieve size fractions (from Qian, 1990)
Figure C.12 - Soil sample subjected to torque
(from Qian, 1990)
References


