Development of a High Speed Rigid Pavement Analyzer – Phase I Feasibility and Need Study

National Concrete Pavement Technology Center
Tech Center

Final Report
May 2008

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IOWA STATE UNIVERSITY
Institute for Transportation

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16. Abstract

Objectives: The long-term goal of this effort is to develop a device that will measure, at highway speeds, pavement structural condition which is known to impact the performance of PCC and composite pavements. Such a system needs to be able to process the collected measurements into an immediately usable form in near real-time. It is believed that the recent improvements in computers, laser measuring devices, and similar technologies will make the development of such a system possible. The goal of the work specifically described herein is to, in fact, determine if the required technologies exist and to what extent a market exists for such a device. Scope: The proposed research project will investigate the feasibility and practicality of developing a device for measuring the structural condition of PCC and composite pavements.

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DEVELOPMENT OF A HIGH SPEED RIGID PAVEMENT ANALYZER – PHASE I FEASIBILITY AND NEED STUDY

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May 2008

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1. INTRODUCTION

1.1 Need and Goals

The national pavement system is a remarkable asset. Attributes of the pavement system that make it so remarkable include both the shear size and the performance expectations associated with it. Management of such a large, complex, and important asset has many challenges and those challenges exist and change throughout the pavement's life. One of the most challenging aspects of owning/managing a pavement system is determining optimum times to perform maintenance, repair, or replacement. Clearly the first step in making such decisions is to understand how the pavement is performing relative to its expected use.

Nondestructive evaluation (NDE) is a key and important tool for effective management of infrastructure assets of all types and the management of pavements is no different. NDE technologies allow owners to more completely determine the condition of assets and to forecast when repair/replacement will be needed. Principally the current state-of-the-practice for rigid and composite pavements is to evaluate pavements at the project level (i.e., a single section of pavement at a time). While these assessments, which are usually made with a falling weight deflectometer (FWD), provide valuable information, the information is limited to discrete sections that are typically already suspected of needing attention. Making FWD measurements on a network level (i.e., system wide) is limited by many factors, including: time, cost, safety to testing personnel, safety of the travelling public, and others.

Although the above mentioned limitations exist, it is also recognized that there is a need to collect information on the performance of more miles of pavement. This need is the driving force behind the work described herein. The specific goal of this project was to investigate the feasibility of developing a system level technique or technology for measuring the structural performance of rigid and composite pavements. Similar systems exist for the measurement of flexible pavements (note that these systems exist in primarily experimental forms). However differences in behavior between pavement types limits the applicability of currently available systems to rigid and composite pavements.

1.2 Scope

The scope of this work consisted of four principal components. The first component consisted of a review of the currently available systems (mentioned above) that have been used on flexible pavements. The second component consisted of the high-level development of system configurations. This development was not intended to be a specific system configuration but rather to provide a basic idea as to what the measurement platform might consist of. The third component was a review and high-level evaluation (both theoretical and experimental as was practical) of available measurement technologies. This evaluation was intended to determine if required measurement qualities exist in available technologies. The final component of this work consisted of an evaluation of the market need for such a measurement system. This component was intended to determine if roadway owners would utilize such a system if it was available.

1.3 Report Content

This report is divided into six chapters. Chapter 2 summarizes the research teams understanding of the types of measurements needed for such a system to be useful. Conceptual system configurations are discussed in Chapter 3. The technologies identified are presented in Chapter 4 along with summaries of the evaluations completed. Chapter 5 presents the previously mentioned market need study and Chapter 6 presents the project summary and recommendations of the work.

2. MEASUREMENT NEEDS AND STATE-OF-THE-PRACTICE

This chapter describes the types of measurements that are desired from a device that would measure the structural performance of a rigid pavement. Keeping in mind that the goal was to develop a network level device, the research team determined that the primary measurements of importance are the pavement system modulus and joint load transfer efficiency (LTE). Each of these two measurement needs are described in the following sections. For background, a brief summary of currently available techniques for making similar measurements on either flexible pavements or on rigid pavements at low-speeds is given.

2.1 Measurement Needs

NDE plays an important role in pavement management. Various methods have been developed over the last several decades for testing pavements. These methods can generally be categorized as either seismic-based or deflection-based methods. Seismic-based methods measure the velocities at which low-strain waves propagate through the pavement. Deflection-based methods, such as the FWD test mentioned previously, involve applying a large force to the pavement and measuring the induced deflections. Both methods are used to determine the elastic properties of the pavement. These elastic properties are then used to predict the pavement capacity and remaining life (Bay and Stokoe, 1998).

The modulus is one of the key properties that determines the strains and displacements in a pavement structure. Several NDE approaches are available to determine the modulus of paving materials in existing pavements. These systems or methods can be categorized by the pavement properties that are measured by the test equipment.

- Pavement structural response—Deflection based testing.
- Material elastic response— Resonant Frequency Test (ASTM C215), Ultrasonic pulse velocity (ASTM C516), Spectral Analysis of Surface Waves (SASW), and Impact-Echo testing.

2.1.1 Pavement System Modulus

FWD equipment measures pavement surface deflections from an applied dynamic load that simulates a moving wheel (FAA 2004). During operation, a variable load, usually 40 kN (9,000 lb), is constrained to fall vertically under gravity onto a 30 cm (11.82 in.) diameter spring-loaded plate resting on the pavement surface (Sharma and Stubstad, 1980). The response of the pavement to impulse loading is normally measured with a set of velocity transducers (geophones) placed at different radial distances from the center of the plate. The FWD surface deflections (e.g., D_0, D_8, D_{12}, D_{18}, D_{24}, D_{36}, D_{48}, D_{60}, and D_{72}) are often collected at several different locations, at the drop location (0) and at radial offsets of 203-mm (8-in.), 254-mm (12-in.), 457-mm (18-in.), 610-mm (24-in.), 914-mm (36-in.), 1219-mm (48-in.), 1524-mm (60-in.), and 1829-mm (72-in.). Figure 1 displays a schematic view of FWD deflection basin and typical sensor spacings.

Sensor Offsets

Sensor Arrangement									
(mm)	0	203	305	457	610	915	1,220	1,525	1,830
(in)	0	8	12	18	24	36	48	60	72
Uniform Spacing	✓		✓		✓	✓	✓	✓	✓
SHRP Specification	✓	✓	✓	✓	✓	✓		✓	

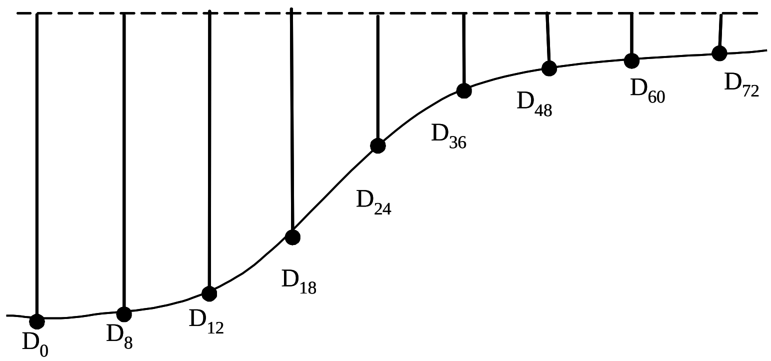


Figure 1. FWD Sensor Spacings and Schematic View of the FWD Deflection Basin

A significant amount of research has focused on algorithms to interpret deflection data from specific structures. Backcalculation is the "inverse" problem of determining material properties of pavement layers from its response to surface loading. Typically, FWD deflection measurements are used to backcalculate the in-situ elastic moduli of each pavement layer. The backcalculated moduli themselves provide an indication of layer condition. They can also be used in an elastic layered or finite element program to predict critical pavement responses (stresses, strains and deflections) under applied loads.

Numerous backcalculation techniques have been developed based on both static and dynamic analysis. The fundamental discrepancies among the developed backcalculation models arise from the type of forward response model (linear or nonlinear; static or dynamic) and the optimization procedure (least squares, database search method, etc.) carried out for the determination of appropriate layer modulus values (Ullidtz, 2000; Goktepe et al., 2005). However, static, linear or quasi-nonlinear layered elastic model based backcalculation programs appear to still be the most common among State Highway Agencies (SHA).

Most backcalculation programs involve the use of numerical integration subroutines that are capable of calculating FWD pavement deflections and other parameters, given the stiffnesses (or moduli) of the various pavement layers and their thicknesses. If all assumptions are correct (e.g., each layer is an elastic layer, is isotropic and homogeneous, and all other boundary conditions) then it is possible to iterate various combinations of moduli until there is a virtually perfect match between the measured and theoretical FWD deflections. In this manner, a solution to the problem of deriving moduli from deflections is obtained (Stubstad et al., 2006).

A serious drawback to the above approach is the fact that one or more of the many input assumptions mentioned above may be incorrect and therefore do not apply to the actual in situ pavement system where FWD was used to measure deflections. Also, the reliability of the solution is dependent upon the seed moduli used as an input. This makes backcalculation an ill-posed process in which minor deviations between measured and computed deflections usually result in significantly different moduli. In many cases, various combinations of modulus values essentially produce the same deflection basin (Mehta and Roque, 2003). In spite of these potential drawbacks, many of the moduli derived through backcalculation will appear both reasonable and rational, based on common engineering sense and a working knowledge of pavement materials. This conclusion appears to be especially true when relatively intact, well-defined, and undistressed pavement sections are tested with FWD.

The process of static backcalculation of layer moduli from FWD data is not always robust enough to tolerate inherent measurement and modeling errors and often "judgment" is needed in interpreting the results. Undoubtedly this has negatively impacted the use of backcalculation to interpret FWD data and, in some cases, a decline in the use of FWD testing for structural evaluation of in-service pavements. For rigid pavements, FWD testing has been used for evaluating the structural properties of the pavement system as well as for detecting voids under concrete slab and load transfer efficiency of joints and cracks.

Various computer programs are available to perform backcalculation analysis. The AREA method for rigid pavements (Ioannides et al. 1989; Ioannides 1990; and Barenberg et al. 1991), ILLI-SLAB (Foxworthy and Darter 1989), ILLI-BACK (Ioannides 1990), best fit algorithm (Hall et al. 1997; Smith et al. 1996), DIPLOBACK (Khazanovich and Roesler 1997), and MODCOMP (Irwin 1994) are examples of FWD interpretation programs and algorithms for rigid and composite pavements. In rigid pavement systems, the backcalculated pavement layer properties are the elastic modulus of the slab (E_{pcc}) and the coefficient of subgrade reaction (k_s) for slab-on-grade pavement systems. For composite pavement systems, the AC modulus is also backcalculated in addition to E_{pcc} and k_s. Researchers at Iowa State University have focused on developing Artificial Neural Networks (ANN) based pavement structural analysis and backcalculation tools for flexible, rigid and composite pavements (Ceylan et al., 2007).

Several methods are available for backcalculating the PCC slab properties, base properties, and the k_s from FWD deflection data. Each method has its own strengths and limitations. The following procedures are typically considered for rigid pavements:

- Backcalculation software and procedures based on elastic layered analysis
- Backcalculation procedures specifically developed for rigid pavements that are based on slab on elastic solid or slab on dense-liquid models that can further be classified as:
    - AREA method-based procedures
    - Best-fit-based procedures

Backcalculation based on multi-layer elastic theory generally requires an iterative procedure described in the previous paragraphs. This approach has been implemented in computer programs such as WESDEF (Van Cauwelaert et al., 1989). Backcalculation based on plate theory, on the other hand, is a closed-form solution. ILLI-BACK (Ioannides, 1990) is a good example of this approach. Hall and Alaeddin (1991) extended this approach by establishing a correlation between the deflection basin and relative stiffness of the pavement. As the relative stiffness is related to the moduli of the pavement according to plate theory, these moduli can be back-calculated from the measured deflection data (Lin et al., 1998).

The following equations provide the closed-form solution for backcalculating E_{pcc} and k_s (Ioannides, 1990; Hall and Alaeddin, 1991):

$$AREA = 6 * \left[1 + 2\left(\frac{d_{12}}{d_0}\right) + 2\left(\frac{d_{24}}{d_0}\right) + \left(\frac{d_{36}}{d_0}\right)\right] \quad (2.1)$$

$$l_k = \left[\frac{E_{pcc} D_{pcc}^3}{12(1-\mu_{pcc}^2)k} \right]^{1/4} \tag{2.2}$$

$$l_e = \left[\frac{E_{pcc} D_{pcc}^3 (1-\mu_s^2)}{6(1-\mu_{pcc}^2)E_s} \right]^{1/3} \tag{2.3}$$

$$l_k = \left[\frac{\ln\left(\frac{36 - AREA}{1812.279}\right)}{-2.559} \right]^{1/0.228} \tag{2.4}$$

$$l_e = \left[\frac{\ln\left(\frac{36 - AREA}{4521.676}\right)}{-3.645} \right]^{1/0.187} \tag{2.5}$$

$$k_s = \left(\frac{P}{8 D_0 l_k^2} \right) \left\{ 1 + \left(\frac{1}{2\pi} \right) \left[\ln\left(\frac{a}{2 l_k} \right) - 0.673 \right] \left(\frac{a}{l_k} \right)^2 \right\} \tag{2.6}$$

$$E_{PCC} = \left(\frac{12 l_k^4 k_s (1-\nu^2)}{h_{PCC}^3} \right) \tag{2.7}$$

where
l_k = dense liquid radius of relative stiffness (in.)
l_e = elastic solid radius of relative stiffness (in.)
D_{pcc} = PCC thickness (in.)
μ_{pcc} = PCC Poisson's ratio
μ_s = subgrade poisson's ratio
a = equivalent radius (in.)

Two important issues that significantly influence rigid pavement backcalculation results are the degree of bonding between layers and the thickness of the various system layers. The backcalculated pavement moduli is very sensitive to the pavement layer thickness. Even a small change in the assumed concrete layer thickness can cause considerable differences in the backcalculated elastic moduli (Ioannides, 1990).

Over the past 30 years, many research studies have addressed the interpretation of pavement deflection measurements as a tool to characterize pavement-subgrade systems, with many of the main findings appearing in American Society for Testing and Materials (ASTM) special technical publications, including those of Bush and Baladi (1989) and Tayabji and Lukanen (2000).

In 1991, the Strategic Highway Research Program (SHRP) conducted a review of the features and capabilities of most of the then-available backcalculation programs, for the purpose of recommending which program or programs should be used in analysis of deflection data collected in the Long-Term Pavement Performance (LTPP) studies (SHRP, 1991). Uzan (1994) presented a synthesis of different backcalculation procedures (both linear and nonlinear) and a discussion of their limitations. Johnson and Baus (1996) investigated a number of basin-matching backcalculation programs.

Recently, Stubstad et al. (2006) developed a new approach, called forwardcalculation, to determine layered elastic moduli from in-situ FWD load-deflection data. This approach was used to conduct a comprehensive review and evaluation of LTPP backcalculation data. It was suggested that, although backcalculation is certainly rigorous and scientific, the user must be aware of its limitations and assumptions, such as linear-elasticity, homogeneity, and isotropic behavior, in addition to the assumption of being horizontally identical.

2.1.2 Load Transfer Efficiency

The LTE of cracks and joints profoundly affects the performance of concrete pavements (Khazanovich and Gotlif 2003). Poor LTE may lead to longitudinal cracking and excessive faulting of jointed concrete pavements (JCP) and could accelerate punchout development in continuously reinforced concrete pavements (CRCP). These distresses could lead to roughness and poor ride quality. Conversely, joints and tightly closed transverse cracks with high LTE do not typically cause any pavement serviceability problems.

When a traffic load is applied near a joint in a PCC pavement, both the loaded and unloaded slabs deflect as a portion of the load is transferred to the unloaded slab. As a result, deflections and stresses in the loaded slab may be significantly less than if there was a free edge. The magnitude of reduction in stresses and deflections at a joint depends on the joint's LTE.

Traditionally, LTE at the joint is determined based on the ratio between of the maximum deflection at the joint of the loaded slab and the deflection of the unloaded slab measured just across the joint from the maximum deflection. Two equations (2.8) and (2.9) for the deflection LTE are used most often (Khazanovich and Gotlif 2003):

$$LTE = \frac{d_u}{d_l} \times 100 \tag{2.8}$$

$$LTE^* = \frac{2d_u}{d_l + d_u} \times 100 \tag{2.9}$$

where,
LTE and LTE^* = Load transfer indexes
d_l= The maximum deflection at the joint of the loaded slab
d_u= The corresponding deflection at the joint of the unloaded slab

Joint deflection LTE values may range from 0 percent (no load transfer) to 100 percent (full load transfer). If a joint exhibits a poor ability to transfer load, then the deflection of the unloaded slab is much less than the deflection of the loaded slab and both LTE indexes will have values close to 0. If a joint's load transfer ability is very good, then the deflections at the both sides of the joint are nearly equal and both indexes have values close to 100 percent. Figure 2.1 illustrates the concept of deflection load transfer for two extreme cases: a joint with full load transfer and a joint with no load transfer.

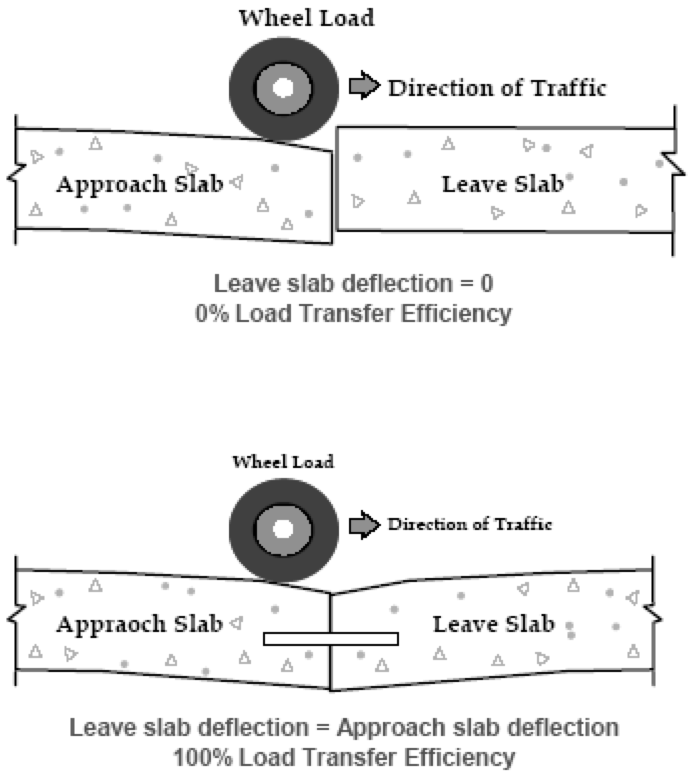


Figure 2.1. Illustration of poor and good load transfer across a joint (NCHRP 2004)

Moreover, the two previously given indices are related by the following equation (2.10) (Khazanovich and Gotlif 2003):

$$LTE^{*} = 2 \times \left(1 - \frac{1}{1 + \frac{LTE}{100}} \right) \times 100 \qquad (2.10)$$

Therefore, these indexes are related, and if one of them is known, the other can be determined. However, the deflection LTE using the index from equation 2.8 because it is much more widely used (Khazanovich and Gotlif 2003).

A joint LTE based on stress can be defined as the following equation (2.11) (Khazanovich and Gotlif 2003):

$$LTE_{\sigma} = \frac{\sigma_u}{\sigma_l} \times 100 \tag{2.11}$$

where,
LTE_{σ} = Load transfer efficiency in stress
σ_l= The maximum stress at the joint of the loaded slab
σ_u= The corresponding stress at the joint of the unloaded slab

NDE can be used to evaluate the LTE of joints and cracks in rigid pavements (AASHTO 1993). LTE testing begins with the placement of the FWD load plate 6 in from the joint (or crack). A haversine load is then imparted to the pavement while the deflections across the joint or crack is recorded. The sensors for measuring deflections are placed at the center of the plate and 12 in from the center of the load plate across the joint (or crack). LTE tests are usually performed in the outer wheelpath of the outside lane. Deflection data should be collected for both the approach and leave side of the joint. As a minimum, deflection data should be collected with the load plate on the leave side of the joint, which results in more conservative values of load transfer efficiencies (AASHTO 1993). It is preferable that LTE testing of this type be done at three load levels—8 kips, 12 kips, and 16 kips. Also, it is recommended that testing should be performed across joints (or cracks) every 100 to 500 ft. However, depending on the length of the project and the availability of resources this can be increased to every 1000 ft (AASHTO 1993).

2.2 State-of-the Practice

Nondestructive deflection testing has been an integral part of the structural evaluation and rehabilitation process for many decades (NCHRP, 2004). In its earliest applications, total measured pavement deflection under a particular load arrangement was used as a direct indicator of structural capacity. Several agencies developed failure criteria that related the maximum measured deflection to the number of allowable load repetitions. With accumulation of knowledge and experience over the years, several algorithms have been developed for transforming deflections into pavement layer properties such as elastic modulus and Poisson's ratio.

2.2.1 Deflection-Based Measurement Systems

2.2.1.1 Manual Static or Rolling Wheel Methods

The four major types of manual equipment which can be mentioned under this category for flexible pavement systems are: (1) the Plate Load Test, (2) the Benkelman Beam, (3) Curvature Meter, and (4) the French Flexigraph laser equipment.

The Plate Load Test registers deflection as a result of a static, circular load. This test is used in Denmark for testing of stiffness of unbound materials during construction of new pavements (Hilderbrand, 2002).

The Benkelman Beam was developed in the USA in the early 1950s and has been used widely since then. It may be the best known deflection measuring device in the world (Huang, 1993). The device consists of a measurement probe hinged to a reference beam supported on three legs. The probe is placed between the dual tires of the rear axle of a loaded truck, and as the truck moves forward the rebound deflection of the probe is measured. The test method is simple, but slow, and it only provides maximum deflection (Hilderbrand, 2002). A major problem associated with the Benkelman Beam is the fact that its reference frame is usually within the deflection basin to be measured and because it is slow viscoelastic effects are significant. Figure 2.2 shows a Benkelman Beam.

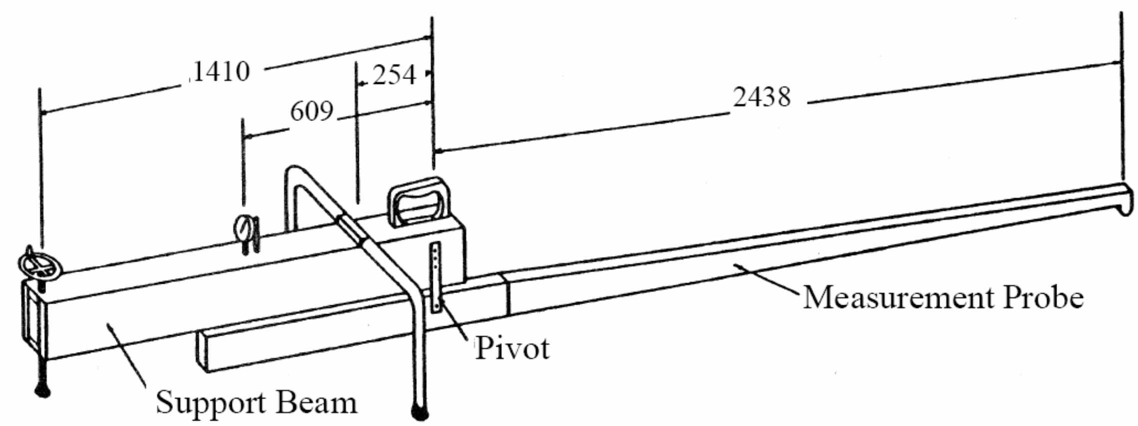


Figure 2.2. Benkelman Beam (all dimensions in mm) (after NCHRP, 1999).

2.2.1.2 Automated Rolling Wheel Methods

This category of test devices includes devices with automatic recording of deflection data during slow movement of a truck. The primary devices in the category are the French LaCroix Deflectograph (LaCroix, 1963; Champion et al., 1968; Prandi, 1967), and the traveling deflectometer developed by the California Department of Transportation (1979). Both devices perform automated Benkelman Beam tests (as described in AASHTO T-256) under both rear wheels. The load applied by traveling deflectometers is usually 80 kN. The major problem with these devices is the lack of an unmoving reference for the deflection readings, which is often due to the limited length of the measuring beam, which also prohibits recording of a full deflection basin. Furthermore, the slow driving speed (2-5 km/h) is not representative of real traffic speeds, and poses a problem on roads with medium to high volume traffic. Figure 2.3 displays a California traveling Deflectometer with Benkelman Beam placed between the dual tires of its rear axle.

Figure 2.3. California traveling Deflectometer (Andren, 2006)

The first generation Danish Deflectograph (Figure 2.4) was developed in 1972 and put into operation two years later (Nielsen, 1981; Nielsen, 1981a). A 15-m long trailer carries an 8-m long truss framework with Benkelman Beam type probes. The measuring procedure is the same as for the Lacroix or California Traveling Deflectograph, where the probes are placed on the road surface to measure the deflection from the constantly moving truck. The probes are then automatically moved to a new position for a new measurement cycle. This deflectograph was sometimes called the "grasshopper", due to their similar movements while jumping along the ground/pavement (Andren, 2006). An interesting historical review on this device can be found in a paper by Banke (2001).

Figure 2.4. The Danish first generation Deflectograph (Christensen, 2003)

The second generation Danish Deflectograph (Jansen, 1990) was completed and placed in regular operation in 1988. Although the second generation was a complete rebuild from the first generation, the working principle, with minor modifications, is the same. The new deflectograph could operate in curves and the operating speed was raised to 7 km/h. The second generation Danish Deflectograph is no longer in use, but the trailer is used in the new Danish High Speed Deflectograph described below. Both of the Danish deflectographs were one-of-a-kind and used only in Denmark (Andren, 2006).

The Department of Main Roads, New South Wales, Australia purchased a Lacroix Deflectograph in 1975 and one more in 1978. Loosely based on the Lacroix concept, the Deflectolab (Hill et al., 1988) was constructed between 1984 and 1987. Almost every detail on the Deflectolab project is given in a paper by Hill et al. (1988). The Deflectolab differs from other deflectographs in that the Benkelman Beam type probe arm is mounted behind the rear axle. The measuring cycle then starts with probes being positioned between the dual tires and the unloading is recorded. The operating speed is 4 km/h, and samples are assessed variably every 4 to 20 meters.

The Curviamètre (Figure 2.5) which is used in France and Belgium, consists of a truck with a rear axle load of 60-80 kN. At 15 km/h, the device records deflections by three geophones mounted on a 15-m loop chain, which is supported by steel drums above the rear axle of the truck, one in front of the axle and one behind. The chain is placed on the pavement surface one meter in front and three meters behind the loading wheel, and passes through the dual wheels such that one geophone at a time is on the pavement surface. The device is mainly used for network measurements of bearing capacity. The limiting factors for this device include is its low operating speed, its in-ability to drive in curves, and the limited deflection data sampling frequency (Hilderbrand, 2002).

Figure 2.5. The Belgian Curviamètre (COST 325, 1997)

The Russian UNK-systems were developed starting in 1975 by Sidenko et al. (1985). The UNK-1 apparently suffered from construction problems and never had any notable use. In 1977, the UNK-2 system was constructed. The UNK-2 works according to the same principle as the French Curviamètre, with one difference: the UNK-2 uses a strain-gauge mounted on a steel plate on the chain to assess the deflection. The system was used for at least five years in the Ukraine and Moldova with satisfactory results. The operating speed was 5 km/h, and the test points were 8 meters apart. A similar, but trailer mounted, system, UNK-3, was developed to allow for measurements on a broader variety of roads. A completely new concept was made with the UNK-4 in 1980. A four-meter beam was controlled with a mechanism that made the beam move periodically along the vehicle. The operating speed was 3 km/h and the sample distance 3 meters. The UNK-4 system (Figure 2.6) was used for routine surveys on the Ukrainian road network, and apparently both the efficiency and accuracy of the UNK-4 may have been higher than that of the Lacroix system. No information has been found on the present status of the UNK-systems, or any other Russian rolling deflectograph (Andren, 2006).

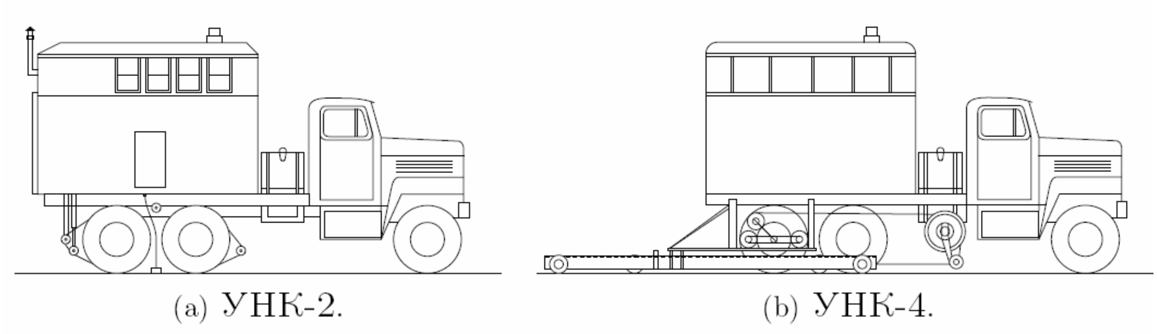


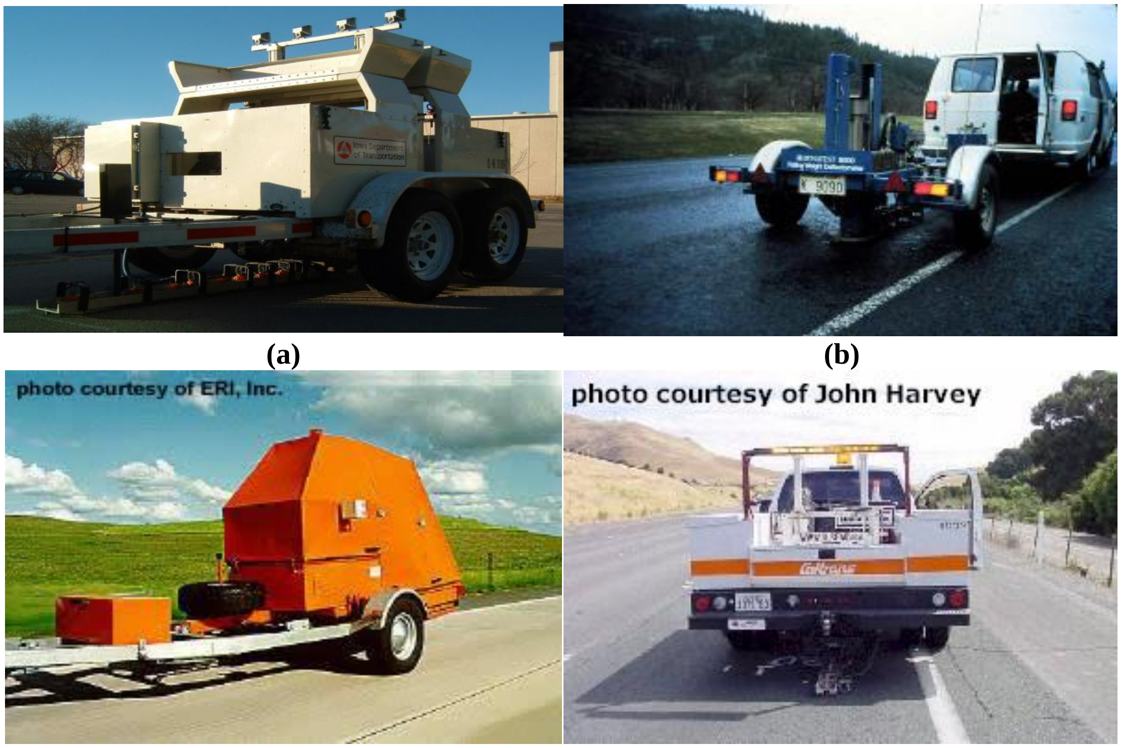
Figure 2.6. The Russian UNK-2 and UNK-4 Deflectographs (Andren, 2006)

2.2.1.3 Stationary Impulse Load Methods

This category of test methods comprises the most widely used types of equipment for nondestructive pavement deflection testing. These methods include the FWD, the Dynaflect, and the Road Rater.

FWD testing has become the principal NDE technique to structurally evaluate in-service pavements over the last twenty years. FWD can be used for structural evaluation of all three major pavement types (flexible, rigid, and composite). The history of the FWD equipment is well-documented by Bohn (2007). FWD is often preferred over destructive testing methods because FWD testing is faster and does not entail the removal of pavement materials. In addition, the testing apparatus is easily transportable. Pavement modulus is "backcalculated" from the observed dynamic response of the pavement surface to an impulse load (the falling weight).

Several types of FWD equipment are shown in Figure 2.7. The FWD can either be mounted in a vehicle or on a trailer and is equipped with a weight and several velocity sensors. To perform a test, the vehicle is stopped and the loading plate (weight) is positioned over the desired location (see Figure 2.8). Sensors are then lowered to the pavement surface and the weight is dropped. The advantage of an impact load response measuring device over a steady state deflection measuring device is that it is quicker, the impact load can be easily varied and it more accurately simulates the transient loading of traffic.



(a) (b)

(c) (d)

Figure 2.7. FWD test equipments: (a) Iowa DOT FWD: JILS-20, (b) DYNATEST FWD, (c) KUAB FWD, and (d) JILLS FWD (Muench et al., 2005)

Figure 2.8. FWD bottom view with sensor locations (Muench et al., 2005)

The Dynaflect is also a stationary trailer mounted device which applies a static load of 9 kN combined with a dynamic load of 4.45 kN applied at a frequency of 8 Hz (Huang, 1993). The load is transferred to the pavement surface by a pair of steel wheels spread 0.5 meters apart. The resulting deflections are recorded with five geophones mounted at 300 mm intervals between the steel wheels, the first one being directly between the steel wheels. In Figure 2.9, a photograph of a Dynaflect device is shown.

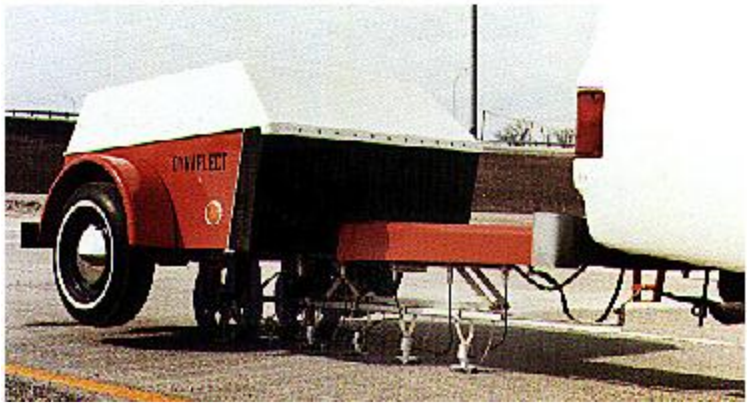


Figure 2.9. Dynaflect (Geo-Log, Inc. 2001)

The Road Rater (Figure 2.10) is a device similar to the Dynaflect in many respects. The major difference is the method by which the dynamic load is provided and the loading magnitude (Hildebrand, 2002). While the Dynaflect produces the dynamic load by a pair of eccentric discs rotating in opposite directions, the Road Rater uses a hydraulic vibrator. The Road Rater is capable of applying static loads of 11-26 kN and dynamic loads of 2-36 kN at a frequency of 5-70 Hz. Three or four transducers are used to register pavement surface deflection. According to Huang (1993), the major limitation of both the Dynaflect and the Road Rater is the loading system capabilities: the Dynaflect has a fixed magnitude and frequency of loading, while the Road Rater applies too low loads with a light version of the device, while a heavier version requires a very high static load.

Figure 2.10. Road Rater (Muench et. al., 2005)

2.2.1.4 Automated Mobile Dynamic Load Methods

Within the last ten years, efforts have been made in several places around the world to produce a high speed monitoring device for measuring pavement bearing capacity. The reason for these activities is the increasing amount of traffic on major roads everywhere which is making stationary tests like the FWD test difficult for highway capacity reasons as well as for safety reasons (Hilderbrand, 2002).

All seismic-based and deflection-based nondestructive methods commonly used in the U.S. are performed at discrete points along a pavement. These results are then used to characterize the entire pavement. The more tests performed, the better the predictions, though there is never complete assurance that all critical locations have been tested. Another weakness of discrete measurements is that it is difficult to separate the effects of the elastic properties of the pavement from the pavement geometry. For example, larger deflections will be measured near a joint or free edge. However, in discrete tests it is difficult to differentiate increased deflections created by joints and edges from increased deflections created by a softer pavement. Although statistics provide a useful tool for estimating pavement properties from discrete measurements, the ideal approach is to measure continuous properties of the entire pavement (Bay and Stokoe, 1998).

Continuous deflection profiles can be used to locate precisely the softest pavement locations for remedial actions, quantify the extent of various pavement conditions, and determine the pavement performance at cracks, joints, and intact sections (Bay and Stokoe, 1998). Five primary systems are actively in the development stage for flexible pavements:

1. Rolling Dynamic Deflectometer (RDD)
2. Rolling Weight Deflectometer (RWD)
3. Rolling Wheel Deflectometer
4. Rolling Deflection Meter (Swedish) and
5. High Speed Deflectograph (Danish).

None of the devices are yet ready for routine testing, but since it is expected that this type of equipment will play a major role regarding monitoring of network bearing capacity in the coming years, a short description of some of tthe devices is provided in the following sections. All devices are based on trucks or semitrailers and all, except the Rolling Dynamic Deflectometer, use laser sensors and perform measurements at normal traffic speed (up to 80 km/h) (Hilderbrand, 2002).

2.2.1.4.1 Rolling Dynamic Deflectometer

The Rolling Dynamic Deflectometer (RDD) (Bay et al., 1995) applies a specified dynamic load to the pavement surface via two sets of dual wheels mounted side by side. Deflections are measured at a driving speed of 5 km/h with a rolling accelerometer mounted in front of the load wheels.

The RDD was developed at The University of Texas at Austin. It was originally constructed by modifying a Vibroseis truck. Particularly useful in oil prospecting, the Vibroseis trucks apply large dynamic forces to the ground in order to generate seismic waves. The hydraulic vibrators mounted on the RDD transmit sinusoidal forces in the 5–100 Hz range to the road surface and the rolling sensors assess the deflections (Bay et al., 1995; Garrow and Hudson, 1997). Field results from the RDD can be found in papers by Bay with various coauthors (Bay et al., 1998; Bay et al., 1999). A photograph of the Texas RDD is shown in Figure 2.11 and the major components of the RDD are illustrated in Figure 2.12.

Figure 2.11. The Texas RDD (Bay and Stokoe, 1998)

Continuous deflection profiles determined with the RDD can be used to: 1) assess the overall stiffness of a pavement; 2) differentiate the relative stiffnesses of different regions; 3) detect cracks, joints, and weak regions; 4) assess the performance of cracked or jointed regions; 5) delineate the regions of the pavement influenced by joints and cracks; and 6) identify areas where additional discrete testing should be performed. The RDD is a powerful tool having the potential for: 1) designing pavement repairs and retrofits, 2) estimating the remaining life of pavements, and 3) functioning as a quality assurance and quality control system during the construction of new pavements (Bay and Stokoe, 1998).

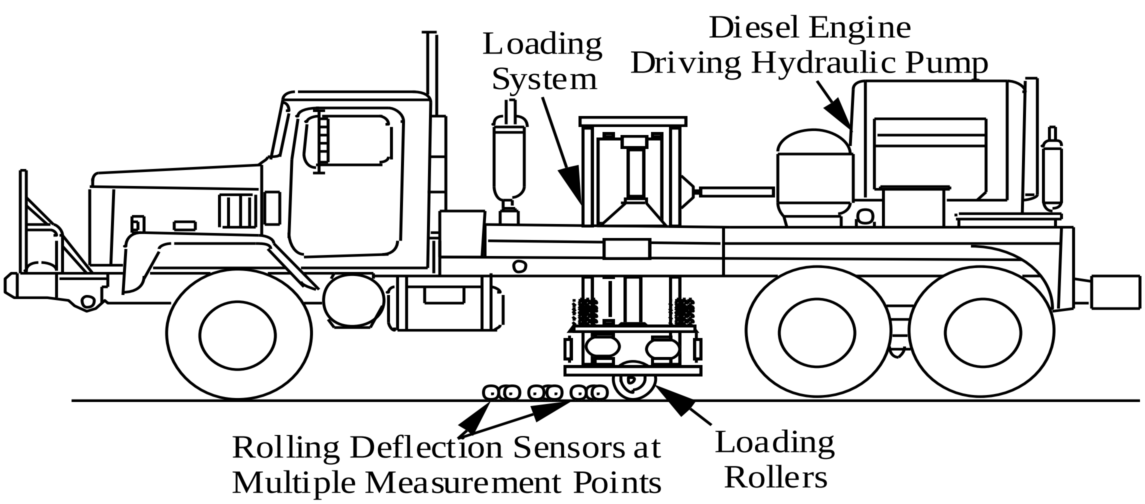


Figure 2.12. Major components of the RDD (Bay and Stokoe, 1998)

2.2.1.4.2 High Speed Deflectograph

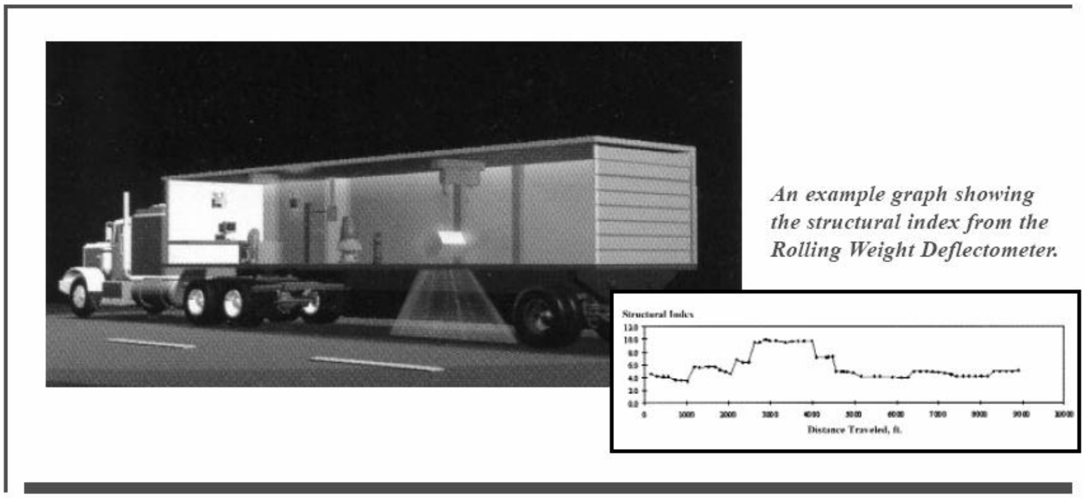
The High Speed Deflectograph (Hildebrand et al., 1999) uses a two laser Doppler sensors mounted in front of the loading wheel measure the vertical velocity of the pavement surface resulting from the load. The velocity measurement can be integrated to provide the deflection at a single point and with a higher accuracy than possible with distance laser sensors. The High Speed Deflectograph (HSD) prototype (Figure 2.13) is primarily capable of scanning a road network with regard to the pavement bearing capacity such that the network can be classified into groups with good, fair and poor bearing capacity.

Figure 2.13. Prototype of High-Speed Deflectograph (HSD) (Hilderbrand, 2002)

The primary quantifiable benefit of the HSD is its production capacity. With a driving speed during measurement of 70 km/h and five effective work hours per day, 1,750 km can be tested within a week. This means that all the lane-km of the Danish state road network (approximately 6,000 km) can be tested in four weeks. Similarly, it will require approximately 12 weeks to test all lane-km of county roads in Denmark (approximately 20,000 km). Thus, the total period needed to monitor the structural condition of the major road network in Denmark is approximately 16 weeks (Hilderbrand, 2002).

2.2.1.4.3 Rolling Weight Deflectometer (RWD)

Applied Research Associates, Inc. (ARA) is developing a high-speed Rolling Wheel Deflectometer (RWD) (Figure 2.14) for structural evaluation of flexible highway pavements. The research and development has been sponsored primarily by the Federal Highway Administration (FHWA) and the Small Business Innovation Research (SBIR) program (Johnson and Rish, 1995; Johnson et al., 1995; Johnson and Rish, 1996). RWD testing is a nondestructive method for measuring the structural response of highway pavements. It is designed to measure continuous deflection profiles at normal highway speeds..



The Rolling Weight Deflectometer travels at highway speeds while collecting deflection response data.

Figure 2.14. Illustration of Rolling Weight Deflectometer (RWD) (Johnson et al., 1995)

The RWD utilizes a "spatially coincident" methodology for measuring pavement deflection. Three lasers are used to measure the unloaded pavement surface (i.e., forward of and outside the deflection basin), and a fourth laser, located between the dual tires and just behind the rear axle, measures the deflected pavement surface (Figure 2.15). Deflection is calculated by comparing spatially coincident scans as the RWD moves forward. In other words, the profile of the undeflected pavement surface is subtracted from the profile of the deflected pavement surface measured at the same exact locations. This method was originally developed by the Transportation and Road Research Laboratory (TRRL) and furthered by Dr. Milton Harr at Purdue University. It was later employed on the Dynatest/Quest prototype RWD.

It is important to remember that at 55 mph and the 2 kHz sampling rate used by the RWD results in a reading being taken approximately every 0.5 in. The random error associated with the individual deflection readings can be very high, due to factors including equipment limitations and pavement factors. For example, due to pavement texture, one laser may read the top of an aggregate while the next laser to pass over that point may read the valley between two adjacent aggregates. This type of error is random, resulting in an approximately equal distribution of overestimated deflections, as underestimated deflections. If a sufficiently large number of readings are averaged, the random error is reduced to the point that the overall mean is not significantly affected by the random noise. For the Texas demonstration, ARA determined that a 100-ft interval was sufficient to reduce random error. This sample unit length is also suitable for network-level purposes.

Figure 2.15. RWD prototype (ARA, 2004)

Field studies have shown that the RWD experiences a warming-up effect, prior to stabilization of readings. Frequent, hard-braking of the RWD while testing the short loops resulted in a continuous decrease in deflections over the course of the day, making the RWD data on the instrumented sections unstable. It is important to note that frequent, hard-braking of the RWD is not typical for routine production testing, and therefore, the majority of the Texas data, collected on roadways other than the instrumented sections, are considered more indicative of expected RWD performance under actual field conditions. Overall, ARA believes that the RWD has demonstrated its usefulness as a network-level structural evaluation tool for flexible highway pavements, although several aspects, such as the influence of thermal effects, need to be investigated further so that improvements can be made to the RWD's accuracy and repeatability (ARA, 2004; ARA, 2005).

2.2.2 Material-based Measurement Systems

This category includes NDE testing devices using ultrasonic or stress waves to measure material elastic properties, and the category includes the Resonant Frequency Test (ASTM C215), Ultrasonic pulse velocity (ASTM C597), spectral analysis of surface waves, and impact-echo.

2.2.2.1 Resonant frequency test (ASTM C215)

Powers originally devised the resonant frequency method in 1938. He discovered that the resonant frequency of a material can be matched with a harmonic tone produced by materials when tapped with a hammer (Malhorta and Carino 1991). Since then, the method has evolved and incorporated the use of electrical equipment for measurement.

An important property of any elastic material is its natural frequency of vibration. A material's natural frequency of vibration can be related to its density and dynamic modulus of elasticity. These relationships for resonant frequency were originally derived for homogenous and elastic materials. However, the method also applies to concrete specimens if the specimens are large in relation to their constituent materials (Malhorta and Carino 1991).

The study of physics has determined resonant frequencies for many shapes, including slender rods, cylinders, cubes, prisms and various other regular three-dimensional objects. Young's dynamic modulus of elasticity of a specimen can be calculated from the fundamental frequency of vibration of a specimen according to equation (2.`1) (Malhorta & Carino 1991):

$$E = \frac{4\pi^2 L^4 N^2 d}{m^4 k^2} \tag{2.12}$$

where,
E = Young's dynamic modulus of elasticity
d = Density of the material
L = Length of the specimen
N = Fundamental flexural frequency
k = Radius of gyration about the bending axis
m = A constant (4.73)

ASTM has described a standard test that covers measurement of the fundamental transverse, longitudinal and torsional resonant frequencies of concrete specimens for the purpose of calculating dynamic Young's Modulus of elasticity (ASTM C-215-97, 2001). This test method calculates the resonant frequencies using two types of procedures, the forced resonance method or the impact resonance method.

The forced resonance method is more commonly used than the impact resonance method due to the ease of testing and interpretation of results (Ferraro 2003). The forced vibration method uses a vibration generator to induce vibration in the test specimen while the vibration pickup transducer is coupled to the specimen. The driving frequency is varied until the pickup signal reaches a peak voltage. The specimen's maximum response to the induced vibration occurs at the resonant frequency. Figure 2.16 illustrates the typical setup of a resonant frequency device. The vibration generator is coupled to the right side of the specimen while the pickup is coupled to the left.

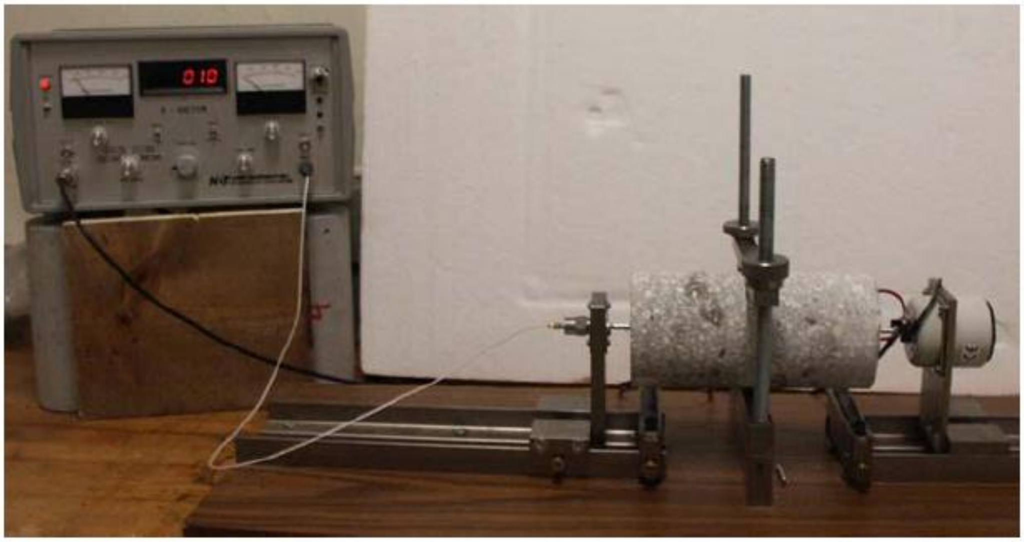


Figure 2.16. Typical forced resonant frequency setup (Ferraro 2003)

2.2.2.2 Ultrasonic pulse velocity (ASTM C597)

Ultrasonic testing is an NDE method that is used to obtain the properties of materials by measuring the time of travel of stress waves through a solid medium. The time of travel of a stress wave can then be used to obtain the speed of sound or acoustic velocity of a given material.

During this type of testing a pulse is sent from the sending transducer to the receiving transducer through the concrete specimen as see in Figure 2.17 (Ferraro 2003). The relationship of a specimen's acoustic velocity is simply calculated from a time and a length measurement. It should be noted that cracks, flaws, voids and other anomalies within a material specimen could increase time of travel therefore decreasing the material's acoustic velocity. However, assuming the specimen in Figure 2.17 is free of anomalies, its acoustic velocity can be calculated simply.

Figure 2.17. Typical ultrasonic test procedure (Ferraro 2003)

The experiment shown in Figure 17 provided the user with a quantitative result. The pulse velocity acoustic velocity of stress waves through a concrete mass is related to its physical properties including a function of modulus, the mass density, and poisson's ratio. The relevant equation for wave speed is equation (2.13) (ASTM C597-97):

$$V = \sqrt{\frac{E(1-\nu)}{\rho(1+\nu)(1-2\nu)}} \tag{2.13}$$

where,
E = Young's modulus of elasticity
ρ = Mass density
ν = Poisson's ratio

In order to transmit or receive the pulse, the transducers must be in full contact with the test medium, otherwise an air pocket between the transducer and test medium may introduce an error in the indicated transit time. There are three possible configurations in which the transducers may be arranged, as shown in Figure 2.18. These are: (a) direct transmission; (b) semidirect transmission; and (c) indirect or surface transmission. The direct transmission method is the most desirable and the most satisfactory arrangement because maximum energy of the pulse is transmitted and received with this arrangement. (Popovics 2003)

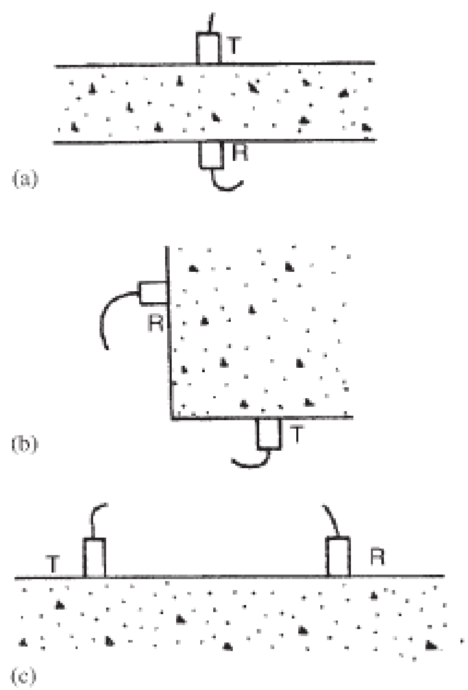


Figure 2.18. Possible transducer configurations for the ultrasonic pulse velocity method: (a) direct; (b) semi-direct; (c) indirect (Popovics 2003)

2.2.2.3 Spectral analysis of surface waves

The Spectral Analysis of Surface Waves (SASW) test method method was originally developed and used for the assessment of elastic properties of pavements and soil layers in geotechnical engineering

(Nazarian 1984; Nazarian et. al. 1995), and later was applied to thin concrete slabs (Bay and Stokoe 1990). This method has the following advantages (Cho and Lin 2005): (1) it requires measurement and access to only one side of the object; (2) when an impact is used to create a source wave, the majority of energy generated from the impact is imparted in the form of surface waves while the remainder goes into body waves; (3) the damping due to geometrical spreading for surface waves (cylindrical) is smaller than for body waves (spherical wave front); (4) a stiffness profile (e.g., Young's modulus or shear modulus) can be obtained without knowing the layer thicknesses.

In the SASW test method, an impulsive load is applied on the surface of the pavement. A variety of sources can be used to generate the impact, from hand held hammers of different sizes (small hammers are sufficient for high frequency excitation) to drop weights (heavier weights for low frequencies). Figure 2.19 (Roesset, 1998) illustrates the schematic of the SASW test method. The passage of the wave train generated by the impact is monitored by two or more vertical receiver stations. In the typical arrangement the distance between receivers is equal to the distance from the source to the first receiver. The electrical signals recorded by the receivers are digitized and transformed to the frequency domain by a dynamic spectral analyzer using a Fast Fourier Transform algorithm. The analyzer also automatically provides the cross spectrum from which the phase difference between the two signals can be obtained as a function of frequency. The inter-arrival time and the phase velocity can then be easily computed. For a given arrangement of source and receivers, the test can provide the dispersion of surface waves which means that surface waves of different frequencies propagate at different depths. Thus by measuring the propagation velocity of waves of different frequencies, the variation of velocity (or stiffness) with depth is obtained (Addo, 2000).

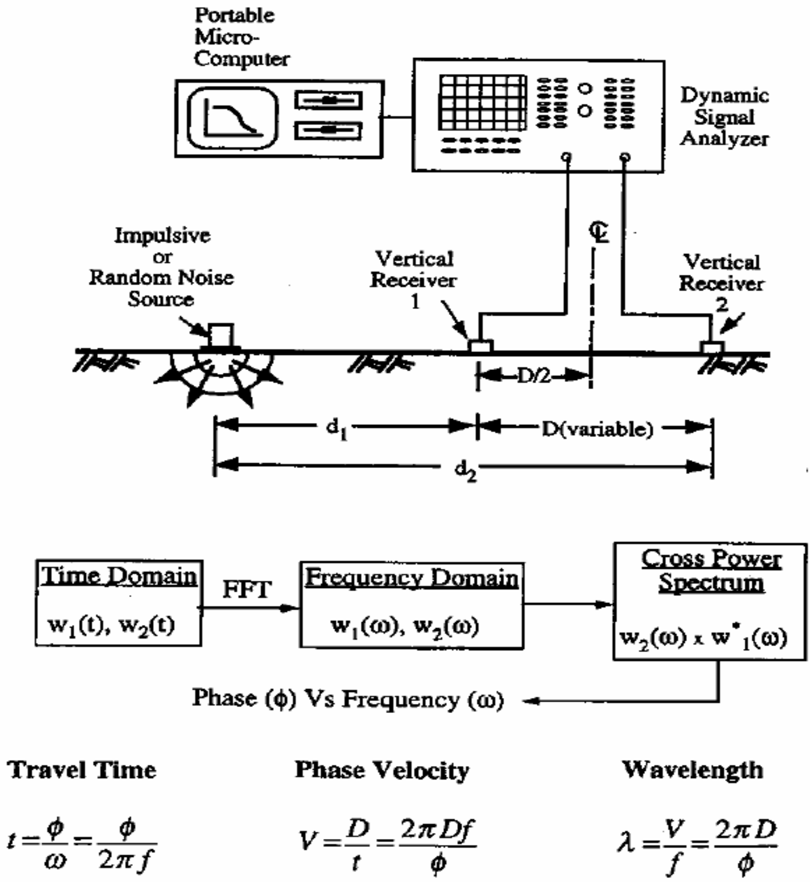


Figure 2.19. Schematic showing the SASW test method (Roesset 1998)

An iterative numerical process called inversion is then used to determine the variation of shear wave velocity with depth (Addo 2000). The shear modulus and the dynamic Young's modulus may then be determined from equations (2.14) and (2.15):

$$G = \rho V_S^2 \tag{2.14}$$

$$E = 2G(1+\nu) \tag{2.15}$$

where,
G = Shear modulus
E = Young's modulus of elasticity
ρ = Mass density
V_s = Shear wave velocity
ν = Poisson's ratio

In the original SHRP program (Nazarian, et. al. 1993), the Seismic Pavement Analyzer (SPA) shown in Figure 2.20 was developed. The Primary applications of the SPA include pavement structure evaluation in terms of elastic moduli and layer thicknesses, detection of voids or loss of support under rigid pavements, delamination in rigid pavements and bridge decks, and subgrade evaluation (Gucunski and Maher 2002). The SPA incorporates five seismic techniques for those purposes: Ultrasonic Body-Wave (UBW), Ultrasonic Surface-Wave (USW), Impact Echo (IE), Impulse Response (IR) and Spectral Analysis of Surface Waves (SASW).

The SPA is a trailer-mounted device, which applies loads of different magnitude and frequency to the pavement surface. All load levels are significantly lower than those of traffic, and the test frequencies are higher than for real traffic loads. Therefore laboratory tests of pavement materials is required to establish a basis for comparison between results from the SPA and the FWD. The load impacts of the SPA produce surface waves which are registered by accelerometers and geophones (Hildebrand 2002). This is similar to FWD, the Dynaflect, and the Road Rater. The data analysis, however, is fundamentally different than that of FWD results since the SPA is a seismic test, which involves determining the propagation velocity of waves of different frequencies through the tested medium and converting the observed response into material properties.

Figure 2.20. Seismic Pavement Analyzer (SPA)(Gucunski and Maher 2002)

2.2.2.4 Impact-Echo

Impact-echo is a method for NDE of concrete and masonry structures that is based on the use of impact-generated stress (sound) waves that propagate through concrete and masonry and are reflected by internal flaws and external surfaces (Sansalone and William 1998). A short-duration mechanical impact, produced by tapping a small steel sphere against a concrete or masonry surface, is used to generate low-frequency stress waves that propagate into the structure and are reflected by flaws and/or external surfaces. Surface displacements caused by reflections of these waves are recorded by a transducer, located adjacent to the impact. The resulting displacement versus time signals is transformed into the frequency domain, and plots of amplitude versus frequency (spectra) are obtained. Multiple reflections of stress waves between the impact surface, flaws, and/or other external surfaces give rise to transient resonances, which can be identified in the spectrum, and used to evaluate the integrity of the structure or to determine the location of flaws (Sansalone and William 1998).

3. MEASUREMENT APPROACHES

The research team has concluded that there are essentially two different deflection-based measurement approaches that could be taken for a HSPA system. These two measurement approaches are to make measurements in either a time coincident fashion (i.e., multiple measurements at the same time) or a position coincident fashion (i.e., measurements are made at the same geometric position by multiple sensors). Each of these is briefly described below with a summary of the advantages and disadvantages of each. Following these descriptions of the two general types of measurement philosophies, the general system configuration developed and proposed by the research team is described.

3.1 Time Coincident

A time coincident measurement approach is one in which multiple sensors take measurements at the same point in time but at different geometric locations. With data processing these measurements could be used to extract information about one point relative to another. In the case of the HSPA, one might think of this as taking a position measurement away from a load (i.e., an unloaded location) at the same time a position measurement is taken at a loaded location. By subtracting the two, one may be able to calculate/estimate the relative deflection under load.

The advantage of making this type of a measurement is that it is relatively easy to control when measurements are made (and how quickly). This allows assurance that two measurements have commonality of some type. The disadvantage of making this type of information is that obtaining a deflection requires making a secondary calculation (with associated assumptions) to determine the deflection at a point. For a rigid pavement, variations in the pavement such as tining make geometric assumptions problematic.

3.2 Position Coincident

A position coincident measurement approach is one in which multiple sensors take measurements at the same geometric position but at different times (due to the movement of the sensor). With data processing one can extract information about the same point when loaded or unloaded. This leads to a more direct calculation of pavement deflection.

The advantage of making this type of a measurement is that no interpolation between measurements is needed and one can obtain a direct measure of deflection under load. The difficulty with this measurement approach is ensuring that the measurements are made at exactly the same position in a measurement environment where the test vehicle speeds can vary.

3.3 Proposed System Configuration

It is the research team’s general understanding that other high-speed measurement systems are based upon a position coincident measurement philosophy. With this type of an approach, attempts are made to make position measurements at the same point and to then make inferences about that point. In critically reviewing these approaches, it is apparent that several key assumptions must be made in the processing of the data. These assumptions (e.g., vehicle speed, orientation of measurement platform, etc.), in the research team’s opinion, are the primary source of the errors of the system. Interestingly, even before such a critical review, the research team had concluded that a time coincident system was the more likely configuration that could achieve the needed accuracy.

As will be seen in the following chapter, the research team has identified two approaches for making condition assessment of rigid pavement systems: distance measurement and direct material-based measurement. The following description applies only the distance measurement aspect of the proposed system (note the direct material based measurement approach will be easily configured once a proof-of-concept can be conducted). Similar to other systems, the proposed approach utilizes a "large" vehicle (i.e., a truck) to which the system components would be mounted via a rigid beam. The sketch shown in Figure 3.1 illustrates the rigid beam and the truck wheel around which the components would be installed. As can be seen, this system utilizes a number of distance measurement sensors place "far" forward and behind the wheel load with one sensor placed directly in line with the wheel and one approximately 12 in. forward of the wheel. Note that several measurement sensors will need to be placed forward and behind of the wheel load; however, the exact number needed is unknown at this point and would need to be established in subsequent work. However it is expected that between 3 and 20 might be needed. It is additionally anticipated that the orientation of the rigid beam may need to be monitored with an inclinometer/tiltmeter such that a baseline coordinate system can be established.

The fundamental principle behind this system configuration is that the measurements made forward and behind the wheel (presumably in locations unaffected by the wheel load) will be used to estimate the height of the pavement at the wheel load when it was unloaded. With such an estimate and the measured position under load, the deflection under load can then be estimated. The sensor just forward of the wheel load can similarly be used to measure deflection and would be useful in assessing joint load transfer efficiency as defined previously. With this approach the two most important pavement measurements could be made.

The fundamental assumption with this approach is that one can estimate the position of an unloaded pavement from measurements made of the pavement forward and behind that section. Clearly there are many influences that can impact the accuracy of those estimates. These include both global and local pavement variations. Global pavement variations are derived from vertical alignment differences (e.g., vertical curves) and local pavement variations are derived from pavement surface characteristics (e.g., tining, roughness, etc.). It is felt that pavement surface characteristics are normally distributed throughout a pavement system and when measurements are averaged over a length of pavement (say, every 0.1 mile) that an average assessment can accurately be made. To study the influence of vertical alignment differences a brief parametric study was conducted and indicates that these errors can likely be eliminated through proper statistical consideration (e.g., a combination of averaging and projections).

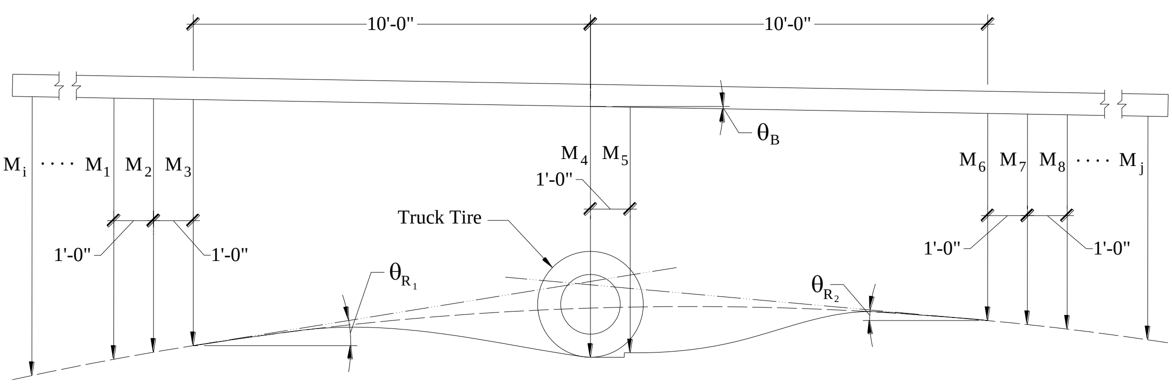


Figure 3.1. Proposed system configuration

4. AVAILABLE TECHNOLOGIES

4.1 Distance Measurement

4.1.1 Lasers

Lasers are used on current flexible pavement deflection analyzers, and resolutions on the order of those necessary to measure concrete pavement deflections are commercially available. However, a current limitation lies in the inability to accurately position the laser source in 3D space (absolute) and determine accurate deflections where desired deflections are an order of magnitude or two smaller than surface irregularities, imperfections and debris. The following information is provided for use with new methodologies to make meaningful *relative* measurements.

Lasers are commercially available with extreme accuracy, large stand-off distances and real time surface compensation for changing surfaces. For example, Micro-Epsilon provides the optoNCDT 1810/2210[i] long range sensor with a measuring range of 50mm (2”), a resolution of 5µm (0.2 mils), a spot size of 500µm (0.5mm), a measuring rate of 2.5kHz (spacing of 0.42 inches @88fps) and a stand-off distance of 500mm (0.5 meters or 20”). For a larger measuring range, Micro-Epsilon’s ILD 2220-200 sensor has a measuring range of 200mm (8”), a resolution of 3µm (0.1 mils), a spot size of 1300µm (1.3mm), a measuring rate of 20kHz (spacing of 0.05 inches @88fps) and a stand-off distance of 230mm (0.23 meters or 9”).

Laser measurements may be made at discrete points or by scanning. Laser scanners are available that can measure at frequencies up to 500 kHz. There are two principle methods used to measure distances with lasers – time of flight and triangulation. Time of flight scans are appropriate for long distances whereas triangulation techniques provide much better accuracies but are limited to shorter ranges. For the purposes of roadway deflection measurements, triangulation would be the preferred technique as precision for time of flight measurements is limited to the millimeter range.

4.1.2 Sonic

Sonic technologies are also available for distance measurement. One particularly promising technology if exemplified in sensors offered by Ultrasonic Arrays, Inc. Ultrasonic Array’s technology has been used in measuring the deformation of railway tracks on iron ore trains. Ultrasonic Array’s DMS-5000 is used for the non-contact distance measurements to any material in hostile environments, fixed or scanning. The company reports testing the TMS-5000 in a simulated 35 mph environment (on a rotating lathe) with no degradation in accuracy. To provide for accurate measurements, a column of air is continuously blown (from an air hood) and monitored to provide for a constant sound velocity. In this way, measurement are continuously calibrated. Up to 125 measurements per second can be made (spacing of 8 inches @88fps) with one transducer. Stand-off distance is 0.2 – 24 inches. At 4 inches standoff, accuracy is better than 0.1% of stand-off distance or 0.001*4 = 4 mils. The air hood has the additional advantage of blowing loose debris from the measurement area. Spot size is 0.85 inches, and average distance and RMSE are reported for each measurement.

4.1.3 Photogrammetry

Photogrammetry is the art of measuring elevations from photographs taken from an overhead vantage point. Using the process of triangulation, computations are performed on measurements taken from overlapping photographs from known locations and angles. To accomplish the measurements, an operator or analysts must be able to identify the same location in two photographs – traditionally a manual process. Accuracy of the elevation measurements depend on quality of optics used, height above target, ground control and precision of location of aerial platform.

Softcopy or digital photogrammetry uses the same mathematical techniques as traditional photogrammetry, where film is replaced by digital images. Measurements are made on computer displays rather than with high quality optics.[ii] To determine the absolute orientation of an aerial image, six parameters need to be defined: the location of the image with respect to a Cartesian coordinate system, X_c, Y_c, and Z_c, and the pointing direction, ω, φ, and κ. Given the orientation of two overlapping images, elevations for common points on the ground may be determined by triangulation. As in the case of traditional photogrammetry, elevations are usually determined manually.

Due to the large number of elevations required for systematic assessment of concrete surfaces, manual interpretation is not practical. Recent work, however, promises to use image mapping techniques to find corresponding matching points that would allow automating the process. One such process, the Automatic Terrain Measurement or ATM technology, however, generates too many errors to make it practical. However, this technique has only been applied to conventional aerial photography. Close up photogrammetry of a rough concrete surface using high speed using high resolution cameras is a new application to be tested.

Another limitation of the photogrammetric technique is the required parameters for positioning the images probably cannot be known to sufficient accuracy on a moving vehicle (1 mil accuracy on the ground requires much better than one mil accuracy for image orientation). However, some of the parameters may not be needed for relative measurements. Also, for high speed cameras at highway speeds, as there is much less freedom of movement in the Z and X (transverse) directions, it is possible that measurements of image position may be estimated along these axes.

If only relative measures of elevation are practical, it would appear that photogrammetric techniques are likely only to be potentially useful for time coincident measurement for deflection basin (shape) measurements. If cameras can be configured in such an array to photograph the entire (or most of the) deflection basin at one time, sufficiently accurate measurements of elevation (Z) may be possible. It is also possible that relative photogrammetric measurements could also be made to assess the relative height of two slabs for joint load transfer computations.

4.1.3.1 CCD camera technologies

Typically available CCD cameras have speeds up to about 200 frames per second at resolutions on the order of 640 pixels. For a truck traveling at 88 feet per second (1,100,000 mils/s.), a 200 frame per second camera will take an image every 5.3 inches (5300 mils). If a resolution on the order of one mil is desired, each pixel should be on the order of one mil (1000dpi). A 640 pixel image would therefore be 0.64 inches (16mm) in length. For photogrammetric purposes, an overlap of approximately 60 percent between images is desired. Clearly there would be no overlap between 0.64 inch (16mm) images spaced at 5.3 inches. To provide a 60 percent overlap of 0.64 inch (16mm) images, an image would have to be captured approximately every 0.38 inch (9.6 mm). This would require a camera speed on the order of 2800 frames per second at 640 pixel resolution, or approximately 1000 frames per second at 1500 pixel resolution. High speed cameras are available with the speed and resolution required for these measurements[iii]. Very fast shutter speeds (5μs) are available to eliminate blurring.

4.1.3.2 Multispectral possibilities

Traditional photogrammetry is performed with panchromatic film. Sometimes, color (visible) and false color (near infrared) photography is used. Other spectra are available, and range from microwave (RADAR) to hyperspectral (bands as narrow as 0.01 micrometers over a wide wavelength range, typically at least 0.4 to 2.4 micrometers, running from the visible through near infrared to middle infrared wavelengths).

4.1.3.3 High Speed Cameras

On November 16, 2007, the project team performed a preliminary test of a high speed, high resolution camera (Photron Fastcam®). The primary purpose of the test was to evaluate the camera for use in measuring paint glass bead application speeds and movement. While in the field, the camera was trained on a concrete joint to investigate the potential for detecting movement on the order required for concrete deflection measurements. The test was performed on a poor quality concrete pavement on Iowa State Highway 210 just west of Slater, Iowa. The camera captured images at 1000 frames per second with a shutter speed of 1/4000 seconds. A test vehicle (loaded dump truck with legal weight axles) was run over the joint at 4 mph and 55 mph. Example images taken from the test are shown in the figures below. As can be seen in these examples, it appears that it may be possible to use this type of image. Significant difficulty would exist in mounting and operating the camera in a reliable manner.

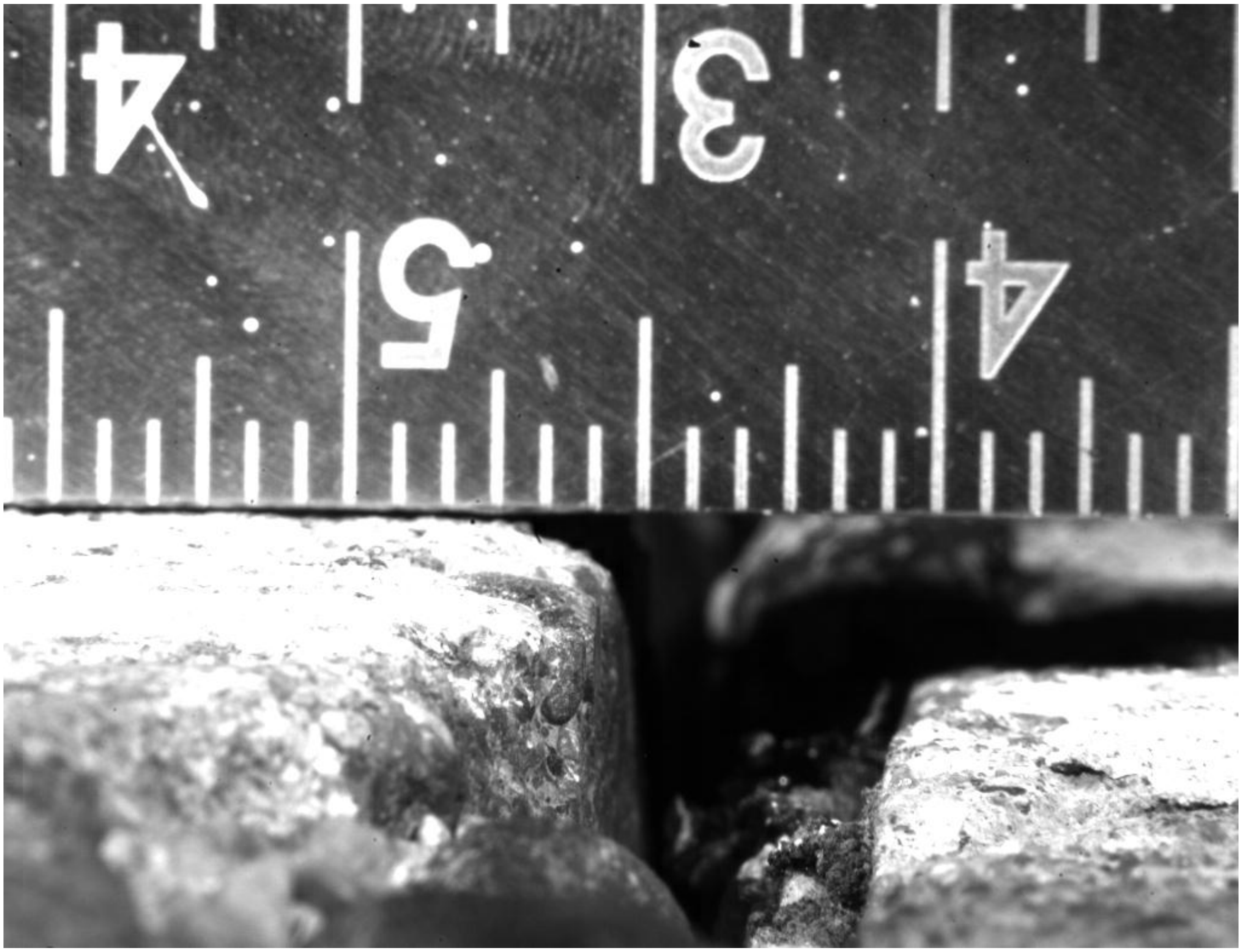


Figure 4.1. Sample still frame from Photron Fastcam® illustrating faulting of approximately ¼ inch

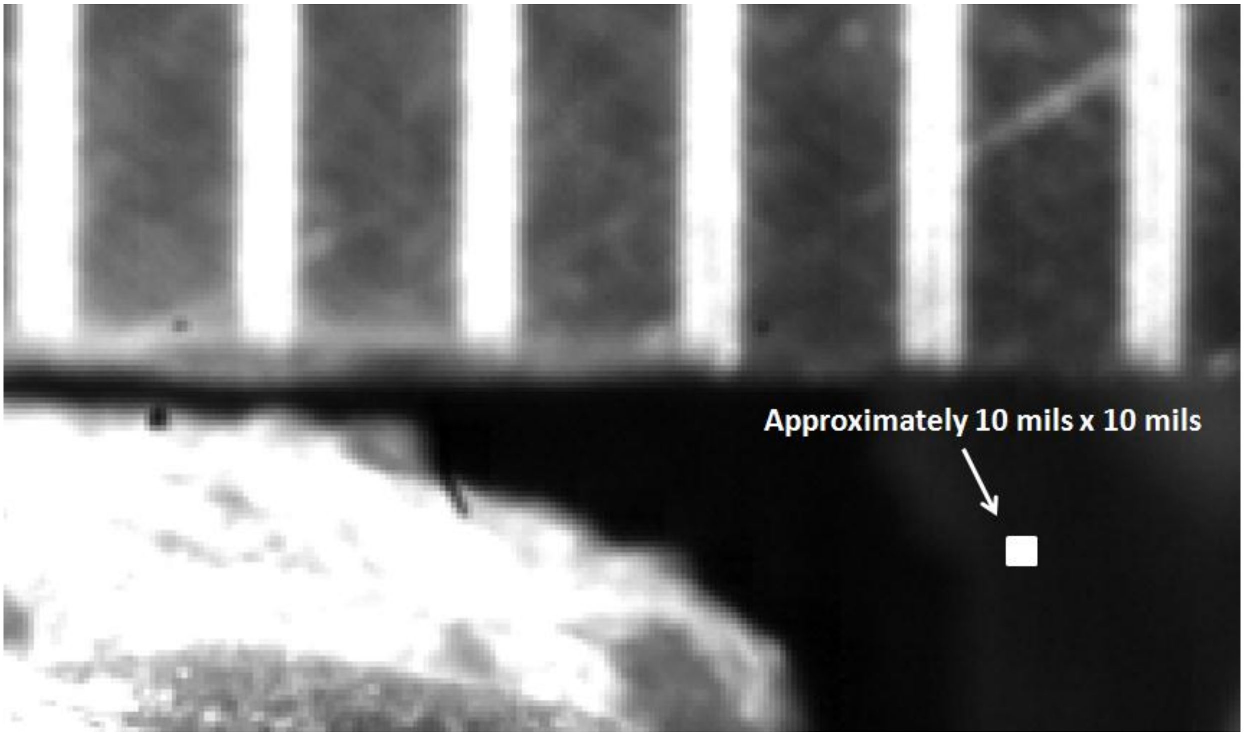


Figure 4.2. Image illustrating approximate resolution of Photron Fastcam® (pixels on order of one mil)

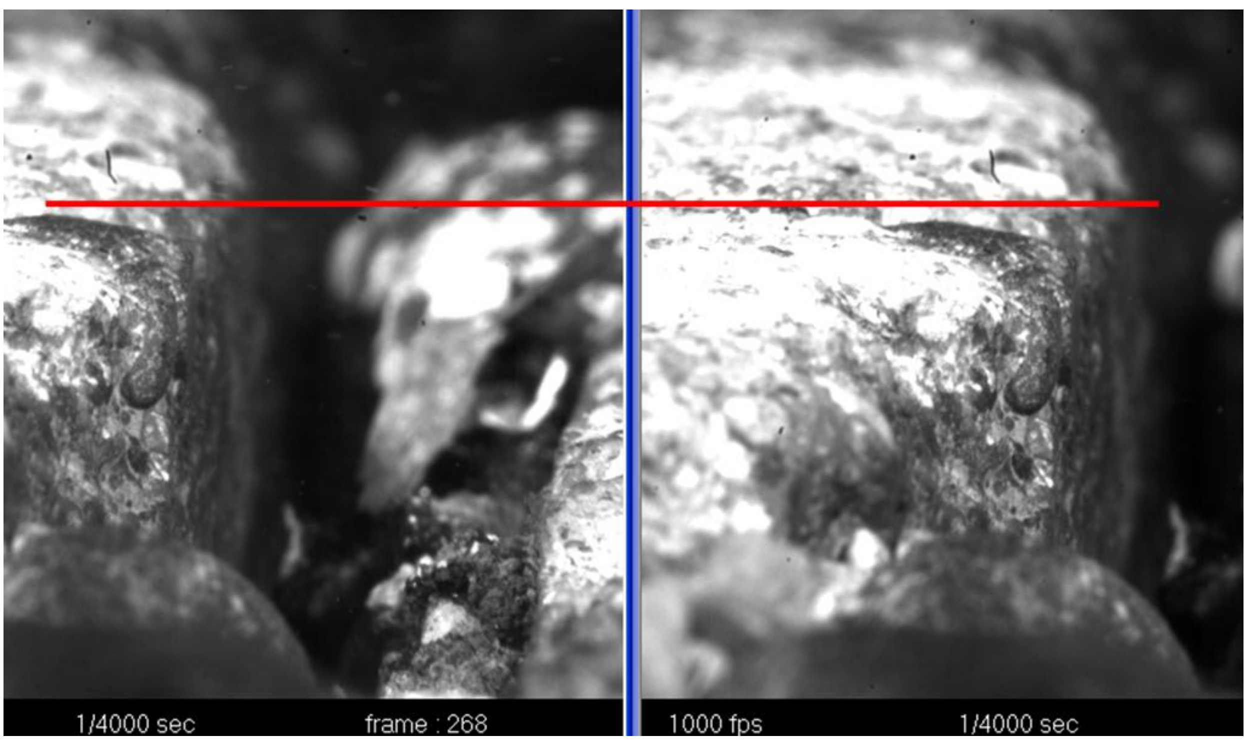


Figure 4.3. Camera captures fault movement on order of 10 mils

4.2 Direct Material Property Measurement

The use of guided waves for non-destructive testing has become increasingly common over the past few years. One of the aspects that make guided waves attractive for testing is their ability to propagate along surfaces over distances of tens of centimeters and therefore to allow a significant increase in scanning speed. Other aspects are the possibility of material characterization and economical flaw detection in layered media such as pavement systems (Strycek et al., 1997). The dispersion curve of surface waves in a layered medium provides information about shear wave velocity as a function of depth that can be used to estimate stiffness profile of the test site. The most commonly used methods to measure surface wave dispersion curves are Spectral Analysis of Surface Waves (SASW) and Multi-channel Analysis of Surface Waves (MASW).

Air-coupled surface wave sensing for concrete was first proposed by Zhu and Popovics (2002). The testing scheme is shown in Fig. 4.4. A directional microphone was used to detect leaky surface waves emitted from a concrete slab, where transient impact sources were generated by a hammer or a steel ball impactor. The experimental study showed that air-coupled sensors are very sensitive and have a high signal-to-noise ratio even after propagation over large distances (up to 10 m), which is valuable for rapid scanning of large-scale structures. The hyper-cardioid sensing field of the directional microphone significantly reduces the effect of the direct acoustic waves and ambient noise. These test results showed the potential of applying air-coupled sensors to surface wave dispersion curve measurement.

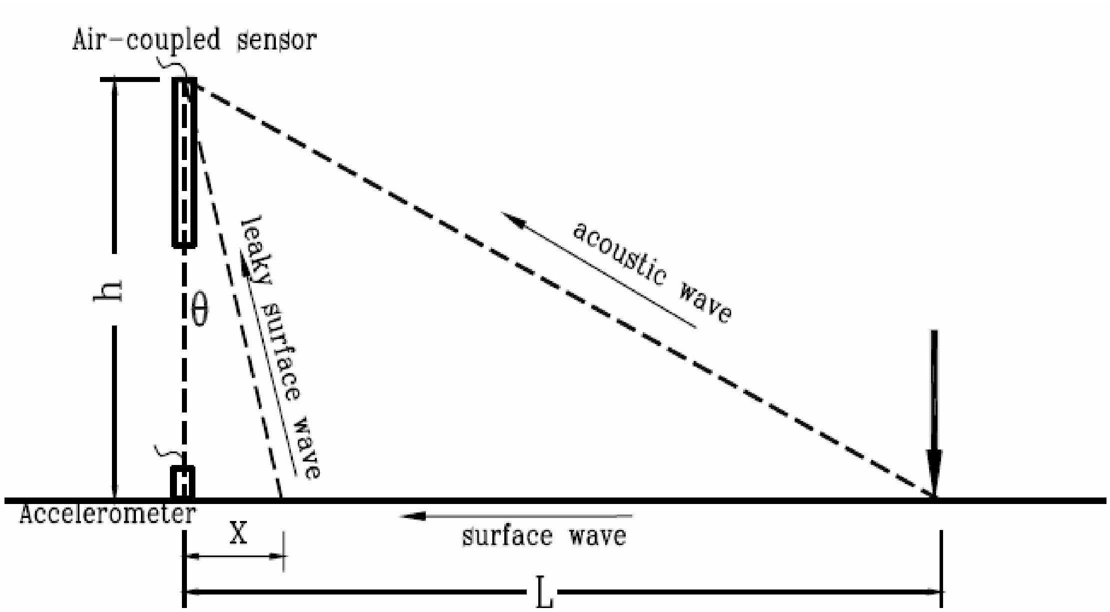


Figure 4.4. Leaky surface wave detection scheme using an air-coupled transducer (Zhu and Popovics, 2002)

Recently, Zhu and Popovics (2006) reviewed the progress of air-coupled surface wave sensing methods in NDE of concrete structures. Their theoretical analysis showed that air-coupled sensors are able to detect leaky surface waves that are excited by a transient point load in concrete. Experimental results showed that air-coupled sensors are able to replace contact sensors in traditional surface wave tests, and therefore greatly improve test efficiency.

For the work presented here, some proof-of-concept testing was completed using the air-coupled surface wave method by researchers at ISU's Center for Nondestructive Evaluation (CNDE) (Fig. 4.5). The implementation principle was to use an impact source (such as studs embedded in a tire of a moving vehicle) to excite sound waves in a concrete roadway. An array of microphones would be used to measure the leaky Lamb or leaky Rayleigh sound waves propagating in the concrete. The results of this testing are concrete wave velocity (possibly including frequency dependence) as a proxy for stiffness, plus some measure of sound wave absorption. The expectation is that concrete wave velocity and sound absorption will correlate with roadway condition and the presence of subsurface damage. Figure 4.6 shows superimposed time-domain and frequency-domain acoustic waveforms from a series of impact (ball drop) tests at different standoff distances on the floor of ISU's Howe hall basement. Figure 4.7 shows these same time-domain waveforms lined up in a waterfall plot. The approximate wavespeed is easily visible as the arrival slope in Fig. 4.7.

Figure 4.5. Proof-of-concept testing using the air-coupled surface wave method at ISU

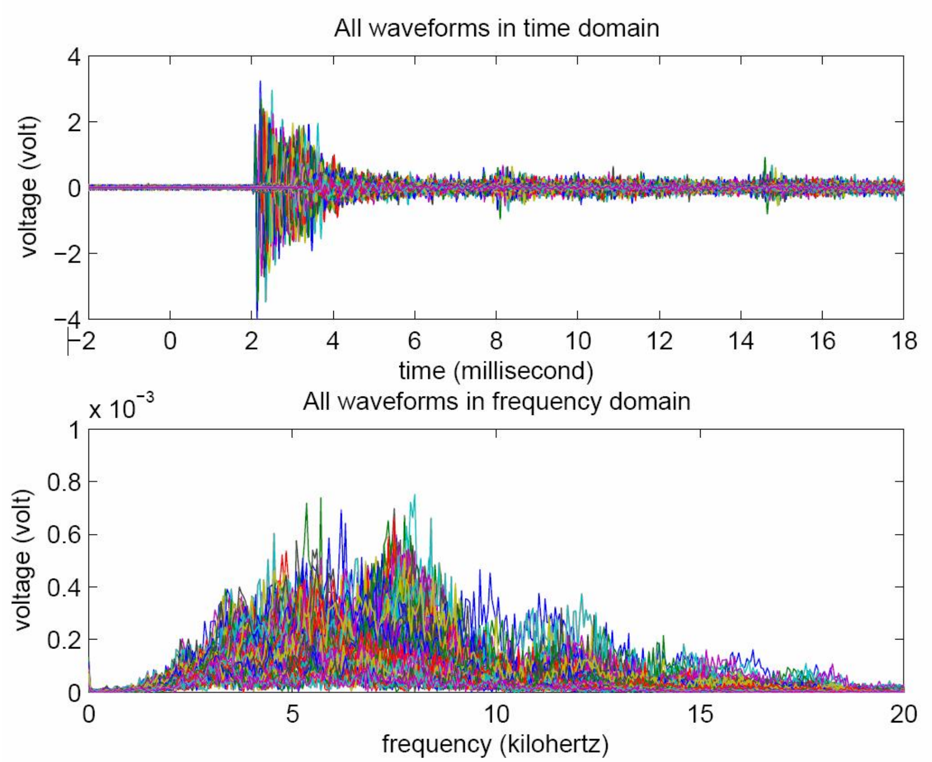


Figure 4.6. Superimposed time-domain (top) and frequency-domain (bottom) acoustic waveforms

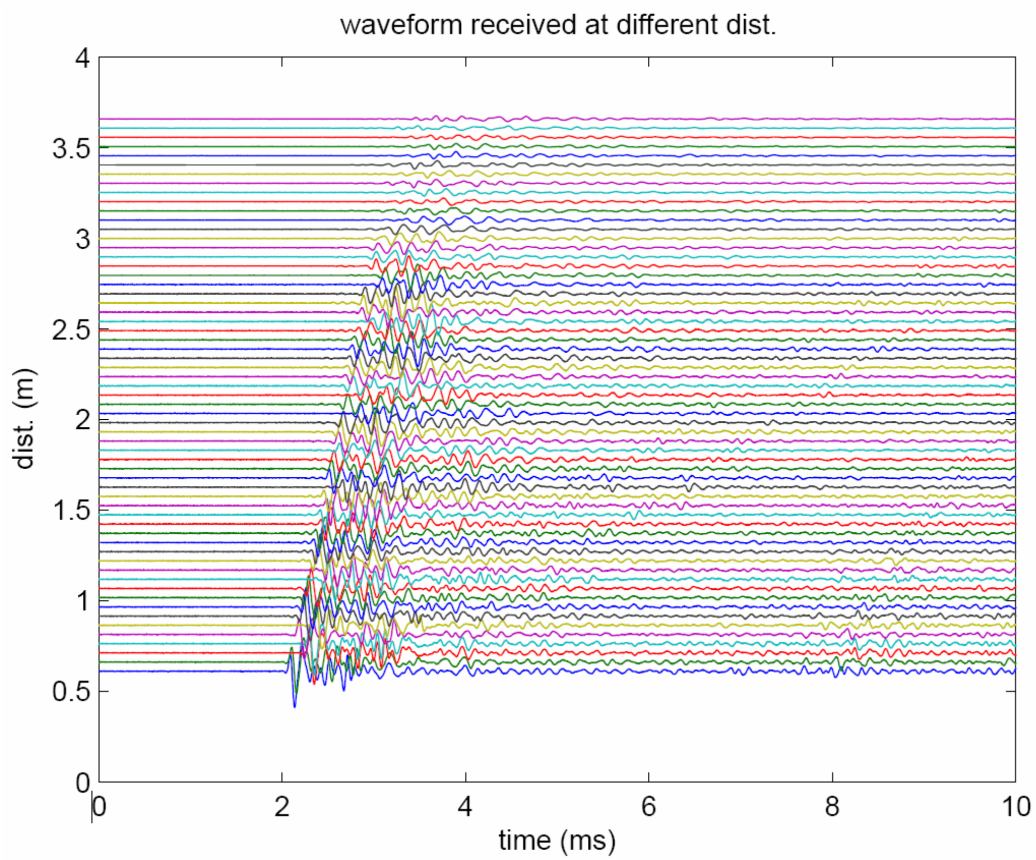


Figure 4.7. Waterfall plot of time-domain waveforms

A better way to view these same data is after a two dimensional Fourier transform into the spatial frequency/temporal frequency domain as seen in Fig. 4.8. On this plot, the phase velocity can be directly evaluated from the coordinates (temporal frequency divided by spatial frequency) and the group velocity from the slope. In this case we observe a wavespeed of approximately 1.95 m/ms. In a realistic device, since each waveform would require a separate microphone, many fewer than the roughly 60 waveforms shown here would likely be used, but more advanced signal processing and automatic identification algorithms (such as root-MUSIC) would be applied to compensate.

A controlling factor of the practicability of this system is whether the ambient noise level on roadways is too high to allow this system to perform adequately. To address this issue, a preliminary field noise test was conducted, comparing audio noise levels on a highway with the signal levels observed in the laboratory. The preliminary conclusion based on these data is that such a device should perform well as long as traffic is light. While the noise of a passing car was well below the signal level, a passing semitrailer may cause the signal to noise ratio to drop below 1.0.

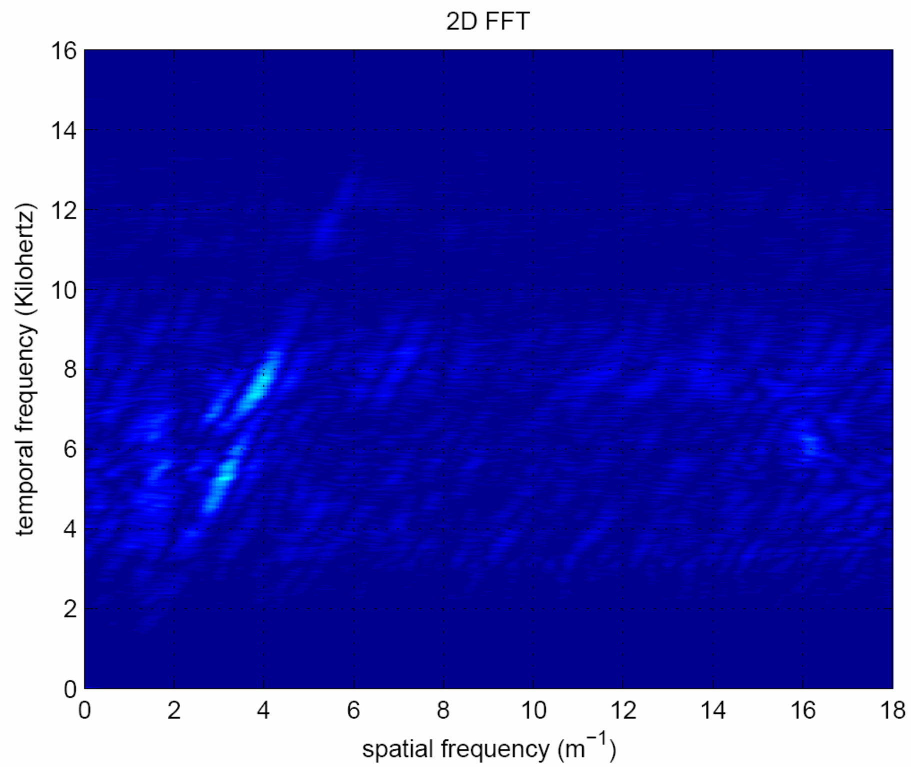


Figure 4.8. Two-dimensional Fourier transform of waveforms

Quite evidently, a large number of technical questions, issues, and challenges remain some of which are listed below:

- Suitability of acoustic attenuation/wavespeed for evaluating concrete condition.
- Airflow and turbulence effects due to running these tests at speed (wind tunnel testing).
- Presence of nearfield road/engine noise from the vehicle carrying the measurement system.
- Effect of sound reflections off of vertical embankments or tunnel walls on system performance.
- Design of an array using lower cost microphones.
- Development of a suitable source of impact noise such as a studded tire.
- Vibration and bouncing of the sensor array during test vehicle motion.
- Construction of a suitable prototype.

5. MARKET NEED

A brief market needs assessment was conducted for network level structural evaluation for rigid and composite pavements based on the general concept of the measurement system. This assessment was intended to determine if roadway owners would utilize such a system if it was available and assess their interest level in using the results as part of their pavement management system (PMS).

To accomplish this task, a simple survey was sent to the members of the Falling Weight Deflectometer (FWD) users group. Sixteen (16) transportation agencies responded representing 14 state DOTs and 2 Provincial governments in Canada. Even though this is not a complete sample of the 50 state DOTs, the survey results are indicative of the interest in the measurement system.

The survey included the following 5 questions:

1. *How many lane-miles of concrete pavements does your agency own? If you have a break down in terms of plain concrete, jointed plain concrete, jointed reinforced concrete, or continuously reinforced concrete, please provide those miles separately.*

2. *Would your agency be interested in network level structural evaluation of concrete pavements conducted at highway speeds? If you answer NO, please skip to question 5.*

3. *The cost to conduct network level evaluation of pavements (IRI, rutting, or distress) can range from $20 per lane mile for IRI and rutting to over $100 per lane mile for distress. From your experience with FWD testing, what would be a reasonable cost for continuous structural evaluation?*

4. *Would your agency be interested in owning the equipment or having a service contract to provide the data?*

5. *Does your agency use structural information (or structure related) as part of a pavement management system?*

Survey Results

1. The majority of the respondents (11 out of 16) were interested in the measurement system. Only 4 said they were not interested and 1 agency didn't own any rigid pavements. Of those 11 that were interested, 7 already use some sort of structural evaluation as part of their pavement management system.

2. In terms of cost, the agencies that were interested indicated that they would be willing to invest in a service contract for data collection at the following rates:
    a. 3 agencies indicated willingness to pay $200 per mile
    b. 5 agencies indicated willingness to pay between $50 and $100 per mile
    c. 2 agencies indicated willingness to pay less than $50 per mile

3. In terms of number of miles to be covered, the agencies interested had a total of 71,000 miles compared to less than 2,000 for those that indicated no interest.

The survey indicates that there is a potential market for such a measurement system. Clearly, the consequences of making a bad decision (based on incomplete, inaccurate, or old data) can be very costly to the highway agencies maintaining these large infrastructure systems. For example, the cost of an additional inch of an asphalt overlay is roughly $50,000 (based on Iowa DOT estimates for 2008). The decision on how many inches are needed is based on the remaining structural capacity of the pavement. Without objective measurements, engineers do not have the proper information to make those decisions.

6. SUMMARY AND RECOMMENDATIONS

This work described in this report started with many big questions. The primary objective was to determine if it might be possible to develop a device capable of analyzing the performance of rigid and composite pavements and, if so, did a market exist for such a technology. The intent of this project was broad in nature and the questions to be answered also broad. However, the research team has developed the following conclusions:

- It appears that it is theoretically possible to make the measurements needed to manage a system-wide rigid pavement inventory. It should, however, be pointed out that the identified technologies would appear to be required to operate at the limits of their accuracy and precision. With the rapid advance of technological capabilities, this may optimistically become a non-issue.
- The use of time-coincident measurements, which according to reviewed literature has never been utilized, is an important component of developing the above conclusion. It appears to the research team that attempting to make use of position-coincident measurements may be leading to the observed difficulties with other systems on rigid pavements.
- Due to overall magnitude, the measurement of joint LTE appears to be the easiest pavement condition to assess. The measurement of joint LTE may be measured with sufficient accuracy with high speed digital cameras. It may also be measured sufficiently with distance measuring devices.
- Sonic based technologies seem promising for making both indirect (i.e., deflection based) and direct measures of pavement system modulus.
- The market needs study indicates that there is interest in the use of a system for making system-wide PC pavement measurements. The costs users might be willing to pay for such measurements appear to be in line with other similar types of measurements and what might be possible with such a device.

Based upon the conclusions described above and the entire collection of work summarized herein, the research team has developed the following recommendations:

- A phase 2 of this research should be conducted to further evaluate the two approaches for making pavement modulus measurements (e.g., sonic for distance measurement and sonic for direct measurement). This study should primarily be conducted in controlled laboratory setting with a small field study. The need for such a study is to further verify that the measurement needs can be met by the identified technologies and proposed system configurations.
- At the conclusion of phase 2, the phase 1 and phase 2 work should be subjected to an exhaustive peer review. This peer review will help to determine if further development should continue.

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[i] http://www.micro-epsilon.com/staticcontent/PDF/Prod_EN/Datasheet_-_optoNCDT_1810-50_2210_-_en.pdf

[ii] Frank L. Scarpace, *Softcopy Photogrammetry,* University of Wisconsin, Madison, April 15, 2003. Available online at http://www.ncgia.ucsb.edu/ncrst/meetings/20030415MSN/presentations/Scarpace2.ppt, accessed January 4, 2008

[iii] See, for example: http://www.redlake.com/