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Final Report

Development of a Structural Index as an Integral Part of the Overall Pavement Quality in the INDOT PMS

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INDOT Research

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Introduction

Transportation agencies spend billions of dollars annually on pavement maintenance and rehabilitation to meet public, legislative, and agency expectations. Knowledge of the structural condition of a highway pavement is crucial for pavement management at both the network level and the project level, particularly when the system monitoring, evaluation, and decision-making are to be made in a context of multiple criteria that include structural condition. A key aspect of the performance criteria for multiple criteria decision making is that the criteria must be amenable to scaling so that it can be duly incorporated in the overall utility function.

The main objectives of this research study are:

- To develop a pavement structural strength index (SSI), scaled logistically from zero to a 100, based on the falling weight deflectometer (FWD) deflection measurements,
- To formulate SSI in such a manner to be used as an index or employ the value of “100 – SSI” as a deduct value from pavement distresses surface index, and
- To develop models by which the SSI could be estimated given functional class, age, and drainage condition wherever deflection measurements are not available.

Extensive literature review of existing information related to pavement structural capacity assessment was conducted.

Necessary data was collected from the Indiana Department of Transportation (INDOT) pavement management databases and deflection measurements available at INDOT Research and Development for both project and network levels. Information from INDIPAVE (a database that includes data on weather conditions, highway classification, traffic, and other information at over 10,000 one-mile pavement sections in the State of Indiana) were also employed. Weather information was also collected from the Indiana State Climate Office. The data includes information on 12,250 road sections from 1999 to 2007.

Data was classified by pavement surface type (whether it is asphalt or concrete) and system classification (whether it is an interstate, a non-interstate but part of the national highway system (NHS), or a non interstate and not a part of the national highway system (non-NHS).

Findings

Data collected for center deflection δ_1 in mils, 1/1000 inch (corrected for a load of 9000 pounds and a temperature of 68 F), international roughness index, IRI, (in inches per mile), rut depth in inches, and pavement condition rating, PCR, (historically used by INDOT as surface distresses index) revealed the following information about the network pavement condition in Indiana;

- For asphalt pavement surfaces, average δ_1 is 3.1, 5.0 and 7.9 mils for interstates, NHS, and non-NHS respectively. Average IRI is 76, 102 and 110 inches per mile. Average PCR is 89, 90 and 89 (out of a 100). Average rut depth is 0.09, 0.14 and 0.13 inch.
- For concrete pavement surfaces, average δ_1 is 3.6, 4.9 and 8.9 mils for interstates, NHS, and non-NHS respectively. Average IRI is 98, 103 and 144 inches per mile. Average PCR is 94, 93 and 86 (out of a 100).

The following equation is developed to calculate the pavement structural strength index (SSI);

$$SSI_{jk} = 100\left(1-\alpha e^{-\frac{\beta}{\delta_1{}^{\gamma}}}\right)$$

where δ_1 is the pavement surface deflection and α, β, and γ are regression coefficients based upon pavement surface type (whether it is asphalt or concrete, *j*) and highway classification (whether it is interstate, NHS, or non-NHS, *k*).

Regression analyses were also employed to establish a relationship between SSI and $\boldsymbol{\delta_1}$ and a deduct value, DV, (100-SSI) is developed;

$$\mathbf{DV = 0.0034\delta_1^3 - 0.2062\delta_1^2 + 0.3224\delta_1}$$

This deduct value is recommended to be incorporated into the pavement surface distresses index.

Finally, models by which the SSI could be estimated given functional class, age, and drainage condition wherever deflection measurements are not available were developed and calibrated. However, these models still need additional refinements and calibration.

Implementation

This research study developed a pavement structural strength index (SSI) and associated deduct values using the falling weight deflectometer (FWD) deflection measurements. The SSI and/or its associated deduct value are recommended to be incorporated into the pavement surface distresses index.

Network level FWD program needs to be revived.

Decisions for pavement preservation and rehabilitation treatments should continue to be driven by pavement structural capacity.

Additional research is recommended to produce prediction models, trigger values and remaining life estimations based on structural capacity.

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CHAPTER 1. INTRODUCTION

1.1 Background and Problem Statement

Pavements constitute one of the major components of highway assets. Today, most highway agencies use some form of pavement management systems (PMS) to manage their pavement assets, aiming to improve the structural and functional performance of pavements at a given budget level. The evaluation of pavement structural condition is an integral component in pavement management. The determination of structural capacity first requires the monitoring or the measuring of some structural characteristic or condition of the pavement. It then involves the analysis of the resulting data, using either a theoretical or empirical basis, to estimate the load-carrying capacity (or remaining service life) of the pavement under expected traffic conditions. This information would then allow agency to decide the type, timing and cost of a treatment to perform on a given pavement section and the allocation of funds within the entire network to perform that treatment.

Typically, the deflections or layer material properties are used to represent the structural strength or condition of a pavement section. Measurement of the structural condition and evaluation of the structural capacity can be performed at both project and network levels, depending on the scope of application shown in Figure 1.1. At the project level (on a specific pavement section or project), this information is used to serve as the as-built record, to provide input to overlay design, to determine the as-built structural adequacy, and to estimate the remaining service life of the pavement. At the network level, the information can be used to determine average network structural conditions, to predict deterioration behaviors, evaluate future structural inadequacies, plan for future work program and assess future funding requirements (Haas et al., 1994).

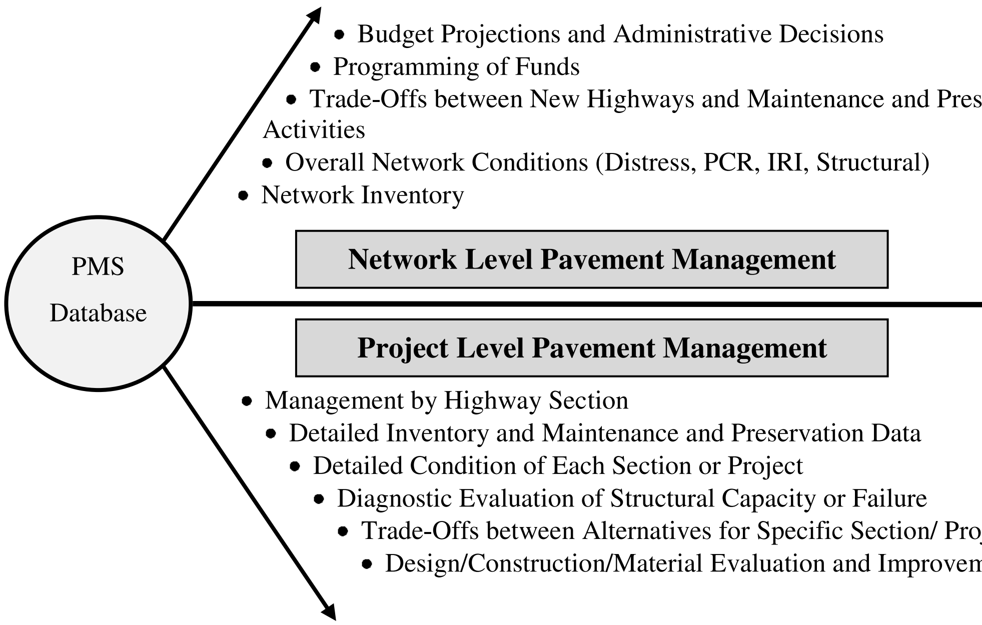


Figure 1.1: PMS Functions at Network and Project Levels

Due to the expense in data collection and effort in performing structural capacity analysis, structural condition is mostly evaluated at the project level (i.e. specific pavement sections or projects) and seldom at the network level (i.e. entire highway network). Even with the advent of non-destructive testing techniques to measure the pavement structural condition (such as the falling weight deflectometer test), structural data collection at the network level is still a costly affair in terms of both time and money. Today, most highway agencies can only collect structural condition information (in the form of deflections or back-calculated material properties from falling weight deflectometer tests) at a frequency ranging between three and five years for the entire highway network (Uddin, 2006). This results in a lack of a comprehensive network level database on pavement structural condition, causing to state highway agencies to consider only functional performance (i.e. surface distress and roughness) in their pavement management decisions.

Noting the difficulties state highway agencies face in collecting network level pavement structural condition information and the current lack of consideration of structural performance, the present research explores the possibility to develop an alternate structural strength index that can be used in network-level pavement management. In Indiana, the index must complement agency's existing practice of collecting pavement structural condition data using falling weight deflectometers every five years. Under a multi-criteria multi-objective pavement or asset management framework, the developed structural strength index should be scalable and bounded. Typically, performance measures have to be scaled, weighted and amalgamated before performing the actual ranking, prioritization and programming (Sinha and Labi, 2007). By ensuring that the structural strength index is bounded before-hand, the need for further assumptions on bounds and additional mathematical treatments are eliminated.

To allow a more frequent update on structural condition information, statistical models have to be developed to estimate the structural strength index in the absence of material property or deflection data. Ideally, the statistical models should provide an annual update of the structural condition database. The complete structural condition database can then provide the basis for developing pavement structural performance models and allowing other network-level pavement management applications.

It is incomplete to develop a structural strength index without developing a usable set of thresholds or triggers for pavement maintenance, rehabilitation and reconstruction (M,R&R) activities. At the project level, these triggers are crucial in determining the type of treatment to perform, and when and where to perform a particular treatment. At the network level, triggers and decision matrices are required to plan, budget and program pavement activities for the entire highway network.

The development of such a structural strength index is expected to aid in the incorporation of pavement structural performance in existing pavement management systems and maintenance management systems, and enhance the integration of these systems into an asset management framework. The realization of this study shall ensure a more efficient and effective budget allocation, planning and programming to address structural deficiencies of pavements in a highway network.

1.2 Objectives of Study

The objectives of this research study can be described as follows:

1. To develop a structural strength index that can be readily applied in network level pavement management.
2. To develop a set of statistical models that allow the prediction of the structural strength index in the absence of detailed structural condition information.

1.3 Scope of Study

The scope of study is as follows:

- Coverage: The present study is focused on the state highway system in Indiana. The entire highway network of interstates, national highway system (NHS) highways, non-interstate non-NHS highways (more than 22,000 one-mile segments) is used as the primary statistical unit for the analyses.
- Analysis period: The study period ranges from 2004 to 2007 as this is the common overlap of availability of existing data from various sources. In some cases, data was obtained for the period before 2004 to facilitate the statistical analyses and to evaluate the structural improvement of maintenance, rehabilitation and reconstruction activities.
- Pavement type: The type of pavements considered in this study is dependent on the surface layer material and load transfer mechanism exhibited by the pavement structure. In this study, the two types of pavement considered are flexible/composite pavements and rigid pavements.
- Geo-climatic region: The present study utilized data from pavement sections from various locations in the state highway network. Statewide subgrade information can be inferred from the Indiana Soil Survey (Soil Survey Division Staff, 1993) while statewide weather information is obtained from the Indiana Climate Office at Purdue University (Indiana State Climate Office, 2008).

- Pavement activity type: On the basis of current practices, the following pavement activities are considered – maintenance, rehabilitation and reconstruction. This classification shall provide the basis of subsequent trigger and decision matrix development.

1.4 Outline of Report

The present report is divided into eight chapters. *Chapter 1* briefly discusses the existing issues related to pavement structural condition data collection and evaluation at the network level and the impetus of developing a structural strength index that can complement existing network level data collection methods and is applicable in network level pavement management. A literature review of the existing methods to evaluate structural condition and its application in pavement management is presented in *Chapter 2*. Also, past research on trigger and decision matrix development for the different structural condition indices is described. The study framework and the methodology in the developing a network-level structural strength index is discussed in *Chapter 3*. *Chapter 4* describes the data collection effort to facilitate the development of the structural strength index. The dataset included information on pavement structural condition, traffic loading, weather and other related data. The development of the structural strength index that can be used to quantify structural performance within a highway network is discussed in *Chapter 5*. Statistical models that can be used to estimate structural strength index in the absence of deflection data are presented in *Chapter 6*. *Chapter 7* concludes with the main findings of the research study and provides recommendations for future research.

CHAPTER 2. LITERATURE REVIEW

In this chapter, a review of existing literature related to the development of structural condition indicators and their use in pavement management is discussed. First, the basic concepts of network level structural condition monitoring are introduced. Then, various methods that can be used to evaluate the structural integrity of pavements are discussed. Past research on the development of structural condition indices for both project and network levels are described. Last, the pitfalls of the currently used structural condition indices are discussed.

2.1 Structural Condition Monitoring on a Highway Network

Structural evaluation is routinely conducted to assess pavement structural integrity and load carrying capacity. Structural condition monitoring on a regular or periodic basis is typically accomplished within the scope of a network monitoring plan. Figure 2.1 shows a framework used by state highway agencies to collect structural condition data within a pavement management system (Hudson and Finn, 1974). There are several issues related to the monitoring plan, as discussed below:

Functional vs. Structural Evaluations: Both functional and structural performances are important in network level pavement management. Serviceability observations below an acceptable level have been practiced commonly in most highway agencies to trigger structural improvements at both network and project levels (Theberge, 1987; Lamptey et al., 2005). For example, a rough pavement may have adequate strength and requires only a functional overlay, but it may also be a result of structural inadequacy and requires a structural overlay. To complete this interrelationship, structural indicators resulted from the evaluation must also be capable to predict or estimate remaining life of the pavement for expected load repetitions.

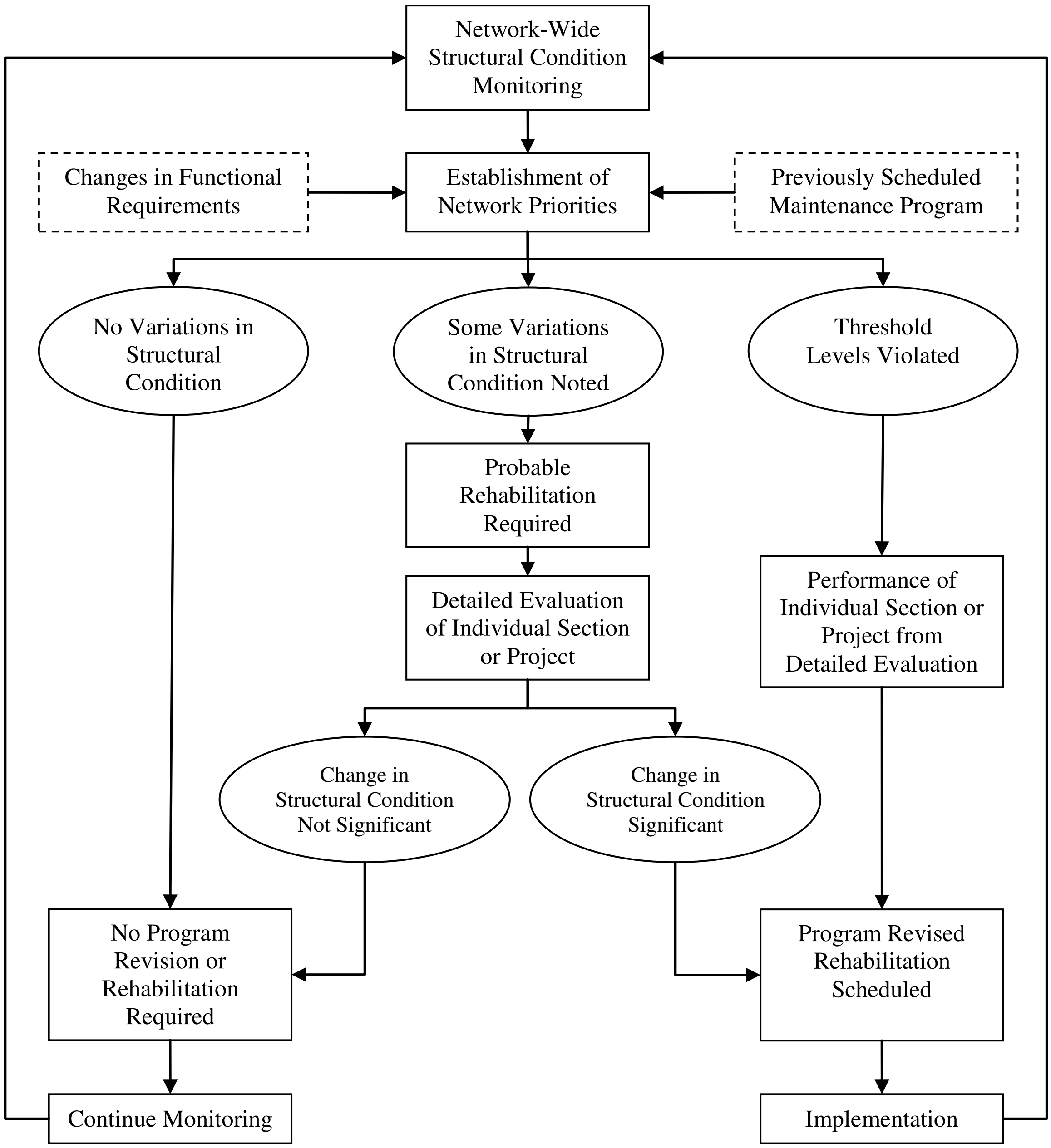


Figure 2.1: Framework for Structural Condition Monitoring within a Pavement Management System *(Source: Hudson and Finn, 1974)*

Criteria for Detailed Evaluation: In network evaluation program, some threshold criteria are needed. This can take the form of limiting deflection levels for a specified load, expected number of load repetitions, limiting index for structural conditions or others (Asphalt Institute, 2000; AASHTO, 2008). Evaluation can take the form of periodic inspection (annually, biannually, etc.) or routine inspection by maintenance and field engineers to report any unusual changes in structural behaviors of pavements. For example, the Indiana Department of Transportation performs periodic inspection of structural conditions once every five years (Noureldin et al., 2005).

Destructive vs. Nondestructive Evaluations: Testing methods can be classified as destructive and nondestructive. For pavements, destructive evaluation usually involves a test pit for sampling and testing the component materials in the laboratory or in the field while non-destructive evaluation requires no disturbance of pavement materials. These shall be discussed in later sections of this chapter.

Project vs. Network Level Evaluation: Ideally, a highway agency should collect detailed structural information of pavement on every analysis section (typically a homogenous pavement section of a standard length). However, the realities are that highway agencies are operating on a limited budget and are managing an extensive highway network. Due to the expense of data collection and analysis, and limited funds available for data collection, highway agencies typically perform a network level structural inspection via nondestructive testing for the entire network at a frequency of three to five years (Noureldin et al., 2005; Uddin, 2006). While this information allows an indication of structural performance, the information collected at the network level typically do not yield enough specific information required for project level pavement management (e.g. material properties such as asphalt dynamic modulus, drainage characteristics of base materials for the design of a structural overlay or reconstructed pavement structure, and local failures of pavement sections). This warrants the need to apply a set of criteria to select a set of pavement sections for detailed evaluation at the project level. Here the information obtained (such as detailed structural properties of pavement layers and materials) can be used for project level pavement management applications (such as treatment selection).

2.2 Methods to Evaluate Pavement Structural Integrity

Structural evaluation can be assessed using (a) direct visual inspection and distress measurement of physical structure, (b) measurement of layer thickness and material strength by physical destructive field coring and laboratory testing, and (c) nondestructive testing, which is desirable in most cases. Structural condition indices, based on objective measurements, indicate structural integrity and load-carrying capacity.

2.2.1 Surface Distress Identification and Evaluation

Pavement surface distress can be a manifestation of the deterioration of pavement structural condition. Distress information also provides clues to the predominant failure mechanisms of the pavement structure, leading to the selection of appropriate corrective maintenance strategies. The identification of various distress types to be included for measurement in a routine pavement condition survey is made on the basis of the experience of the agency personnel. Specific variables recorded and the units in which they are measured vary from agency to agency.

2.2.1.1 Pavement Surface Distress Identification

Manuals and standards have been developed for detailed distress identification and measurements. For example, the Distress Identification Manual for the Long-Term Pavement Performance Studies (LTPP) (Miller and Bellinger, 2003) provides guidelines for identification of distress type and severity. For asphalt pavements, including asphalt overlays on asphalt or concrete pavements, distresses can be classified into cracking; patching and potholes; surface deformation; surface defects; and miscellaneous distresses as shown in Table 2.1. For jointed and reinforced Portland cement concrete pavements (JCP), including concrete overlays on PCC pavements, distresses can be classified into cracking; joint deficiencies; surface defects; and miscellaneous distresses, as shown in Table 2.2.

Table 2.1: Asphalt Concrete Pavement Surface Distresses Considered in LTPP

Distress Categories/Type	
Cracking	Surface Deformation
Fatigue Cracking	Rutting
Block Cracking	Shoving
Edge Cracking	Surface Defects
Wheel Path Longitudinal Cracking	Bleeding
Non-Wheel Path Longitudinal Cracking	Polished Aggregate
Reflection Cracking at Joints	Raveling
Transverse Cracking	Miscellaneous Distress
Patching and Potholes	Water Bleeding and Pumping
Patch/ Patch Deterioration	Lane-to-Shoulder Dropoff
Potholes	

Table 2.2: Jointed Concrete Pavement Surface Distresses Considered in LTPP

Distress Categories/Type	
Cracking	Surface Defects
Corner Break	Map Cracking
Durability Cracking (D cracking)	Scaling
Longitudinal Cracking	Polished Aggregate
Transverse Cracking	Popouts
Joint Deficiencies	Miscellaneous Distress
Transverse Joint Seal Damage	Blowups
Longitudinal Joint Seal Damage	Faulting of Transverse Joints and Cracks
Spalling of Longitudinal Joints	Lane-to-Shoulder Dropoff
Spalling of Transverse Joints	Lane-to-Shoulder Separation
	Patch/ Patch Deterioration
	Water Bleeding and Pumping

The Indiana Department of Transportation also developed the Pavement Condition Data Collection Manual (INDOT, 1998) where surface distresses are classified for flexible/composite pavements and jointed concrete pavements (shown in Tables 2.3 and 2.4). The distress data is evaluated via automated data collection procedure. Note that in this case the surface deformation (i.e. rutting and shoving) and pavement roughness are evaluated separately from pavement surface condition rating. On the other hand, Chapter 52 of the INDOT Design Manual (INDOT, 2007) offers a somewhat different interpretation on distress classification, with a more in-depth view on both functional and structural distresses, as shown in Table 2.5.

Table 2.3: Composite/Flexible Pavement Distresses Considered in INDOT Condition Data Collection Manual

Distress Category	Distress Types
Cracking	Alligator Cracks
	Transverse Cracks
	Block Cracks
	Longitudinal Cracks
	Edge Cracks
	Widening Cracks
Patching and Potholes	Patching
	Potholes
Surface Defects	Raveling
Surface Deformation	Rutting
Miscellaneous	Pumping
	Maintenance actions

Table 2.4: Concrete Pavement Distresses Considered in INDOT Condition Data Collection Manual

Distress Category	Distress Types
Cracking	Corner Breaks
	D Cracks
	Transverse Cracks
	Longitudinal Cracks
Joint Deficiencies	Transverse Joint Spalling
	Longitudinal Joint Spalling
	Transverse Joint Seal Damage
Miscellaneous	Pumping
	Faulting

Table 2.5: Distress Types Considered in INDOT Design Manual

Asphalt Pavement	Concrete Pavement
Block Cracking	Alkali-Silica Reactivity
Flushing	Blow-ups
Frost Heave	Corner Breaks
Longitudinal Cracking	D-cracking
Polishing	Faulting
Raveling	Joint/Joint Seal Failure
Reflective Cracking	Longitudinal Cracking
Rutting	Polishing
Shoulder Drop-off	Poor rideability
Stripping	Pop-outs
Thermal Cracking	Punch-outs
Alligator/Fatigue Cracking	Transverse Cracking
Weathering	Scaling
	Spalling
	Structural Failure

A review of existing literature revealed a variety of guidelines on pavement distress identification among state highway agencies (Hudson and Uddin, 1987; Shahin and Walther, 1990; SHRP, 1993; INDOT, 1998; ASTM, 1999; Miller and Bellinger, 2003; INDOT, 2007). Broadly, distress types can be classified into the following three groups:

Cracking: Examples of load-induced cracking in asphalt (or flexible) pavements are wheel-path longitudinal cracks and alligator cracks (or fatigue cracks). Transverse cracks (or thermal cracks) are typically caused by accumulated temperature variation that the asphalt layer has experienced. Block cracking is environmentally associated and is caused by shrinkage of hot-mix asphalt and daily thermal cycles. Reflection cracking appears as regularly spaced transverse cracks on the asphalt layer paved over an old jointed concrete pavement or a strong base stabilized with cement or lime. Cracking on concrete pavements can appear as linear (mostly load-associated), durability or "D" cracking (due to freeze thaw expansion of certain aggregate types), map cracking (due to alkali-silica reaction or over-finishing of concrete surface), and shrinkage cracks.

Deformation: Examples of deformation distresses in asphalt pavements are rutting (associated with poor asphalt mixes and interaction with repeated traffic loads and high ambient temperatures), shoving and bumps (related to poor asphalt mix design or construction), and depression (caused by the settlement in the subgrade). Faulting of transverse joints and depression comprise a common distress in concrete pavements and are caused by a combination of factors such as repeated traffic loads, insufficient dowel transfer, water-filtration, loss of materials below joints, and temperature gradient of the concrete slab.

Surface Defects: Examples of surface defects in asphalt pavements are flushing or bleeding (indicating excessive asphalt content), raveling and weathering (interaction of mix problems and environment), potholes (appearing more in spring – thawing interaction with repeated traffic loads), and patches (indicating the repair of localized areas of distresses or utility cuts and trenches in the pavement). Examples of surface defects in concrete pavements are joint deterioration, punch-out, corner break, spalling, aggregate pop-outs, and pumping (subsurface water damage and interaction with traffic load repetitions).

2.2.1.2 Pavement Surface Distress Data Collection

Distress data can be collected or monitored by manual visual inspections or with the help of photographic and video recording in the field followed by data interpretation in the office.

Visual Inspection: Visual inspection and observation is the most common method of condition monitoring and is generally through a survey conducted from the windshield of a slow-moving van. Such surveys are labor intensive, expensive and subject to inspector's judgments. However, in some emergency cases, the manual inspection is necessary, such as post-catastrophe (hurricane, flood or earthquake) evaluation of the condition of highway pavements. The cost of data collection can be reduced through sampling for a small network size.

Analog Imaging: The predominant use of analog imaging is in photographing (with 35-mm film) and videotaping. Images obtained can be of high quality, but they are not easily converted to digital format for computer storage and manipulation. Analog imaging has been less frequently used in recent years owing to the maturing of digital technology. The photographic method, popularly known as photo logging, was used by a few agencies for many years. It probably became most well known for its adoption as the method of choice for the LTPP program. The photo logging methodology essentially consists of photographing the pavement surface, usually with 35-mm film, and reduction of distress data through review of the film at a workstation. Photo logging vans typically use a downward facing camera and possibly one or more facing forward or in another direction, depending on user needs. Much of the work is done at night using lighted cameras to overcome problems with shadows cast by survey vehicles, traffic, or roadside features that can mask pavement features critical to proper distress evaluation. In most cases, photo logging is continuous over what INDOT defines as a roadway section or sample of a roadway section.

Digital Imaging: The use of digital cameras is rapidly becoming the preferred method of pavement imaging to capture pavement surface distress data (McGhee, 2004). Survey vehicle configuration is similar to that for videotaping in that one or two cameras capture the pavement image while any number may be used for other data required by the agency. Again, special lighting may be used to overcome shadowing problems. A major force behind the move toward digital imaging of pavements is the opportunity to reduce distress data from those images through automated methods. There are two types of cameras currently used to digitally image a pavement surface. These are known generally as area scan and line scan methods, although some vendors are using other terminologies (Wang and Li, 1999).

2.2.2 Destructive Evaluation of Pavement Structural Condition

It occasionally becomes necessary to undertake destructive testing by removing portions of the pavement structure at the project level to ascertain a particular problem and to determine how failure occurs. In general, destructive evaluation is performed when a particular pavement section has already displayed evidence of distress or on special road section (e.g. experimental test section or as part of the contractual requirements).

Destructive testing also gives an opportunity to collect material samples from different pavement layers and subgrade. For example, destructive testing has been used at the AASHO Road Test (AASHO, 1962) and the SHRP and LTPP studies (SHRP, 1989a, 1989b). Destructive testing techniques include coring in bound layers, boring in soft layers, and dynamic cone penetrometer (DCP) testing in subgrade soils (Uddin, 2002). The process also involves removing samples from various layers, examining the samples in the field, and then testing them in the laboratory. Only a limited number of samples are tested in the laboratory (for example, resilient modulus test of subgrade soils) because of the time and cost involved. Then the results are inferred to the remaining units or used as independent tests to verify and validate the results of deflection testing.

Table 2.6 shows some of the typical laboratory tests that can be conducted on pavement samples and cores to determine the strength of the materials. These tests are essential in determining the structural capacity of the pavement to carry expected traffic loading and estimating the remaining service life of the pavement. Readers can refer to standard texts on pavement materials (Huang, 2004; Papagiannakis and Masad, 2008) and standards and specifications listed in the references for a more detailed discussion of testing procedures for structural strength of pavement materials. The advantages of destructive testing to determine direct strength properties of the materials and thickness layers must be weighed against the disadvantages of removing portions of the pavement and replacing it with patches, and possibly degrading the property of that section where the sample is extracted.

Table 2.6: Standard Laboratory Tests for Strength of Pavement Materials

Pavement Materials	Material Property	AASHTO Specification*	ASTM Standard*
Subgrade and Aggregates	Resilient Modulus	AASHTO T307	-
	Modulus of Subgrade Reaction	AASHTO T221 AASHTO T222	ASTM D1195 ASTM D1196
	California Bearing Ratio	AASHTO T193	ASTM D1883 ASTM D4429
	R-Value	AASHTO T190	ASTM D2844
Asphalt Mixture	Compressive Strength	AASHTO T167	ASTM D1075
	Dynamic Modulus	AASHTO TP62	ASTM D3497
	Diametrical Tensile Strength	AASHTO T283	-
	Indirect Tensile Strength	AASHTO T322	ASTM D4867
Concrete Mixture	Compressive Strength	AASHTO T22	ASTM C39
	Flexural Tensile Strength	AASHTO T97 AASHTO T177 AASHTO T198	ASTM C78 ASTM C293 ASTM C496

* Refer to list of references for details of testing procedures.

2.2.3 Nondestructive Evaluation of Pavement Structural Condition

Nondestructive testing (NDT) provides response measurements, which are used to infer the information about the physical structure of the pavement from behavioral evaluations. Typically, these techniques are selected over destructive methods due to lower cost, less interruption to traffic, less damage to pavement and the ability to make a sufficient number of measurements to quantify variability. While non-destructive testing has these advantages over destructive testing methods, it should always be remembered that the nondestructive testing techniques evaluate only the response of the pavement and not the physical properties directly.

2.2.3.1 Deflection Measurement

Deflection measurements have long been used to evaluate the structural capacity of in-situ pavements. They can be used to backcalculate the elastic moduli of various pavement layers, evaluate load transfer efficiency across joints and cracks in concrete pavements, and determine the location and extent of voids underneath concrete slabs. Based on the type of loading applied to the pavement surface, NDT deflection testing can be divided into three categories: static or slow-moving loads, steady vibration and impulse loads.

Static or Slow Moving Load: The Benkelman beam, California traveling deflectometer and the LaCroix deflectometer are some of the devices in this category. The Benkelman beam, which was developed by during the WASHO Road Test (HRB, 1955), and has been used extensively by highway agencies for pavement research and overlay design around the world. It consists of a simple lever arm attached to a light-weight aluminum or wood frame, as shown in Figure 2.2.

Measurements are made by placing the tip of the beam probe between the dual tires of a loaded truck (usually 18 kip or 80 kN axle load) at the point where the deflection is to be determined. As the loaded vehicle moves away from the test point, rebound or upward movement of the pavement is measured by the dial gauge. This equipment is versatile and is simple to operate. However, it is slow and labor-intensive. In some cases, particularly on stiff pavements, the support legs itself may be within the influence of the loaded tire's deflection basin, causing inaccuracies in deflection measurement (Huang, 2004).

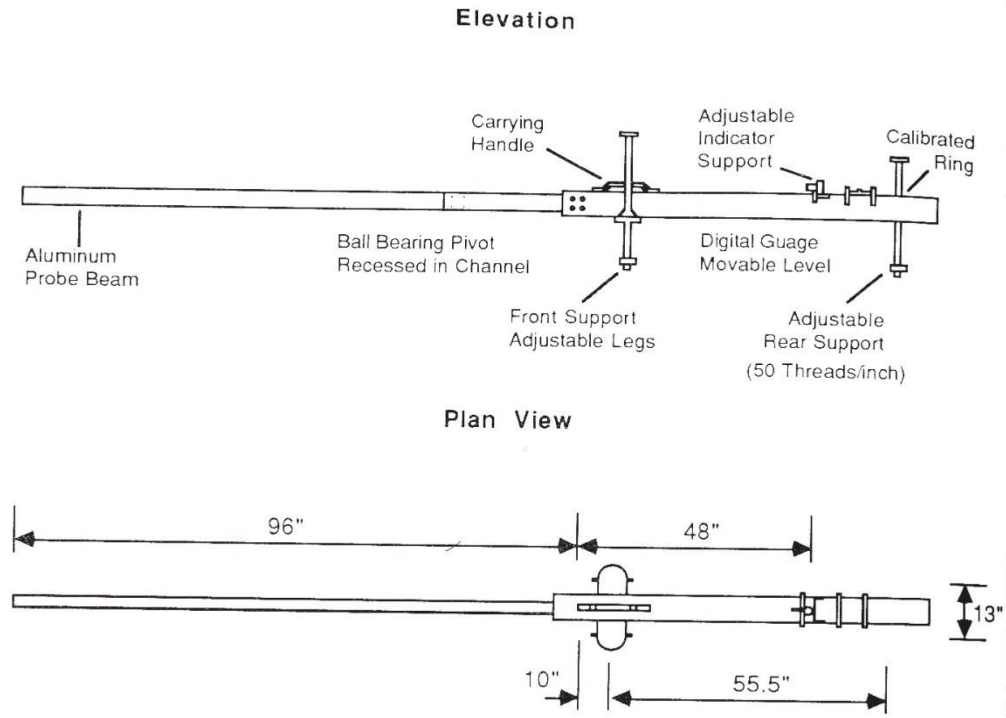


Figure 2.2: Illustration of a Benkelman Beam

The traveling deflectometer, developed by California Division of Highways, is used to measure deflections while a truck, generally with an 18 kip (80 kN) rear axle, is moving. The LaCroix deflectometer was developed in France and is used extensively in Europe. Like the California traveling deflectometer, the system measures deflection under both rear wheels.

A major problem with Benkelman beam, California traveling deflectometer and the LaCroix deflectometer is the difficulty to obtain an immovable measurement for making deflection measurements. This makes their validity on stiffer pavements questionable since the pivot itself can be within the deflection basin of the load. Also, the devices suffer from the disadvantage that static or slow-moving loads do not represent the actual transient or impulse loads imposed on the pavement by traffic. Therefore they cannot be applied mechanistically to pavement design and evaluation with extensive empirical correlations.

Steady-State Vibration: Steady-state vibratory devices, including Dynaflect, and Road Rater, produce a sinusoidal force imposed on a static load as illustrated by Figure

2.3. The deflections are generated by vibratory devices that impose a sinusoidal dynamic force over a static force. The magnitude of the peak-to-peak dynamic force is less than twice that of the static force and the device always applies a compressive force of varying magnitude on the pavement. Deflections are measured by acceleration or velocity sensors placed under the center of the load and at specified distance from the center usually at 1-ft (0.3 m) intervals.

One advantage of this type of equipment over the static equipment is that a reference point is not required (since we are tracking velocity or the change in deflection). An inertial reference is used so that the change in deflection can be compared to the magnitude of the force. The disadvantages of the method are that actual loads applied to the pavements are not in the form of steady-state vibration and that the use of relatively large static load could have some damaging effect on stress-sensitive pavement materials.

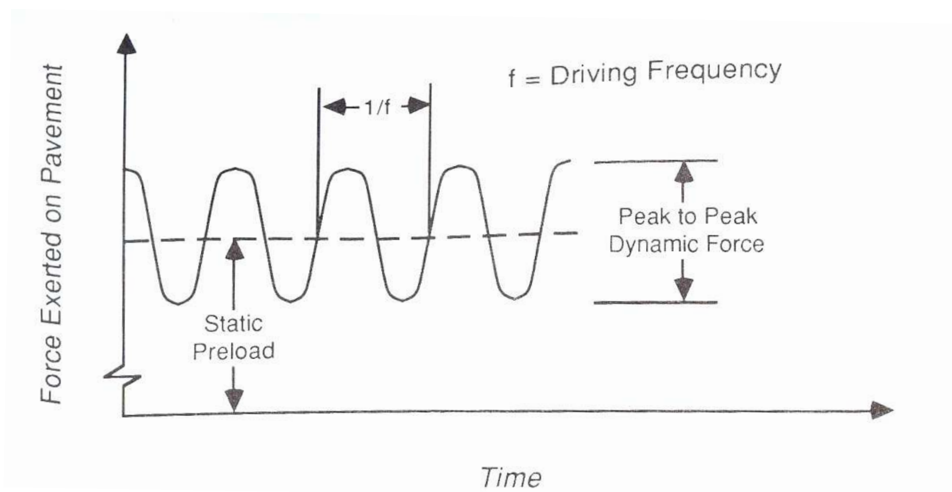
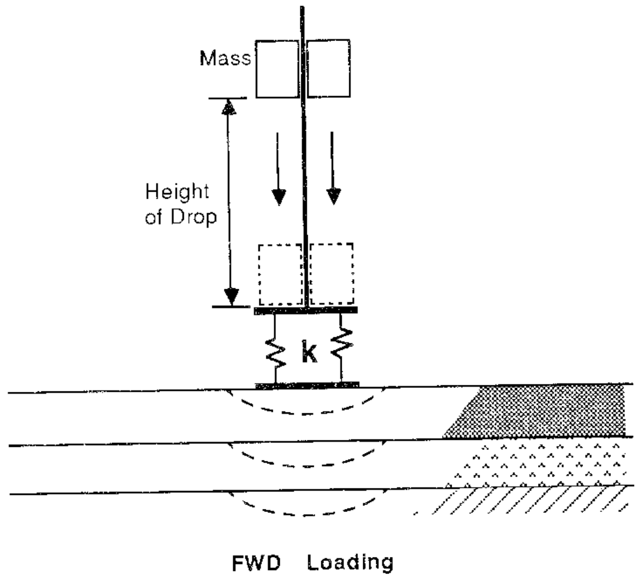


Figure 2.3: Typical Dynamic Force Output of Steady State Vibratory Devices

Impulse Load: Devices in this category deliver a transient force impulse to the pavement surface and include the various types of falling weight deflectometer (FWD). Figure 2.3 illustrates the basic principles of FWD. By varying the amount of weight and the height of the drop, different impulse forces can be generated. The normal operation is to move the trailer-mounted device to the test location, lower the loading plate and transducers hydraulically to the pavement surface, complete the test sequence by dropping the weight at each height selected, lift the loading plate and sensors and tow the device to the next site. The major advantages of the impulse loading device are the ability to model a moving wheel accurately in both magnitude and duration and the use of a relatively small static load compared with impulse loading.

One of the most widely used FWD among highway agencies in the United States is the Dynatest system. The impulse force is created by dropping a weight of 110, 220, 440 or 660 lb (50, 100, 200 or 300 kg) from a height of 0.8 to 15 in. (20 to 381 mm). By varying the drop height and weight, a peak force ranging from 1500 to 24,000 lb (6.7 to 107 kN) can be generated. The load is transmitted to the pavement through a loading plate to provide a load pulse in the form of a half-sine wave with a duration between 25 and 30 ms. The magnitude of the load is measured by a load cell while deflections are measured by seven velocity transducers mounted on a bar that can be lowered automatically to the pavement surface with the loading plate. One of the transducers is located at the center of the plate while the other six are located up to 7.4 ft (2.25 m) from the center (see Figure 2.4). The Dynatest FWD is also equipped with a microprocessor-based control console that can be fitted on the passenger side of the front seat of a standard automobile.

The obtained deflection data can be used to establish homogeneous pavement design sections. The peak deflection bowl and thickness information can be analyzed to backcalculate elastic moduli of different pavement layers and subgrade using the static layered linear elastic theory for flexible pavements and plate theories for rigid pavements. This can be considered as project-level analysis. Deflection data are not used for structural evaluation at the network level (Uddin and Torres-Verdin, 1998). In some cases more simplified indicators from deflection data are used for the network level, such as a structurally adequate (not needing structural maintenance or rehabilitation) or inadequate (needing detailed deflection testing and analysis) pavement section based on a limiting maximum deflection criterion. These indicators for project and network level analyses shall be discussed in the next section.



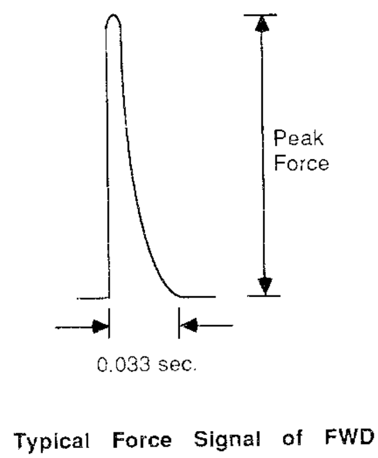


Figure 2.4: Basic Principle of the Falling Weight Deflectometers (FWD)

2.2.3.2 Other Nondestructive Testing Methods

Structural integrity of pavements can also be checked using other nondestructive evaluation (NDE) methods which do not subject the pavement to actual loading (such as heavy load deflection testing) or destructive testing. These NDE methods include seismic evaluation (such as the wave propagation) (Nair, 1971), vibration methods (such as the modal analysis), acoustic methods, electromagnetic method, and electrical resistivity methods (Hudson et al., 1987). Other noncontact and nondestructive testing technologies for the structural evaluation include the ground penetrating radar (GPR), infrared thermography, high-speed video and related optical methods, Moire technique, and ultrasonic sensors. Non-contact GPR data is used for determining surface layer thickness. Van-mounted GPR equipment has been used successfully to evaluate pavement surface layer thickness nondestructively at the network level (Corley-Lay and Morrison, 2001; Noureldin et al., 2005; McGovern et al., 2006).

2.3 Indicators for Pavement Structural Condition and Capacity

There exist many indicators that can be used to represent the pavement structural strength (or condition). In this section, some of the commonly-used indicators are discussed.

2.3.1 Pavement Distress Condition and Indices

The most common form is the pavement distress condition index. Almost all distresses can be identified at three severity levels (low, medium, and high) and the extent of each distress can be classified at three extent levels (low, medium, and high). A distress index, analysis to calculate a summary statistics, such as PCI, can be computed based on distress severity and extent measurements.

Distress surveys are carried out to assess the degree of physical pavement deterioration, which is a function of:

- Type of distress
- Severity of distress
- Extent of distress (amount or density of distress)

Each of the above three characteristics of pavement distress has a significant influence on the determination of the overall pavement deterioration. Because there are many types of distresses and a variety of ways to define severity levels and extent measurement, it is important to use or adapt standard procedures for distress identification and measurements of extent at each severity level. For practical and meaningful performance evaluation of a network, most distress data is combined into an overall pavement condition index. Washington state PMS first used the concept of deduct values for each measured distress from a perfect score of 100 for an excellent pavement with no distress, and a combined index of Pavement Condition Rating (PCR) was established (LeClerc and Nelson, 1982). The condition index data from the PMS database can be used to develop performance models for different environmental regions and different functional classes, which are reliable for a specific region and reflect the effect of pavement construction and maintenance practices. This concept is also adapted by other state highway agencies such as the Indiana Department of Transportation (INDOT, 1998) and Ohio Department of Transportation (Kanok et al., 2006).

The PAVER distress survey procedure combines the effect of various distress types and measurements of distress severity and extent into a single index, PCI to evaluate overall pavement condition of the surveyed section. PCI varies between 0 and 100. A value of 100 implies the pavement is in excellent condition and zero means a failed pavement (Shahin and Walther, 1990). Network-level PAVER analysis software has been implemented successfully, resulting in an ASTM standard (ASTM, 1999). The visual distress survey methodology and PCI calculation have also been adapted by many agencies and service providers for distress data measured by wind-shield surveys and interpreted in the office using video records.

Similar approaches are adopted in the mechanistic empirical pavement design guide (MEPDG) which was approved by AASHTO in late 2007 as the provisional pavement design procedure in the United States (AASHTO, 2008). In the design guide, distresses are identified using the procedures used in the Long-Term Pavement Performance (LTPP) program. According to the MEPDG, individual distresses are to be evaluated along with other parameters such as material properties and deflections to determine the structural adequacy of the pavement.

2.3.2 Pavement Material Properties, Critical Stresses and Critical Strains

Pavement material property is a direct indicator for pavement structural condition. Material properties can be determined from laboratory or field experiments mentioned in Table 2.6. These properties are important in the evaluation of critical pavement stresses and strains which in turn form the basis for determining the structural capacity (or the remaining life) of a pavement section via a mechanistic approach. So far, this has been the primary approach adopted in the development of the mechanistic empirical pavement design guide (MEPDG) (AASHTO, 2008) and has been widely researched (Luo and Prozzi, 2007; AASHTO, 2008; Ashaban et al., 2008; Muthadi and Kim, 2008). The MEPDG has listed a number of material properties (shown in Table 2.7) that are crucial to evaluate pavement (in terms of stresses and strains in the pavement structure) against following possible failure criteria for both flexible and rigid pavements:

- Flexible pavements
    - Vertical compressive stress limit at the top of the subgrade
    - Vertical surface deflection limit at the pavement surface
    - Critical tensile strain at the bottom of the asphalt or concrete layer
- Concrete pavements
    - Critical bending stress limits (both tensile and compressive) and deflection limits due to curling (i.e. temperature differentials)
    - Critical bending limits and deflection limits due to loading (center, edge and corner)

Based on the given information and the failure criteria, the remaining fatigue life of the pavement can be determined from Miner's hypothesis in the form of cumulative damage index:

$$DI = \sum \left(\frac{n}{N_f} \right) \tag{2.1}$$

where DI is the cumulative damage index (which is a function of material properties, stresses and strains experienced by the pavement), n is the number of axle load applications to date, and N_f is the allowable number of load applications to failure. From Equation (2.1), a DI of 0 indicates that the pavement has not experienced any form of traffic loading (such as a newly constructed pavement) while a DI of 1 indicates a pavement structural failure. There are different equations to define DI for both flexible and rigid pavements, and using Equation (2.1), the remaining life of a pavement section ($= N_f - n$) can be determined as an indicator of remaining structural capacity (AASHTO, 2008).

Table 2.7: Major Material Properties Considered in the MEPDG

Material Category	Material Inputs		
	Inputs for Critical Response (i.e. stresses and strains) Computations	Additional Inputs for Distress and Transfer Functions	Additional Input for Climatic Effects
Hot Mix Asphalt (HMA)	• Time-temperature dependent dynamic modulus (E*) of HMA mixture • Poisson ratio	• Tensile strength • Creep compliance • Coefficient of thermal expansion	• Surface shortwave absorptivity • Thermal conductivity • Heat capacity • Asphalt binder viscosity (stiffness)
Portland Cement Concrete	• Modulus of elasticity (E) • Poisson ratio • Unit weight • Coefficient of thermal expansion	• Modulus of rupture • Split tensile strength • Compressive strength • Cement content • Water-cement ratio • Shrinkage	• Surface shortwave absorptivity • Thermal conductivity • Heat capacity
Unbound base, subbase and subgrade materials	• Seasonally adjusted resilient modulus (M_r) • Poisson ratio • Unit weight • Coefficient of lateral pressure	• Gradation parameters	• Plasticity index • Gradation parameters • Effective grain sizes • Specific gravity • Hydraulic conductivity • Optimum moisture content

2.3.3 AASHTO Structural Indicators for Flexible and Rigid Pavements

For the past few decades, highway agencies have been using the structural indicators developed in the AASHO Road Test to determine the structural integrity of pavements. This includes the concepts of structural number for flexible pavements, effective slab thickness for rigid pavements (AASHTO, 1993). This section shall briefly discuss the use of these indicators to represent structural condition.

2.3.3.1 Effective Structural Number in Flexible Pavements

The effective structural number (SN) is an index value that combines layer thicknesses, structural layer coefficients, and drainage coefficients which is computed from the following equation:

$$SN = a_1 D_1 + a_2 D_2 m_2 + a_3 D_3 m_3 \qquad (2.2)$$

where D_1, D_2, D_3 represents the thickness of the surface, base and subbase layers respectively; a_1, a_2, a_3 represents the layer coefficients of the surface, base and subbase layers respectively; and m_2, m_3 represents the drainage coefficients of the base and subbase layers respectively.

When evaluating the structural condition of any existing pavement, the term "effective structural number" or SN_{eff} is used instead. The effective structural number of an existing pavement (SN_{eff}) may be determined from (a) results of non-destructive tests (NDT) (using a deflection-based procedure), (b) results of condition survey, or (c) remaining life analysis.

<u>NDT Deflections:</u> The determination of SN_{eff} from results of NDT is based on the assumption that the structural capacity of a pavement is a function of its total thickness and overall stiffness (i.e. pavement layer moduli). Given the deflections obtained from a falling weight deflectometer, the pavement layer moduli can be estimated by back-calculation using multilayer elastic theory. This area has been widely-researched upon in the past two decades (Uzan, 1994; Fwa and Chandrasegaram, 2001; Noureldin et al., 2005). The relationship between SN_{eff}, thickness and estimated stiffness from the back-calculation algorithms can then be described as follows (AASHTO, 1993):

$$SN_{eff} = 0.0045 D \sqrt[3]{E_p} \qquad (2.3)$$

where D is the total pavement thickness in inch and E_p is the pavement stiffness in psi.

<u>Condition Survey:</u> The method of determination of SN_{eff} from condition survey involves making an engineering judgment in assigning layer coefficients and drainage

coefficients to the various layers of the existing pavement, and calculating the SN_{eff} using the structural number equation shown in Equation (2.2). AASHTO (1993) provided a table (Table 2.8) with suggested layer coefficients for various pavement materials depending on the level of deterioration observed during visual inspections.

Table 2.8: Suggested Layer Coefficients for Existing AC Pavement Layer Materials

Material	Surface Condition	Coefficient
AC surface	Little or no alligator cracking and/or only low-severity transverse cracking	0.35 – 0.40
	<10% low-severity alligator cracking and/or <5% medium- and high-severity transverse cracking	0.25 – 0.35
	>10% low-severity alligator cracking and/or <10% medium-severity alligator cracking and/or >5 – 10% medium- and high-severity transverse cracking	0.20 – 0.30
	>10% medium-severity alligator cracking and/or <10% high-severity alligator cracking and/or >10% medium- and high-severity transverse cracking	0.14 – 0.20
	>10% high-severity alligator cracking and/or >10% high-severity transverse cracking	0.08 – 0.15
Stabilized base	Little or no alligator cracking and/or only low-severity transverse cracking	0.20 – 0.35
	<10% low-severity alligator cracking and/or <5% medium- and high-severity transverse cracking	0.15 – 0.25
	>10% low-severity alligator cracking and/or <10% medium-severity alligator cracking and/or >5-10% medium- and high-severity transverse cracking	0.15 – 0.20
	>10% medium-severity alligator cracking and/or < 10% high-severity alligator cracking and/or >10% medium- and high-severity transverse cracking	0.10 – 0.20
	>10% high-severity alligator cracking and/or >10% high-severity transverse cracking	0.08 – 0.15
Granular base or subbase	No evidence of pumping, degradation, or contamination by fines	0.10 – 0.14
	Some evidence of pumping, degradation, or contamination by fines	0.00 – 0.10

Source: AASHTO, 1993

<u>Remaining Life Analysis:</u> The determination of SN_{eff} from remaining life analysis is based on the fatigue damage concept that the structural capacity of a pavement diminishes gradually as the pavement is subjected to increasing number of traffic loads. The remaining life of a pavement, as a percentage of its design life can be represented by the following equation:

$$R_L = 100\left(1 - \frac{N_p}{N_{1.5}}\right) \tag{2.4}$$

To calculate R_L, the total amount of traffic the pavement has carried to date (N_p) and the total amount of traffic the pavement could be expected to carry to a terminal serviceability index (PSI) of 1.5 ($N_{1.5}$) need to be determined. $N_{1.5}$ can be estimated using the AASHTO pavement design nomograph shown in Figure 2.5 (AASHTO, 1996) and using a terminal PSI of 1.5 and a reliability of 50%. Given R_L, the effective structural number SN_{eff} can then be determined:

$$SN_{eff} = CF \times SN_0 \tag{2.5}$$

where CF is the condition factor, which is a function of R_L (shown in Figure 2.6) and SN_0 is the structural number of the new pavement.

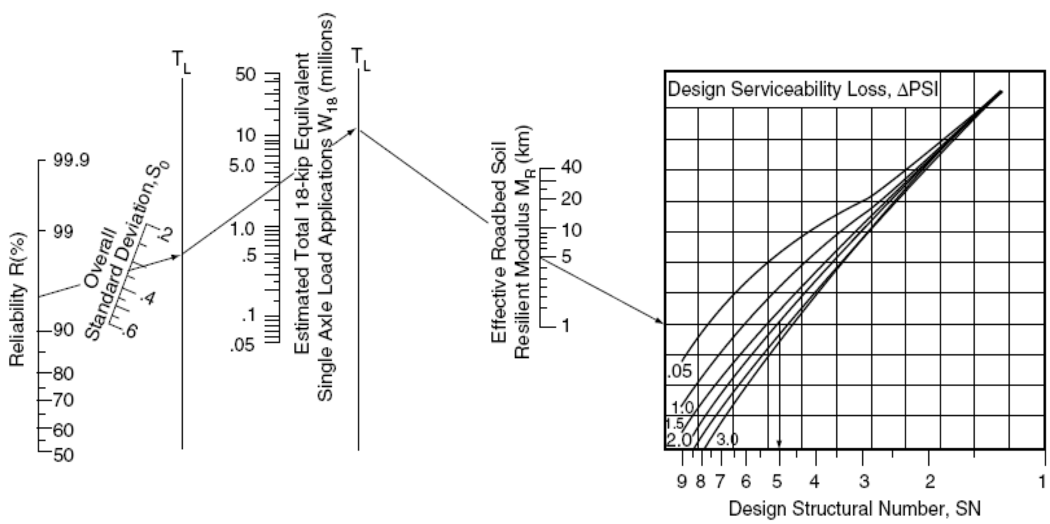


Figure 2.5: Design Chart for Flexible Pavements using Mean Values for each Input
(Source: AASHTO, 1993)

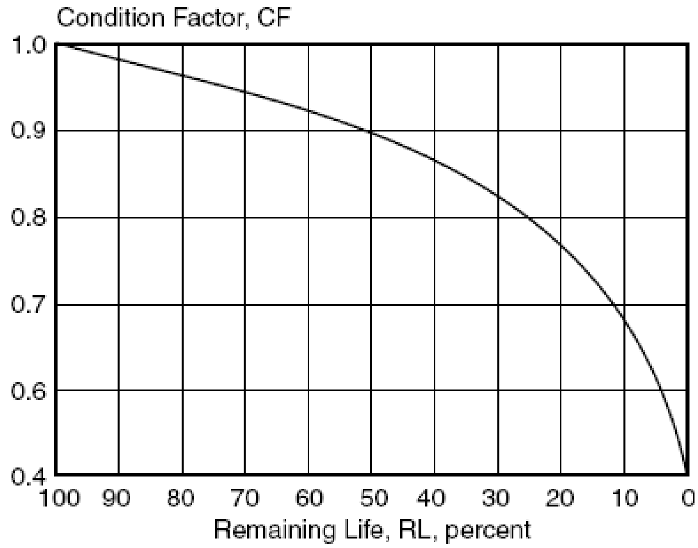


Figure 2.6: Relationship between Condition Factor and Remaining Life *(Source: AASHTO, 1993)*

2.3.3.2 Effective Slab Thickness for Rigid Pavements

For rigid pavements, the concept of effective thickness of the slab is used. This concept is somewhat analogous to the structural number used for flexible pavements and is illustrated in the AASHTO design nomograph (Figure 2.7) where the slab thickness D can be determined given the effective modulus of subgrade reaction *k*, concrete elastic modulus E_c, concrete modulus of rupture S_c, joint load transfer, drainage of base and traffic loading.

The effective slab thickness of an existing rigid pavement (D_{eff}) can be determined from (a) results of condition survey, or (b) remaining life analysis.

<u>Condition Survey:</u> Based on the condition of the existing slab, its effective thickness is computed as:

$$D_{eff} = F_{jc} F_{dur} F_{fat} D \tag{2.7}$$

where D is the thickness of the existing slab, F_{jc} is the joints and cracks adjustment factors, F_{dur} is the durability adjustment factor, and F_{fat} is the fatigue adjustment factor. When there are no deteriorated transverse joints or cracks, or if all such defects are effectively repaired, F_{jc} can be taken as 1.00. Otherwise, F_{jc} can be assigned according to a practically linear equation joining the points of F_{jc} = 1.00 for zero deteriorated transverse joints and cracks, and F_{jc} = 0.56 for 200 such joints and cracks per mile. F_{dur} has the value of 1.00 if there are no signs of durability problems, 0.96 to 0.99 if there is some durability cracking but no spalling, and 0.88 to 0.95 if both cracking and spalling exist. F_{fat} has a value of 0.97 to 1.00 if very few transverse cracks and punchouts exist, 0.94 to 0.96 if a significant number of transverse cracks and punch-outs exist, and 0.90 to 0.93 if a large number of transverse cracks and punch-outs exist.

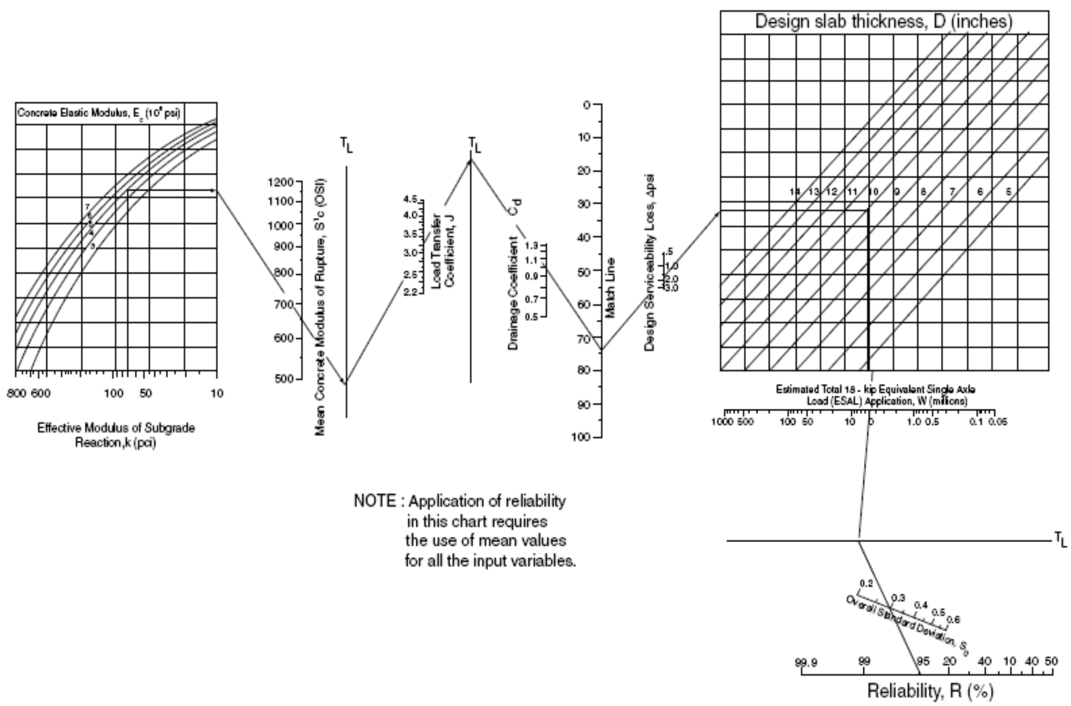


Figure 2.7: Rigid Pavement Thickness Design Chart *(Source: AASHTO, 1993)*

<u>Remaining Life Analysis:</u> Based on the percent remaining life of the existing pavement, its effective thickness can be estimated by the following equation:

$$D_{eff} = C_F D \tag{2.8}$$

where D is the thickness of the parent slab, and C_F the condition factor determined from Figure 2.6. To determine the condition factor C_F; the remaining life of the parent pavement must first be computed using Equation (2.6). $N_{1.5}$ can be determined using the AASHTO thickness design nomograph for rigid pavements shown in Figure 2.7

(AASHTO, 1993). An implicit assumption of this approach is that the pavement has been constructed and maintained in accordance with the original design without durability or distress problem.

2.3.4 Structural Capacity or Condition Indices

It is often desirable to express pavement structural condition (or adequacy) through an index that can be easily understood by non-technical people or managers unlike technical terms like material strength, deflections or structural numbers. It is also preferable that the index is bounded on a 0 to 5 or 0 to 10 scale so that is of a form compatible to pavement serviceability index PSI (0 to 5 scale) and pavement condition index or rating (PCI or PCR) (0 to 10 or 0 to 100 scale). The index has to allow pavement engineers to answer the questions:

- What is the maximum load the pavement can withstand without causing excess immediate distress, given the value of the index?
- How many more load repetitions can the pavement withstand, given the value of the index?

The first question is applicable when highway engineers are faced with the issue of posting load limits during spring thaw periods or with issuing permits for trucks with load exceeding legal weight limits. Here, the engineer is concerned about a single or a relatively limited number of load applications causing excessive damage to the pavement structure. The second question is a situation of repeated load applications where engineers need to use this information to estimate the remaining life of the pavement. Therefore the index must be simple to understand to non-engineers. Currently, there is no widely accepted method for developing a universally accepted condition index. This section shall discuss some of the structural capacity or condition indices that have been developed from prior research.

2.3.4.1 Structural Adequacy Index

Most early methods to develop a structural adequacy index are based on comparing the representative rebound deflection (RRD) and the design rebound deflection (DRD) on flexible pavements (Haas et al., 1994). Similar approach is also adopted by Scullion (1988). The representative rebound deflection (RRD) is evaluated from temperature-adjusted Benkelman beam rebound deflection measurements at every 50 to 100 m for a given pavement section can be defined as:

$$RRD = \bar{x} + 2\sigma \qquad (2.9)$$

where $\bar{x}$ is the mean of the temperature-adjusted deflections in the pavement section and σ is their standard deviations. The design rebound deflection (DRD) is a set of deflection limits developed by the Asphalt Institute (1981) and is a function of the traffic level anticipated over the future life of the pavement section (expressed in terms of ESALs). Based on these two deflections, Haas et al. (1994) developed the structural adequacy index (SAI) defined by Equation (2.10).

$$SAI = 5 - (Density) \qquad (2.10)$$

The density values are evaluated from Table 2.9 using for a given set of traffic level and RRD. The beauty of the SAI is that the index is of a bounded scale and is compatible with other indicators such as the PSI or PCR when it comes to network level pavement management applications. However, the SAI is only applicable for flexible pavements and not for rigid pavements.

Noureldin et al. (2005) proposed a structural condition indicator based on ESALs and the center deflection under the center sensor. This indicator uses the subjective terms of “excellent”, “very good”, “good”, “fair”, and “poor” to describe the pavement structural condition. Table 2.10 illustrates how the structural condition indicator can be determined using deflection measurements and cumulative ESAL loading. While simple to understand by non-engineers, the indicator suffers from several problems. First, the indicator shown in Table 2.10 is valid for all pavement types. However, asphalt and concrete pavements behave fundamentally different and testing procedures are also different. Second, the indicator used is a categorical variable. Unlike ordinal variables where the numerical value of the index clearly indicates the strength of the pavement, the categorical indicator lacks the differentiation within a level or across levels. For example, a deflection of 4 mils is considered “very good” but a deflection of 3.9 mils is considered “excellent” (even thought the difference is only 0.1 mil. This difference if applied in conventional pavement management system would lead to inefficient allocation of resources and pavement management strategies.

Table 2.9: Density Values for Evaluating Structural Adequacy Index in Flexible Pavements

RRD – DRD (mils)	Percentage of Individual Deflection Observation > DRD								
	< 30%			30% - 60%			> 60%		
	Traffic Level								
	Low	Medium	High	Low	Medium	High	Low	Medium	High
0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
0.1	0.0	0.3	0.3	0.3	0.3	0.5	0.3	0.5	0.8
0.2	0.3	0.5	0.8	0.5	0.8	1.0	0.8	1.0	1.5
0.3	0.5	0.8	1.5	0.8	1.0	2.0	1.3	1.5	2.5
0.4	0.8	1.0	2.0	1.3	1.5	2.5	1.5	2.0	3.0
0.5	1.0	1.5	2.5	1.5	2.0	2.8	2.0	2.5	3.3
0.6	1.3	2.0	3.0	2.0	2.8	3.3	2.5	3.0	3.8
0.7	1.5	2.8	3.3	2.0	3.0	3.5	2.8	3.3	4.0
0.8	1.8	2.8	3.5	2.3	3.3	4.0	2.8	3.5	4.3
0.9	2.0	3.0	3.8	2.5	3.5	4.3	3.0	3.8	4.5
1.0	2.0	3.0	4.3	2.5	3.8	4.5	3.0	4.0	4.8
1.1	2.3	3.3	4.5	2.8	3.8	4.5	3.3	4.3	4.8
1.2	2.3	3.3	4.5	2.8	4.0	4.8	3.3	4.3	5.0
1.3	2.5	3.3	4.5	3.0	4.0	4.8	3.3	4.5	5.0
1.4	2.5	3.5	4.8	3.0	4.3	5.0	3.5	4.5	5.0
1.5	2.5	3.5	4.8	3.0	4.3	5.0	3.5	4.8	5.0
1.6	2.8	3.8	4.8	3.3	4.5	5.0	3.8	4.8	5.0
1.7	2.8	3.8	5.0	3.3	4.5	5.0	3.8	4.8	5.0
1.8	2.8	4.0	5.0	3.5	4.8	5.0	3.8	5.0	5.0
1.9	3.0	4.0	5.0	3.5	4.8	5.0	4.0	5.0	5.0
2.0	3.0	4.5	5.0	3.8	5.0	5.0	4.0	5.0	5.0
2.1	3.0	4.5	5.0	3.8	5.0	5.0	4.0	5.0	5.0
2.2	3.3	4.8	5.0	3.8	5.0	5.0	4.3	5.0	5.0
2.3	3.3	4.8	5.0	4.0	5.0	5.0	4.3	5.0	5.0
2.4	3.3	5.0	5.0	4.0	5.0	5.0	4.3	5.0	5.0
2.5	3.5	5.0	5.0	4.3	5.0	5.0	4.5	5.0	5.0
2.6	3.5	5.0	5.0	4.3	5.0	5.0	4.5	5.0	5.0
2.7	3.8	5.0	5.0	4.3	5.0	5.0	4.5	5.0	5.0

Source: Haas et al., 1997

Table 2.10: Structural Condition Table Using Deflection Measurements for Pavement

Structural Condition Indicator	Center deflections (mils) under Different Cumulative ESAL Level (millions)					
	> 30	10 – 30	3 – 10	1 – 3	0.3 – 1	< 0.3
Excellent	< 4	< 5	< 6	< 8	< 10	< 12
Very Good	4 – 6	5 – 7	6 – 8	8 – 10	10 – 12	12 – 14
Good	6 – 8	7 – 9	8 – 10	10 – 12	12 – 14	14 – 16
Fair	8 – 10	9 – 11	10 – 12	12 – 14	14 – 16	16 – 18
Poor	> 10	> 11	> 12	> 14	> 16	> 18

Source: Noureldin et al., 2005.

2.3.4.2 Structural Condition Index using Structural Number

Zhang et al. (2003) attempted to develop a structural condition index (SCI) based on the FWD deflection readings of a pavement and the total thickness of the pavement layers. Deflection data from the falling weight deflectometer (FWD) was routinely collected and the elastic moduli of the pavement layers can be evaluated using back-calculation algorithms. Using the structural number concept as described in the earlier sections, they developed the following structural condition index:

$$SCI = \frac{SN_{eff}}{SN_0} \tag{2.11}$$

where SN_{eff} is the effective structural number and SN_0 is the required structural number (i.e. that of a newly-constructed pavement). This index can be interpreted as the ratio of the existing structural capacity of the pavement to the structural capacity of a new pavement. While this method is similar to that adopted by the AASHTO 1993 pavement design guide for the design of overlays, it is only valid for flexible pavements, due to the use of structural number concept.

Other researchers (Ali and Tayabji, 1998; Chakroborty et al., 2006) have also adopted similar approach in the development of structural condition indices and they suffer the same flaw as the research performed by Zhang et al. (2003).

2.4 Issues with Current State-of-the-Art and Need for Research

This chapter has discussed the issues associated with collecting pavement structural condition data and evaluating the structural condition in the highway network. The task of collecting pavement structural data and performing structural capacity analysis is both costly and time-consuming and most highway agencies lack a comprehensive network level database detailing the structural condition of pavements. Consequently, these agencies tend to consider only functional performance (i.e. surface distress and roughness) in making network level decisions (e.g. assessment of overall network condition, production of a prioritized work program, scheduling of activities, and the allocation of funds).

There is therefore a need to develop an indicator that allows highway agencies to estimate the structural condition of the highway network annually or at least in the event where there is no recent data on structural condition. Structural indicators discussed in this chapter are not adequate to meet the requirements for network level pavement management where decisions have to be made on a monthly or annual basis. For example, data collected from destructive testing are of a project level scale and field tests and experiments are costly to perform throughout a network on a frequent basis. Surface distresses, while useful in network level pavement management, is not a strong indicator of structural strength or capacity. This leaves us with the option of nondestructive testing which is in fact a promising approach for developing a structural indicator suitable for network level pavement management.

Although various forms of structural condition indicators were developed in the past, they were either restrictive (i.e. only valid for either flexible or rigid pavements), or were unbounded on a scale (e.g. structural number or effective thickness). For example, structural condition indicators, based on the structural number concept or structural adequacy indices, require either the FWD deflections or back-calculation of elastic moduli on FWD data. However, FWD tests are generally performed on the network level on a frequency of three to five years and the information cannot accommodate the need for annual evaluation. Also, structural condition indicators such as those based on

Miner's hypothesis, typically require some form of correlation to surface distresses, where automated surface distress identification techniques are still in their infancy and the issue of data quality is still unresolved (McGhee, 2004). Besides the need for the structural condition (or strength) indicator to represent the structural performance of the entire network, it also has to be bounded (on a five-point or ten-point scale). This is necessary for ready interpretation for highway managers as well as the general public.

The present research aims to develop a structural strength index that can be readily applied in network level pavement management without the difficulties associated with the current procedures. A set of models shall be developed to allow the prediction of the structural strength index in the absence of detailed structural condition information. The next chapter shall discuss the scope of work performed and the methodology adopted in this research study.

CHAPTER 3. STUDY FRAMEWORK

The need to have a structural condition indicator that is compatible with other functional indicators (such as the pavement condition rating and pavement serviceability index) is well recognized. This chapter explains the framework and methodology adopted to develop such an indicator to enable pavement engineers and managers to estimate pavement strength and to assess pavement structural quality at the network level.

3.1 Overview of Study Framework

Figure 3.1 shows the overall framework of the study. The main objective of this study is to develop a structural strength index (SSI) that can provide engineers and managers an indication of the pavement structural condition and allows simple application within the network level pavement management system. In order to develop the structural strength index, deflections from the routine falling weight deflectometer (FWD) testing on the highway network are utilized. This is because most highway agencies collect FWD deflection data on a routine basis at the network level. Noting that the frequency of these tests ranges between three and five years, the study proposes to develop deflection prediction models that allow managers to predict the expected FWD deflections at any given point in time. This is geared towards the development of a complete structural information database that agencies can use when planning for maintenance, rehabilitation and reconstruction (M, R & R) activities. Given the structural strength index, the study will demonstrate its use in network level pavement management.

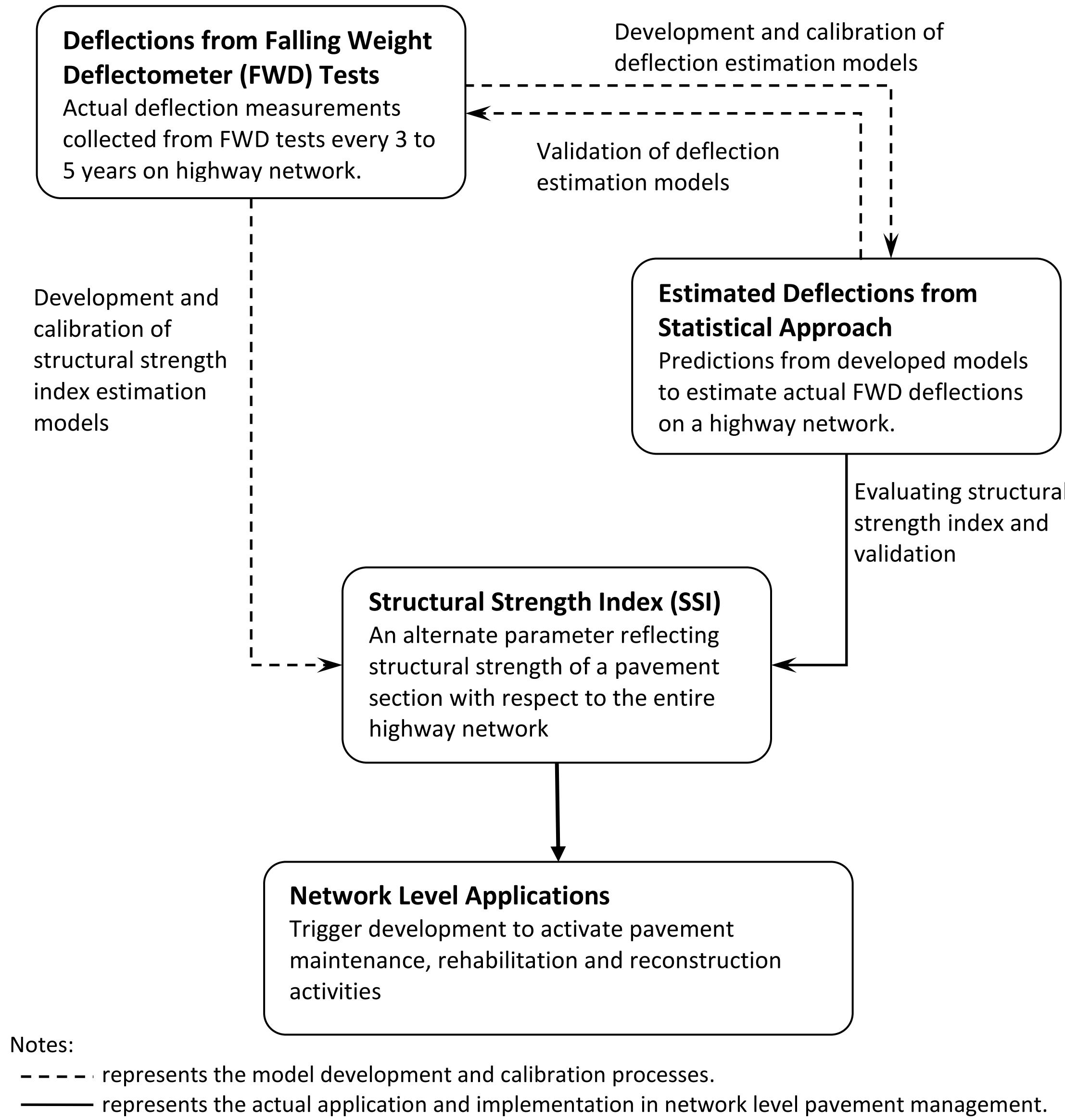


Figure 3.1: Framework of Proposed Study

3.2 Study Approach

3.2.1 Development of the Structural Strength Index as a Measure of Pavement Structural Condition

The study shall first set out to develop a structural strength index using a statistical approach. Previous research had noted that the choice of structural condition indicator is dependent on pavement type, pavement materials and thickness design (Haas et al., 1994; AASHTO, 1993; Zhang et al., 2003; Chakroborty et al., 2006; AASHTO, 2008). The present study shall follow these basic principles when developing an alternate structural strength index to measure pavement structural condition. Figure 3.2 shows the basic concept adopted in this research. As shown in Figure 3.2, it is noted that deflection values from the FWD tests will be one of the primary inputs used to determine the structural strength index. This will allow agencies to evaluate structural strength in an almost real-time fashion when conducting their routine FWD tests on the highway network. In general, the structural strength index (SSI) can be expressed mathematically as:

$$SSI = f\left(\delta_i\right) \tag{3.1}$$

given that

$$\delta_i = g\left(x_i\right) \tag{3.1a}$$

$$SSI \in \left[0,100\right] \tag{3.1b}$$

This study sets to develop SSI models based on δ_i, which is the deflection at the ith sensor of a typical FWD test (Figure 3.2). It is noted that δ_i is a function of pavement type, material properties, pavement layer or slab thicknesses design (i.e. load transfer mechanisms), cumulative traffic loading, and cumulative weather effects. Therefore, Equation (3.1a) has to be considered when developing a structural strength indicator where x_i represents the different variables that can affect deflection δ_i. Furthermore, it is essential for SSI to be bounded so that it is compatible with other parameters such as the pavement serviceability rating (PSR) and the pavement condition rating (PCR) when determining pavement M, R & R activities at the network level. In this study, we consider that SSI is bounded between 0 and 100, as shown in Equation (3.1b).

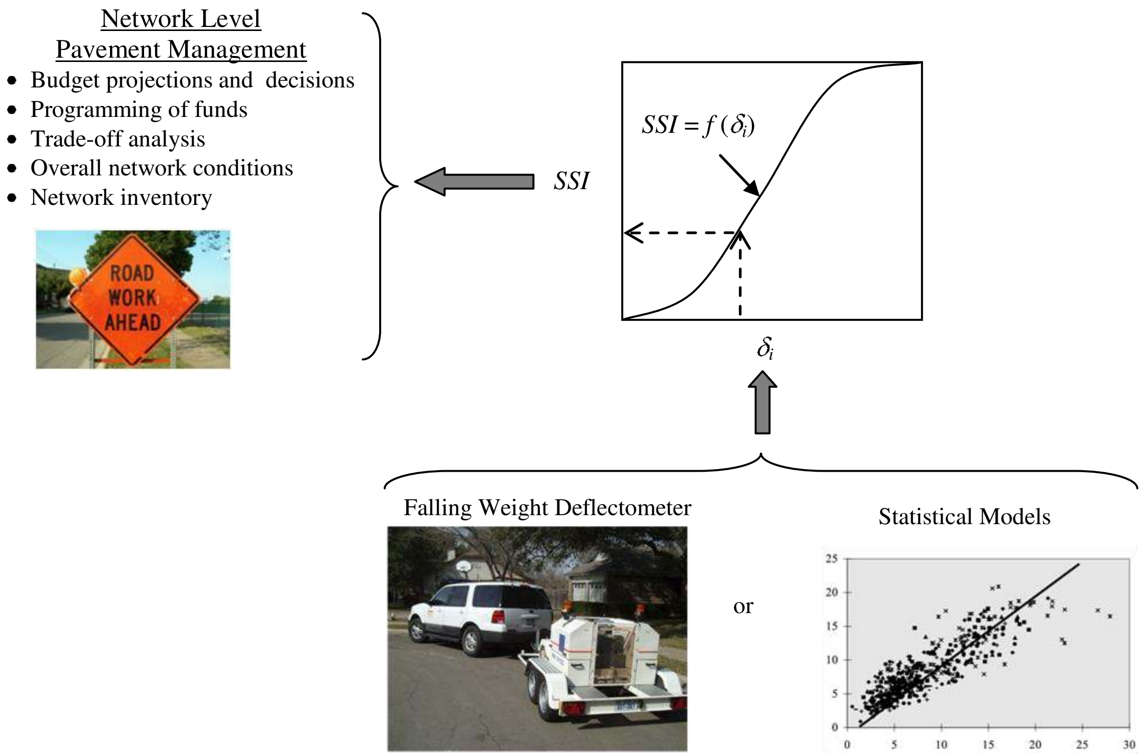


Figure 3.2: Basic Concept Used in the Research

3.2.2 Development of Statistical Deflection Prediction Models

Because most highway agencies perform network level deflection-testing every three to five years and network-level pavement management decisions are made at least on an annual basis, the current network level structural strength evaluation practice is insufficient to allow a comprehensive management of our highway networks. This study therefore proposes the use of statistical deflection prediction models in the form of Equation (3.2):

$$\delta_i = g \quad x_i \tag{3.2}$$

where x_i are variables indicating the cumulative traffic loading, cumulative weather effects, pavement type, material properties and pavement layer or slab thicknesses. Note that in this case, in order to differentiate from the actual FWD deflection δ_i, we use the notation δ_i to represent the estimated FWD deflection from the statistical model. The estimated deflections δ_i are validated against actual deflections δ_i obtained from the network level FWD tests.

Given the estimated deflection from Equation (3.1a), the structural strength index SSI can then be estimated using Equation (3.3), as shown in Figure 3.2.

$$SSI = f \quad \delta_i \tag{3.3}$$

and

$$SSI \in \; 0,100 \tag{3.3a}$$

The structural strength index can be used for network level pavement management decisions.

CHAPTER 4. DATA DESCRIPTION

This chapter describes the data used in the study. Different data categories required for the study (pavement functional condition, pavement structural condition, roadway inventory, traffic and weather data) are discussed in terms of the type of data collected and how the data is collected.

4.1 Pavement Functional Condition Data

Functional condition data consists of three items: International Roughness Index (IRI), rutting (for flexible pavements) and Pavement Condition Rating (PCR) data. IRI and PCR are used for both flexible and rigid pavement while rutting only applies to flexible pavements. The IRI and rut data is collected annually on state-maintained highways by a vendor using a data collection van (as shown in Figure 4.1). The van is equipped with a laser profiler and 5-point rut bar that collects the IRI and rut depth of the pavement surface respectively. In addition, the van is equipped with three pan-tilt-zoom video cameras that record a panoramic view of the road (for identification of roadway inventory) and two video imaging cameras that record a view of the pavement surface.

4.1.1 International Roughness Index

International Roughness Index (IRI) is a measure of the roughness of the road. Roughness is the distortion or “bumpiness” of the pavement surface that causes a vehicle operator or rider to experience an uncomfortable ride. Since roughness is caused by the longitudinal distortion of the pavement surface, the way to measure it is to measure the longitudinal profile of the wheel path. The most common index used is the international roughness index (IRI). It measures the "bumpiness" of the pavement in terms of inches per mile, i.e., the higher the number the rougher the ride. IRI data is collected according to the AASHTO PP 37-04 standard on “Determination of International Roughness Index (IRI) to Quantify Roughness of Pavements” (AASHTO, 2004). In this study, IRI data is collected continuously along a highway but the values are reported every 1-mile or 0.25 mile. Excellent pavements are found to have IRI in the 0 to 80 range, good pavements are in the 80 to 115 range, fair pavements are in the 115 to 150 range, and poor pavements are over 150 (INDOT, 1997).

Figure 4.1: Illustration of a Data Collection Van

4.1.2 Rutting on Asphalt Pavements

Rutting is transverse deformation along pavement wheel paths of a pavement. It is measured using five lasers mounted on a rut bar on the data collection van, as shown in Figure 4.1. Two lasers are mounted at the edge of the pavement, two lasers are mounted over the wheel paths and one is mounted over the center of the lane as shown in Figure 4.2. The rut is estimated by determining the difference in height between the wheel path and the level established by averaging heights at the edge of pavement and the center of the lane and is done for both the right and left wheel paths. Rut data is collected using the AASHTO PP 38-00 standard on "Determining Maximum Rut Depth in Asphalt Pavements" (AASHTO, 2003). In this study, rut depths are collected continuously along a highway but are reported in every 1-mile or 0.25 mile. A severely rutted pavement is one which has an average rut larger than 0.25 inch (INDOT, 1997).

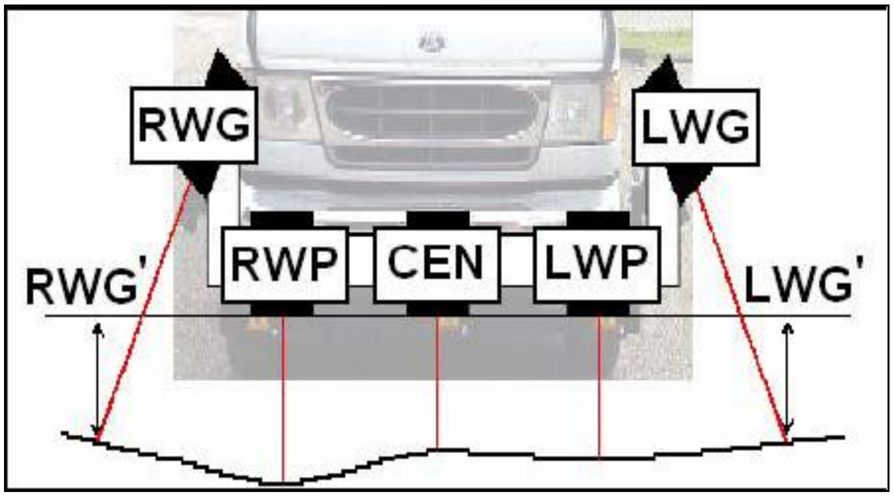


Figure 4.2: Illustration of a Five-Point Rut Bar to Measure Rut Depth

4.1.3 Pavement Condition Rating

Pavement condition rating (PCR) is a measure of the pavement surface distress and is measured by conducting automated pavement condition surveys. The distresses measured generally include such factors as surface defects, cracking and patching. These surveys are conducted by viewing the distresses such as transverse cracking, longitudinal cracking, blocking cracking on the road for a distance of 500 ft at the beginning of each mile and rating each distress for severity and extent. It is assumed that the first 500 ft of each mile is representative of the entire mile. The severity and extent are combined together to determine deduct points for each distress present on the road segment. The weighted values of the deduct points are subtracted from 100 to determine the PCR. The rating goes from 100 to 0 with excellent pavements in the 100 to 90 range, good pavements in the 90 to 80 range, fair pavements in the 80 to 70 range, and poor pavements below 70. (INDOT, 1997). Table 4.1 summarizes the pavement functional condition data used in the study discussed thus far.

Table 4.1: Pavement Functional Condition Data used in the Study

Data Type	Description	Method of Data Collection
International Roughness Index (IRI)	Measures the ride of the pavement surface in inches per mile, Used for flexible or rigid pavement	Measured continuously by laser sensors, aggregated by mile and by tenth of a mile
Rut Depth (for flexible pavements)	Measures the transverse deformation of the pavement surface in inches. Used for flexible pavement only	
Pavement Condition Rating (PCR)	Measures the level of distress on the pavement surface 0-100 scale with 100 being excellent and 0 being very poor. Used for flexible or rigid pavement	Measured on the first 500' at each reference post using people rating the pavement from video. PCR is based on the distresses rated by viewers and summated. The distresses include but are not limited to transverse, block, and fatigue cracking

4.2 Pavement Structural Condition

While there are several different ways to estimate the structural condition of the pavement, this study will use deflections from FWD testing as an estimator of pavement structural condition. Both network and project-level data are used in this study. The difference between project and network level deflection testing lies in the testing needs and frequency, and also the testing procedures. Network-level deflection testing involves deflection testing of the entire highway network every three to five years. Typically, three test sites are sampled in every mile and a standard 9000-lb drop-weight is used, as shown in Figure 4.3 (Noureldin et al., 2005). Project-level deflection testing is generally performed on individual project sites according to the specific needs of the project. For the present study, three test locations are selected within the project site and three different drop heights are used, according to ASTM standards (ASTM, 2000). Data collected from the FWD tests includes the weight (lbs) and pressure (psi) of the drop on the pavement, the deflection (mils) at various points along the FWD and the air and pavement surface temperature (°F), as shown in Table 4.2.

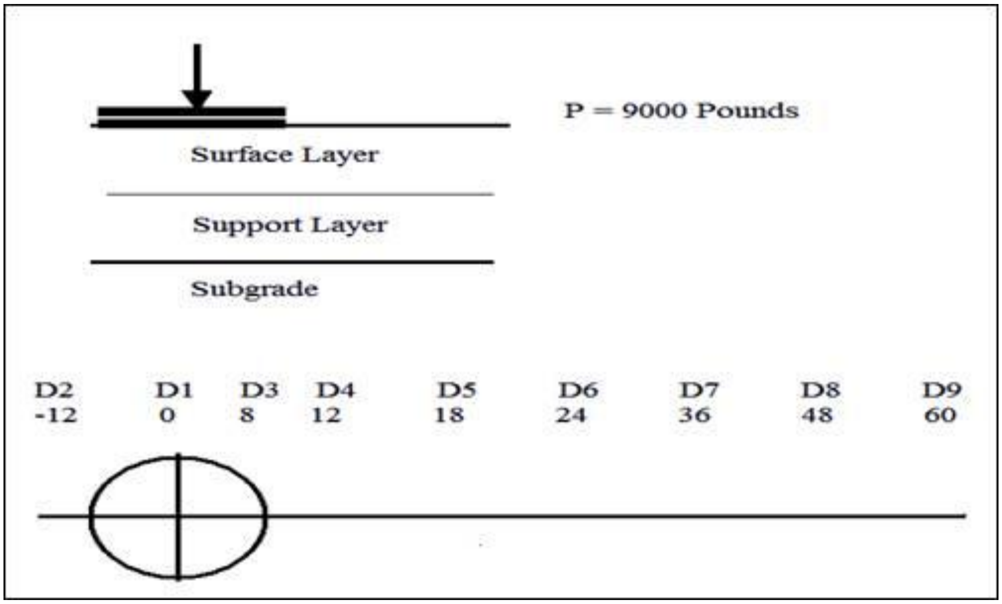


Figure 4.3: Schematic diagram of the FWD (Noureldin et al., 2005)

Table 4.2: Pavement Structural Condition Data used in Study

Data Type	Description
Drop#	1, 2, or 3 (each drop represent a weight used for the testing)
Pressure(psi)	Pressure on the pavement from drop, (Based on the size of the plate that sits on the pavement on which the weight is applied)
Load(lbs)	Load of the drop (9000 lbs for this study)
D1(mils)	Sensor 1 measurement of the deflection (located at the center of the plate)
D2(mils)	Sensor 2 measurement of the deflection (located 12“ from center of plate towards front of the FWD device)
D3(mils)	Sensor 3 measurement of the deflection (located 8“ from center of plate)
D4(mils)	Sensor 4 measurement of the deflection (located 12“ from center of plate)
D5(mils)	Sensor 5 measurement of the deflection (located 18“ from center of plate)
D6(mils)	Sensor 6 measurement of the deflection (located 24“ from center of)
D7(mils)	Sensor 7 measurement of the deflection (located 36 “ from center of plate)
D8(mils)	Sensor 8 measurement of the deflection (located 48“ from center of plate)
D9(mils)	Sensor 9 measurement of the deflection (located 60“ from center of plate)
Mean Temp(°F)	Average of the Air Temp(°F) and Surface Temp(°F)
Air Temp(°F)	Ambient air temperature
Surface Temp(°F)	Pavement Surface Temperature

The center deflection (D_1 in Figure 4.3) was found to be an excellent indicator of the overall strength of a section of pavement (Noureldin et al., 2005). A stronger pavement section is indicated by a lower deflection and a weaker pavement section is indicated by a higher deflection, for a given pavement type and pavement structure. The center deflection is corrected for temperature using the air and pavement surface temperature using Equation (4.1):

$$D_{1\ corrected} = \alpha \ D_1 \tag{4.1}$$

where $(D_1)_{corrected}$ is the center deflection corrected for temperature effects, D_1 is the actual center deflection measured from the FWD test and α is the correction factor defined in Table 4.3.

Table 4.3: Center Deflection Temperature Correction Factors

Mean Pavement Temperature, °F	41	50	59	68	77	86	95	104	113	122
Temperature Correction Factor	0.74	0.81	0.90	1.00	1.11	1.22	1.34	1.46	1.59	1.72

4.3 Road Inventory, Pavement Structure and Pavement Work Data

Table 4.4 summarizes the different road inventory, pavement structure and pavement work data used in the study. In this study, roadway inventory data was extracted from the INDOT Pavement Management System. Roadway information was collected by visual observation of the roadway while driving the network and is continually updated using aerial mapping and visual observation. Information on the roadway includes the route number and location of the road, the system that the road is on, and if the road is rural or urban. The system indicates if the road section is an interstate, a national highway system (NHS), or state route.

Pavement work data includes information on when and the type of pavement work performed on a road. It includes restorative work such as resurfacing and major work such as rehabilitation, reconstruction, or replacement of the pavement. This information is updated maintained by listing each pavement contract in the inventory in its correct location on the road. The information includes the type of pavement work performed, the year of the work, and the type of pavement surface.

Pavement structure data describes the type of the material under the surface. It is classified as rigid, flexible, composite, and un-designed. An un-designed pavement is an older pavement that was not designed to modern standards. These pavements were laid in the 1920s through the 1930s and have been overlaid over the ensuing years. The roads that have un-designed pavement are minor roads that have low AADT and are not considered in this study.

Table 4.4: Road Inventory, Pavement Structure and Pavement Work Data used in the Study

Data Type	Description
Road	Name of the road (i.e. S_46, I_65, U_40).
Reference Post	The reference post nearest the location and the distance to the post (in miles) (i.e. RP_003 + 0.39).
System	Is the road on the interstate (I), non-interstate national highway system (N), or non-national highway system (O)?
Year of Last Work	The year that restorative pavement work was performed at the location?
Type of Last Work	What restorative pavement work was performed?
Pavement Surface	Is the pavement section concrete (J) or asphalt (A)?
Pavement Structure	Is the pavement structure under the surface of the pavement rigid (R), flexible (F), composite (C) (Composite pavement consists of asphalt laid over a concrete pavement) or undersigned (U) (Undesigned pavement refers to a section in which the pavement was laid without any design.)?

4.4 Traffic Data

Traffic data consists of annual average daily traffic (AADT) and percent truck traffic. This data is collected and processed by INDOT based on guidelines provided by the FHWA in its Traffic Monitoring Guideline (FHWA, 2001). The traffic data is collected on a three year cycle by dividing the state maintained road network into thirds and collecting one third of the traffic data every year using coverage and classification counts on homogenous road segments. Table 4.5 summarizes the type of traffic data collected in this study.

Table 4.5: Traffic Data used in the Study

Data Type	Description

Data Type	Description
AADT	Annual average daily traffic on the road
Percent Truck Traffic	Percent commercial truck traffic on the pavement section

4.5 Subgrade Data

Subgrade data describes the existing soil condition under the pavement structure. This data was obtained using the INDOT's GIS database and database from the Natural Resources Conservation Service of the US Department of Agriculture. The soil data consists of the type and soil properties, as shown in Table 4.6. This study considers the drainage capability of the subgrade underneath the pavement structure as it is known that a poorly-drained subgrade will result in premature pavement structural failure.

Table 4.6: Subgrade Data used in the Study

Item (Variable)	Description
MUKEY	Key identifier used for mapping soil types
MUNAME	Name of soil type
DRCLASSDCD	Dominant condition drainage class - natural drainage condition of the soil refers to the frequency and duration of wet periods.
DRCLASSWETTEST	Wettest condition drainage class - natural drainage condition of the soil refers to the frequency and duration of wet periods.

Natural drainage class can be used to describe the drainage condition of the subgrade and is referred to as the frequency and duration of wet periods of the soil under conditions similar to those under which the soil developed. The natural drainage classes can be inferred from observations of landscape position and soil morphology. In many soils, the depth and duration of wetness is related to the quantity, nature, and pattern of soil mottling. Correlation of drainage classes and soil mottling are made through field observations of water tables, soil wetness, and landscape position (Natural Resources Conservation Service, 2007). The classes are defined as follow:

Excessively drained subgrade: Water is removed very rapidly from the subgrade. The occurrence of internal free water commonly is very rare or very deep. The subgrade is commonly coarse-textured and has a very high hydraulic conductivity.

Somewhat excessively drained subgrade: Water is removed from the subgrade rapidly. Internal free water occurrence commonly is very rare or very deep. The subgrade is commonly coarse-textured and has high saturated hydraulic conductivity.

Well drained subgrade: Water is removed from the subgrade readily but not rapidly. Internal free water occurrence commonly is deep or very deep; annual duration is not specified.

Moderately well drained subgrade: Water is removed from the subgrade somewhat slowly during some periods of the year. Internal free water occurrence commonly is moderately deep and transitory through permanent. The subgrade have a moderately low or lower saturated hydraulic conductivity in a layer within the upper 1 m, periodically receive high rainfall, or both.

Somewhat poorly drained subgrade: Water is removed slowly so that the subgrade is wet at a shallow depth for significant periods in a year. The occurrence of internal free water commonly is shallow to moderately deep and transitory to permanent. The soils commonly have one or more of the following characteristics: low or very low saturated hydraulic conductivity, a high water table, additional water from seepage, or nearly continuous rainfall.

Poorly drained subgrade: Water is removed so slowly that the subgrade is wet at shallow depths periodically or remains wet for long periods. The occurrence of internal free water is shallow or very shallow and common or persistent. Free water at shallow depth is usually present. This water table is commonly the result of low or very low saturated hydraulic conductivity of nearly continuous rainfall, or of a combination of these.

Very poorly drained subgrade: Water is removed from the subgrade so slowly that free water remains at or very near the ground surface during much of the year. The occurrence of internal free water is very shallow and persistent or permanent. The subgrade is commonly level or depressed and frequently ponded. If rainfall is high or nearly continuous, slope gradients may be greater (Soil Survey Division Staff, 1993).

4.6 Climate Data

Climate data consists of temperature and precipitation data obtained from the Indiana Climate Office at Purdue University. Daily rainfall and temperature are collected at various weather stations across Indiana and is summarized to monthly averages temperatures and monthly total rainfalls for the different weather regions shown in Figure 4.4 and Table 4.7. There are nine weather regions in Indiana that are defined by the Indiana Climate Office and divide state into geographical regions based on adjacent counties as shown in Table 4.8.

Figure 4.4: Map of Weather Regions in Indiana

Table 4.7: Climate Data Considered in this Study

Item (Variable)	Description
Weather Region	Weather region as defined by NOAA of pavement section
Average High Temperature	Average monthly high temperature of the weather region
Average Low Temperature	Average monthly low temperature of the weather region

Item (Variable)	Description
Average Temperature	Average monthly temperature of the weather region
Precipitation	Monthly precipitation of the weather region

Table 4.8: Indiana Counties in each Weather Region

Climate Division	Counties
1 (Northwest)	Benton, Jasper, La Porte, Lake, Newton, Porter, Pulaski, Starke, White
2 (North central)	Carroll, Cass, Elkhart, Fulton, Kosclusko, Marshall, Miami, St. Joseph, Wabash
3 (Northeast)	Adams, Allen, De Kalb, Huntington, Lagrange, Noble, Steuben, Wells, Whitley
4 (West central)	Clay, Fountain, Montgomery, Owen, Parke, Putnam, Tippecanoe, Vermillion, Vigo, Warren
5 (Central)	Boone, Clinton, Grant, Hamilton, Hancock, Hendricks, Howard, Johnson, Madison, Marion Morgan, Shelby, Tipton
6 (East central)	Blackford, Delaware, Fayette, Henry, Jay, Randolph, Union, Wayne
7 (Southwest)	Daviess, Dubois, Gibson, Greene, Knox, Martin, Pike, Posey, Spencer, Sullivan, Vanderburgh, Warrick
8 (South central)	Brown, Crawford, Floyd, Harrison, Jackson, Lawrence, Monroe, Orange, Perry, Washington
9 (Southeast)	Clark, Dearborn, Jefferson, Jennings, Ohio, Ripley, Scott, Switzerland

CHAPTER 5. DEVELOPMENT OF STRUCTURAL STRENGTH INDEX

As noted in the previous chapters, existing structural strength indicators developed in past literature (e.g. structural number, structural adequacy index, etc.) are typically not applicable at the network level. This is because of the extensive cost and time needed to achieve a comprehensive structural condition database for network level pavement management if those developed structural strength indicators were to be used. Furthermore, existing structural indicators were either restrictive or unbounded on a scale. This chapter therefore discusses the development of a structural strength index that highway agencies can use to evaluate pavement strength for network level pavement management. In particular, deflections from the network-level falling weight deflectometer (FWD) testing are used in this study. A statistical approach to develop a structural strength index is proposed and statistical models to evaluate pavement structural strength index are developed.

5.1 Key Considerations when Developing a Structural Strength Indicator

5.1.1 Properties of a Structural Strength Indicator

When developing a structural strength indicator that is applicable to network level pavement management, there are several considerations that have to be made:

- Measurable: It indicator should be possible (and easy) to measure the indicator in an objective manner. Also, performance measure levels can be readily developed from the indicator.
- Realistic: It should be possible to collect pavement structural condition data without excessive effort, cost and time, at the network level.

- Forecastable: For network and planning level pavement management, the indicator must allow prediction of structural performance at a future time.
- Bounded: For network level pavement management involving multiple criteria, the indicator has to be bounded on a scale.
- Defensible: The indicator has to be clear and concise so that the manner of assessing and interpreting its level can be communicated not only between engineers, but also to managers, government executives and the general public.
- Account for fundamental difference between flexible and rigid pavements: The indicator must consider the fundamental difference between flexible and rigid pavements in terms of mechanical behavior.
- Account for design differences within a pavement type: The indicator must allow for the fact that different pavement thickness design could lead to different pavement structural bearing capacity and hence a different structural strength index.

As noted in Chapter 2, structural strength indicators developed in the literature can be categorized into: functional distress indicators (e.g. pavement condition rating); material properties (e.g. layer elastic modulus); AASHTO structural indicators (e.g. structural number and effective slab thickness); direct deflection measurements; and other structural condition indices (e.g. structural adequacy index). Table 5.1 summarizes the performance of these indicators against the desired properties of the structural condition indicator. From the table, a few points are noted:

- Functional distress indicators such as the pavement condition rating and the individual distress ratings are subjective in nature (due to the way the distresses are evaluated) and can not differentiate between designs of different layer thicknesses. This is despite the fact that these indicators are bounded and are easily understood by non-engineers.
- Material properties and elastic moduli are too cumbersome to be applied at the network level. While they are measurable and forecastable, these indicators are not bounded. Furthermore, these indicators are difficult to understand by non-engineers.

- AASHTO structural indicators allow the objective measurement of pavement structural capacity. However, a comprehensive evaluation at the network level is not realistic. Furthermore, these indicators are not bounded on a scale.
- Direct deflection values, while are easily measured by falling weight deflectometers at the network level, are unbounded. However, the use of deflection alone as an index without any consideration to pavement and design types can create difficulty in structural strength interpretation.
- The use of the structural adequacy index (SAI) developed by Haas et al. (1994) seems the most promising of all the structural condition indicators considered. However, the SAI itself does not consider pavement thickness or design type in its formulation.

Table 5.1: Performance of Existing Indicators against the Desired Properties of an Ideal Structural Condition Indicator

Structural Condition Indicator	Measurable in an Objective Manner	Realistic at Network Level	Forecastable	Bounded	Defensible	Differentiation between Flexible and Rigid Pavements	Differentiation between Different Design Types
Pavement Condition Rating	(a)	✓	✓	✓	✓	✓	×
Distress Severity and Extent Rating	(a)	✓	✓	✓	✓	✓	×
Layer Elastic Moduli	✓	×	✓	×	(b)	✓	×
Material Strength (e.g. flexural strength of concrete, tensile strength of asphalt mixture)	✓	×	✓	×	(b)	✓	×
AASHTO Structural Indicators (e.g. Structural Number, Effective Slab Thickness)	✓	×	✓	×	✓	✓	✓
Direct Deflection Measurement	✓	✓	✓	×	(b)	×	×
Structural Adequacy Index	✓	✓	✓	✓	✓	✓	×

Notes:

(a) Pavement condition rating and individual distress ratings while measurable are primarily subjective.

(b) Material properties, layer moduli and deflections are easily understood within the engineering community, but are less easily understood by managers, executives and the general public

5.1.2 Investigating Correlation between Functional and Structural Conditions

Highway agencies traditionally make use of pavement functional conditions such as pavement roughness (measured in terms of the international roughness index IRI), surface distresses (measured in terms of pavement condition rating PCR) and rut depths on flexible pavements to select pavement maintenance, rehabilitation or reconstruction projects at the project selection level and to δprogram and budget these activities at the network level. While the use of pavement roughness and/or surface distresses to activate maintenance activities are reasonable, these indicators alone might not be representative of the pavement structural condition. It is obvious that a pavement near the point of structural failure will exhibit poor IRI and have numerous surface distresses. However, it is also possible for a pavement of good structural condition to have a poor IRI or exhibit surface distresses. It is therefore of interest to investigate if the functional characteristics of the pavement is sufficient to represent the structural condition of the pavement.

In this study, structural condition is represented by the center deflection δ_1 obtained from the network-level falling weight deflectometer test using a 9000-lb drop weight (see Figure 4.3). Details of the test are described earlier in Chapter 4. Deflection information is collected for the entire Indiana highway network from 2004 to 2007. It is noted that the Indiana Department of Transportation follows a five-year cycle in pavement structural condition evaluation at the network level, and essentially, the dataset used in this study corresponds to approximately one data collection cycle. It is also noted that for jointed PCC pavements, FWD tests were conducted on the center of a PCC slab, i.e. deflections due to center loading on concrete slab are used in the study. Functional data (IRI, PCR and rut depths on asphalt pavements) were also collected for every deflection test location during the same period.

The Spearman rank correlation coefficients were evaluated to determine if there is any collinearity between the center deflection and functional condition indicators (IRI, PCR, rut depths on flexible pavements). For a given pair of indicators (X_i, Y_i), the Spearman rank correlation coefficient ρ_{xy} is defined as:

$$\rho_{xy} = 1 - \frac{6\sum d_i^2}{n \;\; n^2 - 1} \tag{5.1}$$

where d_i (= $x_i - y_i$) is the difference between the ranks of corresponding values X_i and Y_i and n is the number of samples in the data set. Since there are four indicators in question, the number of combination pairs evaluated in the study is ${}^4C_2$ = (4!)/(2!)(2!) = 6 for each set of data.

Table 5.2: Kolmogorov–Smirnov Test Results for Structural and Functional Indicators

Pavement Type	Functional Class	Structural or Functional Indicator	Sample Size n	Kolmogorov–Smirnov statistic D_n (p-value)
Flexible	Interstates	FWD Test Center Deflection δ_1	4766	0.127 *(0)*
		IRI	4766	0.111 *(0)*
		PCR	4766	0.070 *(0)*
		Rut Depth	4766	0.069 *(0)*
	Non-Interstates NHS	FWD Test Center Deflection δ_1	7979	0.127 *(0)*
		IRI	7979	0.093 *(0)*
		PCR	7979	0.008 *(0)*
		Rut Depth	7979	0.155 *(0)*
	Non-Interstates Non-NHS	FWD Test Center Deflection δ_1	21766	0.120 *(0)*
		IRI	21766	0.073 *(0)*
		PCR	21766	0.063 *(0)*
		Rut Depth	21766	0.191 *(0)*
Rigid	Interstates	FWD Test Center Deflection δ_1	915	0.056 *(0)*
		IRI	915	0.085 *(0)*
		PCR	915	0.204 *(0)*
	Non-Interstates NHS	FWD Test Center Deflection δ_1	1180	0.093 *(0)*
		IRI	1180	0.163 *(0)*
		PCR	1180	0.123 *(0)*
	Non-Interstates Non-NHS	FWD Test Center Deflection δ_1	912	0.087 *(0)*
		IRI	912	0.062 *(0)*
		PCR	912	0.125 *(0)*

In this study, the choice of Spearman rank correlation test was selected over the standard Pearson correlation test because the Pearson test requires variables to be normally distributed whereas the Spearman test is a nonparametric test requiring no assumption on the distribution. The Kolmogorov–Smirnov tests were performed on center deflections, IRI and PCR for both asphalt and PCC pavements, and rut depths for asphalt pavements to test for normality and the results are shown in Table 5.2. It can be observed from the test results that the four different variables are non-normal at a 95% significance level, justifying the use of the Spearman rank correlation test.

To test for collinearity, the following hypothesis test was conducted on each combination pair:

$$H_0: \rho_{xy} = 0 \tag{5.2a}$$

$$H_1: \rho_{xy} \neq 0 \tag{5.2b}$$

The null hypothesis states that the Spearman rank correlation coefficient is zero, indicating a lack in linear dependency between the two variables in a combination pair. On the other hand, the alternate hypothesis states that the Spearman rank correlation coefficient is non-zero, indicating that there is some form of linear dependency between the two variables in the combination pair. Table 5.3 shows the results of the hypotheses tests. It was found that there is no linear dependency between the center deflection δ_1 from the FWD test and the functional indicators (IRI, PCR and rut depths) for all pavement types and functional classes at a 95% significance level.

Table 5.3: Results of Spearman Rank Correlation Tests

Pavement Type	Functional Class	Combination Pair	Sample Size	Spearman Rank Correlation Coefficient ρ_{xy} (*p*-value)	Reject H_0? [Equation (5.2)]*
Flexible	Interstates	Center Deflection vs. IRI	4766	-0.039 *(0)*	Yes
		Center Deflection vs. PCR	4766	-0.151 *(0)*	Yes
		Center Deflection vs. Rut	4766	0.179 *(0)*	Yes
	Non-Interstates NHS	Center Deflection vs. IRI	7979	0.106 *(0)*	Yes
		Center Deflection vs. PCR	7979	-0.141 *(0)*	Yes
		Center Deflection vs. Rut	7979	0.095 *(0)*	Yes
	Non-Interstates Non-NHS	Center Deflection vs. IRI	21766	0.149 *(0)*	Yes
		Center Deflection vs. PCR	21766	-0.077 *(0)*	Yes
		Center Deflection vs. Rut	21766	0.109 *(0)*	Yes
Rigid	Interstates	Center Deflection vs. IRI	915	0.319 *(0)*	Yes
		Center Deflection vs. PCR	915	0.053 *(0)*	Yes
	Non-Interstates NHS	Center Deflection vs. IRI	1180	-0.185 *(0)*	Yes
		Center Deflection vs. PCR	1180	-0.204 *(0)*	Yes
	Non-Interstates Non-NHS	Center Deflection vs. IRI	912	0.220 *(0)*	Yes
		Center Deflection vs. PCR	912	-0.474 *(0)*	Yes

* Hypothesis tests were conducted at a 95% significance level.

This finding seems to imply that the structural condition as represented by the center deflection δ_1 from the FWD test is independent of the pavement functional conditions. One may argue that a structurally poor pavement can have a high IRI and a low PCR and vice versa, offering some form of collinearity. The observation made in this study can be explained by Table 5.4, which shows the summary statistics of the data used in this section. It is noted from the table that for a typical highway agency, parameter values indicate that conditions (both structural and functional) are relatively good. This is expected since highway agencies have to maintain their highway network according to specific agency goals. For example, FHWA require states to ensure that 95% of travel on the NHS (in terms of vehicle-miles travelled) experience acceptable ride quality (defined as an average IRI of 170 inches/mile). In this case, it is possible for pavement structural and functional conditions to be uncorrelated, given that most pavements are already in a relatively good or fair condition.

Table 5.4: Summary Test Statistics

Pavement Type	Functional Class	Structural/ Functional Indicator	Sample Size	Mean	Standard Deviation
Flexible	Interstates	Center Deflection δ_1 (10^{-3} inch)	4766	3.1	1.4
		IRI (inch per mile)	4766	76.0	34.0
		PCR	4766	89.0	7.0
		Rut Depth (inch)	4766	0.09	0.03
	Non-Interstates NHS	Center Deflection δ_1 (10^{-3} inch)	7979	5.0	2.3
		IRI (inch per mile)	7979	102.0	43.8
		PCR	7979	90.1	6.3
		Rut Depth (inch)	7979	0.14	0.10
	Non-Interstates Non-NHS	Center Deflection δ_1 (10^{-3} inch)	21766	7.9	4.2
		IRI (inch per mile)	21766	110.1	47.0
		PCR	21766	89.2	6.9
		Rut Depth (inch)	21766	0.13	0.11
Rigid	Interstates	Center Deflection δ_1 (10^{-3} inch)	915	3.6	1.4
		IRI (inch per mile)	915	97.8	28.9
		PCR	915	93.5	5.8
	Non-Interstates NHS	Center Deflection δ_1 (10^{-3} inch)	1180	4.9	1.6
		IRI (inch per mile)	1180	103.2	51.0
		PCR	1180	92.5	4.4
	Non-Interstates Non-NHS	Center Deflection δ_1 (10^{-3} inch)	912	8.9	4.5
		IRI (inch per mile)	912	143.8	48.8
		PCR	912	86.2	8.6

5.2 Development of a Structural Strength Index to Measure Pavement Structural Condition

Since most highway agencies already have some form of network-level data collection system in place for FWD testing, the study sets out to make use of FWD deflections as the basis of developing a structural strength index. If we assume that $(D_{ijk})_n$ is a variable denoting the *n*th sensor deflection for any pavement segment *i* with pavement type *j* and design type and $(\delta_{ijk})_n$ is a given deflection measured by the *n*th sensor during a FWD test on a homogeneous pavement segment *i*, pavement type *j* and design type *k*, we can determine a comparative structural performance of segment i within the pavement family. Here, segments belonging to the same pavement family can be defined as segments having the same pavement type and similar designs (e.g. pavement structural thickness). The comparative performance of the pavement segment within the pavement family in the highway network, $(Y_{ijk})_n$ can be defined by Equation (5.3).

$$ {Y_{ijk}}_{n} = P\left[{D_{jk}}_{n} > {\delta_{ijk}}_{n} \right] = 1 - F\left[{\delta_{ijk}}_{n} \right] \qquad (5.3) $$

The structural performance of the pavement segment *i* is defined here to be the probability of pavements in the given family (*j*, *k*) having a deflection larger than $(\delta_{ijk})_n$.

If we were to make use of only the center deflection $(\delta_{ijk})_1$ (i.e. 1st sensor in the FWD test) to compare the pavement structural performance of a given segment *i* in a given family (*j*, *k*), we can define a term called "Structural Strength Index" or SSI by refining Equation (5.3). In this case, SSI is defined as the probability that pavements in a given family (*j*, *k*) having a deflection larger than $(\delta_{ijk})_1$.

$$ SSI = 100P\left[{D_{jk}}_{1} > {\delta_{ijk}}_{1} \right] = 100 \quad 1 - F\left[{\delta_{ijk}}_{1} \right] \qquad (5.4) $$

In this case, only the FWD center deflection data is used since it reflects the overall structural capacity of the pavement (Noureldin et al., 2003). Note that these deflections have to be normalized to a standard load (generally 9,000 lb, or 40 kN, for highways) and a standard temperature (generally 68°F, or 20°C). Since we only make use of one deflection point and a standard load, the SSI cannot be used to determine individual layer material properties nor can we use it to determine defects in individual pavement layers.

The SSI is good for evaluating the overall pavement structural condition and hence is only valid for network level pavement structural evaluation.

A few key steps are required to develop the structural strength index SSI as shown in Figure 5.1. They are:

Step 1: Determine the different pavement families (j, k). This can be achieved by first identifying the type of pavements managed by the highway agency and their structural design.

Step 2: Determine $f[(\delta_{ijk})_1]$ and $F[(\delta_{ijk})_1]$ which are the probability distribution and cumulative probability distribution of the center deflections δ_1 for a given family (j, k).

Step 3: Determine the structural strength index SSI functions using Equation (5.4).

Table 5.5 compares the performance of the SSI against other structural conditions developed in the literature. The SSI can be said to be:

- Measurable: The SSI can be easily measured by the FWD test using the deflection of the center sensor. The FWD test is an objective method to measure the structural response of the pavement.
- Realistic: The SSI makes use of deflection that can be easily evaluated at the network level.
- Forecastable: Since the SSI is an index based on the probabilistic distribution of FWD deflections in a highway network, models have to be further developed to allow planners to forecast SSI deterioration on pavements. This is currently not considered in this study.

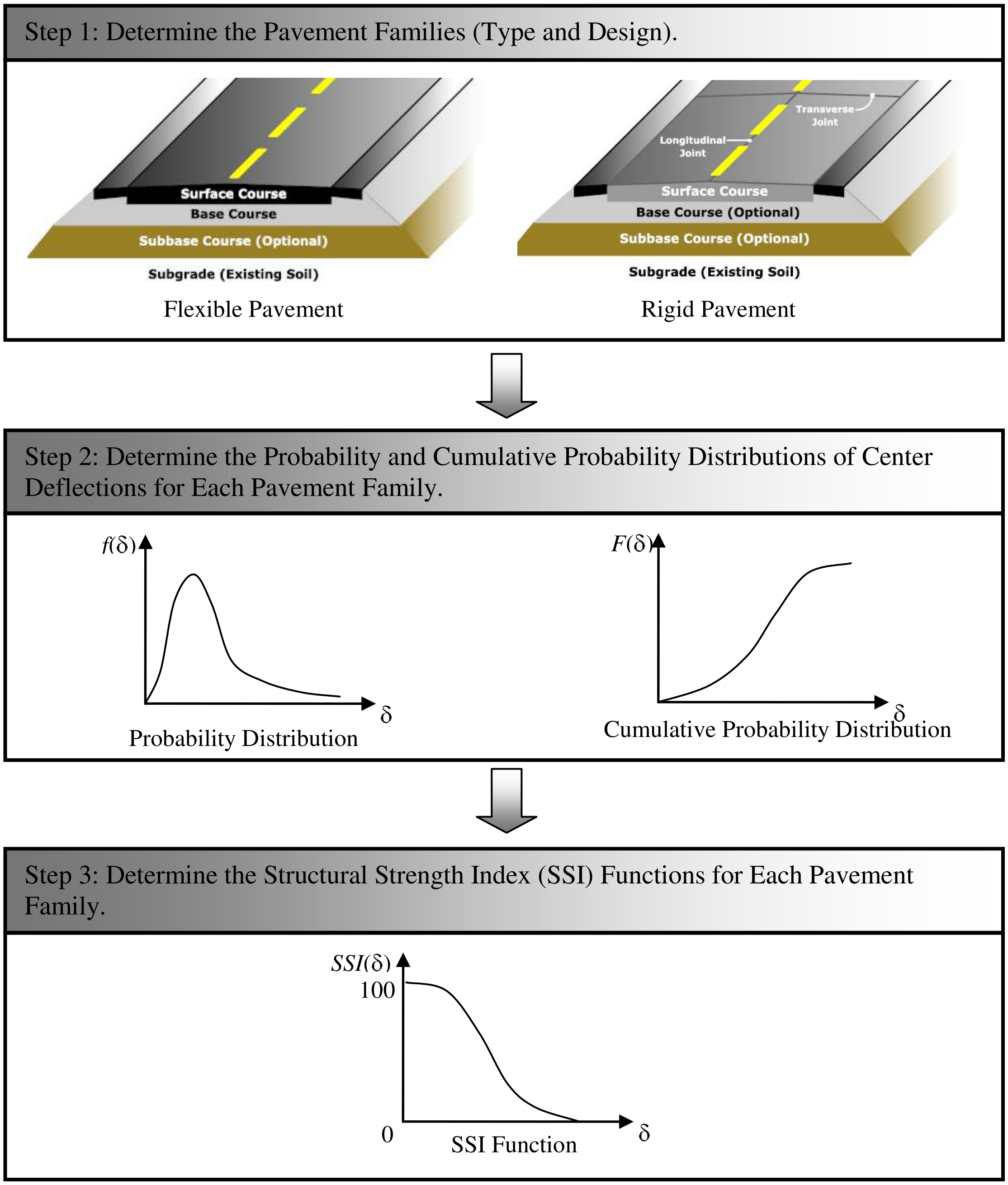


Figure 5.1: Methodology to Develop Structural Strength Index (SSI) Functions

- Bounded: The SSI is bounded between 0 and 100 and hence can be easily integrated in a multi-objective multi-criteria transportation asset management system.

- Defensible: The SSI essentially provides a 0 (very poor) to 100 (excellent or perfect) scale, which is easily comprehensible by both engineers and non-engineers. Since the SSI provides the comparative performance to other pavement sections in the same highway network, policy makers can easily understand the physical meaning of SSI.
- Account for fundamental difference between flexible and rigid pavements: The SSI considers the fundamental difference between flexible and rigid pavements through categorization by pavement families.
- Account for design differences within a pavement type: The SSI allows consideration by pavement thickness design through categorization by pavement families.

These properties of the SSI represent the ability of the indicator to measure structural strength at the network level and provides potential for managers to consider pavement structural condition in network-level pavement management.

Table 5.5: Comparison between Structural Strength Index and other Structural Indicators

Structural Condition Indicator	Measurable in an Objective Manner	Realistic at Network Level	Forecastable	Bounded	Defensible	Differentiation between Flexible and Rigid Pavements	Differentiation between Different Design Types
Pavement Condition Rating	(a)	✓	✓	✓	✓	✓	×
Distress Severity and Extent Rating	(a)	✓	✓	✓	✓	✓	×
Layer Elastic Moduli	✓	×	✓	×	(b)	✓	×
Material Strength (e.g. flexural strength of concrete, tensile strength of asphalt mixture)	✓	×	✓	×	(b)	✓	×
AASHTO Structural Indicators (e.g. Structural Number, Effective Slab Thickness)	✓	×	✓	×	✓	✓	✓
Direct Deflection Measurement	✓	✓	✓	×	(b)	×	×
Structural Adequacy Index	✓	✓	✓	✓	✓	✓	×
Structural Strength Index	✓	✓	**(c)**	✓	✓	✓	✓

Notes:

(a) Pavement condition rating and individual distress ratings while measurable are primarily subjective.

(b) Material properties, layer moduli and deflections are easily understood within the engineering community, but are less easily understood by managers, executives and the general public.

(c) Models have to be developed to develop forecasting models based on structural strength index SSI.

5.3 Determination of Structural Strength Index Functions: A Case Study for Indiana

The methodology stated in the previous section is applied to determine a set of structural strength indices using the state of Indiana as an illustration. Details of data collected for this study are found in Chapter 4.

5.3.1 Determining Pavement Families by Type and Design

The first step in evaluating the SSI functions is to determine the different pavement families. Pavement families are defined by pavement type (flexible or asphalt and rigid or PCC pavements) and pavement structural design. While it is relatively easy to determine the pavement type at the network level through network level pavement condition and roadway inventory surveys, maintaining a detailed database on pavement thicknesses at the network level is a challenging task for most highway agencies. Recognizing that most highway agencies do not have a detailed database on pavement structural design at the network level, this study proposes to use the pavement functional class as a proxy for pavement structural design. The basis for this assumption is that pavement structural design is fairly standardized within the same functional class (i.e. interstate, non-interstate NHS and non-interstate non-NHS highways). Given the pavement types *j* (asphalt/PCC) and functional classes *k* (interstate, non-interstate NHS and non-interstate non-NHS highways), six different combinations can be obtained:

- Flexible Interstate
- Flexible Non-Interstate NHS
- Flexible Non-NHS
- Rigid Interstate
- Rigid Non-Interstate NHS
- Rigid Non-NHS

There is a need to check if any of these six families can be combined. The Mann-Whitney-Wilcoxon (MWW) test was performed to test whether any pair of pavement families has the same distribution for the center deflection δ_1:

H_0: The δ_1 distributions of the two pavement families are the same. (5.5a)

H_1: The δ_1 distributions of the two pavement families are not the same. (5.5b)

The acceptance of the null hypothesis would mean that the distributions of the pair of pavement families are the same and hence two families can be combined. The non-parametric MWW test is used in this study because deflection distribution is found to be non-normal. This renders the use of traditional parametric tests invalid.

Table 5.6 shows the test results for the six different combination pairs of pavement families. It can be observed that the null hypothesis was rejected for all the six different combination pairs. This means that SSI functions have to be developed for the six different design categories (satisfying one of the desirable properties of a structural strength indicator).

Table 5.6: Mann-Whitney-Wilcoxon Test Results for Different Combination Pairs of Pavement Families

Pair No.	Pavement Family	Sample Size *n*	MWW *U*-Statistic	*z*-Statistic [a] (*p*-value)	Reject H_0? [Equation (5.5)][b]
1	Flexible Interstate	4712	8127540	-56.85 *(0)*	Yes
	Flexible Non-Interstate NHS	8534	32084667		
2	Flexible Interstate	4712	6926284	-93.44 *(0)*	Yes
	Flexible Non-NHS	21966	96577508		
3	Flexible Non-Interstate NHS	8534	43619671	-72.59 *(0)*	Yes
	Flexible Non-NHS	21966	143838173		
4	Rigid Interstate	969	218281	-18.81 *(0)*	Yes
	Rigid Non-Interstate NHS	905	658664		
5	Rigid Interstate	969	57353	-29.29 *(0)*	Yes
	Rigid Non-NHS	714	634512		
6	Rigid Non-Interstate NHS	905	110156	-22.80 *(0)*	Yes
	Rigid Non-NHS	714	536013		

Notes:
[a] When sample size is large, the MWW *U*-statistic is approximately normally distributed. The *z*-statistic for *U* can therefore be estimated.
[b] Hypothesis tests were conducted at a 95% significance level.

There is also a need to determine if the two different pavement types (flexible and rigid) can be combined. The Mann-Whitney-Wilcoxon (MWW) test is again performed to test whether the two pavement types have the same distribution for the center deflection δ_1:

H_0: The δ_1 distributions of the two pavement types are the same. (5.6a)

H_1: The δ_1 distributions of the two pavement types are not the same. (5.6b)

Table 5.7 shows the test results for the three different combination pairs of pavement families. It can be observed that the null hypothesis was rejected for all the three different combination pairs. This means that we have to differentiate pavement types when developing SSI functions (satisfying one of the desirable properties of a structural strength indicator).

Table 5.7: Mann-Whitney-Wilcoxon Test Results for Different Pavement Types

Pair No.	Pavement Family	Sample Size *n*	MWW *U*-Statistic	*z*-Statistic [a] (*p*-value)	Reject H_0? [Equation (5.5)][b]
1	Flexible Interstate	4712	3028306	-16.03 *(0)*	Yes
	Rigid Interstate	969	1537621		
2	Flexible Non-Interstate NHS	8534	5627116	-22.65 *(0)*	Yes
	Rigid Non-Interstate NHS	905	2096154		
3	Flexible Non-NHS	21966	9467276	-9.44 *(0)*	Yes
	Rigid Non-NHS	714	6216448		

Notes:
[a] When sample size is large, the MWW *U*-statistic is approximately normally distributed. The *z*-statistic for *U* can therefore be estimated.
[b] Hypothesis tests were conducted at a 95% significance level.

5.3.2 Additional Curve Combination

In addition to these six families of curves, one additional combination will be determined:

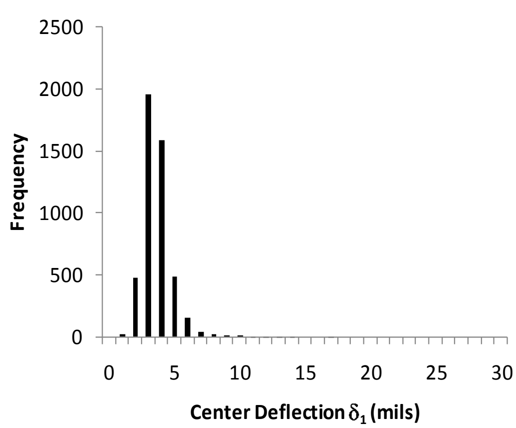
- All pavement types and functional classes combined.

It may be argued that the differences in the deflections of a pavement are not significant in terms of pavement type and functional class. A high deflection is poor regardless of the pavement type and functional class. By using one SSI curves as opposed to six curves, the implementation of the SSI in a PMS may be simplified.

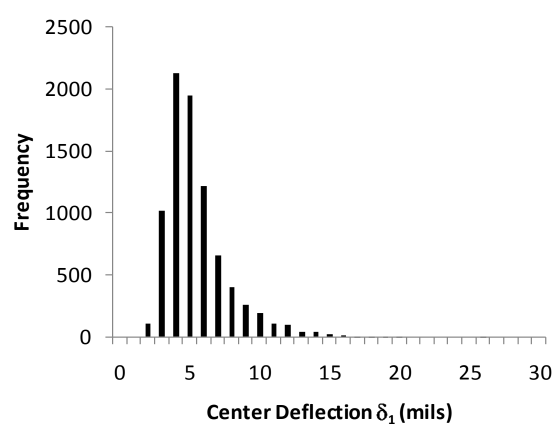
5.3.3 Determination of Cumulative Probability Functions of Center Deflections for Each Pavement Family

The second step involves determining the cumulative probability functions for the six different pavement families. Figure 5.2 and 5.3 show the histograms and cumulative probability distribution of each pavement family. It can be observed that:

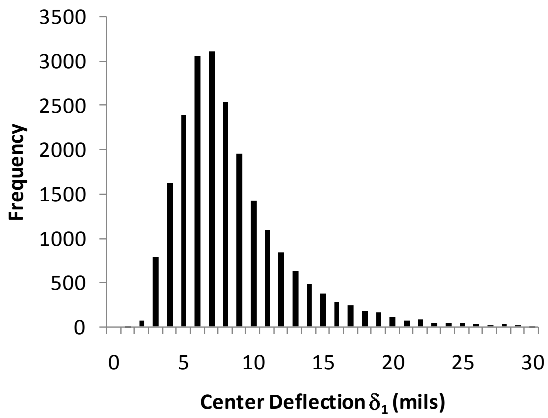
- The distributions of δ_1 for all six pavement families are non-normal. The distributions are left-skewed.
- The mean and mode of the distribution increases from interstate class to non-interstate NHS class to non-interstate non-NHS class. Similarly, the variance increases from the interstate class to non-interstate NHS class to non-interstate non-NHS class. This behavior is expected since interstate pavements are designed and maintained to a more stringent deflection level as compared to non-interstate pavements.



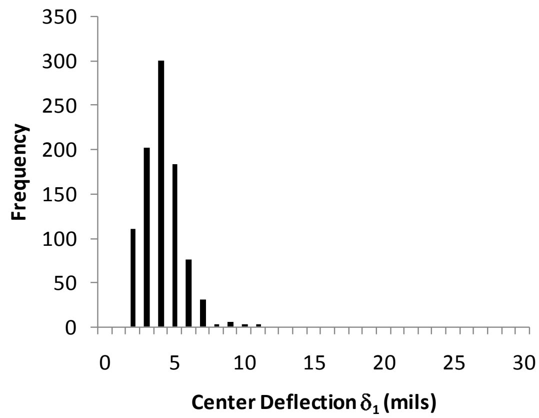
(a) Flexible Interstate



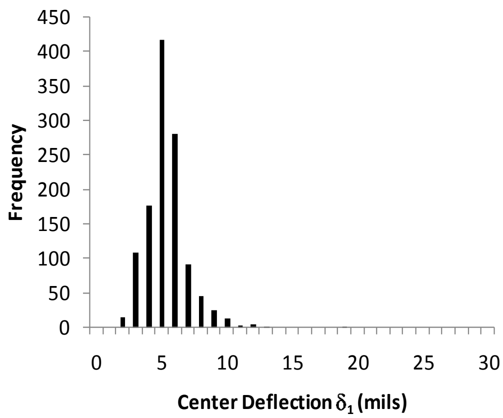
(b) Flexible Non-Interstate NHS



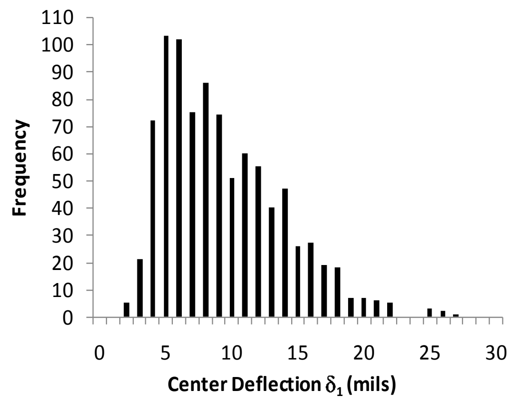
(c) Flexible Non-Interstate Non-NHS



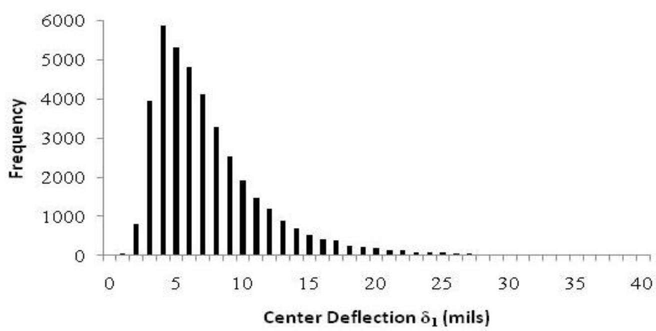
(d) Rigid Interstate



(e) Rigid Non-Interstate NHS

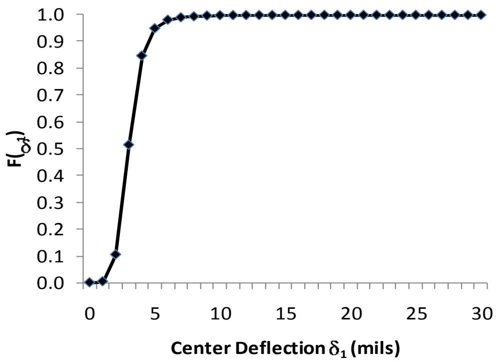


(f) Rigid Non-Interstate Non-NHS

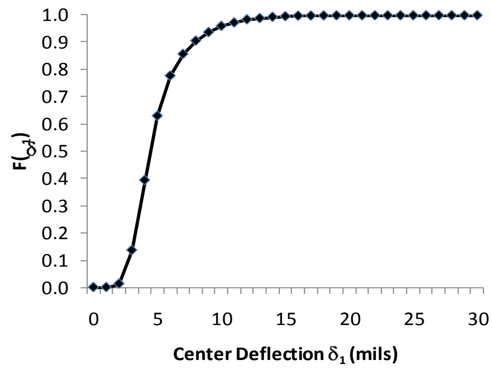


(g) All Pavement Types and Functional Classes

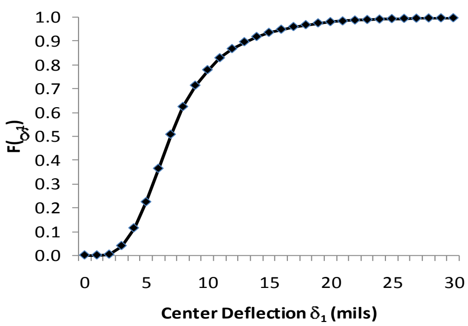
Figure 5.2: Histograms for Center Deflection δ_1 of Different Pavement Families



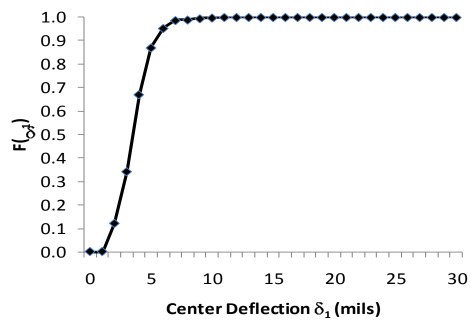
(a) Flexible Interstate



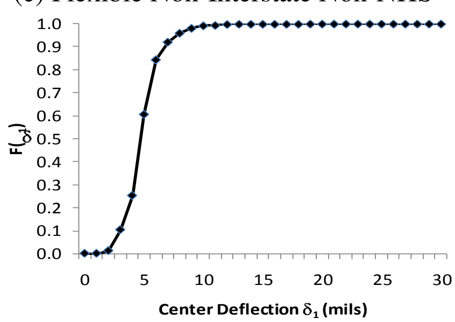
(b) Flexible Non-Interstate NHS



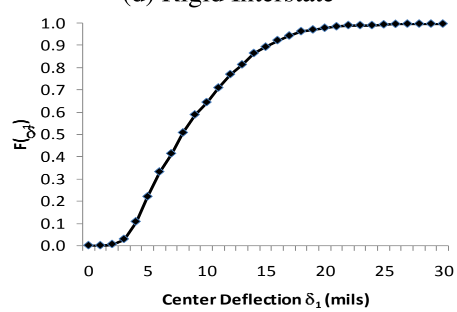
(c) Flexible Non-Interstate Non-NHS



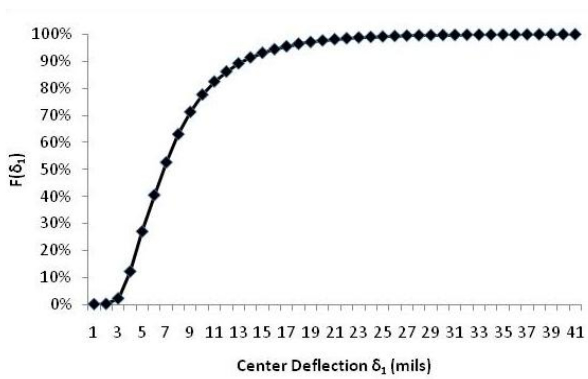
(d) Rigid Interstate



(e) Rigid Non-Interstate NHS



(f) Rigid Non-Interstate Non-NHS



(g) All Pavement Types and Functional Classes

Figure 5.3: Cumulative Probability Distributions for Center Deflection δ_1 of Different Pavement Families

5.3.4 Determination of Structural Strength Index Functions

It is noted from Figure 5.2 and 5.3 that all six pavement families have the same shape. This means that a generic structural strength index (SSI) function can be developed to describe the cumulative probability distribution of each pavement family. It is assumed that SSI can be described by a generic form shown in Equation (5.6):

$$SSI_{jk} = 100\left(1-\alpha e^{-\frac{\beta}{\delta_1{}^{\gamma}}}\right) \qquad \text{for given pavement family } (j,k) \qquad (5.6)$$

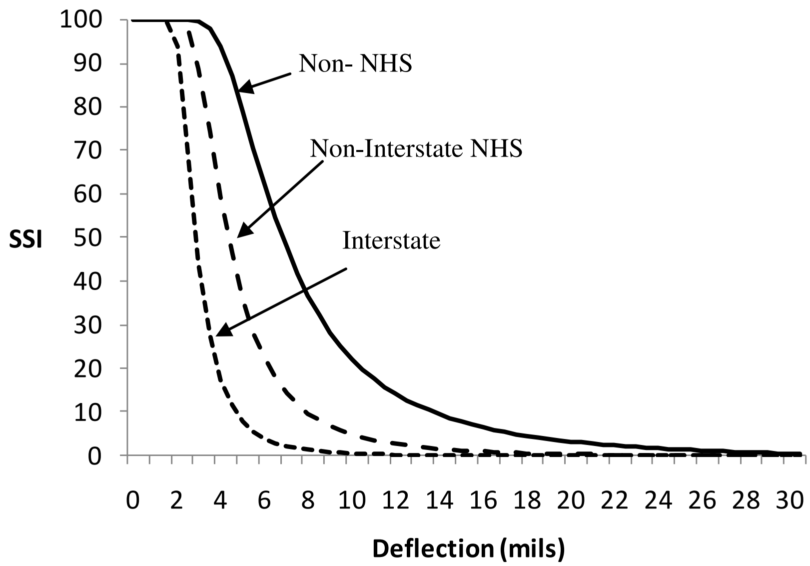
where δ_1 is the deflection measured at the center sensor (in mils), and α and β are coefficients that has to be determined. A structural strength index of 0 indicates that the segment is the structurally worst pavement segment in the entire highway network while a structural strength index of 100 represents a pavement segment is the structurally best within the network. By performing regression analyses on each pavement family, the coefficients α and β can be determined. The results are shown in Table 5.8. The r^2 for each equation is at least 0.99 which indicates an excellent fit to the data. Equation (5.6) can therefore be used to calculate the SSI for the pavement and system type.

Table 5.8: Coefficients in Equation (5.6) for Different Pavement Families

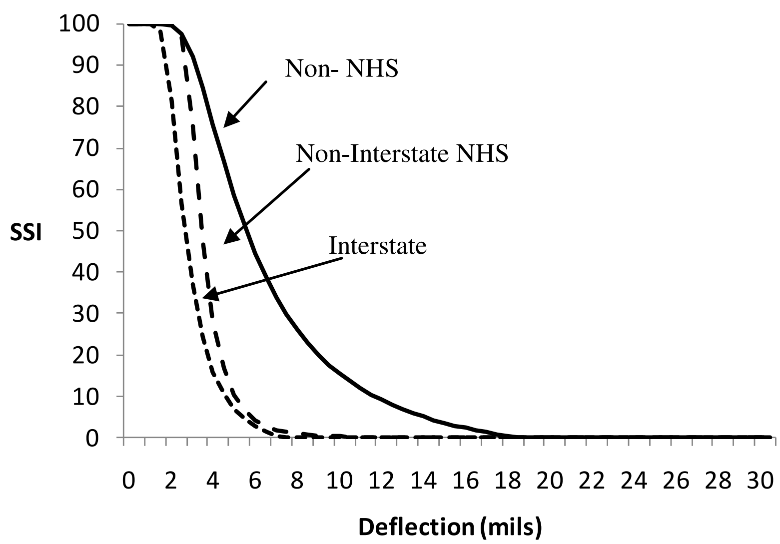
Pavement Family	Coefficients of Model [Equation (5.6)]			Regression Coefficient r^2
	α	β	γ	
Flexible Interstate	1.0013	40.303	3.853	0.998
Flexible Non-Interstate NHS	1.0035	66.811	3.106	0.998
Flexible Non- NHS	1.0124	100.838	2.586	0.999
Rigid Interstate	1.0345	14.301	3.056	0.999
Rigid Non-Interstate NHS	1.0017	338.056	4.995	0.999
Rigid Non- NHS	1.0717	23.600	1.999	0.999
All Pavements & Func Class				

Figure 5.4 shows a graphical representation of the SSI functions for different pavement families. It can be observed that:

- SSI is defined for a larger range for flexible pavements than rigid pavements. This is expected since the elastic modulus for rigid pavements are much larger than that for flexible pavements.



(a) Flexible Pavements



(b) Rigid Pavements

Figure 5.4: SSI Functions for Different Pavement Families

- For both flexible and rigid pavements, SSI for Interstate pavements tends to be defined over a smaller range than non-Interstate NHS pavements and non-NHS pavements. This is expected since interstate pavements are designed to handle heavier traffic loads (i.e. they have larger pavement thicknesses and materials of better strength).

- For a given deflection, SSI for interstate pavements is lower than that for non-interstate NHS and non-NHS pavements. For example, a flexible pavement with a 4 mils deflection will result in an SSI of 17 on Interstate pavements, 59 on non-Interstate NHS pavements and 94 on non-NHS pavements. This is expected since Interstate pavements are subject to a more stringent deflection guideline compared to the other pavement types.

Using the information in Table 2.10 that Noureldin et al. (2005) developed for determining thresholds under different traffic loading, SSI thresholds can be developed. Cumulative ESAL levels are estimated from the cumulative truck traffic for the different highway systems and the deflections are estimated by interpolation. Corresponding SSI thresholds for both flexible and rigid pavements are shown in Table 5.9.

Table 5.9: SSI Thresholds for Indiana Pavements

Pavement	System	Measure	Excellent	Very Good	Good	Fair	Poor
Flexible	Interstate	Deflection (mil)	0-4	4-6	6-8	8-10	>10
		SSI	95-100	90-95	85-90	80-85	<80
	Non-Interstate NHS	Deflection (mil)	0-6	6-8	8-10	10-12	>12
		SSI	90-100	85-90	80-85	75-80	<75
	Non-NHS	Deflection (mil)	0-8	8-10	10-12	12-14	>14
		SSI	85-100	80-85	75-80	70-75	<70
Rigid	Interstate	Deflection (mil)	0-4	4-6	6-8	8-10	>10
		SSI	95-100	90-95	85-90	80-85	<80
	Non-Interstate NHS	Deflection (mil)	0-6	6-8	8-10	10-12	>12
		SSI	90-100	85-90	80-85	75-80	<75
	Non-NHS	Deflection (mil)	0-8	8-10	10-12	12-14	>14
		SSI	85-100	80-85	75-80	70-75	<70

5.4 Summary of Findings

This chapter has discussed the development of the structural strength index. A structural condition indicator has the desirable properties of being measurable, and distinguishable by pavement types and designs. The concept of structural strength index (SSI) is proposed where the SSI is defined as the probability that pavements in a given family have a deflection larger than the measured deflection in a given highway segment. It is further shown in the chapter that the SSI models satisfy the desirable properties of a structural condition indicator. Using the case of Indiana as an illustration, SSI models are then estimated to provide managers with means to measure pavement structural condition for network level pavement management. The use of SSI can therefore be a viable approach to measure comparative pavement structural condition in a highway network.

CHAPTER 6. DEVELOPMENT OF DEFLECTION PREDICTION MODELS

The previous chapter has demonstrated that pavement deflections (in particular the center deflection δ_1 from the network-level FWD testing) can be used to characterize the strength of pavements in a highway network. While the developed structural strength index (SSI) models can allow the agency to plan for pavement activities at the network level, applications are still limited since network-level deflection testing is performed on a three- to five-years cycle. Consequentially, statistical models were developed to predict center deflections δ_1 (and SSI) that can be used on an annual basis.

6.1 Methodology

In this study, ordinary least square (OLS) regression was performed to develop deflection prediction models. It was assumed that the deflection model is of the following form:

$$\delta_1 = \beta_0 + \sum_{i=1}^{n} \beta_i x_i \tag{6.1}$$

where δ_1 is the deflection of the center sensor during the network level FWD test (measured in mils), x_i are the independent variables and β_i are the coefficients that have to be estimated. The FWD test is assumed to be performed at a single load level (9000-lb). For rigid pavements, δ_1 is the deflection of the center sensor for the center-loading (i.e. mid-slab) FWD test.

Table 6.1 shows a list of the variables considered, including pavement surface age (defined as year of the FWD testing minus the year of last pavement treatment), the pavement type (flexible or rigid), center deflection δ_1, average annual daily traffic or AADT, the percentage of truck traffic, average annual truck traffic, cumulative traffic loading since last pavement treatment, dominant condition drainage class of subgrade, average maximum daily temperature, average minimum daily temperature, cumulative average daily temperature variation since last pavement treatment.

In model development, pavement structural information (such as material layer thicknesses and pavement design) was not available. This is typical in most highway agencies where structural information is not frequently collected or updated in the network level pavement management system. In absence of thickness and design information, truck count was used instead as a proxy for pavement thickness and design.

As noted in Table 6.1 data such as average temperatures, rainfall, and truck traffic can be collected annually. This means that the proposed model can be used to predict center deflection δ_1 in the absence of FWD deflection or material information, allowing a complete database for network-level pavement management.

Table 6.1: Variables Considered in Study

Variable	Description
δ_1	Center deflection of the pavement section (mils)
Rural	Rural Road Indicator: 0 = Urban, 1 = Rural
Flex	Flexible Pavement Indicator: 0 = Rigid, 1 = Flexible
System	System Type: 1 = Interstate, 2 = Non-NH, 3= Non-NHS
Drainage	Dominant condition drainage class
%Comm	Percent of commercial traffic
Trucks	Truck Count (in thousands) – Proxy for Pavement Thickness
AADT	Current Annual Average Daily Traffic (in thousands)
Surf_Age	Surface age (years)
AvHiTemp	Average high temperature for weather region (^oF)
AvLoTemp	Average low temperature for weather region (^oF)
Avg_Temp	Average temperature for weather region (^oF)
Avg_Rain	Average rainfall for weather region (inch)
NorthReg	Weather Regions 1,2,3,4,5,6 grouped together
Delta_Tem	AvHiTemp – Avg_Temp (^oF)
CumTrk	Number of trucks since last pavement work in millions of trucks
CumTemp	Delta_Tem * Surf_Age (^oF-year)

The same pavement families that were used in developing the SSI were used to develop deflection prediction models. The six cases are:

- Flexible interstate

- Flexible non-interstate NHS
- Flexible non-NHS
- Rigid interstate
- Rigid non-interstate NHS
- Rigid non-NHS

These models can allow the prediction of SSI for network level management, as shown schematically in Figure 6.1.

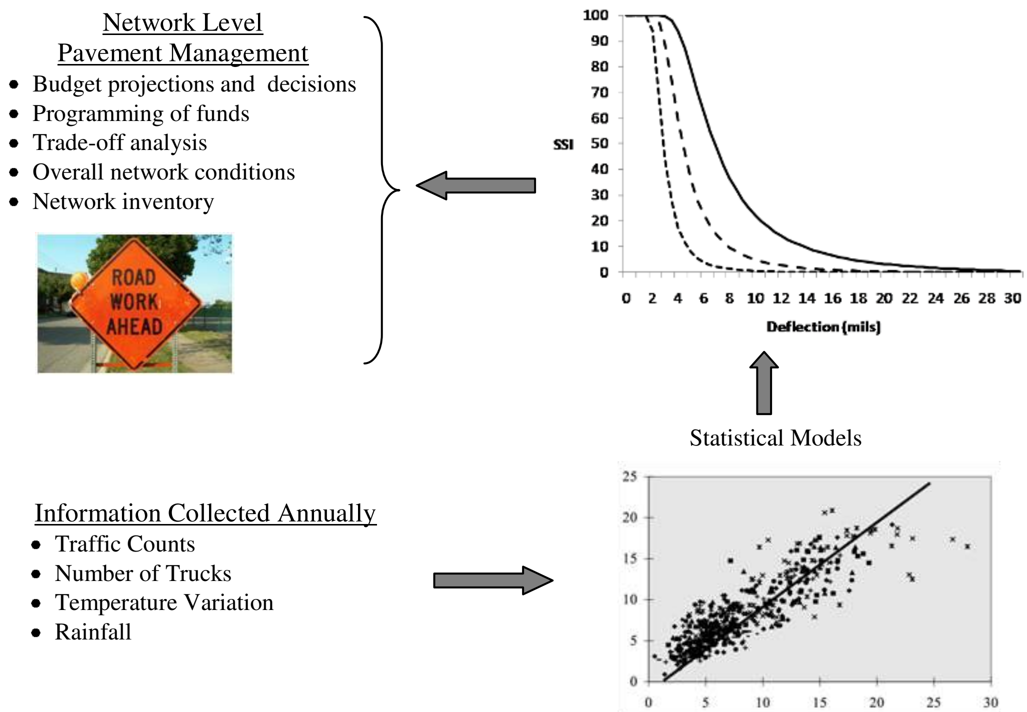


Figure 6.1: Use of Deflection Prediction Model in Pavement Management

6.2 Development of Deflection Prediction Models

Deflection models were developed for all six pavement families using the following functional form:

$$\ln \delta_1 = \beta_0 + \sum_{i=1}^{n} \beta_i x_i \tag{6.2}$$

where δ_1 is the deflection of the center sensor during the network level FWD test (measured in mils), x_i are the independent variables and β_i are the coefficients that have to be estimated. The logarithmic of center deflection was chosen because it produce a better fit as compared to a lineal center deflection.

6.2.1 Deflection Prediction Models for Flexible Pavements

Table 6.2 shows the deflection prediction models for both flexible pavements. The following can be observed from the table:

- In general, when the cumulative truck traffic increases, natural logarithm of the center deflection $\ln(\delta_1)$ increase. This is expected because pavement structure deteriorates with increasing truck traffic loading, resulting in a larger deflection. The only exception is the non-interstate NHS flexible pavement family.
- When the cumulative temperature variation increases, natural logarithm of center deflection $\ln(\delta_1)$ increases. Asphalt undergoes aging with weathering, causing a loss in structural capacity and an increase in deflection. The only exception is the non-interstate NHS flexible pavement family.
- When the rainfall increases, center natural logarithm of center deflection $\ln(\delta_1)$ increases. This is due to increased weathering which leads to asphalt aging and degradation of base and subgrade strength.
- Truck count was used in the models as a proxy for pavement thickness and design. A higher truck count would warrant a thicker pavement and a more stringent design. In general, a larger truck count (and pavement thickness) would result in a smaller center deflection δ_1 and natural logarithm $\ln(\delta_1)$. The exception to this behavior is the non-interstate NHS flexible pavement families.
- Rural highways can also be viewed as a proxy of pavement thickness and design. Typically, pavements on rural highways are expected to carry less traffic and tend to be designed to a lower standard as compared to pavements on urban highways. This means that deflection tends to be lower for rural highway pavements as compared to urban highway pavements.

- The developed deflection prediction models exhibit a rather low r^2. While these models allow the identification of possible factors affecting deflections in flexible pavements, the models cannot be applied in practice to predict actual pavement deflections.

Table 6.2: Deflection Prediction Models for Flexible Pavements

(a) Model Results

Pavement Family	Variable	Coefficient	Standard Error	b/Std Error	P[|Z|>z]	Mean of X
Flexible Interstate	Constant	1.7345	0.1314	13.2050	0.0000	-
	CUMTEMP	-0.0066	0.0012	-5.4370	0.0000	97.9795
	CUMTRK	0.0160	0.0029	5.5400	0.0000	42.3022
	TRUCKS	-0.0515	0.0101	-5.0780	0.0000	12.7209
	RURAL	-0.1293	0.0432	-2.9970	0.0027	0.1750
Flexible Non-NHS	Constant	1.3141	0.0519	25.3410	0.0000	-
	CUMTEMP	0.0022	0.0004	5.4750	0.0000	117.0772
	AVG_RAIN	0.0488	0.0306	1.5950	0.1107	0.3566
	CUMTRK	-0.0138	0.0037	-3.7620	0.0002	10.8090
	TRUCKS	0.0258	0.0162	1.5910	0.1115	2.6812
	RURAL	0.1258	0.0356	3.5320	0.0004	0.1765
Flexible Non-Interstate NHS	Constant	2.0443	0.0177	115.7370	0.0000	-
	CUMTEMP	0.0002	0.0001	1.8310	0.0670	127.1212
	AVG_RAIN	0.0604	0.0113	5.3350	0.0000	0.3034
	CUMTRK	0.0103	0.0047	2.1810	0.0292	2.6770
	TRUCKS	-0.2104	0.0231	-9.1270	0.0000	0.6347
	RURAL	-0.2163	0.0167	-12.9280	0.0000	0.1055

(b) Summary Statistics

Pavement Family	Flexible Interstate	Flexible Non-Interstate NHS	Flexible Non-NHS
Observations	537	816	7865
r^2	0.0628	0.0645	0.0854
Log-Likelihood	-197.6304	-370.5077	-4812.6679
Durbin-Watson Statistic	1.9034	2.0441	2.0108
Mean of $\ln(\square_1)$	1.0830	1.5368	1.9619
Std Dev of $\ln(\square_1)$	0.3615	0.3942	0.4666

6.2.2 Deflection Prediction Models for Rigid Pavements

Table 6.3 shows the deflection prediction models for rigid pavements. The following can be observed from table:

- In general, when the cumulative truck traffic increases, natural logarithm of center deflection $\ln(\delta_1)$ increase. This is expected because pavement structure deteriorates with increasing truck traffic loading, resulting in a larger deflection.
- When the cumulative temperature variation increases, natural logarithm of center deflection $\ln(\delta_1)$ increases. Concrete slabs undergo fatigue through repeated warping due to temperature variations. This causes a loss in structural capacity and results in an increase in deflection. The only exception is the non-interstate non-NHS rigid pavement family.
- When the rainfall increases, natural logarithm of center deflection $\ln(\delta_1)$ increases. This is due to increased weathering which leads to deterioration of joints and loss of base and subgrade support.
- Truck count was used in the models as a proxy for pavement thickness and design. A higher truck count would warrant a thicker pavement and a more stringent design. In general, a larger truck count (and pavement thickness) would result in a smaller center deflection δ_1 and natural logarithm $\ln(\delta_1)$. The exception to this behavior is the rigid non-interstate NHS pavement family.
- Rural highways can also be viewed as a proxy of pavement thickness and design. Typically, pavements on rural highways are expected to carry less traffic and tend to be designed to a lower standard as compared to pavements on urban highways. This means that deflection tends to be lower for rural highway pavements as compared to urban highway pavements.
- The developed deflection prediction models exhibit acceptable r^2 values. These models can therefore used in the pavement management systems to predict actual pavement deflections.

Table 6.3: Deflection Prediction Models for Rigid Pavements

(a) Model Results

Pavement Family	Variable	Coefficient	Standard Error	b/St.Er.	P[|Z|>z]	Mean of X
Rigid Interstate	Constant	2.6063	0.1502	17.3560	0.0000	-
	CUMTEMP	-0.0048	0.0006	-7.7930	0.0000	218.0201
	CUMTRK	0.0149	0.0023	6.4320	0.0000	65.4100
	TRUCKS	-0.1306	0.0159	-8.1900	0.0000	10.0180
	RURAL	-0.2713	0.0947	-2.8660	0.0045	0.0683
Rigid Non-NHS	Constant	0.9778	0.0452	21.6230	0.0000	-
	CUMTEMP	0.0017	0.0002	10.7400	0.0000	199.7880
	AVG_RAIN	0.4254	0.0329	12.9260	0.0000	0.2538
	CUMTRK	-0.0294	0.0031	-9.5980	0.0000	10.4187
	TRUCKS	0.1955	0.0161	12.1040	0.0000	2.1385
Rigid Non-Interstate NHS	Constant	2.6160	0.0509	51.3970	0.0000	-
	CUMTEMP	-0.0013	0.0003	-4.0010	0.0001	170.5222
	CUMTRK	0.0115	0.0054	2.1250	0.0344	9.2552
	TRUCKS	-0.3695	0.0441	-8.3710	0.0000	1.1986
	RURAL	-0.2572	0.1002	-2.5660	0.0108	0.0411

(b) Summary Statistics

Pavement Family	Rigid Interstate	Rigid Non-Interstate NHS	Rigid Non- NHS
Observations	249	599	316
r^2	0.3471	0.2667	0.5003
Log-Likelihood	-77.8065	-73.7409	-115.3490
Durbin-Watson Statistic	2.07098	2.0012	2.0752
Mean of $\ln(\square_1)$	1.2030	1.5394	2.0436
Std Dev of $\ln(\square_1)$	0.4101	0.3199	0.4939

6.3 Validation and Application to Determine Structural Strength Index

In order to validate the results of the models, 60 pavement segments not used in model development were randomly selected and the deflection was estimated using the models shown in Tables 6.2 and 6.3. In addition, comparable SSI values were calculated for both actual and predicted deflection values. The predicted deflections were then compared to the actual deflection. Table 6.4 shows the validation data for both and SSI values.

Table 6.4: Deflection Prediction Model Validation

Pavement	System	CumTemp	Ave_ Rain	Cum_Truck	Truck	Rural	Actual Deflection	Predicted Deflection
Flexible	Interstate	84	2.34	17.32	6.78	0	3.68	3.03
Flexible	Interstate	56	2.13	28.67	15.71	0	2.96	2.76
Flexible	Interstate	133	3.69	57.18	12.05	0	3.07	3.15
Flexible	Interstate	196	1.12	74.28	12.72	1	1.60	2.32
Flexible	Interstate	164	3.69	78.51	13.44	0	2.74	3.37
Flexible	Interstate	62	3.69	27.44	12.53	0	2.46	3.07
Flexible	Interstate	62	4.89	26.89	12.28	0	3.13	3.08
Flexible	Interstate	123	3.69	57.99	13.24	0	3.27	3.21
Flexible	Interstate	71	5.25	21.32	8.34	0	2.77	3.25
Flexible	Interstate	95	3.61	53.70	16.35	0	2.81	3.08
Flexible	NHS	217	3.71	8.46	1.10	1	9.57	7.46
Flexible	NHS	88	4.42	9.99	3.04	0	4.10	5.28
Flexible	NHS	149	3.81	6.15	1.53	0	2.17	5.94
Flexible	NHS	128	1.60	6.57	2.00	1	5.70	5.82
Flexible	NHS	86	4.83	0.67	0.23	0	3.88	5.67
Flexible	NHS	172	0.90	6.63	1.30	0	6.46	5.35
Flexible	NHS	110	0.90	3.01	0.83	1	4.02	5.51
Flexible	NHS	82	4.89	9.17	3.14	1	2.04	6.14
Flexible	NHS	88	4.42	9.99	3.04	0	4.85	5.28
Flexible	NHS	136	7.21	7.87	1.66	0	5.37	6.68
Flexible	Non-NHS	120	2.34	0.64	0.18	0	4.87	8.73
Flexible	Non-NHS	138	3.65	1.71	0.39	0	12.18	8.96
Flexible	Non-NHS	70	5.19	1.84	0.72	1	6.67	7.28
Flexible	Non-NHS	214	1.59	4.65	0.61	1	5.48	5.99
Flexible	Non-NHS	95	3.20	3.00	1.03	0	8.45	7.46
Flexible	Non-NHS	214	1.59	4.65	0.61	1	6.50	5.99
Flexible	Non-NHS	262	3.37	1.18	0.14	0	8.73	9.57
Flexible	Non-NHS	124	9.33	3.21	0.68	0	9.84	11.67
Flexible	Non-NHS	178	4.16	3.49	0.53	0	5.27	8.87
Flexible	Non-NHS	125	1.92	2.79	0.70	0	10.55	7.46
Rigid	Interstate	204	2.34	44.53	7.18	0	5.07	3.87
Rigid	Interstate	380	3.65	81.62	6.78	0	3.37	3.04
Rigid	Interstate	211	3.61	119.34	16.35	0	3.00	3.45
Rigid	Interstate	192	2.34	39.60	6.78	0	4.18	4.01
Rigid	Interstate	126	3.61	71.60	16.35	0	1.43	2.54
Rigid	Interstate	71	1.14	14.08	6.43	0	6.20	5.12
Rigid	Interstate	238	1.14	50.70	6.95	0	3.28	3.72
Rigid	Interstate	380	3.65	81.62	6.78	0	3.46	3.04
Rigid	Interstate	204	2.34	44.53	7.18	0	2.78	3.87
Rigid	Interstate	238	1.14	50.70	6.95	0	6.13	3.72
Rigid	NHS	77	1.19	9.87	4.51	0	9.60	9.06
Rigid	NHS	173	2.16	2.85	0.60	1	9.86	9.23
Rigid	NHS	77	1.19	6.24	2.85	0	5.13	7.29
Rigid	NHS	77	1.19	6.24	2.85	0	5.80	7.29
Rigid	NHS	77	1.19	9.87	4.51	0	6.42	9.06
Rigid	NHS	221	3.61	10.85	1.42	0	17.09	17.27
Rigid	NHS	77	1.19	6.24	2.85	0	6.19	7.29
Rigid	NHS	77	1.19	9.87	4.51	0	7.42	9.06
Rigid	NHS	344	2.48	48.60	4.44	0	8.29	7.81
Rigid	NHS	77	1.19	6.24	2.85	0	4.33	7.29
Rigid	Non-NHS	48	1.14	0.41	0.28	0	8.90	11.66
Rigid	Non-NHS	196	7.94	15.26	2.09	0	3.65	5.84
Rigid	Non-NHS	249	2.41	3.61	0.43	0	9.19	8.80
Rigid	Non-NHS	249	2.41	3.61	0.43	0	9.50	8.80
Rigid	Non-NHS	249	2.41	3.61	0.43	0	10.38	8.80
Rigid	Non-NHS	356	2.91	28.11	2.33	0	3.34	5.03
Rigid	Non-NHS	30	2.73	0.31	0.28	0	9.46	11.91
Rigid	Non-NHS	356	2.91	28.11	2.33	0	5.37	5.03
Rigid	Non-NHS	22	4.04	0.70	0.96	1	9.26	7.27
Rigid	Non-NHS	356	2.91	28.11	2.33	0	4.18	5.03

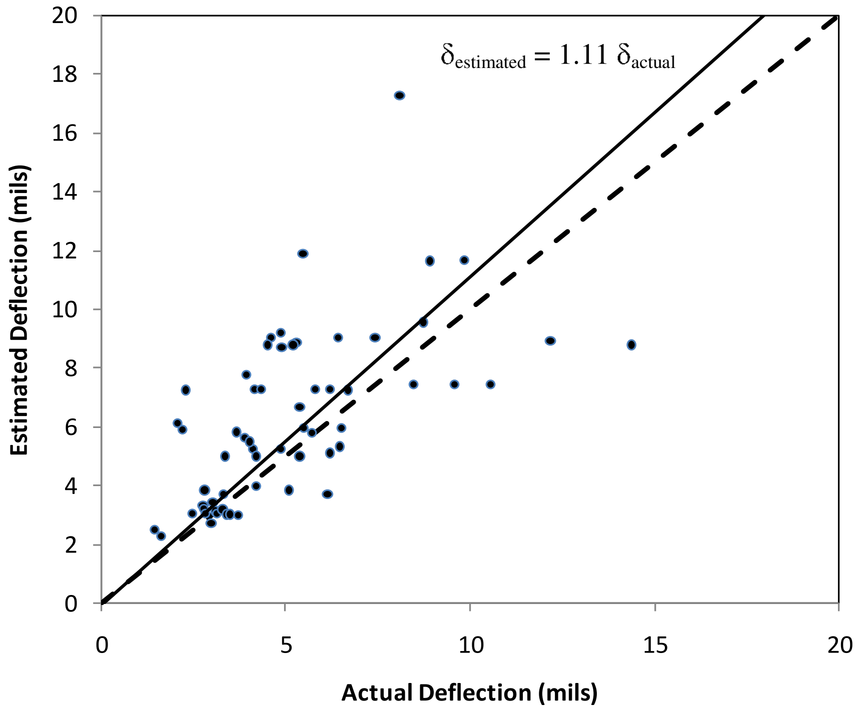


Figure 6.2: Comparison between Estimated and Actual Deflections

Figure 6.2 shows the comparison between the estimated and actual deflections. It is observed that the model tends to overestimate center deflection δ_1 by about 11%. This is acceptable for practical pavement management purposes since the models provide conservative estimates of the deflections and hence pavement strength.

Table 6.4 also demonstrates the validity of the framework proposed in Chapter 3. SSIs can be evaluated with and without actual FWD deflections without great loss of practicality. If FWD tests are performed on a given segment in a given year, the actual SSI can be calculated using the SSI models developed in Chapter 5. In the event where there is no data in a given year, the developed models in this chapter can be applied to estimate the FWD deflections and the SSIs. Thus highway agencies will be able to consider pavement structural performance in addition to the current pavement functional performance when making annual network-level pavement management decisions.

6.4 Chapter Summary

This chapter has presented the development of pavement deflection prediction models that can be used for network level pavement management. A statistical approach was adopted to relate FWD deflection with parameters such as cumulative truck traffic, temperature variation, truck count and rural indicator (proxies for pavement thickness design). An illustration is also provided in the chapter to demonstrate how the deflection models can be used to determine pavement structural strength index. The variables used in the models can be easily obtained by highway agencies annually and the models essentially can be used for the prediction of pavement deflections in the absence of FWD tests or pavement coring. This can greatly enhance the completeness of the pavement structural condition database that is crucial in network level pavement management.

CHAPTER 7. CONCLUSIONS

7.1 Main Findings

Pavement structural capacity assessment is an essential component for pavement management at both the network level and the project level. At the network level, this essential component is needed to determine the minimum funding for meeting the goal of an overall acceptable pavement structural condition for the network. At the project level, it is needed to guide the pavement manager to identify the appropriate pavement treatment (ranging between doing nothing and complete reconstruction).

At both levels of management, there is an increasing trend towards decision making in a multiple criteria context. A key aspect of the multiple criteria decision making is that the criteria must be amenable to scaling so that they can be incorporated in the overall utility function. Almost all SHAs measure the in-situ structural condition of their pavements using the falling weight deflectometer (FWD), whose output is the pavement surface deflection in mils (1/1000 inches).

This report presents an index to assess the structural condition of a pavement using deflection data from FWD measurements as part of a multiple criteria decision-making at the network level. Pavement deflection measurements were scaled to a structural strength indicator (SSI) ranging from 0 to 100 (excellent structural condition). The scale selected was logistic in shape and was developed on the basis of cumulative distribution. The use of the cumulative probability distribution was merely to establish the performance upon which a logistic functional form was specified. Any other method could have been used to establish the reference data, such as expert opinion or monetary equivalents of each condition level. As such, the developed index is not to be viewed as being tied to a probability distribution or interpreted as such. Thus, the developed index could be used across time and space, very much like the pavement serviceability rating PSR. Using the relationship between this index and deflection number, the value of structural condition for a given pavement section can be established given the deflection in mils. A modeling framework to predict pavement deflection values and estimate the SSI values given its functional class, surface age, and soil drainage conditions is also developed.

7.2 Recommendations for Future Research

In answering the questions posed at the beginning of this thesis, a wide range of research directions can be pursued in the future. The most important effort would be to investigate the stability of the SSI function developed for Indiana and its transferability to other states. This can be done by collecting deflection data from other highway agencies and ascertaining what calibration parameters would be needed to enhance the transferability of the models.

With regard to SSI prediction methods for use at pavement sections lacking deflection data, it is critical that these models are as reliable as possible. If this is not done, SSIs could be estimated incorrectly leading to denial of structural preservation treatments when they are due, or application of such treatments long before they are actually due at a given pavement section. In this regard, future work could include refinement of these models in a variety of ways. First, the assumption of linearity of the explanatory variables, particularly, surface age, is unduly restrictive and could be relaxed. This is because it is of interest to examine and track the true relationship between pavement structural condition and age, and there is reason to believe that there not only is a very gradual deterioration in pavement structural condition over time, but also there is likely to be a non-linear shape of this relationship. Secondly, the variables that surrogate pavement thickness could be replaced by the true pavement thicknesses derived from coring tests and/or GPR measurements. This would obviate the problems of endogeneity engendered by the use of surrogate variables such as traffic loading or functional class. Third, better prediction models with a greater fit to actual deflection measurements have to be developed for each pavement family with particular attention to flexible pavements.

Furthermore, future research could establish decision matrices on the basis of a reasonable number of factors and also at high levels of granularity for each factor. Such matrices would recommend a set of appropriate alternative structural treatments for a given level of structural condition, traffic loading, soil drainage conditions, etc.

Finally, future research could examine the various institutional mechanisms by which pavement structural condition (in terms of the developed index) could be incorporated into the matrix of performance measures for pavement as well as overall highway asset management. The present thesis, has strived to help in setting the stage for efforts to be carried out at the next level.

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