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Guide for Mechanistic-Empirical Design

OF NEW AND REHABILITATED PAVEMENT STRUCTURES

FINAL REPORT

PART 2. DESIGN INPUTS

CHAPTER 5. EVALUATION OF EXISTING PAVEMENTS FOR REHABILITATION

NCHRP

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DISCLAIMER

PART 2—DESIGN INPUTS

CHAPTER 5
EVALUATION OF EXISTING PAVEMENTS FOR REHABILITATION

This chapter provides an introduction to project-level pavement evaluation, guidance for data collection, and an overall condition assessment and problem definition for existing flexible and rigid pavements.

2.5.1 INTRODUCTION

Reliable and cost-effective design of a rehabilitation project requires the collection and detailed analysis of key data from the existing pavement. Such data are often categorized as follows:

- Traffic lane pavement condition (e.g., distress, smoothness, surface friction, and deflections).
- Shoulder pavement condition.
- Past maintenance activities.
- Pavement design features (e.g., layer thicknesses, shoulder type, joint spacing, and lane width).
- Geometric design features.
- Layer material and subgrade soil properties.
- Traffic volumes and loadings.
- Climate.
- Miscellaneous factors (e.g., utilities and clearances).

This chapter provides procedures and guidance for performing project-level evaluation of pavement structures for use in rehabilitation type selection (see PART3, Chapter 5) and in rehabilitation design (see PART 3, Chapters 6 and 7). It also provides guidance for determining those inputs that are considered essential for the different types of rehabilitation design. The project-level evaluation program incorporated into this Guide covers three common pavement types—flexible, rigid, and composite. It discusses the procedures used for pavement evaluation and the types of input data required for both assessing existing pavement condition and designing recommended rehabilitation alternatives.

2.5.1.1 Major Aspects of Project-Level Pavement Evaluation

Project-level data evaluation usually consists of a detailed analysis of all aspects of pavement condition, resulting in the identification of specific problems and their causes. The data types required for analysis range from simple data, such as the pavement design features and pavement geometrics, to detailed data obtained from destructive testing (e.g., asphalt concrete [AC] dynamic modulus and portland cement concrete [PCC] elastic modulus), nondestructive testing (e.g., deflection testing), and drainage surveys.

Overall pavement condition and problem definition can be determined by evaluating the following major aspects of the existing pavement:

- Structural adequacy (load related).
- Functional adequacy (user related).
- Subsurface drainage adequacy.
- Material durability.
- Shoulder condition.
- Extent of maintenance activities performed in the past.
- Variation of pavement condition or performance within a project.
- Miscellaneous constraints (e.g., bridge and lateral clearance and traffic control restrictions).

The structural category relates to those properties and features that define the response of the pavement to traffic loads; the data will be used in mechanistic-empirical design of rehabilitation alternatives. The functional category relates to the surface and subsurface characteristics and properties that define the smoothness of the roadway, or to those surface characteristics that define the frictional resistance or other safety characteristics of the pavement's surface.

Subsurface drainage and material durability may affect both structural and functional condition. Shoulder condition is very important in terms of rehabilitation type selection and in affecting project cost. Variation within a project refers to areas where there is a significant variability in pavement condition. Such variation may occur along the length of the project, between lanes (truck lane versus other lanes), among cut and fill portions of the roadway, and at bridge approaches, interchanges, or intersections.

Miscellaneous factors, such as joint condition for jointed concrete pavements and reflection cracking for composite pavements, are important to the overall condition such pavements but should be evaluated only where relevant.

Lastly, project-level pavement evaluation cannot be complete unless all possible constraints that may be encountered during rehabilitation (such as the availability of adequate bridge clearance for placing overlays and traffic control restrictions) are documented. This is a very important consideration in selecting feasible rehabilitation alternatives and for life cycle cost analysis (LCCA).

2.5.1.2 Definition of Project-Level Pavement Evaluation

This chapter provides overall guidance for identifying the types and root causes of distress on existing pavements. It also provides information on the data required for providing a cost-effective rehabilitation design for defective pavements. As shown in figure 2.5.1, the pavement evaluation and rehabilitation selection and design process can be subdivided into three phases (*1*). For all three phases, considerable amounts of analysis and engineering judgment should be applied to define the problems of existing pavements and to develop cost-effective solutions. A detailed description of phase I (evaluation) is presented in this chapter; phase II (selection and preliminary design) is described in Chapters 5, 6, and 7 in PART 3 of this Guide. Phase III (LCCA) is described in Appendix C.

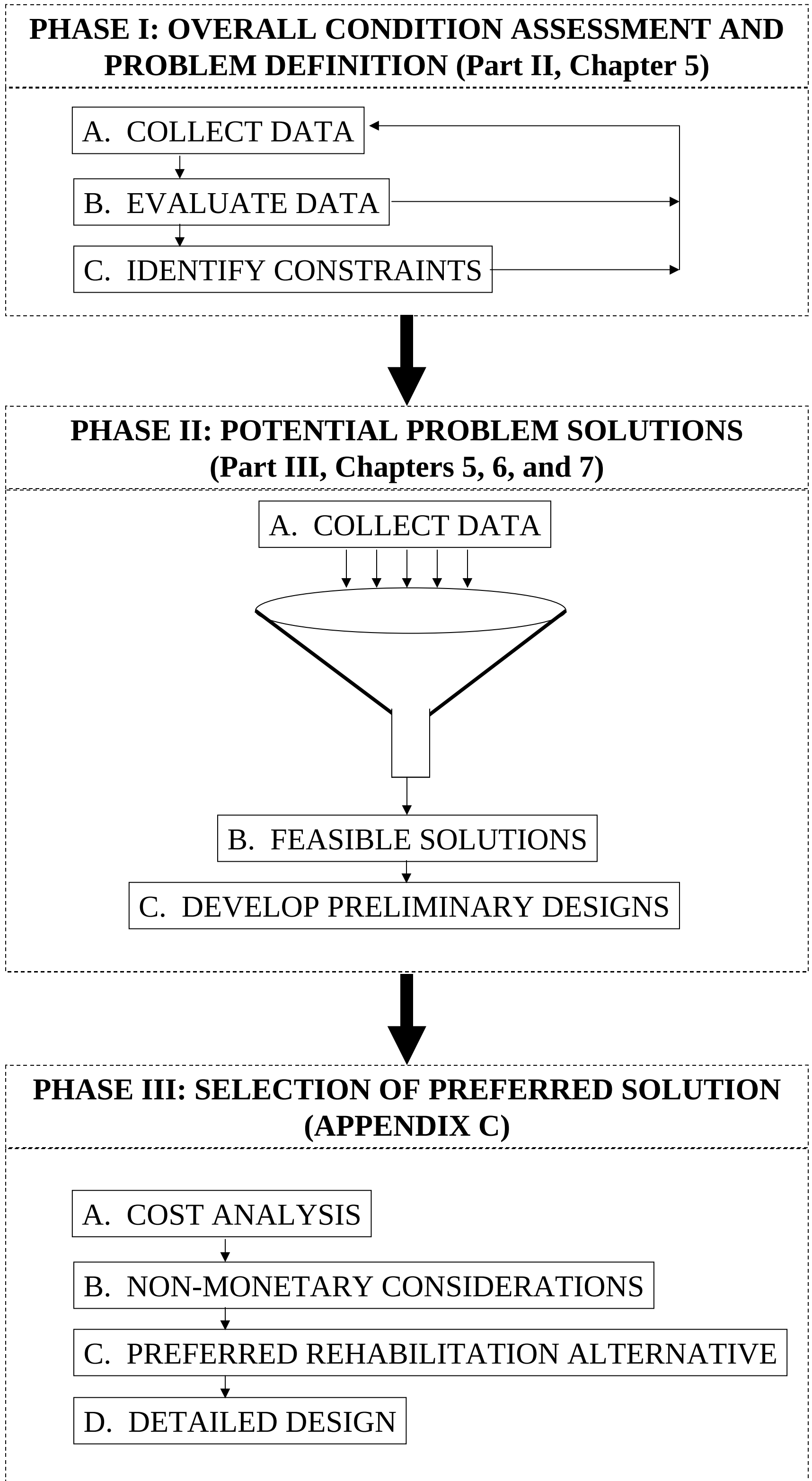


Figure 2.5.1. The pavement rehabilitation selection process (*1*).

Phase I—Overall Condition Assessment and Problem Definition (Evaluation)

The first step in the pavement rehabilitation selection process involves assessing the overall condition of the existing pavement and fully defining the existing pavement problems. To avoid making an inaccurate assessment of the problem, the engineer should collect and evaluate sufficient information about the pavement. High-speed nondestructive testing data such as ground penetration radar (GPR) and profile testing should be considered to assist in making decisions related to timing of the improvement and additional data collection efforts needed.

Table 2.5.1 contains a comprehensive checklist of factors designed to hone in on the problems that should be addressed. This list should be modified to suit the project's specific needs. It is vital that the agencies develop procedures and guidelines for answering the questions on their list. Information on many of the factors in table 2.5.1 can be obtained from the agency's existing pavement management system; however, depending on how regularly data are collected and how recent the latest data are, there may be the need to supplement the pavement management data with more current field survey and testing data. The data to be collected and the steps for determining an assessment of the pavement's current structural or functional condition are (*2*):

1. Historic data collection (records review).
2. First field survey.
3. First data evaluation and determination of additional data requirements.
4. Second field survey.
5. Laboratory characterization.
6. Second data evaluation.
7. Final field evaluation report.

2.5.1.3 Level of Data Collection

Data required for overall condition assessment and problem definition can be collected in a variety of ways that are categorized as Level 1, 2, or 3. These levels are defined according to source and reliability, similar to the definition of inputs in PART 2, Chapters 1 through 4. Descriptions of the different levels of data are summarized in section 2.5.2 of this chapter. The premature failure of many rehabilitated pavements can be traced to inadequate problem definition. Therefore, it is important that the evaluation process be implemented adequately to ensure reliable results.

2.5.1.4 Field Evaluation Plan

The design engineer should prepare an evaluation plan that outlines all activities required for investigating and determining the causes of pavement defects and for selecting and designing an appropriate repair strategy for those defects. A well-planned pavement evaluation process should also be within the resources of the agency, and it should address the traffic control requirements for rehabilitation alternatives.

Table 2.5.1. Checklist of factors used in overall pavement condition assessment and problem definition.

Facet	Factors	Description
Structural adequacy	Existing distress	1. Little or no load/fatigue-related distress 2. Moderate load/fatigue-related distress (possible deficiency in load-carrying capacity) 3. Major load/fatigue-related distress (obvious deficiency in current load-carrying capacity) 4. Load-carrying capacity deficiency: (yes or no)
	Nondestructive testing (deflection testing)	1. High deflections 2. Are backcalculated layer moduli reasonable? 3. Are joint load transfer efficiencies reasonable?
	Nondestructive testing (GPR testing)	1. Determine layer thickness
	Nondestructive testing (profile testing)	1. Determine joint/crack faulting
	Destructive testing	1. Are cores strengths and condition reasonable? 2. Are the layer thicknesses adequate?
	Previous maintenance performed	Minor, Normal, Major
	Has lack of maintenance contributed to structural deterioration?	Yes, No, Describe__________________
Functional adequacy	Smoothness Cause of smoothness deficiency	Measurement _______________ Very Good, Good, Fair, Poor, Very Poor Foundation movement Localized distress or deterioration Other
	Noise	Measurement _______________ Satisfactory, Questionable, Unsatisfactory
	Friction resistance	Measurement _______________ Satisfactory, Questionable, Unsatisfactory
Subsurface drainage	Climate (moisture and temperature region)	Moisture throughout the year • Seasonal moisture • Very little moisture • Deep frost penetration • Freeze-thaw cycles • No frost problems
	Presence of moisture-accelerated distress	Yes, Possible, No
	Subsurface drainage facilities	Satisfactory, Marginal, Unsatisfactory
	Surface drainage facilities	Satisfactory, Marginal, Unsatisfactory
	Has lack of maintenance contributed to deterioration of drainage facilities?	Yes, No, Describe__________________
Materials durability	Presence of durability-related distress (surface layer)	1. Little or no durability-related distress 2. Moderate durability-related distress 3. Major durability-related distress
	Base erosion or stripping	1. Little or no base erosion or stripping 2. Moderate base erosion or stripping 3. Major base erosion or stripping
	Nondestructive testing (GPR testing)	1. Determine areas with material deterioration/moisture damage (stripping)

Table 2.5.1. Checklist of factors used in overall pavement condition assessment and problem definition (continued).

Facet	Features	Description
Shoulder adequacy	Surface condition	1. Little or no load-associated/joint distress 2. Moderate load-associated/joint distress 3. Major load-associated/joint distress 4. Structural load-carrying capacity deficiency: (yes or no)
	Localized deteriorated areas	Yes, No Location:
Condition/ performance variability	Does the project section include significant deterioration of the following: • Bridge approaches • Intersections • Lane to lane • Cuts or fills	 Yes, No Yes, No Yes, No Yes, No
	Is there a systematic variation in pavement condition along project (localized variation)?	Yes, No
	Systematic lane to lane variation in pavement condition	Yes, No
Miscellaneous	PCC joint damage: • Is there adequate load transfer (transverse joints)? • Is there adequate load transfer (centerline joint)? • Is there excessive centerline joint width? • Is there adequate load transfer (lane-shoulder)? • Is there joint seal damage? • Is there excessive joint spalling (transverse)? • Is there excessive joint spalling (longitudinal)? • Has there been any blowups?	 Yes, No Yes, No Yes, No Yes, No Yes, No Yes, No Yes, No Yes, No
	Past maintenance • Patching • Joint resealing	 Yes, No Yes, No
	Traffic capacity and geometrics • Current capacity • Future capacity • Widening required now	 Adequate, Inadequate Adequate, Inadequate Yes, No
Constraints?	Are detours available for rehabilitation construction?	Yes, No
	Should construction be accomplished under traffic?	Yes, No
	Can construction be done during off-peak hours?	Yes, No, Describe________________
	Bridge clearance problems	Describe________________
	Lateral obstruction problems	Describe________________
	Utilities problems	Describe________________
	Other constraint problems	Describe________________

Results from the field evaluation plan should help to identify specific details of the project that may have a significant effect on the performance of the repair strategy or rehabilitation design. A sample step-by-step procedure for collecting and evaluating existing pavements is outlined in table 2.5.2 and explained in the paragraphs that follow.

Table 2.5.2. Field data collection and evaluation plan.

Step	Title	Description
1	Historic data collection	This step involves the collection of information such as location of the project, year constructed, year and type of major maintenance, pavement design features, materials and soils properties, traffic, climate conditions, and any available performance data.
2	First field survey	This step involves conducting a windshield and detailed distress survey of sampled areas within the project to assess the pavement condition. Data required includes distress information, drainage conditions, subjective smoothness, traffic control options, and safety considerations. Detailed procedures for collecting pavement distress/condition data are given in section 2.5.2.4.
3	First data evaluation and the determination of additional data requirements	Determine critical levels of distress/smoothness and the causes of distress and smoothness loss using information collected during the first field survey. This list will aid in assessing preliminarily existing pavement condition and potential problems. Additional data needs will also be assessed during this step.
4	Second field survey	This step involves conducting detailed measuring and testing such as coring and sampling, profile (smoothness) measurement, skid resistance measurement, deflection testing, drainage tests, and measuring vertical clearances.
5	Laboratory testing of samples	This step involves conducting tests such as material strength, resilient modulus, permeability, moisture content, composition, density, and gradations, using samples obtained from the second field survey.
6	Second data evaluation	This involves the determination of existing pavement condition and an overall problem definition. Condition will be assessed and the overall problem defined by assessing the structural, functional, and subsurface drainage adequacy of the existing pavement. Condition assessment and overall problem definition also involve determining material durability, shoulder condition, variability in pavement condition along project, and potential constraints. Additional data requirements for designing rehabilitation alternatives will also be determined during this step.
7	Final field and office data compilation	Preparation of a final evaluation report.

Steps 1 and 2: Historic Data Collection and First Field Survey

Regardless of the level of input data adopted for pavement evaluation, the field collection and evaluation process and problem definition phase of pavement rehabilitation should begin with an assembly of historic data and preferably some benchmark data. This information may be obtained from a windshield field survey of the entire project followed by a detailed survey of selected areas of the project (steps 1 and 2 of the field evaluation plan presented in table 2.5.2).

Steps 1 and 2 of the field collection and evaluation plan should, as a minimum, fulfill all the data requirements to perform an overall problem definition. The following activities should be performed:

- Review construction and maintenance files to recover and extract information and data pertinent to pavement performance and response.

- Review previous distress surveys and the pavement management records, if available, to establish performance trends and deterioration rates.
- Review previous deflection surveys.
- Review previous pavement borings and laboratory test results of pavement materials and subgrade soils.
- Perform a windshield survey or an initial surveillance of the roadway's surface, drainage features, and other related items.
- Identify roadway segments with similar or different surface and subsurface features using the idealized approach (discussed in the next section of this chapter). In other words, isolate each unique factor that will influence pavement performance.
- Identify the field testing/materials sampling requirements for each segment and the associated traffic control requirements.
- Determine if the pavement performed better or worse than similar designs.

The information gathered in this step can be used to divide the pavement into units with similar design features, site conditions, and performance characteristics for a more detailed pavement evaluation. Also, because limited time and funds are allotted to this portion of the evaluation process, each agency should develop a standard data collection/evaluation procedure that best suits its information, personnel, and equipment resources. The information gathered in this phase can only be used to detect any inadequacies that should be rectified during rehabilitation. It is not sufficient for designing rehabilitation alternatives such as a structural overlay, retrofit subdrainage systems, or for reconstruction of the entire project.

<u>Step 3: First Data Evaluation and Determination of Additional Data Requirements</u>

Using the information and data gathered in steps 1 and 2, a preliminarily overall pavement condition analysis can be performed. Also, the information gathered can be used to determine if the specific project is a candidate for pavement preservation options. This is achieved by using the data obtained to evaluate the following major aspects of the existing pavement:

- Structural adequacy.
- Functional adequacy including foundation movement.
- Subsurface drainage adequacy.
- Material durability.
- Shoulder condition.
- Variation of pavement condition.
- Miscellaneous constraints.

If any of these aspects is inadequate, then more detailed data will be required to determine the extent and severity of the problem and for use in the design of all feasible rehabilitation alternatives. Step 3 is very important since it helps agencies reduce considerably the list of additional data requirements, making the overall pavement assessment and problem definition process more cost-effective.

Step 4 and 5: Second Field Survey and Laboratory Characterization

Steps 4 and 5 involve conducting detailed measuring and testing, such as coring and sampling, smoothness measurement, deflection testing, skid resistance measurement, drainage tests, and measuring vertical clearances on the project under evaluation. The data collected at this stage should be guided by the data needs determined at the end of the first evaluation phase in step 3. Steps 4 and 5 will also involve conducting tests such as material strength, resilient modulus, permeability, moisture content, composition, density, and gradations, using samples obtained from the second field survey. Field data collection, laboratory characterization, and data manipulation should be done according to established guidelines from test standards such as AASHTO, ASTM, LTPP, SHRP, and State and local highway agencies. Section 2.5.2 of this chapter presents a detailed description of the methods used for data collection.

Step 6 and 7: Second Data Evaluation and Final Field Evaluation Report

Using the data collected during steps 1 through 5, the final pavement evaluation and overall problem definition can be conducted. The data required for determining feasible rehabilitation alternatives (outlined in PART 3, Chapter 5) will also be prepared during this step. Step 7 documents the details of the pavement evaluation process, the data obtained specifying levels of input, and problems identified in a final evaluation report. All of this information will be utilized in the design of rehabilitation strategies as described in PART 3, Chapters 6 and 7.

2.5.2 GUIDES FOR DATA COLLECTION

2.5.2.1 Overview

One of the most critical aspects of pavement evaluation is the collection of reliable data on the existing pavement facility. This is because all major decisions regarding existing pavement problems and feasible rehabilitation alternatives depend on the accuracy and integrity of the data assembled. Several types of data are collected for the pavement evaluation process, ranging from easy-to-obtain data, such as the pavement geometric features and structure, to the more complex material properties and pavement response variables, such as resilient modulus and deflections obtained through testing.

Figure 2.5.2 shows a timeline of when various data types are collected. Any data collected before pavement evaluation, regardless of type, is *historic*. It includes site-, design-, and construction-related data assembled from inventory, monitoring, and maintenance data tables established throughout the pavement life. Data collected during pavement evaluation, such as visual surveys, nondestructive, and destructive testing are described as *benchmark* data. The source of data for pavement evaluation and rehabilitation design is important because it relates to the reliability of the entire process. For example, layer thickness information collected during the evaluation process through coring is described as benchmark data. The same data obtained from the files containing test data collected during construction is described as historic. A successful and thorough pavement evaluation program will require both benchmark and historic data, since some data by definition will always remain historic (e.g., traffic). However, in situations where the data can be obtained from both sources, benchmark data will tend to be more reliable. The following are brief summaries of the typical data types used in pavement evaluation.

Historic Data

Historic data basically consists of inventory data and monitoring data, as shown in figure 2.5.2. The constituents of historic data are described in the following sections.

Inventory Data

Inventory data basically consist of data necessary to identify the project under evaluation. This consists of the geometric details of a project and describes the design features and material properties of the structural constituents of the pavement.

All of these data remain constant up to the time of evaluation (with the exception of material properties and climate that change with time) unless the pavement undergoes significant maintenance or repairs. Usually, inventory data are obtained from the project's as-designed or as-constructed drawings, plans, and records. Inventory data are not always reliable due to potential differences in as-designed and as-constructed design plans and pavement properties and the general inaccuracies associated with such data. Inventory data should be used for pavement characterization only when there are no other alternatives.

Monitoring Data

Monitoring data include distress, surface friction, longitudinal profile measurements, nondestructive testing such as deflection testing, and destructive testing that consists of coring, sampling, and laboratory characterization. Typically, these data are collected on a periodic basis to provide a historic database for monitoring pavement performance. Monitoring data also include the past traffic estimates (measured in terms of the axle type, load, and frequency) and a detailed list of all significant maintenance activities that have been performed on the pavement since construction.

Benchmark Data

Benchmark data consist of data obtained through visual surveys and testing of the pavement or samples retrieved from the pavement.

2.5.2.2 Data Required for Overall Condition Assessment and Problem Definition

Data required for overall condition assessment and problem definition can be categorized from Levels 1 to 3, according to source and reliability. Descriptions of the different levels of data are summarized in table 2.5.3. Table 2.5.3 shows that various kinds of information (whether historic or benchmark) are required for a comprehensive pavement evaluation. The information required can be obtained directly from the agency's historic data tables (inventory or monitoring tables) or by conducting visual surveys, performing nondestructive testing, and performing destructive testing as part of pavement evaluation.

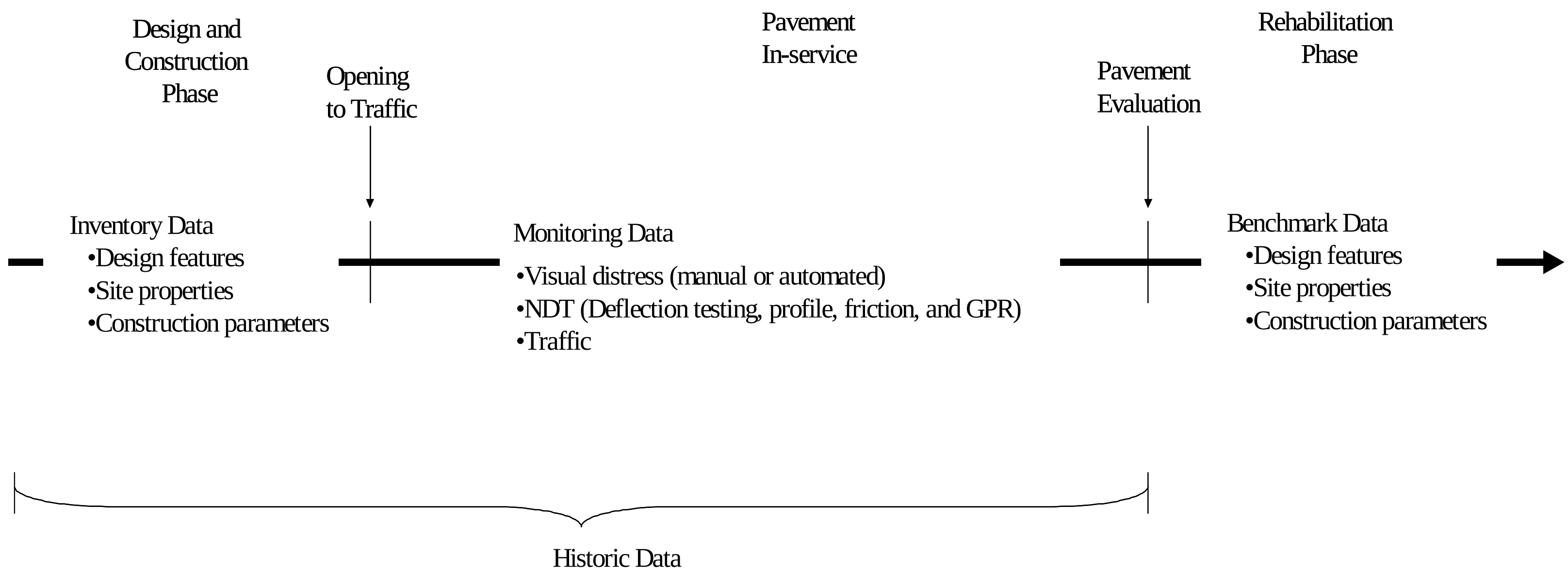


Figure 2.5.2. Timeline for data acquisition.

Table 2.5.3. Definition of input levels for pavement evaluation.

Features	Factor	Level of Data		
		1	2	3
Structural adequacy	Load-related distress	50 to 100 percent visual survey of the entire project	10 to 50 percent visual survey of entire project	Windshield survey of entire project
	Nondestructive testing (deflection testing)	Perform NDT at intervals less than 500 ft along project	Perform NDT at intervals greater than 500 ft along project	Use historic data or perform limited NDT at selected locations along project
	Nondestructive testing (GPR testing)			
	Nondestructive testing (profile testing)			
	Destructive testing (coring, DCP)	Perform coring at intervals less than 2000 ft along project	Perform coring at intervals greater than 2000 ft along project	Use historic data or perform limited coring at selected locations along project
	Maintenance data	Historic data and visual survey	Historic data	Historic data
Functional evaluation	Nondestructive testing (profile testing)—IRI	Perform testing along entire project	Perform testing along selected sample units within project	Use historic data (pavement management data)
	Nondestructive testing (friction testing)—FN	Perform testing along entire project	Perform testing along selected sample units within project	Use historic data (pavement management data)
Subsurface drainage	Climate data	Refer to Chapter 3 in PART 2 of this Design Guide		
	Moisture-related distress	100 percent drainage survey of the entire project	100 percent drainage survey of sample area along project	Windshield survey of entire project
	Signs of moisture-accelerated damage	100 percent drainage survey of the entire project	100 percent drainage survey of sample area along project	Windshield survey of entire project
	Condition of subsurface drainage facilities	100 percent drainage survey of the entire project	100 percent drainage survey of sample area along project	Windshield drainage of entire project
	Condition of surface drainage facilities	100 percent drainage survey of the entire project	100 percent drainage survey of sample area along project	Windshield survey of entire project
Materials durability	Durability-related surface distress	100 percent visual survey of the entire project	100 percent visual survey of sample area along project	Windshield survey of entire project
	Base condition (erosion or stripping) or contamination	Perform testing every 50 ft along project	Perform testing every 500 ft along project	Use historic data or perform limited testing at selected locations along project

Table 2.5.3. Definition of input levels for pavement evaluation, continued.

Features	Factors	Level of Data		
		1	2	3
Shoulder	Surface condition (distress and joint)	100 percent visual survey of the entire project	100 percent visual survey of selected sample units within project	Windshield survey of entire project
Variability along project	Identification of areas of likely variability and condition of such areas	100 percent survey* of the entire project	100 percent survey* of selected sample units within project	Windshield survey* of entire project
Miscellaneous	PCC joint condition	100 percent visual survey of the entire project	100 percent visual survey of selected sample units within project	Windshield survey of entire project
	Traffic capacity and geometrics	100 percent visual survey of the entire project	100 percent visual survey of selected sample units within project	Windshield survey of entire project
Constraints	Are detours available?	100 percent visual survey of the entire project	100 percent visual survey of selected sample units within project	Windshield survey of entire project
	Should construction be accomplished under traffic	100 percent visual survey of the entire project	100 percent visual survey of selected sample units within project	Windshield survey of entire project
	Can construction be done during off-peak hours	100 percent visual survey of the entire project	100 percent visual survey of selected sample units within project	Windshield survey of entire project
	Bridge clearance problems	100 percent visual survey of all bridges in entire project area	100 percent visual survey of selected sample units within project	Windshield survey of bridges in entire project area
	Lateral obstruction problems	100 percent visual survey of the entire project	100 percent visual survey of selected sample units within project	Windshield survey of entire project
	Utilities problems	100 percent visual survey of the entire project	100 percent visual survey of selected sample units within project	Windshield survey of entire project

* All relevant surveys (e.g., visual, drainage). Levels 1 and 2 typically are benchmark data while level 3 consists of a limited form of benchmark data obtained from windshield surveys and historic data.

The activities performed as part of assembling historic data from inventory or monitoring data files include a review of past construction and maintenance data files to recover and extract information and data pertinent to pavement design features, material properties, and construction parameters, borings logs, and laboratory testing of layer materials and subgrade soils.

The review should also include past pavement management records for information on past distress surveys and maintenance activities. A thorough review of past records could also yield information on pavement constraints such as bridge clearances and lateral obstruction. Two kinds of information that should be assembled as part of the historic data are traffic and climate-related data.

The traffic data required include past and future traffic estimates that are required as input for determining current and future pavement structural adequacy. PART 2, Chapter 4 of this Guide provides guidance on the traffic-related data required and how they can be obtained. Climate variables such as precipitation and freeze-thaw cycles may also be required as inputs for rehabilitation design and structural adequacy analysis. PART 2, Chapter 3 of this Guide provides guidance on the climate-related data required and how they can be obtained.

Visual Surveys

Visual surveys range from a casual windshield survey conducted from a moving vehicle to the more detailed survey that involves trained engineers and technicians walking the entire length of the project (or selected sample areas) and measuring and mapping out all distresses identified on the pavement surface, shoulders, and drainage systems (*2*). Recently, automated visual survey techniques have become more common and are being adopted for distress surveys and pavement condition evaluation. Several methods are available to measure and quantify distress.

In most cases, the raw data collected during the survey needs to be transformed for use in pavement evaluation and analysis (e.g., converting the number of low-, medium-, and high-severity transverse cracking in a 500-ft sampled section into percentage of slabs cracked). Because of the vast differences in visual data collection methods, the procedures used for transforming them for use in pavement evaluation will not be discussed in this Guide. Users should, however, ensure that their data are transformed and compatible with the distress quantities used in this Guide.

Nondestructive Testing Data

Nondestructive testing (NDT) is a term used to describe the examination of pavement structure and materials properties through means that do not induce damage or property changes to the structure. NDT ranges from simple techniques such as using GPR to determine in-situ layer thickness and condition, profile testing to determine pavement surface smoothness, friction testing to determine pavement surface-vehicle tire skid resistance, through to the well-established method of deflection testing, using a Falling Weight Deflectometer (FWD) (*3*).

Though the most widely used forms of NDT are deflection, profile, and friction testing, other forms of NDT (such as GPR) are becoming state-of-the-art technologies. NDT typically has the following advantages (*1*,*3*):

- Reduces the occurrence of accidents due to lane closures.
- Reduces costs.
- Improves testing reliability.
- Provides vital information for selecting between rehabilitation options.
- Provides data for rehabilitation (overlay) design.

A key disadvantage to some NDT, such as deflection testing, is lane closures.

Destructive Testing

Destructive tests require the physical removal or damage of pavement layer material to obtain a sample (either disturbed or undisturbed) for laboratory characterization or to conduct an in-situ dynamic cone penetrometer (DCP) test. Destructive testing ranges from simple tests such as coring (and determining the pavement layer thicknesses by measuring core lengths) to performing dynamic modulus testing on retrieved AC cores or determining the elastic modulus and strength of PCC cores. Other forms of destructive testing that are less common are:

- Trenching of hot mix AC pavements to determine material condition and permanent deformation.
- Lifting of slabs of jointed concrete pavements (JCP) to determine subsurface material conditions.

Trenching consists of cutting a full depth, 4- to 6-in-wide strip of pavement, full width of a traffic lane, and removing it to observe the condition of the different pavement layers over time. If rutting is present, it allows the engineer to determine where the rutting is located and the cause of rutting (consolidation or plastic flow). Trenching also allows the engineer to determine if and where stripping-susceptible asphalt layers lie in the pavement section. Destructive tests such as trenching generally help improve evaluation of the causes of surface distresses.

Destructive testing has many limitations, particularly when conducted on moderate to heavily trafficked highway systems (e.g., risk to testing personnel). Practical restraints—in terms of time and money—severely limit the number and variety of destructive tests conducted on routine pavement evaluation studies (*1, 3*). Destructive testing also has some vital advantages, including the observation of subsurface conditions of pavements layers and bonding between layers.

Destructive testing could also include the milling of an AC overlay in an AC/PCC composite pavement to make it possible to visually examine the joint area of the PCC for deterioration.

2.5.2.3 Establishing Fundamental Analysis Segments

The first step in pavement evaluation involves dividing the pavement project into segments possessing similar design features, site properties, or pavement conditions. Segmentation can also be done for data collection purposes. Some tests require specific minimum lengths of measurement, such as a 0.1-mile length for smoothness testing. Figure 2.5.3 shows a typical project divided into five segments. The five segments were selected based on different cross-sectional designs and traffic levels.

The figure shows that although the surface layer has uniform thickness, the base is made up of different materials; therefore, a difference in performance can be expected. Segments with different conditions or performance characteristics will likely have different problems and should therefore be evaluated separately. Analysis segments can also be identified by evaluating pavement response to testing (e.g., defection testing, surface profile).

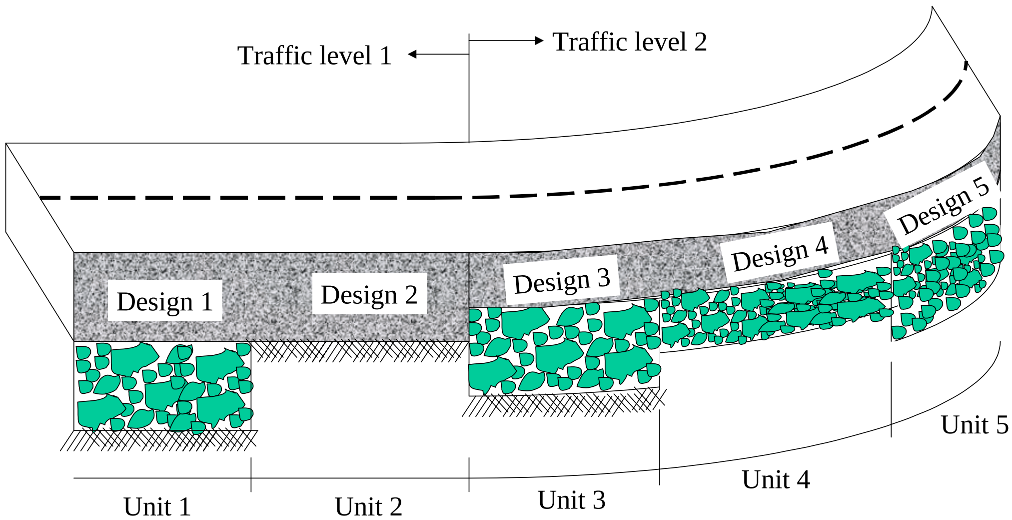


Figure 2.5.3. Example of pavement project segmented into five units with similar properties for evaluation.

Normally, data for this type of segment differentiation are obtained from plans, specifications, and other kinds of historic data. Data from high-speed GPR testing and profilometers for determining smoothness are becoming increasingly helpful in establishing fundamental analysis segments (26, 27). Figure 2.5.4 illustrates the typical plot of a pavement response variable as a function of distance along the highway segment.

Typically, there is considerable variability in measurements of response variables such as smoothness with distance. Nonetheless, it is still possible to observe points of significant change in response because at such points the mean of the response variable of segments on either side will be noticeably different. For instance, for the variable response shown in figure 2.5.4, four separate units can be identified for evaluation.

The variations in pavement design, site, and construction properties for the pavement project shown in figure 2.5.3 and the variations in pavement response variables for the pavement project shown in figure 2.5.4 are called the “between-unit variability.” Between-unit variability reflects the fact that statistically homogeneous units may exist within a given rehabilitation project and may be used in segmenting the pavements for evaluation, condition assessment, and problem definition, and eventually for rehabilitation design. The ability to delineate the general boundary locations of these units is critical in pavement evaluation and rehabilitation because these units form the basis for the specific evaluation and rehabilitation alternatives to be conducted.

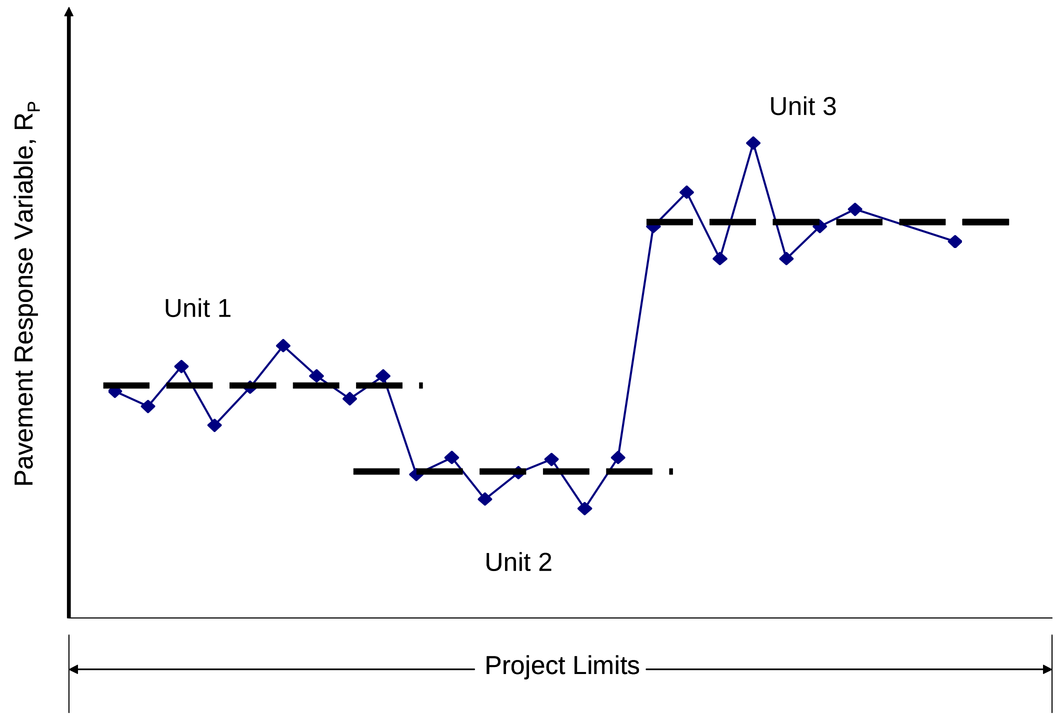


Figure 2.5.4. Typical plot of a pavement response variable as a function of distance along the highway segment.

If unit delineation is not done carefully, the possibility of gross inefficiencies in pavement evaluation and the determination of rehabilitation strategy will increase considerably, and the units identified could be either under-designed (i.e., premature failure) or over-designed (uneconomical use of materials). As noted earlier, the units identified should reflect all statistically homogeneous units that may exist within the given rehabilitation project.

There are several methods available for dividing a given project under evaluation into smaller and more manageable units based on the between-unit variability within the project. Two common approaches are the "idealized approach" and the "measured pavement response approach," discussed and described in the following paragraphs (*1*).

<u>Idealized Approach</u>

To delineate a pavement length, the engineer should isolate each unique factor influencing potential pavement performance. These factors include:

- Pavement type.
- Differences in construction history (including rehabilitation and major maintenance).
- Differences in pavement cross section (including layer material type and thickness).
- Differences in subgrade type and foundation support.
- Differences in past and future traffic.

- Differences in pavement condition such as the levels of smoothness and distress. Evaluation should include lane-to-lane variability in pavement condition.

Under ideal circumstances, the engineer will use a historic pavement database to evaluate these factors. Figure 2.5.5 illustrates how the factors listed above are used to determine analysis units that are characterized by a unique combination of pavement performance factors. The validity of the final units is directly related to the accuracy of the historic pavement information available.

If accurate records have been kept, the idealized approach has more merit in delineating unique units than a procedure that relies on current observations of pavement surface distress because changes in one or more design factors (which indicate points of delineation) are not always evident through observation of pavement surface distress.

When delineating pavement analysis units, the most difficult factor to assess (without measurement) is the subgrade (foundation) factor. While records may indicate a uniform soil subgrade, the realities of cut and fill earthwork operations, variable compactive effort, drainage, topographic positions, and groundwater table positions often alter the in-situ response of subgrades, even along a "uniform soil type." The depth to bedrock should also be considered in delineating pavement analysis units since it has a significant effect on pavement foundation strength and support.

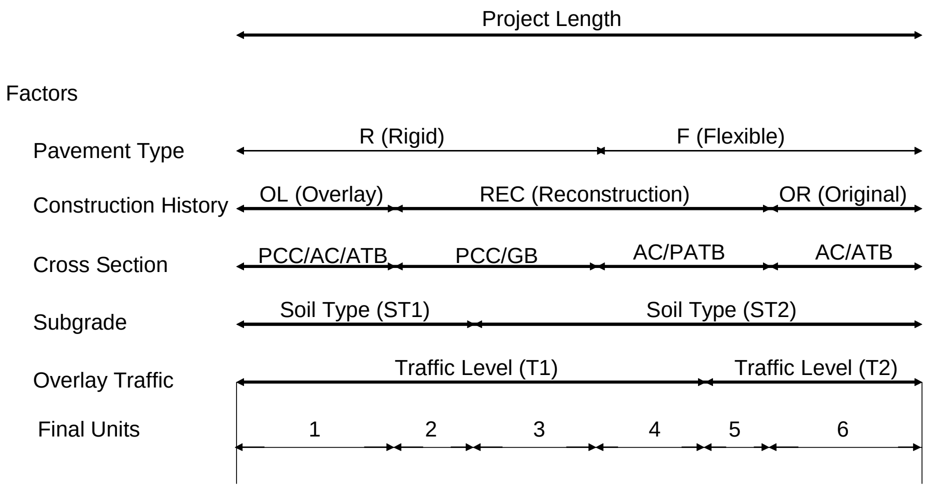
Frequently, the engineer cannot accurately determine the practical extent of the performance factors noted and should rely upon the analysis of a measured pavement response variable (e.g., deflection, smoothness, and distress) for unit delineation. This is done by developing a plot of the measured response variable versus distance along the project.

Figure 2.5.6 shows an example of such a plot. While this example uses smoothness as the pavement response variable, the procedure is identical for any other type of pavement response variable selected.

Once the plot of a pavement response variable has been generated, it may be used to delineate units with uniform condition through several methods. The simplest of these is visual examination to determine where relatively unique units occur. Several analytical methods are also available to help delineate units, such as the recommended "cumulative difference" procedure.

<u>Measured Pavement Response Approach</u>

The cumulative difference procedure is based on the simple mathematical fact that when the variable Z_c (defined as the difference between the area under the response curve at any distance, X, and the total area developed from the overall project average response at the same distance) is plotted as a function of distance along the project, unit boundaries occur at the location where the slopes of the plot Z_c versus X change sign. Figure 2.5.7 is an example plot of the cumulative difference variable (Z_c) for the data shown in figure 2.5.6. For this example, 11 preliminary analysis units can be outlined clearly.



Unit No.	**1**	**2**	**3**	**4**	**5**	**6**
Pavement type	Rigid	Rigid	Rigid	Flexible	Flexible	Flexible
Construction history	Overlay	Reconstruction	Reconstruction	Reconstruction	Reconstruction	Original
Cross section	PCC/AC/ATB	PCC/GB	PCC/GB	AC/PATB	AC/PATB	AC/ATB
Subgrade	ST1	ST1	ST2	ST2	ST2	ST2
Traffic	TL1	TL1	TL1	TL1	TL2	TL2

Figure 2.5.5. Idealized method for analysis of unit delineation.

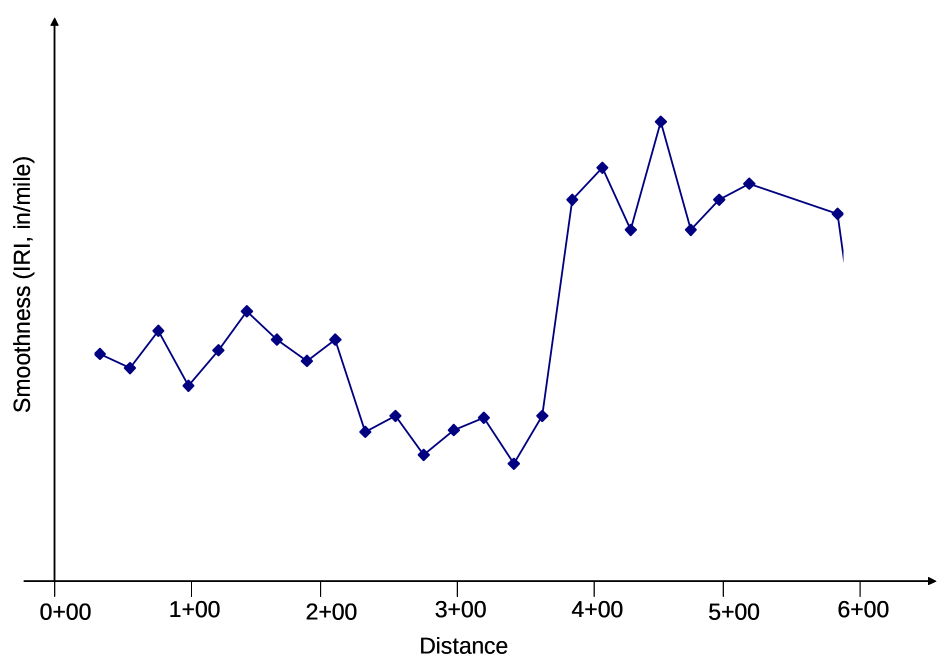


Figure 2.5.6. Smoothness results versus distance along a project.

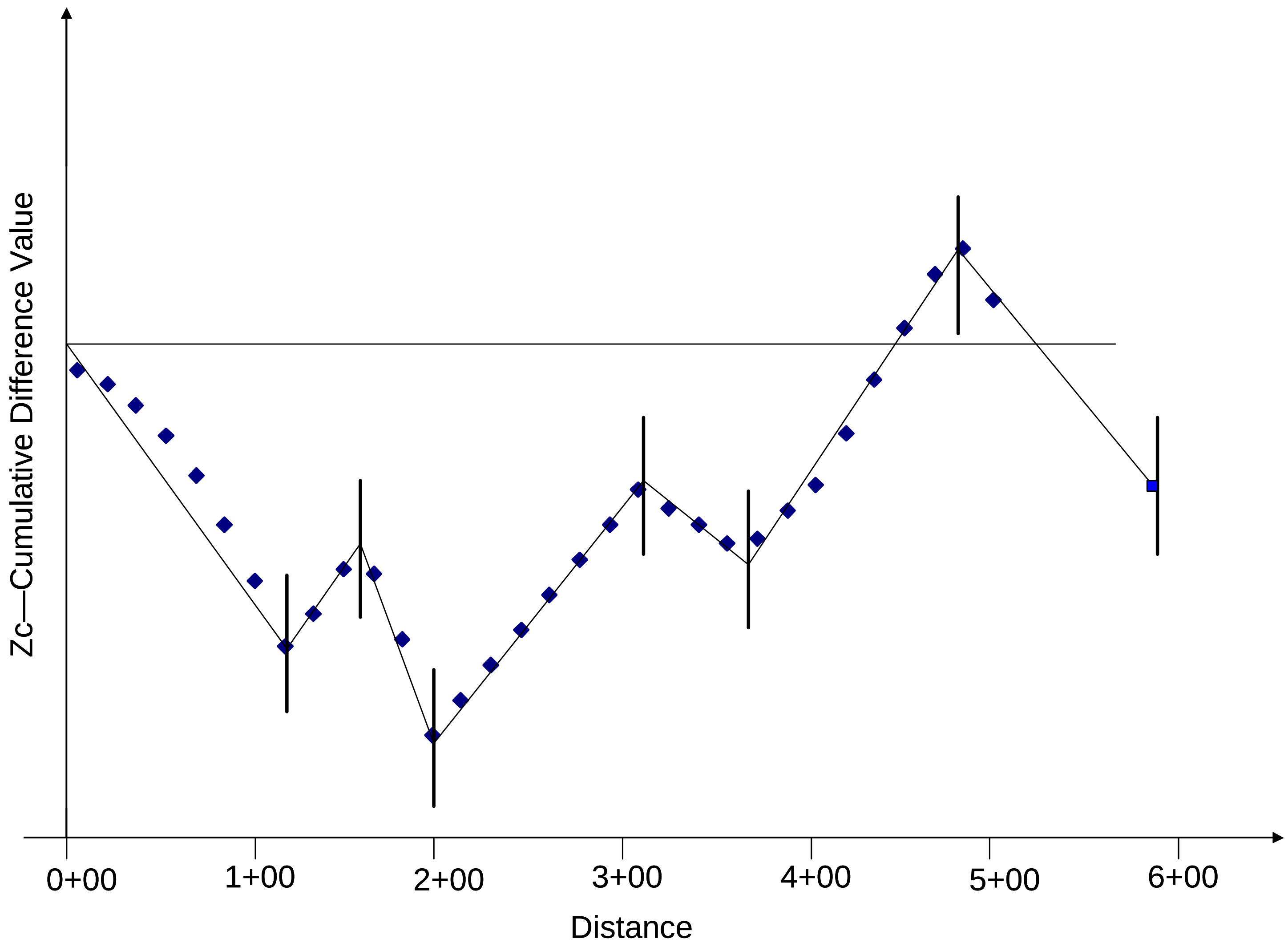


Figure 2.5.7. Delineating analysis units by cumulative difference approach.

Engineers can further evaluate the identified units to determine whether two or more can be combined for practical construction considerations and economic reasons. The combination of units should be done relative to the sensitivity of the mean response values for each unit on performance of future rehabilitation designs.

After delineating the project in units for evaluation, the inherent diversity of the required variables (e.g., layer thickness) within each unit should be assessed because it is an important source of variability required for rehabilitation design. This source of variability is called ”within-unit variability.” Within-unit variability is important because it is used to characterize the variability of the pavement properties within a unit for use in designing rehabilitation alternatives. It is also used in characterizing variability within the unit for determining the reliability of alternative rehabilitation designs.

For both the idealized and measured pavement response approaches, the segments should be of practical length for construction. Localized deteriorated areas should be specifically addressed within any segment so it does not reoccur and cause failure after rehabilitation.

2.5.2.4 Distress Survey

A key input required for the determination of feasible rehabilitation alternatives is pavement condition. Although pavement condition is defined in different ways by different agencies, it almost always requires the identification of several distress types, severities, and amounts through on-site visual survey. Manual distress surveys, automated distress surveys, photologs, and low-level aerial photographs can all be used in specific situations to aid in economically collecting data for determining pavement condition. The use of automated techniques could significantly reduce the time of data collection and the time from data collection to decision making and the start of rehabilitation which is the time critical for minimizing damage of distressed pavements by heavy loads. This section describes the types of distresses common to flexible, rigid, and composite pavements and procedures for measuring them.

<u>General Background</u>

Accurate condition surveys, which assess a pavement’s physical distress, are vital to a successful evaluation effort because condition survey results form the basis adequacy, as presented in section 2.5.3 in this chapter. Thus, an intensive survey is highly recommended before any detailed pavement evaluation—pavement condition assessment and problem definition—is attempted.

While engineers accept the necessity for condition or distress surveys in broad terms, specific methodologies for such surveys vary from agency to agency. Each agency should develop a survey approach consistent with its use of the data generated, as well as its available manpower and financial resources.

Minimum Information Needs

When pavement condition surveys are conducted, certain information should be available if the engineer is to make knowledgeable decisions regarding pavement condition assessment and problem definition and, hence, rehabilitation needs and strategies. The following data are required for pavement evaluation:

1. Type—Identify types of physical distress existing in the pavement. The distress types should be placed in categories according to their causal mechanisms.
2. Severity—Note level of severity for each distress type present to assess degree of deterioration.
3. Quantity—Denote relative area (percentage of the lane area or length) affected by each combination of distress type and severity.

A detailed visual distress survey should address each of the requirements listed above. Although the parameters of each category may vary from agency to agency, the procedures used in conducting such surveys are similar (manual or automated) and can be adapted or modified to suit local conditions. For this Design Guide, distress identification for flexible, rigid, and composite pavements will be based on the *Distress Identification Manual for the Long-Term Pavement Performance Project* (*4*). Figures 2.5.8 through 2.5.13 show examples of the distress types of interest at various levels of severity.

Although the *Distress Identification Manual for the Long-Term Pavement Performance Project* was developed as a tool for the LTPP program, the manual has broader applications and provides a common language for describing cracks, potholes, rutting, spalling, and other pavement distresses required for pavement evaluation. The manual is divided into three sections, each focusing on a particular type of pavement: AC-surfaced, JCP, and continuously reinforced concrete pavement (CRCP) (*4*).

Figure 2.5.8. Example of transverse joint faulting in jointed plain concrete pavement.

Figure 2.5.9. Example of transverse cracking in jointed plain concrete pavement.

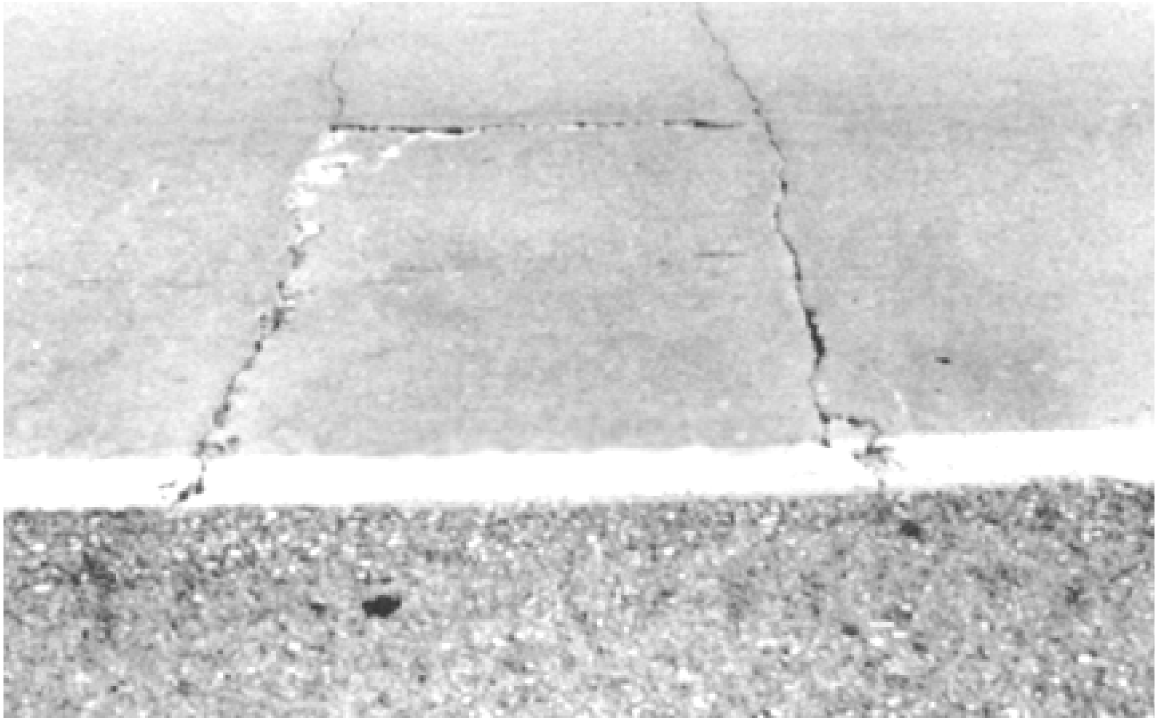


Figure 2.5.10. Example of punchout in continuously reinforced concrete pavement.

Figure 2.5.11. Example of fatigue cracking in hot mix AC pavement.

Figure 2.5.12. Example of permanent deformation in hot mix AC pavement.

Figure 2.5.13. Example of longitudinal cracking in hot mix AC pavement.

Tables 2.5.4 through 2.5.6 present a summary of common distress types for flexible, jointed concrete, and continuously reinforced concrete pavements (*3, 4, 5, 6, 7, 8*).

Pavement Conditions of Concern

When conducting visual surveys, engineers should note all distress types present especially the distress types critical for pavement condition assessment and problem definition. The following sections briefly describe the distress types and severities of concern for flexible, rigid, and composite pavement (*1, 5 ,6, 7, 8*).

Flexible Pavements

For flexible pavements, the distress types that are critical include those that have a detrimental impact to the ride quality of the pavement surface and can cause accelerated deterioration of the pavement structure. Flexible pavements with one or more of the following conditions will continue to deteriorate and cause further accelerated distress after rehabilitation, if adequate remedial measures are not taken:

- Flexible pavements with high deflections and brittle hot mix asphalt surfaces (high modulus, low tensile strain at failure).
- Weak surface or saturated unbound aggregate base/subbase materials and/or subgrade soils.
- Expansive soils subjected to seasonal variations in moisture.

Table 2.5.4. General categorization of flexible and composite pavement distress.

General Description	Distress Type[1, 2, 3]	Major Contributing Factors
Cracking	Fatigue Cracking	Load
	Long. Cracking (wheelpath)	Load
	Reflection Cracking	Load, materials, climate, construction
	Transverse Cracking	Materials, climate
	Block Cracking	Materials, climate, construction
Surface deformation	Rutting	Load, materials
	Shoving	Load
Surface defects	Raveling	Materials, climate, construction
	Bleeding	Materials, climate, construction
Miscellaneous distress	Lane-to-Shoulder Drop-off	Materials, climate, construction
	Pumping	Load, materials, climate, construction
Patching and potholes	Patch Deterioration	Load, materials, climate, construction
	Potholes	Load

1. Note that the severity of the distresses listed is typically aggravated by harsh climatic conditions and a lack of adequate drainage.
2. Most of the distresses listed influence the functionality of the pavement which is typically characterized by smoothness and surface friction.
3. Some distresses such as reflection cracking and rutting have multiple causes.

Table 2.5.5. General categorization of JCP distress.

General Description	Distress Type[1, 2, 3]	Major Contributing Factors
Cracking	Corner breaks	Load
	Longitudinal cracking	Load, materials, climate, construction
	Transverse cracking	Load
Joint deficiencies	Transverse joint seal damage	Materials, climate, construction
	Longitudinal joint seal damage	Materials, climate, construction
	Spalling of longitudinal joints	Materials, climate, construction
	Spalling of transverse joints	Materials, climate, construction
PCC durability	Durability cracking	Materials, climate, construction
	ASR	Materials, climate, construction
Surface defects	Map cracking	Materials, climate, construction
	Scaling	Materials, climate, construction
	Polished aggregate	Materials, climate, construction
	Popouts	Materials, climate, construction
Miscellaneous distress	Blowups	Materials, climate, construction
	Faulting of transverse joints and cracks	Load, materials, climate, construction
	Lane-to-shoulder dropoff	Materials, climate, construction
	Lane-to-shoulder separation	Materials, climate, construction
	Patch deterioration	Load, materials, climate, construction
	Water bleeding and pumping	Load, materials, climate, construction

1. Note that the severity of the distresses listed is typically aggravated by harsh climatic conditions and a lack of adequate drainage.
2. Most of the distresses listed influence the functionality of the pavement which is typically characterized by smoothness and surface friction.
3. Some distresses such as longitudinal cracking have multiple causes.

Table 2.5.6. General categorization of CRCP distress.

General Description	Distress Type[1, 2, 3]	Primarily Materials, Climate, or Construction Related
Cracking	Punchouts	Load, materials, climate, construction
	Longitudinal cracking	Load, materials, climate, construction
	Transverse cracking	Materials, climate, construction
Joint deficiencies	Longitudinal joint seal damage	Materials, climate, construction
	Spalling of longitudinal joints	Materials, climate, construction
PCC durability	Durability cracking	Materials, climate, construction
	ASR	Materials, climate, construction
Surface defects	Map cracking	Materials, climate, construction
	Scaling	Materials, climate, construction
	Polished aggregate	Materials, climate, construction
	Popouts	Materials, climate, construction
Miscellaneous distress	Blowups	Materials, climate, construction
	Lane-to-Shoulder dropoff	Materials, climate, construction
	Lane-to-Shoulder separation	Materials, climate, construction
	Patch deterioration	Load, materials, climate, construction
	Water bleeding and pumping	Load, materials, climate, construction

1. Note that the severity of the distresses listed is typically aggravated by harsh climatic conditions and a lack of adequate drainage.
2. Most of the distresses listed influence the functionality of the pavement which is typically characterized by smoothness and surface friction.
3. Some distresses such as longitudinal cracking have multiple causes.

- Frost susceptible soils subjected to freezing temperatures.
- Hot mix asphalt surface and bases that are susceptible to moisture damage and stripping.
- Brittle hot mix asphalt mixtures (low adhesion or strength) that have severe block, transverse, or longitudinal cracking.
- Hot mix asphalt surface mixtures with severe rutting or surface distortion.

Rigid Pavements

For jointed concrete pavements, the distress types that are critical are those associated with the joints and cracks. Pavements with deteriorated joints create localized areas of weakness that cannot be bridged by flexible overlays (full-depth repairs are required before rehabilitation with overlays). Such localized areas of weakness may be addressed with minimal repairs if overlaid by a separated (unbonded) PCC layer. Also, joints with poor load transfer and that have deteriorated badly will continue to deteriorate and can cause further distress after rehabilitation if adequate remedial measures are not taken.

The conditions of existing CRCP that are critical include deteriorated and working transverse cracks, steel rupture, punchouts, pumping, expansion joints, durability problems, and the amount of previous repairs. Because of the closely spaced cracks in CRCP, it is a relatively flexible pavement; therefore, adequate foundation support is critical, and the condition survey should examine any localized conditions that indicate the lack of support. Tables 2.5.7 and 2.5.8 summarize the key distress types and conditions that are of concern for flexible and rigid pavements (3, 5, 6, 7, 8).

Table 2.5.7. Conditions of concern for existing flexible pavements.

Factor	Possible Condition	
Fatigue Cracking	Interconnected cracks forming a complete pattern	Over 45 percent of the wheel path area; crack deterioration around edges
		Severely deteriorated cracks over 20 percent of wheel path area; pieces move under traffic
Longitudinal Cracks in Wheel Path	Longitudinal Cracks in or adjacent to wheel path	Mean crack width exceeds 0.25 in and has some deterioration in at least one wheel path over 25 percent of the length.
Reflection Cracking	Transverse or longitudinal cracks that have a mean width that exceeds 0.25 in with some crack deterioration	
Transverse Cracking	Crack spacing is less than 100 ft	Mean crack width exceeds 0.25 in with crack deterioration
Block Cracking	Transverse and longitudinal cracks that form a grid pattern	Mean crack width exceeds 0.25 in and has some crack deterioration
Rutting	Average depth between both wheel paths exceeds 0.5 in	
Shoving	Depression-swells exceed 25 percent of wheel path area	Stripping in HMA Layer
		High asphalt content mixtures
Raveling	Surface is rough and pitted, loose of coarse aggregate over more than 50 percent of the lane area	Moisture sensitive HMA mixture
		Thin film thickness (low asphalt content)
Bleeding	Loss of surface texture over more than 25 percent of the wheel path area	Stripping in HMA mixtures
		High asphalt content mixtures
Pumping	Pumping along construction joints or cracks along lane	Saturated aggregate base materials or soils
Lane-to-Shoulder Drop-off	Average difference in elevation of the traffic lane and shoulder exceeds 0.8 in	
Depressions/Swells, Longitudinal Profile	HMA pavements supported by expansive soils that are subjected to seasonal variations in moisture content	
	HMA pavements supported by frost-susceptible soils that have insufficient cover to prevent frost penetration into the soil	
Surface Transverse Profile	Rutting	Greater than 0.5 in
	Shoving	More than 25 percent of wheel path area
Patch/Patch Deterioration	Loss of material around the edges of the patch and/or pumping around the edges	
Potholes	Depth of the pothole exceeds 1 in with a frequency of more than 2 per mile	
Uniformity of Support	Localized area with high deflections or frequent abrupt changes in deflection magnitude and basin curvature	
Deflection Softening Structural Response	Deflection increase with increasing load at an accelerated rate	

Table 2.5.8. Conditions of concern for existing rigid pavements.

Pavement Type	Factor	Possible condition
Jointed concrete pavement	Transverse joints	Poor load transfer (< 50 percent) and high deflections
		Presence of severe spalling/disintegration, pumping, faulting
		Working transverse joints and cracks (severely spalled or faulted that exhibit poor load transfer less than 50 percent)
		Corner breaks
	Longitudinal joints	Wide joint opening, corrosion of tie bar, poor load transfer, pumping, spalled/deteriorated
	PCC slab-cracking	Working transverse cracks[1], working longitudinal cracks, corner breaks, shattered slabs, movement or rocking of slabs when loaded
		Badly shattered slabs (four or more pieces that rock under load)
	PCC slab durability	Severe "D" cracking and reactive aggregates
	Drainage	Pumping and other evidence of poor subsurface drainage
	Uniformity of support	Relatively excessive deflections at certain locations
	Surface profile	Settlements, heaves, and loss of smoothness, and number of existing and new repairs prior to overlay per unit sampling area
	Existing expansion joints	Presence of exceptionally wide joints (greater than 1 in) or full-depth, full-lane-width AC patches per unit sampling area
	Overlay lane width	Same as overlay, or narrower than overlay, and is widening required
CRCP	Transverse cracks	Wide working cracks, steel rupture, poor crack load transfer, spalling/disintegration, pumping, faulting
	Longitudinal joints	Wide joint opening, corrosion of tie bar, poor load transfer, spalled/deteriorated
	Punchouts and punchout potential	Wide working closely-spaced transverse cracks, steel rupture, poor foundation support, and existing punchouts
	PCC slab durability	Severe "D" cracking, reactive aggregates
	Drainage	Pumping and other evidence of poor subsurface drainage
	Uniformity of support	Relatively excessive deflections at certain locations
	Surface profile	Settlements, heaves, loss of smoothness, and number of existing and new repairs prior to overlay per unit sampling area
	Existing expansion joints	Presence of exceptionally wide joints (greater than 1 in) or full-depth, full-lane-width AC patches per unit sampling area
	Potential overlay lane width	Same as overlay, or narrower than overlay, and is widening required

[1]Working cracks are spalled or faulted cracks that exhibit poor load transfer (<50 percent deflection transfer).

Composite Pavements

The key distress type of concern for composite pavements is reflection cracking. A deteriorated and working reflected transverse crack will lead to water infiltrating into the pavement system, ultimately leading to accelerated failure. The amount and state of reflected cracks should therefore be noted during distress surveys to aid in the pavement condition assessment and problem definition process (*3*).

Visual Distress Data Collection Methods

There are two main methods used in visual distress data collection, namely manual and automated.

Manual Surveys

Manual surveys consist of a visual walk through the pavement section by experienced field inspection crews. Data pertaining to specific distress types, severities, and quantities are collected by mapping their exact location on distress maps. The distresses are drawn on the map at the scaled location using symbols appropriate to the pavement type. Distresses that are not described in the Distress Identification Manual are usually photographed for future analysis.

Automated Surveys (Photologging)

Automated surveys gained popularity in the mid-1990s when State highway agencies began converting from manual to a fully or semi-automated process that utilizes video survey techniques for distress surveys and overall pavement condition evaluation. Reasons for moving to the automated process included:

- Increased safety (no survey crews parked on shoulders of busy highways).
- Less impact on the traveling public.
- Ability to automatically measure pavement rideability, rutting, and faulting.
- Increased uniformity of data collection and data processing activities.
- Warehousing of survey data on videotapes and databases.

Automated surveys typically consist of a specially modified vehicle that houses an extensive set of computers and sensors, including lasers, inertial measurement units, accelerometers, ultrasonic transducers, digital cameras and other advanced technology subsystems. The purpose of all this technology is to collect information and critical data about the pavement condition at a very fast pace by measuring and recording up to 36 different pavement surface characteristic—ranging from roughness and rutting to detailed asset inventories using multiple camera imagery—while traveling at posted speed limits. Data collected using this type of technology have been found to be comparable and compatible with standard manual survey techniques. A wide variety of data can be collected continuously at highway speeds, including:

- Longitudinal profile.
- Transverse profile/rutting.
- Grade, cross-slope.
- Pavement texture.
- Pavement distress.
- Global Positioning System (GPS) coordinates.
- Panoramic right-of-way video.

Once the videotape and sensor data are collected, the information is processed by trained and experienced personnel. As part of the data processing effort, the rating personnel identify predominant distress types, severities, and quantities. In addition, sensor data associated with pavement roughness, wheel path rutting, and joint faulting are processed using automated, computer-based techniques.

Summary

The distress evaluation provides valuable information for determining the causes of pavement deterioration, its condition, and eventually its rehabilitation needs. The distresses need to be classified according to the underlying cause (load, moisture, temperature/climate, materials, or a combination thereof). Pavement drainage should be evaluated closely. If moisture is accelerating pavement deterioration, the engineer should determine how the water is accelerating the deterioration, where it is coming from, and what can be done to prevent or minimize it.

2.5.2.5 Smoothness Measurements/Data

Pavement smoothness is widely regarded as a critical measure and indicator of pavement performance because it is the parameter that is most evident to the roadway user. Smoothness of a pavement's surface also has a significant effect on vehicle operating costs and safety. Smoothness is usually an index that quantifies the deviations on a pavement surface. The pavement surface is characterized by measuring its profile (*9*).

A profile is defined as a two-dimensional slice of the road surface, taken along an imaginary line. Profiles taken along a transverse line show the superelevation and cross slope surface, plus rutting and other damage. Longitudinal profiles show the design grade, smoothness, and texture. Pavement smoothness is characterized using the longitudinal profile. The use of the transverse profile to characterize rutting and other pavement distress is explained in the *LTPP Distress Identification Manual* (*4*).

<u>Definition of Smoothness</u>

Smoothness is defined as "the longitudinal deviations of a pavement surface from a true planar surface with characteristic dimensions that affect vehicle dynamics, ride quality, and dynamic pavement load" (*10*). It is also referred to as *ride quality* or *roughness*. Several indices are used to quantify the measured profile of a pavement, including Profile Index (PI), the Mays Roughness Index, and the International Roughness Index (IRI).

The lack of smoothness on a pavement's surface may be caused or exacerbated by several factors, particularly, the irregularities built into the pavement surface during construction, referred to as *initial smoothness*. The initial pavement smoothness generally deteriorates when the pavement is exposed to traffic and climate-related loads that cause the development of distress and other defects on the pavement surface. Some of the factors that cause smoothness loss include (*9, 10*):

- Localized pavement distress (i.e., depressions, potholes, and cracks).
- Traffic (which causes distresses such as corrugations of flexible pavements).
- Environmental processes, combined with pavement layer material properties such as poor drainage, swelling soils, freeze-thaw cycles, and non-uniform consolidation of subgrade, results in change in longitudinal profile.
- Warping and curling of long concrete slabs.

Smoothness Measuring Systems (Profilers)

There are many instruments and test methods used to measure the "true profile" for an imaginary line on the road. A profiler is an instrument used to produce a series of numbers related in a well-defined way to a true profile. Profilers do not always measure true profile, exactly. They measure the components of true profile that are needed for a specific purpose—computing smoothness (9, *11*). Profilers work by combining three measurements—a reference elevation, the height relative to the reference, and the longitudinal distance—in different ways based on the design of the profiler. Three types of equipment are commonly used in measuring pavement profiles—the rod and level (static profiler), Dipstick (static profiler), and the inertial profiler. Of these, the most common type of equipment used by State highway agencies is the inertial profiler, which is described in the following section (*9*).

Inertial Profiler

The inertial profiler combine the same three measurements required for pavement profilers (a reference elevation, the height relative to the reference, and the longitudinal distance) to measure the profile of a pavement (9). The inertial reference is provided by an accelerometer. Data processing algorithms convert the acceleration measure to an inertial reference that defines the instant height of the accelerometer in the host vehicle. The height of the ground relative to the reference is therefore the distance between the accelerometer (in the vehicle) and the ground directly under the accelerometer. This height is measured with a non-contacting sensor, such as a laser or ultrasonic transducer. The longitudinal distance of the instruments is usually picked up from the vehicle speedometer (9, *11*).

An inertial profiler not only works at highway speed, it requires a certain speed even to function. For example, even the best inertial profilers do not work well at speeds less than 10 mph. Locating the accelerometer and sensor over the proper imaginary line is difficult and requires an experienced driver (9, *11*).

Even though inertial profilers do not produce the same plot of profile as static profilers, they still provide high accuracy for smoothness indices calculated from pavement profiles. In general, smoothness indices computed from inertial profiles are more reliable than measures obtained statically because the inertial systems are mostly automated and eliminate the many potential sources of human error.

Also, with the development of more accurate sensors and faster computers, more reliable results are being obtained. For example, early systems performed the profile calculations electronically and required that the vehicle operate at constant forward speed. Modern inertial profilers correct for minor variations in speed and perform the calculations numerically with on-board computers. Instructions for using an inertial profiler are provided by the manufacturer. Figure 2.5.14 illustrates the components of an inertial profiler.

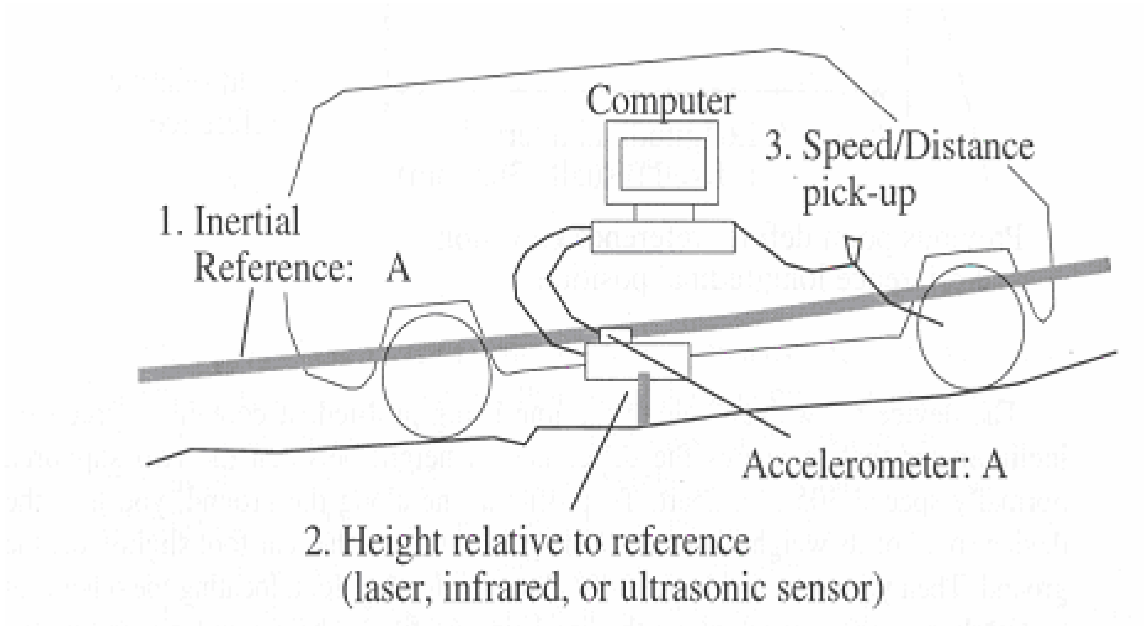


Figure 2.5.14. Components of an inertial profiler.

Smoothness Measuring Indices

Several smoothness indices are used for characterizing a pavement's smoothness. They are calculated with the following basic four-step approach:

1. Obtain the raw profile measurements of the pavement surface from a profiler.
2. Filter the raw profile data to eliminate wavelengths that are not of interest. Some analyses involve several filters applied in sequence.
3. Accumulate (or reduce) the filtered profile to a single index by accumulating the absolute values of the numbers, or accumulating the squared values. The result is a single cumulative number.
4. Convert the accumulated number to an appropriate scale. This involves dividing by the number of profile points or the length of the profile, to normalize the smoothness by the length covered. For example, many historical roughness indices have had units of inches/mile. A scale factor may be used to obtain standard units. A transformation equation may be used to convert from a profile-based scale to an arbitrary scale.

The main advantage of using profilers to determine smoothness is that it is flexible. Several statistics or profile indices can be obtained from the same profile and each statistic can potentially describe a different characteristic of the profile. For this Guide, the index used for characterizing pavement surface smoothness is IRI measured in in/mile.

IRI is generally reproducible, portable, and stable with time and is the first widely used profile index where the analysis method is intended to work with different types of profilers. Also, IRI is currently the most widely used smoothness index, and it is being adopted by several State agencies as the key indicator showing whether a pavement is functionally inadequate and should be rehabilitated (*3*).

The algorithm used for computing IRI is based on the quarter-car model. The quarter-car model is just as its name implies, a model of one corner (a quarter) of a car. It includes one tire, represented with a vertical spring, the mass of the axle supported by the tire, a suspension spring and a damper, and the mass of the body supported by the suspension for that tire. This quarter-car simulation is meant to be a theoretical representation of the response-type systems in use at the time the IRI was developed, with the vehicle properties of the "golden car" adjusted to obtain maximum correlation to the output of those systems. NCHRP Report 228 described the quarter-car model and the algorithm used in computing IRI (*12*). Figure 2.5.15 shows IRI ranges represented by different classes and conditions of highways.

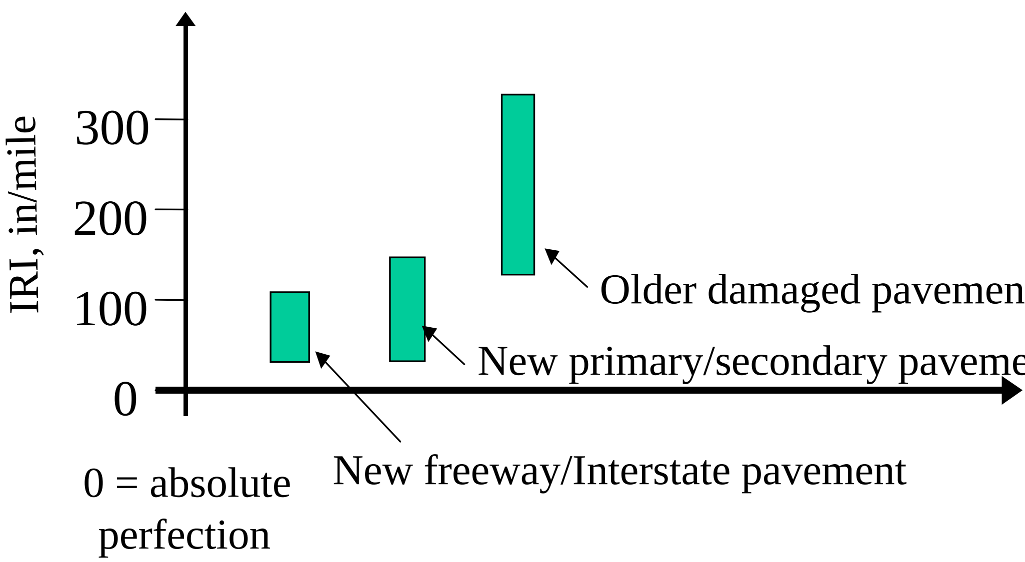


Figure 2.5.15. IRI ranges represented by different classes of road.

2.5.2.6 Surface Friction

Pavement surface friction is widely regarded as an indicator of safety of vehicles on highways because it is a measure of the force that resists sliding of vehicles tires on a pavement. Friction resistance is the force developed when a tire that is prevented from rotating slides along the surface of the pavement. ASTM E-867, "Standard Terminology Relating to Traveled Surface Characteristics," defines friction resistance as "the ability of the traveled surface to prevent the loss of traction" (*13*). Although friction resistance is often thought of as a pavement property, it is actually a property of both the pavement surface characteristics and the vehicle's tires.

Friction resistance is measured and reported by various highway agencies using different test methods. The most common among these are Friction Coefficient (μ), Skid Number (SN), Friction Number (FN), British Pendulum Number (BPN), and the International Friction Index (IFI) (*14*).

2.5.2.7 Drainage Survey

Distress in flexible, rigid, and composite pavements is often either caused or accelerated by the presence of moisture in the pavement structure. When evaluating the condition of existing pavements, engineers should investigate the role of drainage improvements in correcting declining pavement performance. It is also important to recognize when a pavement's distresses are not moisture-related and, therefore, cannot be remedied by drainage improvements (*1*).

A comprehensive drainage condition survey, an essential part of any pavement evaluation strategy, is presented in PART 3, Chapter 1 of this Guide. The survey has been designed to reveal moisture-related distresses that may be caused or accelerated by moisture in the pavement structure such as pumping, D-cracking, joint deterioration, faulting, and corner breaks. Further, the drainage survey also shows pavement damage due to freezing and subsequent thawing including differential frost heave and spring breakup (evidence of loss of support).

Results from the drainage survey are used later in this chapter to evaluate the effectiveness of the pavement's existing drainage facilities, define potential drainage related problems, and in PART 3, Chapter 5 of this Guide, to recommend feasible rehabilitation alternatives.

2.5.2.8 Nondestructive Testing

Several NDT systems are available for use in pavement evaluation. These systems or methods can be categorized by the pavement properties that are measured by the test equipment (*1, 3*):

- Pavement structural response—Deflection testing.
- Layer thickness and determination of pavement anomalies—Ground penetrating radar testing.
- Material elastic response and determination of material anomalies—Seismic analysis of surface waves (SASW), impact echo (I-E), and impulse response (I-R) testing.

The most widely used deflection testing devices are the impulse loading devices (e.g., FWD). These devices use velocity transducers or seismometers to measure pavement surface deflection and the deflection basin of the loaded pavement, making it possible to obtain the pavement's response to load and the resulting curvature under load. Another class of devices that measures pavement deflection is the rolling wheel deflectometer, but this is still under development (*15).*

Seismic testing of pavement systems is possible using the Spectral Analysis of Surface Waves (SASW) test method. The SASW methodology was initially developed under the Strategic Highway Research Program (SHRP) and has been greatly enhanced through the development of new equipment and computer programs. Seismic analysis involves determining travel time of Rayleigh waves through the tested medium and converting the observed response into material properties. Seismic testing is usually performed under small strain conditions, typically less than 0.001 percent. They have been shown to be useful in determining modulus values and thickness of surface layers. A schematic showing the seismic testing of pavement systems is presented in figure 2.5.16.

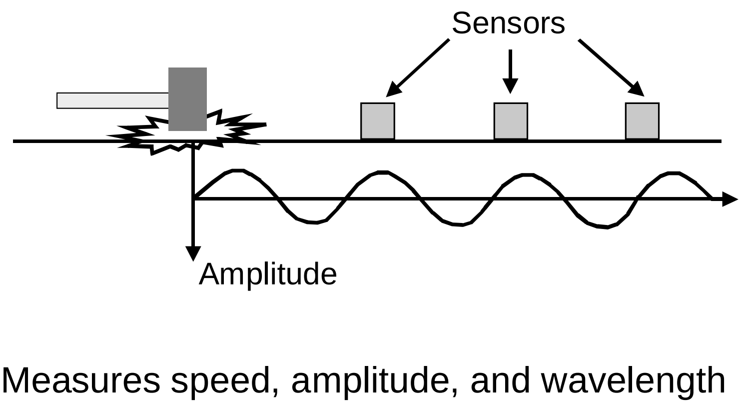


Figure 2.5.16. Schematic showing the seismic testing of pavement systems.

It is recommended that NDT data be used in conjunction with the information from distress surveys for effective interpretation of the data. NDT should be performed prior to performing destructive tests, such as coring and materials excavation, to better select the locations of such tests.

<u>Deflection Testing</u>

Nondestructive deflection testing has been an integral part of the structural evaluation and rehabilitation process for many decades. In its earliest applications, the total measured pavement deflection under a particular load arrangement was used as a direct indicator of structural capacity. Several agencies developed failure criteria that related the maximum measured deflection to the number of allowable load repetitions.

With the accumulation of knowledge and experience over the years, several algorithms have been developed for transforming deflections obtained through testing into pavement layer properties such as elastic modulus and Poisson's ratio using various kinds of backcalculation algorithms and software.

Deflection testing equipment operates by applying a load to the pavement system and measuring the resulting maximum surface deflection or the surface deflection basin. Deflection testing results are used to determine the following:

- Asphalt concrete pavements.
    - Elastic modulus of each of the structural layers (at non-distressed locations).
    - Structural adequacy (at non-distressed locations).
- Concrete pavements.
    - Concrete elastic modulus and subgrade modulus of reaction (center of slab).
    - Load transfer across joints (across transverse joints in wheelpath).
    - Void detection (at corners).
    - Structural adequacy (at non-distressed locations).

Equipment Type

There are different types of commercially available deflection testing devices. The devices are grouped based on loading mode—impulse, steady-state dynamic, and static. The impulse devices are the most recently developed and better simulate the load from a moving tire. A brief description of this type of device is presented in this section.

Impulse Type Deflection Testing Equipment

The most commonly used impulse type deflection testing equipment is the FWD. The FWD is a trailer-mounted device that delivers a transient force impulse to the pavement surface. The equipment uses a weight that is lifted to a given height on a guide system and is then dropped. The falling weight strikes a set of rubber buffers mounted to a 12-in circular foot plate, which transmits the force to the pavement. A thin ribbed rubber pad is always mounted under the foot plate.

By varying the mass or the drop height or both, the impulse load can be varied. This load may be varied between 2,500 lb to 27,000 lb for regular types of FWD. Seven deflection sensors measure the surface deflections caused by the impulse load. The first deflection sensor is always mounted in the center of the loading plate, while the rest are positioned at various spatial distances up to 6 ft from the load center. From all deflections recorded, peak values are stored and displayed. Load pulse base widths usually range from 20 ms to 60 ms for various equipment manufacturers.

FWD testing is commonly conducted at pavement temperatures between 40°F and +90°F. In special cases, such as when investigating the influence of frost on a pavement system, testing can be performed at temperatures below freezing. Testing at temperatures above +90°F is usually not recommended since viscous properties of the asphalt become predominant, whereas pavement structure modeling and analysis is generally based on elastic properties. When deflection measurements are taken on an asphalt concrete pavement, the results should be corrected (standardized) to a particular type of loading system and normalized to an arbitrarily defined set of climatic conditions. In general, measured deflections should be adjusted to a reference pavement temperature (usually 70 ^{o}F) to account for the effect of temperature on asphalt-treated materials modulus.

Also, because deflection testing is usually conducted at a particular time of the year (i.e., month or season) the backcalculated layer moduli (and modulus of subgrade reaction for PCC pavements) represent only that period in the year. The layer moduli should be transformed to account for the effect of changes in seasons (e.g., dry versus wet and freeze versus non-freeze). Procedures for adjusting moduli for the effect of climate are presented in PART 2, Chapter 3. The effect of seasonal moisture and temperature on pavement deflection is presented in figure 2.5.17.

The FWD load level used for testing AC and PCC pavements may affect the back calculated moduli obtained from the deflection data, particularly for unbound granular and subgrade

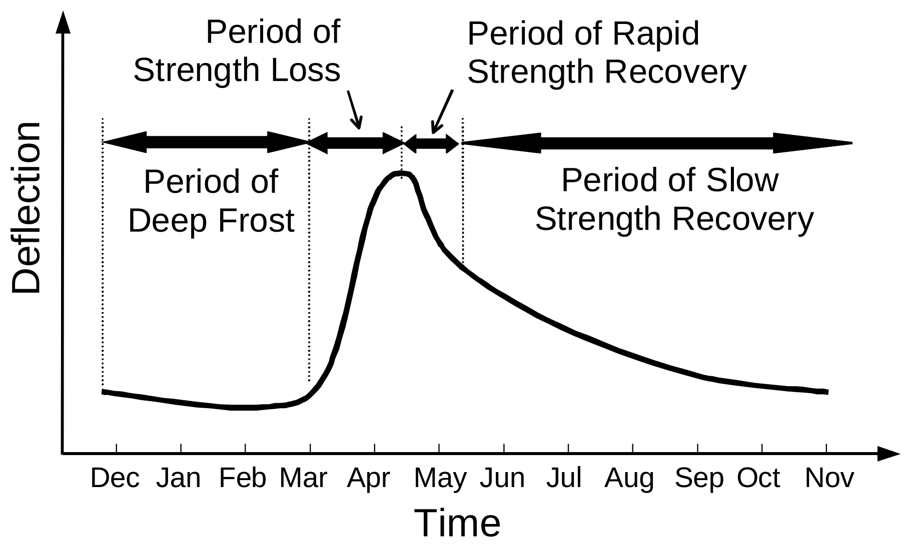


Figure 2.5.17. Seasonal effects on pavement deflection (*16*).

materials. Normally, the heavier the load the lower the back calculated modulus of unbound layers. Thus, it is recommended that the FWD load should be in the range of 9 to 12 kips so that the layer moduli predictions will be representative of pavement response under heavy truck wheel loads. In addition, with the deeper deflection zone caused by larger dynamic loads, additional weaknesses in the pavement structure may be located.

Test spacing depends on length of the road and level of investigation. For project-level evaluations, test spacing may vary from 100 ft to 500 ft. Testing may be performed in the outer wheelpath, in the center of the lane, or both. Figures 2.5.18 through 2.5.21 show schematics of the FWD test layout and the stress zone of a pavement under FWD testing.

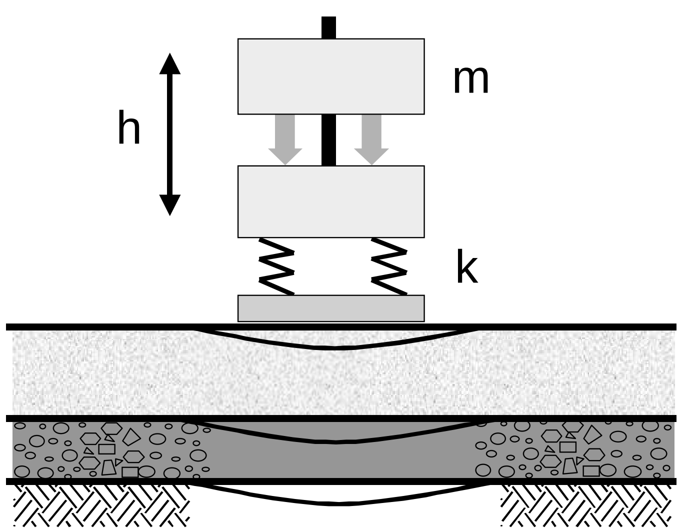


Figure 2.5.18. Schematic (elevation) view of the impulse type FWD test equipment (*16*).

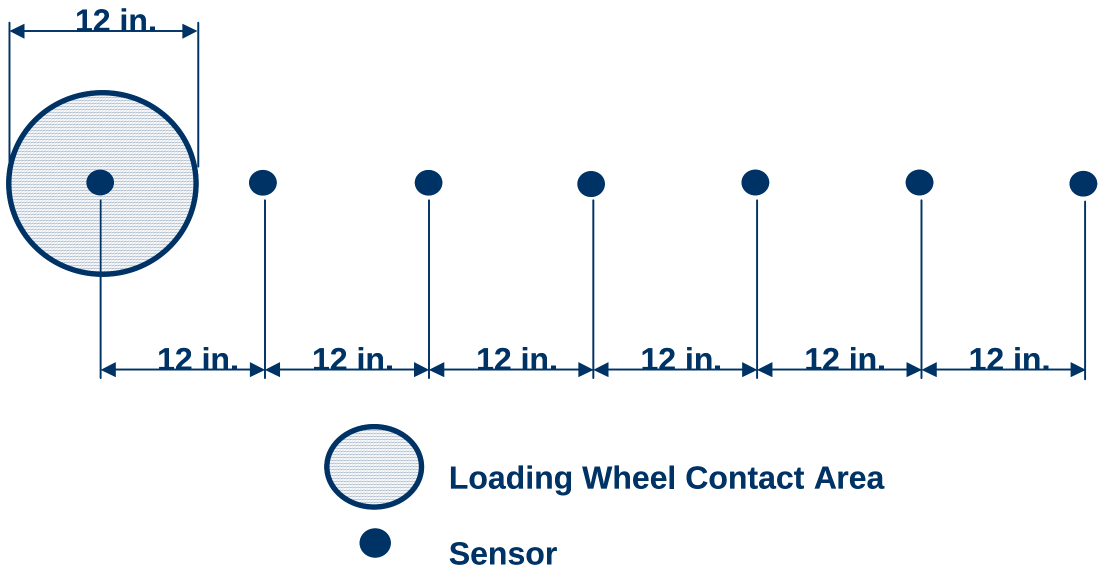


Figure 2.5.19. Plan layout of the FWD test showing loading location and position of sensors.

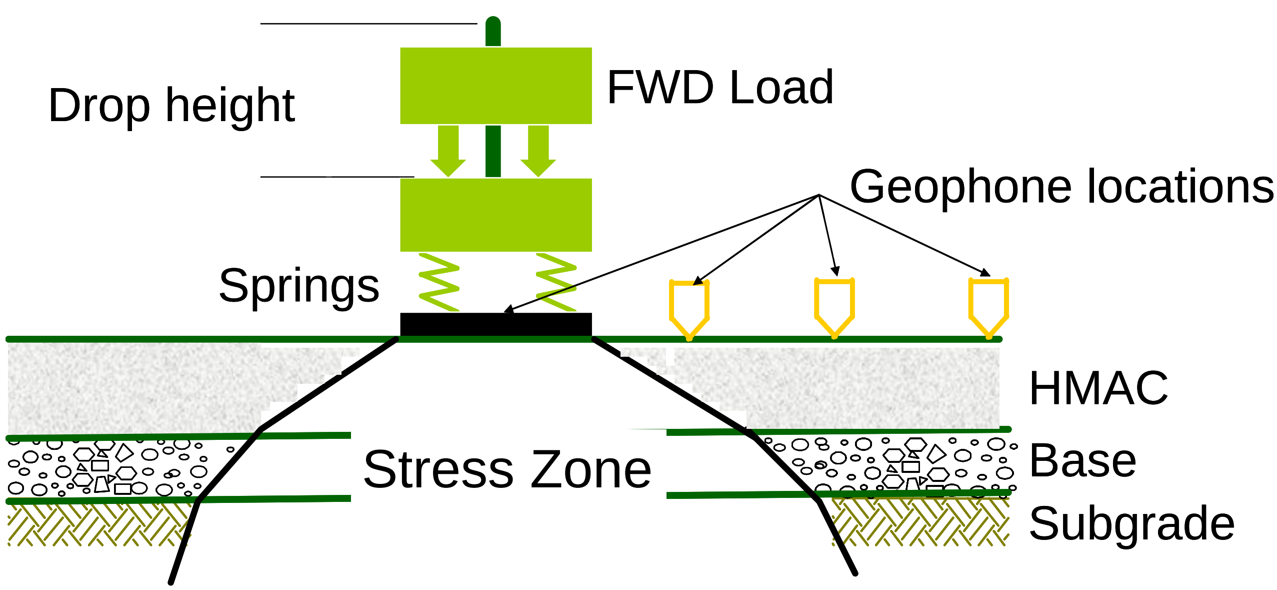


Figure 2.5.20. Schematic of the stress zone with a pavement under FWD testing.

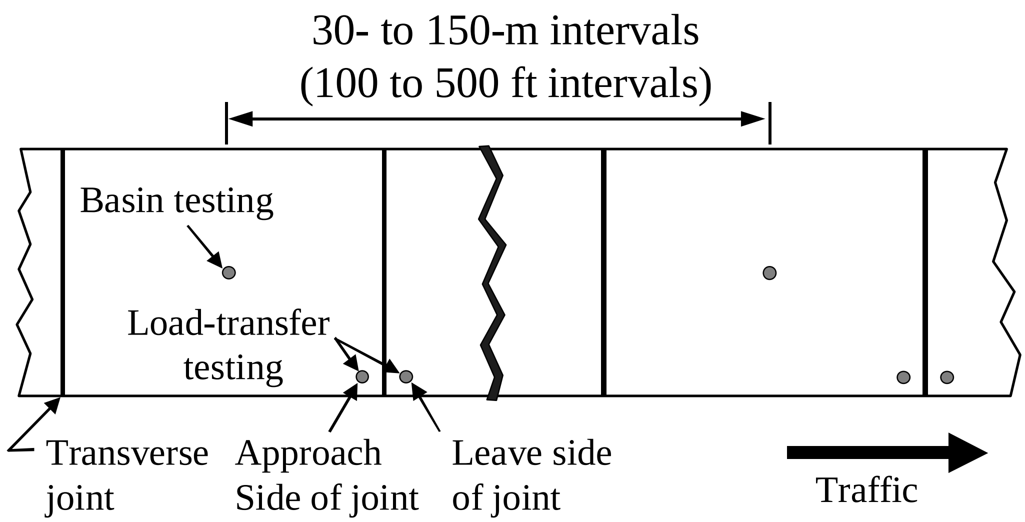


Figure 2.5.21. Examples of testing location and frequency for FWD testing for JPCP (*16*).

The use of the deflection test data for backcalculating asphalt concrete pavement elastic modulus (for each of the structural layers), backcalculating concrete pavement PCC elastic modulus, subgrade modulus of reaction, load transfer across joints, and for void detection are described in the next few sections.

Backcalculation of Asphalt Concrete Pavements Elastic Modulus (for Each Structural Layer)

This section presents a summary of the procedures used in backcalculating flexible pavement layer properties (*17, 18*).

Computational Software and Approaches

One of the more common methods for analysis of deflection data is to backcalculate the elastic properties for each layer in the pavement structure and foundation. Backcalculation programs provide the elastic layer modulus typically used for pavement evaluation and rehabilitation design. At present, interpretation of deflection basin test results usually is performed with static-linear analyses, and there are numerous computer programs that can be used to calculate these elastic modulus values (Young's modulus) (*17, 18*).

There are three basic approaches to backcalculating layered elastic moduli of pavement structures: 1) the equivalent thickness method, 2) the optimization method, and 3) the iterative method. Layer thickness is a critical parameter that should be reasonably accurately known for nearly all backcalculation programs, regardless of methodology, although some programs claim to be able to determine a limited set of both Young's modulus and layer thickness. Many of the software packages available are similar, but the results can be different as a result of the assumptions, iteration technique, backcalculation, or forward calculation schemes used within the programs.

Within the past couple of decades, there have been extensive efforts devoted to improve backcalculation of elastic-layer modulus by reducing the absolute error or Root Mean Squared (RMS) error to values as small as possible. The absolute error term is the absolute difference between the measured and computed deflection basins expressed as a percent error or difference per sensor; the RMS error term represents the goodness-of-fit between the measured and computed deflection basins. These improvements have spawned standardization procedures and guidelines to ensure that there is consistency within the industry and to improve upon the load-response characterization of the pavement structural layers. ASTM D 5858, *Standard Guide for Calculating In Situ Equivalent Elastic Moduli of Pavement Materials Using Layered Elastic Theory* is a procedure for analyzing deflection basin test results to determine layer elastic moduli (i.e., Young's modulus). Detailed algorithms used to compute layer elastic moduli are presented in several references (*19, 20*).

Limitations of the Backcalculation Process—Layered Elastic Analyses

Most backcalculation programs are limited by the number and thickness of the layer used to define the pavement structure. They are also limited by assuming that the behavior of pavement layer materials under loading is linear elastic defined using Young's modulus.

Unbound pavement materials and soils exhibit for the most part, non-linear behavior—either stress-hardening or stress-softening. Thus, the calculated layer modulus represents an "effective" Young's modulus that adjusts for stress-sensitivity and discontinuities or anomalies (such as variations in layer thickness, localized segregation, cracks, slippage between adjacent layers, and the combinations of similar materials into a single layer).

Most backcalculation programs use some sort of iterative or optimization technique to minimize the difference between the calculated (for a specific set of elastic layer properties) and measured deflection basins. Obviously, the absolute error (percent error per sensor) and RMS error (goodness-of-fit) vary from station-to-station and depend on the pavement's physical features that have an effect on the deflection basin measured with the FWD. For example, thickness variations, material density variations, surface distortion, and cracks, which may or may not be visible at the surface, can cause small irregularities within the measured deflection basin, which are not consistent with the assumptions of elastic layer theory.

These irregularities result in differences between the measured and calculated deflection basins. In fact, some of the differences between the calculated and measured basins are so large that the solution is considered highly questionable or that no elastic layered solution exists for that measured deflection basin for the simulated pavement structure (layer type and thickness).

Backcalculation of Concrete Slab Elastic Modulus and Subgrade Modulus of Reaction

This section presents a summary of the procedures used in backcalculating rigid pavement surface and subgrade strength properties. Detailed descriptions of the software and algorithms used with nondestructive response measurements of pavement structures for computing pavement strength parameters are provided in several references (*19*, *20*).

Computational Approaches

Rigid pavements are generally analyzed as slab on grade with or without a base or subbase. In the past decade, much progress has been made in the development of reliable methods for backcalculation of concrete slab, base layer, and subgrade moduli from deflection measurements. Nevertheless, backcalculation for rigid pavements remains a challenging problem. To obtain realistic results from backcalculation, a thorough analysis of all factors that influence the final results is required. Some of these factors are sensor configuration, base layer type, joint spacing, and temperature conditions during testing. Several methods for backcalculating the PCC slab, base, and subgrade moduli or moduli of subgrade reaction (k-value) are available. Each method has its strengths and its limitations. The following are algorithms specifically developed for rigid pavement; based on slab on elastic solid or slab on dense liquid models (*19*):

- AREA method-based procedures.
- Best Fit-based procedures.

Both backcalculation procedures/algorithms are based on plate theory and are used to backcalculate layer material properties—elastic modulus, Poisson's ratio, and modulus of subgrade reaction. The Best Fit method solves for a combination of the radius of relative stiffness, ℓ, and the coefficient of subgrade reaction, k, that produce the best possible agreement between the predicted and measured deflections at each sensor (*19*). The AREA method, which was described in the 1993 AASHTO Guide, estimates the radius of relative stiffness as a function of the AREA of the deflection basin. This estimation, along with the subsequent calculation of subgrade k and slab modulus of elasticity, E, is made using simple closed form equations (*1*, *19*). Both methods are based on Westergaard's solution for the interior loading of a plate consisting of a linear elastic, homogeneous, and isotropic material resting on a dense liquid foundation. Under a load distributed uniformly over a circular area of radius a, the distribution of deflections, w(r), may be written as (*19*):

$$w(r) = \frac{p}{k}\, f(r, l) \qquad (2.5.1)$$

$$f(r) = 1 - C_1(a_l)\, ber(s) - C_2(a_l)\, bei(s) \qquad for\ 0 < r < a \qquad (2.5.2)$$

$$f(r) = C_3(a_l)\, \ker(s) + C_4(a_l)\, kei(s) \qquad for\ r > a \qquad (2.5.3)$$

where

p	=	applied load intensity (pressure) = $P/(\pi a^2)$
P	=	total applied load
a_l	=	(a/l) dimensionless radius of the applied load

r	=	radial distance measured from the center of the load
l	=	$(D/k)^{1/4}$ radius of relative stiffness of plate-subgrade system for the dense liquid foundation
D	=	$Eh^3/12(1-\mu^2)$ flexural rigidity of the plate
k	=	modulus of subgrade reaction
s	=	(r/*l*) normalized radial distance
E	=	plate elastic modulus
μ	=	plate Poisson's ratio
h	=	plate thickness
ber, bei	=	Kelvin Bessel functions
ker, kei	=	Kelvin Bessel functions

The Kelvin Bessel functions may be solved using appropriate series expressions available in the literature (*19*). Detailed algorithms for the Best Fit and AREA methods used to compute the PCC elastic modulus and modulus of subgrade reaction are presented in several references (*18*, *20*).

Void Detection Testing and Analyses

Detection of voids under joints and cracks in rigid pavements is one of the important uses of the FWD. The FWD deflection data can be analyzed in several ways to estimate the approximate size of voids under a concrete pavement. Presented in the following sections are two methods for detecting voids under a concrete slab (*1*).

Corner Deflection Profile Method

This method requires the measurement of slab corner deflections under a constant load (preferably 9 kips) for all joints along a section of pavement. Measured deflections for the approach and leave slab corners for each joint within the project are then plotted as shown in figure 2.5.22. Usually the approach slab corners have little deflections and are most likely to have full support and therefore no voids. A “field-generated criterion” developed based on a maximum deflection value of the approach slab corner and larger than the apparent full support or no-void value of the approach slab corner, can then be selected and used as for identifying the corners with a high possibility of voids. Slab corners with deflections higher than the field-generated criterion may have voids. For example, the deflection measurements in figure 2.5.22 show approximately 0.020 in to be a reasonable maximum deflection criterion separating slab corners with or without a high possibility of voids (*1*).

One shortcoming of this void detection method is that a single criterion may not be appropriate if joint load transfer and slab corner deflections vary widely from joint to joint. High variability in pavement deflections may be due to variability in subgrade conditions and pavement structure. The change in temperature conditions over the testing period can also significantly affect deflection measurements. Furthermore, this method does not provide any indication of the probable size of voids that may be present. Consequently, this method should be viewed as an approximate approach to void detection (*1*).

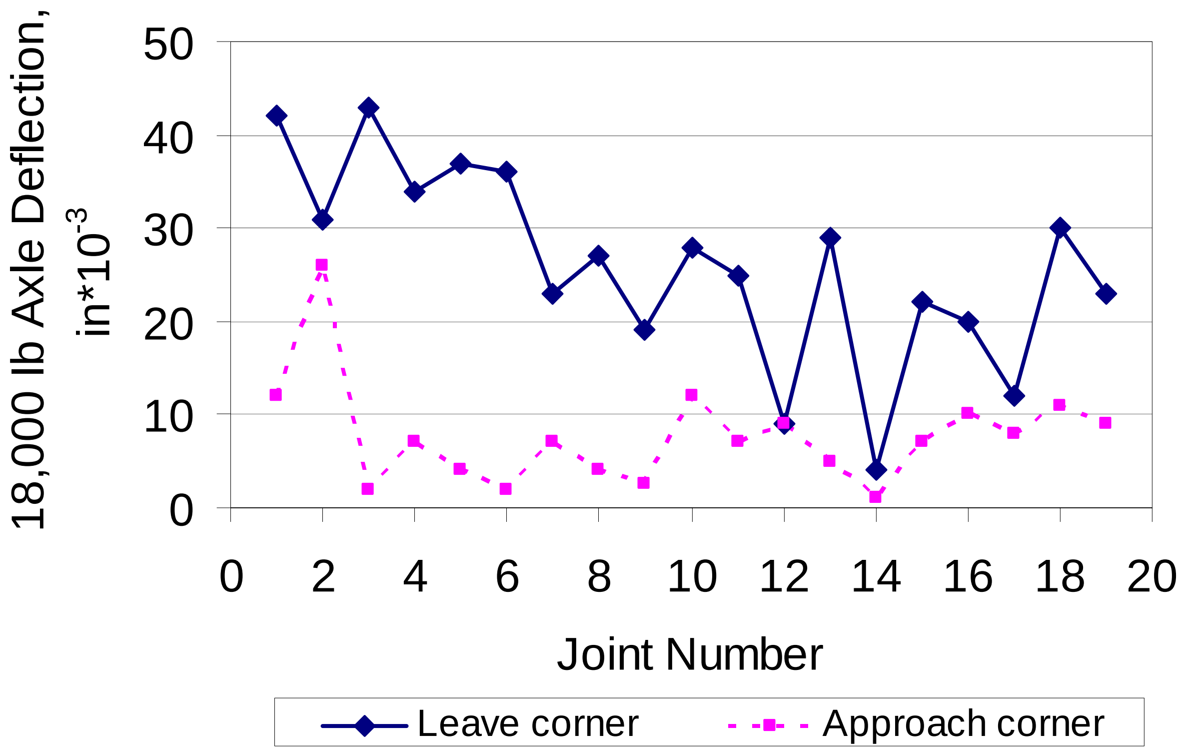


Figure 2.5.22. Profile of corner deflection for JPCP.

Variable Load Corner Deflection Analysis Method

In this method, corner deflections are measured at three load levels (6, 9, and 12 kips) to establish the load vs. deflection response for each test location. The measured deflections are then plotted against the load level, and a linear regression is performed to plot a best-fit line through the data points. The best-fit line is extended back to zero load to determine the y-intercept. If the pavement slab is fully supported, the y-intercept should be close to zero (theoretically, the intercept should be zero, however, allowing for possible errors in measurement, effect of slab curling, and so on the actual intercept should be close to zero). The presence of voids causes the best-fit line to shift up because of the additional deflection resulting from the loss of support. The result is a positive intercept that indicates the relative size of the void. An intercept of 0.002 in or more indicates a possible presence of voids. This method is illustrated in figure 2.5.23 (*1*). An example plot of calculated voids along a project is shown in figure 2.5.24.

A significant amount of variability is typically associated with any field test data. To obtain reliable results, measuring the testing variability is highly recommended. This can be accomplished by making multiple drops at each load level and conducting a t-test to determine if the calculated void is statistically significant. Even though some test protocols calls for three drops at each load level. It may not be necessary to make three drops at each load level at every testing point, but a sufficient number of multiple-drop tests should be conducted at each day of testing to establish testing variability.

Other Considerations for Void Detection Testing

The temperature conditions during deflection testing are extremely important to void detection. Several studies have shown that pavement slabs can have a significant amount of residual negative effective temperature gradients (i.e., the slabs are curled up). A fair amount of positive temperature gradient is usually required (e.g., 9 ^{o}F) for pavement slabs to reach the flat condition. When pavement slabs are exposed to negative temperature gradients (such as during nighttime or early morning), a significant amount of voids may be present under the slab corners, even if there are no foundation problems (*1*).

To ensure that built-in curling is not falsely identified as an erosion problem, the deflection testing for void detection should not be conducted when the pavement slabs are exposed to a significant negative temperature gradient. Higher midday temperatures should also be avoided during deflection testing to minimize the possibility of joint lockup and slab curl, especially if the Corner Deflection Profile method is used. On cool, overcast days, deflection testing may be performed throughout the day (*1*).

In general, a deflection device capable of simulating heavy truckloads should be used for the deflection testing. The preferred testing equipment is the FWD. There are no standards in terms of the location and quantity of joints to be tested; this is left to the discretion of the engineer upon conducting a visual field survey. The recommended testing pattern depends on the method of void detection used (*1*).

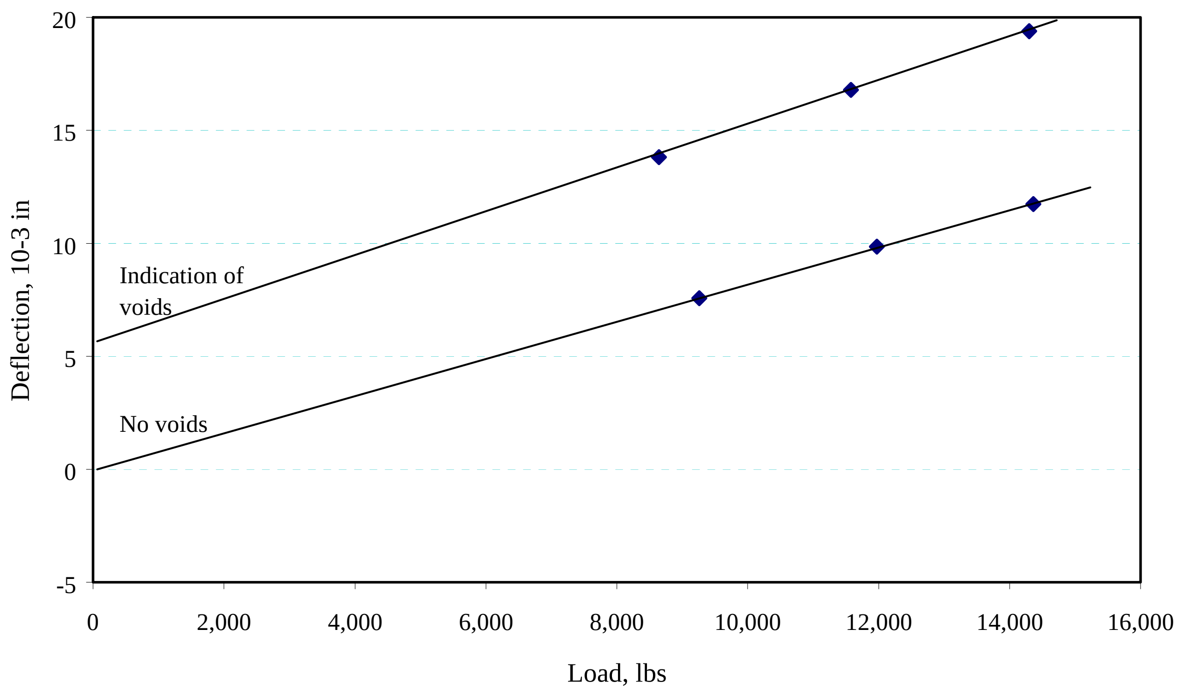


Figure 2.5.23. Void detection using the Variable Load Corner Deflection method.

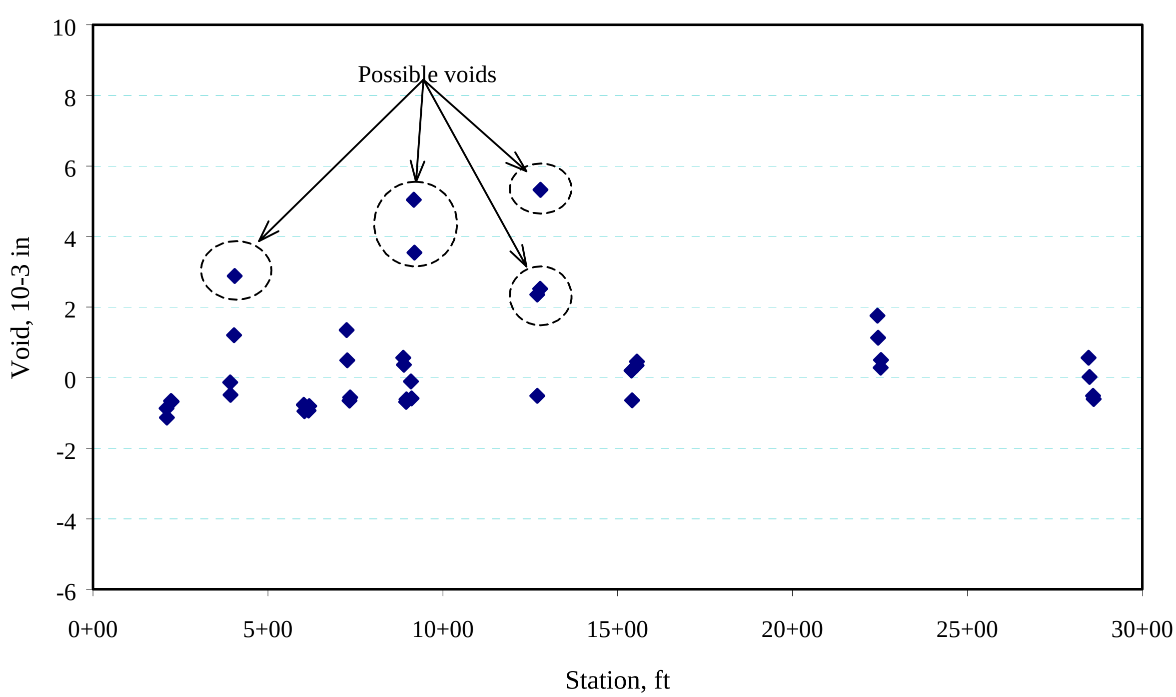


Figure 2.5.24. An example plot of calculated voids at joints along a project.

Load Transfer Efficiency

In addition to backcalculation of pavement layer and subgrade properties and void detection, deflection testing can also be used to evaluate the load transfer efficiency (LTE) of joints and cracks in rigid pavements (*1*).

LTE testing begins with the placement of the FWD load plate 6 in from the joint (or crack) measured from the center of the plate to the joint or crack. A haversine load is then imparted to the pavement while the deflections across the joint or crack is recorded. The sensors for measuring deflections are placed at the center of the plate and 12 in from the center of the load plate across the joint (or crack). LTE tests are usually performed in the outer wheelpath of the outside lane. Deflection data should be collected for both the approach and leave side of the joint. As a minimum, deflection data should be collected with the load plate on the leave side of the joint, which results in more conservative values of load transfer efficiencies (*1*).

Testing should be done at a minimum at one load level. It is preferable to test at three load levels—8 kips, 12 kips, and 16 kips. Also, it is recommended that testing should be performed across joints (or cracks) every 100 to 500 ft. However, depending on the length of the project and the availability of resources this can be increased to every 1000 ft (*1*).

Figure 2.5.25 illustrates the concept of deflection load transfer for two extreme cases: a joint with full load transfer and a joint with no load transfer. Joint deflection load transfer efficiency values may range from 0 percent (no load transfer) to 100 percent (full load transfer). The load transfer efficiency described above is the deflection load transfer because it is the ratio of the deflection of the unloaded side to the deflection of the loaded side. The deflection load transfer is not the same as stress load transfer, and studies indicate that there is not a one-to-one relationship between deflection load transfer and stress load transfer. The stress load transfer as described in the 1993 AASHTO Guide is not a required input for the structural analysis in the Design Guide. Using the deflection data obtained load transfer efficiency in terms of the measured deflection can be calculated using the following formula:

$$LTE = \frac{\delta_u}{\delta_l} * 100 \qquad (2.5.4)$$

where

LTE = load transfer efficiency, percent

δ_u = deflection on unloaded side of joint or crack measured 6 in from the joint/crack

δ_l = deflection on loaded side of joint or crack measured beneath the load plate the center of which is placed 6 in from the joint/crack

To ensure that the computed LTE reflects the actual LTE and joint or crack condition several other factors should be considered during testing. The important factors are presented in the following sections.

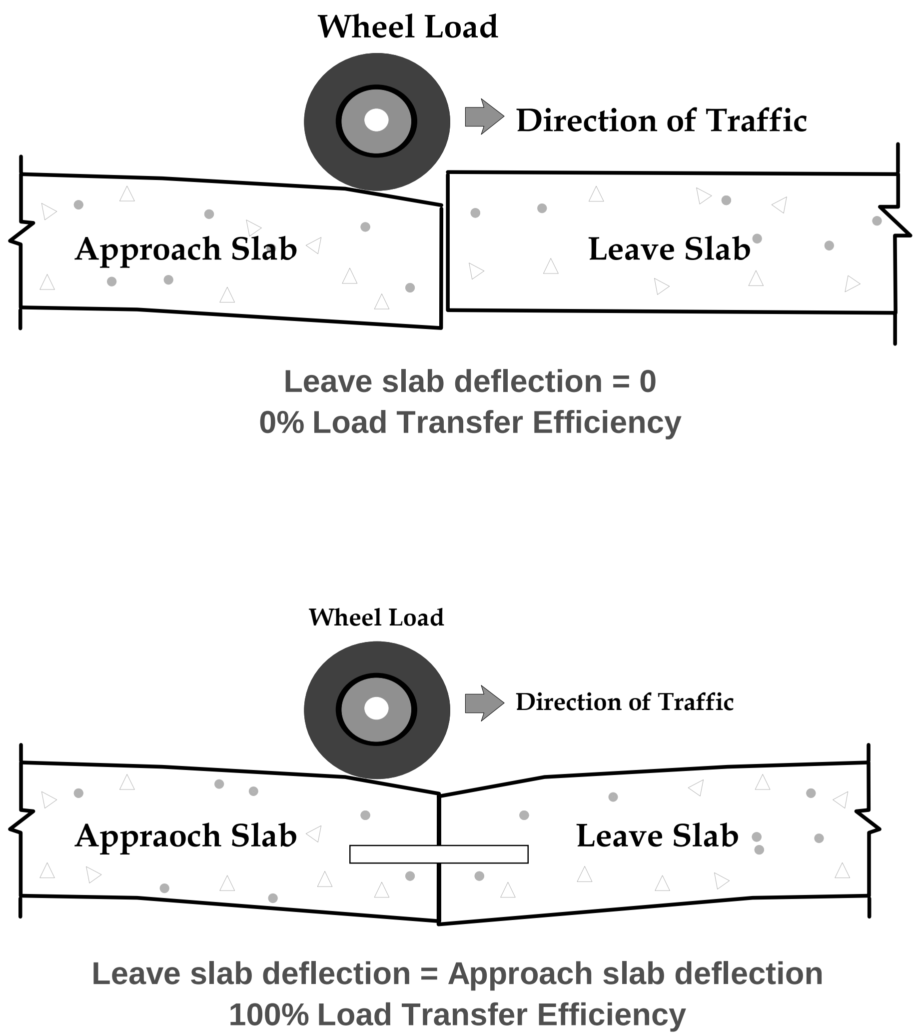


Figure 2.5.25. Illustration of poor and good load transfer across a joint.

Temperature Adjustment

Testing should be avoided during hot portions of the day (after 11 am, typically) to avoid joint lockup. On cool, overcast days, deflection testing may be performed throughout the day. When a large negative thermal gradient exists (during portions of the night), slab curling may be significant and may affect measured load transfer. The specific temperature adjustment to load transfer efficiencies will depend on the increments used to accumulate damage for top-down cracking, which is a major distress affected by joint load transfer.

Seasonal Adjustment

For the response modeling and calculation of structural damage, it is desirable to know the percentage of time that a pavement will have load transfer efficiencies less than 25 percent, 25 to 50 percent, 50 to 75 percent, 75 to 90 percent, and greater than 90 percent. Seasonal adjustment is made to the calculated load transfer efficiency values based on the testing time of year and on engineering experience. This is used to estimate the percentage of time the joints in the pavements will have load transfer efficiencies in the specified ranges.

Other Information

In addition to the actual deflection data, visual distresses present at the joint or crack should be recorded and quantified. Joint (and crack) distress information is useful in analyzing and filtering the results obtained from the LTE calculation. The joint distresses that need to be noted include, transverse joint spalling, transverse joint faulting, pumping/erosion, D-cracking, and alkali-silica reactivity (ASR). The presence of subsurface deterioration at a joint or crack can be determined by coring through the joint/crack and comparing the results to nondestructive testing and distress survey results. The load transfer rating as related to the load transfer efficiency is shown in table 2.5.9.

Table 2.5.9. Load transfer efficiency quality.

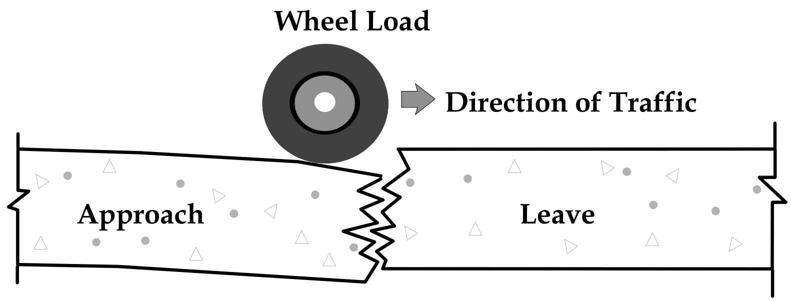
Load Transfer Rating	**Load Transfer Efficiency (percent)**
Excellent	90—100
Good	75—89
Fair	50—74
Poor	25—49
Very Poor	0—24

Evaluating Cracks in JCP

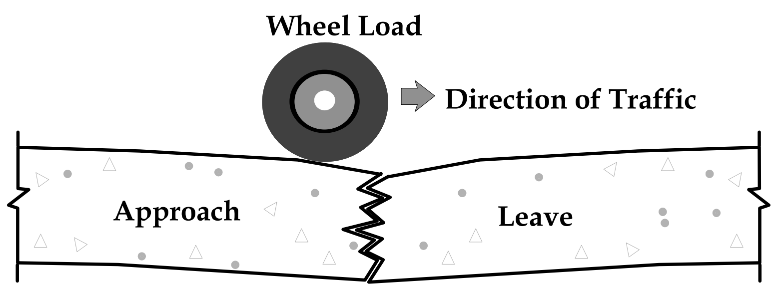
Load transfer efficiency calculation can be used to evaluate the level of deterioration of JCP cracks. Figure 2.5.26 shows an illustration of poor and good load transfer across a crack in a JCP. Crack LTE is a critical measure of pavement condition because it is an indicator of whether the existing cracks will deteriorate further. For jointed plain concrete pavements (JPCP), cracks are held together by aggregate interlock, while for jointed reinforced concrete pavements (JRCP) cracks are held together by the reinforcing steel and aggregate interlock. In general, cracks with a good load transfer (LTE greater than 75 percent) hold together quite well and do not significantly contribute to pavement deterioration. However, over time, the cracks in JPCP and JRCP deteriorate partially due to rupture of reinforcing steel for JRCP and loss of aggregate interlock for both JPCP and JRCP. Cracks with poor load transfer (LTE less than 50 percent) in are working cracks and can be expected to deteriorate to medium and high severity levels and exhibit faulting over time. These cracks are candidates for rehabilitation.

Ground Penetrating Radar

GPR is a well-established nondestructive method of investigating the internal composition of many naturally occurring materials such as rocks, earth and gravel, and man-made materials like concrete, brick, and asphalt. It can also be used to detect metallic and non-metallic pipes, sewers, cables, cable ducts, voids, foundations, reinforcing rods in concrete, and a whole host of other buried objects. It is also used to investigate the depth and makeup of different strata layers and is frequently used to survey areas of land before digging takes place.



Poor load transfer across crack



Good load transfer across crack

Figure 2.5.26. Illustration of poor and good load transfer across a crack in a joint concrete pavement.

This section presents information on the fundamentals of ground penetrating radar and how it used in characterizing pavement structure (layer thickness). A schematic showing a given pavement structure under GPR testing is shown in figure 2.5.27 (*2*).

General Principles

The radar system sends out pulses of electromagnetic (EM) energy and works by detecting the electrical echo caused when the pulse meets (electromagnetic) discontinuities, such as pipes, voids, or abrupt changes in material properties. By moving the radar across the surface of a pavement the reflected waves are used to create an image of the profile of the layers within the pavement system. The measured output is a time profile of how long it took the EM pulse to penetrate the pavement system and bounce back to the GPR receiver and is expressed as the Two Way Travel Time (TWTT). TWTT is the time taken for the signal to leave the transmit antenna, bounce off the target and finally be detected by the receive antenna. Radar data is collected over a time period or time window. The longer the time window over which the radar observes the returning signal, the further that signal will have traveled. This time profile can then be converted into a true depth.

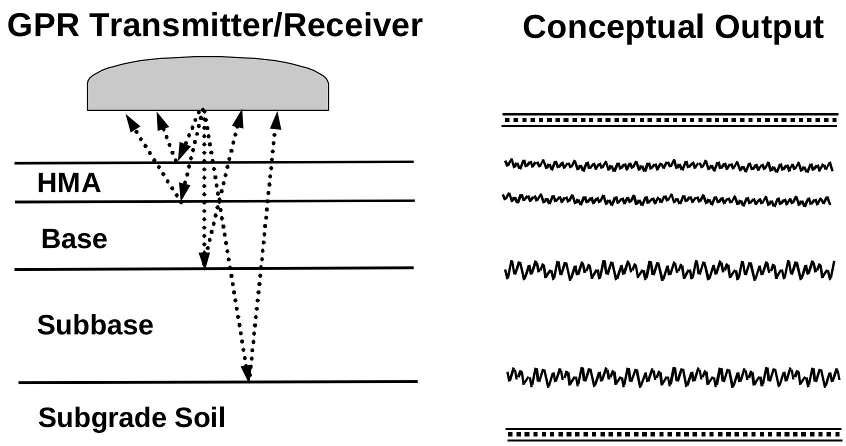


Figure 2.5.27. A schematic showing a pavement structure under GPR testing (*2*).

Use for Determining True Depth

A radar wave travels at different rates through different ground materials, due to their differing electromagnetic properties. This implies that the true depth of a layer can only be determined if the speed of propagation of the radar signal through the materials can be determined. This would have been relatively simple if a pavement system consisted of a uniform media with perhaps isolated targets buried within it. However, in many cases, the ground being surveyed can be quite complex, making simple calibration difficult. The radar pulses are electromagnetic in nature, they have an extremely short duration (about 0.001 microseconds), and they travel at close to the speed of light.

Table 2.5.10 shows the velocities in different media (compared with the speed of light, c). The typical GPR has the ability to use a variety of antennas centered on different frequencies for measuring the thickness profiles of pavements constructed with different material types. The center frequency of an antenna is defined as the mean of the range of frequencies that the antenna is sensitive to. Summaries of capabilities of the common antenna frequencies are presented in table 2.5.11.

Table 2.5.10. Estimates of the speed of radar pulses in different media (2, 21).

Medium	**Speed (as a fraction of the speed of light, c)**
Air	1.0
Dry geological material	0.33 to 0.5
Wet geological material	0.2 to 0.33
Water	0.11

Table 2.5.11. Summaries of capabilities of the common antenna frequencies (2, 21).

Antenna Mean Frequency, MHz	**Penetration Depths, m**	**Resolution, m**
500	2	0.05
1000	0.5 to 0.75	0.01

Without actual calibration, a speed of c/3 is assumed to be a reasonable approximation of the actual propagation velocity. At this velocity, it takes 0.02 microseconds to travel from the surface to a reflector at 1 m depth and back again. The velocity of the propagating pulse is determined by the "dielectric constant" of the subsurface material. The dielectric constant, ϵ, relates the velocity of propagation in air (c) to the velocity of propagation in the dielectric medium (v). Typical dielectric constants are summarized in table 2.5.12.

Table 2.5.12. Typical dielectric constants (2, 21).

Medium	**Dielectric Constant**
Air	1.0
Dry geological material	4 to 9
Wet geological material	9 to 25
Water	81

As discussed, the raw output data from a GPR is presented as a radargram. As the antennas are moved across the surface, the transmitter radiates short sharp pulses and the receiver records the echoes. This is analogous to firing a shot in a mountainous area and listening to the echoes. The radar system constructs amplitude versus time traces as the antennas are moved across the subsurface. These traces are plotted next to each other showing recorded amplitudes versus chainage along the profile, and time (depth) into the ground. The resulting radargram appears in the form chainage (horizontal axis) versus time (vertical axis). The simplest conversion from time to depth requires knowledge of the velocity of the pulse in the ground. Typical time-to-depth conversion factors are given in table 2.5.13.

Table 2.5.13. Typical time-to-depth conversion factors (2, 21).

Medium	**Dielectric Constant, ms/m**
Air	0.006
Dry geological material	0.012 to 0.02
Wet geological material	0.02 to 0.035
Water	0.06

The interpretation of GPR data has been a source of suspicion among engineers. Very rarely are the test data of such quality that they can be interpreted literally. Radar works best as a technique for interpolating between known quantities, such as between boreholes, or extrapolating from known pavement profiles. For most typical applications, the engineer or geophysicist cannot interpret radar data blindly (except for "search-and-identify" projects such as confirming the presence of buried cables or pipes underground). For applications, such as, determining (1) the depth of pipes and cables beneath a pavement or (2) the pavement layer thicknesses GPR can be applied in the following two ways:

- Use for the identification of potential sites for ground-truthing exercises (drilling).
- Use to interpolate between (or extrapolate from) ground truth sites.

Brief descriptions of GPR applications for pavement evaluation are presented in the following sections.

Use of GPR for Search-and-Identify

Search-and-identify is the most trivial application of radar. Usually very little is known of the site other than that there may be some buried object (e.g., pipes). GPR is deployed in the hope that it can prevent a large and painstaking digging exercise. Detecting anomalies in radar patterns over the site identifies possible locations of the buried objects. The identified locations are then excavated. Failure to find anomalies in radar patterns is not unusual because a general knowledge of the site should exist so that testing is performed in the vicinity of the buried object. Failure to do this could lead to a wild search with very little chance for success. Also, some ground conditions are not uniform, making it difficult to interpret radar signals. In such situations, GPR is normally used as a cheap substitute for an expensive excavation program.

Use for the Identification of Potential Sites for Ground-Truthing Exercises (Drilling)

The application of radar to identify drilling sites is reasonably straightforward. A rapid GPR survey is conducted over the area of interest. Depending on the anomalies identified in radar patterns, sites with typical or atypical structures can be identified for further investigation.

Use to Interpolate Between (or Extrapolate From) Ground Truth Sites

The interpolation between, or extrapolation from, ground-truth sites is less trivial. Here radar can be used to "connect the dots" between drill holes and, sometimes, stratigraphies as determined by other geophysical techniques. The use of ground-truth helps, as literal interpretation of radar data is usually very difficult. Close cooperation between the engineers and geophysicists is needed for accurate interpretation.

Limitations

GPR relies on the transparency of the geological materials (in the frequency band in which radar works) to allow one to "see" into the ground. Electrically conductive media are partially opaque to radar and limit the range over which radar can be used. Overall, conductivity is increased by the presence of clay minerals, salts, and water, especially in combination. Conductive ore-bodies also limit penetration. Sometimes, different geological materials have similar dielectric properties. As radar relies on the electromagnetic contrast between materials, the interface between the materials may be invisible. This is especially true in unweathered rock, where the contrasts are expected to be low in any case.

2.5.2.9 Destructive Pavement Testing

Background

Historically, destructive and nondestructive tests have been used in combination for evaluating pavement strength and performance—assessment of the existing pavement condition. Experience has shown that nondestructive testing techniques alone may not always provide a reasonable or accurate characterization of the in-situ material properties, particularly for those of the top pavement layer. It is recognized good practice to supplement nondestructive tests with the use of destructive testing methods. Destructive testing ranges from coring of materials for observation and testing to the removal of a layer of pavement by milling to observe the underlying pavement condition. Another common and useful form of destructive testing is the DCP test.

Destructive tests require the physical removal of pavement layer material to obtain a sample (either disturbed or undisturbed) for observation of material condition (e.g., bonding, AC stripping, PCC D-cracking, or PCC ASR) or to conduct an in-place test. Such testing has many limitations, particularly when conducted on moderate to heavily trafficked highway systems. Practical restraints in terms of time and money severely limit the number and variety of destructive tests conducted on routine rehabilitation studies.

Major Parameters

During the data collection process, the engineer should accumulate enough information on the in-place condition of the pavement system to determine the precise cause of the distress. The parameters of the actual data collected will vary from project to project. To illustrate, if a rigid pavement is experiencing extensive pumping after 15 to 20 years of service, the rehabilitation required is probably routine, and a minimum field sampling and testing program will probably suffice. On the other hand, if a rigid pavement is experiencing extensive pumping after only a few years in service, more extensive field testing and data collection may be necessary to pinpoint the exact cause of the distress and the appropriate rehabilitation measures. Such pumping may be the result of material erosion (improper compaction) or excessive joint deflection (inadequate load transfer).

It is the responsibility of the engineer to determine the extent of the data collection for a specific project, and to minimize cost by avoiding the collection of unnecessary information. There are, however, several major parameters that should be viewed as highly recommended in any data collection process. They are as follows:

- In-situ material properties (e.g., modulus and strength).
- Layer thicknesses.
- Layer material type.
- Examination of cores to observe general condition and material durability.

An important aspect of destructive testing is to test the retrieved core samples for strength and to determine or confirm the layer thickness and material type. Other useful information obtained through visual examination of cores is the pavement layers general condition, the presence, severity, and extent of distress, indication of stripping for AC cores and D-cracking and ASR for PCC cores, bonding condition between layers, and the presence of defects such as cracks, voids, layer separation, aggregate distribution, bleeding. Other features of the material that can be noted are the general type and shape of aggregate such as rounded gravel or angular crushed stone.

Necessity for Destructive Testing

There are three sources of information available to the engineer during the data collection process: historic data, destructive testing, and nondestructive testing. One or more of these sources may be used to fulfill the data collection parameters listed above. While the emphasis thus far has been on nondestructive testing, destructive testing may play a vital role in field sampling and testing.

The use of a limited number of destructive tests to verify/modify material properties estimated from either NDT or historic data is sound engineering practice worthy of consideration. Also, these tests may be used to determine drainage conditions and identify problem layers. Test pits may also be of use in this area.

For rigid pavements, one of the more significant material properties influencing performance is the flexural strength (modulus of rupture) of the concrete. General correlations between splitting tensile strength and flexural strength may be used as a source of input since cores can be obtained from the pavement.

The determination of pavement layer material type cannot be made through NDT. While historic information may be available, the extreme importance and sensitivity of this variable calls for the use of destructive testing to verify/modify the available historic information. Layer material type can usually be identified from historic pavement information, unless special circumstances dictate otherwise. A limited amount of coring at randomly selected locations may be used to verify the historic information.

In summary, while NDT is largely preferred to destructive testing, a complementary destructive test program is needed to ensure the accuracy of data obtained. This system will also ensure that only accurate data will be used in the rehabilitation design.

Selecting Test Protocols and the Required Number of Tests

Most agencies use their local test protocols to perform laboratory analysis to characterize pavement material properties. Such test protocols provide guidance on method of sampling, material preparation, test equipment or apparatus, frequency of calibration of test equipment, test procedure, calculation of results and data analysis, and reporting of results.

It is recommended that agencies follow the guidance provided in the test protocols to obtain consistent and reliable results.

Sampling should be designed for each analysis unit (delineated using the guideline presented in section 2.5.2) based on the unit length and design or construction features (e.g., intersection, bridge approach). Analysis units are pavement segments, which exhibit statistically uniform attributes and performance. These units should form the basis for a field sampling and testing program. The sampling and testing plan should also identify within-unit variability (associated with any parameter) for use in design reliability computations. PART 2, Chapter 2 of this Guide provides a summary of test types and protocols recommended for materials characterization of flexible and rigid pavements.

Dynamic Cone Penetrometer

This section describes the DCP, its use, and the application of data obtained by its use. The DCP is typically used to measure soil strength. It can also be used for determining pavement layer thickness by identifying sudden changes in strength within the pavement layer system.

Description

The DCP consists of a 5/8-in diameter steel rod with a steel cone attached to one end which is driven into the pavement or subgrade by means of a sliding dual-mass hammer. A schematic of the DCP is shown in figure 2.5.28. The angle of the cone is 60 degrees, and the diameter of the base of the cone is 0.79 in. The cone is hardened to increase service life. The diameter of the cone is 0.16 inch larger than that of the rod to ensure that the resistance to penetration is exerted on the cone.

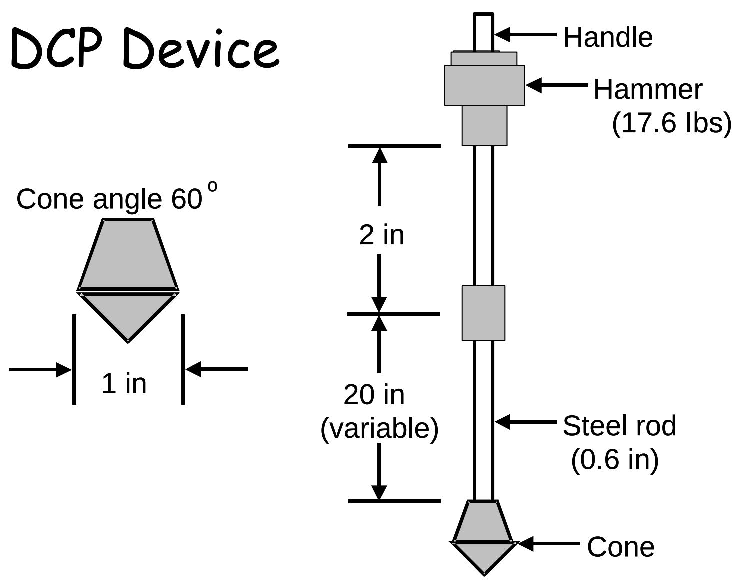


Figure 2.5.28. Schematic of the manual DCP (*16*) (Not to scale).

The DCP is driven into the soil by dropping either a 17.6-Ib or 10.1-Ib sliding hammer from a height of 22.6 in. The cone penetration caused by one blow of the 17.6-Ib hammer is essentially twice that caused by one blow of the 10.1-Ib hammer.

The 10.1-Ib hammer is more suitable for use and yields better test results in weaker soils having a California Bearing Ratio (CBR) value of 10 or less. The 17.6-Ib hammer penetrates high-strength soils quicker and may be preferred when these soils are encountered. However, the 10.1-Ib hammer can be used on soils up to CBR 80.

The depth of cone penetration is measured at selected penetration or hammer-drop intervals, and the soil shear strength is reported in terms of DCP index (based on the average penetration depth resulting from one blow of the 17.6-Ib hammer). The average penetration per blow of the 10.1-Ib hammer should be multiplied by 2 to obtain the DCP index value. The DCP is designed to penetrate soils to depths of 36 in. Individual DCP index values are reported for each test depth resulting in a soil-strength-with-depth profile for each test location.

Soil Strength Evaluations with DCP

The number of test measurements required, test location, depth of measurements, and frequency of recording data with depth vary with the type and properties of the pavement to be evaluated and with the time available for conducting the tests. For this reason, there are no hard-and-fast rules for the number of tests required in evaluating pavements. Soil conditions are extremely variable. The strength range and uniformity of the soils or existing pavement materials will generally control the number of measurements necessary.

In all cases, it is advisable to first test those spots that appear to be weakest, since the weakest conditions are critical for pavement evaluation. Penetrations in areas that appear to be firm and uniform may be few and widely spaced. In areas of doubtful strength, penetration tests should be more closely spaced. No less than three penetration tests should be made in each area having similar soil conditions.

Soil strength usually increases with depth, but in some cases a thin, hard crust will overlay a soft layer, or the soil will contain thin layers of hard and soft material. For this reason (and the fact that trucks with heavy wheel loads will exert considerable stresses to the soil up to depths of 36 in or more), it is recommended that testing be conducted to a minimum depth of 36 in unless prevented by a very hard layer at a lesser depth. Testing depths may be reduced for pavements with light traffic.

Application of Test Data

The Design Guide Software allows users to input DCP test results directly or indirectly depending on the models of choice for converting the raw penetration data into layer moduli as follows:

- Directly input the penetration test result.
- Convert the penetration test results into CBR using locally calibrated models and then input layer CBR.
- Convert the penetration test results into modulus using locally calibrated models and then input layer modulus.

Models used in the Design Guide Software to convert penetration rate to modulus are presented in PART 2, Chapter 2.

Other Considerations in Using the DCP for Pavement Evaluation

In general, soil strength and properties are affected immediately and significantly by climate. The magnitude in change in properties varies according to material type. Gravel layers within the pavement structure are generally less affected by rainfall than other soil types, such as clay. Nevertheless, both soil types will freeze if exposed to extended periods of sub-zero temperatures. DCP test results are therefore only useful if interpreted within the context of the soil condition. A blind use of test data could result in significant deviations from the true material strength properties.

The soil type in which testing is to be performed should also be taken into consideration to develop appropriate test strategies. For example, DCP tests in highly plastic clays are generally accurate for depths to approximately 12 in. At deeper depths, clay sticking to the lower rod may indicate higher CBR values than the actual values. Oiling the penetration rod will help prevent the clay from sticking to the penetration rod; however, it will not significantly improve the test results. A 2-in diameter (or larger) auger can be used to open the test hole up after each 12-in DCP test penetration. This will eliminate clay lower-rod friction problems and allow the test to accurately measure the clay soil strength for an additional 12 in. However, sands occur in a loose state, and relatively dry sands show no DCP index values for the top few inches. They only show increasing DCP index values with depth, and this should be considered when evaluating the strength properties of such materials regardless of the DCP index values.

Finally, DCP testing should not be conducted in very stiff or high strength soils. If a layer is encountered with a penetration of less than 1 in after 10 blows with the 17.6-Ib hammer (20 blows with the 10.1-Ib hammer), then testing should be stopped to prevent damage to the equipment. The depth of the stiff layer may be determined by coring or drilling with an auger. For thin stiff layers, DCP testing should proceed through the cored access hole after the depth of stiff layer has been recorded. The DCP is generally not suitable for soils with significant amounts of coarse aggregates that will be retained on a 2-in sieve.

2.5.3 OVERALL CONDITION ASSESSMENT AND PROBLEM DEFINITION

The final step in the pavement evaluation process is to prepare an overall condition assessment. If this is done properly, then the next phase of identifying feasible rehabilitation alternatives is relatively easy (PART 3, Chapter 5). Such an assessment helps fully define the problems that should be addressed or corrected, leading to the development of a set of feasible rehabilitation solutions.

This section presents guidelines for assessing the overall state of a pavement and defining its key problems, in preparation for identifying feasible, cost-effective rehabilitation alternatives. Areas of assessment include pavement structural, functional, and drainage adequacy, materials durability, maintenance applications, shoulders adequacy, and variability within the project.

As illustrated in table 2.5.14, information gathered from the various surveys and tests discussed in the previous section of this chapter can and should be used in assessing overall pavement condition.

Table 2.5.14. Areas of overall condition assessment and corresponding data sources.

Area of Assessment	Data Source					
	Distress Survey	Smoothness Testing	Friction Testing	Drainage Survey	Nondestructive Testing	Destructive Testing
Structural Adequacy	√			√	√	√
Functional Adequacy	√	√	√			
Drainage Adequacy	√			√	√	√
Materials Durability	√			√	√	√
Maintenance Applications	√					
Shoulders Adequacy	√				√	
Variability Along Project	√	√	√	√	√	√
Miscellaneous	√			√	√	√

To assess current adequacy in any of the categories (e.g., structural, functional, drainage, and so on) the extent and severity of related distresses is compared with the value ranges provided for different highway classes. A pavement is considered to have failed in the given category if it meets or exceeds any of the threshold values specified under the “inadequate” category. When a pavement exhibits these levels of distress, the rate of deterioration is such that maintenance treatments become cost-prohibitive, lane closures excessive, and larger-scale remedial action is needed.

A pavement with one or more related distresses in a given category in the “marginal” category is one that will soon need rehabilitation. The establishment of a “trigger” value for each distress within the ranges listed under “marginal” allows an agency time to plan, design, and implement a rehabilitation activity prior to the pavement reaching a structurally inadequate condition.

The rehabilitation strategy adopted for a given pavement should be based on the categories that are inadequate or marginal. Specific rehabilitation strategies should be based on the local agency guidelines. Some guidance is provided in PART 3, Chapter 5 on possible rehabilitation alternatives available.

2.5.3.1 Structural Adequacy

The structural adequacy of a pavement can be viewed in terms of current or future structural adequacy.

Current structural adequacy represents the difference between the existing level of pavement deterioration and the level of deterioration deemed as structural failure by an agency. A pavement that has not deteriorated to the point of structural failure is considered structurally adequate, whereas one that has reached or surpassed the agency-specified threshold for deterioration is considered structurally deficient.

Future structural adequacy refers to a pavement's ability to support projected traffic loadings (obviously, a pavement that is currently structurally deficient has no ability to carry future loads without significant maintenance and repair). It is measured in terms of the amount of additional serviceable life (age or traffic loadings) an existing pavement can provide without undergoing any structural improvements and without heavy maintenance and repair. This remaining serviceable life is determined through a detailed analysis of structural distresses using mechanistic based models and various key inputs concerning the design, past performance, and current state of the pavement.

A thorough assessment of structural adequacy makes use of the following four key evaluation activities:

- Evaluation of the current types, severities, and extents of load-related distresses (and their progression overtime if available).
- Evaluation of in-situ material samples via coring, removal of small sections of pavements, visual examination, and testing.
- Analysis of pavement response to loading characteristics, as generated by deflection testing.
- Estimating damage to the existing pavement structure and, thus, the remaining life of the pavement.

Evaluation of Load-Related Distresses

Surface distresses provide valuable insight into a pavement's current structural condition. The presence of significant load-associated distresses, as identified in a visual condition survey, generally indicates that a pavement is currently approaching or has reached structural inadequacy. Such distresses in flexible pavements include fatigue cracking (including longitudinal cracking in the wheelpath) and rutting. For rigid pavements, they include transverse cracking and transverse joint/crack faulting in JPCP and JRCP and longitudinal cracking in JPCP, JRCP, and CRCP, punchouts in CRCP, and patch/patch deterioration in all three concrete pavement types.

To assess current structural adequacy, the extent and severity of load-related distresses can be compared with the value ranges provided for different highway classes in tables 2.5.15 and 2.5.16. A pavement is considered to have failed structurally if it meets or exceeds any of the threshold values specified under the "inadequate" category. When a pavement exhibits these levels of distress, the rate of deterioration is such that maintenance treatments become cost-prohibitive, lane closures excessive, and larger-scale remedial action is needed.

Table 2.5.15. Distress types and severity levels recommended for assessing rigid pavement structural adequacy (at the time of evaluation).

Load-Related Distress	Highway Classification	Current Distress Level Regarded As:		
		Inadequate	Marginal	Adequate
JPC Deteriorated Cracked Slabs (medium- and high-severity transverse and longitudinal cracks and corner breaks), % slabs	Interstate/ Freeway	>10	5 to 10	<5
	Primary	>15	8 to 15	<8
	Secondary	>20	10 to 20	<10
JRC Deteriorated Cracked Slabs (medium- and high-severity transverse cracks and corner breaks), #/lane-mi	Interstate/ Freeway	>40	15 to 40	<15
	Primary	>50	20 to 50	<20
	Secondary	>60	25 to 60	<25
JPC Mean Transverse Joint/Crack Faulting, in	Interstate/ Freeway	>0.15	0.1 to 0.15	<0.1
	Primary	>0.20	0.125 to 0.20	<0.125
	Secondary	>0.3	0.15 to 0.3	<0.15
JRC Mean Transverse Joint/Crack Faulting, in	Interstate/ Freeway	>0.3	0.15 to 0.3	<0.15
	Primary	>0.35	0.175 to 0.35	<0.175
	Secondary	>0.4	0.2 to 0.4	<0.2
CRC Punchouts (medium- and high-severity), #/lane-mi	Interstate/ Freeway	>10	5 to 10	<5
	Primary	>15	8 to 15	<8
	Secondary	>20	10 to 20	<10

Table 2.5.16. Distress types and levels recommended for assessing current flexible pavement structural adequacy.

Distress Type	Highway Classification	Distress Level Regarded As:		
		Inadequate	Marginal	Adequate
Fatigue Cracking, percent of wheel path area.	Interstate-Freeway	>20	5 to 20	<5
	Primary	>45	10 to 45	<10
	Secondary	>45	10 to 45	<10
Longitudinal Cracking in Wheel Path, ft/mi	Interstate-Freeway	>1060	265 to 1060	<265
	Primary	>2650	530 to 2650	< 530
	Secondary	>2650	530 to 2650	< 530
*Reflection Cracking, crack width, in	Interstate-Freeway	> 0.5	0.25 to 0.5	< 0.5
	Primary	> 0.75	0.5 to 0.75	< 0.5
	Secondary	> 0.75	0.5 to 0.75	< 0.5
Transverse Cracking, spacing, ft	Interstate-Freeway	< 100	100 to 200	> 200
	Primary	< 60	60 to 120	> 120
	Secondary	< 60	60 to 120	> 120
Rutting, mean depth of both wheel paths, in	Interstate-Freeway	> 0.4	0.25 to 0.4	< 0.25
	Primary	> 0.6	0.35 to 0.6	< 0.35
	Secondary	> 0.8	0.4 to 0.8	< 0.4
Shoving, percent of wheel path area.	Interstate-Freeway	>10	1 to 10	None
	Primary	>20	10 to 20	<10
	Secondary	>45	20 to 45	<20

* Composite AC/PCC pavements.

A pavement with one or more load-related distresses in the “marginal” category is one that will soon need rehabilitation. The establishment of a “trigger” value for each distress within the ranges listed under “marginal” allows an agency time to plan, design, and implement a rehabilitation activity prior to the pavement reaching a structurally inadequate condition.

The results of visual condition surveys can also be used to determine variability within the project. Differences in load-related distress levels throughout the project length, between lanes, or at specific locations (e.g., bridge approaches, intersections) give the engineer more information to work with when formulating overall rehabilitation design strategies.

The extent of differences in distresses between traffic lanes (carrying significantly different amounts of heavy trucks traffic) is important to note in the evaluation. This provides a direct indication of structural adequacy of the pavement.

Lastly, it should be noted that the values in tables 2.5.15 and 2.5.16 represent general guideline limits as to what is considered adequate, marginal, and inadequate. Designers may specify alternative limits, based on their own experiences and agency policies/practices.

<u>Evaluation and Testing of In-Situ Material Samples</u>

The results of visual distress surveys can be supplemented with information collected through the visual examination of cores and material testing. Core samples taken from an existing pavement, for instance, not only serve to verify layer thicknesses and conditions, they can also be used to identify the causes of observed distresses (e.g., debonding of AC layers, stripping of AC, PCC durability problems), tested for evidence of unbound granular material degradation and/or contamination, and tested for AC dynamic modulus, PCC compressive strength and indirect tensile strength. Resulting modulus and strength results can be compared with typical design values or used in a simulation of pavement response under loading.

In some cases, it may be useful to remove (by milling) a small section of pavement surfacing (AC overlay of PCC joint) to observe the underlying pavement condition.

<u>Analysis of Pavement Response</u>

A third method of evaluating an existing pavement’s structural adequacy involves characterizing pavement response to loading by NDT. Coupled with data from a complete visual distress survey and coring and materials testing program, NDT data can provide a very reliable assessment of current structural adequacy. More important, however, is the fact that the data compiled from all three evaluations (distress survey, coring, and NDT) can be used to estimate existing pavement damage and, in turn, determine structural adequacy.

NDT Data

Although deflection data are often used to quantify the variability of pavement strength within a project, such data can also be used to backcalculate AC modulus, base modulus, PCC elastic modulus and the resilient moduli of underlying pavement layers, including the subgrade, and the modulus of subgrade reaction. The backcalculated values can be compared to the typical material stiffness characteristics to identify weak material layers and gauge current structural adequacy. Table 2.5.17 list typical values found in the Long Term Pavement Performance (LTPP) database.

Table 2.5.17. Typical laboratory tested /backcalculated pavement layer material stiffness and strength characteristics (*19*, *22*, *23*, *24*).

Strength Variable	**Material Type**	**Range of Test Values**		
		Low	**High**	**Mean**
Modulus	AC*	300,000	1,500,000	500,000
	PCC	3,000,000	7,500,000	4,500,000
Resilient Modulus, psi	AC-treated base	100,000	500,000	250,000
	Lean concrete base	500,000	2,500,000	1,500,000
	Cement-treated base	250,000	1,000,000	600,000
	Soil cement	50,000	100,000	75,000
	Granular base	15,000	40,000	30,000
	Granular subbase	8,000	25,000	15,000
	Coarse subgrade	7,000	20,000	12,000
	Fine subgrade	3,000	7,000	5,000
Flexural Strength, psi	PCC	500	900	700
Compressive Strength, psi	PCC	2,500	7,000	5,500

* Tested at 68 ^{o}F and 0.1 cycles/sec.

As an example, a pavement with a mean PCC elastic modulus below 3,000,000 psi would be considered structurally deficient, since the modulus typically ranges from 3,000,000 to 7,000,000 psi. A low elastic modulus value would likely be attributed to distress within the PCC material, such as cracking or durability problems.

For rigid pavements, deflection test results can also be used to compute the subgrade dynamic modulus of reaction. The backcalculated k-value’s can be compared to the subgrade strength characteristics provided in table 2.5.18, to determine the general condition of the subgrade and to identify areas of weakness. It is also a useful tool for identifying variability in pavement foundation strength along the project.

The material strength values in tables 2.5.17 and 2.5.18 serve as guidelines for comparison with actual values. Agencies are encouraged to develop their own matrix of typical strength values for assessing relative pavement structural condition.

DCP Data

DCP test data can also be used to assess the structural condition of the existing foundation. As discussed in the previous section of this chapter, DCP test results can be correlated with unbound base/subgrade support/stiffness parameters, such as CBR and resilient modulus. The stiffness values obtained through correlation can then be compared with the typical values to identify weak material layers and assess current structural adequacy.

Table 2.5.18. Typical FWD backcalculated dynamic modulus subgrade reaction for rigid pavements (*19*, 20, *22*).

Subgrade Material Type	**Range of Dynamic Modulus of Subgrade Reaction, Values, psi/in***		
	Low	**High**	**Mean**
Gravel	350	450	400
Coarse sand	200	400	300
Fine sand	150	300	225
Silt	25	165	95
Silty gravel	300	500	400
Plastic clay	25	255	140
Moderately plastic clay	25	215	120
Highly plastic clay	40	220	130

* Dynamic k-value is approximately twice the conventional static k-value.

Summary

The structural adequacy of a pavement can be viewed in two different terms, current structural adequacy and future structural adequacy. The procedures required for both current and future structural adequacy assessment are summarized in the following sections.

Current Structural Adequacy

Current structural adequacy is assessed through a combination of:

- Load-related distress.
- Material durability.
- Backcalculated layer elastic moduli.
- Visual examination of pavement cores to determine layer condition, bonding, and other defects.
- Physical testing of cores to determine moduli and strength.

The engineer using the assembled information and data assesses the current structural adequacy of the existing pavement following the guidelines presented in this section. A thorough assessment will answer the following questions:

- What is the current structural adequacy of the existing pavement?
- Does the pavement need a structural improvement now?

Future Structural Adequacy

Future structural adequacy can only be assessed through a structural analysis of the existing pavement that takes the following into account:

- Past damage from traffic and climate-related loads.
- Current condition.

- Expected future traffic and climate-related damage.

Procedures for estimating past damage based on current condition determined using procedures presented in this chapter are included in PART 3, Chapters 6 and 7. Estimated past damage is an important component for assessing the future structural adequacy of rehabilitated pavements (e.g., JPCP restoration, concrete overlays over exiting hot mix AC, and hot mix AC over existing hot mix AC). Past damage estimates are however not required for PCC over existing concrete pavements or where existing PCC pavements are to be fractured before overlay.

2.5.3.2 Functional Adequacy

The functional adequacy of a pavement is a measure of how well the pavement is performing its intended function of providing a smooth, safe ride to the highway user (*25*). The two primary components of functional adequacy are smoothness and friction resistance. As mentioned previously in this chapter, smoothness is synonymous with ride quality and rideability. It can be measured and reported using a vast array of surveying equipment and techniques and reported using several indices. The Design Guide uses IRI to report pavement smoothness (*26*). Suggested smoothness criteria are provided in table 2.5.19; frictional resistance criteria may be suggested by the highway agencies.

Table 2.5.19. Recommended IRI levels for assessing pavement smoothness.

<table>
<tr><th rowspan="2">Pavement Type</th><th rowspan="2">Highway Classification</th><th colspan="3">IRI (in/mile) Level Regarded As:</th></tr>
<tr><th>Inadequate (Not smooth)</th><th>Marginal (Moderately Smooth)</th><th>Adequate (Smooth)</th></tr>
<tr><td rowspan="3">Flexible and rigid</td><td>Interstate-Freeway</td><td>>175</td><td>100 to 175</td><td><100</td></tr>
<tr><td>Primary</td><td>>200</td><td>110 to 200</td><td><110</td></tr>
<tr><td>Secondary</td><td>>250</td><td>125 to 250</td><td><125</td></tr>
</table>

Table 2.5.19 offers guideline IRI values for assessing the adequacy of pavement smoothness. It lists the ranges of adequate, marginal, and inadequate IRI for the major classes of highways. As can be seen, high-type pavements with an IRI greater than 175 in/mi create significant user discomfort and are therefore considered functionally inadequate. For secondary highways, on the other hand, IRI values up to 250 in/mi are considered adequate because secondary highways are generally much lower volume than Interstates/freeways and travel speeds are often lower.

2.5.3.3 Drainage Adequacy

A significant amount of information on subdrainage is given in PART 3, Chapter 1. This section focuses on the evaluation of existing pavements only. As outlined in the *Techniques for Pavement Rehabilitation* manual, drainage adequacy is a function of external and internal factors (*25*). External drainage factors are the climatic conditions in an area that regulate the supply of moisture to the pavement. Locations with high amounts of annual precipitation (>20 in/year), high-intensity rainfall, intensive seasonal precipitation, and freeze-thaw conditions generally require special consideration for drainage.

Internal drainage factors are the roadway design characteristics and material properties that influence the action of moisture in a pavement system. Such factors consist of the permeability of the pavement surface, the drainability of base/subbase layers and the subgrade soil, the cross-sectional design of the pavement structure (i.e., "bathtub" designs), the longitudinal grade of the roadway and ditches, and the inclusion of surface and subsurface drainage facilities.

The effects of all these factors on drainage adequacy are evidenced by pavement surface distresses and/or deficiencies in existing drainage facilities. In addition to gathering and evaluating the existing pavement's design and construction records, proper assessment of drainage requires conducting both visual distress surveys and drainage surveys, and examining in detail the respective results.

Visual Distress Survey Results

Data from visual distress surveys will reveal the types and extents of distresses present in the pavement that are either caused by or accelerated by moisture. Such distresses include, but are not limited to, fatigue cracking, rutting, and stripping for flexible pavements and pumping, faulting, corner breaks, D-cracking, and reactive aggregate for rigid pavements. Drainage is considered inadequate or marginal if any of these key distress types are present to the extents listed in table 2.5.20.

Drainage Survey Results

Information collected from drainage surveys will indicate the presence, condition, and functionality of surface and subsurface drainage facilities, such as ditches, longitudinal edge drains (outlets in particular), transverse drains, permeable and daylighted bases, joint and crack sealants, and various drainage structures (e.g., culverts, storm drains, curb-and-gutter).
The survey will also produce information concerning the pavement cross-slope, longitudinal grades, and locations of cut and fill, all of which factor into the influence of moisture on a pavement. Based on the results of drainage surveys, pavement drainage is considered inadequate or marginal if any of the following conditions are prevalent throughout the project:

- Water remains in pavement joints and cracks for an extended period of time following a rainfall.
- If present, joint and crack sealants are failing to perform their intention of keeping water from entering the pavement system.
- Water ponds on the travel lane or shoulder, due to build-ups or other obstructions at the lane or shoulder edge.
- Outlets on subsurface drainage facilities (edge drains, daylighted bases) are obstructed or are below ditchlines.
- Subsurface drainpipes or membranes are obstructed (as determined by video cameras).
- Ditchlines are at or above the top of the subgrade.
- Ditches have considerable amount of standing water and/or water-loving vegetation (e.g., cattails and willows), due to obstructions or flat slope.
- Inlets are obstructed or not at grade.

Table 2.5.20. Distress types and levels recommended for assessing drainage adequacy.

Moisture-Related Distress	Highway Classification	Distress Level Regarded As:		
		Inadequate	Marginal	Adequate
AC stripping	Interstate-Freeway	Signs of the distress	No signs of the distress	No signs of the distress
	Primary	Signs of the distress	No signs of the distress	No signs of the distress
	Secondary	Signs of the distress	No signs of the distress	No signs of the distress
AC pumping (fines from underlying layers)	Interstate-Freeway	Signs of the distress	No signs of the distress	No signs of the distress
	Primary	Signs of the distress	No signs of the distress	No signs of the distress
	Secondary	Signs of the distress	No signs of the distress	No signs of the distress
JPC and JRC Pumping (all severities), % joints	Interstate/ Freeway	>25	10 to 25	<10
	Primary	>30	15 to 30	<15
	Secondary	>40	20 to 40	<20
JPC Mean Transverse Joint/Crack Faulting, in	Interstate/ Freeway	>0.15	0.1 to 0.15	<0.1
	Primary	>0.20	0.125 to 0.20	<0.125
	Secondary	>0.3	0.15 to 0.3	<0.15
JRC Mean Transverse Joint/Crack Faulting, in	Interstate/ Freeway	>0.3	0.15 to 0.3	<0.15
	Primary	>0.35	0.175 to 0.35	<0.175
	Secondary	>0.4	0.2 to 0.4	<0.2
PCC Durability (all severity levels of D-cracking and reactive aggregate durability distress)	All	Predominantly medium- and high-severity	Predominantly low- and medium-severity	None or some low-severity
JPC and JRC Corner Breaks (all severities), #/mi	Interstate/ Freeway	>25	10 to 25	<10
	Primary	>30	15 to 30	<15
	Secondary	>40	20 to 40	<20

Drainage assessment can also be benefited by data obtained from coring and material testing. The permeability and effective porosity of base/subbase materials, as determined through laboratory tests or calculated from gradations, can be used to quantify drainability. Moreover, using county soil maps, the drainability of the subgrade soil can be estimated. A comprehensive procedure for assessing the need for drainage facilities is described in PART 3, Chapter 1.

2.5.3.4 Material Durability

Material durability problems are the result of adverse chemical or physical interactions between a paving material and the environment. In rigid pavements, the problems occasionally manifested and the interactions causing the problems are as follows:

- D-cracking in PCC—the fracture of layer aggregate particles, and subsequently the PCC mortar, as a result of water freezing (and expanding) in the pores of moisture-susceptible coarse aggregate.
- Alkali reactivity distress in PCC—map cracking and joint deterioration resulting from the reaction of high silica or high carbonate aggregates and alkalies (sodium and potassium) in portland cement. The reaction produces a gel that absorbs water and swells, thus fracturing the cement matrix.
- Freeze-thaw damage in PCC—spalling and scaling of PCC in freeze-thaw climates due to inadequate entrained air voids. The lack of entrained air restricts the internal expansion of water in concrete during periods of freezing and thawing.
- Steel corrosion in doweled and continuously reinforced PCC—for pavements located in regions where de-icing salts are used, the salts can lead to the corrosion of embedded steel dowels or reinforcing steel (including tie bars).
- Treated base/subbase disintegration—stripping of asphalt cement by water in asphalt-treated materials, or the disintegration of cement-treated materials due to freeze thaw cycles.
- Unbound base/subbase contamination by fines from subgrade.

In flexible pavements, the problems occasionally manifested and the interactions causing the problems are as follows:

- Moisture damage (stripping) identified by pavement shoving, bleeding, or rutting—caused by the trapping of moisture in lower or intermediate layers of the pavement as a result of by high voids/low density of the mix or the presence of excessive amount of material passing the No. 200 sieve in the mix resulting in a high fines/asphalt ratio.
- Raveling—disintegration of the AC material caused by the lack of adequate compaction, constructed in cold or wet weather, use of dirty aggregate, use of a mix with inadequate asphalt cement "dry" mix, or the over heating of the AC mix during construction.
- Bleeding—caused by excessive amounts of asphalt in the mix, low air voids, or excess amounts of prime or tack.
- Unbound base/subbase contamination by fines from subgrade.

Pavement material durability is best assessed using information from visual condition surveys and from core samples. Tables 2.5.21 and 2.5.22 provide guidelines for determining whether material durability is an issue that should be addressed, based on the types and levels of durability-related distress observed for flexible and rigid pavements, respectively.

Table 2.5.21. Distress types and levels recommended for assessing flexible and composite pavement material durability.

Durability-Related Distress	Highway Classification	Distress Level Regarded As:		
		Inadequate	Marginal	Adequate
Raveling, percent of total area	Interstate-Freeway	Loss of coarse aggregate	—	Loss of fine aggregate
	Primary	>50	10 to 50	<10
	Secondary	>100	45 to 100	<45
Rutting, mean depth of both wheel paths, mm	Interstate-Freeway	>10	6 to 10	<6
	Primary	>13	9 to 13	<9
	Secondary	>20	10 to 20	<10
Shoving, percent of wheel path area.	Interstate-Freeway	>10	1 to 10	None
	Primary	>20	10 to 20	<10
	Secondary	>45	20 to 45	<20
Block Cracking, crack width, mm	Interstate-Freeway	Noted	—	None
	Primary	>6	—	<6
	Secondary	>6	—	<6
Bleeding, percent of wheel path area	Interstate-Freeway	>10	5 to 10	<5
	Primary	>25	10 to 25	<10
	Secondary	>50	20 to 50	<20
Stripping (Treated Base/Subbase)	All	Unable to recover majority of cores due to disintegration or stripping Some pumping of fines onto shoulder may be observed	Unable to recover some cores due to disintegration or stripping Some pumping of fines onto shoulder may be observed	Cores are predominantly intact No sign of pumping of fines from beneath pavement
Unbound granular base contamination	All	Contamination of unbound granular base/subbase with fines from subgrade		

Table 2.5.22. Distress types and levels recommended for assessing rigid pavement material durability.

Durability-Related Distress	Highway Classification	Distress Level Regarded As:		
		Inadequate	Marginal	Adequate
PCC Durability (D-cracking and ASR)	All	Predominantly medium- and high-severity, with significant spalling and disintegration at joints	Predominantly low- and medium-severity, with some spalling and disintegration at joints	None or predominantly low-severity
JPC and JRC Patch/Patch Deterioration (medium- and high-severity), % surface area	Interstate/ Freeway	>10	5 to 10	<5
	Primary	>15	8 to 15	<8
	Secondary	>20	10 to 20	<10
CRC Patch/Patch Deterioration (medium- and high-severity), % surface area	Interstate/ Freeway	>5	2 to 5	<2
	Primary	>10	5 to 10	<5
	Secondary	>15	8 to 15	<8
PCC Longitudinal Joint Spalling (medium- and high-severity), % length	Interstate/ Freeway	>50	20 to 50	<20
	Primary	>60	25 to 60	<25
	Secondary	>75	30 to 75	<30
JPC Transverse Joint Spalling (medium- and high-severity), joints/mi	Interstate/ Freeway	>50	20 to 50	<20
	Primary	>60	25 to 60	<25
	Secondary	>75	30 to 75	<30
JRC Transverse Joint Spalling (medium- and high-severity), joints/mi	Interstate/ Freeway	>25	10 to 25	<10
	Primary	>30	15 to 30	<15
	Secondary	>40	20 to 40	<20
Unbound granular base contamination	All	Contamination of unbound granular base/subbase with fines from subgrade		
Stripping (Treated Base/Subbase)	All	Unable to recover majority of cores due to disintegration or stripping Some pumping of fines onto shoulder may be observed	Unable to recover some cores due to disintegration or stripping Some pumping of fines onto shoulder may be observed	Cores are predominantly intact No sign of pumping of fines from beneath pavement

2.5.3.5 Maintenance Applications

Visual distress surveys and discussions with local maintenance personnel will provide the information necessary to determine if past maintenance on a pavement has reached an excessive level. The amount of full- and partial-depth patching (AC or PCC) that exists as a result of alligator cracking, transverse crack deterioration, slab cracking, joint deterioration, punchouts, and other major surface distresses is the primary indicator of past maintenance. However, other maintenance treatments, such as selected resurfacing, crack sealing and joint resealing, spot grinding of joints, and subsealing of slabs, should also be taken into consideration.

Levels of maintenance and their condition (e.g., patch deterioration) should be considered in determining the structural adequacy of an existing pavement. Suggested guidance as to what levels of maintenance are deemed to reduce a pavement structural condition to marginal or inadequate for the three different highway classes are provided in table 2.5.23. As can be seen, these levels are defined in terms of the percent surface area that has been (a) patched through partial- or full-depth repairs and selected resurfacing and (b) treated through microsurfacing, spot grinding, or subsealing. It is assumed that the patches, resurfacing, spot grinding, and so on are deteriorated.

Table 2.5.23. Patching levels recommended for assessing past maintenance.

Maintenance Application[1]	**Highway Classification**	**Distress Level Regarded As:**		
		Inadequate	**Marginal**	**Adequate**
%Surface area of flexible pavement with deteriorated patching and other repairs	Interstate/ Freeway	>15	8 to 15	<8
	Primary	>20	10 to 20	<10
	Secondary	>25	12 to 25	<12
%Surface area of JPC and JRC with deteriorated patches, resealed joints, replaced slab, and so on	Interstate/ Freeway	>15	8 to 15	<8
	Primary	>20	10 to 20	<10
	Secondary	>25	12 to 25	<12
%Surface area of CRC pavement with deteriorated patching and other repairs	Interstate/ Freeway	>8	3 to 8	<3
	Primary	>12	5 to 12	<5
	Secondary	>15	10 to 15	<10

[1]Pavements with excessive amounts of deteriorated repairs and other maintenance activities are most likely to be structurally compromised. This should be considered in evaluating the structural adequacy of a pavement. The effect of deteriorated repairs on the functionality of the pavement will be detected by determining the smoothness of the pavement as outlined in table 2.2.19.

2.5.3.6 Shoulders Adequacy

The design and condition of existing shoulders should also be considered when doing an overall assessment of a pavement because planned rehabilitation is largely dependent upon the types of materials (AC, PCC, granular) that make up the shoulder and the kinds of distresses that exist. Condition information needed to assess shoulder adequacy should include, as a minimum, data from visual distress surveys. NDT data, if available, can be particularly beneficial to this analysis if the shoulders are expected to support significant truck loadings in the future.

The same surface distress types used to assess pavement structural adequacy should be used to assess the adequacy of the shoulders. For tied concrete shoulders, such distresses include transverse cracking and corner breaks, pumping, faulting, patch/patch deterioration, and longitudinal and transverse joint spalling. For AC shoulders, such distresses include fatigue cracking, thermal cracking, raveling and weathering, rutting, potholes, and patch/patch deterioration. Also to be considered for both shoulder types are lane–shoulder drop-offs and lane–shoulder separation.

Generally, the threshold levels for these distresses appearing in shoulder pavements will be somewhat higher than the levels specified for pavements in traffic lanes.

2.5.3.7 Variability Along the Project

There are several forms of variability within a project that should be considered when assessing overall condition. Such variability is generally defined in terms of changes in structural or functional conditions that are the result of variations in pavement structure, construction quality, traffic loadings, subgrade properties, and topography. The main forms of variability, which can and should be delineated using distress, NDT, and possibly destructive testing data, include the following:

- Variations of condition over the project length.
- Lane-to-lane variations in condition (a valuable observation because of the typical difference in truck traffic between lanes).
- Variations occurring at intersections or interchanges (due to slower moving trucks).
- Variations occurring at bridge approach and leave areas (due to settlements, pushing, and excessive joint openings).
- Cut and fill section variations.

By delineating locations with different conditions and comparing the results with comprehensive design and construction data, the causes of variation may be able to be identified. And, by knowing the causes of variation, a more appropriate set of rehabilitation alternatives can be developed which better address the problems that do exist along the project.

2.5.3.8 Miscellaneous

<u>Rigid Jointed Pavements</u>

Deficiencies of transverse, centerline, and lane-to-shoulder joints of jointed concrete pavements may be manifested as several types of distress, including inadequate load transfer, joint seal damage, separation, pumping, faulting, spalling, and cracking. In general, distresses located close to the pavement joint result in a weakening of the joint, which creates excessive deflections and reduced load transfer. The reduced load transfer, in turn, leads to additional joint deterioration and cracking.

The overall joint condition can be assessed by determining the extent and severity of joint distresses and the ability of a joint to transfer load from one side of a joint to the other (load transfer efficiency, LTE). Good load transfer efficiency is a major factor in a pavement's structural performance. Table 2.5.24 presents guideline distress levels for determining the adequacy of PCC joints.

Table 2.5.24. Distress types and levels recommended for assessing PCC joint condition.

Joint-Related Distress	Highway Classification	Distress Level Regarded As:		
		Inadequate	**Marginal**	**Adequate**
JPC and JRC Mean Load Transfer Efficiency, %	All	<60	60 to 80	>80
JPC and JRC Transverse Joint Seal Damage (medium- and high-severity), % joints	All	>40	15 to 40	<15
JPC and JRC Pumping (all severities), % joints	Interstate/Freeway	>25	10 to 25	<10
	Primary	>30	15 to 30	<15
	Secondary	>40	20 to 40	<20
JPC Mean Transverse Joint/Crack Faulting, in	Interstate/ Freeway	>0.15	0.1 to 0.15	<0.1
	Primary	>0.20	0.125 to 0.20	<0.125
	Secondary	>0.3	0.15 to 0.3	<0.15
JRC Mean Transverse Joint/Crack Faulting, in	Interstate/ Freeway	>0.3	0.15 to 0.3	<0.15
	Primary	>0.35	0.175 to 0.35	<0.175
	Secondary	>0.4	0.2 to 0.4	<0.2

2.5.4 SUMMARY

The assessment of overall pavement condition requires that the following eight facets of the existing pavement be investigated thoroughly:

- Structural adequacy (current and future).
- Functional adequacy.
- Drainage adequacy.
- Materials durability.
- Maintenance applications.
- Shoulders adequacy.
- Project variability.
- Miscellaneous constraints

Information needed to assess these facets are obtained from a variety of sources, including design and construction records, visual distress surveys, smoothness testing, drainage surveys, NDT, destructive testing, and maintenance records.

After completing each investigation, the engineer will better understand the real problems of the existing pavement. Once the problems have been clearly defined, the next task will involve identifying the rehabilitation strategies that best address the problems in a cost-efficient manner.

As noted in previous sections, the pavement rehabilitation selection process should involve assessing the overall condition of the existing pavement and fully defining the existing pavement problems. To avoid making an inaccurate assessment of the problem, the engineer should collect and evaluate sufficient information about the pavement. Table 2.5.1 can help in this regard. It contains a comprehensive list of factors designed to hone in on the problems that should be addressed. This list should be modified by each agency to suit their particular needs. It is vital that agencies develop procedures and guidelines for consistently answering the questions on their list.

Finally, it is important that the project engineer prepare a brief report that summarizes the data analysis and findings. The report should highlight the following:

- Project identification (including a brief statement on the history of the project and the major goals of the evaluation/rehabilitation project).
- Brief statement on the major findings of previous evaluations where available.
- Purpose of this evaluation.
- Dimensions of this evaluation (indicate whether this is a global or a partial evaluation. If global, specify the major dimensions or criteria of the evaluation. If partial, indicate the focus (e.g., structural evaluation).
- Sources of data and methods of used for data collection (include a brief justification for choosing these particular sources and methods).
- Summary and interpretation of results (e.g., major findings and strengths and deficiencies requiring attention)
- Remedial action and rehabilitation alternatives (i.e., actions recommended as a result of the evaluation and reasons for the recommendations).

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