ROAD AND BRIDGE WORKS

THE REPUBLIC OF UGANDA

MINISTRY OF WORKS AND TRANSPORT

ROAD DESIGN MANUAL

Volume 3: Pavement Design

Part IV: Pavement Rehabilitation

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TABLE OF CONTENTS

APPENDICES

ABBREVIATIONS AND DEFINITIONS

AADT Average Annual Daily Traffic

AASHTO American Association of State Highway and Transportation Officials

AC Asphalt concrete. High-quality hot mixture of asphalt cement and well-graded, high-quality aggregate, thoroughly compacted into a uniform dense mass.

BCEOM Bureau Central d'Etudes d'Outre Mer (France)

CBR California Bearing Ratio

CEBTP Centre d'Etudes du Batiment et des Travaux Publics (France)

CEM I Portland Cement US 310-1 (US = Uganda Standard)

CRCP Continuously Reinforced Concrete Pavement

CSRA Committee of State Road Authorities (South Africa)

DCP Dynamic Cone Penetrometer

Drainage Coefficients (m_2, m_3)

Factors used to modify layer coefficients in flexible pavements as a function of how well the pavement structure can handle the adverse effect of water infiltration

Equivalent Standard Axles (ESA)

Summation of equivalent 8.2 t single axle loads used to combine mixed traffic to design traffic class

FWD Falling Weight Deflectometer

IRI International Roughness Index

Is Global damage index (used in the VIZIR method)

JPCP Jointed Plain Concrete Pavement

JRCP Jointed Reinforced Concrete Pavement

Layer Coefficients (a_1, a_2, a_3)

The empirical relationship between structural number (SN) and layer thickness which expresses the relative ability of a material to function as a structural component of the pavement

LCPC Laboratoire Central des Ponts et Chaussees (France)

Maintenance Routine work performed to keep a pavement, under normal conditions of traffic and forces of nature, as nearly as possible in its as-constructed condition.

NDT Non-Destructive Testing

Overlay One or more courses of asphalt construction on an existing pavement. The overlay often includes a leveling course, to correct the contour of the old pavement, followed by a uniform course or courses to provide needed thickness.

PMS Pavement Management System

Q_I Pavement Quality Rating (used in the VIZIR method)

Reconstruction

The process by which a new pavement is constructed, utilizing mostly new materials, to replace an existing pavement.

Recycling The reuse, usually after some processing, of a material that has already served its first-intended purpose.

Rehabilitation Work undertaken to significantly extend the service life of an existing pavement. This may include overlays and preoverlay repairs, and may include complete removal and reconstruction of the existing pavement, or recycling of part of the existing materials.

Roadbase A layer of material of defined thickness and width constructed on top of the subbase, or in the absence thereof, the subgrade.

RRD Representative Rebound Deflection

S1 to S6 Subgrade strength classes used to characterize the subgrade in pavement design (cf *MoWT-Volume III, Pavement Design Manual (Part 1, Flexible Pavement Design Guide)*.

SN_{eff} Effective structural number of an existing pavement.

SN_{new} Structural number of a new pavement.

Structural Number (SN)

An index number into which thickness of flexible pavement layers may be converted through the use of suitable layer coefficients related to the type of material being used in each layer of the pavement structure

Subbase The layer of material of specified dimensions on top of the subgrade and below the roadbase.

Subgrade The surface upon which the pavement structure and shoulders are constructed.

Surfacing The asphalt surfacing of a flexible pavement or the concrete slab of a rigid pavement.

T1 to T8 Traffic classes used to characterize the anticipated traffic in terms of ESA for flexible pavement design purposes (cf *MoWT-Volume III, Pavement Design Manual (Part 1, Flexible Pavement Design Guide)*.

T_1, T_2, T_3 Thicknesses (in centimeters) of pavement surface, base and subbase layers (existing or required)

TRL Transport Research Laboratory (UK)

VIZIR A method for quality evaluation of paved roads developed by the LCPC and used in PMS implementation

VOC Vehicle Operating Cost

1 INTRODUCTION

1.1 Purpose and Scope of the Manual

The purpose of this Asphalt Pavement Rehabilitation and Overlay Design Manual - Part 4 is to give specific guidance and recommendations to the engineers responsible for the maintenance and rehabilitation of existing pavements.

This Asphalt Pavement Rehabilitation and Overlay Design Guide - 2004 is based on a review of the design standards of several countries. Most chapters are based closely on the Transport Research Laboratory Overseas Road Note 31: A Guide to the Structural Design of Bitumen-Surfaced Roads in Tropical and Sub-Tropical Countries (ref.1). Of note is the fact that this reference document and companion TRL documents have drawn on the experience of TRL and collaborating organizations in several tropical and sub-tropical countries.

For that portion of the text dealing with surface treatments, the text is based once again on a TRL source, Overseas Road Note 3: A Guide to Surface Dressing in Tropical and Sub-Tropical Countries (ref. 2).

Other major reference sources include AASHTO, and, in particular, the AASHTO Guide for Design of Pavement Structures, as revised in 1993 (ref.6). References of the Asphalt Institute were reviewed for asphalt concrete and other hot- and cold-mix types. South African and Kenyan references were reviewed to assist in the development of a design well suited for the eastern African region.

As the network of paved roads in Uganda evolves, it is likely that a gradual shift in emphasis will occur, from new design and construction to maintenance and rehabilitation of the existing paved network.

The design of new pavements is covered in the *MoWT-Volume III, Pavement Design Manual (Part 1, Flexible Pavement Design Guide and Part 2 Rigid pavement Design Guide)*. The purpose of this manual is to provide, as a complement, a guide for the design of the most common rehabilitation solutions applicable to paved roads, and for the selection of an appropriate rehabilitation alternative for a particular project.

This manual provides design procedures for the rehabilitation of both flexible and rigid pavements. The procedures utilize data, which are collected during desk studies as well as by means of fieldwork.

Several rehabilitation alternatives, including asphalt overlays, are usually available to the design engineer from a technical viewpoint; the manual provides guidance regarding the choices, practical and economical, between the various alternatives.

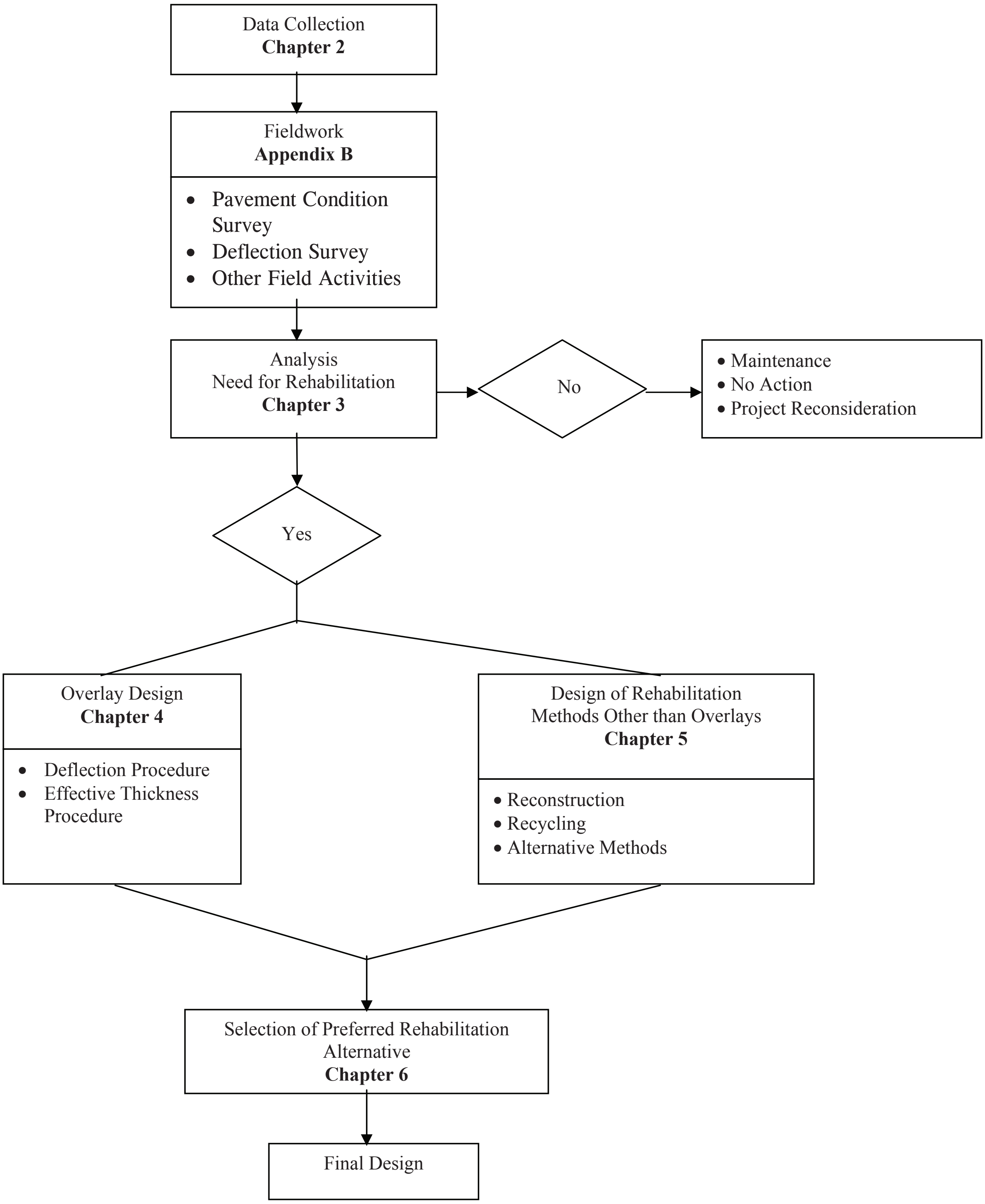
Careful consideration to sound engineering practice shall be observed in the use of the Manual, and under no circumstances shall the Manual waive professional judgment in applied engineering. As this Manual due to Technological development and change, requires periodic updating, comments and suggestions on all aspects from any concerned body, group or individual as feedback during its implementation is expected and will be highly appreciated.

1.2 Organization of the Manual

The manual is organized in a manner designed to satisfy its purpose and scope as identified above, and to guide its user along a natural progression following a recommended procedure.

The procedure and the organization of the manual are illustrated in the flow diagram in Figure 1.1.

In Chapter 2, the user of the manual is taken through the steps required to collect an array of data relevant to the rehabilitation project. This data includes various elements from different sources (e.g. past road inventories, as-built pavement structures, unit prices, etc.), essentially gathered as part of a desk study, and preferably prior to initiating the collection of additional data in the field. One of the important aspect of this task consists of collecting relevant traffic data.


Data Collection
Chapter 2
Fieldwork
Appendix B
• Pavement Condition Survey
• Deflection Survey
• Other Field Activities
Analysis
Need for Rehabilitation
Chapter 3
No
• Maintenance
• No Action
• Project Reconsideration
Yes
Overlay Design
Chapter 4
• Deflection Procedure
• Effective Thickness Procedure
Design of Rehabilitation Methods Other than Overlays
Chapter 5
• Reconstruction
• Recycling
• Alternative Methods
Selection of Preferred Rehabilitation Alternative
Chapter 6
Final Design

Details of the data collection that needs to be carried out in the field are presented in Appendix B. Of primary importance among these are a detailed pavement condition survey and a deflection survey. Other tasks to be performed in the field are field testing and sampling of materials, drainage survey, etc. The methods presented cover both flexible and rigid pavements.

Chapter 3 outlines the method used to verify or confirm the need for rehabilitation (as opposed to maintenance) for a particular project, using the data collected which are normally more complete, recent and reliable than the data available at the time of the initial determination. In some cases, the need for rehabilitation may not be confirmed, in which case more cost-efficient projects or maintenance expenditures may need to be sought.

In Chapter 4, flexible and rigid pavements are treated separately, and for each, two methods of overlay design are considered. Chapter 5 details the various alternatives to overlay, including reconstruction and recycling.

Finally, as the project nears the completion of its design phase, the user of the manual will be guided through methods aimed at selecting, from among the feasible rehabilitation alternatives, the alternative best suited to optimize the satisfaction of all the rehabilitation objectives, considering economic and practical concerns. This is the purpose of Chapter 6.

1.3 The Pavement Management Context

Pavement management, in its broadest sense, encompasses all the activities involved in the planning, design, construction, maintenance, evaluation and rehabilitation of the pavement portion of a public works program. A pavement management system (PMS) is a set of tools or methods that assist decision makers in finding optimum strategies for providing, evaluating, and maintaining pavements in a serviceable condition over a given period of time. The function of a PMS is to improve the efficiency of decision-making, expand its scope, provide feedback on the consequences of decisions, facilitate the coordination of activities within the agency, and ensure the consistency of decisions made at different management levels within the agency.

In this sense, pavement "design", as covered by *MoWT-Volume III, Pavement Design Manual (Part 1, Flexible Pavement Design Guide and Part 2 Rigid pavement Design Guide)* and "rehabilitation", as covered in this manual, are essential parts of the overall pavement management process, a conceptual illustration of which is given in Figure 1.2 (after Ref. 3 and 4).

As can be seen on Figure 1.2, it is convenient and customary to describe a PMS in terms of two generalized levels: (1) the network management level, and (2) the project level.

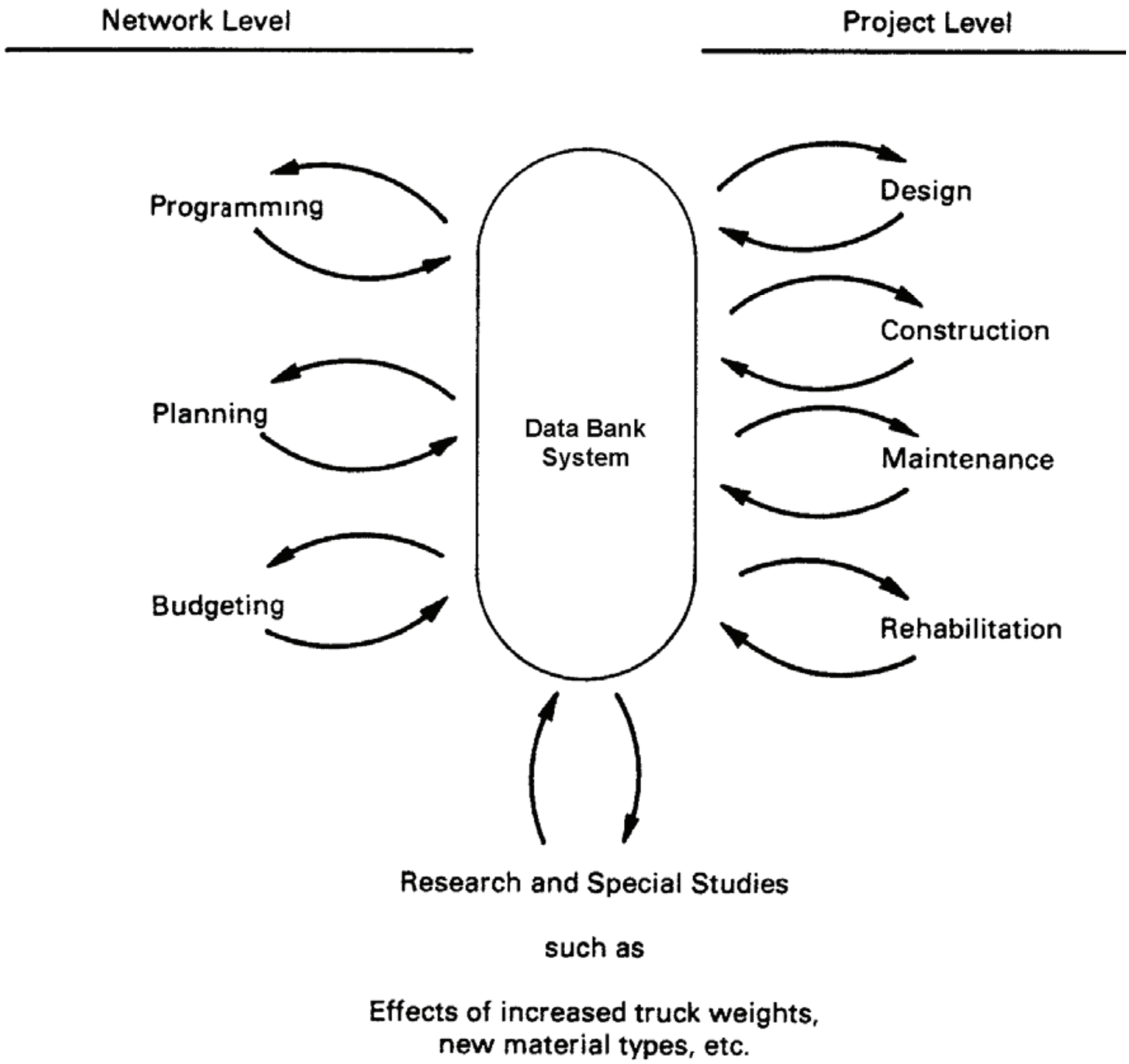


Figure 1.2 Activities of a Pavement Management System

At the network level, the pavement management system gives information relevant to the development of programs involving the road infrastructure considered as a group of itineraries, identified by their functional class and their main technical characteristics and level of traffic. Basically, the pavement management system deals primarily with network level where key administrative decisions that affect programs for road networks are made. It allows one to evaluate and compare series of maintenance activities and maintenance and rehabilitation alternatives, and to test them with regard to budget constraints.

The project level is the appropriate level for evaluating the detailed pavement conditions of a section of an itinerary, and for designing technical solutions for routine, periodic maintenance, or rehabilitation, according to road management priorities, selected cost-efficient policy alternatives, and budget constraints. Here again, by comparing the benefits and costs associated with several alternative activities, an optimum strategy is identified that will provide the desired benefits at the least cost over a selected analysis period.

1.3.1 Maintenance and Rehabilitation

Two terms, "maintenance" and "rehabilitation", deserve particular attention in order to fully understand and better benefit from this manual. As already indicated, the PMS aims at allocating resources between maintenance and rehabilitation, and an outline of the method used for this purpose is given in Chapter 4. It is therefore appropriate to define what is covered by these two groups of activities.

a) Maintenance

The following excerpt from the introduction to the Asphalt Institute Manual Series No. 16, *Asphalt in Pavement Maintenance* (Ref. 6) is considered appropriate to define maintenance, as well as to provide some insight into two general categories of maintenance:

"Pavement maintenance is not easy to define. Maintenance departments generally agree what it is, but there are some minor differences. Some call pavement improvement "maintenance". Others include only the work that keeps the pavement in its as-constructed condition.

Taking all these into consideration, the definition that seems to fit best is:

- Pavement maintenance is work performed from time to time to keep a pavement, under normal conditions of traffic and forces of nature, as nearly as possible in its as-constructed condition.

Distinctions are usually made between forms of maintenance, based on their required frequency. The International Road Maintenance Handbook (Ref. 7) uses the grouping of "routine" and "periodic" maintenance, while other sources (e.g. TRRL, Ref. 8) use "routine", "recurrent", "periodic" and "urgent". The following excerpt from Reference 8 illustrates these categories:

"… There are four categories:

- **routine** maintenance, required continually on every road, whatever its engineering characteristics or traffic volume
- **recurrent** maintenance, required at intervals during the year with a frequency that depends on the volume of traffic using the road
- **periodic** maintenance, required only at intervals of several years
- **urgent** maintenance, needed to deal with emergencies and problems calling for immediate action when a road is blocked.

Examples of activities within these categories are as follows:

Routine:
Grass cutting; drain clearing; recutting ditches; culvert maintenance; road signs maintenance

Recurrent on paved roads:
Repairing pot-holes; patching; repairing edges; sealing cracks

Periodic on paved roads:
resealing (surface dressing, slurry sealing, fog spray, etc.); regravelling shoulders; road surface marking

Urgent:

Removal of debris and other obstacles; placement of warning signs and diversion works."

b) Rehabilitation

The following excerpt from the AASHTO Guide (Ref. 3) illustrates the essential difference between maintenance and rehabilitation:

"Major rehabilitation activities differ markedly from periodic maintenance activities (sometimes called normal, routine and/or preventive maintenance) in that the primary function of the latter activity is to preserve the existing pavement so that it may achieve its applied loading, while rehabilitation is undertaken to significantly increase the functional life."

In this manual, the following definition of rehabilitation, which also introduces the major rehabilitation categories, will be retained:

"Work undertaken to significantly extend the service life of an existing pavement. This may include overlays and pre-overlay repairs, and may include complete removal and reconstruction of the existing pavement, or recycling of part of the existing materials."

Rehabilitation is the subject of this manual, and is given further scrutiny hereunder.

i) Rehabilitation Alternatives

In the context of pavement management at a project level, and as noted earlier, several rehabilitation alternatives are usually available after technical evaluation, and will need to be compared. It is therefore appropriate, before a more detailed description of these alternatives is given, to outline their main characteristics and the classification used in this manual.

For simplicity, rehabilitation is subdivided into two major categories:

(1) Rehabilitation Methods with Overlays
(2) Rehabilitation Methods other than Overlays

It is realized that various agencies define overlays in different ways, e.g. some sources would consider the addition of unbound courses as overlay. For clarity, and consistent with the scope of this manual, the following definition of an overlay will be adopted for use in this manual:

"One or more courses of asphalt construction on existing pavement. The overlay often includes a leveling course, to correct the contour of the old pavement, followed by a uniform course or courses to provide needed thickness."

Bearing in mind the distinction between maintenance and rehabilitation, thin overlays will not be emphasized in the manual, nor will resurfacing with surface dressing or overlays of short (spot) length. It is noteworthy that asphalt concrete overlays of 5 cm are considered periodic maintenance in the context of the Pavement Management System (Ref. 5).

It is also realized, and it should be recognized, that some of the methods which fall under “rehabilitation methods other than overlays” may be used/required as pre-overlay treatments in major rehabilitation work.

Rehabilitation methods other than overlays include, as broad primary categories:

- Reconstruction: in this category, little or no contribution is expected from the existing pavement materials and the materials needed for rehabilitation will be mostly new materials.
- Recycling: the rehabilitation takes advantage of the existing pavement materials, which are re-used in part or as a whole, in the construction of the rehabilitated pavement.

The above categories are general in nature, and some combinations of methods are possible and indeed used. For instance, the materials of an existing surface treatment and road base may be recycled as the road base or sub base of a reconstructed pavement, with new materials brought on site to construct the upper layers, e.g. road base and AC surface.

In addition to the above primary categories of non-overlay rehabilitation methods, other rehabilitation techniques (some of them, as noted, being also applicable prior to an overlay) can be applied to pavements to significantly extend their lives without the placement of an overlay, or may delay recycling or reconstruction for several years. These techniques, however, in order to truly qualify as rehabilitation techniques, must satisfy several criteria:

1) They must be applied only to pavements which are structurally adequate to support future traffic loadings over the design period without structural improvement from an overlay. Only structurally adequate pavements, or pavements restored to a structurally adequate state, are candidates for rehabilitation without overlay.
2) They must address the cause(s) of the pavement distress and be effective in both repairing existing deterioration and preventing its recurrence. For this, a combination of techniques may be required (one repair method and one preventive technique).

If each of the repair and preventive methods meet the pavement’s needs and satisfy the imposed constraints (such as available funding and minimum life extension), then they qualify as feasible rehabilitation alternatives.

If the alternative considered fails to satisfy the above criteria, it will be better classified under the term of maintenance.

Examples of major rehabilitation methods that may be employed as non-overlay techniques include:

(1) Full-Depth Repair
(2) Partial Depth Patching
(3) Joint-Crack Sealing
(4) Subsealing-Undersealing
(5) Grinding and Milling
(6) Subdrainage
(7) Pressure Relief Joints

(8) Load Transfer Restoration
(9) Surface Treatments

These methods are defined and described further in Chapter 5 of the manual.

Finally, non-overlay rehabilitation methods also include, as required when these elements have become deficient, geometric improvements and/or drainage improvements or restoration.

2. DATA COLLECTION

In this chapter, general guidance is given relative to the gathering of data other than the data which needs to be collected in the field.

It is recommended that this collection and review of data be conducted prior to the field activities, as it can influence the latter with regard to their scope and organization.

The collection includes a search through the PMS records, as detailed below. Conversely, the results of this collection, to the extent that they can update and/or complete the PMS records, should be made available to the Branch or Office in charge of the PMS records.

2.1 PMS Records

These records are among the first sources of data to be sought and collected. Normally, the data required for the PMS are regularly updated. They fall into three categories, as follows:

1) Data specifically collected for the PMS. This data deals with road inventory and road conditions. It includes the results of visual inspection, roughness and deflection surveys, and geotechnical investigations.

The visual inspection includes information about road geometry, road inventory features (length; width; profile; alignment; type of structures encountered; etc.), and about road damage (deformation; cracks; potholes; rutting; gullies; etc.). This information is extremely valuable in assessing the road condition; it is usually gathered along the entire length of the road.

Roughness is normally measured using a Bump Integrator and expressed through the International Roughness Index (IRI). Typical values of the IRI, depending upon the type of road and its condition, are as follows:

Table 2.1 Condition of the Road vs. IRI

IRI Ranges	Road Condition
Lower than 6	very good
6 to 11	good
11 to 15	fair
15 to 19	poor
Larger than 19	very poor

While the road roughness is not directly involved in the structural design of the rehabilitation procedures presented in this manual, it may influence the choice and the method of effecting one alternative, e.g. by indicating a probable need for a leveling course prior to applying a structural overlay course.

Deflection surveys for the PMS are usually carried out using a Benkelman beam. Except for a variation in the magnitude of the load application, the procedure is similar to that described in this manual in Appendix B, and the user of this manual should refer to these sections for details. A review of the PMS deflection data and a comparison with the results of the deflection survey undertaken for the specific project should give insight into the evolution of the load carrying capacity of the pavement and/or increased confidence in the results of the specific survey.

Data specifically collected for the PMS also include detailed information about the subgrade, pavement materials, and pavement structure. This information would have been collected on sample sections by means of test pits with retrieval of samples for laboratory analysis. The data from the test pits would normally include:

a) Layers of the pavement structure: type, thickness, and maximum size of aggregates of each layer.

b) Subgrade: in-situ dry density; in situ moisture content; gradation; Atterberg limits; classification according to AASHTO and/or USCS systems; and in situ CBR obtained from correlation with DCP testing.

Here again, this data can be compared, and be a useful complement, to the project specific geotechnical data collected in the field as detailed in Appendix B.

2) Data required for the PMS, but that has usually been collected separately in the field and then forwarded to or obtained by the Branch or Office in charge of the PMS. This essentially includes traffic and axle load data.

The data should normally include AADTs, classification of the traffic among the various vehicle categories, and axle load data within each category. The data available from the Branch or Office in charge of the PMS may already have been processed and be readily available only in terms of traffic classes and cumulative equivalent axles. It will then be necessary to collect the raw data in order to reprocess them according to the procedure detailed further in this chapter (Section 2.3).

The usefulness of the traffic data thus collected will depend greatly on whether they are sufficiently up-to-date. Nevertheless, they should be a valuable complement to the evaluation of the traffic made for the specific project under consideration. In some cases, they may be sufficiently recent, reliable, and complete to be used, at least in part, for the project.

3) The third category of data normally available from the Branch or Office in charge of the PMS includes archive data such as theoretical pavement structures; ages of the pavement structures; unit prices of road works; vehicle operating costs and geoclimatic data.

General information about pavement structure and history should normally have been made available to the PMS through MoWT's Maintenance Divisions or in the stations, and includes:

- number and thickness of the pavement layers (surfacing, roadbase and subbase), together with a description of the material of each layer (asphalt, concrete, surface treatment, crushed basalt aggregates, etc.)
- the date of construction or rehabilitation of the pavement structure
- the type and date of major periodic maintenance activities

This data is very important to properly assess the road condition.

Unit rates of road works should also be available among the data in the PMS records. These costs will be useful in the economic comparison of the feasible rehabilitation alternatives, as detailed in Chapter 6 of this manual.

Vehicle operating costs (VOC) should not normally have an impact on the design of a specific project already selected for rehabilitation.

2.2 Other Data

A review of existing documents, in addition to those available from the Branch or Office in charge of the PMS, should be conducted prior to field activities.

Such a review includes essentially a desk study similar to that undertaken for the feasibility study of any road project.

The geological environment of the project should be researched, together with the geotechnical characteristics corresponding to the geological formations along the alignment.

Sources of information regarding past investigations, published or not, are to be gathered and reviewed. They may include activities only loosely related to road construction concerns (e.g. agricultural, hydrology, mining).

Existing reports, maps, mineral resource surveys, boring logs, and any relevant data need to be collected and compiled. This is particularly valuable in identifying road construction materials.

Of particular importance, when available, are as-built plans of the road section under consideration or of adjacent projects, together with any memoirs or design reports (including pavement design reports and/or pavement evaluation reports) and maintenance records.

2.3 Traffic Data

The rehabilitation procedures presented in this manual require a characterization of the traffic expected to be carried by the road after rehabilitation. Such a characterization is done in terms of a cumulative number of equivalent standard axles (ESAs). This method of characterization is necessary because the deterioration of paved roads due to traffic results from both the magnitude of the individual wheel loads and the number of times these loads are applied. Equivalency factors are used to convert traffic volumes into cumulative equivalent standard axles loaded at 8.2 metric tonnes.

The process by which the cumulative number of ESAs is determined is illustrated in Figure 2.1, and is essentially as described in the MoWT-Volume III, Pavement Design Manual (Part 1, Flexible Pavement Design Guide). The user of this rehabilitation manual should refer to these chapters for such details.

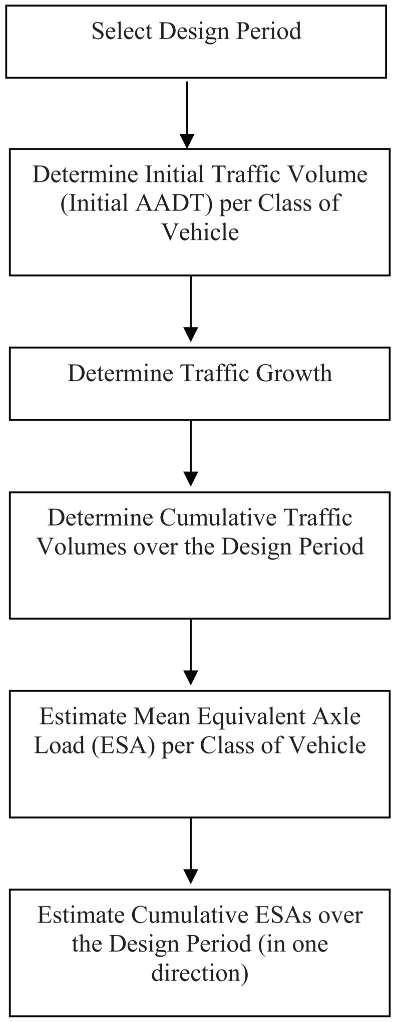


Figure 2.1 Traffic Evaluation

Some notes are, nevertheless, worth mentioning, as follows:

- The design period of a rehabilitation project may be subject to constraints of a different nature than those relative to a new construction project. In some cases, for instance, a rehabilitation project may be necessary during a relatively short period, during which an alternate alignment is being designed and/or constructed. The design period should be carefully ascertained at the outset of the rehabilitation design.

- Vehicle classification, initial traffic volume determination and cumulative traffic volumes over the design period shall be as detailed in the respective chapters of the MoWT-Volume III, Pavement Design Manual (Part 1, Flexible Pavement Design Guide).

- The conversion of traffic volumes into cumulative ESAs over the design period, as detailed in the MoWT-Vol*ume III, Pavement Design Manual (Part 1, Flexible Pavement Design Guide)* is effected through the use of axle load equivalency factors. These equivalency factors could be calculated using standard formulae; and are relative to single axle loads. For vehicles with multiple axles, i.e. tandems, triples, etc., each axle in the multiple group is considered separately, and weighted separately in axle load surveys.

It is noted that the rehabilitation methods presented in this manual are derived from methods proposed by the Asphalt Institute (e.g. Ref. 11), which uses equivalency factors for tandem axles. The use of equivalency factors considering each axle separately, is comparatively conservative and is recommended.

3 ANALYSIS FOR REHABILITATION

3.1 Introduction

Upon completion of the data collection and fieldwork, the engineer normally has all the elements necessary to perform the rehabilitation design. It is worth, however, evaluating at this point, at the project level, the need for rehabilitation by applying the method of determination used by the PMS mostly at the network level.

If the need for rehabilitation is not confirmed, this may indicate that the most recent data collected for the project has refined the earlier, tentative determination that rehabilitation was needed for a particular project, and that rehabilitation should be reconsidered. It may also indicate, on the other hand, that the project under consideration is of such a nature and/or presents such overriding characteristics that have not yet been entirely incorporated into the overall PMS system, which does not operate on a strictly technical basis.

The purpose of this chapter is to outline the method used in the PMS to differentiate between the needs for rehabilitation or maintenance for flexible pavements. The method is given in detail for reference in Appendix A (copied from Ref. 12).

3.2 Damage Quantifiers

During the pavement condition survey (cf. Appendix B.1), two types of damage (A and B) have been noted, together with their extent and severity level. Type B damage generally influences the type of work to be done only if there is no Type A damage. In the method, a global visual index Is is used, which is based solely on Type A damage (indicative of structural condition).

The global visual index Is is calculated from three damage groups:

- cracking and crazing;
- deformation and rutting;
- repairs.

The index Is (from 1 to 7) is calculated following the flow chart given in Figure 3.1. Ratings of 1 and 2 reflect good surface conditions requiring no work or for which work can safely be postponed. Ratings 3 and 4 represent an intermediate surface condition, serious enough to trigger maintenance work in the absence of any other consideration. Ratings 5, 6 and 7 represent very poor surface conditions requiring major maintenance or rehabilitation work.

3.3 Pavement Quality Rating

A pavement quality rating Q_i is estimated by combining the value of the index I_s as defined above, characterizing the “visual condition”, with the results of deflection measurements qualifying the combined “bearing capacity” of the pavement structure and its subgrade support.

The deflection of a homogeneous section of road is characterized by its value “d” (in $^1/_{100}$ mm). In the PMS method, “d” is defined as “m + 1.3 ō”, “m” being the mean value of the deflection measurements and “ō” their standard deviation.

No cracking or deformation	0

Visual Examination

(1) Cracking Index If	Extent / Severity	0 to 10%	10 to 50%	>50%
	1	1	3	3
	2	2	3	4
	3	3	4	5

Deformation Index Id	Extent / Severity	0 to 10%	10 to 50%	> 50%
	1	1	3	3
	2	2	3	4
	3	3	4	5

(1) Separate calculation for longitudinal cracking and crazing.
The larger of the two indices is used.

First damage index

Id \ If	0	1-2	3	4-5
0	1	2	3	4
1-2	3	3	4	5
3	4	5	5	6
4-5	5	6	7	7

Extent / Severity	0 to 10%	10 to 50%	> 50%
1	0	0	0
2	0	0	+1
3	0	+1	+1

Correction for repairs

Surface
Damage
Index
Is
Scale from (best) to 7 (worst)

Final rating

Figure 3.1: Determination of Damage Index I_s

Note: In the PMS study (Ref. 5), a choice has been made to measure (with a Benkelman Beam) the deflections between the twin-wheels of an axle loaded at the legal limit in the country. In order to calculate "d" as indicated above, it is suggested that a linear interpolation be used, from the measurements made with an axle loaded at 8.2 metric tonnes as prescribed in Appendix B. Although imperfect, this provides a reasonable consistency to link the design methods presented in this manual with the PMS procedure.

The characteristic deflection “d” having been determined, it is then compared with two threshold values d1 and d2, which divide general deflection magnitudes into three ranges as follows:

d1	value below which pavement performance is generally good
d2	value above which pavement performance is poor
d1-d2	range of indecision

The values of d1 and d2 are not necessarily fixed and are subject to revision as the PMS evolves on a network level. They may also be traffic dependent. The current values should be obtained from the Branch or Office in charge of the PMS.

As an illustration, the following values are **selected as “threshold” deflection values:**

For roads with a surface treatment

d1	90/100 mm
d2	115/100 mm

For roads with surfacings of asphalt concrete of more than 8 to 10 cm

d1	60/100 mm
d2	80/100 mm

The deflections were chosen on the basis of a small number of test sections. The deflection “threshold” should be reviewed from time to time and the designer is expected to verify the current values.

Based on the deflection “d” compared to d1 and d2, the characteristic deflection of a particular road falls into one of three classes, as illustrated in Figure 3.2. Figure 3.2 also defines the overall Pavement Quality Rating Qi based on the possible combinations of deflection value and Surface Damage Index I_s.

Based on the Quality Rating Q_i, the method provides the following options, as illustrated in Figure 3.2:

- For quality ratings Q1 to Q3, no major rehabilitation work is required. The road works to be performed include routine maintenance and/or periodic maintenance.
- For quality ratings Q4 to Q6, this indicates a zone of indetermination where visual inspection and deflection values appear inconsistent. A procedure is given in Appendix A to reassess the rating and to reclassify it as either in the range Q2-Q3 or Q7-Q8.
- For quality ratings Q7 to Q9, a rehabilitation is required (e.g. in the form of a structural overlay).

Surface damage index Is \ Deflection	Class 1	Class 2	Class 3
	d1	d2	
1 - 2 Little or no cracking or no deformation	Q1 (maintenance)	Q3 (maintenance)	Q6 (to be reclassified)
3 - 4 Cracks with little or no deformation, deformation without cracks	Q2 (maintenance)	Q5 (to be reclassified)	Q8 (overlay)*
5 – 6 - 7 Cracks and deformation	Q4 (to be reclassified)	Q7 (overlay)*	Q9 (overlay)*

* or other rehabilitation method

Figure 3.2 Pavement Quality Rating (Q_i) and Required Road Works

4. DESIGN OF ASPHALT OVERLAYS

4.1 Asphalt Overlays of Flexible Pavements

4.1.1 General

Asphalt overlays may be used to correct both surface deficiencies (raveling, roughness, slipperiness) and structural deficiencies. Surface deficiencies in asphalt pavements usually are corrected by thin resurfacings (functional overlays), but structural deficiencies require overlays designed on factors such as pavement properties and traffic loadings (structural overlays).

There are many instances when a surface treatment will not accomplish what is needed. Examples are depressions or severe raveling. In such cases, a thin overlay should be used over any required leveling course. Thin overlays usually range from 2.5 cm to 5 cm thick using a fine-grained top size dense mix. These are considered maintenance.

The overlay design procedures in the remainder of this section provide an overlay thickness to correct a structural deficiency. If no structural deficiency exists, a thin overlay may still be required to correct a functional deficiency.

This section covers the design of structural overlays by means of one or several lifts of asphalt concrete.
It is assumed that this option is feasible, i.e. that the condition of the existing pavement is not such that it dictates substantial removal and replacement of the existing pavement. Such conditions would include:

- A large amount of very severe alligator cracking,
- Excessive rutting which can be attributed to unstable existing materials,
- Seriously deteriorated stabilized roadbase requiring an excessive amount of repairs prior to overlay operations,
- Contaminated granular roadbase,
- Excessive stripping of the existing AC surface.

Two methods of overlay design are recommended, namely a deflection procedure (adapted from the Asphalt Institute, Ref. 11) and an effective thickness (or component analysis) procedure (adapted from AASHTO, Ref. 3). It is recommended that both methods always be used for comparison purposes. It is unlikely that the methods will agree exactly, and sound engineering judgment is required to estimate the possible reasons for the discrepancies and make a choice or a compromise between the results obtained by both methods.

Preference may be given to the effective thickness procedure when the history of pavement construction is well known and the destructive testing results are such that the quality of the materials is also well known. When the results and the records indicate the possibility of significant variation and uncertainty in the structure of the existing pavement, or extensive localized repairs, it is probably preferable to rely on statistical deflection results indicative of the overall load carrying capacity of the system comprising the pavement and its supporting subgrade.

4.1.2 Deflection Procedure

The deflection procedure recommended herein uses the results of a deflection survey conducted with a Benkelman Beam as described in Section B.3.1

The steps involved in the procedure are as follows:

Step 1: Determine a representative rebound deflection (RRD)
Step 2: Determine the design future traffic in terms of cumulated equivalent standard axles (ESAs).
Step 3: Determine the required overlay thickness.

Step 1
The individual deflection measurements recorded during the deflection survey must be adjusted by a temperature adjustment factor which can be read from Figure 4.1. The determination of the mean pavement temperature can be made in accordance with the procedure given in Appendix D.

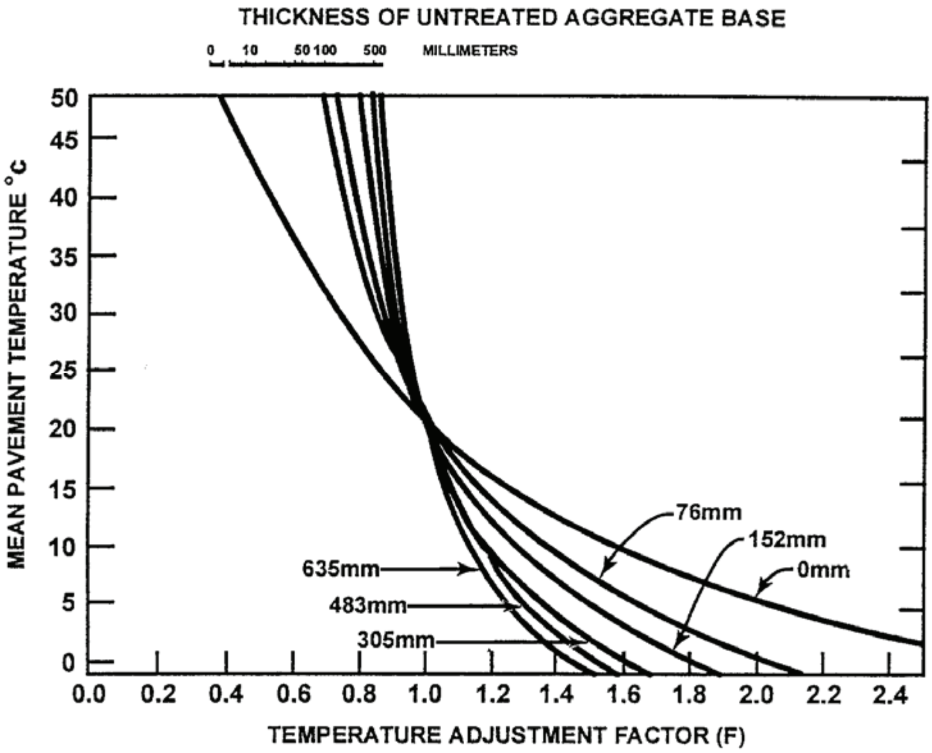


Figure 4.1 Temperature Adjustment Factor for Benkelman Beam Deflections

The results of deflection surveys conducted with a **FWD** are well-suited for use, after back-calculation of pavement parameters, as an input to the effective thickness procedure.

The mean and standard deviation of the adjusted individual deflection readings are then calculated. The representative rebound deflection RRD is taken as:

$$RRD = (x + 2s) c$$

Where:
x is the arithmetic mean of the individual deflection measurements adjusted for temperature
s is the standard deviation of the adjusted individual measurements
c is a critical period adjustment factor

The critical period is the interval during which the pavement is most likely to be damaged by heavy loads. The Asphalt Institute (Ref. 11) recommends the following methods for determining the critical period adjustment factor:

a) Obtain a continuous record of measured rebound values for a similar pavement in a similar environment and on a similar subgrade, and determine the most critical period. Then either:

1) Make the rebound measurements during the critical period, in which case the adjustment factor, c, equals 1.0. Or:
2) Make the rebound measurements at any time and adjust to the critical deflection by letting the adjustment factor, c, equal the ratio of the critical period deflection to the deflection for the date of the test.

b) If no record of comparable deflection data is available, make the rebound measurements at any time and make any needed adjustments using engineering judgment.

Example

Rebound deflections were measured for a section of asphalt pavement under uniform temperature conditions. The measurements were made under an 8.2 tonne axle load and the pavement temperature was estimated to be 17° C. The aggregate roadbase of the pavement was 350 mm thick. Using Figure 4.1, the temperature adjustment factor is F = 1.04. The measurements were made at the end of the rainy season and may reasonably be considered to be representative of the critical period, hence c = 1.0. Individual and adjusted measurements, and calculated representative deflection, are given below:

Measured deflections (under 8.2 tonne axle), mm	**Deflections adjusted for temperature (factor 1.04), mm**
0.60	0.62
0.56	0.58
0.46	0.48
0.70	0.72
0.80	0.83
0.68	0.71
0.57	0.60

Measured deflections (under 8.2 tonne axle), mm	Deflections adjusted for temperature (factor 1.04), mm
0.71	0.74
0.53	0.55
0.86	0.90
Mean adjusted value (mm)	0.673
Standard deviation (mm)	0.130
Representative deflection (mm)	0.932

Step 2

The traffic analysis outlined in Section 2.3 of this manual (and further detailed in the *MoWT-Volume III, Pavement Design Manual (Part 1, Flexible Pavement Design Guide)* is to be used to determine the design number of equivalent standard axles (ESAs) expected to be carried by the road after overlay. As a reminder, this design number of ESAs is expressed in equivalent 8.2 tonne axles.

Step 3

To find the thickness of asphalt concrete overlay required, enter the overlay thickness design chart, Figure 4.2, with the RRD obtained in Step 1, move vertically to the curve representing the design ESA (from Step 2), and move horizontally to the Overlay Thickness scale.

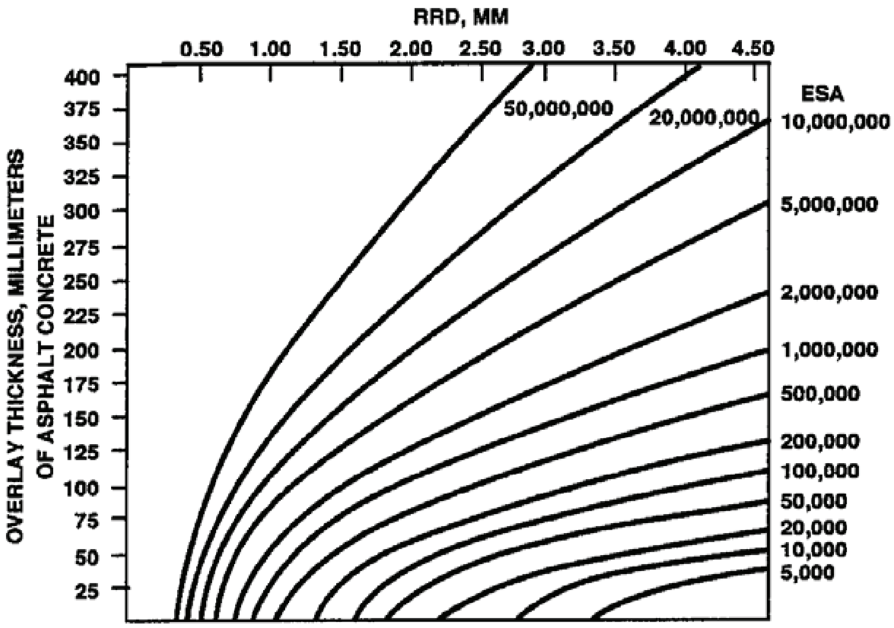


Figure 4.2: Overlay Thickness Design Chart

Example

The pavement considered in the example in Step 1 is being considered for a future traffic of 5 x 106 ESAs. With a RRD of 0.93 mm, the required overlay thickness is found from the chart to be 75 mm.

4.1.3 Effective Analysis Procedure

The required thickness of AC overlay is computed as:

$$T_0 = \frac{(SN_{new} - SN_{eff})}{a_0}$$

where:

T_0 = required overlay thickness in centimeters
SN_{new} = structural number of a new pavement (centimeters)
SN_{eff} = effective structural number of the existing pavement (centimeters)
a_0 = structural coefficient of the AC overlay

The same coefficient as given for new AC surface course materials may be used for the structural coefficient a0 of the AC overlay.

It may be noted that the structural numbers have the same dimension as a thickness.

SN_{new} is computed as indicated in **Appendix E**. It requires the determination of the required structure of a new pavement for the specific subgrade strength and traffic applicable to the project. The procedure given **in Appendix E** lists structural layer coefficients for the conversion of the required structure into SN_{new}.

SNeff requires knowing the existing pavement structure and using the equation:

$$SN_{eff} = a_1T_1 + a_2T_2m_2 + a_3T_3m_3$$

Where:

T_1, T_2, T_3 = thicknesses (in centimeters) of existing pavement surface, roadbase, and subbase layers

a_1, a_2, a_3 = corresponding structural layer coefficients
m_2, m_3 = drainage coefficients for granular roadbase and subbase

The thicknesses T_i are determined from the previously collected data and field work, as indicated in Chapter 2 and Appendix B.

The coefficients a_i may be determined from Table 4.1, which lists suggested layer coefficients for commonly used materials. Other suggested coefficients, for stabilized roadbase materials, are given in Table 4.2.

Table 4.1
Suggested Layer Coefficients for Existing AC Pavement Layer Materials

MATERIAL	SURFACE CONDITION	COEFFICIENT
AC Surface	Little or no alligator cracking and/or only low-severity transverse cracking	0.35 to 0.40
	<10 percent low-severity alligator cracking and/or <5 percent medium- and high-severity transverse cracking	0.25 to 0.35
	>10 percent low-severity alligator cracking and/or <10 percent medium-severity alligator cracking and/or >5-10 percent medium- and high-severity transverse cracking	0.20 to 0.30
	>10 percent medium-severity alligator cracking and/or <10 percent medium-severity alligator cracking and/or >10 percent medium- and high-severity transverse cracking	0.14 to 0.20
	>10 percent high-severity alligator cracking and/or >10 percent high-severity transverse cracking	0.08 to 0.15
Granular Roadbase or Subbase	No evidence of pumping, degradation, or contamination by fines. Some evidence of pumping, degradation, or contamination by fines.	0.10 to 0.14 0.00 to 0.10

Table 4.2
Additional Suggested Layer Coefficients for Stabilized Roadbase Materials

MATERIAL	SURFACE CONDITION	COEFFICIENT
Stabilized Roadbase	Little or no alligator cracking and/or only low-severity transverse cracking	0.20 to 0.35
	<10 percent low-severity alligator cracking and/or <5 percent medium- and high-severity transverse cracking	0.15 to 0.25
	>10 percent low-severity alligator cracking and/or <10 percent medium-severity alligator cracking and/or >5-10 percent medium- and high-severity transverse cracking	0.15 to 0.20
	>10 percent medium-severity alligator cracking and/or <10 percent high-severity alligator cracking and/or >10 percent medium- and high-severity transverse cracking	0.10 to 0.20
	>10 percent high-severity alligator cracking and/or >10 percent high-severity transverse cracking	0.08 to 0.15

It must be realized that relatively limited guidance is available for the selection of layer coefficients for in-service pavement materials. Engineers are encouraged to use judgment and to build experience in the selection of the coefficients, particularly with regard to local materials and pavement behavior.

The drainage coefficients m2 and m3 may be determined on the basis of Table 4.3 and 4.4 further below.

Table 4.3: Quality of Drainage

Quality of Drainage	Water Removed Within
Excellent	2 hours
Good	1 day
Fair	1 week
Poor	1 month
Very Poor	(water will not drain)

Table 4.4
Recommended m_i Values for Modifying Structural Layer Coefficients of Untreated Roadbase and Subbase Materials

	Percent of Time Pavement Structure is Exposed to Moisture Levels Approaching Saturation			
Quality of Drainage	Less Than 1%	1-5%	5-25%	Greater Than 25%
Excellent	1.40-1.35	1.35-1.30	1.30-1.20	1.20
Good	1.35-1.25	1.25-1.15	1.15-1.00	1.00
Fair	1.25-1.15	1.15-1.05	1.00-0.80	0.80
Poor	1.15-1.05	1.05-0.80	0.80-0.60	0.60
Very Poor	1.05-0.95	0.95-0.75	0.75-0.40	0.40

Example

An existing pavement is made up of the following layers:

- 5 cm AC surfacing (T_1)
- 15 cm granular roadbase (T_2)
- 15 cm subbase (T_3)

The AC surface shows less than 10 percent of low-severity cracking, very little medium- and high-severity transverse cracking, and can be assigned (cf. Table 4.1) a structural layer coefficient a_1 of 0.30. Roadbase and subbase courses show no evidence of degradation or contamination. The coefficients a_2 and a_3 may both be taken as 0.12. The quality of drainage is considered fair and the pavement structure is exposed to levels near saturation on the order of 5%. Both coefficients m_2 and m_3 can be taken as 1.00.

SN_{eff} is calculated as:

$SN_{eff} = 0.30x5 + 0.12x1.00x(15 + 15) = 5.1$

It is contemplated to overlay the pavement for an expected traffic class T6. The subgrade strength class is S4. The structural number of an adequate pavement structure for these conditions is $SN_{new} = 9.13$ (see Appendix E).

The required overlay thickness is: $T_0 = (9.13 – 5.1)/0.44 = 9.15$ cm (which may be rounded up to 10 cm).

Note: The above method is based on the determination of SN_{eff} essentially from an assessment of the quality of the pavement layers from visual, field (e.g., DCP) and/or laboratory testing. Alternatively, SN_{eff} may be estimated from the results of deflection (FWD) testing as outlined in Appendix F.

4.1.4 Surface Preparation for Overlay

In the design of overlays and the adoption of the overlay as a rehabilitation solution, the construction feasibility should be verified first (besides the economic constraints) with reference to factors such as:

- Traffic control, traffic disruption;
- Materials and equipment availability; and
- Construction problems such as utilities, bridge clearances, side slope extension.

Having determined the feasibility, careful and correct preparation of the existing pavement, prior to construction of overlays, is essential to good construction and to maximum overlay performance. The overlay thickness is designed to correct a below-average pavement condition, but not to provide the extra structural strength needed for localized weak areas. If the overlay thickness is based on the weakest condition in the section, it would be over-designed for the rest of the section and thus be needlessly costly. Therefore, the weaker areas must be corrected to provide a uniform foundation for the overlay.

Some of the factors which need consideration in preparation of the existing pavement are as follows:

Pre-Overlay Pavement Repairs

If distress in the existing pavement is likely to affect the performance of the overlay, it should be repaired prior to the placement of the overlay. Much of the deterioration that occurs in overlays results from deterioration that was not repaired in the existing pavements. The cost tradeoffs of pre-overlay repair and overlay type should also be considered.

Severe alligator cracking and linear cracks, rutting and surface irregularities should be repaired prior to overlay of AC pavements.

The pre-overlay repairs generally fall in the maintenance categories. One particular pre-overlay operation to consider is an effective reflection crack control.

Reflection Crack Control

Reflection cracks are a frequent cause of overlay deterioration. The thickness design procedures described in the preceding sections do not consider reflection cracking. Pre-overlay repairs (patching and crack filling) may help delay the occurrence and deterioration of reflection cracks. Additional reflection crack control measures include:

- Pavement fabrics;
- Crack relief layers. These are composed of open-graded coarse aggregate and a small percentage of asphalt cement; and
- Increased overlay thickness.

Subdrainage

As indicated in Appendix B, the existing subdrainage condition of the pavement should be evaluated since it has a great influence on how well the overlay performs. Removal of excess water from the pavement cross-section will increase the strength of the pavement layers and subgrade, and reduce deflections.

Milling-Recycling

Milling, with or without the intent of recycling the milled materials, can improve the performance of the overlay, by removing some of the cracked and hardened materials and by minimizing existing rutting or other significant distortions.

Surface Recycling

This process may be considered as analogous to pre-overlay surface preparation or an in-place variant of cold milling and recycling. The asphalt pavement surface is heated in place, scarified, remixed, relaid, and rolled. Asphalts, recycling agents, new asphalt hot-mix, aggregates, or a combination of these may be added to obtain a desirable mixture. When new asphalt hot-mix is added, the finished product may be used as the final surface; otherwise, an asphalt surface course should be used.

Shoulders

Overlaying traffic lanes generally requires that the shoulders be overlaid to match the grade line of the traffic lanes. In selecting an overlay material and thickness for the shoulder, the designer should consider the extent to which the existing shoulder is deteriorated and the amount of traffic that will use the shoulder. For example, if trucks tend to park on the shoulder at certain locations, this should be considered in the shoulder overlay design.

If an existing shoulder is in good condition, any deteriorated areas should be patched. An overlay may then be placed to match the shoulder grade to that of the traffic lanes. If an existing shoulder is in such poor condition that it cannot be patched economically, it should be removed and replaced.

4.2 Asphalt Overlays of Rigid Pavements

4.2.1 Deflection Procedure

Deflection tests measure the response of a pavement to a specified load. Depending on the type of pavement, the measured deflection indicates the necessity for overlay. The thickness of the overlay can be reduced by limiting the deflections by way of undersealing.

In Jointed Plain Concrete Pavements (JPCP), the differential deflection should be. 0.05 mm, and mean edge deflection should be. 0.36 mm. In Continuously Reinforced Concrete Pavement (CRCP), the deflection should be less than 0.015 mm. If the above criterion is exceeded, overlaying with or without undersealing is required. The deflections are assumed to be reduced by 0.2 percent per millimeter of overlay thickness. The thickness of overlay can be obtained from the chart shown in Figure 4.3, based on the temperature differential and the length of the slab. Alternatives 2 and 3, mentioned in the figure, can be used to reduce the overlay thickness in sections B and C of the chart.

Example

Slab = 13.5 m Jointed Plain Concrete Pavement (JPCP)

High Temperature = 28° C

Low Temperature = -16° C

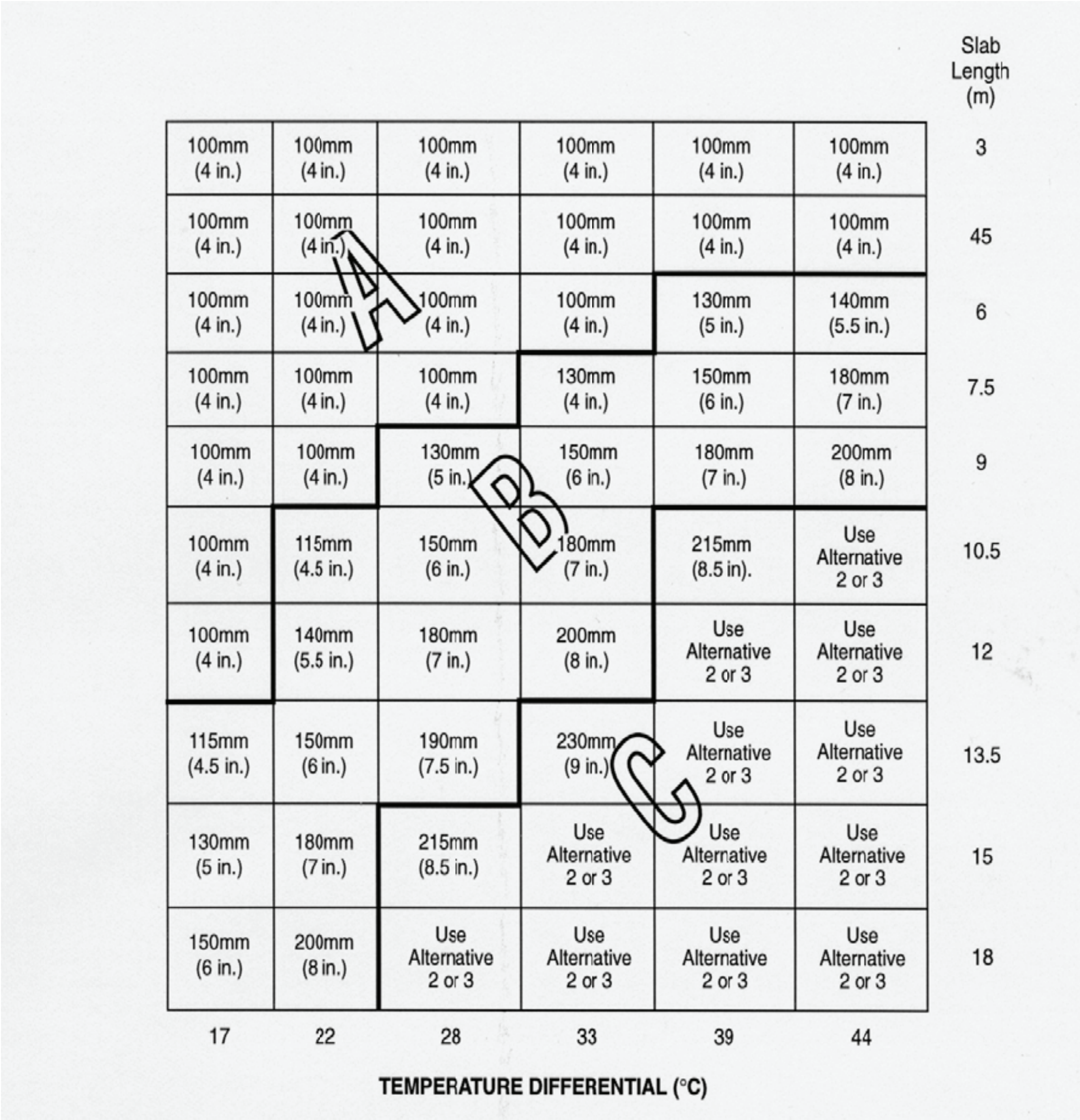
Δt = 44° C

Measured Benkelman Beam deflections:

d_1 = 0.85 mm
d_2 = 0.63 mm

Diff. Deflection = $d_1 - d_2$ = 0.85 – 0.63 = 0.22 mm

Mean Deflection = $(d_1 + d_2)/2$ = (0.85 + 0.63)/2 = 0.74 mm



Slab Length (m)	17	22	28	33	39	44
3	100mm (4 in.)	100mm (4 in.)	100mm (4 in.)	100mm (4 in.)	100mm (4 in.)	100mm (4 in.)
45	100mm (4 in.)	100mm (4 in.)	100mm (4 in.)	100mm (4 in.)	100mm (4 in.)	100mm (4 in.)
6	100mm (4 in.)	100mm (4 in.)	100mm (4 in.)	100mm (4 in.)	130mm (5 in.)	140mm (5.5 in.)
7.5	100mm (4 in.)	100mm (4 in.)	100mm (4 in.)	130mm (4 in.)	150mm (6 in.)	180mm (7 in.)
9	100mm (4 in.)	100mm (4 in.)	130mm (5 in.)	150mm (6 in.)	180mm (7 in.)	200mm (8 in.)
10.5	100mm (4 in.)	115mm (4.5 in.)	150mm (6 in.)	180mm (7 in.)	215mm (8.5 in).	Use Alternative 2 or 3
12	100mm (4 in.)	140mm (5.5 in.)	180mm (7 in.)	200mm (8 in.)	Use Alternative 2 or 3	Use Alternative 2 or 3
13.5	115mm (4.5 in.)	150mm (6 in.)	190mm (7.5 in.)	230mm (9 in.)	Use Alternative 2 or 3	Use Alternative 2 or 3
15	130mm (5 in.)	180mm (7 in.)	215mm (8.5 in.)	Use Alternative 2 or 3	Use Alternative 2 or 3	Use Alternative 2 or 3
18	150mm (6 in.)	200mm (8 in.)	Use Alternative 2 or 3	Use Alternative 2 or 3	Use Alternative 2 or 3	Use Alternative 2 or 3

TEMPERATURE DIFFERENTIAL (°C)

Figure 4.3: Chart for selecting Asphalt Concrets Structural Overlay Thickness for PCC Pavement

NOTES:

(1) Temperature differential (Δt) is the difference between the highest normal daily maximum temperature and the lowest normal daily maximum temperature for the hottest and coldest months, based on a 30-year average.

(2) Alternative 2: Crack and seat the slab into smaller sections with thinner overlay.

(3) Alternative 3: Utilize a crack-relief layer with drainage system with thinner overlay.

Alternative 1: Thick Overlay

Step 1—Enter the design chart, Figure 4.3, with Δt = 44°C and slab length = 13.5 m
The required overlay is more than 230 mm. Therefore, use Alternative 2 or 3.

Alternative 2: Reduce Slab Length

Step 1—Break the slab into 6.8 m sections.

Step 2—Enter the design chart, Figure 4.3, with Δt = 44° C and slab length = 6.8 m

Required overlay = 160 mm (by interpolation)
Use 165 mm

Step 3—Check vertical mean deflection:

At the rate of 0.2 percent per millimeter of overlay thickness, the reduction in deflection is:
165 x 0.74 x 0.002 = 0.24 mm
Remaining deflection is: 0.74 – 0.24 = 0.50 mm > 0.36 mm allowable.
Therefore, undersealing is required.

4.2.2 Effective Thickness Procedure

If the overlay is being placed for some functional purpose such as roughness or friction, a minimum thickness overlay that solves the functional problem should be placed. If the overlay is being placed for the purpose of structural improvement, the required thickness of the overlay is a function of the structural capacity required to meet future traffic demands and the structural capacity of the existing pavement. The required overlay thickness to increase structural capacity to carry future traffic is determined by the following equation:

$$D_{ol} = A(D_f - D_{eff})$$

Where:

D_{ol} = Required thickness of AC overlay, cm
A = Factor to convert PCC thickness deficiency to AC overlay thickness
D_f = Slab thickness to carry future traffic, cm
D_{eff} = Effective thickness of existing slab, cm

The A factor, which is a function of the PCC thickness deficiency, is given by the following equation:

$$A = 2.2233 + 0.0015(D_f - D_{eff})^2 - 0.0604(D_f - D_{eff})$$

AC overlays of conventional JPCP, JRCP, and CRCP have been constructed as thin as 5 cm and as thick as 25 cm. The most typical thicknesses that have been constructed for highways are 7.5 to 15 cm.

D_{eff} is computed from the following equation:

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

Where:

D = Existing PCC slab thickness, cm
F_{jc} = Joints and cracks adjustment factor
F_{dur} = durability adjustment factor
F_{fat} = fatigue damage adjustment factor

The factors F_{jc}, F_{dur}, F_{fat}, are dependent on the existing condition of the pavement and can be evaluated based on the condition survey detailed in Appendix B.

F_{jc} depends on the total number of unrepaired deteriorated joints, cracks, punchouts and other discontinuities per kilometer in the design lane and is determined using Figure 4.4.

F_{dur} depends on the existing durability problems, such as aggregate distress. Using the condition survey, F_{dur} is determined as follows:

1.00: No sign of PCC durability problems
0.96-0.99: Durability cracking exists, but no spalling
0.88-0.95: Substantial cracking and some spalling exists
0.80-0.88: Extensive cracking and severe spalling exists

The F_{fat} factor adjusts for past fatigue damage that may exist in the slab. It is determined by observing the extent of transverse cracking (JPCP, JRCP) or punchouts (CRCP) that may be caused primarily by repeated loading.

The following guidelines can be used to estimate the F_{fat} factor in the design lane:

0.97-1.00: Few transverse cracks/punchouts exist (none caused by “D” cracking or reactive aggregate distress)
JPCP: <5 percent slabs are cracked
JRCP: <25 working cracks per mile (about 16 per km)
CRCP: <4 punchouts per mile (2 or 3 per km)

0.94-0.96: A significant number of transverse cracks/punchouts exist (none caused by “D” cracking or reactive aggregate distress)
JPCP: 5-15 percent slabs are cracked
JRCP: 25-75 working cracks per mile (16-47 per km)
CRCP: 4-12 punchouts per mile (3 to 8 per km)

0.90-0.93: A large number of transverse cracks/punchouts exist (none caused by “D” cracking or reactive aggregate distress)
JPCP: >15 percent slabs are cracked
JRCP: >75 working cracks per mile (>47 per km)
CRCP: >12 punchouts per mile (>8 per km)

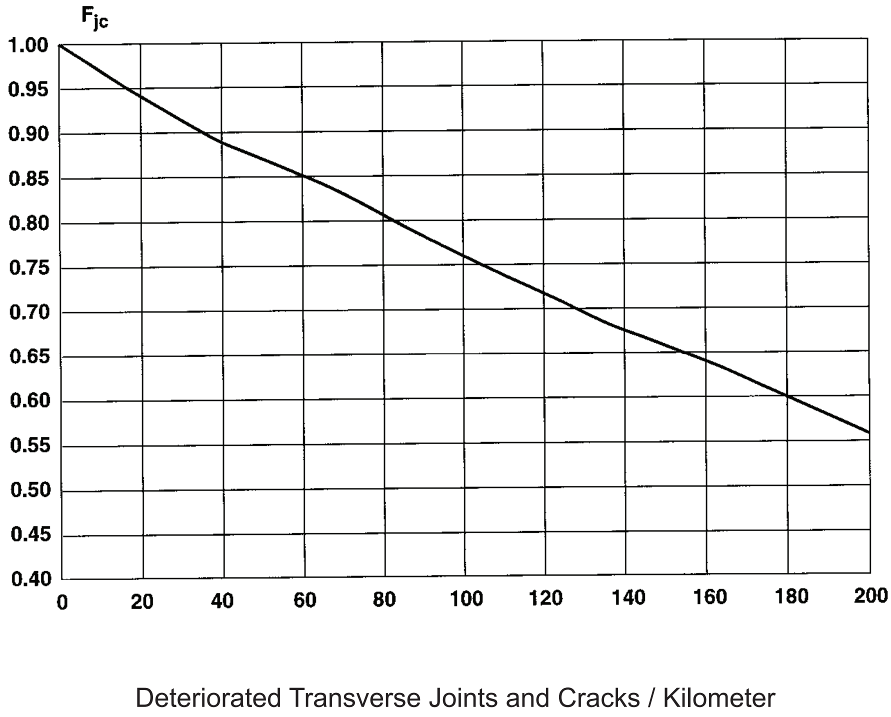


Figure 4.4: F_{jc} Adjustiment Factor

4.2.3 Surface Preparation for Overlays

The following types of distress in JPCP, JRCP and CRCP should be repaired prior to placement of an AC overlay.

Distress Type	Repair Type
Working cracks	Full-depth repair or slab replacement
Punchouts	Full-depth PCC repair
Spalled joints	Full-depth or partial-depth repair
Deteriorated repairs	Full-depth repair
Pumping/faulting	Edge drains
Settlements/heaves	AC level-up, slab jacking, or localized reconstruction

Full depth repairs and slab replacements in JPCP and JRCP should be PCC, dowelled or tied to provide load transfer across repair joints. Full depth repairs in CRCP should be PCC and should be continuously reinforced with steel which is tied or welded to reinforcing steel in the existing slab to provide load transfer across joints and slab continuity.

Installation of edge drains, maintenance of existing edge drains, or other subdrainage improvement should be done prior to placement of the overlay if a sub-drainage evaluation indicates a need for such an improvement.

Pressure relief joints should be placed only at fixed structures, and not at regular intervals along the pavement. The only exception to this is where reactive aggregate has caused expansion of the slab.

4.2.4 Reflection Crack Control

The basic mechanism of reflection cracking is strain concentration in the overlay due to movement in the vicinity of joints and cracks in the existing pavement.

A variety of reflection crack control measures have been used in attempts to control the rates of reflection crack occurrence and deterioration. Any one of the following treatments may be employed in an effort to control reflection cracking in an AC overlay of JPCP or JRCP:

(1) *Sawing and sealing joints in the AC overlay* at locations coinciding with joints in the underlying JPCP or JRCP.
(2) *Increasing AC overlay thickness*. Reflection cracks will take more time to propagate through a thicker overlay and deteriorate more slowly.
(3) *Placing a bituminous-stabilized granular interlayer (large-sized crushed stone crack relief layer), prior to or in combination with placement of the AC overlay* has been effective. See Figure 4.5.
(4) *Cracking and seating JPCP or breaking and seating JRCP* prior to placement of the AC overlay. This technique reduces the size of PCC pieces and seats them in the underlying base, which reduces horizontal (and possibly vertical) movements at cracks.

Reflection cracking can have a considerable (often controlling) influence on the life of an AC overlay of JPCP or JRCP. Deteriorated reflection cracks detract from a pavement's serviceability and also require frequent maintenance, such as sealing, milling, and patching.

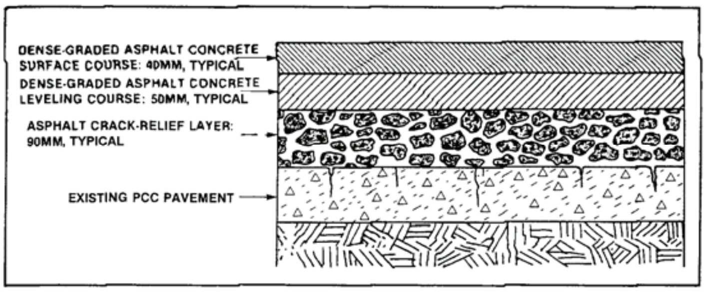


Figure 4.5: Crack-Relief Layer in an Overlay System, Cross-Section

When the pavement has been rendered as uniformly stable as possible, it must be cleaned thoroughly and tack-coated with asphalt before the overlay is placed.

When overlaying a PCC pavement that has been grooved, special treatment is necessary to prevent moisture intrusion. Here, a heavy asphalt tack coat, an asphalt slurry seal, or a sand (fine-graded) asphalt leveling course is used to fill the grooves.

Old PCC surfaces that range from polished to coarse-textured can vary significantly in bonding ability. Polished surfaces can be re-textured to improve their bonding with overlays. However, in most cases, the proper use of a tack coat, selection of the proper mix type and overlay thickness, coupled with correct construction procedures, will prove more economical in ensuring a good bond.

5. DESIGN OF ALTERNATIVE REHABILITATION METHODS

- Reconstruction: In this category, little or no contribution is expected from the existing pavement materials and the materials needed for rehabilitation will be new materials.
- Recycling: The rehabilitation takes advantage of the existing pavement materials, which are reused in part or as a whole in the construction of the rehabilitated pavement.

Other methods which are effective in both repairing existing deterioration and, to some extent, preventing or delaying its recurrence include repairs by patching, crack sealing, milling and surface treatments or seals. These methods, however, are better classified under the term of maintenance (cf. Chapter 1). It is recognized that they are often required for the preparation of the existing pavement prior to overlay (cf. Chapter 4).

The above categories are general in nature, and some combinations of methods are possible and indeed used. For instance, the materials of an existing surface treatment and base course may be recycled as the base or subbase of a reconstructed pavement, with new materials brought on site to construct the upper layers, e.g. roadbase and AC surface.

5.1 Reconstruction

5.1.1 Asphalt Pavement

Since the pavement is to be reconstructed with new materials, generally only the subgrade of the existing pavement will remain. It is assumed in this case that the existing pavement materials are trucked away. If the materials are removed from the roadway and carried to a central plant for processing and later reuse, this central plant recycling is considered a cold recycling technique.

For reconstruction design, therefore, the subgrade strength (CBR) must be evaluated and this evaluation yields the design CBR to use in pavement design.

The rest of the design process for reconstruction follows that of a new pavement design. This design is to be carried out in accordance with the MoWT-Volume III, Pavement Design Manual (Part 1, Flexible Pavement Design Guide).

In case a variant reconstruction procedure is used, whereby, for instance, the existing subbase is left in place, prior to the addition of a new roadbase and surface course, the principles of component analysis, as detailed for the overlay design in Chapter 4, may be applied.

Example

An existing pavement is made up of the following layers:

- Surface dressing
- 20 cm of granular roadbase
- 15 cm of subbase

The condition survey has indicated the need for rehabilitation. The subgrade has been evaluated and is considered properly compacted and homogeneous. Investigations have revealed that the subgrade can be characterized by a CBR within the range of subgrade strength class S4 (CBR between 8 and 14). Traffic studies concluded that the future traffic would be in excess of 6×10^6 ESA but well within the range of traffic class T6 (6 to 10×10^6 ESA).
An adequate new pavement structure (*MoWT-Volume III, Pavement Design Manual (Part 1, Flexible Pavement Design Guide)* consists of:

- 10 cm AC surfacing
- 20 cm granular roadbase
- 17.5 cm subbase

The existing subbase course is of good quality, with a CBR consistently in excess of 30. The existing roadbase is of marginal or poor quality and new sources of adequate quality of crushed stone have been located. It was elected to reconstruct the pavement from the existing subbase (to be preserved) up. The scarified surface dressing and existing base course will be disposed of.

The existing subbase may remain and contribute to the new pavement structure as 15 cm of subbase (at a 1-to-1 ratio with new materials). It is simpler, in view of the abundance of materials, to substitute crushed stone base materials to the missing 2.5 cm of subbase. The reconstruction operations will consequently lay down 22.5 cm of new roadbase and 10 cm of new AC surfacing.

5.1.2 CEM I Concrete Pavement

At some point near the end of the life of a CEM I Concrete pavement, the slab is so badly deteriorated that total reconstruction becomes more cost-effective than resurfacing or restoration. CEM I Concrete pavement conditions that favor reconstruction include:

1. Little or no remaining structural life, as evidenced by extensive slab cracking.
2. Extensive slab settlement, heave, or cracking due to foundation movement (caused by swelling soil or frost heave).
3. Extensive joint deterioration (particularly for short-jointed pavements, since full-depth repair would require replacement of a large percentage of the concrete surface).
4. Extensive concrete deterioration due to poor durability (D-cracking or reactive aggregate distress).

Reconstruction may also be warranted for highways that do not meet required geometric design standards (e.g. lane width, bridge clearance, curve superelevation). On resurfacing projects, it may sometimes be necessary to reconstruct short sections of pavement near bridges to maintain required bridge clearances.
As is the case with asphalt pavement, in reconstruction projects, generally only the subgrade of the existing pavement will remain. For reconstruction design, therefore, the subgrade strength (CBR) must be evaluated.

5.2 Recycling

As long as virgin (new) materials are in abundant supply and within reasonable distance, reconstruction can remain an attractive option. However, when new materials are less plentiful and the costs of haulage become uneconomical, the need to conserve energy and materials favors pavement recycling.
The processes for pavement recycling are conveniently grouped into two categories:

(a) Asphalt Pavement Recycling

This category is further divided into three groups:

- Surface recycling. This type of recycling is described earlier in Section 4.1;
- Cold-mix recycling; and
- Hot-mix recycling.

(b) CEM I Concrete Pavement Recycling

These categories are illustrated by Figure 5.1.

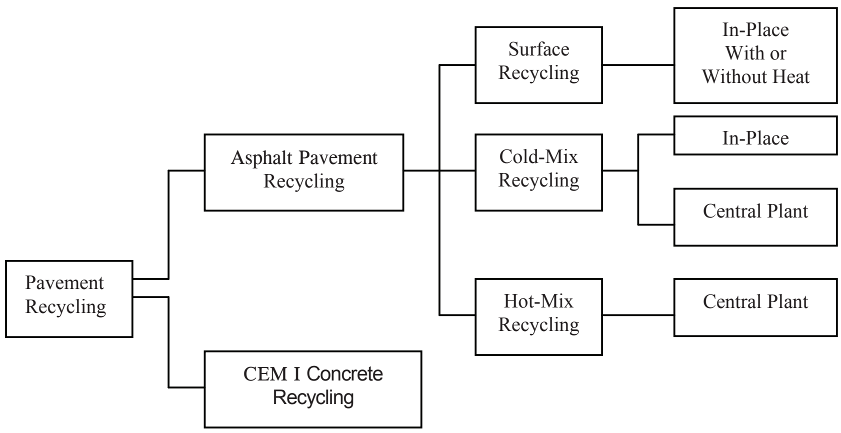


Figure 5.1: Categorization of Recycling Approaches

5.2.1 Recycling of Asphalt Pavement

a) Cold-Mix Recycling

In this process, reclaimed asphalt pavement materials, reclaimed aggregate materials, or both, are combined with new asphalt, and/or recycling agents in place, or at a central plant, to produce cold-mix roadbase mixtures. An asphalt surface course is required. This is a common method for recycling old pavements, including both the surface and untreated roadbase materials.

From the viewpoint of thickness design, the design procedure is similar to that of a reconstruction. The required structure of a new pavement must be determined in accordance with the (*MoWT-Volume III, Pavement Design Manual (Part 1, Flexible Pavement Design Guide)*. A component analysis helps to determine the contribution of the recycled materials.

Example

The same example is used as given above for reconstruction, with the following modification: in this case, the subbase is still left in place, but the existing roadbase and surface treatment are being recycled as a bituminous roadbase. The materials are recycled in-place and may be attributed a structural coefficient of 0.25 (see Appendix F).

Keeping the same 1:1 ratio between the existing 15 cm of subbase to be left in place and 15 cm of the required subbase, the deficiency to be made up to complete the required structure is:

- 10 cm of AC surfacing
- 20 cm of granular roadbase
- 2.5 cm of subbase

Using structural layer coefficients of 0.44, 0.14 and 0.11, respectively, this corresponds to a deficit of structural number of 7.5.

One solution consists therefore of laying down 12.5 cm of recycled roadbase (contributing 12.5x0.25 i.e. 3.1 to the SN) and the originally required 10 cm of AC surfacing (contributing 10x0.44 i.e. 4.4 to the SN).

However, alternate solutions providing the same SN may be considered (e.g. 17 cm of recycled roadbase and 7.5 cm of AC surfacing). This illustrates a commonly advocated advantage of recycling.

Cold-mix recycling involves ripping, scarifying, pulverizing, or crushing the old pavement. It is either hauled to a central site and upgraded with asphalt cement, emulsified asphalt or other stabilizing agents (lime, cement, fly ash in combination with lime or cement, or calcium chloride), or it is treated in place. Treatment in place may be achieved by blading, rotary mixers or a travel plant. Before beginning the project, representative samples should be obtained. The asphalt content and consistency and the gradation of the aggregates should be determined. Then the materials should be blended and a mix design developed. The real economical advantage of cold-mix recycling often lies in the ability to upgrade an old pavement structure with a relatively thin wearing course with limited need for new aggregates. After the recycled mix is placed, it should be capped with an AC wearing course or a surface treatment.

b) Hot-Mix Recycling

In this process, reclaimed asphalt pavement materials, reclaimed aggregate materials, or both, are combined with new asphalt and/or recycling agents and/or new aggregate, as necessary, in a central plant to produce hot-mix paving mixtures.

Two methods are used to process the old pavement: (1) Scarifying and removing the material to a crushing plant where it is sized, or (2) sizing the material in-place with rotating drummilling equipment or hammermills and hauling the sized materials to a central location for later processing. The reclaimed asphalt pavement contains both asphalt and aggregate. Testing is conducted to determine the characteristics of the reclaimed asphalt pavement (e.g. asphalt extraction, gradation of the aggregate and consistency of the extracted asphalt).

Equipment to centrally hot-process the recycled material can be, for convenience, separated into three general categories: direct flame heating, indirect flame heating, and superheated aggregate. The techniques and related operations are illustrated in Figure 5.2.

From the viewpoint of thickness design, the design procedure is identical to that given for cold-mix recycling. It should only be noted that hot-mix recycling lends itself to higher quality control, hence a higher layer structural coefficient may be assigned to the recycled materials.

5.2.2 Recycling of CEM I Concrete

In most cases where reconstruction is justified, the existing concrete can be recycled to reduce the cost of reconstruction.

Concrete pavement recycling involves breaking up the old pavement on grade, loading and hauling the material to a crushing plant, and processing it at the plant to produce recycled concrete aggregate (RCA). The product of this process is an aggregate that can be used in place of virgin aggregate in any component of the pavement structure where aggregate is used, including:

1. Untreated, dense-graded aggregate roadbase
2. Cement- and asphalt-stabilized roadbases
3. Lean concrete roadbase
4. CEM I concrete surfacing
5. Asphalt concrete surfacing
6. Fill
7. Filter material
8. Drainage layer or edge drains

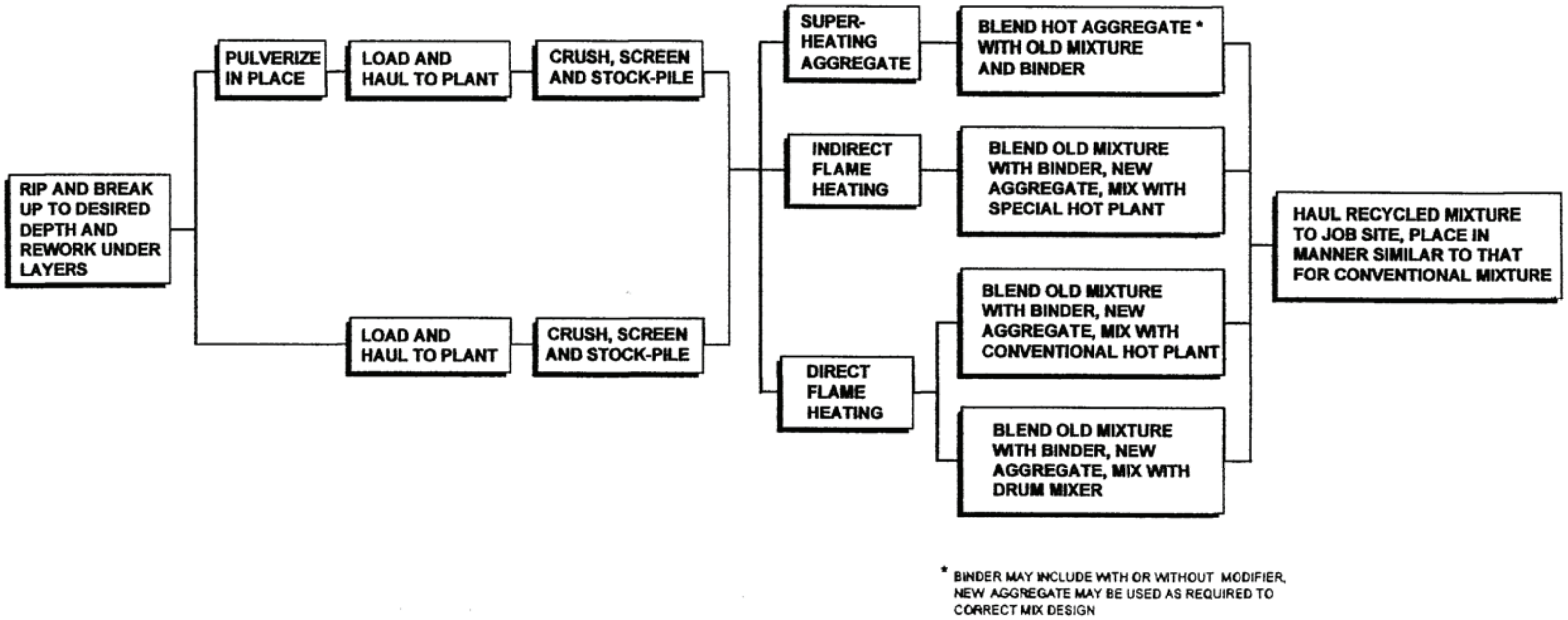


Figure 5.2: Central Plant Hot-Mix Recycling Techniques

Recycling of CEM I Concrete pavements is not limited to those pavements that contain sound aggregate. Pavements containing reactive aggregate can be recycled into new concrete using fly ash to control expansion of the reactive aggregate. Badly D-cracked pavements have also been successfully recycled into new pavements. The common practice in recycling D-cracked pavement is to limit the maximum size of the recycled concrete aggregate, so that any aggregate particles that contain sizable voids are fractured before being used in the new concrete mix.

One limitation of small aggregate size is poor aggregate interlock at joints and cracks, especially on JRCP and CRCP. Additional large virgin aggregate may need to be added so that sufficient aggregate interlock will develop.

A CEM I Concrete pavement that has an AC overlay can be recycled, but the two layers must be recycled separately. Asphalt concrete should not be recycled for aggregate in a CEM I Concrete mix, because the asphalt cement will inhibit entrainment of air in the concrete mix.

Whether or not a particular project should be recycled depends on numerous factors, including suitability of the pavement for recycling, availability and cost of virgin aggregate, cost of disposing of old pavement material if it is not recycled, approximate cost of recycling, policy toward recycling, and the extent of local contractors' experience with recycling. CEM I Concrete pavement recycling has proven to be both economical and environmentally advantageous. Laboratory and field studies have shown that a high quality concrete, with improved durability, can be produced using recycled concrete aggregate. Significant savings in material transportation and disposal costs are possible through recycling, particularly in urban areas.

CEM I Concrete pavement recycling can be divided into two groups:

- surface recycling
- central plant recycling

Both surface and central plant recycling techniques have been utilized on roadways containing Portland cement concrete. However, surface recycling techniques applied to concrete pavements is most often considered a pavement removal operation. The removed pavement can be recycled. Figure 5.3 describes some of the CEM I concrete recycling options.

Surface Recycling

Surface recycling techniques involve the use of cold milling or cold planing techniques that are capable of economically removing up to approximately two inches of concrete in a single pass. Traffic can operate for extended periods of time on the milled surface, or an asphalt concrete overlay can be placed.

The sequence of operation involving surface recycling is as follows:

1. Establishing desirable grade line.
2. Milling, grinding, or planing the pavement to the desired depth.
3. Clean-up involving rotary broom and vacuum equipment.
4. Disposal or recycling of the millings.

Surface recycling operations, involving overlays, use the sequence of operations as described above, with the addition of a tack coat and asphalt concrete overlay.

Pavement milling operations are suitable for the removal of localized severe surface undulations caused by swelling clays, etc.; for removal of pavement prior to overlay along gutters, at bridge approaches, and other areas where a feathered edge of asphalt concrete or CEM I concrete is likely to abrade; and for improved drainage, surface texture, and skid resistance.

An added advantage of surface recycling is the increase in bond strength between a milled CEM I concrete and an overlay as compared to a normal overlay operation. This anticipated increase in bond strength may allow the use of thin overlays on CEM I concrete pavements.

Central Plant Recycling

The pavement removal and crushing operations are performed with conventional construction and demolition equipment or specially designed equipment. The old concrete pavement is normally broken such that the size of the resulting slab is normally small enough to be received by the primary crusher. Additional reduction in slab size can be performed at the crushing location. Central plant sizing can be performed with conventional, fixed and portable crushing equipment; however, reinforcing steel may be a problem and may have to be removed at one or more of six processing locations:

1. On the grade during the loading operation,
2. During the locating operation for crushing if stockpiling occurs prior to crushing,
3. At the entry to the primary crushing,
4. On the belt after primary crushing,
5. On the belt after final crushing, or
6. In the stockpile prior to remixing.

Equipment recently developed pulverizes the concrete to smaller sizes on grade and thus more complete steel removal is possible on grade.

Blending and mixing operations in the central plant are standard operations, as are the techniques utilized for the placing and curing of recycled materials. Gradation adjustments (particularly the addition of natural sands) are often made to improve workability.

Structural Design

The structural design of CEM I Concrete pavement using recycled materials can be done, in principle, using the guidelines given in the (*MoWT-Volume III, Pavement Design Manual (Part 2, Rigid Pavement Design Guide)*.

In essence, recycled material is viewed no differently from new or virgin material in either new or rehabilitated pavement systems. Because of this, the most important structural rehabilitation parameter for recycled material is the characterization of the load carrying ability (lab and/or field) of the material.

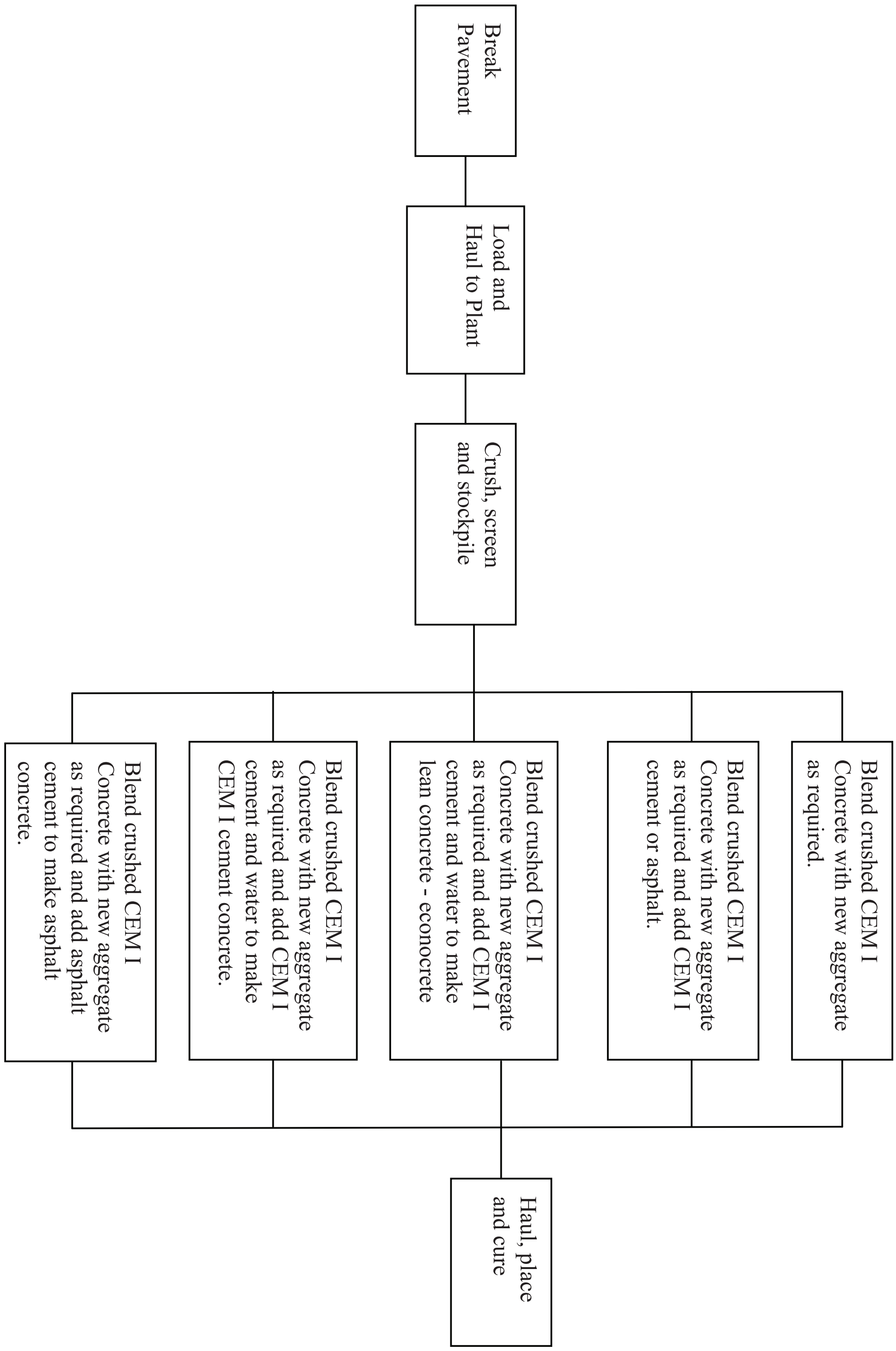


Figure 5.3: CEMI Concrete Pavement Recycling

Literature review indicates that the structural coefficients of recycled materials do reflect the extreme variation of reused materials and recycling processes utilized. However, it is equally apparent that recycled materials have the capacity to be equivalent, or greater, in load spreading capabilities than the original material. With good control on the recycling process, and based on laboratory test data, an appropriate structural coefficient can be evaluated to be used in the design.

Similarly, the properties of concrete made with recycled aggregate show some variations compared to the concrete made with virgin aggregate, as given below.

Concrete Properties

The following is a comparison of the properties of CEM I Concrete made with recycled concrete aggregate and concrete made with virgin aggregate:

1. The compressive strength of recycled concrete is between 60 and 100 percent of the compressive strength of conventional concrete at the same water-cement ratio. Water-reducing admixtures can be used to obtain higher strength concrete at the same cement content without compromising workability.

2. The static modulus of elasticity of recycled concrete is between 60 and 100 percent of the modulus of conventional concrete at the same water-cement ratio.

3. The flexural strength of recycled concrete is between 80 and 100 percent of the flexural strength of conventional concrete at the same water-cement ratio.

4. Recycled concrete has a higher ratio of flexural strength to compressive strength than conventional concrete.

5. Low-strength recycled concrete can be recycled into higher-strength concrete through proper mix design.

6. The durability of concrete made with aggregate susceptible to D-cracking can be substantially increased by limiting the top size of the aggregate.

7. The volume response to moisture and temperature changes of recycled concrete is not significantly different from that of normal concrete.

The fact that concrete containing recycled concrete aggregate has a higher ratio of flexural strength to compressive strength than conventional concrete should be considered if compressive strength is to be used as the criterion for acceptance or for timing the early opening of reconstruction projects.

All of the above comments about recycled concrete aggregate's performance in concrete surfaces apply to its use in lean concrete roadbase courses, as well. Recycled concrete aggregate was used in lean concrete roadbases long before it was used in concrete surfaces because not enough was known about its performance in surface courses. In general, the use of recycled concrete aggregate is restricted to roadbase courses (granular, cement- or asphalt-stabilized, and lean concrete).

6. SELECTION OF PREFERRED ALTERNATIVE

The following excerpt from Part III of the AASHTO Guide (Ref. 3) appears relevant to this section:

"While analytical solutions to portions of the rehabilitation methodology are presented, the engineer must recognize that it may be impossible to accurately determine the optimal rehabilitation solution from a rigorous analytical model. However, the user should not be discouraged from employing this approach but rather feel encouraged to use every available tool at his/her disposal to determine the problem cause, identify potentially sound and economic solution alternatives, and then select the most preferred rehabilitation strategy from sound engineering experience".

6.1 Rehabilitation Factors

Rehabilitation choices should be viewed with reference to several factors, including:

- the decision to use new materials, recycled materials, or a combination of both (noting that recycled materials need not be those obtained from the specific pavement project being rehabilitated);
- the decision to employ full reconstruction, partial reconstruction, a direct overlay, or some combination of reconstruction and overlay.

Due to state of the art limitations regarding the entire rehabilitation process, there is a definite need for feedback on the performance of various rehabilitation methods. The "optimum" solution (from the viewpoint of cost to benefit ratio) may not be attainable for a particular project due to constraints imposed (e.g. funding). There should however be a "preferred" solution which is cost-effective, has other desirable characteristics, and meets the existing constraints.

Some factors which may influence the rehabilitation choices have been listed as data which must be collected prior to the condition survey (cf. Chapter 2). Road inventory is one of them, and the geometric design (i.e. the adequacy of the existing geometry) should be viewed as one major factor to decide between overlay and non-overlay methods. Other potential constraints to be contended with include:

- traffic control problems;
- possible funding limitations;
- desirable service life of the rehabilitation;
- right-of-way;
- availability of materials and equipment; and
- probable contractor's capacities.

On the basis of the above factors and constraints, the design engineer should be able to suggest candidate solutions among those described in Chapters 4 and 5, and among the multiple combinations thereof. After the feasible candidate solutions have been selected, by weighing candidate solutions against project constraints, preliminary design should be prepared. Preliminary designs require only approximate cost estimates.

6.2 Selection of Preferred Solution

There is no infallible method for selecting the most "preferred" rehabilitation alternative for a given project. The selection process requires engineering judgment, creativity and flexibility. Guidance can nevertheless be offered to select a preferred solution. Both monetary and non-monetary considerations apply.

6.2.1 Cost Analysis

The cost is generally considered the most important criterion in the selection process.

Normally, a life-cycle cost analysis is required, and an outline of its main components is given below. However, one must bear in mind a few points regarding life cycle cost analysis as it is applied to the selection of a rehabilitation method.

Life-cycle cost analysis requires inputs of both cost and time. Unfortunately, both of these elements are subject to a large degree of uncertainty. For instance, the effective life of a rehabilitation technique is subject to the following influences:

- the skill and care with which the work is performed;
- the quality of the materials used;
- environmental conditions prevalent in the region where the pavement exists;
- the traffic which uses the pavement; and
- other rehabilitation and maintenance work being performed concurrently.

Even the engineer familiar with the performance of various rehabilitation methods in his or her local area can appreciate the difficulty of selecting appropriate inputs for use in the life-cycle cost analysis. To eliminate as much uncertainty as possible, it is essential to collect rehabilitation performance data whenever available (e.g. from the Office or Branch responsible for the PMS).

- Another important consideration regarding life-cycle cost analysis is that the same rehabilitation techniques, when applied to different pavements, may have variant effects. Furthermore, some methods keep a pavement at a consistently high-condition level, while others may allow the condition of the same pavement to fluctuate. Thus, discrepancy is often not revealed by the cost analysis if user costs are not included in the calculations. It is therefore important to include user costs in a cost analysis.

The major costs to consider in the economic analysis include:

(1) Government costs

   (a) Initial rehabilitation costs. These should include, in particular, the costs of pavement preparation (repairs, etc.) required prior to overlay, if this is the alternative considered in the analysis.
   (b) Future rehabilitation costs (after the selected design period for the rehabilitation design).
   (c) Maintenance costs, recurring throughout the design period.
   (d) Salvage return or residual value at the end of the design period.
   (e) Engineering and administrative costs.
   (f) Traffic control costs, if applicable.

(2) User costs

   (a) Travel time
   (b) Vehicle operation
   (c) Accidents
   (d) Discomfort
   (e) Time delay and extra vehicle operating costs during resurfacing or major maintenance

There are a number of methods of economic analysis that are applicable to the evaluation of alternative strategies.

(1) Equivalent uniform annual cost method, often simply termed the “annual cost method”
(2) Present worth method for:

   (a) costs
   (b) benefits, or
   (c) benefits minus costs, usually termed the “net present worth” or “net present value method”

(3) Rate-of-return method
(4) Benefit-cost ratio method
(5) Cost-effectiveness method

Either the net present worth value or the equivalent uniform annual cost may be used to determine life cycle costs for comparisons of alternate pavement rehabilitation strategies. In either case, it is essential that comparisons only be made for analysis periods of equal length. Details of the equations used in the methods are beyond the scope of this manual. They may be found in the HDM models developed by the World Bank.

6.2.2 Non-monetary Considerations

As with the review of the constraints in the selection of candidate rehabilitation methods, several factors need to be considered in selecting the preferred solution. They include, in particular, service life, duration of construction, reliability of the solution, constructibility and maintainability.

Also, as with monetary considerations, the service life (selected and ascertained during the data collection) of a rehabilitation method is an important factor. This is particularly significant for high-volume roads, for which lane closures and traffic delays pose considerable difficulties. The important time parameter is years of pavement life extension achieved by the rehabilitation methods and should be a factor in almost any decision criterion.

6.3 Preferred Rehabilitation Alternative

The preferred rehabilitation alternative will be selected using first cost considerations, then non-monetary factors. When the cost does not indicate a clear advantage, weighing factors may be assigned to the non-monetary considerations. Each factor is evaluated and multiplied by its weight, and a final “score” is calculated. This procedure is in relatively common use to select the preferred rehabilitation alternative.

Such a method for measuring several rehabilitation alternatives against criteria that cannot be expressed in monetary units is illustrated in Figure 6.1.

Detailed design, plans and cost estimates are normally prepared after the selection of the preferred alternative. If major differences occur at this stage, either in cost or in design, it may be necessary to reinvestigate the cost-effectiveness of the solution.

First, the relative importance of each criterion is assigned by the design team. Next, the alternatives are rated according to their anticipated performance in the criterion areas. Then, an alternative’s rating in an area is multiplied by the assigned weight of that factor to achieve a “score”. Finally, all of the scores for an alternative are summed, and the alternative with the highest score is the preferred solution.

	CRITERIA							
	INITIAL COST	DURATION OF CONSTRUCTION	SERVICE LIFE	REPAIRABILITY & MAINTENANCE EFFORT	RIDEABILITY & TRAFFIC ORIENTATION	PROVEN DESIGN IN LOCAL CLIMATE	TOTAL SCORE	RANK
RELATIVE IMPORTANCE	20%	20%	25%	15%	5%	15%	100	
ALTERNATIVE 1	60 / 12	60 / 12	100 / 25	80 / 12	90 / 4.5	100 / 15	80.5	1
ALTERNATIVE 1A	60 / 12	60 / 12	100 / 25	80 / 12	90 / 4.5	100 / 15	80.5	1
ALTERNATIVE 2	60 / 12	60 / 12	70 / 17.5	50 / 7.5	60 / 3	40 / 6	58	5
ALTERNATIVE 2A	60 / 12	60 / 12	70 / 17.5	50 / 7.5	60 / 3	40 / 6	58	5
ALTERNATIVE 3	60 / 12	40 / 8	100 / 25	80 / 12	100 / 5	90 / 13.5	75.5	2
ALTERNATIVE 4	60 / 12	80 / 6	40 / 10	20 / 3	40 / 2	20 / 3	44	8
ALTERNATIVE 5	40 / 8	60 / 12	40 / 10	50 / 7.5	50 / 2.5	30 / 4.5	44.5	7
ALTERNATIVE 6	70 / 14	80 / 16	60 / 12.5	50 / 7.5	80 / 4	40 / 6	60	4
ALTERNATIVE 7	100 / 20	100 / 20	20 / 5	20 / 3	40 / 2	40 / 6	56	6
ALTERNATIVE 8	30 / 6	60 / 12	100 / 25	100 / 15	100 / 5	30 / 4.5	67.5	3

Figure 7.1: Illustrative Method of Selecting Rehabilitation Alternatives

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APPENDIX A

VIZIR METHOD
FOR QUALITY EVALUATION OF PAVED ROADS

A.1 Classification and Quantification of Damages

General

The first part of VIZIR is a method of classification and quantification of damage that forms part of general road maintenance management studies or special route maintenance studies. It is intended to provide a picture of the surface condition of a road at a given time and to identify zones of equal quality (in terms of three damage levels).

These zones of equal quality and these three damage levels are used to determine the nature and type of work required; in some cases, the very identification of the damage determines the solution, while in others it is only one factor in a more complex diagnosis involving other criteria.

The damage classified in the VIZIR method is relevant primarily to flexible pavements with bituminous surfacings. This damage is divided into two categories:

Type A Damage

This characterizes the structural condition of a pavement, affecting either all of its courses and the ground or the surfacing only. This damage is caused by a structural deficiency of the pavement, and its identification is used in the search for a solution in conjunction with other criteria, in particular the bearing capacity as characterized by the static deflection.

Type A damage includes four types:

- Deformation;
- Rutting;
- (fatigue) cracking; and
- crazing.

Type B Damage

This damage leads to repairs that are generally unrelated to the pavement's structural capacity. It may be caused by defective placement, by deficient product quality, or by some special local condition, possibly aggravated by traffic.

Type B damage includes:

- cracking other than fatigue cracking, i.e. longitudinal joint cracks, transverse thermal shrinkage cracks, and longitudinal and transverse clay shrinkage (desiccation) cracks;
- potholes;
- raveling and, more generally, all surfacing defects such as fretting, bleeding, etc.

A.2 Survey and Grading of Damage

The damage is surveyed by an operator who travels the length of the route and records, for any damage:

- its type;
- its severity; and
- its extent, i.e., the length of road affected or, as appropriate, the area.

VIZIR provides a damage topology and, for each type of damage, three levels of severity.

The survey can be done manually, while travelling the road on foot or by car. The operator in this case enters his observations (identification of damage and estimate of its severity) on a route diagram, a document representing the route as a straight line, the scale and precision of which are appropriate to the type of study.

The survey may also be done using the LCPC's DESYROUTE equipment. Carried in a car, it makes it easy to record the type, severity, and location of any damage (position on curved x-axis based on information from the distance indicator). Its data entry and retrieval software allows all sorts of calculations, such as a mean severity index and an extent as a percentage of a given length, or an overall quality index based on a combination of several types of damage. It can also read out all of these data, either in the form of a route diagram identical to the one produced by hand or in the form of files that can be incorporated in a road data bank (for instance the VISAGE road data bank).

To help the operator in his work, VIZIR proposes a catalogue of damage and a method of graphic representation that are summed up in Table 1 (Type A damage) and Table 2 (Type B damage).

In the route diagram, damage is represented by a rectangle of which the background (white, grayish, or black) indicates the level of severity, while the two sides represent the co-ordinates of the beginning and end of the damaged zone, or in other words, its extent.

The severity values given in Tables 1 and 2 are average values suitable for many road networks. However, they can be altered to fit the assigned maintenance objectives and the expected level of severity.

Damage \ Severity	1	2	3
Deformation rutting	Perceptible to user but small $f < 2$ cm	Severe deformations, localised subsidence or rutting $2 \leq f \leq 4$ cm	Deformation severely affecting safety or travel time $f \geq 4$ cm
Cracking	Hair line cracks in wheel tracks or centerline	Open and / or branching cracks	Markedly branched and/or wide open cracks: edges sometimes damaged
Crazing	Fine crazing with no loss of materials large mesh (> 50 cm)	Tighter crazing (< 50 cm) sometimes accompanied by loss of materials, stripping, and incipient potholes	Very open crazing forming blocks (< 20 cm), sometimes accompanied by loss of materials
Patching and Repair	☐ Either rebuilding of part or all of pavement	Surface work related to type A defects	
	☐ Or surface work related to type B defects	☐ Repair has stood up well	☐ Visible damage to repair itself

Table 1: Level of severity of type A damage

Damage \ Severity	1	2	3
Longitudinal joint crack	Hair line isolated	• Wide (1 cm or more) without stripping or • Hair line & branching	• Wide with spalling of edges or • Wide and branching
Pothole	• Number < 5 • Dia, not more than 30 cm	5 to 10 < 5 or Dia. 30 cm Dia. 100 cm	> 10 5 to 10 or Dia. 30 cm Dia. 100 cm
	Per 100 m of pavement		
Movement of material Ravelling, fretting, bleeding, etc.	Localised. Roadbase not visible	Continuous or localised but roadbase visible	Continuous and roadbase visible
	Localised.	Continuous in one wheel track	Continuous and "marked" in one wheel track

Table 2: Level of severity of type B damage

Each type of damage and each associated level of severity, as indicated in Tables 1 and 2, is described in detail in the next sub-section and shown in a photograph attached to the present appendix.

A.3 Description of Type A Damages

This concerns the following types of damage:

1. Deformation, rutting
2. Fatigue cracking and crazing
3. Patching and repairs

Deformation and Rutting

The deformations specific to flexible pavements almost always lead to rutting or subsidence. Their degree of severity is determined by the depth h measured on a straightedge 1.5 m long placed crosswise on the pavement (see following figure).

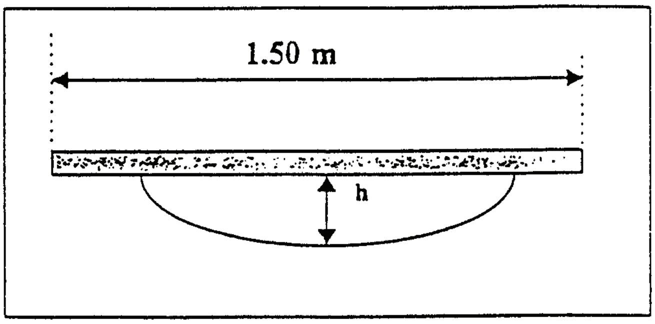

Figure A.1 Measurements of Deformation

Rutting appears in the wheel tracks, on the back of the pavement about 50 to 80 cm from the edge. It is caused by settlement of the materials under heavy, channelized traffic, possibly but not necessarily aggravated by a deficiency of bearing capacity of the soil. There may also be rutting through creep of the bituminous courses only, but this is found most often in semi-rigid pavement or in pavements with a bituminous roadbase.

The following values are given for guidance:

Severity 1	$h < 2$ cm
Severity 2	2 cm $< h <$ 4 cm
Severity 3	$h > 4$ cm

Subsidence affects the entire edge of the pavement. It is a result of deficiencies of bearing capacity or of stability, possibly caused by materials of poor quality or excessively high water contents. Subsidence often occurs in bends and in zones of very high stress: It can be in the form of:
Localized subsidence of severity 1
Subsidence of severity 3 or
Ridge of severity 1, 2 or 3
Longitudinal deformations generally show up in the evenness measurement (sag of a few cm, wavelength from 1 to 20 cm) and must not be counted twice. However, longitudinal deformation in the form of an isolated ridge may properly be recorded with transverse deformations.

Fatigue Cracking

This category does not include cracks resulting from faulty construction, such as the longitudinal joint between two mix spreading bands, or cracks caused by some particular behavior of the material, such as longitudinal or transverse thermal shrinkage cracks or clay shrinkage cracks.
Longitudinal fatigue cracks, on the other hand, are recorded. Most often initially single and isolated,

they evolve toward continuous cracking, sometimes branching, before multiplying in the wheel tracks to the point of becoming very closely spaced.

Severity 1 Single or clearly separated longitudinal cracks
Severity 2 Continuous cracks, branched or clearly open
Severity 3 Extensively branched cracks foreshadowing crazing or wide open cracks

Patching and Repairs

In some damage survey methods, repairs may not be counted. This is true of the *OECD Manual–Road monitoring for question of maintenance – DC1 – 1990*. The LCPC has always held that repairs, like damage, are an integral part of the visual examination. The 1977 guide to the examination of flexible pavements, in fact, says, "a recent repair conceals a problem; frequent repairs call attention to it". Repairs are intended to palliate the deficiencies of a pavement, temporarily or permanently; their number, extent, and frequency in time are elements of the diagnosis. In the VIZIR method, repairs must be rated in the course of the visual examination, because some of them are used in determining the index of pavement appearance. Two cases are distinguished:

- The repair has definitively eliminated the defect, in which case it will appear on the survey document (route diagram) but not be counted when calculating the surface quality index;
- The repair has more or less adequately eliminated the defect, but not its cause, and will doubtless reappear here or there or nearby; the repair is therefore reflected as an aggravating factor in calculating the visual index.

The first category includes repairs of Type B defects or partial or complete rebuilding of the pavement, in good condition. VIZIR rates them as being of severity 1. Rebuilding may be of the surfacing only or of the whole pavement:

Rebuilding of wearing course at longitudinal joint (Type B damage)

Rebuilding of pavement (Type A damage)

Patching of the wearing course (Type B damage)

Structural defects of pavement after repair (Type A)

The second category includes localised pavement repairs of Type A defects, classed as severity 2 if in good condition and severity 3 otherwise. Only severities 2 and 3 may, according to their extent, increase the visual index. Photos 21 and 24 illustrate such cases.

Table A.3: Extent of Patching and Repair vs. Severity

Severity \ Extent	0 to 10%	10 to 50%	> 50%
1	0	0	0
2	0	0	+1
3	0	+1	+1

Table for calculating the correction of surface damage I_s according to the severity and extent of repairs.

A.4 Description of Type B Damages

The common Type B damage in flexible pavements includes:

- Cracking of longitudinal joint;
- Potholes
- Raveling and, more generally, any surface defect such as fretting, bleeding, etc.

There are others that depend on extreme climatic conditions or particular materials, such as:

Thermal shrinkage of bituminous materials;

- Clay shrinkage of pavement courses.

VIZIR is intended for flexible pavements, and so does not cover the thermal shrinkage cracking of hydraulic materials. If they had to be included, they would be in category B.

Cracking of Longitudinal Joint
This is a failure of bonding between two adjacent bands of coated materials. This type of crack is initially single and straight, and can be repaired by treatment specific to such cracking (severity 1). Traffic causes the crack to branch or double and lose material at the edges (severity 2, 3). Repairs are more costly: complete rebuilding of the damaged zone or placement of a new wearing course.

Severity 1	Single hair line crack
Severity 2	Branching crack
Severity 3	Wide, branching crack with loss of materials

Potholes

Potholes most often result from wear or destruction of the wearing course, sometimes from the presence of foreign bodies in the surfacing.

They are small when they first appear. In the absence of maintenance, they grow and reproduce in rows, often with a pitch equal to the circumference of a truck type.

Depending on the degree of severity of these potholes, it may be necessary to rebuild part of the wearing course or, if they have become too deep, of the wearing course and roadbase. In some cases, it may even be necessary to rebuild the whole pavement. If there are too many potholes, it may be necessary to rebuild a considerable length of pavement.

The three levels of severity depend on the nature of the work:

Severity 1	Small number of potholes that can be plugged by ordinary patching
Severity 2	Large number of small potholes in surface
Severity 3	Potholes or crazing formation – pavement must be rebuilt or overlaid

Movement of Material: Ravelling, Fretting, Bleeding, etc.

Movements of materials include ravelling and bleeding: some ravelling is caused by excessive wear of the wearing course and may lead to the formation of potholes. It is normally counted as cracking or crazing.
Ravelling of severity 1 on crazing of severity 2, counted as Type A damage

Other ravelling is caused by defective construction. The following damages are distinguished:

- **Ravelling:** more or less large losses of gravel from surface dressings (deficiency of bonding, hence of the quality and/or quantity of bitumen);
- **Scabbing:** loss of all or part of a thin (<3 cm) wearing course of coated materials and separation from its substrate; this is often preceded by major cracking of the wearing course;
- **Fretting:** deficient implementation due to bad equipment operation: poor distribution of bitumen by the sprayer, hence surface unequally covered by the bitumen or bad gravel distribution on the pavement; and
- **Bleeding:** upward movement of excess bitumen in hot weather.

The following examples show the severity levels that can be manifested by some of the above mentioned pavement defects:

Severity 1 Localised raveling (scabbing) or discontinuous ravelling of plucking type

Severity 2 Continuous ravelling

Severity 3 Very marked generalised fretting

Severity 3 Generalised bleeding

Shoulder and Ditch Erosion

There are other forms of damage that are specific to a climate, a country, or a given traffic pattern and that may require rebuilding of the shoulders as part of maintenance:

- **Lacy edge:** this damage occurs in pavements in which the roadbase and shoulders are of the same type, and is caused by the frequent stopping of vehicles on the shoulders. The extent of the damage is more important than its severity.

Severity 1 Onset of lacy edges

Severity 2 Lace cutting more than 0.50 m into the pavement

Severity 3 Extreme erosion approaching destruction of the pavement

- **Low shoulders:** this damage is caused by maintenance of the shoulders, which gradually become lower than the pavement surface.

Severity 1 From 1 to 5 cm lower

Severity 2 From 5 to 10 cm lower

Severity 3 More than 10 cm lower

- **Ditch and shoulder erosion:** the erosion may take a number of forms (rainwash of ditch; rainwash and destruction of ditch; destruction gravely endangering a part of the pavement). The severity code follows this progression:

Severity 1 Erosion of ditch – repairs limited to ditch

Severity 2 Erosion and rainwash of shoulder

Severity 3 Threat to or destruction of a part of the pavement

A.5 Use of Damage Quantifiers to Determine Maintenance Needs

General

In the previous chapter, two types of damage have been distinguished (A and B). For each type of damage, a scale of classification according to three levels of severity has been provided. Each type of damage is accordingly quantified by two estimators:

- its extent (length of road affected); and
- its severity.

VIZIR quantifies damage to estimate the quality of a pavement, investigated either in connection with a road maintenance management study (network) or with a view to particular work (route). In both cases, even if the levels of precision are different, rehabilitation approaches must be identified.

In the case of type B damage, the maintenance approach follows from the identification of the damage by itself, and no other parameters are needed for the diagnosis. For example, centerline cracking requires bridging of the cracks, lacy edges require rebuilding of the edges and shoulders, and so on.

This is not true of type A damage; the approach will depend on other factors, and the diagnosis will be based on damage, bearing capacity, traffic, etc. It is therefore necessary to establish an overall rating of visual condition, similar to the ratings or classes used for the other parameters. Type A damage leads to major work such as rebuilding or overlaying of the surfacing. This work remedies type B damage in passing: bridging a centerline crack to prevent the infiltration of water is pointless if a wearing course is going to be applied over the pavement. Generally, type B damage influences the type of work done only if there is no type A damage (whence the choice of the order A and B) and, conversely, the global visual index used to qualify the pavement counts only type A damage.

Damage Index Is

The global visual index Is is calculated for a specified length of road from three damage groups:

- cracking and crazing;
- deformation and rutting; and
- repairs.

A cracking index I_f, depending on the severity and extent of cracking or crazing of the length of road in question, is calculated first. When both cracking and crazing are present, the larger of both values is used.

A deformation index I_d, depending on the severity and extent of deformation and rutting is then calculated in a similar manner.

I_f and I_d are combined in a first pavement quality index. It may, as appropriate, be corrected to reflect the severity and extent of certain repairs. It has already been pointed out that some repairs conceal pavement deficiencies, and so are treated as aggravating factors when estimating surface quality.

This correction yields a global damage index I_s that qualifies the pavement over the length chosen for the calculation. I_s ranges from 1 to 7. Ratings 1 and 2 reflect good surface conditions that need no work (or at least on which work may safely be postponed). Ratings 3 and 4 represent a rather intermediate surface condition, bad enough to trigger maintenance work in the absence of any other consideration. Ratings 5, 6 and 7 represent very poor surface conditions requiring major maintenance or overlay work.

The baseline length on which I_s is calculated may depend on the type of study, the database, other parameters included in the diagnosis, and the operator.

For studies of road maintenance management systems, which are global studies, the route diagram is plotted to a scale of about 2 cm per km; the database itself is established with a step on the order of 500 m. I_s may therefore be calculated for 500 m lengths.

For route maintenance planning, the survey is plotted to a scale of about 5 cm per km, and a 200 m step may be used for the calculation.

Finally, in the case of a test section, the route diagram is plotted to a scale of about 20 cm per km; a high degree of precision is attained and I_s may be calculated for 50 m lengths.

When the damage survey is done using DESYROUTE, the software of which includes VIZIR, simply indicate, during data processing, the measurement length on which the calculation should be based and the scale on which the route diagram should be printed out.

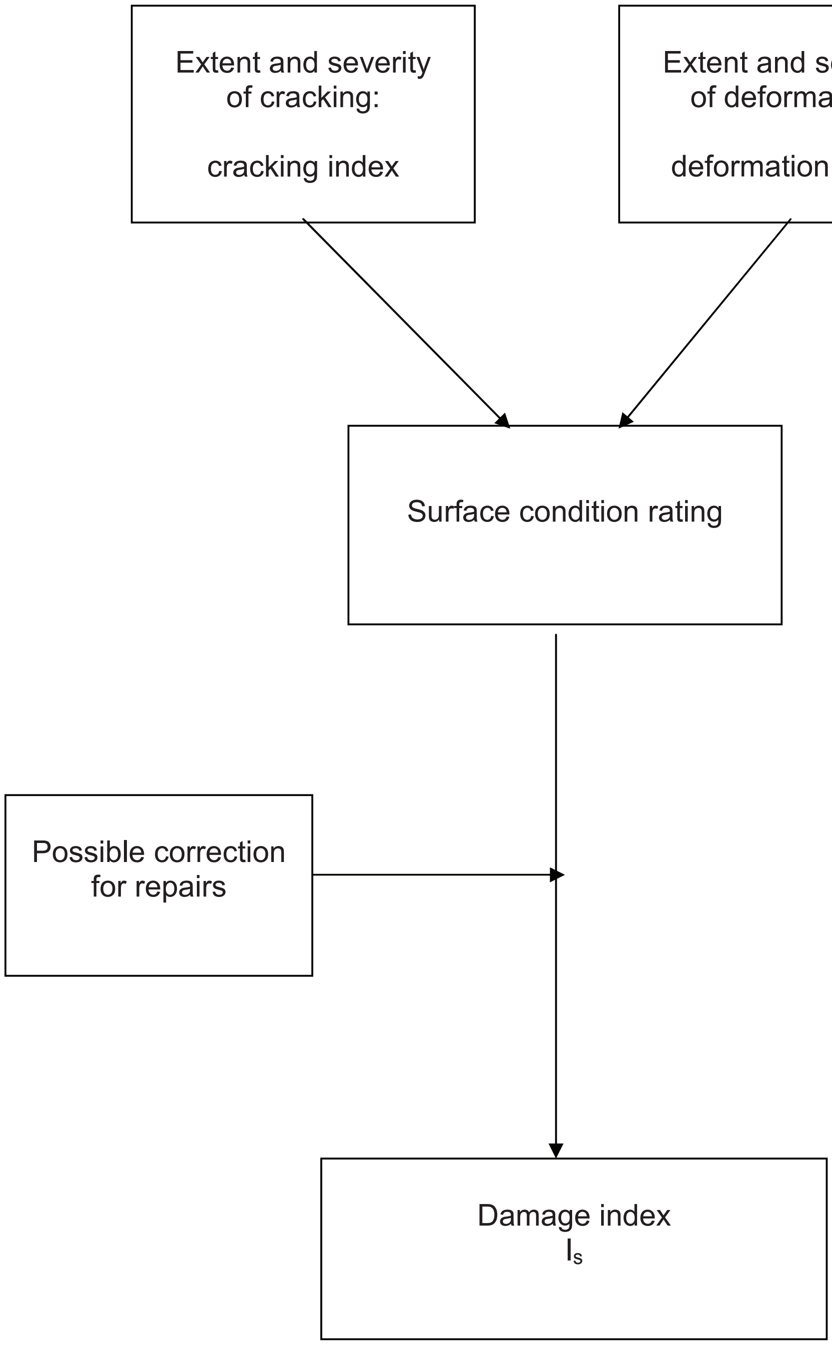

Figure A.2 Principles of Determination of Surface Condition Rating

No cracking or deformation	0

Visual Examination

(1) Cracking Index If	Severity \ Extent	0 to 10%	10 to 50%	>50%
	1	1	3	3
	2	2	3	4
	3	3	4	5

Deformation Index Id	Severity \ Extent	0 to 10%	10 to 50%	> 50%
	1	1	3	3
	2	2	3	4
	3	3	4	5

(1) Separate calculation for longitudinal cracking and crazing.
The larger of the two indices is used.

First damage index

Id \ If	0	1-2	3	4-5
0	1	2	3	4
1-2	3	3	4	5
3	4	5	5	6
4-5	5	6	7	7

Severity \ Extent	0 to 10%	10 to 50%	> 50%
1	0	0	0
2	0	0	+1
3	0	+1	+1

Correction for repairs

Surface
Damage
Index
Is
Scale from (best) to 7 (worst)

Final rating

Figure A.3 Determination of Damage Index I_s

A.6 Development of Solutions

General

The development of maintenance or overlay solutions from a diagnosis combining a visual examination with other parameters such as bearing capacity and traffic is not independent of the context of the study. This section deals with the search for solutions in the context of road maintenance management systems.

The methodology of the studies performed by the LCPC may be represented schematically as follows:

a. Construction of a database, field survey of data, determination of network quality at time t_0

b. Analysis of data, determination of "technical solution", i.e. what should be done to the road network on the basis of its quality image at time t_0 to restore a certain level of service, in the absence of any budgetary constraints.

c. Search for an "optimised solution" reflecting both technical requirements and budgetary constraints, including staging of the work over several years, and time projection of the image of the network at time t_0 and of the technical solution. For this, laws describing change against time must be available. The World Bank's HDM III model proposes such laws for developing countries.

The technical solution is determined in two stages.

- First, "visual condition" and "bearing capacity" are combined; the bearing capacity is indicated by the deflection value and the visual condition is represented by index I_s as determined in the previous section. The combination generates a pavement quality rating on a scale from 1 to 9 and a conclusion as to what needs to be done: nothing, maintenance or an overlay.

- Then, in a second stage, the pavement quality rating and the level of traffic are combined and the work to be done is decided for each pair of values.

Pavement Quality Rating Q_i

The pavement quality rating Q_i is estimated by combining the value of the Index I_s qualifying the pavement surface and the deflection value qualifying the combined bearing capacity of the pavement and foundation soil.

Damage Index I_s

The damage index is divided into three ranges:

Rating 1 or 2	Little or no cracking or deformation; good surface condition requiring no (or only just requiring) immediate maintenance.
Rating 3 or 4	Cracks with little or no deformation, or deformation without cracking; intermediate surface condition, bad enough to trigger maintenance work in the absence of any other consideration.
Rating 5, 6 or 7	Extensive cracking and deformation; poor surface condition requiring major maintenance or overlay work.

Deflection

Deflection is also divided into three ranges, by thresholds d1 and d2:

d1 value above which pavement performance is generally good

d2 value above which pavement performance is poor

d1-d2 range of indecision.

The choice of thresholds d1 and d2 depends on many factors, such as climate, type and thickness of pavement, soils, axle loads, and so on. These values are generally based on a particular country's experience. They may, where possible, be based on the examination of control sections during the general study of the network. If values based on experience are not available in a country, this calibration may be done during the study. As far as Ethiopia is concerned, the issue of the determination of d1 and d2 is discussed in Part III of the present manual.

Table 4 gives the pavement quality rating Q_i as a function of the Index I_s and the deflection value. Three main ranges have to be considered:

Q_1, Q_2, and Q_3 : these ratings mean that there is nothing that needs to be done, or at most maintenance work depending on the traffic carried by the road. When sealing is required, the cracking index is used to determine the data and type of work.

Q_7, Q_8, and Q_9 : these ratings mean that the pavement requires an overlay, the thickness of which is determined by the traffic.

Q_4, Q_5, and Q_6 : this is a zone of indetermination for which the index I_s and the deflection value seem inconsistent. These cases should be further examined and eventually reclassified. The procedure is as follows:

Q_4 : pavement with marked damage in spite of a good bearing capacity. The validity of the deflection measurement should be checked together with the nature of the damages (in particular rutting layers of coated materials, unrelated to the deflection measurement). Depending upon the answer, Q_4 will be reclassified as Q_2 (priority to deflection) or Q_7 (priority to damage).

Q_5 : same analysis as above; allowance will be made for the position of the deflection with respect to the limits and to the traffic; depending upon the answer, may be reclassified as Q_3, Q_7, or Q_8.

Q_6 : pavement having a large deflection value without apparent damage; to validate or unvalidate the surface condition, check the age of the pavement or the date of the most recent work, together with the traffic level. Depending upon the answer, may be reclassified as Q_3 or Q_8.

Surface damage index Is \ Deflection	Class 1	Class 2	Class 3
	d1	d2	
1 - 2 Little or no cracking or no deformation	Q1 (maintenance)	Q3 (maintenance)	Q6 (to be reclassified)
3 - 4 Cracks with little or no deformation, deformation without cracks	Q2 (maintenance)	Q5 (to be reclassified)	Q8 (overlay)*
5 - 6 - 7 Cracks and deformation	Q4 (to be reclassified)	Q7 (overlay)*	Q9 (overlay)*

* or other rehabilitation method

Figure A.4: Pavement Quality Rating (Q_i) and Required Road Works

APPENDIX B

FIELDWORK

B.1 Pavement Condition Surveys

Condition surveys are essentially required to assess a pavement's physical distress and form the basis of a diagnosis regarding the maintenance or rehabilitation needs. Together with drainage, destructive and non-destructive testing, they are mandatory before rehabilitation design.

Flexible Pavements

One of the stated objectives of the PMS is to provide a pavement condition evaluation system. A condition survey conducted for a specific project, in order to be compatible with the PMS of the network, must follow the same method. Also, by using the same method, the results of the specific condition survey can be forwarded to the Office or Branch in charge of the PMS and incorporated into the PMS Road Data Bank.

Following is a summarized description of the method of classification and quantification of visible damage, adapted from the "VIZIR method", as presented in the **Manual of Procedures for PMS (Ref 12)** and copied for reference in Appendix A.

General

The method of classification and quantification of damage is intended to provide a picture of the road surface condition at the time of inspection and to identify zones of equal quality (in terms of three damage levels). The damage is divided into two categories:

- Type A damage, which characterizes the structural condition of the pavement and includes four types as follows:
  - Deformation;
  - Rutting;
  - (fatigue) cracking; and
  - crazing.
- Type B damage, generally unrelated to the pavement structural capacity. This type of damage may be caused either by defective placement, or by deficient materials quality, or by some special local condition, aggravated by traffic. Type B damage includes:
  - cracking other than fatigue cracking, i.e. longitudinal joints and transverse thermal shrinkage cracks;
  - potholes;
  - raveling and, more generally, all surfacing defects such as fretting, bleeding, etc.

Survey and Grading of Damage

The survey is intended to record, for any damage:

- its type;
- its severity; and
- its extent, i.e. the length of road affected, or, as appropriate, the area.

It is to be noted that the method was designed for use with automatic data recording equipment, which facilitates the recording of the distresses and the calculation of quality ratings. Nevertheless, the survey can also be done manually.
The severity of each type of damage is described in detail, in tabulated form as well as by illustrations (photographs) in Appendix A. Following are some examples.

Description of Type A Damage

Rutting appears in wheel tracks about 50 to 80 cm from the pavement edge. It may be due to settlement of the materials under heavily channelized traffic, or occasionally due to creep within the bituminous layers only.

As an example, rutting is characterized by the following levels of severity:

Severity 1 $h < 2$ cm
Severity 2 2 cm $< h <$ 4 cm (see Figure B.1)
Severity 3 4 cm $< h$

Where h is the depth of rutting measured by a straight edge. As indicated earlier, complete illustrations and guidance regarding severity levels are given in Appendix A for rutting, as well as for other types of Type A damage, and the user should refer to that Appendix for complete details.

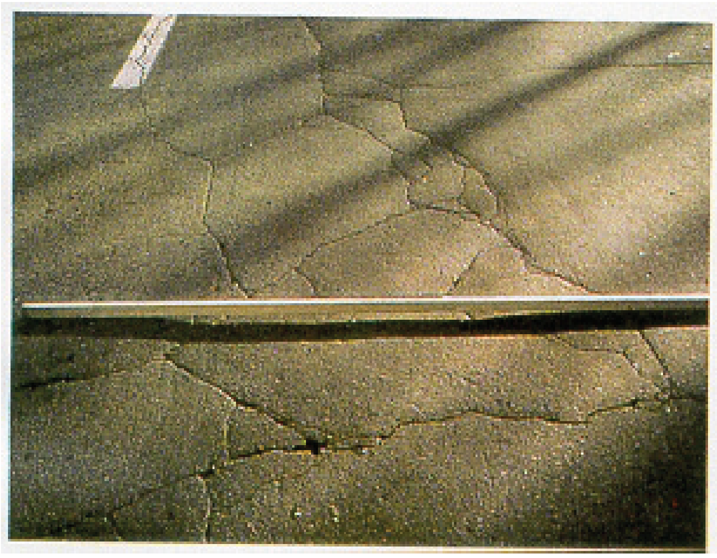

Figure B.1: Rutting – Severity Level 2

Description of Type B Damage

Similarly, damage of Type B is characterized by a severity level, as detailed in Appendix A. One example is that of cracking of longitudinal joints. This type of crack is initially single and straight (Severity 1, e.g. Figure B.2). Under traffic, the crack evolves, branches or doubles, and eventually loses material at the edges (Severity 2, 3).

Figure B.2: Cracking – Severity Level 1

Rigid Pavements

As for flexible pavements, a visual condition (distress) survey is essential for planning rehabilitation efforts. In order to make knowledgeable decisions, the engineer should have the following information:

- Distress type;
- Distress severity; and
- Distress amount.

The distress type will assist in defining probable cause(s) of the distress and ultimately help in selecting a rehabilitation strategy suitable to repair and prevent the recurrence of the problem. The distress is not necessarily related to traffic loads. Tables B.1 and B.2 (from **Ref. 3**) classify the distress types of jointed concrete and continuously reinforced concrete pavements respectively.

The condition survey will also document the severity of the distress and provide a record of the pavement condition at the time of the survey, including the location of the distress. Thus, differences between sections of the pavement (or lanes) will become apparent and guide in the rehabilitation design process.

Definitions of the major distress types and of severity levels suggested for use during the condition survey are presented in **Appendix A**, together with recommended methods of measurement (to record the distress amount).

B.2 Drainage Survey

In the design of pavement rehabilitation, the possible contribution of drainage improvement should be investigated. The existing drainage of the pavement and subgrade may be inadequate and the pavement distress may result from this condition or be accelerated by it.

The pavement condition survey will often indicate moisture-related distress. Distress types in flexible pavements which may either be caused or be accelerated by moisture include stripping, rutting, depression, fatigue cracking and potholes. In rigid pavement, they include pumping, "D" cracking, joint deterioration, faulting and corner breaks.
Subsequent to the pavement condition survey and depending on whether or not drainage-related distress is apparent, a determination needs to be made regarding the need for a specific drainage survey.
It should be recognized that even when moisture-related distress is absent, drainage deficiencies may exist which need to be corrected. Maintenance personnel regularly involved with a particular section of road under evaluation should be consulted in this regard, as they are a good source of relevant information.

Table B.1 General Categorization of Jointed Concrete Pavement Distress (from Ref. 3)

Distress Type	Primarily Traffic Load Caused	Primarily Climate/Materials Caused
1. Blow-up		X
2. Corner break	X	
3. Depression		X
4. Durability “D” cracking		X
5. Faulting of transverse joints and cracks	X	
6. Joint load transfer associated distress	X	X
7. Joint seal damage of transverse joints		X
8. Lane/shoulder dropoff or heave		X
9. Lane/shoulder joint separation		X
10. Longitudinal cracks		X
11. Longitudinal joint faulting	X	X
12. Patch deterioration	X (M, H)	X (L)
13. Patch adjacent slab deterioration	X	X
14. Popouts		X
15. Pumping and water bleeding	X (M, H)	X (L)
16. Reactive aggregate durability distress		X
17. Scaling, map cracking and crazing		X
18. Spalling (transverse and longitudinal joints)	X (M, H)	X (L, M. H)
19. Spalling (corner)		X
20. Swell		X
21. Transverse and diagonal cracks	X (L, M, H)	X (L)

Table B.2 General Categorization of Continuously Reinforced Concrete Pavement Distress (from Ref. 3)

Distress Type	Primarily Traffic Load Caused	Primarily Climate/Materials Caused
1. Asphalt patch deterioration	X	
2. Blow-up		X
3. Concrete patch deterioration	X (M, H)	X (L)
4. Construction joint distress		X
5. Depression		X
6. Durability “D” cracking		X
7. Edge punchout	X	
8. Lane/shoulder dropoff or heave		X
9. Lane/shoulder joint separation		X
10. Localized distress		X
11. Longitudinal cracking		X
12. Longitudinal joint faulting	X	X
13. Patch adjacent slab deterioration	X	X
14. Popouts		X
15. Pumping and water bleeding	X (M, H)	X (L)
16. Reactive aggregate distress		X
17. Scaling, map cracking and crazing		X
18. Spalling	X	X
19. Swell		X
20. Transverse cracking	X (M, H)	X (L, M)

Note**: L, M, H refer to low**, medium and high severity levels.

The first step in drainage evaluation will include a review of the as-built documents collected as part of the data collection discussed in Chapter 2. That will encompass the determination of the initial drainage provisions, and examination of pavement cross-sections and profiles. If moisture-related distress is apparent, then the original drainage system is inadequate (due to design or present condition). The next step of the evaluation will include an examination of topographical features. This may lead to discovering streams or wet areas above the pavement elevation.

The drainage evaluation also requires a site investigation (preferably during a wet season), with the aim of answering such questions as the following:

- Where and how does water move across the pavement surface?
- Where does water collect on and near the pavement?
- How high is the water level in the ditches?
- Do the joints and cracks contain any water?
- Does water pond on the shoulder?
- Does water-loving vegetation flourish along the roadside?
- Are deposits of fines or other evidence of pumping (blowholes) visible at the pavement's edge?
- Do the inlets contain debris or sediment buildup?
- Are the joints and cracks sealed well?

If the original drainage design appears adequate, the site investigation should also verify that it has actually been entirely built as planned, and determine whether changes have taken place since it was built.

Drainage problems often encountered include shallow side ditches, broken or clogged pipes and outlets, permeable shoulders and slow draining aggregate roadbase with no outlets or outlets of insufficient capacity.

Subgrade problems related to drainage include pockets of poor soils (e.g. saturated silts and/or clays, or organic materials) and localized springs, groundwater seepage or non-functioning subsurface drainage systems.

B.3 NDT Deflection Measurements

The use of non-destructive testing (NDT) for the measurement of pavement deflection under applied loading has been part of the process of structural evaluation and rehabilitation design for decades, using various types of equipment (e.g. Ref. 13). The total deflection under load has been used as an indicator of the load-carrying capacity of the pavement, especially for flexible pavements, by correlating allowable repetitions of ESAs to the maximum deflection. This has later been refined by taking into consideration only the "rebound" or "elastic" portion of the deflection as the key indicator of performance, rather than the total deflection under load. Various methods utilizing measurements of the slope (e.g. radius of curvature) of the deflection basin (or deflection bowl) under load have also been developed to further characterize the pavement behavior and structural capacity.

In this manual, emphasis is placed on measurements of the rebound deflection made by means of a Benkelman Beam. This presents the advantages of a certain consistency with the procedures of the PMS, a relatively simple field procedure, and a consistency with the selected overlay design methods presented further in the manual. An alternate deflection survey procedure is also outlined, using a Falling Weight Deflectometer (FWD). This method, which is well suited and actually intended to make use of deflection bowl measurements, requires more logistical support and is more expensive.

Benkelman Beam Rebound Deflection Testing Procedure

The following (adapted from Ref. 11) gives a procedure for determining the static Benkelman Beam rebound deflection at a point on an asphalt pavement structure under specified axle load, tire size, tire spacing and tire pressure. The absolute minimum spacing of sample locations should be 100 m, on either side (in either direction) of the road. Shorter intervals should be adopted if practical, and in areas of special concern (particular distress).

Equipment

Major equipment includes:

1. A Benkelman Beam as per AASHTO T256.

   Painting the beam white or aluminum will reduce temperature effects.

2. A 4.5 metric tonne truck as the reaction load. The vehicle shall have an 8.2 tonne load equally distributed on the two dual wheels of the rear axle. The clear distance between two tires of each dual shall be a minimum of 50 mm. The tires shall be 10.00 x 20, 12 ply, inflated to a pressure of 552 kPa. The use of tires with tubes and rib treads is recommended.

3. Tire pressure-measuring gauge.

4. Standard Iron-Constantan thermocouple wire and a temperature potentiometer (any other surface temperature measuring equipment that gives results comparable to the thermocouple potentiometer equipment may be used).

Procedure

1. The preselected point to be tested is located (usually only the outside lane is tested) and marked on the pavement. Points should be located 0.6 m from the pavement edge if the lane width is less than 3.35 m; 0.9 m from the pavement edge if the lane width is 3.35 m or more.

2. Center one set of dual wheels of the truck over the marked point. A location within 75 mm of the point is acceptable.

3. Insert the probe of the Benkelman Beam between the dual wheels and place on the selected sample point.

4. Remove the locking pin from the beam and adjust the front legs to permit approximately 13 mm travel of the dial gauge stem.

5. Start the buzzer on the beam and record the initial dial reading.

6. Immediately after recording the reading, drive the truck slowly forward 9 m or more.

7. Record the final dial reading. When the dial movement stops, stop the buzzer. The dial movement may resume after a short pause, but no more readings are necessary.

8. Measure the pavement surface temperature[1] in the following manner:

   a. At a point not less than 254 mm from the pavement's edge, drive a small hole into the pavement 3 mm in diameter and 3 mm in depth.

   b. Fill the hole with water or asphalt. Insert a thermocouple with the wire bent at right angles 5 mm from the end, 3 mm into the water or asphalt.

   c. Read the temperature with a temperature potentiometer. At this time, also record the air temperature.

1 One pavement surface temperature measurement an hour usually will be sufficient. Make the measurement at the location currently being tested for deflection.

9. Check the truck tire pressure once a day and correct to the 552 kPa standard equipment, if necessary.

10. Determine the total thickness, to the nearest 25 mm of asphalt-bound components in the pavement structure. This may be done from construction records (cf. Chapter 2), by core drilling, or by a small excavation at the pavement's edge. Determine also the types and general condition of materials in the remainder of the pavement structure, i.e., "granular roadbase, wet" or "granular subbase, saturated and contaminated with silt from subgrade".

Calculations

Subtract the final dial reading from the initial dial reading and record. The total pavement rebound deflection is twice the dial movement during the test (two-to-one is the usual ratio for the Benkelman Beam; some models may be built with a different ratio).

Report

The report shall include the following:

Sample location
Total pavement rebound deflection
Pavement surface temperature
Air temperature
Asphalt pavement thickness

For rigid pavements, the deflection measurements using the Benkelman Beam are made along the edge of the concrete slab, according to the following guidelines essentially taken from those of the Asphalt Institute (Ref. 11):

For two-lane highways, the deflection measurements are made on the outside edge on both sides of the centerline. For divided highways, deflections should be measured on the outermost edge only. Additional deflection measurements should be made at corners, joints, cracks, and deteriorated pavement areas, to determine load-transfer capability. The total vertical movement in these areas is important, but the differential movement from one slab to another is more important, because it may indicate the need for undersealing. The differential movement is considered a measurement of the load-transfer capability of the slab. Load-transfer is defined as the ratio of the deflection on one side of the joint or crack to the deflection on the opposite side. This is illustrated in Figure B.3.

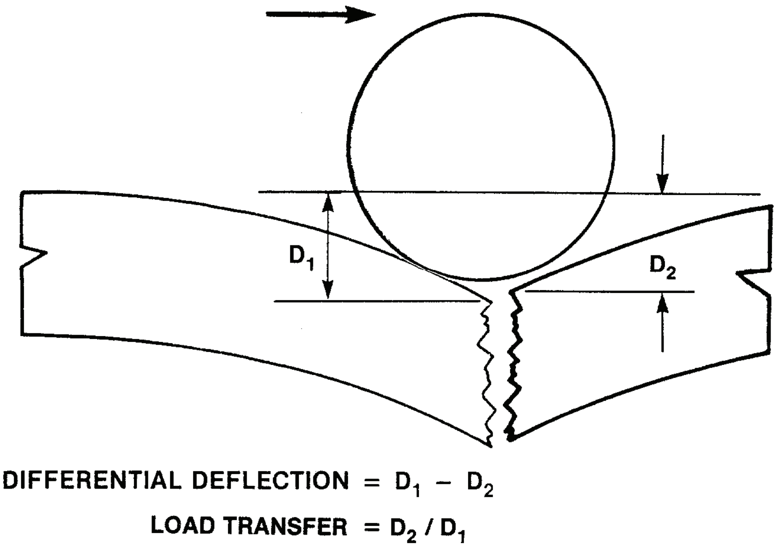

Figure B.3 Differential Deflection and Load Transfer (from Ref. 11)

The deflection testing procedure for rigid pavements is as follows:

The Benkelman Beam should be positioned on the shoulder as shown in Figure B.4.

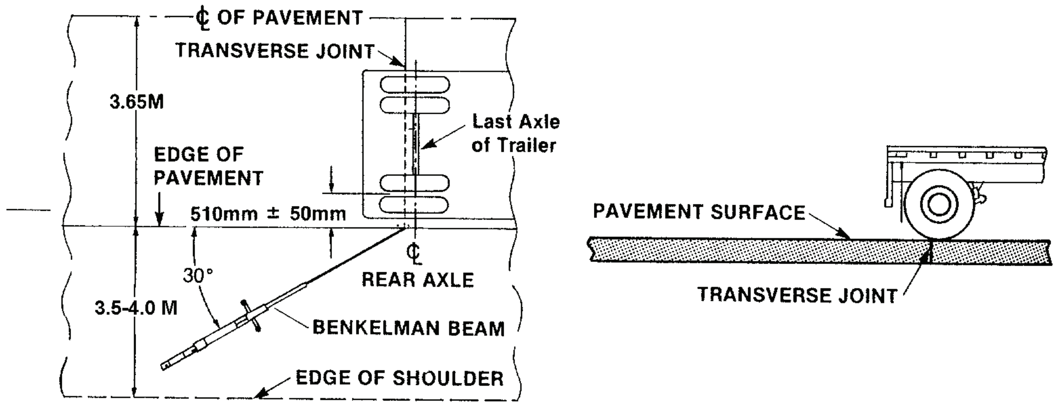

Figure B.4 Position of vehicle and Benkelman Beam for making static rebound deflection measurements on rigid pavements

Measurements should be made on either side of the joint or crack to determine the parameters described in Figure B.3. They should be taken, to the extent practical, during the most severe environmental circumstances, e.g. at the end of a wet season. A suggested minimum spacing is 100 m, with more closely spaced sample locations where cut and fill sections alternate rapidly.

Falling Weight Deflectometer (FWD)

The Falling Weight Deflectometer (FWD) is classed as an impulse deflection device providing a non-destructive means of determining a pavement response to dynamic loading.

Various models and manufacturers (e.g. Dynatest, Foundation Mechanics, KUAB, etc.) exist, which follow the same principle and provide similar information. A typical arrangement is given schematically in Figure B.5.

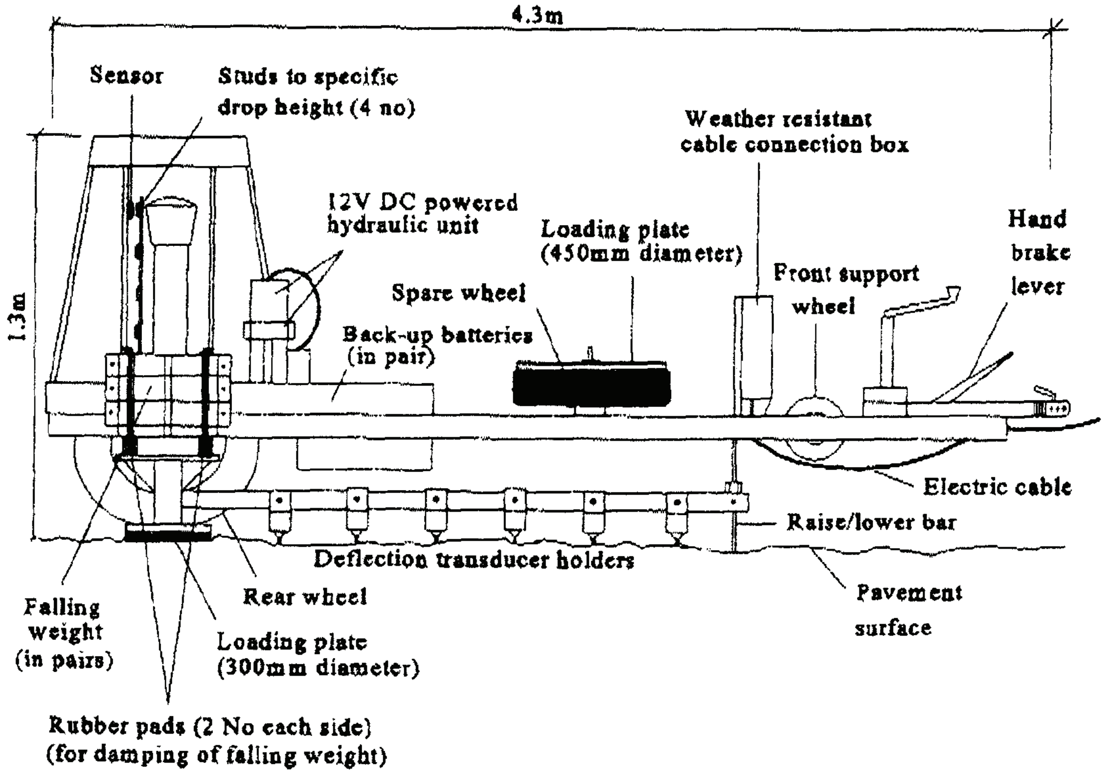

Figure B.5 Falling Weight Deflectometer

Weights are raised to a predetermined height and allowed to fall on a specially-designed plate, transmitting an impulse force to the pavement. The shape of the load pulse obtained is somewhat similar to that obtained from a moving wheel load. Figure B.6 schematically illustrates a typical pavement structure under a FWD dynamic load. By varying the weights dropped and the height from which they are dropped, the force developed upon impulse can be varied, as can the range of wheel load effects simulated. For instance, a typical dynamic force of 40 KN (a generally recommended magnitude) applied to a 30 cm diameter disk by a dropping weight on the order of 60 kg is common.

Usually, six or seven geophones (sensors, or velocity transducers) record the deflections at offset distances from the center of load application. A typical configuration for flexible pavements is given in Figure B.7.

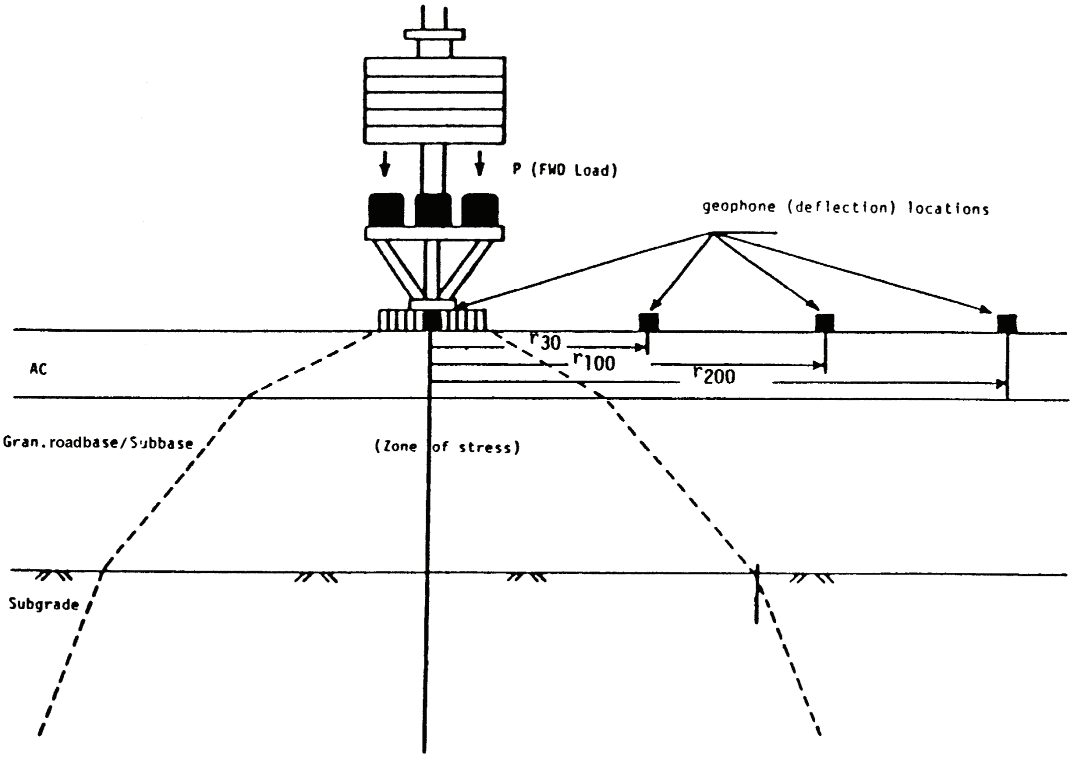

Figure B.6 Schematic of Stress Zone within Pavement Structure under the FWD Load (from Ref. 3)

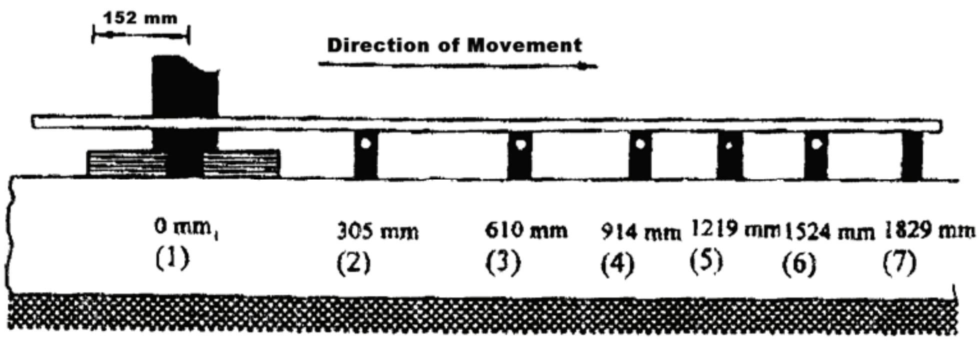

Figure B-7 Deflection Sensors Configuration for Flexible Pavement Testing

The tests should be made in the outer wheelpath at intervals sufficient to adequately assess conditions. Intervals of 25 m to 250 m are typical. Areas that are seriously deteriorated and obviously will require repairs should not be tested, or tested separately so as not to influence statistical analyses of the results.

B.4 Existing Pavement Structure and Subgrade Assessment

The data obtained from PMS records and the subsequent review of those records should serve as the basis for the fieldwork. The PMS data can be reconfirmed by carrying out a specific field study and reviewing the field data.

In general, field testing is categorized into two broad areas: nondestructive testing (NDT) and destructive testing. Destructive tests (such as test pits, DCP tests) require the physical removal of pavement layer material in order to obtain a sample (either disturbed or undisturbed) or to conduct an in-situ test. Such testing has many disadvantages and limitations, particularly when conducted on moderate to heavily trafficked roadways. Practical restraints in terms of time and money severely limit the number and variety of destructive tests conducted on routine rehabilitation studies.

Nondestructive testing, on the other hand, does not necessitate physical disturbance of the pavement. The most widely used form of NDT is associated with the field deflection tests noted in Section B.3.

In spite of their disadvantages, destructive tests are the only definitive means of determining pavement layer thicknesses and layer material type.

Test Pits

Test pits represent one of the common methods of investigation to determine the thickness and type of the various pavement layers and to assess the subgrade. Samples from each pavement layer and subgrade can be collected for visual inspection and subsequent laboratory testing. The test results can be used in the rehabilitation design analysis and to check conformance of the material with standard specifications. Test pits shall be dug through the pavement layers and into the subgrade soil for a minimum total depth of 0.80 m (alternatively, the pit should extend at least 0.20 m below subgrade level).

The spacing of the test pits should depend on sound engineering judgement and be guided by a prior review of all possible documents, as well as a visual pavement condition survey.

However, as a general guideline, one test pit every 500 meters, alternating on either side of the roadway, is recommended. The position of each test pit shall be accurately determined and reported.

DCP Testing

DCP testing frequency can be the same as that of the test pits, i.e. every 500 m. In fact, the DCP test location can be adjacent to the test pit location. However, as mentioned before, depending on the pavement location, the frequency can be altered to better assess the pavement and to optimize the testing program.

The DCP instrument is designed for the rapid in-situ measurement of the structural properties of existing road pavements (with the exception of hard layers like concrete) and subgrade. In cases of material that the DCP cannot penetrate, a hole can be drilled through that layer using a suitable drill. Lower layers of the pavement can then be tested in the normal way.

The DCP test results can be compared with the laboratory test results, and the data collected from review of the possible documents, including PMS records. Additional testing may be necessary in case of discrepancies to better evaluate pavement conditions.

APPENDIX C

TYPICAL RIGID PAVEMENT DISTRESS TYPE-SEVERITY DESCRIPTIONS

Name of Distress: Blow-up

Description:

Most blow-ups occur during the spring and hot summer at a transverse joint or wide crack. Infiltration of incompressible materials into the joint or crack during cold periods results in high compressive stresses in hot periods. When this compressive pressure becomes too great, a localized upward movement of the slab or shattering occurs at the joint or crack. Blow-ups are accelerated due to a spalling away of the slab at the bottom, creating reduced joint contact area. The presence of "D" cracking or freeze-thaw damage also weakens the concrete near the joint, resulting in increased spalling and blow-up potential.

Severity Levels:

L— Blow-up has occurred, but only causes some bounce of the vehicle which creates no discomfort.
M— Blow-up causes a significant bounce of the vehicle which creates some discomfort. Temporary patching may have been placed because of the blow-up.
H— Blow-up causes excessive bounce of the vehicle, which creates substantial discomfort and/or a safety hazard and/or vehicle damage, requiring a reduction.

How to Measure:
Blow-ups are measured by counting the number existing in each uniform section. Severity level is determined by riding in a mid- to full-sized sedan weighing approximately 13.3-16.9 kN over the uniform section at the posted speed limit. The number is not as important as the fact that initial blow-ups signal a problem with "lengthening" or gradual downhill movement—and others should be expected to occur until the maximum distance is down to 300 meters between blow-ups, the distance required to develop full restraint of an interior section.

Name of Distress: Corner Break

Description:

A corner break is a crack that intersects the joints at a distance less than 1.8 m on each side, measured from the corner of the slab. A corner break extends vertically through the entire slab thickness. It should not be confused with a corner spall, which intersects the joint at an angle through the slab and is typically within 0.3 m from the slab corner. Heavy repeated loads, combined with pumping, poor load transfer across the joint, and thermal curling and moisture warping stresses, result in corner breaks.

Severity Levels:

L—Crack is tight (hairline). Well-sealed cracks are considered tight. No faulting or break-up of broken corner exists. Crack is not spalled.
M—Crack is working and spalled at medium severity, but break-up of broken corner has not occurred. Faulting of crack or joint is less than 13 mm. Temporary patching may have been placed because of corner break.
H—Crack is spalled at high severity, the corner piece has broken into two or more pieces, or faulting of crack or joint is more than 13 mm.

How to Measure:

Corner breaks are measured by counting the number that exists in the uniform section. Different levels of severity should be counted and recorded separately. Corner breaks adjacent to a patch will be counted as "patch adjacent slab deterioration".

Name of Distress: Depression

Description:

Depressions in concrete pavements are localized settled areas. There is generally significant slab cracking in these areas, due to uneven settlement. The depressions can be located by stains caused by oil droppings from vehicles and by riding over the pavement. Depressions can be caused by settlement or consolidation of the foundation soil, or can be "built-in" during construction. They are frequently found near culverts. This is usually caused by poor compaction of soil around the culvert during construction. Depressions cause slab cracking, roughness, and hydroplaning when filled with water of sufficient depth.

Severity Levels:

L— Depression causes a distinct bounce of vehicle which creates no discomfort.
M— Depression causes significant bounce of the vehicle, which creates some discomfort.
H— Depression causes excessive bounce of the vehicle, which creates substantial discomfort, and/or a safety hazard, and/or vehicle damage, requiring a reduction in speed for safety.

How to Measure:

Depressions are measured by counting the number that exists in each uniform section. Each depression is rated according to its level of severity. Severity level is determined by riding in a mid- to full-sized sedan weighing approximately 13.3-16.9 KN over the uniform section at the posted speed limit.

Name of Distress: Durability ("D") Cracking

Description:

"D" cracking is a series of closely-spaced, crescent-shaped hairline cracks that appear at a PCC pavement slab surface adjacent and roughly parallel to transverse and longitudinal joints, transverse and longitudinal cracks, and the free edges of pavement slab. The fine surface cracks often curve around the intersection of longitudinal joints/cracks and transverse joints/cracks. These surface cracks often contain calcium hydroxide residue, which causes a dark coloring of the crack and immediate surrounding area. This may eventually lead to disintegration of the concrete within 0.3 to 0.6 m or more of the joint or crack, particularly in the wheelpaths. "D" cracking is caused by freeze-thaw expansive pressures of certain types of coarse aggregates and typically begins at the bottom of the slab, which disintegrates first. Concrete durability problems caused by reactive aggregates are rated under "Reactive Aggregate Distress".

Severity Levels:

L— The characteristic pattern of closely-spaced fine cracks has developed near joints, cracks, and/or free edges; however, the width of the affected area is generally <30 cm wide at the center of the lane in transverse cracks and joints. The crack pattern may fan out at the intersection of transverse cracks/joints with longitudinal cracks/joints. No joint/crack spalling has occurred, and no patches have been placed for "D" cracking.
M— The characteristic pattern of closely-spaced cracks has developed near the crack, joint, or free edge and: (1) is generally wider than 30 cm at the center of the lane in transverse cracks and/or joints; or (2) low- or medium-severity joint/crack or corner spalling has developed in the affected area; or (3) temporary patches have been placed due to "D"

cracking-induced spalling.

H— The pattern of fine cracks has developed near joints or cracks and (1) a high severity level of spalling at joints/cracks exists and considerable material is loose in the affected area; or (2) the crack pattern has developed generally over the entire slab area between cracks and/or joints.

How to Measure:

“D” cracking is measured by counting the number of joints or cracks (including longitudinal) affected. Different severity levels are counted and recorded separately. “D” cracking adjacent to a patch is rated as patch-adjacent slab deterioration. “D” cracking should not be counted if the fine crack pattern has not developed near cracks, joints, and free edges. Popouts and discoloration of joints, cracks, and free edges may occur without “D” cracking.

Name of Distress: Faulting of Transverse Joints and Cracks

Description:

Faulting is the difference of elevation across a joint or crack. Faulting is caused in part by a buildup of loose materials under the approach slab near the joint or crack, as well as depression of the leave slab. The buildup of eroded or infiltrated materials is caused by pumping from under the leave slab and shoulder (free moisture under pressure) due to heavy loadings. The warp and/or curl upward of the slab near the joint or crack due to moisture and/or temperature gradient contributes to the pumping condition. Lack of load transfer contributes greatly to faulting.

Severity Levels:

Severity is determined by the average faulting over the joints within the sample unit.

How to Measure:

Faulting is determined by measuring the difference in elevation of slabs at transverse joints for the slabs in the sample unit. Faulting of cracks is measured as a guide to determine the distress level of the crack. Faulting is measured 30 cm in from the outside (right) slab edge on all lanes except the innermost passing lane. Faulting is measured 30 cm in from the inside (left) slab edge on the inner passing lane. If temporary patching prevents measurement, proceed on to the next joint. Sign convention: + when approach slab is higher than departure slab, – when the opposite occurs. Faulting never occurs in the opposite direction.

Name of Distress: Joint Load Transfer System Associated Deterioration (Second Stage Cracking)

Description:

This distress develops as a transverse crack a short distance (e.g., 23 cm) from a transverse joint at the end of joint dowels. This usually occurs when the dowel system fails to function properly due to extensive corrosion or misalignment. It may also be caused by a combination of smaller diameter dowels and heavy traffic loadings.

Severity Levels:

L— Hairline (tight) crack with no spalling or faulting or well-sealed crack with no visible faulting or spalling.

M— Any of the following conditions exist: the crack has opened to a width less than 25 mm; the crack has faulted less than 13 mm; the crack may have spalled to a low- or medium-severity level; the area between the crack and joint has started to break up, but pieces have not been dislodged to the point that a tire damage or safety hazard is present; or temporary patches have been placed due to this joint deterioration.

H— Any of the following conditions exist: a crack with width of opening greater than 25 mm; a crack with a high-severity level of spalling; a crack faulted 13 mm or more; or the area between the crack and joint has broken up and pieces have been dislodged to the point that a tire damage or safety hazard is present.

How to Measure:

The number of joints with each severity level are counted in the uniform section.

Name of Distress: Joint Seal Damage of Transverse Joints

Description:

Joint seal damage exists when incompressible materials and/or water can infiltrate into the joints. This infiltration can result in pumping, spalling, and blow-ups. A joint sealant bonded to the edges of the slabs protects the joints from accumulation of incompressible materials and also reduces the amount of water seeping into the pavement structure. Typical types of joint seal damage are: (1) stripping of joint sealant, (2) extrusion of joint sealant, (3) weed growth, (4) hardening of the filler (oxidation), (5) loss of bond to the slab edges, and (6) lack or absence of sealant in the joint.

Severity Levels:

L— Joint sealant is in good condition throughout the section with only a minor amount of any of the above types of damage present. Little water and no incompressibles can infiltrate through the joint.

M— Joint sealant is in fair condition over the entire surveyed section, with one or more of the above types of damage occurring to a moderate degree. Water can infiltrate the joint fairly easily; some incompressibles can infiltrate the joint. Sealant needs replacement within 1 to 3 years.

H— Joint sealant is in poor condition over most of the sample unit, with one or more of the above types of damage occurring to a severe degree. Water and incompressibles can freely infiltrate the joint. Sealant needs immediate replacement.

How to Measure:

Joint sealant damage of transverse joints is rated based on the overall condition of the sealant over the entire sample unit.

Name of Distress: Lane/Shoulder Drop-Off or Heave

Description:

Lane/shoulder drop-off or heave occurs when there is a difference in elevation between the traffic lane and shoulder. Typically, the outside shoulder settles due to consolidation or a settlement of the underlying granular or subgrade material or pumping of the underlying material. Heave of the shoulder may occur due to frost action or swelling soils. Drop-off of granular or soil shoulder is generally caused from blowing away of shoulder material from passing trucks.

Severity Levels:

Severity level is determined by computing the mean difference in elevation between the traffic lane and shoulder.

How to Measure:

Lane/shoulder drop-off or heave is measured in the sample unit at all joints when joint spacing is > 15 m, at every third joint when spacing is < 15 m. It is also measured at mid-slab in each slab measured at the joint. The mean difference in elevation is computed from the data and used to determine severity level. Measurements at joints are made 0.3 m from the transverse joint on the departure slab only on the outer lane/shoulder.

Name of Distress: Lane/Shoulder Joint Separation

Description:

Lane/shoulder joint separation is the widening of the joint between the traffic lane and the shoulder, generally due to movement in the shoulder. If the joint is tightly closed or well-sealed so that water cannot easily infiltrate, then lane/shoulder joint separation is not considered a distress.

Severity Levels:

No severity level is recorded if the joint is tightly sealed.

L— Some opening, but less than or equal to 3 mm.

M—More than 3 mm, but equal or less than 10 mm opening.

H— More than 10 mm opening. Gravel or sod shoulders are rated as high.

How to Measure:

Lane/shoulder joint separation is measured and recorded in mm near transverse joints and at mid-slab. The mean separation is used to determine the severity level.

Name of Distress: Longitudinal Cracks

Description:

Longitudinal cracks occur generally parallel to the centerline of the pavement. They are often caused by improper construction of longitudinal joints or by a combination of heavy load repetition, loss of foundation support, and thermal and moisture gradient stresses.

Severity Levels:

L— Hairline (tight) crack with no spalling or faulting, or a well-sealed crack with no visible faulting or spalling.
M— Working crack with a moderate or less severity spalling and/or faulting less than 12 mm.
H— A crack with width greater than 25 mm; a crack with a high-severity level of spalling; or a crack faulted 13 mm or more.

How to Measure:

Cracks are measured in meters for each level of distress. The length and average severity of each crack should be identified and recorded.

Name of Distress: Longitudinal Joint Faulting

Description:

Longitudinal joint faulting is a difference in elevation of two traffic lanes measured at the longitudinal joint. It is caused primarily by heavy truck traffic and settlement of the foundation.

Severity Levels:

Severity level is determined by measuring the maximum fault.

How to Measure:

Where the longitudinal joint has faulted, the length of the affected area and the maximum joint faulting is recorded.

Name of Distress: Patch Deterioration (including replaced slabs)

Description:

A patch is an area where a portion or all of the original slab has been removed and replaced with a permanent type of material (e.g., concrete or hot-mixed asphalt). *Only permanent patches should be considered.*

Severity Levels:

L— Patch has little or no deterioration. Some low severity spalling of the patch edges may exist. Faulting across the slab-patch joints must be less than 6 mm. Patch is rated low severity even if it is in excellent condition.
M— Patch has cracked (low severity level) and/or some spalling of medium-severity level exists around the edges. Minor rutting may be present. Faulting of 6-19 mm exists. Temporary patches may have been placed because of permanent patch deterioration.

H— Patch has deteriorated by spalling, rutting, or cracking within the patch to a condition which requires replacement.

How to Measure:

The number of patches within each uniform section is recorded. Patches at different severity levels are counted and recorded separately. Additionally, the approximate area (in square meters) of each patch and type (i.e., PCC or asphalt) is recorded. All patches are rated either L, M, or H.

Name of Distress: Patch Adjacent Slab Deterioration

Description:

Deterioration of the original concrete slab adjacent to a permanent patch is given the above name. This may be in the form of spalling of the slab at the slab/patch joint, “D” cracking of the slab adjacent to the patch, a corner break in the adjacent slab, or a second permanent patch placed adjacent to the original patch.

Severity Levels:

Severity levels are the same as that described for the particular distress found. A second permanent patch, placed adjacent to a previously-placed permanent patch, will be rated here as medium severity. Temporary patches placed because of this deterioration will also be rated here as medium severity.

How to Measure:

The number of permanent patches with distress in the original slab adjacent to the patch at each severity level will be counted and recorded separately. Additionally, the type of patch (AC or PCC) and distress will be recorded separately.

Name of Distress: Popouts

Description:

A popout is a small piece of concrete that breaks loose from the surface due to freeze-thaw action, expansive aggregates, and/or nondurable materials. Popouts may be indicative of unsound aggregates and “D” cracking. Popouts typically range from approximately 25 mm to 10 cm in diameter and from 13 to 51 mm deep.

Severity Levels:

No degrees of severity are defined for popouts. The average popout density must exceed approximately one popout per square meter over the entire slab area before they are counted as a distress.

How to Measure:

The density of popouts can be determined by counting the number of popouts per square meter of surface in areas having typical amounts.

Name of Distress: Pumping and Water Bleeding:

Description:

Pumping is the movement of material by water pressure beneath the slab when it is deflected under a heavy moving wheel load. Sometimes the pumped material moves around beneath the slab, but often it is ejected through joints and/or cracks (particularly along the longitudinal lane/shoulder joint with an asphalt shoulder). Beneath the slab there is typically particle movement counter to the direction of traffic across a joint or crack that results in a buildup of loose materials under the approach slab near the joint or crack. Many times, some fine materials are pumped out, leaving a thin layer of relatively loose clean sand and gravel beneath the slab, along with voids causing loss of support. Pumping occurs even in pavement sections containing stabilized subbases.

Water bleeding occurs when water seeps out of joints and/or cracks. Many times it drains out over the shoulder in low areas.

Severity Levels:

L— Water is forced out of a joint or crack when trucks pass over the joints or cracks; water is forced out of the lane/shoulder longitudinal joint when trucks pass along the joint; or water bleeding exists. No fines can be seen on the surface of the traffic lanes or shoulder.
M—A small amount of pumped material can be observed near some of the joints or cracks on the surface on the traffic lane or shoulder. Blow holes may exist.
H— A significant amount of pumped materials exist on the pavement surface of the traffic lane or shoulder along the joints or cracks.

How to Measure:

If pumping or water bleeding exists anywhere in the sample unit, it is counted as occurring at highest severity level, as defined above.

Name of Distress: Reactive Aggregate Distresses

Description:

Reactive aggregates either expand in alkaline environments or develop prominent siliceous reaction rims in concrete. It may be an alkali-silica reaction or an alkali-carbonate reaction. As expansion occurs, the cement matrix is disrupted and cracks. It appears as a map-cracked area; however, the cracks may go deeper into the concrete than in normal map cracking. It may affect most of the slab or it may first appear at joints and cracks.

Severity Levels:

Only one level of severity is defined. If alkali-aggregate cracking occurs anywhere in the slab, it is counted. If the reaction has caused spalling or map cracking, these are also counted.

How to Measure:

Reactive-aggregate distress is measured in square feet or square meters.

Name of Distress: Scaling and Map Cracking or Crazing

Description:

Scaling is the deterioration of the upper 3 to 13 cm of the concrete slab surface. Map cracking or crazing is a series of fine cracks that extend only into the upper surface of the slab surface. Map cracking or crazing is usually caused by overfinishing of the slab and may lead to scaling of the surface. Scaling can also be caused by reinforcing steel being too close to the surface.

Severity Levels:

L—Crazing or map cracking exists; the surface is in good condition with no scaling.
M/H—Scaling exists.

How to Measure:

Scaling and map cracking or crazing are measured by area of slab in square meters.

Name of Distress: Spalling (Transverse and Longitudinal Joint/Crack):

Description:

Spalling of cracks and joints is the cracking, breaking or chipping (or fraying) of the slab edges within 0.6 m of the joint/crack. A spall usually does not extend vertically through the whole slab thickness but extends to intersect the joint at an angle. Spalling usually results from (1) excessive stresses at the joint or crack caused by infiltration of incompressible materials and subsequent expansion, (2) disintegration of the concrete from freeze-thaw action of “D” cracking, (3) weak concrete at the joint (caused by honeycombing), (4) poorly designed or constructed load transfer device (misalignment, corrosion), and/or (5) heavy repeated traffic loads.

Severity Levels:

L— The spall or fray does not extend more than 8 cm on either side of the joint or crack. No temporary patching has been placed to repair the spall.
M— The spall or fray extends more than 8 cm on either side of the joint or crack. Some pieces may be loose and/or missing, but the spalled area does not present a tire damage or safety hazard. Temporary patching may have been placed because of spalling.
H— The joint is severely spalled or frayed to the extent that a tire damage or safety hazard exists.

How to Measure:

Spalling is measured by counting and recording separately the number of joints with each severity level. If more than one level of severity exists along a joint, it will be recorded as containing the highest severity level present. Although the definition and severity levels are the same, spalling of cracks should not be recorded. *The spalling of cracks is included in rating severity levels of cracks*. Spalling of transverse and longitudinal joints will be recorded separately. Spalling of the slab edge adjacent to a permanent patch will be recorded as patch adjacent slab deterioration. If spalling is caused by “D” cracking, it is counted as both spalling and “D” cracking at appropriate severity levels.

Name of Distress: Spalling (Corner):

Description:

Corner spalling is the raveling or breakdown of the slab within approximately 0.3 m of the corner. However, corner spalls with both edges less than 8 cm long will not be recorded. A corner spall differs from a corner break in that the spall usually angles downward at about 45° to intersect the joint, while a break extends vertically through the slab. Corner spalling can be caused by freeze-thaw deterioration, “D” cracking, and other factors.

Severity Levels:

L— Spall is not broken into pieces and not loose.
M— One of the following conditions exists: Spall is broken into pieces; cracks are spalled; some or all pieces are loose or absent but do not present tire damage or safety hazard; or spall is patched.
H— Pieces of the spall are missing to the extent that the hole presents a tire damage or safety hazard.

How to Measure:

Corner spalling is measured by counting and recording separately the number of corners spalled at each severity level within the sample unit.

Name of Distress: Swell

Description:

A swell is an upward movement or heave of the slab surface, resulting in a sometimes sharp wave. The swell is usually accompanied by slab cracking. It is usually caused by frost heave in the subgrade or by an expansive soil. Swells can often be identified by oil droppings on the surface as well as riding over the pavement in a vehicle.

Severity Levels:

L— Swell causes distinct bounce of the vehicle which causes no discomfort.
M— Swell causes significant bounce of the vehicle which creates some discomfort.
H— Swell causes excessive bounce of the vehicle which creates substantial discomfort, and/or a safety hazard, and/or vehicle damage, requiring a reduction in speed for safety.

How to Measure:

The number of swells within the uniform section are counted and recorded by severity level. Severity levels are determined by riding in a mid- to full-sized sedan weighing approximately 13.3-16.9 kN over the uniform section at the posted speed limit.

Name of Distress: Transverse and Diagonal Cracks

Description:

Linear cracks are caused by one or a combination of the following: heavy load repetition, thermal and moisture gradient stresses, and drying shrinkage stresses. Medium- or high-severity cracks are working cracks and are considered major structural distresses. They may sometimes be due to deep-seated differential settlement problems. (Note: Hairline cracks that are less than 1.8 m long are not rated.)

Severity Levels:

L— Hairline (tight) crack with no spalling or faulting, a well-sealed crack with no visible faulting or spalling.
M—Working crack with low- to medium-severity level of spalling, and/or faulting less than 13 mm. Temporary patching may be present.
H— A crack with width of greater than 25 mm; a crack with a high-severity level of spalling; or a crack faulted 13 mm or more.

How to Measure:

The number and severity level of each crack should be identified and recorded. If the crack does not have the same severity level along the entire length, the crack is rated at the highest severity level present. Cracks in patches are recorded under patch deterioration.

APPENDIX D

METHOD FOR PREDICTING MEAN PAVEMENT TEMPERATURE

Definition and Purpose:

Mean pavement temperature is an average of the temperatures at the surface, mid-depth, and bottom of the asphalt-bound portion of the pavement, even though it may have been placed in several layers and at different times.

The mean pavement temperature is used to find a *Temperature Adjustment Factor*, required for adjusting pavement deflection values to a standard temperature of 21°C, as indicated in Step 1 of Appendix B.2 of the manual.

Information Required:

The temperature prediction method requires five items of information:

1. Location of test site—for identification of test data to guide in selection of the weather station from which air temperature data must be obtained.

2. Date of test—required to indicate the dates on which air temperature will be needed for pavement temperature adjustment.

3. Maximum and minimum air temperature—needed for each of the five days immediately prior to the date of deflection testing to provide an air-temperature history at the test site.

4. Pavement surface temperature—measured at the time the deflection test is performed.

5. Thickness of asphalt-bound portion of the pavement—required for selection of the proper curves on the pavement temperature chart.

All information listed above, except Item 3, is recorded during performance of the Benkelman beam deflection survey.

Item 3, the five days of air temperature history, prior to the date of the deflection measurements, can be obtained in one of three ways.

1. Hourly air temperatures may be read and recorded for a period of five days preceding each test location in the general vicinity where deflection measurements are made. From these data the maximum and minimum daily temperatures are determined.

2. Obtain from the nearest weather station the maximum and minimum air temperatures for each of the five days proceeding each day of testing. This station should be in a location that records essentially the same temperature readings as at the test site.

3. Obtain the maximum and minimum air temperatures for each of the five days proceeding each day of testing from published weather bureau (or similar agency) data. This is the most practical approach if the evaluation of test results is needed immediately.

When the five-day air temperature history has been obtained, the average of the ten values is determined. This average is used, together with pavement surface temperature to estimate the pavement temperature at any depth. Surface temperature is measured and recorded when the deflection test is performed. The date and time of these are also recorded.

Surface temperature may be measured by inserting a thermometer or thermocouple into a small, 6mm± deep hole or indentation in the surface of the pavement. The hole should be filled with water or asphalt, which should be allowed to come to equilibrium before the temperature is read.

Prediction of Mean Pavement Temperature:

Relationships between pavement surface temperature, air-temperature history, and pavement temperature at any depth have been derived and are shown in condensed form in Figure D-1. It should be used for overlay design only. Pavement temperatures at various depths can be estimated by applying the required information to this chart.

The first step is to add the five-day air temperature history (average of maximum and minimum air temperatures for five days preceding the test) and the pavement surface temperature to determine the value with which to enter the chart. Then, the *Mean Pavement Temperature* is estimated:

(a) Determine pavement thickness and mid-depth.

(b) Enter Figure D-1 on the horizontal scale with the pavement surface temperature. Extend a line vertically to intersect the depth lines. Extend lines horizontally from the depth representing the bottom of the layer to intersect the vertical scale. Interpolate between the depth lines as necessary.

(c) The sum of the surface temperature, mid-depth temperature, and bottom temperature is averaged to provide the Mean Pavement Temperature.

(d) The Mean Pavement Temperature is used to enter Figure 4.1 in the Manual to obtain a temperature adjustment factor for Benkelman beam deflection readings.

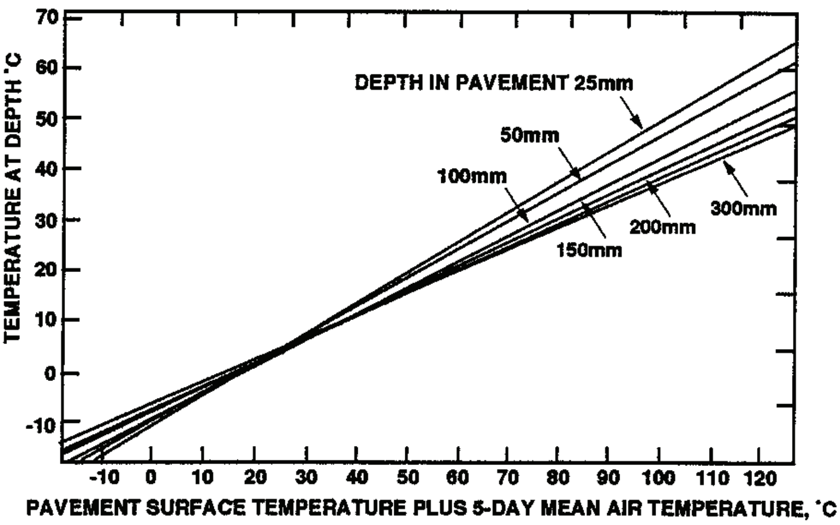

Figure D.1: Predicted Pavement Temperature

Example

Given: Asphalt concrete pavement 10cm thick. Surface temperature is 31°C. Average air temperature for 5 days preceding the test is 22°C. The adjusted surface temperature is 31 + 22 = 53°C (surface temperature plus 5-day average air temperature). Find predicted pavement mean temperature.

1. Surface temperature = 31°C

2. Temperature at 5cm depth = 30°C (From Figure D-1)

3. Temperature at 10cm depth = 27°C (From Figure D-1)

4. Pavement mean temperature =

$$\frac{31^\circ + 30^\circ + 27^\circ}{3} = 29^\circ$$

(Use this temperature in Figure 4.1 of the manual to find temperature adjustment factor, F.)

APPENDIX E

DERIVATION OF SN_{new}, STRUCTURAL NUMBER OF A NEW PAVEMENT

SN_{new} is required in the component analysis procedure (**Section 4.1 of the manual**) to determine a required asphalt overlay thickness. SN_{new} is computed in three steps as follows:

- Step 1: Select an appropriate required structure of a new pavement for the specific subgrade strength and traffic applicable to the project. The structure selected is characterized by the thicknesses T_i of its component layers, i.e. T_1, T_2, T_3= thicknesses of required pavement surfacing, roadbase and subbase layers respectively.

- Step 2: To each of the layers determined in Step 1, assign an appropriate structural layer coefficient a_i.

  The following structural layer coefficients are recommended:

  - Bituminous surface: a_1 = 0.44
  - Bituminous roadbase: a_1 = 0.30 (note: use 0.25 for in-place recycled materials)
  - Cement or lime stabilized roadbase: a_2 = 0.15 to 0.20
  - Granular roadbase: a_2 = 0.14
  - Cement or lime stabilized subbase: a_3 = 0.12
  - Granular subbase: a_3 = 0.11
  - Granular capping layer: a_3 = 0.09

- Step 3: compute SN_{new} as:

$$SN_{new} = a_1T_1 + a_2T_2 + a_3T_3$$

Example

The example of Section 4.1 is considered: subgrade strength class S4, anticipated future traffic class T6.

- Step 1:

  An adequate new pavement structure consists of:

  - 10 cm AC surfacing
  - 20 cm granular roadbase
  - 17.5 cm subbase

- Step 2:

  Structural layer coefficients are assigned as follows: a_1 = 0.44; a_2 = 0.14; a_3 = 0.11

- Step 3:

$$SN_{new} = a_1T_1 + a_2T_2 + a_3T_3 = 0.44x10 + 0.14x20 +0.11x17.5 = 9.13$$

APPENDIX F

DERIVATION OF SN_{eff}, EFFECTIVE STRUCTURAL NUMBER OF AN EXISTING PAVEMENT

The NDT method of SN_{eff} determination follows an assumption that the structural capacity of the pavement is a function of its total thickness and overall stiffness. The relationship between SN_{eff} and stiffness is:

$$SN_{eff} = \frac{6T_t}{1{,}000}\sqrt[3]{E_p} \qquad \text{(Equation 1)}$$

where T_t = total thickness (in centimeters) of all pavement layers above the subgrade

E_p = effective modulus of pavement layers above the subgrade (kPa)

E_p may be back-calculated from deflection data as follows.

The data required for the calculations is:

d_o = deflection measured at the center of the load plate (and adjusted to a standard temperature of 20° C), cm

P = applied load, kg

D = total thickness of pavement layers above the subgrade, cm

d_r = deflection at a distance r from the center of the load, cm

r = distance from center of load, cm

a = NDT load plate radius, cm

With the above data, using Figure F-1, the ratio of E_p/M_R can be calculated.

Where M_R = subgrade resilient modulus, kPa

$$= \frac{23.54P}{d_r r}$$

Based on the ratio of E_p/M_R and known value of M_R, E_p can be computed in kPa. SN_{eff} is then calculated using Equation 1.

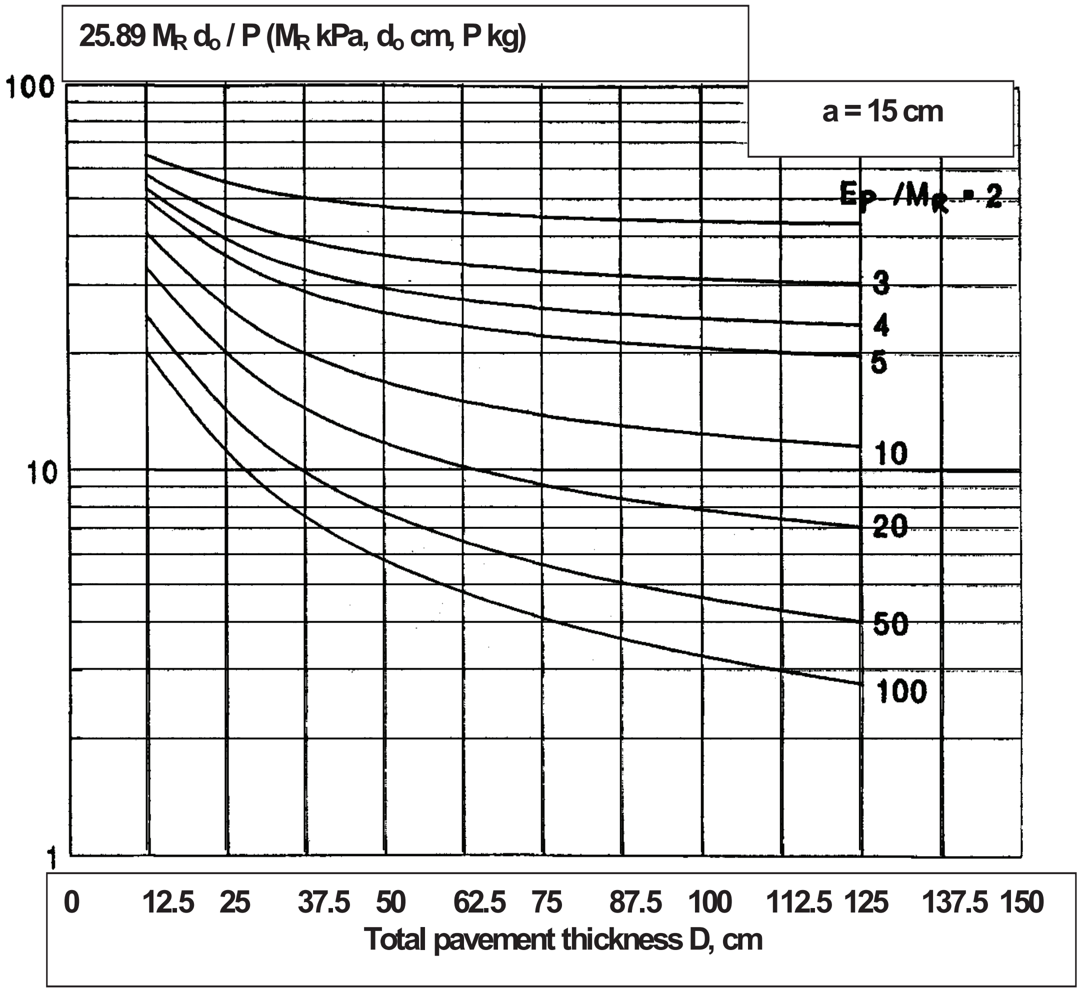

Figure F.1 Determination of E_p/M_R