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REPUBLIC OF THE PHILIPPINES
DEPARTMENT OF PUBLIC WORKS & HIGHWAYS

Feasibility Study of the Road Improvement Project on the Pan-Philippine Highway (Philippines-Japan Friendship Highway)

FINAL REPORT

GUIDE FOR PAVEMENT REHABILITATION DESIGN

(VOLUME VI)

SEPTEMBER, 1987

JAPAN INTERNATIONAL COOPERATION AGENCY

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PREFACE

From the time the initial improvements were undertaken up to present, the Pan-Philippine Highway (Philippines-Japan Friendship Highway) has played the very vital role in the country's economic development. At present, however, the Highway suffers many deficiencies including pavement deterioration problem which if not corrected, will impede the momentum of the socio-eocnomic development currently being pursued for the country.

Medium-Term Philippine Development Plan (1987-1992) specifically points out the strategy that priority shall be given to the maintenance of existing and soon-to-be completed infrastructure to prolong their useful lives, reduce costs to the users, and postpone huge investments for their major rehabilitation on replacement. Rehabilitation and restoration, as well as improvement and upgrading of existing facilities shall take the precedence over replacement and new construction as low-cost measures to provide acceptable levels of infrqstructure services.

In line with the stress on development policies, the Government of the Philippines thru the Department of Public Works and Highways (DPWH) has decided to pursue the Feasibility Study of the Road Improvement Project on the Pan-Philippine Highway (Philippines-Japan Friendship Highway) (The Feasibility Study) with a technical assistance from the Government of Japan.

In response to the request of the Government of the Philippines, the Government of Japan has decided to conduct the Feasibility Study thru the Japan International Cooperation Agency (JICA), which is the official agency responsible for the implementation of technical cooperation programs.

This Guide was drawn up compiling the investigation result and design outputs on pavement rehabilitation through undertaking the Feasibility Study. Since the investigations were confined to Sta. Rita-Aritao Sections (about 200 km) and Calamba-Calauag Section (about 180 km) of the Pan-Philippines Highway. This Guide may, therefore, have some deficiencies particularly on the following.

- Types of pavements included
- Types of pavement distresses discussed
- Construction executions/methods of rehabilitation
- Maintenance Works
- Pavement material properties
- Construction costs

However, this Guide intends to instigate engineering discussion among those who are engaged in the field of road engineering encompassing pavement rehabilitation during road planning, design and construction stages.

Therefore, this Guide is sincerely desired to be positively utilized, whenever applicable, for the development of the road engineering.

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CHAPTER 1

INTRODUCTION

CHAPTER 1
INTRODUCTION

1.1 The Feasibility Study

The objectives of the Feasibility Study are;

- To identify and establish the needed improvement works to upgrade the functional efficiency of the Study Sections.
- To prioritize the road segments for which improvement works are required.
- To conduct the feasibility study of typical improvement work proposed within the prioritized segments.

The Study Section is;

- Sta. Rita - Aritao Section (approximately 200 kilometers)
- Calamba - Calauag Section (approximately 180 kilometers)

The Final Report of the Feasibility Study composes of;

Volume I; Executive Summary
Volume II; Main Text
Volume III; Appendix
Volume IV; Drawings
Volume V; Guide for Design of Road Function Improvement
Volume VI; Guide for Design of Pavement Rehabilitation

This report is Volume VI of the Final Report. Whenever necessary, other Volumes are desired to be used for reference.

1.2 Limitation of the Guide

As mentioned, the main objectives of the Feasibility Study are to identify the road section where pavements require the rehabilitation, and to propose the adequate rehabilitation methods. The investigations were confined to the specified sections of the Highway. Main focus of the Feasibility Study were put on the design of portland cement concrete pavements. This Guide may, therefore, have the following limitations.

Types of pavement distress

The pavement investigation are all portland cement concrete (PCC). Distress discussed are mainly the cracks of PCC pavements.

Construction execution/methods for New Construction and Rehabilitation

Discussions are mainly focused on design of PCC pavement. Construction execution/methods are not discussed.

Maintenance Works

The routine/periodic maintenance are not the subject of the Feasibility Study. Those are, therefore, not discussed.

Pavement Material Properties

The very limited samples of subgrades, subbases and concrete slab were investigated. No detailed discussion on properties are made.

Construction Costs

The costs used in the Feasibility Study are November 1986 prices nearside Metro Manila. Whenever necessary, the costs should be updated. Moreover, the costs vary in different localities and construction condition. Such adjustments, therefore, are inevitable.

It is, therefore, noted that the study outputs, e.g. economic pavement types, most economical performance periods of initial pavement structures and economic indicators, should be adjusted accordingly.

<u>Rehabilitation Works discussed in this Guide</u>

The following new/reconstructed pavement and overlay methods are discussed in this Guide.

- New Construction/Reconstruction of PCC Pavement
- New Construction/Reconstruction of AC Pavement
- PCC Overlay - PCC Existing
- AC Overlay - PCC Existing
- AC Overlay - AC Existing

<u>Major Rehabilitation Works not discussed in this Guide</u>

The major rehabilitation works which are not included in this Guide are as follows:

- Full Depth Pavement Repair
- Partial Depth Pavement Repair
- Joint and Crack Sealing
- Subsealing of Concrete Pavements
- Grinding/Milling of Pavements
- Subdrainage Design
- Pressure Relief
- Restoration of Joint Load Transfer
- Surface Treatments

AASHTO Guide 1986 recommends on the selection of candidate methods to repair distress and prevents its recurrence as shown in Table 1.2-1 and 1.2-2. The detailed explanations on these methods are available in AASHTO Guide 1986.

CANDIDATE REPAIR AND PREVENTIVE METHODS

TABLE 1.2-1 RIGID PAVEMENT DISTRESS

Joint/Crack Distress	Repair Methods	Preventive Methods
Pumping		1. Reseal Joints 2. Restore Load Transfer 3. Subdrainage 4. Edge Support (PCC Shoulder/Edge Beam)
Faulting	1. Structural Overlay	1. Subseal 2. Reseal Joints 3. Restore Load Transfer 4. Subdrainage 5. Edge Support
Slab Cracking	1. Replace/Recycle Lane	1. Subseal loss of support 2. Restore Load Transfer 3. Structural Overlay
Joint or Crack Spalling		
Blowup		1. Pressure Relief Joint 2. Resealing Joints/Cracks
Punchouts		1. Polymer of epoxy grouting 2. Subseal loss of support 3. Rigid Shoulders

TABLE 1.2-2 ASPHALT PAVEMENT DISTRESS

Distress	Repair Methods	Preventive Methods
Alligator Cracking	Full-Depth Repair	Crack Sealing (May slow down alligator cracking)
Bleeding	Apply Hot Sand	
Block Cracking	Seal Cracks	
Depression	Level-Up Overlay	
Polished Aggregate	Skid Resistant Surface Treatment Slurry Seal	
Potholes	Full-Depth Repair	Crack Sealing and Seal Coats
Pumping	Full-Depth Repair	Crack Sealing and Seal Coats
Raveling and Weathering	Seal Coats	Rejuvenating Seal
Rutting	Level-Up Overlay and/or Cold Milling	
Swell	Removal and Replacement	Paved Shoulder encapsulation

1.3 Organization of the Guide

This Guide, mainly dealing with the design of new (or reconstructed) PCC pavements is organized into thirteen (13) chapters. These Chapters are compiled and arranged to follow the procedures for pavement rehabilitation design, as illustrated in Figure 1.3-1.

Chapter 1 specifies the scope, limitation and organization of this Guide.

Chapter 2 provides the general informations on pavement design principles covering pavement management system, design concept, pavement performance, alternative design strategy and reliability.

Chapter 3, 4 and 5 contain the basic engineering informations on pavement technology encompassing pavement material properties, traffic loading and pavement deteriorations.

Chapter 6 discusses the procedures to identify road sections where pavements rehabilitation will be required.

Chapter 7 deals with the detailed engineering surveys to obtain data necessary to develop a sound pavement rehabilitation design.

Chapter 8, 9, 11 and 12 details the pavement design procedures. Chapter 8 discussed the preparation and/or selection of design input data and chapter 9 demonstrates the basic structure design under the specific conditions.

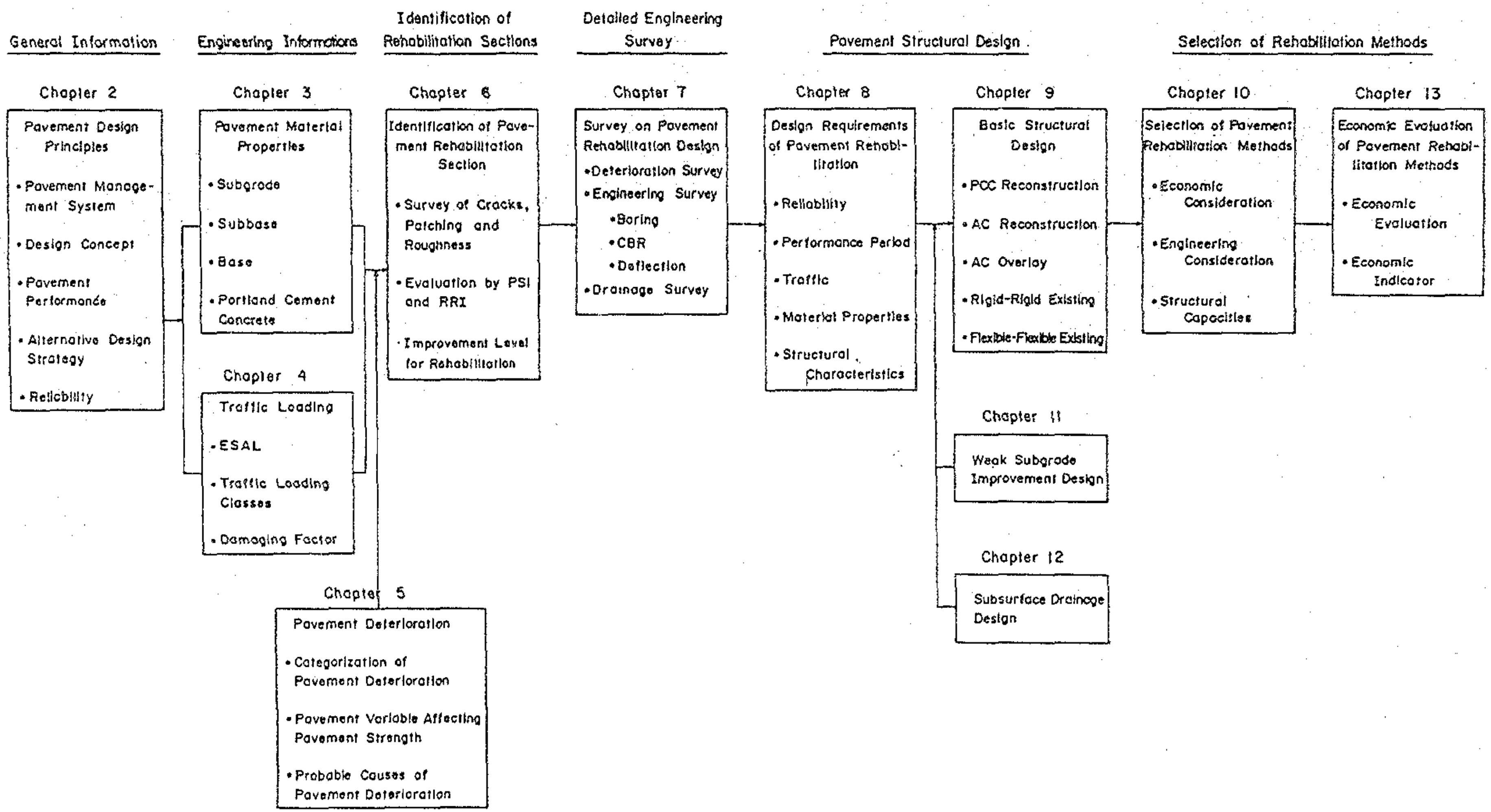
While, Chapter 11 provides the information on weak subgrade improvement design and chapter 12 subsurface drainage design.

Chapter 10 and 13 examines the selection criteria and/or procedures of adequate rehabilitation methods. Chapter 10 summarizes the economic and engineering consideration as well as structural capacities in selecting rehabilitation methods.

Chapter 13 discusses the economic evaluation procedures on rehabilitation methods and provide the economic indicators for the convenience of planning pavement rehabilitation project.

FIGURE 1.3-1 ORGANIZATION OF THIS GUIDE

—Procedures for Pavement Rehabilitation Design —



CHAPTER 2
PAVEMENT DESIGN PRINCIPLES

CHAPTER 2
PAVEMENT DESIGN PRINCIPLES

This chapter contains general principles on pavement design. The background information related to 1) pavement management system, 2) design concepts, 3) pavement performance, 4) alternative design strategy, and 5) reliability are provided.

2.1 Pavement Management System

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.

Pavement management is described in terms of two generalized levels: (1) the network management level, sometimes called the program level, where key administrative decisions that affect programs for road networks are made, and (2) the project management level, where technical management decisions are made for specific projects.

Pavement management systems can provide several benefits at both the network and project levels. Foremost among these is the selection of cost-effective alternatives. Whether new construction, rehabilitation, or maintenance is concerned, PMS can help management achieve the best possible value for the public.

At the network level, the management system provides information pertinent to the development of a program of new construction, maintenance, or rehabilitation that will optimize the use of available resources.

At the project level, detailed consideration is given to alternative design, construction, maintenance, or rehabilitation activities for a particular roadway section or project within the overall program. By comparing the benefits and costs associated with several alternative activities, an optimum strategy is identified that will provide the desired benefits or service levels at the least total cost over the analysis period.

The relationship of each activity at both levels, network and project is illustrated on Figure 2.1-1. The representative works under each activity are listed in Table 2.1-1.

This Guide complies with the terminology on pavement work classification defined in this pavement management system.

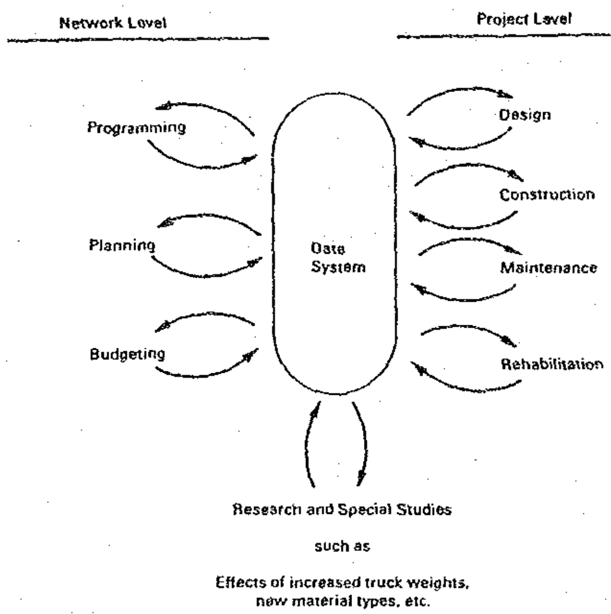


Figure 2.1-1 Activities of a pavement management system

TABLE 2.1-1 MAJOR WORKS OF PAVEMENT MANAGEMENT SYSTEM

Network Management Level (Program Level)	Project Level Design	Construction	Maintenance	Evaluation	Rehabilitation
. Assess Network Deficiencies . Establish Priorities . Budgeting	. Information on Materials Traffic, Environment Costs, etc. . Alternative Design Strategies . Analysis . Economic Evaluation and Optimization	. Specifications and Contracts . Construction Schedules and Opreation . Quality Control . Construction Records	. Standards and Schedules . Maintenance Operations . Budget Control . Maintenance Records	. Periodic Monitoring . Evaluation Records	. Problem Definition . Potential Problem Solutions . Selection of Preferred Solution . Implementation
			<u>Major Maintenance Works</u> . Shoulders . Related Drainage . Thin resurfacing of short length . Patching . Filling Potholes . Sealing cracks and joints . Repair of minor failures	<u>Major Periodic Monitoring</u> . Structural Evaluation (Existing Distress, Load-Carrying Capacity) . Functional Evaluation (PSR/PSI, Roughness)	<u>Major Rehabilitation Works</u> . Resurfacing to provide structural capacity or serviceability . Replacing or restoring malfunctioning joints . Substantial pavement undersealing when essential for stabilization . Removing and Replacing deteriorated materials . Reworking or strengthening of bases or subbases . Cracking and sealing of PCC pavement with AC . Adding underdrains

2.2 Design Concepts

The main concepts on pavement rehabilitation design adopted in this Guide are serviceability and performance concepts recommended by AASHTO in the following guide.

- AASHTO Guide for Design of Pavement Structures

  American Association of State Highway and Transportation Officials, 1986. (hereinafter called AASHTO Guide 1986)

The major changes from AASHTO Interim Guide for Design of Pavement Structures-1972 include the following considerations.

- Reliability
- Soil Support Value
- Layer Coefficient (Flexible Pavements)
- Drainage
- Environment
- Tied Shoulder and widened lanes
- Subbase erosion
- Life cycle costs
- Rehabilitation
- Pavement Management
- Load Equivalency Values
- Traffic
- Low-volume road
- Mechanistic-Empirical procedure

The main guides/texts used as reference in preparing this Guide are listed in Appendix 2-1.

2.3 Pavement Performance

Current concepts of pavement performance include some consideration of functional performance, structural performance, and safety. This Guide is primarily concerned with functional and structural performance. The survey methods for functional performance are discussed in Chapter 6.

2.3.1 Structural Performance

The structural performance of a pavement relates to its physical condition; e.g., occurrence of cracking, faulting, reveling, or other conditions which would adversely affect the load-carrying capability of the pavement structure or would require maintenance.

2.3.2. Functional Performance-Present Serviceability Index

(1) Serviceability-Performance Concept

The functional performance of a pavement concerns how well the pavement serves the user. In this context, riding comfort or ride quality is the dominant characteristic . In order to quantify riding comfort, the "serviceability-performance" concept was developed by the AASHO Road Test staff in 1957.

The serviceability-performance concept is based on five fundamental assumptions, summarized as follows;

- Highways are for the comfort and convenience of the travelling public (User).
- Comfort, or riding quality, is a matter of subjective response or the opinion of the User.
- Serviceability can be expressed by the mean of the ratings given by all highway Users and is termed the serviceability rating.
- There are physical characteristics of a pavement which can be measured ojectively and which can be related to

subjective evaluations. This procedure produces an objective serviceability index.

- Performance can be represented by the serviceability history of a pavement.

(2) Present Serviceability Index

The serviceability of a pavement is expressed in terms of the present serviceability index (PSI). The PSI is obtained from measurements of roughness and distress, e.g., cracking, patching and rut depth (flexible), at a particular time during the service life of the pavement. Roughness is the dominant factor in estimating the PSI of pavement.

The equations to determine the level of serviceability of the surviving pavement are described in Chapter 6.

(3) Initial and Terminal Serviceability Index

The scale for PSI ranges from 0 through 5, with a value of 5 representing the highest index of serviceability. For design it is necessary to select both an initial and terminal serviceability index.

The initial serviceability index (p_i) is an estimate by the user of what the PSI will be immediately after construction. Values of P_i, established for AASHO Road Test conditions were 4.2 for flexible pavements and 4.5 for rigid pavements. Because of the variation of construction methods and standards, it is recommended that more reliable levels be established by each agency based on its own conditions.

The terminal serviceability index (p_i) is the lowest acceptable level before resurfacing or reconstruction becomes necessary for the particular class of highway. An index of 2.5 or 3.0 is often suggested for use in the design of major highways, and 2.0 for highways with a lower classification. For relatively minor highways, where economic considerations dictate the initial expenditures be kept low, a p_t of 1.5 may be used.

2.3.3 Functional Performance-Rehabilitation Requirement Index

(1) Rehabilitation Requirement Concept

The present serviceability index defined by AASHO is estimated based on subjective assessments by road user, therefore the dominant factor is roughness. However, the physical distress is likely to influence a decision to initiate maintenance or rehabilitation. It is rather true for the pavements which the initial roughness (just after construction) are high. For these pavements, physical condition, more precisely cracking, should be given higher priority in identifying sections to be rehabilitated.

In accordance with this consideration, the rehabilitation requirements index (RRI) is proposed in the Feasibility Study. Highway/Maintenance/Construction engineers evaluate the necessity of rehabilitation instead of road users for PSI. RRI is obtained from measurement of cracking, patching and roughenss. Comparing with PSI, RRI is more closely related to physical distress, especially cracking.

(2) Rehabilitation Requirement Index

In the Feasibility Study, the equations to determine the level of rehabilitation requirement were established for Rigid Pavement in this country. PSI formula was also developed in accordance with AASHO's concept. See Chapter 6.

RRI was justified to have the reasonable compatability with AASHTO's PSI, therefore RRI can be directly used for rehabilitation equations in AASHTO Guide 1986.

2.4 Alternative Pavement Design Strategies

2.4.1 Economic Evaluation by Life-Cycle Costs Analysis

Project feasibility is determined at the network level, by comparison with other potential projects based on engineering economy. There are management decisions required to determine the feasibility and programming of a project.

When a project is economically feasible, there is a requirement to achieve maximum economy within that project. This within project economy is achieved by considering a variety of alternatives capable of satisfying the overall project requirements.

In pavement design, this involves the selection of the most economic pavement type including the most economical performance period of the initial pavement structure and the planned stage construction (planned rehabilitation).

Life-cycle costs analysis was coined to emphasize the need for a complete cost analysis to determine the most economic pavement type. Life-cycle costs refer to all costs and all benefits which are involved in the provision of a pavement during its complete life cycle. The costs include construction costs and maintenance cost, while the benefits are in general composed of savings in traffic cost and savings in maintenance cost. Based on these analyis, the most economical alternative of pavement type can be decided.

Basic structural designs described in Chapter 8 are performed on the most economical pavement types identified based on the life-cycle costs analysis.

2.4.2 Stage Construction (Planned Rehabilitation)

If the initial pavement structure is designed to withstand for the total traffic expected over the analysis period, the initial investment might be high, and the initial pavement structure may not be economical. This is especially true if life-cycle economic analysis are to be performed, where the trade-offs between the thickness designs of the initial pavement structure and any subsequent overlays can be evaluated. In such cases, stage construction alternatives are to be considered.

For this reason, the performance period of the initial pavement structure should be decided by life-cycle economic analysis.

It is also important to check the constraint on minimum performance period of the initial pavement structure before some major rehabilitation works is performed. The limit may be controlled by such factors as the public's perception of how long a "new" surface should last, the funds available for initial construction, life-cycle cost and other engineering consideration.

2.4.3 Performance Period of Initial Pavement Structures

(1) Analysis on Most Economical Performance Period

When a pavement project is planned and assured economically feasible, it is required to achieve the maximum economy within that project. It involves a detailed economic evaluation of the possible alternatives within the project to optimize the project investment. It is essential in economic evaluation that all costs occuring during the life of the facility (Life-cycle costs) be included.

An economic evaluation should consider many possible alternatives within the constraints of time and resources. It involves the selection of pavement types (rehabilitation method) and planned stage construction (performance period of initial pavement structures).

Analysis on most economical performance period of initial pavement structures were made, by an electric computer, selecting most economical pavement structure from alternatives of each proposed rehabilitation method under the following conditions:

- Rehabilitation Methods

  5 methods, Refer to Figure 9.1-1

- Alternative of Each Rehabilitation Method

  25 Alternatives for Heavy and Extra Heavy Loading Traffic and 15 Alternatives for Light Loading Traffic;Refer to Figure 9.1-1.

- Traffic Loading Classes

  Heavy Loading Traffic, 5 cases (A to E)
  Extra Heavy Loading Traffic, 5 cases (F to J)
  Light Loading Traffic, 3 cases (L_1 to L_3)

- Strength of Subgrade ; 3 cases of CBR (3, 6 and 10)

- Design Variables ; Strength of Concrete

$$S'c = 580 \text{ psi}$$
$$Ec = 3.28 \times 10^6 \text{ psi}$$

- Drainage Factor ; Cd = 0.9
- Load Transfer Coefficient ; J = 4
- Analysis Period ; 25 years

The output of analysis by an electric computer is presented in Appendix 16-2 of volume II.

While, Figure 2.4-1 graphically shows most economical alternative and the most economical performance period of initial pavement structure for PCC Reconstruction, AC Reconstruction and AC Overlay-PCC Existing for the case of Traffic Loading Class E and CBR 6 which is most representable along the Feasibility Study Section.

(2) Recommended Performance Periods

Table 2.4-1 summarizes the performance periods of the initial pavement structures recommended based on the analysis for the five (5) proposed rehabilitation methods, subclassifying traffic loading class into five (5).

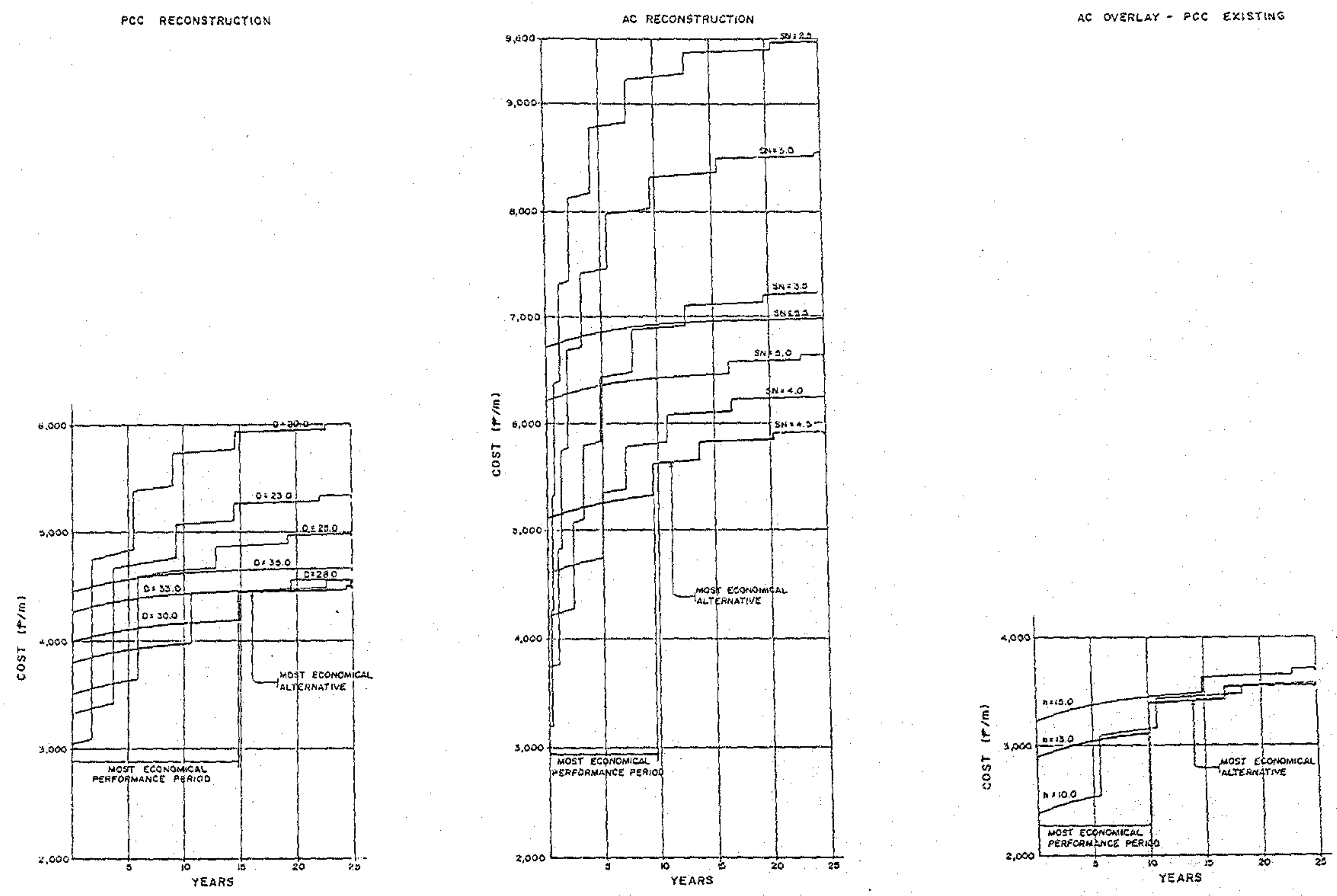


Figure 2.4-1 ALTERNATIVE DESIGN STRATEGIES - PLANNED STAGE, CONSTRUCTION, PERFORMANCE LIFE OF REHABILITATION

NOTE : TRAFFIC LOADING CLASS : E

C B R : 6

TABLE 2.4-1 RECOMMENDED PERFORMANCE PERIOD OF INITIAL PAVEMENT STRUCTURE

Traffic Loading Class	P C C Reconstruction	A C Reconstruction	PCC Overlay -PCC Existing	AC Overlay -PCC Existing	AC Overlay -AC Existing
L-1, L-2, L-3 ESAL's ≅ 0.03×10^6	20 years 1) or Min. Thickness 13 cm	15 years	20 years 1) or Min. Thickness 13 cm	25 years 1) or Min. Thickness 5 cm	10 years 1)3) or Min. Thickness 3 cm
A, B, C ESAL's = $0.031 \sim 0.4 \times 10^6$	15 years	12 years	15 years	12 years 1) or Min. Thickness 10 cm	5 years 2)3) or Max. Thickness 10 cm
D, E ESAL's = $0.41 \sim 1.0 \times 10^6$	15 years	8 years	15 years	8 years 3)	5 years 2) 3) or Max. Thickness 10 cm
F, G ESAL's = $1.1 \sim 2.0 \times 10^6$	15 years	8 years	15 years	8 years 3)	Not Applicable
H, I, J ESAL's = $2.1 \sim 3.5 \times 10^6$	12 years 2) or Max. Thickness 35 cm	5 years 2) or Max. SN 5.5	12 years 2) or Max. Thickness 33 cm	5 years 2) 3) or Max. Thickness 15 cm	Not Applicable

NOTE: 1) Performance period is governed by the minimum structural requirement as the case may be.
2) Performance period is governed by the maximum pavement structure as the case may be.
3) Not applicable where performance period is too short (less than 5 years) even if the maximum pavement structure is applied (see "Basic Design")

2.5 Reliability

In AASHTO Guide 1986, the new concept of pavement design reliability was introduced.

A final summary description of the reliability concept is given by the following definition:

> The reliability of a pavement design-performance process ; is the probability that a pavement section designed using the process will perform satisfactorily over the traffic and environmental conditions for the design period.
>
> Basically, it is a means of incorporating some degree of certainty into the design process to ensure that the design alternatives will last the analyis period. The reliability design factor accounts for chance variations in both traffic prediction (w_{18}) and the performance prediction (W_{18}), and therefore provides a predetermined level of assurance (R) that pavement sections will survive the period for which they were designed.

AASHTO suggests levels of reliability as shown in Table 2.5-1.

TABLE 2.5-1 SUGGESTED LEVELS OF RELIABILITY FOR VARIOUS FUNCTIONAL CLASSIFICATIONS

Functional Classification	Recommended Level of Reliability Urban	Rural
Interstate and other freeways	85 - 99.9	80 - 99.9
Principal Arterials	80 - 99	75 - 95
Collectors	80 - 95	75 - 95
Local	50 - 80	50 - 80

NOTE: Results based on a survey of the AASHTO Pavement Design Task Force

Standard normal deviate values are shown in Table 2.5-2.

TABLE 2.5-2 STANDARD NORMAL DEVIATE (Z_R) VALUES CORRESPONDING TO SELECTED LEVELS OF RELIABILITY

Reliability, R (percent)	Standard Normal Deviate, Z_R
50	-0.000
60	-0.253
70	-0.524
75	-0.674
80	-0.841
85	-1.037
90	-1.282
91	-1.340
92	-1.405
93	-1.476
94	-1.555
95	-1.645
96	-1.751
97	-1.881
98	-2.054
99	-2.327
99.9	-3.090
99.99	-3.750

Overall standard deviation (combined standard error of the traffic projection and performance prediction) are reported as follows.

0.30 - 0.40 Rigid Pavements

0.40 - 0.50 Flexible Pavements

Since the selection of reliability level greatly affects the structural thickness of pavement, it should be decided by the authorities responsible for pavement design. In this Guide, the reliability of 50% was adopted.

CHAPTER 3

PAVEMENT MATERIAL PROPERTIES

CHAPTER 3
PAVEMENT MATERIAL PROPERTIES

This chapter summarizes discussion on specification requirements and design value of pavement material properties, e.g. subgrade, subbase, base and portland cement concrete. The test result on cement, aggregate and concrete by the Technical University of the Philippines are refered the material properties of asphalt were not contained.

3.1 Subgrade

3.1.1 Specification Requirements

Standard Specification 1972 requires that the subgrade shall be brought to a firm, unyielding surface by rolling the entire area with an approved roller weighing not less than 10 tons.

The requirements of General Specification 1976 on compaction and tolerance are shown in Table 3.1-1 and 3.1.2, respectively.

TABLE 3.1-1 COMPACTION REQUIREMENTS

SOIL TYPE Classification according to AASHTO M 145	TEST METHOD for determining moisture - density relations	MINIMUM RELATIVE DENSITY Field dry density as % of maximum dry density as determined by the specified test method
A - 1 A - 2 - 4 A - 2 - 5 A - 3	AASHTO T 180 (4.54 kg rammer) Method D	95%1/, 90%2/
A - 2 - 6 A - 2 - 7 A - 4 A - 5 A - 6 A - 7	ASSHTO T 99 (2.50 kg rammer) Method D	100%1/, 95%2/

NOTE: 1/ Subgrade on embankment
2/ Existing ground

TABLE 3.1-2 TOLERANCES OF SUBGRADE LEVEL, SUB-BASE AND BASE

		Subgrade Level	Sub-Base	Base
Permitted variation from design THICKNESS OF LAYER	mm	-	+20 -20	+10 -10
Permitted variation from design LEVEL OF SURFACE	mm	+20 -30	+10 -20	+ 5 -10
Permitted SURFACE IRREGULARITY measured by 3 m straight-edge	mm	30	20	5
Permitted variation from design CROSSFALL or CAMBER	%	±0.5	±0.3	±0.2
Permitted variation from design LONGITUDINAL GRADE over 25 m length	%	±0.1	±0.1	±0.1

3.1.2 Design Value

Several methods are proposed to estimate strength of subgrade for pavement design.

(1) AASHTO Guide 1986 Method

Resilient Modulus of Roadbed (M_R)

The definitive material property used to characterize roadbed soil for pavement design in the Guide is the resilient modulus (M_R). The procedure for determination of M_R is given in AASHTO Test Method T274.

The resilient modulus is a measure of the elastic property of soil recognizing certain nonlinear characteristics. The resilient modulus can be used directly for the design of flexible pavements but must be converted to a modulus of subgrade reaction (k-value) for the design of rigid or composite pavements.

The following correlations between M_R, CBR and R-value are suggested.

M_R (psi) = 1500 x CBR
(Reasonable for fine-grained soil with a soaked CBR of 10 or less)

M_R = 1000 + 555 (R-value)
(Used for fine-grained soil with R value of 20 or less)

Effective Roadbed Soil Resilient Modulus (Flexible Pavement)

The seasonal resilient modulus values may be determined by correlations with soil properties, i.e, clay content, moisture, PI, etc.

An effective roadbed soil resilient modulus (M_R, Psi) is then established which is equivalent to the combined effect of all the seasonal modulus value. In establishing M_R, the relative damage values corresponding to each seasonal modulus were proposed.

Effective Modulus of Subgrade Reaction (Rigid Pavement)

Since an effective modulus of subgrade reaction (k-value, Pci) is dependent upon several factors besides roadbed soil resilient modulus, the following factors are proposed to be taken into consideration.

- Types and thickness of subbase
- Seasonal roadbed soil resilient modulus (MR)
- Subbase elastic (resilient) modulus (ESB)
- Composite modulus of subgrade reaction for each season (K)
- Adjustment by rigid foundation
- Relative damage by thickness of Concrete Slab (Ur)
- Potential loss of support (LS)

Example analysis on K value are reported in Appendix 3-1, assuming roadbed soil resilient modulus of 6500 psi for dry season and 5000 psi for wet seasons in this country.

(2) PCA Method

Portland Cement Association suggest the following in Thickness Design for Concrete Highway and Street.

Since the plate-loading is time consuming and expensive, the k value is usually estimated by correlation to simpler tests such as the California Bearing Ratio (CBR) or R-value tests. The result is valid because exact determination of the k value is not required; normal variations from an estimated value will not appreciably affect pavement thickness requirements. The relationships shown in Figure 3.1-1 are satisfactory for design purposes.

(3) Corps of Engineers Method

Corps of Engineers suggests unit dry weight, field CBR and subgrade modulus in accordance with the Unified Classification System of Soils originally developed by Casagrande. See Table 3.1-3.

(4) Recommendation

As PCA mentions, CBR may be valid, especially rigid pavement design. However, it is recommended, particularly for flexible pavement that resilient modulus of several type of subgrade should be investigated beforehand so that the result can be commonly used in this country.

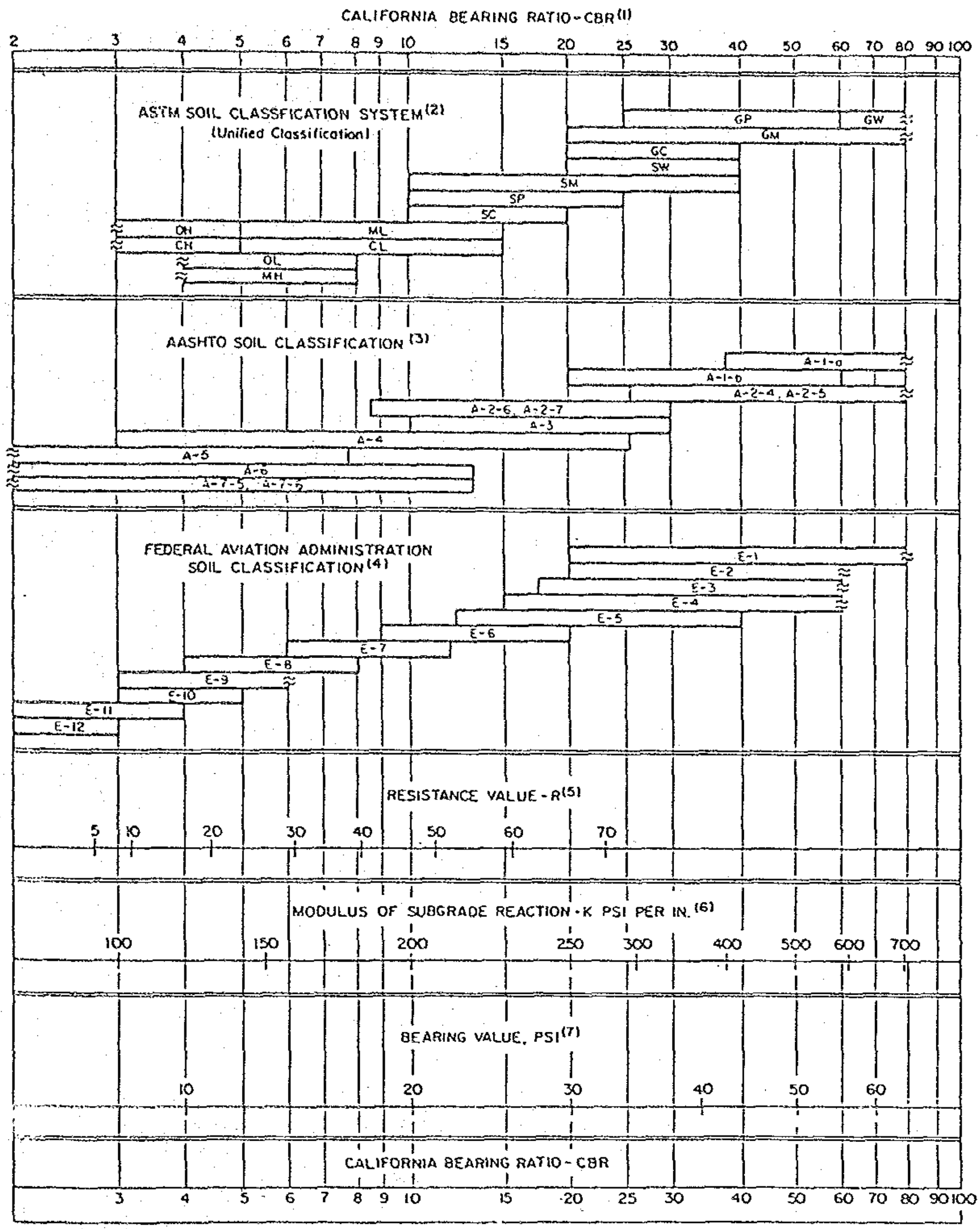


FIGURE 3.1-1 APPROXIMATE INTERRELATIONSHIPS OF SOIL CLASSIFICATIONS AND BEARING VALUES

TABLE 3.1-3 CHARACTERISTICS PERTINENT TO ROAD AND RUNNING FOUNDATIONS

Major Divisions (1)	(2)	Letter (3)	Name (4)	Value as Foundation When Not Subject to Frost Action (5)	Value as Base Directly under Wearing Surface (6)	Potential Frost Action (7)	Compressibility and Expansion (8)	Drainage Characteristics (9)	Compaction Equipment (10)	Unit Dry Weight (pcf) (11)	Field CBR (12)	Subgrade Modulus k (pci) (13)
Coarse-grained soils	Gravel and gravelly soils	GW	Gravel or sandy gravel, well graded	Excellent	Good	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment, steel-wheeled roller	125–140	60–80	300 or more
		GP	Gravel or sandy gravel, poorly graded	Good to excellent	Poor to fair	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment, steel-wheeled roller	120–130	35–60	300 or more
		GU	Gravel or sandy gravel, uniformly graded	Good	Poor	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	115–125	25–50	300 or more
		GM	Silty gravel or silty sandy gravel	Good to excellent	Fair to good	Slight to medium	Very slight	Fair to poor	Rubber-tired equipment, sheepsfoot roller, close control of moisture	130–145	40–80	300 or more
		GC	Clayey gravel or clayey sandy gravel	Good	Poor	Slight to medium	Slight	Poor to practically impervious	Rubber-tired equipment, sheepsfoot roller	120–140	20–40	200–300
	Sand and sandy soils	SW	Sand or gravelly sand, well graded	Good	Poor	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	110–130	20–40	200–300
		SP	Sand or gravelly sand, poorly graded	Fair to good	Poor to not suitable	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	105–120	15–25	200–300
		SU	Sand or gravelly sand, uniformly graded	Fair to good	Not suitable	None to very slight	Almost none	Excellent	Crawler-type tractor, rubber-tired equipment	100–115	10–20	200–300
		SM	Silty sand or silty gravelly sand	Good	Poor	Slight to high	Very slight	Fair to poor	Rubber-tired equipment, sheepsfoot roller, close control of moisture	120–135	20–40	200–300
		SC	Clayey sand or clayey gravelly sand	Fair to good	Not suitable	Slight to high	Slight to medium	Poor to practically impervious	Rubber-tired equipment, sheepsfoot roller	105–130	10–20	200–300
Fine-grained soils	Low compressibility LL < 50	ML	Silts, sandy silts, gravelly silts, or diatomaceous soils	Fair to poor	Not suitable	Medium to very high	Slight to medium	Fair to poor	Rubber-tired equipment, sheepsfoot roller, close control of moisture	100–125	5–15	100–200
		CL	Lean clays, sandy clays, or gravelly clays	Fair to poor	Not suitable	Medium to high	Medium	Practically impervious	Rubber-tired equipment, sheepsfoot roller	100–125	5–15	100–200
		OL	Organic silts or lean organic clays	Poor	Not suitable	Medium to high	Medium to high	Poor	Rubber-tired equipment, sheepsfoot roller	90–105	4–8	100–200
	High compressibility LL > 50	MH	Micaceous clays or diatomaceous soils	Poor	Not suitable	Medium to very high	High	Fair to poor	Rubber-tired equipment, sheepsfoot roller	80–100	4–8	100–200
		CH	Fat clays	Poor to very poor	Not suitable	Medium	High	Practically impervious	Rubber-tired equipment, sheepsfoot roller	90–110	3–5	50–100
		OH	Fat organic clays	Poor to very poor	Not suitable	Medium	High	Practically impervious	Rubber-tired equipment, sheepsfoot roller	80–105	3–5	50–100
Peat and other fibrous organic soils		Pt	Peat, humus, and other	Not suitable	Not suitable	Slight	Very high	Fair to poor	Compaction not practical			

[a] From Corps of Engineers.

JRA recommends a method to determine design CBR as follows.

Determining the Design CBR

1) Determination of CBR value for individual spots. For any given location, where multiple strata of different soil types or conditions exist within 1.0 m depth from the subgrade level, the average CBR value of the soils within this depth should be taken as the CBR value for that location. In calculating the average value, the filter course should be ignored.

For the calculation of the average CBR value, the following formula is applied.

$$CBR_m = \left\{ \frac{h_1\, CBR_2^{1/3} + h_2\, CBR_2^{1/3} + \ldots\ldots + h_n\, CBR_n^{1/3}}{100} \right\}^3$$

where CBR_m: average CBR of the location in question

CBR_1, CBR_2, ..., CBR_n: CBR value of soil strata No. 1, 2, ..., n

h_1, h_2, ..., h_n: Thickness of soil strata No. 1, 2, ..., n in cm. $h_1+h_2+\ldots\ldots+h_n = 100$

2) Determination of the design CBR. The road section to be constructed with a uniform pavement thickness is determined based on the results of preliminary studies and the CBR test. The design CBR is determined based on CBR values of individual locations within the road section, with extreme values discarded, by the following formula.

$$\text{Design CBR} = \text{Average value of CBR of individual locations} - \frac{(\text{Max. CBR} - \text{Min. CBR})}{C}$$

where C is a coefficient given in Table 3.1-4.

TABLE 3.1-4 VALUES OF C FOR CALCULATING DESIGN CBR

No. of Values Available (n)	2	3	4	5	6	7	8	9	10 or More
C	1.41	1.91	2.24	2.48	2.67	2.83	2.96	3.08	3.18

3.2 Subbase

3.2.1 Specification Requirements

Standard Specification 1972 calls for several requirements for aggregate subbase including maximum dimension of particle, liquid limit, plasticity index, compaction,etc.

General Specification 1976 requires more detailed regulations including grading.

Requirements for subbase by these specification are summarized and compared with the requirement by Japan Road Association as presented in Appendix 3-2.

3.2.2. Design Value

(1) AASHTO Guide 1986 Method

As described in 3.1.2, the effect of subbase (thickness and elastic modulus of subbase) is included in estimating effective modulus of subgrade reaction for rigid pavement design.

For flexible pavement design, AASHTO structural layer coefficients are assigned for subbase materials depending on elastic modulus of materials. A value of this coefficient is used to convert actual layer thickness into structural number (SN).

(2) PCA Method

PCA defines subgrade and subbase support in terms of Westergaard modulus of subgrade reaction (K), (a 30 inch diameter plate). The values shown in Table 3.2-1 are recommended by PCA based on the Burmister analysis of two-layer systems and plate-loading tests made to determine k values on subgrades and subbases for full-scale test slabs. Design k values for cement-treated subbases meeting these criteria are given in Table 3.2-2.

TABLE 3.2-1 EFFECT OF UNTREATED SUBBASE ON K-VALUES

Subgrade k value, pci	Subbase k value, pci			
	4 in.	6 in.	9 in.	12 in.
50	65	75	85	110
100	130	140	160	190
200	220	230	270	320
300	320	330	370	430

TABLE 3.2-2 DESIGN K-VALUES FOR CEMENT-TREATED SUBBASES

Subgrade k value, pci	Subbase k value, pci			
	4 in.	6 in	8 in.	10 in.
50	170	230	310	390
100	280	400	520	640
200	470	640	830	-

Based on these values recommended by PCA, Figure 3.2-1 is prepared by the Study Team to graphically show the effect of subbase.

(3) Recommendations

1) Comparative Analysis on effect of subbase

The comparative study was made in order to arrive at the economical thickness of subbase, based on the effect of subbase shown in Figure 3.2-1.

Materials of Subbase assumed

Item 108 Aggregate Subbase

- Main Materials Screened Aggregate
- CBR Value (Subbase) 20
- Layer Coefficient 0.95
- Unit Price ₱208.55/m^3

Effects of K Value estimated

Combined K Values of Subgrade and Subbase

Subgrade CBR	Subbase Thickness 10 cm	20 cm	30 cm
3	100	130	170
6	180	210	270
10	220	250	310
15	250	280	350

Outputs

The output of the comparative study shows the following:

Case 1: Thickness of subbase increased by 10 cm.
Cost; increase by ₱160/m - 2 lanes
k value; increase from 210 to 270 pci
(Subgrade CBR = 6)

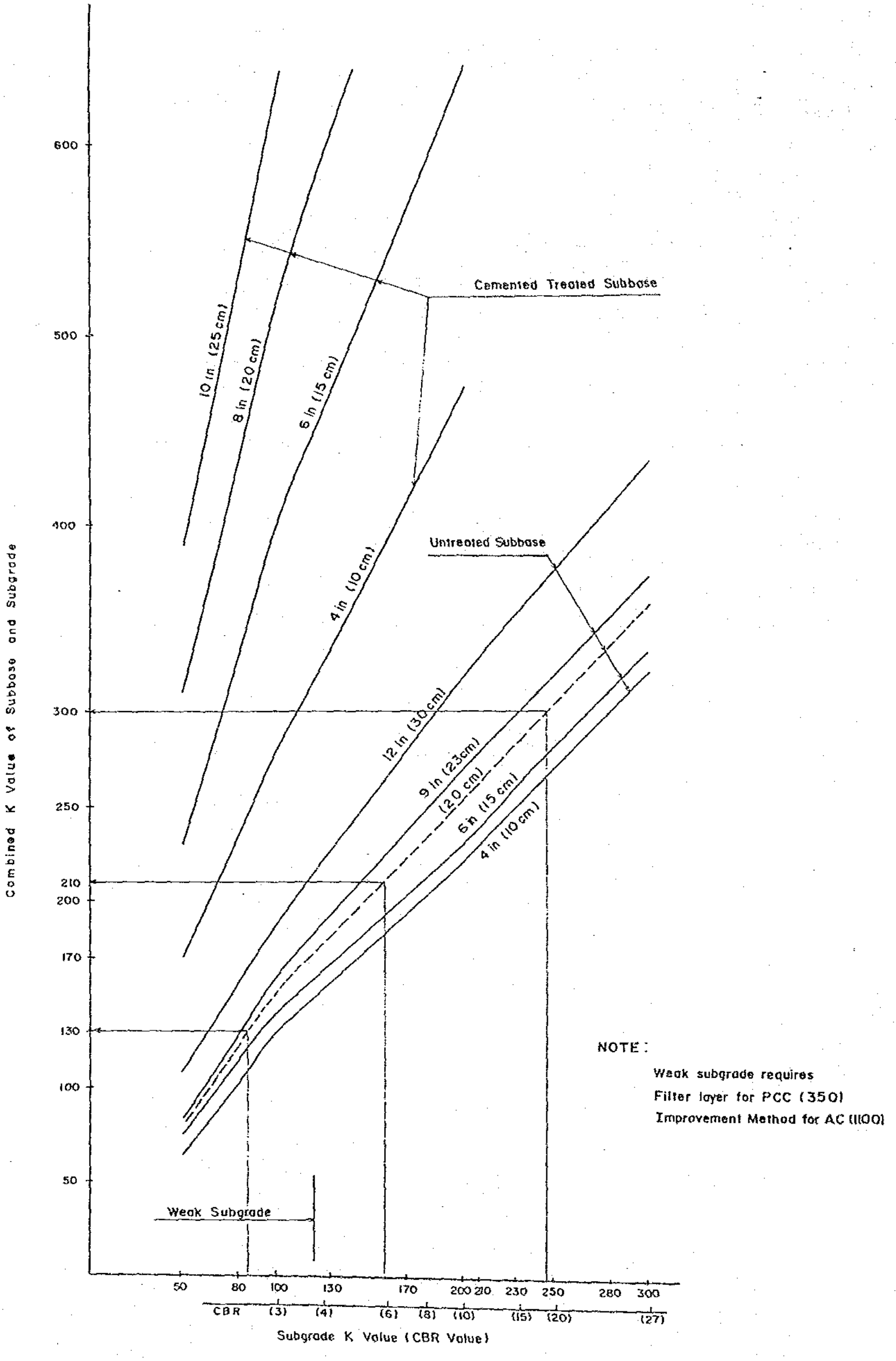


FIGURE 3.2-1 EFFECT OF SUBBASE

Initial Performance Period; extended from
14.4 to 15.4 by 1.0 years (B)
14.9 to 15.7 by 0.8 years (E)
13.9 to 14.5 by 0.6 years (I)

Case 2; The thickness of concrete slab decreasing by 2 cm and the thickness of subgrade increases by 10 cm.

Cost; - 23.93 x 6.7 m = -₱160.3/m - 2 lanes
+ 20.86 x 7.7 m = +₱160.6/m - 2 lanes

k value; decreased from 270 to 210 pci, by 60 pci

Initial Performance Period; shortened from
19.2 to 14.9 years, by 4.3 years (D)
18.5 to 14.8 years, by 3.7 years (G)

2) Recommendations

The combined effects of subgrade and subbase may be estimated based on the recommendation by PCA, for the rigid pavement design.

It is noted from the output of the comparative study that even the subbase thickness increases by 10 cm, the thickness of concrete slab required remain almost the same, especially for CBR value of 3 and 6 and traffic loading classes of G, H, I and J. Therefore, it may not be advisable to decrease the concrete slab thickness by adding thicker subbase. However, it never mean that the quality of subbase can be ignored.

3.3 Base

For rigid pavement, AASHTO, PCA and TRRL do not necessarily recommend provision of base. This Guide follows the same idea.

For flexible pavement, base is the efficient layer among pavement layers. Specification requirements are summarized in Appendix 3-2, for reference.

3.4 Portland Cement Concrete

3.4.1 Specification Requirement

Standard Specification 1972 calls for the several requirements for cement, fine aggregate, course aggregate and water supply. These requirements are summarized in Appendix 3-2, together with requirements of General Specification 1976 and Japan Road Association for reference.

Table 3.4-1 summarizes specification requirements for portland cement concrete pavement.

3.4.2 Experimental Studies on Concrete

The Integrated Research and Training Center in the Technological University of the Philippines is conducting a series of experimental studies on the design and manufacture of concrete in the Philippines with the cooperation of Japan International Cooperation Agency's Experts. The major test results are presented in Appendix 3-3.

3.4.3 Design Value

(1) AASHTO Guide 1986

<u>Modulus of Elasticity</u>

Modulus of elasticity for portland cement concrete, E_c has a correlation with S'c.

$$Ec = 57000\ (f'c)^{0.5}$$

Where:

Ec = PCC elastic modulus (psi)
$f'c$ = PCC compressive strength (psi)

TABLE 3.4-1 SPECIFICATION REQUIREMENTS FOR PORTLAND CEMENT CONCRETE PAVEMENT

	Standard Specification (1972)	General Specification (1972)	Japan Road Association
Maximum Size of Coarse Aggregate (mm)	40, 50, 25	60, 40	40, 30, 25
Consistency (Slump) (cm)	5 to 7.5	-	2.5
Water Cement Ratio (%)	57	-	45
Quantity of Coarse Aggregate Per Unit Volume of Concrete (%)	67 (Gravel - Crushed Stone)	-	76 - 73 (Gravel - Crushed Stone)
Air Content (%)	-	-	4
Unit Center of Water (kg)	-	-	115 - 125
Design Strength (kg/cm^2)	37 (14 day flexural strength)	43 (28 day flexural strength)	45 (28 day flexural strength)
TargetStrength* (kg/cm^2)	-	-	52 (45 x 1.15)

NOTE: *Quality Control Strength

Modulus of rupture (flexural strength)

The value of 0.175 is generally used as a conversion factor changing compressive strength to modulus of rupture, based on the data gathered from the Material Testing Laboratory of DPWH.

(2) PCA Method

Aside from modulus of rupture, PCA considers the increase of strength with age and fatigue relationship as shown in Figure 3.4-1 and 2.

(3) Recommended Value

Referring to test results on cement, aggregate and concrete by Technical University of the Philippines as presented in Appendix 3-3 and the compressive strength tests of 15 pieces of existing concrete pavements in the Feasibility Study, the regulation of BPH Memorandum Circular No. 48 is considered reasonable which calls for as follows.

Richer mixes may be required, if necessary to produce concrete with a minimum 14-day modulus of rupture of 600 psi (42 kg/cm^2) when tested by the mid-point method or 525 psi (37 kg/cm^2) when tested by the third point method, without extra cost to the Government.

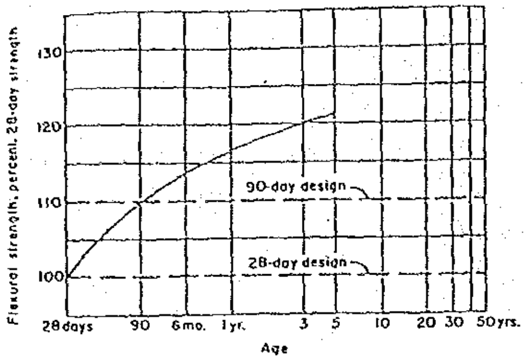


Figure 3.4-1 Flexural strength, age, and design relationships.

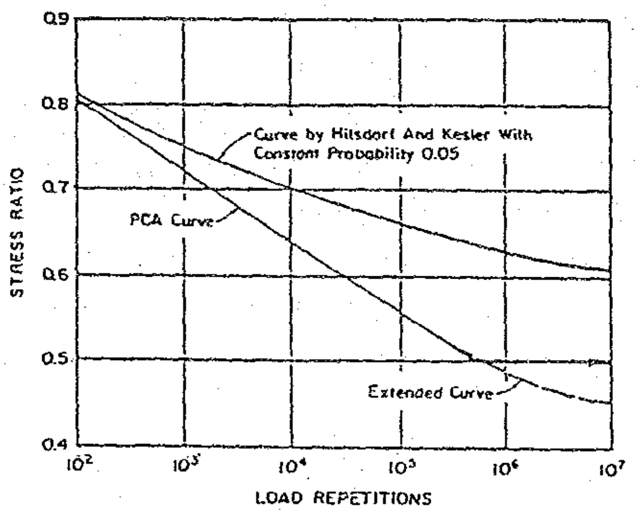


FIGURE 3.4-2 FATIGUE RELATIONSHIPS
(PORTLAND CEMENT ASSOCIATION)

CHAPTER 4

TRAFFIC LOADING

CHAPTER 4
TRAFFIC LOADING

This Chapter deals with traffic loading classes and damaging effects to pavement imposed by traffic. This involves the discussion on the equivalent single axle load (ESAL).

4.1 Equivalent Single Axle Load

In the pavement structural design, appropriate traffic input factors should be determined. Of primary concern are the number and weights of axle loads expected to be applied to the pavement during a given period of time. Since the axle loads from the light cars contribute very little to structural deterioration of the pavement, only heavy trucks and buses are considered for the purpose of designing pavement thickness.

The result of the AASHO Road Test has shown that damaging effect on pavement performance of the passage of an axle load of any mass can be represented by a number of 18-kip equivalent single axle loads or ESAL. This concept has been applied to the design equation of AASHTO Guide. The load equivalency factors derived from the AASHO Road Test are available in the same Guide. Appendix 4-1 shows the load equivalency factors for single, tandem and triple axles under Pt of 2.5. Table 4.1-1 shows the factors for slab thickness 9 inches, Pt of 2.0 for single and tandem axles.

TABLE 4.1-1 AXLE LOAD EQUIVALENCY FACTOR

Slab Thickness = 9 in; Pt = 2.0

Axle Load			Axle Load Equivalency Factor	
Kips	Tons	KN	Single	Tandem
2	0.907	8.889	.0002	.0001
4	1.814	17.778	.002	.0005
6	2.721	26.667	.010	.002
8	3.628	35.556	.032	.005
10	4.535	44.444	.081	.012
12	5.442	53.333	.175	.025
14	6.349	62.222	.338	.047
16	7.256	71.111	.601	.081
18	8.163	80.000	1.00	.132
20	9.070	88.889	1.58	.204
22	9.977	97.778	2.38	.305
24	10.884	106.667	3.47	.441
26	11.791	115.556	4.88	.620
28	12.698	124.444	6.70	.850
30	13.605	133.333	8.98	1.14
32	14.512	142.222	11.8	1.50
34	15.420	151.111	15.3	1.95
36	16.327	160.000	19.5	2.49
38	17.234	168.889	24.6	3.13
40	18.141	177.778	30.7	3.89
42	19.048	186.667	38.0	4.78
44	19.955	195.556	46.6	5.82
46	20.862	204.444	56.7	7.02
48	21.769	213.333	68.4	8.40
50	22.676	222.222	82.0	9.98
52	23.583	231.111		11.8
54	24.490	240.000		13.8
56	25.397	248.889		16.1
58	26.304	257.778		18.7
60	27.211	266.667		21.6
62	28.118	275.556		24.9
64	29.025	284.444		28.5
66	29.932	293.333		32.6
68	30.839	302.222		37.1
70	31.746	311.111		42.1
72	32.653	320.000		47.6
74	33.560	328.889		53.6
76	34.467	337.778		60.3
78	35.374	346.667		67.7
80	36.281	355.556		75.7
82	37.188	364.444		84.4
84	38.095	373.333		94.0
86	39.002	382.222		104.0
88	39.909	391.111		116.0
90	40.816	400.000		128.0

4.2 Traffic Loading Classes

Estimating the initial and future traffic volumes and loading for pavement structural design requires substantial study and analysis for each number of heavy trucks and buses, traffic growth, axle loads and axle configurations.

For the purpose of simplicity and convenience, traffic loadings are classified into (thirteen) 13 classes using the number of ESAL application at the initial year, as shown in Table 4.2-1.

TABLE 4.2-1 STANDARD TRAFFIC LOADING CLASSES

(At Initial Year)

	Traffic Loading Class	Number of ESAL At Initial Year
Light Loading Traffic	L - 1	0.005×10^6
	L - 2	0.01
	L - 3	0.03
Heavy Loading Traffic	A	$0.03 - 0.1 \times 10^6$
	B	0.11 - 0.2
	C	0.21 - 0.4
	D	0.41 - 0.7
	E	0.71 - 1.0
Extra Heavy Loading Traffic	F	$1.1 - 1.5 \times 10^6$
	G	1.6 - 2.0
	H	2.1 - 2.5
	I	2.6 - 3.0
	J	3.1 - 3.5

4.3 Traffic for Basic Design and Application

The basic structural designs are analyzed by an electric computer for the basic cases of traffic loading and traffic growth rate.

The traffic loading used for the basic design are the standard traffic loading classes shown in Table 4.2-1 and the traffic growth rate of 4.5 percent which is the average value along the Study Section.

For the sections with the traffic growth rate, other than 4.5 percent, the output of basic design can be applied by adjusting the number of ESAL at the initial year. The adjusted number of ESAL at the initial year, as shown in Table 4.3-1 can be estimated assuming that the cummulative number of ESAL for 10 years are the same with the basic design cases.

TABLE 4.3-1 BASIC DESIGN AND APPLICATION

Traffic Loading Classes	Basic Design: Traffic Growth Rate	Basic Design: Number of ESAL at Initial Year	Application: Traffic Growth Rate	Application: Number of ESAL at Initial Year	Traffic Growth Rate	Number of ESAL at Initial Year	Traffic Growth Rate	Number of ESAL at Initial Year	Traffic Growth Rate	Number of ESAL at Initial Year
A	4.5%	3.5×10^6	3.5%	3.67×10^6	4.0%	3.58×10^6	5.0%	3.42×10^6	5.5%	3.34×10^6
B	4.5%	3.0×10^6	3.5%	3.14×10^6	4.0%	3.07×10^6	5.0%	2.93×10^6	5.5%	2.86×10^6
C	4.5%	2.5×10^6	3.5%	2.62×10^6	4.0%	2.56×10^6	5.0%	2.44×10^6	5.5%	2.39×10^6
D	4.5%	2.0×10^6	3.5%	2.09×10^6	4.0%	2.05×10^6	5.0%	1.95×10^6	5.5%	1.91×10^6
E	4.5%	1.5×10^6	3.5%	1.57×10^6	4.0%	1.54×10^6	5.0%	1.47×10^6	5.5%	1.43×10^6
F	4.5%	1.0×10^6	3.5%	1.05×10^6	4.0%	1.02×10^6	5.0%	0.98×10^6	5.5%	0.95×10^6
G	4.5%	0.7×10^6	3.5%	0.73×10^6	4.0%	0.72×10^6	5.0%	0.68×10^6	5.5%	0.67×10^6
H	4.5%	0.4×10^6	3.5%	0.42×10^6	4.0%	0.41×10^6	5.0%	0.39×10^6	5.5%	0.38×10^6
I	4.5%	0.2×10^6	3.5%	0.21×10^6	4.0%	0.20×10^6	5.0%	0.20×10^6	5.5%	0.20×10^6
J	4.5%	0.1×10^6	3.5%	0.10×10^6	4.0%	0.10×10^6	5.0%	0.10×10^6	5.5%	0.10×10^6

NOTE:

$$\text{ESAL}(i\%) = \text{ESAL}(4.5\%) \times \frac{1.045^{10} - 1}{0.045} \times \frac{i}{(1+i)^{10} - 1}$$

4.4 Simplified Method for Estimation of ESAL

Generally, informations on traffic volumes by types of vehicles are obtained from the agencies concerned. Aside from these informations, data on axle load distribution pattern by type of axle and by types of vehicles are required for the estimation of the number of ESAL's application. However, data on these traffic loading are hardly obtained. In such cases, the following simplified method may be applied especially for the preliminary pavement design, but with some special cares.

Damaging factor (D F.) is defined:

$$DF = \frac{\text{No. of ESAL Application}}{\text{Actual Traffic Volume}}$$

When the damaging factors at the road section being designed are assumed by types of vehicles, the number of ESAL's application can be easily calculated using the formula mentioned above.

The damaging factor assumed in the Feasibility Study are shown below for reference.

Damaging Factor (Pan-Philippine Highway)

North Section ----- Manila Bound; 5 to 9
Cagayan Bound; 2 to 3
South Section ----- Manila Bound; 3 to 4
Bicol Bound; 2 to 3
Secondary Road ---- 1to2(assumed only)

Refer to Chapter 5.2.

CHAPTER 5
PAVEMENT DETERIORATION

CHAPTER 5
PAVEMENT DETERIORATION

This chapter discusses the probable causes of PCC pavement deterioration/ distress. Analyzed are the effects of reliability, initial serviceability index, strengths of concrete, subgrade and subbase for rigid pavement. The informations in this Chapter are desired to be positively utilized in PCC pavement design.

5.1 Categorization of Pavement Distress

The evaluation of pavement condition includes consideration of specific problems that existsin the pavement. This requires a determination of types and causes of pavement distresses,aswell as the extent of pavement deterioration.

Pavement distresses can be categorized as being caused either by traffic loads or non-load factors, including design, construction, poor-durability materials, and climate factors.

Federal Highway Administration presents the information on classification of pavement distress with three distress severity levels in FHWA/RD-81/080 Study entitled "A Pavement Moisture Accelerated Distress Identification System", which is contained in AASHTO Guide, 1986.

In the Feasibility Study, this classification of distress mentioned above was adopted as a principle guide. Pavement distress items classified by FHWA Study and the detailed description with severity levels and method of measurement are presented in Appendix 5-1. The general categorization of jointed concrete pavement distress is presented in Table 5.1-1.

References were also obtained from Organization for Economic Co-Operation and Development (OECD), Road Research, "Catalogue of Road Surface Deficiencies, 1978."

TABLE 5.1-1 GENERAL CATEGORIZATION OF JOINTED CONCRETE PAVEMENT DISTRESS

	Distress Type	Primarily Caused By Traffic Load	Primarily Caused By Climate/Materials
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 of 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 Det.	X	X
14.	Popouts		X
15.	Pumping and Water Bleeding	X (M, H)	X (L)
16.	Reactive Agg. Dur. Dist.		X
17.	Scaling, Map Cracking and Crazing		X
18.	Spalling (Trans. and Long. Jts.)	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)

NOTE:

H; High Severity

M; Medium Severity

L; Low Severity

5.2 Damaging Effects of Axle Loads

A major object of the AASHO Test was to establish, using normal road construction and normal road vehicles, the relative damaging effects of a wide range of axle loads. The number of passages of each axle load required to cause the same damage as one passage of a 'standard' axle of 8160 kg (18000 lb) was expressed as an equivalence factor. The equivalence factors are shown in Table 4.2-1 and Appendix 4-1.

For example, the equivalence factors are 0.0002 for 910 kg (2000 lb), 1 for 8160 kg (18.00 lb) and 22.8 for 18,140 kg (40,000 lb). The equivalence factor of 0.0002 for the 910 kg (2000 lb) axle means that 5000 passes of such an axle do the same amount of damage as one pass of the 'standard' axle. Similarly the factor of 22.8 for the 18 140 kg (40 000 lb) axle means that 23 passes of a 'standard' axle are equivalent to one pass of the very heavy axle. The Equivalence Factors were found not to vary very much either with the thickness or the type of the pavement (flexible or concrete). The absolute damage of course varied with these factors, but not the relative damage. The damaging effect of axles thus increases steeply with the axle load, the relationship being represented by approximately a fourth-power law.

TRRL terms the damaging factor expressed in terms of standard axle per 100 commercial axles. Sufficient data are not yet available to relate damaging factors closely to different types of road. The values recommended in Road Note 29 are:

Motorways and Trunk Roads carrying over 1000 commercial vehicles per day in each direction	40 standard axles per 100 commercial axles
Roads carrying between 250 and 1000 commercial vehicles per day in each direction	30 standard axles per 100 commercial axles
All other roads	20 standard axles per 100 commercial axles

(The traffic intensities refer to the time of construction).

The use of these factors in pavement design for the traffic in the two directions may lead to some difference between the lives of the left-hand traffic lanes in the two directions.

In this Guide, the damaging factor is termed as standard axle per vehicles. See Chapter 4.

5.3 Pavement Variables Affecting Pavement Strength

(1) Average and Variation Ranges of Pavement Variables

There are several pavement variables affecting the performance and the service life of the pavement, some are severely and others not so much. In evaluating the probable causes of the pavement deterioration, it is quite meaningful to estimate the effects of these variables against pavement strength.

In estimating the degree/severity of these effects, AASHTO Design Equation was used as a model because the equation involves most number of pavement variables than others do.

The average values of variables were set at the mathematical averages in this country. The variation ranges of pavement variables were also given taking into consideration the maximum and minimum values in this country. These values were obtained from the Feasibility Study. See Table 5.3-1.

(2) Analysis on Effects of Pavement Variables

The analysis to evaluate the effects of pavement variables was done by a computer with a program for AASHO Design Equation. When calculating the effect of the one pavement variable, other variables were kept in the average values.

Figure 5.2-1 illustrates the degree of the effects. The effects are converted to the slab thickness to obtain the equivalent slab thickness due to change in the values of pavement variables.

Within the practical ranges of pavement variables in this country, the order of effects are summarized as shown in Table 5.3-2.

TABLE 5.3-1 Average and Variation Range of Pavement Variables

			Average	Range of Variation			Remarks
				Poor		Good	
Reliability	ZR	Standard Normal Deviation	0	90 % (-1.282)	~	95 % (-1.645)	
	So	Standard Error of Prediction	0	0.3			Constant
Initial Seviceability Index	Po,(R)	Initial Psi Measured by Roughness	4.5(150)	4.1 (250)	~	4.5(150)	( ) Roughness
Concrete	D	Thickness of Slab (inch)	9	8	~	1.2	
	S'c	Modulus of Rupture (Psi)	500	450 (2600)	~	650(3700)	( ) Compressive Strength
	Ec	Modulus of Elasticity (Psi)	3.05×10^6	2.87×10^6	~	3.47x10	
Subgrade on Subbase	K	Modulus of Subgrade (Pci) Reaction	250	80(3)	~	550(56)	( ) CBR
Transverse Joint	J	Load Transfer Coefficient	4.0	4.4	~	3.6	
Drainage System	Cd	Drainage Coefficient	0.9	0.7	~	1.1	

Note: 1.) Average values is the mathematics average obtained from slabs surveyed.
2.) When analyzing the affecting ratio of one variable, other variables were kept constant at average values.

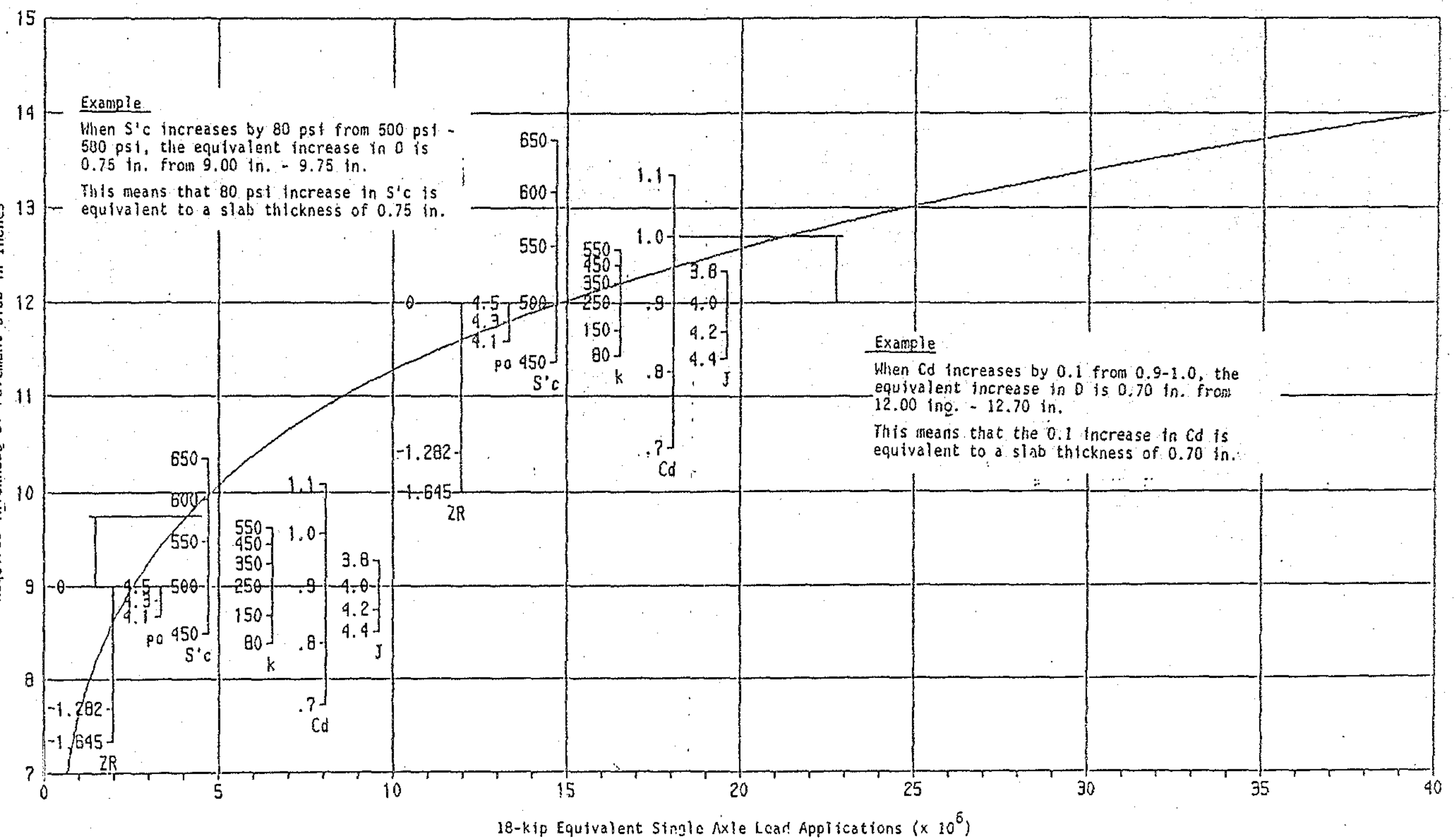


FIGURE 5.3-1 EQUIVALENT SLAB THICKNESS OF PAVEMENT VARIABLES

TABLE 5.3-2 EFFECTS OF PAVEMENT VARIABLES

	Range	In Case of D = 9 in.	In Case of D = 12 in.
Drainage Coefficient	0.7 - 1.1	2.36 in.	2.90 in.
Concrete	450 - 650 psi	1.86 in.	2.34 in.
Reliability	0 - 95%	1.67 in.	2.00 in.
Subgrade	80 - 550	1.24 in.	1.12 in.
Load Transfer	4.4 - 3.8	0.76 in.	0.94 in.
Initial RRI	4.1 - 4.5	0.32 in.	0.41 in.

Among pavement variables, drainage coefficients show the greatest effect that thickness of slab may differ by 2.36 in. (6.0 cm) if drainage factor changes from 0.7 (very poor) to 1-1 (excellent).

The concrete strength and reliability have almost the same effects, about 1.8 in. (4.6 cm) in the range of 450 psi (32 78/cm^2) and 605 psi (46 kg/cm^2).

The strength of subgrade gives rather small effect, about 1.3 in. (3.3 cm) in the range of K = 80 (CBR = 3) and K = 550 (CBR = 56).

5.4 Probable Causes of Pavement Deterioration

The analysis and evaluation on probable causes of pavement deteriorations undertaken under the Feasibility Study are summarized below. Refer to Volume 2 of the Feasibility Study for the detailed discussion.

Figure 5.4-1 shows a relation between occurence of cracking and thickness of concrete slab. From the figure, it is understood that the lack of concrete slab thickness may be one of the causes.

Figure 5.4-2 indicates a relation between occurance of cracking vs. fatigue ratio. Fatigue ratio are calculated by Portland Cement Association Method. Refer to Appendix 8-1. Fatigue ratio shows a little correlation with the occurance of cracking, therefore, the causes may be due to fatigue rather than erosion.

From Figure 5.4-2 and 5.4-3, the similar observation may be made.

The assessment of the probable causes of pavement deterioration are summarized as follows:

(1) Traffic

The primary causes might be over-weighed traffic. The basis of this inference are:

- Fatigue ratio is considerably high comparing with erosion ratio (PCA and Westergaard)
- Transverse cracks are predominant.
- Relative damaging effects of traffic are high.

  (8.1 for Manila bound along the North Study Section)
  (3.6 for Manila bound along the South Study Section)

(2) Concrete Slab

The quality of concrete vary in wide ranges. The modulus of rupture vary from 430 psi to 630 psi. The required value is 525 psi (BPH Memo Circular No. 48) at 14 days.

The thickness of concrete slab is thin (8.60 - 11.25 inches) in general and varies in wide ranges. It did not tally with the number of ESAL estimated.

(3) Subbase and Subgrade

In some sections, the CBR values of subbase are lower (the lowest is 3) than those of subgrades. This might be due to saturation/erosion of subbases. Although the resilient modulus of subbase and subgrade has relatively small effects to the structural strength of pavement in case of concrete pavement, the localized loss of support and void beneath concrete slab may significantly involve the pavement performance.

(4) Drainage System

The drainage system involves the most remarkable effects to the pavement performance among those pavement variables. Moreover, the drainage factors is changeable in accordance with the condition of sections/spots.

In the Study Sections, the highest drainage factor is estimated as 1.1 (good, San Jose-Aritao) and the lowest 0.8 (poor, many sections).

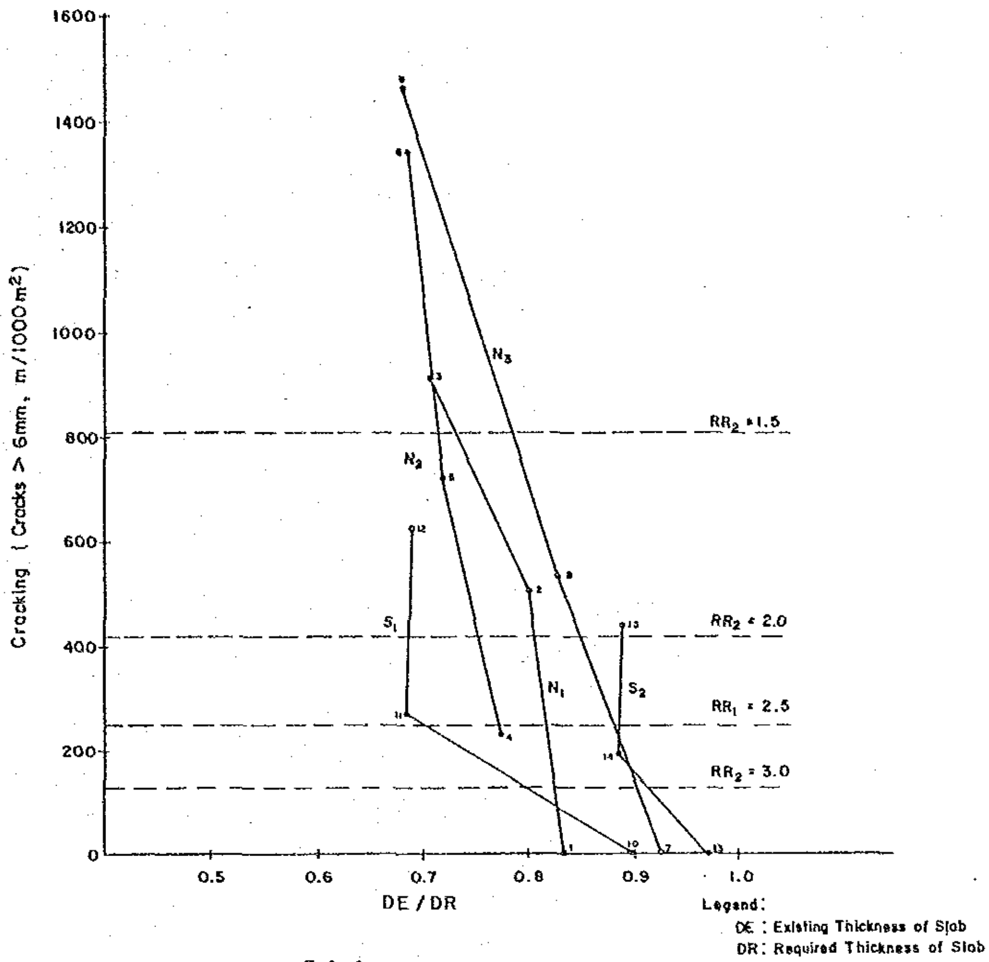


Figure 5.4-1 Cracking and Thickness of Slab

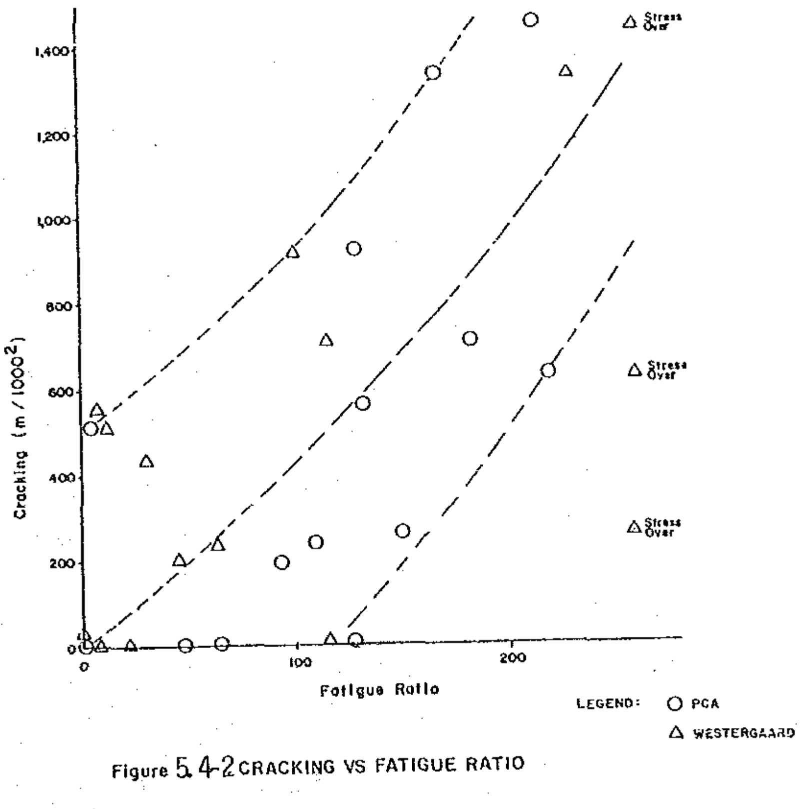


Figure 5.4-2 CRACKING VS FATIGUE RATIO

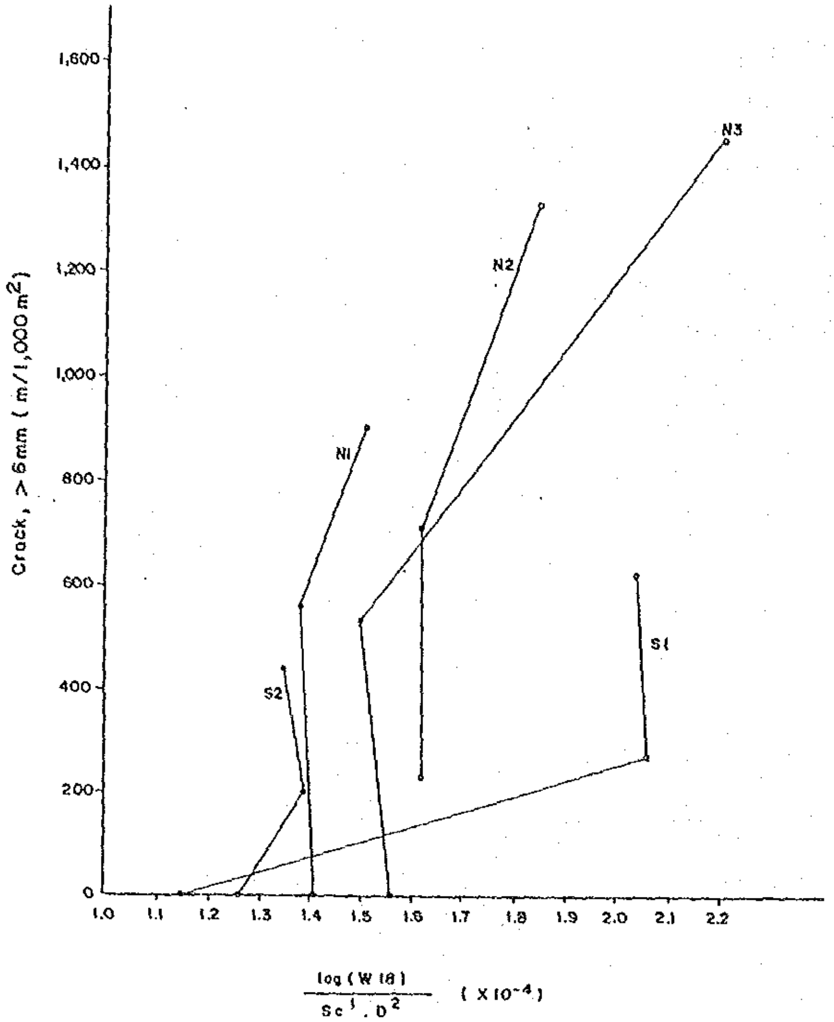


Figure 5.4-3 CRACKING VS STRESS

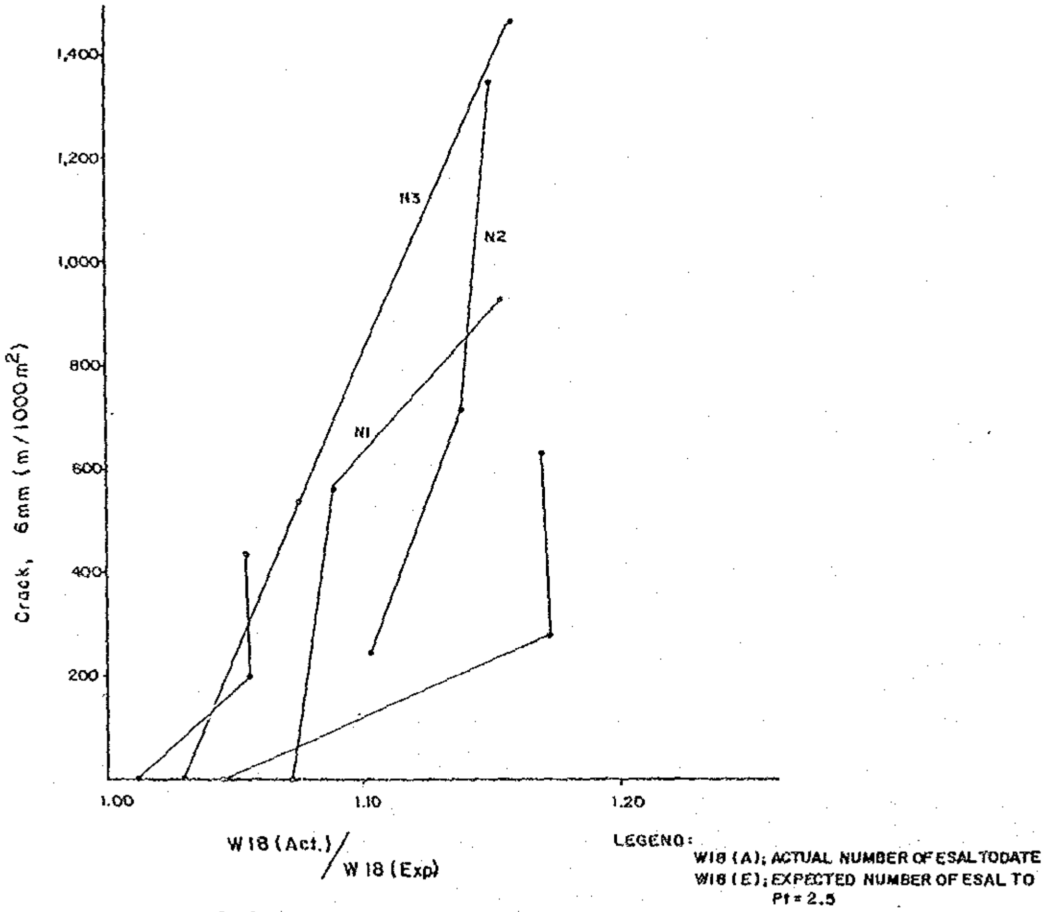


Figure 5.4-4 CRACKING VS ESAL APPLICATIONS

CHAPTER 6

IDENTIFICATION OF PAVEMENT REHABILITATION SECTION

CHAPTER 6
IDENTIFICATION OF PAVEMENT REHABILITATION SECTION

The pavement condition survey should be conducted to evaluate the present surface condition of pavements and identify the road sections where the pavement rehabilitation will be required. The factors mainly involved for evaluation are riding comfort and physical distress of pavements.

In evaluating pavement performance to identify the road sections for rehabilitation, the evaluation index adopted in this Guide are:

- Present Serviceability Index (PSI)
- Rehabilitation Requirement Index (RRI)

The evaluation methods of both indeces are briefly discussed hereunder. Refer to Chapter 2.

6.1 Pavement Surface Condition Survey

There are various types of pavement deficiencies as shown below, among others.

- Cracks
- Patching
- Roughness
- Faulting
- Sealant Failure
- Pumping

However, type of deficiencies used in both methods are only cracks, patching and roughness, the survey methods of which are briefly mentioned.

6.1.1 Crack Survey

Class of Cracks

Cracks are classified in accordance with AASHTO Road Test.

Class 1; Fine cracks not visible under day surface condition for a man with good vision standing at a distance of 15 ft.

Class 2; Cracks that can be seen at a distance of 15 ft. but exhibited only minor spalling such that the opening at the surface is less than 1/4 inch.

Class 3; Any crack spalled at the surface to a width of 1/4 inch or more for at least one-half its length.

Class 4; Any crack which has been sealed.

In the Feasibility Study, cracks under Class 1 and 2 are called as fine crack and Class 3 and 4 as wide crack. Cracks are also categorized into four (4) types by their sharpness and location, e.g. 1) longitudinal crack, 2) transverse crack, 3) corner crack and 4) Block/Randum crack.

Cracking

Cracking is termed as the amount of cracks in the pavement surface and express in linear meter of cracks under class 3 and 4 per 1000 sq. meter of surface area.

Cracking Index

Cracking index is the total length of all cracks under Class 1, 2, 3 and 4 per 1000 sq. meter of surface area.

Survey Method

The length of cracks may be roughly estimated by each slab taking into account its proportion to a longitudinal or transversal length of a slab. Although no tape may be used for the measurement, it can be estimated accurately enough for practical purposes. At the first stage of the survey, the surveyors should be trained with the illustration of cracking levels.

Whenever the accuracy in surveying cracks is required, the actual length of cracks should be measured by a tape.

6.1.2 Patching Survey

Patching

The area of pavement surface patched with asphalt concrete is expressed in square meter of patching per 100 sq. meter of surface area.

Survey Method

The area of patching may be estimated by the judgement of surveyors with the knowledge of civil engineering. But the actual measurement is recommended, if accuracy is in question.

6.1.3 Roughness Survey

Roughness Index

The roughness of the road surface can be surveyed by the Bump Integrator equipped to a Toyota Land Cruiser. The reading of the counter of the integrator gives the number of inches of unidirectional movement of the rear axle of the test vehicle and converted to Roughness Value (RV).

$$RV\ (cm/km) = \frac{\text{Actual Reading} \times 25.4}{\text{Length of the Section (Km)}}$$

Survey Method

The bump integrator is a device which produces an electric impulse for a particular amount of movement of an axle relative to the frame of the test vehicles. The pulses are counted and expressed as a total amount of movement per length of road. The survey engineer with a long experience is desired for the roughness survey. The test vehicle should be driven at 30 kph in the year most suited to measurement. The intensity of the integrator should be recorded at every 200 m with the reading of the odometer.

The data obtained from pavement surface condition survey should be well compiled in the format, an example of which is shown in Figure 6.1-1 and Figure 6.1-2.

Figure 6.1-1 OCULAR SURVEY OF PAVEMENT DEFICIENCIES AND GENERAL INFORMATIONS

Bound : ______________ Date : ______________

Section : ______________ Surveyor : ______________

KM.	Town / City	General Information					Crack (%)										Patching	Faulting	Sealant Failure	Pumping
		T	L	S	D	CY	Type	Crack Length per 100 m of a lane (m)												
								0	50	100	150	200	250	300	AVE.		m	Each		NO.
+000		1 2 3	1 2 3 4 5	1 2 3	1 2 3		1 2 3 4												1 2	
+100		1 2 3	1 2 3 4 5	1 2 3	1 2 3		1 2 3 4												1 2	
+200		1 2 3	1 2 3 4 5	1 2 3	1 2 3		1 2 3 4												1 2	
+300		1 2 3	1 2 3 4 5	1 2 3	1 2 3		1 2 3 4												1 2	
+400		1 2 3	1 2 3 4 5	1 2 3	1 2 3		1 2 3 4												1 2	
+500		1 2 3	1 2 3 4 5	1 2 3	1 2 3		1 2 3 4												1 2	
+600		1 2 3	1 2 3 4 5	1 2 3	1 2 3		1 2 3 4												1 2	
+700		1 2 3	1 2 3 4 5	1 2 3	1 2 3		1 2 3 4												1 2	
+800		1 2 3	1 2 3 4 5	1 2 3	1 2 3		1 2 3 4												1 2	
+900		1 2 3	1 2 3 4 5	1 2 3	1 2 3		1 2 3 4												1 2	

General Information

Topography (T)

1. Flat (F)
2. Rolling (R)
3. Mountainous (M)

Location (L)

1. Rice Field (R)
2. Plowed Field (P)
3. Coconut Field (C)
4. Forest (F)
5. Wasteland (W)

Section Type (S)

1. Cut (C)
2. Embankment (E)
3. Cut / Embankment (C/E)

Types of Cracking

1. Longitudinal Cracking (L)
2. Transverse Cracking (T)
3. Corner Cracking (C)
4. Block / Random Cracking (A)

Drainage Condition (D)

1. Good (G)
2. Fair (F)
3. Bad (B)

Construction Year (CY)

Sealant Failure

1. No Failure
2. Failure

Figure 6.1-2 ROUGHNESS SURVEY

ROAD NO. ________ NAME ______________________________ RUN NO. ________

FROM ____________ TO ____________ DRIVER ____________ RECORDER ____________

SPEEDOMETER (AT START) ________ AT END ________ FACTOR ________ ACT. DIST. ________ DATE ________

TACHOMETER (AT START) ________ AT END ________ FACTOR ________ ACT. DIST. ________ ST. TIME ________

ODOMETER (AT START) ________ AT END ________ FACTOR ________ ACT. DIST. ________ FIN. TIME ________

ODOMETER	INTEGRATOR		PAVEMENT		STRUC. JCT.	REMARKS
	INTENSITY	DIFF.	TYPE	COND.		

6.2 Evaluation of Surface Condition

6.2.1 Evaluation of PSI (AASHTO)

The section deals with the evaluation method by the present serviceability index established by AASHTO.

The serviceability of a pavement is expressed in terms of the present serviceability index (PSI). The PSI is obtained from measurements of roughness and distress, e.g., cracking, patching and rut depth (flexible), at a particular time during the service life of the pavement. Roughness is the dominant factor in estimating the PSI of pavement.

The equations to determine the level of serviceability of the surviving pavement are expressed as follows:

(1) Flexible Pavement

$$p = 5.03 = 1.91 \log (1+ \overline{SV}) - 0.01 \sqrt{C+P} - 1.38\overline{RD}^{\,2}$$

in which

p = the present serviceability index;
$\overline{SV}$ = the mean of the slope variance in the two wheelpaths;
C = cracking; the area, in square feet per 1000 sq. ft. of pavement surface exhibiting class 2 or class 3 cracking.
P = patching; the area of skin/deep patching in square feet per 1000 sq. ft. of pavement surface.
$\overline{RD}$ = a measure of rutting in the wheel paths.

(2) Rigid Pavement

$$p = 5.41 - 1.80 \log (1+\overline{SV}) - 0.09 \sqrt{C+P}$$

or

$$p = 5.41 - 18.0 \log (0.40R - 33) - 0.09 \sqrt{C+P}$$

in which

p = present serviceability index;

$\overline{SV}$ = mean of the slope variance in the two wheelpaths; and

R = the roughometer reading in inches per mile.

C = cracking, the total linear feet of class 3 and class 4 cracks per 1000 sq. ft. of pavement area.

P = patching; the area of skin/deep patching in square feet per 1000 sq. ft. of pavement surface.

In this equation, roughness was measured by the Bureau of Public Roads Roughometer, while roughness in this Feasibility Study by the Bump Integrator. No relation between two measurements by both equipments are authorized. TRRL proposed very loose relation for the comparison between surface irregularity and riding quality.

Based on this very loose relation between two measurement of roughness, AASHO PSI formula was converted, as follows. Refer to Appendix 13-3 of Volume III.

$$PSI = 5.41 - 1.80 \log (0.20R-30) - 0.05 \sqrt{C+3.3p}$$

(for rigid pavement)

R,C,P: as defined for the RRI/PSI formula in this study

It is again noted that the above formula includes the estimation in the conversion of roughness value and the accuracy of the formula is therefore not warranted.

6.2.2 Evaluation by PSI/RRI in the Philippines

(1) PSI in the Philippines

The present serviceability is a subjective assessment by the road users using their own guideline and judgement. It is understood that the basis of judgement may be swayed by the tolerableness of road users, national characters as well as economic conditions of the country since comfort or riding quality is a matter of subjective response or the opinion of the users. Each country, therefore, may have their own rating.

In order to ascertain the basis for the Philippines, the assessment of ridability was conducted under the Feasibility Study.

Following the methods employed in AASHO Test, the present serviceability rating on surface condition were conducted under the Feasibility Study.

Each member of the rating composed of road users, drivers and engineers were asked to rate the serviceability/comfort using their own judgement.

Based on these ratings the formula to evaluate the Present Serviceability Index (PSI) was established as follows:

$$PSI = 7.75 - 2.0 \log (R) - 0.06 \sqrt{C+P} \qquad (r = 0.745)$$
$$PSI = 8.05 - 2.2 \log (R) - 0.06 \sqrt{C} \qquad (r = 0.728)$$
$$PSI = 9.33 - 2.9 \log (R) \qquad (r = 0.566)$$
$$PSI = 2.93 - 0.07 \sqrt{C+P} \qquad (r = 0.642)$$
$$PSI = 2.88 - 0.08 \sqrt{C} \qquad (r = 0.601)$$

Where:

R: Roughness in cm per km

C: Cracking (Total of class 3 and Class 4 cracks in m per 1,000 m^2)

P: Patching in m^2 per 1,000 m^2

r: Coefficient of correlation between PSI and corresponding PSR.

The formula which includes R, C and P is recommended.

(2) RRI in the Philippines

As mentioned, the present serviceability index is a subjective assessment by the road users using their own guideline and judgement, theindex does not necessarily identify the sections where the rehabilitation works are needed, when judged from the engineering point of view.

As described in chapter 2, the concept introduced by AASHO Road Test 1952, is based on the principle that the road user is not directly interested in the amount of cracking or deformation present in the pavements over which he drives. He is primarily interested in the ability of the road to provide a comfortable and safe ride. Riding quality is not however, necessarily related to the structural condition of the pavement.

In order to simply assess the structural capacity, highway/maintenance/construction engineers were requested to evaluate the surface condition based mainly on cracking and patching, while the road users assessed in case of PSI.

The index derived from this rating method was called as Rehabilitation Requirement Index (RRI), the formula of which was established by the Feasibility Study, as follows:

$$RRI = 7.53 - 1.5 \log (R) - 0.11 \sqrt{C+P} \qquad (r = 0.756)$$
$$RRI = 7.97 - 1.7 \log (R) - 0.11 \sqrt{C} \qquad (r = 0.740)$$
$$RRI = 10.89 - 3.2 \log (R) \qquad (r = 0.448)$$
$$RRI = 3.93 - 0.12 \sqrt{C+P} \qquad (r = 0.731)$$
$$RRI = 3.87 - 0.13 \sqrt{C} \qquad (r = 0.706)$$

(for rigid pavement)

Where, R, C, P and r as defined above.

The formula involving R, C and P is recommended.

In case where roughness measuring equipment is not available, the RRI formula can be replaced with

$$RRI = 3.93 - 0.12 \sqrt{C+P}$$

6.2.3 Average Cracking and Roughness in the Philippines

Average cracking and roughness in AASHO's Road Test were estimated as discussed in Appendix 13-3 of Volume II. Based on the result of pavement surface condition survey, the average cracking and roughness in the Study Section were also estimated for each level of RRI and PSI.

Table 6.2-1 and Figure 6.2-1 summarize the comparison of RRI/PSI and AASHO's PSI.

These comparison shows:

1) In comparison with RRI and AASHO's PSI, the former indicates longer length of cracking and higher roughness where the index are lower than 3.0.

2) In comparison with PSI and AASHO's PSI, the former indicates shorter length of cracking and higher roughness where index are lower than 3.0.

3) In comparison with RRI and PSI, the former always indicates longer length of cracking and higher roughness.

TABLE 6.2-1 AVERAGE CRACKING AND ROUGHNESS CONDITION

AASHTO Road Test			The Study in the Philippines					
PSI	Cracking ($m/1000\ m^2$)	Roughness (Cm/Km)	RRI	Cracking	Roughness	PSI	Cracking	Roughness
4.0	12	175	4.0	5	160	4.0	0	80
3.5	28	190	3.5	12	270	3.5	0	130
3.0	50	220	3.0	60	290	3.0	4	220
2.5	110	260	2.0	140	320	2.5	35	280
2.0	220	300	2.0	250	340	2.0	150	320
1.5	390	360	1.5	400	370	1.5	360	360

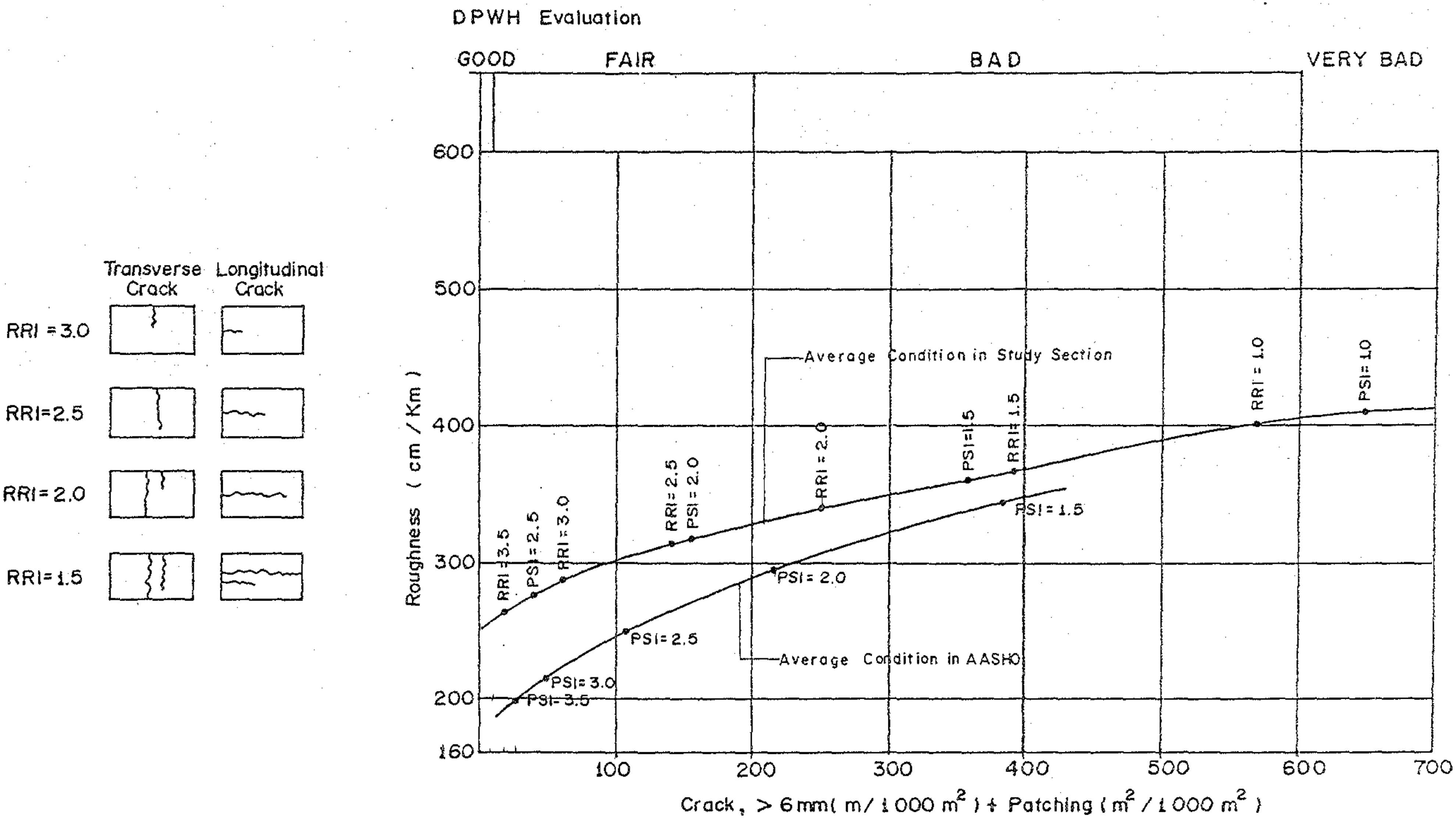


FIG. 6.2-1 AVERAGE RELATIONSHIP BETWEEN RRI AND ROUGHNESS, CRACKING, PATCHING

6.3 Improvement Level for Pavement Rehabilitation

6.3.1 Serviceability Requirement for Pavement Rehabilitation

Since highways are defined as facilities for the comfort and convenience of the travelling public, the serviceability-riding quality-should be given by precedence over other consideration when the pavement rehabilitation is proposed. It should, however, be within the technical justification and the possible financial arrangements.

(1) AASHTO Guide 1986

AASHTO Guide 1986 suggest the following guidelines.

The primary measure of serviceability is the Present Serviceability Index (PSI) which ranges from 0 (impossible road) to 5 (perfect road). The terminal serviceability index (Pt) is the lowest acceptable level before resurfacing or reconstruction becomes necessary and suggested as follows:

Design of Major Highways; Pt = 2.5 or higher
Design of Highways with a low classification; Pt = 2.0
Design of Minor Highways; Pt = 1.5 (due to economic consideration)

(2) Survey in the Feasibility Study

One criterion for identifying a minimum level of serviceability may be established on the basis of public acceptance.

During the surface condition survey, the assessors were asked to say whether the sections were acceptable or not. In this way, a level of acceptability on the rating scale was established.

The survey results are summarized in Figure 6.3-1 showing the percent of people stating acceptable at each level of PSR and RRR.

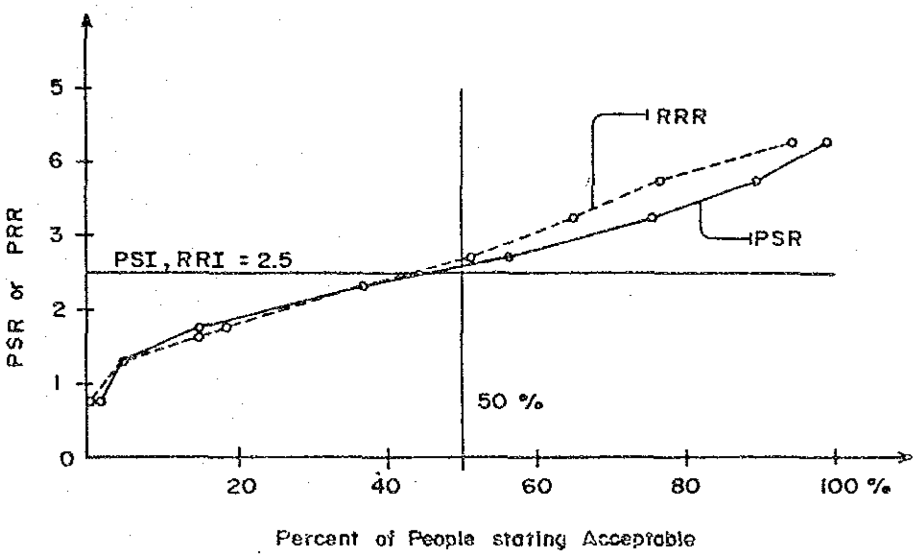


FIGURE 6.3-1 SERVICEABILITY AND ACCEPTABILITY

6.3.2 Engineering Requirement for Pavement Rehabilitation

(1) Failure Criteria of TRRL

Based on experimental pavements, Transport and Road Research Laboratory (TRRL) defines failure condition for reinforced concrete pavements is that the total length of cracking in each 100 m of the left-hand traffic exceeds 250 m. This is the time when wide cracks (width exceeding 1.2 mm. at the surface) are developing at a rate which makes reconstruction or overlaying a cheaper alternative to crack repair. See Figure 6.3-2.

For unreinforced concrete pavement, TRRL suggests that a substantial overlay would represent the economic solution to cracking when one in three of the slabs was cracked.

The 'failure' criterion used for experimental reinforced concrete roads in Britain is 250 m of total crack length per 100 cm of traffic lane. Experience shows that when such pavements are approaching the end of their lives about one-third of the total cracking falls into the Class 3 and Class 4 categories as defined in relation to the AASHO Road Test. It indicates that the British failure condition would correspond to a PSI level of about 2.

(2) Maintenance/Rehabilitation Criteria of JRA

The Road Maintenance/Rehabilitation Manual (Japan Road Association, JR 1978) proposes the criteria for maintenance/rehabilitation for concrete pavement as shown in Table 6.3-1.

TABLE 6.3-1 CRITERIA FOR MAINTENANCE/REHABILITATION (CONCRETE PAVEMENT)

	Rutting Depth (mm)	Faulting (mm)	Skid Resistence Coefficient	Longitudinal (mm)	Cracking[2/] Index (Cm/m^2)	Joint Failure
Highway with Heavy Traffic	30-40	15	0.25	5.0[1/]	30	Whenever failures are observed
Highway with Light Traffic	40-50	-	-	-	50	

[1/] Measured by 3 m Profilometer

[2/] Cracks measured are those cracks reaching the bottom of the slab

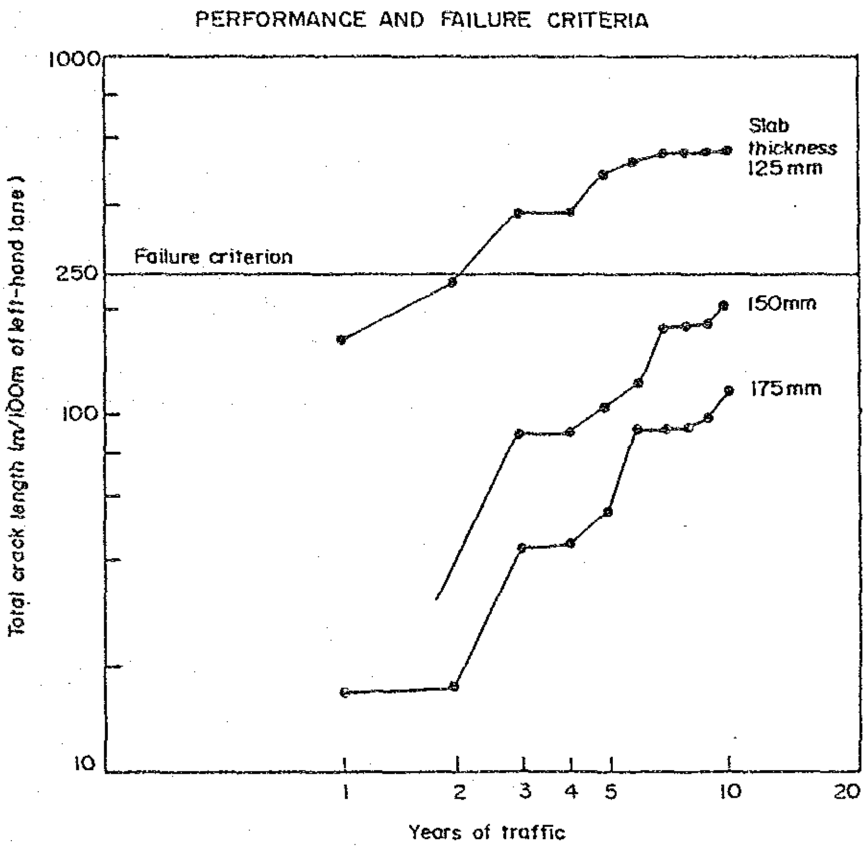


Figure 6.3-2 Development of cracks with age of road (reinforced concrete)

6.4 Recommended Improvement Level for Pavement Rehabilitation

(1) Comparison on Rehabilitation Criteria

Table 15.3-1 summarizes the comparison on the rehabilitation criteria required by AASHTO, TRRL and JRA. The corresponding RRI's in the Study were calculated based on the same length of cracking of each guideline and roughness values associated with that length of cracking in the Study Section.

It is observed from Table 6.4-1 that

- Failure criteria is about RRI of 2.0.
- Rehabilitation criteria

  Major Highway: more than RRI of 2.5
  Highway and a low Classification: RRI of 2.0 or 2.2
  Minor Highway: RRI of 1.5

TABLE 6.4-1 COMPARISON ON REHABILITATION CRITERIA

A A S H T O	AASHTO Guide 1986			The Study		
	Suggested Terminal Serviceability	Typical Pavement Condition		Corresponding RRI	Typical Pavement Condition	
		Roughness	Cracking		Roughness	Cracking
Major Highway	3.0 or 2.5	220 260	50 110	3.2 2.7	280 310	50 110
Highway with a lower Classification	2.0	300	220	2.2	340	220
Minor Highway	1.5	360	390	1.5	380	390

T R R L	T R R L			The Study		
	Total Truck Length			Corresponging RRI	Typical Pavement Condition	
	(m/100 m)	($m/1000\ m^2$)	Cracking		Roughness	Cracking
Failure Criteria	250	810	270	1.9	350	270

Cracking; 1/3 of Total Crack Length

J R A	Japan Road Association			The Study		
	Mix Allowable Cracking Index			Corresponding RRI	Typical Pavement Condition	
	(cm/m^2)	($m/1000\ m^2$)	Cracking		Roughness	Cracking
Freeway	20	200	100	2.7	310	100
Highway with Heavy Traffic	30	300	150	2.5	320	150
Highway with Light Traffic	50	500	250	2.0	340	250

Cracking; 1/2 of Mix Allowable Cradking Index

(2) Recommended Improvement Level of Pavement Rehabilitation

From the comparison and observation, the improvement level for pavement rehabilitation as shown in Table 6.4-2 is recommended.

TABLE 6.4-2 RECOMMENDED IMPROVEMENT LEVEL FOR PAVEMENT REHABILITATION

Highway Class	RRI	Typical Pavement Condition	
		Roughness	Cracking
Major Highway	2.5	320	140
Highway with a low Classification	2.0	340	250
Minor Highway	1.5	370	400

NOTE; Roughness; cm/km
Cracking; $m/1000\ m^2$ cracks under Class 3 and 4

Pan-Philippine Highway, being the Study Road, is undoubtedly the most important Major Highway in the country, therefore, RRI of 2.5 is recommended as the improvement level.

It is, however, noted that pavements with 2.5 to 3.0 of RRI should be given attention when the pavement rehabilitation program is planned. Because, the cracks of these pavements might be progressed and RRI might be less than 2.5 when the actual rehabilitation work is started.

CHAPTER 7

SURVEYS FOR PAVEMENT REHABILITATION DESIGN

CHAPTER 7

SURVEYS FOR PAVEMENT REHABILITATION DESIGN

Accurate condition survey which assess a pavement physical distress are vital to a successful rehabilitation effort. Discussed in this Chapter are deterioration survey, engineering survey and drainage survey to provide the necessary information to develop a sound rehabilitation strategy. Thus, an intensive survey is mandatory before any rehabilitation design are attempted.

7.1 Deterioration Survey

When pavement deterioration survey is conducted, there is a *minimum information requirement necessary to make knowledgeable* decisions regarding rehabilitation needs and strategies. These *information requirements are:*

(1) Distress Type - Identify types of physical distress existing in the pavement. The distress types should be placed in categories according to their casual mechanisms.

(2) Distress Severity - Note level of severity for each distress type present to assess degree of deterioration.

(3) Distress Amount - Denote relative area (percentage of the project) affected by each combination of distress type and severity.

A technically sound engineering deterioration survey must address each one of these needs, although the parameters of each category may vary. Refer to Appendix 5 for categorization and level of distress.

A thorough deterioration survey is an invaluable tool in the rehabilitation process. If properly conducted, the survey identifies distress types present which, in turn, assists the engineer in defining probable causes of the distress. Only with the proper identification of probable cause(s), it is possible to select the rehabilitation strategy (overlay or nonoverlay) that will both repair and prevent the problem.

It is emphasized that periodic condition surveys provide the capability to assess impending distress and estimate the probable rate of future pavement deterioration. Thus, recognition of the initial stages of rigid pavement, pumping, for example, may allow nonoverlay rehabilitation approaches to be used as compared to the costly rehabilitation of slab fracture, faulting, and joint damage. A more extensive deterioration survey is essential when embarking on a major rehabilitation project so that the best and most economic solution may be achieved.

AASHTO Guide 1986 recommends to utilize nondestructive deflection testing (NDT deflection) to evaluate the in-situ structural capacity of a pavement.

7.2 Engineering Survey

The evaluation of the overall structural capacity of the pavement system, especially for rigid pavements is a complicated assessment. While deflection criteria similar to those using a Benkleman Beam are in common use for structural evaluation and rehabilitation design, AASHTO Guide 1986 recommends the use of maximum elastic deflection in combination with an indicator of the radius of curvature of the pavement under load, which can be measured by nondestructive testing (NDT) method.

NDT deflection data are utilized for the following purposes.

- Evaluation of the In-Situ Structural Capacity of the pavement
- Rigid Pavement Joint/Load Transfer Analysis
- Rigid Pavement Slab-Void Detection.

In this Guide, however, only conventional methods are briefly outlined.

7.2.1 Geo-technical Survey

Wherever weak foundation/soft grounds exist, geo-technical survey (boring with standard penetration test) should be conducted together with labaratory tests to obtain the following data.

- Soil stratum
- Level of ground water
- Natural moisture content
- Atterberg limit
- Sieve analysis
- Unconfined compression test
- Consolidation test

Based on data obtained, settlement analysis should be carried out to account for the influence due to consolidation settlement of weak foundation/soft grounds against pavement deterioration, as follows.

Settlement Height

$$Sc = \frac{e_o - e_i}{1 + e_o} \cdot H$$

Where:

Sc = Settlement height

e_o = Initial void ratio

e_i = Void ratio after settlement

H = Thickness of settlement layer

Settlement Speed

$$t = \frac{(H/2)^2}{Cr} \cdot Tr$$

Where:

t = Settlement speed (see)

Cr = Coefficient of consolidation (cm^2/sec)

Tr = Coefficient of Time
0.848 for 90% consolidation

7.2.2 CBR Test

CBR test intends for determining the bearing value of subgrade and subbase. It is prefered that the CBR test should be conducted at every locations where soil test types and drainage condition differ.

The main data obtained from CBR test are:

- Natural and optimum moisture content
- Consistency
- Particle size distribution
- Specific gravity
- Compaction at natural and optimum moisture content
- CBR value at natural and optimum moisture content

From CBR value obtained, design CBR value may be determined by the methods mentioned in Chapter 3.

7.2.3 Deflection Survey

As discussed, AASHTO Guide 1986 recommends NDT deflection measurement for the structural evaluation of pavements. Also, the Asphalt Institute limits the use of Benkelman or Dynaflect only for asphalt surfaced roads.

In line with these recommendations from two agencies, this Guide recommends no adoption of deflection survey by Benkelman/ Dynaflect for rehabilitation design of PCC Pavement.

It is however noted that these conventional deflection measurements are still effective tools to estimate in-situ stability of pavement.

7.3 Drainage Survey

Distress in rigid pavements is often either caused or accelarated by the presence of moisture in the pavement structure. When designing pavement rehabilitation, it is important to assess probable causes of distress which may be moisture related damage.

In drainage evaluation, the following should be examined:

Goemetry

. longitudinal grades
. transverse grades
. widths of pavement layers
. layer thickness
. cut-and-fill depths
. slopes and dimensions of surface drainage
. features (ditches, culverts, etc.)
. in-place subsurface drainage

Topography

. feature influencing surface/subsurface movement of water
. lakes/streams
. seasonally wet/dry (climatic zone)
. soil map

Site Condition

. places where water moves across pavement surface
. places where water collects near pavement
. water level in ditches
. existence of water in cracks and joints
. ponding of water on shoulder
. deposits of fine or other evidence of pumping visible at pavement's edge.

CHAPTER 8

DESIGN REQUIREMENTS OF PAVEMENT REHABILITATION

CHAPTER 8
DESIGN REQUIREMENTS OF PAVEMENT REHABILITATION

This chapter discusses the preparation and/or selection of the inputs required for new (or reconstruction) pavement and rehabilitation design.

8.1 Design Requirements for New/Reconstruction.

Table 8.1-1 identifies all possible design inputs required for new/reconstruction design of pavement.

8.1.1 Design Variables

(1) Time Constraints

Time constraints involves the selection of performance and analysis period inputs which affect (or constrain) pavement design from the dimensions of time.

Performance Period of Initial Pavement Structures

Performance period refers to the period of time that an initial pavement structures will last before it needs rehabilitation. The most economical performance periods for type of pavements are discussed in Chapter 2. Refer to Table 2.4-1.

Analysis Period

Analysis period means the period of time for which the analysis is to be conducted. In general, an analysis period of 25 years is adopted.

TABLE 8.1-1 DESIGN REQUIREMENTS

Category	Description
1. Design Variable	
1.1 Time Constraints	
. Performance Period	Life of Initial Pavement Structure
. Analysis Period	Planned Stage construction; 25 years
1.2 Traffic	W_{18} = 18 kip Equivalent Single Axle Load (ESAL) Application Traffic Loading Classes; 10 classes (A to J)
1.3 Reliability	Z_R = 1.645 for 95% Reliability, R S_o = 0.3~0.4 for Standard Error } not considered
1.4 Environmental Impact	
. Roadbed Swelling	PSI_{SW} = Loss of PSI; not considered
2. Performance Criteria	
2.1 Serviceability	$PSI = P_o - P_t = P_o - \Delta PSI_W - \Delta PSI_{SW}$ (ΔPSI_{SW}; not considered)
3. Material Properties for Structural Design	
3.1 Effective Roadbed Soil Resilent Modules (Flexible)	MR (pci); estimated based on CBR, 8 cases (2~20)
3.2 Effective Modulus of Subgrade Reaction (Rigid)	K-Value (pci); estimated based on CBR, 8 cases (2~20)
3.3 Pavement Layer Materials Characterization	E_{SB} = Modulus of Subbase (8,000 psi) E_{BS} = Modulus of Base (22,000 psi) E_{AC} = Modulus of Asphalt Concrete (350,000 psi) E_C = Modulus of Elasticity of PCC(3.28×10^6 psi)
3.4 PCC Modulus of Rupture (Rigid) (Flexural Strength)	S'_c = Estimated Mean Value for PCC Modulus of Rupture (psi); 580 psi
3.5 Structural Layer Coefficient (Flexible)	Asphalt Concrete Layer Coefficient ; 0.39 Bitumen Stabilized : 0.2 Mechanically Stabilized (Crushed Stone); 0.125 Crusher Run (Crushed Gravel) ; 0.105 Subbase ; 0.095
4. Pavement Structural Characteristics	
4.1 Drainage	Flexible m = Layer Coefficent Modifying Factor; 0.8 Rigid CD = Drainage Coefficient; 0.9
4.2 Load Transfer (Rigid)	
. Jointed Pavement	
. Tied shoulder or Widened Outside Lane	J = Load Transfer Coefficient; 4
4.3 Loss of Support (Rigid)	LS = Loss of Support 1.0 ~ 3.0 for unbounded granular materials 2.0 ~ 3.0 for fine granular or natural subgrade materials 0 ~ 1.0 for cement Treated Granular Base
5. Reinforcement Variables (Rigid)	
5.1 Slab Length	Depending on local conditions, subbase type, course aggregate, etc.
5.2 Working Stress	
5.3 Friction Factors	

(2) Traffic

The design analysis are based on cummulative expected 18-kip equivalent single axle loads (ESAL) during the analysis period (W18). The traffic loading classes applicable in this country is proposed in Chapter 4. Refer to Table 4.2-1.

(3) Reliability

Reliability concept is introduced in AASHTO Guide 1986 to account for chance variations in both traffic prediction and performance prediction as mentioned in Chapter 2. Refer to Table 2.5-1.

However, reliability is concluded not to be considered in the Feasibility Study after the discussion with representative of DPWH. It is, therefore, noted that reliability of the basic structural design of this Guide is judged as 50 percent which is the lowest level suggested by AASHTO Guide 1986.

(4) Environmental Impacts

In this Guide, serviceability loss due to roadbed swelling (PSI_{SW}) is not accounted for, because the effects of seasonal temperature and moisture changes on material properties are not analyzed.

8.1.2 Performance Criteria

The primary measure of serviceability is the Present Serviceability Index (PSI) which ranges from 0 (impossible road) to 5 (perfect road). The original or initial serviceability (Po) observed at the AASHO Road Test were;

Po = 4.5 for Rigid Pavement
   = 4.2 for Asphalt Pavement

AASHTO Guide 1986 suggest the lowest allowable PSI or terminal serviceability Index (Pt).

Pt = 2.5 for major highway
= 2.0 for highway with lesser traffic/lower classification
= 1.5 for relatively minor highway where economic consideration dictatesthat initial expenditures be kept low.

Inthe Feasibility Study, the terminal serviceability of 2.5 was adopted taking into account the importance and traffic volume of the Pan-Philippine Highway . Refer to Table 6.4-2.

8.1.3 Material Properties for Structural Design

(1) Effective Roadbed Soil Resilient Modulus, (Flexible Pavement)

To determine effective roadbed soil resilient modulus (MR) exclusively for the design of flexible pavements based on serviceability criteria, laboratory resilient modulus test (AASHTO T274) should be performed on representative samples in stress and moisture conditions. Seasonal resilient modulus should also be determined based on conditions of primary moisture seasons, dry and wet seasons.

In this Guide, effective roadbed soil resilient modulus (MR) are estimated only based on soil classification and CBR Test results because of absence of available data, as shown in Table 8.1-2.

(2) Effective Modulus of Subgrade Reaction (Rigid Pavement)

An effective modulus of subgrade reaction (k-value) should be developed for rigid pavement design, accounting for seasonal modulus value, effects of subbase characteristics, effects of rigid foundation, relative damage of slab thickness due to 18 kip ESAL, loss of support ect.

In this Guide, effective modulus of subgrade reaction (k-value) are estimated, taking into account the effect of subbase, as shown in Table 8.1-2.

The detailed discussions on estimation of k-value are presented in Chapter 3 and Appendix 3-1.

TABLE 8.1-2 STRENGTH OF ROADBED/SUBGRADE

CBR of Subgrade	k (pci) of Subgrade	MR (pci) of Subgrade	k (pci)
2	50	2,500	80
3	100	4,000	130
4	120	5,000	170
6	160	6,000	210
8	180	7,000	230
10	200	8,000	250
15	230	12,000	280
20	250	15,000	300

NOTE: K; estimated based on the suggestion by Portland Cement Association.

(3) Pavement Layer Materials Characteristics

In this Guide, modulus of pavement layer materials are estimated following the suggestion by AASHTO Guide, 1986 as follows:

- Moduli for Subbase, E_{SB}; 8,000 psi (at 68°F)
- Moduli for Base, E_{BS}; 22,000 psi
- Moduli for Asphalt Concrete, E_{AC}; 350,000 psi
- Moduli for Portland Cement Concrete E_c; 3.28×10^6 psi

$$E_c = 57,000\ ('c)^{0.5}$$

Where:

Ec = PCC elastic modulus (psi)

'c = PCC compressive strength (psi)

(4) PCC Modulus of Rupture (Rigid Pavement)

The modulus of rupture (flexural strength) of portland cement concrete required by the design procedure is the mean value determined after 28 days using third-point loading (AASHTO T97, ASTM C78) and estimated as follows:

S'_c (mean) = Sc + Z (SDs)

S'_c = estimated mean value for PCC Modulus of Rupture (psi); 580 psi in the Study

Sc = construction specification of concrete modulus of rupture (psi)

SDs = estimated standard deviation of concrete modulus of rupture (psi)

Z = standard normal variate

In this Guide, the modulus of rupture of 580 psi (40 kg/cm^2) is adopted in accordance with the specification requirement of 525 psi (36.8 kg/cm^2) at 14 days in the country.

(5) Structural Layer Coefficients (Flexible Pavement)

A value for layer coefficient is assigned to each layer material in the pavement structure in order to convert actual layer thickness into structural number (SN).

In this Guide, the structural layer coefficients (Ai values are assumed as follows.

TABLE 8.1-3 STRUCTURAL LAYER COEFFICIENTS, Ai

Layer Material	Layer Coefficient
Asphalt Concrete Surface Course	0.39
Bitumen Stabilized	1.2
Mechanically Stabilized (Crushed Stone)	0.125 (CBR 40, R value 70)
Crusher Run (Crushed Gravel)	0.105 (CBR 25, R value 60)
Subbase	0.095 (CBR 8, value 40)

8.1.4 Pavement Structural Characteristics

(1) Drainage

The effects of certain levels of drainage on predicted pavement performance are important consideration in pavement design.

Table 8.1-4 presents the general definitions corresponding to different drainage levels from pavement structure.

TABLE 8.1-4 DRAINAGE LEVELS

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

In accordance with drainage level, AASHTO recommends m_i values for modifying structural coefficients of base and subbase materials for flexible pavements and value of drainage coefficients Cd for rigid pavements.

In this Guide, m_i of 0.8 and Cd of 0.9 are used. Refer to Table 13.3-2 and 13.3-3.

(2) Load Transfer (Rigid Pavement)

The load transfer coefficient, J, is a factor used in rigid pavement design to account for the ability of a concrete pavement structure to transfer (distribute) load across discontinuities, such as joints or cracks.

In this Guide, load transfer coefficient of 4 is used considering from effect of plain joint.

(3) Loss of Support

This factor, LS, is included in the design of rigid pavements to account for the potential loss of support arising from subbase erosion and/or differential vertical soil movements. It is treated in the actual design procedure by diminishing the effective or composite k-value based on the size of the void that may develop beneath the slab. Refer to Appendix 3-1 for detailed discussion.

8.1.5 Reinforcement Variables

Reinforcement variables are not discussed in this Guide.

8.2 Design Requirements for Overlay Design

Aside from the design inputs required for new (or reconstructed) pavement, the following additional inputs should be prepared for overlay design.

- Analysis unit delineation
- Existing pavement layer properties
- Existing pavement subgrade properties
- Design properties of overlay layers
- Effective in-situ structural capacity of existing pavement (by visual condition factor approach)
- Remaining life value of existing pavement prior to overlay (RLx) (by visual condition survey approach)
- Remaining life of overlaid pavement system (RLy) (by selection of desired terminal serviceability)
- Remaining life factor (FRl)

These design requirements are discussed in Chapter 9.

CHAPTER 9

BASIC STRUCTURAL DESIGN OF PAVEMENT REHABILITATION METHODS

CHAPTER 9

BASIC STRUCTURAL DESIGN OF PAVEMENT REHABILITATION METHODS

This chapter analyzes the structural design of pavement rehabilitation methods assuming the basic pavement conditions in the Philippines.

The design analysis was made in accordance with AASHTO Guide 1986. The design methods of PCA, TRRL and JRA are briefly explained in Appendix 9.3

9.1 Basic Assumptions

9.1.1 Major Design Criteria

Major design criteria assumed for one basic structural design of pavement rehabilitation methods are summarized in Table 9.1-1. Table 9.1-2 showsthe assumed strength of subgrade. Refer to Chapter 8 for the detailed discussion.

9.1.2 Typical Cross Section of Pavement Structures

Figures 9.1-1 specifies the structural design terms of flexible and rigid pavements as recommended AASHTO Guide 1986.

Figure 9.1-2 shows an example of cross section designed for PCC and AC pavements, and AC overlay.

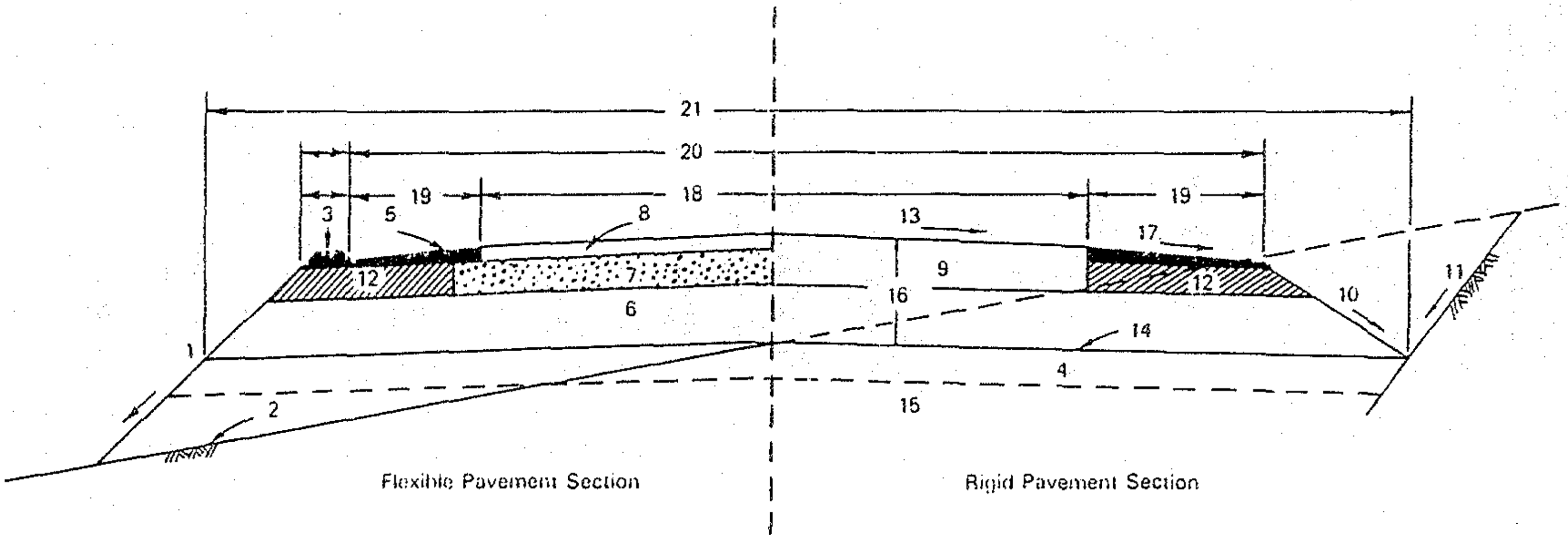
For PCC pavements, the thickness of subbase is always constant , 20 cm. This thickness was determined based on analysis of the combined effectiveness of subgrade and subbase. Refer to Appendix 3-1.

TABLE 9.1-1 Major Design Criteria

	Concrete/PCC Pavement	Asphalt/AC Overlay
Reliability	not considered	not considered
Performance Period	Traffic Loading A-E; 15 yrs. Traffic Loading H-J; 12 yrs.	Traffic Loading A,B; 12 yrs. Traffic Loading D-G; 8 yrs. Traffic Loading H-J; 5 yrs.
Traffic Loading Class	See Table 4.2-1	See Table 4.2-1
Serviceability	Initial; 4.5 Terminal; 2.5	Initial; 4.2 Terminal; 2.5
Effective Roadbed Soil Resilient Modulus	———	MR assumed by CBR See Table 9.1-2
Effective Modulus of Subgrade Reaction	K value assumed by CBR See Table 9.1-2	———
Pavement Layer Material Characteristics	Subbase ESB = 8000 psi Base EBS = 22000 psi AC EAC = 350000 psi PCC EC = 3.20×10^6 psi	Subbase ESB = 8000 psi
PCC Modulus of Rupture	525 psi (36.8 kg/cm^2); 14 days 580 psi (40.0 kg/cm^2(; 28 days	-
Structural Layer Coefficient	AC = 0.39 Bitumen Stabilized Base = 0.2 Mechanically Stabilized " = 0.125 Crushed Run Base = 0.105 Subbase = 0.095	-
Drainage	CD = 0.9	m = 0.8
Load Transfer Coefficient	4	-
Loss of Support	1	-
Visual Construction Factor of PCC Slab (RRI = 2.5)	0.4	-

TABLE 9.1-2 STRENGTH OF SUBGRADE

CBR of Subgarde	k (pci) of Subgrade	MR (pci) of Subgrade	k (pci)
2	50	2,500	80
3	100	4,000	130
4	120.	5,000	170
6	160	6,000	210
8	180	7,000	230
10	200	8,000	250
15	230	12,000	280
20	250	15,000	300



1 · FILL SLOPE
2 · ORIGINAL GROUND
3 · DIKE
4 · SELECTED MATERIAL OR PREPARED ROADBED
5 · SHOULDER SURFACING
6 · SUBBASE
7 · BASE COURSE
8 · SURFACE COURSE
9 · PAVEMENT SLAB
10 · DITCH SLOPE
11 · CUT SLOPE
12 · SHOULDER BASE
13 · CROWN SLOPE
14 · SUBGRADE
15 · ROADBED SOIL
16 · PAVEMENT STRUCTURE
17 · SHOULDER SLOPE
18 · TRAVEL LANES
19 · SHOULDER
20 · ROADWAY
21 · ROADBED

Structural Design Terms

Figure 9.I-1 Typical section for rigid or flexible pavement structure.

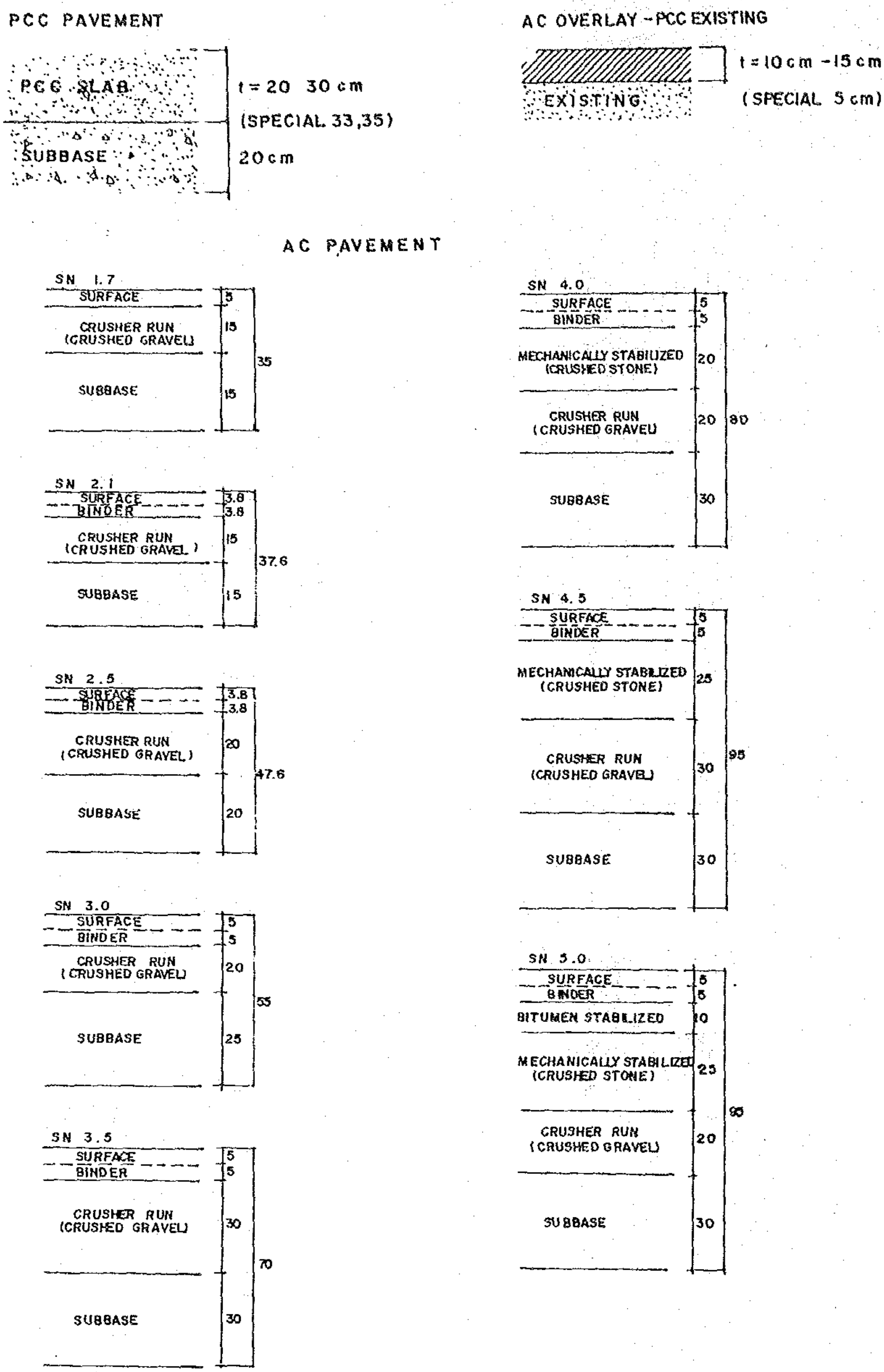


Figure 9.1-2 EXAMPLE OF CROSS SECTION DESIGN

The typical cross sections with various values of structural number are presented. The layer thickness analysis should be performed taking into consideration the unit cost of each layer.

9.1.3 Proposed Rehabilitation Methods

The following five (5) rehabilitation works are studied. See Figure 9.1-3.

- PCC Reconstruction
- AC Reconstruction
- Rigid Overlay - Rigid Existing
- Flexible Overlay - Rigid Existing
- Flexible Overlay - Flexible Existing

Figure 9.1-3 also summarizes second rehabilitation works and the succeeding works.

Typical cross section of proposed rehabilitation works are reported in Appendix 9-1.

9.1.4 Stage Construction (Planned Rehabilitation)

Because of heavy traffic loads, the initial pavement structures required involve the huge amount of the initial investment. Therefore, the stage construction (the planned rehabilitation) strategy are recommended in order to save the initial investment, thus facilitate rehabilitation project at the earliest possible time.

After the serviceability of initial pavements reaches at RRI of 2.5 (design performance period of initial pavement structure), the second rehabilitation work will be performed as shown in Table 9.1-3.

Figure 9.1-4 demonstrates the planned rehabilitation strategy.

Figure 9.1-3 . PROPOSED REHABILITATION METHODS

PROPOSED REHABILITATION METHODS \ STAGE CONSTRUCTION	FIRST REHABILITATION WORKS INCLUDING ALTERNATIVES	SECOND REHABILITATION WORKS	THIRD / AFTER THIRD REHABILITATION WORKS
PCC RECONSTRUCTION	CONCRETE SLAB (D), SUBBASE (20 cm.) (ALTERNATIVES) (D = 13, 15, 18 cm.) D = 20, 23, 25, 28, 30, 33, 35 cm.	AC OVERLAY 10 cm. (5 cm.) (MIN. REQUIREMENT DUE TO REFLECTION CRACKS)	AC OVERLAY 5 cm. (5 cm.)
AC RECONSTRUCTION	AC, BASE, SUBBASE (SN) (ALTERNATIVES) (SN = 1.3, 1.7, 2.1, 2.5, 3.0) SN = 2.5, 3.0, 3.5, 4.0, 4.5, 5.0, 5.5	AC OVERLAY 5 cm. (5 cm.)	AC OVERLAY 5 cm. (5 cm.)
RIGID OVERLAY RIGID EXISTING	CONCRETE SLAB (H), SEPARATION COURSE, EXISTING {D = 23 cm. = 20 cm. (20 cm.) (ALTERNATIVES) H = 13, 15 cm. H = 18, 20, 23, 25, 28 cm.	CONCRETE OVERLAY H 20 cm.	CONCRETE OVERLAY H 20 cm.
FLEXIBLE OVERLAY - RIGID EXISTING	EXISTING {D = 23 cm. = 20 cm. H (ALTERNATIVES) H = 5, 8 cm. H = 10, 13, 15 cm. (10 cm. MIN. REQUIRED DUE TO REFLECTION CRACKS)	AC OVERLAY 5 cm.	AC OVERLAY 5 cm.
FLEXIBLE OVERLAY - FLEXIBLE EXISTING	EXISTING AC { SN = 2.5 { H (ALTERNATIVES) (H = 3, 5, 8 cm.) H = 5, 8, 10 cm.	AC OVERLAY 5 cm. (3, 5 cm.)	AC OVERLAY 5 cm. (3, 5 cm.)

NOTE : ( ) FOR LIGHT LOADING TRAFFIC

TABLE 9.1-3 STAGE CONSTRUCTION (PLANNED REHABILITATION)

Rehabilitation Methods	Initial Rehabilitation (at the stage)	Second Rehabilitation	Third Rehabilitation
2- Lane PCC Reconstruction	PCC Reconstruction	10 cm AC Overlay	5 cm AC Overlay
1- Lane PCC Reconstruction	PCC Reconstruction	PCC Reconstruction	PCC Reconstruction
2- Lane AC Overlay	10 cm AC Overlay	5 cm AC Overlay	5 cm AC Overlay
2- Lane AC Reconstruction	AC Reconstruction	5 cm AC Overlay	5 cm AC Overlay

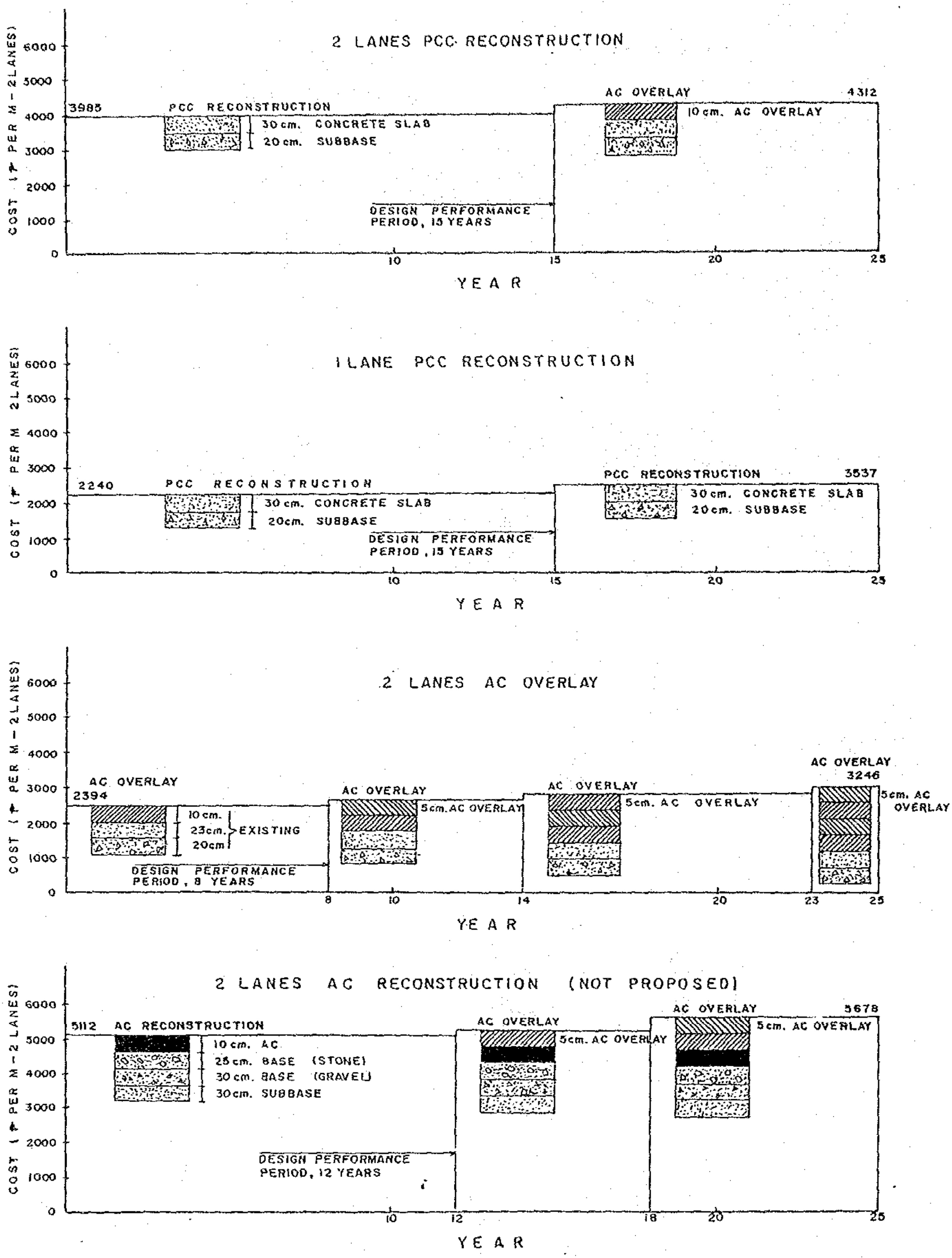


Figure 9.1-4 PLANNED REHABILITATION STRATEGY
TRAFFIC LOADING CLASS D
CBR=6

9.2 Rigid Pavement Design

These sections describes the design for portland cement concrete pavements, including plain jointed (JCP); jointed reinforced (JRCP), and continuously reinforced (CRCP).

Rigid pavement joint design, rigid pavement reinforcement design and prestressed concrete pavement are not discussed in this Guide but available in AASHTO Guide 1986.

(1) Basic Design Equation

The AASHTO design procedure is based on AASHO Road Test pavement performance algorithm. The basic design equation based on serviceability-performance concept for rigid pavement in AASHTO Guide 1986 is as follows.

Figure 9.2-1 presents the nomograph used for determining the slab thickness required for estimation total 18-kip Equivalent Single Axles Load Application and effective modulus of subgrade developed by AASHTO Guide 1986.

In the Feasibility Study, a computer program to solve this equation was developed. The output of basic structural design by an electric computer are summarized in Section 9.5.

Basic Design Equation for Rigid Pavements

$$\log_{10}(W_{18}) = Z_R x S_o + 7.35 x \log_{10}(D+1) - 0.06 + \left[\frac{\mathrm{Log}_{10}\frac{\Delta PSI}{4.5-1.5}}{1 + \frac{1.624x10^7}{(D+1)^{8.46}}} \right]$$

$$+ (4.22 - 0.32 x p_t) \log_{10} \left[\frac{S_c x C_d x (D^{0.75} - 1.132)}{2.15.63 x J \left[D^{0.75} - \frac{18.42}{(E_c/k)^{0.25}} \right]} \right]$$

Where:

W_{18} = predicted number of 18-kip equivalent single axle load applications,

Z_R = standard normal deviate,

S_o = combined standard error of the traffic prediction and performance prediction,

D = thickness (inches) of pavement slab,

Δ PSI = difference between the initial design serviceability index, P_o, and the design terminal serviceability index, P_t,

S_c = modulus of rupture (psi) for portland cement concrete used on a specific project,

J = load transfer coefficient used to adjust for load transfer characteristics of a specific design,

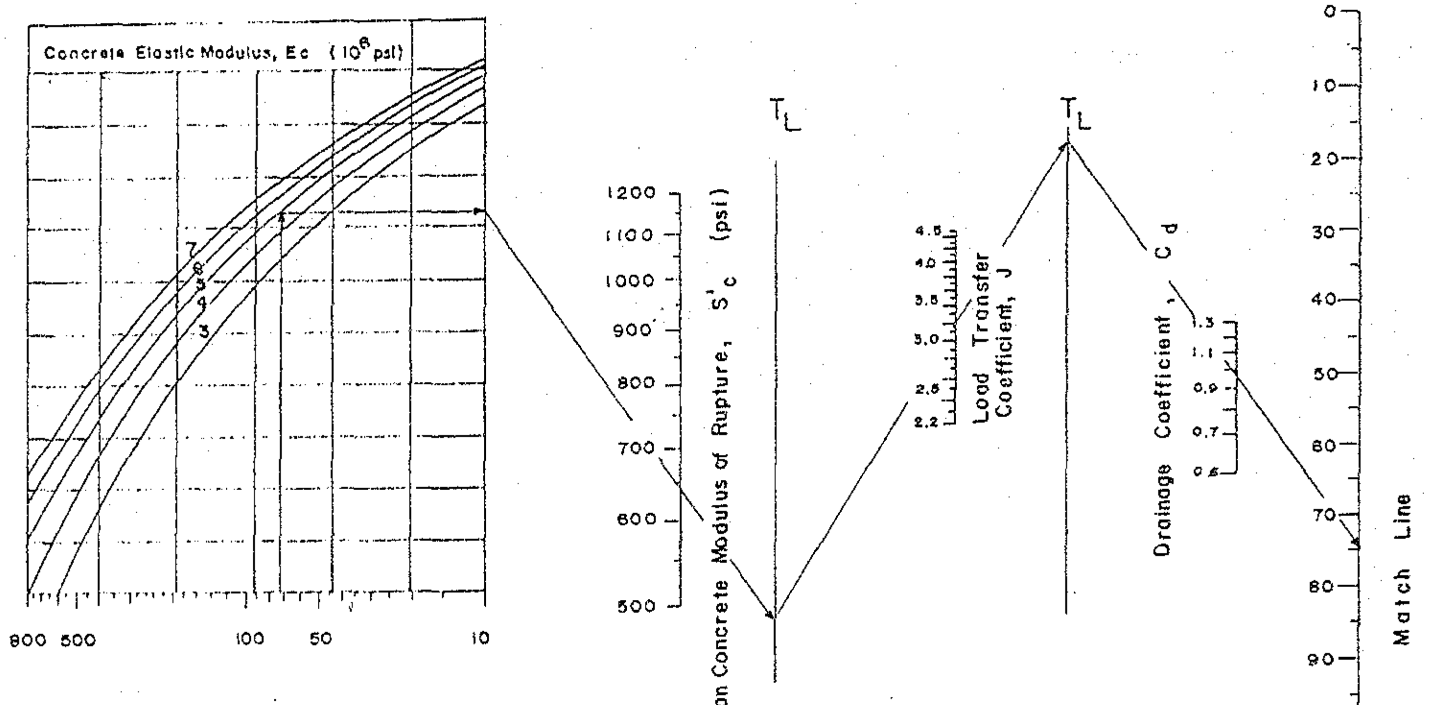
C_d = drainage coefficient

E_c = modulus of elasticity (psi) for portland cement concrete, and

k = modulus of subgrade reaction (pci)

NOMOGRAPH SOLVES:

$$\log_{10} W_{18} = Z_R * S_O + 7.35 * \log_{10}(D+1) - 0.06 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.5 - 1.5}\right]}{1 + \frac{1.624 * 10^7}{(D+1)^{8.46}}} + (4.22 - 0.32p_t) * \log_{10}\left[\frac{S'_c * C_d \left[D^{0.75} - 1.132\right]}{215.63 * J \left[D^{0.75} - \frac{18.42}{(E_c/k)^{0.25}}\right]}\right]$$



Example:

k = 72 pci
E_c = 5 x 10^6 psi
S'_c = 650 psi
J = 3.2
C_d = 1.0

So = 0.29
R = 95% (Z_R = -1.645)
ΔPSI = 4.2 − 2.5 = 1.7
W_{18} = 5.1 x 10^6 (18 kip ESAL)
Solution: D = 10.0 inches (nearest half-inch, from segment 2)

FIGURE 9.2-1 Design chart for rigid pavement based on using mean values for each input variable (Segment I)

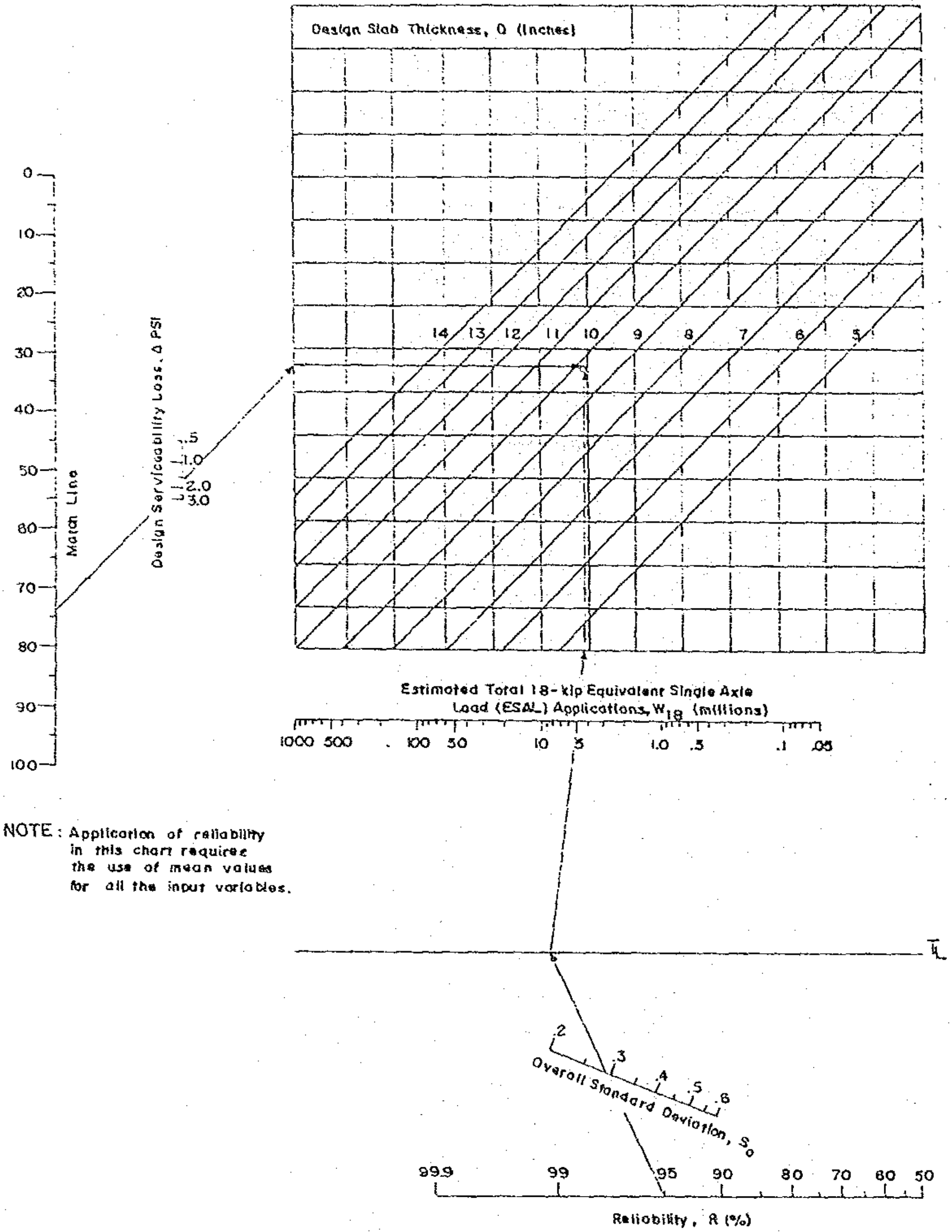


FIGURE 9.2-2 Design chart for rigid pavements based on using mean values for each input variable (Segment 2)

9.3 Flexible Pavement Design

(1) Basic Design Equation

The basic design equation based on serviceability - performance concept for flexible pavement in AASHTO Guide 1986 is as follows:

Basic Design Equation for Flexible Pavements:

$$\log_{10}(W_{18}) = Z_R \times S_o + 9.36 \times \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.2-1.5}\right]}{0.40 + \frac{1094}{(SN+1)^{5.29}}} + 2.32 \times \log_{10}(M_R) - 8.07$$

Where:

W_{18} = predicted number of 18-kip equivalent single axle load application.

Z_R = standard normal deviate.

S_o = combined standard error of the traffic prediction and performance prediction.

Δ PSI = difference between the initial design serviceability index, P_o, and the design terminal serviceability index P_t, and

M_R = resilient modulus (psi)

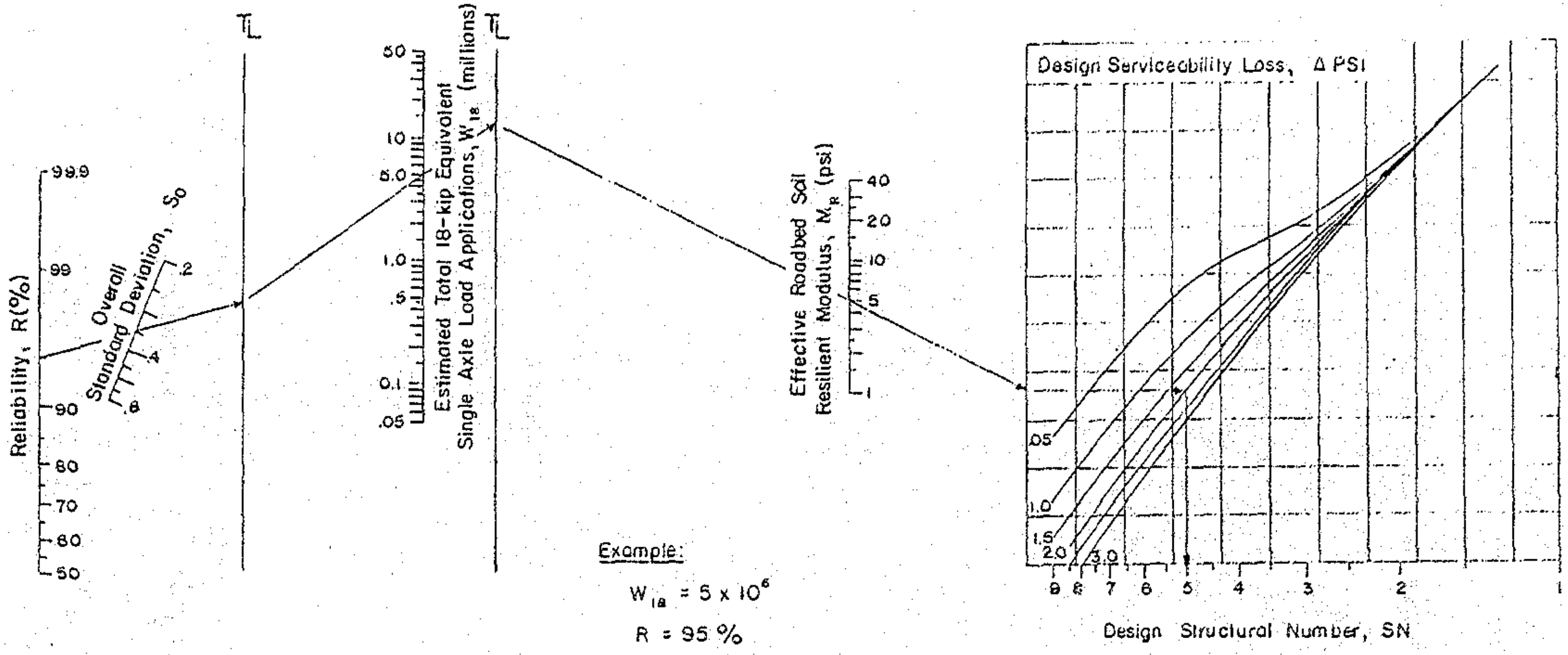
SN = structural number

The design nomograph to solve the equation is prepared in AASHTO Guide 1986 as shown in Figure 9.2-1.

In the Feasibility Study, a computer program was developed to analyze the equation. The outputs of basic design equation by electric computer are summarized in Section 9.5

NOMOGRAPH SOLVES:

$$\log_{10} W_{18} = Z_R * S_0 + 9.36 * \log_{10}(SN+1) - 0.20 + \frac{\log_{10}\left[\frac{\Delta PSI}{4.2 - 1.5}\right]}{0.40 + \frac{1094}{(SN+1)^{5.19}}} + 2.32 * \log_{10} M_R - 8.07$$



Example:

$W_{18} = 5 \times 10^6$

R = 95 %

S_0 0.35

M_R = 5000 psi

ΔPSI = 1.9

Solution: SN = 5.0

Figure 9.3-1 Design chart for flexible pavements based on using mean values for each input.

(2) Selection of Layer Thickness

The design is based on identifying a flexible pavement Structural Number (SN) to withstand the projected level of axle load traffic for the analyis period.

Once the design Structural Number (SN) for an initial pavement structure is determined, it is necessary to identify a set of pavement layer thickness. The following equation provides the basis for converting SN into actual thicknesses of surfacing, base and subbase;

$$SN = a_1D_1 + a_2D_2m_2 + a_3D_3m_3$$

Where:

a_1, a_2, a_3 = layer coefficients representative of surface, base and subbase course, respectively (see Section 2.3.5)

D_1, D_2, D_3 = actual thicknesses (in inches) of surface, base, and subbase courses, respectively.

m_2, m_3 = drainage coefficients for base and subbase layers, respectively. (see Section 2.4.1).

Since it is generally impractical and uneconomical to place surface, base or subbase courses less than the minimum thickness, the value shown in Table 9.3-1 are provided as minimum practical thicknesses for each pavement course.

TABLE 9.3-1 FLEXIBLE PAVEMENT

Unit = Inches

Traffic, ESAL's	Asphalt Concrete	Aggregate Base
Less than 50,000	1.0 (or surface treatment)	4
50,001 - 150,000	2.0	4
150,001 - 500,000	2.5	4
500,001 - 2,000,000	3.0	6
2,000,001 - 7,000,000	3.5	6
Greater than 7,000,000	4.0	6

9.4 Pavement Overlay Design

9.4.1 General Overlay Methodology

The general overlay methodology presented herein is applicable to all types of overlay placed on any type of pavement structure. This methodology employs the serviceability-traffic (performance) relationships, which rely on life-cycle cost concepts to select a minimum economic overlay strategy.

Figure 9.4-1 diagrams the key relationships and concepts of the general overlay methodology. They are: (a) serviceability-traffic repetitions, (b) structural capacity-traffic repetitions, and (c) pavement condition factor-traffic repetitions.

In Figure 9.4-1, the overall pavement condition factor, C, is related to the effective capacity by the following;

$$SC_{xeff} = Cx.SC_{o}$$

When the concept of remaining life is considered, the general overlay equation becomes:

$$SC_{OL}{}^{n} = SC_{y}{}^{n} - F_{RL}\ (SC_{xeff})^{n}$$

Where:

F_{RL} : The remaining life factor which accounts for damage of the existing pavement as well as the desired degree of damage to the overlay at the end of the overlay traffic. It is always less than or equal to a value of 1.0.

Sc_{o} : See Figure 9.4-1

SC_{OL} : See Figure 9.4-1

SC_{y} : See Figure 9.4-1

SC_{eff}: See Figure 9.4-1

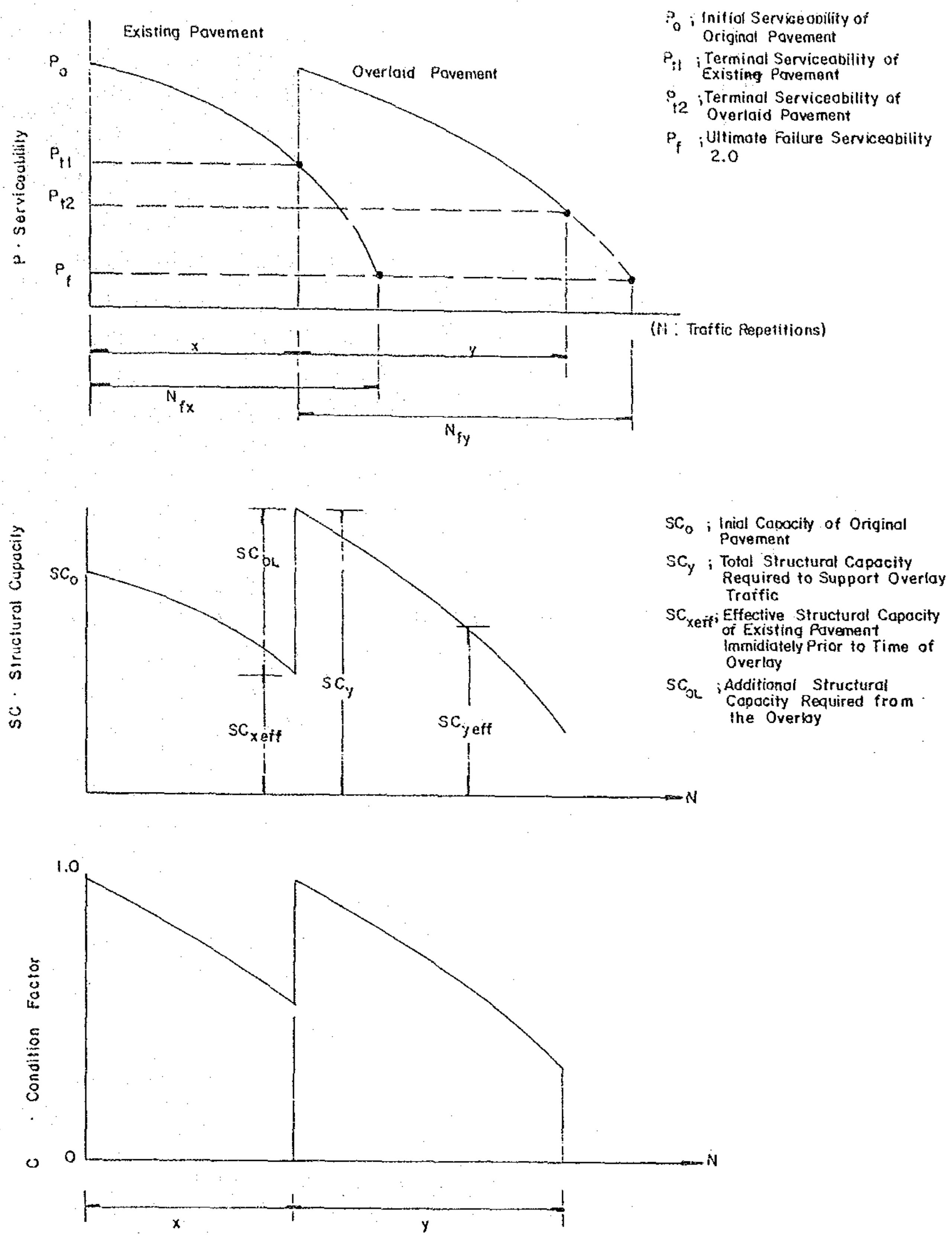


Figure 9.4-1 Relationship between serviceability-capacity condition factor and traffic.

9.4.2 Overlay Design Procedure

There are seven (7) steps in the overlay design procedure, as shown in Figure 9.4-2.

Step 1; Analysis Unit Delineation

The first step in the overlay process is the clear delineation of basic analysis units. The objective is to determine boundaries along the project length that subdivide the rehabilitation project into statistically homogenous pavement units possessing uniform pavement cross sections, subgrade (foundation) support, construction histories, and subsequent pavement condition.

Step 2; Traffic Analysis

The purpose of the traffic analysis step is to determine the cumulative 18 ESAL repetitions along a pavement length from the date the pavement was originally opened to traffic through the end of the anticipated overlay period.

Step 3; Materials and Environmental Study

Design values for the layer materials used in the rehabilitation process may be categorized into three major groups:

- Existing pavement layer properties
- Existing pavement subgrade (foundation) properties
- Design properties of overlay layers

AASHTO Guide 1986 recommends the Pavement Layer Moduli Prediction Method of NDT (Nondestructive Testing).

GENERAL OVERLAY EQUATION; $SC_{OL}^{\,n} = SC_y^{\,n} - F_{RL}(SC \times eff)^n$

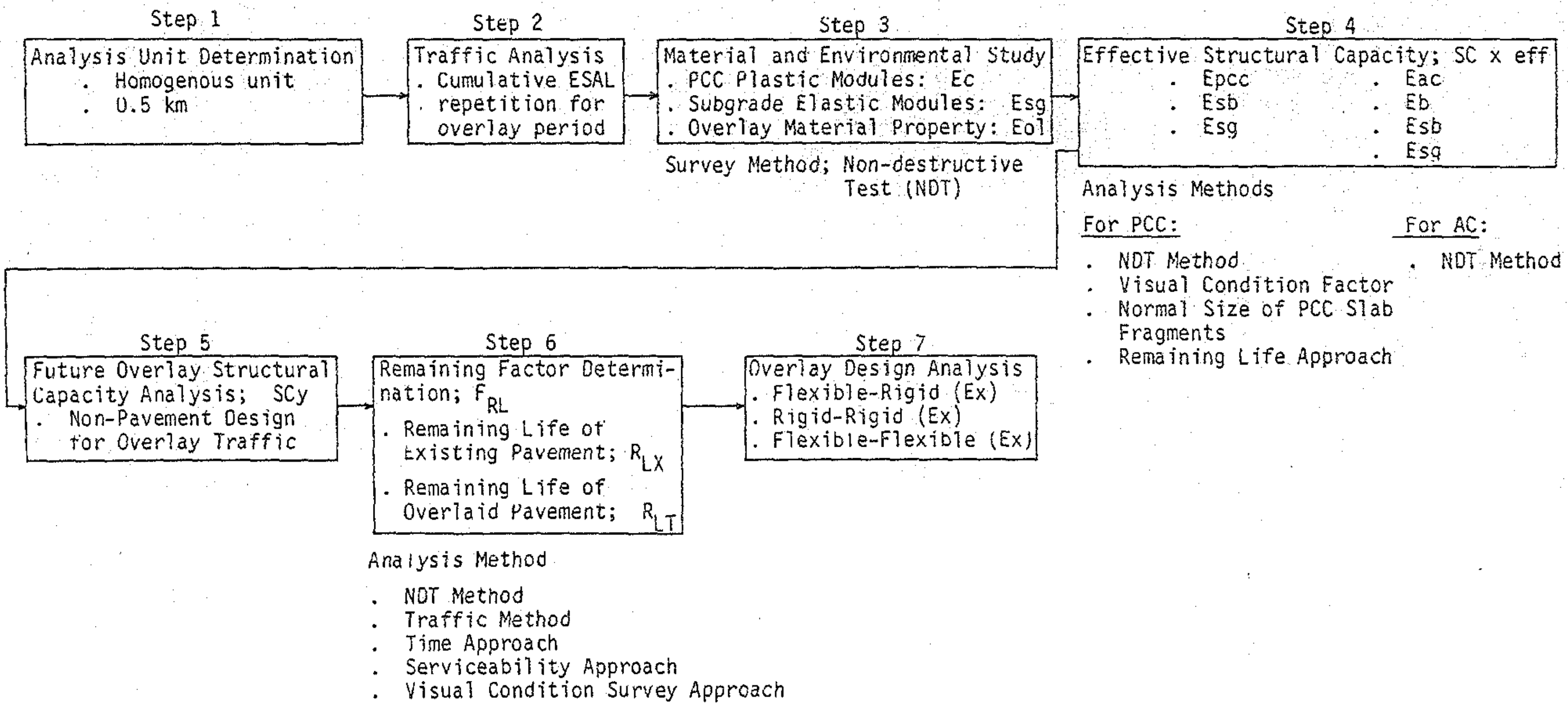


FIGURE 9.4-2 OVERLAY DESIGN PROCEDURE

Step 4; Effective Structural Capacity Analysis

The fourth step in an overlay analysis is to estimate effective (in situ) structural capacity of the pavement to be overlaid. Information regarding material properties derived in the previous step is used to arrive at this parameter.

Rigid Pavements

Aside from two (2) method of NDT, AASHTO Guide 1986 recommends approximate procedures to estimate effective structural capacity of the pavement to be overlaid. They are;

- Visual Condition Factor Approach

  The relationship between Cv (Visual condition factor) and DxeH value (Effective Thickness of PCC Slab) are proposed as shown in Table 9.4-1.

- Normal Size of PCC Slab Fragments

- Remaining Life Approach

Flexible Pavements

Only NDT method can be applied.

Step 5; Future Overlay Structural Capacity Analysis (SCy)

The major objective of this step is simply to determine the total structural capacity of a new pavement required to carry repetitions in the overlay period to a terminal serviceability of P_{t2}, using the same existing subgrade (foundation) support for the design value. The analysis assumes that the existing pavement (SC_{xeff}) does not exist over the foundation. Consequently, this step in overlay process is simply a new pavement design for either a flexible system or rigid system.

TABLE 9.4-1 SUMMARY OF VISUAL (C_v) AND STRUCTURAL (C_x) CONDITION VALUES

Layer Type		Pavement Condition	C_v Visual Condition Factor Range	C_x Structural Condition Factor Value
Asphaltic	1.	Asphalt layers that are sound, stable, uncracked and have little to no deformation in the wheel paths.	0.9 - 1.0	.95
	2.	Asphalt layers that exhibit some intermittent cracking with slight to moderate wheel path deformation but are still stable.	0.7 - 0.9	.85
	3.	Asphalt layers that exhibit some moderate to high cracking, have ravelling or aggregate degradation and show moderate to high deformation in wheel path.	0.5 - 0.7	.70
	4.	Asphalt layers that show very heavy (extensive) cracking, considerable ravelling or degradation and very appreciable wheel path deformations.	0.3 - 0.5	.60
P C C	1.	PCC pavement that is uncracked, stable and undersealed, exhibiting no evidence of pumping.	0.9 - 1.0	.95
	2.	PCC pavement that is stable and undersealed but shows some initial cracking (with tight, non-working cracks) and no evidence of pumping.	0.7 - 0.9	.85
	3.	PCC pavement that is appreciably cracked or faulted with signs of progressive crack deterioration: slab fragments may range in size from 1 to 4 sq. yds. pumping may be present.	0.5 - 0.7	.70
Pozzolanic Base/ Subbase	1.	Chemically stabilized bases (CTB, LCF...) that are relatively crack free, stable and show no evidence of pumping.	0.9 - 1.0	.95
	2.	Chemically stabilized bases (CTB, LCF...) that have developed very strong pattern or fatigue cracking, with wide and working cracks that are progressive in nature: evidence of pumping or other causes of instability may be present.	0.3 - 0.5	.60
Granular Base/ Subbase	1.	Unbound granular layers showing no evidence of shear or densification distress, reasonably identical physical properties as when constructed and existing at the same "normal" moisture - density conditions as when constructed.	0.9 - 1.0	.95
	2.	Visible evidence of significant distress within layers (shear or densification), aggregate properties have changed significantly due to abrasion, intrusion of fines from subgrade or pumping, and/or significant change in in-situ moisture caused by surface infiltration or other sources.	0.3 - 0.5	.60

SPECIAL NOTES:

1. The visual condition factor, C_v is related to the structural condition factor, C_x, by:

$$C_v = C_x^2$$

2. The structural condition factor, C_x, and not the C_v value, is the variable used in the structural overlay and design equation (for all overlay-existing pavement types). It is defined by:

$$SC_{self} = C_v\, SC_o$$