EAROMAR VERSION 2 TECHNICAL REPORT

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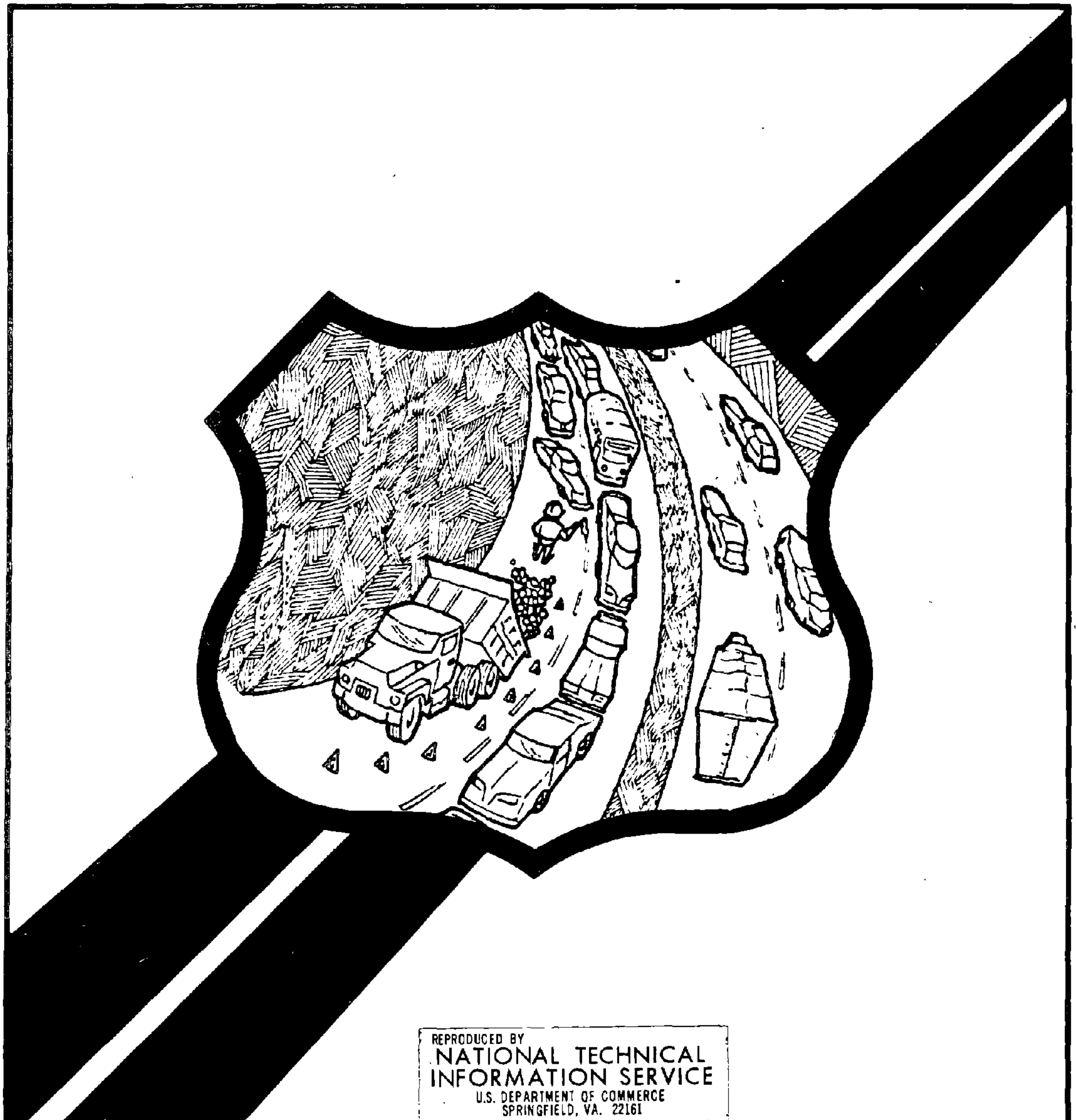

FOREWORD

Highway administrators and engineers must select among alternative pavement investment and maintenance strategies. These decisions should be based upon economic analyses of the impacts expected for each pavement management strategy. The FHWA system EAROMAR predicts structural performance and simulates freeway operation to evaluate life-cycle roadway costs. "EAROMAR Version 2, Final Technical Report" presents a detailed analysis of the technical aspects of pavement life-cycle costing. The other four volumes are:

FHWA/RD-82/085, "Executive Summary"
FHWA/RD-82/087, "Users Manual"
FHWA/RD-82/088, "Program Documentation"
FHWA-IP-82-13, "Case Studies."

Sufficient copies of this report are being distributed by FHWA Bulletin to provide a minimum of one copy to each regional office, division office, and State highway agency. Direct distribution is being made to the division offices.

Richard E. Hay, Director
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NOTICE

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15. Supplementary Notes

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16. Abstract Decisions among competing pavement investment and maintenance strategies must be based upon economic analyses considering both the costs and impacts of each strategy. Such analyses are sensitive to several local factors, including initial pavement design and construction, traffic loads, climate, maintenance and rehabilitation policy, maintenance technology, and unit costs. Of particular importance here are maintenance and rehabilitation actions, whose effects on pavement performance have not been studied extensively or quantified in the past.

To enable highway administrators and engineers to account better for the interactions among the several factors above in influencing strategy selection, we have redesigned and recoded FHWA's EAROMAR system to produce a second version of this product. EAROMAR simulates freeway operational and structural performances to predict life-cycle roadway costs. These costs include highway agency expenditures for roadway reconstruction, rehabilitation and maintenance, and user costs of vehicle operation, travel time, (including congestion), and accidents.

This report details the design concepts and technical relationships built into the modified EAROMAR system. Innovative features include prediction of maintenance and rehabilitation requirements based upon the damage occurring in the pavement, and detailed treatments of maintenance policy, maintenance technology, and traffic flow and congestion.

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CHAPTER 1

INTRODUCTION

1.1 BACKGROUND TO STUDY

In 1975 urban highways accounted for less than one-sixth of the total mileage in the U.S. Federal Aid system. Nevertheless, they sustained more than one-half the total system usage, measured in vehicle miles traveled (VMT). As shown in Table 1, this relatively high ratio of usage to length held true across all major designations of Federal Aid system roads in urban areas.

The pattern is most striking for the Interstate highways, no doubt owing to their uniformly superior design standards directed toward high-volume, high-speed travel. The 22 per cent (by length) of Interstate freeways classified as urban roads in 1975 carried 52 per cent of Interstate VMT nationwide. The relative distributions of average daily traffic volumes in the urban Interstate network are shown in Table 2, indicating that some 30 per cent of these roads experienced over 40,000 vehicles per day.

Occurrences of similarly high volumes were also recorded on other urban portions of the Federal Aid system, but to a much lesser degree. In fact, Tables 1, 3 and 4 imply that flows exceeding 40,000 vehicles per day were characteristic of only one to two per cent of non-Interstate urban roads nationwide, these heavily used roads comprising for the most part divided, controlled-access expressways and wide arterials. Thus, the movement of high-volume traffic can be associated almost exclusively with controlled-access urban freeways typified by Interstate program construction over the past twenty years.

An important consequence of the sustained heavy volumes on these highways has been their deterioration under traffic loadings in relatively short periods of time, raising the problem of how to maintain or rehabilitate them adequately, safely and economically. Several repair projects have already been undertaken on urban roads built during the early years of the Interstate program, providing case study lessons in the difficulties involved. The facts that many additional freeways were completed under Interstate funding in the 1960s, and that such heavily-trafficked pavements often start to show distress after only 10-15 years' service, indicate the continued significance of this problem through the coming years.

This relatively rapid rate of pavement deterioration observed, plus the difficulty in repairing structural damage while contending with high volume traffic, and the general trend toward rehabilitation of the existing urban network in lieu of new road construction, have all renewed interest in premium pavements or so-called "zero maintenance" designs for major urban highways. Premium pavements are pavements expected to pro-

TABLE 1

URBAN HIGHWAY LENGTH AND USE, 1975

(Data Expressed as Percentage of Total Mileage and VMT Respectively for the Federal Aid System as a Whole)

	INTERSTATE	OTHER FEDERAL AID PRIMARY	FEDERAL AID SECONDARY	FEDERAL AID URBAN	TOTAL URBAN
MILEAGE (% of Federal Aid System)	1	4	3	6	14
VEHICLE MILES TRAVELED (% of Federal Aid System)	13	16	5	16	50

SOURCE: Highway Statistics, 1975 (1), Tables FM-1 and VM-2.

TABLE 2

DISTRIBUTIONS OF AVERAGE DAILY TRAFFIC VOLUMES ON URBAN INTERSTATE HIGHWAYS, 1975

(Data Based on National Totals for the 50 States Plus District of Columbia)

	INTERVALS OF AVERAGE DAILY TRAFFIC					
	0 - 10,000	10,000- 20,000	20,000- 30,000	30,000- 40,000	> 40,000	Totals*
Mileage	1,056	2,081	1,896	1,199	3,024	9,413
Kilometrage	1,700	3,350	3,053	1,930	4,869	15,155
Per Cent	11	22	20	13	32	98

*157 miles (253 km.), or 2 percent, are unclassified.

SOURCE: Highway Statistics, 1975 (1), Table INT-15.

TABLE 3

DISTRIBUTIONS OF AVERAGE DAILY TRAFFIC VOLUMES ON NON-INTERSTATE URBAN HIGHWAYS

(Data in Per Cent)

	INTERVALS OF AVERAGE DAILY TRAFFIC				
	0- 20,000	20,000- 30,000	30,000- 40,000	> 40,000	TOTAL
1. Non-Interstate Federal Aid Primary - Urban	80	12	4	4	100
2. Federal Aid Urban	91	6	2	1	100

SOURCE: Highway Statistics, 1975 (1), Table FM-110

TABLE 4

CHARACTERISTICS OF NON-INTERSTATE ROADS
HAVING DAILY TRAFFIC VOLUMES GREATER
THAN 40,000 VPD

1. NON-INTERSTATE FEDERAL AND PRIMARY SYSTEM - URBAN*

Undivided	24%
Divided, No or Partial Access Control	29
Divided, Full Access Control	47
Total	100%

2. FEDERAL AND URBAN SYSTEM*

Undivided	30%
Divided, No or Partial Access Control	36
Divided, Full Access Control	34
Total	100%

*Daily volumes of 40,000 or greater were observed within each group predominantly on roads of 48 foot width (15 m.) or wider.

SOURCE: Highway Statistics, 1975 (1), Table FM-110 vide maintenance-free service under projected traffic and environmental conditions for at least twenty years, and to require only routine structural maintenance for ten to twenty years thereafter.

By their nature premium pavements are capital intensive strategies, and their justification requires an assessment of the economic benefits of such pavements (to both the sponsoring agency and the motoring public) in comparison to their higher initial cost. Where anticipated benefits exceed costs, an economic warrant is established for premium pavement design and construction.

Current methods of pavement design and evaluation are not generally applicable to analyzing the need for premium pavements. Such methods typically incorporate rather arbitrary assumptions as to, for instance, the suitable level of pavement serviceability to be afforded and over what duration (e.g., producing designs sufficient to sustain pavement PSI above 2.5 over a 20-year life); or the level of maintenance to be provided (e.g., "normal" maintenance assumed, although this effort is not explicitly defined). Furthermore, in incorporating these types of assumptions, many pavement design and evaluation methods reduce the inherent economic problem -- to minimize total pavement-associated costs -- to a simpler, but not equivalent, technical problem.

Such simplifications break down under a premium pavement analysis. For example, neither the 20-year design life, nor the terminal serviceability value of 2.5 above, may be optimal under the high-volume traffic conditions for which premium pavements are most appropriate. Also, assumptions of "normal" maintenance as applied to conventional designs fail to account for the significant problems in repairing pavements during peak hour urban flows. Difficulties in maintenance and rehabilitation under high levels of demand include greatly increased congestion and associated costs of traffic delay, safety hazards to both the maintenance work crew and the motoring public, space restrictions and tight work schedules which may affect work production and quality, and increased unit costs of maintenance during nighttime or weekends to avoid rush-hour disruptions.

1.2 STUDY OBJECTIVE

This research project was undertaken in response to the issues above, with the objective to develop a set of procedures to provide economic warrants for premium pavements. From an historical perspective, this project was intended to update findings developed for the Federal Highway Administration in 1974 by Byrd, Tallamy, MacDonald and Lewis (2), findings which were incorporated within a computerized analysis procedure -- the EAROMAR system (Economic Analysis of Roadway Occupancy for Maintenance And Rehabilitation).

Our update and revisions of the engineering, economic, and computer design aspects of EAROMAR have been focused in three major areas: (1) introduction of new concepts in, for example, the description of the analysis problem, maintenance policy specification, maintenance scheduling and resource management, and treatment of differential cost inflation within the economic analysis; (2) incorporation of new models and data regarding, for example, pavement deterioration and user cost predictions; and (3) restructuring the computer program code itself to produce a more modular organization, facilitating future revisions, expansion, and adaptation to specific locales. Because the collective effect of these changes has been major, we have produced a new, redesigned, and recoded version of the EAROMAR system.*

The technical concepts, models and procedures developed under this research for application to premium pavement evaluation are described in this report. The computerized model encompasses flexible, rigid, and composite pavement designs, and accounts for relevant physical, environmental, economic and policy-related factors which interact to influence pavement performance and costs. It considers explicitly the operational, safety, and cost aspects of road occupancy of heavily trafficked routes. Results contributing to economic warrants comprise costs of pavement construction (and any subsequent reconstruction or overlays); pavement maintenance costs; user-associated costs for vehicle operation, travel time, and accident occurrence; and changes in levels of vehicle emissions.

1.3 METHOD OF APPROACH

Premium pavement warrants depend upon the interaction of several technical, economic and socio-political criteria which are specific to a region and time and can therefore be evaluated only on a case-by-case basis. These include, for example:

- Road structural and operational characteristics
- Projected traffic volume and composition
- Local environmental conditions
- Policies regarding pavement maintenance and rehabilitation
- Local practices on scheduling of maintenance work
- Prevailing unit costs for labor, equipment and materials needed for pavement repair
- Values of travel time, economic costs attributable to highway accidents, and appropriate rates of discount.

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* For simplicity the name "EAROMAR" when used in this report will refer to the new version of the computerized analysis, unless qualified by phrases denoting "the original" or "the initial" system.

On a national basis these criteria governing the analysis may vary widely. Throughout this project we have therefore identified the problem not as one to develop specific premium pavement warrants, since this would be impossible to do in any general way; but rather to develop a flexible, practical procedure which local highway administrators could apply to determining warrants under their particular situations. A premium pavement justified in one locale will not necessarily be warranted in another, where maintenance practices, unit costs, or other factors above may differ.

Economic warrants for premium pavements are built from the EAROMAR system's estimates of pavement-related construction, maintenance, rehabilitation, reconstruction, and user operating costs, discounted through an analysis period. These costs are obtained by simulating road operational and structural performance through successive seasons within years, accounting in each period for the collective influences of pavement characteristics, imposed loadings, environmental factors, maintenance policies, and other factors enumerated above on pavement damage and corresponding maintenance or rehabilitation requirements. By performing the analysis for several different pavement designs and maintenance policies, one may compare the total discounted costs of each strategy to identify the least-cost combination of design and maintenance. Where road occupancy for pavement maintenance or rehabilitation will cause substantial increases in user costs (due to increased travel time and congestion, inefficient vehicle operation, increased risk of accidents, and increased vehicle emissions), a premium pavement requiring essentially no maintenance may be justified.

The major components of the EAROMAR simulation are identified in Table 5. To the extent possible each component has been addressed in as flexible and comprehensive a manner as possible. For example, specification of maintenance, reconstruction, traffic, and economic data may vary both over time and along the length of the road. Program design is modular, allowing each component in Table 5 to be easily updated or replaced as need be. All data are provided the system via free-format input conventions.

1.4 OUTLINE OF REPORT

The following chapters describe the function of each model component in Table 5.

Route characteristics, construction projects, strategy specifications and economic data are described in Chapter 2.

Chapter 3 presents damage prediction models for flexible, rigid and composite pavements.

Chapter 4 discusses treatment of pavement repair (whether by maintenance, overlay, or reconstruction), and develops our approach to maintenance policy specifications and maintenance management.

Chapter 5 describes the engineering aspects of road user operations, including representation of travel demand and simulation of traffic flow under both free-flow and congested conditions.

Chapter 6 details the treatment of road user costs, including vehicle operating costs, travel time costs, accident costs, and pollution emissions.

Chapter 7 concludes the report, summarizing the major features of the EAROMAR system and identifying potential system applications.

TABLE 5

COMPONENTS OF THE EAROMAR SYSTEM

ROUTE CHARACTERISTICS

- Geometry and Capacity
- Pavement
- Environment
- Administrative Sections
- Initial Construction Costs

CONSTRUCTION PROJECTS

- Overlays
- Reconstruction or Additional Construction

TRAFFIC DEMAND

- Vehicle characteristics
- Trip Purpose
- Volume and Growth
- Daily and Hourly Distributions

DAMAGE PREDICTION

- Flexible Pavement
- Rigid Pavement
- Composite Pavement

MAINTENANCE POLICIES

- Flexible Pavement
- Rigid Pavement
- Composite Pavement

TABLE 5

COMPONENTS OF THE EAROMAR SYSTEM

(continued)

MAINTENANCE MANAGEMENT

- Scheduling
- Unit Cost and Production
- Resource Limitations

ROAD OPERATING CHARACTERISTICS

- Free-flow Speed
- Congestion and Queueing
- Speed Change Cycles

USER CONSEQUENCES

- Vehicle Operating Costs
- Travel Time
- Accident Costs
- Pollution Emissions

ECONOMIC DATA

- Discount Rates
- Differential Inflation
- Accident Costs

STRATEGY SPECIFICATIONS

CHAPTER 2

SCOPE, CONCEPT, AND ORGANIZATION OF THE EAROMAR ANALYSIS

2.1 OVERVIEW

Cost streams to determine economic warrants for premium pavements are predicted by EAROMAR through its simulation of highway operation and pavement performance, encompassing the several sets of parameters listed in Table 5. This chapter begins our description of the modeling approach employed, defining system scope, concept, and organization. The general precepts introduced in this chapter will set the stage for more detailed technical explanations presented later in this report.

We start below with the first component of problem definition, the specification of route initial characteristics. For the sake of clarity and convenience, all subsequent influences on pavement deterioration, performance, and costs over time are considered under the organization of strategies in Section 2.3. It is important to bear in mind, however, that the data discussed in Sections 2.2 and 2.3 respectively must interrelate with one another during the analysis. Furthermore, both route characteristics and strategy specifications may be varied by a manager over a series of analyses to test different conventional versus premium pavement options. The nature of this data interaction and suggestions of the types of alternatives that can be evaluated are discussed in Section 2.4. Section 2.5 concludes the chapter, with an explanation of economic parameters influencing cost predictions, illustrating the treatment of inflation and discounted annual totals.

2.2 ROUTE CHARACTERISTICS

General

Route characteristics encompass the geometric, structural, operational and environmental descriptors which must be known to carry out an analysis through EAROMAR. These characteristics are used, first, to define engineering and environmental variables needed, for example, to predict pavement damage or to estimate volume-capacity relationships; and second, to identify administrative divisions of the route length for which separate estimates of costs will be maintained.

The route studied may be one already existing or one newly constructed at the beginning of the analysis period. If it is an existing route, traffic loading histories and the condition of the pavement surface at the start of the analysis are provided by the manager in the pavement descriptions. If it is a new route, managers may input the costs of initial construction, if it is desired that these costs be included in the economic analysis.

The facility to be analyzed within the EAROMAR system is termed a route - a length of highway comprising usually two or more roadways. Virtually any controlled-access divided highway, whether existing, newly-constructed, or planned, may be considered. There are no restrictions on the length of route one may define; this will depend solely on the scope of the premium pavement analysis one wishes to carry out. Moreover, routes may consist of two or more roadways representing physically distinct pavements and traffic flows - denoting, for instance, directional flows, express versus local lanes, or auto-only versus mixed traffic lanes. Each roadway is simulated independently in the EAROMAR analysis, and separate cost and performance results can be obtained for individual roadways.

Distance along a route within the EAROMAR analysis is denoted by a milepost system assumed to run along the imaginary centerline of the route. Locations on all roadways are tied to this single system; thus, variations in the lengths of component roadways due to minor differences in their respective geometrics are ignored. However, roadways need not all start and end at the same mileposts, nor need they be of the same length. (As we will show later, they also do not have to be constructed at the same time.) These features make it possible to represent changing route configurations over its length, in terms of the number of roadways (and, hence, total number of lanes) available to serve traffic.

Roadways

Each roadway within the route is identified by a name and described in terms of its length and location, direction of traffic flow, capacity, horizontal and vertical alignment, and pavement characteristics. Roadways may differ from one another in any or all of these characteristics, even at the same milepost. Moreover, the capacity, horizontal and vertical alignment, and pavement parameters associated with a roadway may vary over its length independently of one another. More detailed explanations of each roadway characteristic follow in the sections below.

NAME, LENGTH, AND DIRECTION OF FLOW

The roadway name serves to identify it, both to enable managers to associate specific information and policies with individual roadways, and to label analysis results such as roadway traffic flows, costs, pavement performance, and the like. The names assigned to roadways are arbitrary, and depend upon the inclination of the manager conducting the analysis. Normally, however, it is good practice to label roadways according to their commonly understood traffic service; e.g., "Northbound" or "Southbound," "Inbound" or "Outbound," or for circumferential highways, "Clockwise" or "Counterclockwise."

Roadway length and location are denoted by a beginning and an ending milepost, measured by the route milepost system described earlier. The direction of traffic flow is specified in relation to the milepost convention, as proceeding either "upstation" or "downstation." It is assumed that the direction of flow applies to the entire roadway throughout a 24-hour day. Thus, roadways with counter-flow lanes or reversible traffic flows are not considered within the analysis.

CAPACITY

The treatment of capacity within EAROMAR follows relationships presented in the Highway Capacity Manual (3), and is discussed in detail in Chapter 5. Data provided in the capacity block are used primarily in support of these capacity calculations, but are also needed for pavement deterioration and maintenance computations involving, for example, pavement surface area.

Data on roadway lanes and shoulders that must be provided by the user include:

1. The number of travel lanes, and the lane width in feet;

2. The widths of the right and the left shoulders respectively;

3. The design lane factor; and

4. A measure of the practical lane flow or impedance to flow.

The first two items are self-explanatory. The design lane factor, a decimal number ranging from zero to one, is used in estimating rates of pavement deterioration. It specifies the percentage of total traffic using the most heavily travelled roadway lane, and thus is a function of the lane distribution observed on the roadway. Finally, data on practical flow values are provided either directly by the average vehicles per hour per lane observed, or indirectly by a decimal factor (from zero to one) accounting for both lane width and side friction corrections to ideal flow values. (Refer to Chapter 5.)

Any of these capacity characteristics may be varied arbitrarily along the roadway length. Where different characteristics begin (e.g. a change in the number of lanes, or in side friction characteristics), the user would simply specify the milepost location of the change, followed by the new capacity value(s).

HORIZONTAL AND VERTICAL ALIGNMENT

Specification of the horizontal alignment encompasses the following information:

1. Horizontal curvature, expressed optionally by the degree of curvature or by the radius of the curve in feet;

2. The design speed of the roadway, in miles per hour;

3. The speed limit enforced on the roadway, in miles per hour; and

4. The accident rate, in occurrences per million vehicle miles miles travelled (MVMT).

The vertical alignment includes simply the slope of the longitudinal profile, in percent positive or negative.

Geometric characteristics may likewise be varied arbitrarily over roadway length. However, judgment should be exercised in maintaining a reasonable ratio of the length interval between changes in geometry to the total route length under study. Geometric characteristics are appled in assessing user costs on each roadway section* as compared to those on level tangent sections of otherwise identical characteristics. (Refer to Chapter 6.) If a route of some length is being analyzed, then a very detailed description of each geometric change may lead to a large number of user cost and other calculations (one at each change in geometry), rendering the simulation inefficient for the additional information gained. In these situations it may be advisable to consolidate many small sections into a larger section of equivalent total length and representing a composite of the contributing characteristics. Another approach is to model all roadways as level tangent sections throughout. This should provide an adequate first approximation when examining a broad range of alternatives. As one narrows the set of feasible solutions, more detailed geometric descriptions may be provided if necessary.

PAVEMENT

Managers may define three types of pavement structures with EAROMAR: flexible (asphalt concrete surface), rigid (portland cement concrete surface), and composite (asphalt concrete surface over portland cement concrete base). The procedure to describe roadway pavements is versatile enough to accommodate the unique designs sometimes used for premium pavements. The characteristics of the surface layer are specified first, followed by descriptions of each succeeding layer up to and including the subgrade. Any member of layers may be defined, with no restrictions on the type or function of each layer. This makes it possible, for example, to include specialized layers within the pavement structure (such as for crack relief or drainage), or to develop nonstandard designs for premium pavements.

Information required for the surface layer is as follows:

1. The type of structure (flexible, rigid or composite);

2. Surface conditions present at the start of the analysis period (in terms of the Present Serviceability Index PSI, individual damage components discussed in Chapter 3, and traffic loading history);

3. Surface thickness, in inches;

*See Note 1 at the end of this chapter regarding definition of roadway sections.

4. The AASHTO layer coefficient, as developed in (4);

5. The elastic moduli of the surface mix, and the stiffness of the bitumen, for flexible and composite pavements;

6. The elastic modulus and the rupture modulus of the portland cement concrete, for rigid pavements;

7. The thermal coefficient of expansion, and the susceptibility of the aggregate to blowups, for rigid pavements; and

8. The type and spacing of joints, for rigid pavements.

For each succeeding layer in the pavement structure (excluding the subgrade) the following information must be provided:

1. The name of the layer assigned by the manager (used for identification purposes only);

2. Layer thickness, in inches;

3. The AASHTO coefficient of the layer material; and

4. The elastic modulus of the layer material.

Data on subgrade conditions include the following:

1. The subgrade strength, expressed in terms of its static CBR, its elastic modulus in kips per square inch), its AASHTO "S" value, or its "k" value (in pounds per cubic inch); and

2. Drainage characteristics, expressed as "Good," "Fair," or "Poor." The technical interpretations of these terms will be given in Chapter 3.

Many items in the above lists are self-explanatory or familiar to pavement designers and engineers. Others, such as aggregate susceptibility to blowups, are specialized technical parameters related to the prediction of pavement damage, and will be discussed in detail in Chapter 3. Following we present simply some comments on pavement descriptions in general within EAROMAR.

The pavement structure may be varied arbitrarily over the roadway length to represent changes in pavement design or construction. Modifications may be made in the number and type of layers included, the ordering of the layers, layer thicknesses, materials properties, or subgrade characteristics. Pavement specifications may also differ among two or more roadways to reflect non-uniform subgrade conditions or unequal traffic loadings.

The descriptions of flexible and composite pavement surfaces contain several elastic moduli for the asphalt concrete, including its dynamic or complex modulus and its diametral resilient modulus. The reason for distinguishing among various definitions of modulus is that each of the flexible pavement damage models discussed in Chapter 3 employs a different measure of elastic response of the asphalt concrete. We have retained within EAROMAR the distinctions among moduli implied in the original derivations of these damage equations. Managers must therefore provide values for each type of elastic modulus, even if values of all elastic or dynamic moduli are considered to be equal for a given flexible or composite pavement.

Elastic moduli are also required for all underlying layers of the pavement. For granular bases this modulus is stress-dependent. Therefore the values provided by a manager should approximate the state of stress within which the base layer is expected to serve.

Subgrade strength may be expressed in one of the following four ways: static CBR, elastic modulus (in kips per square inch), AASHTO soil-support or "S" value, or modulus of foundation support or "k" value (pounds per cubic inch). For convenience a manager need input only one of these four measures; equivalent values of the other three will be computed automatically, using conversion equations described in Chapter 3. (All four measures of subgrade strength are required internally within EAROMAR for use by different pavement damage models.)

Materials properties of pavement surfaces, underlying layers, and subgrade may be input either as constant throughout the year, or as varying by season. In the latter case such variations reflect the effects of temperature and moisture fluctuations on pavement materials properties.* Moreover, it is not necessary that all pavement characteristics represented in EAROMAR be treated in the same way at the same time. For example, one may specify the elastic moduli of the pavement surface to vary by season (reflecting temperature effects), but input base layer moduli and subgrade CBR as constant throughout the year. Or, one may hold surface and base values constant (reflecting annual averages), but vary subgrade properties seasonally to reflect periods of high groundwater. In each case a manager should consider what seasonal effects are prevalent in his or her area, and what data are available on changes in materials properties throughout the year. (Other time-dependent changes in pavement characteristics and how they are treated within EAROMAR will be discussed in Chapter 3.)

*Pavement materials properties are not now represented as direct functions of temperature (or moisture) within EAROMAR. However, there is no reason that they cannot be in the future, since temperature and moisture data are provided for each season defined within the year, as will be discussed shortly.

Administrative Areas

The route under study may traverse several highway administrative jurisdictions, such as foreman repair sections or maintenance districts. Conditions within each jurisdiction may be important to a premium pavement analysis, particularly as they affect the ability to provide needed maintenance. For example, individual foreman sections or districts may exhibit different levels of labor, equipment, or materials availability to repair highway pavement now or in the future.

Managers may simulate these issues of maintenance supply through what are termed administrative areas (or more simply, areas) within EAROMAR. Areas define an arbitrary but contiguous set of divisions of the route length. Each area is denoted by a name (assigned by the user) and beginning and ending mileposts. By implication an area also divides all component roadways into segments bounded by the two mileposts; therefore, in effect areas establish subsets of the total route maintenance responsibility. The relationships among areas, roadways, and the route milepost system are illustrated in Figure 1.

All data on maintenance scheduling, technology, unit costs, and resource availability are provided within EAROMAR by area, as we shall see in Chapter 4. Thus, areas can be used to reflect the local impacts of maintenance budget constraints, the competition for scare maintenance resources among this and other routes in the network, and differences in maintenance practices among foremen or district engineers. Furthermore, a separate series of output reports on highway performance and costs is available by area, to assist a manager in pinpointing what segments of the route under study might be the most favorable condidates for premium pavements.

Environmental Conditions

Moisture and temperature patterns throughout the year influence both the types and the respective rates of damage sustained in pavements. To account for these effects within EAROMAR, users may describe environmental conditions affecting the route in question. Environmental variables capture broad regional influences, rather than localized variations along the route length. Therefore only one set of environemental information is required of the manager; this description applies uniformly to all roadway sections, and is not modified during the analysis.

The EAROMAR system permits two independent methods of describing the route environment: (1) division of the year into seasons characterized by individual temperature and moisture averages; and (2) specification of the AASHTO regional factor. Each of these conventions is discussed further below.

SEASONS

Seasons within EAROMAR are arbitrary divisions of the year, among

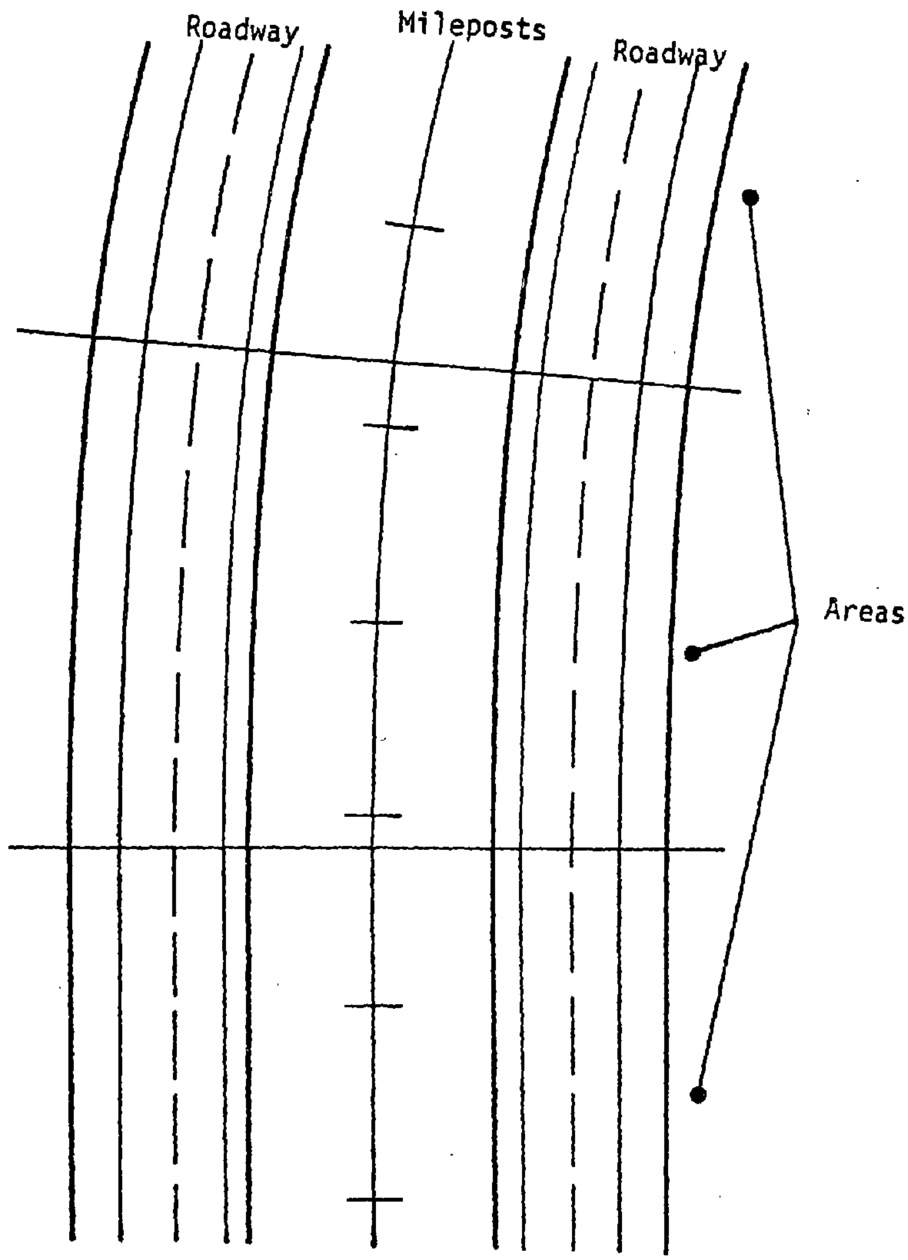

FIGURE 1. ROADWAYS, AREAS AND MILEPOSTS OF A ROUTE

which one may simulate periodic fluctuations in temperature and moisture levels. Any number of seasons between one and twelve may be defined; the length of each season is likewise arbitrary, so long as the sum of the durations of all seasons equals twelve months. For each season one must specify the following information:

1. The name of the season (any unique identifier, to be selected by the user);

2. The duration of the season, in months;

3. The average seasonal temperature, in degrees Fahrenheit; and

4. The average seasonal moisture level, in inches total rainfall.

Several examples of different seasonal descriptions are shown in Table 6. A manager has complete independence (save for the guidelines above) in deciding how seasons are to be structured in a typical year. Thus, the cases in Table 6 are for illustration only, and any other examples satisfying the conditions above could just as readily have been chosen.

Example A in Table 6 illustrates a fairly conventional division. It is based simply on the astronomical definitions of four seasons typical in temperate zones. Sometimes, however, models of pavement damage may be more appropriately based on subdivisions of individual seasons or upon departures from the astronomical calendar. Distinctions between early and late fall, or between the wet and dry periods of spring, may be useful as indicated in Example B. Moving on to Example C, we see that with a maximum of twelve seasons permitted, a user may find it convenient to divide the year into months particularly if monthly temperature and moisture data are readily available. Finally, if no seasonal variations are desired, the manager may simply default to an annual simulation, as shown in Example D.

Whatever seasons are defined by a user become an integral part of EAROMAR simulation over time. In addition to pavement damage histories, both traffic volume projections and maintenance scheduling may be treated as seasonally dependent (as will be explained in Chapters 4 and 5). Furthermore, output reports on highway performance and costs can be stratified by season of the year. Thus, beyond their role in organizing environmental data, seasons form a structural component of the analysis, and will be referred to many times in subsequent chapters.

REGIONAL FACTOR

Several of the pavement deterioration relationships to be presented in Chapter 3 are based upon results of the AASHO Road Test (5). The AASHTO equations do not include temperature and moisture conditions as independent variables, but rather incorporate these (and other environmental effects) within a weighting function termed a regional factor.

TABLE 6

EXAMPLES OF DEFINITIONS OF SEASONS

Season	Duration months	Mean Temp °F	Mean Seasonal Moisture inches rainfall
EXAMPLE A			
Spring	3	55	18
Summer	3	75	13
Fall	3	60	9
Winter	3	30	11
EXAMPLE B			
Wet Spring	1	39	9.0
Dry Spring	2	48	9.7
Summer	3	75	7.2
Early Fall	2	70	8.5
Late Fall	2	53	4.5
Winter	2	37	7.4
EXAMPLE C			
January	1	36	2.1
February	1	37	5.3
March	1	45	4.2
April	1	56	5.7
May	1	66	4.5
June	1	75	4.0
July	1	79	2.7
August	1	77	1.2
September	1	71	3.7
October	1	60	4.6
November	1	48	3.3
December	1	37	1.2
EXAMPLE D			
Annual	12	65	4.5

The regional factor is therefore identified as a separate environmental input to EAROMAR.

The regional factor is an empirically derived value to correlate the AASHO Road Test results with expected pavement performance in other regions of the country, where moisture, temperature, drainage, and frost conditions likely differ from those observed at the Road Test site. The reional factor is measured over a scale of 0.2 to 5.0, with the following guideline values:

1. 0.2 to 1.0 for frozen roadbeds;

2. 0.3 to 1.5 for dry roadbeds; and

3. 4.0 to 5.0 for saturated and thawing roadbeds. (5)

The regional factor at the Road Test site is defined to be 1.0. Where roadbed conditions are on actual pavement vary throughout the year, an annual average of the above guideline values must be used.

It should be noted that the regional factor does not substitute for the seasonal temperature and moisture data described above. Rather, the two sets of information are used in a complementary way, wtih the regional factor applicable to AASHTO pavement relationships, and the seasonal data to more mechanistic formulations. Nevertheless, one should take care that the value of the regional factor is consistent with the seasonal information provided.

Construction Projects

Projects are undertaken over time to restore or upgrade the characteristics of the route, and thus their implementation should correctly be discussed in section 2.3. However, since the data which a manager must provide in the projects area corresponds to those input under the initial road description, it is helpful to consider their specification here.

Projects may be applied to accomplish the following changes or improvements in route configuration:

1. To create a new roadway within the route;

2. To extend the length of an existing roadway;

3. To change the capacity, horizontal alignment, vertical profile, or pavement cross-section of an existing roadway; or

4. To overlay an existing pavement.

Items 1 through 3 are considered to be construction activities within EAROMAR, while item 4, an overlay, is treated separately.

SUBSEQUENT ROADWAY CONSTRUCTION OR RECONSTRUCTION

Project Descriptions. Construction projects within EAROMAR may be used to modify any part of the route description explained earlier. For example, new roadways may be added to simulate a major future expansion of capacity, say, from two roadways to four. Or, where a route comprises multiple roadways (of perhaps unequal length), certain roadways may be extended at a future date under EAROMAR projects. Finally, on a given roadway such projects may be used to simulate the addition of new lanes or shoulders; road reconstruction to upgrade horizontal or vertical alignment; or rebuilding (as opposed to overlaying) the pavement structure.

In each case the description of the project scope replicates that for initial roadway conditions described earlier. For new roadways or extensions to existing roadways, data on name, length, and (for new roadways only) direction of traffic flow; roadway capacity; horizontal and vertical alignment; and pavement design must be provided by the user exactly as for initial roadway conditions. The conventions discussed earlier for varying these characteristics along roadway length (with subsequent creation of roadway sections) likewise apply. For modifications to existing roadways only the block containing the affected data needs to be input. For example, if a lane were to be added along a length of roadway, only the capacity block would have to be entered under beginning and ending mileposts delimiting the roadway segment of interest.

Timing. The timing of construction work is important for two reasons. First, constraints on the season in which work starts may reflect technical requirements regarding temperature or moisture conditions necessary to successful project accomplishment. Second, both the start time and the duration of proposed work influence the degree of workzone interference with the normal traffic stream. It may be possible to lessen traffic disruption by adjusting the timing of construction, as well as workzone configuration.

Within EAROMAR project timing is controlled by two parameters specified by the user: (1) the season(s) during the year in which project work may commence; and (2) the proposed duration of work, in months.* Any number of seasons may be listed as the potential starting time of work, so long as each season has been corretly defined in the "seasons" block earlier. Where a number of contiguous seasons are referred to (e.g. spring, summer, and fall) the "to" keyword convention may be used (e.g. spring to fall).

Project duration should encompass the entire period the workzone is to occupy the road. For example, assume that work commences, say, in spring, and continues actively through summer, fall, and the subsequent spring and summer. Very little work can be accomplished in winter, but

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* A third aspect of project scheduling is the year in which work will commence. This information is contained in the strategy specifications, and is therefore discussed in section 2.3.

(since work is not finished) the barricades delimiting the workzone will remain standing. If we assume the duration of each season to be three months, the total project duration in this case is 18 months.

Roadway Closure. Three types of closures for construction projects are possible within EAROMAR at the command of the user:

1. Lane restrictions, in which one or more lanes on a given roadway are closed for repair work, with traffic constrained to the remaining lanes;

2. Crossovers, in which an entire roadway is closed for work, with traffic diverted to another roadway; and

3. Detours, in which an entire roadway is closed for work, with traffic diverted to a temporary bypass not part of the route in question.

These closure types duplicate those used in the simulation of maintenance within EAROMAR; therefore, illustration of their use and explanations of technical details will be presented in Chapter 4, with the analytic treatmetn of closures following in Chapter 5.

There are, however, two characteristics of road closures unique to the treatment of projects (in comparison with maintenance). First, closures for projects are assumed to occupy the roadway 24 hours per day until all project work is completed. (By comparison, with maintenance activities there is the option to confine roadway occupancies to those periods when the work crew is actually on site. See section 4.3.) Thus, in the example for project duration earlier, EAROMAR will simulate a closure zone present during all hours of the day throughout the 18-month duration of the project. This relative permanence of project closure zones, and the significant delays that may result in traffic movement are felt to represent realistic operational aspects of highway construction.

Second, the physical limits of the closure zone may extend any distance beyond the length of the roadway actually to be reconstructed. (This is also different from the treatment of maintenance, wherein closure zones essentially coincide with the maintenance workzone itself.) This feature permits managers to exercise a considerable degree of control over the placement of the road occupancy. For example, ends of the closure zone may be located at the mileposts of interchanges at which traffic will be diverted from and returned to the roadway. Again, the objective is to model the impacts of highway construction on traffic as realistically as possible.

Project Costs. The costs of roadway construction are provided by the user for each project. These costs should be consistent with (1) the level of effort required to produce the end product, whether construction of a new or extended roadway, or changes in roadway capacity, alignment, or pavement; (2) the technology of construction envisioned, and (3) project duration and roadway closure configuration to be used. This simplified costing approach has been adopted since our interests in EAROMAR do not require a simulation of the construction process itself. The premium pavement analysis considers only the effects of construction activity on the traffic stream, incorporation of construction output within an updated road description, and the costs of accomplishing this improvement.

Costs may be provided by the manager in one or more of the following categories:

1. A lump sum cost to cover any or all components of the project;

2. A unit cost in dollars per centerline mile to cover any or all components of the project; and

3. A lump sum cost to cover erection and dismantling of the roadway closure, construction of temporary bypass lanes, setting up warning messages and detour signs, and the like.

The division of total project costs between items 1 and 2 is completely at the discretion of the user, depending upon the scope of project work and to what extent different project mileposts are to be investigated as strategic alternatives (section 2.3). For example, if the scope of (re)construction is well-defined over a specific length of roadway, it may be appropriate to provide all project costs as a lump sum total. On the other hand, we shall see in section 2.3 that a project may be executed over different roadway lengths as governed by the strategy specifications. In this case it may suffice to provide project costs on a unit mile basis. Finally, in the general case total project costs will exhibit both fixed and variable components. Users may therefore provide both lump sum and per-mile figures respectively.

Item 3 above is self-explanatory. Costs associated with the roadway occupancy are segregated from the costs of actual construction work to allow managers to substitute different closure options if required, and to cost each one separately.

OVERLAYS

Overlays form a special subset of the construction project activities treated within EAROMAR. The purpose of an overlay is to restore the riding surface or to increase pavement strength by adding a new surface layer. Within the EAROMAR analysis, overlays are simulated by updating roadway pavement information to incorporate the structural contribution of the new layer, as determined by its thickness and materials properties.

Overlays may consist of either flexible or and rigid pavement,* and may be

---

* A rigid overlay on a flexible or rigid pavement results in a rigid pavement. A flexible overlay atop a flexible pavement results in a flexible pavement. A flexible overlay atop a rigid pavement results in a composite pavement under the conventions adopted within EAROMAR. However, the simulation of rigid overlays is discouraged; see section 3.5.

any thickness or length. (Of course, the overlay thickness and materials properties specified should be realistic in light of available pavement technology and the limitations of construction practice.)

Data to characterize the overlay are analogous to those already discussed for initial roadway pavement conditions and for construction projects. Overlay thickness, materials properties (layer coefficient, modulus, joint spacing, and so on as appropriate to a flexible or a rigid material), and the resulting improvement in surface conditions (in terms of PSI and values of individual damage components) are provided exactly as for a pavement surface layer described in initial roadway construction. Project timing, roadway closures, and costs incident to placing an overlay are input as for construction projects.

2.3. SPECIFICATION OF STRATEGIES

Several problem elements in Table 5 are dynamic over both space and time. Values of these parameters may vary simultaneously among roadways and along roadway length, while depending as well upon the analysis year. Some of these parameters are under the direct control of the user (we may refer to these as policy or control variables), and include pavement maintenance policies, maintenance management issues, and the nature and timing of construction investments. Other aspects of the problem are not subject to direct management control, but nevertheless significantly affect the outcome of the analysis. These condition or state variables include future traffic volume and composition, assumed rates of damage induced in pavements, and economic scenarios.*

Since system conditions are subject to uncertainty over time, and since policy specifications are modified as a result of management decisions, the effects of changes in both policy and state variables must be investigated within premium pavement options. Furthermore, a number of options must be analyzed to determine whether a warrant for premium pavements exists. The capabilities required to organize policy and condition information, to apply this information correctly, and to manipulate it through a series of analyses are provided in EAROMAR through the definition of strategies. A strategy designates what values will be assumed by each of the policy and state variables, and how these values will change over both roadway location and time. The format of strategy definition permits managers to readily adjust the values and the location and time dimensions of policy and state variables, so that successive strategies may be readily defined.

The following sections provide more detailed information on the formulation and use of strategies within EAROMAR. First we describe the construction of strategies - the type of information included, and assignment of data to roadway locations and time periods. We then illustrate

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*Strategies within EAROMAR comprise changes in both policy variables and state variables. Although state variables are not subject to management control, managers must still account for them in their analyses; and in particular, managers must project the uncertain future values of state variables such as traffic demand, pavement damage, and economic discount and inflation rates. On the other hand, to say that policy variables are under direct management control does not mean that managers have complete freedom in determining respective values. For example, maintenance standards, maintenance scheduling, and resource allocations are often strongly influenced by local statutes, budget constraints, safety guidelines, civil service regulations and the like. Nevertheless, within these constraints highway administrators have some discretion over such policy areas, and it is this discretion in each area that is treated as the policy variable. Furthermore, note that the inclusion of both state and policy variables within the definition of strategies promotes a very versatile approach, able to represent many different situation existing throughout the country.

how successive strategies may be defined through a series of EAROMAR simulations to obtain the range of cost results needed to justify premium pavements.

Structure of Strategy Specifications

ACCESS OF INFORMATION

If one assembled all the data associated with the several policy and state variables, and particularly with the separate assignments of different values over roadway locations and time, the resulting collection would be sizable and somewhat difficult to manipulate. For example, the modification of any of this information to test different strategies might require considerable editing to update values or to reassign data to different roadway sections and analysis years. The approach adopted within EAROMAR is therefore not to include the body of policy and state variables within the strategy specifications, but rather to access this information in blocks and assign blocks to specific roadway sections and specific analysis periods simultaneously. This method requires only very succinct statements within the strategy formulations, statements which can be edited easily and rapidly to define new strategies.

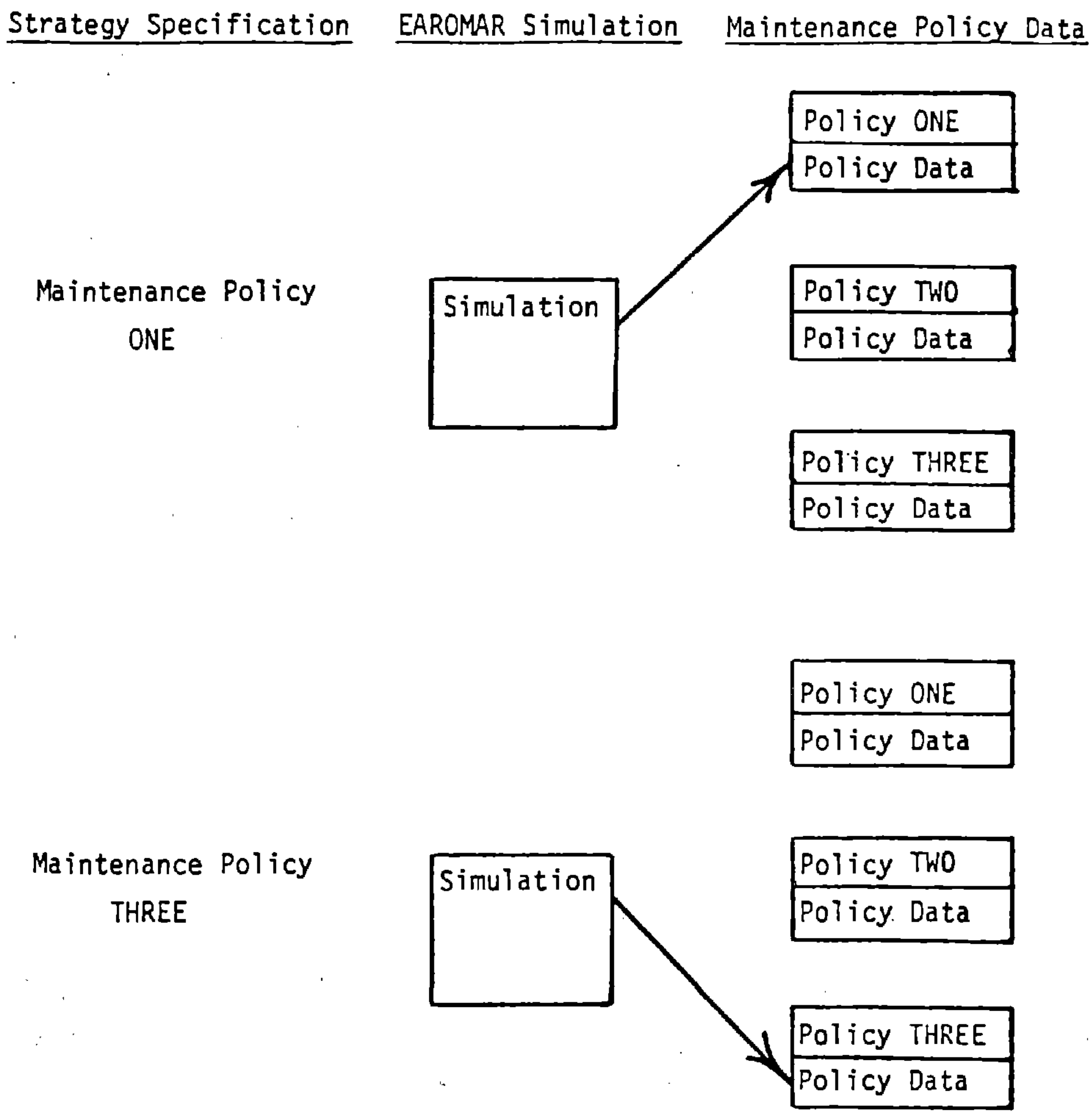
To see how two systems of accessing informations blocks works, consider the example shown in Figure 2 . A particular policy variable, maintenance policy, is chosen for illustration; however, all other policy and state variables would be treated in like fashion.

Different maintenance policies must be defined prior to formulating the strategy. (The information required to do this is explained in Chapter 4.) Each different policy, comprising a different set of maintenance data, is given a unique name by the user to identify it to the EAROMAR system. All maintenance policies relevant to the analysis must be defined at this point; i.e. if a policy is to be used at any roadway location, at any time during the analysis period, the data to define this policy must be organized within a named block prior to strategy specification. If the same maintenance policy applies to several roadway locations or to multiple time periods, the policy block needs to be entered only once.

Within the strategy specifications, then, it is necessary only to refer to the name of the maintenance policy to invoke the data contained therein. As shown in the first example in Figure 2, identification of maintenance policy ONE causes the EAROMAR simulation to execute all data associated with that maintenance policy. If, in some subsequent simulation, it is desired to test a different policy, the manager merely invokes the name of this new policy instead. The second example in Figure 2 shows maintenance policy THREE replacing policy ONE in the simulation.

FIGURE 2

ACCESSING BLOCKS OF
INFORMATION UNDER STRATEGIES



ASSIGNMENT OF INFORMATION

The remaining issue is the assignment of information blocks to appropriate roadway locations and analysis periods. This is also accomplished within the strategy statement. In fact, the general form of a strategy specification is as shown in Figure 3. The block name identifies a particular set of policy or condition data as discussed above. Roadway lengths to which these data apply are identified either by roadway name and milepost or by area name, depending upon the particular policy or state variable. Finally, the designation of time periods follows a rule of succession: strategy specification remains valid over a given roadway length until it is superceded by another specification. There are several options to establish the time at which policy or condition data are invoked:

1. By stating directly the analysis year in which the data are to be invoked;

2. By stating the name of a construction data project at whose completion the data will be invoked; and

3. If the policy variable is itself a construction project, by stating levels of road performance at which the project should be undertaken. Relevant road performance measures include the average daily traffic, the pavement PSI, pavement damage components, or the time since the project was last completed.

If we again consider maintenance policies as examples of how policy and state variables are treated within strategies, we can develop the following statements as illustrations:

```
POLICY ONE
     MILEPOST NORTHBOUND  17.5  26.2
     START YEAR (1985)                                    (1)

POLICY TWO
     MILEPOST SOUTHBOUND  0.0  10.7
     START PROJECT  OVRLY-13                              (2)
```

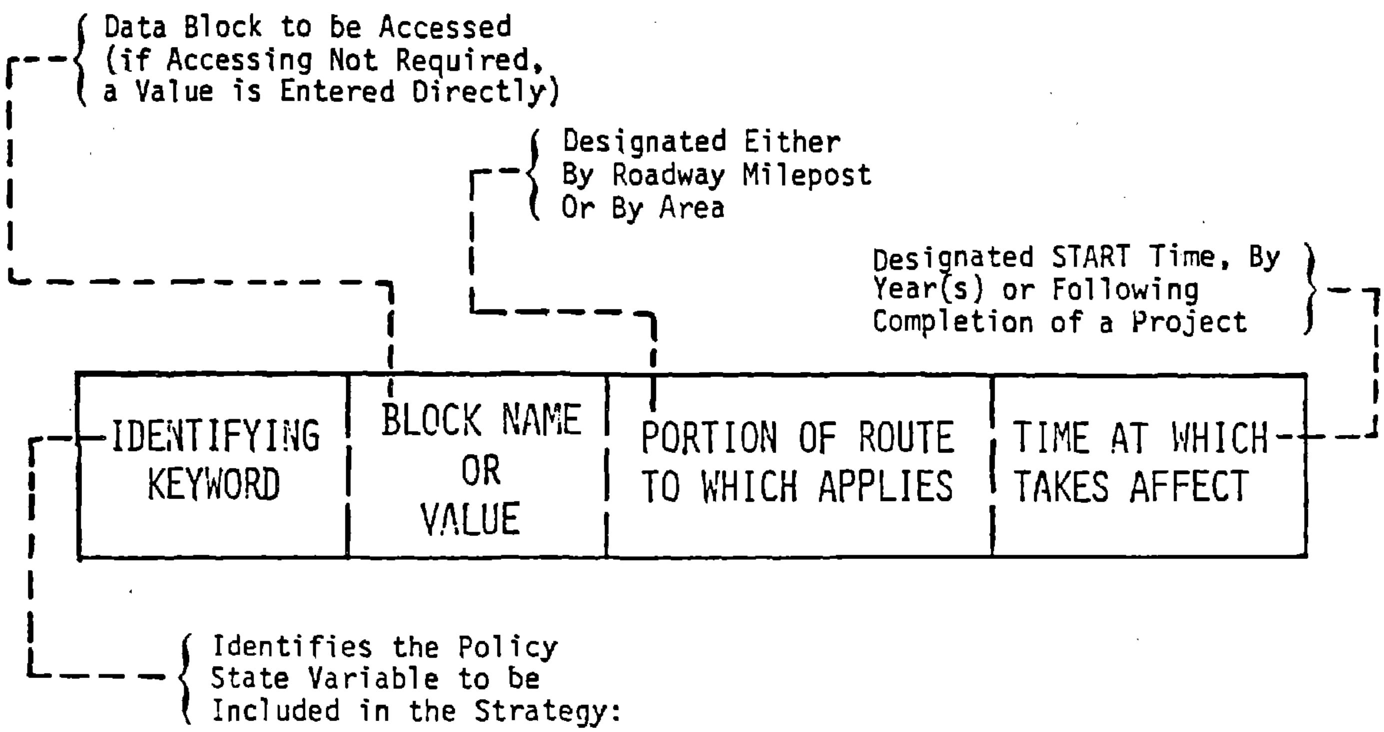
In the first example all data associated with maintenance policy ONE will be assigned to the Northbound roadway between mileposts 17.5 and 26.2, beginning in 1985. In the second example the Southbound roadway between mileposts 0.0 and 10.7 will be assigned maintenance policy TWO following completion of the pavement overlay, project OVRLY-13.

The examples above illustrate relatively simple, but nevertheless useful, constructions of strategy statements. However, it is also possible to develop more complex statements using what are called Boolean expressions. For example, consider the following:

FIGURE 3

GENERAL FORM OF THE

STRATEGY SPECIFICATION



- . PROJECT - construction, reconstruction or overlay project
- . AADT - traffic volume, average arrival vehicles per day
- . GROWTH - annual traffic growths or growth rate
- . TRAFFIC - traffic sets containing descriptions of traffic composition
- . POLICY - maintenance policies
- . DAMAGE - projected rates of pavement damage
- . SCHEDULING - maintenance technology, unit cost, and scheduling information
- . RESOURCES - maintenance resources available
- . SCENARIO - economic scenarios

```
POLICY THREE
     MILEPOST  30.0  42.7
     START YEAR  (1990) OR PROJECT RECONST                (3)
```

In this case an option on starting maintenance policy THREE is desired. The policy will be invoked either during year 1990, or following completion of the construction project RECONST, whichever comes first. In general, strategy specifications can incorporate any Boolean expression involving years or projects, including those encompassing multiple years or multiple projects (e.g. YEAR (1985 1990) AND (PROJECT OVRLY-13 OR PROJECT RECONST)). However, the effective use of Boolean expressions requires an understanding of their construction and syntax. At this point we must therefore defer more detailed explanations of Boolean expressions until Chapter 4, where they will be thoroughly reviewed in relation to the definition of maintenance policies. (Also note in the example above that mileposts are not qualified by roadway name. In this case the specification applies to the lengths of all roadways between the two mileposts.)

A further point concerns the special treatment of projects alluded to in item 3 above. The execution of projects can be controlled by specifying start years or other projects preceding the one in question, as described above for any other strategy specification. However, in addition to these controls projects can also be invoked in response to the condition of the highway itself, with relevant parameters again organized within Boolean expressions. Consider the following example:

```
PROJECT  CONST-ONE
   MILEPOST INBOUND  0.0  7.5
   START (AADT GT. 7000) OR (PSI LT 3.0)
       OR (INTERVAL GE 7)                                 (4)
```

First, the mileposts here do not denote the extent of project work -- recall from section 2.2 that limits of work are established by mileposts contained in the project description itself. Rather, these mileposts indicate the portion of roadway over which the Boolean expression for start time is to be evaluated.*

The interpretation of the Boolean expression itself is as follows. The project CONST-ONE will commence when any of the following three conditions are satisfied between mileposts 0.0 and 7.5 on roadway INBOUND:

1. The average daily traffic volume exceeds 70,000 vehicles per day;

---

* Of course, the mileposts for evaluation of the START expression may coincide with the limits of the project, but it is not always necessary or desirable to do this. For example, if the construction of a new roadway is contingent upon traffic levels observed on an existing roadway, then the two milepost specifications must differ.

2. The average pavement PSI falls below 3.0; or

3. The time since the last performance of project CONST-ONE is at least seven years.*

Again, other conditions could have been included by manipulation of the Boolean expression. The flexibility of such expressions will be explored in Chapter 4.

EXAMPLE

An example of a complete strategy specification is given in Figure 4. Information on initial traffic levels (AADT) and traffic growth rates (GROWTH) are straightforward and included directly in the strategy specifications. Other data, however, are accessed by name, and must be contained in data blocks as described earlier. For example, for the project PROJ-1 there must exist a project description identified by that name and containing the type of information outlined in section 2.2. Similarly, SET-1 refers to descriptions of traffic composition which will be explained in Chapter 5; OPTION-1, PROJECTION-1, and DECISION-1, to maintenance scheduling options, projections of resource availability, and maintenance policy decisions respectively, all of which are explained in Chapter 4; PREMIUM and MODERATE-RATE, to sets of pavement damage rates, to be covered in Chapter 3; and MOD-INFLATION and HIGH-INFLATION, to economic scenarios, which will be discussed in section 2.5.

Use of Strategies

Several strategies may be tested against different route descriptions developed in section 2.2 to establish what combinations of traffic flows, maintenance and reconstruction policies, maintenance scheduling and resource situations, and economic factors justify premium pavements. The recommended method of approach is illustrated in Figure 5.

Route characteristics represent the initial conditions of the problem. For purposes of a premium pavement analysis, one of the most important elements of the highway description is the pavement design itself. Pavement layer thicknesses and materials properties, and the variation in these characteristics along the highway length, embody decisions on the extend to which conventional vs. premium pavements are to be employed. The relative impacts of different premium and conventional pavements may be evaluated by including the respective pavement designs within separate descriptions of the highway facility (denoted schematically by I and II in Figure 5), and analyzing these different pavement configurations within a series of EAROMAR simulations. Although pavement design has been chosen here as the element of route description of perhaps greatest interest, in fact any other geometric, operational, or environmental

---

* This type of condition, controlled by the INTERVAL keyword, is intended for use with overlay projects.

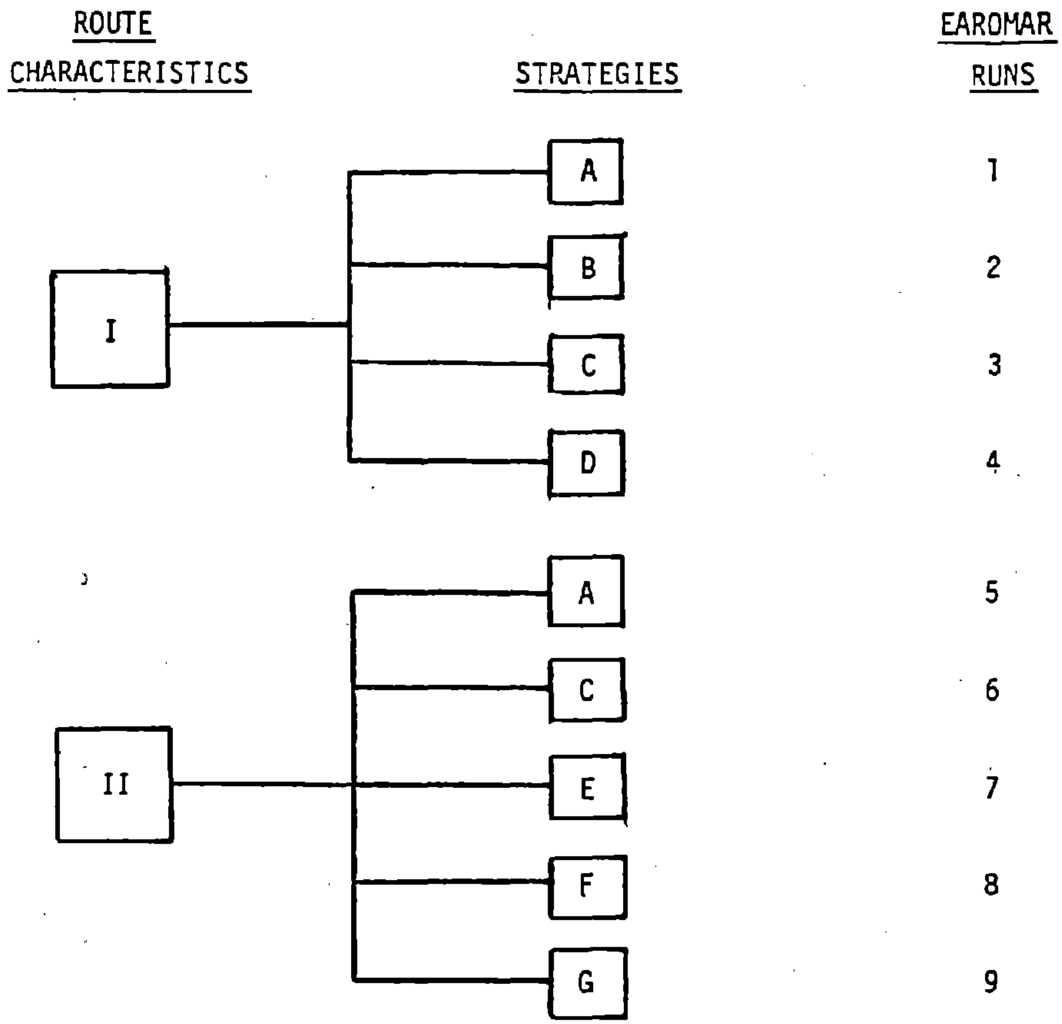
FIGURE 4

EXAMPLE OF STRATEGY SPECIFICATIONS

```
AADT 30000 MILEPOST 0 10 START YEAR (1980)

GROWTH GEOMETRIC 2  MILEPOST 0 10 START YEAR (1980)
GROWTH LINEAR 100  MILEPOST 0 10 START YEAR (1985)

TRAFFIC SET-1  MILEPOST 0 10 START YEAR (1980)

PROJECT PROJ-1  MILEPOST 5 10 START PS1 LT 3.0 AND AADT GT 60000
SCHEDULING OPTION-1  AREA LEV-TAN-PREM START YEAR (1980)
SCHEDULING OPTION-1  AREA LEV-CUR-PREM START YEAR (1980)
SCHEDULING OPTION-1  AREA UP-TAN-PREM START YEAR (1980)
SCHEDULING OPTION-1  AREA DOWN-TAN-PREM START YEAR (1980)
SCHEDULING OPTION-1  AREA LEV-TAN-FLEX START YEAR (1980)
SCHEDULING OPTION-1  AREA LEV-TAN-RIGD START YEAR (1980)
SCHEDULING OPTION-1  AREA LEV-TAN-COMP START YEAR (1980)
SCHEDULING OPTION-1  AREA FLEX;HI-CAP START YEAR (1980)
SCHEDULING OPTION-1  AREA FLEX;LOW-CAP START YEAR (1980)

RESOURCES PROJECTION-1  AREA LEV-TAN-PREM START YEAR (1980)
RESOURCES PROJECTION-1  AREA LEV-CUR-PREM START YEAR (1980)
RESOURCES PROJECTION-1  AREA UP-TAN-PREM START YEAR (1980)
RESOURCES PROJECTION-1  AREA DOWN-TAN-PREM START YEAR (1980)
RESOURCES PROJECTION-1  AREA LEV-TAN-FLEX START YEAR (1980)
RESOURCES PROJECTION-1  AREA LEV-TAN-RIGD START YEAR (1980)
RESOURCES PROJECTION-1  AREA LEV-TAN-COMP START YEAR (1980)
RESOURCES PROJECTION-1  AREA FLEX;HI-CAP START YEAR (1980)
RESOURCES PROJECTION-1  AREA FLEX;LOW-CAP START YEAR (1980)

POLICY DECISION-1   MILEPOST 0 10

DAMAGE PREMIUM   MILEPOST 0 5
DAMAGE MODERATE-RATE MILEPOST 5 10

SCENARIO  MOD-INFLATION  HIGH-INFLATION
```

FIGURE 5

APPLICATION OF STRATEGIES
TO ROUTE CHARACTERISTICS



(Note: Designations of route characteristics by Roman numerals, and strategies by letters, are schematic representations for use in this figure, and are not employed in the use of EAROMAR) characteristic of the highway may be tested in a similar fashion.

Strategies encompass all time-dependent aspects of the problem. In developing different strategies a manager should assess the importantce of both policy and state or condition variables to the economic analysis. For example, policy variables encompassing pavement maintenance or rehabilitation actions constitute administrative alternatives to a premium pavement, and should be investigated in some depth. On the other hand, condition or state variables involving traffic projections, rates of pavement damage, and economic scenarios are subject to uncertainity, and managers may wish to bracket their extimates and gauge the sensitivity of the results.

Figure 5 shows several strategies tested under each route description. In general the different strategies should reflect adjustments in both policy and state variables, although the relative emphasis between the two must depend upon the situation at hand (e.g. the quality of data available, the number of feasible management options, and construction or maintenance cost constraints). Also, the strategies should be consistent with the route characteristics to which they correspond. For example, one should not have to assign a strategy including substantial overlay projects to route characteristics embodying a premium pavement. At the same time, the strategies must themselves be internally consistent; maintenance policies and project schedules should complement one another, and both should be realistic in light of projected traffic demands. Note in Figure 5 that strategies A and C are deemed to be appropriate to both route descritpions, while other strategies are applied selectively. There is no restriction on the number of strategies that may be examined under a given route description.

Each combination of route and strategy requires a separate EAROMAR simulation or run and generates separate cost and performance results as indicated in Figure 5 . If the route descriptions and strategies have been correctly approached, the results of all runs may be compared to determine whether premium pavements are economically justified throughout any portion or all of the route. To help the user understand how route descriptions and strategies influence cost results in a given run section 2.4 outlines the simulation process employed within EAROMAR, and illustrates the evaluation of cost totals.

2.4 OVERVIEW OF THE EAROMAR SIMULATION

In this section we present a conceptual outline of the EAROMAR analysis, tracing the general flow of the calculations leading to highway performance and cost results. Elements of the simulation are discussed in general terms consistent with the information presented under route characteristics and strategies earlier. The detailed analytical relationships used in predicting pavement damage, maintenance, traffic operations, and costs are explained Chapters 3 through 6.

Initialization

To begin the EAROMAR simulation requires a series of preparatory operations and checks that are referred to collectively as program initialization. A schematic of the initialization phase is shown in Figure 6 .

The primary task here is to process and validate information provided by the user. Route descriptions are assembled within an internal representation of the facility to be studied. At the same time, strategy specifications are evaluated to establish linkages to blocks of user-supplied data concerning project descriptions, maintenance policies and management information, pavement damage rates, traffic descriptions, and economic data. Many bookkeeping aspects of the analysis are resolved: for example, roadways are partitioned into sections; initial pavement conditions are established; traffic is distributed throughout the route; and economic and other indicators are set to base year values.

If processing of all information proceeds to completion without error, the simulation begins. An internal clock registers the first season of the first year. Roadways are analyzed in sequence; and within each roadway, each section is simulated individually. Potentially three types of simulations may be conducted at the section level, corresponding to performance of pavement maintenance, undertaking a construction project, and modeling traffic operations. These paths are denoted by A, B, and C respectively in Figure 6 .

Simulation of Maintenance

The simulation of maintenance is diagrammed in Figure 7 .

POLICIES AND WORKLOADS

Maintenance is treated within EAROMAR as a demand - responsive action. This means that maintenance requirements are not extrapolated from historical trends of past work performed, but rather are based directly upon the type and amount of pavement damage predicted. How much damage is to be repaired among the several maintenance activities is a management decision expressed through maintenance policies.

Quality standards within each policy specify the percentage of damage

FIGURE 6

START OF EAROMAR

SIMULATION

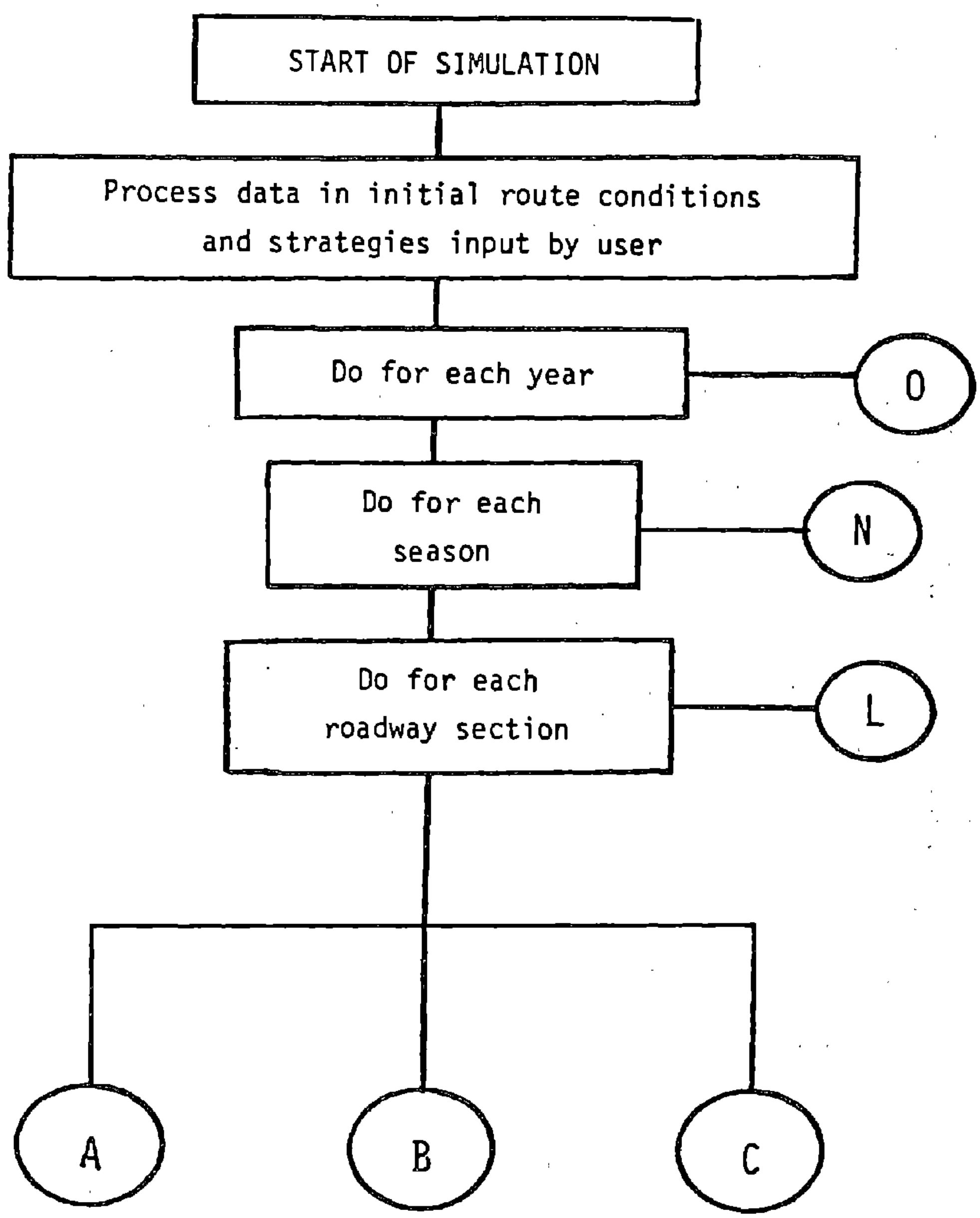
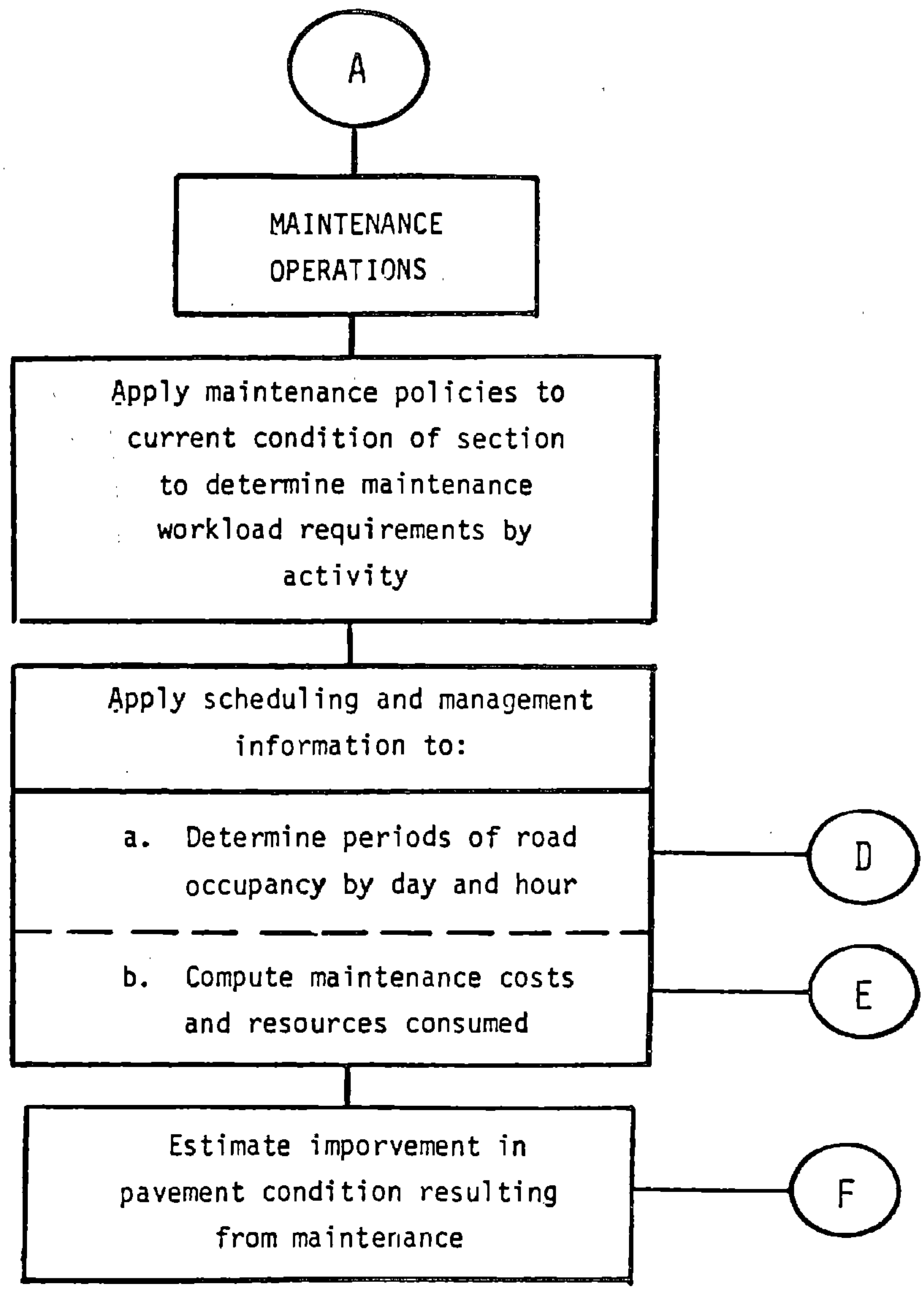


FIGURE 7

SIMULATION OF

MAINTENANCE OPERATIONS



to be repaired under each activity, and when maintenance is to be undertaken. The quantity of damage remedied under each activity defines a maintenance workload. The maintenance policy in effect for the given roadway section and year is determined from the strategy specifications. Development of maintenance policies is described in Chapter 4, section 4.2.

MAINTENANCE SCHEDULING AND MANAGEMENT

Application of maintenance quality standards to the total accumulated pavement damage results in a total maintenance workload for each activity in the current season. The maintenance workload is then used as the basis for scheduling work and estimating seasonal maintenance costs.

Scheduling. Maintenance costs depend to some extent upon the time of day in which work is carried out. Nighttime or weekend work typically involves premium labor time. Nevertheless, such additional costs may be warranted to avoid excessive congestion and hazards during more heavily traveled periods.

Maintenance work may therefore be scheduled within the EAROMAR analysis in the following ways:

- By seasonal distributions -- the allowable seasons during the year when maintenance may be performed for each activity; and

- By daily and hourly distributions -- the allowable hours during weekdays and weekends when maintenance may be performed for each activity.

Descriptions of maintenance technologies to be employed for each activity (in terms of labor, equipment and materials resources required and associated production rates) are provided with the scheduling information. Also, maintenance wages may be adjusted by both type of day (weekday or weekend) and by arbitrary blocks of hours within a day (to distinguish hours for which time-and-a-half or double time will be paid, for example). Other maintenance unit costs (for materials and equipment) are assumed to remain constant through a week. These adjustments are incorporated automatically in the maintenance cost algorithm discussed below.

Furthermore, one may also describe the configuration of the work zone to be used for each activity or combination of activities. This description includes the type of closure, its length, and number of through traffic lanes to be maintained. Closure characteristics affect the relative disruption of the traffic stream and associated user costs during roadway occupance, as noted by D in Figure 7 . These interactions will be clarified shortly.

Data on maintenance scheduling and technology, unit costs of maintenance resources, and descriptions of closure zones are all contained within or more maintenance scheduling blocks provided by the manager, and are assigned by roadway location and time through scheduling specifications in the EAROMAR strategies. Guidance on organizing scheduling information is given in Chapter 4, section 4.3.

Costs and Resources Consumed. Maintenance costs and resources consumed are computed by activity, based on estimated workloads, crew production rates, unit resource requirements, and unit costs for labor, equipment and materials provided above. The production rate is in units of damage repaired per hour. Unit resource requirements are the number of men, and the quantity and type of equipment and materials to be used. Unit costs are the dollars-per-hour costs for labor and equipment, and unit quantity costs for materials. This breakdown is quite flexible, since it segregates maintenance demand (activity workload), productivity (the production rate), technology (the resource requirement), and cost (the unit costs). This should make it easier, first, to adapt EAROMAR to the requirement of different state agencies throughout the nation; and second, for individual users to update EAROMAR over time.

The computation of maintenance costs proceeds as follows. The production rate, applied to the workload, determines the aggregate length of time maintenance crews will spend repairing that type of damage that season. Aggregate time, multiplied by unit resource requirements in labor and materials, provides man-hour and equipment-hour estimates. Likewise, the extend of damage, multiplied by unit material requirements, determines the quantity of materials placed. (The resources consumed are stored for later checks against resource availability, as indicated by E in Figure 7.) The quantities of resources used, multiplied by corresponding unit costs, give the total estimated costs for each activity to repair specific types of damage. To these costs are added other costs associated with the activity, such as for mobilization and traffic control. Costs are then summed over all types of damage repaired to arrive at seasonal totals for this roadway section, which are stored for later tabulation (E in Figure 7). Details on costing procedures used within EAROMAR are given in Chapter 4, section 4.4.

Damage Repaired. The immediate effects of maintenance are measured by the extent of damage corrected and the improvement (if any) in the surface PSI. However, pravement repairs may also have positive effects on both user consequences (e.g. higher tolerable speeds, lower rates of fuel and tire consumption), and upon rates of future pavement damage. An attempt is made to account for both of these effects within EAROMAR, thus accounting explicitly for the benefits of maintenance performance. The calculations of quantity of damage repaired are illustrated in Chapter 4, section 4.2. An assessment of the influence of current damage levels on expected rates of future pavement damage is given in Chapter 3. Finally, the interactions between pavement surface and user consequences are discussed both in Chapter 5, section 5.3, and Chapter 6.

Simulation of Construction Projects

The simulation of construction projects (including both subsequent (re)construction and overlays) is similar in approach to that for maintenance. A flow chart of the process is shown in Figure 8.

By their nature projects are undertaken at discrete intervals much less frequently than maintenance activities. Therefore, a separate check on project scheduling is performed, as indicated by the first operation in Figure 8 . The determination of whether or not the project is to be performed now is predicated upon the governing strategy specification by the manager.

All information on the type and duration of the planned roadway occupancy, on project costs, and on changes in route characteristics to result from the project is contained in the project description reviewed in section 2.2. Again, these data are contained in a block identified by the project name, and are assigned to the appropriate roadway section through the strategy specifications.

The keys labeled G, H, and I in Figure 8 indicate that specific data or results are stored or passed to other parts of the program for later use. Road occupancy information will be retrieved during the simulation of traffic operations to account for any congestion induced by the work zone. Project costs will be included as a separate item in the analysis cost summaries. Finally, improvements gained through the project (whether in the pavement itself, or in geometric or capacity related features of the highway) will be incorporated in an updated route description for use in subsequent periods of the simulation.

Traffic Operations and User Consequences

The simulation of traffic and traffic-related effects within EAROMAR is pictured in Figure 9 . As part of a premium pavement analysis, the consideration of the traffic stream has three objectives:

1. To assess the impacts of maintenance scheduling on traffic disruption and congestion;

2. To estimate changes in user-consequences as influenced by pavement condition and roadway occupancy for maintenance or rehabilitation; and

3. To estimate pavement damage over time resulting from traffic loadings.

TRAFFIC DEMAND

Warrants for premium pavements -- and indeed, any pavement management decisions involving high traffic demand -- are sensitive to the volume and composition of traffic affected. Furthermore, traffic predictions are

FIGURE 8

SIMULATION OF

CONSTRUCTION PROJECTS

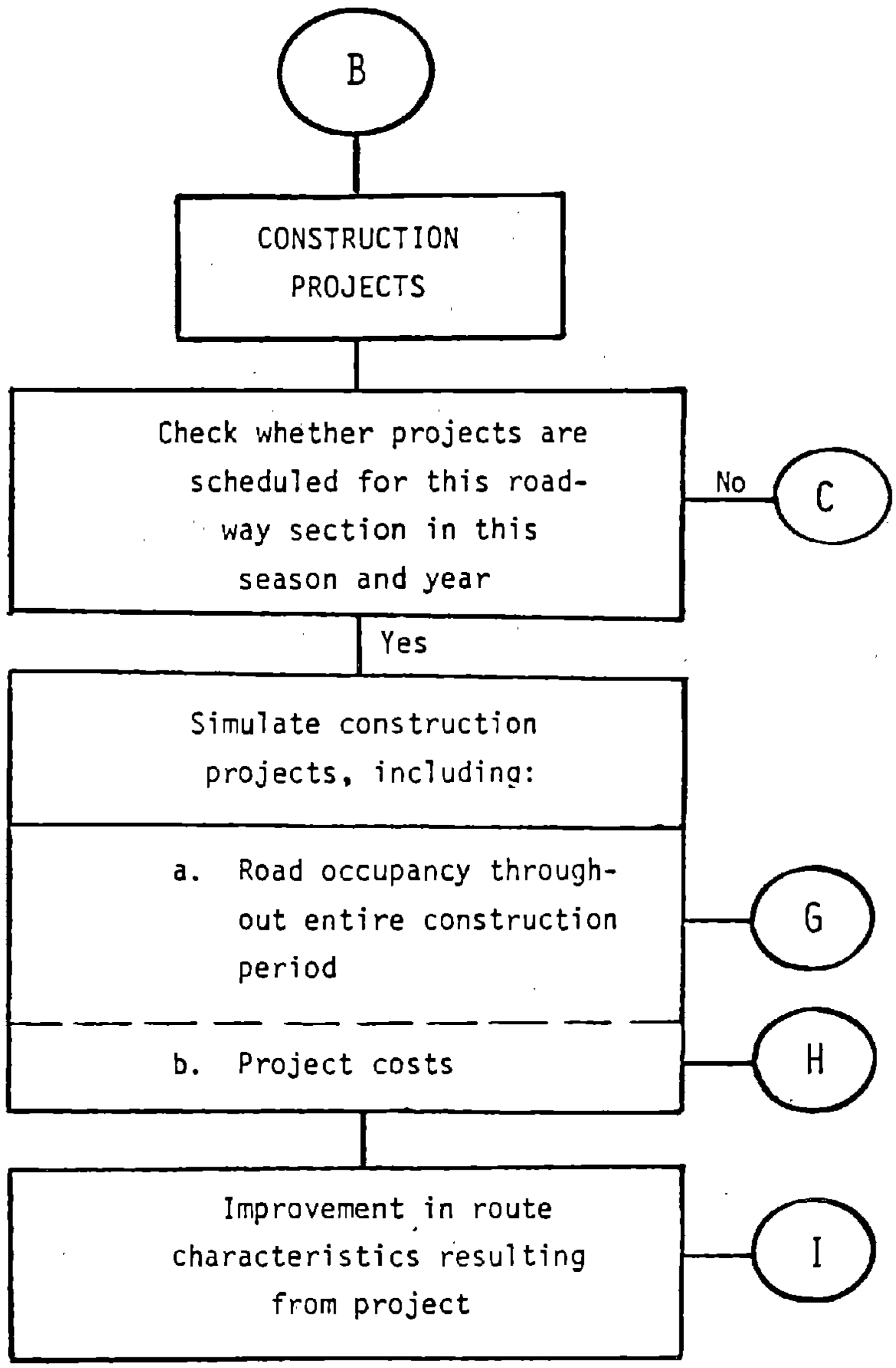
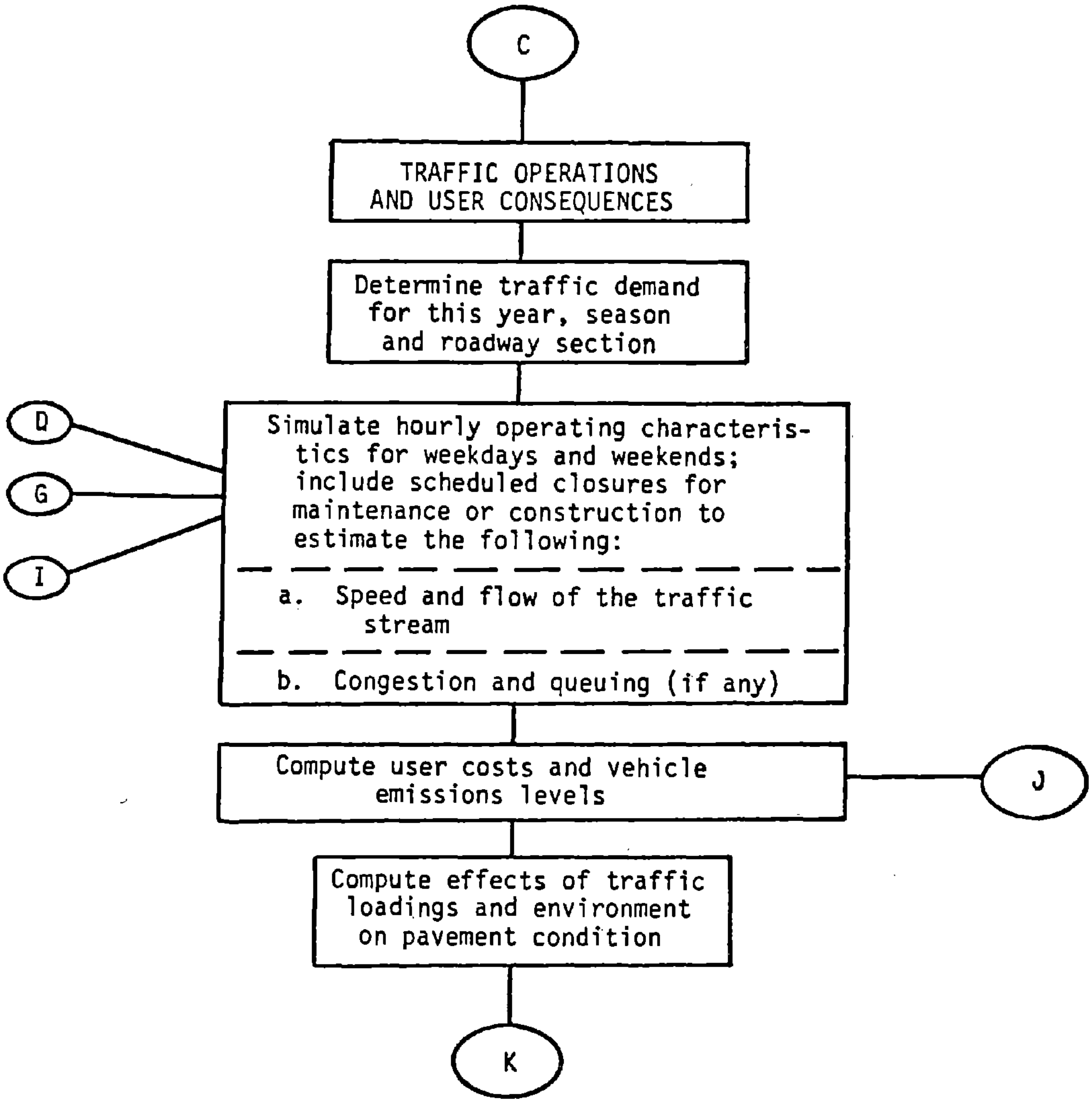


FIGURE 9

SIMULATION OF
TRAFFIC OPERATIONS AND COSTS



subject to uncertainty, and it is often desirable to perform sensitivity analyses of selected pavement alternatives over different assumptions of demand. Within the EAROMAR analysis roadway traffic volumes, annual growth rates, and traffic composition are each treated as state variables controllable by the user through strategy specifications.

Basic traffic demand over a roadway section is expressed in terms of AADT, which may grow over time at a linear or geometric rate; moreover, both the types of growth and their rates are themselves variable over time. AADT may be adjusted by season as well as by weekday versus weekend splits; these adjusted AADT values are then decomposed into respective hourly flows to represent variations in demand patterns throughout each type of day.

Traffic composition is reflected by distributions of trip purposes and vehicle types within the traffic mix. Both the individual classes of trip purposes and the types of vehicles to be included within the analysis may be defined by the user with no restrictions. Thus, through strategy specifications, one may tailor the descriptions of travel demand to exactly those projected for a particular route under study.

By use of the strategy specifications, separate traffic descriptions may be applied to each roadway to represent, for example, directional flows or local-express splits. Furthermore, both volume and composition may vary simultaneously over the roadway length and over time to model, for instance, a route traversing several zones of different economic, social, or demographic character, where factors affecting traffic demand, growth, or composition are likely to vary now or in the future. Examples of the many possibilities in representing anticipated traffic volumes and composition are contained in Chapter 5, Section 5.2.

HOURLY OPERATING CHARACTERISTICS

Hourly operating characteristics describe the level of service afforded motorists in terms of average spped attainable, hourly vehicle flows, and congestion or queuing, which are derived from comparing hourly variations in travel demand to roadway capacity. Since separate demand predictions and capacity characteristics may be provided for each roadway in the analysis, simulations of traffic operations are carried out for each hour of the day for each roadway section.

Under normal conditions the capacity of a roadway varies over its length as a function of changes in the number of lanes, geometric properties, side clearances, and so forth. Roadway occupancy for maintenance or rehabilitation causes a temporary local decrease in capacity, depending upon the closure characteristics specified for maintenance activities or projects, and the required duration of maintenance work.

Within a given season and year, and for each individual roadway section, the EAROMAR system simulates road operating characteristics considering both daily (weekday versus weekend) and hourly variations in traffic demand, simultaneously accounting for any road occupancy determined by the maintenance polices, project commitments, and scheduling requirements discussed earlier. Uncongested flows are estimated using speed-flow relationships developed from the Highway Capacity Manual (3). Where hourly demand exceeds local capacity (whether due to normal rush hour peaks or to occupancy for pavement repair), congested flows are simulated over both the roadway length and time. A speed change cycle is also introduced upon entry of the flow into the congested zone. Details on the simulation of the traffic stream are covered in Chapter 5.4 for congested flow.

Since both variations in travel demand and performance of maintenance work are considered on an hourly basis, the EAROMAR analysis is capable of looking at maintenance scheduling as a decision variable in its own right. The tradeoff to be investigated is that of the additional costs of performing maintenance during off-peak hours versus the reduced disruption of the traffic stream, with attendant decreases in user-related costs.

USER CONSEQUENCES

User consequences are estimated within the system based upon simulated operating characteristics for each roadway section. Models are included to compute vehicle operating costs, travel time and costs, accident costs, and pollution levels as functions of speed, speed changes, congestion, the characteristics of the vehicular traffic, and the current condition of the pavement surface. These calculations are performed for each hour of the weekday and weekend, and thus reflect the costs of delays and interruptions to normal movement induced by maintenance or project workzones. Details on user cost and vehicle emissions models are contained in Chapter 6.

Variations in user costs among different components of the traffic stream are automatically taken into account. For example, costs attributable to fuel consumption and emissions will vary by vehicle type. Values of travel time, on the other hand, are a function of trip purpose. Data upon which these distinctions can be made are provided in the descriptions of travel demand discussed earlier.

PAVEMENT DAMAGE

The last item identified in Figure 9 concerns the effects of traffic, in combination with environmental influences, on pavement damage. Models to predict pavement damage are included within EAROMAR for two purposes.

First, highway maintenance is often a demand-responsive activity, in that work is done after damage has appeared. Therefore, to be able to estimate future maintenance requirements accurately, one must be able to predict the type and amount of damage expected to occur, and when it will occur. (Put another way, to investigate a premium pavement design, one should be able to confirm that no damage requiring maintenance is likely to occur within the first 20 years of life, and that only routine maintenance will occur for the next 20 years.)

Second, the condition of the highway surface affects user response, and may have some bearing on speed, vehicle operating costs, and accident frequencies. Again we would like to know the type, amount, and timing of expected damage to the pavement surface.

Pavement damage is computed in terms of individual structural damage components (cracking, distortion, potholes, etc.); other types of surficial distress (e.g. incident to materials durability) or loss of skid resistance are not included. The rate of pavement damage is computed on a season-by-seadon basis as a function of the structural and environmental characteristics of each roadway section, load-associated data computed from travel demand above, and the existing pavement condition determined by damage and maintenance histories. The seasonal dependence of the damage models allows explicit consideration of effects of temperature and moisture on the rate of damage accumulated; moreover, recall from our discussions earlier that traffic loadings themselves may be seasonally adjusted. Derivations of the damage models used with EAROMAR are presented in Chapter 3.

As an alternative to employing the built-in models to predict damage, users may wish to input directly their own estimates of pavement deterioration for one or more distress modes. The direct user input is a very desirable feature, particularly for those types of distress where insufficient data exist for a good model, and where local experience is important. However, users must in this case insure that some parity exists between the levels of distress input directly, and data provided for the same years in other areas (e.g. traffic loadings, maintenance policies).

Pavement damage is assumed to accrue throughout the season in question, with all damage incurred totaled at season's end (as will be seen shortly). Thus, the damage accumulated during one season influences maintenance prediction and scheduling in the following season. Or, referring to the simulation of maintenance in Figure 7 earlier, the then "current condition" of the roadway section pavement is based in part on damage attributable to the preceeding season.

Seasonal Summaries

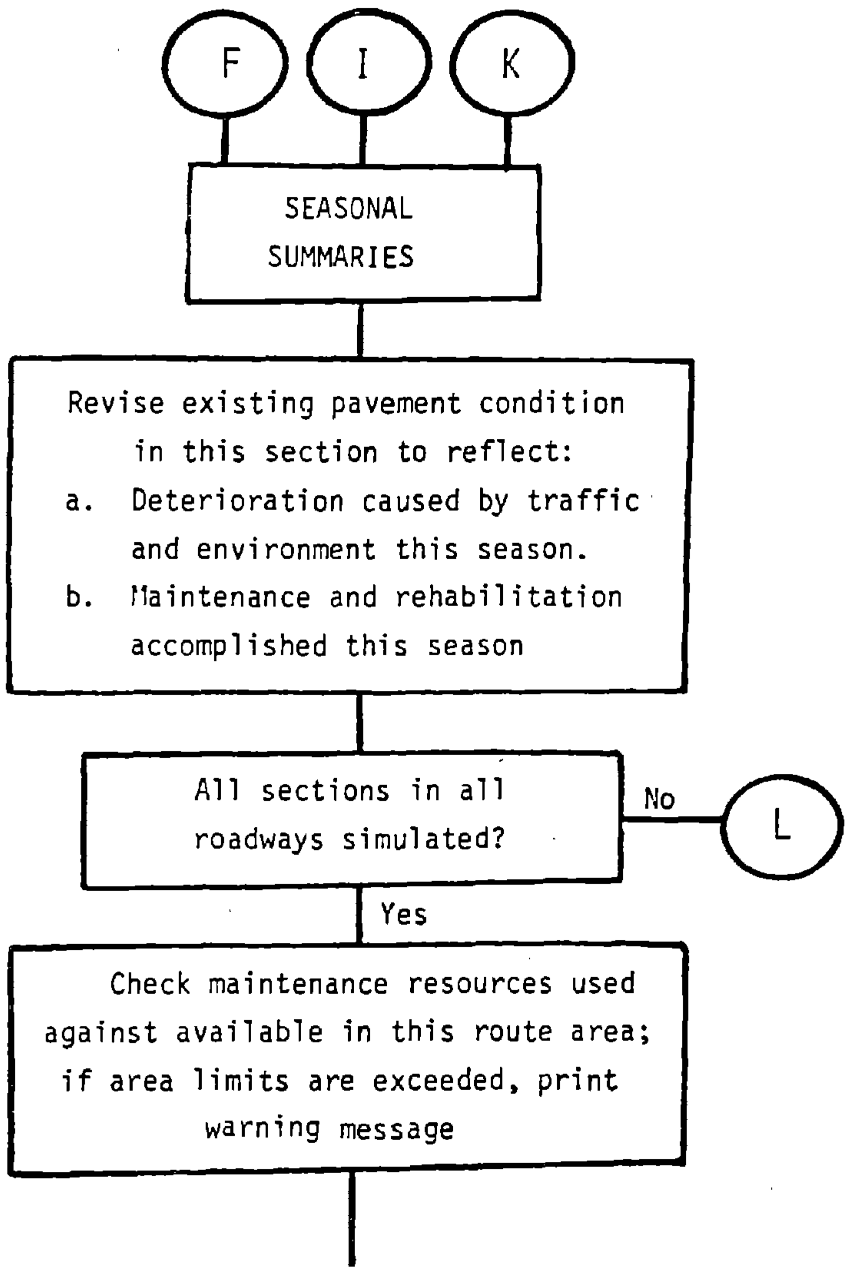
Figure 10 summarizes operations concluding the simulation of a given roadway section through one season. These operations generally entail consolidating and summarizing results on highway performance and costs.

Estimates of section pavement condition result from both positive and negative influences experienced during the just-completed season. Positive influences include the maintenance performed and any larger scale corrective actions achieved through projects. Negative factors include the effects of traffic loads and environment in causing new damage. The offsetting effects of repairs and newly occurring distress are accounted for at the level of individual damage components. The net improvements or deteriora-

FIGURE 10

TABULATION OF

SEASONAL SUMMARIES



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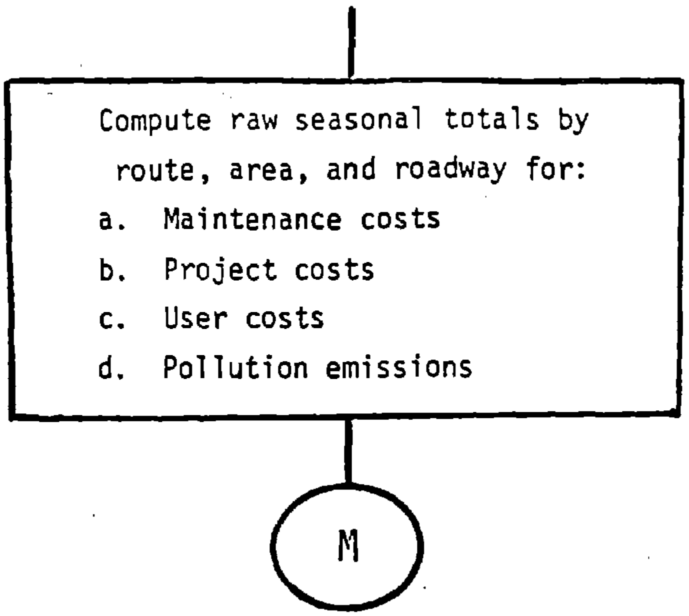
(FIGURE 10, cont.)



tion, applied to respective conditions from the previous season, determine new damage and pavement index values that will be used in the following season's simulation of maintenance and traffic operations in this section.

The process described in Figures 6 through 10 is repeated -- first, for all remaining sections in this roadway, then for sections in all other roadways in turn. Within each roadway sections are processed sequentially in the direction counter to traffic flow, to handle congestion and queueing effects (see Chapter 5, Section 5.4).

When all sections have been simulated, results are compiled, tabulated, and checked. For example, maintenance resources consumed are compared against resource constraints by area, to insure that the final amount of work called for under maintenance policies does not exceed one's ability to physically perform the work. Also, seasonal cost totals for construction projects, maintenance, and user consequences are obtained by roadway, area, and the route as a whole.

Annual Summaries

When all seasons within the year have been completed, relevant cost data are summarized on an annual basis by roadway, area, and for the route as a whole. Cost totals are then adjusted for relative inflation rates and for one or more discount or vestcharge rates specified by the user (see Section 2.5).

Annually adjusted factors such as traffic growth rates and inflation rates are updated before proceeding with the simulation of the next analysis year. In addition, all strategy specifications are reviewed to determine what traffic sets, maintenance data, economic scenarios and so forth need to be revised by assessing new data blocks. See Figure 11.

The simulation of all years within the analysis period marks the completion of the study. Discounted cost totals are printed, which can be interpreted as described in the following section.

Interpretation of Results

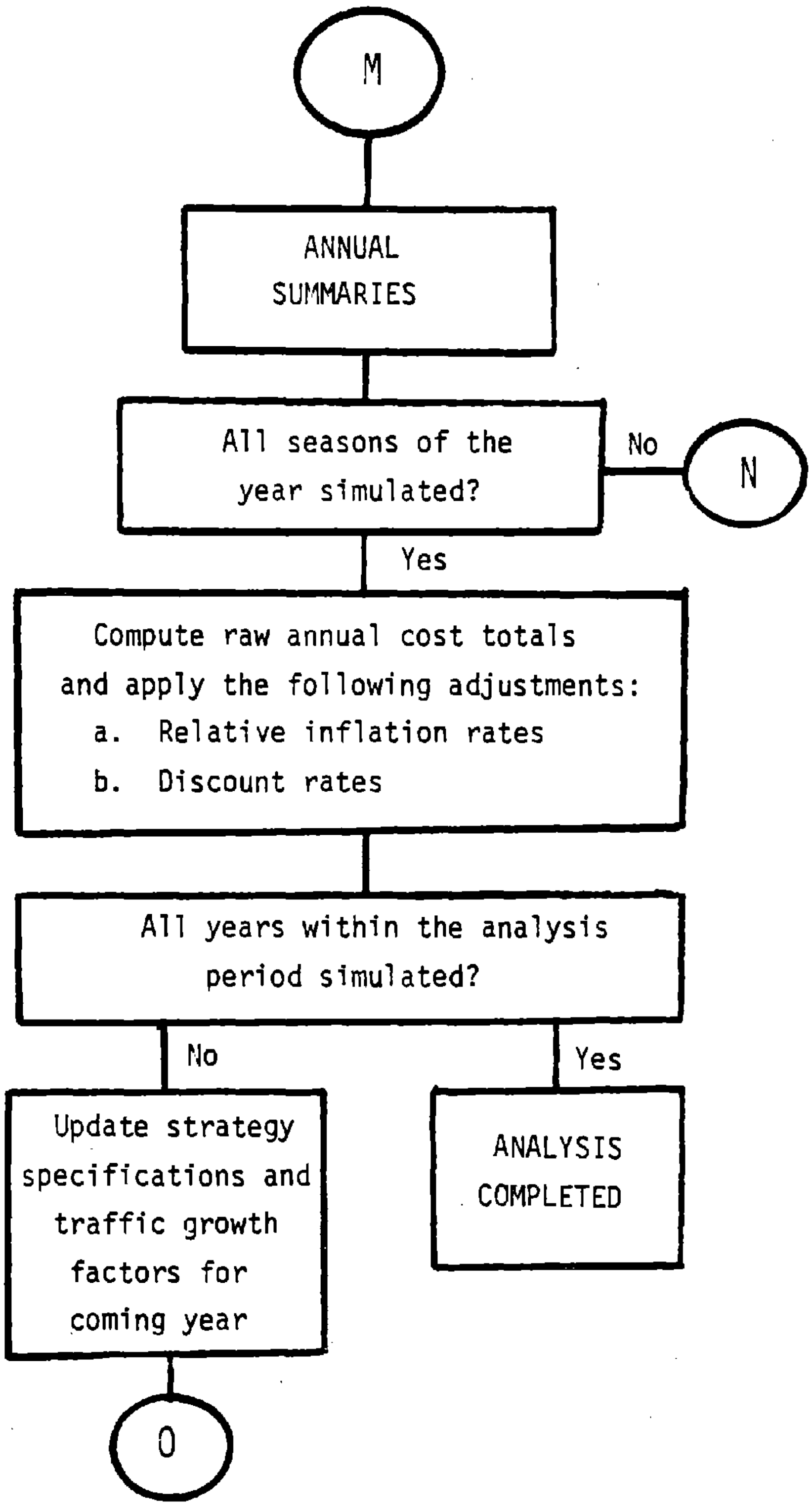
Development of premium pavement warrants -- and in fact, evaluation of any pavement strategy -- implies a comparison among different management options. For example, a premium pavement is justified only if the total discounted costs incident to its lifetime are less than those of a conventional pavement. The same arguement is true for issues of maintenance policy, maintenance scheduling, and maintenance resources applied to a particular highway. Furthermore, the future projections of factors such as traffic demand or pavement deterioration that are necessary to estimate total discounted costs are subject to uncertainty. In these situations it is desirable to test the sensitivity of different strategies to variations in these factors.

For these reasons, the EAROMAR system is designed to be used in an iterative sense, whereby several pavement alternatives may be investigated

FIGURE 11.

COMPLETION OF

ANNUAL SIMULATIONS



and compared based upon a series of road simulations. Within a series of runs one may vary factors influencing traffic growth and composition, maintenance policy, maintenance scheduling, resource limitations, economic data, or any combination of these using strategy specifications. Variations in pavement design, or in other aspects of route characteristics, may be addressed through repeated applications of the EAROMAR model, by adjusting route descriptions. Application of these concepts was illustrated in Figure 5.

Within each season maintenance costs and user costs are calculated as described above. Seasonal totals are summed for each year, the total annual costs discounted at specified rate(s), and the discounted totals accumulated. At the completion of the simulation the discounted maintenance and user costs are displayed, together with initial construction costs provided under the route characteristics.

Analysis of these results is illustrated in Figure 12. Although this example relates to warrants for premium pavements, a similar comparisons could be done for evaluation of other pavement strategies or maintenance management issues. The total discounted costs for premium pavements reflect the significant initial investment required, plus user costs incident to travelling on the highway during the analysis period. Some maintenance costs may be present but are negligible. The discounted costs of conventional pavements represent initally lower costs of construction, but higher maintenance costs and correspondingly higher user costs due to road occupancy and the lower average condition of the pavement surface.

If the total discounted costs of the premium pavement are less than those of a conventional pavement, an economic warrant is thereby established for the premium pavement. If not, then the warrant is not established, and conventional pavements appear economically justified. In the latter case one may, however, wish to investigate different options between conventional designs versus maintenance policy, or in maintenance scheduling for a given policy, to see if total discounted costs for the conventional design may themselves be reduced through such management changes.

An alternative but equivalent means of comparing different pavement strategies is illustrated in Figure 13. Here we consider the same pavement alternatives as shown in Figure 12; however, we have chosen not to estimate construction costs for either of the initial pavements. The discounted totals thus represent maintenance and user cost changes only, as predicted by the EAROMAR simulation.

These results are interpreted as follows. If the costs associated with the premium pavement are higher than those for the conventional design, a premium pavement is not justified under any circumstances. However, if the opposite is true, then the following statement holds:

- The difference between the discounted costs of the two pavement designs in Figure 13 defines the additional amount one

FIGURE 12

EXAMPLE ANALYSIS OF PREMIUM

PAVEMENT WARRANTS

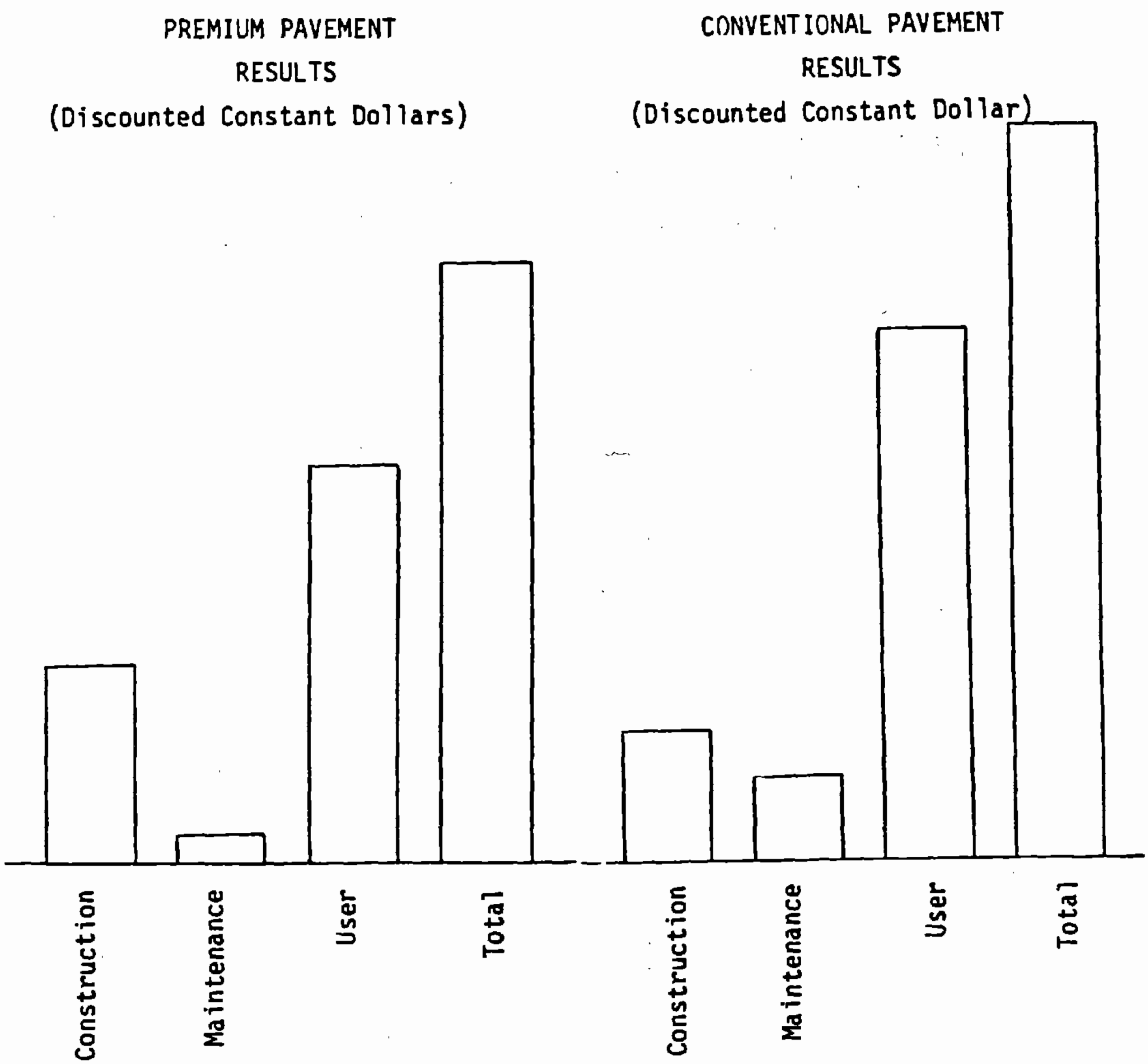
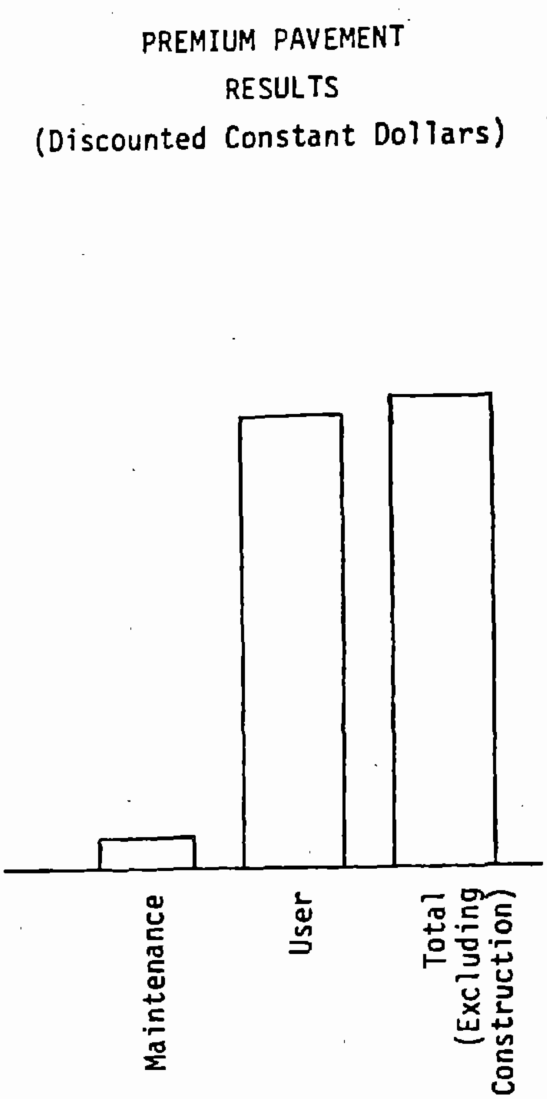
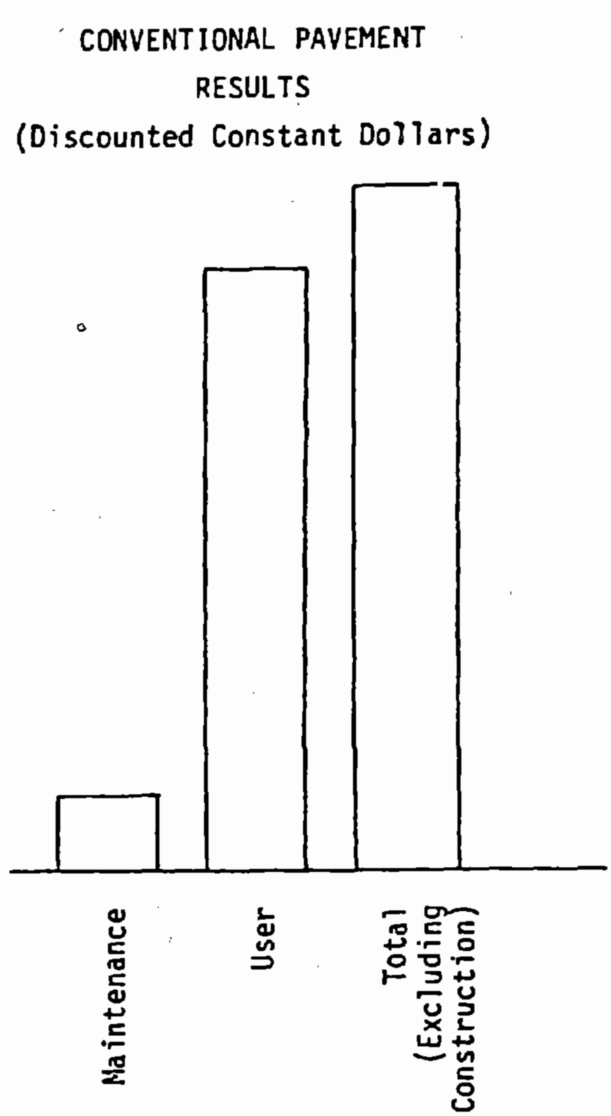


FIGURE 13

EQUIVALENT ANALYSIS OF PREMIUM

PAVEMENT WARRANTS





should be willing to pay <u>over and above the costs of a conventional pavement</u> to install a premium pavement. If the premium pavement envisioned can in fact be built within this addiitonal amount, the premium pavement is warranted. If the cost of the premium pavement, less that of the conventional pavement, exceeds this cost differential, the premium pavement is not economically justified.

For example, suppose the discounted maintenance and user costs of the premium pavement in Figure 13 are $30 million less than those of a conventional design. If the cost of the conventional pavement were $20 million, then any costs of a premium pavement for this section of road would be justified to a limit of $50 million; costs above this limit would not be warranted.

Again, the selection of the method of evaluating EAROMAR results is arbitrary, since both are equivalent and, if correctly applied, will lead to the same decision.

2.5 ECONOMIC SCENARIOS

Projections of future construction, maintenance, and road user costs are affected by anticipated trends in the general economic climate through the analysis period. Within EAROMAR the economic situation is represented by a set of information referred to as an economic scenario. Each scenario comprises the following data specified by the manager:

1. One or more discount rates reflecting the opportunity costs of money to be assumed in the analysis;

2. A set of inflation rates, variable over time, for individual cost components; and

3. A set of costs associated with different severities of motor vehicle accidents.

Scenarios are assigned through strategy specifications in each run. Furthermore, multiple scenarios may be specified in a given run, making it easy for a manager to test the sensitivity of analysis results to (for example) different inflationary patterns.

Following are more complete descriptions of each component of an economic scenario.

Discount Rates

EAROMAR represents an economic (as opposed to a financial) analysis; therefore a discount rate (or a vestcharge rate, or the minimum attractive rate of return) is included to reflect the opportunity cost of money.

The choice of discount rate may affect the selection of an optimal pavement strategy within EAROMAR, since it directly influences the comparative worths of capital-intensive vs. maintenance-intensive alternatives. Although this fact appears to be well understood in the highway profession, there is not unanimity on how to select a rate for a given analysis, with values ranging from zero percent to rates approximating the cost of borrowing used in practice.

Although it is not within the scope of this work to develop guidelines for determing the correct discount rate, nevertheless it is our opinion that non-zero rates are appropriate for public investments. This feeling, coupled with the inherent difficulty in fixing a specific rate, led us to build the following capability within EAROMAR: to perform analyses over a range of discount rates simultaneously, and to display results as illustrated in Figure 14. Any number of discount rates, of any percentage values, may be specified by the manager.

The advantage of this strategy is that users may test the

ROADWAY: NORTHBOUND OR WESTBOUND

SCENARIO: NO-RELATIVE-INFLATION

	YEAR	CONSTRUCTION COSTS (MILLIONS)	MAINTENANCE COSTS (MILLIONS)	VEHICLE OPERATING COSTS (MILLIONS)	VEHICLE TRAVEL TIME COSTS (MILLIONS)	ACCIDENT COSTS (MILLIONS)	TOTAL COSTS (MILLIONS)
ANNUAL	1974	37.020	0.0	5.954	5.960	0.000	48.935
ANNUAL	1975	0.0	0.0	7.124	6.625	0.000	13.749
ANNUAL	1976	0.0	0.0	8.448	7.736	0.000	16.183
ANNUAL	1977	0.0	0.0	8.527	7.832	0.000	16.359
ANNUAL	1978	0.0	0.0	8.599	7.928	0.000	16.527
ANNUAL	1979	0.0	0.0	8.675	8.020	0.000	16.695
ANNUAL	1980	0.0	0.0	8.754	8.151	0.000	16.904
ANNUAL	1981	0.0	0.0	8.828	8.242	0.000	17.070
ANNUAL	1982	0.0	0.0	8.906	8.334	0.000	17.240
ANNUAL	1983	0.0	0.0	8.977	8.443	0.000	17.420
TOTAL	1974-1983	37.020	0.0	82.792	77.271	0.000	197.083
DISCOUNTED	5%	35.257	0.0	63.104	58.935	0.000	157.297
DISCOUNTED	8%	34.278	0.0	54.404	50.835	0.000	139.517
DISCOUNTED	10%	33.654	0.0	49.556	46.322	0.000	129.533
DISCOUNTED	12%	33.053	0.0	45.329	42.388	0.000	120.771
DISCOUNTED	15%	32.191	0.0	39.947	37.380	0.000	109.519

FIGURE 14. EXAMPLE DISCOUNTED COST RESULTS

sensitivity of the final outcome to the discount rate. If one policy or strategy dominates the others over the range of rates investigated, then no further refinements are called for.

If the choice of rate does affect the outcome, then one must determine whether the break point lies above or below the rate intended for this analysis; or, if the results are highly sensitive, then one must make a precise determination of the rate to be used. Over a number of analyses, however, this strategy will require certainly no more time, and in many cases less time, than would the requirements that the user explicitly state one discount rate prior to undertaking analysis.*

This feature of multiple discount rates is really a type of sensitivity analysis, and it should not confuse the fact that discount rates cannot vary over time, nor can they be "mixed" within an analysis. With reference to Figure 14, each line represents in effect the results of a separate, complete analysis. Thus, in comparing two alternatives, the manager should insure that both results are computed at the same discount rate.

Inflation Rates

Inflation distorts the dollar value of resources consumed over time. It then becomes necessary to talk about two dollar values: current dollars, which are the inflated dollars or the actual cost of a resource at the time it is consumed; and constant dollars, which are the dollars one would have paid for the resource in some reference year. The ratio of current dollars to constant dollars defines the aggregate (or accumulated) rate of inflation for that time period in relation to the reference year.

Current dollars are suitable in a financial analysis, where one must deal with actual numbers of dollars at the time they are spent. This would be the case, for example, in estimating a budget, or in arranging financing for a project. However, the type of analysis conducted by

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* We do not mean to imply here the discount rates should be fixed after the analysis is performed, since this may impair the impartiality of the study. What is intended here is essentially a check that can be done quickly and accurately. For example, if an agency typically uses a discount rate in the vincinty of 8-10%, and the EAROMAR results indicate alternative A superior to alternative B at all rates between 0% and 20%, then clearly A is the preferred alternative. However, if A were preferable at 8%, and B at 10%, then a precise rate should be fixed before proceeding with further analyses. These additional analyses should then include, if possible, more detailed estimates of future costs, and perhaps non-economic factors affecting the relative worth of each alternative.

EAROMAR is an economic analysis, where one must resort to constant dollars. The effect of inflation in a constant dollar analysis arises only to the extent that inflation rates differ among several resources considered.

For example, take an analysis in which 1980 is the reference year, and assume a general rate of inflation (1980-1981) of 12%. However, further assume that the material price of asphalt increases by 15% over the next year. Looking at 1981 in constant dollars, everything except asphalt remains at the same level of 1980 dollars, since the effect of general inflation has been merely to increase the (current) price, but not to change the relative value of the resources.

This last statement is not true, though, regarding the asphalt. Its relative value in constant dollars has increased by an amount equal to its relative rate of inflation:

$$\frac{1.15}{1.10} = 1.045 \qquad (5)$$

or a 4.5 percent increase. Therefore the constant dollar estimate for asphalt in 1981 is 4.5% higher than the equivalent estimate in 1980. The discounted economic analysis would use the 4.5%-inflated figure.

The treatment of inflation in EAROMAR can therefore be visualized as in Table 7. Managers may specify a general rate of inflation, as well as individual rates of inflation for specific components, all varying over time if desired. Although it might be useful in some instances to specify separate rates for each different resource to be used, we feel this would complicate the system needlessly and, given the uncertainty in projecting inflation rates into the future, would not be justified under EAROMAR's objectives. Instead we have suggested that five rates be considered, corresponding to maintenance labor, equipment, and material, fuel, and value of travel time.

The rates specified by the user are the nominal annual rates (i.e. based on current dollars). The system computes the corresponding relative rates of inflation automatically for each resource category and each year, using the following relationship:

$$1 + r_r = \frac{1+r}{1+r_o} \qquad (6)$$

where r_r is the relative or differential annual inflation rate for this item;

r is the nominal annual inflation rate for this item estimated by the user;

and r_o is the general annual inflation rate.

Then, as the simulation proceeds year by year, the costs of labor, equipment, materials, fuel and travel time will be adjusted annually

TABLE 7

TREATMENT OF INFLATION

WITHIN EAROMAR

A. DATA PROVIDED BY USER

Category	Year Rate Takes Effect	Annual Rate
General rate	1980	10%
Labor	1980 1990	11 8
Equipment operation	1980	9
Materials	1980	10
Fuel	1980 1990	15 12
Travel time	1980	5

B. CONVERSION TO RELATIVE OR DIFFERENTIAL RATES

Category	Year Rate Takes Effect	Annual Rate
General rate	1980	-
Labor	1980 1990	0.9% -1.8%
Equipment operation	1980	-0.9%
Materials	1980	0%
Fuel	1980 1990	4.5% 1.8%
Travel Time	1980	-4.5%

based on the appropriate differential or relative rate, as indicated in Table 7B.

Accident Costs

Accident costs are a function of the legal compensation environment under which the premium pavement analysis is carried out, and in this sense are included as parameters describing the local economic situation. Separate costs-per-occurrence can be provided by the manager for (1) accidents causing property damage only, (2) accidents causing injury, and (3) accidents causing fatalities. Obviously this information encompasses only the monetary costs of motor vehicle accidents; however, managers are free to adopt as wide a latitude as they feel appropriate in identifying what costs need to be reflected under each accident category. Use of these entries in the highway user cost calculations will be described in Chapter 6.

Use of Multiple Scenarios

The data contained in economic scenarios do not affect the simulation in progress, but rather are applied as adjustments to simulation results (as indicated in Figure 11). Therefore it is possible to test multiple economic scenarios within a single run, rather than among several runs as would be required for other state or policy variables.

An illustration of how this is accomplished in EAROMAR is shown in Figure 15. The strategy specifications define three individual scenarios, differentiated by their assumed patterns of inflation. These specifications will cause the system to access the appropriate blocks of economic information provided by the user, illustrated in the lower part of Figure 15. Within each scenario the discount rates, time-dependent inflation rates, and accident costs are entered.

Taking the "Low-Inflation" scenario as an example, we see that the discount rates to be tested are 6, 8, 10, 12 and 14 percent. The general rate of inflation (following the INFLATION keyword in Figure 15) is projected to be 5% in 1980, 6% in 1981, 5% from 1982 to 1996, and 5.5% from 1996 through the conclusion of the analysis. Similar interpretations are applied to the individual inflation rates projected for maintenance labor, equipment, and materials; fuel, and value of travel time. The system will then automatically convert these nominal rates to relative or differential rates as described earlier. Accident costs under this low-inflation scenario are estimated to be $1000 per occurrence for property-damage-only; $5000 per occurrence for injury-related accidents; and $100,000 per occurrence for fatal accidents. The NO-INFLATION and HIGH-INFLATION scenarios contain analogous information. (Note that for the NO-INFLATION case, no inflation rates need be entered.)

As construction, maintenance, and user costs are tallied by the

FIGURE 15

EXAMPLE APPLICATIONS

OF ECONOMIC SCENARIOS

Strategy Specification (Scenarios to be tested):

```
NO-INFLATION
LOW-INFLATION
HIGH-INFLATION
```

Data for each Scenario:

```
SCENARIO NO-INFALTION
     DISCOUNT 6 8 10 12 14
     ACCIDENT 1000 5000 100,000
     END SCENARIO

SCENARIO LOW-INFLATION
     DISCOUNT 6 8 10 12 14
     INFLATION 1980 5 1981 6 1982 5 1996 5.5
     LABOR 1980 4
     EQUIPMENT 1980 4 1987 5
     MATERIALS 1980 5 1990 7
     TIME 1980 3
     FUEL 1980 10 1990 12
     ACCIDENT 1000 5000 100,000
     END SCENARIO

SCANARIO HIGH-INFLATION
     DISCOUNT 6 8 10 12 14
     LABOR 1980 8
     INFLATION 1980 10 1990 12
     EQUIPMENT 1980 8 1987 11
     MATERIALS 1980 12 1999 14
     TIME 1980 7
     FUEL 1980 15 1990 20
     ACCIDENT 2000 10,000 100,000
     END SCENARIO
```

EAROMAR simulation, each scenario in Figure 15 would be applied to adjust costs for differential inflation, to compute accident costs, and to discount cost totals over time (at multiple rates if so indicated). A separate set of cost reports would then be produced under each scenario. Thus, managers can identify immediately the influence of different anticipated economic scenarios on the relative costs and benefits of a given pavement strategy.

NOTES TO CHAPTER 2

1. Within the EAROMAR simulation roadways are divided into lengths called "sections" so that each section exhibits homogenous characteristics. Many factors discussed in this chapter define boundaries between roadway sections: locations of changes in roadway capacity, alignment or pavement structure; locations of changes in traffic volumes, compositions or growth rates; limits of project workzones; limits of administration areas; and locations where maintenance policies change, to name the more significant ones. While the division of a roadway into sections is internal to the simulation and not apparent to the user, managers should realize the following:

1. All internal calculations of roadway performance and costs (including pavement deterioration, maintenance, traffic flow and congestion, and user costs) are done section by section;

2. The efficiency of the simulation is roughly proportional to the number of sections to be analyzed; and

3. The number of sections created by the simulation can be controlled by the manager, directly influencing the computational speed and cost of the simulation.

The greatest efficiency is achieved if the various milepost specifications in the route and strategy descriptions (e.g. milepost locations denoting changes in alignment, capacity, pavement structure, administrative areas, traffic, maintenance policy, etc.) coincide with one another as much as possible, and are the minimum number needed to describe the problem effectively. Of course there is a tradeoff between computational efficiency and the level of detail required in the analysis. Nevertheless, if two adjacent sections differ from each other in only minor respects, one should consider combining them.

CHAPTER 3

PAVEMENT DAMAGE RELATIONSHIPS

3.1 RATIONALE AND APPROACH

Rationale

Pavement damage relationships are included within EAROMAR for two reasons. First, pavement maintenance is a demand-responsive activity, in that work is done to correct damage that has already appeared, and possibly to prevent more severe future damage. Therefore, to estimate future maintenance requirements accurately, one must be able to predict the type and amount of damage expected to occur, and when it will occur. (Put another way: to investigate a premium pavement design, one should be able to confirm that no damage requiring maintenance is likely to occur within the first 20 years of life, and that only routine maintenance will occur in the 10-20 years thereafter.) Second, the condition of the highway surface affects pavement serviceability to users, and may (under certain conditions) affect traffic speed and vehicle operating costs. Again, we would like to know the type, amount, and timing of expected damage to the pavement surface.

Many factors combine to influence both the type and the rate of pavement damage: (1) the quality of initial pavement design and construction; (2) vehicle loadings to which the pavement is subjected over time; (3) environmental conditions encompassing local subgrade soils, temperature, moisture, and sunlight; and (4) the care and renewal of the pavement through maintenance, rehabilitation and reconstruction. As a result, a large number of analytic models have been described in the literature to simulate at least some of the complex interactions among all these factors. However, these models differ widely in scope, objective, mathematical formulation and range of independent variables, and the selection of candidates for use within EAROMAR must be judged in light of the requirements of a premium pavement analysis. Several items are worthy of note.

As an economic analysis EAROMAR requires predictions of pavement <u>performance</u>, as opposed to <u>design</u>. While certain equations or models used in design can also be applied to pavement performance, in general, design procedures are predicted on controlling the level of specific modes of damage (e.g. preventing the onset of cracking), or of maintaining some adequate level of serviceability throughout the design life. From a performance point of view we would like instead to predict how a given pavement will behave under a given set of loads, environmental factors and maintenance policies. In lieu of controlling damage, we need to know how much damage will in fact occur.

A related point is the relative abundance for some damage mechanisms of theoretical derivations based on laboratory tests, but which have little correlation to field data. For example, Finn, et al (6) cite this situation in the prediction of fatigue cracking in flexible pavements. Again, what are needed in EAROMAR are measures of field distress over time as functions of the several independent variables above.

A third consideration is the effect of maintenance rehabilitation, or overlay on pavement performance. There is very little in the research literature (particularly in a quantitative sense) on the effects of pavement maintenance in improving either the current condition and serviceability of the road surface, or the future rate of pavement deterioration. The same statement is true regarding overlays and pavement rehabilitation. A particularly difficult question in assessing the performance of an overlaid pavement is the structural contribution to be attributed to the damaged original layers.

With these considerations in mind we reviewed the literature to identify candidate models to predict the serviceability or the damage in flexible, rigid, and composite pavements, as functions of the several variables listed earlier. Both empirical and mechanistic models were studied. Each type of model presented both advantages and disadvantages for use in premium pavement studies.

Empirical models (such as the serviceability equations developed from AASHO Road Test data (4, 5) relate pavement performance or serviceability in the field to gross measures of traffic, environmental, and pavement parameters, and are generally derived from statistical correlations between the respective input and output variables. They have the advantage of being simple to use and, in cases where models have been developed in a comprehensive way, they account for many of the load, environmental, structural and materials parameters of interest in this study. Their disadvantages are, first, that the approaches taken among the many available empirical models differ widely; and second, they are not necessarily based on an understanding of pavement behavior. As a result, empirical models show considerable variation in their respective independent variables and, at times, in the goodness of fit between field observations and model predictions.

On the other hand, mechanistic models have been developed based on an understanding of flexible and of rigid pavement response to loads and environment. Furthermore, several of these models have been field tested with generally good results. However, the disadvantage of mechanistic procedures (with reference to our requirement in EAROMAR) is that they are based on computations of stresses and strains (spatially and temporally dependent) that are difficult to capture in a closed-form relationship. Hence, large-scale computer systems are necessary to obtain mechanistic solutions (based, for example, on finite-element theory or on layered-system theory) and it would be difficult to merge all the mechanistic models that would be required within the EAROMAR framework.

Some general comments apply to both the empirical and the mechanistic approaches. First, none of the models reviewed accounts for the effects of maintenance, rehabilitation or overlays as discussed above. Therefore, at this time the beneficial aspects of pavement repair can be modeled only indirectly. Second, the available models predict only those damage components that are closely allied with current formulations of pavement serviceability (e.g. cracking, rutting, roughness), or (for mechanistic models) those components attributable to recognized modes of fracture or distortion (e.g. fatigue cracking, temperature cracking, rutting). No models are currently available to predict more localized forms of pavement distress which are nevertheless important in predicting future requirements for maintenance (e.g. potholes, base failures, joint filler deterioration, pumping).

Approach

Therefore, the strategy adopted in this project is to propose performance relationships based on models available in the literature, accounting to the greatest extent possible for relevant structural, materials, load and environmental factors. Ideally this could have been done by identifying appropriate mechanistic models, executing the models for a range of input factors, and fitting closed-form relationships to the mechanistic results. Unfortunately, it was not possible to accomplish this for the several damage components required, given the resources available for this part of the investigation. However, from published analyses of both flexible and rigid pavement fatigue cracking we did obtain essentially closed form approximations of mechanistic predictions for these damage modes. For the other distress mechanisms of interest we adapted empirical models when available.

All pavement damage relationships are applied within EAROMAR for each road section on a seasonal basis. This permits estimates of damage to be sensitive to road physical and operational characteristics (pavement layer structure, materials properties, vehicle loadings as a function of traffic volume and composition) and to seasonal adjustments in ambient temperature and moisture.

For each season of each year we are interested in using these models to predict the rate of damage occurrence in lieu of the absolute amount accumulated. The reason is that the net amount of pavement damage present at any given time depends upon both the damage history and the amount of past maintenance performed. However, available models are unable to account for the effects of maintenance in their predictions of total damage accumulated. Therefore, in using these models it is necessary to convert the predictions of cumulative damage into the effective rate at which additional damage is now being induced. This is done by subtracting a model's estimate of prior cumulative damage (at time t-1) from the current estimate (for time t):

$$\text{Incremental damage occurrence }(t) = \text{Cumulative Damage }(t) - \text{Cumulative Damage }(t-1) \qquad (7)$$

The net amount of damage present at the end of any time period t is then given by:

Net Cumulative Damage (t) = Net Cumulative Damage (t-1)

+ Incremental Damage Occurrence (t)

- Damage Repaired Through Maintenance, Rehabilitation or Overlay (t). (8)

The rate of damage occurrence is sensitive to time-dependent and environmental influences on pavement materials properties, and to the remedial effects of preventive maintenance. However, while environmental influences and aging are discussed in the literature, the beneficial effects of maintenance in reducing the rate of future damage occurrence have not been derived quantitatively. Within EAROMAR both these effects are accounted for through adjustments in pavement materials properties over time (either by season or by year). Environmental influences and aging are modeled by time- and seasonal-dependent fluctuations in layer and subgrade properties.* The effects of maintenance are simulated by damage-dependent adjustments in these same properties. In this way we may say that the effects of maintenance on the preservation of the highway investment are accounted for at least indirectly.

Effects of traffic loading on pavement damage are computed using a design lane concept. A design lane factor ranging from 0 to 1 may be specified by the user for each roadway section as discussed in Chapter 2. This factor represents the percentage of total traffic on the most heavily used or "design" lane; the performance of this lane is taken to characterize the pavement performance in the roadway section overall. Within each damage model any representation of traffic (e.g. average vehicles per day; cumulative number of trucks; cumulative number of standard axle loadings) is reduced by the design lane factor. For convenience the application of the design lane factor is understood in all the damage models which follow, and it will not be shown explicitly in any of the pavement damage equations.

We discussed above that for some localized distress mechanisms (e.g. potholes, base failures) generalized damage models have not been published in the research literature. To accommodate these types of failures within the EAROMAR framework (since they too affect maintenance requirements) we allow a user to specify estimated rates of damage accumulations over time. Such rates may also be specified (if desired) for one or more damage components addressed by the pavement models; in these cases the user's inputs will override model predictions for those damage components. Users may provide time-dependent damage rates, variable along the roadway length, under the strategy specifications described in Chapter 2. However, such rates will be applied exactly

*Temperature and moisture effects on materials properties can be simulated by the user through seasonal variations in layer materials properties, as explained in Chapter 2.

as input, with no further adjustments. Therefore, managers should insure that any damage rates provided are consistent with the pavement section, anticipated traffic volume and composition, and local environmental factors influencing pavement damage.

Sections 3.2, 3.3 and 3.4 describe the models developed for flexible, rigid and composite pavements respectively. Section 3.5 discusses the time-varying aspects of the models, considering effects of environment and past maintenance performed on materials properties. These relationships have been adapted from existing performance or design models which were considered compatible with the EAROMAR system structure and data requirements. The models have been implemented in a modular fashion; thus, they may be easily updated as new or improved relationships are derived through further research.

3.2 FLEXIBLE PAVEMENT

The flexible pavement damage components and their treatment within EAROMAR are identified in Table 8. Models to predict flexible pavement damage require the data listed in Table 9 from the roadway descriptions provided by the user (discussed in Chapter 2). An explanation of each of the flexible pavement models follows.

Fatigue Cracking

Flexible pavement fatigue is manifested by alligator cracking appearing in the wheelpaths, induced by excessive tensile stresses and strains at the bottom of the asphalt concrete surface layer (if the base is granular) or at the bottom of a stabilized base layer. Darter and Barenberg (7) cite as causes repeated or heavy traffic loadings (in relation to pavement structural capacity), loss of pavement support due to excessive moisture or shear failure in the sub-layers, and hardening of the asphalt concrete surface with time.

Fatigue cracking, together with other sources of cracking, was one of the distress mechanisms correlated with reductions in serviceability at the AASHO Road Test (4). Three levels of cracking were identified:

1. Class 1 - fine, disconnected hairline cracks
2. Class 2 - formation of alligator cracking
3. Class 3 - progression of alligator cracking to more severe spalling and loosening of individual blocks of pavement.

Only when the extent of cracking reached Class 2 or Class 3 was it included in the reduction in serviceability index.

Mechanistic formulations of fatigue are typically based on Miner's Law (8), which postulates a linear accumulation of damage independent of the order in which loads are applied and with no healing effects, according to the relationship:

$$D = \sum_i \frac{n_i}{N_i} \qquad (9)$$

where D is the cumulative fatigue damage,

n_i is the number of load applications during period i, and

N_i is the total number of allowable loads in period i as determined from fatigue relationships for the pavement structure and materials properties.

The number of applied loads n_i depends upon traffic volume and

TABLE 8

TREATMENT OF FLEXIBLE PAVEMENT
DAMAGE COMPONENTS WITHIN EAROMAR

	Damage Component Or Serviceability Index	Prediction Model Included Within EAROMAR	User May Input Rate Directly
1.	Linear Cracking	X	X
2.	Areal Cracking	X	X
3.	Rutting	X	X
4.	Roughness	X	X
5.	Potholes		X
6.	Base Failures		X
7.	Pavement-Shoulder Joints		X
8.	Present Serviceability Index	X	X

TABLE 9

DATA REQUIRED FOR
FLEXIBLE PAVEMENT
DAMAGE MODELS

Model	Input Data Required
All Models	Layer Thicknesses
Areal Cracking	Diametral Resilient Modulus Complex Modulus Subgrade Resilient Modulus AASHTO Layer Coefficients
Linear Cracking	Pavement Age Stiffness of Asphalt Cement Freezing Index
Roughness	AASHTO Regional Factor AASHTO Layer Coefficients AASHTO Subgrade Support Value
Rutting	Elastic Modulus Subgrade Resilient Modulus

composition during time period i; in EAROMAR the applicable time periods are seasons within years. The allowable number of load applications N_i is estimated from fatigue curves relating the load limit to the initial tensile stress or strain at the bottom of the surface or stabilized layer, as a function of materials properties. (Again, any time dependencies in materials properties are accounted for by season within EAROMAR.) The application of this concept within EAROMAR therefore requires the following determinations:

1. Estimates of the maximum tensile stress or strain at the bottom of the pavement surface or stabilized layer, as a function of applied load, pavement structure and materials properties;

2. Estimates of cumulative fatigue damage according to eq. (9) based upon the total number of applied loads vs. the number of allowable loads determined by the tensile stress or strain from 1.; and

3. A relationship between damage predicted by Miner's law (from 2. above) and field manifestations of fatigue cracking.

Each of these steps is developed analytically below.

MAXIMUM TENSILE STRENGTH OR STRAIN

Mechanistic models are available (e.g. refs. 9, 10) to determine layer stresses and strains in flexible pavements. However, as discussed earlier, the use of such mechanistic models would be cumbersome within the EAROMAR framework and would not take proper account of the effects of seasonal maintenance. What are needed instead are closed-form approximations to the mechanistic solutions - approximations which yield the desired stress-strain data as a function of appropriate layer thicknesses and materials properties.

One example of such an approximation was developed by Finn and others in work under NCHRP Project 1-10B (6). The following equation was regressed from their PSAD program:

$$\begin{aligned} \log \varepsilon = \; & 3.355 - 0.72219 \log T_1 - 0.089108 \log T_2 \\ & - 0.065293 \log T_3 - 0.53784 \log E_1 \\ & - 0.26563 \log E_2 - 0.12667 \log E_3 \\ & - 0.7358 \log E_4 + 0.45913 \log L \end{aligned} \tag{10}$$

where

ε = maximum horizontal tensile strain in asphalt layer, micro-inches/inch;

E_1 = complex modulus $|E^*|$ of surface layer psi x 10^5(30,000 to 2.75 x 10^6psi);

E_2 = modulus of base layer, ksi (15 to 50 ksi);

E_3 = modulus of subbase layer, ksi (7 to 50 ksi);

E_4 = modulus of subgrade, ksi (3 to 50 ksi);

T_1, T_2, T_3 = thickness of surface (3 to 6 inches), base (3 to 9 inches), and subbase (0 to 12 inches); and

L = axle load, kips (18 or 30);

From the point of view of the EAROMAR design, a relationship of this type is potentially useful; however, eq. (10) assumes a fixed three-layer system plus subgrade, whereas the pavement input for EAROMAR is not limited to any particular number of layers. Furthermore, the fact that the moduli of untreated base layers are stress-dependent may preclude application of this relationship to other than the layer thickness and modulus values tested.

To address the problem of a variable number of layers in general, it was decided to approximate the pavement structure using a two-layer solution for which relationships yielding tensile strain in the asphalt surface layer are available. Under this approach treated and untreated base/subbase layers are converted to equivalent thicknesses of surface layer, using the AASHTO layer coefficients as follows:

$$t_i' = t_i \cdot \frac{a_i}{a_1} \text{ and } t_1 = \sum_i t_i' \tag{11}$$

where

t_i' is the effective thickness contributed to the surface thickness t_1 by each base or subbase layer i;

t_i is the actual thickness of layer i;

a_i is the AASHTO structural coefficient of layer i; and

a_1 is the AASHTO structural coefficient of the flexible surface layer.

The equivalent thickness contributions of each base/subbase layer are then added to the actual thickness of the surface layer to obtain an equivalent surface thickness t_1.

Curves to solve for the initial tensile strain at the bottom of the asphalt layer in the resulting two-layer system are taken from

Santucci (11), and are illustrated in Figure 16. These curves were constructed for selected subgrade modulus values (E_2), surface layer thicknesses (t_1), and surface layer diametral resilient modulus (E_1), using multi-layered elastic theory developed by the Chevron Research Company for a 9-kip equivalent wheel load under dual tires. By applying linear regression techniques to the curves, the following model is obtained to predict the maximum critical tensile strain:

$$\varepsilon_t = A \exp [-B \log E_1] \tag{12}$$

$$A = 10^{(b-mt_1)} \tag{13}$$

$$b = 4.8183 \exp [-0.0103\ E_2] \tag{14}$$

$$m = 0.04818 \exp [-0.0245\ E_2] \tag{15}$$

$$B = 1.831 \exp [-CE_2] \tag{16}$$

$$C = 0.02235 \exp [-0.0512\ t_1] \tag{17}$$

where

ε_t is the maximum initial tensile strain in the surface layer, 10^{-6} in/in;

E_1 is the diametral resilient modulus of the flexible surface layer as input by the user, ksi;

t_1 is the effective surface thickness in inches in a two-layer representation, computed from eq. (11); and

E_2 is the resilient modulus of the subgrade as input by the user, ksi.

If the subgrade strength is entered by a static CBR value instead of the resilient modulus, the following conversion is made:

$$E_2 = 1.5\ \text{CBR} \tag{18}$$

where

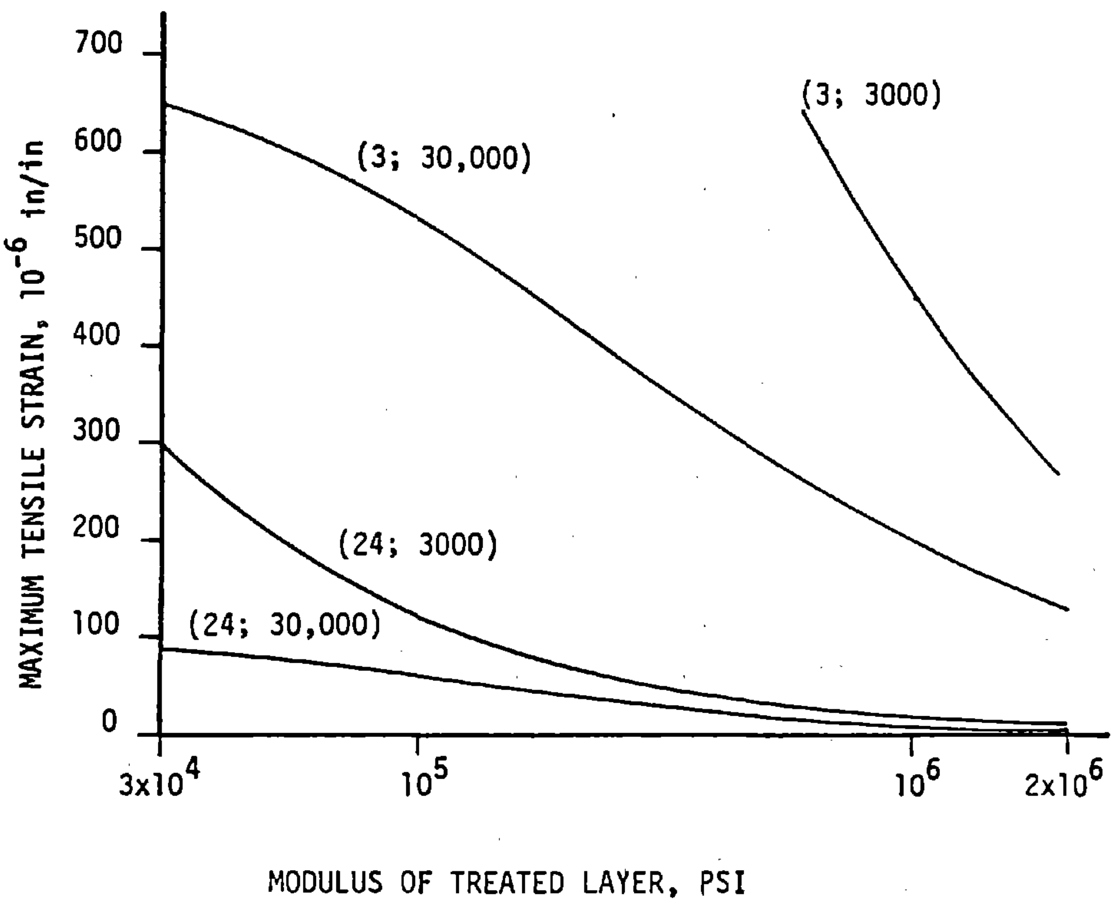
E_2 is the subgrade resilient modulus in ksi; and

CBR is the static CBR input by the user.

FATIGUE DAMAGE

Predictions of the number of loads N_f necessary to cause fatigue failure have been developed extensively in the literature. However, such curves are based on laboratory tests, generally with little correlation to field experience to account for the relaxation times between traffic loads and resulting differences in crack propagation rates.

Finn and others (6) have attempted to transform laboratory fatigue curves obtained by Monismith into field distress prediction equations for fatigue cracking, by determining a shift factor for the



(Numbers in parentheses refer to thickness of treated layer, in inches, and resilient modules of subgrade, in psi) (1 in = 2.54 cm; 1 psi = 6.9kN/m^2)

FIGURE 16. EXAMPLES OF RELATIONSHIPS TO PREDICT TENSILE STRAIN (AFTER(11))

laboratory equation based upon analyses of the AASHO Road Test results. Their findings indicated that the number of traffic loads required to initiate fatigue distress is on the order of 13 to 18 times that predicted by constant-stress laboratory tests.

Because of limitation in the data available from the AASHO Road Test, field distress models could be estimated at only two levels of cracking: (1) cracking less than or equal to 10 percent of the wheelpath area; and (2) cracking equal to or greater than 45 percent of the wheelpath area. The equations presented in ref. (6) are as follows:

$$\log N_f\ (\leq 10\%) = 15.947 - 3.291 \log \varepsilon_t - 0.854 \log E^* \quad (19)$$

$$\log N_f\ (\geq 45\%) = 16.086 - 3.291 \log \varepsilon_t - 0.854 \log E^* \quad (20)$$

where

N_f is the number of loads of constant stress necessary to cause fatigue cracking;

ε_t is the initial tensile strain at the bottom of the asphalt layer for the applied stress, 10^{-6} in/in; and

E^* is the complex modulus of the asphalt concrete surface, ksi

For purposes of the EAROMAR analysis we would like to interpret eqs. (19) and (20) in terms of total pavement area rather than wheelpath area. Since the wheelpaths for that study were assumed to total about two meters or six feet in width, eq. (19) would apply to cracking less than or equal to 5 percent of pavement area; and equation (20), to cracking equal to or greater than about 25 percent of pavement area. This interpretation does not change any numerical parameters in these equations, but simply the description of their range of applicability.

The identification of separate equations for different levels of cracking, however, is unsuitable for the EAROMAR requirements, since it is the extent of cracking which we seek to predict in the first place. In the absence of any better field-laboratory correlations of fatigue cracking, it appears reasonable to select a relationship midway between eq. (19) and (20). The following equation is proposed for use in EAROMAR; it represents an increase in predicted allowable loads to initiate fatigue distress of about 15 times that given by laboratory results.

$$\log N_f = 16 - 3.291 \log \varepsilon_t - 0.854 \log E^* \quad (21)$$

where

N_f is the number of load applications of constant stress to cause fatigue cracking;

ε_t is the initial tensile strain at the bottom of the asphalt layer computed from eq. (12), 10^{-6} in/in; and

E* is the asphalt concrete complex modulus input by the user, ksi,

The application of eq (21) within each season of each year will define a number of allowable loads N_i. From traffic volume and composition data (discussed in Chapter 5) the actual number of seasonal loads n_i can also be estimated. The ratio of these two items can then be computed and summed to obtain estimates of cumulative fatigue damage according to Miner's hypothesis, eq (9). The one remaining step is then to relate cumulative damage to the extent of areal cracking predicted on the pavement surface.

AREAL FATIGUE CRACKING

A relationship between observed areal cracking and fatigue damage according to Miner's Law was reported by Darter and Barenberg (7) and is reproduced in Figure 17. Cracking here includes Classes 1, 2 and 3, whereas only Classes 2 and 3 were included in the formulation of the AASHTO serviceability equation. Also, the scatter in the data in Figure 17 indicate that, at best, only an approximate relationship is possible at this time.

The data in Figure 17 suggest a relationship as follows:

$$CI = 210\ (\log D)^{0.947},\ D \geq 1.0 \qquad (22)$$

where

CI is the area of cracking, sf/1000 sf; and

D is the cumulative damage.

We have modified eq. (22) to convert the units of predicted damage to square feet of cracking per lane mile, consistent with other EAROMAR damage prediction models. The resulting relationship used within EAROMAR is then:

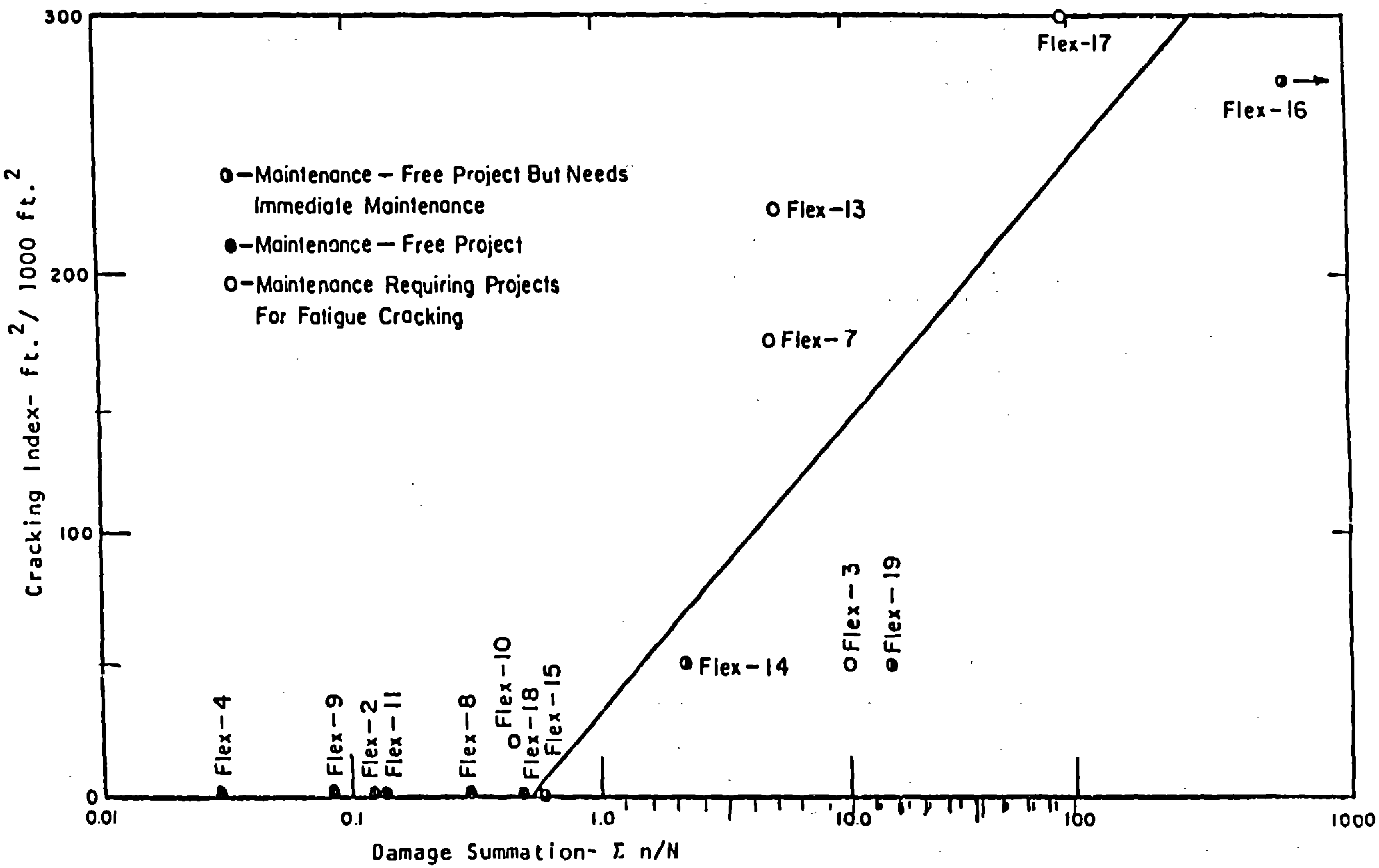


Figure 17. ACCUMULATED FATIGUE DAMAGE VERSUS CRACKING INDEX FOR FLEXIBLE PAVEMENTS (7)

$$C = 13300\ (\log D)^{0.947},\ D \geq 1 \qquad (23)$$

where

C is the area of fatigue cracking, sf/lane mile; and

D is the cumulative damage predicted by eq. (9).

Linear Cracking

Linear cracking in flexible pavements is generally attributed to thermal or shrinkage mechanisms, whether low-temperature shrinkage cracking or thermal fatigue active within the surface layer, or reflection cracking arising in a stabilized base layer or frozen granular base layer.

Several mechanistic studies have been performed to estimate pavement reflection cracking initiated in the base layer. Carpenter, Lytton and Epps (12) investigated cracking of pavements in West Texas, using finite element analysis to establish freeze-thaw contraction of the unbound base layer as an important contributing mechanism. Pretorius and Monismith (13) applied finite element techniques to simulate fatigue cracking of an asphalt concrete surface placed over a soil cement base in which shrinkage cracks have already developed. Carpenter and Lytton (14) report a procedure to predict thermally induced cracking based on the concept of susceptibility of frozen unstabilized base course material to crack formation and propagation. All of these studies, however, rely on the application of theoretical models to predict stress/strain distributions through the pavement layers, and do not afford sufficient data from which closed-form approximations can be derived. Thus, at this time we are unable to include a model for flexible pavement reflection cracking within EAROMAR.

Both low-temperature and thermal-fatigue mechanisms have been investigated analytically for flexible pavements. Shahin and McCullough (15) and Shahin (16) describe a stochastic model to treat both of these temperature-related phenomena, and have obtained fairly good agreement with observed highway cracking in Canada. However, again this model has not been reduced to closed-form relationships, and its method of predicting thermal fatigue (by calculating, for each time period and each pavement section, cumulative damage levels at different normal-distribution significance levels, and then selecting that significance level corresponding to cumulative damage equal to 1.0) would likely prove cumbersome under the EAROMAR approach.

We have therefore selected a more adaptable model of temperature-related linear cracking, based upon work by Hajek and Haas (17). This model estimates the amount of transverse cracking over time due to low-temperature shrinkage of the asphalt concrete surface. Thermal fatigue is not included. The following equation was developed by Hajek and Haas based upon regression analyses of 32 pavement sections in Canada:

$$10^{I} = 2.497 \times 10^{30} \times S^{(6.7966 - 0.8740\, t + 1.3388a)}$$
$$\times (7.054 \times 10^{-3})^{d} \times (3.193 \times 10^{-13})^{m}$$
$$\times\, d^{0.6026\, S} \qquad (24)$$

where

I is a cracking index equalling the number of full and half-transverse cracks per 500-ft section of road;

S is the stiffness of the original asphalt cement determined for temperature m and loading time of 20,000 sec by modified McLeod method, ($kg/cm^2 \times 10^{-1}$);

t is the thickness of all asphalt concrete layers, in.;

a is the age of asphalt concrete layers, years;

d type of subgrade (dimensionless code: 2 is clay, 3 is loam, and 5 is sand); and

m is the winter design temperature, deg C x -0.10.

Some minor modifications have been made in this model to have it conform to the EAROMAR framework. These are described below.

First, the winter design temperature is defined as "the lowest temperature at or below which only 1 percent of the hourly ambient air temperatures in January occur for the severest winter during a 10-year period." (17) However, to obtain and analyze climatological data necessary to determine this design temperature might prove cumbersome for premium pavement studies in different regions of the country. Instead, it was decided to compute this design temperature from the freezing index, using a correlation suggested in (17) and shown in Figure 18 developed from data for Ontario and southern Manitoba. Also, the units of this relationship were converted to a Fahrenheit scale. The resulting equation is then:

$$T_w = -8.064 \log^2 FI + 61.27 \qquad (25)$$

where

T_w is the winter design temperature, °F; and

FI is the local freezing index (input by the user).

Second, it was felt desirable to suppress the dimensionless "d" term expressing subgrade type. This is not to say that subgrade type has no influence on crack frequency; nevertheless, the use of an arbitrary term is awkward, since it does not identify what characteristics of the subgrade are important to this analysis, and thus it cannot be extended to other subgrade types. The results of a sensitivity analysis performed by Hajek and Haas (as illustrated in Figure 19) indicate that the effect of subgrade type on cracking is insignificant except at extremely low design temperatures (Obs. 29 in Figure 19, -40°C or -40°F). Based upon this reasoning we fixed the value of "d" equal to 3.

Third, although eq. (25) is based upon the total thickness of all asphalt layers, there is no provision within EAROMAR for labeling individual layers by material type. (The preference is to characterize materials by their relevant physical parameters.) Data from Table 8 in reference (18) suggest that asphalt concrete and asphalt treated layers are associated with AASHTO layer coefficient values of about 0.25 or higher. For purposes of this model, then, the total asphalt thickness is taken to be the sum of the thicknesses of all contiguous layers, beginning with the surface layer, having layer coefficients greater than or equal to 0.25.

Finally, we adopted a standard system of units of measurement, rewrote eq. in its equivalent logarithmic form, and converted the units of the predicted cracking index I into the predicted length of cracking in lineal feet per lane mile. The resulting model used within EAROMAR is:

$$I = (861.1 - 110.7\ t + 169.6\ A) \log (S/0.1419)$$

$$+ 88.0\ T_w + 257\quad S + 220.5 \qquad (26)$$

where

I is the amount of cracking, lineal feet per lane mile;

t is the thickness in indices of all asphalt concrete layers having an AASHTO layer coefficient of at least 0.25;

A is the age of asphalt concrete layers, years;

S is the stiffness of the asphalt cement, ksi, determined for temperature T_w and loading time of 20000 seconds by the modified McLeod method; and

T_w is the winter design temperature computed for eq. (25), °F.

The simulation of all linear cracking occurs within EAROMAR during the coldest season of the year, as determined from mean seasonal temperatures input by the user.

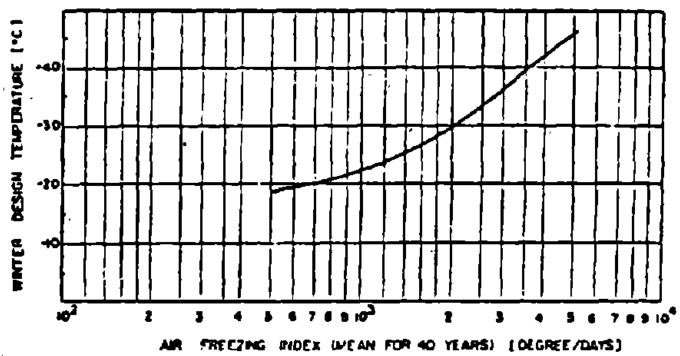


FIGURE 18. WINTER DESIGN TEMPERATURE VS. FREEZING INDEX (17)

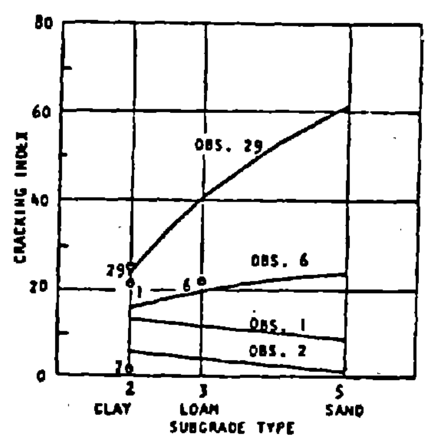


FIGURE 19. EFFECT OF SUBGRADE ON CRACKING INDEX (17)

Longitudinal Roughness

The longitudinal roughness of a pavement is a measure of the deviation of its longitudinal profile (measured, for example, in the wheelpath) from a smooth reference plane. Results of the AASHO Road Test (4) indicate that roughness plays a significant role in determining ride quality perceived by the user, as measured by Present Serviceability Index (PSI).

Despite the importance of this parameter, however, roughness has received relatively little attention (in comparison with rutting and cracking) among models to predict pavement damage. One likely reason is that roughness results from non-homogenous deformations in the pavement surface, and accounting for this inherent variability in pavement response is difficult using deterministic methods. As an example of the approach required to estimate roughness as a function of traffic and the environment, Brademeyer (19) applied a spatial auto-correlation function representing differences in materials properties and quality of construction along the pavement surface. However, this idea has not yet been reduced to a closed-form relationship suitable for use within EAROMAR.

To obtain a model usable within EAROMAR we rely instead on the high degree of correlation observed between roughness and PSI. Models are available to estimate the decline in serviceability as a function of pavement characteristics, cumulative traffic loads, and environment. Since the level of serviceability measured by PSI is dominated by the amount of roughness (expressed as slope variance), it is possible to calculate approximately the amount of roughness present at a given PSI level. This procedure thus allows one essentially to predict roughness as a function of relevant pavement, traffic, and environmental variables.* Derivation of the model is described below.

PAVEMENT SURFACE DETERIORATION OVER TIME

The deterioration of pavement serviceability over time is given by the general AASHTO Road Test equation (4 , 5);

$$G_t = \beta (\log W_t - \log \rho) \tag{27}$$

* We will <u>not</u> use this set of relationships to predict the future pavement PSI within EAROMAR. PSI will be estimated instead in the traditional way, as a function of roughness, rutting, and cracking, as described in a later section. All we are proposing here is the transformation of a time-dependent PSI function to an equivalent function to predict roughness.

where

G_t is the logarithm of the ratio of loss in serviceability at time t to the potential loss taken to a point where $p_t = 1.5$;

β is a function of design and load variables influencing the shape of the p vs. W curve;

W_t is the cumulative axle load applications at the end of time t;

ρ is a function of design and load variables that denotes the expected number of axle load applications to a serviceability index of 1.5; and

p_t is the serviceability at the end of time t.

Moavenzadeh and Brademeyer (20) have rendered this relationship in the following form:

$$p_t = p_o - 2.7 \times [\, 1.585 \times W_t \times R \times (SN + 1)^{-9.36} \times 10^{-.372(S - 3)} \,]^{\beta} \qquad (28\ )$$

where

p_t is the serviceability at end of time t;

p_o is the initial serviceability;

W_t is the cumulative number of equivalent 18-kip single axle loads at the end of time t in the design lane;

R is the regional factor;

SN is the pavement structural number;

S is the soil support value; and

$$\beta = 0.4 + 1094/(SN + 1)^{5.19}. \qquad (29)$$

All of the independent variables in eq. (28) are known to the EAROMAR simulation. The initial serviceability p_o of a flexible pavement can be taken from AASHO Road Text experience to be 4.2. The number of 18-kip axle applications will be computed from EAROMAR's simulation of traffic flow over time and from vehicle characteristics, as described in Chapter 5. Pavement structural number is computed from layer descriptions input by the user (Chapter 2), while regional factor and soil support value are likewise provided by the user as part of the roadway description. If the subgrade strength is entered by a static CBR value instead of the soil support value, the following conversion is made:*

$$S = 1.45 + 3.46 \log CBR \tag{30}$$

where

S is the AASHTO soil support value; and

CBR is the static CBR.

CORRELATIONS OF ROUGHNESS WITH SERVICEABILITY

Although loss of pavement serviceability is associated with cracking and rutting as well as roughness, data on 74 selected pavement sections in Appendix F of the AASHO Road Test Report (4) indicate that longitudinal roughness typically accounts for 85 to 95 per cent of any decline in PSI. Plots of slope variance vs. serviceability such as shown in Figure 20 also indicate a strong correlation between these two variables. Thus if a relationship between PSI and roughness can be defined, eq. (28) can be used to predict roughness in lieu of serviceability.

The desired relationship is suggested in Figure 21 (38), which compares roughness measurements and serviceability ratings from Ontario and Minnesota respectively. The very close agreement between the two plots leads to the following equation:

$$R = 355 \times 10^{-PSI/5} \tag{31}$$

where

R is the pavement roughness in inches per mile; and

PSI is the present serviceability index.

PROPOSED MODEL

We stressed earlier that the purpose of eq (28) is not to predict pavement serviceability as such, but rather to lead to a prediction of roughness. To emphasize this point we combine equations (28) and (31) to suppress any reference to PSI. The resulting model for longitudinal roughness is then:

$$\log R = 1.71 + 0.54 \left[1.585 W_t \times RFx (SN + 1)^{-9.36} \times 10^{-.372(S - 3)} \right]^{\beta} \tag{32}$$

where all variables are as defined earlier.

---

* This equation is based on a comparison of soil support value with static CBR values in the AASHTO Interim Guide (5).

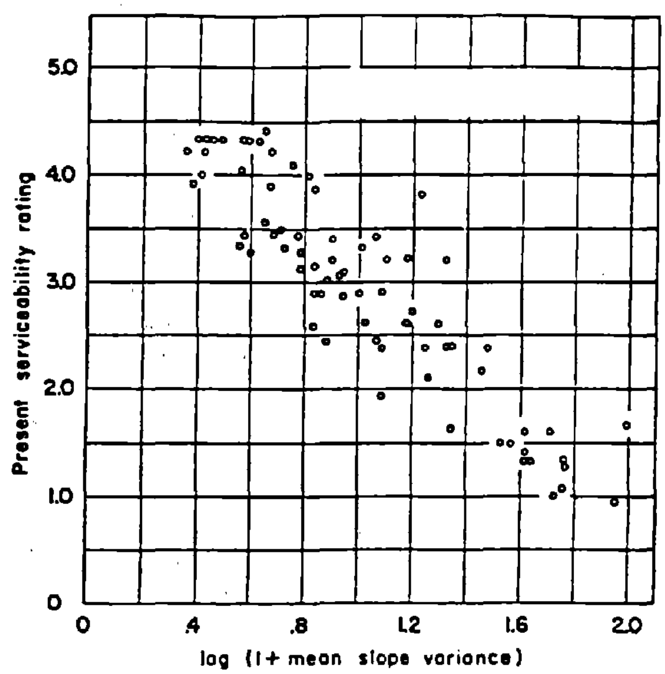


FIGURE 20. SERVICEABILITY VS. LOG (1+SLOPE VARIANCE)
FOR 74 FLEXIBLE PAVEMENTS (4)

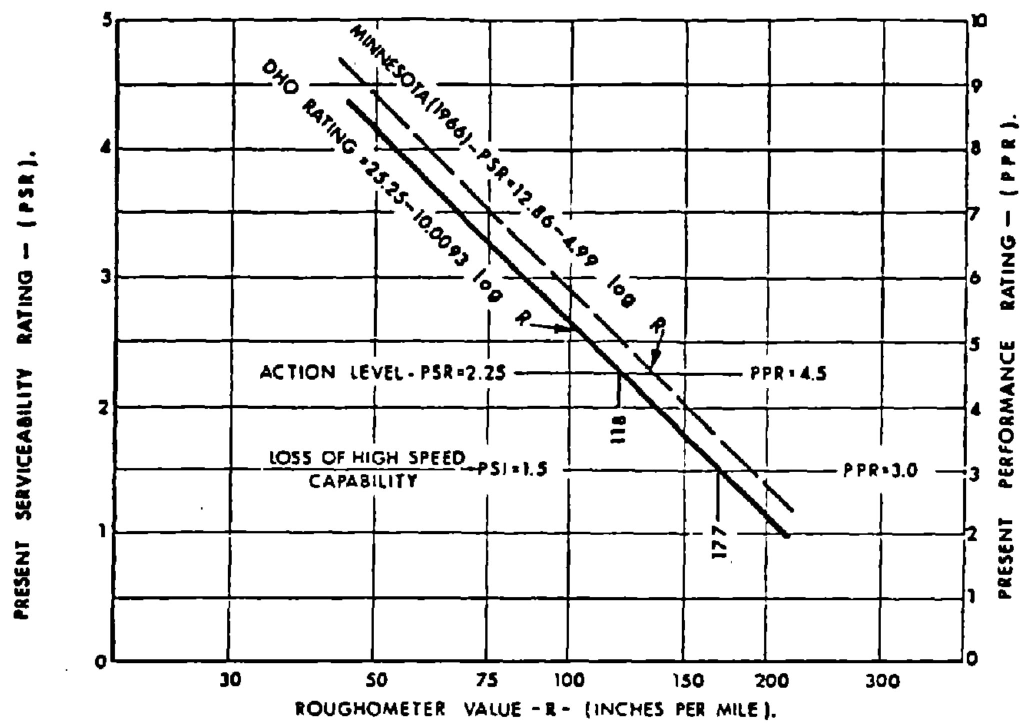


FIGURE 21. COMPARISON BETWEEN SERVICEABILITY AND ROUGHNESS (38)

Rutting

Rutting in the wheelpaths results from the permanent deformation in one or more pavement layers, influenced by the number of traffic loadings, layer properties, and environment. Analysis of the AASHO Road Test sections determined that lateral movement of material in the subbase accounted for most of the rutting observed, although some was attributable also to increases in density among the various layers. The rate of rutting appears to decrease over time, at least as long as pavement structure integrity is maintained.

Rutting has been addressed in mechanistic models such as VESYS (9) and PDMAP (6). Again, these solutions entail a detailed simulation of the pavement structure, and would require additional research to obtain closed-form approximations to be usable within EAROMAR.

An approach more suitable for use within EAROMAR was reported by Meyer and Haas (21) based upon analyses of the Brampton Test Road. Their regression model is as follows:

$$\begin{aligned} RD = & - 1.0318 + 1.2067\ AT + 0.0803\ N - 2.3684\ \ln (AT) \\ & + 0.1896\ \ln (AT \times N) + 1.1639\ E_1 \ln (AT) \\ & - 0.0216\ E_2 N - 0.4114\ E_1 N \ln (AT) + 0.0456\ E_2 N \ln (AT) \end{aligned} \quad (33)$$

where

- RD is the permanent deformation in inches;
- AT is the effective asphalt thickness in inches/10 (with 1" of hot mix = 2" granular base = 3" subbase);
- E_1 is the modulus of the asphalt layer, psi/10^6;
- E_2 is the modulus of the subgrade, psi/10^4; and
- N is the cumulative number of 18-kip single axle loads/10^5.

Although it is difficult to obtain physical insight into pavement behavior from this model, the equation is of closed form and includes the key influencing parameters as independent variables. (Environmental effects can be accounted for by specifying seasonal dependence of the layer moduli, as discussed in Chapter 2.) The specification of the equivalent asphalt thickness, however, is arbitrary; for use within EAROMAR we would have to key the layer conversion factors above to values of the AASHTO layer coefficients. A suggested way to do this is as follows:

AASHTO Layer Coefficient	Equivalent Asphalt Thickness Used in Rutting Model
< 0.14	0.33
0.14 - 0.25	0.5
> 0.25	1.0

Our original intent was to use this model directly within EAROMAR to predict rutting. However, trials with different flexible pavement designs produced anomalous results. Eq. (33) is insensitive to pavement modulus E_1 when the (equivalent) asphalt thickness is around 10 inches (or AT is about 1.0), and certain terms in the model exhibit reversals of sign as E_1 is varied. Furthermore, because of logarithmic terms in N, the model produces invalid results at low levels of cumulative traffic (i.e. when the pavement is new and N is small).

We therefore restructured the model to retain its sensitivity to traffic loadings and materials properties, but to eliminate inconsistent behavior. Eq.(33) was exercised for different combinations of E_1, E_2 and AT over ranges of N. New functional forms were then fitted to these results. The final model developed for use within EAROMAR is as follows:

$$RD = R_{max} \left(1 - \exp[-W/W^*]\right) \tag{34}$$

$$\ln R_{max} = 1.95 - 1.04\ H + 0.85\ E_1 - 0.40\ E_2 \tag{35}$$

$$\ln W^* = 3.06 - 0.72\ H + 3.50\ E_1 - 0.68\ E_2 \tag{36}$$

where

RD is the mean rut depth in inches;

R_{max} is the limiting rut depth in inches;

W is the cumulative number of 18-kip single axle loads/10^5

W^* is $\sqrt{W_{max}}$, where W_{max} is the asymptotic number of axle loads/10^5;

H is the effective asphalt thickness in inches/10 (as computed above for AT);

E_1 is the modulus of the asphalt concrete input by the user, in ksi/10^3; and

E_2 is the subgrade modulus input by the user, in ksi/10.

Serviceability

The serviceability of a flexible pavement is expressed by its present serviceability index (PSI). In any season and year during the EAROMAR simulation the PSI is estimated as a function of current surface damage according to the AASHTO relationship (4):

$$PSI = 5.03 - 1.91 \log (1+SV) - 1.38\ RD^2 - 0.01\sqrt{C + P} \quad (37)$$

where

PSI is the present serviceability index;

SV is the mean slope variance in the wheelpaths, radians^2 x 10^6;

RD is the mean rut depth, inches;

C is the area of class 2 plus 3 cracking, sf per 1000 sf; and

P is the area of pavement patching, sf per 1000 sf.

The growth in surface damage, leading to declines in PSI, is computed from the damage models discussed above. Rutting is predicted using eq. (34); areal cracking, from eq. (23); and lineal cracking, from eq. (26). Lineal cracking is included in the PSI computation within EAROMAR; a conversion of 1 lf of crack = 1 sf of cracking is assumed. Slope variance (x 10^6) is obtained from the estimate of roughness in eq. (32), using the following conversion*:

$$SV = 0.000117\ R^{2.334} \quad (38)$$

where

SV is mean slope variance in radians^2 x 10^6; and

R is the longitudinal roughness, in inches per mile.

All damage terms in eq. (37) are converted to equivalent quantities per lane mile before computing PSI on a lane-mile basis.

---

* This relationship is estimated from observations of both roughness and slope variance for selected flexible pavement sections reported in Appendix F of the AASHO Road Test Report (4).

The quantity of surface damage present at any time is also a function of past pavement maintenance and rehabilitation performed. Calculations of the amount of damage repaired in response to the maintenance policy specified are explained and illustrated in Chapter 4, and these adjustments in cumulative pavement damage are also taken into account in the calculation of PSI. (Note that eq. (32) used in the roughness model includes neither maintenance nor the seasonally-dependent variables which build the damage models. These shortcomings justify our earlier position that eq. (32) is inappropriate to predict serviceability within EAROMAR and is to be viewed merely as a constituent of the roughness model.)

Summary of Flexible Pavement Damage Relationships

Below we summarize the equations developed for the flexible pavement damage models within EAROMAR. To impose consistency among these models developed from different sources, we have restructured some of the equations to unify notation and to express constants generally to no more than three significant digits to the right to the decimal.

FATIGUE CRACKING

Maximum Tensile Strain.

$$\varepsilon_t = K_1 \exp\,[-K_2 \log M_R] \tag{39}$$

$$K_1 = 10^{-m} \; H + b \tag{40}$$

$$m = 0.0482 \exp\,[-0.0245\, E_2] \tag{41}$$

$$b = 4.818 \exp\,[-0.0103\, E_2] \tag{42}$$

$$K_2 = 1.831 \exp\,[-K_3\, E_2] \tag{43}$$

$$K_3 = 0.0244 \exp\,[-0.0512\, H] \tag{44}$$

$$H = \sum_i h_i' = \frac{1}{a_1} \sum_i h_i\, a_i \tag{45}$$

where

ε_t is the maximum initial tensile strain in the surface layer, 10^{-6} in/in;

M_R is the diametral resilient modulus of the flexible surface layer, in ksi, input by the user;

H is the equivalent surface thickness in inches in a two-layer representation of the pavement structure;

E_2 is the resilient modulus of the subgrade, in ksi, input by the user ;

h_i' is the effective thickness in inches contributed to the surface by each base or subbase layer i;

h_i is the actual thickness of layer i, in inches, input by the user;

a_i is the AASHTO structural coefficient of layer i input by the user; and

a_1 is the AASHTO structural coefficient of the flexible surface layer input by the user.

<u>Fatigue Damage</u>. The number of allowable loads is computed as follows:

$$\log N_i = 16 - 3.291 \log \varepsilon_t - 0.854 \log E_i^* \tag{46}$$

where

N_i is the number of load applications of constant stress to cause fatigue cracking through season i;

ε_t is the initial tensile strain at the bottom of the asphalt layer computed from eq. (39), 10^{-6} in/in; and

E_i^* is the asphalt concrete complex modulus, in ksi, input by the user for season i.

The applied loads contributing to fatigue are computed as follows:

$$n_i = \sum_{j=1}^{2} F_{ij} \times w_{ij} \tag{47}$$

where

n_i is the number of fatigue loads applied during season i;

F_{ij} is the numberof weekdays (j=1) or weekends (j=2) within season i, computed internally; and

w_{ij} is the number of 18-kip single axle loads in the design lane computed in EAROMAR by season and type of day.

The fatigue damage according to Miner's hypothesis is computed as follows:

$$D_i = \frac{n_i}{N_i}\ ; \qquad D = \sum_i D_i \tag{48}$$

where

D_i is the fatigue damage occurring in season i; and

D is the cumulative fatigue damage; and all other variables are as defined above.

<u>Areal Fatigue Cracking</u>.

$$ACRACKS = 13300\ (\log D - 1.176)^{0.947}, \tag{49}$$

$$D \geq 15.$$

ACRACKS is the amount of areal fatigue cracking, in square feet per lane mile added this season; and

D is the cumulative fatigue damage estimated by eq. (48).

LINEAR CRACKING

$$LCRACKS = (861.1 - 110.7\ H' + 169.6\ A) \times \log\ (M_s/0.142) + 55.0\ T_w + 257\ M_s + 220.5, \quad (50)$$

$$T_w = -8.064\ \log^2 FI + 61.27 \quad (51)$$

where

LCRACKS is the amount of lineal thermal cracking, lineal feet per lane mile;

H' is the sum of the thicknesses, in inches, of all contiguous pavement layers (beginning with the surface layer) having AASHTO layer coefficients greater than or equal to 0.25;

A is the age of the asphalt concrete pavement, in years, input by the user and updated during the simulation;

M_s is the stiffness of the asphalt cement, in ksi, determined for temperature T_w and loading time of 20,000 seconds by the modified McLeod method;

T_w is the winter design temperature, °F; and

FI is the local freezing index, input by the user.

Linear cracking is simulated for a flexible pavement only during the coldest season of the year, as determined from seasonal temperature data input by the user.

LONGITUDINAL ROUGHNESS (52)

$$\log \text{ROUGHNESS} = 1.71 + 0.54\,[1.585\, W_t \times R \times (SN+1)^{-9.36} \times 10^{-0.372(S-3)}]^{\beta},$$

$$\beta = 0.4 + 1094/(SN+1)^{5.19} \qquad (53)$$

where

ROUGHNESS	is the pavement roughness in inches per mile;
W_t	is the cumulative number of 18-kip single axle loads in the design lane at the end of time t computed during the simulation;
R	is the AASHTO regional factor input by the user;
SN	is the pavement structural number computed from layer data input by the user; and
S	is the AASHTO subgrade soils support value input by the user.

RUTTING

$$RUTS = R_{max}(1-\exp[W_t/W_t^*]) \qquad (54)$$

$$\ln R_{max} = 1.95 - 1.04\ H'' + 0.85\ E_1 - 0.40\ E_2 \qquad (55)$$

$$\ln W_t^* = 3.06 - 0.72\ H'' + 3.50\ E_1 - 0.68\ E_2 \qquad (56)$$

$$H'' = \frac{1}{10} \sum_i F_t \times h_i \qquad (57)$$

where

RUTS is the permanent deformation of the pavement surface due to rutting, in inches;

R_{max} is the limiting rut depth in inches;

W_t is the cumulative number of 18-kip axle loads in the design lane, computed during the simulation;

W_t^* is equal to $\sqrt{W_{max}}$, where W_{max} is the asymptotic number of axle loads/10^5;

H'' is the effective asphalt layer, in ksi/10^3, input by the user;

E_1 is the modulus of the asphalt layer, in ksi/10^3, input by the user;

E_2 is the subgrade modulus, in ksi/10, input by the user;

F_t is a factor used to compute the equivalent asphalt thickness, according to the following relationship:

AASHTO Layer Coefficient	F_t
< 0.14	0.33
0.14 - 0.25	0.50
> 0.25	1.0

and

h_i is the thickness of pavement layer i, in inches, input by the user.

SERVICEABILITY

$$PSI = 5.03 - 1.91 \log (1+SV) - 1.38 (RUTTING')^2 - 0.01 \sqrt{C+P} \quad (58)$$

$$SV = 0.000117 (ROUGHNESS')^{2.334} \quad (59)$$

$$C = ACRACKS'/63.36 + LCRACKS'/63.36 \quad (60)$$

$$P = AREA_{Patched}/63.36 \quad (61)$$

where

PSI is the present serviceability index;

SV is the mean slope variance, in $radians^2 \times 10^6$;

RUTTING' is the net mean rut depth, in inches, resulting from both accumulation of damage and past maintenance performed;

C+P is the area of cracking plus patching, in sf/1000sf;

ROUGHNESS' is the net roughness, in inches per mile, resulting from both accumulation of damage and past maintenance performed;

ACRACKS'
LCRACKS' are the net amount of cracking in sf/lane mile and lf/lane mile respectively, resulting from both accumulation of damage and past maintenance performed; and

$AREA_{Patched}$ is the cumulative area of pavement patching simulated by the model's maintenance routines, in square feet per lane mile.

3.3 RIGID PAVEMENT

Rigid pavement damage components and their treatment within EAROMAR are identified in Table 10. Models to predict rigid pavement damage require the data listed in Table 11 from the roadway descriptions provided by the user (Chapter 2). Explanations of each of the rigid pavement models follow.

Lineal Cracking

Both transverse and longitudinal cracks appear in rigid pavement slabs. Field studies of plain jointed and jointed reinforced concrete pavements by Darter and Barenberg (7) indicate, however, that transverse cracking is generally the more important of the two in considering potential zero-maintenance performance. Therefore the prediction of lineal cracking by EAROMAR focuses on transverse cracking alone.

Transverse cracking may be induced by fatigue or by environmental stresses. Fatigue can arise through a number of causes: excessive traffic loads (in terms of the weight or the cumulative number of vehicle passes); inadequate slab thickness or strength; or loss of subgrade support (through erosion). Environmentally induced damage can result through curling and slab friction (which depend on temperature and slab length), and joint lockup (especially where deicing salts are used).

The model described below is based upon relationships developed by Darter (22) for the design of zero-maintenance plain jointed concrete pavement. This procedure empahsizes load fatigue and slab curling as the primary damage mechanisms. The presentation below follows a sequence similar to that employed for flexible pavement fatigue earlier.

MAXIMUM TENSILE STRESS OR STRAIN

Maximum tensile stresses at the edge of a slab were computed by Darter using a finite element program to analyze the effects of slab thickness, load configurations and location, subgrade support, and temperature gradient (variable throughout the day). These results were then used to derive independent equations for load stress (STRL) due to traffic and curl stresses (STRC) due to thermal gradients in the absence of loads. It was found that load and curl stresses are additive if an adjustment factor R is applied first to the curl stress term. The resulting system of regression equations for total stress at the slab edge when subjected to an edge load is as follows:

$$STRT = STRL + (R)STRC \tag{62}$$

where

STRT = total resultant stress in the longitudinal direction at the bottom of the PCC slab edge when the wheel load is located at the slab edge (load is single axle or tandem axle) ;

TABLE 10

TREATMENT OF RIGID PAVEMENT
DAMAGE COMPONENTS WITHIN EAROMAR

	Damage Component Or Serviceability Index	Prediction Model Included Within EAROMAR	User May Input Rate Directly
1.	Linear Cracking	X	X
2.	Areal Cracking		X
3.	Roughness	X	X
4.	Faulting	X	X
5.	Joint Stripping		X
6.	Spalling	X	X
7.	Blowups	X	X
8.	Pumping Joints	X	X
9.	Pavement-Shoulder Joints		X
10.	Present Serviceability Index	X	X

TABLE 11

DATA REQUIRED FOR RIGID PAVEMENT DAMAGE MODELS

Model	Input Data Required
Linear Cracking	Layer Thickness PCC Modulus of Rupture Modulus of Foundation Support Average Daily Temperature Slab Length PCC Coefficient of Expansion Pavement Age
Roughness	Layer Thicknesses PCC Modulus of Rupture PCC Elastic Modulus Modulus of Foundation Support
Faulting	Layer Thicknesses Joint Spacing Pavement Age Drainage Characteristics Percent Trucks Subbase Modulus
Spalling	Joint Spacing Pavement Age
Blowups	Joint Spacing Pavement Age Aggregate Susceptibility
Pumping	Layer Thicknesses PCC Slab Length Drainage Characteristics

STRL = stress at bottom of PCC slab edge when load is located at slab edge (no thermal curling stress);

STRC = stress at bottom of PCC slab edge caused by curling of slab due to thermal gradient (no traffic load);

R = adjustment factor for STRC so that it can be combined with STRL to give correct STRT,

The R ranges from about 0.8 to 1.5 depending on slab/foundation conditions.

The regression equations determined for these stresses are as follows:

Load Stress for single axle load: (63)

$$STRL = [LOAD/(18.0H^2)][17.35783 + 0.078\ ES - 0.05388H^3/k + 7.41722\ \log_{10}(H^3/k)]$$

Load stress for tandem axle load: (64)

$$STRL = [LOAD/(36.0H^2)][14.09599 + 0.10522ES - 0.09886H^3/k + 6.2339\ \log_{10}(H^3/k)$$

Curl Stress:

$$\begin{aligned} STRC - [(G)(ET)/(5 \times 10^{-6})][&0.006712k + 79.07391\ \log_{10}k + 11.72690L \\ &- 0.00720kL - 3.22139L\ \log_{10}k - 0.06883LES - 0.59539ES\ \log_{10}k \\ &- 204.39477H/k - 38.08854L/H - 8.36842H\ \log_{10}k + 0.07151ESH \\ &+ 0.05691LES\ \log_{10}k + 0.20845LH\ \log_{10}k + 0.00058LHk - 0.00201LESH \cdot \log_{10}k] \end{aligned} \quad (65)$$

R adjustment factor:

$$\begin{aligned} R = 0.48039 &+ 0.01401H - 0.00427ES - 0.27278G - 0.00403L + 0.19508\ \log_{10}k \\ &+ 0.45187G\ \log_{10}H - 0.00532G^2 + 0.01246GL - 0.00622GL\ \log_{10}k \\ &+ 8.7872\ \log_{10}(H^3/k)/H^2 + 0.00104GES - 0.11846G\ \log_{10}(H^3/k) \\ &+ 0.07001\ \log_{10}(ES + 1.0) - 0.01331G\ \log(ES + 1) \end{aligned} \quad (66)$$

where

LOAD = total load on single or tandem axle, pounds;

H = PCC slab thickness, inches;

G = thermal gradient through slab, °F/in.;

k = modulus of foundation support (top of subbase, pci) ;

L = slab length, ft.;

ES = erodability of support along slab edge, inches; and

ET = thermal coefficient of contraction of PPC/°F .

There are several comments on the use of these equations in EAROMAR. Within EAROMAR are expressed in terms of equivalent 18-kip (8.2 MT) single axle loadings; therefore only the equation for single axle loads is referenced from eqs. (63) and (64), with the variable LOAD set equal to 18,000 lbs (8.2 MT). Second, the width of erodability along the slab edge, ES, is dependent upon the extent of pumping, and will be estimated by the pumping model to be discussed shortly. Third, if the modulus of subgrade support is not input directly by the user, it is estimated using the relationship

$$k = 51.93\ CBR^{0.5876} \tag{67}$$

where

k is the modulus of foundation support, in pci; and

CBR is the static CBR.

Finally, the daytime and nighttime temperature gradients through a slab of arbitrary thickness have been related to slab thickness and seasonal average daily temperature, using data for several cities developed in (22). Estimates for daytime (0700 - 1900) and nighttime (1900 - 0700) are given respectively by:

$$G_d = \frac{(T/4)-1}{H + 2} \tag{68}$$

$$G_n = -\frac{(T/20)+5}{H + 2} \tag{69}$$

where

G_d, Gn are daytime and nighttime thermal gradients in the PCC slab in a given season (°F/in);

T is the seasonal average daily temperature in °F input by the user; and

H is the PCC slab thickness, in inches, input by the user.

FATIGUE DAMAGE

Fatigue damage is calculated using Miner's Law in a manner similar to that discussed for flexible pavement fatigue. This procedure requires estimates of the number of applied loads n_i during each time period i, and the total number of loads to failure N_f.

Applied Loads. The number of traffic loads applied each season is predicted from traffic volume and composition data discussed in Chapter 5. However, the point of application of wheel loads with respect to slab geometry, and the temperature conditions (varying by time of day) under which the loads are applied, influence the magnitude of the edge stress and thus the contribution to fatigue damage. Both these factors must therefore be accounted for in estimating the number of applied loads within the design lane.

The critical location for fatigue damage has been shown to lie at the mid-point of the longitudinal slab edge (22). The magnitude of the stresses at this location is influenced by the distance of the vehicle's outer wheel from the slab edge, a function of the lateral distribution of vehicles in the outer (assumed to be the design) lane. For purposes of calculating fatigue damage, Darter considers only that percentage of loads falling within 6 inches (15 centimeters) of the slab edge.

The lateral distribution of vehicles on a highway depends upon local road geometry and individual vehicle and driver characteristics, and should properly be measured as part of a rigid pavement study. As general guidelines, however, observations cited in (22) indicate that the average distance from outer wheel to slab edge lies typically within the range of 11 to 21 inches (28 to 53 cm), with variations in lateral placement normally distributed and having a standard deviation of 10 inches (25 cm). Statistics on the frequencies of truck wheel loads at various distances from the slab edge, as a function of the mean lateral displacement, are given in Table 12 (20).

To incorporate these effects of load location within EAROMAR we have adopted, as a general approximation, a normal distribution of lateral tire-to-slab-edge distance (according to Table 12) with an assumed mean lateral displacement of about 18 inches (38 cm). These assumptions result in an estimate of loads within 6 inches (15 cm) of the slab-edge as follows:

$$n_6 = 0.12 n_{dl} \qquad (70)$$

where

n_6 is the number of loads falling within 6 inches (15 cm) of the slab edge; and

n_{dl} is the total number of loads computed for the design lane.

Table 12. PERCENTAGE OF TRUCK WHEEL LOADS AT VARIOUS LATERAL DISTANCES FROM SLAB EDGE (22)

Position on Slab (or Shoulder), D	Mean Lateral Displacement ($\bar{D}$), ins. 12	24	36	42	48
< -3 ins.	6.68	0.35	0.01	--	--
-3 to +3 ins.	11.73	1.44	0.04	0.01	--
+3 to +9 ins.	19.80	4.89	0.30	0.04	0.01
9 to 15 ins.	23.58	11.73	1.44	0.30	0.04
15 to 21 ins.	19.80	19.80	4.89	1.44	0.30
21 to 27 ins.	11.73	23.58	11.73	4.89	1.44
27 to 33 ins.	4.89	19.80	19.80	11.73	4.89
33 to 39 ins.	1.44	11.73	23.58	19.80	11.73
39 to 45 ins.	0.30	4.89	19.80	23.58	19.80
45 to 51 ins.	0.04	1.44	11.73	19.80	23.58
51 ins.	0.01	0.35	6.68	18.41	38.21
	100.00	100.00	100.00	100.00	100.00

*Data computed from normal distribution with standard deviation = 10 ins. (254 mm), and mean $\bar{D}$ as indicated.

A second consideration in treating load effects on rigid pavement fatigue is the time of day during which loads are applied. Time of day is correlated with temperature gradients in the slab which affect thermal curling stresses, as discussed earlier. The number of loads imposed during daytime vs. nighttime, respectively, influences the pattern of superposition of load and environmental fatigue damage.

Traffic loadings within EAROMAR are simulated, within each season, on an hourly basis throughout the day (for both weekdays and weekends) as described in Chapter 5. For purposes of estimating rigid pavement fatigue, "daytime" is interpreted to run from 7 AM to 7 PM, with "nighttime" consuming the remaining twelve hours.

The number of applied loadings for purposes of rigid pavement fatigue thus computed as follows:

$$n_{id} = \sum_{j=1}^{2} D_{ij} \times (n_6)_{ijd} \tag{71}$$

$$n_{in} = \sum_{j=1}^{2} D_{ij} \times (n_6)_{ijn} \tag{72}$$

where

n_{id}, n_{in} are the numbers of fatigue loads incurred in season i during daytime (d) and nighttime (n) respectively;

j denotes the type of day (weekday or weekend);

D_{ij} is the number of weekdays (j=1) or weekends (j=2) within season i; and

n_6 is the number of seasonal 18-kip axle loadings predicted to fall within 6 inches (15 cm) of the slab edge.

Allowable Loads. The number of allowable loads N_f to rigid pavement fatigue depends upon the ratio of the total stress at the pavement slab edge to the PCC flexural strength. Total stress as a function of both traffic loads and slab curling is computed by the system of equations (62) - (66). Flexural strength is predicted by a time-dependent modulus of rupture, according to the relationships (22):

$$F_t = F_A \times F_{28} \tag{73}$$

$$F_A = 1.22 + 0.17 \log A - 0.05(\log A)^2 \tag{74}$$

where

F_t is the PCC modulus of rupture at any time t, in ksi;

F_A is a growth factor depending upon PCC pavement age;

F_{28} is the 28-day mean modulus of rupture of the PCC in ksi, input by the user; and

A is the time in years since the pavement slab was constructed, as input by the user and updated during the EAROMAR simulation.

The fatigue relationship developed in (22) is for design purposes, in that it represents not the mean fatigue curve, but rather a more conservative estimate providing a confidence interval of about one decade of load applications (or about 76 percent). The relevant equations incorporated within EAROMAR are as follows:

$$\log N_{id} = 16.61 - 17.61\ (STRT_d/(F_i \cdot 1000)) \tag{75}$$

$$\log N_{in} = 16.61 - 17.61\ (STRT_n/(F_i \cdot 1000)) \tag{76}$$

where

N_{id}, N_{in} are the numbers of allowable loads to failure through season i for daytime and nighttime (d and n) respectively;

$STRT_d$, $STRT_n$ are the total (load plus curl) stresses in psi at the slab edge for daytime and nighttime, computed using equation (62); and

F_i is the PCC flexural strength through season i computed using equation (73), in ksi.

Damage Equation. Based upon the results of the preceding two sections, the fatigue damage relationship incorporating Miner's hypothesis is constructed within EAROMAR as follows:

$$D = \sum_i \frac{n_{id}}{N_{id}} + \sum_i \frac{n_{in}}{N_{in}} \tag{77}$$

where

D is the cumulative fatigue damage;

n_{id}, n_{in} are the numbers of applied loads during daytime and nighttime predicted by eqs (71) and (72) respectively; and

N_{id}, N_{in} are the numbers of allowable loads predicted from PCC fatigue considerations for daytime and nighttime using eqs. (75) and (76) respectively.

LINEAR FATIGUE CRACKING

As with flexible pavement, it is necessary to relate the magnitude of rigid pavement fatigue damage to the macroscopic cracking likely to be observed in the field. Such a relationship is suggested in Figure 22 (22), and can be approximated by the equation

$$CI = 8.24 \, D^{0.33191}, \quad D \geq 0 \tag{78}$$

where

CI is the rigid pavement cracking in SF/1000 SF; and

D is the PCC fatigue damage computed form eq. (77).

Equation (78) can then be modified to convert the units of predicted damage to linear feet of cracking per lane mile, consistent with other EAROMAR damage prediction models. The resulting relationship used within EAROMAR is then:

$$C = 522 \, D^{0.332} \tag{79}$$

where

C is the amount of fatigue cracking, ft/lane mile; and

D_i is the cumulative fatigue damage predicted by eq. (77).

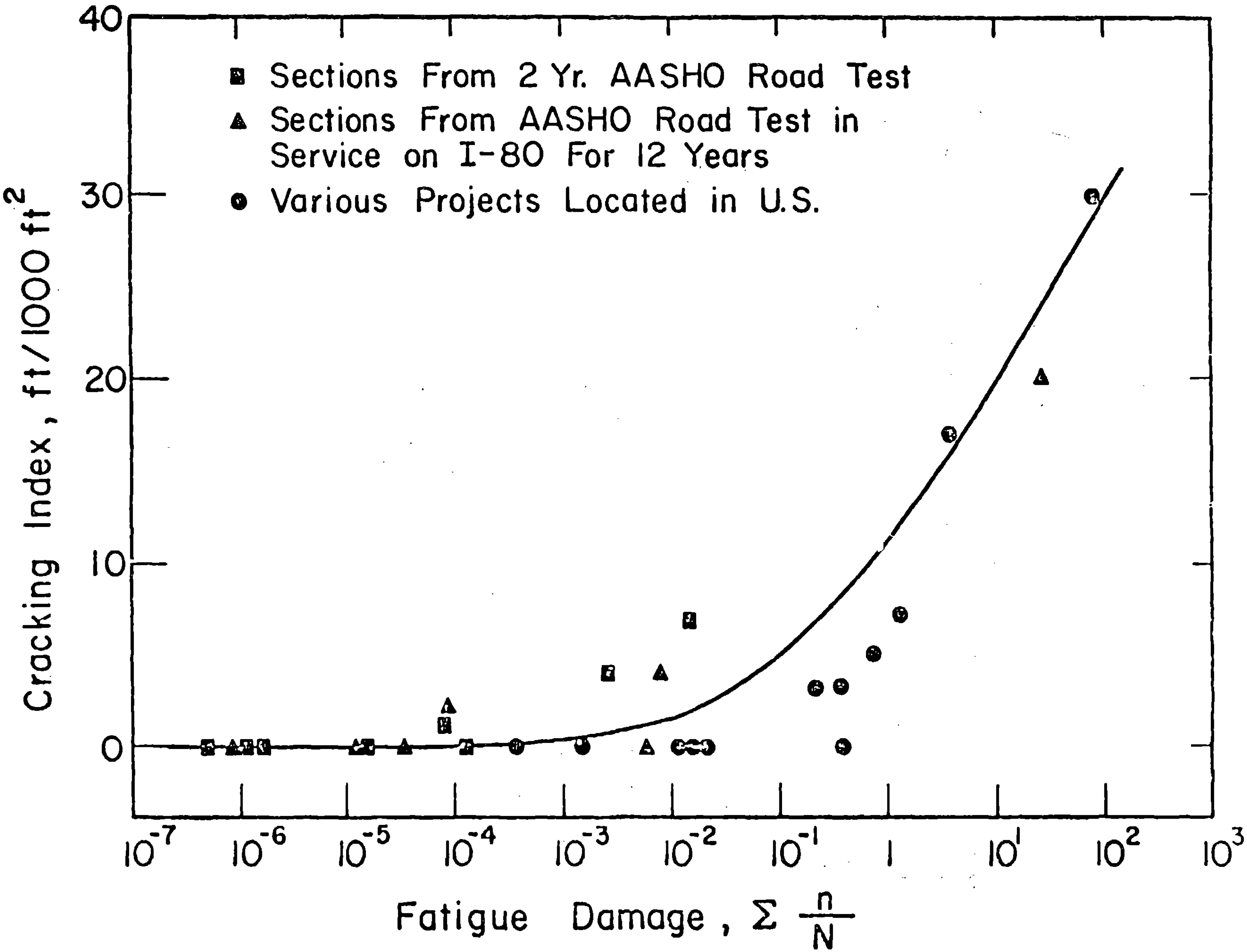


FIGURE 22. EFFECT OF FATIGUE DAMAGE ON CRACKING OF PCC PLAIN JOINTED CONCRETE PAVEMENTS. (22)

Faulting

Faults are vertical dislocations between adjacent slabs, at a transverse joint. Faulting is usually attributed to the action of moving traffic in causing a buildup of water-borne particles under the approach slab and erosion of base material on the leave side of the joint. The lack of effective load transfer at joints is also a major factor; faulting is generally more severe at non-doweled transverse joints, although doweled pavements also experience faulting (22).

NON-DOWELED PAVEMENTS

Studies of joint performance with regard to faulting in plain non-doweled concrete pavements were reviewed by Brokaw (23), who sought to relate the effects of joint roughness to user dissatisfaction and to pavement serviceability as defined by AASHTO. By converting measurements of faulting to slope variance at joints only, and by correlating these measurements with observation of pavement thickness and age, vehicle loads, and subgrade soil, Brokaw obtained the following relationships for plain non-doweled concrete pavement:

For A-1, A-2 and A-3 subgrade soils,

$$SVF = 1.92\ TA^2/D^{5.47}; \qquad (80a)$$

and for A-4, A-5, A-6 and A-7 subgrade soils,

$$SVF = 1.11\ TA^2/D^{4.60} \qquad (80b)$$

where

- SVF is the slope variance due to faulting at joints, radians^2 x 10^6 ;
- T is the average 2-way ADT of tractor - semi-trailer and combination vehicles;
- A is the age of the pavement in years; and
- D is the thickness of the slab, in inches.

More recently Packard (24) updated the review of field studies, and proposed a revised design procedure for controlling joint faulting of non-doweled pavements with short joint spacings (30 ft [9m] or less). Packard's model evolved from the premise that faulting represents a special kind of roughness in rigid pavements, whose effects on quality of ride are not well accounted for by AASHTO concepts of serviceability and slope variance. He contended that since the AASHTO pavements had experienced no significant faulting during the Road Test period, "severely faulted pavements affect serviceability much more adversely than indicated by the AASHTO model." Also, since slope variance is dependent upon not only the magnitude of the faults but also the joint spacing, Packard chose the average fault F_{avg} (measured in 32nds of an inch [0.8 mm]) as the measure of faulting for which it would be easier to assign physical interpretation.

By analyzing data on the faulting of non-doweled pavements in five states, Packard obtained the following relationship:

$$F_{avg} = 1.29 + 48.95 \frac{(TA^2)^{0.465}}{D^{3.9}} S^{0.610} (J-13.5)^b \qquad (81)$$

where

F_{avg} is the average fault, in 32nds of an inch (0.8 mm);

T is the number of tractor - semi-trailer and combination trucks in one direction (average number per day during A years of service);

A is the age of the pavement, in years;

D is the slab thickness, inches;

S is a factor denoting the drainage characteristics of the subgrade (1 for good, 2 for poor);

J is the joint spacing, in feet; and

b is a factor depending upon subbase characteristics, equaling 0.241 for granular bases, and 0.037 for stabilized bases.

With some minor modifications this model can be used to predict faulting within EAROMAR. Let us discuss each of the changes or clarifications in turn.

Measure of Faulting. The quantity F_{avg} is a measure of field damage that correlates well with observations of various pavements, as illustrated in Figure 23 (24). Within EAROMAR, however, such measures are used not only to express current pavement condition, but also to estimate maintenance requirements. The workload to maintain faulted joints is based on the number of such joints, rather than the absolute magnitude of faulting per se; therefore the quantity F_{avg} must be converted to an equivalent number of joints requiring attention.

This conversion is based upon an observation by Brokaw (23) that statistics such as "100 percent of joints faulted 0.15 [3.8 mm] and more" can be used as ratings of pavement riding comfort. This approach is further supported by Packard (24), who notes that serviceability declines gradually and uniformly until faults reach a level of 7/32 to 8/32 of an inch (0.22-0.25 in, or 5.6-6.4 mm), at which time the ride quality diminishes rapidly thereafter. Within the EAROMAR simulation, then, the extent of faulting is expressed by the number of faults of 0.25 in. (6.4 mm) per lane mile, using the following calculation:

$$F = \frac{5280}{J} \times \frac{F_{avg}}{32 \times 0.25} = \frac{165}{J} \times \frac{F_{avg}}{0.25}, \qquad (82)$$

$$F \leq 5280/J$$

where

F is the average number of faults of 0.25 inch (6.4 mm) per lane mile; and

F_{avg}, J are as defined earlier.

Traffic. The measure of traffic loadings T in equation (81) is an imprecise one, given that the weights of combination trucks can vary widely. In the absence of better information, however, the definition of T in equation (81) must be adhered to.

Within the EAROMAR simulation the number of tractor-semi-trailer and combination trucks can be approximated by using the percent of trucks in the traffic stream, defined (in the model) as the percentage of vehicles having a passenger-car-equivalent greater than 2.0 This percent of trucks, multiplied by the seasonally-adjusted AADT, will yield an estimate of the number of tractor-semi-trailer and

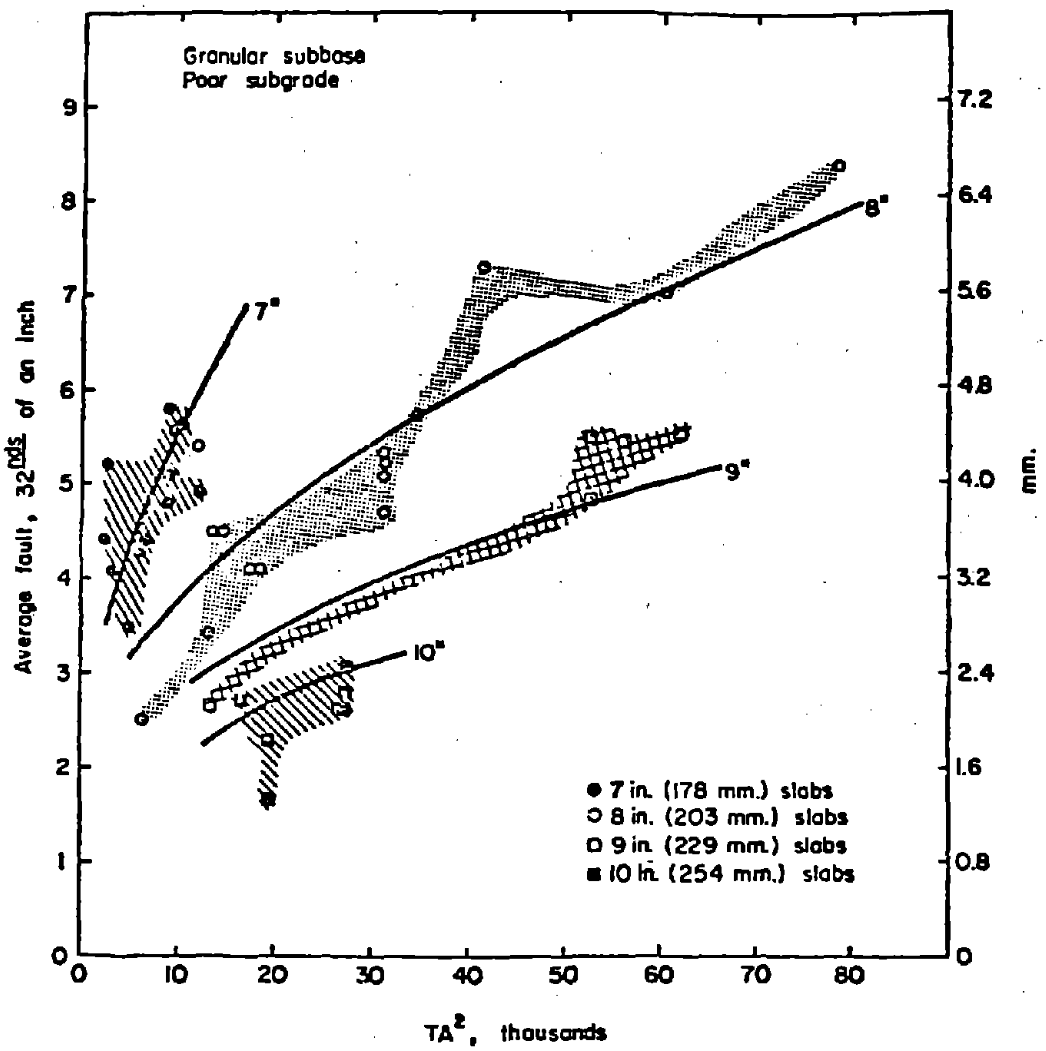


FIGURE 23. JOINT FAULTING IN MINNESOTA, NORTH DAKOTA, AND WISCONSIN (20-FT JOINT SPACING, POOR SUBGRADE, GRANULAR SUBBASE). (24)

combination trucks per day on a given roadway.* (See Chapter 5 for further explanations of traffic-related calculations.)

One further point with regard to the traffic specification is that the truck count defined for T in equation (81) applies to <u>all</u> lanes in one direction (presumably one roadway of a divided highway). However, it is not stated explicitly in (24) whether the measured faulting represented by F_{avg} also applies to all lanes in this direction, or rather to a single "design lane." From the context of the discussions in (24) it appears that the damage (faulting) is indeed measured in a single lane (the outer lane); and for rural highways similar to those surveyed in (24), the assumption that most trucks use the outer lane is a valid one.

On highly traveled roadways, however, truck loads tend to be distributed across the typically six to eight lanes provided, as discussed earlier in this chapter and in Chapter 5. It is therefore reasonable to assume that the factor T is equation (81) should be reduced by the design lane factor, as are vehicle loads. This adjustment has been incorporated within the EAROMAR system's predictions of faulting.

<u>Subgrade</u>. The subgrade drainage factor S in equation (81) embodies a qualitative assessment of design and environmental factors influencing faulting. Again lacking more precise information on this effect, we associate the values suggested in equation (81) with inputs for pavement drainage provided by the user to EAROMAR, as follows:

Drainage Characteristics Input to EAROMAR	Value of S in Equation (81)
Good	2.0
Fair	1.5
Poor	1.0

*Note that the value of T defined in eq.(81) represents the average number of trucks per day <u>during A years of service assuming 5% annual growth</u>; within EAROMAR we use the seasonally adjusted average per day <u>within a given year</u>. These differences in computing average daily volumes may lead to distortions in the fault predictions. However, Packard (24) himself noted the likelihood of "large inherent variation...associated with the truck traffic factor" in his source data, due to fluctuations in actual daily counts about the average, and to increasing magnitudes of loads and proportions of heavy trucks over time. Therefore any error introduced by the calculation of average daily truck traffic within EAROMAR is not expected to affect the results significantly.

Subbase Stabilization. The contribution of stabilized bases to reducing the severity of faulting is accounted for by the exponent "b" in equation (81). Within EAROMAR the distinction between stabilized and unstabilized materials must be reflected by some discrimination in materials properties discussed in Chapter 2. Domenichini (25) points out that the in situ properties of stabilized subbases are difficult to correlate with standard laboratory values, since they vary over time and are influenced by shrinkage and cracking. He notes that the elastic moduli of cement-treated subgrades, for example, have been reported variously as 12,000 MPa to 500 MPa (1.8×10^6 to 7.5×10^4 psi), with the latter value comparable to that of a good unbound granular base. The assignment of values to the exponenet b in equation (81) is therefore based on the subbase modulus input by the user (Chapter 2), as follows:

Subbase Modulus, ksi	Value of b in Equation (81)
≤ 75	0.241
> 75	0.037

Proposed Model. Consolidating these adjustments to Packard's model, we obtain the following relationship for faulting of non-doweled pavements:

$$F_n = \frac{165}{Jx0.25} [1.29 + 48.95 \frac{(P_T VA^2)^{0.465}}{D^{3.9}} S^{0.610} (J-13.5)^b], \qquad (83)$$

$$F_n \leq 5280/J$$

where

F_n is the average number of faults of 0.25 inch (6.4 mm) per lane mile predicted for non-doweled pavements;

J is the joint spacing input by the user, in feet;

P_T is the fraction of trucks in the traffic stream as calculated in Chapter 5;

V is the seasonally adjusted average daily traffic in the design lane, as calculated in

Chapter 5 and adjusted by the design lane factor input by the user;

A is the simulated age of the pavement surface, in years;

D is the slab thickness input by the user, in inches ;

S is the factor denoting the drainage characteristics of the subgrade, based upon inputs by the user (1 for good, 1.5 for fair, 2 for poor); and

b is a factor representing the contributions of stabilized vs. unstabilized subbase, based upon elastic modulus input by the user (0.241 for $E \leq 75$ ksi [≤ 517 MPa]; 0.037 for $E > 75$ ksi),

DOWELED PAVEMENTS

We were not able to locate in the literature a model for doweled pavements comparable to equation (83). However, information comparing the performance of doweled vs. non-doweled pavements in Florida indicates the relative decrease in faulting attributable to positive load transfer at joints. This information can be used to develop a relationship for faulting of doweled rigid pavements in EAROMAR.

The Florida data were summarized by Darter (22) as shown in Figures 24 and 25. Classifications of test sites by pavement age, slab thickness, and traffic loadings are obscured by data aggregation. Nevertheless, the greater faulting experienced by the non-doweled pavements is apparent, and the relationships inferred from Figures 24 and 25 can be taken as composite models covering a range of situations.

The distributions in Figures 24 and 25 may be reduced to mean fault depths, which can then be expressed as an equivalent percentage of joints having faults of 0.25 inch (6.4 mm). Results of these calculations are shown in Table 13, demonstrating the relative performance of doweled vs. non-doweled pavements over time.

The ratios of faulted joints for doweled and non-doweled slabs respectively are computed for the three pavement ages in Table 13. Based upon these Florida data, doweled pavements can be expected to exhibit one-third to one-fourth the faulting predicted for non-doweled sections, with this ratio declining with increasing pavement

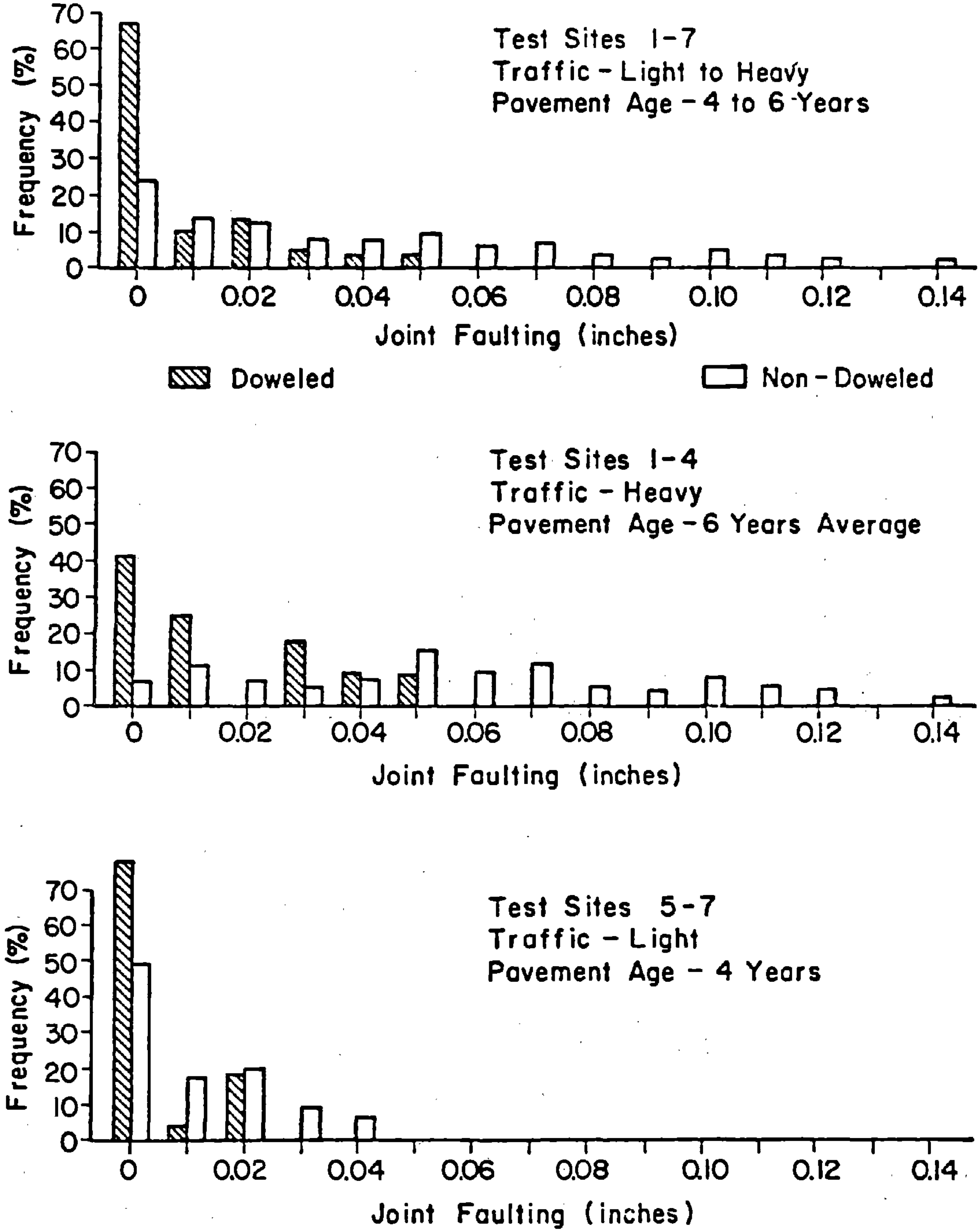


FIGURE 24. COMPARISON OF JOINT FAULTING IN FLORIDA FOR DOWELED AND NON-DOWELED JOINTS.(22)

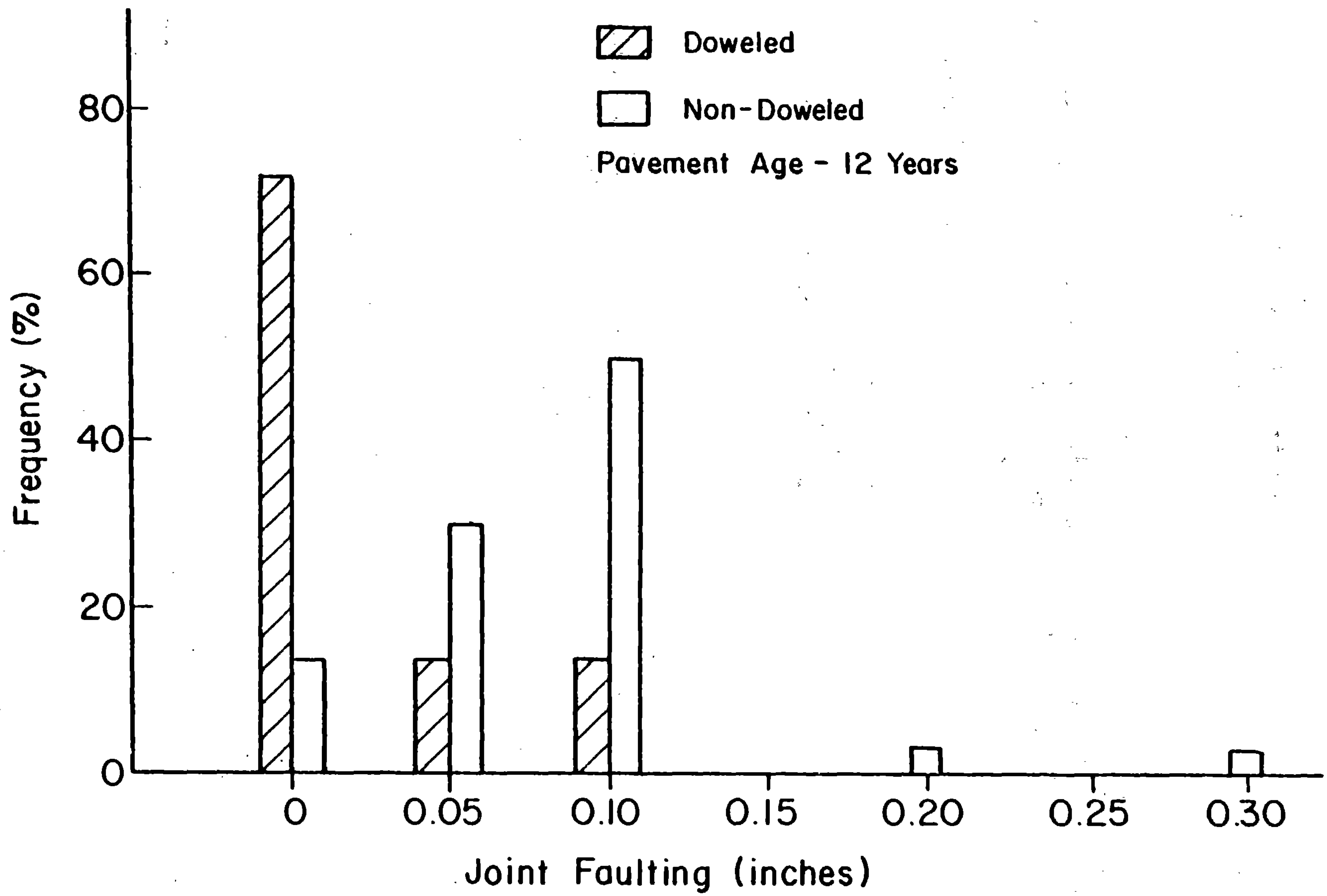


FIGURE 25. COMPARISON OF JOINT FAULTING BETWEEN DOWELED AND NON-DOWELED JOINTS - FLORIDA TEST SITES 3, 6, and 7 - RIVER GRAVEL AND CRUSHED STONE AGGREGATE IN PCC.(22)

TABLE 13

FAULTING IN DOWELED AND

UNDOWELED PAVEMENTS

(Refer to Figures 24 and 25)

	Percent of Joints					
Faulting/Joint (inches-mm)	Sections 5-7, 4 yrs old		Sections 1-4, 6 yrs old		Sections 3, 6, 7 12 yrs old	
	D	ND	D	ND	D	ND
0.00 - 0.00	78	48	41	8	72	14
0.01 - 0.25	4	17	25	10	0	0
0.02 - 0.51	18	19	0	7	0	0
0.03 - 0.76		9	17	10	0	0
0.04 - 1.02		7	9	7	0	0
0.05 - 1.27			8	15	14	30
0.06 - 1.52				9	0	0
0.07 - 1.78				10	0	0
0.08 - 2.03				4	0	0
0.09 - 2.29				3	0	0
0.10 - 2.54				8	14	49
0.11 - 2.79				4		0
0.12 - 3.05				4		0
0.14 - 3.56				1		0
0.20 - 5.08						4
0.30 - 7.62						3
Total Percentage of Joints	100	100	100	100	100	100
Avg. Fault/Joint						
Inches	0.004	0.001	0.052	0.053	0.021	0.081
mm	0.10	0.28	0.39	1.35	0.53	2.06
Avg. % Joints Faulted*	1.6	4.4	6.1	21.2	8.4	32.4

*Assuming 0.25 in, or 6.35 mm, per faulted joint

D - doweled; ND - not doweled

age. The time dependency can be accounted for by the following correction

$$F_{d/n} = \frac{1}{1 + \sqrt{A}} \quad (84)$$

where

$F_{d/n}$ is the ratio of the numbers of 0.25 inch (6.4 mm) faults predicted for doweled and non-doweled pavements respectively; and

A is the age of the doweled or non-doweled pavement, in years

Combining equations (83) and (84), we obtain the model for faulting of doweled pavements in EAROMAR:

$$F_d = F_n \times F_{d/n} \quad (85)$$

where

F_d is the average number of faults of 0.25 inch (6.4 mm) per lane mile predicted for doweled pavements; and

Fn, F_{dn} are defined earlier.

Joint Seal Deterioration

"Joint damage" within EAROMAR refers to the deterioration, stripping, or non-performance of joint sealants in plain-jointed concrete pavements. Review of the literature indicates that useful lifetimes and performance of different sealants are product-specific, and depend as much upon details of joint design as upon more general structural, traffic, and environmental conditions surrounding the pavement. For these reasons we do not model the deterioration of joint sealants within EAROMAR, but rather have the user specify annual rates of deterioration (indicating average sealant lifetime), as noted in Table 10. However, because of disagreement in the literature regarding the apparent usefulness of joint sealants, we have included some comments below on these issues and their implications for the treatment of pavement damage within EAROMAR.

Two themes recur in the literature on joint sealants. The first is that sealants provide continuity to an otherwise discontinuous concrete surface, to prevent (1) infiltration of free water to the underlying pavement layers, (2) infiltration of incompressibles to the pavement joint, and (3) infiltration of snow-melting brine to load-transfer devices (LTDs). Modes of damage commonly associated with these penetrations of the joint are pumping, faulting, spalling, blowups, mid-slab cracking, joint movement, and LTD failure (22, 26, 27, 28, 29). According to some authorities, "unsealed joints are a major contributor to concrete pavement deterioration" (30), and "for modern jointed concrete pavements built on well-constructed foundations, joint construction and sealing problems are the prevelent cause of premature pavement failure" (26).

The second theme is that despite this conventional wisdom regarding joint sealants, correlations of joint filler deterioration with expected increases in pavement damage are not always apparent. In their survey of premium jointed concrete pavements, Darter and Barenberg (7) found considerable joint filler stripping, but no connection between sealer damage and structural maintenance activity. Darter (22) noted little spalling or blowups in relation to sealer performance, as determined from field studies and interviews. Ray (29) mentioned that California requires sealed joints in new construction only in mountainous areas, relying instead upon short joint spacings (averaging 15.5 ft, or 4.7 m) and erosion-resistant subbases in JCP design. He also cited extensive European practice in leaving joints unsealed, and referred to conclusions by the Permanent International Association of Road Congresses (31) that unsealed joints perform acceptably if (1) traffic is light, (2) traffic is heavy but the climate is dry, or (3) traffic is heavy and the climate wet, but the pavement is doweled.

New York State has conducted an extensive study of preformed compression sealers, covering 56 test sections over a 10-year period (26). The study monitored the performance of the sealers (e.g. infiltration of incompressibles on top, at the edge, and below sealers) and associated changes in pavement condition, primarily spalling. Most spalls observed were small and occurred within 2-3 years, leading the investigators to conclude that they were porbably due to joint sawing during construction. No correlation was found between amount of spalling (or joint movement) and amount of debris above the joint sealer. (However, comparisons with poured-sealant joints and unsealed joints were not included in this study.)

As a result, we have limited the interaction between joint sealant damage and other damage components within EAROMAR to that of water infiltration alone. An unsealed joint is considered analogous to an open crack, the treatment of which is discussed in section 3.5. No direct linkages are now included in EAROMAR between joint damage and faulting, spalling, blowups or pumping. While this decision is open to change in the future, the modeling of interactions among these damage components will require much better quantitative information, isolating the effects of the joint seal itself, than is now available. Also, the relative contributions of infiltration through pavement joints vs. through cracks or joints at the pavement-shoulder boundary will have to be accounted for.

Many joint sealers are now on the market, and their properties and relative performance have been documented in several studies(22, 26, 28, 30, 32, 33, 34, 35, 36). In general, hot-poured seals using bituminous materials, rubberized asphalt, or polysulfides have lives of about 1-5 years; preformed compression sealers perform acceptably for 5-10 years or more. (Of course, relative cost must be considered as well.) Because of this considerable variability, and the contributing influences of good construction practices (depth and width of joint cut; clearing of joints) to sealant performance, users specify directly the projected annual deterioration of joint sealants to be simulated within EAROMAR. For example, an annual deterioration rate of 0.10 would mean that, on average, 10 per cent of pavement joints will fail each year; this is equivalent to an average sealant life of 10 years. A deterioration rate of 0.5 implies failure of 50% of joints each year, or an equivalent sealant life of 2 years. In this way the deterioration rate specified by the user can be used to represent the range of both the observed lifetimes of sealants and the effects of joint construction and environment peculiar to a given state.

Spalling

Spalling refers to the disintegration of the concrete surface near joints or cracks, evidenced by pieces of pavement breaking away to leave a rough riding surface. Darter and Barenberg (7) cite several design, performance, and environmental factors that may cause spalls, including joint spacing (with longer spacings resulting in more severe spalling), infiltration of incompressibles into joints, problems in joint construction or load transfer devices, freeze-thaw cycles, and deterioration of the portland cement concrete.

Only some of these causes can be accounted for well within a prediction model. Joint spacing and climate can be described unambiguously, and incorporated within a time-dependent relationship. However, deficient design and construction practices at joints, and deterioration of the concrete material, are difficult to capture in a general way. The infiltration of incompressibles into transverse joints may proceed at first from the pavement-shoulder joint and from the subbase or subgrade; later, fines and debris may enter from the pavement surface, as joint sealants deteriorate or are stripped away. But the value of joint sealants themselves in affecting maintenance requirements is debatable(as discussed in the previous section): thus, the relationship between sealant deterioration and subsequent spalling is not clear at this time.

Data on the percent of joints spalled over time as a function of joint spacing are reported by Darter (22) for the Michigan Test Road, as shown in Figure 26. The reduction in the incidence of spalling with decreases in joint spacing is clearly indicated. The increase in joints spalled over time is substantial, due probably to combinations of influencing factors cited above. Note that one may infer the number of joints spalled from Figure 26, but not the total pavement area spalled.

The trends in Figure 26 were developed for use within EAROMAR in the following way. The curves in Figure 26 (estimated by Darter) were reduced to the following functional forms:

$$F_s = 1 - e^{-\alpha\,(L-8)}, \tag{86}$$

$$\alpha = 0.0000162\ A^{3.0806} \tag{87}$$

where F_s is the fraction of joints spalled

L is the joint spacing (slab length), in feet, input by the user; and

A is the age of the pavement, in years, as input by the user and updated during the simulation.

To estimate maintenance requirements within EAROMAR, however, we need the area of spalling rather than the number of spalled joints. To relate these two quantities we have assumed an average spalled area for

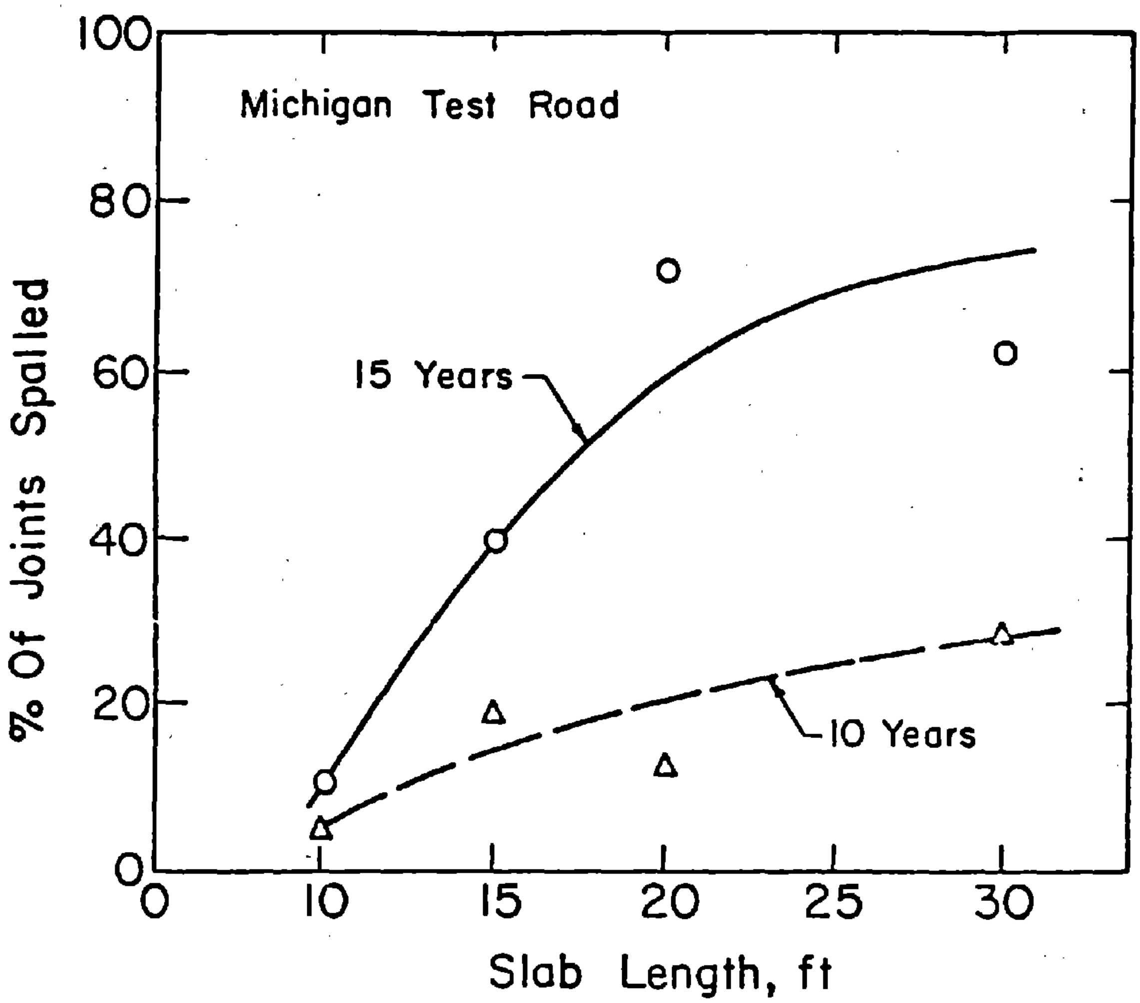


FIGURE 26. EFFECT OF JOINT SPACING ON SPALLING OF JOINTS. (22)

joint of 2 sf/joint. (This value may be revised as better field data become available.) Also, we have converted predictions of spalls to an equivalent lane-mile basis. The resulting equation used within EAROMAR is then:

$$SPALLS = \frac{5280}{L} \times F_s \times AREA_{Spall} \tag{88}$$

where SPALLS is the area of spalled pavement in square feet per lane mile;

L is the joint spacing (slab length), in feet, input by the user;

F_s is the fraction of joints spalled, computed by eq. (86); and

$AREA_{Spall}$ is the spalled area per affected joint, assumed to be 2 sf/joint.

Pumping

Pumping refers to the ejection of water-borne subbase or subgrade materials from beneath a slab, caused by the action of traffic across a pavement joint or crack. Pumping does not occur immediately following pavement construction, but rather develops over time through gradual development of voids beneath the slab, due to cumulative traffic loadings and slab warping, and filling of these voids with free water. Design against pumping generally involves use of free-draining subbase materials (to prevent buildup of water beneath the slab) or provision of erosion-resistant (e.g. stabilized) subbases to withstand the scouring effects of water ejection.

Although the mechanisms underlying pumping are well recognized in the literature (e.g. (37)), no models exist to predict the number of pumped joints for a given pavement, traffic and environment that would be needed to assess maintenance requirements for rigid pavements. For EAROMAR we have therefore developed a model based upon data on pumping observed at the AASHO Road Test (4). Given the structural orientation of the Road Test, these data stress the influences of cumulative axle loads and slab thickness on the development of pumping.

The amount of pumping at the Road Test was expressed volumetrically by a pumping index, in cubic inches of material ejected per inch of pavement edge. The pumping index is strongly correlated with slab thickness and equivalent 18-kip axle applications, as shown in Tables 14-16 derived from ref. (4) for non-reinforced sections. Data in these tables were regressed to obtain the following relationships:

$$P_I = mW_{18} \tag{89}$$

$$\log m = 1.07 - 0.34\ H \tag{90}$$

where P_I is the pumping index, in cubic inches per inch of slab length;

W_{18} is the cumulative number of 18-kip axle loads, in thousands, computed during the simulation; and

H is thickness of the PCC slab, in inches, input by the user.

To translate the pumping index into an equivalent number of joints experiencing pumping (as might be required in assessing maintenance needs), one must assume a volume of material pumped per joint. From the data in Tables 14-16, say the pumping index for a moderately pumped pavement were 100. This corresponds to 6.3 million cubic inches, or 136 cubic yards [104 cu.m.] of material pumped per mile, or approximately 2.6 cu. yds. [2 cu.m.] pumped per 100 feet of pavement lane. Joint spacing for non-reinforced sections at the AASHO Road Test was 15 feet [14.6 m]. We

TABLE 14

AASHO ROAD TEST PUMPING INDEX AT 1,114,000 LOAD APPLICATIONS

OR PSI OF 1.5,

PCC Slab Thickness = 5 inches

Axle Load, Kips	Axle Equivalency	Unweighted Load Applications, Thousands	Equivalent 18-Kip Single Axle Loads, Thousands	Pumping Index, Cubic Inches per inch of slab length
6S	0.01	1114	11	4
		"	"	6
		"	"	6
		"	"	2
12S	0.21	"	234	53
		"	"	83
		"	"	63
		"	"	88
24T	0.50	705	353	73*
		1114	557	65
		901	451	106*
		771	386	146*
18S	1.00	716	716	191*
		353	353	91*
		291	291	147*
32T	1.42	343	487	202*
		328	466	101*
		289	410	118*

*Pumping Index at PSI = 1.5.

TABLE 14 (con't)

AASHO ROAD TEST PUMPING INDEX AT 1,114,000 LOAD APPLICATIONS

OR PSI OF 1.5,

PCC Slab Thickness - 5 inches

Axle Load Kips	Axle Equivalency	Unweighted Load Applications, Thousands	Equivalent 18-Kip Single Axle Loads, Thousands	Pumping Index, Cubic Inches per inch of slab length
40T	3.55	1114	3955	37
		"	"	67
		"	"	47
		898	3188	98*
30S	7.79	878	6839	150*
		1114	8678	159*
		"	"	168
48T	7.27	"	8099	164
		"	"	133
		"	"	105*

*Pumping Index at PSI = 1.5.

TABLE 15

AASHO ROAD TEST PUMPING INDEX AT 1,114,000 LOAD APPLICATIONS

OR PSI OF 1.5,

PCC Slab Thickness = 8 inches

Axle Load Kips	Axle Equivalency	Unweighted Load Applications, Thousands	Equivalent 18-Kip Single Axle Loads, Thousands	Pumping Index, Cubic Inches per inch of slab length
12S	0.18	1114	200	18
		"	"	18
		"	"	18
24T	0.45	"	501	24
		"	"	31
		"	"	27
18S	1.00	"	1114	24
		"	"	20
		"	"	24
		"	"	21
32T	1.47	"	1638	35
		"	"	29
		"	"	39
		"	"	30
22.4S	2.28	"	2540	33
		"	"	47
		"	"	97
		1111	2533	122*

*Pumping Index at PSI =1.5.

TABLE 16

AASHO ROAD TEST PUMPING INDEX AT 1,114,000 LOAD APPLICATIONS

OR PSI OF 1.5,

PCC Slab Thickness = 11 inches

Axle Load Kips	Axle Equivalency	Unweighted Load Applications, Thousands	Equivalent 18-Kip Single Axle Loads, Thousands	Pumping Index, Cubic Inches per inch of slab length
22.4S	2.40	1114	2674	23
		"	"	2
		"	"	3
40T	3.94	"	4389	35
		"	"	0
		"	"	12
30T	9.14	"	10182	15
		"	"	19
		"	"	20
		"	"	12
48T	8.55	"	9525	25
		"	"	22
		"	"	21
		"	"	24

might assume that most, but not necessarily all, joints were pumping in a typical lane. These considerations would lead one to estimate a range of perhaps 0.35 - 0.55 cu. yds. [0.27 - 0.42 cu. m] of material pumped per joint. A midpoint of 0.45 cu yds/joint [0.35 cu m/joint] would correspond to, say, a volume of 6 ft x 12 ft x 2 in deep [1.8 m x 3.65 m x 5 cm deep] -- i.e. a zone extending half of a typical lane width, and about 6 ft [1.8 m] on each side of the joint.

The number of joints per lane mile experiencing pumping can then be estimated as follows:

$$\text{PUMPING} = \frac{63360 \text{ in/mi} \times P_I \text{ cu in/in}}{0.45 \text{ cu yds/}_{\text{joint}} \times 46656 \text{ cu in/}_{\text{cu yd}}}$$

$$= 3P_I \text{ joints/mile} \qquad (91)$$

where PUMPING is the number of pumped joints per lane mile; and

P_I is the pumping index computed in eq. (89).

Improvements in subsurface drainage will reduce the incidence of pumping, although no quantitative information is available in the literature to relate the extent of this reduction to subbase permeability. However, conceptually we may propose a reduction factor dependent upon the qualitative description of pavement drainage (Good, Fair, or Poor) input by the user (Chapter 2). Arbitrary values this factor of 0.2, 0.6, and 1.0 for Good, Fair, and Poor drainage respectively have been assumed within EAROMAR; these may be revised in the future as field data become available. The pumping equation used within EAROMAR is then:

$$\text{PUMPING} = 3\, F'_D\, P_I\ , \qquad (92)$$

$$\text{PUMPING} \le 5280/L$$

where

PUMPING is the number of pumped joints per lane mile;

F'_D is a factor denoting the effects of subbase drainage quality input by the user (Good = 0.2; Fair = 0.6; Poor = 1.0);

P_I is the pumping index, in cubic inches per inch of slab length, computed in eq. (89); and

L is the length of the PCC slab, in feet, input by the user.

The inequality limits the number of pumped joints to the total number of joints in the section.

Recall in eqs (63) - (66) the use of an erodability variable ES, denoting a longitudinal strip of width ES inches along the outer slab edge under which subbase support was assumed to have been eroded. This variable, used to predict linear cracking, can be related to the pumping index as follows:

$$ES = \frac{P_I}{DEPTH_{Pump}} \tag{93}$$

where ES is the width of eroded subbase, in inches;

P_I is the pumping index, cu in/in, computed in eq. (89); and

$DEPTH_{Pump}$ is the average depth of material pumped from beneath the pavement surface, assumed to be 2 inches (5 cm).

Blowups

Blowups are instances of shattering or buckling of the rigid pavement surface occurring at joints or cracks. In his development of the original EAROMAR system, Butler (2) noted that "a number of patterns have been identified but no quantitative mechanism has been developed predicting blowups"; this situation continues today.

NCHRP Synthesis No. 19 (36) summarized the circumstances surrounding blowups, noting that most empirical observations focus on the number of blowups vs. environment, joint spacing and pavement length between blowups. Little is known about the state of stress at the time of a blowup, the materials properties of the concrete and their variation near joints, the amount of debris in the joints and contributions of other pavement elements (subbase, drainage, shoulders, slab thickness and joint sealants). Most blowups occur in spring or early summer, following a hot spell and a recent rain, and in the US are typically associated with physical causes (e.g. stress relief due to buildup of debris in joints) rather than chemical ones. Blowups are rarely observed in pavements having joint spacings less than 20 ft. (6 m), or in pavements less than 5 years old; however, once blowups begin, they continue for a period, usually at frequencies no more than one per mile (0.6 per km) per year.

Most authorities agree that incompressibles in the joint are undesirable, and may cause blowups by restricting slab movement and weakening the joint through spalling. Soft aggregates that weaken the concrete may also contribute to blowups. Blowup frequency per lane-mile changes with age, as shown in Figure 27 (2).

Finney (32) focused on the problem of soft aggregates in relation to blowup frequency, as shown in Figures 28 and 29. He found that aggregate heterogeneity, defined by the following relationship

$$H = \sin p\pi \tag{94}$$

where

H is the aggregate heterogeneity; and p is the proportion of carbonate in the aggregate

was a better predictor of blowup frequency than carbonate content. Note that the increases in blowup frequency over time shown in Figures 28 and 29 agree with the trend implied in Figure 27.

In developing a model of blowups for EAROMAR we consolidated the effects of joint spacing, pavement age and aggregate properties discussed above. While it would have been desirable to include the effects due to infiltration of incompressibles into joints, data to build an analytic model suitable to EAROMAR were not available at this time. Construction of the model to predict blowups within EAROMAR proceeded as follows.

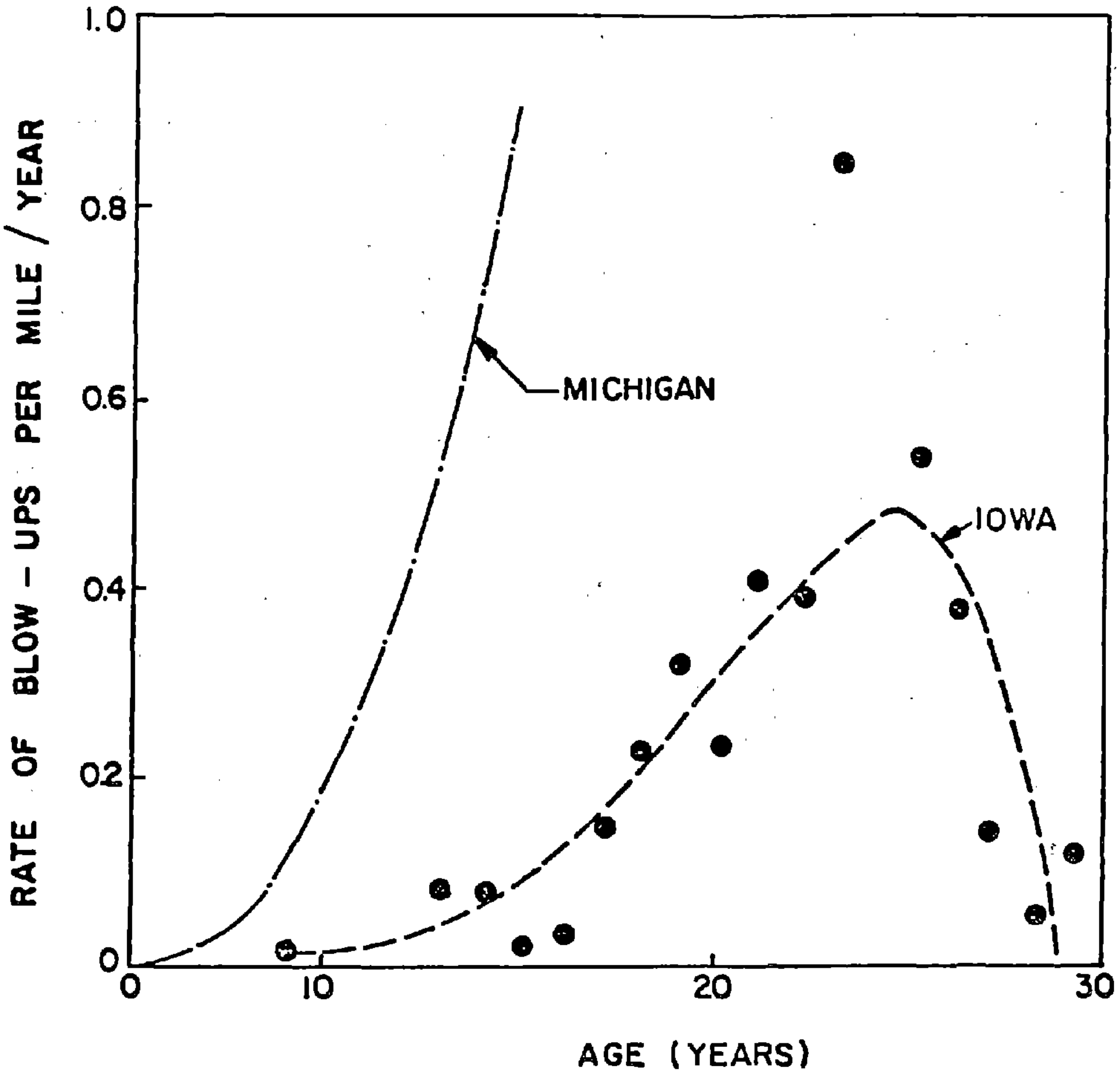


FIGURE 27. EXAMPLE OF BLOWUP HISTORIES IN MICHIGAN AND IOWA (2)

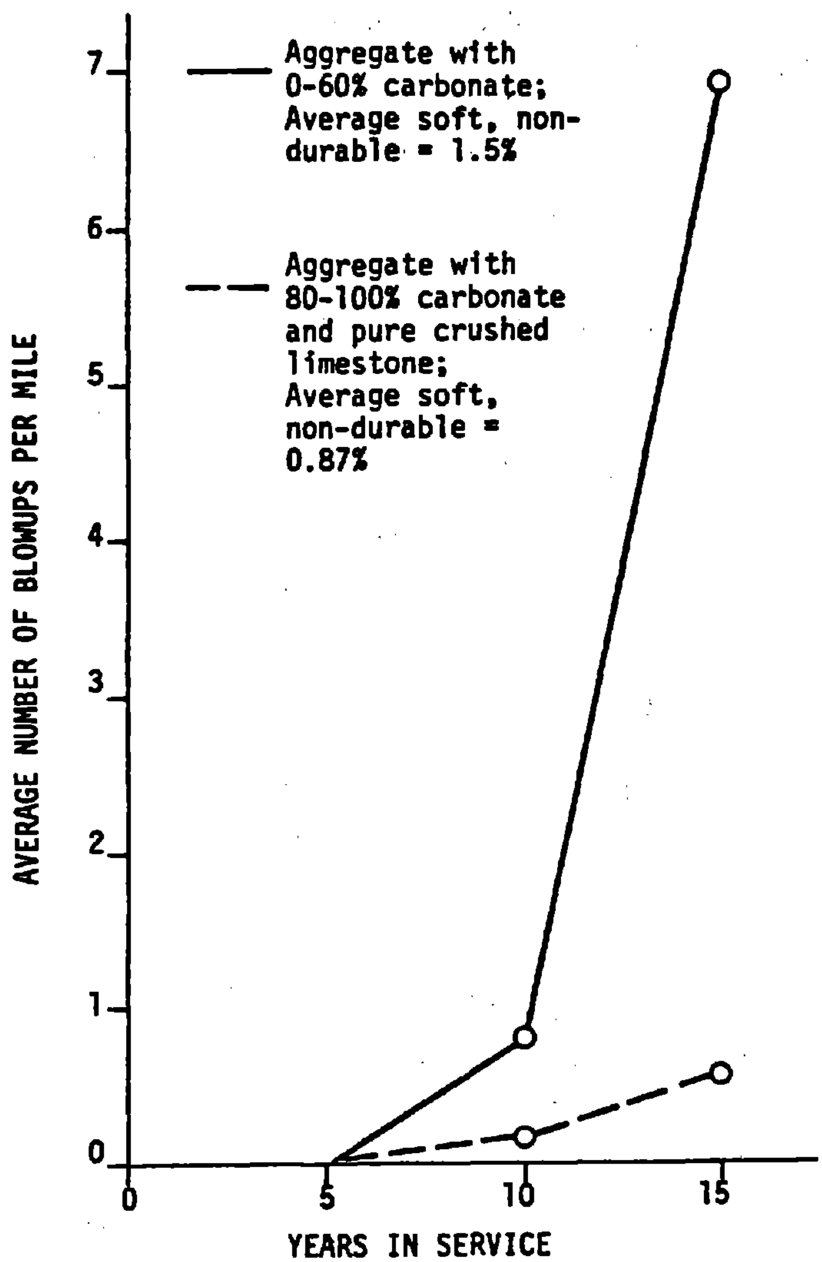


FIGURE 28. BLOWUP FREQUENCIES VS. AGE AND AGGREGATE CLASSIFICATION (AFTER (32))

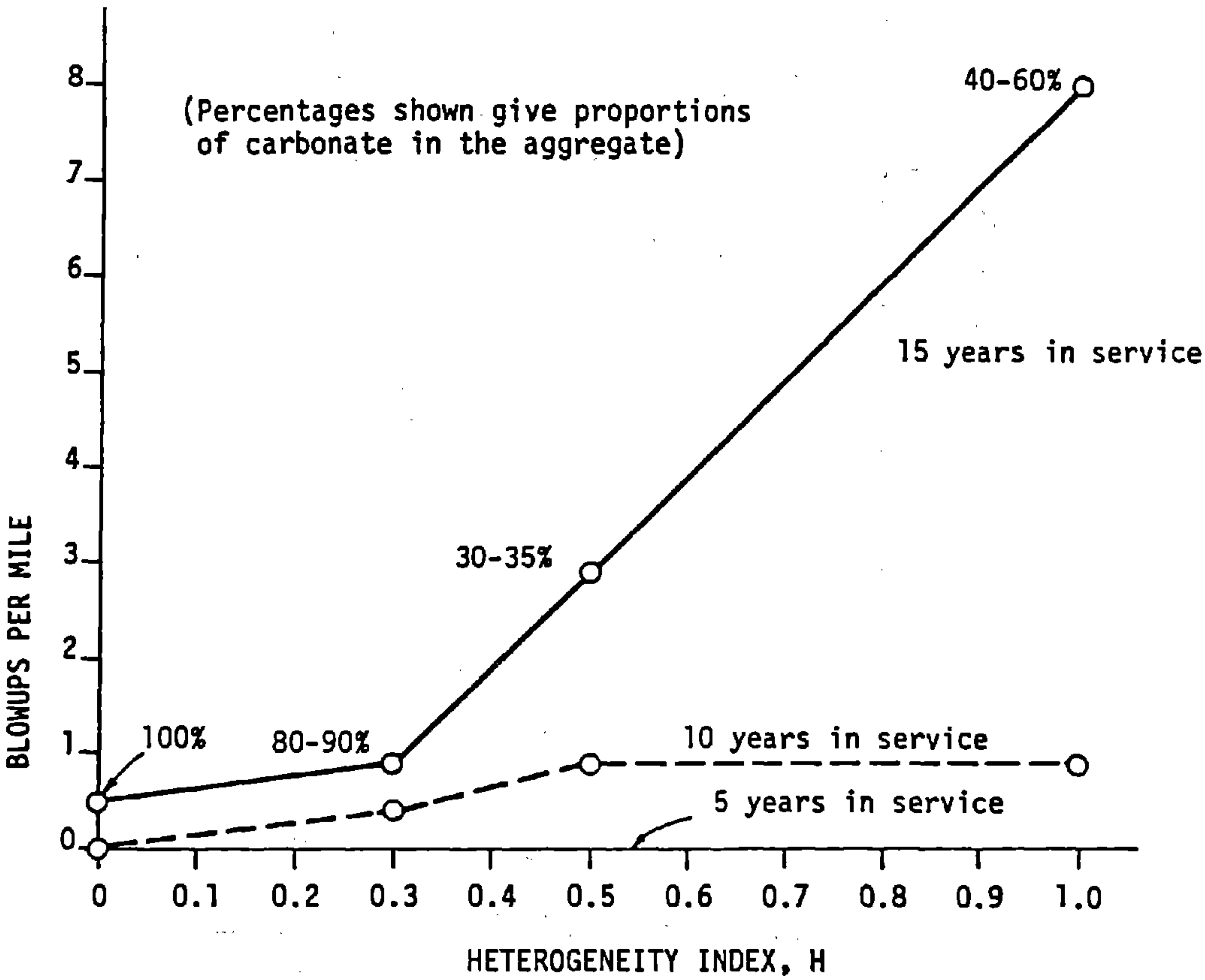


FIGURE 29. BLOWUP FREQUENCY VS. AGGREGATE HETEROGENEITY (AFTER (32))

JOINT SPACING

Blowups are unlikely at joint spacings of less than 20 feet (6m). However, we wished to represent a rapidly increasing possibility of blowups (all other factors being equal) at spacings above 30 feet (9m). Mathematically the most direct way to do this using a smooth function was through a logit distribution of the following form:

$$B_L = 1/(1+\exp-[L-25]) \tag{95}$$

where

B_L is a weighting factor varying from zero to one, expressing the likelihood of blowups as a function of joint spacing; and

L is the joint spacing (slab length), in feet, input by the user.

PAVEMENT AGE

Figures 27-29 demonstrate that blowups are rarely observed on pavements less than 5 years old. Figure 27 charts the relative blowup frequency over a 30-year period observed in Iowa (2), in which the annual occurrence per lane mile peaked at 25 years or so, after which the incidence of blowups declined. Figures 28 and 29 cover only a 15-year interval, and therefore show blowups increasing monotonically with pavement age (corresponding to the upward side of the curve in Figure 27).

To capture these effects we proposed a bell-shaped curve modeled by the equation for a normal distribution:

$$B_A = K \exp-[(\chi-m)/\sigma]^2 \tag{96}$$

where

B_A is a prediction of blowup frequency per lane mile, dependent upon pavement age;

K is a calibration constant;

χ is the random variable (in this case, pavement age);

M is the value of pavement age at which B_A is maximum; and

σ is the standard deviation of pavement age, indicating the width of the distribution used to approximate the Iowa data in Figure 27.

Experiments with this function against the data in Figures 27-29 suggested the following values for the parameters above:

$K = 3$
$M = 25$ years
$\sigma = 7.5$ years

AGGREGATE PROPERTIES

The homogeneity index defined by eq. (94) provides a convenient numerical measure to reflect aggregate properties affecting blowups. We expanded upon this concept to consider an aggregate susceptibility factor or index, ranging from 0 to 1, that encompasses the homogeneity index in eq. (94), and which may be adjusted by the user to reflect other materials properties as desired.

The trends in Figures 28 and 29 indicate that, for a given time in service, the incidence of blowups increases rapidly as the homogeneity index approaches 1.0. Referring now to Figure 27, we hold to the assumptions that blowups are neglibible after 30 years, and that the frequency of blowups is on the order of 1-2 per lane-mile (.05-1.2/lane-kilometer) at the maximum. Then, under the functional form proposed in eq. (96), the increase in blowups observed with increasing homogeneity index can be accounted for by both an increase in the magnitude of K, and a shift in the value m to an earlier age (i.e. the peak blowup frequency both becomes larger and occurs earlier for a pavement with a higher homogeneity index). The results of these adjustments are incorporated in the final form of the model relationship given below.

PROPOSED MODEL

The final form of the blowup model used in EAROMAR, incorporating the several considerations above, is as follows:

$$\text{BLOWUPS} = 3F_{agg} \;\; \exp-([A-25+5F_{agg}]/7.5)^2$$
$$\times \; 1/(1+\exp-[L-25]) \qquad (97)$$

where

BLOWUPS is the predicted number of blowups per lane mile per year;

F_{agg} is a factor varying from zero to one denoting the susceptibility of the PCC aggregate to blowups, as input by the user (may reflect homogeneity index, or other parameters as judged by the user);

A is the age of the PCC pavement, in years, input by the user and updated during the simulation; and

L is the PCC joint spacing (slab length), in feet, input by the user.

In contrast with many of the other models discussed in this chapter, which predict cumulative damage over time, eq. (97) yields the <u>rate</u> of damage occurrence. Its predictions are therefore already in the form required by EAROMAR (see section 3.1). Because blowups are associated with hot weather, the results of eq. (97) are applied (in any year) only during that season having the maximum mean temperature specified by the user. Finally, the frequency of blowups varies linearly with F_{agg} in eq. (97), implying that if the aggregate index is zero, the number blowups is also zero. This result may not be realistic (e.g. if iowups are due to other causes); in such cases the model may give better results if a small, non-zero value (e.g. 0.2) is assigned to F_{agg}.

To illustrate the interactions among the several variables involved, Table 17 gives results estimated by eq. (97) for selected values of joint spacing, aggregate susceptibility, and pavement age.

TABLE 17

EXAMPLE PREDICTIONS OF BLOWUP FREQUENCIES

(NUMBER PER LANE MILE)

Age of Pavement, Years	JOINT SPACING, FEET								
	15			25			40		
	AGGREGATE SUSCEPTIBILITY								
	0.2	0.5	0.8	0.2	0.5	0.8	0.2	0.5	0.8
0	0	0	0	0	0	.0	0	0	.001
5	0	0	0	0	.003	.013	.001	.006	.025
10	0	0	0	.009	.047	.140	.018	.093	.279
15	0	0	0	.071	.276	.633	.142	.552	1.266
20	0	0	0	.226	.671	1.179	.451	1.342	2.358
25	0	0	0	.295	.671	.903	.589	1.342	1.806
30	0	0	0	.158	.276	.284	.316	.552	.569

(1 mile = 1.6 km)

(1 ft = 0.305 m)

Roughness

Rigid pavement roughness refers to distortions along the longitudinal profile of the pavement, excluding those due to faulting. Estimates of roughness between joints were made by Brokaw (23) in terms of slope variance, according to the relationship:

$$SVO = (A+1)^n - 1 \tag{98}$$

where

SVO is the part of slope variance between joints;

A is the pavement age, in years; and

n is an exponent depending upon type of subgrade soil (0.58 for A-1, A-2, and A-3 subgrade soils; 0.70 for A-4 through A-7 subgrade soils).

However, it was desired to have roughness sensitive to variables additional to those of pavement age and subgrade type. Also, descriptions of soils within EAROMAR are in terms of materials properties rather than designations by type or class. Therefore we adopted an approach similar to that taken for flexible pavements, relying on the correlation between roughness and serviceability (4).

PAVEMENT SURFACE DETERIORATION OVER TIME

An equation to predict rigid pavement serviceability over time was developed at the AASHO Road Test; however, researchers have found this relationship to be unrealistic over longer time periods or at different sites (2, 22, 24). Recently Darter (22) developed a modified serviceability equation that appears to fit more closely the serviceability trends of pavement sections in service for 16 years. These equations have been adapted for use within EAROMAR.

The approach developed by Darter is based upon the following functional form:

$$W'_{18} = [\rho \ln (\frac{3}{y} - 1) + \beta] \tag{99}$$

$$\beta = -50.08826 - 3.77485H + 30.64386 \sqrt{H} \tag{100}$$

$$\rho = -6.69703 + 0.13879H^2 \tag{101}$$

$$y = P2 + \frac{3}{\exp(-\beta/\rho) + 1} - P1 \tag{102}$$

where

W'_{18} is the total equivalent 18-kip single axle loads, in millions, to reduce the serviceability index from P1 to P2;

P2 is the current or terminal serviceability index;

P1 is the initial serviceability index; and

H is the PCC slab thickness, in inches.

This equation was then further modified by Darter to include materials porpetties, with the following result:

$$\log W_{18} = \log W'_{18} + (3.892 - 0.706P2)\, Y \qquad (103)$$

$$Y = \log\left[\frac{F28}{690}\;\frac{4\log\left(\dfrac{8.789H^{0.75}}{M}\right) + 0.359}{4\log\left(\dfrac{Z^{0.25}(0.540H^{0.75})}{M}\right) + 0.359}\right] \qquad (104)$$

$$M = (1.6a^2 + H^2)^{\frac{1}{2}} - 0.675H \qquad (105)$$

$$Z = E/k \qquad (106)$$

W_{18} is the corrected number of equivalent 18-kip single axle loads, in millions, to reduce the serviceability index from P1 to P2;

where W'_{18} = is the result predicted by eq. (99);

F_{28} is the modulus of rupture used in design, in psi;

H = is the slab thickness, in inches;

a is the radius of the applied edge load on the slab, in inches;

E is the modulus of elasticity of the PCC, in psi; and

k is the modulus of foundation support on top of the subbase, in pci.

These equations have been modified for use within EAROMAR as follows.

Consider first the simpler form of the model given by eq. (99). It can be transformed algebraically into an equivalent function of PSI deterioration more suitable to our application:

$$P2 = P1 - \frac{3}{1 + \exp(-\beta/\rho)} + \frac{3}{1 + \exp((W_{18}-\beta/)/\rho)} \qquad (107)$$

where all variables are as defined earlier.

Note that for $W_{18} = 0$, P2=P1.

Moving now to the adjustments for materials properties, we suppressed the radius of the contact area of applied edge loading, by assuming a pressure of 75 psi (517 kPa) for a tire carrying 4500 lbf (20 kN), yielding a contact radius of 4.37 inches (1.7 cm). The resulting equation for M is then:

$$M = (30.56 + H^2)^{\frac{1}{2}} - 0.675H. \tag{108}$$

The basic form of the adjusted model in eq. (103) is:

$$W = \frac{W'A}{B^{P2}} \tag{109}$$

where W,W' are the adjusted and unadjusted 18-kip axle loads, in millions;

A,B are constants or functions of Y, and therefore constant for any given pavement and materials properties; and

P2 is the current or terminal serviceability index.

The exponent P2 in eq. (109) renders this adjustment intractable when we try to incorporate it in the form of eq. (107). To overcome this problem we considered the variation in the ratio A/B^{P2} for P2 =(4.5 to 1.5), and selected the midpoint of this range. This allowed us to define a new variable X as follows, which is contant for given pavement and materials properties:

$$X = 10^{1.774Y} \tag{110}$$

where Y is defined by eq. (104).

By repeating the derivation of eq. (107), but now with the materials adjustment included, we arrived at the following relationship:

$$P2 = P1 - \frac{3}{1+\exp(-\beta/\rho)} + \frac{3}{1+\exp((W_{18}-\beta)/\rho X)}, \tag{111}$$

Note that for this new equation, however, when $W_{18} = 0$, P2 is not equal to P1, because of the inclusion of the variable X in the third term. To satisfy this boundary condition while retaining the materials adjustment, we imposed symmetry with respect to X between the second and third terms in eq. (111). (The modified second term does not vary with W_{18}; therefore this change does not affect the rate of serviceability deterioration.)

The final form of the equation adopted for EAROMAR is thus as follows:

$$P2 = P1 - \frac{3}{1+\exp(-\beta/\rho X)} + \frac{3}{1+\exp((W_{18}-\beta)/\rho X)} \qquad (112)$$

where P2 is the current or terminal serviceability;

P1 is the initial serviceability, assumed to be 4.5;

β is as defined by eq. (100);

ρ is as defined by eq. (101); and

W_{18} is the cumulative number of 18-kip axle loads, in millions.

CORRELATIONS OF ROUGHNESS WITH SERVICEABILITY

Correlations of roughness with serviceability were based upon data on rigid pavement sections presented in (4). Regression analyses of these data yielded the following relationship:

$$R = 360 - 72\ P2 \qquad (113)$$

where R is the pavement roughness in inches per mile; and

P2 is the current value of the present serviceability index.

PROPOSED MODEL

Equations (112) and (113) may now be combined to yield the model for rigid pavement roughness within EAROMAR:

$$R = 360 - 216\ (1.5 - \frac{1}{1+\exp(-\beta/\rho X)} + \frac{1}{1+\exp((W_{18}-\beta)/\rho X)} \qquad (114)$$

where R is the rigid pavement roughness between joints in inches per mile;

β is as defined by eq. (100);

ρ is as defined by eq. (101);

X is as defined by eq. (110); and

W_{18} is the cumulative member of 18-kip axle loads, in millions, as computed during the EAROMAR simulation.

Serviceability

Serviceability of rigid pavements is expressed by its present serviceability index (PSI). In any season and year during the EAROMAR simulation the PSI is estimated as a function of current surface damage according to the AASHTO relationship (4):

$$PSI - 5.41 - 1.80 \log (1+SV) - 0.09 \sqrt{C+P} \qquad (115)$$

where

PSI is the present serviceability index;

SV is the mean slope variance in the wheelpaths, radians^2 x 10^6;

C is the lineal feet of Class 3 plus Class 4 cracks per 1000 sf of pavement area*; and

P is the area of pavement patched, in sf/1000 sf.

The growth in surface damage, leading to declines in PSI, is computed using the damage models discussed above. Slope variance is computed as the sum of contributions to slope variance from roughness (eq. (114)) and faulting (eqs. (83) and (85)). The contribution due to roughness was derived from regression analysis of data on rigid pavement sections contained in (4):

$$SVR = 0.000145 \, R^{2.255} \qquad (116)$$

where SVR is the slope variance, in radians^2 X 10^6, due to roughness between joints; and

R is the rigid pavement roughness between joints, in inches per mile.

*A Class 3 crack is one opened or spalled at the surface to a width ¼-inch (6 mm) or more over at least half its length. A Class 4 crack is one that has been sealed. Crack length is measured by the projection of the crack parallel or perpendicular to the pavement centerline.

The contribution due to faulting was adapted from a relationship between slope variance and faulting cited by Packard (24):

$$SVR = \frac{0.001512}{L} [L\ F_{avg}\ F]^{1.723} \tag{117}$$

where SVR is the slope variance, in radians^2 x 10^6, due to faulting;

L is the number of joints (slab lenght), in feet, input by the user;

F_{avg} is the height of an average fault, in inches (assumed to be 0.25 inch, or 6 mm); and

F is the number of faults per lane mile computed by eq. (83) or (85).

Total slope variance is then computed as

$$SV = SVR + SVF. \tag{118}$$

Both lineal and areal cracking are included in the PSI computation within EAROMAR; a conversion of 1 lf of crack = 1sf of cracking (1sm cracking = 3.28 m of crack), is assumed. Lineal cracking is computed within EAROMAR using eq. (79); rate of areal cracking is input by the user.

Summary of Rigid Pavement Damage Relationships

Below we summarize the equations developed for the rigid pavement deterioration models within EAROMAR. As with the flexible pavement models earlier, we have restructured some of the equations to achieve more consistent notation among the several models presented and to express constants generally to no more than three significant digits to the right of the decimal.

THERMAL AND FATIGUE CRACKING

Maximum Tensile Stresses.

$$\sigma_T = \sigma_L + F_1 \cdot \sigma_c \tag{119}$$

where σ_T is the total stress in psi in the longitudinal direction at the bottom of the PCC slab when the wheel load is at the slab edge;

σ_L is the portion of total stress in psi due to an 18-kip single axle load (with no thermal curling stress), as given by:

$$\sigma_L = \frac{1000}{H^2} [17.358 + 0.078\ e - 0.0539\ H^3/k + 7.417 \log (H^3/k)] \ ; \tag{120}$$

F_1 is an adjustment factor given by

$$\begin{aligned} F_1 = {} & 0.48 + 0.014\ H - 0.00427\ e - 0.273\ G \\ & - 0.00403\ L + 0.195 \log k + 0.452\ G \log H \\ & - 0.00532\ G^2 + 0.0125\ GL - 0.00622\ GL \log k \\ & + 8.787\ (\log [H^3/k])/H^2 + 0.00104\ Ge \\ & - 0.118\ G \log (H^3/k) + 0.0700 \log (e+1) \\ & - 0.0133\ G \log\ (e+1) \end{aligned} \tag{121}$$

σ_c is the portion of total stress in psi due to curling arising from a thermal gradient (with no traffic load), as given by:

$$\begin{aligned} \sigma_c = \frac{GK}{5\text{x}10^{-6}} [& 0.00671\ k + 79.074 \log k + 11.727\ L \\ & - 0.00720\ kL - 3.221\ L \log k - 0.0688\ Le \\ & - 0.595\ e \log k - 204.395\ H/k - 38.089\ L/H \\ & - 8.368\ H \log k + 0.072\ H\ e + 0.05691\ Le \log k \\ & + 0.208\ LH \log k + 0.00058\ LH\ k - 0.00201\ Le\ H \log k]; \end{aligned} \tag{122}$$

G is the thermal gradient in the PCC slab, in °F/in, estimated by the following equations for daytime (0700-1900) and nighttime (1900-0700) respectively:

$$G_d = \frac{T/4-1}{H+2}, \text{ and} \tag{123}$$

$$G_n = -(\frac{T/20+5}{H+2}); \tag{124}$$

and where

H is the PCC slab thickness, in inches, input by the user;

e is the erodability of support along the slab edge due to pumping, in inches, as computed by the pumping model;

k is the modulus of foundation support at the top of the subbase, in pci, input by the user;

T is the seasonal average daily temperature in °F input by the user;

L is the length of the PCC slab, in feet, input by the user; and

K is the thermal coefficient of expansion of the PCC slab, ft per ft per °F, input by the user.

Separate calculations of F_1 and σ_c are made for daytime and nighttime within each season, resulting in daytime and nighttime estimates for season i of the total stress, σ_{Tid} and σ_{Tin} respectively.

Fatigue Damage. The applied loads contributing to fatigue are computed as follows:

$$w_{id} = \sum_{j=1}^{2} F_{ij} \times w'_{ijd} \tag{125}$$

$$w_{in} = \sum_{j=1}^{2} F_{ij} \times w'_{ijn} \tag{126}$$

$$w'_{ijd} = 0.12 \times \sum_{h=8}^{19} w_{ijh} \tag{127}$$

$$w'_{ijn} = 0.12 \times \sum_{\substack{h=20 \text{ to } 24, \\ 1 \text{ to } 7}} w_{ijh} \tag{128}$$

where

- i is an index denoting season;
- j is an index denoting type of day within the season (j=1=weekday; j=2=weekend);
- d is an index denoting daytime;
- n is an index denoting nighttime;
- h is an index denoting hour of day (1 through 24);
- w_{id}, w_{in} are the numbers of fatigue loads applied in season i during daytime and nighttime respectively;
- F_{ij} is the number of weekdays (j=1) or weekends (j=2) within season i;
- w'_{ijd}, w'_{ijn} are the numbers of 18-kip axle loadings predicted to fall within 6 inches (15 cm) of the slab edge, by season, type of day, and day or night split; and
- w_{ijh} is the number of 18-kip axle loadings in the design lane computed in EAROMAR by season, type of day, and hour.

The number of allowable loads is computed as follows:

$$\log N_{id} = 16.61 - 17.61 \; (\sigma_{Tid}/M^{i}_{rup}) \tag{129}$$

$$\log N_{in} = 16.61 - 17.61 \; (\sigma_{Tin}/M^{i}_{rup}) \tag{130}$$

$$M^{i}_{rup} = (1.22 + 0.17 \log A - 0.05 \log^2 A) \; M^{28}_{rup} \tag{131}$$

where

N_{id}, N_{in} are the numbers of allowable loads to failure through season i for daytime and nighttime respectively;

M^i_{rup} is the modulus of rupture of the PCC pavement, in ksi;

A is the time since the pavement slab was constructed, in years, as the input by the user and updated during the EAROMAR simulation; and

M^{28}_{rup} is the 28-day mean modulus of rupture, in ksi, input by the user.

The damage according to Miner's hypothesis is computed as follows:

$$D_i = \frac{w_{id}}{N_{id}} + \frac{w_{in}}{N_{in}} \tag{132}$$

$$D = \sum_i D_i \tag{133}$$

where

D_i is the fatigue plus curl stress damage occurring this season;

D is the cumulative fatigue plus curl stress damage; and

all other variables are as defined above.

The amount of linear cracking on the pavement surface is then calculated as follows:

$$LCRACKS = 522\ D^{0.333} \tag{134}$$

where

LCRACKS is the amount of linear cracking, in square feet per lane mile.

FAULTING

Non-doweled Pavements.

$$\text{FAULTS (non-doweled)} = \frac{5280}{LF_{avg}}[1.29 + 48.95 \left(\frac{P_T\ VA^2}{H^{3.9}}\right)^{0.465}$$

$$\times F_D^{0.610} (L - 13.5)^6], \qquad (135)$$

$$\text{FAULTS (non-doweled)} \leq 5280/L \qquad (136)$$

where

FAULTS (non-doweled) is the number of faulted joints per lane mile predicted for non-doweled pavements;

L is the joint spacing (slab length) input by the user, in feet;

F_{avg} is the average height of fault, assumed to be 0.25 inches (6mm);

P_T is the fraction of trucks in the traffic stream, as calculated in Chapter 5;

V is the seasonally adjusted average daily traffic in the design lane; as calculated in Chapter 5 and adjusted by the design lane factor input by the user;

A is the age of the pavement surface, in years, as input by the user and updated during the simulation;

H is the slab thickness input by the user, in inches;

F_D is a factor denoting the drainage characteristics of the subgrade, based upon inputs by the user (1 for good, 1.5 for fair, 2 for poor); and

b is a factor representing the contribution of stabilized vs. unstabilized subbases, depending upon the elastic modulus input by the user (0.241 for $E \leq 75$ ksi [≤ 517 MPa]; 0.037 for $E > 75$ ksi), Doweled Pavements.

$$\text{FAULTS (doweled)} = F_{dn} \times \text{FAULTS (non-doweled)} \tag{137}$$

$$F_{dn} = 1/(1+A)^{1/2} \tag{138}$$

where

FAULTS (doweled) is the number of faulted joints per lane mile predicted for doweled pavements; and

A is the age of the doweled or non-doweled pavement, in years.

SPALLING

$$SPALLS = \frac{5280}{L} \times AREA_{spall} \tag{139}$$

$$F_s = (1-e^{-\alpha}(L-8), \tag{140}$$

$$\alpha = 0.0000162\ A^{3.0806} \tag{141}$$

where

SPALLS is the area of spalled pavement in square feet per lane mile;

L is the joint spacing (slab length), in feet, input by the user;

$AREA_{Spall}$ is the spalled area per affected joint, assumed within the model to be 2 sq ft per joint (0.37 sm per joint);

A is the age of the pavement, in years, as input by the user and updated during the simulation.

BLOWUPS

$$BLOWUPS = 3 \times F_{agg} \times \exp-([A - 25 + (5 \times F_{agg})]/7.5)^2$$

$$\times 1/(1 + \exp - [L - 25]) \qquad (142)$$

where

BLOWUPS is the predicted number of blowups per lane mile per year;

F_{agg} is a factor varying from zero to one denoting the susceptibility of the PCC aggregate to blowups, input by the user;

A is the age of the PCC pavement, in years, input by the user and updated during the simulation; and

L is the PCC joint spacing (slab length), in feet, input by the user.

(Note that this model predicts the annual rate of blowup occurrence. This rate is applied during the season having the maximum mean temperature specified by the user.)

PUMPING

$$PUMPING + 3 \times F'_D \times P_I \,, \tag{143}$$

$$PUMPING \leq 5280/L; \tag{144}$$

$$P_I = m\, W_{18} \tag{145}$$

$$\log m = 1.07 - 0.34\, H \tag{146}$$

$$e = P_I/DEPTH_{pump} \tag{147}$$

where

PUMPING is the number of pumped joints per mile;

F'_D is a factor denoting the quality of subbase drainage as input by the user (good = 0.2; fair = 0.6; poor = 1.0);

L is the length of the PCC slab, in feet, input by the user;

P_I is the pumping index, in cubic inches per inch of slab length;

W_{18} is the cumulative number of 18-kip axle loads, in thousands, computed during the simulation;

H is the thickness of the PCC slab, in inches, input by the user;

e is the erodability of support along the slab edge due to pumping, in inches; (used in the linear cracking model); and

$DEPTH_{pump}$ is the average depth of material pumped from beneath the pavement surface, assumed to be 2 inches (5 cm).

ROUGHNESS

$$\text{ROUGHNESS} = 360-216\left[1.5 - \frac{1}{1+\exp(-\beta/\rho X)} + \frac{1}{1+\exp([W_{18}-\beta]/\rho X)}\right] \quad (148)$$

$$\beta = 50.088 - 3.775\ H + 30.644\ \sqrt{H} \quad (149)$$

$$\rho = -6.697 + 0.139\ H^2 \quad (150)$$

$$X = 10^{1.774\ Y} \quad (151)$$

$$Y = \log\left(\frac{M_{rup}^{28}}{0.69}\right)\frac{4\log\left[\frac{8.789\ H^{0.75}}{F_2} + 0.359\right]}{4\log\left[\frac{Z^{0.25}\ (0.54\ H^{0.75})}{F_2} + 0.359\right]} \quad (152)$$

$$F_2 = (30.56 + H^2)^{1/2} - 0.675\ H \quad (153)$$

$$Z = 10^3 \times E/k \quad (154)$$

where

ROUGHNESS is the longitudinal roughness of the pavement surface, in inches per mile;

W_{18} is the cummulative number of 18-kip axle loads, in millions, computed during the simulation;

H is the PCC slab thickness, in inches, input by the user;

M_{rup}^{28} is the 28-day modulus of rupture of the PCC pavement, in ksi, input by the user;

E is the elastic modulus of the PCC pavements, in ksi, input by the user; and

k is the modulus of foundation support on top of the subbase, in pounds per cubic inch, input by the user.

SERVICEABILITY

$$p = 5.41 - 1.80 \log (1 + SV) - 0.09 \sqrt{C+P} \qquad (155)$$

$$SV = 0.000145 \text{ ROUGHNESS}^{2.255} + \frac{0.001512}{L} (L\ F_{avg}\ \text{FAULTS}) \qquad (156)$$

$$C = \text{LCRACKS}/63.36 + \text{ACRACKS}/63.36 \qquad (157)$$

$$P = \text{AREA}_{\text{Patched}}/63.36 \qquad (158)$$

where

p is the present serviceability index defined by AASHTO;

ROUGHNESS is the pavement surface roughness, in inches per mile, computed during the simulation;

L is the joint spacing, in feet, input by the user;

F_{avg} is the average height of fault, assumed to be 0.25 in (6 mm);

FAULTS is the number of faulted joints per lane mile;

LCRACKS is the pavement linear cracking, in feet per lane mile, computed during the simulation;

ACRACKS is the pavement areal cracking, in square feet per lane mile, computed during the simulation; and

$\text{AREA}_{\text{Patched}}$ is the pavement patching simulated by the models maintenance routines, in square feet per lane mile.

3.4 COMPOSITE PAVEMENT

Composite pavements within EAROMAR are pavements having a flexible surface with one of the underlying layers consisting of a rigid material. Properties of the surface layer and all successive pavement layers are specified by the user exactly as described for flexible pavements (i.e. in terms of physical dimensions and materials properties).

The damage components considered for composite pavements, and their treatment within EAROMAR, are identified in Table 18. Damage models used to predict composite pavement condition over time are identical to those described for corresponding flexible pavement damage modes in section 3.2; and readers are referred to that section for explanations of model derivations.

Table 18

TREATMENT OF COMPOSITE PAVEMENT

DAMAGE COMPONENTS WITHIN EAROMAR

Drainage Component or Serviceability Index	Prediction Model Included Within EAROMAR	User May Input Rate Directly
1. Linear Cracking	x	x
2. Areal Cracking	x	x
3. Rutting	x	x
4. Roughness	x	x
5. Potholes		x
6. Pavement-Shoulder Joints		x

3.5 ADDITIONAL PAVEMENT CONSIDERATIONS

In this section we consider two additional aspects of pavement structure, not directly involving damage prediction, but nevertheless important to the simulation of pavement performance. The first concerns the interactions among the environment, pavement materials properties, and resulting damage incurred. The second concerns the treatment of overlays and the evaluation of pavement strength and layer properties following an overlay.

Interactions Among Environment, Materials Properties and Damage

Several types of interactions among damage components and environmental influences have already been described in Chapters 2 and 3 in relation to their treatment within EAROMAR. These interactions include the following:

1. Seasonal variations in pavement layer and subgrade moduli imput by the user, allowing one to reflect the changes in materials properties due to variations in temperature (or other seasonal effects) throughout the year;

2. Inclusion of age-dependent terms within several of the damage equations derived in sections 3.2 and 3.3;

3. Limitations of particular damage occurrence to certain seasons of the year -- e.g. blowups, to the hottest season, and cold-weather cracking in flexible pavement, to the coldest season of the year; and

4. Interactions between two damage components, such as the contribution of erodability due to pumping to linear cracking in rigid pavements.

Regarding this last point, we would like to have seen more direct interactions between other pavement damage components, such as between pumping and faulting, or blowups and joint infiltration. The disagreement among engineers on the contribution of incompressibles in joints to subsequent spalling was cited in section 3.3. Defining these additional interactions conclusively is not now possible, given the closed-form models available and the fact that they originate from many different sources. It is hoped that in the future, as knowledge from comprehensive mechanistic models is applied more widely, the effects of one damage mechanism on another will become clearer.

In this section we wish to address a specific interaction important to maintenance and premium pavement evaluation: the infiltration of water into cracks and joints, with resulting potential weakening of the pavement structure. This mechanism is important because a good deal of structural maintenance and rehabilitation is devoted to preserving the integrity of the pavement surface. The benefits of such work are often justified in part by the reduction in water infiltration, but typically no quantitative evidence of impacts on future pavement damage is provided.

The lack of current information on the effects of water infiltration and drainage has been cited by Cedergren (39). Using data from several road tests and test tracks, he calculated relative damage factors, ranging from 5 or 10 to 1, to 70,000 to 1, for wet vs. dry conditions respectively. Although the trend indicating shortened pavement life with increasing traffic loads under wet conditions is clear, the wide range of these estimates precludes their applications to predicting pavement performance.

The approach followed in the model within EAROMAR bases the amount of water entering the pavement structure on the seasonal rainfall and the extent of cracking in the pavement surface. Reduction in pavement strength is dependent upon the length of time the sublayers remain saturated, which is a function of the amount of water having entered the pavement and the drainage characteristics of the sublayers input by the user. The model considers only water entering the pavement structure through discontinuities in the surface (typically the most significant source); groundwater sources and side infiltration are not included. The technical relationships employed are based upon work by Moulton (40), supplemented by data presented in (39) and by assumptions on pavement materials behavior.

GENERAL MODEL CONSIDERATIONS

In pavements subject to rainfall one may distinguish three periods associated with wet weather, in addition to the period corresponding to dry conditions:

1. The time during which rain is falling, in which the pavement sublayers may or may not be building up to saturation;

2. If rainfall is sufficiently heavy or the sublayers of sufficiently low permeability, the time during which the sublayers are saturated or sufficiently wet to affect materials properties and structural behavior; and

3. The time during which any residual water, not sufficient to affect pavement behavior, is drained off.

We reviewed data for selected cities in their months of maximum rainfall, as shown in Table 19. Seldom do the total days of precipitation greater than 0.1 inch (2.5 mm) exceed 10, and the number of days in which the precipitation exceeds 0.5 inch (12.7 mm) is typically seven or fewer. However, the period of saturation following a rain can last from 5 - 20 days, except in those pavements having exceptionally good drainage qualities (39). Therefore in our model we considered only the second period above -- the period (after it stops raining) during which the pavement is significantly wet or saturated -- as the time relevant to estimating changes in rate of pavement damage, and neglected the time during which the rain is actually falling (period 1 above). (This assumption was made to simplify the model derivation; there is no reason why the time during rainfall could not also be included if desired.)

Drainage characteristics are input by the users in qualitative descriptions -- Good, Fair, or Poor -- as described in Chapter 2. For use in the drainage model these descriptions must be reduced to quantitative measures of subsurface permeability. Cedergren (39) presented coefficients of permeability for standard bases and subbases of about 0.02 - 20 ft/day (0.6 - 610 cm/day), and for open graded bases, about 3000 - 250,000 ft/day (900 - 75,000 m/day). Based on these data we defined the following correspondence between user descriptions of drainage quality and coefficient of permeability used in model calculations:

Poor 0.1 ft/day, or 0.03 m/day
Fair 100 ft/day, or 30.5 m/day
Good 10,000 ft/day, or 3050 m/day.

TABLE 19

RAINFALL DATA FOR SELECTED CITIES
IN PEAK MONTH OF 1978

	Total Precip. (inches)	No. Days Precip.	Average Daily Precip. (inches - mm)
Boston	8.12	15	0.54 - 13.7
San Francisco	6.20	16	0.39 - 9.9
Seattle	6.05	16	0.38 - 9.7
Los Angeles	7.70	10	0.77 - 19.6
Miami	2.57	11	0.23 - 5.8
Chicago	6.38	12	0.53 - 13.5
New Orleans	12.53	17	0.74 - 18.8

Thus, typical bases and subbases lie in the Poor to Fair range under this designation.

Quantifying the deleterious effects of water on pavement life requires estimates of (1) the reduction in sublayer materials properties during the time the pavement is significantly wet; and (2) the duration of this period of weakened strength (in terms of the three wet periods described earlier, the length of time between the starts of the second and third periods respectively). Unfortunately the answers to these key questions are not well supported by field documentation.

Von Quintus et al (41) presented data on seasonal changes in plate bearing capacity for a pavement having granular base, frost-susceptible subgrade, and high water table. Based upon the September bearing strength normalized to a value of 100, seasonal variations at this site ranged from about 20 in the spring-thaw months to over 140 in the frozen winter months. Values of relative damage factors between wet and dry periods were discussed earlier; Cedergren (39) calculated values of about 10-40 to 1 for the AASHTO Road Test. However, these data are not tied to detailed materials properties. We have therefore assumed that during the time of substantial pavement wetness, individual layer moduli are reduced by 50%.

Determining the time during which the pavement is sufficiently wet to affect performance is more difficult. Equations are available relating degree of drainage (i.e. percent of water removed from a saturated layer) to time, but again these data are not tied to changes in layer materials properties or in pavement performance. As a conservative estimate we have calculated drainage times on the basis of an assumed degree of 0.8. The implication of this and the preceding assumption is that in the time required to drain 80 percent of the water from a saturated layer, the sublayer moduli will be considered to be reduced in value by 50 percent in the EAROMAR simulation.

Based upon these general formulations, the following model relationships have been derived.

ESTIMATES OF DURATION OF PAVEMENT WETNESS

The duration of pavement wetness is determined by the interaction of water inflow and outflow characteristics of the pavement structure. As explained in the preceding section, outflow characteristics dominate the particular model within EAROMAR, and it is these relationships which will be explained first. Following the description of outflow equations, we will consider the influence of inflow parameters on the model.

The time required to drain a saturated subsurface layer is captured within the relationships shown in Figure 30 (40). The normalized time factor, t/m, is dependent upon U, the degree of drainage achieved; the width of road L to be drained; the depth of the drainage layer H_d; and the transverse slope of the drainage layer S:1. From our earlier discussion, we have assigned a value of 0.8 to U, the degree of drainage achieved, taken to be the point at which wetness no longer affects pavement structural behavior. Pavement cross-slopes typically vary from 1/8" - 1/4" per foot (1-2 cm/m); we have therefore assumed

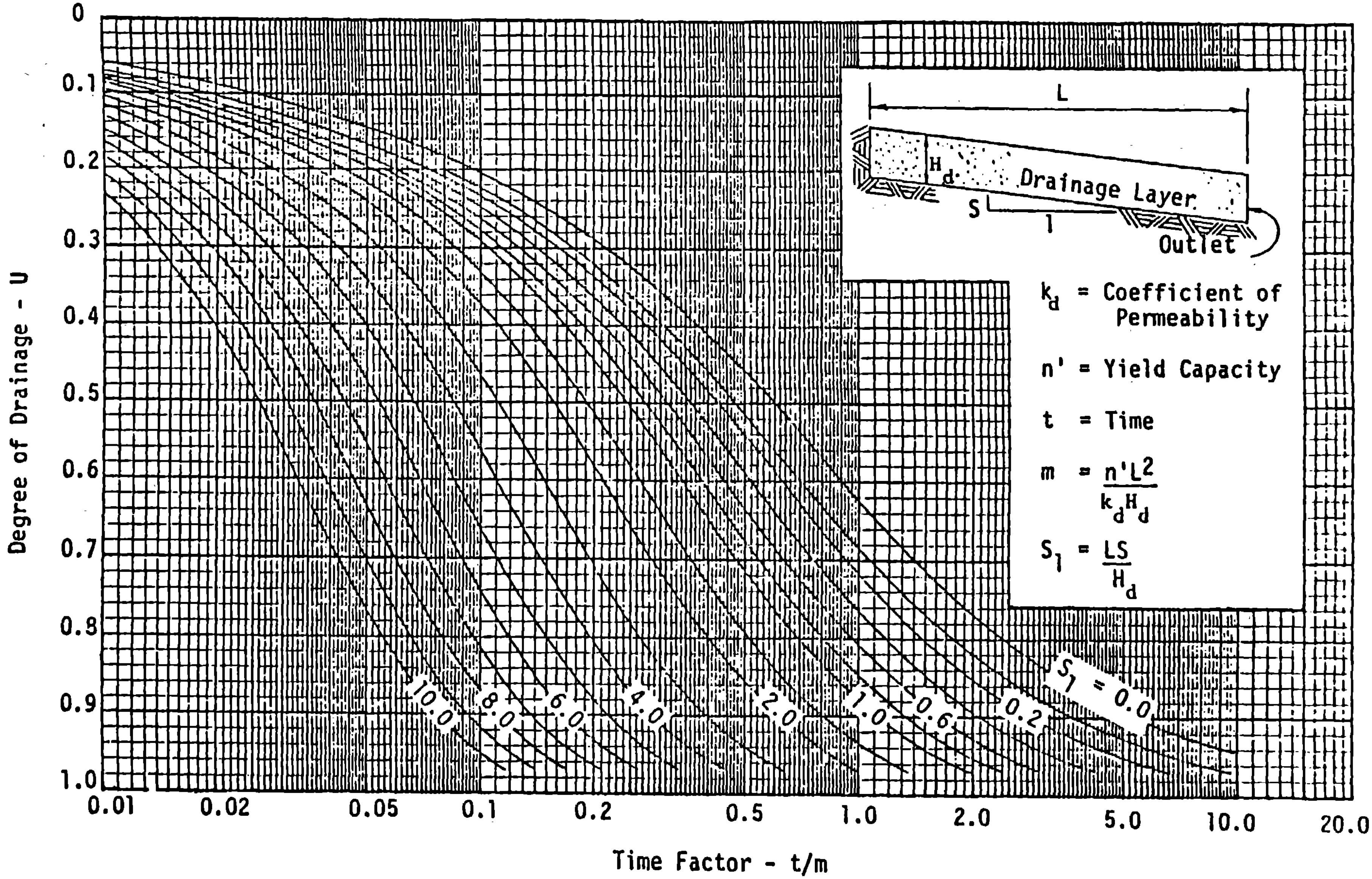


FIGURE 30: TIME DEPENDENT DRAINAGE OF SATURATED LAYER (40)

S, the slope factor, to be a constant equal to 0.015. Also, we have taken L, the width of road drained, conservatively to be equal to the sum of the widths of all lanes plus shoulders in the roadway. Finally, we have assumed H_d, the depth of the pavement drainage layer, typically to be about 1 ft (0.3 m). Based upon these assumptions, we fit the following function to data points generated from Figure 30:

$$t/m = 2.5e^{-2S'} \tag{159}$$

$$S' = 0.015\ L/H_d \tag{160}$$

$$L = (N_{lanes} \times W_{lane}) + \Sigma\ W_{shldr} \tag{161}$$

where t/m is a normalized time of drainage;

H_d is the thickness of the drainage layer, assumed to be 1 ft (0.3 m);

N_{lanes}, W_{lane} are the number and width, respectively, of lanes in this roadway; and

W_{shldr} are the widths of the left and right shoulders in the roadway.

The denominator of the normalized time is a function of the yield capacity of the drainage layer:

$$m = \frac{nL^2}{k_d H_d} \tag{162}$$

where

m is the normalizing factor;

n is the yield capacity (effective porosity) of the drainage layer;

L is the width of roadway drained, defined by eq. (161);

k_d is the coefficient of permeability of the pavement drainage layer; and

H_d is the thickness of the drainage layer, assumed to to 1 ft (0.3 m).

Values of n can be estimated from Figure 31 (40), using the coefficients of permeability assigned earlier to users qualitative descriptions of drainage:

TABLE 20. VALUES OF DRAINAGE PARAMETERS

Drainage Quality Input	Coefficient of Permeability, k_d	Yield Capacity, n
Good	10^4 ft/day (3050 m/day)	0.23
Fair	10^2 ft/day (30.5 m/day)	0.08
Poor	10^{-1} ft/day (0.03 m/day)	0.055

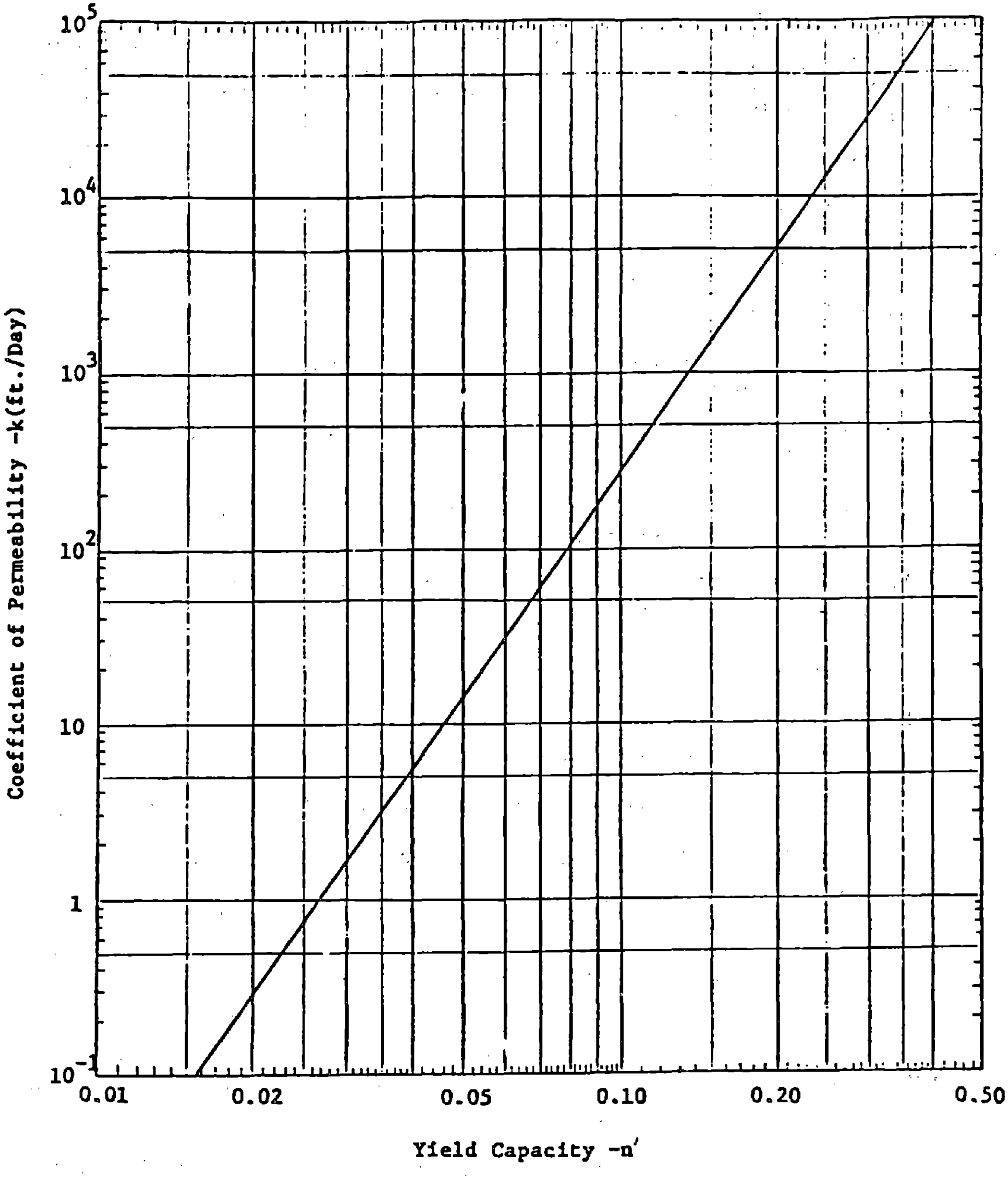


FIGURE 31. CHART FOR DETERMINING YIELD CAPACITY (EFFECTIVE POROSITY) (40)

With these values one can solve eq. (162) for m, and determine the normalized time t/m from eq. (159). The time of drainage corresponding to the period in which we have assumed pavement structural behavior is affected is then given by:

$$t_{drain} = m\ (t/m) \tag{163}$$

where

t_{drain} is the time to achieve a degree of drainage of 0.8, in days;

m is the normalizing factor computed in eq. (162); and

t/m is the normalized time computed in eq. (159).

COMBINING INFLOW AND OUTFLOW CONSIDERATIONS

Eq. (163) gives the drainage time of a saturated pavement layer once the rainfall has stopped. However, the quantity of water to be drained depends not only on the seasonal rainfall, but also on the condition of the pavement surface -- the number of open cracks and joints. We have treated both of these contributing factors to inflow as multipliers of the time computed in eq. (163). Thus, if either the seasonal rainfall (for a fixed amount of cracking) or the amount of cracking or open joints (for a fixed seasonal rainfall) increases, the time during which pavement structural response is affected will also increase. If either the seasonal rainfall or the open cracks and joints in the pavement are negligible, the time the pavement is affected will also be negligible.

Consider first the seasonal rainfall input by the user (Chapter 2.). To convert total precipitation to an equivalent time or duration comparable to the duration predicted in eq. (163), we require an assumed rainfall intensity. Data in Table 19 and in (39 , 40) suggest that a daily intensity of 0.5 in/day (12.7 mm/day) is reasonable as a composite national figure.

Findings reported in (39) showed that substantial quantities of water can enter even very narrow cracks in a pavement under field test conditions. (Cracks 1/8-inch [3 mm] wide admit more than 95% of water falling at an intensity of 2 in/hr [50 mm/hr], even with steep pavement transverse slopes. Cracks as narrow as 0.035-inch [0.89 mm] can absorb 70% or more of runoff at the same intensity.) In practice there rates may be reduced somewhat, due to debris at the bottom of the crack or to buildup of water in the crack. Nevertheless, infiltration rates become quite high at low levels of cracking or open joints in the pavement surface.

To model this relationship we have assumed the fraction of water inflow to be a negative exponential function of cracking and open joints, subject to assumed boundary conditions. Specifically, if there are no open cracks or joints in the pavement surface, water infiltration is reduced to zero. At cracking (or open joints) covering 50 percent of the pavement surface, infiltration is assumed to equal 99 percent of all water falling on the pavement area.

By combining the above assumptions, and incorporating a definition of the total area of discontinuities in the pavement surface, we obtain the following relationships:

$$t_{wet} = \frac{r_{season}}{i_{avg}} \left(1-e^{-9C}\right) t_{drain} \tag{164}$$

$$C = \frac{1}{5280} \quad \frac{LCRACKS + ACRACKS}{W_{lane}} + \frac{SHOULDER \times W_{wet}}{2\ W_{lane}\ N_{lane}}$$

$$+ JOINTS \times W_{wet} \tag{165}$$

where

t_{wet} is the duration of pavement wetness, in days, during which structural response is assumed to be affected;

r_{season} is the seasonal rainfall, in inches, input by the user

i_{avg} is the daily rainfall intensity, assumed to equal 0.5 inches (12.7 mm);

C is the fraction of pavement area having cracks or open (unsealed) joints;

t_{drain} is the time, in days, to drain the saturated pavement sublayers;

LCRACKS, ACRACKS, SHOULDER, JOINTS are quantities of damage components per lane mile computed in sections 3.2 and 3.3 or input by the user*;

W_{lane}, N_{lane} are the width of lane, in feet, and the number of lanes in the roadway respectively, as input by the user; and

W_{wet} is the width of subsurface zone wetted by an open joint, assumed to be 6 feet (1.8 m).

REDUCTIONS IN PAVEMENT STRENGTH

Pavement characteristics are affected by water infiltration in two ways. First, the strengths of granular bases and subgrades are reduced by 50 percent, as described in the general model formulation. Second, the AASHTO regional factor is adjusted to reflect saturated conditions above and beyond those assumed by the user in initial program input.** Resulting model relationships are as follows:

---

*LCRACKS in lf/lane mile; ACRACKS in sf/lane mile; SHOULDER (the length of open shoulder joint) in ft per roadway mile; and JOINTS in number of "damaged" or open joints per lane mile.

$$F_{red} = \frac{t_{season} - 0.5\ t_{wet}}{t_{season}} \qquad (166)$$

$$R' = \frac{(5t_{wet} + R[t_{season} - t_{wet}])}{t_{season}} \qquad (167)$$

where

F_{red} is a reduction factor applied to the moduli of granular pavement layers and to the CBR and moduli of the subgrade;

t_{season} is the length of the season, in days, determined from season information input by the user;

t_{wet} is the duration of pavement wetness, in days, computed from eq. (164);

R' is the AASHTO regional factor corrected for additional wetness due to a cracked pavement surface; and

R is the regional factor input by the user.

Note that eq. (166) and (167) apply a time-averaged correction (under wet vs. dry conditions) to the pavement materials properties and regional factor. Multiplication by 0.5 in eq. (166) reflects the assumed loss in materials strengths under wet conditions; the coefficeint 5 in eq. (167) reflects the value of the regional factor associated with saturated conditions.

The effects of water infiltration on pavement performance are therefore modeled indirectly within EAROMAR, through the materials-related and environmental adjustments indicated above. This makes it possible, however, to consider interactions between load-related and environmental influences on pavement damage (using the equations in sections 3.2 and 3.3) and to see what effects unsealed cracks and joints have on the rate of future pavement damage. The latter relationship in turn allows one also to investigate the benefits achieved through the routine maintenance actions of sealing joints and cracks.

**Regional factor captures moisture-related effects on pavement performance in AASHTO-based models. Therefore in reducing the strength of the subgrade due to water infiltration we adjust only the regional factor, not the soil support value.

Pavement Overlays

Pavement overlays may be specified by the user under construction activities, as described in Chapter 2. User specifications include not only the physical description of the proposed overlay (thickness, materials properties), but also policy statements on the timing and extent of the work to be performed. The analytical task within EAROMAR is to simulate the structural performance of the overlaid pavement, accounting for the residual strength of the original pavement and the contribution of the new surface layer. This process is described below for flexible overlays on non-rigid and rigid pavements respectively.* In addition, the treatment of reflection cracking is presented for overlaid rigid pavements.

FLEXIBLE OVERLAYS OF FLEXIBLE PAVEMENTS

Although methods have been developed to design overlays of flexible pavements, little research has been devoted to overlay performance, including the structural response and materials properties of the old pavement under a new surface layer. The performance models in EAROMAR were therefore adopted from data developed for existing design procedures.

For design purposes the strengths of existing pavements due to be overlaid are reduced from their as-constructed values. AASHTO (5) recommended that the layer coefficient of the existing surface, typically 0.44 for high-quality bituminous mixes, be lowered to 0.24 (or to 0.40 for "new" pavements). ARE Inc. (42) related elastic moduli of the existing surface to the class of cracking present when overlaid, as shown in Table 21. The degree to which cracking has progressed is explicit, but not the extent of cracking.

TABLE 21

REDUCTIONS IN FLEXIBLE SURFACE MODULUS
USED IN OVERLAY DESIGN

Surface Cracking	Modulus
Uncracked	Equal to Overlay Modulus
Class 2 Cracking	70,000 psi (483,000 kN/m^2)
Class 3 Cracking	20,000 psi (138,000 kN/m^2)

*Our emphasis in model developement was on flexible overlays for all pavements. In principle a rigid overlay may be specified within EAROMAR, and the model will accept the resulting pavement structure for stimulation. However, we do not consider the rigid pavement damage equations in EAROMAR appropriate to rigid overlays, and discourage use of the rigid overlays at the present time.

We have consolidated these ideas in EAROMAR in the following way. The value of the reduced AASHTO layer coefficient for the surface was taken to be dependent upon both pavement age and the amount of cracking present when overlaid. We assumed that at an age of 10 years, in the absence of any cracking, a pavement surface would function as a reasonably good black base following an overlay. If the original surface coefficient were 0.44, and that of the equivalent base 0.30, this would mean a reduction in the layer coefficient of about 70% at 10 years.

We further assumed that cracking of about 5,000 sf/lane mile (83 sf/ 1,000 sf, or 0.8 m^2/m^2) at age of 10 years reduced the coefficient of the layer (when overlaid) to 0.24 - that of an "old" bituminous pavement in the AASHTO Guide. This value is about 55 percent of the value when new.

We then conbined these assumptions on the basis that age and cracking both reduce the strength of the pavement surface simultaneously. The resulting equation to estimate the AASHTO coefficient of a flexible surface when it is overlaid is as follows:

$$a' = a_o \exp(-0.0357\ A - 0.0000575\ ACRACKS) \qquad (168)$$

where a' is the AASHTO coefficient of the former surface layer following an overlay;

a_o is the original surface layer coefficient input by the user;

A is the age of the pavement surface, in years, input by the user and updated during the simulation; and

ACRACKS is the amount of areal cracking, in sf/lane-mile, computed during the simulation as described in section 3.2.

Treatment of the surface elastic, complex and resilient moduli followed upon the guidelines suggested in (42). However, within EAROMAR the value of 75 ksi (517 MPa) defines the boundary between stabilized and unstablized pavement bases. We therefore adjusted the modulus values in Table 21 slightly so that a pavement subject to Class 2 cracking would continue to perform as a stablized base. We also associated extents of cracking with the moduli values in Table 21. estimated from data for road sections contained in (5). The resulting adjustments to surface elastic, complex and resilient moduli simulated within EAROMAR following an overlay are given in Table 22.

TABLE 22

VALUES OF FLEXIBLE PAVEMENT MODULUS ASSUMED IN EAROMAR

Surface Cracking	Modulus
1. < 5,000 sf/lane-mile ($< 0.08\ m^2/m^2$)	Equal to Modulus Value Input for that Layer
2. 5,000-15,000 sf/lane-mile ($0.08 - 0.24\ m^2/m^2$)	76 ksi (524 MPa)
3. > 15,000 sf/lane-mile ($> 0.24\ m^2/m^2$)	20 ksi (138 MPa)

FLEXIBLE OVERLAYS OF RIGID PAVEMENTS

Rigid pavement overlays are handled much in the same way as described for flexible pavement overlays above. However, in addition to weakening of the old surface layer, we consider reflection cracking originating in the rigid surface and propagating through the newly-placed overlay.

Weakening of Old Surface Layer. The modulus of portland cement concrete used in pavements typically ranges from 2 to 5 million psi (13.8 to 34.5 million kPa). ARE Inc. (43) recommended retaining this value for overlay design if the surface exhibited no cracks of only Class 1 or Class 2 cracking, or adopting a value of 500,000 psi (3.45 million kPa) if the surface exhibited Class 3 or Class 4 cracking. (A modulus of 70,000 psi, or 483,000 kPa, was suggested for surfaces to be broken up mechanically before placing the overlay.)

Layer coefficients for overlaid rigid pavements, developed by Louisiana, were presented in the AASHTO Interim Guide (5). These values range from 0.40 for an "old" surface, to 0.20 for one that is "old, failed," to 0.10 for one that is "old, pumping." For comparison, a "new" concrete surface was associated with a value of 0.5.

We consolidated these effects due to age and damage, considering that an uncracked surface 10 years old (with an initial layer coefficient of 0.5) should have a revised layer coefficient when overlaid of 0.4; and a 10 year old surface with cracking of 5,000 lf/lane-mile (0.26 m/sm), a revised layer coefficient of 0.20. The resulting equation incorporated within EAROMAR to estimate these reductions is as follows:

$$a' = a_o \exp(-0.0223\ A - 0.00014\ LCRACKS) \qquad (169)$$

where a' is the AASHTO coefficient of the former surface layer following an overlay;

a_o is the original surface layer coefficient input by the user and updated during the simulation; and

LCRACKS is the amount of lineal cracking, if lf/lane-mile, computed during the simulation as described in section 3.3.

Reductions in elastic modulus values as a function of surface condition prior to an overlay are given in Table 23.

TABLE 23

VALUES OF RIGID PAVEMENT MODULUS ASSUMED IN EAROMAR FOR OVERLAYS

Surface Cracking	Modulus
1. < 5,000 lf/lane-mile ($<0.26\ m/m^2$)	Equal to Modulus Value Input for that Layer
2. $\geq$ 5,000 lf/lane-mile ($\geq 0.26\ m/m^2$)	500 ksi (3.45 million kPa)

Reflection Cracking. Reflection cracks are cracks originating at discontinuities in the underlying concrete slab, which propagate through the overlay layer. Crack formation can be due to differential horizontal or vertical movements at cracks or joints in the underlying slab. Many methods have been tried in the field, with varying success, to reduce reflection cracking, including breaking up the old pavement surface, placing granular layers or fabrics to act as bond breakers between the overlay and concrete slab,reinforcing the overlay, increasing overlay thickness, and sawing joints in the overlay (43, 44, 45, 46).

Apparently because of the different potential mechanisms involved, no generally used models of reflection cracking exist. For a performance model within EAROMAR we have therefore relied on data presented in (43) and supported by other sources, indicating that crack reflection appears in overlays a very short time after construction, and that nearly 100 percent of joints and cracks in the underlying slab can be reflected within a few years. Thicker overlays resist cracking as shown in Figure 32.

The model within EAROMAR is based upon the simplifying assumption that only joints, and not cracks, in the PCC slab will initiate reflection cracking. From the data in Figure 32, the following relationships were derived to simulate reflection cracking in an overlay of a rigid pavement:

$$C_{reflect} = \frac{3}{H_o} (1 - \exp[-A/3]) \times \frac{5280}{L} \times W_{lane} \qquad (170)$$

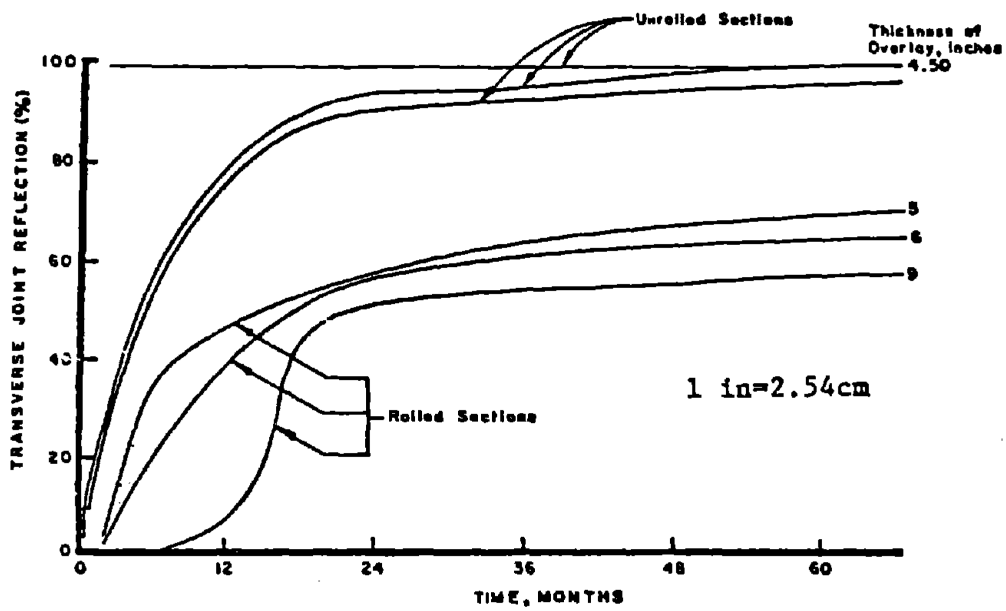


FIGURE 32: EFFECT OF INCREASED OVERLAY THICKNESS AND PAVEMENT BREAKING BY ROLLING ON REFLECTION CRACKING (43)

$$C_{reflect} \leq \frac{5280}{L} \times W_{lane} \qquad (171)$$

$$LCRACKS = LCRACKS + C_{reflect} \qquad (172)$$

where $C_{reflect}$ is the extent of reflection cracking in the overlay surface, in lineal feet per lane-mile;

H_o is the thickness of the overlay, in inches, input by the user;

A is the age of the overlay surface, in years, computed during the simulation;

L is the joint spacing, in feet, of the underlying PCC slab (input by the user for the old PCC surface);

W_{lane} is the width of the roadway lane, in feet, input by the user; and

LCRACKS is the amount of linear cracking, in lf/lane-mile, simulated for the overlay surface.

This model applies only when the overlay surface is placed directly above the PCC slab. If the user wishes instead to simulate an interlayer (whether a gravel cushion, fabric, or some other type), he may involve the construction PROJECT option in lieu of the OVERLAY option within EAROMAR, specify the pavement structure following the overlay (PCC slab, interlayer, overlay surface), and provide all other project details as he would have for the overlay. The EAROMAR system will then simulate the resulting pavement structure as a flexible or a composite pavement, but will not invoke eqs. (170-172). (The user may account for any reflection cracking he feels may appear by specifying a damage rate for LCRACKS in his input.)

A similar approach may be used if the PCC slab is to be broken up before overlaying. In this way EAROMAR can account at least indirectly for several of the procedures used to control reflection cracking.

CHAPTER 4

MAINTENANCE POLICIES, SCHEDULING AND COSTS

4.1 INTRODUCTION

Over the past fifteen years highway maintenance has made increasing use of management techniques for work accomplishment analysis, planning and budgeting. The development of maintenance management systems in the 1960's through the 1970's instituted practices of systematic data collection, work reporting, and extrapolation of past trends to future planning estimates.

In the 1970's, the completion of many roadbuilding programs (including much of the Interstate system) created large additions of road physical inventory requiring maintenance. At the same time, factors affecting the economical timing and performance of maintenance were changing, including higher traffic volumes, higher allowable vehicle weights, and increased costs for maintenance labor and materials. These trends combined to force a new awareness of maintenance as a demand-responsive activity; i.e., one whose levels of performance and costs cannot be estimated adequately from extrapolation of past trends, but rather must be based upon rational assessments of anticipated future deterioration of the highway system. This awareness led to development of damage prediction models for use in maintenance management and road project evaluation systems.

However, it is generally impossible for an agency to muster the resources necessary to accomplish all required maintenance work at a given time; indeed, it may not even be economically efficient to do so. The allocation of scarce resources among maintenance activities requires conscious policy decisions by highway administrators regarding the types of maintenance activities to be funded, the intensity of these activities, and their scheduling among various budget periods and maintenance districts. This fact led to research to formalize maintenance policies in terms of quality standards, and to consider as well the impacts of particular levels of maintenance performance in terms of preservation of road investment, user consequences, and safety (47).

The treatment of pavement maintenance within the EAROMAR system is drawn directly from these state-of-the-art developments in maintenance management. Three important characteristics of maintenance planning are embodied in this approach: (1) the specification of maintenance policy to address predicted pavement damage, which determines what maintenance will be performed; (2) the scheduling of maintenance operations by district on a seasonal, daily and hourly basis, which determines when the specified maintenance will be performed; and (3) the identification of labor, equipment, and materials requirements, crew production rates, and unit resource costs, which determine the cost of the planned maintenance.

Based upon these user-supplied data (which collectively define a total maintenance program in an engineering, economic and technological sense), EAROMAR estimates the impacts of the level of maintenance performed in terms of:

1. preservation of investment: improvements in the pavement's current condition and rate of future deterioration;

2. user consequences: changes in operating cost, travel time, and vehicle emissions; and

3. safety: variations in accident likelihood and severity.

The following sections discuss the policy, scheduling, and cost aspects of the maintenance simulation respectively. Consistent with the scope of this research, emphasis is placed upon the structural (i.e., load-induced) components of pavement deterioration and repair. Thus, factors such as skid resistance or construction quality control are excluded. In discussing maintenance policy, scheduling or cost considerations individually, the following sections venture in some cases into considerable explanatory detail, and therefore stand independently of one another. Nevertheless, one should realize that the simulation of the total maintenance program results from the interaction of all three components; and by skillful specification of input data within each of the respective areas, one may represent virtually any realistic maintenance situation.

4.2 DEFINITION OF MAINTENANCE POLICY

Background

Many states have incorporated models within their management systems to predict annual maintenance requirements or costs. Because these relationships are often based upon historical data or upon regression analyses drawn from existing practices, they include implicitly a particular level of maintenance performance — namely, the standards to which the roads have been or are currently being maintained. Moreover, to the extent that such models estimate directly the cost per unit inventory (or the work effort per unit inventory), without considering the road deterioration or damage (or the specific maintenance policy) leading to the projected maintenance requirement, these models develop maintenance predictions based upon work outputs rather than inputs, and are therefore ill-equipped to treat differences in input values (e.g., productivity, unit costs, changes in maintenance technology) among geographic regions or over time. To illustrate the implications of these facts for premium pavement analysis, let us consider some examples from the original EAROMAR system.

Figures 33 , 34, and 35 illustrate maintenance prediction relationships in the original EAROMAR system for the activities of patching bituminous concrete pavements, patching PCC pavements, and mudjacking respectively. Figures 33 and 34 are in terms of maintenance dollars per lane mile, while Figure 35 is in terms of aggregate dollars. Note, however, that all are measures of output (i.e., total work performed), and are limited to considering the supply of maintenance services.

Even if such data were used to back-compute probable levels of damage, these results would be limited for the following reasons:

1. They are based upon the one particular maintenance policy inherent in the source data, and no other. As a corollary, the maintenance level-of-effort and cost requirements over time in Figures 33-35 are based upon one particular road condition history, and no other. Thus, one may not be able to relate these results to other pavements, where, for example, more extensive initial maintenance was performed, improving the subsequent damage history, and therefore reducing later maintenance required; or, where maintenance was initially deferred, causing higher rates of subsequent damage, and eventually leading to more extensive remedial action required.

2. The relationships in Figures 33-35 are insensitive to factors influencing rate of deterioration, and therefore the demand for maintenance work. These influencing items include magnitude and frequency of physical loads, environmental conditions, and quality of initial design and construction, as were discussed in Chapter 3.

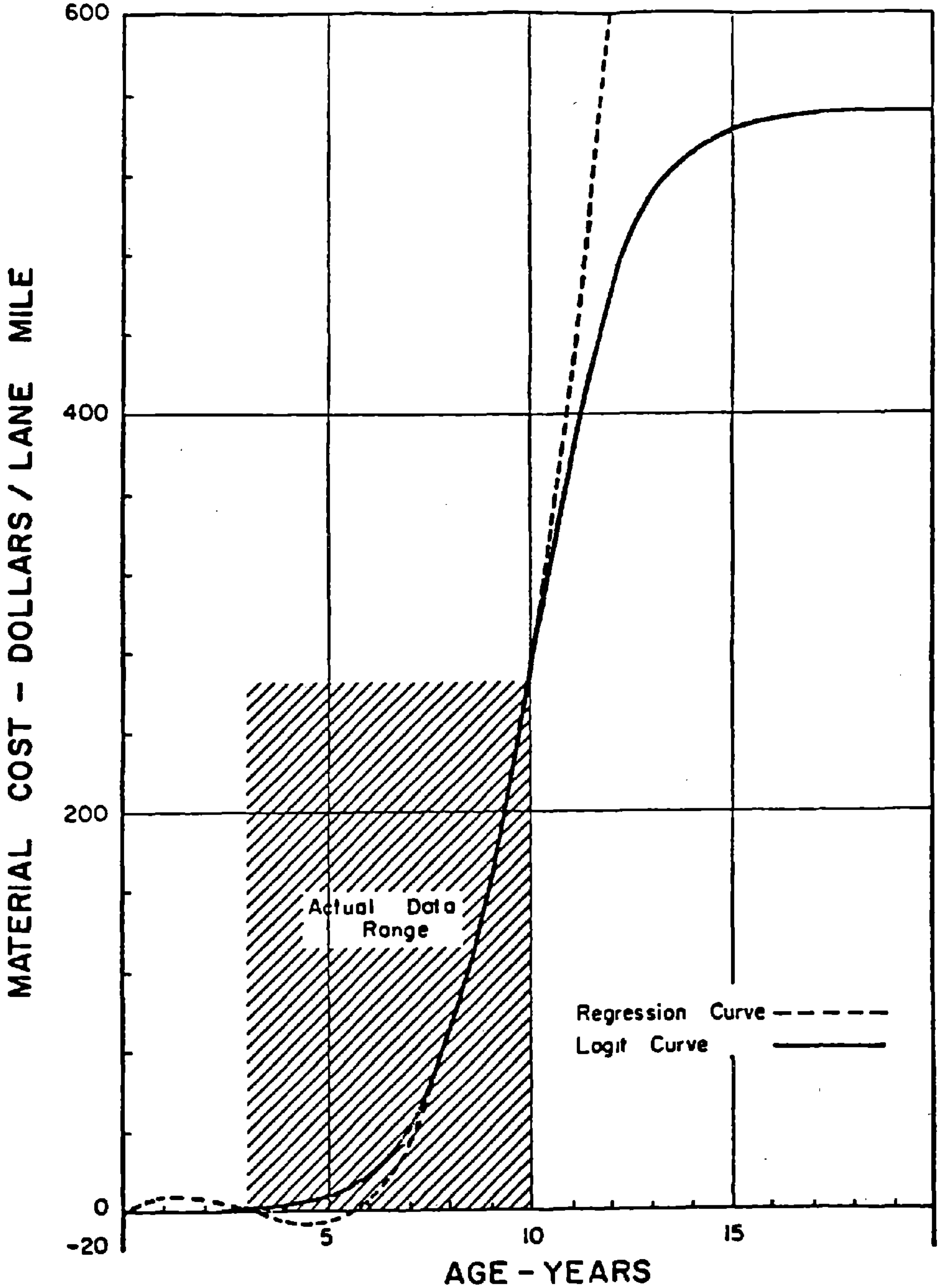


FIGURE 33: PATCHING CURVES FOR BITUMINOUS CONCRETE PAVEMENTS (2)

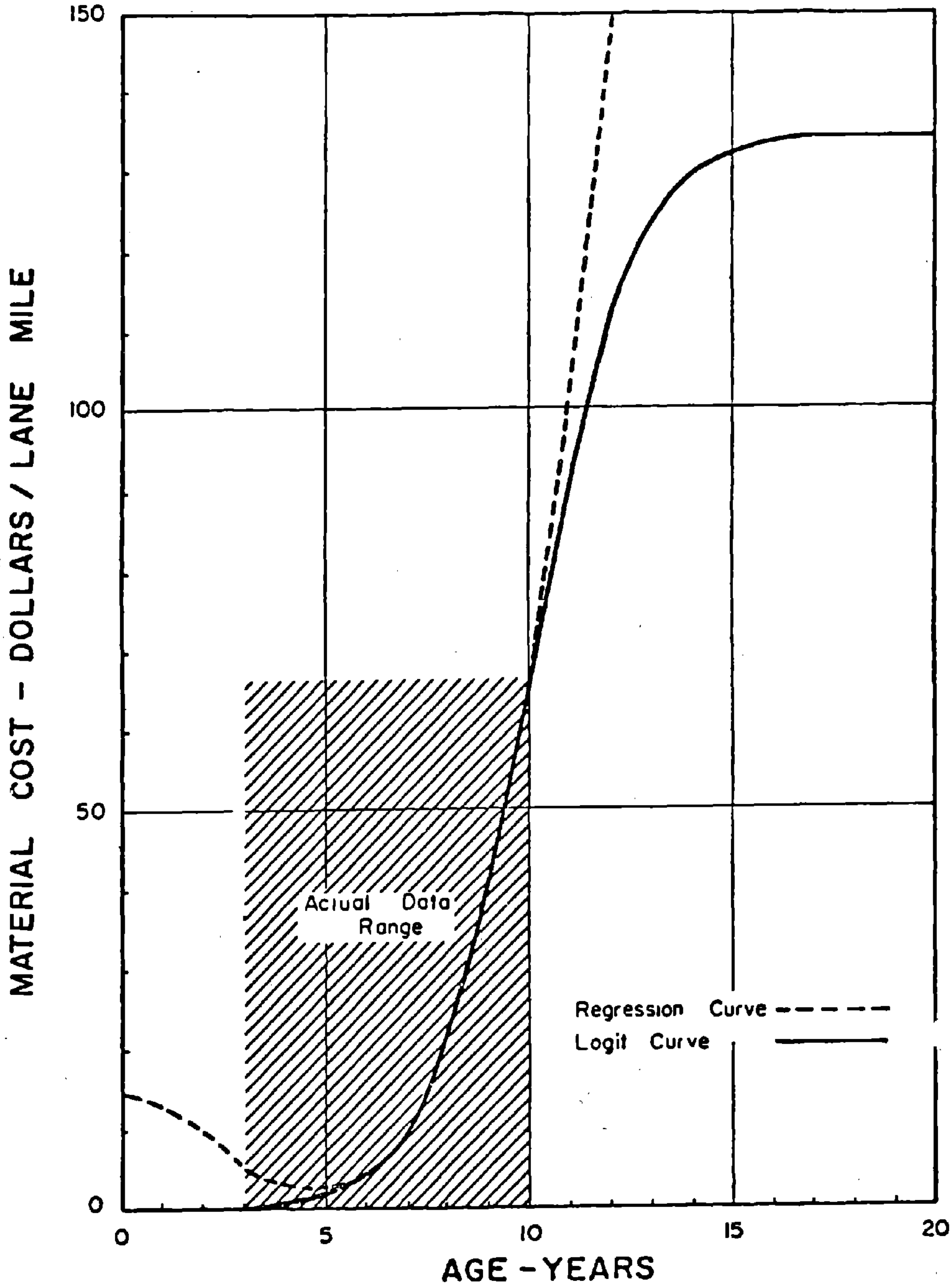


FIGURE 34: PATCHING CURVES FOR PORTLAND CEMENT CONCRETE PAVEMENTS (2)

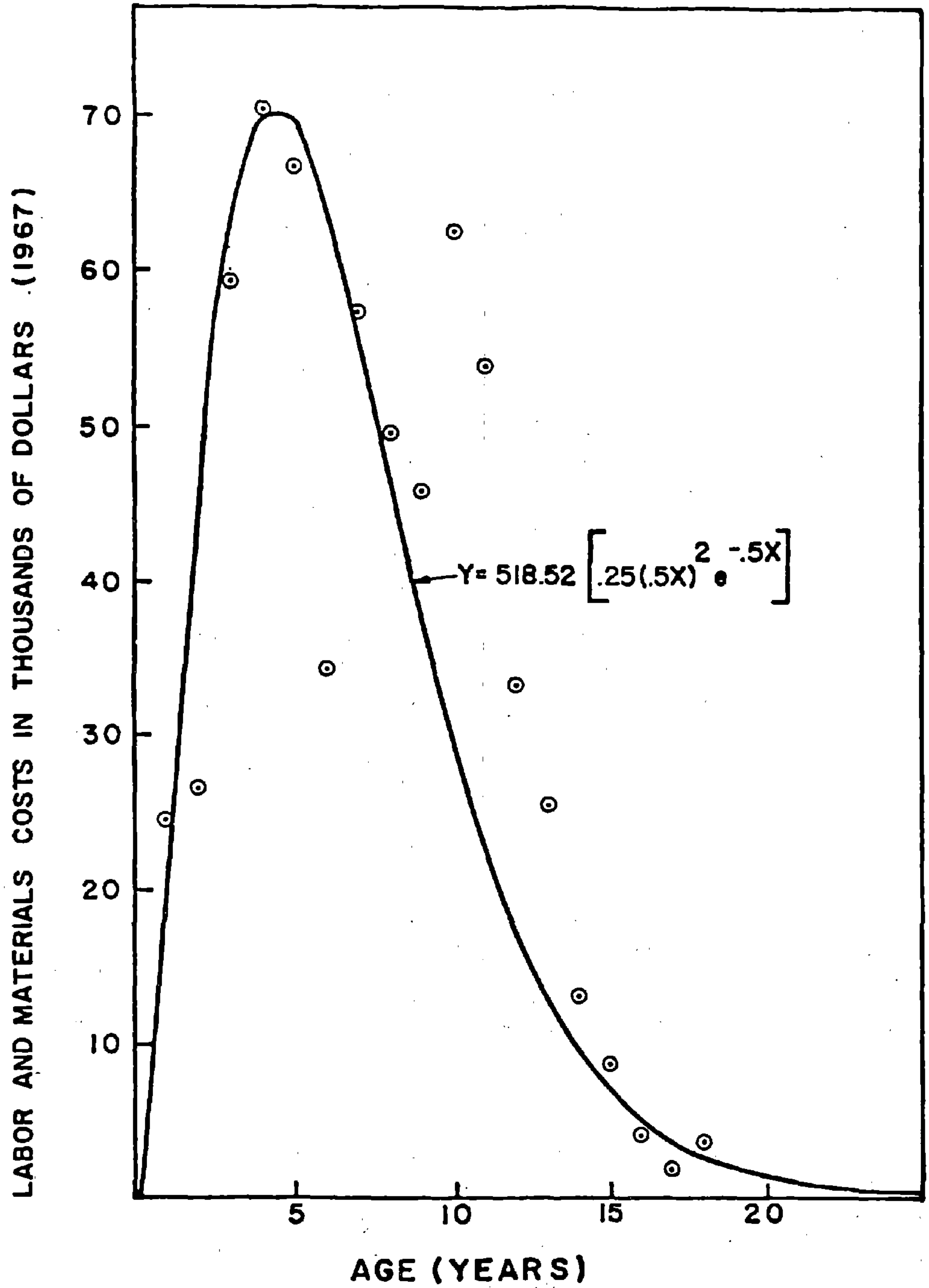


FIGURE 35. CURVE FIT TO MUDJACKING EXPENDITURES AS REPORTED BY THE OHIO TURNPIKE AFTER CONVERSION TO 1967 DOLLARS (2)

3. It is difficult to adjust the data in Figures 33-35 to account for the following changes, whether individually or collectively:

- Maintenance technology (a function of labor, equipment, and materials employed)

- Crew productivity (a function of the size of the crew, labor skills and motivation, and equipment and materials employed)

- Unit costs (a function of local economic and institutional conditions, prevailing safety practices, methods of procurement, work space or time limitations, and classes of labor, equipment or materials employed.)

To circumvent these problems in maintenance prediction it is necessary to go beyond the supply of maintenance services to consider the demand for maintenance work, and to cost this work on the basis of inputs to the maintenance process -- labor, equipment and materials. Maintenance demand arises through both a physical dimension -- the condition (or damage) of the pavement, a function of the combined actions of road usage, response of the physical systems, and environmental influences -- and a policy dimension -- the desired or specified level of service, expressed through quality standards. Demand-responsive representations of maintenance are essential to assess changes in pavement condition and rate of deterioration over time (i.e., measures of preservation of investment), and impacts of pavement maintenance upon attendant user considerations regarding cost, safety and vehicle emissions.

The discussions of maintenance policy to follow employ these demand-responsive characteristics in terms of quality standards. Also, use is made of the existing pavement condition as determined by the deterioration models in Chapter 3. To relate quality standards to specific components of damage, the EAROMAR analysis includes a set of well-defined maintenance activities that may be simulated. These activities are discussed for each pavement type below.

Maintenance Activities

Pavement maintenance encompasses several actions in response to the type of pavement surface, evident damage, climate, traffic, and desired level of service involved. Approaches to maintenance differ somewhat from state to state, for a number of possible reasons: variations among the several influencing factors above, administrative or definitional differences, or local preferences among several maintenance technologies available, to name a few. Nevertheless, it is possible to categorize maintenance actions for pavement within a general set of activities, within which one may express different mixes of labor, equipment and materials employed, and extent of damage repaired.

The set of activities included within EAROMAR is used to simulate all maintenance work projected through the analysis period. Activities are related to one or more damage modes predicted by the pavement deterioration models in Chapter 3, and are also keyed to maintenance policy specifications to be described later in this chapter. The activities represent typical operations recognizable to highway maintenance personnel, and are flexible enough in their descriptions to allow for realistic variations in methods of performance, sizes of crews employed, or specialized equipment or materials required for a given roadway.

We should also point out that in the following technical discussions of maintenance we have intentionally included overlays. Because of their potential for structural contribution to the existing pavement, overlays are not generally considered a maintenance activity (they are rather a strengthening or betterment); and in fact we treat overlays as a project, and not a maintenance activity, within the EAROMAR simulation.* Nevertheless, overlays obviously correct several types of surficial distress, and may improve measured road condition considerably, thereby affecting the amount of routine or periodic maintenance required. It is this interaction between the remedial effects of overlays and those of less intensive maintenance activities that we wish to focus on below.

FLEXIBLE PAVEMENTS

Flexible pavement activities modeled include crack filling, seal coating, pothole repair, skin patching, and deep patch and base repair, in addition to overlays.

Technical Review. Before we describe the treatment of these maintenance activities within EAROMAR, it may be useful to review briefly their technical aspects, particularly as to how their practice varies among different states. General guidelines to the applicability of these activities are as follows:

1. Crack filling is applicable where moderate fatigue, transverse, or longitudinal cracking occur, but with no surface deformation present.

2. Seal coating may be applicable where intensive cracking and/or spalling, raveling or surface disintegration occur, particularly where large areas are involved. It is not suitable for correcting surface deformations.

3. Pothole repairs are appropriate when such depressions appear in the pavement surface.

4. Skin patching is applicable where there is some evidence of surface deformation, such as rutting or other forms of transverse or

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* Projects within the EAROMAR system are described in Chapter 2.

longitudinal roughness.

5. Deep patching and base repairs are appropriate where evidence of base failure exists. Thus is it often applicable in conjunction with (or prior to) skin patching or overlay operations.

6. Overlays are applicable where surface deformation greater than can be remedied by skin patching are evident over extensive areas.

Although the above descriptions correspond to generally understood terminology, state maintenance departments may adopt somewhat different, or more refined, definitions based upon past experience and local practice. In addition, the threshold levels at which maintenance should be performed, the technology employed for maintenance accomplishment, and the specified finished conditions following maintenance also vary within limits.

Tables 24 through 26 summarize data for skin patching, surface sealing, and deep patching activities, illustrating typical variations among state maintenance department practices. These characteristics have been abstracted from the performance standard sheets of the respective highway maintenance departments. The implications of these data for EAROMAR are that a degree of adaptability is highly desirable in describing the technology, performance, costs, and remedial effects of each maintenance activity. This adaptability is embodied within both the policy specifications and the activity management information provided by the user to the EAROMAR system, as will be described in subsequent sections.

Labor-Intensive vs. Equipment-Intensive Technology. One of the major decisions to be made in maintenance management, and which significantly affects costs, is at what point repairs by machine should replace handwork. In general, the decision to use the more productive and economical machine work is made when that area to be maintained is sufficiently large. Exactly how large is a decision which is often made by maintenance personnel and can be guided by the fact that to mobilize a spreader, trucks, ancillary machinery and the work crews necessary with this equipment, at least a whole day's work effort in continguous areas is necessary. The decision may also be affected by whether or not a private contractor can offer a competitive price compared to the regular agency operations.

The conceptual relationship between various maintenance activities and when they should be undertaken in response to specific damage components is outlined in Table 27. For the surface defects in particular, where the more extensive areas are involved machine methods are indicated. Thus seal coating would replace crack filling where cracking is intensive and extends over large areas. Also, skin patching would normally be performed by machine methods over the larger areas.

	NEVADA	NEVADA	ILLINOIS
Activity No.			
Activity Name	Surface patching - Premix (hand).	Surface Patching - Premix (machine).	Pavement repairs, bituminous
Description	Premix and liquid asphalt hand placed.	Premix and liquid asphalt placed by machine.	Bituminous concrete and asphalt primer tack coat laid by machine including rolling.
Damage Condition	Vertical differential of pavement in any direction exceeds half an inch or differential with paved shoulder is three quarters of an inch.	As above.	Pavement surface spalling, raveling or showing signs of settlement.
Maintenance Level	Restore loss of surfacing by raveling or other causes, which affect riding qualities or surface seal.	As above.	Roadway surface should be maintained to provide a reasonably comfortable riding surface and provide a safe travel surface.
Workload Rate	0.75 cubic yards per mile of 24 foot bituminous surface.	6 cubic yards per mile of 24 foot bituminous surface.	This acitivty can be either a spot location or a continuous mileage location depending upon the extent of work. The activitiy includes bituminous repair of concrete, bituminous, or overlay pavement which has deteriorated due to cracking, raveling, spalling or rutting.

TABLE 24

Comparison Between State Performance Standards - Skin Patching

	COLORADO	COLORADO	MASSACHUSETTS
Activity No.	483	484	621
Activity Name	Patching - machine overlay, and levalling.	Patching - hand.	Surface treatment (bituminous concrete method).
Description	Bituminous mix and tack coat placed by machine.	Bituminous mix and tack aisle.	Bituminous concrete and liquid asphalt.
Damage Condition	Deterioration of bituminous surface due to settlement, or raveling.	Deterioration of bituminous surface due to potholes and depressions.	Extended surface deterioration (5 or more sq. yds.) indicated by cracking, spalling, rutting and with no indication of base failure.
Maintenance Level	Restore asphalt surface to original condition.	Restore loss of asphalt surface to level of exisiting roadway surface.	Restore to a smooth, level, impervious surface.
Workload Rate	10.0 tons per mile of 24 foot bituminous surface per year.	0.8 tons per mile of 24 foot bituminous surface.	This activity is to be carried out to remedy areas of bituminous surface roadway and/or shoulder surfaces with hot bituminous concrete, 1/2" $\pm$ in depth, to correct poor pavement surfaces, spalling or rutting and to maintain a rideable surface until roadway is completely resurfaced. The activity includes the use of dump trucks, pavers, and rollers. Average daily production is approximately 200 tons.

TABLE 24

Comparison Between State Performance Standards - Skin Patching

(continued)

	NEVADA	NEVADA	NEVADA
Activity No.			
Activity Name	Surface patching - spot seal.	Seal coat - sand.	Seal coat - flush
Description	Liquid asphalt and sand.	Liquid asphalt and sand with mechanical equipment.	Aspahlt emulsion and sand with mechanical equipment.
Damage condition	Cracking and checking of surface has become general and perveious to water.	Pavement surface is cracked or checked allowing penetration of water.	Surface raveling, cracking or other deterioration determined by inspection.
Maintenance Level	Maintain surface of pavement imperveious to water to prevent cracking of old or oxidized surface.	Maintain asphalt surface to prevent penetration by water.	Renew old asphalt surface and seal small cracks and surface voids.
Workload Rate	--	1700 square yards per mile of 24 foot bituminous surface.	2800 square yards per mile of 24 foot bituminous surface.

TABLE 25

Comparison Between State Performance Standards - Seal Coat

	NEVADA	COLORADO
Activity No.		
Activity Name	Seal coating - chip, sand, fog, slurry	Seal coat - chips
Description	Liquid asphalt and aggregate.	Liquid asphalt and stone chips with mechanical equipment.
Damage Condition	Deterioration of bituminous surface due to cracking and raveling.	When asphaltic surface becomes checked and raveled so as to be permeable to water and surface loss is occuring, and the work is of such and extent as to preclude the use of regular maintenance operations.
Maintenance Level	Restore or renew deterioration surface.	A renewed and revived surface that will prevent penetration of water into underlying base material.
Workload Rate	Production 42000 square yards daily. Apply a chip, sand or fog or slurry coat to continuous sections of bituminous roadway surfaces to see cracks and rejuvenate dry, weathered surfaces to prevent further surface deterioration.	500 square yards per mile of 24 foot bituminous surface.

TABLE 25

Comparison Between State Performance Standards - Seal Coat
(continued)

	NEVADA	COLORADO	MASSACHUSETTS
Activity No.	233	421	282
Activity name	Base and surface repair.	Base Stabilization and repair, activity #164.	Deep patch and base reapir, activity #612.
Description	Aggregate base, premix, and liquid asphalt.	Remove and replace base and surface material using premix bituminous material and/or required base material to correct severe cracking, upheavals and base failures. Materials required are base aggregate, bituminous mix, graders, rollers, etc.	Base material, liquid asphalt and bituminous concrete placed by machines including loaders, rollers and pamper.
Damage Condition	Pavement disintegration occurring along with vertical and lateral movement of the surface, with showing of fines in and around cracks.	Deterioration of bituminous and concrete surfaces due to unstable base material.	Pavement disinte-ration with vertical and lateral movement of the surface with a showing of fines in and around cracks.
Maintenance Level	Maintain the surface of the highway in a condition providing reasonable comfort and safety to public traffic.	Removal of unsuitable base material and replacement of base and surface to restore proper conditions.	Restore base and replace surface to level of existing roadway.
Workload Rate	0.1 cubic yards per mile of 24 foot bituminous surface. Work may extend to pavement sub-base.	0.1 tons per mile of 24 foot surface.	Removal and replacement of all areas of bituminous roadway, shoulder surfaces and PCC pavement, including removal and replacement of base material using hot bituminous material and required base material to correct severe cracking, upheavals potholes clusters, frost boils and base failures. Average daily production 120 square yards.

TABLE 26. Comparison Between State Performance Standards - Deep Patch and Base Repair

Table 27

COMPARISON OF DAMAGE WITH APPROPRIATE MAINTENANCE ACTIVITIES FOR FLEXIBLE PAVEMENT

TYPE OF DEFECT	MAINTENANCE ACTIVITIES	DAMAGE COMPONENT
Surface	Crack filling -or- Seal coat, slurry, sand	Surface fatigue cracking, transverse and longitudinal, with no significant surface deformation or spalling: i) In limited areas -or- ii) Over extensive contiguous areas
	Seal coat, possibly with gravel, stone chips or sand	Spalling and ravelling with no significant deformation, but possibly some cracks also
	Skin patch (hand) -or- Skin patch (machine)	Minor (0.5 in. $\pm$) rutting, transverse or longitudinal roughness: i) In limited areas -or- ii) Over extensive areas
	Overlay	Extensive areas of severe (1 1/2 in. $\pm$) rutting, transverse or longitudinal roughness but no indication of pavement or base damage
Full Depth	Full depth pavement repair	Base failure--almost always includes one or more of the above surface defects but is identified by severe surface deformation characteristics also
Full or Partial Depth	Pothole repair	Potholes--can occur in conjunction with any of the above conditions.

Interaction Among Activities. From the discussion above, it is clear that the beneficial effects of some flexible pavement activities overlap those of others. That is, there does not exist a simple one-to-one correspondence between specific damage components and associated maintenance activities. For a given damage component more than one activity may be applicable, as with cracking (where both crack filling and sealing are reasonable actions). On the other hand, for a given maintenance activity more than one damage component may be repaired; skin patching, for instance, may correct both roughness and cracking.

The relationship between damage components and relevant maintenance activities may therefore be represented conceptually as in Figure 36 . The shaded areas indicate the potential remedial effects of each activity. Note that minor damage will almost always be repaired by activities designed to correct the more severe deficiencies that occur in the same areas. For example, skin patching will normally obviate the need for separate crack filling operations because the applied tack coat and preparatory work will seal the cracks prior to laying the patch. Also, the patch itself will assist in sealing the cracks.

Further note that the activities are not themselves mutually exclusive. For example, the deep patch and base repair activity may occur in conjunction with skin patching when surface deformations are present due to base failure. Pothole patching may occur in conjunction with any of the other activities.

These types of considerations are not a problem in field performance of maintenance, where visual inspections determine the type, extent, and likely causes of damage at a pavement location, and result in the scheduling of appropriate corrective actions. In representing maintenance within a conceptual model, however, the relationship between specific types of damage and relevant maintenance or betterment activities needs to be formalized by a well-defined set of rules.

One reason for this need is that pavement condition can only be predicted in an average sense over some length of road segment. Quantities of each component of damage are estimated in proportion to total pavement area as described in Chapter 3, but in no sense are the models precise enough to predict combinations of distress mechanisms at a particular roadway location. Thus, although the EAROMAR system may estimate, for example, both cracking and surface deformations, one does not have the capability of investigating what proportions of these distress manifestations occur together within a localized area, and are therefore likely attributable to a single cause (e.g., base failure). The most one can say is that both cracking and deformations are indicated by the models, and each must be repaired by an appropriate maintenance activity. This line of thought argues for a well-defined correspondence between the several activities to be simulated and respective damage components.

A second reason for more direct damage-activity relationships is that a conceptual model requires a clear definition of what activities to assign

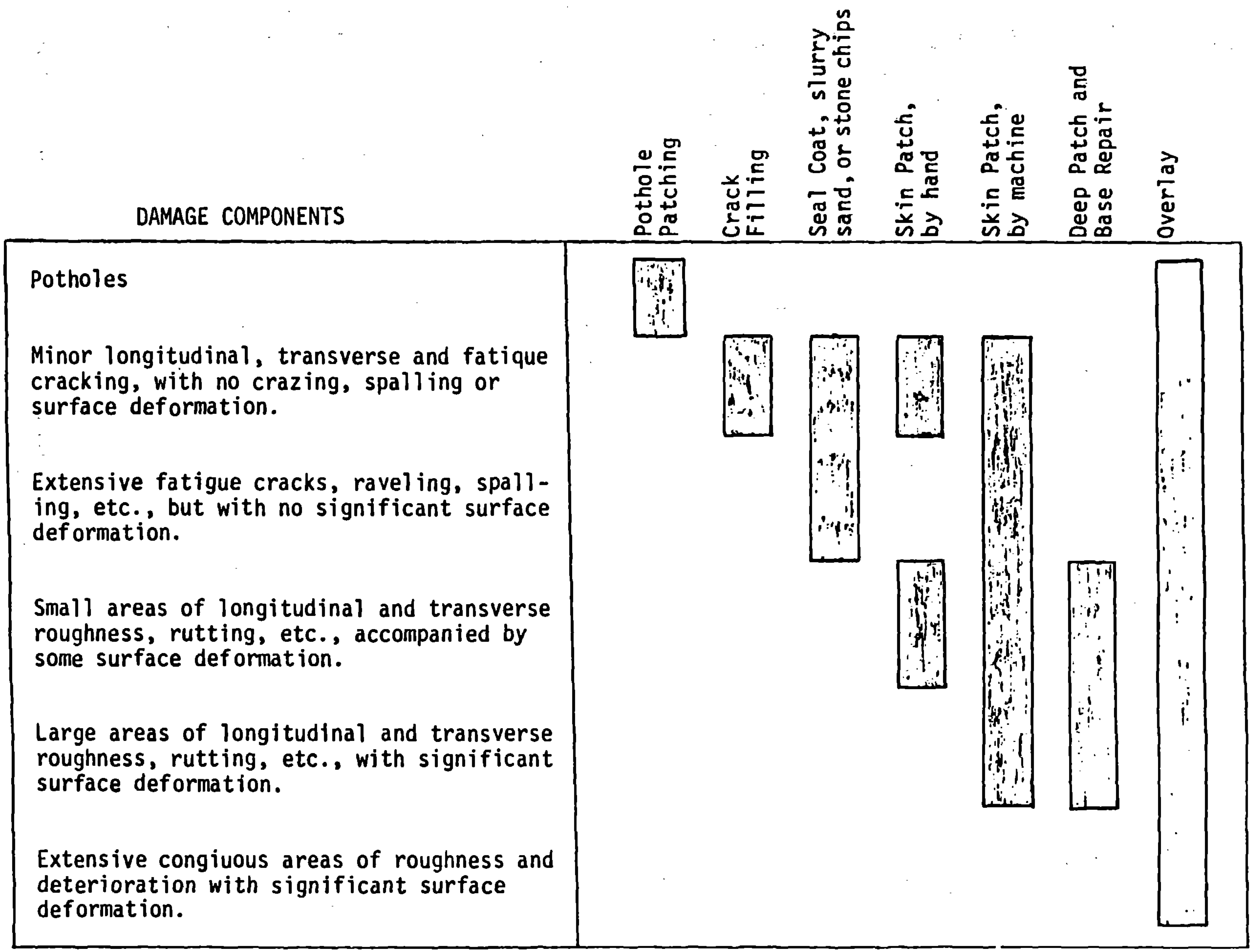

FIGURE 36. REMEDIAL EFFECTS OF VARIOUS MAINTENANCE ACTIVITIES

in repairing simulated damage or distress. Referring to our discussions above, we may target both crack filling and sealing to the repair of cracks, depending on their extent. If severe cracking occurred, we would then want to simulate the sealing activity, but we would also want to suppress crack filling, which would now be redundant. On the other hand, if the cracking were not extensive, we would want to model crack filling only, foregoing the sealing until the pavement condition warranted this more intensive operation. Note that specifications of combinations of crack filling and sealing would be difficult to model, since we are again dealing with average predictions of damage within a roadway section, and one cannot say what proportions of cracking would be suitable for sealing versus crack filling. These arguments therefore call for a more formal hierarchy among the several activities to be simulated.

Activity Precedence. The needs of both a hierarchy among flexible pavement activities and a clear-cut correspondence between activities and damage components in the EAROMAR analysis may be satisfied by the entries in Table 28 . Table 28 represents a formalization for modeling purposes of the ideas introduced in Figure 36 . The P's denote the primary damage component which the activity is intended to correct. The X's denote other components which will also be corrected within the roadway area in which primary work is performed. The order of activities listed denotes a hierarchy of corrective actions.

These activities are processed beginning with the more intensive actions and proceeding toward the least; i.e., from the bottom of the list upward. For a given damage component, any portion of that damage corrected by an intensive action will correspondingly reduce the level of effort required for less intensive operations. In evaluating the combined beneficial effects of several maintenance activities at a given location and time, it will be assumed, first, that quantities of damage components represent averages over each section; and second, that certain damage components may be associated with one another.

Treatment Within EAROMAR. Let us consider an example of how maintenance activities are simulated under this approach. Table 29 presents data for a one-mile roadway section exhibiting several modes of distress. The percentages at the right indicate the relative areas of the two-lane pavement surface subject to the respective damage components (again, average values over both lanes throughout the one-mile length). From Chapter 3, roughness is assumed to exist uniformly over the entire pavement surface. Rutting is assumed to occur within wheel tracks three feet (0.9m) wide (resulting in the estimated 7,000 sy for the two-lane, one-mile length). A base failure is taken as 12 ft. x 12 ft., or 16 sy (13.4 sm). Potholes are assumed to be 4 sf (0.35 sm) in area. The quantity of each damage component reparied will depend upon the particular maintenance strategy adopted.*

* For simplicity, we have shown cracking in an areal sense only. If lineal cracking is present also, it will be repaired to the same proportion that areal cracking is filled or sealed in each of the examples below.

TABLE 28

RELATIONSHIP OF ACTIVITIES TO
FLEXIBLE PAVEMENT DAMAGE COMPONENTS

	PAVEMENT DAMAGE COMPONENT						
Maintenance Activity	Lineal Cracking	Areal Cracking	Rutting or Transverse Roughness	Potholes	Longitudinal Roughness	Shoulder Distress	Base Failure
Crack Filling	P					X	
Seal Coat	X	P					
Pothole Repair	X	X		P			
Skin Patch	X	P	X		X		
Deep Patch and Base Repair	X	X		X	X		P
Overlay	X	X	X	X	P	X	

TABLE 29

EXAMPLE OF FLEXIBLE PAVEMENT MAINTENANCE SIMULATION

PAVEMENT DESCRIPTION { Flexible surface, 2-lane section one mile long; total surface area = 14,000 sq. yds. = 11,800 sm

Damage Component	Severity	Pavement Area Affected
Roughness	120 in./mile (1,900 mm/km)	100% or 14,000 sy (11,800 sm)
Rutting	0.75 in. (1.9 cm)	50% or 7,000 sy (5,900 sm)
Potholes	5 per lane-mile	∿ 0% or 4.4 sy (3.7 sm)
Cracking	12,600 sf/lane-mile (2,340 sm)	20% or 2,800 sy (2,340 sm)
Base Failure	3 per lane-mile	0.7% or 96 sy (80 sm)

FIGURE 37

EXAMPLES OF DIFFERENT MAINTENANCE OPTIONS TO REPAIR PAVEMENT IN TABLE 29

OPTION 1: Overlay 100% of pavement area

Damage Component	Severity and Area	Repaired Condition
Roughness*	120 in/mile over 14,000 sy (1,900 mm/km over 11,800 sm)	51 in/mile over 14,000 sy (810 mm/km over 11,800 sm)
Rutting	0.75 in over 7,000 sy (1.9 cm over 5,900 sm)	0 in over 7,000 sy (0 cm over 5,900 sm)
Potholes	5 per lane mile =40 sf = 4.4 sy = 3.7 sm	0
Cracking	12,600 sf/lane-mile = 2,800 sy = 2,340 sm	0
Base Failure	3 per lane mile	0

OPTION 2: Repair all base failures; place skin patches to correct 50% of cracking

Damage Component	Severity and Area	Repaired Condition
Roughness	120 in./mile over 14,000 sy (1,900 mm/km over 11,800 sm)	120 in./mile over 14,000 sy (1,900 mm/km over 11,800 sm)
Rutting	0.75 in. over 7,000 sy (1.9 cm over 5,900 sm)	same
Potholes	5 per lane-mile, = 40 sf or 4.4 sy = 3.7 sm	0 per lane-mile
Cracking	12,600 sf/lane-mile = 2,800 sy = 2,340 sm	12,150 sf/lane-mile = 2,700 sy = 2,260 sm
Base Failure*	3 per lane-mile	0 per lane-mile

(continued next page)

* Primary damage component for this activity

(Continued)
FIGURE 37 EXAMPLES OF DIFFERENT MAINTENANCE OPTIONS TO REPAIR PAVEMENT IN TABLE 29

Damage Component	Severity and Area	Repaired Condition
B. Effects of skin patching		
Roughness	120 in./mi. over 14,000 sy (1,900 mm over 11,800 sm)	113 in./mi. over 14,000 sy (1,795 mm/km)
Rutting	0.75 in. over 7,000 sy (1.9 cm over 5,900 sm)	0.61 in. over 7,000 sy (1.5 cm over 5,900 sm)
Potholes	0 per lane-mile	Same
Cracking*	12,150 sf/lane-mile = 2,700 sy = 2,260 sm	6,075 sf/lane-mile = 1,350 sy = 1,130 sm
Base Failure	0 per lane-mile	Same

OPTION 3: Repair all base failures and potholes; fill 75% of visible cracks

Damage Component	Severity and Area	Repaired Condition
A. Effects of base repair		
Roughness	120 in./mi. over 14,000 sy (1,900 mm/km over 11,800 sm)	120 in./mile over 14,000 sy (1,900 mm/km over 11,800 sm)
Rutting	0.75 in. over 7,000 sy (1.9 cm over 5,900 sm)	Same
Potholes	5 per lane-mile, = 40 sq = 4.4 sy = 3.7 sm	0 per lane-mile
Cracking	12,600 sf/lane-mile = 2,800 sy = 2,340 sm	12,150 sf/lane-mile = 2,700 sy = 2,260 sm
Base Failure*	3 per lane-mile	0 per lane-mile

(continued next page)

*Primary damage component for this activity

(Continued)
FIGURE 37 EXAMPLES OF DIFFERENT MAINTENANCE OPTIONS TO REPAIR PAVEMENT IN TABLE 29

Damage Component	Severity and Area	Repaired Condition
B. Effects of potholes repair		
Roughness	120 in/mile over 14,000 sy (1,900 mm/km over 11,800 sm)	Same
Rutting	0.75 in. over 7,000 sy (1.9 cm over 5,900 sm)	Same
Potholes*	0 per lane-mile	Same
Cracking	3,038 sf/lane-mile = 675 sy = 565 sm	Same
Base Failure	0 per lane-mile	
C. Effects of crack filling		
Roughness	120 in/mile over 14,000 sy (1,900 mm/km over 11,800 sm)	Same
Rutting	0.75 in. over 7,000 sy (1.9 cm over 5,900 sm)	Same
Potholes	0 sf/lane-mile	Same
Cracking*	12,150 sf/lane-mile = 2,700 sy = 2,260 sm	3,038 sf/lane-mile = 675 sy = 565 sm
Base Failure	0 per lane-mile	Same

*Primary damage component for this activity

There are a number of maintenance or rehabilitation options available, depending upon management decisions as to what activities should be undertaken, and the thresholds at which each activity will be performed. Translation of these policy issues into user-defined activity specifications within the EAROMAR system will be described later. For the time being, assume that the user wishes to test three alternative policy options, described qualitatively in Figure 37.

For each option, EAROMAR must evaluate the total range of activities specified, selecting first the most intensive action (i.e., the one farthest down on the activity list in Table 28). It will then identify the primary damage component for that activity (i.e., the component denoted by P in Table 28), retrieve the total quantity of damage present, and compute the portion of the primary damage to be corrected by that activity. The repair of the primary damage component will occur over a certain area of roadway; if any other damage modes occur within that area, and if they are also corrected by the activity (i.e., if they are denoted by X in Table 28), they will likewise be simulated as being repaired. The system will repeat the process for each succeeding activity (moving up the activity list in Table 28) until all specifications within the maintenance policy have been satisfied.

Consider Option 1 in Figure 37 , calling for an overlay over the entire roadway section. From Table 28 , an overlay is the most intensive maintenance or rehabilitation activity listed, in that it corrects all surface deficiencies. The extent of overlay is specified in the project description; usually it includes all roadway lanes. The net effect is that the entire roadway section will be overlaid, and all damage components will be repaired over their entire areas. This outcome is indicated under Option 1 in Figure 37. The roughness level of 51 in./mile (810 mm/km) is assumed to be the minimum achievable under current construction methods.

Option 2 is a more limited policy involving base repair and skin patching. Base repair is evaluated first in the activity hierarchy. Its primary damage component is base failure, of which 96 sy (80 sm) are now present in the example roadway section (3 base failure/lane x 2 lanes x 16 sy or 13.4 sm per failure.) The activity specifications under Option 2 call for all base failures to be repaired; therefore 96 sy (80 sm) of the pavement surface will be subjected to this activity. At the same time, deep patching will correct any localized roughness, potholes, or cracking; therefore, each of these distress modes will also be corrected to a maximum extent of 96 sy (80 sm). The damaged areas prior to the deep patch activity, and the respective amounts remaining following repair, are summarized under "A" in Option 2 in Figure 37.

The quantities shown in the "repaired condition" column under Option 2-A are computed as follows. First, the primary damage component is considered; since all base failures (totaling 96 sy or 80 sm) were repaired, there are no remaining base failures -- zero (0) per lane-mile is indicated. Potholes and cracking are assumed to have existed within the zones of base failure, and therefore to be repaired within the pavement area encompassed by deep patching. Of the 4.4 sy (3.7 sm) of potholes initially simulated, all are repaired, since 4.4 sy < 96 sy (3.7 xm < 80 sm). An analogous calculation holds for cracking; only now, not all cracking is repaired, since the area of cracking is greater than the pavement area repaired. The calculation of cracked area remaining is as follows:

$$1 - \frac{96 \text{ sy repaired}}{2800 \text{ sy existing}} \times 2,800 \text{ sy} = 2,700 \text{ sy}$$

$$1 - \frac{80 \text{ sm repaired}}{2340 \text{ sm existing}} \times 2340 \text{ sm} = 2260 \text{ sm}, \qquad (173)$$

The roughness measure is in inches per mile (mm/km); it is assumed to be corrected in proportion to the percent of total pavement area repaired by the activity. In this case, the ratio (96 sy repaired/14000 sy total pavement area; 80 sm repaired/11800 sm total pavement area) is negligible, and roughness remains at 120 in/mile (1900 mm/km).

The second activity under Option 2 is to patch the pavement to repair 50 percent of the cracking. The calculations are summarized in Option 2-B in Figure 37. For the primary distress mode of cracking, the area to be patched is 1350 sy (1130 sm) -- one-half of 2700 sy (2260 sm). Other damage modes affected by patching are rutting and roughness. Roughness shows improvement as a function of the area patched:

$$1 - \frac{1350 \text{ sy}}{14000 \text{ sy}} \times (120 - 51) + 51 = 113 \text{ in/mile}$$

$$1 - \frac{1130 \text{ sm}}{11800 \text{ sm}} \times (1900 - 810) + 810 = 1795 \text{ mm/km}, \qquad (174)$$

For rutting we assume that patching one-half the area of ruts is equivalent to reducing the mean rut depth by one-half. (This assumption follows from our statement earlier that we can consider only average conditions within a roadway section.) The improvement in rutting due to patching is calculated as follows:

$$1 - \frac{1350 \text{ sy}}{70000 \text{ sy}} \times 0.75 \text{ in} = 0.61 \text{ in}$$

$$1 - \frac{1130 \text{ sm}}{5900 \text{ sm}} \times 1.9 \text{ cm} = 1.5 \text{ cm}, \qquad (175)$$

The final road condition resulting from this maintenance policy option is shown in the rightmost column under Option 2-B. Not surprisingly, the pavement is improved to a lesser degree than that achieved by the overlay in Option 1. Base failures and cracking are substantially improved due to the particular activites performed; other manifestations of distress are only moderately or marginally corrected.

To give some further examples of this approach, Option 3 in Figure 37 denotes a maintenance policy comprising base repairs, patching of all potholes, and filling 75% of visible cracks (which could be interpreted as filling of all cracks greater than some minimum width). The incremental effects of each activity performed in sequence is given under A, B, and C respectively in Option 3. Note from the final road condition that this policy could be considered less intensive than Option 2: while base failures and potholes are completely repaired, the crack filling under Option 3 did not gain the benefits of the skin patching in Option 2, in terms of correcting cracking, ruts, and roughness.

Many other maintenance options could of course have been conceived in Figure 37. However, our purpose was not to demonstrate maintenance policy (the full range of maintenance policies in EAROMAR will be described in the section on Quality Standards); but rather, to demonstrate how the interactions among flexible pavement activities in Figure 36 can be accommodated within a conceptual model. Within the EAROMAR analysis we have formalized these interactions by first defining the specific damage modes for which each activity is relevant; and second, listing activities in the order in which they are to be processed. Table 28 defines both of these relationships. The implementation of this approach (as illustrated in Figure 37) simplifies reality to some degree, as in the adoption of average damage measures throughout roadway section length, and the associations among several damage modes (e.g., base failure, cracking, and potholes). Nevertheless, if these abstractions are borne in mind when defining maintenance policy, the simulation of flexible pavement maintenance can well represent realistic road situations.

RIGID PAVEMENTS

Rigid pavement activities to be modeled include crack filling, patching, joint filler replacement, slab replacement, and mudjacking, in addition to overlays.

Technical Review. Guidelines on the applications of each of these activities are as follows:

1. Crack filling is applicable where transverse, corner, longitudinal and diagonal cracking occur, but with no surface deformation present.

2. Patching corrects localized roughness or deterioration due to faulting or spalling.

3. Joint filler replacement may be required where existing filler has been stripped or no longer functions as an effective seal.

4. Slab replacement may be necessary where faulting and corner cracking cannot be remedied by surface patching or where blowups or D-cracking occur. In general, slab replacement is necessary when structural adequacy cannot be maintained by less expensive methods.

5. Mudjacking is undertaken to correct excessive faulting and pumping of the slab.

Table 30 summarizes these relationships between damage components and appropriate maintenance activities. Examples from three states of the standards governing the mudjacking activity are shown in Table 31.

Technology. The question of when certain activities are appropriate is usually a matter for field inspection and judgement, and often depends upon the practice of a particular state. The scale of technology that can be successfully employed is also dictated by the often localized nature of rigid pavement damage, occurring at slab boundaries or cracks.

For example, crack filling and joint seal replacement are activities which must be conducted using hand labor. Neither lend themselves to fully automated or large-scale operations such as surface sealing of extensive areas of pavement.

Patching of faulted and spalled slabs can be done either in small areas by hand or, where extensive areas of damage exist, by machine laying and finishing. Indications of blowups, D-cracking and pumping necessitate full-depth slab repairs (or mudjacking in the case of pumping) that are independent of other structural maintenance. Where extensive areas of surface roughness occur, and where spot-patches may have been placed previously, it may be appropriate to overlay the entire pavement. No universally accepted point at which an overlay is desirable appears to be available. However, considerations of rideability, indicated (for example) by a PSI of 2.5, have been used.

TABLE 30

COMPARISON OF DAMAGE AND APPROPRIATE MAINTENANCE ACTIVITIES RIGID PAVEMENT

TYPE OF DEFECT	MAINTENANCE ACTIVITIES DESCRIPTION	DAMAGE COMPONENT DESCRIPTION
Surface	Crack filling	Various forms of slab cracking but no significant deformation or settlement. Includes the following: Transverse cracking Corner cracking Longitudinal cracking Diagonal cracking
	Patching	Surface deterioration but with minimal deformation including: Faulting Spalling
	Joint replacement	Joint filler stripping
	Overlay	All of the above and 'D' cracking, below.
Full depth	Slab replacement	Blowups 'D' cracking Corner cracking
	Mudjacking (May also necessitate crack filling, patching and joint replacement at the same site)	Pumping Faulting

TABLE 31

COMPARISON BETWEEN STATE PERFORMANCE STANDARDS - MUDJACKING

ITEM	STATE: ILLINOIS	NEW YORK	CALIFORNIA
Activity N:	414	C-11	02-511
Activity name	Mudjacking	Lime jacking rigid pavement	Mudjacking
Description/ Damage Condtion/ Maintenance level	To fill cavities under the pavement and to lift areas of the pavement that have settled in order to eliminate dips and bring the pavement up to proper grade. To eliminate cavities and deter further erosion under culvert aprons, paved ditch and slope walls. Continued erosions may result in the complete loss of these facilities.	Lime jacking rigid pavement is the restoring of cement concrete pavement to the original line and grade where slabs have settled, by forcing a slurry of portland cement and ground limestone through holes in the slabs with a mud jack. It is to be done according to Quality Guideline, Sec. 1.350.	Roadbed deficiencies which immediately affect safety, riding quality, and capital investment, such as excessive bridge approach slab settlement and abrupt vertical variations should be given first priority in roadbed maintenance. When surface deviations exceed 1/2 inch between adjacent slabs. Corrections should be scheduled.
Workload Rate	4 - 6 Cu. Yd. daily	40 holes per 8-hr. day	3.45 Sq. Yd. per Manhour

Treatment Within EAROMAR. The treatment of rigid pavement activities within EAROMAR is analogous to that of flexible maintenance earlier. Table 32 formalizes the hierarchy of rigid pavement activities and the correspondence between activities and damage components repaired. The simulation of these activities in sequence proceeds in much the same way as described in the examples in Figure 37 for flexible pavements.

COMPOSITE PAVEMENTS

Maintenance activities for composite pavements include crack filling, patching, sealing, and pothole repairs, in addition to overlays. Their technical descriptions are similar to those given under flexible pavements earlier. The correspondence between activities and damage components and their order of simulation are given in Table 33.

Maintenance calculations for all pavements also require standardization of the units of repair for each activity. Work units incorporated within EAROMAR to measure repair quantities are listed in Table 34.

TABLE 32

RELATIONSHIP OF ACTIVITIES TO
RIGID PAVEMENT DAMAGE COMPONENTS

PAVEMENT DAMAGE COMPONENT

Maintenance Activity	Lineal Cracking	Areal Cracking	Longitudinal Roughness	Faulting	Joint Filler Stripping	Spalling	Blow-ups	Pumping	Shoulder Distress
Crack Filling	P								X
Patching		P	X	X		P			
Joint Replacement					P				
Mudjacking			X	P				P	
Slab Replacement		X				X	P		
Overlay	X	X	X	X		X			

TABLE 33

RELATIONSHIP OF ACTIVITIES TO
COMPOSITE PAVEMENT DAMAGE COMPONENTS

PAVEMENT DAMAGE COMPONENT

Maintenance Activity	Lineal Cracking	Areal Cracking	Rutting	Longitudinal Roughness	Potholes	Shoulder Distress
Crack Filling	P					X
Sealing	X	P				
Pothole Repair	X	X			P	
Patching	X	P	X	X		
Overlay	X	X	X	X	X	

PAVEMENT TYPE	ACTIVITY	WORK UNIT
FLEXIBLE	Crack Filling Patching Base Repair Sealing Pothole Repair	Lineal Feet Square Feet Number Square Feet Number
RIGID	Crack Filling Patching Joint Repair Slab Repair Mudjacking	Lineal Feet Square Feet Number Number Number
COMPOSITE	Crack Filling Patching Sealing Pothole Repair	Lineal Feet Square Feet Square Feet Number

TABLE 34

ACTIVITY WORK UNITS

Quality Standards

GENERAL CONCEPT

The examples in Figure 37 illustrate the effects of different maintenance policies on both the type and extent of remedial work required and the resulting condition of the pavement surface. Maintenance policies are expressed in EAROMAR through "quality standards" defining the thresholds at which work should be performed and the portion of accumulated damage to be repaired. Quality standards can be applied to control the timing and the intensity of pavement maintenance over the route length, rendering the maintenance function sensitive to both the rates of damage accumulation and the previous maintenance performed. As instruments of maintenance policy, quality standards are thus an integral part of the demand-responsive approach to maintenance.

To visualize the application of quality standards, consider the situation for one mode of pavement damage and one maintenance activity -- say, lineal cracking and crack filling, respectively -- in Figure 38 . The top curve, Figure 38 A, represents the accumulation of lineal cracking over time (as a function of cumulative traffic loadings) in the absence of any maintenance. Figure 38 B shows the effects of two alternative quality standards, Q_1 and Q_2, on the levels of cracking observed over time. It is assumed here that in each case all cracking exhibited is fully corrected. A more general situation, however, is that at any given time only a portion of the total cracking on the roadway is repaired through maintenance. Figure 38 C illustrates two different applications for a given threshold Q: one results in relatively frequent but minor repair R_1, while the other undertakes less frequent but major repair R_2. Note that neither R_1 nor R_2 are sufficient to fill all cracks on the roadway. Several comments are worthy of note.

First consider the effects of two alternative standards Q_1 and Q_2 in Figure 38 B: The different quality standards result (not unexpectedly) in two different trends in cracking observed over time. If we adopt a simple time average for illustration, the higher quality standard Q_1 results in a better (i.e., lower) average level of cracking C_1.

Second, the frequency of crack filling under Q_1 is greater than that under Q_2, in that $t_1 < t_2$. Influencing this result, however, is the fact that we have assumed all cracks to be repaired under both Q_1 and Q_2. The actual values of t will depend on the relative percentages of cracks actually filled under each standard (as will be discussed for Figure 38C below). Nevertheless, it is correct to say that for a given deterioration curve (Figure 38A), specifying the quality standard is equivalent to establishing some implicit frequency of maintenance.

Third, it is useful to define a quality standard using a unit of measure commensurate with that of the corresponding damage mode. In Chapter 3 predictions of lineal cracking were expressed in units of lf per lane mile. The threshold at which crack filling commences would

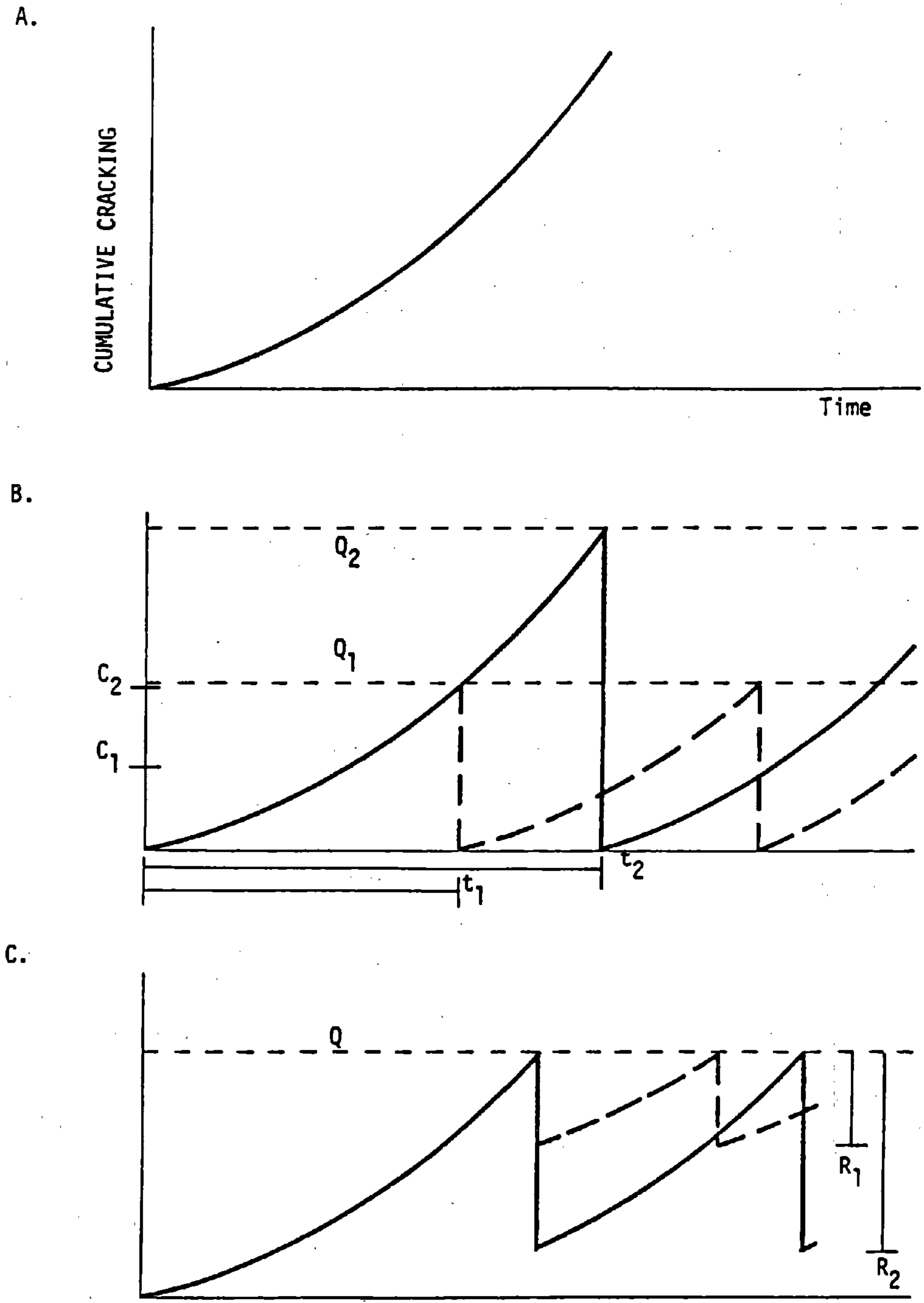

FIGURE 38. CONCEPTS OF QUALITY STANDARDS

therefore likely be some level of total lineal cracking observed, also in lf per lane mile or some index derived from this measure. This correspondence between the quality standard and the unit of pavement damage is shown clearly on the ordinate in Figure 38 B.

Finally, consider the mechanism available to control the extent of repair R under a given threshold Q, as postulated in Figure 38 C. The procedure is to restrict quality standards and associated maintenance actions to classes of damage most critical to road integrity and performance. For example, one could specify that only those cracks greater than a certain width be filled; the result is that less than 100% of all cracks would be repaired. Furthermore, one could employ this mechanism among several activities to simulate policies of deferred maintenance, or of options between hand- vs. machine-operations described earlier. Filling of cracks, say, could be confined to a relatively small operation, with more extensive repairs accomplished during pavement sealing. Qualifications of this type intentionally limit the extent of repair R in comparison to the total damage accumulated in the pavement.

The relationships in Figure 38 thus define an approach to predict maintenance work requirements as a function of both the condition of the pavement and designated maintenance policies. Moreover, since the damage function in Figure 38 accounts for the quality of both initial construction and of subsequent maintenance, rehabilitation and reconstruction, broad maintenance-investment tradeoffs can also be explicitly evaluated. The demand-responsive approach pictured in Figure 38 is therefore fundamental to addressing the types of strategic decisions inherent in establishing premium pavement warrants.

IMPLEMENTATION WITHIN EAROMAR

The implementation of maintenance quality standards within EAROMAR is accomplished by a series of statements constructed as shown in Figure 39 . Statements of this type may be defined by the user for each combination of maintenance activity and its corresponding primary damage component in Tables 28 , 32 , or 33 . These statements, in turn, may be aggregated among all relevant activities into what are termed maintenance policies. More will be said about policies shortly; first, let us review in detail the elements of the quality standard presented in Figure 39.

The first two elements define the pavement type and maintenance activity for which the standard is applicable. "Pavement type" is required for two reasons. First, it is conceivable that the route under study exhibits different pavement surfaces, either over its length or among different roadways. Second, in such a case it may be desirable to specify two different standards for (essentially) the same activity on the two different surfaces. Inclusion of "pavement types" with the "activity" identification makes it possible, for instance, to define separate "flexible pavement crack filling" and "rigid pavement

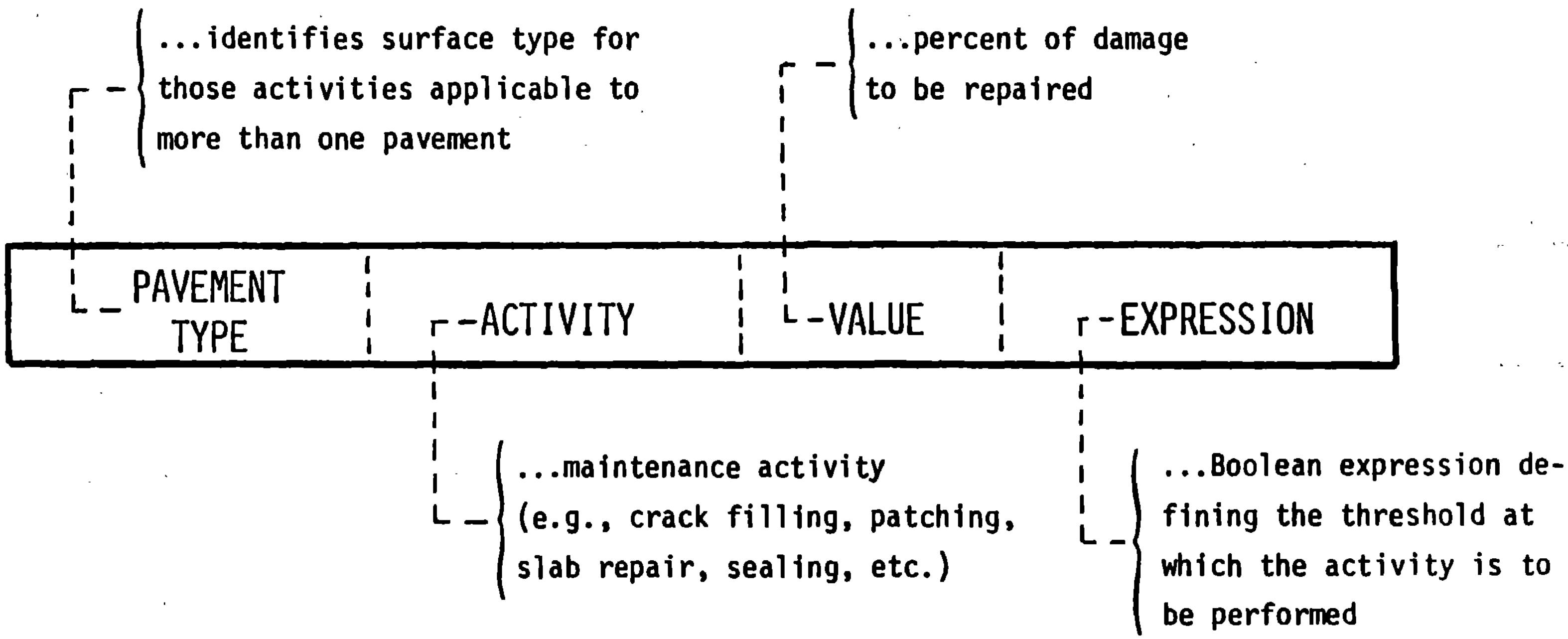

FIGURE 39

TYPICAL CONSTRUCTION OF A QUALITY STANDARD crack filling" standards within the same maintenance policy.

The "percent of damage repaired" is the numerical representation of the magnitude of repair R in Figure 38. This value is interpreted as the percentage of total (primary) damage to be corrected by that maintenance activity. As discussed in relation to both Figure 38 and Figure 37 earlier, from a value of less than one hundred per cent one would infer that only the more severe manifestations of damage are to be remedied (e.g., only cracks greater than a certain width are to be filled, or cracking greater than a certain area sealed). However, it is important to note that since the EAROMAR simulation treats only the mean damage components over a roadway section, this implication is not addressed explicitly within the analysis.

The final element in Figure 39 is an expression that corresponds to the quality standard threshold Q in Figure 38. The point at which a manager feels it prudent to undertake a particular maintenance activity is not always simple to define. It may depend, for instance, on the accumulation of some minimum amount of damage, or on a pavement index falling below some acceptable value. On the other hand, for reasons of scheduling and logistical efficiency, a manager may wish to specify directly the frequency of maintenance performance, based on his knowledge of the rate of damage accumulation. Furthermore, a manager may at the same time wish to control the timing of maintenance for reasons of work quality.* For instance, he may specify that crack filling be done in fall and early winter when the cracks have opened in width.

All of these factors can be incorporated precisely and succinctly within logical statements called <u>Boolean expressions</u>. A Boolean expression is a statement that can be evaluated to a TRUE or FALSE result. A Boolean expression may be simple, containing only one component yielding TRUE or FALSE; or, it may be complex, containing multiple components each of which can be evaluated as TRUE or FALSE. In the latter case, the components of the statement are connected to each other using one of three logical operators: NOT, AND, and OR (evaluated in that order). The expression components yielding TRUE or FALSE may be of two types: (1) a comparison of a keyword with some constant, and (2) a predicate keyword followed by an argument list.

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* Maintenance may also be scheduled for reasons of resource availability and minimization of traffic interference, as will be described in section 4.3 Within EAROMAR, both of these scheduling capabilities are provided as separate and independent controls for use by the manager.

Logical Comparisons. A logical comparison takes the following form:

$$keyword \quad operator \quad constant \tag{176}$$

where: *keyword* is one of a set of reserved words within the EAROMAR system, identifying:

(1) individual damage components,
(2) pavement PSI,
(3) the time since this activity was last performed, or
(4) the current roadway traffic volume in AADT.

operator is one of a set of six comparison operators:

(1) EQ - equal to
(2) NE - not equal to
(3) GT - greater than
(4) LT - less than
(5) GE - greater than or equal to
(6) LE - less than or equal to

and *constant* is a constant value specified by the user, consistent in meaning and units of measure with the *keyword*.

The complete set of comparison keywords is listed in Table 35 . Let us consider some examples of their use.

The unit of measure of lineal cracking is lf per lane-mile. Suppose a manager wished to commence crack filling when 5000 lf/lane-mile appeared in a pavement. Within his quality standard for the crack filling activity, then, he would include the following expression:

LCRACKS GE 5000 (177)

So long as average cracking in the pavement remained below 5000 lf/lane-mile, the Boolean expression (177) would be evaluated as FALSE, and the crack filling activity would not be executed. However, as soon as lineal cracking exceeded the 5000 lf/lane-mile threshold specified by the user, crack filling would be simulated, subject to the scheduling constraints in section 4.3.

As an alternative, suppose the manager now wished to set a policy for crack filling not as a function of the actual amount of damage in the pavement, but rather on a periodic basis -- say, every two years. The governing expression would then be as follows:

TABLE 35

LIST OF COMPARISON KEYWORDS

AADT
INTERVAL
PSI
LCRACKS
ACRACKS
RUTS
POTHOLES
ROUGHNESS
FAULTS
JOINTS
SPALLS
BLOWUPS
PUMPING
SHOULDER
BASE-FAILURE

INTERVAL GE 2 (178)

The "GE" operator is used here instead of the "EQ" operator denoting strict equality, since scheduling constraints (section 4.3) or (more generally) other components of a complex Boolean expression may preclude work from being performed exactly two years after the previous accomplishment of this activity. Use of the less restrictive "GE" comparison allows the manager's intent (to perform crack filling at two-year intervals) to be fulfilled as soon as possible, but is flexible enough to allow for any potential delays due to scheduling constraints or other influences.

In moving to complex expressions we must first define how the logical operators NOT, AND, and OR are interpreted. NOT changes the logical meaning of the expression it precedes. If [Expression] = TRUE, then NOT [Expression] = FALSE and vice versa. AND is a conjunction joining two expressions; if both expressions are TRUE, the result is TRUE. Otherwise, the result is FALSE. OR is also a conjunction joining two expressions; however, for the result to be TRUE, at least one of the expressions must be TRUE. If both expressions are FALSE, the result is FALSE. These rules are summarized in more concise notation in Figure 40.

Note that the regular order of priority among these operators is NOT, AND, then OR. That is, all factors modified by NOT will be evaluated first; then all expressions joined by AND; finally, the remaining expressions joined by OR. This order of precedence may, however, be modified by parentheses, as illustrated by the fifth example for AND and OR respectively.

Consider the following complex expressions:

(LCRACKS GE 5000 OR PSI LT 3.5) AND INTERVAL GE 2 (179)

NOT(LCRACKS LT 5000 AND PSI GE 3.5) AND INTERVAL GE 2 (180)

In both expressions parentheses are used to modify the order in which the logical operators are evaluated. In (179), the parentheses cause the two expressions joined by OR to be evaluated first; the result is then combined with that of the INTERVAL term, according to the rules of AND. The meaning of this expression (if assigned to a crack filling activity) is that crack filling will be performed if either the extent of cracking exceeds 5000 lf/lane-mile or the PSI falls below 3.5, but in no case more frequently than every two years. (Note the change in meaning if the parentheses are removed.)

Expression (180) is logically equivalent to (179). Within the parentheses the sense of the comparisons have been reversed, and OR replaced with AND. The parentheses cause the expressions joined by AND to be evaluated first. The result is then modified by NOT, and that result then evaluated together with the INTERVAL term. Although

FIGURE 40

OPERATIONS INVOLVING
LOGICAL OPERATORS
<u>NOT</u>, <u>AND</u> and <u>OR</u>

Given:	[Expression One] , [E1]	=	TRUE	(181)
And:	[Expression Two] , [E2]	=	FALSE	(182)
Then:	[E1] AND [E2]	=	FALSE	(183)
	[E1] AND NOT [E2]	=	TRUE	(184)
	NOT [E1] AND [E2]	=	FALSE	(185)
	NOT [E1] AND NOT [E2]	=	FALSE	(186)
	NOT [E1 AND E2]	=	TRUE	(187)
Also:	[E1] OR [E2]	=	TRUE	(188)
	[E1] OR NOT [E2]	=	TRUE	(189)
	NOT [E1] OR [E2]	=	FALSE	(190)
	NOT [E1] OR NOT [E2]	=	TRUE	(191)
	NOT [E1 OR E2]	=	FALSE	(192)

(180) is a less intuitive rendition of (179), logically the two expressions say the same thing.

Predicate With Argument List. The predicate with argument list takes the following form:

predicate (*list*) (193)

where *predicate* = one of two reserved words within the EAROMAR system, identifying:

(1) a year within the analysis period
(2) a season within the analysis period

and *list* = a list of one or more values specified by the user, consistent in meaning with the *predicate*.

This convention for the Boolean expression is used within EAROMAR to identify specific seasons or years in which work should be performed. Thus, for the example

YEAR (1980 1983 1989 1992 2000) (194)

this expression would be evaluated as TRUE in the respective years listed, and maintenance would be simulated (again subject to scheduling constraints) only in those years shown. In all other years, (194) would result in a FALSE evaluation. Of course, the years listed under the YEAR keyword must fall within the analysis period defined in Chapter 2 if the expression is to be TRUE.

A similar approach applies to seasonal policy specifications. For instance, the expression

SEASON (FALL SPRING) (195)

would generate a requirement for maintenance in the fall and spring of every year (unless modified by a YEAR expression), but at no other time during the annual simulation. Again, the seasons listed must fall within the set defined under general route characteristics in Chapter 2 if the expression is to be TRUE.

Both season and year may be governed by a complex statement such as the following:

SEASON (FALL SPRING) AND YEAR (1980 1983 1989)

(196)

Also, where many contiguous years or seasons are to be referenced, one may use a built-in "TO" convention, as in these expressions:

YEAR (1990 TO 1995) (197)

SEASON (SPRING TO FALL) (198)

Expression (197) assigns a TRUE value to each of the six years 1990, 1991, 1992, 1993, 1994 and 1995, while (198) results in a TRUE value during spring, summer, and fall (assuming this was the sequence of seasons defined in the initial route characteristics).

Predicates with argument lists may be used in conjunction with logical comparisons, with all expressions joined by AND or OR or modified by NOT as described earlier. For example, a reasonable expression would be:

(PSI LT 3.0 OR INTERVAL GE 5)
AND NOT YEAR (1995 TO 2000)
(199)

If this expression applied to the crack filling activity, work would commence when the PSI fell below 3.0 or when at least five years had passed since cracks were last filled. In no case, however, would cracks be filled from year 1995 to 2000. (Perhaps an overlay is scheduled in year 2000, and maintenance work becomes uneconomical as that time approaches).

Default Value for Expressions. The Boolean expressions are useful for controlling the threshold at which work commences under a maintenance activity. Where no expression is entered in the quality standard, the implication is that the activity is always performed if any primary damage is present.* Within the simulation the maintenance activity will thus respond to existing primary damage in all seasons of all years, so long as the governing policy remains in effect (see next section). An equivalent procedure is to enter the keyword ALWAYS in lieu of a Boolean expression; this keyword has a perpetual value of TRUE and would therefore be processed in the same way as the default expression value.

ASSEMBLING STANDARDS WITHIN POLICIES

The preceding sections have described the mechanisms of constructing quality standards for individual maintenance activities within EAROMAR. When taken collectively, however, quality standards among

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* If no work at all is to be performed under a given maintenance activity, the activity should be excluded from the definition of the maintenance policy. Refer to the next section.

all activities must embody an integrated approach to the role of maintenance in fulfilling pavement serviceability requirements. To consider the objectives of EAROMAR specifically, the evaluation of premium pavement investments against competing maintenance alternatives requires a systematic treatment of the total pavement maintenance effort.

For these reasons, quality standards are organized within what are termed "maintenance policies" within EAROMAR. Each policy consists of a complete set of standards, one standard for each combination of pavement surface type and maintenance activity relevant to the study. Only those activities included within an active policy will be simulated within EAROMAR; activities excluded from policies are not executed.

An example of a policy specification encompassing three pavement types is shown in Figure 41. No special constraints govern the organization of a policy. However, from our discussions above it is recommended that each policy represent some readily identifiable characteristic or approach distinguishing it from alternative policies. For example, separate policies can be defined to represent, respectively, a high level and a low level of maintenance. Or, a given policy may be based on a preventive maintenance approach, while another may be oriented more toward deferred maintenance concepts.

Each policy is denoted by an identifier which enables it to be referenced in the specification of strategies described in Chapter 2. Thus, over the length of the route different policies may be active at the same time. Also, at a given route location policies may be varied over time. This dynamic character of the maintenance policies, coupled with the flexibility afforded the user in defining component quality standards, provides a versatile management instrument to replicate existing situations in various states and to test new maintenance strategies.

POLICY DECISION-1

FLEXIBLE CRACK-FILLING 50 LCRACKS GT 500

RIGID CRACK-FILLING 100 LCRACKS GT 1000

FLEXIBLE PATCHING 100 INTERVAL GE 2 AND RUTS GT 0.5

RIGID PATCHING 100 INTERVAL GE 3

COMPOSITE PATCHING 100 INTERVAL GE 4

BASE-REPAIR 100 ALWAYS

FLEXIBLE SEALING 100 ACRACKS GT 10000 OR AADT GT 40000

JOINT-REPAIR 100 INTERVAL GE 5

SLAB-REPAIR 100 BLOWUPS GE 1

MUDJACKING 75 PUMPING GE 10 AND INTERVAL GE 2

COMPOSITE POTHOLE-FILLING 100 POTHOLES GE 1

FLEXIBLE POTHOLE-FILLING 100 POTHOLES GE 1

END POLICY

FIGURE 41

EXAMPLE OF A POLICY SPECIFICATION ENCOMPASSING THREE PAVEMENT TYPES

4.3 MAINTENANCE SCHEDULING AND RESOURCE CONSTRAINTS

Background

The prediction of maintenance requirements as a function of pavement condition and maintenance policy is a demand-side relationship determining what work ought to be done. To fulfill these work requirements, however, introduces a new set of management issues associated with maintenance supply and the question of what work can be done. Supply-side relationships consider the maintenance technology proposed, associated resource requirements, and scheduling of periods of road occupancy.

The implications of the supply-side analysis fall in two broad areas. One is the consumption of scarce maintenance resources and generation of costs. Maintenance cost considerations are particularly important in a premium pavement analysis, where work requirements can be substantial (owing to heavy use of the route in question) and where repairs must often be accomplished rapidly and during premium time to avoid severe congestion. The second is the interference with the traffic stream occasioned by the workzone itself -- another major planning consideration on heavily trafficked routes. The degree of inconvenience and increased costs borne by motorists will depend on the severity of the roadway closure for the maintenance workzone and the hours of occupancy.

In this section we develop the fundamental relationships for maintenance supply, looking at maintenance technology and resource consumption, work scheduling, and physical characteristics of road closures. In section 4.4 we will relate technology and scheduling to maintenance costs. Interactions between the workzone and the traffic stream will be analyzed under the treatment of roadway capacity, congestion and queueing in Chapter 5.

Maintenance Technology

A maintenance technology determines what quantities of what resources must be combined to complete an activity. However, just as maintenance standards were seen earlier to vary from state to state (e.g., Tables 24 through 26), so too do maintenance technologies. Table 36, for instance, reports various mixes of labor, equipment and materials from several states for hand-patching with premix. In developing the EAROMAR system, we therefore felt that maintenance technologies could not be assumed *a priori* within the model relationships, but rather must be specified by a user, based upon local standards and practices.

The information needed to describe maintenance technologies is very similar to that reported in Table 36 . Its definition and treatment within the EAROMAR analysis is explained in the sections below.

STATE and ACTIVITY	LABOR	EQUIPMENT	MATERIALS
ALABAMA (Spot premix patching, hand operation)	2 truck drivers 1 laborer 2 flagmen	1 flat truck 1 dump truck 1 hot pot 1 portable roller	Premixed bituminous material Liquid asphalt
DELAWARE (Premix patching, hot mix; paved travel way)	1 foreman 2 equipment operators 1 laborer 1 equipment operator 1 equipment operator 1 laborer	1 pickup 2 dumptrucks 1 compressor 1 roller (steel) 1 vibratory compactor 1 cement saw (if required)	Bituminous concrete Tack Select borrow (if required)
IOWA (Spall patching; roadway surface)	2 men	1 dumptruck 1 premix heater 1 tack tank	Bituminous blade mix, Commercial premix, Liquid tack

TABLE 36

(continued next page)

VARIOUS MIXES OF
LABOR, EQUIPMENT & MATERIALS FROM SEVERAL STATES
FOR HAND-PATCHING WITH PRE-MIX

(continued from previous page)

STATE AND ACTIVITY	LABOR	EQUIPMENT	MATERIALS
MINNESOTA (Skin patch)	1 section truck driver-raker 1 tack & raker 1 pick-up driver and section truck driver-shoveler 1 sweeper-shoveler and raker	1 2½ ton dual-section truck (haul hot mix) 1 2½ ton dual-section truck 1 tack tank 1 6-pack pickup	Tack and hot plant mix Bituminous surface material (fine mix)
VIRGINIA (Premix patching)	1 foreman 4 operators 2 maintenance help	2 2½ ton trucks 1 roller 1 kettle	13 tons premix asphalt 65 gallons RC2 Asphalt

TABLE 36

VARIOUS MIXES OF
LABOR, EQUIPMENT & MATERIALS FROM SEVERAL STATES
FOR HAND-PATCHING WITH PRE-MIX

RESOURCE CLASS

Maintenance resources are categorized within EAROMAR as either labor, equipment, or materials. As seen from Table 36, however, many different kinds of these factors of production are used even within a single activity, and their combination varies from state to state. For these reasons, labor, equipment, and materials are disaggregated within EAROMAR. Each kind of resource, or each individual factor of production, is termed a resource class.

The definition of resource classes is at the freedom of the user, with no restrictions as to the type or number of classes. For example, labor classes could comprise one item (e.g., maintenance workers) or several, as indicated in Table 36 (e.g., foremen, drivers, operators, laborers). Equipment classes could include, for instance, dumptrucks, pickup trucks, trailers, signboards, asphalt kettles, pavers, and so forth. Materials could encompass any item put in place or consumed during the maintenance process, such as liquid asphalt, portland cement or bituminous concrete, aggregate, grout, and the like. The only requirement (and a logical one) is that the list of resouce classes encompass <u>all</u> factors to be referenced subsequently in descriptions of technologies by activity. This is not a restrictive requirement, since the user himself defines the activity technologies as well. What is implied here, however, is that the user must make conscious decisions on the level of detail to be represented in the technological descriptions, and then provide consistent information throughout the input process.

Associated with each resource class are factor prices that will be used in the computation of maintenance costs described in section 4.4. These unit costs should include all cost elements of work in place to be attributed to the maintenance function: direct costs, as well as indirect costs associated with work performance. How these costs are determined is again at the discretion of the user.

For example, if indirect labor costs for on-site foremen are charged against the maintenance function within the user's organization, then the costs of such supervision should be included in the EAROMAR description. This can be done in different ways. A labor class for supervisory personnel can be explicitly declared, and a wage rate attributed directly to it. Another option is not to isolate supervisory personnel as a separate labor class, but rather to apportion their expenses as an indirect cost burden on wages of direct cost workers (resulting in a "loaded" wage rate). Similar arguments hold for costs associated with payroll benefits, equipment depreciation, and materials storage, among other such items.

<u>Wage Rates</u>. Individual wage schedules are assigned for each labor class. To allow for the possibility of maintenance work outside of normal working hours, wage schedules include adjustment factors for time of day worked and for separate weekday vs. weekend

rates. The specification of wage rates is as follows.

Each labor class is assigned a base wage rate: the normal rate which that class receives during "straight time" work. The user may then specify optional wage factors, applicable over some period during either a weekday or weekend. The wage rate simulated for this labor class will then be the product of the base rate and the adjustment factor, or:

$$\mathrm{Wage}_L^{dh} = \mathrm{Wage}_L^{o} \cdot \mathrm{Factor}_{dh} \qquad (200)$$

when Wage_L^{dh} is the hourly wage at which labor class L will be simulated

Wage_L^{o} is the base hourly wage for labor class L

and Factor_{dh} is a multiplicative factor, a function of the type of day d (weekday or weekend) and hour of day h

Table 37 illustrates the declaration of labor classes and assignment of wage schedules. From the hours of 9:00 a.m. through 5:00 p.m. in this example, all workers will receive their base rate: $10.00/hour for foremen, $8.00/hour for drivers, and $5.00/hour for workers. During other hours on weekdays all labor classes accrue time-and-one-half wages. On weekends all labor classes are paid double-time. Table 37 is merely an example of the types of wage distinctions that can be made. The user is free to specify any time distribution and wage adjustment factors that model correctly his local situation.

Equipment and Materials Unit Costs. The unit costs of equipment and materials are assumed to be constant by hour and day. Unit operating costs in dollars per hour (including depreciation if appropriate) are provided by the user for each equipment class listed. Unit materials costs in dollars per unit quantity are provided for each materials class.* Table 38 gives some examples of equipment class descriptions, while Table 39 illustrates the declaration of materials classes and costs.

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* The unit of quantity used for materials is arbitrary and at the control of the user. The only stipulation is that the user must remain consistent between the unit chosen as the basis of cost, and the unit specified in the maintenance technology in the following section. For instance, if aggregate is to be measured in tons, then both the unit costs and the rates of resource consumption in the maintenance technology must be based on tons. If the unit of quantity is cubic yards, both must be based on cubic yards. This stipulation applies to all materials classes.

LABOR CLASSIFICATION	BASE RATE	ADJUSTMENTS Period Day-Hour	Factor
Workers	$5.00	Weekday	
		2400 - 0800	1.5
		0800 - 1600	1.0
		1600 - 2400	1.5
		Weekend	
		2400 - 2400	2.0
Drivers	$8.00	Weekday	
		2400 - 0800	1.5
		0800 - 1600	1.0
		1600 - 2400	1.5
		Weekend	
		2400 - 2400	2.0
Foremen	$10.00	Weekday	
		2400 - 0800	1.5
		0800 - 1600	1.0
		1600 - 2400	1.5
		Weekend	
		2400 - 2400	2.0

TABLE 37

DECLARATION OF
LABOR CLASSES & ASSIGNMENT OF WAGE SCHEDULES

EQUIPMENT	COST, $/HR
Dumptrucks	$100
Kettle	10
Router	15
Compressor	15
Pickup	25
Loader	50
Roller	75
Chip-Spreader	100
Broom	35
Tamper	10
Backhoe	35
Distributor	100
Paver	100
Concrete-Saw	25
Water-Tank	10
Applicator	5
Mudjacker	100
Vibrator	5

TABLE 38

EXAMPLES OF
EQUIPMENT CLASS DESCRIPTIONS

MATERIALS	COST PER UNIT QTY.
LIQUID-ASPH	$50.00 / GAL
ASPH-EMULSION	50.00 / GAL
CHIPS	50.00 / TON
BIT-CONC	50.00 / TON
GRAVEL	1.00 / TON
PC-CONCRETE	25.00 / CY
SEALANT	50.00 / GAL
MESH	0.50 / SF
EPOXY-ADDITIVE	75.00 / GAL
CURING-COMPOUND	10.00 / GAL
DIRT	0.50 / CY
CEMENT	3.00 / SACK
WATER	0.0005 / GAL

TABLE 39

DECLARATION OF
MATERIALS CLASSES AND COSTS

RESOURCE CONSUMPTION AND WORK PRODUCTION

A maintenance technology is nothing more than a method or process by which work is accomplished. In describing a method for each activity within EAROMAR, we wish to focus on both the input and the output sides of this process. Inputs are measured by resources consumed, while outputs are characterized by rate of work production.

In developing descriptions of the methods, users may address several issues: for example, (1) choices between labor-intensive vs. machine-intensive methods discussed earlier in section 4.2; (2) labor/machine ratios dictated by machine operational requirements or governing labor work rules; (3) the implications of safety regulations in requiring additional flagmen, arrow boards, or other safety precautions; and (4) the effects of available labor skills, quality of supervision and reliability of equipment on work production. As with other data discussed in this chapter, the user has virtually complete freedom in specifying different methods for each maintenance activity. However, it is vitally important that the description represent a practical and consistent technological package. The number of laborers and their skills; the number and types of equipment; and types and quantities of materials -- all must be compatible with one another. Also, production rates must be realistic, given the resource mix proposed. These requirements must be met if estimates of maintenance schedules and costs are to be reasonable.

Resource Consumption. Inputs to the maintenance process are specified for each activity on the basis of a typical crew. Data required are as follows:

1. the number of persons required within each labor class;

2. the number of pieces required within each equipment class; and

3. the materials quantity per work unit required, for each materials class.

Within each materials class, the unit of measure for materials quantity must agree with the basis of unit cost. The work unit for each activity is fixed within the simulation logic as indicated in Tab 35. Note that, in agreement with the demand-responsive approach to maintenance prediction presented in sections 4.1 and 4.2, work units are based on elements of maintenance demand (i.e., measures of damage components) rather than maintenance supply (e.g., tons of concrete patching placed). This feature permits a manager to vary the maintenance technology independently of the activity work unit, and to simulate (if need be) novel or experimental maintenance technologies.

Work Production. Outputs of the maintenance process are specified for each activity as an average production rate, in work units accomplished per hour. This hourly production should be taken as that of a typical crew for the corresponding activity, and should reflect the particular types and quantities of resource classes developed for the method input description above.

Example. Table 40 illustrates descriptions of technology for two maintenance activities: flexible pavement sealing and rigid pavement slab repair. Any resource class may be included in these descriptions, as long as it has been previously declared to the system (e.g., in Tables 37 , 38, or 39). Note the use of the demand-side work unit in the materials consumption rates and the work production rate.

Maintenance Scheduling and Occupancy

The occupancy of a road for maintenance or rehabilitation raises a number of management issues regarding the timing and duration of work, and potential disruption of the traffic stream. Within EAROMAR these effects are simulated using two sets of information provided by the user: (1) maintenance schedules, and (2) descriptions of closure zones.

SCHEDULING

The scheduling capabilities within EAROMAR afford a user the opportunity to reflect supply-side considerations associated with, for example, the limited availability of maintenance crews, the consolidation of several activities within one workzone, or the desire to minimize interference with the normal flow of traffic. Whereas the demand-side relationships in section 4.2 determine how much work will be done annually, the scheduling data specify when this work will be accomplished during a simulated year.

Schedules are defined on a seasonal, daily, and hourly basis, consistent with the simulation of traffic operations (described in Chpater 5). Each schedule also identifies a group of activities to be accomplished during the same occupancy period. For example, if both crack filling and joint sealing (two separate activities within EAROMAR) are to be performed within the same work zone, these can be declared within the same activity group within EAROMAR, and will be scheduled together.

Table 41 illustrates the application of scheduling information within EAROMAR. The rationale for grouping activities here is the season in which they are normally performed; however, any other basis could just as well have been used. Different times of road occupancy have been specified for both weekdays and weekends among the various groups to illustrate the flexibility available to a manager. Again, any number of time periods, each covering any portion

ACTIVITY	LABOR	EQUIPMENT	MATERIALS Type	Quantity	PRODUCTION RATE
FLEXIBLE PAVEMENT SEALING	1 Foreman 4 Workers 10 Drivers	5 Dumptrucks 1 Loader 2 Rollers 1 Chip Spreader 1 Broom 2 Pickups	Asphalt Emulsion Chips	0.014 Gal/SF 0.001 Tons/SF	7920 SF/Hr.
RIGID PAVEMENT SLAB REPAIR	1 Foreman 6 Workers	3 Dumptrucks 1 Loader 1 Roller 1 Compressor 1 Concrete Saw 1 Vibrator	PC-Concrete Mesh Epoxy Additive Curing Compound	2.24 CY/Repair 0.8 SF/Repair 0.8 Gal/Repair 0.72 Gal/Repair	0.125 Repairs/Hr.

TABLE 40

DESCRIPTIONS OF TECHNOLOGY
FOR TWO MAINTENANCE ACTIVITIES

GROUP	ACTIVITIES	SEASON	WEEKDAY (24-hr. clock)	WEEKEND (24-hr. clock)	MOBILIZATION TIME (hours)
Activities Performed In Autumn Only	Flexible Crack Filling Flexible Sealing	Autumn	1000-1500	1000-1600	0.5
Cold Weather Activities Performed From Autumn through Spring	Flexible Pot-hole Filling	Autumn to Spring	0900-1200	---	0.5
Warm Weather Activities Performed From Spring through Autumn	Base Repair Joint Repair Slab Repair Flexible Patching Rigid Patching Mudjacking	Spring to Autumn	0900-1700	1000-1600	0.5

TABLE 41

EXAMPLES OF SCHEDULING INFORMATION WITHIN EAROMAR of a 24-hour day, may be defined.

Associated with each group of activities is a maintenance workzone, the characteristics of which will be explained in the next section. From a scheduling standpoint, however, the setting up and dismantling of this workzone may consume some time that would not then be available for productive work. This time, estimated and provided by the user, is shown as "mobilization time" in Table 41 . Mobilization time may in fact be used to represent any non-direct consumption of time by the crew, including travel time to and from the site, workzone mobilization and demobilization, crew breaks, and miscellaneous or lost time (if not already accounted for in the crew production rate described earlier).

In its simulation the EAROMAR system subtracts mobilization time from the daily time allotted in the schedule; the balance is then available for direct repair work. For example, for the first group of activities in Table 41 (flexible crack filling and sealing), the daily hours available are five on weekdays (from 10:00 a.m. to 3:00 p.m.), and six on weekends (from 10:00 a.m. to 4:00 p.m.). Mobilization is allowed as one-half hour. Therefore, a total of 4½ hours per day on weekdays and 5½ hours per day on weekends will be available for crack-filling and sealing.

The use of scheduling information to estimate the total duration of maintenance work and to arrive at maintenance costs will be explained in section 4.4.

CLOSURES

Types. Three types of closures for pavement repair are defined within EAROMAR:

1. Lane restrictions, in which one or more lanes on a given roadway are closed for repair work, with traffic constrained to the remaining lanes;

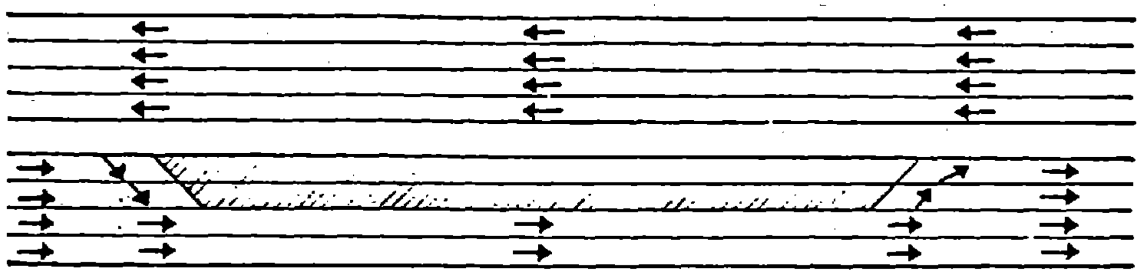
2. Crossovers, in which an entire roadway is closed for work, with traffic diverted to another roadway; and

3. Detours, in which an entire roadway is closed for work, with traffic diverted to a temporary bypass not part of the route in question.

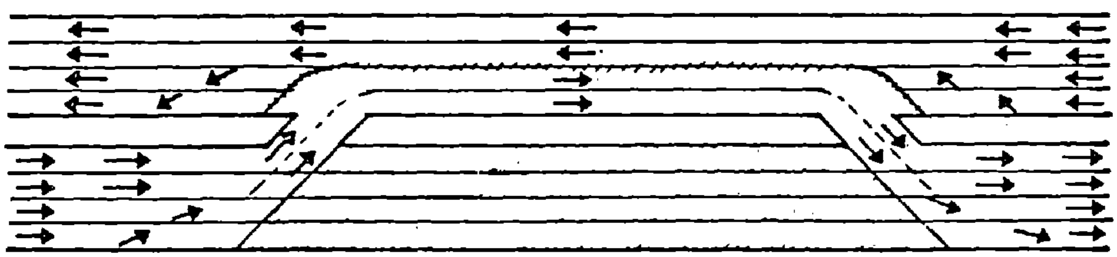
The types of closures are illustrated in Figure 42.

Closures affect traffic operations by introducing a temporary reduction in route capacity, the analytic treatment of which is described in Chapter 5. At this point, suffice it to say that the type of closure influences the impact of maintenance occupancy on traffic flow, and also must be compatible with the method of work

A. LANE REDUCTION OR RESTRICTION



B. CROSSOVER



C. DETOUR

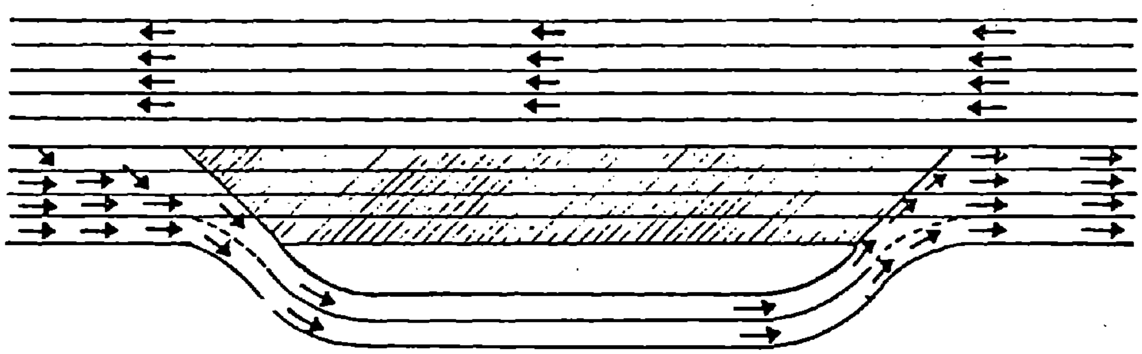


FIGURE 42. TYPES OF ROADWAY CLOSURES SIMULATED

performance. For example, where maintenance operations can proceed on a limited scale involving only one or two lanes at a time, a lane restriction may be the most appropriate type of closure. Only the traffic on the affected roadway is disrupted, and construction of temporary lanes is generally not called for.

Where the magnitude of operations or logistical considerations require that the entire roadway be closed, the choice involves the relative merits of a crossover vs. a detour for the case at hand. A crossover generates reduced capacity and ensuing congestion for two roadways instead of one, but restricts disruption to within the existing route right-of-way, and requires relatively little temporary construction. On the other hand, a detour involves the rerouting of traffic onto existing parallel thoroughfares outside the route right-of-way, or onto a temporary bypass constructed around the workzone. This option eliminates the roadway interactions present in crossovers, but its efficiency depends obviously on the relative capacities of alternate routes available or on the degree to which a manager is willing to invest in the construction of temporary roadways. The costs and effectiveness of different closure configurations can be evaluated as part of an EAROMAR analysis.

Extent. The longitudinal extent of a closure zone is specified by the user in terms of the length of workzone proper, and the length of taper leading into the workzone. These elements are illustrated for a lane restriction in Figure 43.

The accomplishment of maintenance work is assumed to take place within the confines of the workzone proper. However, in simulating traffic operations (Chapter 5), the reduction in capacity is over the total length of the workzone plus taper.

In Chapter 2, we mentioned that, for purposes of the EAROMAR simulation, a roadway is divided into sections of uniform geometric and structural characteristics. Figure 44 illustrates two possibilities regarding the relationship between the length of a roadway section and that of the occupancy zone (workzone plus taper) specified. If the section length exceeds the zone length, then a reduction in capacity is simulated over the zone length only (refer also to Chapter 5). For purposes of positioning the zone during the simulation, the occupancy is assumed to begin at the downstream (as traffic flows) end of the section, and to extend upstream for the required length, as shown in Figure 44 A. If, however, the occupancy zone length exceeds the section length, then the reduction in capacity is simulated over the entire section length, with the excess zone length (i.e., zone length minus section length) ignored. (Any maintenance work required in adjacent sections will be accounted for in the simulation of those sections individually.) This situation is illustrated in Figure 44 B.

Transience. In cases where affected lanes can be easily and

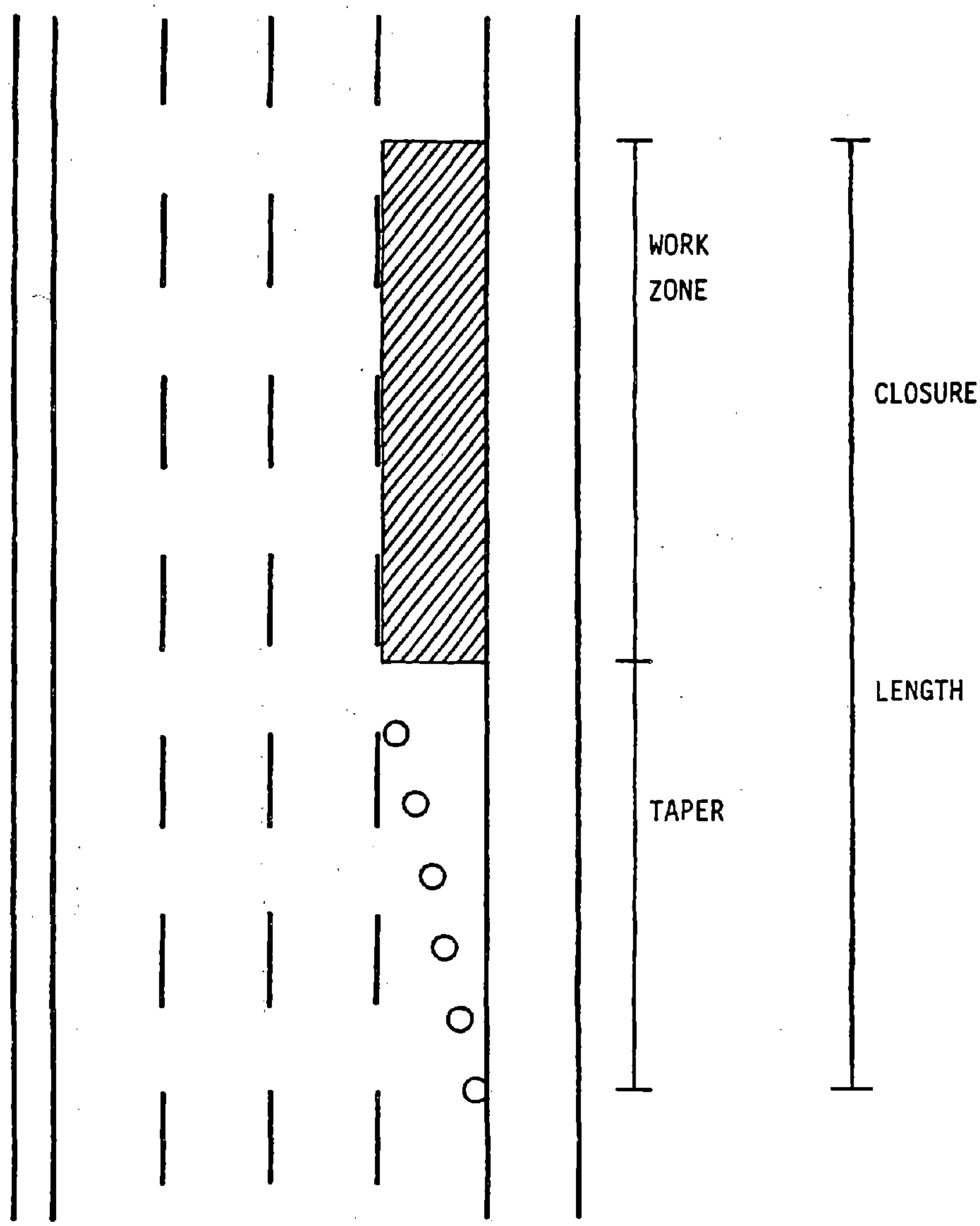
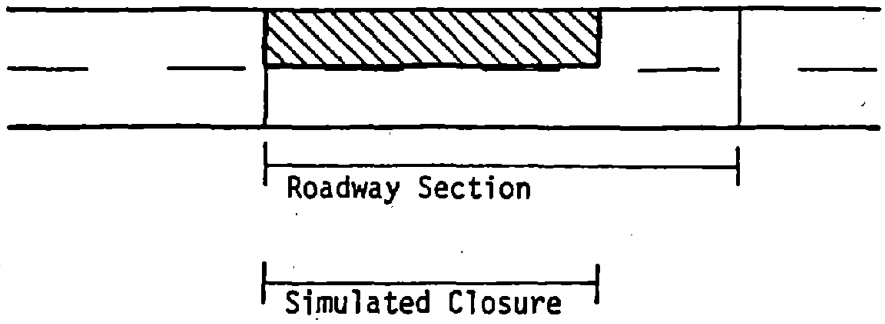


FIGURE 43. CLOSURE LENGTH DUE TO MAINTENANCE WORKZONE

A. CLOSURE LENGTH LESS THAN SECTION LENGTH



B. CLOSURE LENGTH GREATER THAN SECTION LENGTH

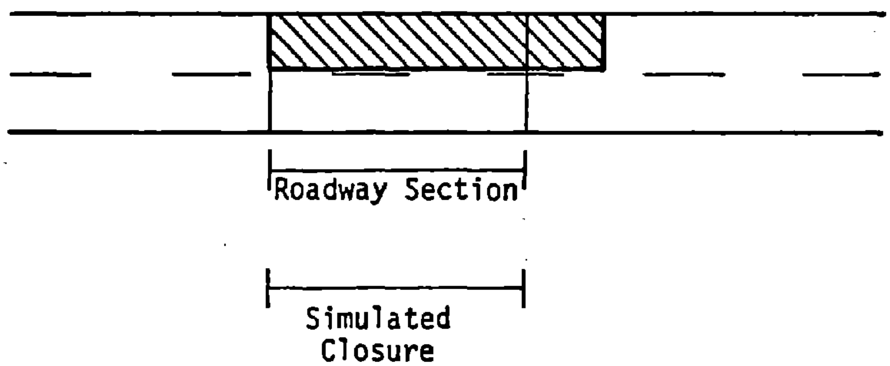


FIGURE 44. DETERMINATION OF CLOSURE LENGTH
TO BE SIMULATED

safely reopened to traffic after each day's maintenance operations, lane closures will be in force only during the hours scheduled for road work. Crack filling and routine patching are typical examples of activities that can be completed within a day, or at least resumed from day to day. In these situations, barriers which can be positioned and moved quickly and cheaply -- truck-mounted signboards, cones, flagmen -- will likely be used. The implication here is that neither the type of maintenance work being performed nor the extent of pavement area being repaired require a 24-hour-per-day closure of the affected lanes.

In other situations, generally involving major maintenance or rehabilitation activities such as substantial patching, slab repair, or overlays, it may be necessary to retain lane closures continuously in place over periods of several days or more, until work is completed. Usually more permanent barriers such as wooden barricades or precast concrete wall sections are employed. The implication here is that lane closures remain in effect during all hours of the day, regardless of whether maintenance crews are actually on site. The criterion for removing the lane closure is not the completion of a day's work, but rather the completion of the total repair.

The EAROMAR system provides the capability to simulate either type of closure at the command of the user. This choice does not affect the simulation of maintenance work itself, since this is determined solely by the interaction between demand-side considerations (quality standards and rates of deterioration discussed in section 4.2 and Chapter 3) and supply-side factors (maintenance scheduling discussed earlier). The transience or permanance of the workzone does, however, affect the impedance seen by roadway traffic throughout a 24-hour day.

Within EAROMAR, a workzone that reduces roadway capacity only during those hours in which maintenance work is scheduled is referred to as an "onsite" closure. The time and cost of mobilization is tallied for each work period simulated.* A workzone that remains on the roadway continuously from the start of repair until all work is completed, independent of scheduling considerations, is called a "total" closure. In the latter case the reduction in road capacity attributable to the workzone is simulated 24 hours per day until all maintenance within the workzone is completed. Mobilization time and costs are tallied once for each workzone.

Resource Constraints

Suppose a manager adjusted maintenance policies to increase the amount of work called for under the quality standards. If the increase

* If maintenance is scheduled in two or more separate time periods per day, mobilization time and cost will be computed for each such period.

were large enough, the resulting work requirements would exceed the maintenance dollars available for that route in that year. Put another way, the manager would lack the necessary labor, equipment or material resources to fulfill all the work he had intended to do.

Resource constraints are an important consideration within EAROMAR because they act as a supply-side check on the demand-side work projections generated through the quality standards. They inform a manager whether his specified policy (no matter how desirable) is realistic given the scarcity of maintenance resources and his assumed commitments to other routes of the road netword under his jurisdiction.

Resource constraints are typically discussed among maintenance managers in terms of budget dollar limitations. Within EAROMAR the alternative is used: i.e., individual constraints on the total labor time (by class), total equipment time (by class of equipment) and total material quantity (by class of material) for the route under analysis. Two reasons underlie this approach. First, EAROMAR represents an economic, as opposed to a financial, analysis of pavement strategies. Whereas imposition of budget limitations would require comparisons of current or nominal dollars in financial terms, EAROMAR computes costs in constant dollars for discounted economic comparison. Second, it is conceivable that only one type or class of resource may be exceeded in a particular policy. For example, a policy may require more labor time than allocated to this route, but all other resource projections may remain adequate. The approach embodied within EAROMAR makes it possible to discern these individual overruns of resource availability (which could not be detected under a simple budget constraint), and to respond with appropriate management actions.

If total annual resource availability is exceeded for any resource class within the EAROMAR simulation, a warning message is printed to the user. No other corrective action is taken, and the maitenance work is simulated as if no constraint existed. Although other courses of action could have been adoped (e.g., attempting to contrain the amount of work actually simulated), in fact these other possibilities would not be entirely consistent with the demand-responsive approach outlined in section 4.2, and would be difficult to implement in practice.

Under the demand-responsive approach maintenance requirements are governed by explicit policy decisions (represented by the quality standards), and not by resource constraints or any other factor. This is not to say that resource constraints should be ignored, for they are a very real and important aspect of maintenance planning. What we are saying, however, is that the effects of resource constraints must be explicitly considered but as a part of the formulation of maintenance policy, either by adjusting quality standards to meet the resource constraints, or by enacting management decisions to eliminate the constraints.

Where resource constraints exist several options are open to managers, and it would be impossible to anticipate within the EAROMAR system how a manager might respond to a specific situation. One option, mentioned above, is to adjust quality standards to see what (presumably) lower level of maintenance must be tolerated to meet the resource constraints. Another is to increase resources available to the route under question through overtime or reallocation of resources from other routes in the system. (In the latter case the deleterious effects of any such transfers on the other routes in the system would have to be accounted for.) A third is to reflect the increase in resources available to the route that would be obtained by additional temporary or permanent hires or by going out to contract. A fourth is to investigate a more efficient way of accomplishing maintenance, as for example by switching from hand to machine methods as discussed in section 4.2. This could be simulated in EAROMAR by adjustments in both the quality standards and the maintenance technologies discussed earlier.

Finally, it is important to realize that several maintenance activities are simulated within EAROMAR, all of which compete for the same pool of resources. If resource availability is exceeded at any time, the question of which activities should be cut back to what extent is a difficult one that likewise cannot be anticipated by the EAROMAR system. Again, this question is better addressed by the manager himself, who is able to judge the severity of the resource shortage, and weigh this against the competing options of quality standard reductions vs. increases in resource commitments.

An example of resource constraints provided by a user to EAROMAR is shown in Table 42 . All constraints are assumed to be on an annual basis.

Incorporation Within Strategies

The requirements for maintenance scheduling and lane occupancy may vary along the length of the route and over time, due to changes in roadway geometry, pavement characteristics, or traffic levels. The imposition of resource constraints may also need to vary over length and time to represent conditions in different maintenance jurisdictions (e.g., districts or foremen areas) traversed by the route, or projected changes in employment levels, purchases of maintenance equipment, and the like.

Within the EAROMAR simulation, these variations can be represented through the definition of strategies described in Chapter 2. Information on the supply of maintenance services is organized within two independent blocks: (1) a "scheduling" block, comprising the information on resource class declarations, unit costs, maintenance technology, scheduling, and roadway occupancy covered earlier in this section; and (2) a "resources" block containing data or resource con-

RESOURCE	ANNUAL CONSTRAINT	UNIT MEASURE
LABOR		
Workers	10,000	Hours
Drivers	6,000	"
Foremen	3,000	"
EQUIPMENT		
Dumptrucks	2,000	Hours
Kettle	2,000	"
Router	2,000	"
Compressor	2,000	"
Pickup	2,000	"
Loader	2,000	"
Roller	2,000	"
Chip Spreader	2,000	"
Broom	2,000	"
Tamper	2,000	"
Backhoe	2,000	"
Distributor	2,000	"
Paver	2,000	"
Concrete Saw	2,000	"
Water Tank	2,000	"
Applicator	2,000	"
Vibrator	2,000	"
Mudjacker	2,000	"
MATERIALS		
Liquid Asph	8,000	Gal
Asph Emulsion	8,000	Gal
Chips	500	Ton
Bit Conc	30	Ton
Gravel	20	Ton
PC Concrete	150	CY
Sealant	200	Gal
Mesh	1,000	SF
Epoxy Additive	10	Gal
Curing Compound	5	Gal
Dirt	100	CY
Cement	100	Sack
Water	1,000	Gal

TABLE 42

EXAMPLES OF RESOURCE CONSTRAINTS

straints discussed above.

Different blocks of "scheduling" and "resource" information may be defined and applied by area and time under the strategy specifications. "Area" was chosen in lieu of "roadway" as the basis of assignment for this maintenance information, since the determinants of scheduling and occupancy decisons are likely to be found in the changing character of the route as a whole along its length, rather than in individual roadways. Furthermore, issues of maintenance technology and resource constraints are tied to maintenance jurisdictions, again a regional concept. The "areas" defined within EAROMAR may be used to designate different maintenance jurisdictions or zones of generally different road characteristics simultaneously.

4.4 MAINTENANCE COSTS

The calculation of maintenance costs within EAROMAR accounts for both the demand-side elements discussed in section 4.2 and the supply-side factors considered in section 4.3. The simulation progresses on a seasonal basis, considering each maintenance activity in turn, within each roadway section. The basic approach to the cost calculation is pictured in Figure 45 ; details on the steps of this process are presented below.

Workload, Time, and Resource Requirements

GENERAL

The basis for predicting maintenance costs within EAROMAR is the maintenance workload. Workload is simply the total maintenance requirement by season, roadway section, and activity, computed as the product of the current pavement condition and the respective quality standard. Examples of workload calculations under different quality standards were given in Figure 37 .

Workloads are expressed in terms of the activity work units shown in Table 34. At the same time, both activity production rates and resource consumption rates (e.g., Table 40) also include the activity work unit within their dimensions. This correspondence among the different terms of the problem render the calculation of required time and resources a straightforward one. An example will illustrate.

Suppose that within a roadway section, 50,000 sf of flexible pavement are to be sealed. From the production rate for the sealing activity (Table 40), the total time required for sealing is estimated as:

$$\frac{50{,}000 \text{ sf}}{7{,}920 \text{ sf/hour}} = 6.3 \text{ hours (1 sf = 0.09 sm)} \qquad (201)$$

The resources required for this activity, also given in Table 40 , include one foreman, ten drivers, four workers, and several items of equipment and material. To seal 50,000 sf of pavement in this roadway section will therefore require 6.3 hours of effort by each labor crew member, 6.3 hours of operation by each piece of equipment, and sufficient materials to seal 50,000 sf of pavement. The resulting total resource requirements for this section are displayed in the first column in Table 43.

Calculations of maintenance time requirements and resources consumed analogous to these may be carried out on an area-wide level as well. Assume that the area encompassing the roadway section above has a projected workload for sealing of one million square feet (including the 50,000 sf for the section computed above). The area-wide maintenance time required for sealing would then be 126 hours, and the

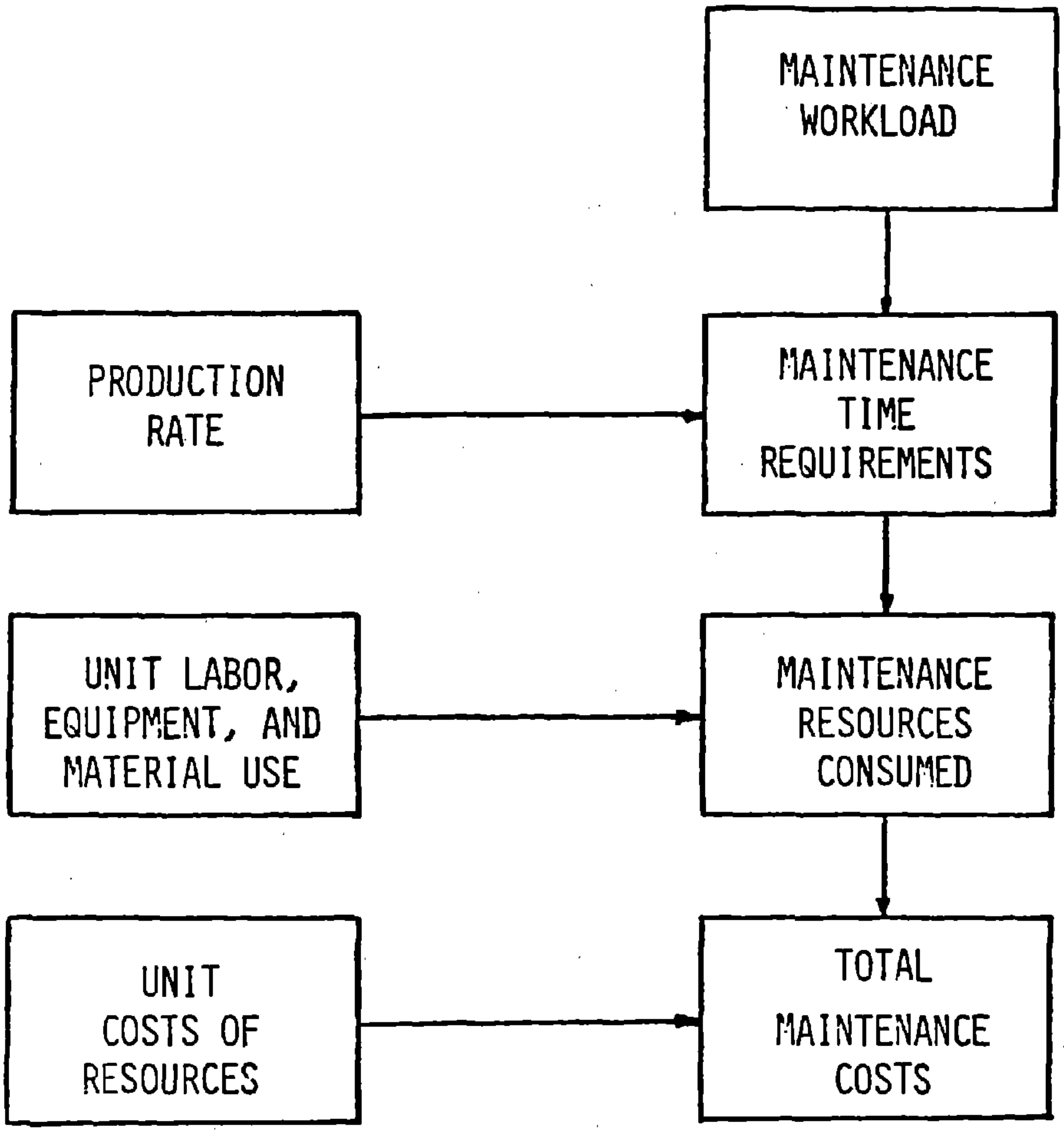


FIGURE 45. CALCULATION OF MAINTENANCE COSTS

TABLE 43

EXAMPLE CALCULATION OF
MAINTENANCE REQUIREMENTS

Resource Class	Seal 50,000 sf (4560 sm)	Seal 1 million sf (92950 sm)
Labor		
Foreman	6.3 MH	126 MH
Workers	25.2 MH	504 MH
Drivers	63.0 MH	1,260 MH
Equipment		
Dumptrucks	31.5 EH	630 EH
Loader	6.3 EH	126 EH
Rollers	12.6 EH	252 EH
Chip Spreader	6.3 EH	126 EH
Broom	6.3 EH	126 EH
Pickups	12.6 EH	252 EH
Materials		
Asphalt Emulsion	700 Gal.	14,000 gal.
Chips	50 T.	1,000 T.

(1T = 0.91 MT; 1 gal. = 3.8 litres)

resources consumed equal to twenty times the quantities computed for the single roadwav section. These results are displayed in Column 2 of Table 43.

SCHEDULING IMPLICATIONS

The application of scheduling data (as typified in Table 41) takes place at the roadway section level, since it is here that the interaction between maintenance occupancy and traffic operations is simulated (see Chapter 5.)* Continuing to our example in Table 41, the task now is to allocate the 6.3 hours attributable to sealing among hours in a day, and among days within the season of the year.

First consider the problem in its general context. In an analysis such as that performed by EAROMAR, which projects costs streams several years into the future, it is unrealistic to attempt to schedule maintenance on specific calendar days. It makes more sense, rather, to think in terms of generic or "typical" categories of days, and then to estimate what proportion of each category of typical day occurs within a season. The distinction between weekdays and weekends immediately defines two broad classes of typical days. Within each such class, however, there are additional divisions of typical days; e.g. those days on which the activity group that includes sealing is performed, those days on which groups of other maintenance activities are performed, or those days on which no maintenance at all is performed. The magnitudes of both maintenance costs and highway user costs accumulated within a season depend on the relative frequencies of each category of typical day occuring within that season.

The characteristics of a typical day are defined by the scheduling information as illustrated in Table 41. This information is interpreted in a totally unbiased manner. For example, if an activity is scheduled on both weekdays and weekends, there is no reason to assume a priori that either the weekday or the weekend would be favored over the other. Similarly, if the time required for an activity group turns out to be less than the total daily hours allotted in the maintenance schedule, there is no basis for assuming that one subset of available hours would be favored over another. The implication is that any maintenance requirement will be prorated over the total weekday and weekend time available. If the hours required for maintenance are less than the total time scheduled, this will be reflected in a fractional number of typical days computed.

These assumptions are embodied in two sets of equations corresponding to "on-site" and "total" closure zones respectively. The equations for on-site closures are as follows:

*Recall that the scheduling block of information is provided by the user by route areas. Therefore a given scheduling block will apply to all roadway sections within the referenced area(s).

$$SUM = 5 \times (WKDHRS - MOBTIM) + 2 \times (WKEHRS - MOBTIM) \quad (202)$$

$$CLSDYS = \frac{7 \times WRKHRS}{SUM} \quad (203)$$

$$WKDDYS = \frac{5 \times CLSDYS}{7} \quad (204)$$

$$WKEDYS = \frac{2 \times CLSDYS}{7} \quad (205)$$

and the equations for total closures are as follows:

$$SUM = (5 \times WKDHRS) + (2 \times WKEHRS) \quad (206)$$

$$CLSDYS = \frac{(7 \times WRKHRS) + MOBTIM}{SUM} \quad (207)$$

$$WKDDYS = \frac{5 \times CLSDYS}{7} \quad (208)$$

$$WKEDYS = \frac{2 \times CLSDYS}{7} \quad (209)$$

where:

- SUM is the weekly sum of scheduled hours available for maintenance work
- WKDHRS is the number of daily weekday hours scheduled for this activity group
- WKEHRS is the number of daily weekend hours scheduled for this activity group
- MOBTIM is the mobilization time specified for this activity group, in hours
- CLSDYS is the number of closure days required to complete all work within the activity group
- WRKHRS is the total time required to complete all maintenance activities within this group, in hours
- WKDDYS is the total number of weekday closure days required
- WKEDYS is the total number of weekend closure days required

and 5,2,7 are the weekday, weekend, and total days per week respectively.

An example will illustrate the application of these relationships to a given roadway section. We estimated earlier that 6.3 hours were required to seal our example flexible pavement surface. From Table 41. crack filling is also an activity within this group, and would therefore be performed with sealing. Suppose that a calculation (anologous to that performed for sealing in equation (201)) yields 5.7 hours required for crack filling. The total maintenance requirement for this activity group is then:

$$\text{WRKHRS} = 6.3 + 5.7 = 12.0 \text{ hrs.} \tag{210}$$

From information in Table 41 we obtain the following:

$$\begin{aligned} \text{WKDHRS} &= 5.0 \text{ hrs.} \\ \text{WKEHRS} &= 6.0 \text{ hrs.} \\ \text{MOBTIM} &= 0.5 \text{ hr.} \end{aligned} \tag{211}$$

Assuming an on-site closure, we then compute the closure days as follows:

$$\text{SUM} = 5(5.0-0.5) + 2\,(6.0-0.5) = 33.5 \text{ hrs. weekly} \tag{212}$$

$$\text{CLSDYS} = \frac{7 \cdot 12}{33.5} = 2.51 \text{ days} \tag{213}$$

$$\text{WKDDYS} = \frac{5}{7} \cdot 2.51 = 1.79 \text{ weekday days} \tag{214}$$

$$\text{WKEDYS} = \frac{2}{7} \cdot 2.51 = 0.72 \text{ weekend days,} \tag{215}$$

Therefore, if there were 90 days within a season comprising, on average, 64 weekdays and 26 weekend days, then about 1.8 of those weekdays and 0.7 of the weekend days would experience maintenance and user costs incident to sealing and crack filling. Such costs would be computed by simulating a weekday closure totaling five hours (of which one-half hour is spent in mobilization), and a weekend closure of six hours (again with one-half hour for mobilization). The road occupancy would be imposed only during those hours given in the scheduling specifications in Table 41.

In contrast, if the occupancy entailed a total closure, and if we assumed an increased mobilization time of 4.0 hours, the closure statistics would be:

$$SUM = 5 \cdot 5.0 + 2 \cdot 6.0 = 37.0 \text{ hours weekly} \quad (216)$$

$$CLSDYS = \frac{7 \cdot 12.0 + 4.0}{37.0} = 2.38 \text{ days} \quad (217)$$

$$WKDDYS = \frac{5}{7} \cdot 2.38 = 1.7 \text{ weekday days} \quad (218)$$

$$WKEDYS = \frac{2}{7} \cdot 2.38 = 0.68 \text{ weekend days} \quad (219)$$

In this case the number of days required to complete work under a total closure is somewhat less than that for an on-site closure. However, it should be realized that total closures are simulated with the workzone occupying the roadway 24 hours per day. Thus in this example, the user costs incident to such all-day closure will be tallied for 1.7 weekday days and 0.68 weekend days during this season.

The times required to complete other activity groups within the maintenance program are computed in a similar way. In the simulation of their costs and impacts, activity groups are assumed to be independent of one another. Thus, on any roadway section at a given time the vehicle stream may experience effects due to roadway occupancy for one activity group, but never more than one such group. Stated another way, the number of different "typical" days to be simulated depends upon the number of individual activity groups, but never upon combinations of activity groups. This assumption of independence allows one to treat both maintenance and user costs attributable to each group in a linearly additive way.

RESOURCE USAGE

Resource usage is monitored within each season at the area level, following completion of all maintenance activities on all relevant roadway sections. Data on resource consumption by activity are compiled in much the same way as shown for flexible pavement sealing in Column 2 of Table 43. The total resource usage is then obtained for each resource class, by summing the respective quantities used in each activity throughout the area. If the predicted usage for any class exceeds the limits illustrated in Table 42, a warning message is printed as described in Section 4.3.

Cost Predictions

Maintenance costs predicted by EAROMAR follow directly from the resource requirements estimated above and the unit costs provided by the user as discussed in Section 4.3. Costs are computed for each resource class according to the relationship:

Cost (resource class, activity)

= Usage (resource class, activity, schedule)

x Unit Cost (resource class, schedule) (220)

where "usage refers to the actual (as opposed to scheduled) time spent on maintenance by labor and equipment, and where the unit cost includes time-dependent adjustments denoted by the "schedule" parameter. These maintenance costs are then aggregated within the general categories of labor, equipment and materials by maintenance activity.

Mobilization costs are added to the maintenance costs computed above. For on-site closures the costs of mobilization are tallied each time the workzone occupies the roadway. For total closures the mobilization costs are computed once for each workzone. The relevant equation for on-site closures is then:

Mobilization Costs (activity group) = fixed component

x (Scheduled periods per day x Number of typical days) (221)

+ Mobilization Time x (Hourly Labor Charges + Hourly Equipment Charges)

and for total closures:

Mobilization Costs (activity group) = fixed Component

+ Mobilization Time x (Hourly Labor Charges + Hourly Equipment Charges) (222)

where the mobilization time and fixed cost component are provided by the user, hourly labor charges include the sum of time-adjusted wages for all crew members, and hourly equipment charges include the sum of hourly equipment rates for all classes and pieces of equipment.

Cost Results

The labor, equipment, and materials components of maintenance costs including mobilization costs, are individually inflated at the respective relative rates discussed in Chapter 2. Maintenance cost totals are then reported by maintenance activity, season, and year. Maintenance cost data are available through EAROMAR by individual roadway, area or route as a total.

CHAPTER 5

ROADWAY OPERATIONAL CHARACTERISTICS

5.1 TRAVEL DEMAND

Introduction

The demand upon a road facility is conventionally expressed as the traffic magnitude (volume or flow) and its composition (by trip purpose and/or vehicle type). Moreover, demand is dynamic, in that it varies simultaneously over route length, direction, and time. An understanding of these dynamic demand characteristics is important to the economic justification of premium pavements, because they contribute to or interact with several technical and economic components of the analysis:

1. Pavement damage. Road traffic subjects the pavement to structural loadings, causing deterioration and eventually failure of the pavement system. For a given traffic projection the rate of pavement deterioration, and hence the probability that time to failure will exceed desired pavement life, may be controlled through pavement thickness and materials quality specifications (and a correct accounting of environmental effects) during design and construction, complemented by appropriate maintenance or rehabilitation actions during service life. Traffic-induced stresses are functions of cumulative numbers and gross weights of vehicles applied, the seasons during which applications take place, and vehicle characteristics such as the number and spacing of tires, tire pressures, and vehicle speeds.

2. Maintenance scheduling. Apart from purely technical requirements, maintenance work schedules can be strongly influenced by travel demand patterns, particularly on high volume roads. For those maintenance and rehabilitation actions requiring sizable work zones or substantial periods of occupancy, agencies may investigate off-peak, weekend, or off-season work periods when travel demand is lowest. The premium prices commanded for work during other than regular hours, or within tight time and space constrictions, must be justified by the minimum disruption to the traffic stream and attendant benefits to the users in increased travel time savings and reduced vehicle operating costs and accident and pollution potentials.

3. User consequences. Beyond their proportional relationship to numbers of vehicles, user-related costs and benefits are also functions of traffic composition; thus, for given traffic volumes or flows, the observed average speeds, congestion characteristics, vehicle operating costs, travel time values, and accident and pollution levels will differ depending upon distributions of trip purposes and vehicle types in the traffic mix. Since user consequences are a factor in maintenance scheduling, traffic compositon may cause second-order corrections to costs and benefits discussed in item 2 above.

It is possible to construct an aggregate description of travel demand consolidating these several characteristics, based upon empirical or semi-empirical relationships appearing in the current literature. (See note 1 at end of chapter.) These relationships are generally familiar to highway practitioners and consistent with current procedures for data collection. Development of this description will begin with broad or long-term characteristics of demand (e.g., overall daily volume; rates of overall annual growth). From these aggregate quantities, more detailed or short-term effects will be derived (e.g., descriptions of the traffic mix; seasonal, daily or hourly variations). Then, in subsequent sections, we will treat specific aspects of road operation necessary for the analysis of premium pavements (definition of road closure configurations in response to maintenance occupancy; and treatment of resulting congestion and queueing).

Traffic Volume

One of the most commonly used measures of travel demand is annual average daily traffic (AADT), defined as the total yearly volume divided by the number of days in the year. Separate values of AADT may be specified for each roadway, as described in Chapter 2.

VARIATION ALONG ROUTE LENGTH

The AADT typically varies along route length, according to the cumulative impact of the point-to-point travel requirements of each motorist. For high volume urban freeways, AADT counts are generally available for individual route lengths between interchanges, as illustrated in Figure 46.

Within the EAROMAR system these demand characteristics can be described using the milepost convention introduced in Chapter 2. For example, the specifications in Table 44 could be used to model the situation shown in Figure 46 assuming two roadways and a 50-50 directional distribution.

TABLE 44

VARIATION IN AADT
ALONG ROUTE LENGTH

MILEPOST INTERVAL	AADT	
	ROADWAY 1	ROADWAY 2
70.0 - 73.6	35,000	35,000
73.6 - 74.5	36,750	36,750
74.5 - 76.2	37,500	37,500
76.2 - 78.0	38,000	38,000
78.0 - 83.0	39,550	39,550
83.0 - 85.5	38,500	38,500
85.5 - 87.9	37,750	37,750

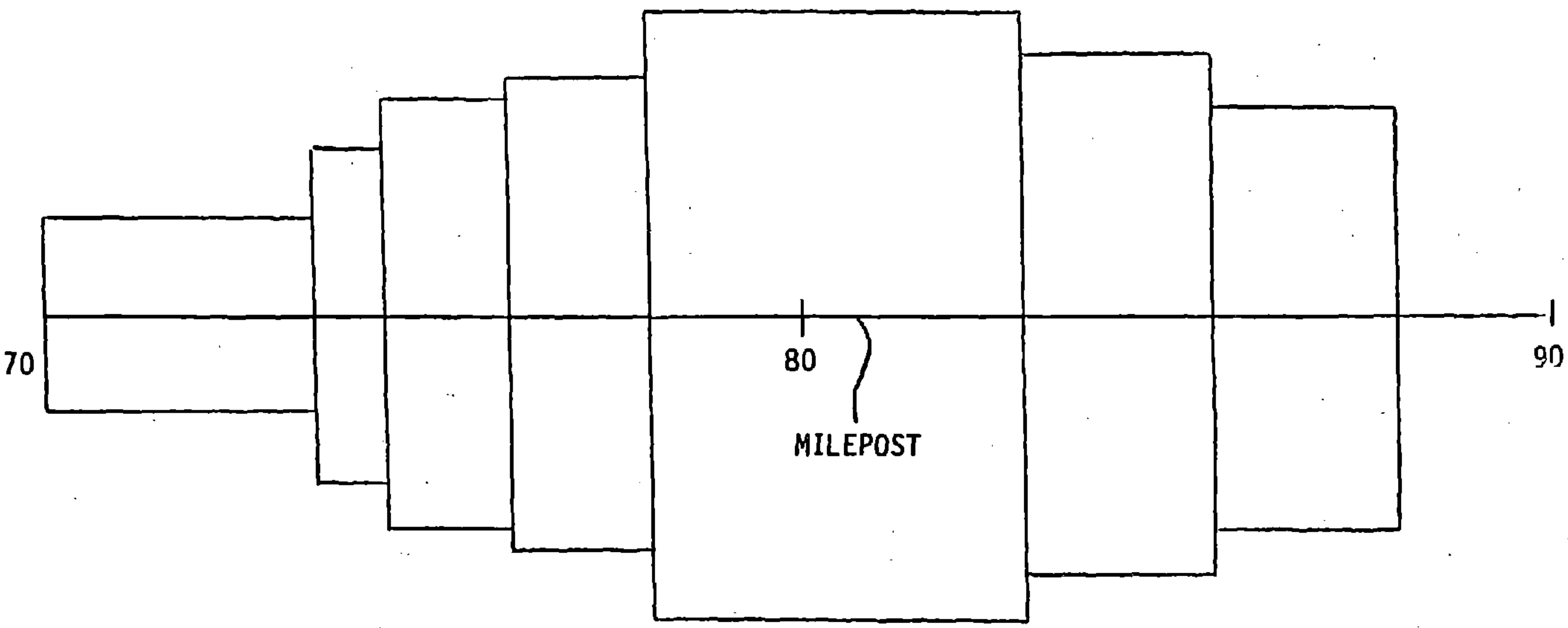


FIGURE 46. VARIATION IN AADT ALONG ROUTE LENGTH

LANE DISTRIBUTION

The lane distribution on multilane roadways may vary widely, depending upon traffic volume, side friction, spacing of exits and percentage of slower-moving vehicles. Figure 47 illustrates observed distributions for six-lane facilities as a function of flow (vehicles per hour), indicating that peak lane usage (in this case, for the middle lane) varied from about 48% to about 37% of total flow.

EAROMAR does not require knowledge of lane distribution for operational purposes, since road closures, congestion, and other flow-related factors can be more efficiently treated in a roadway-by-roadway, rather than a lane-by-lane, simulation. However, lane distribution is important in predicting pavement deterioration (under the "design lane" concept), and in this regard it is useful to mention how this item is treated within the analysis.

Because of the variability of design lane factors observed in practice, EAROMAR permits users to specify these data within the capacity-related characteristics of each roadway. Furthermore, lane factors may be varied over roadway length to represent changes in lane distribution due, for example, to different numbers of lanes or different demand characteristics. From the data presented above, and in Table 45, it is reasonable to assume that a lane distribution factor of 50 to 80 per cent is appropriate for the types of roadways likely to be considered in a premium pavement analysis.

TIME VARIATIONS

By definition, the AADT is a stable quantity, intended to measure only long-term time variations in travel demand. Two types of time variations in AADT are included in the EAROMAR analysis: growth in demand, and future point adjustments. Specification of data governing these variations was discussed as part of strategy definition in Chapter 2.

Traffic Growth. Traffic growth is correlated with changes in patterns of social, economic, or demographic activity. In addition, since the link analysis proposed for EAROMAR is an approximation to the more comprehensive network approach, other factors may also be included in demand growth projections: e.g., traffic diverted from other highways, traffic resulting from modal shifts, or generated traffic.

Two types of growth patterns are included within the EAROMAR analysis: linear growth, and geometric growth. Regardless of which type is specified, traffic growth is assumed to apply to total roadway demand, rather than to any component of it. Linear growth between years T and T+1 is computed as:

$$AADT_{T+1} = AADT_T + \Delta AADT \qquad (223)$$

where ΔAADT is rate of growth in units of incremental numbers of vehi-

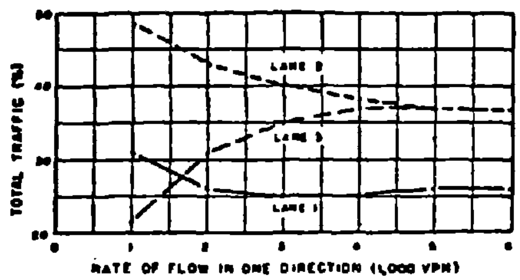


FIGURE 47
EFFECT OF TRAFFIC VOLUME ON LANE
USE FOR SIX-LANE FACILITIES

Source: Highway Capacity Manual, SR 87, 1965. (3)

Number of Traffic Lanes (Two Directions)	*Percentage of Trucks in Design Lane*
2	**50**
4	**45 (35-48)[1]**
6 or more	**40 (25-48)[1]**

[1] *Probable range.*

TABLE 45
PERCENTAGE OF TOTAL TRUCK TRAFFIC
(TWO DIRECTIONS) IN DESIGN LANE

Source: Thickness Design Manual (MS-1), September 1963. (48)

cles, as provided by the user. The increment ΔAADT remains constant from year to year for a given roadway segment, until updated or modified by the user in his strategy specifications.

Geometric growth between years T and T+1 is computed as:

$$AADT_{T+1} = AADT_T \quad (1 + r/100) \tag{224}$$

where r is the average annual rate of growth expressed as a percentage, as provided by the user. Again, the rate of annual growth remains constant for a given roadway segment, until updated or modified in the strategy description.

Future Adjustments. Ocassionally within a road network relatively abrupt changes in traffic assignment occur, due to construction of a new road link, addition of new interchanges, or revisions in levels of service in other modes. Such changes are interruptions in normal growth patterns. Although it is possible to represent such interruptions by short-lived changes in growth rates, at other times it is simpler to recompute route AADT based upon estimates of traffic redistribution. Since the EAROMAR analysis is conducted at a link rather than a network level, the redistribution must be calculated by the user before performing the premium pavement analysis. Nevertheless, it is possible to represent the results of such calculations as point adjustments in AADT, which would be independent of, and would override the predictions of, prior growth patterns.

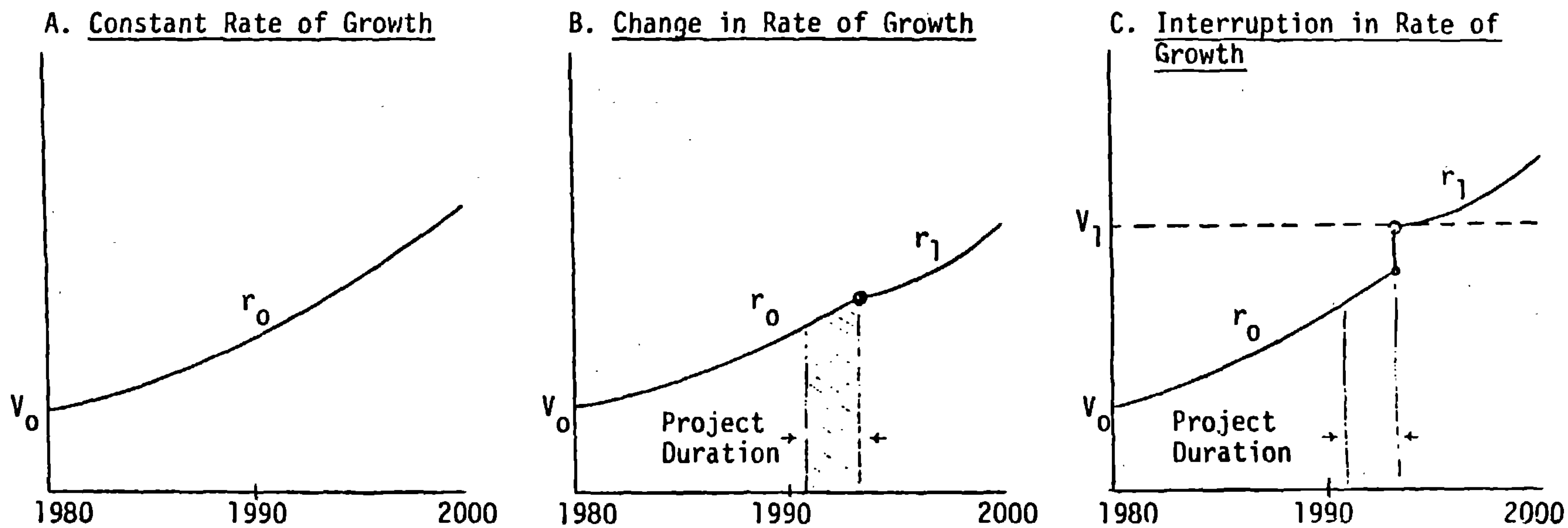
The specification of future adjustments in AADT is accomplished in much the same way described for initial AADT earlier, and is also part of the strategy definition process described in Chapter 2. Such adjustments may vary simultaneously over roadway, location, and time. The year in which an adjustment takes effect may be either input directly by the user or made contingent upon the completion of a project. Once a future AADT level has been assigned, it may be allowed to resume normal growth under the growth pattern previously defined for the roadway segment; or, a new growth pattern may be specified.

Examples. At this point it may be useful to consolidate our discussions above on the several ways in which traffic volume may be treated as a dynamic route characteristic within the EAROMAR analysis. Following are two examples illustrating application of the concepts presented earlier. For simplicity we have focused on time variations of volume at one route location; however, variations along the length of the route would be accomplished simply by adjusting the milepost boundaries of each description of AADT or traffic growth.

Figure 48 illustrates three typical situations of traffic growth, each succeeding one increasing in complexity.

For each case the AADT and growth specifications are shown, accompanied by a schematic graph of resulting traffic demand through one anal-

FIGURE 48. EXAMPLES OF AADT GROWTH PATTERNS AND SPECIFICATIONS



A.

1980: AADT = V_o

Geometric Growth Rate = r_o

B.

1980: AADT = V_o

Geometric Growth Rate = r_o

Project Completion:

Geometric Growth Rate = r_1

C.

1980: AADT = V_o

Geometric Growth Rate = r_o

Project Completion:

AADT = V_1

Geometric Growth Rate = r_1

ysis period. Figure 48A projects a simple geometric growth of r_0 per cent from an inital AADT of V_0, continuing throughout the analysis period. This simple representation may be adequate for many analyses. In Figure 48B we modify this situation to include the effects of a project that is presumed to change the rate of traffic growth. The new rate r_1 governs from the completion of the project through the remainder of the analysis period. In Figure 48C we assume further that project completion will cause redistribution of traffic throughout the road network, and that the resulting volume estimated for the route segment under analysis is V_1. (Note that in this case the estimate of V_1 causes a positive or upward shift from the trend line indicated by prior growth; the shift could just as well have been negative.) Subsequent growth will proceed from V_1 at a rate of r_1 through the remainder of the analysis period.

Figure 49 illustrates a different type of example, where we have assumed steady growth of r_0 to be interrupted in year T by a significant, but short-lived, increase in traffic. (The cause of this increase might be, for example, a planned attraction such as an exposition; temporary diversion of traffic from another source; or, if the change were negative, the results of a fuel shortage.) Figure 49A indicates how this situation could be treated using geometric growth relationships; Figure 49B, employing linear growth conventions. Following the peak, one could specify either resumption of the original growth r_0 for the remainder of the original period, or assume some residual effect of the peak via a new rate r_1 (the latter option is illustrated in Figure 49).

The descriptions of traffic volume presented earlier, when applied correctly in combination with one another, therefore provide considerable flexibility in representing many different types of growth scenarios for traffic volume. Moreover, if future traffic growth is subject to uncertainty, several different scenarios may be defined and tested in a series of runs under the EAROMAR system, to gauge the sensitivity of results to different growth assumptions.

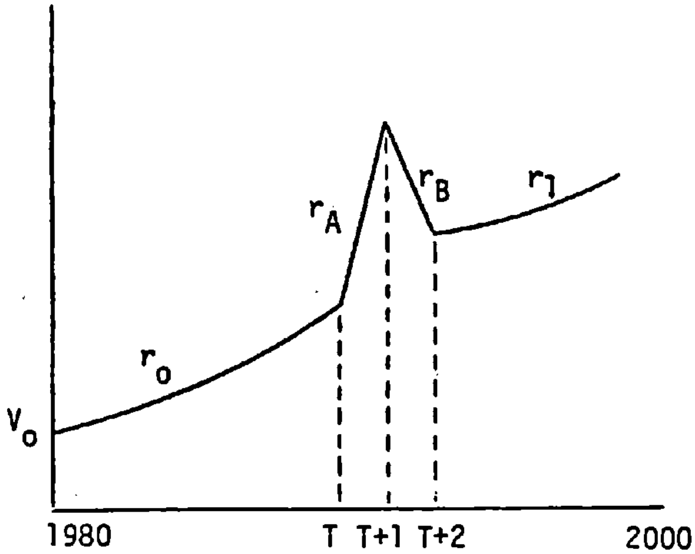
Traffic Composition

There are several important aspects of road operations that require more detailed information on travel demand than that provided by AADT, and perhaps over shorter time intervals than annual daily average. Factors for which we would like additional information include:

1. pavement loadings;
2. volume-capacity relationships, speed-flow relationships, congestion and queueing; and
3. differentiation among representative types of vehicle operating costs, values of travel time, and other user consequences.

FIGURE 49. EXAMPLES OF AADT GROWTH INTERRUPTIONS

A. Using Geometric Conventions

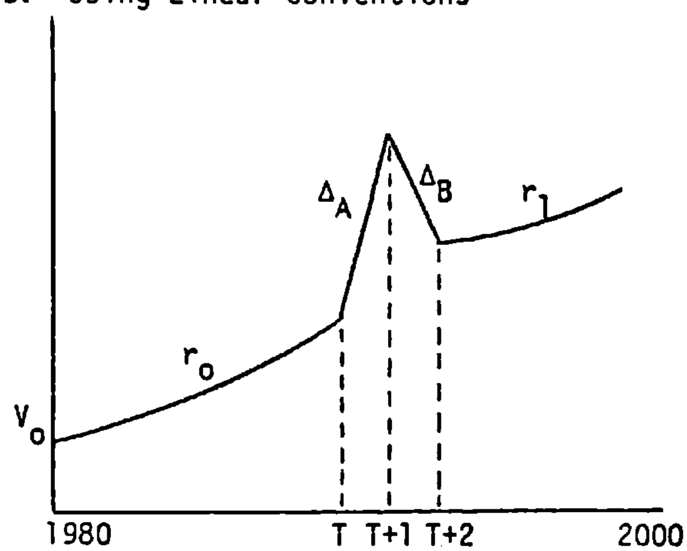


1980: AADT = V_o
Geometric growth
Rate = r_o

T: Rate = r_A
T+1: Rate = r_B
T+2: Rate = r_1

$(r_A \gg r_o)$
$(r_B < 0)$
$(|r_B| \lesssim r_A)$

B. Using Linear Conventions



1980: AADT = V_o
Geometric Growth
Rate = r_o

T: Linear growth
Rate = Δ_A

T+1: Rate = Δ_B

T+2: Geometric growth
Rate = r_1

$(\Delta_\Delta \gg V_o(1+r_o))$
$(\Delta_B < 0)$
$(\Delta_B \leq 0_A)$

These data are conventionally derived from descriptors of traffic composition, particularly regarding distributions of trip purpose and vehicle type in the traffic stream. Each major component of the description of traffic composition and its variation over time and space are discussed below.

TRIP PURPOSE

The distribution of the traffic stream by trip purpose is an important characteristic of travel demand, since it relates the underlying socio-economic need to undertake transportation to a specific user response regarding mode, time, and cost of travel. Trip purpose is therefore often used as a stratifier in expressing individual values of travel time; seasonal, daily and hourly variations in traffic flow; and choice of vehicles to accomplish the transport objective.

Definition. The definition of trip purpose is based upon the following concepts:

1. Trip purpose, and information associated with trip purpose, is defined by the user at the time of the analysis; it need not be pre-defined within the EAROMAR system beforehand.

2. Individual trip purposes must be mutually exclusive; i.e. pertain to readily identifiable and separable components of the traffic stream. Thus, "work trips" and "shopping trips" are two different, mutually exclusive purposes in that trips in one category do not coincide or overlap with trips in the other. If there were combined "work-shopping" trips in the traffic stream, these would have to be defined either in one category or the other above; or a separate purpose, "work-shopping," could be created to include them. In any case, trips should not be "double-counted" in two or more categories;

3. The set of trip purposes must be collectively exhaustive; i.e. the purposes defined must be sufficient to encompass the total volume of AADT.

Given these general conditions, trip purpose may be declared within the analysis via (1) a name or identifier, and (2) the relative percentage of AADT attributable to that purpose.* The identifier will enable the system to recognize each trip purpose declared by the user, and to organize all related information correctly by purpose. The percentage of AADT will distribute the total demand volume, and serve as the basis for more detailed variations over space and time to follow below. The percentages for all trip purposes must sum to 100 per cent.

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* Separate percentages are provided for weekdays and weekends; further distinctions by type of day will be discussed shortly.

TABLE 46

EXAMPLES OF TRIP PURPOSE SPECIFICATIONS

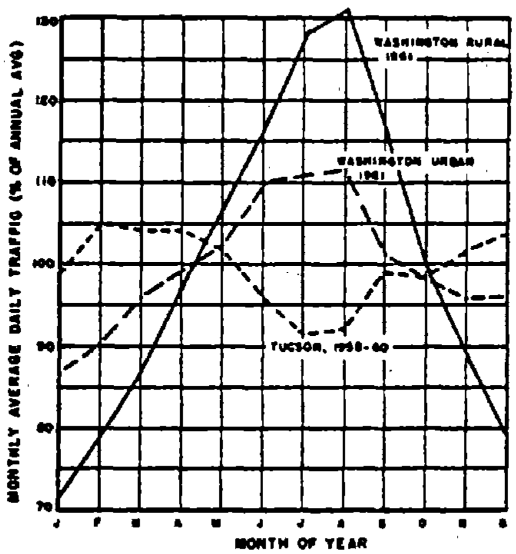
	CATEGORY	WEEKDAY	WEEKEND
A.	Commuting	.30	.15
	Social-Recreational	.15	.25
	Shopping	.05	.10
	Personal business	.10	.15
	Vacation	.05	.05
	School	.10	0
	Freight	.10	.10
	Service	.10	.10
	Passenger	.05	.10
		1.00	1.00
B.	Commuting	.35	.15
	Personal	.30	.35
	Commercial Freight	.25	.30
	Commercial Non-Freight	.10	.20
		1.00	1.00
C.	Commercial	.80	.80
	Non-Commercial	.20	.20
		1.00	1.00
D.	All Trips	1.00	1.00

To illustrate the application of these concepts within the EAROMAR analysis, we have listed in Table 46 several co rect examples of trip purpose definition. The last example is a special case where one in effect suppresses any differences among trip purpose classifications, and treats travel demand as an anonymous stream of motorists. Although this procedure is not necessarily recommended in an analysis, it may have use in certain special situations, and in any case it illustrates the generality of the proposed approach. Furthermore, the specific purposes shown are for illustration only; any other divisions could have been chosen, subject only to the constraint that one provide the associated data required by the analysis under each trip purpose classification. These requirements will be explained in detail below and in Chapter 6.

Seasonal Distribution. Traffic has been observed to vary seasonally by as much as ± 30 per cent from the equivalent annual average, as illustrated for three locations in Figure 50. However, note that the patterns of seasonal variations may differ widely from one another, and in some cases can run counter to one another. This is not an unreasonable finding, in that travel demand is likely to fluctuate depending upon the particular economic character, degree of urbanization, and climate of the region served by a route, and the composite of social, economic and demographic factors inducing travel demand.

Seasonal effects are included within the EAROMAR analysis, first, to relate the demand fluctuations above to route operational requirements and concomitant scheduling of needed maintenance or rehabilitation; and second, to account for differences in numbers of pavement loadings, and pavement response to stress, at various times of the year. From our discussion above it is reasonable to represent seasonal fluctuations in total traffic volume by introducing, instead, individual variations by trip purpose. (The validity of this approach will be further reinforced through the correlation of trip purpose with vehicle type and its effect on pavement loadings, as will be discussed in a following section.)

Recall from Chapter 2 that the number, names, and durations of seasons to be considered in the analysis are initially declared as part of the general route characteristics; therefore, the seasons over which traffic demand will vary must be consistent with the earlier information provided. Quantification of demand variation will be expressed as "relative percentage of AADT" throughout the duration of the season. Therefore, a demand level of "1.3" in a 90-day season implies that, for each of the 90 days, traffic volume will be projected at 130% of the annual daily average. Note that by the definition of AADT, the seasonal levels provided, when weighted by season length in months, must balance to a 12-month average of 1.0.



Source: Highway Capacity Manual, SR 87, 1965.
(3)

FIGURE 50

EXAMPLES OF MONTHLY TRAFFIC VOLUME VARIATIONS

TABLE 47

EXAMPLES OF SEASONAL DISTRIBUTIONS

	SEASON (Index)	LENGTH (Months)	DISTRIBUTION FACTOR	CHECK (2) X (3)	
A.	1	2	0.7	1.4	
	2	2	0.8	1.6	
	3	2	1.0	2.0	
	4	2	1.1	2.2	
	5	2	1.2	2.4	
	6	2	1.2	2.4	
		12		12.0	OK
B.	1	3	0.8	2.4	
	2	3	0.8	2.4	
	3	3	1.1	3.3	
	4	3	1.3	3.9	
		12		12.0	OK
C.	1	3	0.8	2.4	
	2	9	1.06	9.6	
		12		12.0	OK
D.	1	12	1.0	12.0	OK

Table 47 gives examples of correctly specified seasonal distributions. The last example is the degenerate one, where no seasonal variation is assumed; in this case analysis results will be based upon demand volumes equal to AADT that remain uniform throughout the year. Season length is included in the table only to illustrate that distribution factors have been correctly estimated; in actually using the analysis, one would specify season length with the route characteristics discussed in Chapter 2.

Daily and Hourly Variations. A knowledge of daily and hourly variations in traffic volume is fundamental in analyzing high-volume routes for premium warrants. In certain periods of the day demand may already exceed capacity, resulting in congestion and queueing. Additional disruptions in the traffic flow are then likely if temporary worksites or partial route closures are introduced for pavement maintenance and rehabilitation. Because the user-related excess costs in congested flow can be very high due to the large numbers of vehicles typically involved, daily and hourly demand variations may also play a strong role in maintenance scheduling during off-peak periods, affecting the cost and conceivably the quality of work thus performed.

For each trip purpose category one may specify separate hourly distributions of demand for weekdays and weekends. Specification of the required distributions is in terms of percentage of seasonally adjusted AADT within each hour for each trip purpose and type of day. It is assumed that the hourly distributions (whether for weekday or weekend) are independent of seasonal adjustments and remain the same throughout the year. Seasonal adjustments therefore serve to change only total daily volume, and not the hourly distribution of that volume.

Table 48 illustrates the correct assignment of hourly demand distributions for two hypothetical trip purposes. The hourly percentages, when summed, must total 1.0 for each trip-purpose/type-of-day combination.

Integration of Time Variations. Under the general heading of trip purpose we have just discussed several types of time variations in travel demand of intervals shorter than one year. The individual adjustment factors presented may now be combined within a single relationship to yield hourly flows for each trip purpose as measures of demand composition. These flow estimates will be used by the EAROMAR analysis in later computations of volume-capacity, speed-flow, and user consequence values. The general equation is:

$$Q_j = \text{AADT} \times \text{SEASON} \times \text{PURPOSE} \times \text{HOURLY} \qquad (225)$$

where Q_j = projected demand in vehicles per hour for a given trip purpose, analysis year, season, type of day, and hour of day;

j = an index over trip purpose

TABLE 48

EXAMPLES OF HOURLY DISTRIBUTIONS

HOUR	A. COMMUTER TRIPS WEEKDAY	A. COMMUTER TRIPS WEEKEND	B. SCHOOL TRIPS WEEKDAY	B. SCHOOL TRIPS WEEKEND
0001				
0100	0.00	0.00	0.00	0.00
0200	0.00	0.00	0.00	0.00
0300	0.00	0.00	0.00	0.00
0400	0.01	0.00	0.00	0.00
0500	0.03	0.00	0.00	0.00
0600	0.08	0.05	0.00	0.00
0700	0.22	0.15	0.00	0.00
0800	0.19	0.25	0.16	0.20
0900	0.07	0.20	0.32	0.40
1000	0.05	0.05	0.10	0.20
1100	0.05	0.05	0.04	0.00
1200	0.05	0.05	0.02	0.00
1300	0.05	0.00	0.01	0.00
1400	0.04	0.05	0.01	0.00
1500	0.04	0.10	0.01	0.00
1600	0.03	0.05	0.01	0.00
1700	0.03	0.00	0.02	0.00
1800	0.03	0.00	0.06	0.00
1900	0.03	0.00	0.12	0.20
2000	0.00	0.00	0.08	0.00
2100	0.00	0.00	0.04	0.00
2200	0.00	0.00	0.00	0.00
2300	0.00	0.00	0.00	0.00
2400	0.00	0.00	0.00	0.00
	1.00	1.00	1.00	1.00

AADT = projected demand volume for a given analysis year, in average annual daily traffic

SEASON = ratio of average seasonal daily traffic to average annual daily traffic

PURPOSE = percent of adjusted AADT by trip purpose for a given type of day

HOURLY = ratio of hourly vehicle flow to total daily volume for a given trip purpose and type of day.

The total hourly demand in mixed vehicles, for all trip purposes, and by weekday or weekend is then given by:

$$Q = \sum_j Q_j \qquad (226)$$

VEHICLE TYPE

The second aspect of traffic composition of interest in the EAROMAR analysis is vehicle mix within the traffic stream. The vehicle distribution provides information on the number and magnitude of axle loadings to which the pavement will be subjected, and is critical in defining several operational and user-related factors in the route economic analysis, including average speed, volume-capacity ratio, and vehicle operating costs.

Definition. There are several ways in which one might conceivably categorize vehicles within an economic analysis:

1. by generic class (e.g., passenger car, single unit truck, semi-trailer);
2. by weight or fuel type (e.g., 12-Kip gasoline-powered truck; 50-Kip diesel-powered truck);
3. by passenger or cargo capacity, or by commodity carried; or
4. by type of route served or service provided (e.g., local vs. intercity route; common carrier vs. private vs. Government-owned trucking; etc.).

Under this range of potential options it is reasonable to assume that one would like considerable flexibility in definition of types. The EAROMAR system thus permits user definition of vehicles to be included, so long as vehicle technical, operational, and economic characteristics required by the system are available, and vehicle types can be reconciled with limits imposed by system models (which will be described below). In terms of the flexibility of input, the definition of vehicle types therefore follows guidelines similar to those discussed earlier for trip purpose.

Definition of vehicle type requires simply declaration of a name or identifier for each vehicle, enabling the EAROMAR system to recognize all user-defined vehicle categories and to associate related information correctly with each vehicle. Table 49 illustrates several examples of ways in which vehicles may be declared within the system. The last example is a degenerate one in which no differentiation among vehicle types is planned. In this case one would probably treat the traffic stream as consisting uniformly of some hypothetical composite vehicle having weight and operational characteristics close to the averages observed for the traffic stream as a whole. Again, such a strategy is not necessarily recommended, but it, together with the other examples in Table 49 , is indicative of the versatility afforded the user by this procedure.

Several data items characterizing each vehicle must also be provided by the user under each vehicle declaration. To some extent this information may be influenced or constrained by internal EAROMAR relationships for vehicle flow and operating costs; these influences will be explained below. Furthermore, the structuring of vehicle characteristics data and their relation to trip purpose (providing a unified representation of traffic composition) also require explanation. Following are discussions of each element of data required.

Model Type. To compute operating costs for fuel, oil, and tires, each vehicle classification must be represented within the EAROMAR system by a set of internal resource consumption and cost models. We have developed four sets of such models for use within the analysis, described in Chapter 6 for the following general vehicle classifications:

- Automobile
- Pickup truck
- Single unit truck
- Tractor-trailer combination

In specifying vehicle types within the traffic stream, one must therefore select one of the internal models to represent that particular vehicle classification insofar as operating cost predictions are concerned. For example, one may have declared a passenger bus as a vehicle type; in reviewing the cost models, one may judge that the bus operating characteristics will be best represented by the models for single unit truck. These models would therefore be indicated by the user during declaration of the bus vehicle type. Model selection governs operating cost predictions only; other vehicle characteristics would be specified separately, as will be described below.

Weight. Vehicle weight, expressed in pounds, tons, or kips, has a strong influence on vehicle operating costs. In addition, it affects

TABLE 49

EXAMPLES OF VEHICLE TYPE DEFINITIONS

A. Automobile
   Pickup
   Single-Unit-Truck
   Combination-Truck
   Bus

B. Automobile
   Pickup
   Van
   Single-Unit-Truck
   2-S2
   3-S2
   Bus

C. Automobile
   Truck
   Bus

D. Vehicle

the stresses induced in the pavement by the vehicle; however, this latter effect is also a function of axle and tire configuration, and is better represented by equivalency factors to be described shortly.

Gross vehicle weight is applied within the EAROMAR system to adjust vehicle operating cost estimates within general model categories. For example, assume one wishes to model a tractor-trailer combination at an average gross weight of 55 kips (25 MT). The representative operating cost model within the EAROMAR system will be that for a composite 45-kip (20 MT) semi-trailer (see Chapter 6). The weight differential of 10 kips (4.5 MT) will therefore be applied automatically by the system to increase operating costs predicted by the model to account for the heavier weight.

The definition of gross weight must of necessity be an average one, including not only the spectrum of weights among the many individual vehicles falling within the class, but also operational variations in weight over time (e.g., cargo trucks making return trips empty). If necessary one may define more than one vehicle type within the same general class to differentiate among different classes of cargo carried, for example; or one may conduct sensitivity analyses across different assumptions of weight distributions.

Fuel Type. The two primary fuel types for on-the-road vehicles are gasoline and diesel. Most automobiles and pickup trucks are gasoline-powered; however, single unit trucks and tractor-trailer combinations may be powered by either fuel, with diesel predominating in long-haul intercity routes. Therefore, in Chapter 6 we introduce a correction for diesel fuel consumption developed for trucks ranging from 16 kips to 50 kips (7.2 to 22.7 MT) in gross. One would invoke this adjustment by specifying "diesel" as the vehicle fuel type; otherwise, "gasoline" should be specified.

Axle Weight Equivalencies. With the promulgation of research results from the AASHO Road Test, and publication of the resulting AASHTO Interim Guide (3) for pavement design, the 18-kip equivalent single axle load has become a relatively standardized measure of pavement loading. Applying this measure to estimating traffic-induced pavement damage within the EAROMAR analysis will require that the 18-kip (80 kN) single axle equivalency of each vehicle type be estimated.

The procedures to be used are analogous to those now employed by many state highway departments in analyzing loadometer data such as those contained on standard FHWA W4 loadometer tables. The difference is that for the EAROMAR analysis, the axle load computation will be performed for each vehicle type, rather than for a mixed traffic stream. The 18-kip single axle equivalency factor for a given vehicle would be estimated as:

$$W_{18_{\text{vehicle}}} = \sum_{\text{all axles}} \text{Axle equivalency} \qquad (227)$$

The weight on each axle would be determined based upon the average vehicle gross weight (determined earlier) and the assumed distribution of gross weight over each axle, as illustrated in the example in Figure 51.

The axle equivalency factor is a function of axle load, pavement type, terminal serviceability (i.e., PSI at which pavement is considered to be failed) and pavement structural number or slab thickness. Tabulations of equivalency values across these parameters are given in the Interim Guide as shown in Tables 50 through 53 .

For the types of pavements likely to be considered in a premium pavement analysis, a terminal PSI of 2.5 is more appropriate for both flexible and rigid pavements. Furthermore, to simplify calculations for flexible pavements the Interim Guide suggests a tentative value of 3.0 for structural number to select the equivalency factors for the first pass of a pavement design. Although many charts have been published under this assumption (see Table 54), one should bear in mind that a high volume freeway of the type normally considered for premium pavements may have a structural number considerably higher than 3.0 .

The calculations in Figure 51 illustrate the determination of the axle equivalency for the vehicle shown. The factors employed are taken from Tables 52 and 53 assuming a flexible pavement with SN = 4.0, and terminal serviceability = 2.5 .

Several states have developed their own procedures for loadometer table analysis which simplify the calculation shown in Figure 51 . These procedures either (1) consolidate the 2-kip increments of vehicle weight in Tables 50 - 53 into more aggregate groupings; (2) fix the specification of terminal PSI and structural number or thickness at some predetermined values, and use the equivalency factors so indicated; or (3) develop vehicle axle equivalencies for representative classes of vehicles (in a manner similar to that shown in Figure 51), and apply the vehicle equivalencies in lieu of the axle equivalencies in subsequent analyses. If such procedures have been developed within a state, they may likewise be applied in determining the vehicle equivalency for the EAROMAR analysis.

However, the range of axle weights observed on a highway may vary considerably, as illustrated in truck survey results shown in Figure 52. Van Til (63) has studied various methods of equivalency factor determination in use, and found that they may lead in some cases to substantial errors (as compared to the more rigorous AASHTO procedures), especially if they are applied to highway types or regions markedly different from conditions assumed in their derivation. Nevertheless, they represent the types of simplifications possible in the characterization of traffic pavement loadings. If similar procedures were developed or in

TABLE 50

TRAFFIC EQUIVALENCE FACTORS, RIGID PAVEMENT, SINGLE AXLES P_t = 2.5

Axle Load		D – Slab Thickness – inches					
Kips	kN	6	7	8	9	10	11
2	8.9	0.0002	0.0002	0.0002	0.0002	0.0002	0.0002
4	17.8	0.003	0.002	0.002	0.002	0.002	0.002
6	26.7	0.01	0.01	0.01	0.01	0.01	0.01
8	35.6	0.04	0.04	0.03	0.03	0.03	0.03
10	44.5	0.10	0.09	0.08	0.08	0.08	0.08
12	53.4	0.20	0.19	0.18	0.18	0.18	0.17
14	62.3	0.38	0.36	0.35	0.34	0.34	0.34
16	71.2	0.63	0.62	0.61	0.60	0.60	0.60
18	80.1	1.00	1.00	1.00	1.00	1.00	1.00
20	89.0	1.51	1.52	1.55	1.57	1.58	1.58
22	97.9	2.21	2.20	2.28	2.34	2.38	2.40
24	106.8	3.16	3.10	3.23	3.36	3.45	3.50
26	115.7	4.41	4.26	4.42	4.67	4.85	4.95
28	124.6	6.05	5.76	5.92	6.29	6.61	6.81
30	133.4	8.16	7.67	7.79	8.28	8.79	9.14
32	142.3	10.81	10.06	10.10	10.70	11.43	11.99
34	151.2	14.12	13.04	12.94	13.62	14.59	15.43
36	160.1	18.20	16.69	16.41	17.12	18.33	19.52
38	169.0	23.15	21.14	20.61	21.31	22.74	24.31
40	177.9	29.11	26.49	25.65	26.29	27.91	29.90

TABLE 51

TRAFFIC EQUIVALENCE FACTORS, RIGID PAVEMENT TANDEM AXLES P_t = 2.5

Axle Load		D – Slab Thickness – inches					
Kips	kN	6	7	8	9	10	11
10	44.5	0.01	0.01	0.01	0.01	0.01	0.01
12	53.4	0.03	0.03	0.03	0.03	0.03	0.03
14	62.3	0.06	0.05	0.05	0.05	0.05	0.05
16	71.2	0.10	0.09	0.08	0.08	0.08	0.08
18	80.1	0.16	0.14	0.14	0.13	0.13	0.13
20	89.0	0.23	0.22	0.21	0.21	0.20	0.20
22	97.9	0.34	0.32	0.31	0.31	0.30	0.30
24	106.8	0.48	0.46	0.45	0.44	0.44	0.44
26	115.7	0.64	0.64	0.63	0.62	0.62	0.62
28	124.6	0.85	0.85	0.85	0.85	0.85	0.85
30	133.4	1.11	1.12	1.13	1.14	1.14	1.14
32	142.3	1.43	1.44	1.47	1.49	1.50	1.51
34	151.2	1.82	1.82	1.87	1.92	1.95	1.96
36	160.1	2.29	2.27	2.35	2.43	2.48	2.51
38	169.0	2.85	2.80	2.91	3.04	3.12	3.16
40	177.9	3.52	3.42	3.55	3.74	3.87	3.94
42	186.8	4.32	4.16	4.30	4.55	4.74	4.86
44	195.7	5.26	5.01	5.16	5.48	5.75	5.92
46	204.6	6.36	6.01	6.14	6.53	6.90	7.14
48	213.5	7.64	7.16	7.27	7.73	8.21	8.55

Source: AASHTO Interim Guide, p. 109. (3)

TABLE 52

TRAFFIC EQUIVALENCE FACTORS, FLEXIBLE PAVEMENT, SINGLE AXLES P_t = 2.5

Axle Load		Structural Number, SN					
Kips	kN	1	2	3	4	5	6
2	8.9	0.0004	0.0004	0.0003	0.0002	0.0002	0.0002
4	17.8	0.003	0.004	0.004	0.003	0.003	0.002
6	26.7	0.01	0.02	0.02	0.01	0.01	0.01
8	35.6	0.03	0.05	0.05	0.04	0.03	0.03
10	44.5	0.08	0.10	0.12	0.10	0.09	0.08
12	53.4	0.17	0.20	0.23	0.21	0.19	0.18
14	62.3	0.33	0.36	0.40	0.39	0.36	0.34
16	71.2	0.59	0.61	0.65	0.65	0.62	0.61
18	80.1	1.00	1.00	1.00	1.00	1.00	1.00
20	89.0	1.61	1.57	1.49	1.47	1.51	1.55
22	97.9	2.48	2.38	2.17	2.09	2.18	2.30
24	106.8	3.69	3.49	3.09	2.89	3.03	3.27
26	115.7	5.33	4.99	4.31	3.91	4.09	4.48
28	124.6	7.49	6.98	5.90	5.21	5.39	5.98
30	133.4	10.31	9.55	7.94	6.83	6.97	7.79
32	142.3	13.90	12.82	10.52	8.85	8.88	9.95
34	151.2	18.41	16.94	13.74	11.34	11.18	12.51
36	160.1	24.02	22.04	17.73	14.38	13.93	15.50
38	169.0	30.90	28.30	22.61	18.06	17.20	18.98
40	177.9	39.26	35.89	28.51	22.50	21.08	23.04

TABLE 53

TRAFFIC EQUIVALENCE FACTORS, FLEXIBLE PAVEMENT, TANDEM AXLES P_t = 2.5

Axle Load		Structural Number, SN					
Kips	kN	1	2	3	4	5	6
10	44.5	0.01	0.01	0.01	0.01	0.01	0.01
12	53.4	0.02	0.02	0.02	0.02	0.01	0.01
14	62.3	0.03	0.04	0.04	0.03	0.03	0.02
16	71.2	0.04	0.07	0.07	0.06	0.05	0.04
18	80.1	0.07	0.10	0.11	0.09	0.08	0.07
20	89.0	0.11	0.14	0.16	0.14	0.12	0.11
22	97.9	0.16	0.20	0.23	0.21	0.18	0.17
24	106.8	0.23	0.27	0.31	0.29	0.26	0.24
26	115.7	0.33	0.37	0.42	0.40	0.36	0.34
28	124.6	0.45	0.49	0.55	0.53	0.50	0.47
30	133.4	0.61	0.65	0.70	0.70	0.66	0.63
32	142.3	0.81	0.84	0.89	0.89	0.86	0.83
34	151.2	1.06	1.08	1.11	1.11	1.09	1.08
36	160.1	1.38	1.38	1.38	1.38	1.38	1.38
38	169.0	1.75	1.73	1.69	1.68	1.70	1.73
40	177.9	2.21	2.16	2.06	2.03	2.08	2.14
42	186.8	2.76	2.67	2.49	2.43	2.51	2.61
44	195.7	3.41	3.27	2.99	2.88	3.00	3.16
46	204.6	4.18	3.98	3.58	3.40	3.55	3.79
48	213.5	5.08	4.80	4.25	3.98	4.17	4.49

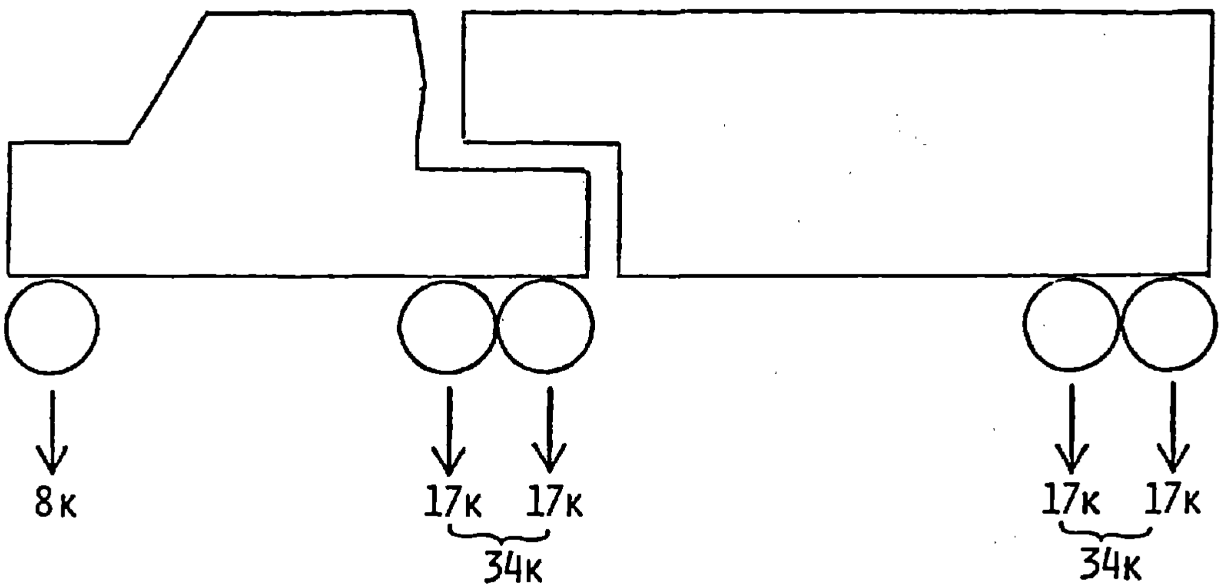
Source: <u>AASHTO Interim Guide</u>, p. 65. (3)

FIGURE 51

EXAMPLE DETERMINATION OF 18-KIP SINGLE AXLE EQUIVALENCY FACTOR FOR A VEHICLE

($SN = 4.0$)

($p_t = 2.5$)



(Equivalency Factors from Tables 52 and 53)

Single Axle @ 8 Kips:		0.0400
Tandem Axles @ 34 Kips:	2 x 1.11	2.2200
Total Vehicle Axle Equiv		2.2600

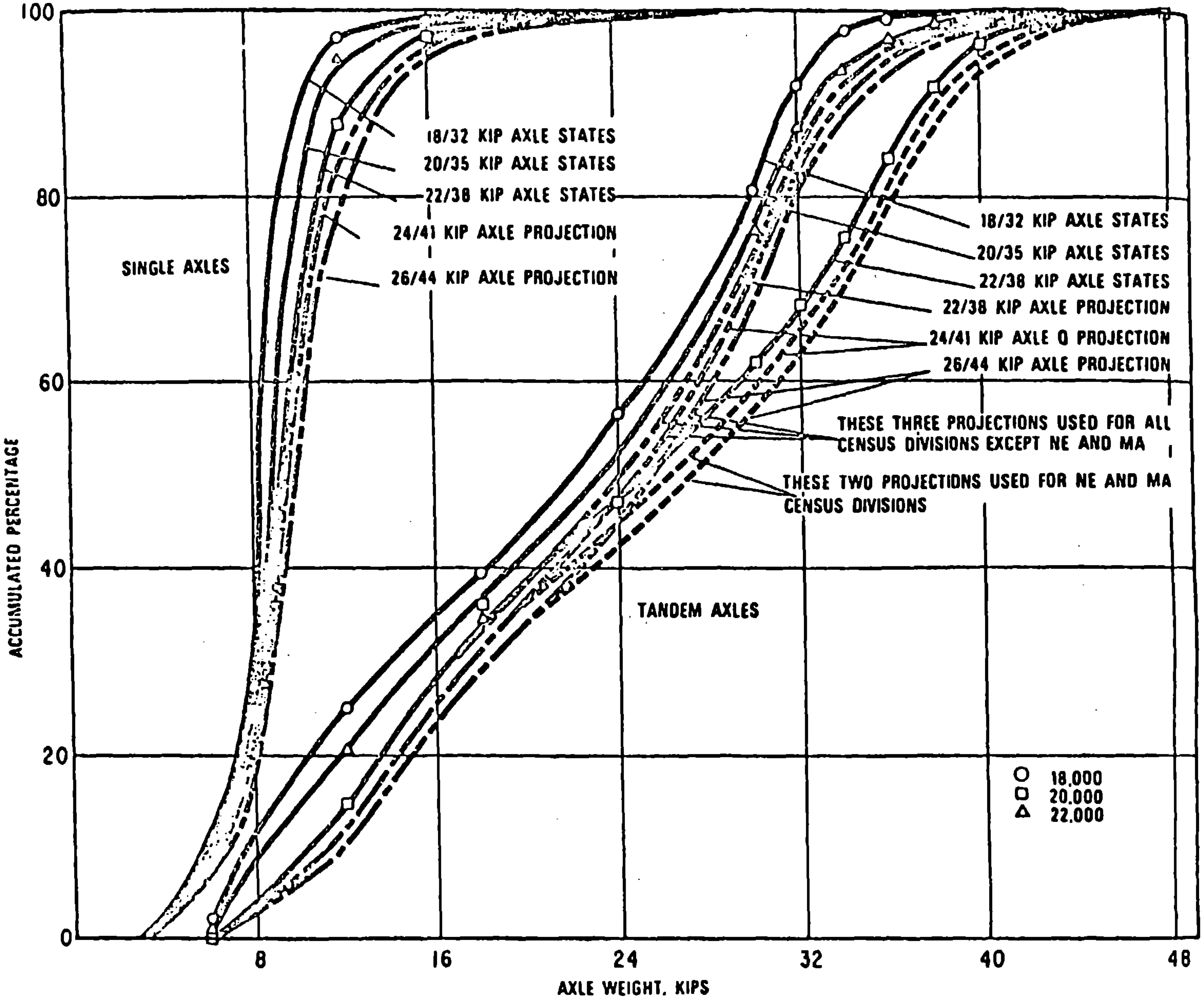


FIGURE 52

EXAMPLES OF CURVES SHOWING DISTRIBUTION OF AXLE WEIGHT BY VEHICLE CLASS FROM A TRUCK WEIGHT STUDY (SOLID CURVES). Source: NCHRP Report 141, P. 68 (49)

TABLE 54

CALCULATION OF DAILY EQUIVALENT LOAD APPLICATIONS

	1970								1990							
	PRESENT LIMITS				PROPOSED LIMITS				PRESENT LIMITS				PROPOSED LIMITS			
VEHICLE CLASS	ADT	DAILY PAYLOAD (LB)	AVG. E18-KIP LOAD APPLI-CATIONS	DAILY ELA	ADT	DAILY PAYLOAD (LB)	AVG. E18-KIP LOAD APPLI-CATIONS	DAILY ELA	ADT	DAILY PAYLOAD (LB)	AVG. E18-KIP LOAD APPLI-CATIONS	DAILY ELA	ADT	DAILY PAYLOAD (LB)	AVG. E18-KIP LOAD APPLI-CATIONS	DAILY ELA
A	B	C	D	E	F	G	H	I	J	K	L	M	N	O	P	Q
1. *Passenger vehicles:*																
Cars	1,850		0.0008	1.480	1,850		0.0008	1.480	3,900		0.0008	3.120	3,900		0.0008	3.120
Motorcycles	10		—*		10		—*	1.480	3,900		—*				—*	
Buses, comm.	12		0.2000	2.400	12		0.2000	2.400	25		0.2000	5.000	25		0.2000	5.000
Buses, school	8		0.3000	2.400	8		0.3000	2.400	10		0.3000	3.000	10		0.3000	3.000
Subtotal	1,880			6.280	1,880			6.280	3,985			11.120	3,985			11.120
2. *Single-unit trucks:*																
Panel, pickups	140		0.0020	0.280	140		0.0020	0.280	300		0.0020	0.600	300		0.0020	0.600
Other 4-tired	10		0.0100	0.100	10		0.0100	0.100	25		0.0100	0.250	25		0.0100	0.250
2D	120	481,200	0.1494	17.928	108	481,200	0.1990	21.691	220	952,820	0.1494	32.868	205	952,280	0.1990	40.795
3A	30	296,580	0.3062	9.186	29	296,580	0.4052	11.751	55	576,345	0.3062	16.841	54	576,345	0.4052	21.881
Subtotal	300			27.494	288			33.822	600			50.659	584			63.526
3. *Tractor semi-trailer combs.:*																
2-S1	25	163,100	0.3744	9.360	21	163,100	0.5049	10.603	40	287,040	0.3744	14.976	35	287,040	0.5049	17.672
2-S2	56	723,072	0.6137	34.367	53	723,072	0.8095	42.904	110	1,590,710	0.6137	67.507	109	1,590,710	0.8095	88.236
3-S2	104	2,351,544	0.7178	74.651	96	2,351,544	0.9546	91.742	350	9,021,950	0.7178	251.230	344	9,021,950	0.9546	328.382
3-S1-2	10															
Subtotal	195			118.378	170			145.249	500			333.713	488			434.290
4. *Truck and full trailer combs.:*	8															
Total	2,383			152.152	2,338			185.351	5,085			395.492	5,057			508.936

* Negligible.

Source: NCHRP Report 141, p. 163. (49)

use for high-volume routes within a particular region or state, there is no reason why those factors could not be employed within the EAROMAR analysis.*

Passenger Car Equivalents. Vehicles such as heavy trucks can influence traffic flow through their size and relative speed, serving to reduce the capacity of a highway under certain conditions. This capacity reduction is measured by the effective number of passenger cars displaced by the vehicle, whether physically (due to vehicle size) or operationally (due to vehicle speed). The units of capacity reduction are termed the vehicle's passenger car equivalent, or "pce".

The passenger car equivalent of a heavy truck can vary widely depending upon the road geometry and surrounding traffic volume and composition. On level grades or downgrades truck speeds can match those of passenger cars, and the passenger car equivalent of a truck will be about 2 due simply to its size. However, on upgrades truck speed is dependent upon the steepness and length of grade, as well as the number of lanes; and the resulting pce value may range from 3 to more than 20.

Unfortunately, no well-defined relationship exists to predict passenger car equivalent as a function of grade or number of lanes. The problem is complicated by the fact that the passenger car equivalent is influenced also by the percentage of heavy trucks in the traffic stream, the distribution of traffic among lanes, driver behavior toward maneuvering, and any psychological intimidation caused by the presence of trucks.

The Highway Capacity Manual (3) reports very limited research in this area, restricted primarily to conditions observed under level of service B. Based upon these observations a set of values for passenger car equivalents has been developed as shown in Table 55. This table is intended for road performance estimates on specific sections of highway (as opposed to long sections encompassing a range of geometric features), and is therefore appropriate to the fairly detailed road descriptions in the EAROMAR analysis discussed in Chapter 2. However, in tabular form the data are rather clumsy to use, and a more streamlined analytic approach was sought.

In considering the values shown in Table 55 , we felt that the variations due to percent of trucks and road grade were the most important ones to capture in the EAROMAR analysis. Adjustments between levels of service A-C and D-E appeared to be of second order; and,

* In general, Van Til found the greatest errors in those methods that had assumed the greatest degree of aggregate or averaged values. Hence the need to confine use of such values to road, vehicle, and seasonal conditions similar to those employed in deriving the particular methods.

TABLE 55

PASSENGER CAR EQUIVALENTS OF TRUCKS ON FREEWAYS AND EXPRESSWAYS, ON SPECIFIC INDIVIDUAL SUBSECTIONS OR GRADES

GRADE (%)	LENGTH OF GRADE (MI)	PASSENGER CAR EQUIVALENT, E_T									
		LEVELS OF SERVICE A THROUGH C FOR:					LEVELS OF SERVICE D AND E (CAPACITY) FOR:				
		3% TRUCKS	5% TRUCKS	10% TRUCKS	15% TRUCKS	20% TRUCKS	3% TRUCKS	5% TRUCKS	10% TRUCKS	15% TRUCKS	20% TRUCKS
0–1	All	2	2	2	2	2	2	2	2	2	2
2	¼–½	5	4	4	3	3	5	4	4	3	3
	¾–1	7	5	5	4	4	7	5	5	4	4
	1½–2	7	6	6	6	6	7	6	6	6	6
	3–4	7	7	8	8	8	7	7	8	8	8
3	¼	10	8	5	4	3	10	8	5	4	3
	½	10	8	5	4	4	10	8	5	4	4
	¾	10	8	6	5	5	10	8	5	4	5
	1	10	8	6	5	6	10	8	6	5	6
	1½	10	9	7	7	7	10	9	7	7	7
	2	10	9	8	8	8	10	9	8	8	8
	3	10	10	10	10	10	10	10	10	10	10
	4	10	10	11	11	11	10	10	11	11	11
4	¼	12	9	5	4	3	13	9	5	4	3
	½	12	9	5	5	5	13	9	5	5	5
	¾	12	9	7	7	7	13	9	7	7	7
	1	12	10	8	8	8	13	10	8	8	8
	1½	12	11	10	10	10	13	11	10	10	10
	2	12	11	11	11	11	13	12	11	11	11
	3	12	12	13	13	13	13	13	14	14	14
	4	12	13	15	15	14	13	14	16	16	15
5	¼	13	10	6	4	3	14	10	6	4	3
	½	13	11	7	7	7	14	11	7	7	7
	¾	13	11	9	8	8	14	11	9	8	8
	1	13	12	10	10	10	14	13	10	10	10
	1½	13	13	12	12	12	14	14	13	13	13
	2	13	14	14	14	14	14	15	15	15	15
	3	13	15	16	16	15	14	17	17	17	17
	4	15	17	19	19	17	16	19	22	21	19
6	¼	14	10	6	4	3	15	10	6	4	3
	½	14	11	8	8	8	15	11	8	8	8
	¾	14	12	10	10	10	15	12	10	10	10
	1	14	13	12	12	11	15	14	13	13	11
	1½	14	14	14	14	13	15	16	15	15	14
	2	14	15	16	16	15	15	18	18	18	16
	3	14	16	18	18	17	15	20	20	20	19
	4	19	19	20	20	20	20	23	23	23	23

Source: Highway Capacity Manual 1965. Highway Research Board Special Report 87, p. 258. (3)

FIGURE 53

TRUCK FACTOR E_t VERSUS GRADE G

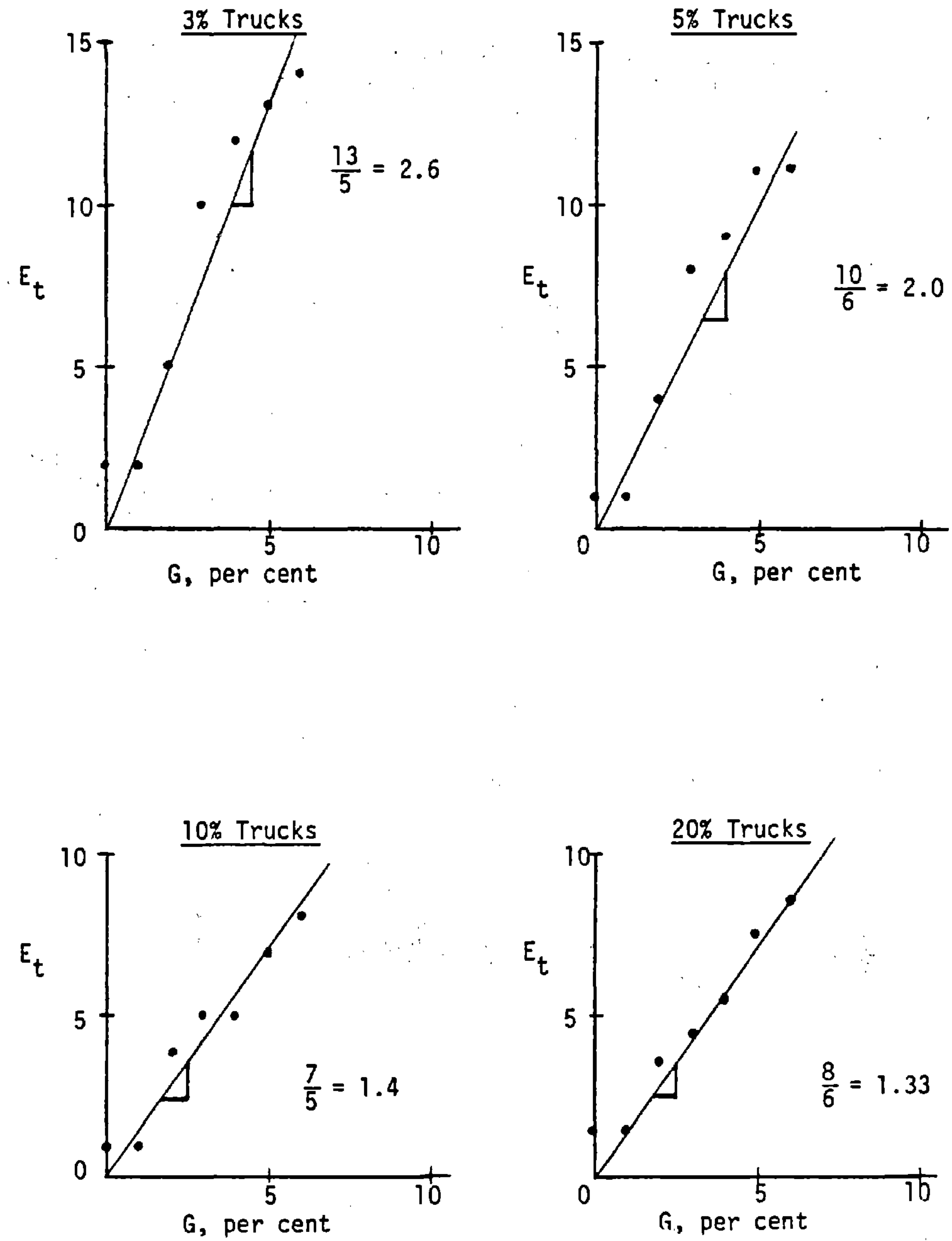
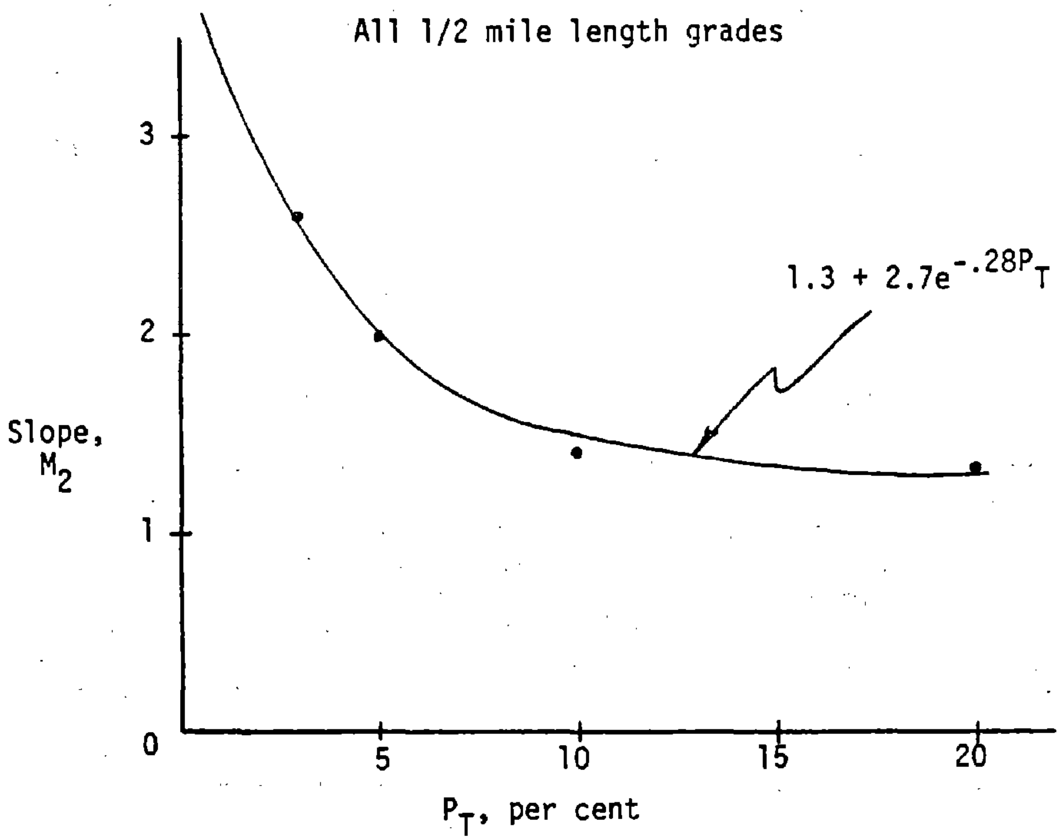


FIGURE 54

SLOPE OF LINEAR RELATIONSHIP (FIGURE 53)
VERSUS PERCENT OF TRUCKS P_t



for high-volume routes likely to be considered in a premium pavement analysis, the occurrence of a sustained grade of any significant length was felt to be improbable. Therefore, we concentrated simply on the data given in Table 55 for levels of service A-C, and assumed a representative length of 1/2 mile for all grades shown.

The variation of passenger car equivalent with grade (assuming 1/2 mile length of grade throughout) is illustrated in Figure 53 for selected values of truck percentage. Note that the relationships are approximately linear, with the slope of the function decreasing with increasing truck percentage. This variation of slope vs. truck percentage is illustrated in Figure 54 ; it can be approximated by a negative exponential curve as shown.

The relationships embodied in Figure 53 and 54 can, therefore, be represented fairly simply in mathematical form by two equations:

$$E_T = mG \tag{228}$$

and

$$m = 1.3 + 2.7e^{-0.28P_T} \tag{229}$$

where:

E_T = passenger car equivalent, pce

G = grade, per cent*

P_T = percentage of trucks, per cent*

A comparison of the predicted passenger car equivalents versus those given for the corresponding grade and percentage of trucks in the <u>Highway Capacity Manual</u> is presented in Table 56. The agreement is quite good across the entire range of grades and truck percentages considered; the maximum errors observed are approximately 20-25%, and these occur infrequently. Furthermore, given the fact that the <u>Manual</u> values themselves are approximations, equations (228) and (229) are considered to be quite adequate for use within the EAROMAR system.

One further refinement need be added: as a simple linear relationship equation (228) passes through the origin, predicting a zero equivalency factor at a zero grade. In reality this is not true, since passenger cars and other relatively mobile vehicles would have a passenger car equivalency of 1.0 on level roads or downgrades, while heavier vehicles would probably have a value of between 1.0 and 2.0. Therefore a minimum or floor is needed for the function; this minimum will be provided by the user as part of the description of each vehicle type.

* Expressed as the numerical percentage, not a decimal number.

TABLE 56

COMPARISON OF
COMPUTED vs. TABULATED
TRUCK pce FACTORS

Grade G, %	Percent of Trucks P_T, %	Truck Factor E_T Computed	Truck Factor E_T from HCM*
2	3	4.9	5
	5	3.9	4
	10	2.9	4
	15	2.7	3
	20	2.6	3
3	3	7.4	10
	5	5.9	8
	10	4.4	5
	15	4.0	4
	20	3.9	4
4	3	10.0	12
	5	7.9	9
	10	5.9	5
	15	5.4	5
	20	5.2	5
5	3	12.3	13
	5	9.8	11
	10	7.3	7
	15	6.7	7
	20	6.5	7

*Highway Capacity Manual (3)

Having derived a simple, closed-form model to relate passenger car equivalents to road and traffic characteristics, we are now ready to consolidate elements of their treatment within the EAROMAR analysis. For each vehicle type declared one will specify its passenger car equivalent on level roads as some number greater than or equal to 1.0. Passenger cars, pickup trucks, and light trucks would typically assume a value of 1.0; buses, about 1.6; and combination trucks and other heavy vehicles, 2.0 or more. All vehicles having pce greater than or equal to 2.0 would be considered as contributing to the "percentage of trucks" used for capacity purposes.

During the simulation of traffic operations on a given road section, the system will compute the composition of travel demand by vehicle type within each hour of the day; the percentage of trucks will then be determined based upon the pce specified above. Road grade is known from the road characteristics provided in Chapter 2. If the grade is algebraically less than 2.0 per cent the passenger car equivalent will be taken directly from the value provided by the user for each vehicle type. If the grade is positive, and greater than 2.0 per cent, equations (228) and (229) will be used instead to estimate passenger car equivalent E_t for each heavy vehicle type. The equivalent total travel demand, in passenger car units, will then be computed as:

$$E = \sum_{\substack{\text{all}\\ \text{vehicle}\\ \text{types } i}} Q_i \times pce_i = \sum_{\substack{\text{all}\\ \text{vehicle}\\ \text{types } i}} AADT \times SEASON \times P_i \times pce_i \qquad (230)$$

where:

E = total hourly passenger car equivalents

Q_i = hourly demand by vehicle type, veh/hr

pce_i = passenger car equivalent by vehicle type

P_i = percentage of adjusted AADT by vehicle type and hour of day.

The source of hourly percentage of AADT by vehicle type, P_i, will be explained in a later section where we consolidate descriptions of demand by trip purpose and vehicle type.

Unit Costs of Consumables. In the course of their operation, vehicles consume fuel, oil, and tires at rates dependent upon the characteristics of the vehicles themselves and road geometric and operational conditions. Predictions of consumption rates and costs are handled by a set of models to be discussed in Chapter 6. As part of the vehicle description, though, unit costs must be provided for each resource category: fuel, in dollars per gallon; oil, in dollars per quart; and tires, in dollars per tire. Costs must reflect products appropriate to the vehicle in question (e.g., fuel type; size of tires) for the initial year of the analysis period. These unit costs will then be inflated each year of the simulation according to the procedures described in Chapter 2. Fuel costs are inflated by the "fuel" inflation rate, while tires are assumed to increase at the general rate of inflation.

Emissions Factor. The vehicle air pollution emissions models presented in Chapter 6 are based upon passenger car emission levels for uniform speeds, for speed changes, and for queueing. These predictions will then be corrected for each vehicle type via a vehicle emission level factor provided by the user, defined to be:

$$\text{Vehicle Emission Level Factor} = \frac{\text{Level of Vehicle Emissions}}{\text{Passenger Car Emissions Under Equivalent Operating Conditions}} \qquad (231)$$

CONSOLIDATION OF TRIP AND VEHICLE CLASSIFICATIONS

Having expanded upon the two components of our description of traffic composition — trip purpose and vehicle type — we would now like to consolidate these two sets of information to unify our representation of demand characteristics. Merging these two sets of data will enable one to completely stratify the traffic stream at any given location and time, and more importantly, to do so in an internally consistent way.

Structure of Relationship. Recall from our previous discussions that declarations of both trip purpose and vehicle type are intended to be at the control of the user. It stands to reason, then, that the pairing of trip categories with vehicle classifications must likewise be by user command, and need not (indeed cannot) be defined by prior convention within the EAROMAR system.

This pairing is accomplished in the EAROMAR analysis through a percentage distribution of vehicle types projected within each trip purpose. For instance, a declaration of work commuter trips could be followed with the specification, say, of "95 per cent autos, 5 per cent pickup trucks" constituting the commuter vehicle mix. Likewise, commercial freight could comprise "40 per cent combinations, 60 per cent single unit trucks"; school trips might have a mix of "85 per cent buses, 15 per cent autos." Note that since the percentages apply to vehicles within each trip category, and not to the traffic demand in total, the percentages must sum to 100 per cent for each trip purpose considered.

Several comments need to be made regarding this procedure. The first concerns consistency among the different elements of data involved. Simply put, the vehicles assigned under each trip category must be drawn from the pool of vehicles previously declared to the system (via the vehicle name or identifier). This requirement insures that all data associated with each vehicle class (e.g., weight, equivalency factors, etc.) will be available when required by the system in assembling the characteristics of the total traffic stream.

Second, the relationship between trip purpose and vehicle type automatically defines and establishes internal links within the EAROMAR system between (1) all information structured by the trip purpose, and (2) all information associated with the vehicle type. These links are illustrated in Figure 55 for the work commuting example earlier. Recall that trip purpose definition includes among other factors seasonal, daily and hourly variations in demand; on the other hand, vehicle type description includes, for instance, pavement and capacity equivalencies. The association between the two enables the system to construct, for example, seasonal, daily, and hourly variations in pavement axle equivalencies, or in numbers of passenger car equivalents. When aggregated among all trip categories (and hence all vehicle classes), these associations become very important in the simulation of road operation, pavement deterioration and maintenance scheduling.

Third, there are no restrictions on the ways in which vehicles may be related by purpose. Again considering our examples earlier, both commuter trips and school trips encompass use of passenger autos. The implication is that both of these trip purposes will share access to the description of the automobile vehicle data, as shown in Figure 56. Moreover, the portion of total demand attributable to automobile travel may now be estimated as the superposition of: (1) the demand generated by work commuter trips; plus (2) the demand arising from school travel; plus (3) the demand patterns of any other trip categories in which auto is designated as a vehicle choice.

The process of superposition is shown schematically in Figure 56, and provides the user considerable versatility and power in structuring travel demand characteristics for various user impact policy studies. For example, Table 57 illustrates one potential way of organizing commercial trucking demand within the EAROMAR analysis. Through interaction of the trip categories of local and intercity frieght (partitioned by general commodity class), and the vehicle mixes within each (comprising different gasoline- and diesel-powered vehicles), one would be able to assess user impacts in a market sense, a vehicle sense, and a fuel consumption sense. Similar breakdowns could be made for other trips or vehicles if warranted within the analysis.

Values of Travel Time. Past research has shown that values of travel time are correlated with trip purpose; further information in this regard is presented in Chapter 6. Nevertheless, it was felt unreasonable to assume that such values would always be uniform within a given trip category, particularly in light of the independence with which one may declare trip categories within the EAROMAR analysis. Therefore, it was decided that values of travel time would be most appropriately provided to the system by vehicle class within a trip category.

TABLE 57

EXAMPLE USE OF TRIP PURPOSE AND VEHICLE CLASS SPECIFICATIONS FOR TRUCKING

PURPOSE	VEHICLES
LOCAL-SERVICE	GAS-PICKUP
	GAS-PANEL
LOCAL-GOODS	GAS-SUT
	DIESEL-SUT
INTERCITY-DURABLES	GAS-SUT
	DIESEL-SUT
	GAS-COMBINATION
	DIESEL-COMBINATION
INTERCITY-AGRICULTURAL	GAS-SUT
	DIESEL-SUT
	GAS-COMBINATION
	DIESEL-COMBINATION
INTERCITY-SERVICE	GAS-PANEL
	GAS-PICKUP

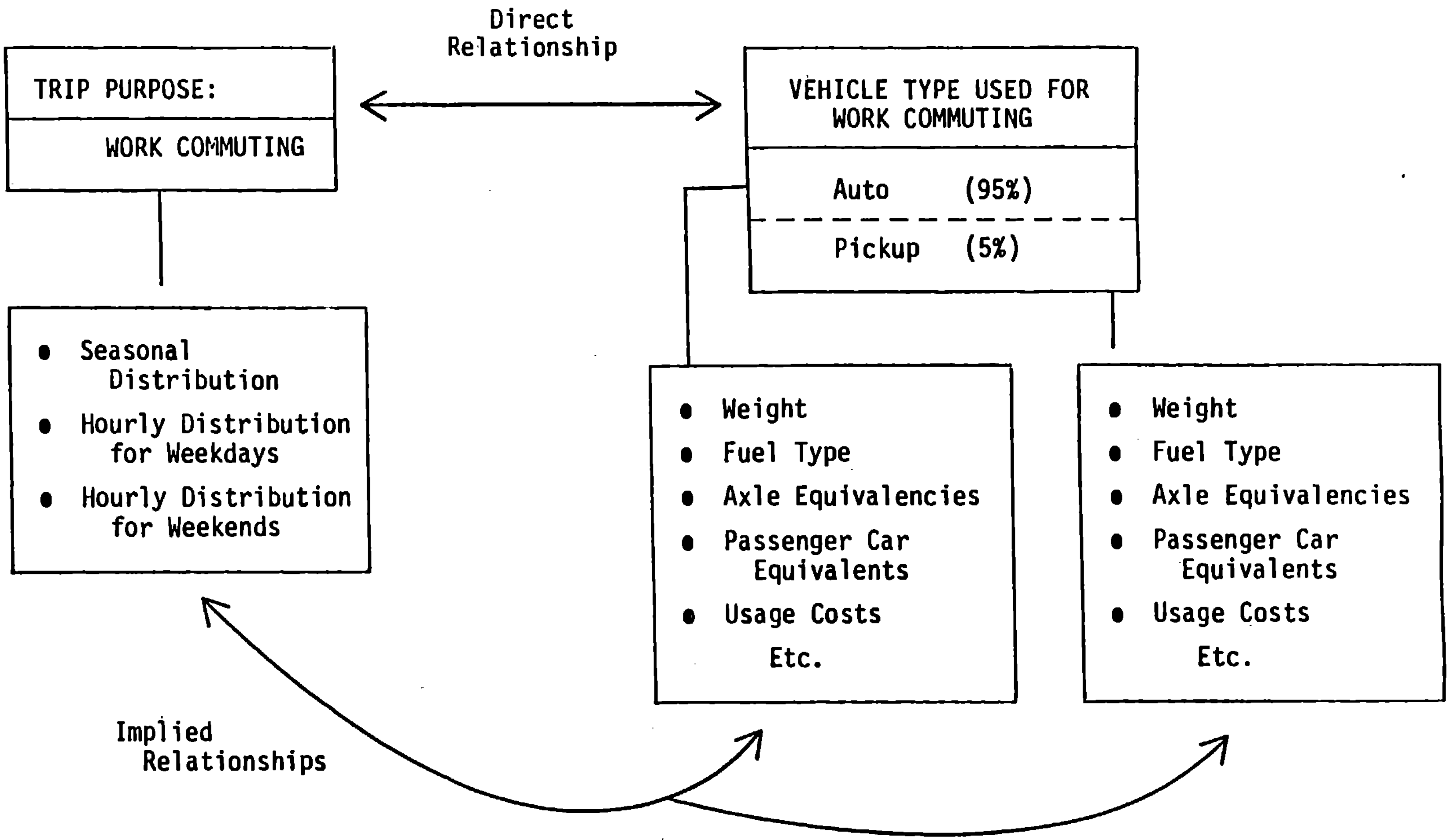


FIGURE 55.

RELATIONSHIP BETWEEN TRIP PURPOSE AND VEHICLE TYPE SPECIFICATIONS

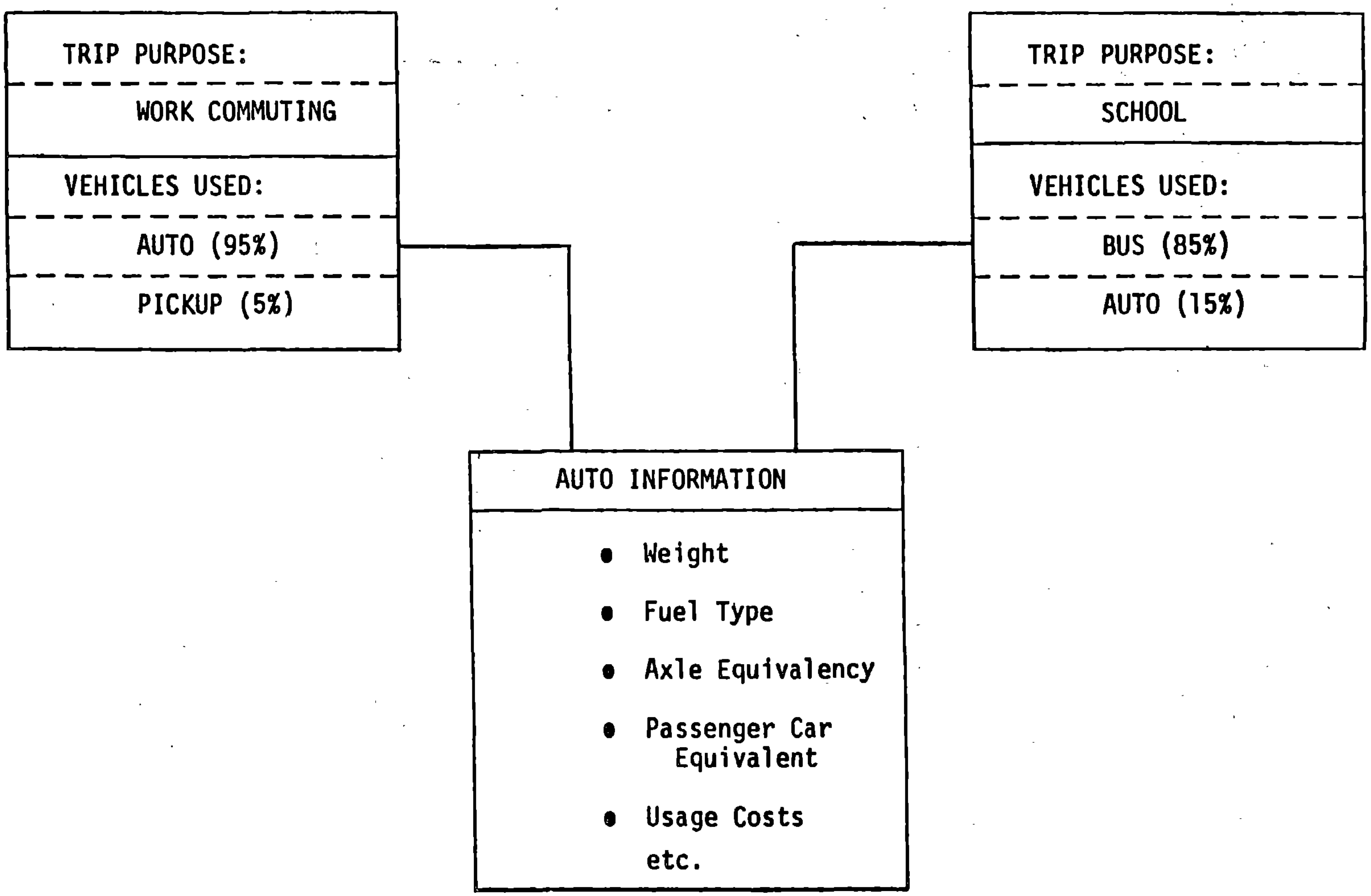


FIGURE 56

SHARING OF VEHICLE DATA BY MULTIPLE TRIP PURPOSE

Figure 57 gives two examples of travel time value specifications. The first example illustrates the case where time values may be determined simply by differing wage scales within broadly defined trip categories. It has been assumed that the wage levels are correlated in a fairly consistent way with vehicle class driven.

The second example, however, illustrates a somewhat different and more imaginative use of the convention. The proposed EAROMAR analysis does not incorporate a breakdown of travel time values by motorist income, for several reasons: (1) there is disagreement among the studies reviewed as to how time values are specifically related to income; (2) it was felt that distributions of income levels for users on particular routes would be difficult to obtain; and (3) the sensitivity of time valuation to income might be overshadowed by uncertainty in the basic value of time concept itself.

Nevertheless, there may arise instances where stratification of time values by income is desired. The second example in Figure 57 corporates three income ranges for passenger car travel, using vehicle type as a proxy for the user income parameter. It is assumed that the technological and economic specifications of three "types" of passenger cars are the same, although this is certainly not a requirement. As with several prior examples, we do not necessarily recommend this approach in any specific analysis. The example is intended to show, however, that through imaginative use of the input capabilities provided by the system, one may construct an analysis according to a structure and level of detail deemed most appropriate for the problem at hand.

Traffic Sets

In Chapter 2, we discussed the desirability of varying traffic descriptions within a series of strategies, to account for the uncertainty in predicting changes in either short-term effects (e.g., seasonal, daily, or hourly distributions) or the composition of the traffic stream (by trip purpose or vehicle type). For convenience in specifying these temporal or spatial changes, it is useful to organize the several data that may be affected into logical organizations, and to refer to each collection of data simply by a name. These collections of data are termed traffic sets within the EAROMAR analysis.

Each traffic set comprises one complete specification of the following information discussed in previous sections:

- Seasonal adjustment to AADT
- Distributions of AADT by trip purpose, type of day (weekday or weekend), and hour of day
- Distributions of vehicle type within each trip purpose

FIGURE 57.

EXAMPLES OF TRAVEL TIME SPECIFICATIONS

Example 1

⋮

PURPOSE: SERVICE-TRUCKING

VEHICLES:

PICKUP	TIME VALUE	= V_1
PANEL	TIME VALUE	= V_2

PURPOSE: FREIGHT-TRUCKING

VEHICLES:

SUT	TIME VALUE	= V_3
COMBINATION	TIME VALUE	= V_4

⋮

Example 2

⋮

PURPOSE: WORK COMMUTING

VEHICLES:

LOW-INCOME CAR	TIME VALUE	= V_1
MODERATE-INCOME CAR	TIME VALUE	= V_2
HIGH-INCOME CAR	TIME VALUE	= V_3

⋮

• Values of travel time by vehicle type and trip purpose.

A number of traffic sets may be defined within an analysis. Traffic sets may differ from one another either in only one of the elements above, in several, or in all elements specified. An example will help to illustrate their practicality and convenience in modeling several diverse but typical situations.

Table 58 identifies three traffic sets defined for a hypothetical problem. For simplicity, only those portions that change from one set to another have been shown. In fact, each set must also include all other elements (seasonal, daily, hourly distributions, etc.) identified above, and for this problem we assume that these other items are the same among the three sets listed.

The sets represent changes in relative traffic composition across both trip purpose and vehicle mix. First, each set denotes successively higher relative percentages of commuting trips within the traffic stream, increasing from 25% to 35%. Second, Set - 2 defines a simultaneous change in composition of commercial traffic, reflecting a greater proportion of single unit trucks than shown in Set - 1. Finally, Set - 3 introduces light trucks into two of the trip categories represented exclusively by auto in Set - 2. Thus, the traffic sets taken collectively define some type of emerging pattern based presumably on independent assessments of future regional economic development and changing preferences in driver behavior.

The application of these sets, through the strategy specifications described in Chapter 2, together with other information necessary to describe travel demand, is shown schematically in Table 59 This table is essentially an extension of the AADT specifications formulated earlier in Table 44; the inclusion of traffic set data serves now to vary the composition of the traffic stream over space and time as well.

Several points are worthy of note. First, it is clear that the several components of demand are varying spatially and temporally; what should be stressed is that these variations may occur in whatever combinations desired to represent the problem at hand. For example, in 1980, the same traffic set is superimposed upon different levels of AADT along the length of the road. The implication is that the demand levels projected on each of the three road segments will have similar composition (in terms of trip purpose and vehicle type), cyclical variations, and peaking characteristics. In later years of the analysis period, growth rates and traffic sets can be manipulated in concert with one another as shown, to vary the effects of these superimpositions at will.

Second, the projection of changes in demand projection, and the locations over which they occur, are arbitrary. Table 59 shows changes at five year intervals, with milepost segments remaining the same for

TABLE 58

EXAMPLES OF TRAFFIC SET SPECIFICATION

TRAFFIC SET - 1

⋮

TRIP PURPOSES		VEHICLE DISTRIBUTIONS	
Commuting - - - -	25%	Auto - 100%	
Commercial - - -	10%	SUT - 50%	3-S2 - 50%
Personal/Social -	50%	Auto - 100%	
Other - - - - - -	15%	Auto - 50%	Light Truck - 50%

⋮

TRAFFIC SET - 2

⋮

TRIP PURPOSES			
Commuting - - - -	30%	Auto - 100%	
Commercial - - -	10%	SUT - 60%	3-S2 - 40%
Personal/Social -	50%	Auto - 100%	
Other - - - - - -	10%	Auto - 50%	Light Truck - 50%

⋮

TRAFFIC SET - 3

⋮

TRIP PURPOSES			
Commuting - - - -	35%	Auto - 95%	Light Truck - 5%
Commercial - - -	10%	SUT - 60%	3-S2 - - - - 40%
Personal/Social -	40%	Auto - 90%	Light Truck - 10%
Other - - - - - -	15%	Auto - 60%	Light Truck - 40%

TABLE 59

EXAMPLE OF COORDINATED TRAFFIC DESCRIPTIONS AND GROWTH

YEAR	ROADWAY MILEPOST SECTION	INITIAL AADT	GROWTH	TRAFFIC SET
1980	0 - 5 5 - 7 7 - 10	70,000 75,000 85,000	+ 2000 AADT + 1000 AADT + 1000 AADT	SET-1 SET-1 SET-1
1985	0 - 5 5 - 7 7 - 10	-- -- --	+ 1000 AADT -- --	-- -- SET-2
1990	0 - 5 5 - 7 7 - 10	-- -- --	0 0 0	SET-2 SET-2 SET-3
1995	0 - 5 5 - 7 7 - 10	-- -- --	-- -- --	SET-3 SET-3 --

each year. In general, however, any year within the analysis period could have been specified;* furthermore, updates of any of the demand components would not have to be confined to milepost segments 0-5, 5-7, and 7-10, but could be defined in terms of any other boundaries (e.g., 0-6, 6-10).

Third, the highway demand at any particular time is the product of the currently defined AADT and the currently defined traffic set. "Currently defined" in this context is taken to mean the following. AADT and traffic set data are assumed to be valid within the year in which they are declared. For example, in 1980 road segment "0-5" will be projected to have 70,000 AADT, and traffic composition and cyclical demand variations as described by Set - 1. On the other hand, rate of traffic growth is assumed to occur from the year in which it is first declared to the following year, and therefore its effects are not tallied within the analysis until that second year. Again looking at segment 0-5 in 1980, the applicable rate of growth is linear at +2000 AADT per year. However, this growth will not be registered until 1981, when AADT will have been simulated to climb from 70,000 to 72,000. To illustrate these points more completely, selected demand projection resulting from the combined specifications in Tables 58 and 59 are compiled in Table 60.

Travel Diversion Effects

One final issue remains to be discussed concerning representation of travel demand: the diversion of vehicles to alternate routes, due to heightened congestion caused by road maintenance or rehabilitation work-zones. In contrast to the development of demand volume and composition already discussed, such diversion patterns are not part of the general literature on traffic demand; rather, they are specific to the particular network under consideration, the demand patterns on that network, and the work-zone size and duration of the occupancy. However, since such road occupancy impacts are germane to the EAROMAR analysis, it was felt that they sould be mentioned in this report.

The magnitudes of such diversions may be estimated from actual traffic counts for high-volume routes under repair. Tables 61 and 62 report the effects of road reconstruction on two major Illinois expressways in 1966 and 1971. Overall two-way daily volumes were reduced by 20-45 per cent during repair or reconstruction, with somewhat higher reductions exhibited in the directional peak flows.

Clearly, such potential reductions can have a marked impact upon demand estimates, and would therefore be desirable to include within

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* A project could also have been specified, at whose completion the traffic set or growth pattern would be implemented. See the discussion of AADT variation over time earlier in this chapter.

TABLE 60

TRAFFIC CHARACTERISTICS RESULTING FROM SPECIFICATIONS IN TABLES 58 AND 59

YEAR	ROADWAY MILEPOST SECTION	COMPUTED AADT	DAILY COMMUTING TRIPS*	DAILY TRIPS BY AUTO**
1980	0-5	70,000	17,500	57,750
	5-7	75,000	18,750	61,875
	7-10	85,000	21,250	70,125
1985	0-5	80,000	20,000	66,000
	5-7	80,000	20,000	66,000
	7-10	90,000	27,000	76,500
1990	0-5	85,000	25,500	72,250
	5-7	85,000	25,500	72,250
	7-10	95,000	33,250	74,340
1995	0-5	85,000	29,750	66,515
	5-7	85,000	29,750	66,515
	7-10	95.000	33.250	74.340

* Illustrates distributions of AADT by trip purpose regardless of vehicle type used -- in this case, auto and pickup for commuting trips.

** Illustrates distributions of AADT by vehicle tvpe for pertinent trio purposes -- in this case. auto use for commuting, personal/ social and other trips.

TABLE 61

Traffic Volume Reductions During Reconstruction Work on Kennedy Expressway, October 1971, Cook County, Illinois*

EXPRESSWAY	AT	NORMAL COUNT, 24-HR (VEH)	VOLUME REDUCTION (%) STAGE I OVER-ALL	STAGE I AM PEAK	STAGE I PM PEAK	STAGE II OVER-ALL	STAGE II AM PEAK	STAGE II PM PEAK
Kennedy	Cumberland	130,000	40	50	50	25	40	40
	Cicero	125,000	35	35	45	25	25	45
	Ohio	210,000	40	40	30	45	35	50
Ryan	55th	230,000	35	—	—	45	—	—
Edens	Wilson	115,000	50	—	—	35	—	—
	Church	105,000	5	—	—	5	—	—
Eisenhower	Sacramento	190,000	0	—	—	+5	—	—
	East	125,000	0	—	—	+5	—	—

[a] Local lane closures on Ryan Expressway; Edens and Eisenhower Expressways open, in full service.
[b] Increase.

TABLE 62

Analysis of Traffic Demand During Reconstruction Work on Edens Expressway, July 1966, Cook County, Illinois*

EXPRESS-WAY	AT	TRAFFIC DEMAND [b] (VEH) BEFORE	DURING	DIFF.	RED. (%)
Edens	Foster Ave.	110,100	87,800	22,300	20
Kennedy	Pulaski	199,700	167,900	31,800	16
	Chicago Ave.	195,500	182,400	13,100	7

[a] Resurfacing on Edens with two lanes closed and four open at all times; one open lane reversed.
[b] 24-hr demand, 2-way.

*SOURCE: Reconditioning High-Volume Freeways in Urban Areas. NCHRP Synthesis of Highway Practice 25. (50)

the EAROMAR analysis. However, further investigation of the problem revealed several difficulties in incorporating these diversion effects within the analysis as it is now conceived.

At the heart of these difficulties lies the fact that the EAROMAR approach is structured at a link, rather than a network, level. Considerations of traffic distribution and diversion therefore cannot be modeled other than in a very approximate way. And the data available, such as those in Tables 61 and 62, are not sufficient to enable one to extend these approximations to general use.

The first problem is how to adjust these factors to be consistent with the more general treatment of travel demand already developed. For instance, seasonal, daily and hourly traffic variations are included in the analysis, and it is reasonable to assume that percentages of demand reduction will likewise vary over time. Yet, the data to support this contention are not available, nor is it clear how one would provide these variations to the analysis in a convenient way.

Second, it is also reasonable to assume that demand reduction due to maintenance congestion will occur primarily within some prescribed distance of the actual workzone limits — within an influence zone, if you will — but will be negligible outside this zone. While Table 61 hints at spatial variation in diversion percentage, it suggests no clues as to the total length of zone in which demand is affected. Determining this length requires knowledge of point-to-point travel desires of each motorist in the traffic stream, as well as estimates of their sensitivity to alternate route choice -- i.e., a network analysis of demand distribution. The problem is compounded by the fact that maintenance workzones move over time, and therefore the spatial variation in demand reduction may itself change rather frequently within the analysis.

Finally, assuming that the demand reduction comprises primarily diversion of traffic to alternate routes, one has no way of estimating revised user costs and travel times on these (perhaps overcrowded) alternates without resorting to a network analysis. If, somehow, demand reductions were effected within the analysis on the route under study, but account were not taken of the additional congestion, time, and operating costs on alternate routes, one could substantially underestimate the cost impact of lane closures during peak periods.

Thus, in volume reductions due to congestion caused by maintenance, we are faced with an aspect of travel demand that should be incorporated within the EAROMAR analysis, but cannot be at this time. Only by extending the EAROMAR framework to a network level can such effects be properly and generally accounted for. Research on a network formulation is beyond the scope of the present study, but is strongly recommended as a future area of work.

5.2 FREE-FLOW OPERATING SPEEDS

Introduction

Road operating characteristics —traffic speed, flow and density— are the primary determinants of the level of service provided by a road facility, and attendant travel costs incurred by its users. Values of these operational measures result from an equilibrium among transportation supply and demand, dynamic over both route length and time.

The next several sections concentrate upon the supply side of this balance as measured by road capacity (i.e., maximum practical flow). Appropriate relationships for the physical and operational contributions to capacity will be developed. Then, predictions of capacity will be combined with the treatment of demand from preceeding sections to estimate equilibrium conditions in terms of volume-capacity ratios.

Using established speed-flow relationships, volume-capacity ratios will be translated into predicted average free-flow operating speeds. Other road factors affecting average speeds, such as speed limits and pavement roughness, will also be discussed.

This portion of Chapter 5 will be limited to considering free-flow conditions (i.e., levels of service A through E). Analysis of congestion and queueing, and associated congested zone speeds and speed changes, will be presented later in the chapter, following discussions of traffic bottlenecks and road closures for maintenance and rehabilitation.

Capacity Relationships

The capacity relationships discussed below are taken from the *Highway Capacity Manual*. (3) Based upon analysis of a considerable body of empirical observations, these relationships are widely used throughout the US, and are familiar to most highway practitioners. Therefore, the emphasis below is on application of pertinent concepts within the EAROMAR analysis, rather than upon their background and derivation. Furthermore, although the *Highway Capacity Manual* is now being revised, current indications are that there will be no substantial changes expected in the basic approach to capacity estimates.

The "ideal" capacity for multilane roads with access control under uninterrupted conditions is normally taken to be 2000 passenger cars per hour. Several geometric and operational factors reduce this number to more practical limits, as identified in Table 63. To simplify capacity calculations, these separate adjustments have been incorporated within only three terms of a general capacity relationship:

$$C = 2000\ N\ W\ T \tag{232}$$

where: C = capacity (mixed vehicles per hour, total for one roadway in one direction)

TABLE 63

FACTORS CAUSING ADJUSTMENTS TO IDEAL UNINTERRUPTED FLOW VALUES

ROADWAY FACTORS

- Lane width
- Lateral clearance
- Shoulders
- Auxiliary lanes
  - Parking lanes
  - Speed change lanes
  - Turning and storage lanes
  - Weaving lanes
  - Truck climbing lanes
- Surface condition
- Alinement
- Grades

TRAFFIC FACTORS

- Trucks, Two-lane
  - Multilane
- Buses
- Lane distribution
- Variations in traffic flow
- Traffic interruptions

Source: Highway Capacity Manual 1965. Highway Research Board Special Report 87, page 109.

N = number of lanes in roadway

W = adjustment for lane width and side clearance (see Table 64)

T = truck factor at capacity

NUMBER OF LANES

Number of lanes is directly available to the EAROMAR system from the route descriptions provided by the user in Chapter 2. No further action is necessary.

LANE WIDTH AND SIDE CLEARANCE ADJUSTMENT

Although the lane width will also be provided in the route characteristics in Chapter 2, information on side clearance will not be called for. In lieu of providing distance to nearest obstruction for both sides of the road, it was felt simpler to have user provide the width and side clearance factor directly from Table 64 for each section of roadway. This information is provided with other "capacity" data within the route descriptions discussed in Chapter 2.

TRUCK FACTOR

The truck factor is determined automatically by the system for each hour simulated throughout the analysis period, using the truck pce factor E_T [from eq. (228)] and percent of trucks P_T in the following relationship:

$$T = 100 / (100 - P_T + E_T P_T) \qquad (233)$$

DEMAND-CAPACITY RATIO

The demand-capacity ratio is determined for each hour of operations simulated by the EAROMAR analysis by dividing total demand (in mixed vehicles per hour) by total capacity (computed above) for each roadway within the route under consideration; or

$$\frac{D}{C} = \frac{\text{Hourly demand computed by eq. (226)}}{\text{Hourly capacity computed by eq. (232)}} \qquad (234)$$

VOLUME-CAPACITY RATIO

Similarly, the volume-capacity ratio is estimated as:

$$\frac{V}{C} = \frac{\text{Hourly flow}}{\text{Hourly capacity}} \qquad (235)$$

Table 64

Combined Effect of Lane Width and Restricted Lateral Clearance on Capacity and Service Volumes of Divided Freeways and Expressways with Uninterrupted Flow

DISTANCE FROM TRAFFIC LANE EDGE TO OBSTRUCTION (FT)	ADJUSTMENT FACTOR,* W, FOR LANE WIDTH AND LATERAL CLEARANCE							
	OBSTRUCTION ON ONE SIDE OF ONE-DIRECTION ROADWAY				OBSTRUCTIONS ON BOTH SIDES OF ONE-DIRECTION ROADWAY			
	12-FT LANES	11-FT LANES	10-FT LANES	9-FT LANES	12-FT LANES	11-FT LANES	10-FT LANES	9-FT LANES
(*a*) 4-LANE DIVIDED FREEWAY, ONE DIRECTION OF TRAVEL								
6	1.00	0.97	0.91	0.81	1.00	0.97	0.91	0.81
4	0.99	0.96	0.90	0.80	0.98	0.95	0.89	0.79
2	0.97	0.94	0.88	0.79	0.94	0.91	0.86	0.76
0	0.90	0.87	0.82	0.73	0.81	0.79	0.74	0.66
(*b*) 6- AND 8-LANE DIVIDED FREEWAY, ONE DIRECTION OF TRAVEL								
6	1.00	0.96	0.89	0.78	1.00	0.96	0.89	0.78
4	0.99	0.95	0.88	0.77	0.98	0.94	0.87	0.77
2	0.97	0.93	0.87	0.76	0.96	0.92	0.85	0.75
0	0.94	0.91	0.85	0.74	0.91	0.87	0.81	0.70

* Same adjustments for capacity and all levels of service.

Source: Highway Capacity Manual 1965. Highway Research Board Special Report 87, p. 256. (3)

For situations where no congestion is present, the ratios (D / C) and (V / C) will be identical. Where congestion does occur, however, (V / C) will be less than (D / C) as discussed in section 5.4.

Speed-Flow Relationships

The operational characteristics of a traffic stream are defined by its speed, flow and concentration. Speed, denoted by u, is defined for a single vehicle as simply rate of movement in miles per hour. For aggregates of vehicles a useful measure is the average running speed, defined as the sum of the distances traversed by all vehicles divided by the sum of the running times. Assuming the same distance D traversed by N vehicles, the average running speed would be computed as:

$$u = \frac{ND}{\sum_{i=1}^{N} t_i} = \frac{ND}{N\bar{t}} = \frac{D}{\bar{t}} = \frac{D}{\frac{1}{N}\sum_{i=1}^{N}\frac{D}{u_i}} = \frac{1}{\frac{1}{N}\sum_{i=1}^{N}\frac{1}{u_i}} \qquad (236)$$

where u_i is the speed of each vehicle i, D is the distance traversed by each vehicle in time t_i, and $\bar{t}$ is the average running time over distance D.

Flow (or volume *) is defined as the number of vehicles passing a point within a given time period; or:

$$q = N / T \qquad (237)$$

where q = vehicle flow ;

N = number of vehicles passing a given point ;

and T = time interval, which for the EAROMAR analysis will be taken as one hour.

Concentration, k, the number of vehicles per unit length of road, is then related to speed and flow via the general relationship:

$$k = q/u; \quad \text{or} \quad u = q/k; \quad \text{or} \quad q = ku \qquad (238)$$

A model of the form of eq. (238)is referred to as a traffic-stream model, and defines a three-dimensional surface relating the operational parameters u, q, and k. For simplicity in representation, however, pertinent relationships are often displayed instead as two-dimensional slices through the surface, yielding speed-concentration and speed-flow curves.

* Flow is normally measured over intervals of less than one hour; volume, over intervals of one hour or longer.

Both theoretical and empirical models are available to represent the speed-flow relationship, which can be used to tie a level of road service (average traffic speed) to road characteristics (flow as constrained by road capacity). Although the precise mathematical forms of these various models differ, all realistic speed-flow curves exhibit the same general shape, consistent with empirical observations of traffic behavior.

First, the free-flow speed of a vehicle approaches the maximum attainable speed* as concentration (and therefore flow) approach zero. The average speed may be somewhat lower than the maximum attainable, because not all drivers choose to operate at the maximum speed that can be realized. However, this average speed will likewise exhibit its maximum value at zero density. Second, speed reduces to zero as concentration reaches the jam concentration k_j, and as flow again goes to zero. Thus, the speed-flow function has two realisitic values on the speed axis at zero flow — one point at the origin, the second at the maximum attainable value. Third, between these zero and maximum values of speed, the speed-flow curve is convex, reaching to a point of maximum flow q at some speed u and critical density k.

To remain consistent with our development of road capacity using procedures recommended in the Highway Capacity Manual (3), we have adapted speed-flow relationships from the empirical curves presented in the Manual for freeways. Brief discussions of both the Manual data and our rendering of these data for use within EAROMAR are described below.

HIGHWAY CAPACITY MANUAL

Highway Capacity Manual procedures are based on analysis of empirical data such as those illustrated in Figure 58 for the free-flow regime on freeways.** Superimposed on this figure are volume-average speed curves developed from the data points shown, and including the regulating effect of speed limits. Except for portions of the curves near critical density (the shaded area in Figure 58), the speed-flow relationships are linear or near-linear; however, no closed form equations are formally introduced in the Manual for these curves.

It should be pointed out that the curves in Figure 58 are included within the Manual only for illustration of average speed values, and are not, strictly speaking, suitable for general demand-capacity calculations. There are two reasons for this. First, the curves make no provisions for incorporating adjustments to reflect various adverse conditions normally found on actual roadways (such as the constraining ef-

* Maximum attainable speed is defined as that speed governed by road design and safe driver practices; i.e., favorable weather conditions and no traffic interferences prevail.

** I.e., in the absence of congestion. Congestion and queueing will be treated in a later part of this chapter.

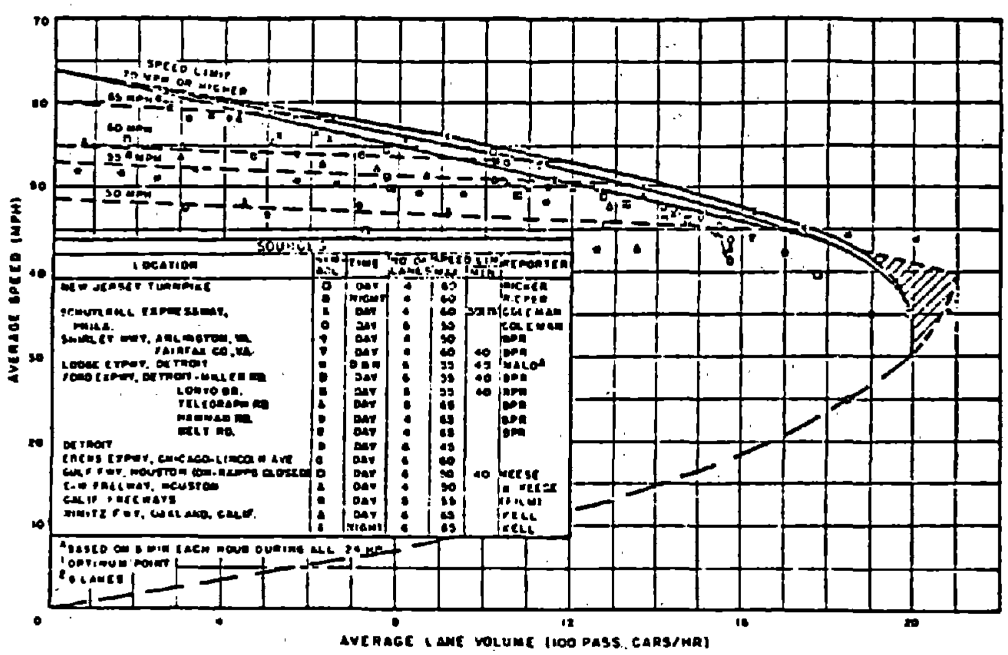

Figure 58

Data and Relationship for Average Speed and Volume

Source: Highway Capacity Manual 1965. Highway Research Board Special Report 87, p. 66. (3)

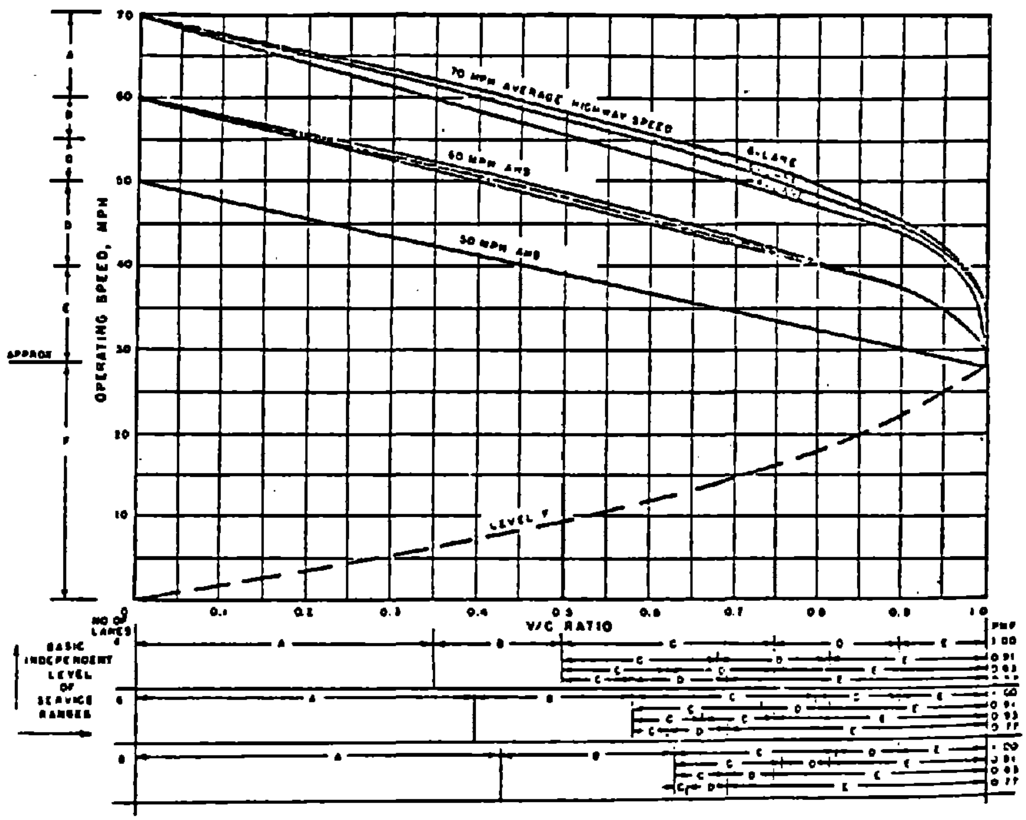

Figure 59

Recommended Operating Speed -- V/C Relationships
from the Highway Capacity Manual

Source: Highway Capacity Manual, 1965. Highway Research Board Special Report 87, p. 264.

fects of roadside obstacles, presence of trucks and other factors discussed earlier). This problem is remedied by replacing absolute volume or flow along the abscissa by the volume-capacity ratio, as will be illustrated shortly. Second, the Highway Capacity Manual bases its level of service concepts on road operating speed* rather than average speed, and therefore a different set of curves is actually recommended for capacity calculations.

These recommended curves are reproduced in Figure 59. Note that the use of the volume-capacity ratio as a measure of flow now enables one to treat different classes of road, numbers of lanes, and other influences on capacity under one generalized procedure. However, the use of operating speed (in lieu of average speed) is inappropriate for the EAROMAR analysis, because it does not reflect overall characteristics of the entire traffic stream, and is therefore unsuitable for calculations of, for example, total user travel time.

EAROMAR RELATIONSHIP

In his development of the original EAROMAR system, Butler (2) derived a set of equations to approximate the Highway Capacity Manual curves for average speed in Figure 60, but replacing absolute volume by the volume-capacity ratio as discussed above. We have modified or simplified these equations for use within the current version. The resulting model is presented in Figure 61 , with interpretations of variables shown in Figure 60. Note that the effects of both highway design speed and speed limit can be explicitly accounted for.

Speed-Roughness Relationship

The relationship between speed and the geometric and operating characteristics of a highway has been covered extensively in the literature. Much less attention, however, has been devoted to the influence of pavement condition on speed and flow. Research on user consequences (speed, vehicle operating costs) as functions of surface condition for paved vs unpaved roads has been limited generally to developing countries; corresponding information on high-type pavements found on US freeways is scarce. Nevertheless, the existence of such a relationship has important implications for the type of analysis conducted through EAROMAR. It would define the benefits of a smooth pavement surface resulting either from good maintenance policies or from investment in a premium pavement.

The one study which was appropriate to our requirements was conducted by Karan and Haas in Ontario in 1974 (51). Data on traffic speed, flows, volume-capacity ratio, speed limits, and pavement roughness (measured by a BPR roughomoter and converted into a Road Condition Index, or RCI) were obtained for 72 road sections. Several regression models were developed to fit tyese data, all of which had good statistical param-

* Operating speed is the maximum attainable speed under prevailing traffic conditions.

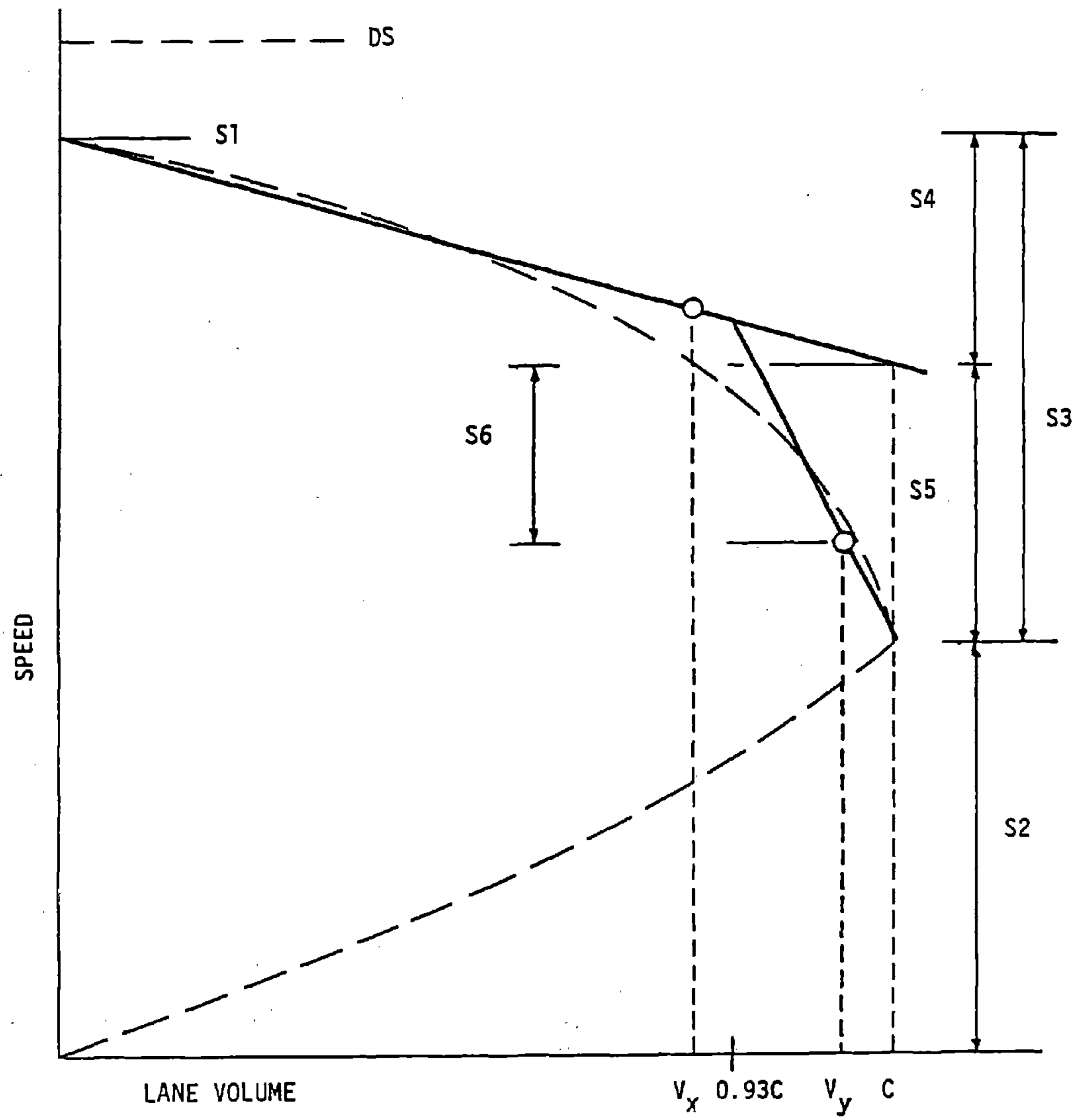

FIGURE 60. RELATIONSHIP BETWEEN SPEED AND VOLUME USED IN EAROMAR

FIGURE 61

SPEED-FLOW RELATIONSHIPS
USED WITHIN EAROMAR

$$S1 = 0.9 \; DS \tag{239}$$

$$S2 = 30 \tag{240}$$

$$S3 = S1 - S2 \tag{241}$$

$$S4 = (0.4 \; DS - 10) \times V / C \tag{242}$$

$$S5 = S3 - S4 \tag{243}$$

$$S6 = 0 \quad (\text{if } V/C \leq 0.93) \tag{244}$$

$$S6 = \frac{[(V/C) - 0.93]}{0.07} \; X \; S5 \quad (\text{if } V/C \geq 0.93) \tag{245}$$

Governing speed due to road design

$$= S_D = S1 - S4 - S6 \tag{246}$$

Governing speed due to speed limit

$$= S_L = (0.9 \; X \; SL) - (3.6 \; X \; \frac{V}{C}) \tag{247}$$

Estimated Free-Flow Average Speed

$$= SPEED = MINIMUM \; (S_D, S_L) \tag{248}$$

Where:

DS = Freeway design speed, mph

C = Capacity of freeway or lane closure in 1000's

SL = Speed limit on freeway or of the lane closure

V = Any volume in 1000's

S = Speed for a given volume V or volume-capacity ratio V/C

Source: EAROMAR Final Report, B.C. Butler, Jr., October 1974, pp. 101 - 103.(2)

eters. One was selected by Karan and Haas based upon subjective evaluation and the model's simplicity.

For use within EAROMAR we have selected a different model from the set developed in (51). This model is also simple in form; moreover, of the four models developed by Karan and Haas, it is the only one which excludes the effects of volume-capacity ratio. This we wanted to do, since volume-capacity ratio is already accounted for in the equations in Figure 61. By converting RCI to the Present Serviceability Index (PSI), and expressing speeds in miles per hour in lieu of kilometers per hour, we obtained the following relationship for use within EAROMAR.

$$S_R = 21.4 + 0.04\ S_L\ PSI + 0.007S_L^2 \qquad (249)$$

where

S_R is the limiting speed due to roughness, mph;

S_L is the observed speed limit, mph; and

PSI is the present serviceability index of the pavement.

Then the free-flow average speed on a given roadway section is estimated as:

$$\text{SPEED} = \text{MINIMUM}\ (S_D, S_L, S_R) \qquad (250)$$

where

SPEED is the predicted average speed of the traffic stream, mph;

S_D is the limiting speed, in mph, due to road design and operational parameters, as computed in eq. (246);

S_L is the limiting speed, in mph, due to the observed speed limit, as computed in eq. (247); and

S_R is the limiting speed, in mph, due to pavement condition, as computed in eq. (249).

5.3 LANE CLOSURES FOR MAINTENANCE AND REHABILITATION

Introduction

Whenever pavement maintenance or rehabilitation work encroaches upon one or more roadway lanes, these lanes are closed to traffic under established procedures to protect the safety of both motorists and maintenance crews. Guidelines for workzone delineation and scheduling are often contained in a procedural manual published by the appropriate state agency or road operating authority, which discusses the recommended type, placement, and extent of workzones, required traffic warning devices and control methods, and allowable times of day for lane closures.

Those aspects of the problem relating to maintenance scheduling and work accomplishment within workzones as part of the EAROMAR analysis were discussed in Chapter 4. In this section, we will consider the operational effects of lane closures on road capacity, and develop ways of representing pertinent closure characteristics and their effects on the traffic stream within the EAROMAR simulation.

Examples of State Practices

The safe and efficient conduct of traffic through temporary workzones has become a topic of considerable interest to highway administrators. Traffic control operations require considerable planning and public information efforts, particularly for high-volume urban freeways requiring maintenance or rehabilitation, where one generally must contend with a combination of large numbers of vehicles already exhibiting congestion during peak flows; right-of-way restrictions often precluding construction of temporary detours; and already crowded conditions on available detour routes. Even where availability of adequate detour measures is not a problem, the tasks of providing advance warning to vehicles entering the workzone area at freeway speeds, and of rechanneling traffic into temporary bypasses of perhaps lower geometric standards, requires established procedures to effectively deploy control devices over a road length extending beyond the actual workzone limits.

Effective workzone procedures have been investigated by several research and performing groups.* The examples given below are taken from NCHRP Syntheses of Highway Practice 1 and 25, and are included to illustrate geometric and operational characteristics of lane closures

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* The subject has been covered in NCHRP Project 20-5, and has been designated an emphasis area by FHWA. Also, Part IV (Traffic Controls for Street and Highway Construction and Maintenance Operations) of the Manual on Uniform Traffic Control Devices is now being revised. Several reports in this area include:

(cont. over)

of interest in estimating capacity reductions (50, 52).

Factors that characterize lane occupancy for maintenance and rehabilitation include:

1. The traffic demand that must be accomodated through the workzone area, including total AADT and any notable peak-period flows;

2. The degree of closure to be employed: whether full (all lanes closed, detour provided) or partial (travel restricted to less than full number of lanes) on the affected roadway;

3. The severity of the road constriction: the number of lanes to be closed (a function of demand and of the type work that must be performed) versus the number of lanes to remain open (a function of demand, total lanes and shoulders available, and temporary lanes to be constructed or provided);

4. The placement of the workzone (outer shoulder, median, or center lane closing; proximity of entrance or exit ramps);

5. The length of the closures, and allowable spacing between successive closures; and

6. Scheduling and duration of work closures; differentiation among moving, short-term, or long-term closures; and daylight closures versus nighttime closures .

OFF-SITE DETOURS.

Figure 62 illustrates the closure plan for using an off-site detour for nighttime repair work on the Edsel Ford Freeway in Detroit.

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* (Footnote from preceding page, cont.:)

Graham, J. L., et al. "Accident and Speed Studies in Construction Zones," Report FHWA-RD-77-80, prepared for FHWA, June 1977 (53).

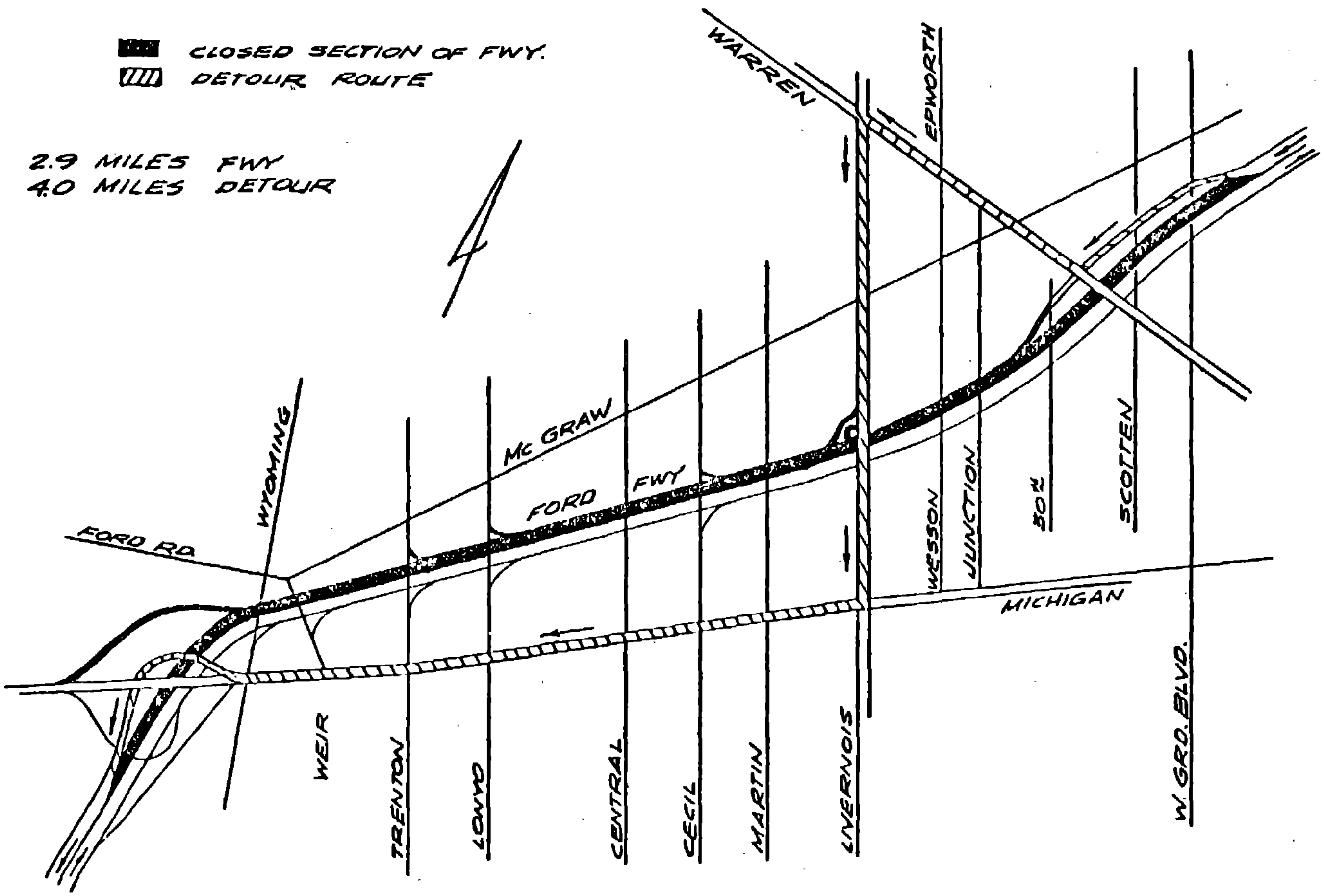
Chipps, J. A., et al. "Traffic Controls for Construction and Maintenance Worksites: A Research Reference Report," Vol. I. Prepared for FHWA by Amer. Public Works Assn., Chicago, October 1976 (54).

Byrd, Tallamy, MacDonald & Lewis. "Techniques for Reducing Roadway Occupancy During Routine Maintenance Activities," NCHRP Report 161 (55).

"Traffic Control for Freeway Maintanance," NCHRP Synthesis of Highway Practice 1, HRB, Washington, D.C., 1969 (52).

"Reconditioning High Volume Freeways in Urban Areas," NCHRP Synthesis of Highway Practice 25, TRB, 1974 (50).

FIGURE 62
Plan for Offsite Detour of a Section of Edsel Ford Freeway, Detroit



Source: NCHRP Synthesis 25, p. 17. (50)

In addition to traffic routing (that included signing and coordination of traffic signals to smooth the flow of traffic), other components of the plan included:

1. Scheduling. Economic studies indicated that the best period for work performance (on a seasonal level) extended from mid-June through August; on weekdays, from 10 p.m. to 5 a.m.; and on weekends, from 2 a.m. to 10 a.m. Note that similar scheduling options can be investigated in the EAROMAR analysis using conventions described in Chapter 4.

2. Freeway Entrances and Exits. It was determined that ramps serving to reroute the freeway traffic around the worksite should be capable of handling at least two lanes of traffic. Also, it was found helpful to close entrance ramps to the freeway a mile or so prior to the detour, to prevent traffic entering the system only to be immediately diverted.

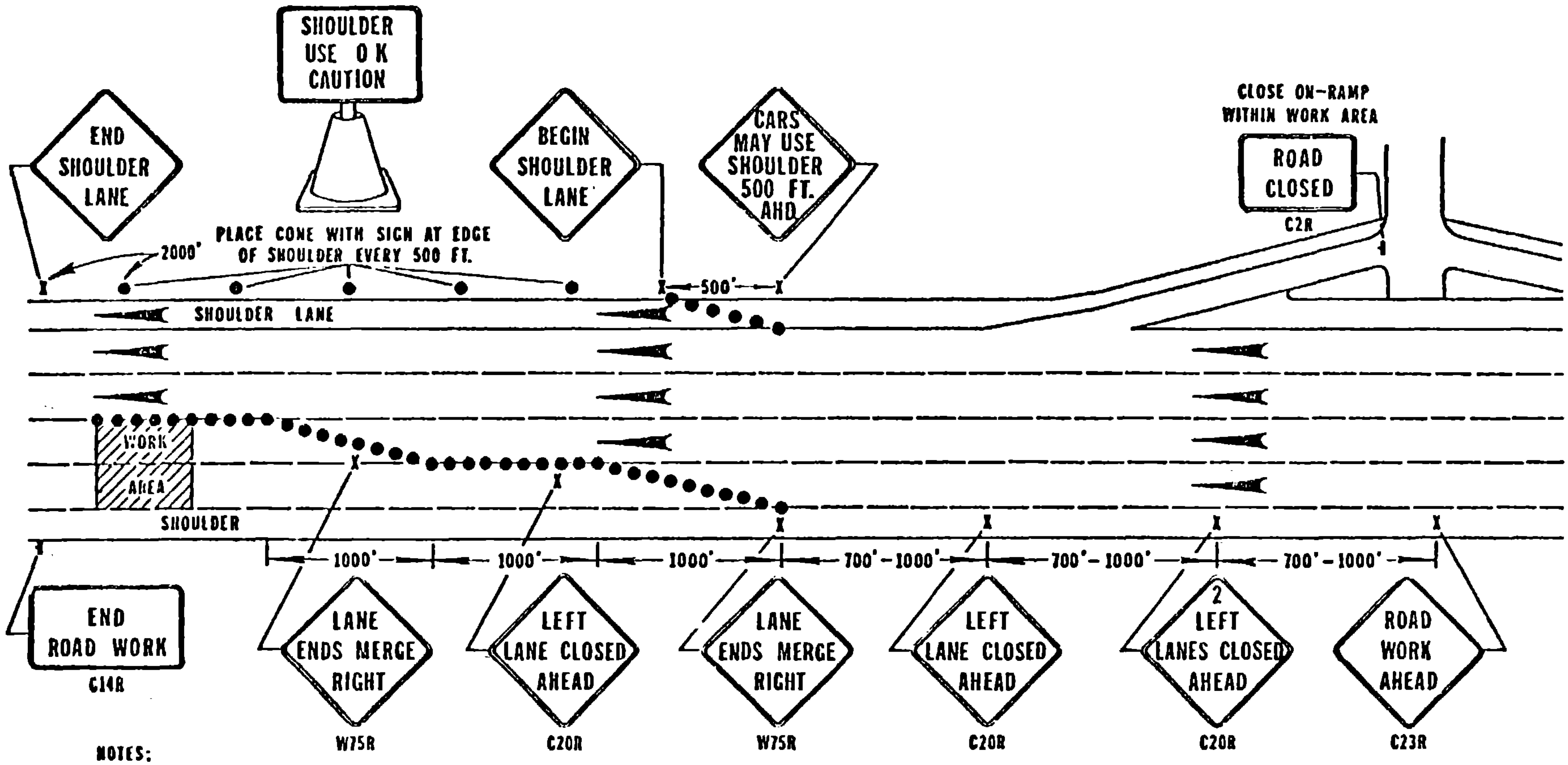
ON-SITE DETOURS

On-site detours can be accomplished by restricting the number of travel lanes available, using paved shoulders as temporary lanes, employing cross-overs to lanes in the opposing traffic roadway, restriping pavements to maintain all lanes at reduced widths, or constructing a temporary bypass within the right-of-way. One example of a plan employing reduced numbers of lanes plus shoulder use is shown in Figure 63.

Figures 64 , 65, and 66 illustrate recommended practices for single-lane (both right and left), double-lane, and center lane closings in daytime for the New Jersey Turnpike. Corresponding diagrams are included in the New Jersey manual for shoulder closures, workzones in the vicinity of ramps, and nighttime work. Figure 67 diagrams a traffic crossover between two roadways to bypass some bridge repair work (prepared by the Illinois State Toll Highway Authority); while Figure 68 , taken from a public information release by the State of Illinois, shows a combination use of lane constrictions and traffic reversal for maintenance and improvements work on the Eisenhower Expressway in Chicago.

Modeling Lane Closures

The treatment of road occupancy within EAROMAR is based on this fact: lane closures for maintenance or rehabilitation affect traffic operations through their reduction in road capacity. Closure characteristics with implications toward capacity reduction fall into the several categories below. Specification of data within each category is provided by the user during the analysis, as part of either the descriptions of maintenance activities or the specification of road projects.



NOTES:

1. Signing for median shoulder lane is same as above but on opposite side.
2. Provide at least 1000' – 2000' between work area and END SHOULDER LANE sign to allow traffic to merge. Do not use shoulder if traffic must merge within the work area.
3. Shoulder lane signs should be covered when shoulder lane is blocked by a stalled vehicle.
4. Do not use shoulder lane when paved width is less than 8' or where there are obstructions such as rolled gutter, drainage structures, restrictions on lateral clearance, or where there are on-ramps within the work area.

Figure 63. Example of Lane Closures and Use of Shoulders for Urban Freeways

Source: NCHRP Synthesis 25, p. 20. (50)

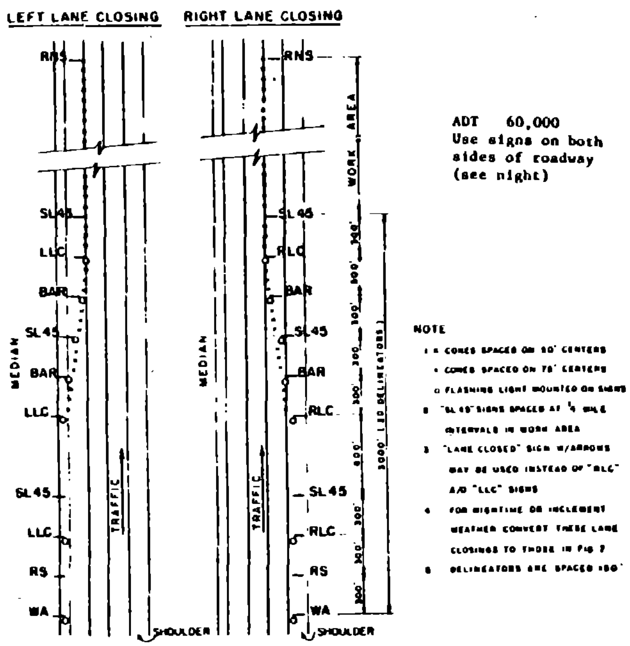


Figure 64. Example of Procedures for Daytime Lane Closing (52)

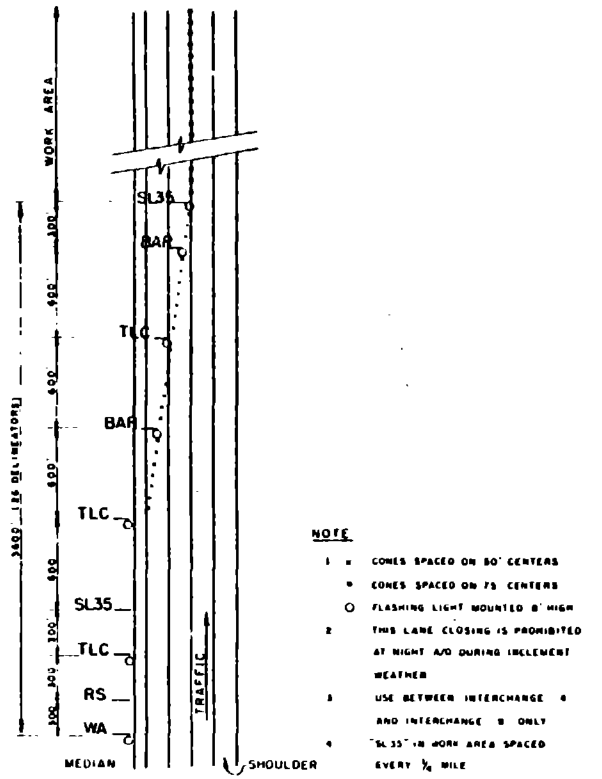


Figure 65. Example of Procedures for Two-Lane Closing (52)

(Source: <u>NCHRP</u> <u>Synthesis</u> <u>1</u>, p. 40)

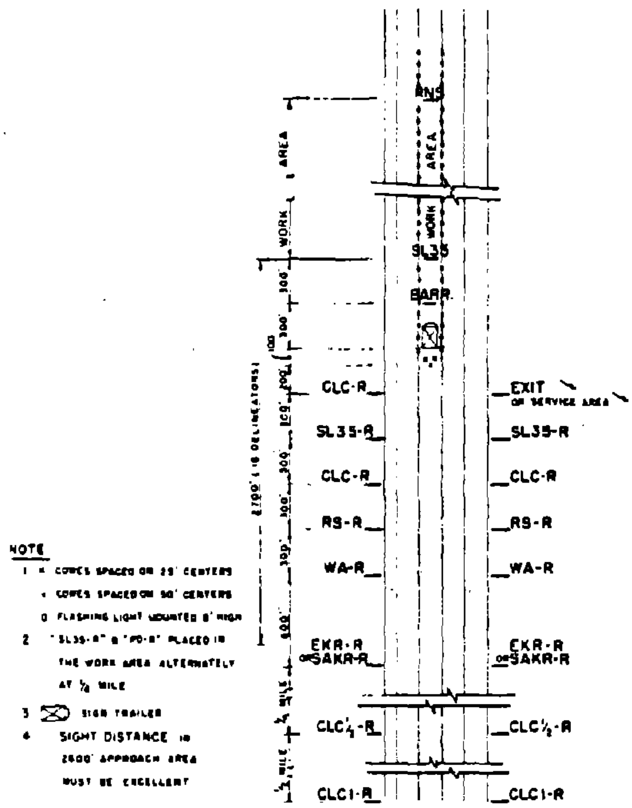
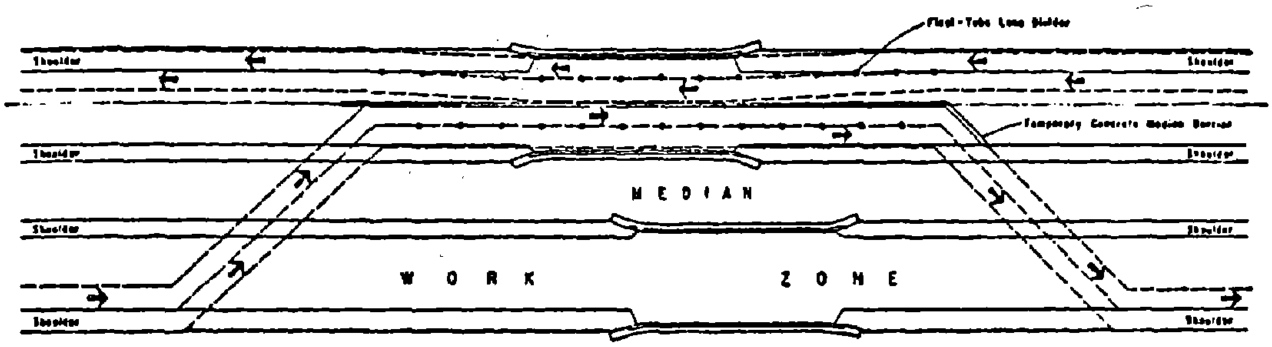


Figure 66
Example of Procedures for Center Lane Closing (52)

Source: NCHRP Synthesis 1, p. 40.



DETAIL OF FLAG CREEK BRIDGE RUN-AROUND

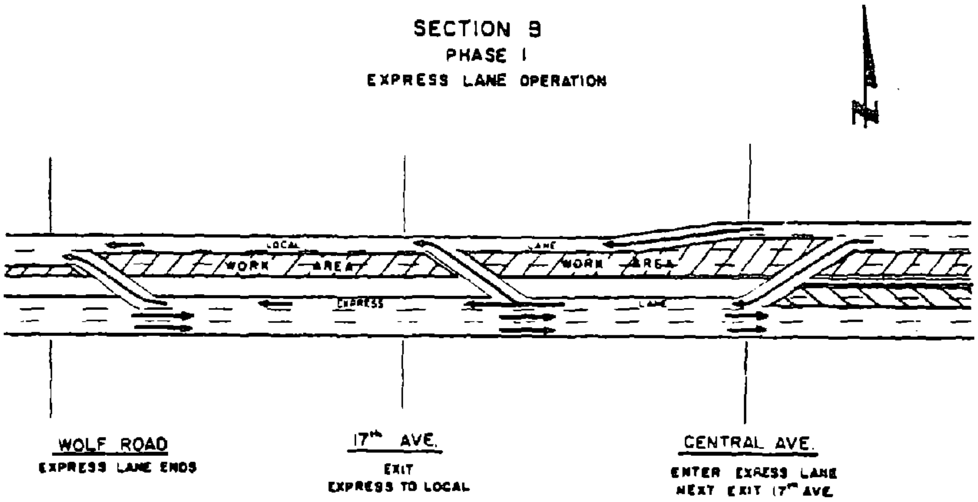
Figure 67.
Example of Traffic Crossover to Bypass Work Zone

Source: NCHRP Synthesis 25, p. 19. (50)

Figure 68

Provision of Local and Express Detour Lanes During Work on the Eisenhower Expressway, Chicago

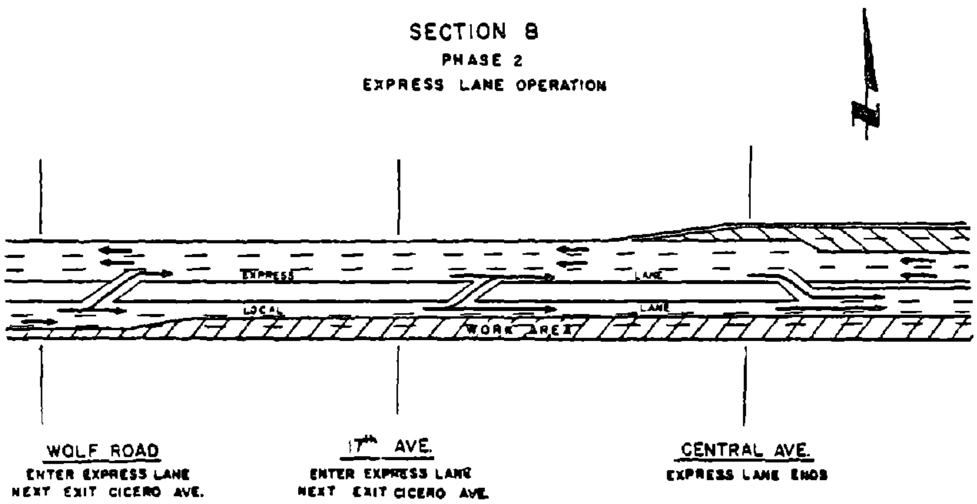
SECTION 9
PHASE 1
EXPRESS LANE OPERATION



EXPRESS LANE OPERATION

1. During Phases 1a & 1b, westbound traffic will have the option to enter the EXPRESS LANE at Central Avenue. The next exit provided, for EXPRESS traffic, will be 17th Ave.
2. Just west of 9th Avenue, the motorist may leave the EXPRESS LANE, for all exits: 17th Ave. to Wolf Rd. Motorists heading for exits west of Wolf Rd. should continue in the EXPRESS LANE.
3. The EXPRESS LANE ends at Wolf Road.
4. Trucks and Buses are restricted to the LOCAL LANE.

SECTION 8
PHASE 2
EXPRESS LANE OPERATION



EXPRESS LANE OPERATION

1. During Phases 2a & 2b, eastbound traffic will have the option of entering the EXPRESS LANE at either Wolf Road or 17th Avenue.
2. For traffic entering the EXPRESS LANE at either of the above, the next exit will be Cicero Avenue.
3. Trucks and Buses are restricted to the LOCAL LANE.

Source: NCHRP Synthesis 1, p. 45. (52)

TYPE OF CLOSURE

Three types of closures can be defined:

1. Lane restrictions, where the roadway on which work is to be performed will remain open, with traffic accomodated over either a reduced number of lanes or lanes of reduced width;

2. Crossovers, where the roadway under repair will be closed and traffic rerouted to an adjacent roadway (if more than two roadways exist, this need not imply reversal of flows); and

3. Detours, where the roadway under repair will be closed and traffic rerouted to an on-site or off-site bypass. The detour may comprise existing streets, newly constructed temporary roads, or a combination of both.

Each type of closure will be described by one or more of the following operational characteristics:

LANES AVAILABLE

The number and width of lanes available to the affected traffic (together with the type of closure) help determine the changes in operational efficiency of the route as described below. The number of lanes available includes all open lanes through the closure zone, whether belonging to the roadway proper or whether temporary lanes constructed for the duration of maintenance work.

CAPACITY

Several options are available for specifying closure capacity, to accommodate the characteristics of the different closure types possible or the nature of the data available.

Reduction Factors. The capacity reduction factor W was explained earlier as a function of lane width and side friction effects. It is possible that a lane closure will reduce the value of this factor. Users may therefore input a revised value W' over the length of the traffic rerouting. This factor can be specified for lane reductions and crossovers, but not detours. The resulting capacity is computed according to the following equation:

$$\text{Closure Capacity, vph} = 2000\ \text{LAVAIL} \times W'T \tag{251}$$

where

LAVAIL is the number of lanes available through the closure zone, as input by the user;

W' is a modified capacity reduction factor for the closure zone, input by the user; and

T is the truck factor defined in eq. (233)

When a crossover is specified, two streams of traffic occupy a single roadway -- the diverted traffic, and the traffic on the receiving roadway (e.g. traffic in the opposing direction on freeways with only two roadways). For crossovers users may therefore specify two reduction factors: W' to govern roadway capacity for the diverted traffic, and W" to govern capacity for traffic already on the receiving roadway. The reduced capacity seen by the diverted traffic is computed by eq. (251). The reduced capacity seen by traffic already on the receiving roadway is computed as follows:

$$\text{Reduced Capacity, vph} = 2000\ (N_{laws} + \text{LTEMP} - \text{LAVAIL}) \times W''\ T \qquad (252)$$

where

N_{lanes} is the number of travel lanes on the receiving roadway, as input by the user;

LTEMP is the number of temporary lanes provided through the closure zone, as input by the user;

LAVAIL is the number of lanes through the closure zone <u>seen by the diverted traffic</u>, as input by the user;

W" is the modified capacity reduction factor through the closure zone for <u>traffic on the receiving roadway</u>, as input by the user; and

T is the truck factor defined in eq. (233).

<u>Flows at Capacity</u>. Lane reductions may present frictional effects greater than those accounted for in eqs. (251) and (252). Or, users may have data available on practical capacities for lane closures based on prior experience. For example, Tables 65 and 66 give roadway capacities observed during maintenance and construction operations on Los Angeles freeways (50).

Flows at capacity may therefore be input directly by the user for lane restrictions and crossovers, but not detours. When crossovers are specified, capacities may be input for each direction of flow. The flow values so provided will substitute for the calculations by eqs. (251) and (252) above.

<u>Detour Length and Speed</u>. The capacity flows estimated above, when compared against hourly roadway demand, can be used to determine average speed through the workzone area, leading to calculation of changes in user costs to be described in Chapter 6. For detours, however, capacities are difficult to estimated. Take, for example, an offsite detour traversing several existing streets, complicated by lack of access control, signals,

and side friction typical of urban areas. To develop composite detour capacities in such cases would be time consuming and difficult to construct within a link-level analysis.

Instead, users provide the length of the detour and average speed through the detour. These length and speed factors will then be applied as required in the user cost calculations in Chapter 6. In simulating congestion, we assume an average detour capacity of 1500 vehicles per hour.

Table 65

Examples of Observed Capacities
on Los Angeles Freeways During Road Work

OBSERVED CAPACITY RATES FOR SOME TYPICAL OPERATIONS ON LOS ANGELES FREEWAYS [a]

No. of lanes, one direction (normal operation)	2	3 or 4	4
No. of lanes open, one direction	1	2	3
TYPE OF OPERATION	CAPACITY, ONE DIRECTION (VPH)		
Median barrier or guardrail repair	1500	3200	4800
Pavement repair, mudjacking, pavement grooving	1400	3000	4500
Striping, resurfacing, slide removal	1200	2600	4000
Pavement markers	1100	2400	3600
Middle lanes, any reason	—	2200	3400

[a] From: Forbes, C. E., et al., "Reducing Motorist Inconvenience Due to Maintenance Operations on High-Volume Freeways." *HRB Spec. Rep. 116* (1971) pp. 181-188.

Table 66

SUMMARY OF TRAFFIC CAPACITIES DURING CONSTRUCTION OF RAMPS AND WIDENING, SOUTHBOUND HOLLYWOOD FREEWAY AT LANKERSHIM

DAY OF WEEK	DATE	TIME OF DAY	WEATHER	4-LANE CAPACITY [a] (VPH)	REMARKS
M	10/27/69	0830	Overcast	6400	Open trench; equipment working both sides.
Th	10/30/69	0700	Sunny	6800	Open trench; loader and trucks working on right.
M	11/2/69	0830	Sunny	6200	Pavement breaker working on right.
Tu	11/4/69	0715	Sunny	6700	Motor grader working on right.
W	11/12/69	0730	Cloudy	6200	Roller and motor grader working on right.
F	11/14/69	0700	Clear	6200	Bottom-dump trucks and motor grader working on right.
Tu	12/16/69	0800	Overcast	6400	Paving ramp.

[a] Maximum observed capacity both before and after construction, 7,600 vph.

(SOURCE: NCHRP Synthesis of Highway Practice 25, p. 22) (50)

DURATION OF OCCUPANCY

Maintenance and rehabilitation work can be scheduled within the EAROMAR analysis by type and hour of day as described in Chapter 4. However, it is not necessary that the duration of road occupancy coincide exactly with periods of actual work performance. For example, for work scheduled to be performed, say, at periods of six hours per day over several successive days, the workzone barriers may or may not be lifted on the road during off-work hours, depending upon the effort needed to set up and remove the barriers, and the serviceability of the affected road area following each day's work.

EAROMAR therefore allows users to differentiate between these two options for road occupancy. "Onsite" occupancy duration will schedule workzone closures to be in place only when maintenance forces are actually on-site and performing work. "Total" occupancy duration will schedule workzone closures to be in place continuously from the time work begins until it is completed (either the same day or on following days). The determination of which option applies should be made by the user depending upon the type of maintenance activity under consideration and locally governing practices. It is assumed that closures for "project" (as opposed to "maintenance") work will be of "total" duration.

LENGTH OF CLOSURE

The lineal extent of a closure is determined in one of two ways. For all maintenance activities (and optionally for projects), the closure location and length coincide with the location and length, respectively, of the workzone plus the taper required for traffic channeling or rerouting.

For "projects" a second option is available, that of specifying milepost delineations for the workzone. This makes it possible to simulate, for example, complete rerouting of traffic from one intersection to another when major rehabilitations are underway, even if the intersection locations do not coincide with the limits of actual work being performed.

DETOUR COST

Where detours involve extensive construction of temporary roads or structures, their costs may form a significant percentage of the total maintenance or rehabilitation expenditure. Users may, therefore, include lump-sum costs of construction and removal of temporary bypass roads, bridges, or other work as part of the closure description.

5.4 CONGESTION AND QUEUEING

Introduction

To this point we have looked at road operating characteristics under free-flow operating speeds; i.e., in the absence of congestion. However, the constricting effects of road closures just described, superimposed upon demand patterns introduced earlier, will likely result under typical urban conditions in increased congestion and queueing, reduced average speeds, and associated increases in user-related costs. The objective of this section is to develop the methods needed to estimate average speed over road length and time under congested conditions, for use later by the user cost relationships in Chapter 6.

Congestion arises on a road section when its demand-capacity ratio exceeds 1.0. Equation (235) defined the V/C ratio under normal operating conditions. This relationship will be used in the EAROMAR simulation to test for congestion now existing (due to peak-hour effects) or that may result in the future from growth in demand.

The introduction of road closures for maintenance or rehabilitation workzones reduces effective road capacity, thereby increasing the V/C ratio (assuming other factors remain the same) and generating increased likelihood of congestion over more sections of the road or over more hours of the day. These capacity reductions were quantified by equations (251) and (252), and illustrated in Tables 65 and 66 . These reduced capacities are applied in a relationship similar to eq. (235) to test for congestion induced by roadway occupancy.

An investigation of congestion and formation of queues and their effects on the traffic stream was carried out by Butler as part of the original EAROMAR development (2). Observing actual traffic streams in rush hour traffic and in areas affected by maintenance workzones, he developed schematic speed profiles for uncongested, on the verge of congested, and queued traffic as shown in Figures 69 through 71 respectively.

These figures define several speed regimes of use in our later development of congestion equations. Approach speed, or AS, defines the free-flow speed typical of the road section when congestion is not present. ZS, or zone speed, is the reduced speed through the zone of restricted capacity (or excessive demand) causing the congestion (i.e., the bottleneck section). QS, or queue speed, is the reduced speed in the queue (if any) approaching the influence zone (i.e., upstream from the bottleneck).

These speed components, and their variation over route length and time, can be related to demand-capacity characteristics through the simulation of roadway operations proposed for the EAROMAR analysis. This simulation will involve computations over each section of roadway for each type and hour of day. When maintenance or rehabilitation work is to be performed, the roadway section will be simulated with the road closure so

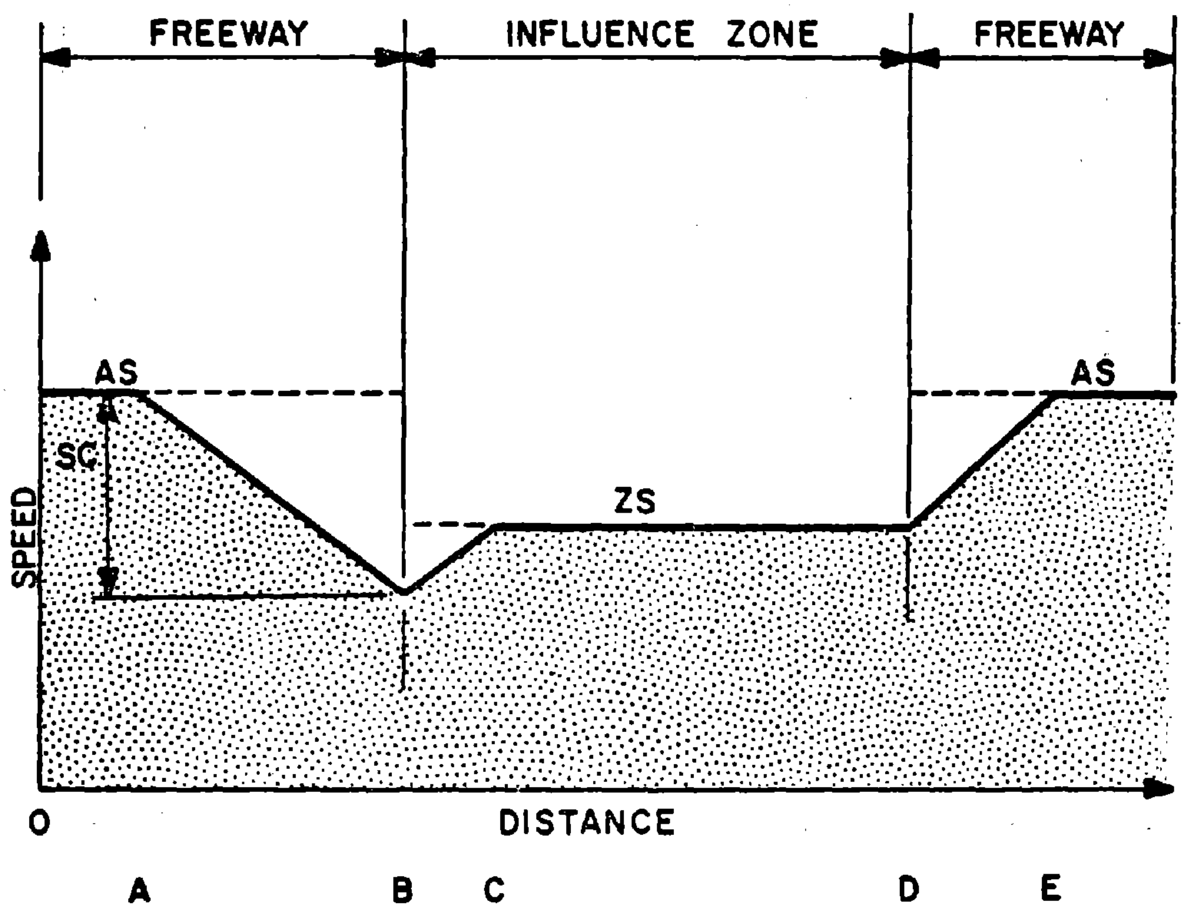


FIGURE 69. SCHEMATIC SPEED PROFILE OF UNQUEUED TRAFFIC OPERATION THROUGH A TRAFFIC CONTROL ZONE.

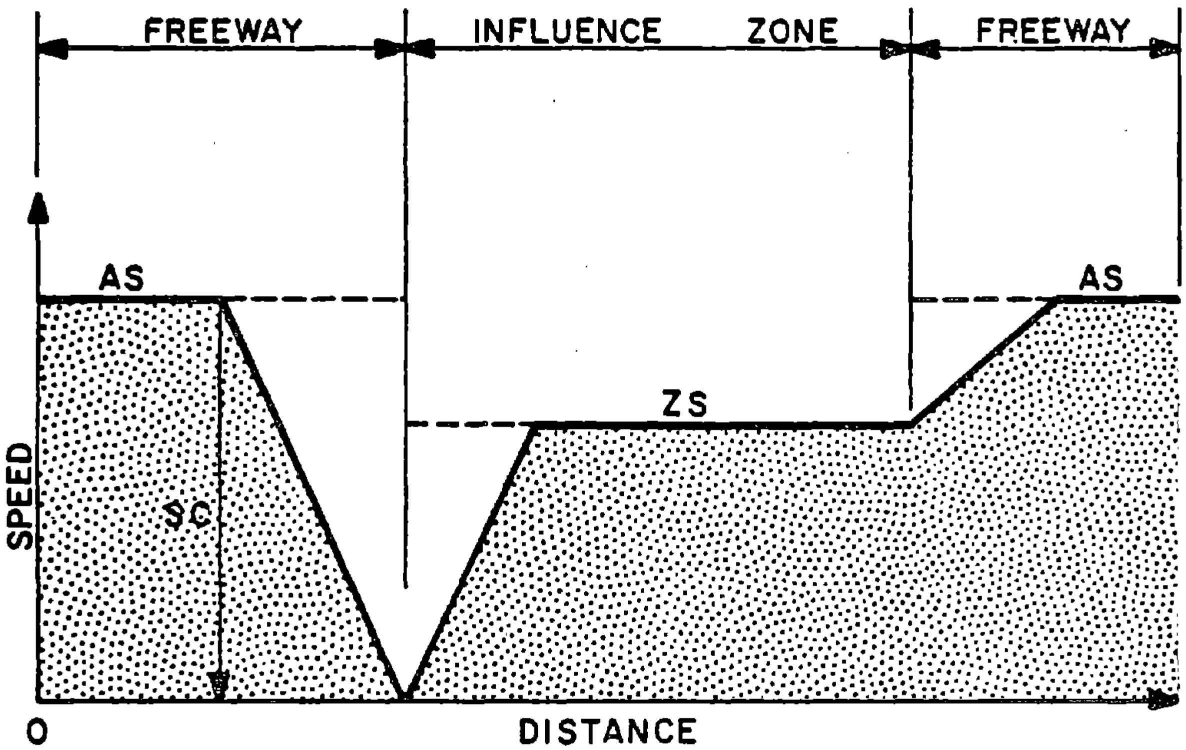


FIGURE 70. SCHEMATIC SPEED PROFILE OF TRAFFIC OPERATION THROUGH A TRAFFIC CONTROL ZONE ON THE VERGE OF QUEUING.

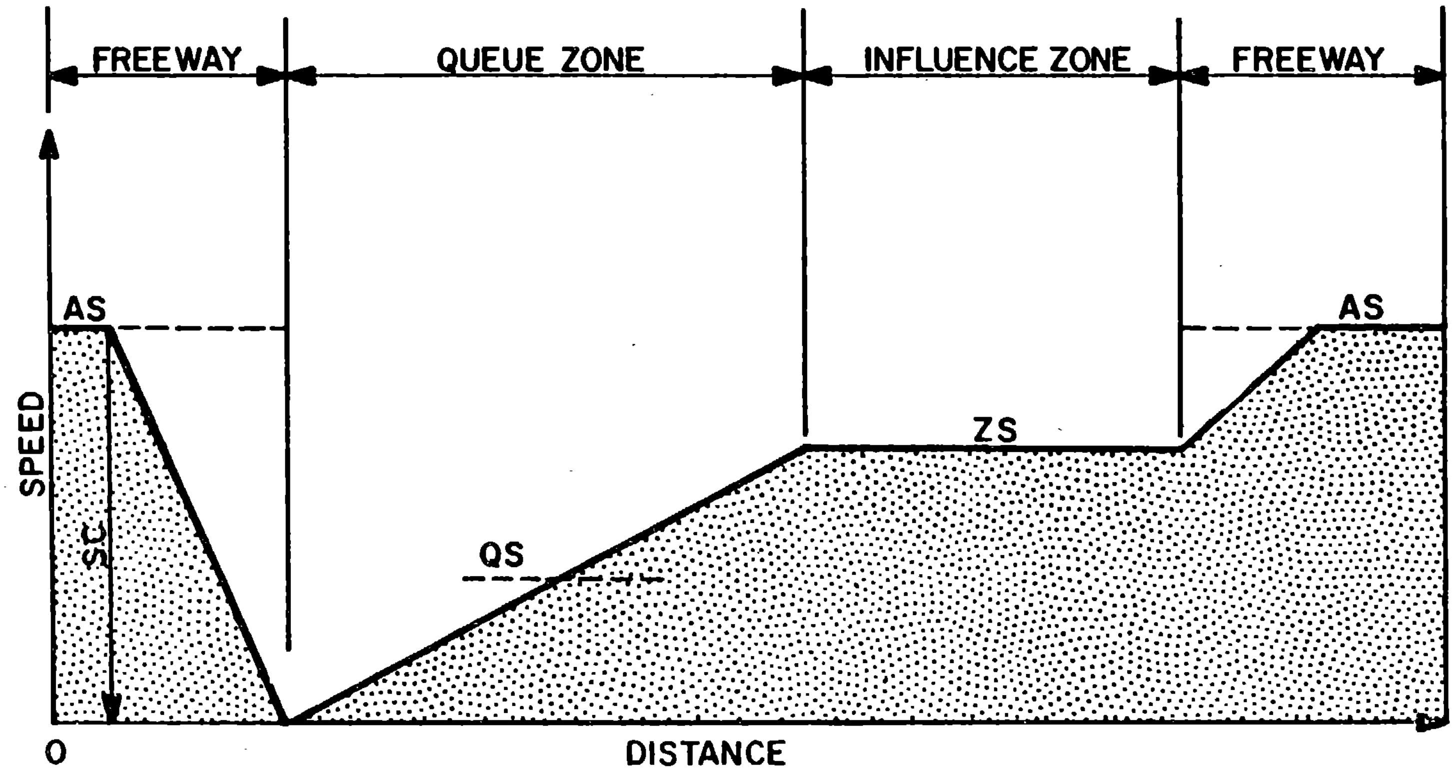


FIGURE 71. SCHEMATIC SPEED PROFILE OF TRAFFIC OPERATION THROUGH A TRAFFIC CONTROL ZONE WHERE A QUEUE HAS OCCURRED.

that increased congestion and attendant user costs can be estimated.

Treatment of Congestion and Queueing in the Literature

The treatment of congestion and queueing in an aggregate sense has been well developed in the literature. The following presentation is based upon work by Butler during the original EAROMAR development (2), subsequent work by Byrd, Tallamy, MacDonald and Lewis for FHWA (56), and by Curry and Anderson in their analysis of road user costs (57).

FORMATION OF QUEUES

The queueing process can be understood through a graphical representation as shown in Figure 72 , with cumulative number of vehicles shown on the vertical axis and time on the horizontal axis. The straight line represents the fixed capacity at a bottleneck; when demand exceeds this capacity, a queue will form upstream from this bottleneck. The variation in demand over time leading to the formation of congestion is shown by the curved line in Figure 72.

In Figure 72 , a queue starts building at time t_1. The queue reaches a maximum at the point t_2 where the slope of the demand curve is equal to that of the capacity line. The maximum queue (in terms of number of vehicles) is given by the vertical distance Q. The vehicle that enters the queue at time t_2 will exit from it at time t_3, encountering a total delay time t_d. As the vehicle flow associated with demand decreases with respect to the bottleneck capacity, the queue will begin to dissipate until it is finally eliminated at time t_4.

QUANTIFICATION OF QUEUE CHARACTERISTICS

The simulation of these queue characteristics (the number of vehicles in the queue, delay times, average length of the queue, and time to dissipate the queue) requires a time-dependent treatment of road demand and capacity relationships. In addition, it should be recognized that both demand and capacity in the approach zone may vary along roadway length. For simplicity, however, the literature sources assume that demand and approach capacity are constant over the roadway length in question. For the time being, we will observe this limitation in discussing the basic equations to simulate congestion. Then, in the following section, we will relax this restriction to develop the more general approach required in the EAROMAR simulation.

Figure 73 illustrates a section of roadway on which queueing is to be analyzed for the easterly traffic flow. The bottleneck section will cause the queue, and corresponds to the region of "zone speed" defined in Butler's speed profiles earlier. Immediately upstream from the bottleneck is the congested zone (identified by the shaded area in Figure 73), corresponding to the region of "queue speed" in the speed profiles. Lying beyond the congested zone is the upstream approach section, corresponding to the region in which "approach speed" governs in the speed profiles.

FIGURE 72
Traffic Volume-Capacity Relationships As
Queueing Occurs

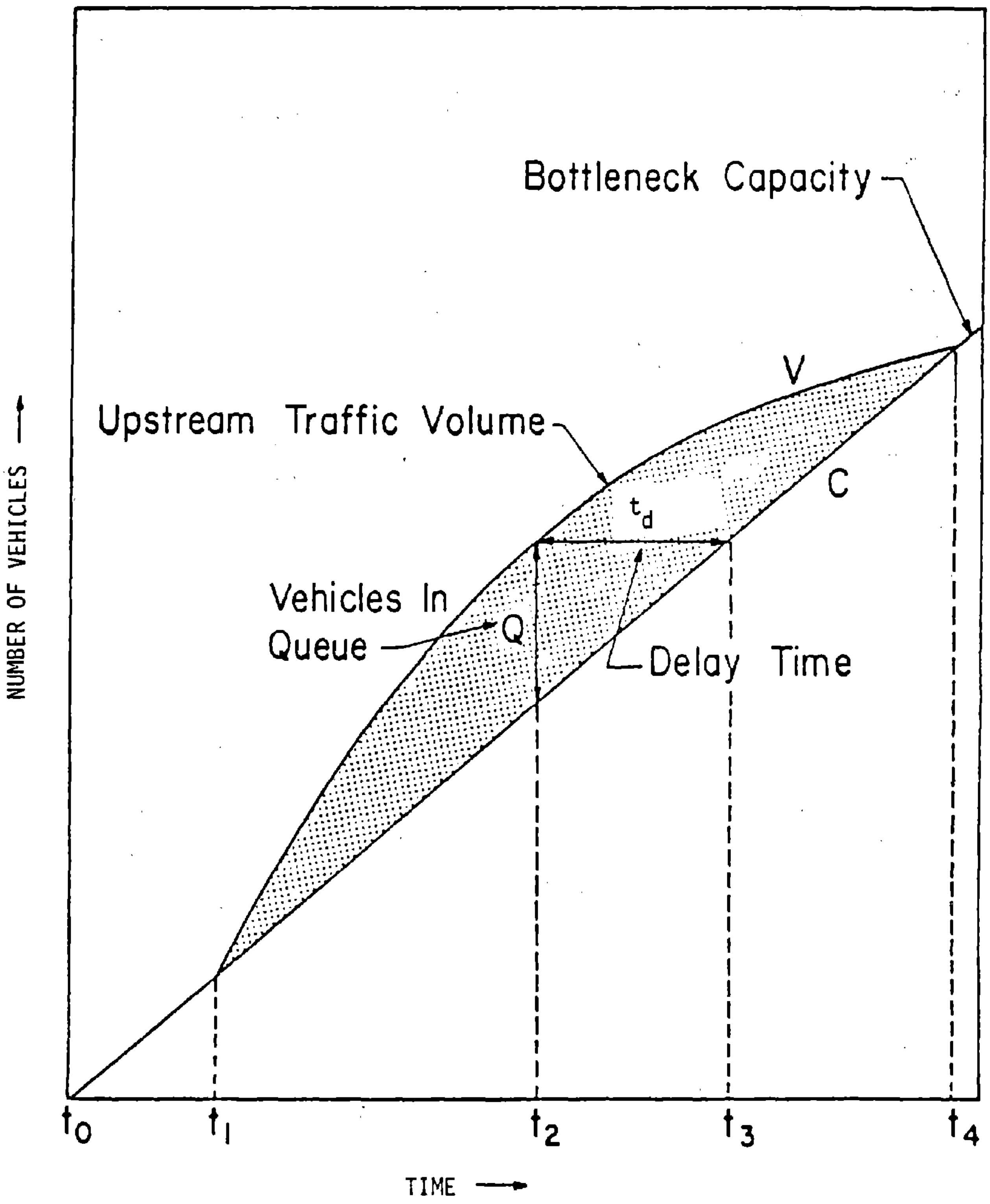
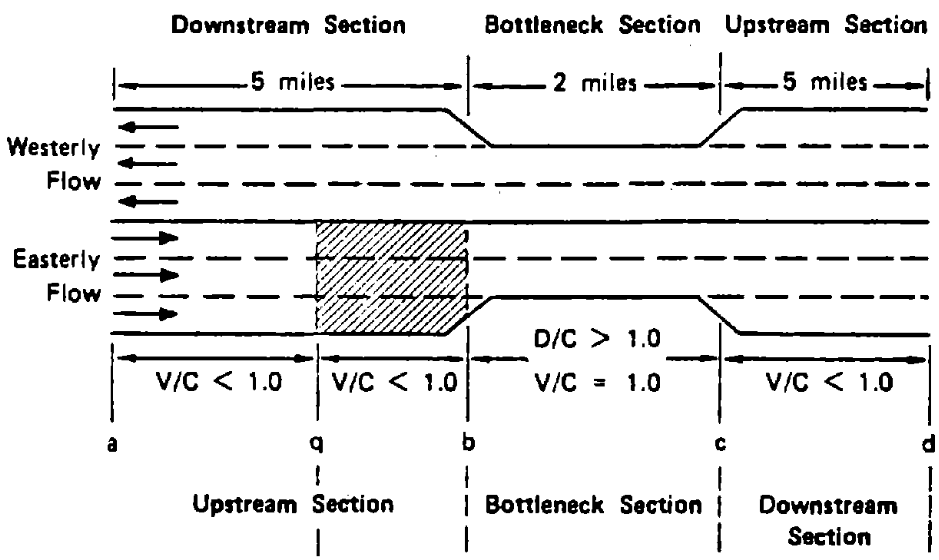
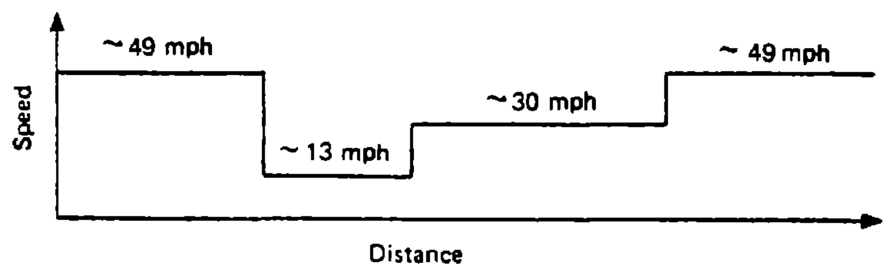


FIGURE 73

Schematic of Roadway Section with Queueing



(a) HIGHWAY SECTIONS



(b) AVERAGE SPEED PROFILE OF EASTERLY FLOW IN PEAK PERIOD

(Source: NCHRP Report 133, p. 14) (57)

Analysis of this situation may be accomplished by two different but equivalent approaches: the shock-wave method, and the deterministic method.

Shock-wave Method. The shock-wave analogy holds that during its buildup (e.g., from t_1 to t_2 in Figure 72), the queue boundary will progress upstream (counter to the flow of traffic) encompassing ever greater numbers of approaching vehicles, similar to the advance of a shock wave in a fluid medium. The speed of the shock wave is determined by the difference in vehicle flows (vehicles per hour) between the approach and congested sections; or:

$$W_t = (-\Delta q_t \ / \ \Delta k_t \ ) \tag{253}$$

where: W_t = speed that the shock wave travels upstream during time period t, mph

Δq_t = difference in vehicle flow during time period t between congested section of roadway and the approach section, vehicles per hour

and Δk_t = difference in traffic density during time period t between congested section and approach section, vehicles per mile.

The average queue length that each vehicle will experience during a time period of T_t hours is:

$$\ell_Q = W_t T_t / 2 \tag{254}$$

The total time to dissipate the queue (equivalent to the interval t_2 to t_4 in Figure 72) is TDQ in hours, estimated as

$$TDQ = \Delta q_t \ T_t \ / \ \Delta q_{t+1} \tag{255}$$

where: T_t = duration of queue buildup

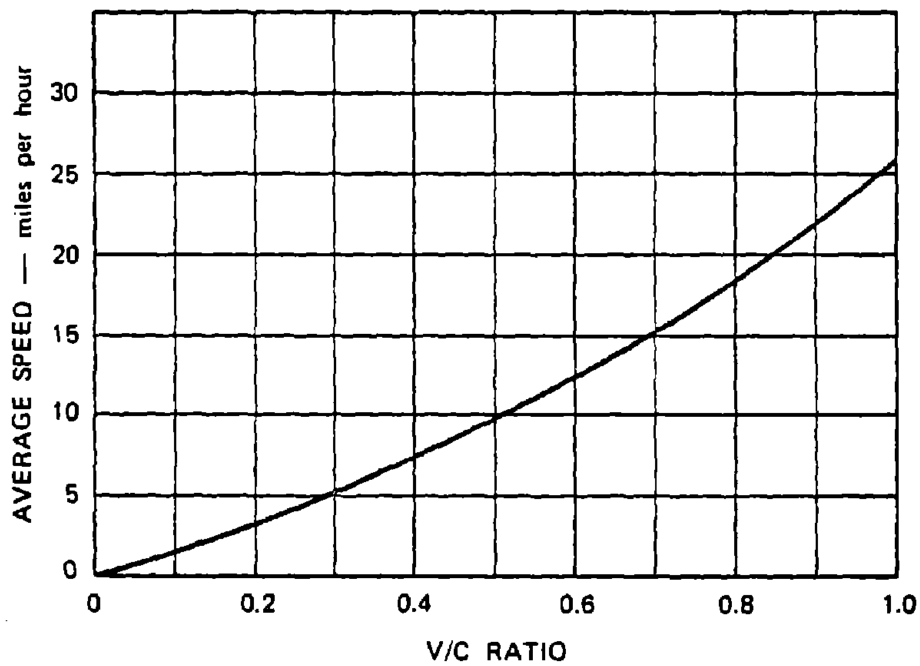
Δq_t = difference in vehicle flows between congested and approach sections during queue buildup

Δq_{t+1} = difference in vehicle flows between congested and approach sections during queue dissipation.

Deterministic Method. The deterministic method conceptualizes traffic as a continuous fluid arriving through the approach zone at flow rate q_1, and released through the bottleneck section at a flow rate q_m. If the arrival rate exceeds the departure rate ($q_1 > q_m$), a queue will form and build. At some later time the arrival rate q_2 will be less than the departure rate ($q_2 < q_m$) and the queue will dissipate.

Figure 74

Average Speed Versus V/C Ratio for
Level of Service



(Source: NCHRP 133, p. 22) (57)

In the following formulation we require the maximum density per lane K_m for queued vehicles. This may be derived from the general traffic stream model in eq. (238) earlier:

$$K_m = \frac{\text{Maximum vehicle flow through queue}}{\text{Speed in queue zone}} \tag{256}$$

The maximum vehicle flow through the queue is determined by the bottleneck capacity estimated from equations (232), (251) or (252). The speed in the queue zone can be estimated from Figure 74, which was developed from observations and research conducted by Curry and Anderson and reported in (57).

The rate of vehicles arriving in the queue during queue buildup is then

$$q_1' = q_1 \left[1 + \frac{q_1 - q_m}{(AS \cdot K_m) - q_1} \right] \tag{257}$$

The rate of queue buildup q_1' is slightly greater than the flow of vehicles approaching the queue q_1 because the queue boundary itself has a non-zero upstream speed (as shown in the shock-wave analogy earlier). If the time of queue buildup (e.g., from t_1 to t_2 in Figure 72) is T_t, the total number of vehicles entering the queue by the end of T_t is

$$N_t = T_t(q_1 - q_m) \tag{258}$$

The time TDQ required to dissipate the queue is:

$$TDQ = T_t(q_1 - q_m) \ / \ (q_m - q_2) \tag{259}$$

(Note the analogy to eq. (255))

and, for the average number of vehicles in the queue $N_t / 2$, the average time spent in the queue is

$$TSQ = \frac{(N_t \ / \ 2)}{q_m} \tag{260}$$

Finally, the average length of the queue is

$$\ell_Q = N_t \ / \ 2k_m \tag{261}$$

(Compare with eq. (254).)

Approach Used Within the EAROMAR Analysis

GENERAL

The preceding discussions of congestion and queuing relationships -- particularly relating to Figures 72 and 73 -- have assumed that both the demand and the approach capacity on the roadway under consideration are uniform along roadway length. Furthermore, eqs. 253 through 261 were developed based on queuing analyses that considered only two levels of demand: peak demand (which causes queue buildup during time T_t), and off-peak demand (which results in queue dissipation during time TDQ).

However, the EAROMAR system design -- and specifically the description of road characteristics in Chapter 2 and the development of demand representation earlier in this chapter -- indicates that these assumptions are too restrictive. Both demand and capacity may vary arbitrarily along route length to define a set of roadway sections; there is no reason to assume that bottleneck, queue, or approach zones would not traverse more than one such roadway section. Furthermore, traffic demand is defined to the hourly level, rendering a "peak / off-peak" distinction as too simplistic. What is required is a tailoring of the theoretical concepts above to the EAROMAR simulation design.

This adaptation is illustrated in the series of flow charts in Figures 75 through 80 . This simulation of road operations is performed within each road section for each hour of the day, consistent with other aspects (e.g., maintenance scheduling) of the EAROMAR design. Within each section average characteristics are computed in terms of demand, capacity, speed, and (if queuing occurs) average length of the queue within an hour.

The simulation is of a one-way type heading upstream. Operational results may be influenced by conditions in the given section or in downstream sections; however, once results are computed, they will not be adjusted for any influences that may lie upstream of the section in question. The simulation therefore accounts for the limiting effects of bottleneck capacities on all affected upstream sections, as well as for continuation of queues through several contiguous sections within an hour, and from hour to hour.

The following paragraphs describe the extention of concepts for queuing and congestion, to the EAROMAR simulation represented in Figures 75 through 80 .

RELATIONSHIPS

The relationships to estimate demand, capacity, and free-flow speed developed earlier in this chapter remain valid for the simulation of congestion. It should be noted, however, that within the simulation these factors must be computed for each hour of the day within each roadway segment.

Within a given hour, traffic conditions in a roadway segment may be influenced by conditions (for the same hour) in the segment immediately downstream. For example, if the downstream segment contains a bottleneck or is fully congested, then the queue will extend through at least a portion of the segment under consideration. We then speak of a segment as "dependent" on downstream effects. On the other hand, where no queuing exists immediately downstream, the segment is independent of downstream effects. Whether or not the segment itself causes congestion upstream depends on its demand-capacity ratio in that hour.

Congestion occurs when the traffic demand within a segment exceeds its hourly flow. Flow may be constrained by the capacity of the segment itself, or by capacity limitations downstream. The capacity limitations may be inherent in the road geometric design, in which case one would expect to observe congestion in the day-to-day roadway operations. On the other hand, a local, temporary capacity limitation may arise from occupancy of the roadway for maintenance or rehabilitation, in which case any additional congestion due to the workzone would also be simulated.

Central to the simulation approach is the following concept: where congestion occurs there is excess demand that is not able to flow through a segment in a given hour. The implications of such excess demand extend in dimensions of both space and time. For example, that portion of demand that is not able to flow through the segment in the specified hour must necessarily clear the segment in some later hour. This simulated delay is analogous to the variable t_d in Figure 72 ; the portion of the hourly demand subject to this delay is D_x in Figures 75 through 80 .

The fact that these delayed vehicles do not flow through the segment in this hour means that they must be "stored" somewhere in the highway system. One likely storage area is the segment in question. However, if the segment does not have the capacity to accommodate all queued vehicles, then some excess queued vehicles must be "passed" to the segment immediately upstream. These excess queued vehicles are identified by the variable XN in Figures 75 through 80 .

It is very important to realize that when we speak of "delayed" or "queued" traffic, we are in fact referring to only <u>one</u> set of vehicles. The reason we have defined <u>two</u> variables, D_x and XN, is to account for the effects of congestion in <u>two</u> dimensions -- time and space, respectively. It follows that the variables D_x and XN cannot serve duplicate functions in calculations of time and space effects; otherwise, the simulation would effectively double count queued vehicles.

Within EAROMAR, the time effects represented by the delay to D_x vehicles are employed in the user cost calculations, particularly those relating to travel time. The space effects incorporating the excess queued vehicles XN are used to identify the length of queue and the number of vehicles in the queue, and thus to determine what segments are affected by the queue.

Calculation of D_x within the simulation of congestion in Figures 75 through 80 is straightforward and requires no further explanation. The treatment of space effects, however, is somewhat more complicated because traffic demand varies along the roadway length, causing discontinuities in the rates of queue buildup among several continuous segments. Within a link analysis such as that employed in EAROMAR, there is no theoretically preferred way to handle this problem; we have thus approached the solution in the following way.

Where a difference in hourly demand equal to DQ exists between two contiguous segments, then DQ traffic must enter or exit the roadway at the interchange implied between the two segments. If congestion exists in both segments, we could make some statements about its effect on the traffic known to be on the roadway itself (since we have relevant volume and capacity information); but we cannot make a priori statements about its effect on the traffic entering or leaving the roadway, since there is no information on the characteristics of the collector or distributor network. An arbitrary but reasonable assumption is that on average during the entire hour, one-half of the difference in demand, or DQ/2, experiences queuing on the roadway proper,* and therefore must be included in the simulation of roadway congestion. The other half (or the remaining DQ/2 component) is assumed to be "stored" on entrance or exit ramps, or in general in other parts of the collector/distributor network, and thus does not experience congestion on the roadway in question. (Again, this does not mean that there is no congestion in other parts of the network. It simply means that we have no information whatever on this subject.)

Under this approach the following relationships for queuing may be adapted from analyses reported in the literature. For a given roadway segment within a given hour the number of vehicles may be computed by:

$$N = DQ/2 + XN' + D_{tot} - V, \quad N \geq 0. \qquad (262)$$

where N = total number of vehicles in queue;

DQ = difference between hourly demand in this segment and hourly demand in downstream segment;

XN' = excess number of vehicles passed from downstream segment;

D_{tot} = total hourly demand in this segment;

and V = flow through this segment.

*I.e., this traffic entered the roadway during the first half-hour and began immediately to experience the congestion; or, it did not exit the roadway until the last half-hour, and had experienced congestion up to that time.

Then the length of queue may be estimated by:

$$L = N/2K \qquad (263)$$

$$K = C/30 \text{ mph} \qquad (264)$$

where the equation for density K represents a conservative approximation of eq. (256). The excess number of vehicles in the queue is computed by:

$$XN = N - 2 \cdot L_s \cdot K, \quad XN \geqq 0 \qquad (265)$$

where L_s = length of this segment.

Finally, the speed in congested flow is computed by a linear approximation to the curve in Figure 74. The equation used is:

$$\text{Congested Speed} = 30 \cdot Q_m / C \qquad (266)$$

where Q_m = the limiting capacity or flow governing this segment (dependent upon the capacity of this segment and upon limiting capacities downstream).

FIGURE 75

Simulation of Road Operational
Dependencies

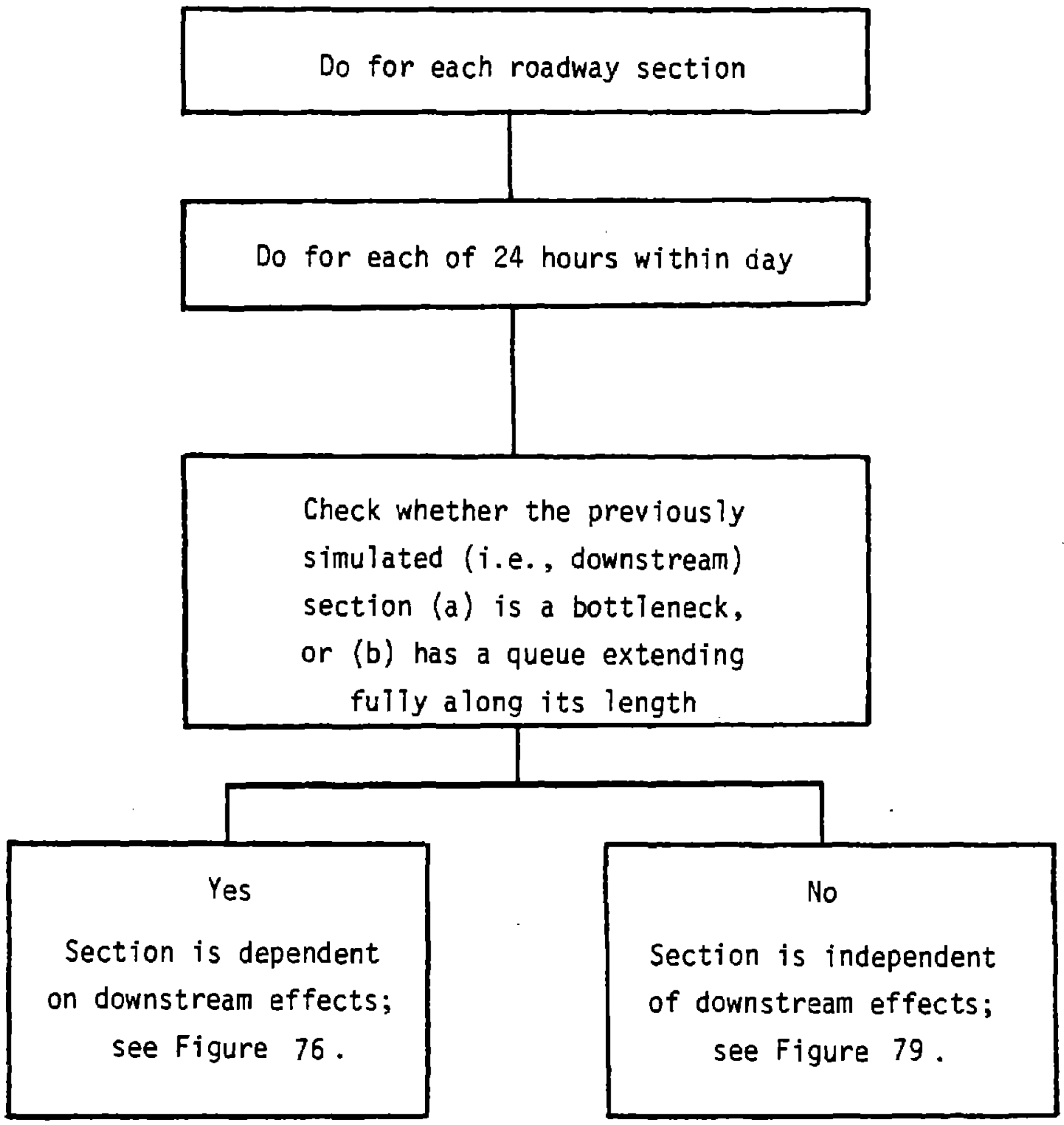


FIGURE 76

Simulation of Roadway Section
Dependent on Downstream Effects

SECTION DEPENDENT ON
DOWNSTREAM EFFECTS

Section dependent upon following information from downstream:

A. Limiting or bottleneck capacity Q_m

B. Excess vehicles in queue XN

Information passed from previous hour within this segment:

A. Excess demand D_x

Compute hourly section demand D_{tot} as sum of:

A. Normal demand D (e.g., eq. 226)

B. Excess demand D_x (if any) from previous hour within this section

(Continued on next page)

(FIGURE 76 , cont.)

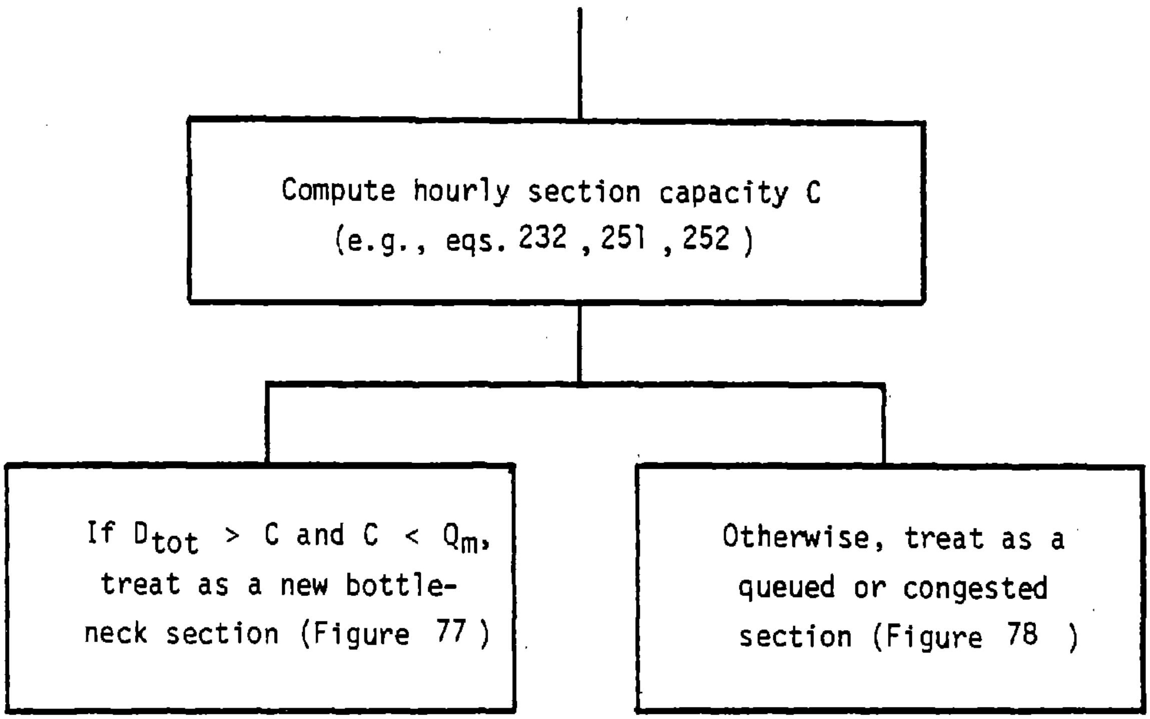


Figure 77

Simulation of Bottleneck Sections

BOTTLENECK SECTION

Determine limiting capacity $Q_m = C$

(pass to upstream section)

A. Set $V = C = Q_m$; $V/C = 1.0$

B. Excess demand $D_x = D_{tot} - Q_m$

(pass to next hour within section)

C. Excess number of vehicles in queue $XN = D_{tot} - Q_m$

(pass to upstream section)

Set speed through bottleneck = 30 mph

Continue processing to next hour or section.

FIGURE 78

Simulation of Congested Sections

QUEUEING PRESENT, OVER AT LEAST PART OF SECTION LENGTH

Compute limiting capacities and hourly flows:

$$Q_m = \text{MIN}\ (Q_m\ ,\ C)$$

(pass to upstream section)

$$V = \text{MIN}\ (Q_m,\ D_{tot})$$

Compute effective volume-capacity ratio as:

$$V/C = Q_m/C$$

Compute congested speed from eq. (266).

Compute N, number of vehicles in queue, using eq. (262)

Compute length of queue, L, using eqs. (263, 264).

(Cont. next page)

(Figure 78 , cont.)

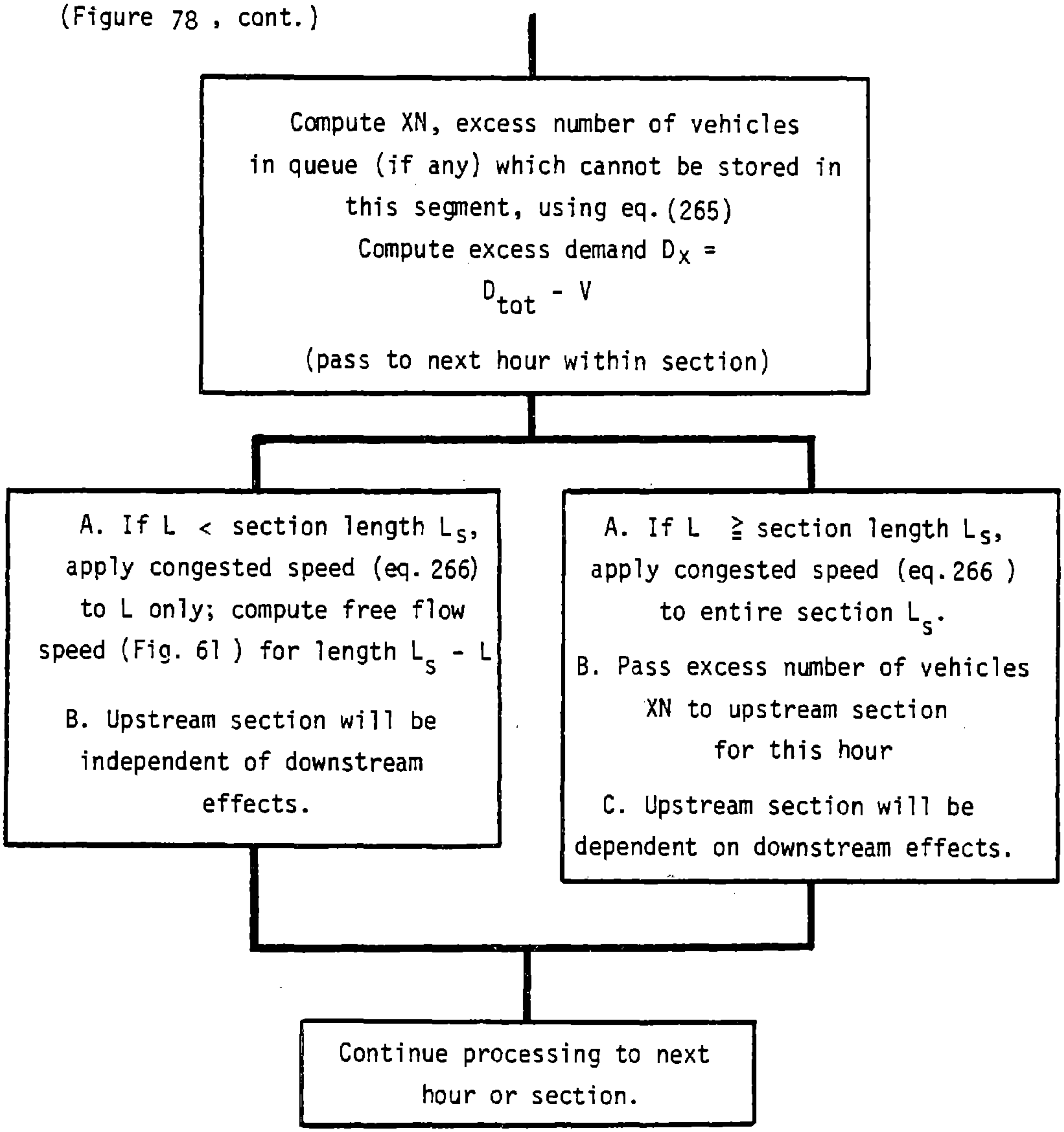


FIGURE 79

Simulation of Roadway Sections
Independent of Downstream Effects

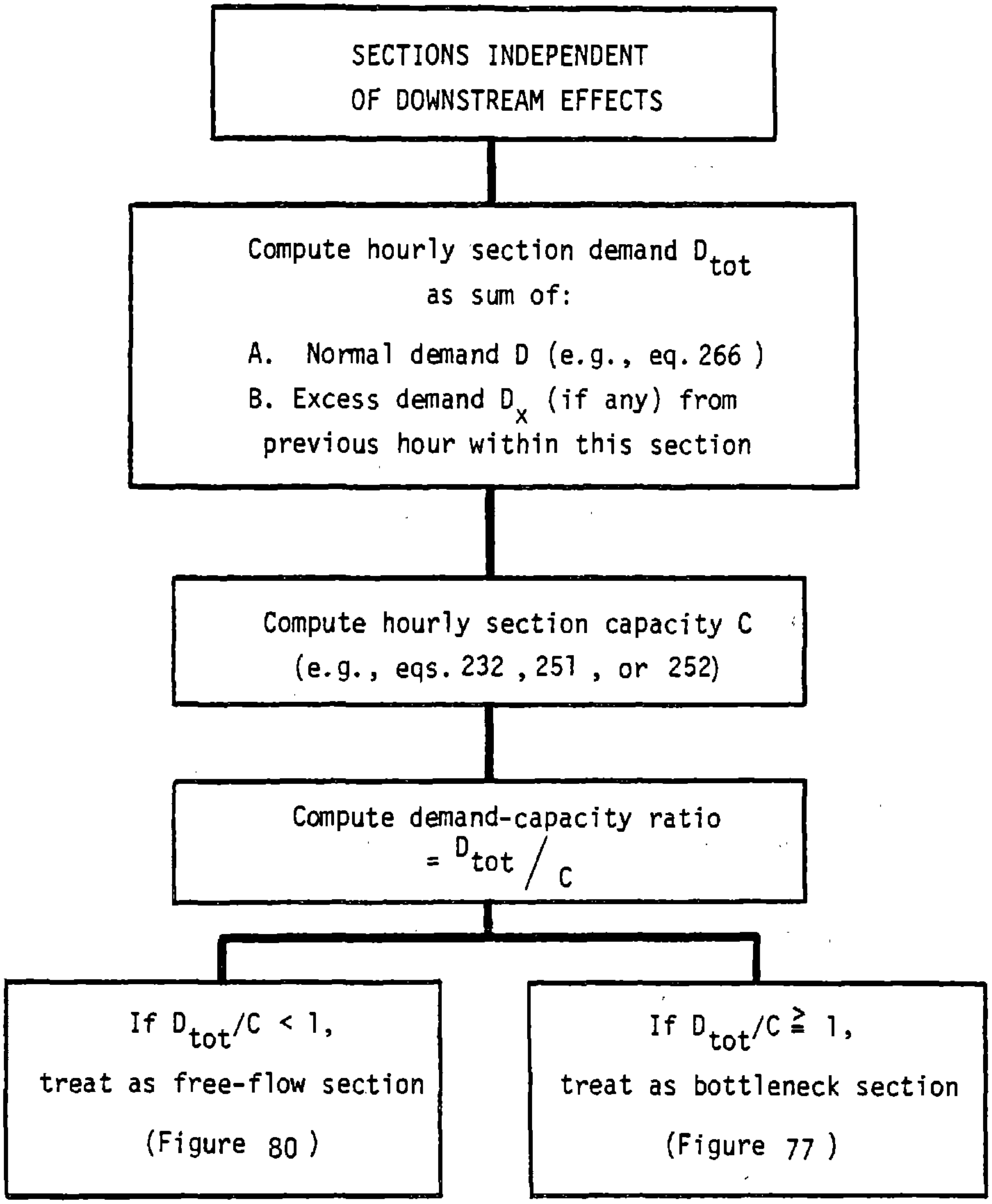
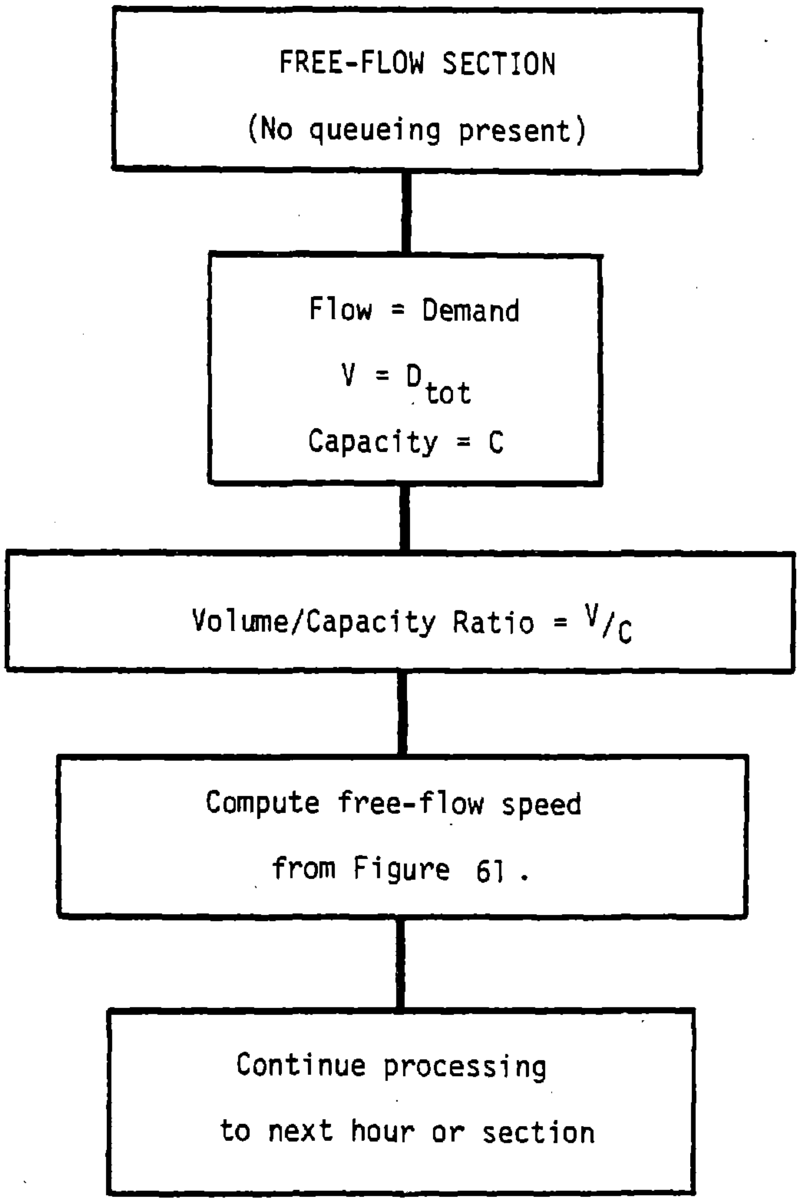


FIGURE 80

Simulation of Free-Flow Roadway Sections



Notes to CHAPTER 5

Note 1.

Our objectives within the EAROMAR system are simply to represent future traffic patterns as opposed to predicting them. Thus, although various models exist in the literature having future travel demand as an output (e.g., macroeconomic models; transportation planning models [as related to land use, activity distribution, or population changes]; and disaggregate demand models), they are beyond the scope of the current EAROMAR analysis. Furthermore, although they could conceivably be coupled with an EAROMAR-type analysis in the future, there are several intermediate issues that would need to be addressed: (1) validation of demand model predictions; (2) data availability; (3) scope of model (urban versus intercity projections); and (4) need for network (as opposed to link) analysis. Until these side issues can be resolved, and the EAROMAR analysis made compatible with assumptions of the demand prediction models, the aggregate, empirical-based methods described in this chapter are considered the most appropriate way to treat demand. They require independent determination by the user of demand volume and characteristics through the analysis period, derived from planning estimates, extrapolation of current trends, or best judgement of future conditions. Where uncertainty exists in these projections, the EAROMAR analysis may be repeated to investigate sensitivity to demand characteristics.

CHAPTER 6

USER CONSEQUENCES

The user consequences due to roadway maintenance occupancy include increases in operating costs and travel times, changes in accident potentials, and increases in pollution emission rates. To effectively evaluate these impacts requires the development of functional relationships between the user consequences given in Table 67, to each of the highway and traffic parameters in the righthand column. The availability of empirical relationships relating the independent and dependent variables will be discussed below, where each of the major user consequence item categories is described separately.

6.1 VEHICLE OPERATING COSTS

Generally, vehicle operating costs are divided into two categories: running costs, which include fuel, oil, tires, parts, and maintenance labor hours; and fixed costs, which include depreciation, insurance, registration, overhead, and so on. Numerous studies have been conducted both here and abroad which have attempted to relate the physical resources consumed under the above classification of running costs to specific parameters of highway design and traffic characteristics, usually within strickly controlled experiments or operator surveys. On the other hand, analysis of the components of fixed costs, primarily depreciation, are typically based on assumptions of lifetime speed/kilometrage relationships, average life expectancies, or vehicle age spectrums. The costs of insurance, overhead, and licensing usually present little difficulty to the analyst, as they are normally available from standard records.

However, the question arises as to which of the above parameters are sensitive to the policies incorporated within the EAROMAR Analysis, and to what extent. These policies, reflecting different pavement construction and maintenance alternatives, affect operating costs only locally, for the duration of the maintenance occupancy, through changes in operating patterns such as speed changes and braking, and throught the extra wear and tear caused by the poor pavement condition prior to the maintenance operations. Roadway occupancy for maintenance can hardly be expected to cause significant impacts to the overall annual or lifetime performance of any particular single vehicle (although the absence of such maintenance activity may, as through a sprung suspension); therefore, the fixed costs in the above categorization are not affected by the alternatives available in an EAROMAR analysis, at least within the framework of current understanding as regards the relationships between road condition, vehicle maintenance, and vehicle depreciation. These points will be elaborated upon in later sections.

TABLE 67

USER IMPACTS OF

HIGHWAY DESIGN AND TRAFFIC PARAMETERS

USER CONSEQUENCE	HIGHWAY & TRAFFIC PARAMETERS
Operating Costs:	Distance
Fuel Consumption	Plus Gradients
Oil Consumption	Minus Gradients
Tire Consumption	Horizontal Curvature
Spare Parts Consumption	Roadway Capacity
Maintenance Labor Hours	Road Surface Conditions
Depreciation Costs	Speed
Travel Time Costs	Speed Changes
	Hourly Traffic Volume
Accident Costs	Traffic Composition
	Vehicle Characteristics
Pollution Levels	Value of Travel Time

Fuel Consumption

No other single item of motor vehicle operating costs had been studied more than fuel consumption. Variables which have been assumed by various investigators to affect fuel consumption rates include gradients, curvature, roughness, tractive resistence, speed, speed changes, gross vehicle weight, horsepower, elevation, wind velocity, vehicle cross section, and tire preseure, among others. Not all investigations have determined the same significance for these parameters, and several parameters have proven insignificant in all cases studied.

Most investigators have modeled fuel consumption as consisting of a base rate on tangent sections which is a function of grade and uniform speed, with correction factors or increments to account for the other features of highway design, such as curvature, volume/capacity ratio, vehicle weight, road condition, and speed changes.

The most comprehensive data source appears to be Claffey (58) and this forms the basis fo rthe development of the internal fuel consumption regression relations. The basic fuel consumption rates on level tangents at uniform speed are modeled as follows, by vehicle type;

$$FL_G = A + B/S + CS^2 \qquad (267)$$

where B is the idle consumption rate, in gallons per thousand hours
S is the vehicle speed, in miles per hour
FL_G is the fuel consumption rate, in gallons per thousand miles
and A and C are functions of the gradient, as follows:

$$C = B/(2S_o^3) \qquad (268)$$

$$A = F_o - 3B/(2S_o) \qquad (269)$$

where S_o is the speed at which the fuel consumption rate is minimal
F_o is the minimal fuel consumption rate.
Expressions for S_o and F_o, and hence for A and C and FL_G as well, have been formulated as follows:

$$S_o = a_o + a_1 G^{a2} \qquad (270)$$

$$F_o = b_o + b_1 G^{b2} \qquad (271)$$

Regression analysis performed on the Claffey data using the above relations provided the constants and correlations presented in Tables 68 and 69 . The r^2 values are for all speed-grade combinations presented in NCHRP 111, and the extremely high correlation indicates that equations (267) through (271) provide a sound theoretical basis for any further regressions on new fuel consumption data. Comparisons with

TABLE 68. Coefficients for Fuel Consumption on Positive Gradients

	Car	Pickup	Six-Tire	Semi-Trailer
B	580	450	650	840
a_0	30	25	20	35
a_1	0	0	-0.024	-5.51
a_2	0	0	2.821	0.806
b_0	44	47	59	163
b_1	6.8	9.75	22.64	84.4
b_2	1.177	1.04	1.11	1.13
r^2	0.98	0.98	0.97*	0.97*

*for $G < 9\ \%$

TABLE 69. Coefficients for Fuel Consumption on Negative Gradients

	Car	Pickup	Six-Tire	Semi-Trailer
B	580	450	650	840
a_0	30	25	20	35
a_1	0.0000444	0.00106	0.067	0.0038
a_2	6.285	4.19	2.96	3.52
b_0	44	47	59	163
b_1	-14.2	-12.6	-12.62	-64.34
b_2	0.445	0.512	0.552	0.344
r^2	0.93	0.95	0.96	0.85

the Claffey data are given in Figures 81 and 82.

To correct for curvature effects, the following regression relations were developed:

$$F_c = 1 + a(S \cdot C^{0.75})^b \quad (272)$$

where F_c is a multiplicative correction factor;
S is the vehicle speed, in miles per hour;
and C is the curvature, in degrees.

The constants obtained from the regression are as given in Table 70 .

The effects of speed change cycles were modeled as follows:

$$\Delta FL_{sc} = a(S \cdot \Delta S^{1.44})^b \quad (273)$$

where ΔFL_{sc} is the excess fuel consumed during a speed change cycle;
S is the initial speed;
ΔS is the magnitude of the speed change.

The constants obtained from the regression are as given in Table 71.

For roughness-fuel consumption relations, the following form was used:

$$F_R = 1 + aS^b\left(\frac{4.5 - PSI}{3.0}\right) \quad (274)$$

where F_R is a multiplicative roughness factor;
S is the vehicle speed;
and PSI is the road serviceability index (where 4.5 is considered perfect and 1.5 is considered badly broken).

The constants obtained from the regression are as given in Table 72 .

Thus, the proposed fuel consumption relation would be:

$$FL = FL_G \cdot F_C \cdot F_R + \Delta FL_{SC} \quad (275)$$

where FL is the total fuel consumption rate;
FL_G is the base rate as a function of grade and speed;
F_C is the curvature correction factor;
F_R is the roughness correction factor;
and ΔFL_{SC} is the excess due to speed change.

A relation of the above type is incorporated for each of the four vehicle types presented in NCHRP 111 (58).

Figure 81. Fuel Consumption on Positive Grades
Passenger Car

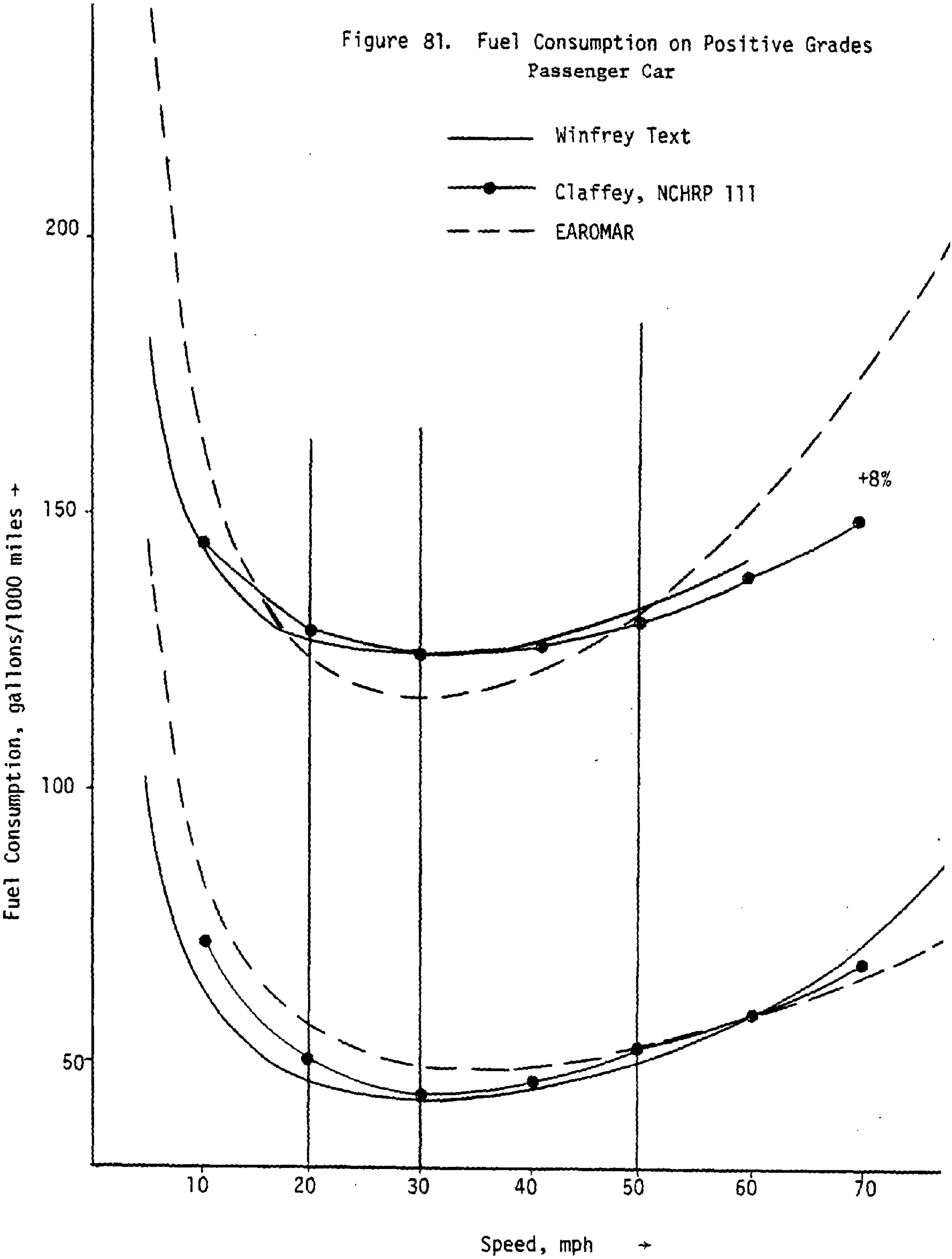


FIGURE 82. Fuel Consumption on Positive Grades

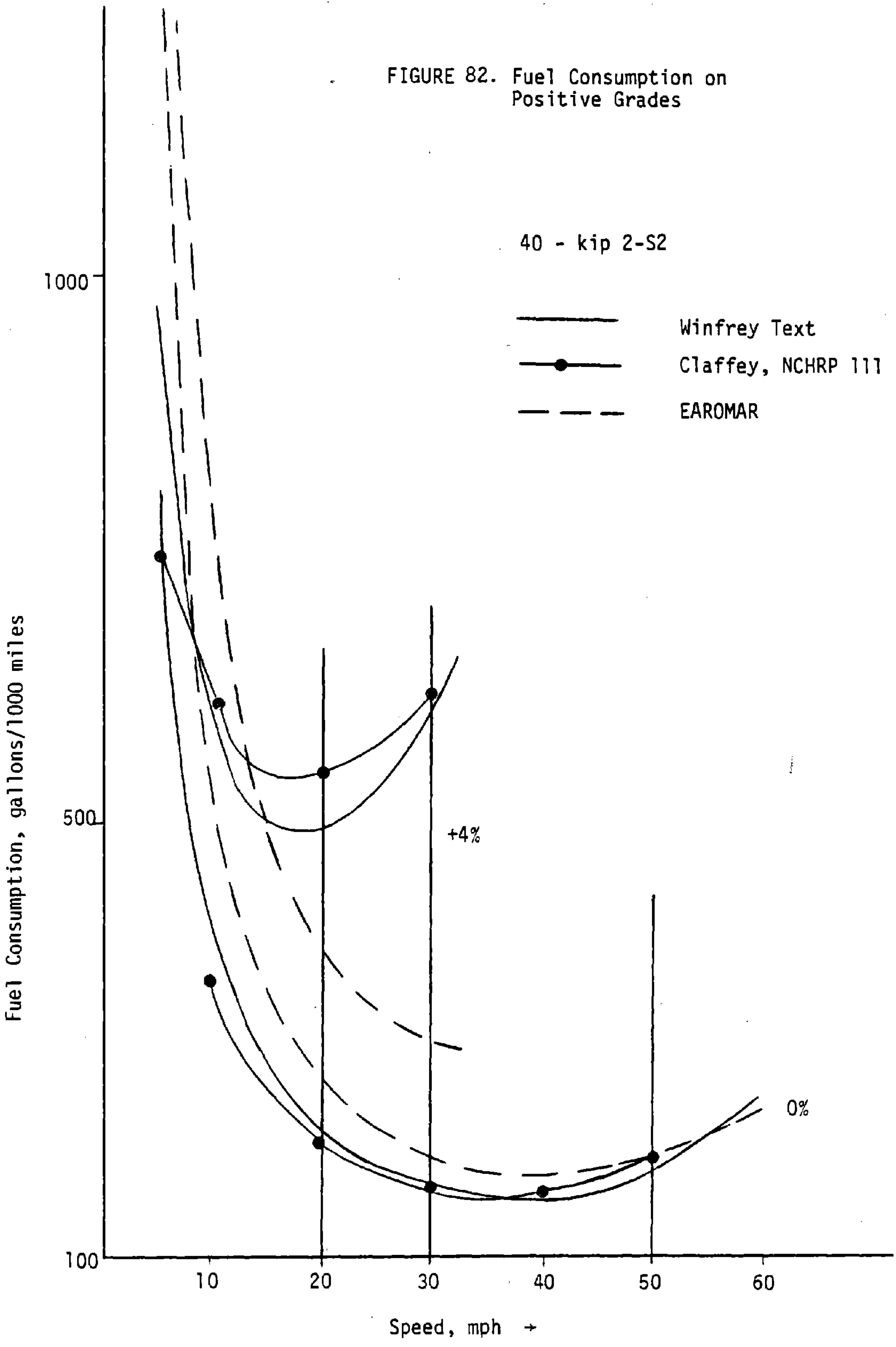


TABLE 70

COEFFICIENTS FOR FUEL CONSUMPTION
ON CURVES

	a	b	r^2
Car	$5.00\ 10^{-7}$	2.49	0.90
Pickup	$1.02\ 10^{-9}$	3.52	0.86
Six-Tire	$3.69\ 10^{-8}$	3.02	0.94
Semi-Trailer	$2.65\ 10^{-6}$	2.14	0.83

TABLE 71

COEFFICIENTS FOR FUEL CONSUMPTION
DURING SPEED CHANGES

	a	b	r^2
Car	.00007784	0.56	0.99
Pickup	.000096	0.51	0.86
Six-Tire	.000279	0.50	0.96
Semi-Trailer	.000615	0.62	0.94

TABLE 72

COEFFICIENTS FOR FUEL CONSUMPTION
INCREASES DUE TO ROUGHNESS

	a	b	r^2
Car	$3.10\ 10^{-5}$	2.50	0.99
Pickup	$2.60\ 10^{-10}$	5.08	0.85
Six-Tire	0.29	1.00	0.85
Semi-Trailer*	0.29	1.00	0.85

*No data presented, so we are assuming same relation as for six-tire truck.

TABLE 73

COEFFICIENTS FOR FUEL CONSUMPTION
AS AFFECTED BY VEHICLE WEIGHT

	CAR	5-kip	12-kip	40-kip
a	0.29	0.29	0.19	0.05
W_0	4	5	12	45

For diesel fuel consumption rates, NCHRP 111 presents gasoline-diesel conversion factors for various truck weights and positive grades. Since no data were presented for negative grades, it was decided to use a gasoline-to-diesel conversion factor of 0.67 for all diesel vehicles.

$$F_D = 0.67 \text{ for diesel vehicles.}$$

$$F_D = 1.00 \text{ for gasoline vehicles.}$$

The factor F_D would then be used to convert total gasoline consumption for a road section to total diesel fuel consumption, for each diesel vehicle type.

A linear fuel consumption/vehicle weight relation was developed from NCHRP 111, as follows:

$$F_W = 1 + a(W - W_0). \tag{276}$$

These factors are shown in Table 73.

The final fuel consumption relation is then as follows:

$$FL = (FL_G \cdot F_C \cdot F_R + \Delta FL_{sc}) \cdot F_D \cdot F_W. \tag{277}$$

Oil Consumption

While the cost of lubricating oil is not generally a major component of running costs, numerous studies have nevertheless attempted to determine functional relationships between oil consumption rates and highway design and traffic parameters. Variables which have been assumed by various investigators to affect oil consumption rates include gradients, curvature, roughness, tractive resistance, speed, speed changes, and gross vehicle weight.

The proposed approach is to model oil consumption as being a proportion of fuel consumption, as follows:

$$OIL = FL \cdot (a_0 + a_1 S + a_2 S^2) \cdot FP/OP = FL \cdot (a_0' + a_1' S + a_2' S^2). \tag{278}$$

where OIL is the oil consumption rate in quarts per 1000 miles;
FL is the fuel consumption rate in gallons per 1000 miles;
S is the vehicle speed, in mph;
FP is the fuel price, in dollars per gallon;
and OP is the oil price, in dollars per quart.

The constants of regression were determined using Winfrey's (59) level tangent data, and are assumed to hold for other highway design conditions. The fuel consumption rate is computed as described in the previous section. The coefficients of the regression analysis are given in Table 74.

TABLE 74

COEFFICIENTS FOR OIL CONSUMPTION
AS A FRACTION OF FUEL CONSUMPTION

	CAR	5-kip	12-kip	40-kip	50-kip DIESEL
FP, \$/GAL.	0.23	0.22	0.20	0.18	0.16
OP, \$/QT.	0.60	0.55	0.40	0.20	0.20
a_0	0.193	0.168	0.102	0.01133	0.06
a_1	-0.00118	-0.00140	-0.00100	0.00125	0.005
a_2	0	0	0	-0.0001833	-0.0001
a'_0	0.0740	0.0672	0.0510	0.0110	0.0480
a'_1	-0.000452	-0.000560	-0.000500	0.001125	0.004000
a'_2	0	0	0	-0.000165	-0.000080
r^2	0.93	0.92	0.93	0.88	1.00
Speed Range } mph	10-60	10-50	10-50	10-50	10-50

Tire Consumption

Tire tread wear is difficult to measure. Since current tires are of high performance quality, it takes thousands of miles to produce enough wear to give reliable measurements. This means that it is difficult to measure tire wear resulting from localized features such as speed changes, braking, or travel around horizontal curves.

The basic tire consumption relation is derived from Winfrey's level tangent data. Corrective factors for speed changes, curvature, and road roughness are developed from Claffey's data. The basic regression model for speed effects was formulated as follows:

$$TC_s = aS^b/TP = a'\ S^b \tag{279}$$

where TC_s is the tire consumption rate due to speed on tangent sections, in tires per thousand miles;
S is the vehicle speed, in mph;
TP is the tire price, in dollars per tire;
and the coefficients of the regression are presented in Table 75.

Following Claffey, tire consumption was assumed to be independent of grade. The effects of curvature were derived from Claffey's data, with the following regression model being used:

$$T_c = 1 + a(S \cdot C^{0.42})^b \tag{280}$$

where T_c is the multiplicative speed-curvature correction factor;
S is the vehicle speed in miles per hour;
and C is the curvature, in degrees.

The results of the regression are found in Table 76. Since only passenger car data are presented by Claffey, the curvature factor was assumed to hold for all vehicle types.

The effects of speed changes were also developed from Claffey, using the following regression model:

$$T_{sc} = a(\Delta S)^b\ /\ TP = a'\ (\Delta S)^b \tag{281}$$

where T_{sc} is the tire consumption rate due to a speed change, in tires per thousand cycles;
ΔS is the magnitude of the speed change;
and TP is the tire price, in dollars per tire.

The results of the regression are presented in Table 77. Since only passenger car data are presented by Claffey, the above rates are assumed to hold for all vehicle types.

TABLE 75

COEFFICIENTS OF TIRE CONSUMPTION
ON TANGENT SECTIONS

	TP, $/TIRE	a	b	a'	r^2
CAR	23.00	0.0185	1.29	0.000804	0.98
5-kip	32.00	0.0269	1.22	0.000841	0.99
12-kip	66.00	0.0621	1.20	0.000941	0.99
40-kip	120.00	0.1098	1.26	0.000915	0.99
50-kip diesel	120.00	0.1490	1.25	0.001242	0.99

TABLE 76

COEFFICIENTS OF TIRE CONSUMPTION
ON CURVES

	a	b	r^2
ALL VEHICLE TYPES	$3.244\ 10^{-8}$	4.33	0.95

TABLE 77

COEFFICIENTS OF TIRE CONSUMPTION
DURING SPEED CHANGE CYCLES

	TP, $/TIRE	a	b	a'	r^2
ALL VEHICLE TYPES	23.00	0.695	1.27	0.03022	0.98

Roughness was found to affect tire consumption on tangents and during speed changes; using the Claffey data and assuming PSI values of 4.5 and 4.2, respectively, for concrete and asphalt pavements, the resulting multiplicative factors are:

$$F_R^S = 0.68 + 1.3(4.5 - PSI) \tag{282}$$

$$F_R^{sc} = 0.49 + 1.77(4.5 - PSI) \tag{283}$$

where F_R^S is the multiplicative roughness factor on tangent sections;

F_R^{sc} is the multiplicative roughness factor during speed changes;

and PSI is the road serviceability index.

Thus, the final tire wear model would be:

$$TC = (TC_S \cdot F_R^S \cdot T_C + T_{sc} \cdot F_R^{sc}) \cdot W/W_0 \tag{284}$$

for each vehicle type.

Maintenance Parts and Labor Costs

With respect to engineering economy analyses, the maintenance expense of a vehicle is an important item. It is affected by distance, gradients, horizontal curves, speed changes, and roadway surface condition. However, since the maintenance expenses of a vehicle and the actual wear of parts is so difficult to relate to these specific features of highway design, it has not yet been possible to measure by field testing the maintenance expense of the overall vehicle with respect to these particular features. To further complicate matters, the maintenance expense of the motor vehicle is highly dependent upon the owner's maintenance policy. Thus, maintenance is the one large item of motor vehicle operating expense which is most difficult to allocate to specific highway design features.

Winfrey relates maintenance cost to speed, while Claffey presents average cost per mile figures. We feel that maintenance cost is not the function of speed that Winfrey presents, but that it is much more related to time, mileage, and driver behavior. This would be represented in the model as a constant annual amount, and hence will not be considered in the economic analysis.

Vehicle Depreciation

Possibly no other single vehicle cost item has produced more theoretical argumentation with less empirical highway design evidence than has vehicle depreciation costs. While the generally accepted definition of total vehicle depreciation, the difference in the initial vehicle cost and its scrap value when it is removed from service, seems quite straightforward, there is little guidance in the literature pointing first to the proportion of total depreciation which might be chargeable to the mileage use of the vehicle and the remaining portion that should be charged to time or simply ownership of the vehicle. Moreover, there is less evidence still of the mileage portion of the total depreciation that should be charged to the factors of highway design and use, such as distance, vertical grades, horizontal curves, speed, speed changes, and roadway surface conditions.

With the exception of the TRRL Kenya Study (60), all the available depreciation relationships assume that the total mileage, and hence the per-mile depreciation, is a function of speed. But is this reasonable? Certainly, if we are talking of average annual speed, the notion would be highly plausible. But we in highway economy studies use a localized link speed reflecting the highway design parameters of condition and geometrics, and in the EAROMAR framework, capacity reductions due to road occupancy. We feel that annual mileage is much more reasonably determined by a driver's needs, travel budget, and preferences, rather than on a localized speed effect produced, possibly, by the performance of highway maintenance. This suggests, by the way, that perhaps a diversion paramater should be introduced to account for the reduced travel demand as maintenance is taking place. Given the above considerations and the general lack of quantitative support for the various approaches found in the literature, we propose that vehicle depreciation be taken as a constant annual amount, and hence need not be included in the economic analysis.

6.2 VALUE OF TRAVEL TIME SAVINGS

There is widespread agreement that the value of travel time savings is a major source of justification for road investments, particularly so for the high-volume highways that are the focus of this research. However, there is no such consensus on the appropriate conceptual and empirical approaches to determining these values. The studies reviewed suggest that the values of travel time savings tend to range from a high for working time, through business travel time and commuting time, to a low for non-work, non-commuting time; furthermore, a definite trend in technique is also emerging. Nevertheless, the following words of caution are found in a recent review of the evaluation of highway improvements:

> "It is advisable to treat travel time as a separate item in economy studies in order that the decision-maker can see readily the amount of overall gains that are priced out on the basis of the dollar value of time and those gains that are actual bona fide reductions in expenditure for travel." (61)

It is apparent that most studies have estimated time values as by-products of single or simultaneous travel choice and demand models, in which the emphasis was on prediction rather than on capturing the concept of the value of travel time. In the desired EAROMAR analysis, we will not be looking at travel demand, but rather at travel supply, for which the requirements of usable travel time are considerably different.

The SRI study by Thomas and Thompson (62) appears to remain as the best source of disaggregate data, including trip purpose and income level. However, it presents considerable difficulties by its introduction into the analysis. Once the value of time is a function of the amount of time saved, then clearly the total amount of time saved per trip must be known. Because motorists on short trips can be expected to have less total time saved by highway improvements than motorists on longer trips, road users would need to be segregated by their total trip length (and perhaps ultimately even by origin and destination).

Suddenly the highway economist needs more information than just the amount of time saved due to highway improvements and the volume of motorists and their incomes, leading directly to a network-level analysis. Since this is not anticipated at this time, some modifications to the SRI results must be developed.

Closed-form approximations of the SRI data have been obtained using a regression model of the form:

$$VTTS = (a_0 + a_1\ \Delta T)(b_0 + b_1\ I) \qquad (285)$$

where $VTTS$ is the value of the travel time savings, in dollars;
ΔT is the time saved, in minutes;
and I is the annual income of the motorist, in thousands of dollars.

The results are given in Table 78.

Since we are not following individual vehicle movements in the simulation, or even the actual time spent in the system by a representative vehicle, we will not know the total trip travel times by trip purpose. Hence, we must use marginal values of travel time in the analysis:

$$MVTTS = \frac{d(VTTS)}{d\Delta T} = a_1 b_o + a_1 b_1 I\ , \qquad (286)$$

and the expected value of marginal travel time savings is as follows:

$$E[MVTTS] = a_1 b_o + a_1 b_1 E[I] = \text{constant} \qquad (287)$$

Thus, the modeler has the choice of using the SRI marginal values and having the average income by trip purpose input by the user, or having

TABLE 78

COEFFICIENTS OF VALUE OF TRAVEL TIME SAVINGS

TRIP PURPOSE	a_0	a_1	b_0	b_1	r^2
VACATION[1]	-0.0360	0.0348	-0.3462	0.2144	1.00
WORK[2]	-0.0504	0.0269	-0.3121	0.1909	0.99
SOCIAL[1]	-0.3465	0.0700	-1.1910	0.2139	0.99
BUSINESS[1]	-0.2652	0.0421	-1.9800	0.5515	0.99
SCHOOL[2]	-0.4887	0.0735	-1.2000	0.1718	0.97

1. Value of time per vehicle
2. Value of time per person

the user input the marginal value of travel time savings directly. The latter is the approach we adopted for the updated EAROMAR analysis, i.e., the user will specify the value of time by trip purpose by vehicle.

6.3 ACCIDENTS

In a recent FHWA report (53), the relationships between construction type, road type, road closure type, and construction zone accident rates were investigated. The major conclusions drawn from the literature review of that study were:

(1) Although several alternative combinations of work area, roadway type and construction scheduling are currently used, little or no data are available to determine the safety effect of the choice of any particular alternative.

(2) There are two prevailing philosophies on speed control in construction zones. One maintains that speeds should always be reduced, while the other claims that normal highway operating speeds should be maintained throughout the zone. Again, there are no data to support the relative safety benefits of either of the philosophies.

(3) The application of traffic control devices in construction zones appears to vary widely between agencies and between construction projects, indicating a general lack of knowledge regarding the safety effectiveness of the various traffic control devices and their locational application.

(4) The high concentrations of accidents at interchanges and transition zones identify those roadway locations where extreme care must be exercised in the selection, utilization, and maintenance of construction zone traffic control devices.

(5) The daily management of traffic operations within a construction zone is an important factor in the safe operation of the zone.

Number of Accidents

The accident study portion of the research looked at the accident experience of construction zone roadways before and during construction. Data from seven states were used in the analysis. The construction data included the type of construction, length, duration, traffic volumes, and type of traffic controls used. Accident data were reduced into several catagories such as type, location, time of occurrence, and severity. The data included 79 projects of which 11 were on six- or eight-lane interstate highways, 24 on four-lane divided interstates, 18 on four-lane divided non-interstates, 3 on five-lane undivided, 4 on four-lane undivided, and 19 were on two-lane roadways. There were 31 urban projects and 48 rural projects. Eleven closure categories were covered.

Although comparative analysis using accident numbers provides some very useful information, the determination of the change in accident rates from the before to the during construction period is a more meaningful measure of the effects of construction. Although the researchers were able to obtain the necessary data to compute accident rates for the before period (length, duration, accident number, and traffic volumes), with the exception of two projects, they were unable to obtain traffic volumes during construction; the states simply did not have that data available. However, the analysis using accident rates provided a method of comparing before to during accidents using only documented accident data, although the lack of construction period traffic volumes forced the authors to compute construction accident rates using the before traffic volumes. They concluded that, overall, the construction projects probably had lower traffic volumes than in their respective before periods, indicating that the actual increase in construction accident rates is probably greater than their results indicate.

The measure taken to try to explain the reported accident rates was the ratio of number of lanes before construction to the number of lanes during construction. The mean accident rate increases for this measure of "interference" are presented in Table 79.

Table 80 presents the mean accident rates for the various work area roadway types, while Table 82 gives the mean accident rates for the various types of construction. Table 81 illustrates the percent increase in accident rates comparing urban and rural projects. Unfortunately, the authors do not present this data disaggregated to the single project level. A more detailed analysis of the disaggregate project data, if available from the authors, may permit a correlation of these closure types with the road degradation types summarized above. In the absence of such an analysis, the following relationship is recommended for incorporation into the roadway occupancy zone accident calculations:

$$\Delta ACC\ (\%) = -15.97 + 63.18\ N_n/N_c \qquad r^2 = 0.96 \qquad (288)$$

where ΔACC is the percentage increase in accident rate due to the roadway occupancy;

N_n is the number of lanes under normal operating conditions;

and N_c is the number of lanes available during the construction or maintenance operations.

This relation, and the data from Table 79, are plotted in Figure 83. which indicates the extremely high correlation of this closure measure

TABLE 79. EFFECT OF DEGRADING VARIOUS ROAD TYPES

Roadway Type	Number Projects	Original Roadway Mean Accident Rate (100 MVM)	Construction Roadway Mean Accident Rate (100 MVM)	% Change
6 or 8-lane Interstate reduced to 2 lanes each direction	8	193.96	204.23	+5.3
6 or 8-lane Interstate reduced to 1 lane each direction	3	227.91	489.22	+114.7
4-lane Interstate reduced to 1-lane each direction	22	136.16	229.55	+68.6
4-lane Interstate reduced to 2-lane, 2-way	2	40.68	100.58	+147.2
4-lane divided reduced to 1-lane each direction	5	314.73	361.46	+14.8
4-lane divided reduced to 2-lane, 2-way	5	177.09	205.16	+15.9
4-lane divided on new alignment	6	249.08	200.52	-19.5
4-lane undivided reduced to 2-lanes	3	801.46	761.99	-4.9
5-lane undivided w/TWLTL reduced to 2-lanes	3	488.25	776.14	+59.0
2-lane reduced to 1-lane	7	363.99	475.73	+30.7
2-lane on new alignment	11	636.77	545.55	-14.3
	75			

TABLE 80

MEAN ACCIDENT RATE BY TYPE OF CONSTRUCTION (100MVM)

TYPE OF CONSTRUCTION	NUMBER PROJECTS	MEAN ACCIDENT RATE BEFORE CONSTRUCTION	MEAN ACCIDENT RATE DURING CONSTRUCTION	PERCENT CHANGE
Resurfacing, Pavement Patching	26	147.73	159.04	+ 7.7
Bridge Work	5	88.27	132.47	+ 50.1
Median Barrier Work	15	187.16	203.87	+ 8.9
Widening of Existing Roadway	12	577.10	593.21	+ 2.8
Upgrading to Interstate Standards	9	167.65	194.65	+ 16.1
Reconstruction of Existing Roadway	2	278.98	371.97	+ 33.3
Construction of New Roadway (New Alignment)	5	213.49	214.27	+ 0.4
Other	1	137.36	137.36	-0-
	75			

TABLE 81. MEAN ACCIDENT RATE BY AREA TYPE

AREA TYPE	Mean Accident Rate (100MVM) Before	During	PERCENT CHANGE
Urban	273.54	304.70	+ 11.4
Rural	140.45	145.59	+ 10.1

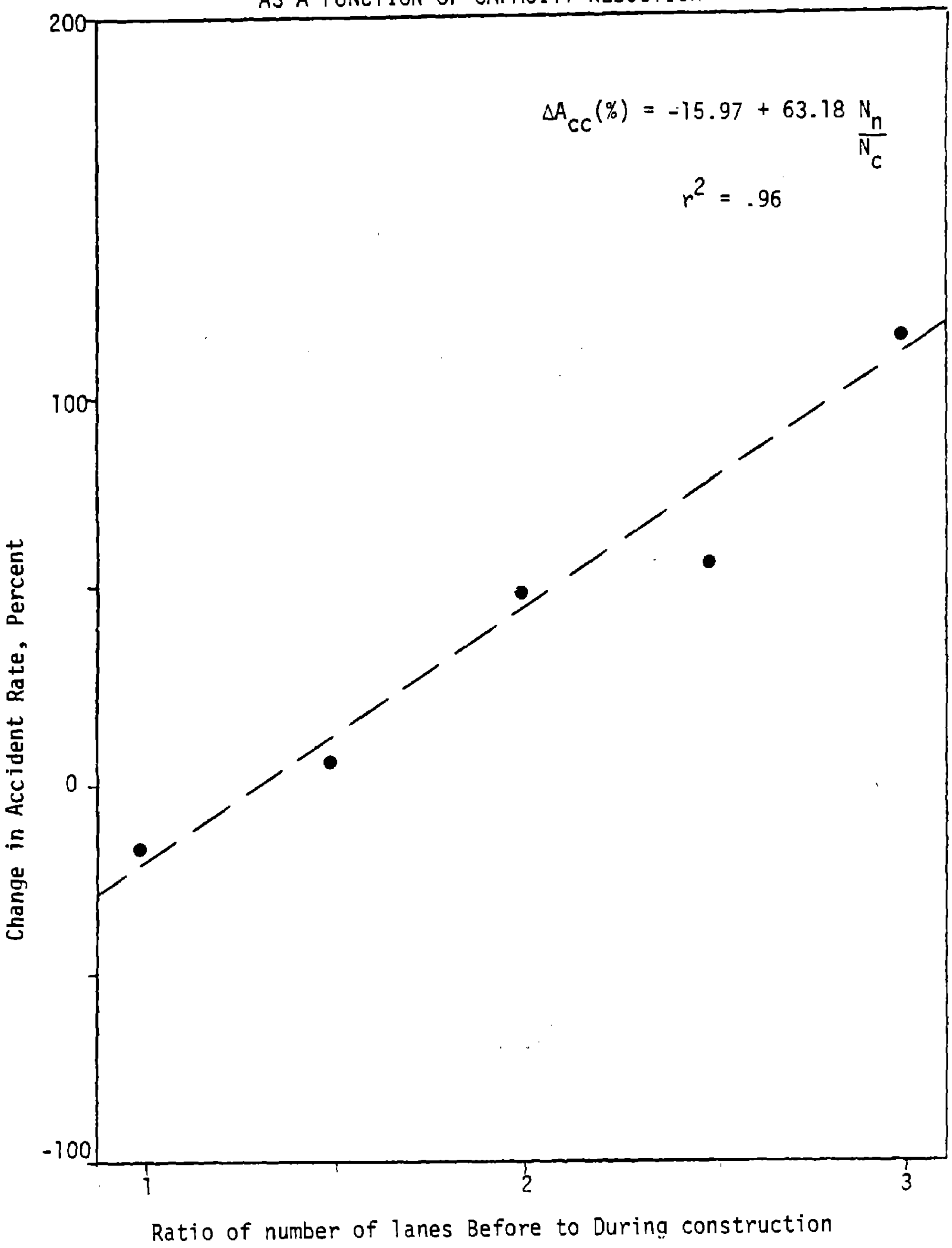
TABLE 82

MEAN ACCIDENT RATE BY WORK AREA ROADWAY TYPE

WORK AREA ROADWAY TYPE	NUMBER PROJECTS	MEAN ACCIDENT RATE (100MVM)
Lane Closure	48	204.23
Cross-over	4	215.28
Temp. By-pass	0	--
Detour	0	--
LC and Cross-over	5	144.29
LC and Temp. By-pass	4	507.91
LC and Detour	10	286.67
Cross-over and Detour	3	154.08
Temp. By-pass and Detour	1	419.35
	75	

FIGURE 83

CHANGES IN ACCIDENT RATES
AS A FUNCTION OF CAPACITY REDUCTION



with the mean accident rate increases.

Another advantage of this formulation is that it reflects actual accident increases due to the presence of the occupancy zone. Therefore, one does not have to resort to assumptions on the deceleration rate in the influence zone or funneling effects.

Thus, we would have the following relationships for the number of accidents with and without road occupancy:

$$N_n^i = AR_n * AHT_i * L \tag{289}$$

$$N_c^i = N_n^i * (0.8403 + 0.6318\ N_n/N_c) \tag{290}$$

where N_n^i is the normal number of accidents in the i^{th} hour ;

AR_n is the normal accident rate, per million vehicle miles ;

AHT_i is the average traffic in the i^{th} hour, in vehicles;

L is the section length, in miles;

N_c^i is the number of accidents in the i^{th} hour in the presence of roadway occupancy ;

N_n is the number of lanes normally operational ;

and N_c is the number of lanes operational during the roadway occupancy.

<u>Accident Costs</u>

The data presented in Table 83 indicates that, in general, roadway occupancy tends to reduce the average severity of accidents while increasing the average accident rate. Although these data are presented in aggregate form, and therefore no information is obtainable about the actual effects of various closure categories, we feel that the severity effects are significant and should be seriously considered for incorporation in the model. This would imply the following data input values (as optional default overrides):

(1) The base year accident rate (AR) for normal annual operation of the facility;

(2) The percentage distribution by severity class (P_i) (property-damage-only, injury, fatal); and

TABLE 83. TOTAL CONSTRUCTION ZONE ACCIDENTS

	Before	During	Change (%)
Total Accidents	8,172	8,785	+7.5
Night Accidents	2,454*	2,685*	+9.4
Severity			
Property-Damage-Only	4,718*	5.226*	+10.7
Injury	2,369*	2,488*	+5.0
Fatal	62*	58*	-6.5
	7,149*	7.772*	
Accident Type			
Right Angle	720	585	-18.8
Rear End	2,614	3,048	+16.6
Side Swipe	939	850	-9.6
Head On	99	114	+15.2
Turning	480	552	+15.0
Ran-Off-Road	706	520	-26.3
Roll	204	225	+10.3
Animal	84	102	+21.4
Fixed Object	941	1,307	+38.9
Fixed Object (Construction Equip.)	--	120	N/A
Other	1,385	1,362	-1.7
	8,172	8,785	
Surface			
Dry	4,190*	4,870*	+16.2
Wet	1,467*	1,443*	-1.6
Ice/Snow	706*	548*	-22.4
Unknown	786*	911*	+15.6
	7,149*	7,772*	
Area			
Urban	4,873	5,149	+6.0
Rural	3,299	3,636	+10.0
	8,172	8,785	

* Does not include State 2 data.

Source: Ref (53)

(3) The average accident cost by severity class (C_i).

From these and the above relations, the total increased accidents and costs, in total and by severity class, could be determined for any ADT and closure category, as follows:

$$N^{i,j} = N^i * P_j \tag{291}$$

$$AC^{i,j} = N^{i,j} * C_j \tag{292}$$

where $N^{i,j}$ is the number of accidents in the i^{th} hour of severity class j (for either normal or occupancy cases);

P_j is the fraction of total accidents of severity class j;

$AC^{i,j}$ is the accident costs in the i^{th} hour for severity class j;

and C_j is the average cost for an accident of severity class j.

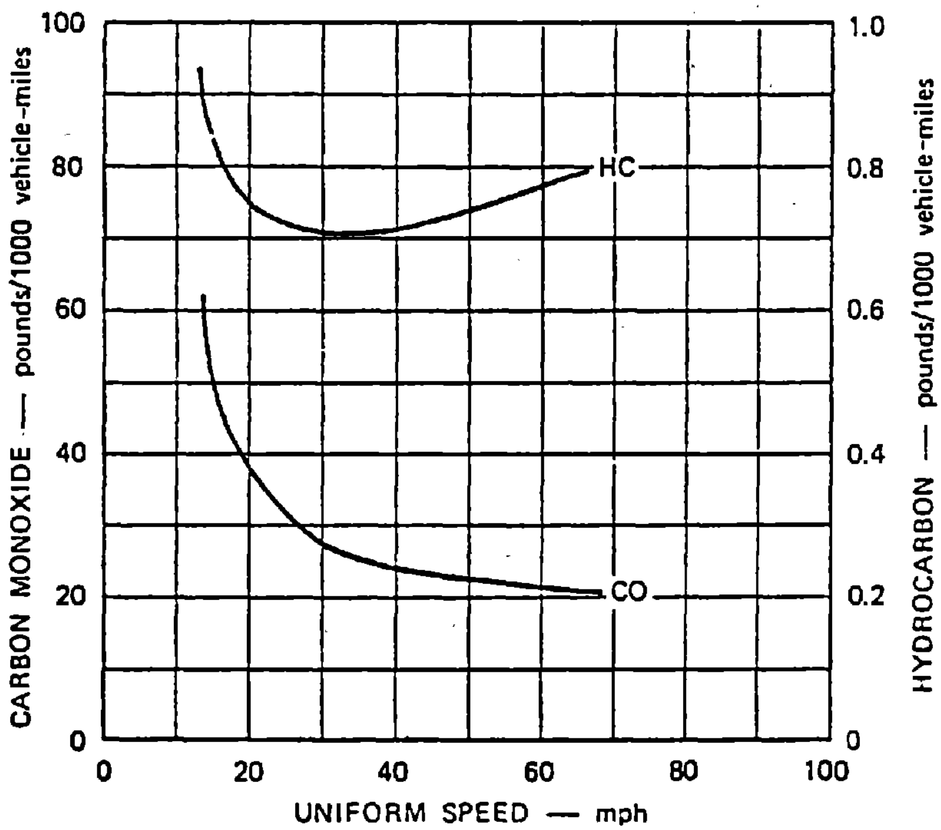
6.4 AIR POLLUTION

The major components of automobile exhaust are the complete oxidation products of the fuel, carbon dioxide and water, and the nitrogen in the air fed to the combustion chamber. Because oxidation is incomplete, carbon monoxide is always present. Minor components, but important ones from an air pollution standpoint, are hydrogen, oxygen, unburned hydrocarbons, partially oxidized hydrocarbons, nitric oxide, and sulfur dioxide.

Although there is a large body of literature available on the evolution of air pollution emission standards and regulations, there is considerably less available on the physical parameters affecting motor vehicle emissions, which are taken by most researchers to be primarily weight and speed. Reference (57) presents graphs of hydrocarbon and carbon monoxide mass emissions for uniform speed, stops from various speeds, and speed-change cycles for a reference automobile (Figures 84-87

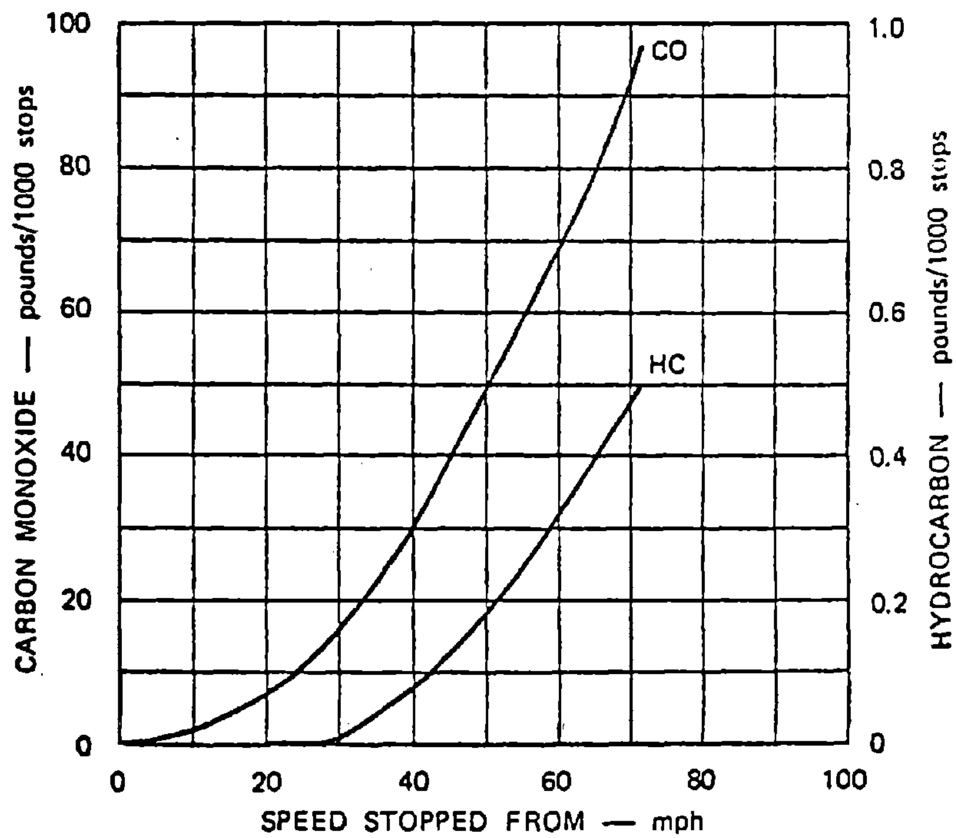
The physical relations determining the emission levels can be modeled by using the relationships presented in (57) (see Figures 84-87 with the omission of the stopping cycle effects, since these are not modeled in the vehicle speed submodel (see Chapter 5). These results have been curve fitted to provide passenger car emission levels as follows:

Figure 84. Hydrocarbon and carbon monoxide emissions per 1,000 miles of driving at uniform speed (reference automobile)



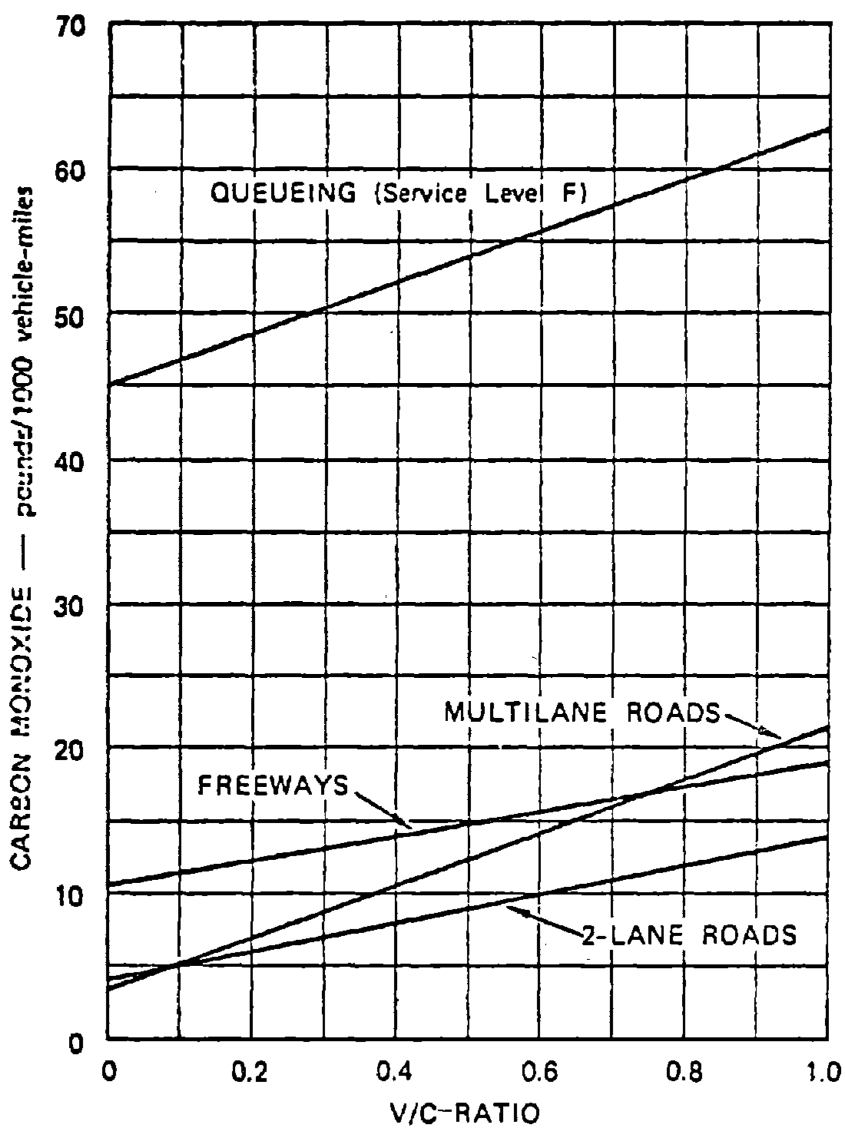
Source: National Cooperative Highway Research Program, NCHRP Report 133, Highway Research Board, p. 39. (57)

Figure 85. Hydrocarbon and carbon monoxide emissions added per 1,000 stops (reference automobile)



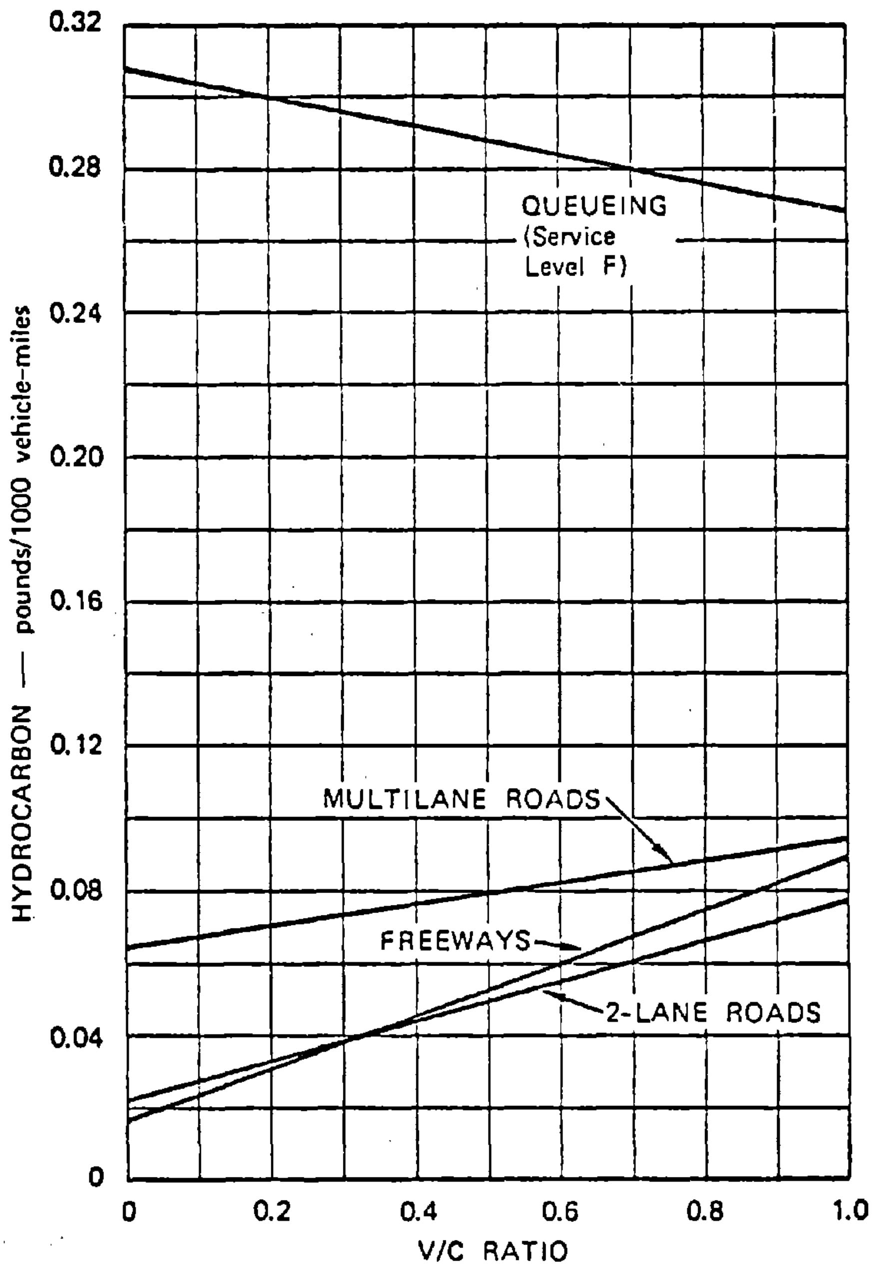
Source: National Cooperative Highway Research Program, <u>NCHRP</u> Report 133, Highway Research Board, p.39 (57)

Figure 86. Carbon monoxide emissions added from speed changes per 1,000 vehicle-miles (reference automobile)



Source: National Cooperative Highway Research Program, NCHRP Report 133, Highway Research Board, p. 40. (57)

Figure 87. Hydrocarbon emissions added from speed changes per 1,000 vehicle-miles (reference automobile)



Source: National Cooperative Highway Research Program, NCHRP Report 133, Highway Research Board, p. 40. (57)

Uniform Speed Emission Levels

$$EL_s = 0.01221 + 0.56167\ S^{-1} \qquad r^2 = 0.99 \tag{293}$$

$$HC_s = EL_s * (1.1640 + 0.0391\ S) * 10^{-2} \qquad r^2 = 0.99 \tag{294}$$

$$CO_s = EL_s * (98.8360 - 0.0391\ S) * 10^{-2} \qquad r^2 = 0.99 \tag{295}$$

where EL_s is the passenger car emission level in pounds per vehicle mile (lb/VM);

HC_s is the hydrocarbon emission level;

CO_s is the carbon monoxide emission level;

and S is the uniform vehicle speed in mph.

Freeway Speed Change Emission Levels

$$EL_{sc} = 0.010523 + 0.008055\ (V/C) \tag{296}$$

$$HC_{sc} = EL_{sc} * (0.22 + 0.20(V/C)\ ) * 10^{-2} \tag{297}$$

$$CO_{sc} = EL_{sc} * (99.78 - 0.20\ (V/C)\ ) * 10^{-2} \tag{298}$$

where EL_{sc} is the passenger car emission level due to speed change cycles on normal freeway operation, in lb/VM;

HC_{sc} is the hydrocarbon emission level;

CO_{sc} is the carbon monoxide emission level;

and V/C is the volume/capacity ratio.

Queueing Emission Levels

$$EL_q = 0.04531 + 0.01746\ (V/C) \tag{299}$$

$$HC_q = EL_q * (0.68 - 0.25\ (V/C)\ ) * 10^{-2} \tag{300}$$

$$CO_q = EL_q * (99.32 + 0.25\ (V/C)\ ) * 10^{-2} \tag{301}$$

where EL_q is the passenger car emission level due to queueing operations, in lb/VM;

HC_q is the hydrocarbon emission level;

and V/C is the volume/capacity ratio.

<u>Total Passenger Car Emissions</u>

$$EL_T = EL_s + EL_{sc} + \delta_q EL_q \quad (302)$$

$$HC_T = HC_s + HC_{sc} + \delta_q HC_q \quad (303$$

$$CO_T = CO_s + CO_{sc} + \delta_q CO_q \quad (304)$$

where δ_q is 1 if queueing occurs, 0 otherwise, and all quantities are as given above.

The results for vehicles other than passenger cars are developed from the above relations using adjustment factors defined by the user.

CHAPTER 7

CONCLUSION AND POTENTIAL APPLICATIONS

7.1 CONCLUSION

The EAROMAR system performs economic analyses of freeway construction, maintenance and rehabilitation policies, encompassing both the structural (i.e. pavement-related) and the operational (i.e. speed- and flow-related) aspects of road performance. This report has documented the technical concepts and relationships incorporated within EAROMAR, organized about route and construction project descriptions, pavement damage, maintenance policies and management, traffic flow, user consequences, economic data, and assembly of this information within strategies to be tested. Although the technical explanations have been segmented in this way for clarity, we have repeatedly stressed the interactions among various system components. One must consider all engineering, economic, and management factors affecting road performance and costs when formulating and evaluating different investment or repair alternatives.

Several new concepts or features have been introduced within the EAROMAR analysis to promote its effectiveness and versatility as a management tool. Among the more important of these are the following:

1. Users have great flexibility in describing route and traffic characteristics. A route may comprise any number of individual roadways, each with its own pattern of traffic loadings. Roadway geometry, capacity, and pavement structure may vary arbitrarily over roadway length, and over time as a result of construction projects. Traffic volume, composition, and growth may likewise vary over roadway length and time.

2. Flexible, rigid, or composite pavements may be simulated on a roadway. Pavement structures are described by component layer thickness and materials properties for any number of layers. Thus it is possible to include non-standard or specialized layers, such as drainage layers or bond-breaking cushions, within the pavement description.

3. Prediction of pavement maintenance requirements is based upon response to the demand for maintenance arising from (a) estimates of pavement damage over time, and (b) maintenance and rehabilitation policies specified by the user. Thus, maintenance workloads and costs are not extrapolated from past trends, but rather are direct functions of the quality of initial pavement design and construction, traffic loading patterns through different seasons of the year, load-environment interactions, past maintenance performed, and current decisions on what work is to be performed, when, and to what level of repair.

4. Key issues of maintenance management are treated in detail. Users may specify completely the maintenance technology to be employed, including crew compositions, equipment and materials requirements, and average production rates. Activities may be scheduled by hour and day within a season; the configuration of the road closure to be used during work performance may also be specified. Maintenance costs are computed using unit costs of each class of labor, equipment and materials, which may be adjusted for inflation.

5. Simulations of traffic speed and flow account for both free-flow and congested situations, by hour of day within each season. Speed and flow within EAROMAR are sensitive to the ratio of demand to roadway capacity, the condition of the pavement surface, and occupancy of the roadway for maintenance or rehabilitation. Consequences to users are monitored in terms of travel time and value of travel time, vehicle operating costs, and accident frequencies and costs. Vehicle pollution emissions are also estimated.

6. The user has the ability to manipulate the set of variables affecting one's analysis through the definition of strategies. Strategy specifications make it possible to change conditions or policies at different roadway locations over time. By correctly defining a route and the strategies to be tested, one may simulate a wide range of construction - rehabilitation-maintenance options.

7. The economic analysis may be tested simultaneously under several assumed discount rates. Differential inflation rates may be applied to maintenance labor, equipment and materials; to motor fuel; and to the value of travel time.

Results of the EAROMAR simulations may be obtained in seasonal, annual, or total period summaries. Examples of the types of reports available are given in Figures 88-92. In addition, simulation traces are available for each roadway section, giving detailed information on hourly traffic flows and speeds, pavement damage, maintenance operations and costs, and vehicle operating costs. These traces supplement the information in Figures 88-92 and are useful in obtaining insight into particular aspects of the analysis.

7.2 POTENTIAL APPLICATIONS

Our original objective in designing the EAROMAR system was to develop procedures for determining economic warrants for premium pavements. Chapters 2 through 6 indicated how we have fulfilled this objective in response to the approach proposed in Chapter 1. It should also be apparent from Chapters 2-6, however, that in its general approach to highway performance and cost modeling, EAROMAR is appropriate to a much larger set of problems involving pavement construction, maintenance and rehabilitation. Below we outline what we see are some potential uses of the EAROMAR system. These are but suggestions, and users may wish to consider other types of analyses as well.

1. Because EAROMAR performs a link-level analysis, it is not a true pavement management system (which adopts a system-wide view). Nevertheless, its structure and logic are compatible with pavement management concepts, and EAROMAR may be used to investigate the same types of investment - maintenance options considered by pavement management systems. Policy alternatives that can be evaluated range from strategic decisions between initial investment vs. subsequent rehabilitation or betterments, to tactical choices among the timing, location and priority of maintenance and rehabilitation. Several commonly discussed management problems fall under this set of options, including economic warrants for premium pavements; deferred maintenance; staged construction; and variations in overlay thickness or frequency.

2. The formulation of maintenance within EAROMAR is also compatible with maintenance management systems in use throughout the country today. For given maintenance policies one may therefore investigate options in work scheduling and configuration of the work zone, or in the method of performing work (e.g. labor-based vs. mechanized technology).

3. The traffic stream simulated within EAROMAR comprises individual vehicle classes whose effects on pavement damage and roadway congestion can be isolated. It is therefore possible to analyze the costs, both to the highway agency and to other roadway users, attributable to a given vehicle class. This procedure is relevant to the problem of cost allocation being addressed by the Federal and several State governments today. Because EAROMAR conducts an economic analysis, it predicts only the costs occasioned -- not necessarily the prices to be charged. Nevertheless, one may consider both pavement-related costs (all costs arising through damage to the pavement by a vehicle, including maintenance costs, rehabilitation costs, and changes in vehicles operating costs due to pavement surface damage), and costs related to traffic flow and congestion (changes in vehicle operating costs and travel time costs caused by the addition of a vehicle class). Cost allocation results may be developed for various pavement designs in different environmental regions.

4. In its maintenance and user consequences routines EAROMAR calculates costs on the basis of resources consumed. It may therefore be used to examine different policies affecting scarce or highly priced resources, such as motor fuel or selected maintenance materials (e.g. asphalt). The effect of future prices rises in fuel can be seen using the inflation adjustment for fuel provided in the economic input (section 2.5). The value of materials can be reflected in either their unit costs or in their scarcity under resource limitations (section 4.4). Although resource consumption rates are not now included in the reports in Figures 88-92, they are displayed in the traces, and it would be possible to modify the program in the future to incorporate this additional information in the analysis results.

FIGURE 88

SEASONAL ROAD CONDITIONS BY ROADWAY

	YEAR	PAVEMENT TYPE	LANE MILES	PSI	CRACKING FT/LM	CRACKING SF/LM	ROUGHNESS (IN/LM)	RUTS (IN)	POT HOLES (#/LM)	FAULTS (#/LM)	DAMAGED JOINTS (#/LM)	SPALLING (SF/LM)	BLOWUPS (#/LM)	PUMPING JOINTS (#/LM)	DAMAGED SHOULDERS (FT/M)	BASE FAILURES (#/LM)
SPRING	1974	FLEXIBLE	17.5	3.7	0.	0.	61.	0.23	1.						0.0	0.0
SUMMER	1974	FLEXIBLE	17.5	3.7	0.	0.	62.	0.32	1.						0.0	0.0
AUTUMN	1974	FLEXIBLE	17.5	3.6	0.	0.	64.	0.36	1.						0.0	0.0
WINTER	1974	FLEXIBLE	17.5	3.6	0.	0.	65.	0.36	1.						0.0	0.0
ANNUAL	1974	FLEXIBLE	17.5	3.6	0.	0.	63.	0.32	1.						0.0	0.0
SPRING	1975	FLEXIBLE	17.5	3.5	0.	0.	66.	0.40	1.						0.0	0.0
SUMMER	1975	FLEXIBLE	17.5	3.9	0.	0.	59.	0.27	1.						0.0	0.0
AUTUMN	1975	FLEXIBLE	17.5	3.8	0.	0.	60.	0.31	1.						0.0	0.0
WINTER	1975	FLEXIBLE	17.5	3.8	0.	0.	61.	0.31	1.						0.0	0.0
ANNUAL	1975	FLEXIBLE	17.5	3.8	0.	0.	61.	0.32	1.						0.0	0.0
SPRING	1976	FLEXIBLE	17.5	3.8	0.	0.	62.	0.34	1.						0.0	0.0
SUMMER	1976	FLEXIBLE	17.5	3.7	0.	0.	62.	0.34	1.						0.0	0.0
AUTUMN	1976	FLEXIBLE	17.5	3.7	0.	0.	63.	0.38	1.						0.0	0.0
WINTER	1976	FLEXIBLE	17.5	3.7	0.	0.	64.	0.38	1.						0.0	0.0
ANNUAL	1976	FLEXIBLE	17.5	3.7	0.	0.	63.	0.36	1.						0.0	0.0
SPRING	1977	FLEXIBLE	17.5	3.6	0.	0.	64.	0.41	1.						0.0	0.0
SUMMER	1977	FLEXIBLE	17.5	3.6	0.	0.	65.	0.41	1.						0.0	0.0
AUTUMN	1977	FLEXIBLE	17.5	3.6	0.	0.	66.	0.43	1.						0.0	0.0
WINTER	1977	FLEXIBLE	17.5	3.5	0.	0.	66.	0.44	1.						0.0	0.0
ANNUAL	1977	FLEXIBLE	17.5	3.6	0.	0.	65.	0.42	1.						0.0	0.0
SPRING	1978	FLEXIBLE	17.5	3.5	0.	0.	67.	0.45	1.						0.0	0.0
SUMMER	1978	FLEXIBLE	17.5	3.5	0.	0.	67.	0.45	1.						0.0	0.0
AUTUMN	1978	FLEXIBLE	17.5	3.4	0.	0.	68.	0.48	1.						0.0	0.0
WINTER	1978	FLEXIBLE	17.5	3.4	0.	0.	68.	0.48	1.						0.0	0.0
ANNUAL	1978	FLEXIBLE	17.5	3.5	0.	0.	67.	0.47	1.						0.0	0.0
SPRING	1979	FLEXIBLE	17.5	3.4	0.	0.	69.	0.49	1.						0.0	0.0
SUMMER	1979	FLEXIBLE	17.5	3.4	0.	0.	69.	0.49	1.						0.0	0.0
AUTUMN	1979	FLEXIBLE	17.5	3.3	0.	0.	70.	0.51	1.						0.0	0.0
WINTER	1979	FLEXIBLE	17.5	3.3	0.	0.	70.	0.51	1.						0.0	0.0
ANNUAL	1979	FLEXIBLE	17.5	3.4	0.	0.	69.	0.50	1.						0.0	0.0
SPRING	1980	FLEXIBLE	17.5	3.3	0.	0.	71.	0.52	1.						0.0	0.0
SUMMER	1980	FLEXIBLE	17.5	3.3	0.	0.	71.	0.52	1.						0.0	0.0
AUTUMN	1980	FLEXIBLE	17.5	3.3	0.	0.	72.	0.54	1.						0.0	0.0
WINTER	1980	FLEXIBLE	17.5	3.2	0.	0.	72.	0.54	1.						0.0	0.0
ANNUAL	1980	FLEXIBLE	17.5	3.3	0.	0.	71.	0.53	1.						0.0	0.0

FIGURE 89

SEASONAL EXPENDITURES BY ROADWAY

	YEAR	CONSTRUCTION COSTS ($1000) LUMPSUM	VARIABLE	TOTAL	MAINTENANCE COSTS BY ACTIVITY ($1000) CRACK FILLING	PATCHING	BASE REPAIR	SEALING	JOINT REPAIR	SLAB REPAIR	MUD JACKING	POTHOLE FILLING	TOTAL
SPRING	1974	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
SUMMER	1974	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AUTUMN	1974	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.3	0.3
WINTER	1974	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
ANNUAL	1974	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.3	0.3
SPRING	1975	78.4	429.1	507.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.2	0.2
SUMMER	1975	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AUTUMN	1975	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.3	0.3
WINTER	1975	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
ANNUAL	1975	78.4	429.1	507.4	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.5	0.5
SPRING	1976	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.3	0.3
SUMMER	1976	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AUTUMN	1976	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.3	0.3
WINTER	1976	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
ANNUAL	1976	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.6	0.6
SPRING	1977	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.3	0.3
SUMMER	1977	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AUTUMN	1977	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.3	0.3
WINTER	1977	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
ANNUAL	1977	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.6	0.6
SPRING	1978	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.3	0.3
SUMMER	1978	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AUTUMN	1978	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.3	0.3
WINTER	1978	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
ANNUAL	1978	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.6	0.6
SPRING	1979	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.3	0.3
SUMMER	1979	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
AUTUMN	1979	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.3	0.3
WINTER	1979	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0
ANNUAL	1979	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.0	0.6	0.6

FIGURE 90

ANNUAL OPERATOR IMPACTS BY ROADWAY

			VEHICLE OPERATING COSTS ($/MILE)				VEHICLE EMISSIONS (LB/MILE)		
	YEAR	VEHICLE TYPE	FUEL	OIL	TIRES	TOTAL	CO	HC	TOTAL
ANNUAL	1974	AUTO	0.061	0.002	0.008	0.071	0.0365	0.0008	0.0373
ANNUAL	1974	PICKUP	0.078	0.002	0.007	0.087	0.0365	0.0008	0.0373
ANNUAL	1974	2D	0.205	0.004	0.014	0.223	0.0365	0.0008	0.0373
ANNUAL	1974	2S2	0.087	0.004	0.026	0.117	0.0365	0.0008	0.0373
ANNUAL	1974	3S2	0.117	0.005	0.029	0.151	0.0365	0.0008	0.0373
ANNUAL	1975	AUTO	0.060	0.002	0.007	0.069	0.0367	0.0008	0.0374
ANNUAL	1975	PICKUP	0.078	0.002	0.007	0.087	0.0367	0.0008	0.0374
ANNUAL	1975	2D	0.205	0.004	0.013	0.222	0.0367	0.0008	0.0374
ANNUAL	1975	2S2	0.087	0.004	0.024	0.115	0.0367	0.0008	0.0374
ANNUAL	1975	3S2	0.116	0.005	0.027	0.148	0.0367	0.0008	0.0374
ANNUAL	1976	AUTO	0.061	0.002	0.007	0.070	0.0368	0.0008	0.0376
ANNUAL	1976	PICKUP	0.078	0.002	0.007	0.087	0.0368	0.0008	0.0376
ANNUAL	1976	2D	0.205	0.004	0.014	0.223	0.0368	0.0008	0.0376
ANNUAL	1976	2S2	0.087	0.004	0.025	0.116	0.0368	0.0008	0.0376
ANNUAL	1976	3S2	0.116	0.005	0.028	0.150	0.0368	0.0008	0.0376
ANNUAL	1977	AUTO	0.061	0.002	0.008	0.072	0.0369	0.0008	0.0377
ANNUAL	1977	PICKUP	0.078	0.002	0.007	0.087	0.0369	0.0008	0.0377
ANNUAL	1977	2D	0.204	0.004	0.015	0.223	0.0369	0.0008	0.0377
ANNUAL	1977	2S2	0.088	0.004	0.027	0.118	0.0369	0.0008	0.0377
ANNUAL	1977	3S2	0.117	0.005	0.030	0.152	0.0369	0.0008	0.0377
ANNUAL	1978	AUTO	0.062	0.002	0.008	0.073	0.0370	0.0008	0.0378
ANNUAL	1978	PICKUP	0.077	0.002	0.008	0.087	0.0370	0.0008	0.0378
ANNUAL	1978	2D	0.204	0.004	0.015	0.223	0.0370	0.0008	0.0378
ANNUAL	1978	2S2	0.089	0.004	0.028	0.120	0.0370	0.0008	0.0378
ANNUAL	1978	3S2	0.117	0.006	0.032	0.154	0.0370	0.0008	0.0378
ANNUAL	1979	AUTO	0.063	0.002	0.009	0.074	0.0371	0.0008	0.0378
ANNUAL	1979	PICKUP	0.077	0.002	0.008	0.087	0.0371	0.0008	0.0378
ANNUAL	1979	2D	0.203	0.004	0.016	0.223	0.0371	0.0008	0.0378
ANNUAL	1979	2S2	0.088	0.004	0.029	0.122	0.0371	0.0008	0.0378
ANNUAL	1979	3S2	0.118	0.006	0.033	0.156	0.0371	0.0008	0.0378
ANNUAL	1980	AUTO	0.063	0.002	0.009	0.074	0.0372	0.0008	0.0379
ANNUAL	1980	PICKUP	0.077	0.002	0.008	0.088	0.0372	0.0008	0.0379
ANNUAL	1980	2D	0.203	0.004	0.017	0.223	0.0372	0.0008	0.0379
ANNUAL	1980	2S2	0.088	0.004	0.030	0.123	0.0372	0.0008	0.0379
ANNUAL	1980	3S2	0.118	0.006	0.034	0.158	0.0372	0.0008	0.0379

FIGURE 91

ANNUAL USER IMPACTS BY ROADWAY

	YEAR	MILLION VEHICLE MILES TRAVELED	TOTAL VEHICLE OPERATING COSTS (MILLIONS)	TOTAL VEHICLE TRAVEL TIME COSTS (MILLIONS)	TOTAL ACCIDENT COSTS (MILLIONS) PDO	INJURY	FATALITY	TOTAL USER COSTS (MILLIONS)	TOTAL EMISIONS (1000 LB)
ANNUAL	1974	78.281	6.288	5.816	0.011	0.028	0.017	12.143	2917.6
ANNUAL	1975	82.586	6.527	6.101	0.012	0.029	0.018	12.668	3092.3
ANNUAL	1976	87.129	6.951	6.456	0.012	0.031	0.019	13.450	3273.6
ANNUAL	1977	87.939	7.129	6.555	0.012	0.031	0.019	13.728	3312.8
ANNUAL	1978	88.750	7.290	6.651	0.012	0.031	0.019	13.985	3351.5
ANNUAL	1979	89.560	7.439	6.743	0.013	0.031	0.019	14.226	3389.7
ANNUAL	1980	90.370	7.577	6.833	0.013	0.032	0.019	14.454	3427.6
ANNUAL	1981	91.181	7.707	6.920	0.013	0.032	0.019	14.672	3465.2
ANNUAL	1982	91.991	7.830	7.006	0.013	0.032	0.020	14.881	3502.6
ANNUAL	1983	92.802	7.948	7.090	0.013	0.033	0.020	15.084	3539.7
ANNUAL	1984	93.612	8.0[illegible]1	7.173	0.013	0.033	0.020	15.280	3576.7
ANNUAL	1985	94.423	7.478	7.219	0.013	0.033	0.020	14.744	3739.4
ANNUAL	1986	95.233	7.297	6.988	0.013	0.033	0.020	14.332	3596.2
ANNUAL	1987	96.044	7.440	7.074	0.014	0.034	0.020	14.561	3633.8
ANNUAL	1988	96.854	7.567	7.155	0.014	0.034	0.021	14.770	3670.8
ANNUAL	1989	97.665	7.683	7.234	0.014	0.034	0.021	14.966	3707.4
ANNUAL	1990	98.475	7.791	7.310	0.014	0.035	0.021	15.150	3743.7
ANNUAL	1991	99.285	7.893	7.385	0.014	0.035	0.021	15.327	3779.8
ANNUAL	1992	100.096	7.990	7.459	0.014	0.035	0.021	15.498	3815.8
ANNUAL	1993	100.906	8.083	7.531	0.014	0.036	0.022	15.664	3851.6
ANNUAL	1994	101.717	8.174	7.603	0.014	0.036	0.022	15.826	3887.3
ANNUAL	1995	102.530	7.753	8.153	0.014	0.036	0.022	15.957	4416.9
ANNUAL	1996	103.338	7.828	7.585	0.015	0.036	0.022	15.464	3933.9
ANNUAL	1997	104.148	7.942	7.664	0.015	0.037	0.022	15.657	3970.9
ANNUAL	1998	104.959	8.043	7.739	0.015	0.037	0.022	15.834	4007.4
ANNUAL	1999	105.769	8.135	7.812	0.015	0.037	0.023	16.000	4043.7
ANNUAL	2000	106.579	8.224	7.884	0.015	0.038	0.023	16.161	4079.8
ANNUAL	2001	107.390	8.307	7.955	0.015	0.038	0.023	16.315	4115.8
ANNUAL	2002	108.200	8.387	8.026	0.015	0.038	0.023	16.466	4151.7
ANNUAL	2003	109.011	8.465	8.096	0.015	0.038	0.023	16.615	4187.8
ANNUAL	2004	109.821	8.545	8.167	0.015	0.039	0.023	16.765	4224.0
ANNUAL	2005	110.632	8.591	8.375	0.016	0.039	0.024	17.020	4381.4
TOTAL 1974-2005		3127.274	248.364	233.758	0.440	1.100	0.667	483.661	119788.2
DISCOUNTED	5%	3127.274	119.384	111.357	0.209	0.523	0.317	231.475	119788.2
DISCOUNTED	8%	3127.274	85.108	79.042	0.149	0.371	0.225	164.670	119788.2
DISCOUNTED	10%	3127.274	70.262	65.104	0.122	0.306	0.185	135.794	119788.2
DISCOUNTED	12%	3127.274	59.329	54.873	0.103	0.258	0.156	114.563	119788.2

FIGURE 92

ANNUAL COST TOTALS BY ROADWAY

	YEAR	CONSTRUCTION COSTS (MILLIONS)	MAINTENANCE COSTS (MILLIONS)	VEHICLE OPERATING COSTS (MILLIONS)	VEHICLE TRAVEL TIME COSTS (MILLIONS)	ACCIDENT COSTS (MILLIONS)	TOTAL COSTS (MILLIONS)
ANNUAL	1974	0.0	0.000	6.288	5.816	0.055	12.160
ANNUAL	1975	0.507	0.001	6.527	6.101	0.058	13.194
ANNUAL	1976	0.0	0.001	6.951	6.456	0.061	13.470
ANNUAL	1977	0.0	0.001	7.129	6.555	0.062	13.747
ANNUAL	1978	0.0	0.001	7.290	6.651	0.063	14.004
ANNUAL	1979	0.0	0.001	7.439	6.743	0.063	14.246
ANNUAL	1980	0.0	0.001	7.577	6.833	0.064	14.474
ANNUAL	1981	0.0	0.001	7.707	6.920	0.064	14.692
ANNUAL	1982	0.0	0.001	7.830	7.006	0.065	14.902
ANNUAL	1983	0.0	0.001	7.948	7.090	0.065	15.104
ANNUAL	1984	0.0	0.001	8.061	7.173	0.066	15.301
ANNUAL	1985	2.093	0.000	7.478	7.219	0.067	16.857
ANNUAL	1986	0.0	0.001	7.297	6.988	0.067	14.353
ANNUAL	1987	0.0	0.001	7.440	7.074	0.068	14.582
ANNUAL	1988	0.0	0.001	7.567	7.155	0.068	14.792
ANNUAL	1989	0.0	0.001	7.683	7.234	0.069	14.987
ANNUAL	1990	0.0	0.001	7.791	7.310	0.070	15.172
ANNUAL	1991	0.0	0.001	7.893	7.385	0.070	15.349
ANNUAL	1992	0.0	0.001	7.990	7.459	0.071	15.520
ANNUAL	1993	0.0	0.001	8.083	7.531	0.071	15.686
ANNUAL	1994	0.0	0.001	8.174	7.603	0.072	15.849
ANNUAL	1995	2.093	0.000	7.753	8.153	0.072	18.072
ANNUAL	1996	0.0	0.001	7.828	7.585	0.073	15.486
ANNUAL	1997	0.0	0.001	7.942	7.664	0.074	15.680
ANNUAL	1998	0.0	0.001	8.043	7.739	0.074	15.857
ANNUAL	1999	0.0	0.001	8.135	7.812	0.075	16.023
ANNUAL	2000	0.0	0.001	8.224	7.884	0.075	16.184
ANNUAL	2001	0.0	0.001	8.307	7.955	0.076	16.339
ANNUAL	2002	0.0	0.001	8.387	8.026	0.076	16.490
ANNUAL	2003	0.0	0.001	8.465	8.096	0.077	16.639
ANNUAL	2004	0.0	0.001	8.543	8.167	0.078	16.789
ANNUAL	2005	0.0	0.001	8.591	8.375	0.078	17.045
TOTAL 1974-2005		4.694	0.018	248.364	233.758	2.207	489.040
DISCOUNTED	5%	2.341	0.009	119.384	111.357	1.050	234.142
DISCOUNTED	8%	1.651	0.006	85.108	79.042	0.745	166.553
DISCOUNTED	10%	1.343	0.005	70.262	65.104	0.614	137.328
DISCOUNTED	12%	1.115	0.005	59.329	54.873	0.517	115.838
DISCOUNTED	15%	0.872	0.004	47.666	43.999	0.415	92.955

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