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COMPUTERIZED METHOD OF PROJECTING REHABILITATION AND MAINTENANCE REQUIREMENTS DUE TO VEHICLE LOADINGS

by

Alberto Garcia-Diaz, Robert L. Lytton, and Dock Burke

of the

Texas Transportation Institute

and

B. Frank McCullough and C. Michael Walton

of the

Center for Transportation Research

Research Report 298/312-1(Final)

Volume 5F

Computerized Methods of Projecting Rehabilitation
and Maintenance Requirements Due to Vehicle Loadings

Research Projects 2-8-80-298 and 3-8-80-312

conducted for

The State Department of Highways and Public Transportation

by the

Texas Transportation Institute
The Texas A&M University System

Center for Transportation Research
The University of Texas at Austin

August 1981

ACKNOWLEDGMENT

This project was sponsored by the Texas State Department of Highways and Public Transportation through its Cooperative Research Program. Work concerning flexible pavements was developed at T.T.I. with Alberto Garcia-Diaz serving as principal investigator and Robert L. Lytton and Dock Burke as project staff members. Work concerning rigid pavements was developed at C.T.R. with Frank McCullough and C. Michael Walton as principal investigators. Mr. Robert L. Mikulin was the Contact Representative for the State Department of Highways and Public Transportation. The authors want to thank Mr. Robert L. Lewis, Mr. Byron Blaschke, Mr. Gerald Peck, and Mr. Robert L. Mikulin, all from SDHPT, for their outstanding cooperation and interest in this project. Proper acknowledgment is extended to Horacio Peña, Jack Allison, Daniel Orlich, Rafael Cal y Mayor, Jack Vathana, and Phung Lim for their excellent assistance.

DISCLAIMER

The views, interpretation, and conclusions expressed or implied in this report are those of the research group. They are not necessarily those of the Texas State Department of Highways and Public Transportation.

ABSTRACT

The goal of this research project is to revise and combine the REHAB and NULOAD computer models into a new approach to forecast pavement rehabilitation costs. The new approach is called RENU and it incorporates the following three main elements: (a) revised pavement performance equations, (b) design-oriented survivor curves, and (c) a procedure to predict the increment in axle loads when higher pay loads are allowed. The most relevant contribution of the new model in the area of flexible pavements is the development of a serviceability/distress approach to investigate the effect of vehicle loading on the life cycle of highways. This approach has the capability to predict if a pavement needs light to medium rehabilitation as a result of distress signs, when the riding conditions (PSI) has not yet reached a terminal value. The new approach is considered more reliable, for Texas flexible pavements, than the AASHTO methodology. In the area of rigid pavements the two most important improvements are the formulation of a modified AASHTO equation to include soil support values, regional factors, design characteristics, and traffic conditions typical of the Texas highway system, and the development of a failure prediction model to estimate maintenance needs.

The RENU approach was built using experimental values of material properties, climatic conditions, design factors, and traffic measurements obtained by the Texas Transportation Institute (TTI), and the Center for Transportation Research (CTR).

Briefly, the overall methodology can be summarized in four steps: (a) a load distribution procedure is incorporated to investigate the shift toward higher loads if a new legal axle load limit is considered,

(b) generation of a pavement performance functions based upon statistical criteria, (c) generation of survivor curves to predict the extent of road rehabilitation requirements in each of the periods of a planning horizon, and (d) determination of rehabilitation costs considering life cycles for both the current and new axle load legal limits.

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Chapter 1

INTRODUCTION

The purpose of this research study is to revise and combine the best elements of the REHAB and NULOAD computer models to develop a new model RENU to forecast pavement rehabilitation costs. The new model incorporates the following elements: (a) revised pavement performance equations, (b) design-oriented pavement survivor curves, and (c) a methodology to predict the increase in axle loads when higher payloads are allowed. The new model will be called RENU.

REHAB [24] is currently being used by the Texas State Department of Highways and Public Transportation (SDHPT) to estimate highway rehabilitation and maintenance funds needed to keep the state road system at an acceptable level of user serviceability. NULOAD [6] is a more recently developed computer model which uses the pavement performance equations formulated by the American Association of State Highway and Transportation Officials (AASHTO) to describe pavement behavior. The AASHTO equations [44] relate soil support values, regional factors, design characteristics, and traffic conditions to pavement serviceability.

As a result of continued preventive maintenance, the riding condition of a pavement may approach a terminal serviceability value in a very slow fashion, so that the need of rehabilitation will most likely be due to the appearance of pavement distress, at a time substantially shorter than that at which the pavement would reach terminal serviceability index. This behavior has been found to be quite common among Texas flexible pavements. The single most important contribution of the new model in the analysis of flexible pavements is the development of a serviceability/

distress approach to investigate the effect of vehicle loadings on the life cycle of highways. Serviceability and distress performance equations have been developed using available data on Texas flexible pavements. The parameters of the equations are estimated using experimental values of material properties, climatic conditions, design factors, and traffic measurements obtained by the Texas Transportation Institute by field observation.

The proposed model for flexible pavements predicts the life cycle for pavements of several types. In order to develop the model, the following types were considered: (a) asphaltic concrete (hot mix) on asphaltic stabilized base (black base), (b) thick asphaltic concrete, (c) asphaltic concrete pavements, (d) surface treated pavement, and (e) overlay. To identify the critical factor causing the need for rehabilitation for pavement sections of a given type, consideration was made of the serviceability condition (ride) and the following kinds of pavement distress: (a) alligator cracking, (b) longitudinal cracking, (c) transversal cracking (d) rutting, (e) flushing, (f) corrugation, (g) patching, (h) ravelling and (i) failures per lane mile.

An analysis of the conditions prevailing in Texas led to two significant considerations in the development of the RENU model. The first consideration is that asphaltic concrete on asphaltic stabilized base and thick asphaltic concrete pavements do not constitute a major part of the present highway mileage and, therefore, were included in the asphaltic concrete type, thus reducing the types of flexible pavements to three. The second consideration is that most pavements in Texas need rehabilitation as a result of critical levels of transverse cracking or alligator cracking, thus reducing the types of distress actually considered in the RENU program to two. If necessary, of course, the above five types of flexible pavements and nine types of distress signs can easily be incorporated in the procedure. The corresponding equations are summarized elsewhere in this report.

Based on condition surveys of Texas rigid pavements, the structural design concept and the maintenance cost estimation procedure of the NULOAD program were revised to increase the accuracy of the predicted mileage to be rehabilitated. Revised survivor curves, modified AASHTO performance equations, and a failure prediction model to estimate maintenance costs are the major contributions in the area of rigid pavement analysis.

The revised performance equation for rigid pavements was developed from extensive Texas pavement data to allow the consideration of local material, especially subbase material. Additionally, in the development of the distress prediction model for rigid pavements, the following signs of distress were included: (a) spalling, (b) pumping, (c) punchouts, and (d) patches. Five types of data were utilized in this analysis: (a) environmental factors, (b) construction factors, (c) traffic, (d) age of pavement, and (e) distress factors.

A brief summary of the overall methodology follows. A load redistribution procedure is incorporated to investigate the shift toward higher loads if a new legal axle load limit is considered. For a given type of pavement, the mileage with critical values of serviceability index is assumed to be distributed according to a probability density function whose parameters are estimated using observed pavement data. Based on this density function, a survivor curve is generated to predict the extent of pavement rehabilitation requirements in each of the periods of a planning horizon. Life cycles are determined for both the current and the new axle load limits, and the corresponding pavement rehabilitation needs are finally translated into dollars.

Chapter 2

SYNTHESIS OF RELATED WORK

Past work on the development and improvement of computerized methods for estimating road rehabilitation requirements are summarized in the following three reports:

(a) "The McKinsey Report" [19], which relates to the original REHAB model.

(b) "The Updated Documentation Report" [28], which contains the input/output instructions for the present REHAB model.

(c) "Effects of Changes in Legal Load Limits on Pavement Cost" [2,3], which refer to the NULOAD model.

Due to the limitation that REHAB does not generate performance and survivor curves, it was felt that NULOAD represented a more effective potential planning procedure. However, NULOAD uses the AASHTO performance equations, which have been found to be unreliable for a large number of Texas pavement sections; additionally, NULOAD actually assumes survivor curves instead of generating them on the basis of obtained data. For this reason, it was decided that the most appropriate option would be the development of a new procedure, RENU, which would be similar to NULOAD but with Texas data-based performance and survivor curves.

The overall development of the new computerized procedure (RENU) was undertaken in two phases. The objective of the first phase of the study was to perform a comparison between REHAB and NULOAD and propose an improved methodology which would take into consideration SDHPT requirements concerning pavement classification, data availability, and district organization of the overall highway system. The results of the first phase of the study are summarized in three volumes. Volume 1 [31] contains the evaluation procedure. This procedure was subdivided into three basic tasks: (a) analysis of initial assumptions of REHAB and NULOAD, (b) evaluation of data needs and data availability, and (c) documentation of findings and recommendations. This third task contains an updated user manual for REHAB. Volume 2 [32] is composed of a detailed flowchart of the program, a FORTRAN list of the computerized procedure, a sample of the program output, and a section with the description of all variables used in the model. Volume 3 [33] contains the NULOAD FORTRAN program and a sample output of this model. The first phase of the study was developed in the period between June 1 and August 31, 1980.

The objective of the second phase of the study was to actually develop the new computerized procedure RENU. This objective was accomplished in the period between September 1 and August 31, 1981. The results of this phase are summarized in two additional volumes. Volume 4 contains a user manual, a FORTRAN listing, and a sample output of RENU. Volume 5, the final report of the study, presents the development, analysis, and discussion of the new procedure, as well as a summary of the results concerning the Texas highway network. The basic topics included in the final report can be listed as follows:

(a) Flexible Pavement Methodology (Chapter 3)

(b) Rigid Pavement Methodology (Chapter 4)

(c) Cost Methodology (Chapter 5)

(d) Load Shifting Procedure (Chapter 6)

(e) Applications of the Model (Chapter 7)

(f) Discussion of Results (Chapter 8)

(g) Conclusions and Recommendations (Chapter 9)

Chapter 3

FLEXIBLE PAVEMENT METHODOLOGY

The performance of a pavement during a specific period can be estimated by the reduction of user serviceability with increasing levels of traffic loads. When this reduction process is represented by a mathematical relationship with known shape and location parameters, it is possible to predict the load traffic required to lower a serviceability index to a specific critical level. Usually the performance of the road is measured in terms of the "Present Serviceability Index" (PSI), which is defined as a measurement of the pavement roughness at any instant of time and based upon a rating scale between 0 and 5.

A critical problem in the analysis of pavement performance is that most of the pavement data available correspond to relatively high levels of PSI. This limitation makes it difficult to predict the performance of older pavements, such as those exhibiting PSI values of 2.5 or less. A traditional approach to pavement rehabilitation is that of upgrading the pavement when the PSI reaches a critical value. By the time the pavement approaches this level, it may have already received a substantial amount of routine maintenance, which may reduce the deterioration rate of the pavement as traffic loading continues to increase.

The purpose of this chapter is to propose and discuss a rehabilitation approach which takes into consideration the effect of routine maintenance upon flexible pavement performance. Briefly, the approach consists of modeling the performance of pavement according to an S-shaped curve which may or may not reach a specified terminal PSI value, as seen in Figures 3-1(a) and 3-1(b). When the curve reaches the terminal PSI, as in Figure 3-1(a), the riding conditions are considered unacceptable and the pavement should be overlaid. When the curve does not reach the critical PSI level, as in Figure 3-1(b), the need for rehabilitation is caused not by a significant loss in riding quality, but rather by the presence of one or more types of distress, such as: rutting, cracking, flushing, and others. In this case the pavement should receive a light type of rehabilitation, perhaps a thin overlay (1 to 2 inches).

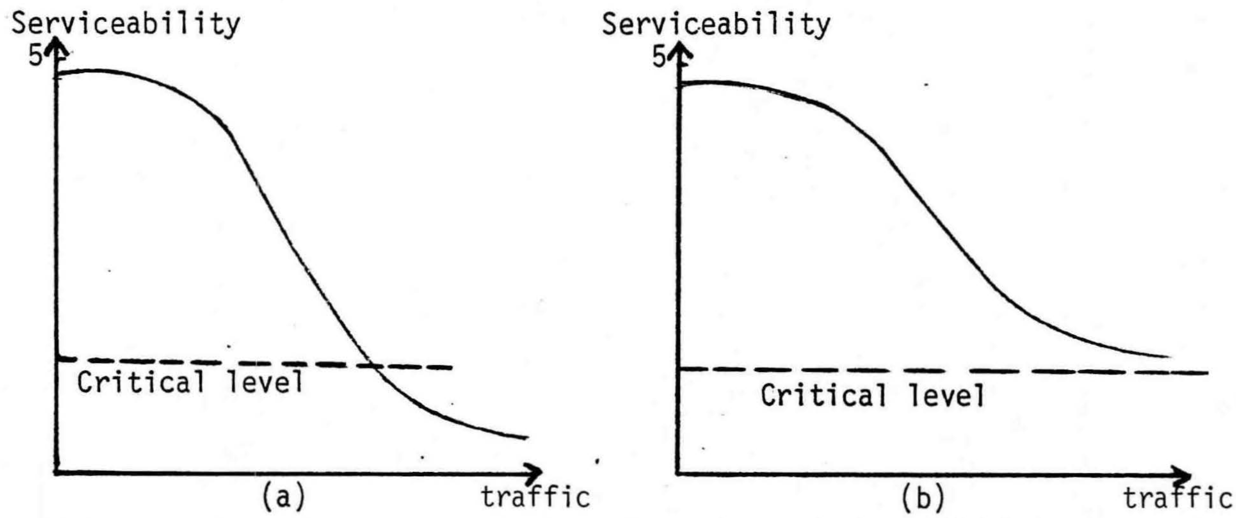


Figure 3-1. PSI Function

This chapter has been divided into four sections. Section 3-1 summarizes the AASHO performance equations, which are currently used in NULOAD. Section 3 develops the proposed serviceability/distress approach, considered more reliable for the analysis of Texas flexible pavements. Section 3.3 discusses the development of the survivor curves used to estimate the percent of surviving miles of a given type of pavement section. Section 3.4 presents the computerized procedure that results from the implementation of the Texas Performance Equations (TPE) and the new survivor curves in the program NULOAD.

3.1 AASHTO Performance Equations

The procedure developed by the American Association of State Highway and Transportation Officials (AASHTO) to predict pavement performance is based upon an extensive road test conducted in Ottawa, Illinois, in the late 1950's and early 1960's. The results were published in 1961 as an Interim Design guide which was later revised in 1972.

In order to support a brief description of the AASHTO equation the following terms must be defined:

(a) Equivalent Single Axle Load Application

(b) Regional Factor

(c) Structural Number

(d) Soil Support Value

Equivalent Single Axle Load Applications

It is a measurement of traffic expressed as an equivalent number of single and tandem axle applications, and obtained as a function of the structural number and critical PSI. Using this factor, traffic can be equated to the number of equivalent 18,000 lb. load applications.

Regional Factor

This factor is used to adapt the AASHTO equations to conditions different from those that existed during the original road test. The values of the regional factor (R) are summarized in Table 3-1 as indicated in the AASHTO Interim Guide [1].

TABLE 3-1. REGIONAL FACTOR

Condition	R Value
Road-bed material frozen to depth of 5 in. or more	0.2-1.0
Road-bed materials, dry summer and fall	0.3-1.5
Road-bed materials, wet spring thaw	4.0-5.0

Structural Number

It is an index number derived from an analysis of traffic, road-bed soil conditions, and regional factor that may be converted to thickness of various flexible pavement layers through the use of suitable layers coefficients related to the type of material being used in each of the pavement structures [43].

Soil Support

Also known as subgrade support value, it is an index of subgrade stiffness which is used in combination with the 18-kip ESALs for a given period of time to compute the design thickness required by the road.

The performance equations developed at the AASHTO Road Test express a pavement damage function in terms of vehicle loading. The damage function is defined as a relative loss in serviceability, and the traffic loading is measured in 18-kip equivalent single-axle load applications. In the formulation of the performance equations, the following notation will be used:

t is years after construction or major rehabilitation

P_t is the serviceability index at year t

P_i is the initial value of serviceability index

P_c is the critical serviceability index

W is the number of 18-kip ESALs that have passed over a pavement

β is a power which differs between rigid and flexible pavements and which depends upon the layer thickness, AASHTO layer coefficient of each layer, and the configuration of wheel loading applied . This function influences the shape of the serviceability curve

ρ is the total number of 18-kip ESALs that will cause the amount of damage corresponding to a value of serviceability equal to 1.5. Additionally, the quantity ρ depends upon layer thicknesses, layer coefficients, and wheel configuration

R is the regional factor

The damage function is defined as the ratio of the loss in serviceability at a given time to the total loss allowed. That is,

$$g = \frac{P_i - P_t}{P_i - P_c} \tag{3-1}$$

Usually P_i is 4.2 and P_c is 1.5, then Eq. (3-1)

$$g = \frac{4.2 - P_t}{2.7} \tag{3-2}$$

As can be seen from Eq. (3-2), the damage function is equal to 0.0 when the pavement is new and becomes 1.0 when the pavement reaches its critical serviceability index. This behavior can be observed in Figure 3-2.

The AASHTO performance equation can be written as:

$$g = (R\,W/\rho)^{\beta} \tag{3-3}$$

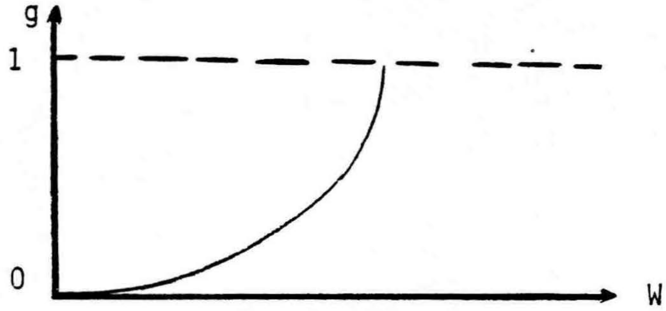


Figure 3-2. Performance Function for AASHTO Approach

Let g_t be the relative loss in serviceability after t periods since last rehabilitation, and let W_t be the corresponding number of 18-kip ESALs. Therefore, from Eq. (3-3),

$$Ln(g_t) = \beta[Ln(R) + Ln(W_t) - Ln(\rho)] \tag{3-4}$$

The parameters β and ρ can be computed in terms of structural design and loading variables. As a result of the AASHTO Road Test, the following relationships were found for β and ρ:

$$\beta = 0.40 + \frac{0.081(L_1 + L_2)\ 3.23}{(SN + 1)^{5.19}} \tag{3-5}$$

$$Ln(\rho) = 5.93\ Ln(SN + 1) - 4.79\ Ln(L_1 + L_2) + 4.33\ Ln(L_2) \tag{3-6}$$

where:

L_1 = load on one single axle or on one tandem axle set

L_2 = axle code (L_2 = 1 for single axle and L_2 = 2 for tandem axle)

Eqs. (3-4), (3-5), and (3-6) can be combined to express W_t, for L_1 = 18,000 pounds, L_2 = 1, P_i = 4.2, and P_c = 1.5, as:

$$Ln(W_t) = 9.36(SN + 1) - 0.20 + \frac{Ln\left[(4.2 - P_t)/2.7\right]}{0.40 + (1094/(SN + 1)^{5.19})} \tag{3-7}$$

In general, the soil subgrade and climatic conditions differ from those encountered in the original experiment. If a soil support value S_i and a regional factor R are included in the analysis, Eq. (3-7) results in the final flexible pavement design equation given below:

$$\text{Ln}(W_t) = 9.36\ \text{Ln}(SN + 1) - 0.20 + \frac{\text{Ln}(4.2 - P_t/2.7)}{0.40 + (1094/(SN + 1)^{5.19})}$$

$$+ \text{Ln}(\frac{1}{R}) + 0.372\ (S_i - 3.0) \qquad (3\text{-}8)$$

From Eq. (3-8), the terminal 18-kip ESALs required to reduce the serviceability index to P_t is given by:

$$W_t = \frac{\rho g^{1/\beta}}{R} \qquad (3\text{-}9)$$

For g = 1, Eq. (3-9) yields $W_o = \frac{\rho}{R}$.

The number of 18-kip ESALs that remains to be carried by the pavement, W_r is equal to W_o - W_t, that is:

$$W_r = \frac{\rho}{R}\ (1 - g^{1/\beta}) \qquad (3\text{-}10)$$

The equivalent annual number of 18-kip ESALs corresponding to W_t can be computed as:

$$W_n = \frac{iW_t}{[(1+i)^n-1]} \qquad (3\text{-}11)$$

where i is the annual growth rate of 18-kip ESALs.

3.2 Texas Pavement Performance Equations

The AASHTO model, represented in Figure 3-2, describes the performance of a pavement as a riding surface in terms of variations in PSI. The performance function of Figure 3-2 keeps the curvature constant along the range of the traffic (or time) variable. A number of observed serviceability values corresponding to Texas flexible pavements indicate that the performance curve should show a reversal of curvature, as illustrated in Figure 3-3. The asymptotic behavior of this curve is due to the reduction of the deterioration rate because of routine maintenance. Once the PSI is relatively stable, the road may need rehabilitation when one or more signs of distress become important, as measured by the area affected and the severity of the distress.

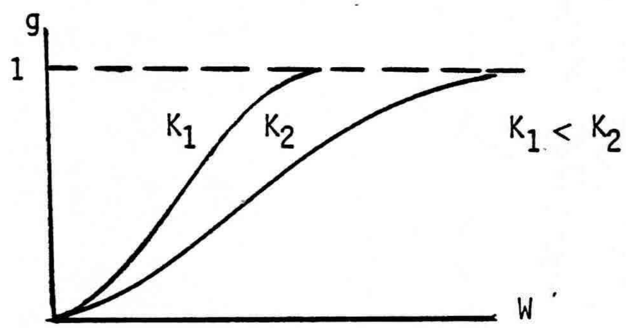


Figure 3-3. Performance Function (loss) for the Texas Performance Approach

3.2.1 Basic Equations for Serviceability Analysis

After examining field data concerning flexible pavements performance, the following function was postulated to represent the relative loss in serviceability index for Texas highways:

$$g(W) = e^{-K/W^n} \qquad (3\text{-}12)$$

where K and n are parameters, and W is the traffic load in 18-kip ESALs. Figure 3-3 shows the behavior of the performance function for different values of K.

As can be verified in Figure 3-3, the performance function $g(w)$ has an inflection point, and an asymptote at $g(W) = 1.0$

The damage function $g(W)$ can also be expressed as the ratio of the loss in serviceability after W 18-kip ESALs to a specified maximum design loss.

Let P_i be the initial PSI (at $W = 0$), P_t be the PSI after W_t 18-kip ESALs, and let P_f be a lower bound on the PSI. Then the relative loss after W_t ESALs can be expressed as:

$$g_t = \frac{P_i - P_t}{P_i - P_f} \qquad (3\text{-}13)$$

Note that Eq. (3-13) is similar to Eq. (3-1) with the exception that the critical value P_c has been substituted with the lower bound P_f.

From Eq. (3-13), it is possible to express P_t as a function of g_t, as follows:

$$P_t = P_i - (P_i - P_f)\, g_t \qquad (3\text{-}14a)$$

Eq. (3-14a) can be further rewritten after using Eq. (3-12). The final result is given by:

$$P_t = P_i - (P_i - P_f)\, e^{-K/W^n} \qquad (3\text{-}14b)$$

Eq. (3-14b) is plotted in Figure 3-4 for different values of K, and in Figure 3-5 for different values of P_f.

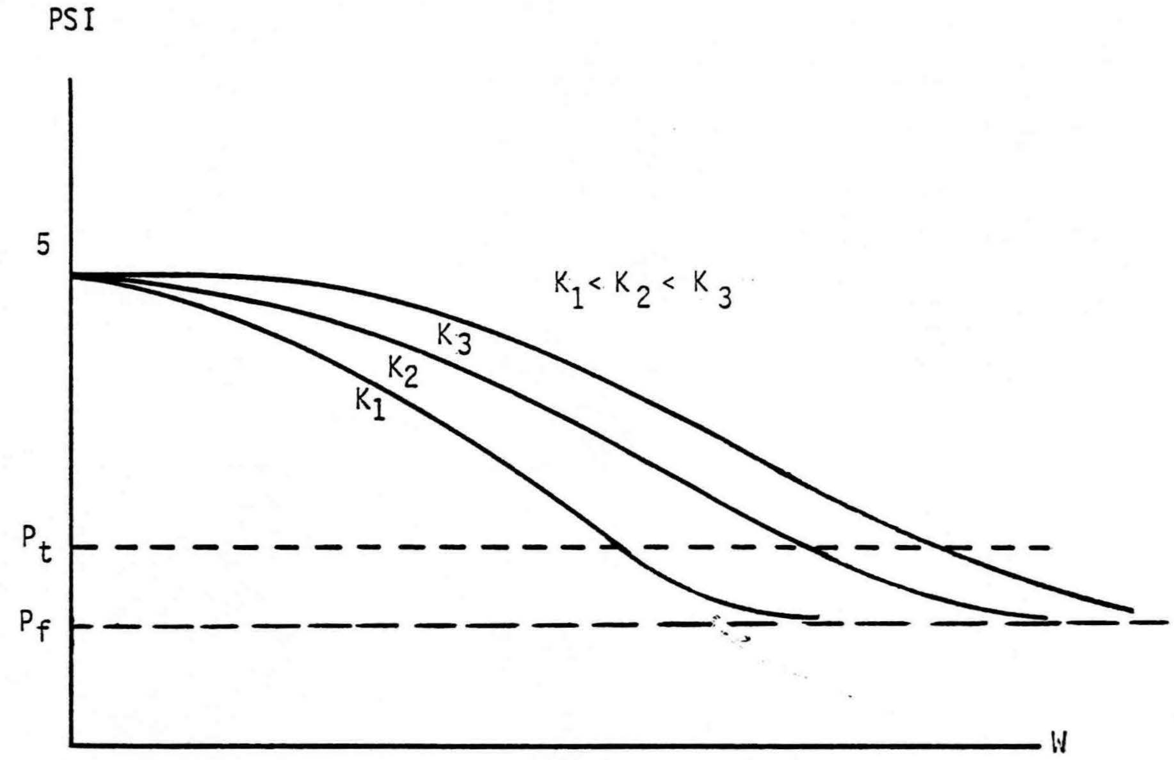


Figure 3-4. Serviceability vs W, for Different K's

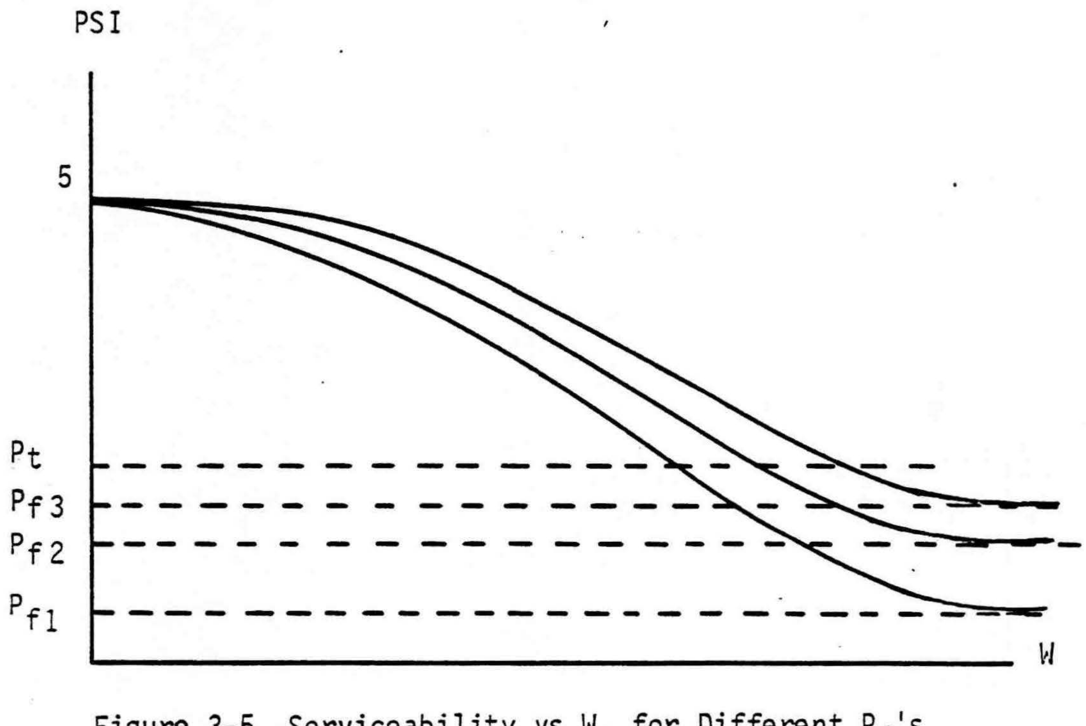


Figure 3-5. Serviceability vs W, for Different P_f's

As illustrated in Figures 3-4 and 3-5, the serviceability value P_f is actually an asymptote of the serviceability curve. The curve has an S-shape which indicates that beyond the inflection point the rate of loss in serviceability is reduced as pavement age increases. This behavior may be explained as a result of routine maintenance over the years. Because of the asymptotic behavior of the curve, a specific terminal value P_t, at which rehabilitation is considered necessary, must satisfy the condition $P_t > P_f$, as shown in Figure 3-4; otherwise, the terminal value P_t is never reached and the pavement is assumed to fail as a result of one or more types of distress. The distress analysis will be presented in Section 3.2.2.

The complete determination of the postulated pavement performance function, Eq. (3-14b), requires the estimation of the parameters K, n, and P_f. The parameters can be estimated according to two different procedures. The first procedure, referred to as the statistical approach, uses past data on traffic loads between rehabilitations along with the theory of maximum likelihood estimators. The development of the statistical approach is shown in Appendices 4 and 6.

The second procedure, referred to as the mechanistic approach, computes the values of each parameter as a function of traffic, design, and climatic variables. For a specific pavement section, these variables are observed and each parameter is computed through regression analysis formulas. The independent variables used in the mechanistic approach are given in Table 3-2. Flexible pavements in the state of Texas can be generally classified into three groups: (a) Hot mix pavements, (b) Surface treated pavements, and (c) Overlays. Average values of the mechanistic properties are also given in Table 3-2. The formulation of the mechanistic approach is summarized in Appendix 6 (parts A and B).

TABLE 3-2. AVERAGE VALUES OF THE MECHANISTIC AND CLIMATIC VARIABLES BY TYPE OF PAVEMENT

Variable	Hot Mix Pavement	Surface Treated Pavement	Overlay
Thornthwaite Index (TI)	3.6	6.2	7.5
Mean Precipitation (PR)	2.0	2.4	2.6
Freeze-thaw cycle (FTC)	54.2	41.9	36.2
Wet-thaw cycle (WFTC)	4.3	3.7	3.3
Mean Annual Temperature (TM)	62.6	64.0	65.1
18-kip ESALs (W)	368,300	94,700	1,089,100
Average daily traffic (ADT)	3,140	567	4,832
Dynaflect I (DMD)	1.17	1.54	1.10
Dynaflect 2 (VOL)	0.42	0.61	0.35
Composite Stiffness (AS)	0.57	0.69	0.76
Subgrade Stiffness (SCI)	0.24	0.25	0.22
Texas triaxial class (TTC)	4.4	5.1	5.2
Liquid Limit (SLL)	39.3	43.6	45.6
Plasticity Index (SPI)	21.1	25.3	27.1
Years since construction (T)	11.7	19.4	26.1
% Subgrade (SPP) Soil passing sieve 200	19.8	19.6	19.6

Note: Every variable name has in parenthesis the name used in the regression equations contained in Appendix 1.

The performance relationship defined in Eq. (3-15) was used in NULOAD as a substitute for the AASHTO equation in the case of Texas flexible pavements. For each of the three most important types of flexible pavements, the parameters K and P_f were computed by the procedures of Appendices 8 and 9. The corresponding results, summarized in Table 3-3, were used in the new program (RENU). As can be seen, both the statistical and mechanistic approaches yield consistent results in K values but are somewhat different in the P_f values.

TABLE 3-3. PARAMETERS OF FLEXIBLE PAVEMENTS PERFORMANCE EQUATIONS (PSI)

Type of Pavement	Mechanistic Approach		Statistical Approach	
	K	P_f	K	P_f
Hot Mix Pavement Rural, Low traffic	41,250.	3.36	47,925.	2.111
Hot Mix Pavement Rural, High traffic	412,500.	3.36	479,250.	2.111
Hot Mix Pavement Urban, Low traffic	103,125.	3.36	119,813.	2.111
Hot Mix Pavement Urban, High traffic	1,031,250.	3.36	1,198,125.	2.111
Surface Treated Pavement - Rural	6,300.	3.24	6,978.	1.974
Surface Treated Pavement - Urban	13,125.	3.24	14,538.	1.974
Overlay, Rural Low traffic	58,500	3.26	51,935.	1.631
Overlay, Rural High traffic	585,500	3.26	519,350.	1.631
Overlay, Urban Low traffic	155,250.	3.26	137,828.	1.631
Overlay, Urban High traffic	1,552,500.	3.26	1,378,275.	1.631

Note: The value of n was assumed equal to 1 to simplify the analysis, see Appendices 6.

3.2.2 Basic Equations for Distress Analysis

The previous approach explained thus far bases the calculation of remaining pavement life upon serviceability index alone. However, it is well known that pavements may be seriously distressed and in need of major rehabilitation before the serviceability index drops to its terminal value. This is particularly true of pavements with severe alligator and transverse cracks. In cases when P_f is higher than P_t or when the remaining life calculated from the serviceability index equation is very long (say 30 to 40 years), the pavement will probably need major rehabilitation due to distress.

The analysis of pavement distress can be accomplished by examining the area ~~of each~~ of the following types of distress: alligator cracking, longitudinal cracking, transverse cracking, rutting, flushing, corrugation, patching, and ravelling However, alligator and transverse cracking are the most important distress types in Texas. The degree or range to which a type of distress is extended can be expressed as the percent of the total pavement surface area in need of repair. The seriousness of the distress may be expressed as crack width, crack depth, relative displacement at a joint, etc. Usually, the severity of a given type of distress can be subjectively estimated by comparing the observed distress with photographs of different levels of severity, such as none, slight, moderate, or severe, and choosing numbers between zero and one (or 0 and 100%) to quantify the seriousness of surface failures. The Table 3-4 shows the rating values for area and severity used in this project.

TABLE 3-4. RANGES FOR AREA AND SEVERITY

AREA		SEVERITY	
Rating	Area Measurement	Range (grade)	Severity Measurement
0	.0005	None	.0005
1	.080	Slight	.167
2	.230	Moderate	.333
3	.500	Severe	.500

The distress equations developed for Texas flexible pavement data are of the same form as the PSI equations,

$$a = e^{-a_0/W^n} \tag{3-15}$$

$$s = s_f e^{-a_1-a_2/W^n} \tag{3-16}$$

where

a is percent of pavement surface area covered by distress

s is severity of distress expressed in numerical form

a_o, a_1, and a_2 are deterioration rate constants

W is traffic load in 18-kip ESALs.

Figure 3-6 illustrates the variation of distressed area for different values of the constant a_o, as the traffic load is changed. The corresponding variation of the degree of distress severity is illustrated in Figure 3-7.

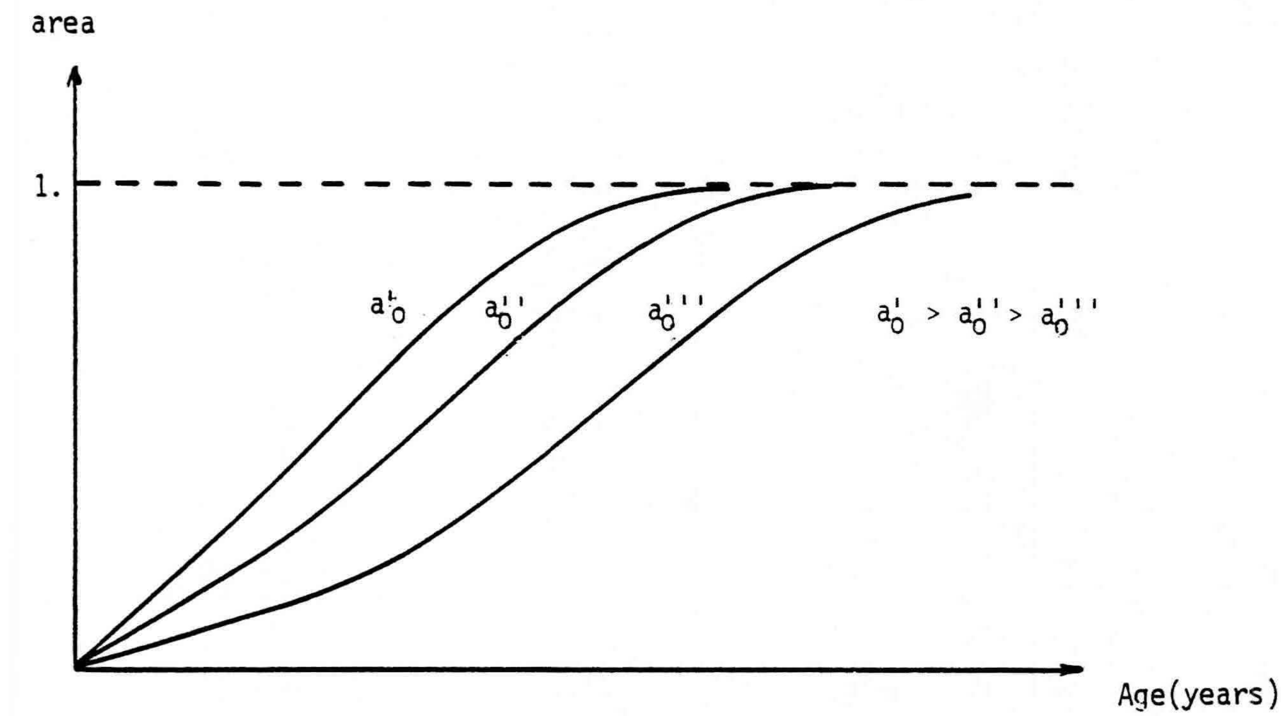


Figure 3-6. Variation of Area in a Distressed Pavement.

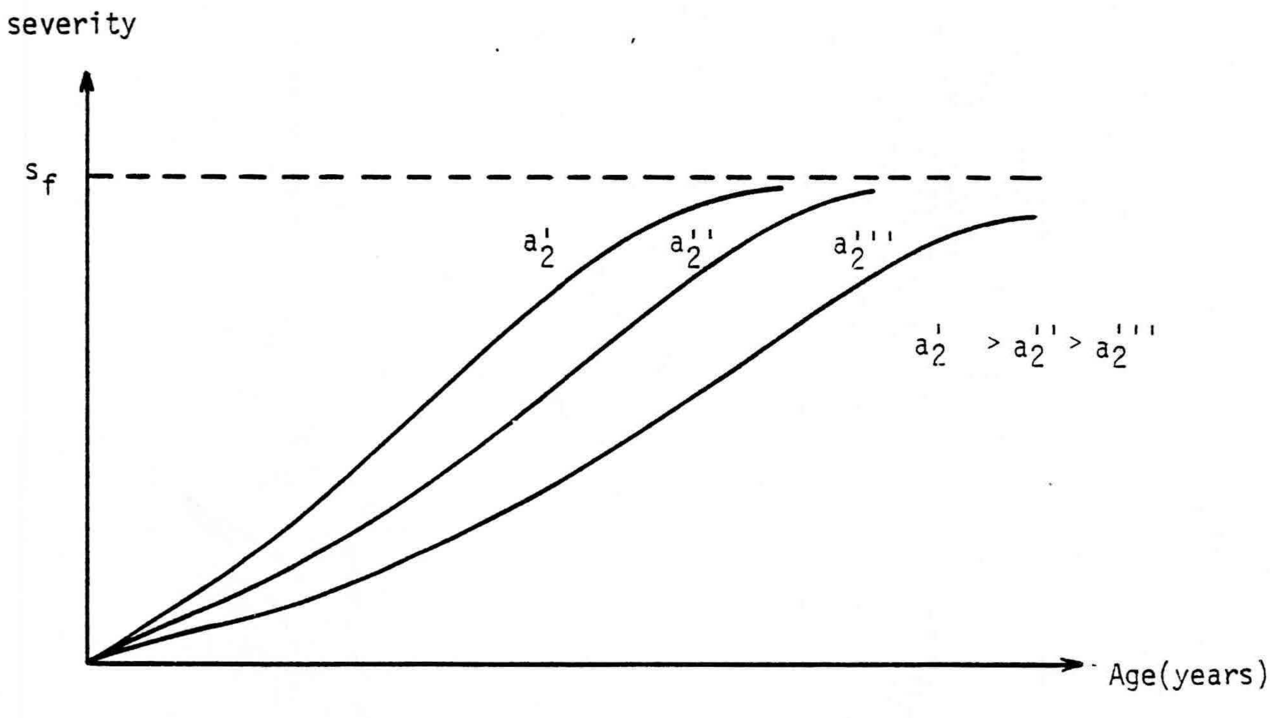


Figure 3-7. Variation of the Severity of the Distressed Pavement

Appendix 6A summarizes the development of a statistical procedure to compute the deterioration rate constants for the distress approach. Appendix 6B summarizes the development of a mechanistic procedure to estimate the same constants. Finally, Table 3-5 contains the results for Texas flexible pavements.

TABLE 3-5. PARAMETERS FOR FLEXIBLE PAVEMENTS PERFORMANCE EQUATIONS (DISTRESS)

Type of Pavement	Mechanistic Approach				Statistical Approach			
	a_0	a_1	a_2	S_f	a_0	a_1	a_2	S_f
Hot Mix, Rural Low traffic	2,000,000	0.40	42,000	0.85	207,944	0.40	39,308	0.85
Hot Mix, Rural High traffic	2,250,000	0.43	110,350	0.80	2,079,442	0.43	120,011	0.80
Hot Mix, Urban Low traffic	480,000	0.52	90,000	0.95	519,860	0.52	91,413	0.95
Hot Mix, Urban High tralfic	5,000,000	0.45	900,000	0.90	5,198,604	0.45	1,033,475	0.90
Surface treated - Rural	12,500	0.25	3,900	0.80	13,863	0.25	4,400	0.80
Surface treated - Urban	35,300	0.45	7,000	0.90	34,657	0.45	6,890	0.90
Overlay, Rural Low traffic	170,000	0.28	30,000	0.75	180,218	0.28	32,620	0.75
Overlay, Rural High traffic	1,170,000	0.44	230,000	0.85	1,802,183	0.44	235,657	0.85
Overlay, Urban Low traffic	420,000	0.39	105,000	0.87	450,546	0.39	106,540	0.87
Overlay, Urban High traffic	4,100,000	0.48	810,000	0.92	4,505,460	0.48	843,575	0.92

Note: In both approaches, the values of a, and S_f have been assumed within a reasonable interval to satisfy the design life of each type of pavement.

3.3 Survivor Curves for Flexible Pavements

Survivor curves are empirical probability functions used to predict the percent of pavement mileage of a specific age which will not need rehabilitation in the short range future. This in turn can be used to estimate the percent of mileage which will need rehabilitation in the near future. This information complemented with data on existing mileage and rehabilitation cost can be used to estimate the funds needed in each period of a specified planning horizon.

Historical pavement data recorded by the Texas Highway Department and Texas Transportation Institute were considered as input to generate survivor curves for the most important types of Texas flexible pavements. However, lack of accurate and sufficient information for older pavements represents an important limitation in the complete determination of survivor functions. Some adjustments were made in order to obtain resulting equations that can be handled by conventional computer procedures.

Currently, the NULOAD program uses normal distribution with assumed mean and standard deviation to generate survivor curves. The new program RENU contains survivor curves generated on the basis of available data for each of the most important types of pavements in Texas.

The survivor functions developed for RENU can be generally written as:

$$V = 1 - e^{-q/W^r} \qquad (3\text{-}17)$$

where V is the percent of surviving mileage,

q is a constant affecting the survivor function,

r is the exponent that affects the 18-kip ESALs,

W is the number of 18-kip ESALs since construction or last rehabilitation.

The basic procedure of RENU to estimate the mileage of a given type of pavement which will (or will not) need rehabilitation is illustrated in Figure 3-8 Figure 3-8(a) represents the distribution of mileage by level of serviceability index. Figure 3-8(b) corresponds to the performance function and shows the traffic loads, W*, at which a critical value of serviceability is reached. Figure 3-8(c) shows the probability density function for the mileage in need of rehabilitation. Figure 3-8(d) is the survivor curve. It gives the percent of pavement mileage with critical performance index which will not fail by the time the traffic load W* is reached.

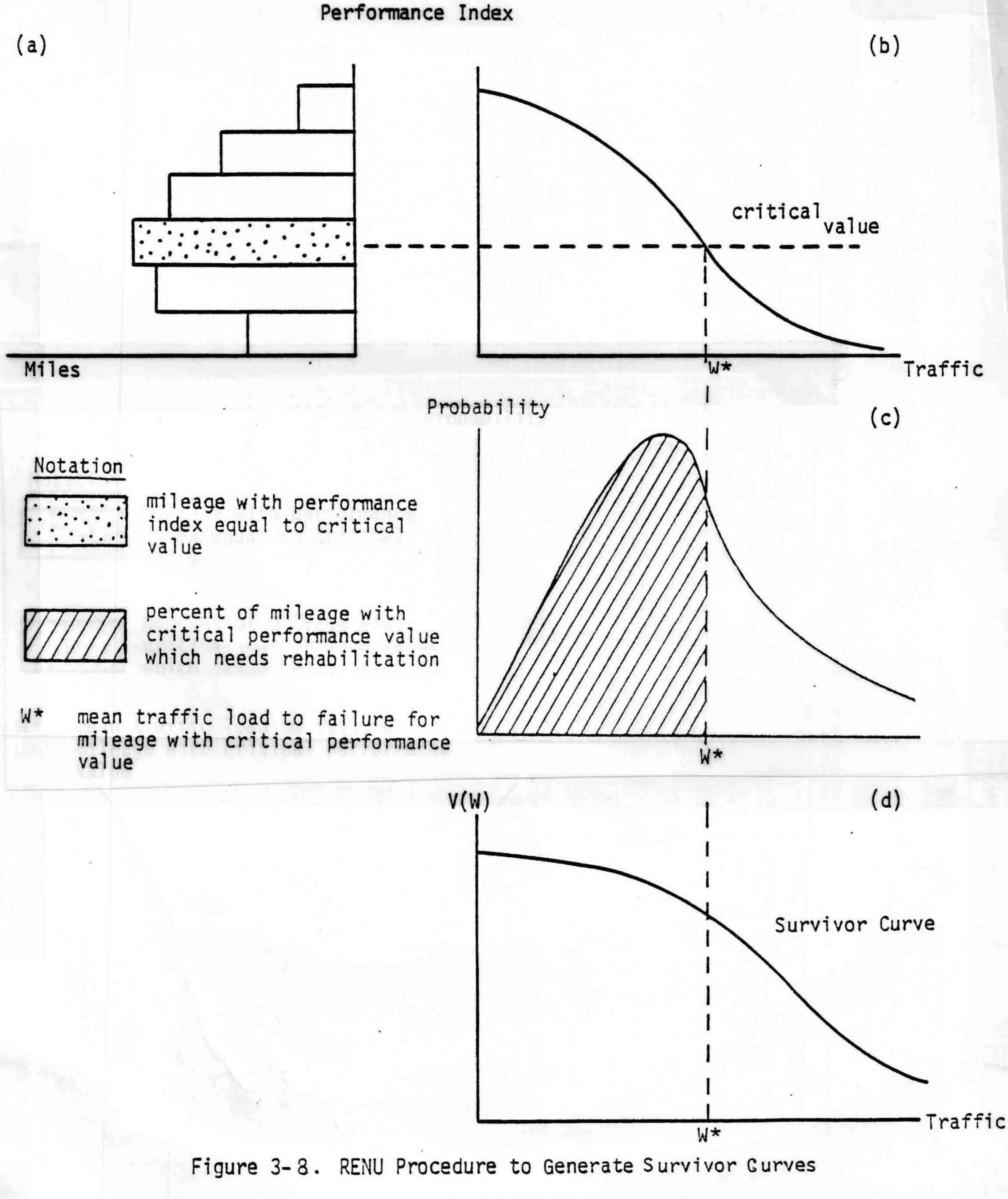


Figure 3-8. RENU Procedure to Generate Survivor Curves

The complete determination of the survivor curve defined by Eq. (3-17) requires the estimation of the parameters q and r. This can be accomplished using the following procedures which are consistent with the methodology illustrated in Figure 3-8.

Step 1: Use the performance functions defined by Eqs. (3-12), (3-15), and (3-16) to generate values of W_t given critical values of the performance index (P_t, a_t and s_t). Define m as the number of values generated.

Step 2: Compute the coefficient of variation (See Appendix 5) and set it equal to $\overline{W}/S_w$, where $\overline{W}$ is the average traffic load corresponding to the m values generated in Step 1 and S_w is the standard deviation estimated from the same set of W's.

Then from Appendix 5 it can be observed that

$$\frac{\overline{W}}{S_w} = \left[\frac{\Gamma(\frac{r - 2}{r})}{\Gamma(\frac{r - 1}{r})} - 1\right]^{1/2} \tag{3-18}$$

where $\Gamma(.)$ is the Gamma function.

Step 3: Use a numerical method to solve the Eq. (3-18) for r.

Step 4: Compute the value of q by either of the two following procedures.

Procedure 1. Set the value of q equal to:

$$q = \left[\frac{\overline{W}}{(\frac{r - 1}{r})}\right]^{r} \tag{3-19}$$

where r is obtained in Step 3. Eq. (3-19) is developed in Appendix 5.

Procedure 2. Compute the value of q by the following expression:

$$q = \frac{m}{\sum_{i=1}^{m} W_i^{-r}} \qquad (3\text{-}20)$$

Eq. (3-20) is explained in Appendix 4.

The application of the procedure defined by Steps 1 through 4 using different levels for the critical index (P_t, a_t, or s_t) allows the generation of a family of functions

$$r = \begin{cases} F_1(P_t) \\ F_2(a_t) \\ F_3(S_t) \end{cases} \qquad (3\text{-}21)$$

where F_1 corresponds to the PSI option and F_2, F_3 to the distress option.

Eq. (3-22) applies to all categories of surface treated pavements. This equation was obtained by regression techniques. The corresponding correlation coefficient was equal to -0.594 :

$$r = 13.53 - 3.85 \ Ln(P_t) \qquad (3\text{-}22)$$

Eq. (3-23) applies to rural, hot mix pavements. The corresponding correlation coefficient was equal to -0.963 :

$$r = 35.72 - 28.07 \ Ln(P_t) \qquad (3\text{-}23)$$

Eq. (3.24) applies to urban, hot mix pavement. The correlation coefficient in this case was equal to -0.0976 :

$$r = 44.22 - 37.30 \ Ln(P_t) \qquad (3\text{-}24)$$

Eq. (3-25) applies to any type of overlaid pavement. The correlation coefficient was equal to -0.599:

$$r = 11.85 - 0.34\ P_t^{\ 3} \qquad (3\text{-}25)$$

Similar equations can be developed for the distress approach, but due to the lack of information, the values of r and q have been computed on the basis of $a_t = 0.5$ and $S_t = 0.5$.

Additionally, similar functions can be developed for the relationship between q and P_t in the PSI case. After investigating several types of algebraic expressions, the following function was found to exhibit the best goodness of fit:

$$Ln(g) = A + BP_t \qquad (3\text{-}26)$$

The parameters A and B depend on the type of flexible pavement, as shown below:

(a) For hot mix pavement the relationships are:

$$Ln(q) = 581.21 - 172.76\ P_t \qquad (3\text{-}27)$$

and

$$Ln(q) = 496.85 - 148.23\ P_t \qquad (3\text{-}28)$$

Eq. (3-27) applies to high traffic and the corresponding correlation coefficient was equal to: -0.958. Eq. (3-28) applies to low traffic and the corresponding correlation coefficient was equal to -0.832.

For surface treated pavements the relationship is:

$$Ln(q) = 111.35 - 5.65\ P_t \qquad (3\text{-}29)$$

The correlation coefficient corresponding to Eq. (3-29) was -0.67.

(c) For overlaid pavements the relationships are:

$$Ln(q) = 235.3 - 64.82 P_t \quad (3\text{-}30)$$

$$Ln(q) = 375.17 - 114.25 P_t \quad (3\text{-}31)$$

Eq. (3-30) applies to low traffic and has a correlation coefficient of -0.602. Eq. (3-31) applies to high traffic and has a correlation coefficient of -0.603.

For the distress approach data on 18-kip ESALs and nature of the failure (area or severity) are not available to develop similar relationships.

Tables (3-6) and (3-7) contain the values of q and r obtained for the principal types of pavement in Texas. Table (3-6) has the values of the parameters for the PSI case, and Table (3-7) for the distress case.

TABLE 3-6. SURVIVOR CURVE PARAMETERS, PSI CASE

Type of Pavement	r_1	q_1
Hot mix pavement Rural, Low traffic	10.0	7.028×10^{54}
Hot mix pavement Rural, High traffic	10.0	7.03×10^{64}
Hot mix pavement Urban, Low traffic	10.0	6.66×10^{58}
Hot mix pavement Urban, High traffic	10.0	6.70×10^{68}
Surface treated pavement Rural	10.0	1.373×10^{44}
Surface treated pavement Urban	10.0	2.115×10^{48}
Overlay, Rural, Low traffic	10.0	2.10×10^{54}
Overlay, Rural, High traffic	10.0	2.10×10^{64}
Overlay, Urban, Low traffic	8.0	4.24×10^{46}
Overlay, Urban, High traffic	10.0	2.0×10^{68}

The numbers in Table 3-7 are average values computed with the same data used to develop Eqs. (3-22) through (3-25). Due to the limited data on distress types, the average values will be used in the RENU program instead of the equations.

TABLE 3-7. SURVIVOR CURVE PARAMETERS, DISTRESS CASE

TYPE OF PAVEMENT	AREA		SEVERITY		
	R_2	q_2	q_3	S_f	r_3
Hot Mix Pavement, Rural	3.0	1.87×10^{16}	2.3×10^{14}	0.8	2.5
Hot Mix Pavement, Rural High Traffic	2.5	1.08×10^{16}	7.3×10^{15}	0.8	2.5
Hot Mix Pavement, Urban Low Traffic	3.0	2.92×10^{17}	5.11×10^{15}	0.85	2.7
Hot Mix Pavement, Urban High Traffic	3.2	6.93×10^{21}	5.1×10^{11}	0.8	1.75
Surface Treated Pavement Urban	2.3	4.45×10^{10}	1.47×10^{9}	0.9	2.0
Surface Treated Pavement Rural	2.25	3.3×10^{9}	3.64×10^{11}	0.86	2.75
Overlay, Rural Low Traffic	3.0	1.22×10^{16}	3.2×10^{14}	0.75	2.75
Overlay, Rural High Traffic	2.5	7.56×10^{15}	1.7×10^{14}	0.95	2.25
Overlay, Urban Low Traffic	2.9	4.99×10^{16}	26×10^{11}	0.93	2.0
Overlay, Urban High Traffic	3.1	9.14×10^{20}	1.3×10^{15}	0.92	2.25

A graphical representation of the survivor curves for the principal types of Texas pavements is given in Appendix 3.

3-4 Implementation of Texas Flexible Pavements Performance Equations

As it has been previously indicated throughout this report, the current version of the NULOAD procedure uses the AASHTO methodology to examine the service life cycle of highways. The fundamental procedure of the program is performed by the LYFCYC subroutine for which a simplified flow chart is given in Figure 3-9 to support further discussion of the RENU program. Figure 3-9 contains the basic methodology for the computation of the 18-kip ESALs and the design of the required pavement; in addition to the design, the program also estimates rehabilitation costs. Steps (1) and (2) of the flow-chart are accomplished through the AASHTO equations [Eq. (3-8)] in NULOAD. In the RENU program the computation of 18-Kip ESALs and PSI values is made through the Texas performance equations [Eqs. (3-12) and (3-15)]. Figure 3-10 shows a flow chart containing the methodology followed to compute 18-Kip ESALs through the Texas performance equations. Basically, the RENU program assigns a failure option (either PSI or distress) to each type of flexible pavements, depending on the values of P_t and P_f.

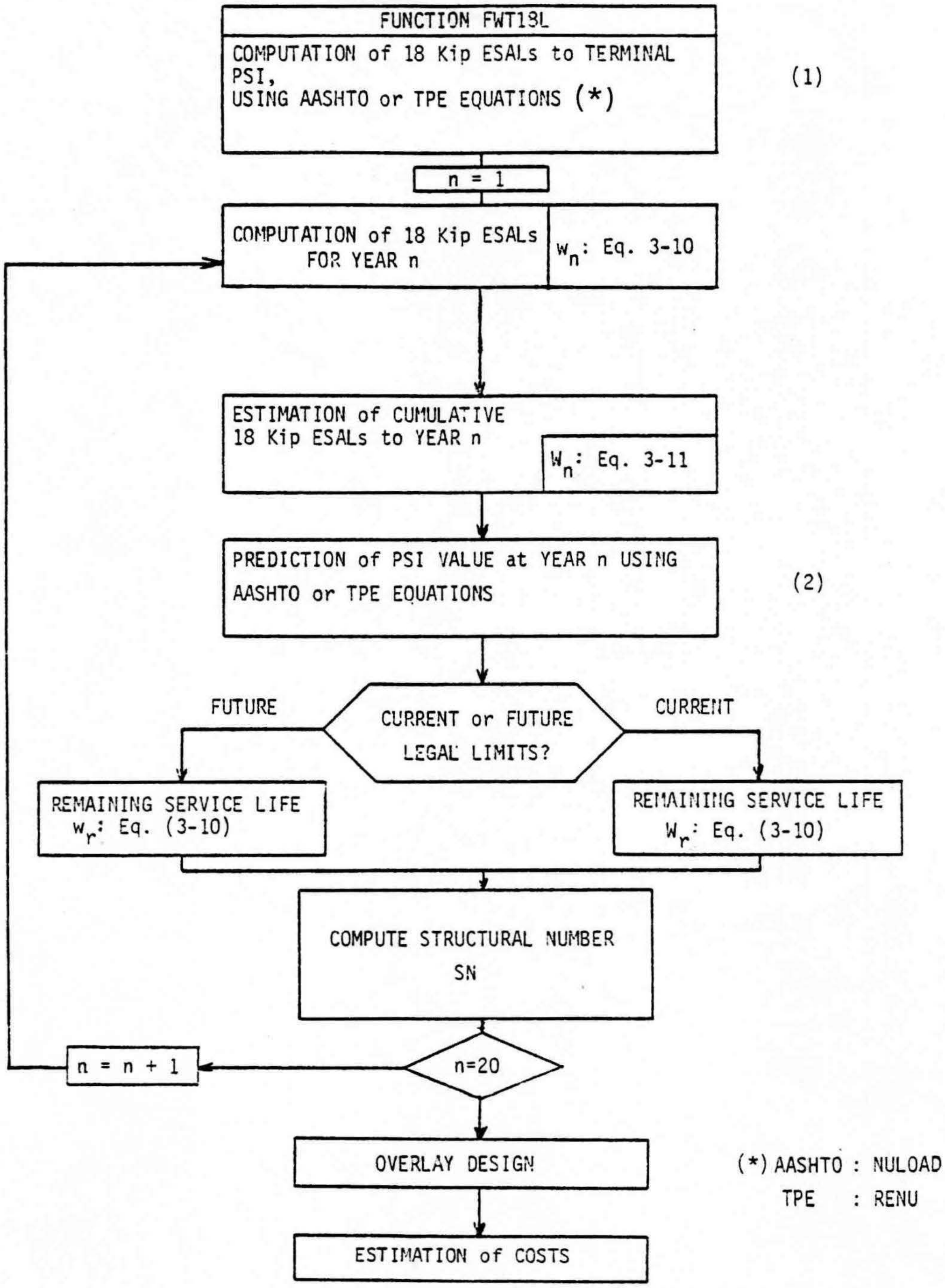


Figure 3-9. Basic Methodology of Subroutine LYFCYL, in RENU or NULOAD

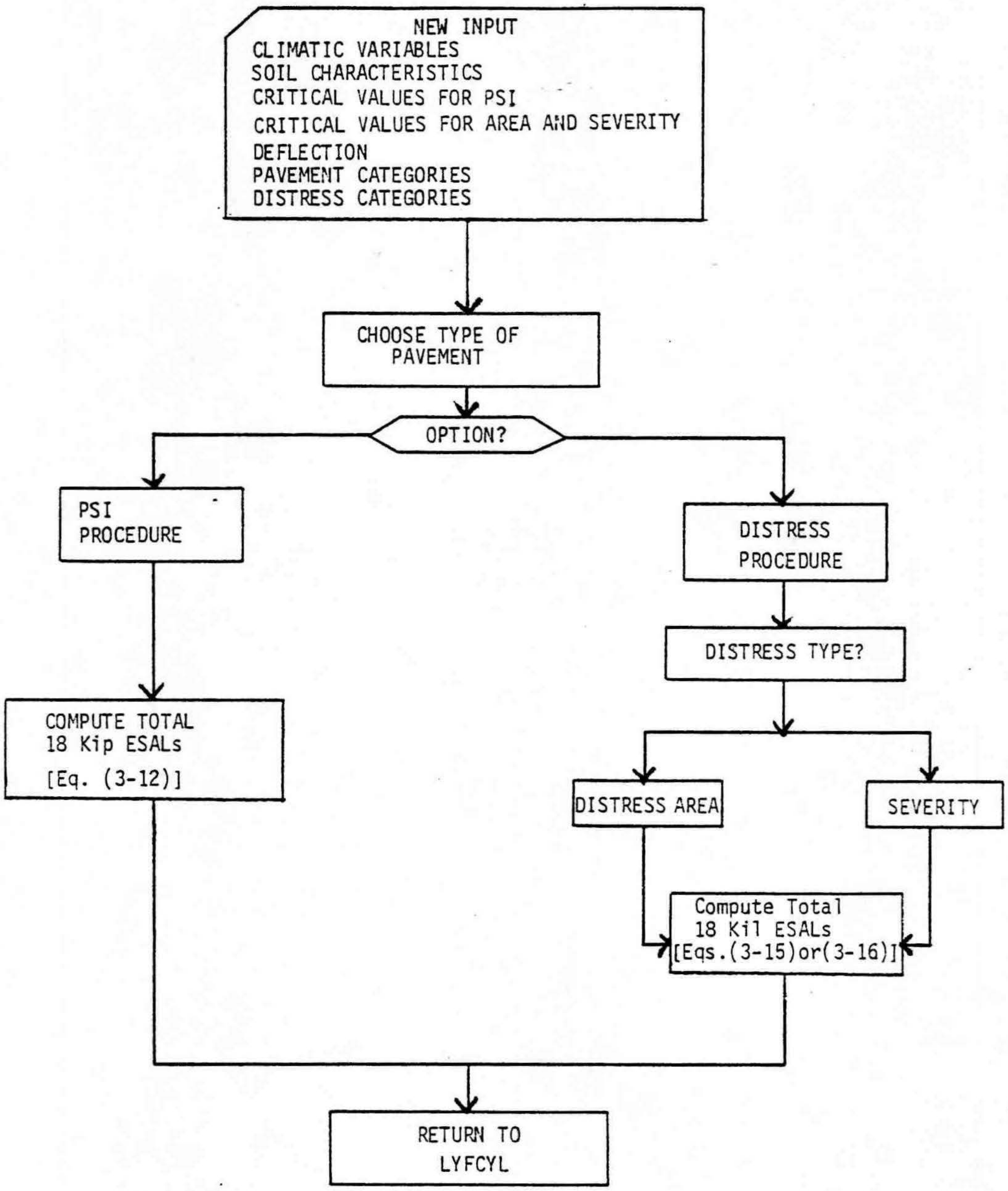


Figure 3-10. Texas Performance Equations Procedure to Compute 18-Kip ESALs

Chapter 4

RIGID PAVEMENT METHODOLOGY

4.1 Modification of AASHTO Equation for Rigid Pavement in Texas

The AASHTO performance equation provides relationships among traffic and pavement performance, structural design, and thickness. Although this equation represents the most comprehensive development of the relationships, the results are for general use. Further, the equation can be modified in order to improve the accuracy of prediction by utilizing local input data. For instance, Texas rigid pavements are normally 8 inches thick and have a K-value in the 60 to 200 pci range. Limestone and siliceous river gravel are two common subbase materials. Pavements reach a terminal level of service with approximately 6,000,000 applications of 18 kips ESAL. Information such as this has been monitored in Texas and has been very useful in updating the general AASHTO performance equation for the state's environment.

The revised AASHTO performance equation was developed to ease the use in the choice of local input data, especially types of subbase material. After modification, the sensitivity of the equation was checked to validate the prediction results as shown in Table 4.1 and Figure 4.1. The major change in the revised AASHTO performance equation is similar to the Strauss performance equation which was developed from extensive Texas rigid pavement data, as shown in Table 4.2.

The input data needed to develop a modified performance relationship for rigid pavements can be unified as follows:

E : Modulus of elasticity of the concrete
K : Modulus of support reaction
D : Thickness of pavement
C : Constant

The general form of the revised AASHTO performance relationship is given by Eq. (4-1):

$$\text{Log } W_t = 7.37 \log (D+1) + 0.06 + \frac{-0.17609}{1 + \frac{1.624 \times 10^7}{(D+1)^{8.46}}}$$

$$+ 3.42 \log \left(\frac{C}{215.63} \right) \frac{D^{0.75} - 1.132}{D^{0.75} - \frac{18.42}{Z^{0.25}}} \qquad (4\text{-}1)$$

where $Z = \frac{E}{K}$.

4.1.1 Siliceous-River-Gravel

Typical values for this subbase material are:

$K = 150$ pci

$E = 6.5 \times 10^6$

$D = 8"$

$$Z = \frac{E}{K} = \frac{6.5 \times 10^6}{150} = 4.33 \times 10^4$$

Assuming $W = W_t$ 18-Kip ESALs, the modified performance relationship (4-1) can be used to obtain Eq. (4-2):

$$\log W_t = 6.79885 + 3.42 \log \left(\frac{C}{215.63}\right) (1.04162)$$

$$= -1.12186 + 3.42 \log (C) \qquad (4\text{-}2)$$

From Eq. (4-2),

$$\log C^{3.42} = \log W_t + 1.12186$$

$$C^{3.42} = W_t \quad 10^{1.12186} \qquad (4\text{-}3)$$

$$C = (13.239 \; W_t)^{0.29240}$$

Assuming $W_t = 6.0 \times 10^6$ in Eq. (4-3), we can write

$$C = 204.157$$

4.1.2 Limestone

A similar procedure can be followed to compute the value of C in the case of limestone subbases:

$$E = 4.4\text{x}10^{6}$$

$$K = 150 \text{ pci}$$

$$Z = \frac{4.4\text{x}10^{6}}{150} = 2.93\text{x}10^{4}$$

$$D = 7.42''$$

$$\log W_t = 6.5992 + (3.42) \log \frac{C}{215.63} \frac{3.364}{3.0883}$$

$$= -1.2550772 + 3.42 \log C \qquad (4\text{-}4)$$

$$\log C^{3.42} = \log W_t + 1.2550772 \qquad (4\text{-}5)$$

Again, assuming $W_t = 6.0 \times 10^6$ in Eq. (4-5), we finally obtain

$$C = 223.31$$

4.1.3 Summary of Modified Performance Equations

The final revised AASHTO performance equation for limestone in Texas is:

$$\log W_t = 7.37 \log (D+1) + 0.06 + \frac{-0.17609}{1 + \frac{1.624\text{x}10^{7}}{(D+1)^{8.46}}}$$

$$+ \; 3.42 \log 1.04 \; \frac{D^{0.75} - 1.132}{D^{0.75} - 18.42/Z^{0.25}} \qquad (4\text{-}6)$$

The revised AASHTO performance equation for siliceous-river-gravel is as follows:

$$\log W_t = 7.37 \log (D+1) + 0.06 + \frac{-0.17609}{1 + \frac{1.624 \times 10^7}{(D+1)^{8.46}}}$$

$$+ 3.42 \log 0.95 \frac{D^{0.75} - 1.132}{D^{0.75} - 18.42/Z^{0.25}} \quad (4\text{-}7)$$

TABLE 4-1. SENSITIVITY STUDY OF THICKNESS OF PAVEMENT, K-VALUE, AND NUMBER OF APPLICATIONS

D	K	Limestone	D	K	Gravel
7.42"	60	4.42×10^6	8.00"	60	4.68×10^6
	100	5.18×10^6		100	5.33×10^6
	150	6.00×10^6		150	6.00×10^6
	200	6.75×10^6		200	6.59×10^6
	300	8.16×10^6		300	7.67×10^6
	600	1.22×10^7		600	1.05×10^7
8.00"	60	7.01×10^6	8.72"	60	8.14×10^6
	100	8.12×10^6		100	9.16×10^6
	150	9.30×10^6		150	1.02×10^7
	200	1.04×10^7		200	1.11×10^7
	300	1.23×10^7		300	1.27×10^7
	600	1.78×10^7		600	1.70×10^7
10.00"	60	2.95×10^7	11.20"	60	4.31×10^7
	100	3.32×10^7		100	4.74×10^7
	150	3.70×10^7		150	5.16×10^7
	200	4.04×10^7		200	5.52×10^7
	300	4.63×10^7		300	6.15×10^7
	600	6.16×10^7		600	7.66×10^7
12.00"	60	1.00×10^8	13.32"	60	1.41×10^8
	100	1.11×10^8		100	1.53×10^8
	150	1.21×10^8		150	1.65×10^8
	200	1.30×10^8		200	1.74×10^8
	300	1.46×10^8		300	1.91×10^8
	600	1.85×10^8		600	2.29×10^8

For K = 150 PCI

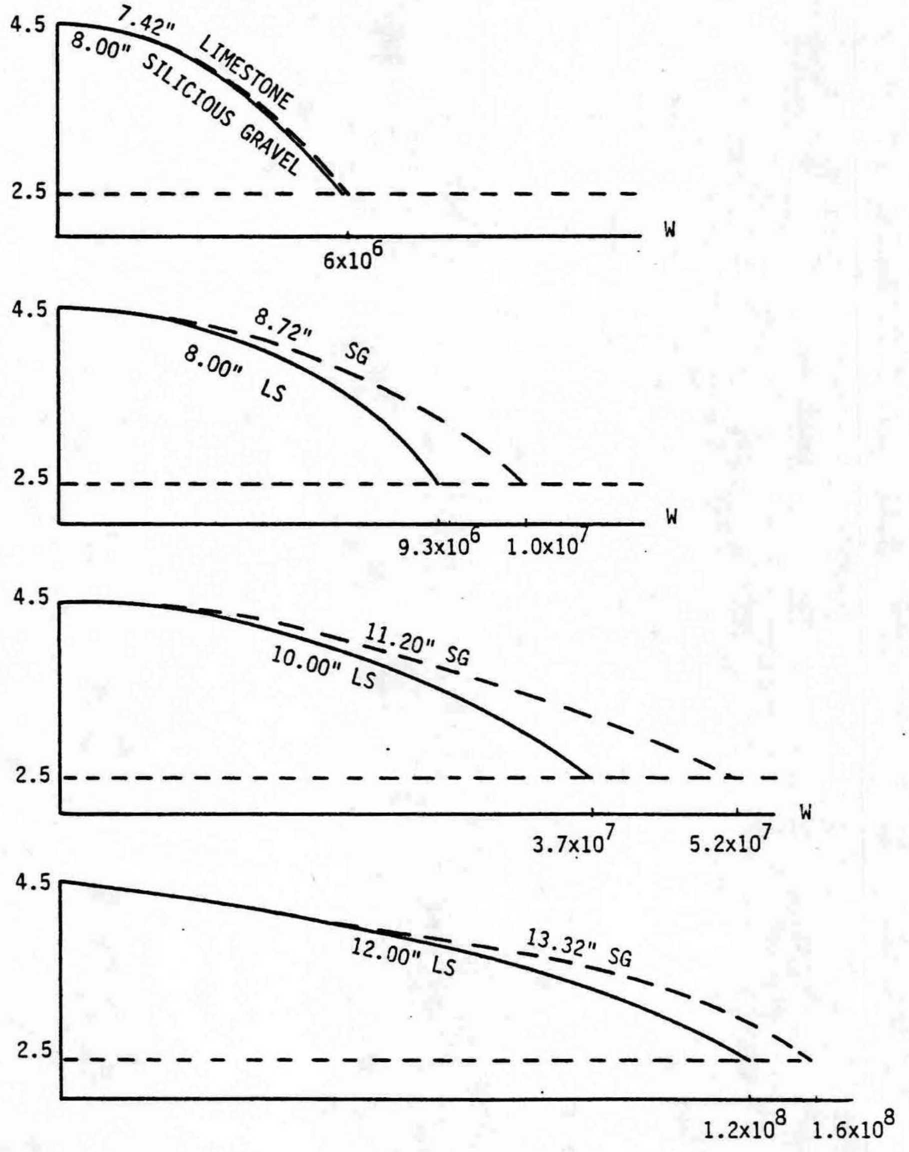


Figure 4-1. Sensitivity of the Revised Equation

TABLE 4-2. THE EQUIVALENT THICKNESS FROM STRAUSS DESIGN EQUATION

D*(Limestone)	D (Siliceous River Gravel)
7.42"	8.00"
8.00"	8.72"
10.00"	11.20"
12.00"	13.32"

* Thickness

4.2 Texas Survivor Curve for Rigid Pavements

The use of survivor curves is a standard method of making management decisions relative to future estimates of time to retirement of physical properties. Physical properties are said to be retired from service when, for one reason or another, they are removed from productive service or altered and used in a second service life. Winfrey [42] developed many survivor curves that fit into three basic types: symmetrical, left-modal, and right-modal. The symmetrical type with the standard deviation of the survivor curve being defined by user input has been selected for use in NULOAD. The stochastic nature of survivor curves makes it very complicated for the user to select the proper standard deviation. For this reason, the revised NULOAD program makes use of the actual survivor curves from previous research [10]. The actual survivor curves, Figure 4.2, will not exactly represent the probability that a pavement of given age will require a timely overlay, but it will give the best approximation of Texas rigid pavement survivor probability. Velasco [10] verified that at present approximately 50 percent of rigid pavements in Texas will be overlaid by the time they are 15 years old. This is based on the assumption that the rigid pavement will have 15 failures per lane-miles per mile at 15 years of age. The field data shows that this assumption is likely to be realistic. Figure 4.2 shows the actual survivor curves for Texas rigid pavements.

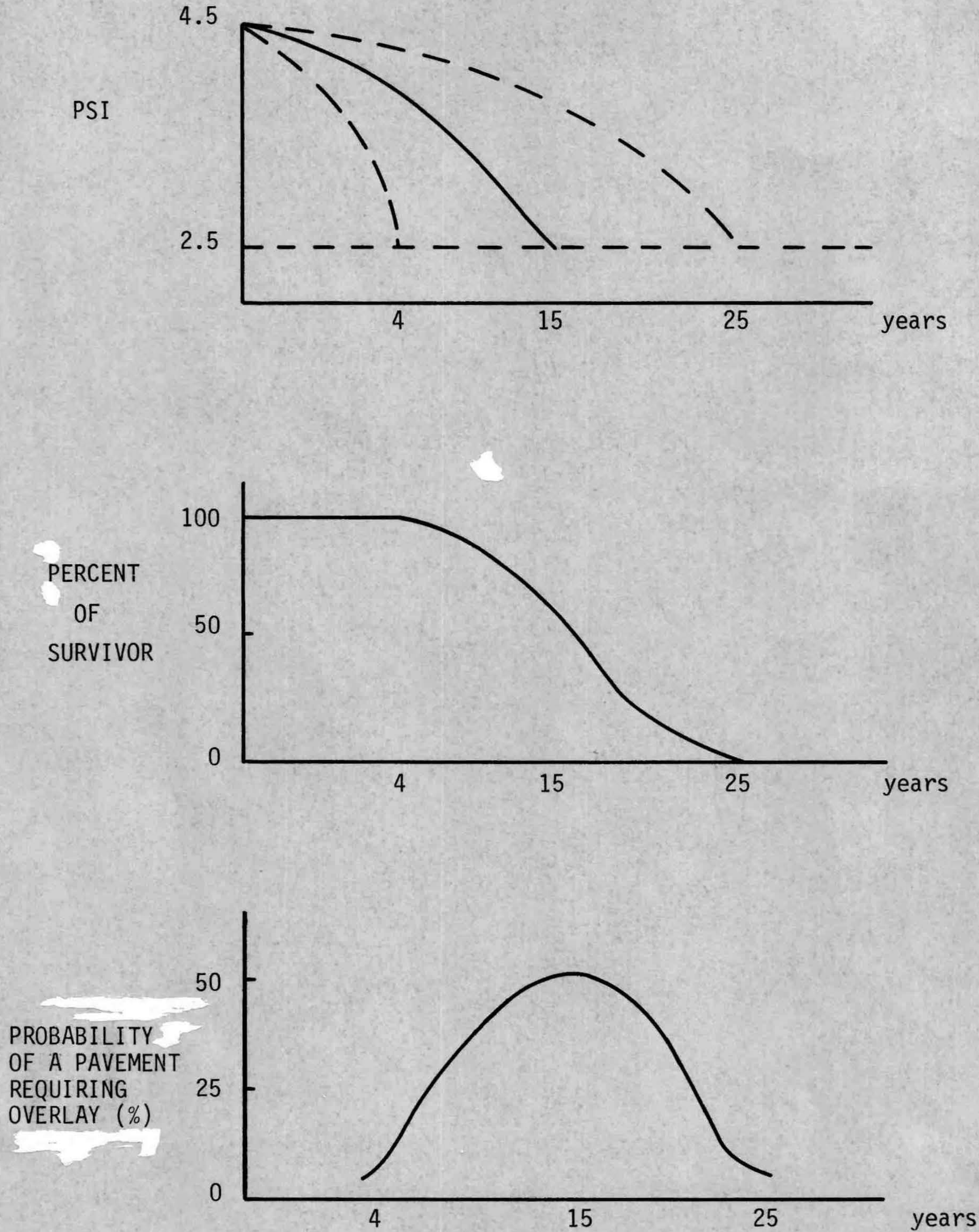


Figure 4-2. Actual Texas Survivor Curve for Rigid Pavements

Chapter 5

ECONOMIC ANALYSIS

The purpose of this chapter is to summarize the basic steps of the methodology followed in RENU to achieve the following two objectives.

(a) Estimate the effects in terms of rehabilitation needs of changes in the legal axle load limits.

(b) Measure the impact of these changes in terms of budget needs for a specified planning horizon.

Although the economic analysis of RENU is similar to that performed by NULOAD, there are a few procedures in RENU which represent important analytical improvements. These procedures are:

(a) Incorporation of the Texas Highway Cost Index to account for future increases in material costs.

(b) Development of a mechanistic procedure to determine the thickness of flexible pavement overlays.

(c) Development of distress prediction models to estimate maintenance for rigid pavements.

5.1 Maintenance Costs

This section presents the analytical tools used to estimate maintenance costs for Texas flexible and rigid pavements. The methodology for flexible pavements is the same already existing in NULOAD: The EAROMAR equations [4] are used to predict maintenance costs for multi-lane freeways as functions of pavement age. For other types of pavements, the EAROMAR results are appropriately modified by multiplying by reduction coefficients reflecting past maintenance data for Texas. The methodology used in RENU for rigid pavements is considered to be more practical than the EAROMAR approach. The number of failures (punchouts and patches) per mile was chosen as the major criterion to predict maintenance needs and costs.

5.1.1 Flexible Pavement Maintenance Costs

RENU has the same maintenance cost options included in NULOAD. These are:

(a) use of the EAROMAR equations
(b) use of historical maintenance data
(c) no consideration of maintenance costs.

The cost models comprising the EAROMAR equations can be classified as follows:

Model 1: Model to estimate the number of square yards of bituminous skin patching per year and per lane mile.

Model 2: Model to estimate crack sealing in bituminous pavements per year and per lane mile.

Model 3: Model to estimate the cost of bituminous base and surface repair per year and per lane mile.

The notation given below is used in the formulation of the flexible pavement maintenance models:

C_1 = cost per square yard of bituminous skin patching

C_2 = cost per linear foot of crack sealing

C_3 = cost per cubic yard of bituminous base and surface repair

T = age of pavement in years

APC = Annual patching cost per lane mile

ASC = Annual sealing cost per lane mile

ABSC = Annual base and surface repair cost

Model 1:

$$APC = \frac{1100\ C_1}{1 + e^{-(T-10)/1.16}} \quad (\$/\text{lane-mile}) \qquad (5\text{-}1)$$

Model 2:

$$ASC = \frac{1000\ C_2}{1 + e^{-(T-10)/1.16}} \quad (\$/\text{lane-mile}) \qquad (5\text{-}2)$$

Model 3:

$$ABSC = \frac{5\ C_3}{1 + e^{-(T-10)/1.16}} \quad (\$/\text{lane-mile}) \qquad (5\text{-}3)$$

The input cost parameters C_1, C_2, C_3 can be obtained from sources such as the 1980 Heavy Construction Cost File [22].

To extend the use of the EAROMAR equations to roadway types other than freeways, samples of past maintenance costs for Interstate Highways, Farm to Market Roads, and U.S. and State Highways were studied to compute average costs per mile for each classification. The reduction factor for a type of pavement is computed as the ratio between the average cost per mile of the given pavement and that for the freeway. Data needed for this analysis were obtained from the SDHPT 1980 maintenance cost files for routine maintenance of bituminous surfaces. The typical routine maintenance actions considered are listed below:

(a) seal coat

(b) edge repair

(c) pot holes

(d) leveling or overlay

(e) correction of bleeding

Table 5.1 summarizes the results of the analysis. As an illustration of the use of this table, the routine maintenance cost for Farm-to-Market roads can be estimated as 38.2% of the cost per mile computed by the EAROMAR equations.

TABLE 5-1. COMPARISON OF MAINTENANCE COSTS

	OBS	MAINTENANCE AVE. EXPENDITURE/LN MILE	% OF INTERSTATE
Interstate	4	$1,027.50	100%
Farm-to-Market	23	391.20	38.2%
State, U.S., other	62	325.10	31.6%

5.1.2 Rigid Pavements Maintenance Costs

Maintenance costs for rigid pavements are expressed as a function of the number of failures per mile of pavement. In Research Project 3-8-75-177, "Development & Implementation of the Design, Construction and Rehabilitation of Rigid Pavements ".

The Center for Transportation Research at the University of Texas at Austin has conducted state wide distress condition survey in 1974, 1978, and 1980. The distress manifestation recorded during these condition surveys were spalling, pumping, punchouts, and patches. Data from condition survey in 1974 and 1978 were used to develop a distress prediction model for CRCP by Noble and McCullough in 1979. Five types of data were utilized for this development of the distress prediction models. Specifically these were data on:

(a) Environmental factors

(b) Construction factors

(c) Traffic

(d) Age of pavement

(e) Pavement distress factors

In accordance with SDHPT criteria, distress failures can be limited to punchouts and repaired patches on the pavement. The selection of the above factors were made on the basis of data availability and the results of an Analysis of Variance (ANOVA) performed prior to regression analysis. The following results were obtained:

$$N = -0.381 - 0.4272x_1 + 0.018864x_2^2 + 0.5532x_3(x_2-x_1) + 0.0005928x_2x_4 + x_5 \quad (5\text{-}4)$$

N = predicted number of failures per mile (punchouts and patches)

x_1 = pavement age at time of condition survey (years)

x_2 = pavement age at future time chosen for distress prediction

x_3 = number of ailures per mile at time of condition survey

x_4 = Texas SDHPT temperature constant (Table 5-2)

x_5 = -5.840 + 1.1856x_2 for pit run gravel subbase aggregate and for other subbase aggregates

TABLE 5-2. TEXAS TEMPERATURE CONSTANT $\overline{\alpha}$

DISTRICT	$\overline{\alpha}$
1	21
2	22
3	22
4	9
5	16
6	23
7	26
8	26
9	28
10	24
11	28
12	33
13	33
14	31
15	31
16	36
17	30
18	26
19	25
20	32
21	38
22	31
23	25
24	24
25	19

Values of 0.672 and 2.436 for R^2 and the mean square error, respectively, show that the equation has an acceptable precision of prediction. The prediction relationship given in Eq. (5-4) requires the following input parameters:

(a) Condition survey data on the number of failures per mile

(b) Pavement age at the time of the survey (expressed in months)

(c) Pavement at time in the future for which the prediction is desired (months)

(d) District number needed to set the temperature constant for a particular district

(e) Subbase aggregate type 0 for limestone and 1 for silicious river

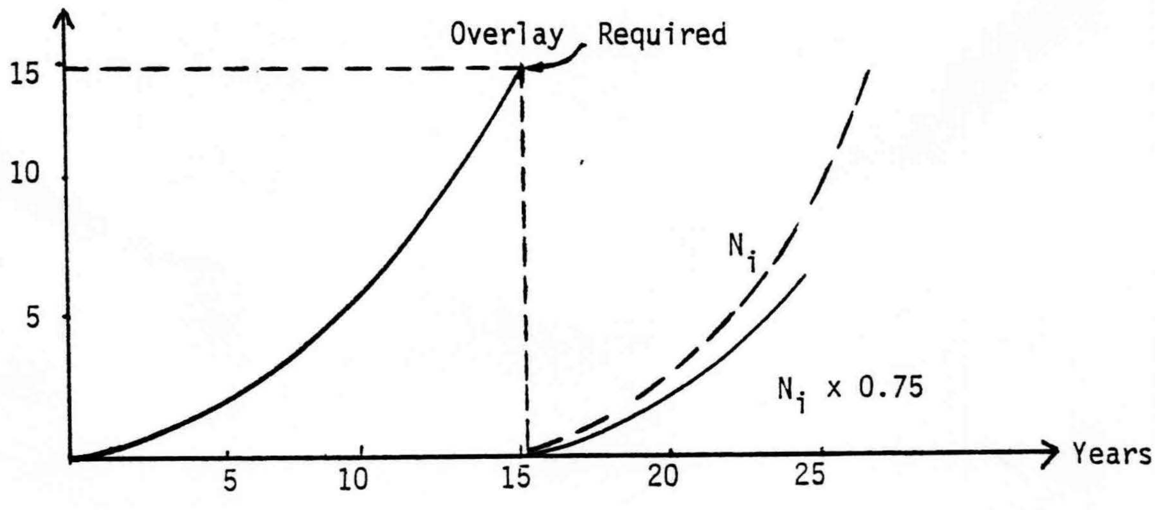


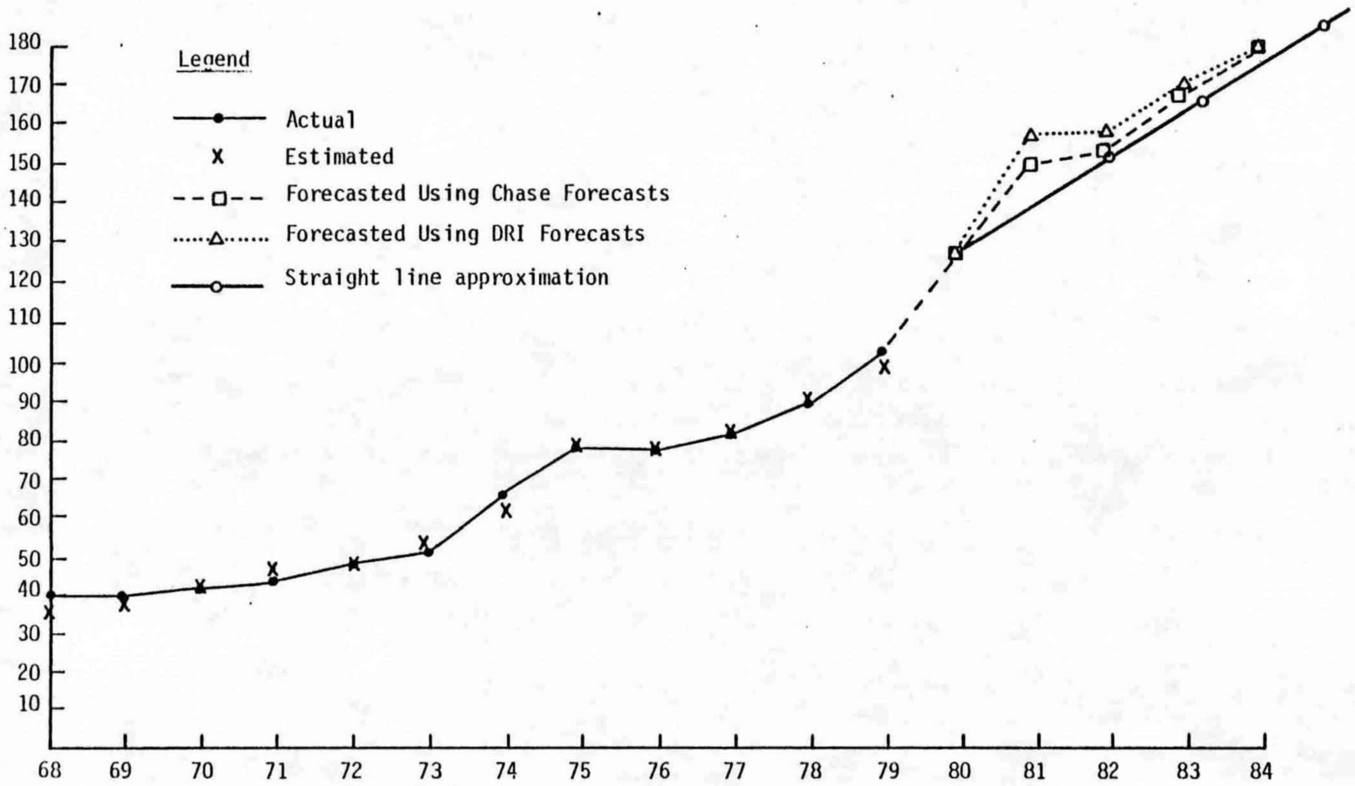
Figure 5-1. Cumulative Failures per Mile per Year

As shown in Figure 5-1, the cumulative number of failures is calculated for each year until this value approaches 15.0, at which time an overlay is needed. After the overlay, the number of failures drops to zero and starts accumulating again at a slower rate. This slower rate could be estimated to approximately 75 percent of the original rate [15].

The number of failures per mile from Eq. (5-4) was developed on the basis of two one-way traffic lanes. In order to estimate the number of failures per lane-mile per year, the lane distribution factor has to apply to the number of failures per mile. This factor ranges between 0.5 to 0.85. In the RENU program a lane distribution factor of 0.65 is used.

5.1.3 Highway Cost Index for Maintenance

The Texas Highway Cost Index has been incorporated into the projection of future maintenance costs. The Maintenance Material Cost Index from the current Forecasts of the Highway Cost Index [35] is input by the user to the program as a constant rate by approximating the projected index to a straight line. Figure 5.2 illustrates a factor of 9% as obtained from the July 1980 report [35].



Years

Figure 5-2. Maintenance Materials Cost Index by Fiscal Years (1979 = 100)

5.2 Rehabilitation Costs

The rehabilitation activity considered in RENU consists of an overlay with asphalt concrete. The rehabilitation cost is a function of the thickness of the overlay, the cost of the materials used in the construction of the overlay, and the width of the shoulders. Two different methodologies are provided to determine the thickness of the overlay. In case of flexible pavements, use is made of elastic layer theory when heavy rehabilitation is needed due to the effect of traffic loadings. In case of rigid pavements, the thickness is determined using modified AASHTO equations.

5.2.1 Flexible Pavements

In the analysis of flexible pavements, RENU allows the consideration of two possibilities. If a pavement fails because of distress, a specified thickness of overlay is applied. The overlay thickness is a user input and can vary from one type of pavement to another. A thick overlay is recommended when the distress is of the type that causes a significant reduction in the structural strength of the pavement.

5.2.1.1 Pavements that Fail Because of Distress

Experience dictates that most pavements in Texas are rehabilitated when a significant amount of distress is present. The user must input the minimum overlay thickness that is recommended for each representative pavement section.

5.2.1.2 Pavements that Fail Because of Serviceability

Elastic layer theory employing the Russian Equations [34] will be utilized to determine the overlay thickness of pavements that fail because of serviceability. The resulting overlay thickness is that which satisfies a maximum dynaflect deflection criterion when subjected to a specified load determined by the number of 18-kip ESALs to be applied during the design period.

Representative pavement sections have been coded into the program including the moduli of elasticity of the different layers. Table 5.3 shows the sections coded into RENU. The dynaflect maximum deflection allowed is based upon the design criteria shown in Table 5-4.

From the Texas performance equations for K, it is possible to express this value as a function of DMD. For the purpose of the present analysis, K will be described by the relationship

$$K = (DMD)^{1/\beta} \tag{5-5}$$

The value of β used in Eq. (5-5) can be obtained by solving this equation after K is set to a specific value which can be found from Eq.(3-15) with n=1, that is,

$$K = - W \text{ Ln } \left(\frac{P_i - P_t}{P_i - P_f}\right) \tag{5-6}$$

for given values of P_i, P_t, P_f, and W.

For a known value of β, the variations in loading (ESALs) can be linked to changes in the dynaflect deflection (DMD) utilizing Eq. (5-7):

$$DMD = -\left[W \text{ Ln } \left(\frac{P_i - P_t}{P_i - P_f}\right)\right]^{\beta} \tag{5-7}$$

TABLE 5-3. LAYER THICKNESS & ELASTICITY MODULI FOR REPRESENTATIVE SECTIONS

Pavement	Layer Thickness (in.)				Modulus of Elasticity (0-overlay)					Subgrade
	1	2	3	4	0	1	2	3	4	
Rural surface treated	.75	6.0	-	-	65,000	20,000	10,600			5,000
Rural Hot Mix (low traffic)	2.0	8.0	-	-	300,000	80,000	15,000			6,000
Rural Overlaid (low traffic)	2.0	2.0	8.0	-	325,000	130,000	90,000	16,000		6,000
Rural Hot Mix (high traffic)	4.0	12.0	-	-	305,000	100,000	16,500			6,000
Rural Overlaid (high traffic)	3.0	4.0	12.0	-	325,000	130,000	90,000	18,500		6,000
Urban surface treated	.75	8.0	-	-	65,000	20,000	12,800			5,100
Urban Hot Mix (low traffic)	2.0	8.0	6.0	-	300,000	85,000	22,000	16,400		6,000
Urban Overlaid (low traffic)	2.0	2.0	8.0	6.0	325,000	130,000	90,000	38,000	19,000	6,000
Urban Hot Mix (high traffic)	4.0	10.0	6.0	-	325,000	95,000	35,000	18,500		6,000
Urban Overlaid (high traffic)	3.0	4.0	10.0	6.0	325,000	150,000	115,000	42,000	22,000	6,000

TABLE 5-4. DYNAFLECT MAXIMUM DEFLECTION CRITERIA FOR REPRESENTATIVE SECTIONS

Pavement	DMD	Design Life 18-Kip ESALs
Rural Surface treated	1.2	20,000
Rural Hot Mix (low traffic)	.8	300,000
Rural Hot Mix (high traffic)	.7	3,000,000
Rural Overlaid Hot Mix (low traffic)	.7	260,000
Rural Overlaid Hot Mix (high traffic)	.6	2,600,000
Urban Surface treated	1.0	50,000
Urban Hot Mix (low traffic)	.7	750,000
Urban Hot Mix (high traffic)	.6	7,500,000
Urban Overlaid Hot Mix (low traffic)	.6	650,000
Urban Overlaid Hot Mix (high traffic)	.5	6,500,000

5.2.2 Rigid Pavements

The required overlay thickness for rigid pavements is determined using the modified AASHTO equations. Once this thickness is known, the cost of overlaying the traffic lanes and the shoulders can be determined. The methodology for determining the cost of the overlay and raising the shoulders up to the edge of the pavement is the same as that used in NULOAD [33].

5.2.3 Highway Cost Index for Rehabilitation

The Surfacing Cost Index from the current Forecasts of the Highway Cost Index [35] is input by the user as a constant rate by approximating the projected index to a straight line. This will account for future price increases in surfacing materials used in the placement of overlays. Figure 5-3 illustrates a factor of 11.8% as obtained from the July 1980 report [35].

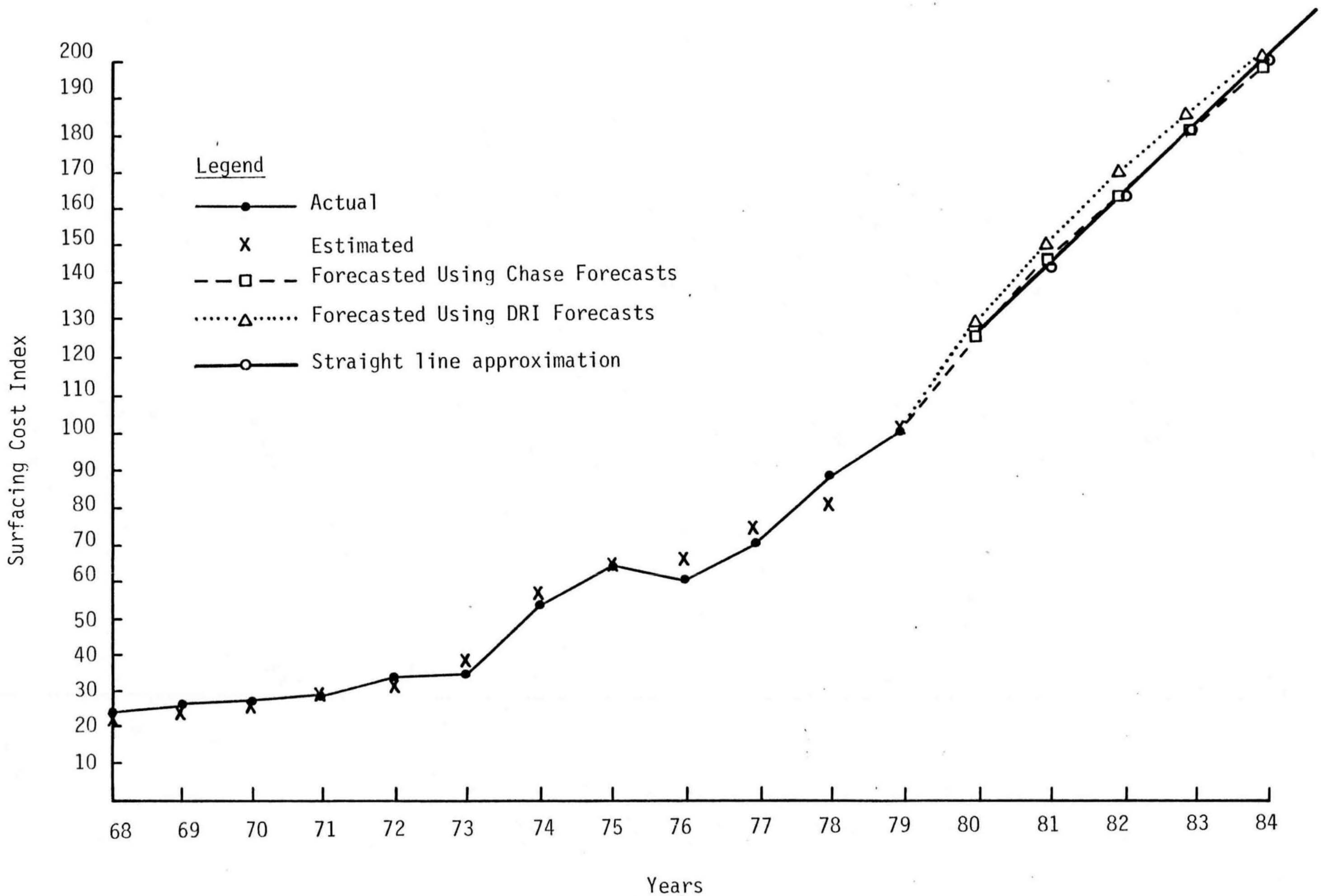


Figure 5-3. Surfacing Cost Index by Fiscal Years (1979 = 100)

Chapter 6

THE SHIFTING PROCEDURE

6.1 The SDHPT Shifting Procedure

In order to evaluate the effect of legal load limit changes on future truck weight distributions, the cumulative percentage of gross vehicle weight (GVW) is shifted, according to tendencies observed in recent years. To accomplish this shifting procedure, the user should supply the appropriate load information for each of the truck types to be considered (basically, truck types 2D, 3A, 3-S2). Although the SDHPT procedure (SSP) currently considers the shifting of the distribution of gross vehicle weight (GVW), it is more useful when related data exist, to shift the distributions corresponding to single, tandem, tridem, steering axle loads, and empty vehicle weight.

The shifting procedure is a simple relationship according to which the existing GVW upper limit is multiplied by a factor that increases linearly from 1.0 to the ratio of practical maximum GVW at present (PMGVWP) to practical maximum GVW in the future. As the GVW increases from the lower limit of the first weight interval to the value of PMGVWP, the factor is linearly increased and at the limit becomes constant and equal to PMGVWF/PMGVWP. The result is the end point of a new interval.

Thus, the shifting is done by calculating a ratio, obtained from past experience, that will give the future vehicle weight distribution for a certain truck type. Afterwards, the relation between the future GVW and the axle weights is calculated manually for each truck type, and the future axle weight distribution is obtained. The empty weight for 1976 to date was estimated by assuming the same distribution prevailing in the years 1970-1974.

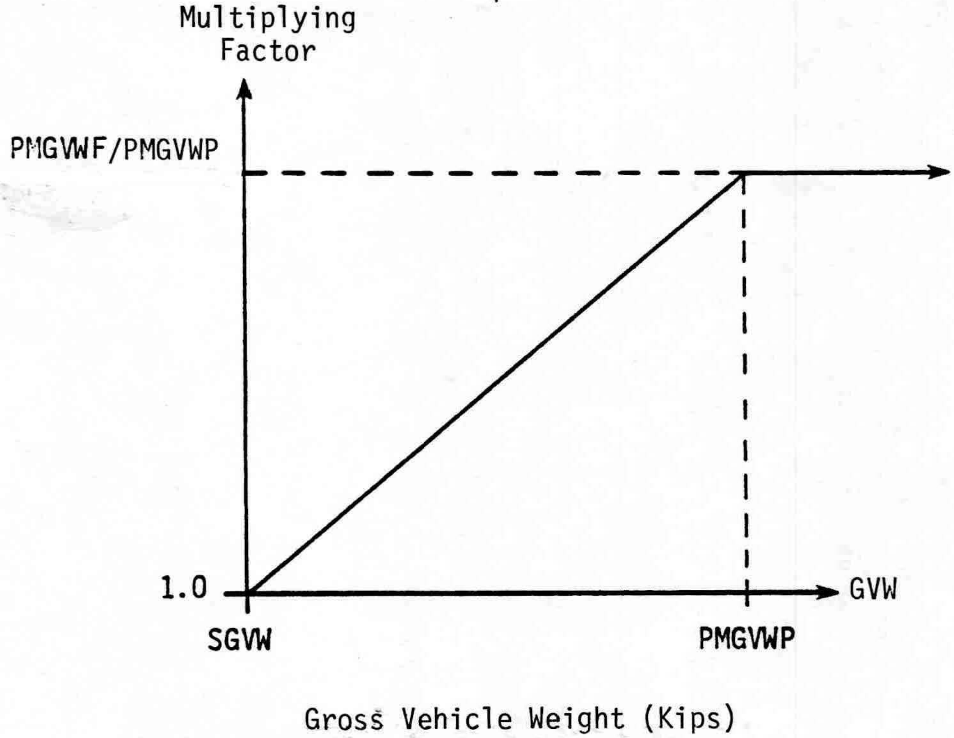
The ratio used for developing the SSP was based on a multiplying factor which is the result of an equation that implies all the different possibilities of a GVW increase for the 4 more common truck types (See Fig. 6-1 in which SGVW is smallest GVW).

The SSP was developed within the NCHRP report #141 and was incorporated into the RENU program to predict the effect of heavier trucks on pavements.

In an analysis of recent truck data, it was found that the weight constraints within the different vehicle types do not all experience a rightward shift, but that only a certain percentage shifts for each truck type. The reason being, that not all the trucks would experience an increment in weight, since some have demand constraints as well as volume constraints that make higher load capacities for them unnecessary.

In order to properly account for these constraints, the lower portion of the GVW cumulative frequency distribution will have to experience less of a shift to the right, or no shift at all. Only those vehicles operating in the upper GVW ranges would truly take advantage of the new allowable weight limits. Only those vehicles operating in the upper GVW distribution should then experience a substantial shift to the right.

Vehicles weighted empty were assumed to remain constant in both scenarios.



for $SGVW \leq GVW \leq PMGVWP$

$$\text{Multiplying Factor} = 1.0 + \frac{\frac{PMGVWF}{PMGVWP} - 1.0}{PMGVWP - SGVW} * (GVW - SGVW)$$

for $GVW \geq PMGVWP$

$$\text{Multiplying Factor} = \frac{PMGVWF}{PMGVWP}$$

Figure 6-1. Multiplying Factor Related to Gross Vehicle Weight for the NCHRP Procedure

6.2 The Modified SDHPT Shifting Procedure (MSP)

In order to modify the GVW distribution shifting procedure, it became necessary to modify only the multiplying factor to be used in the shifting procedure, using 1970-1974 data. Five different analyses were conducted, each using a constant payload, the equivalent to that hauled by 100 vehicles of a certain type operating on a particular highway class under present conditions. The procedure that best fitted the existing conditions was found to be the one that would consider only the shifting from the 50% cumulative, for truck types 2D and 3A and 33% cumulative, for truck types 3-s2 and 2-S1-2 (Fig. 6-2).

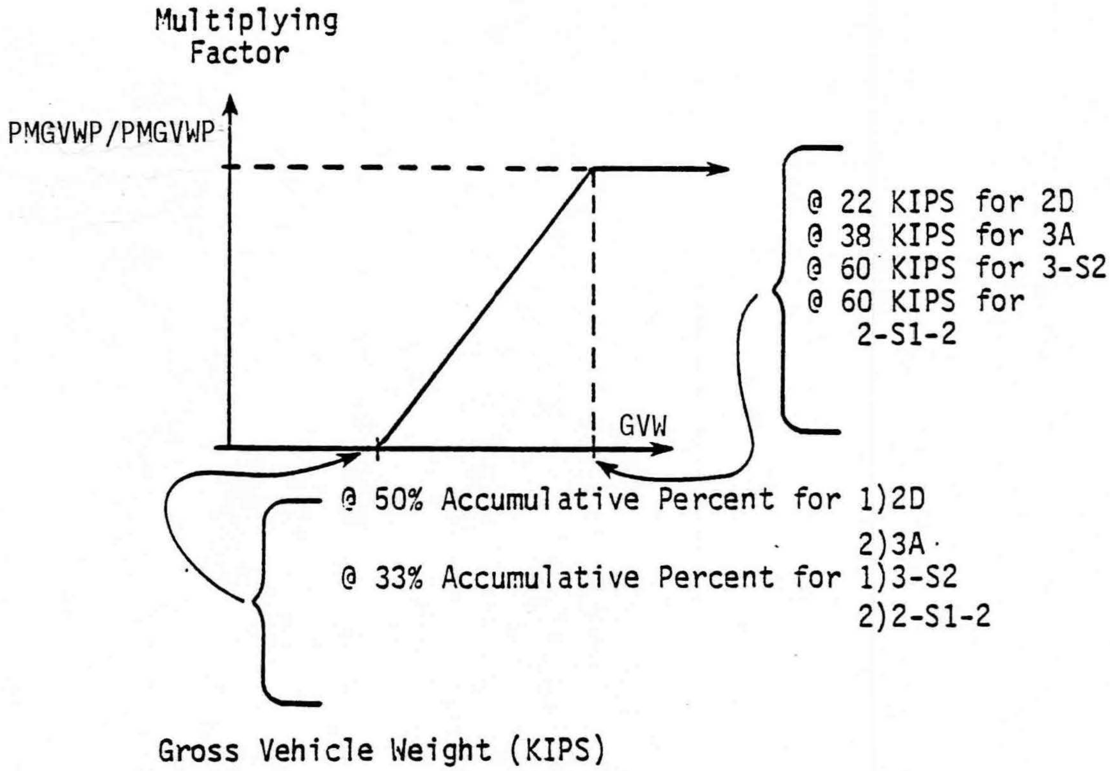
Recently, data from 1976 to 1979 was made available, making it possible to check the assumptions made previously. The following statistics were compared:

1. GVW accumulative frequency based on single year data or data combined for several years
2. GVW distribution histogram
3. Average GVW
4. % of overweight trucks
5. Axle weight accumulative frequency
6. Accumulative frequency vs. GVW for different years

The comparison was made using four common truck types and data for interstate rural highways and other main highways [45].

Some of the observations extracted from the comparison were:

1. A definite increase in GVW is observed from 1971-75 data to 1976-79 data.
2. The assumption that empty or lightly loaded vehicles will not experience the rightward shift due to demand and volume constraints is confirmed.



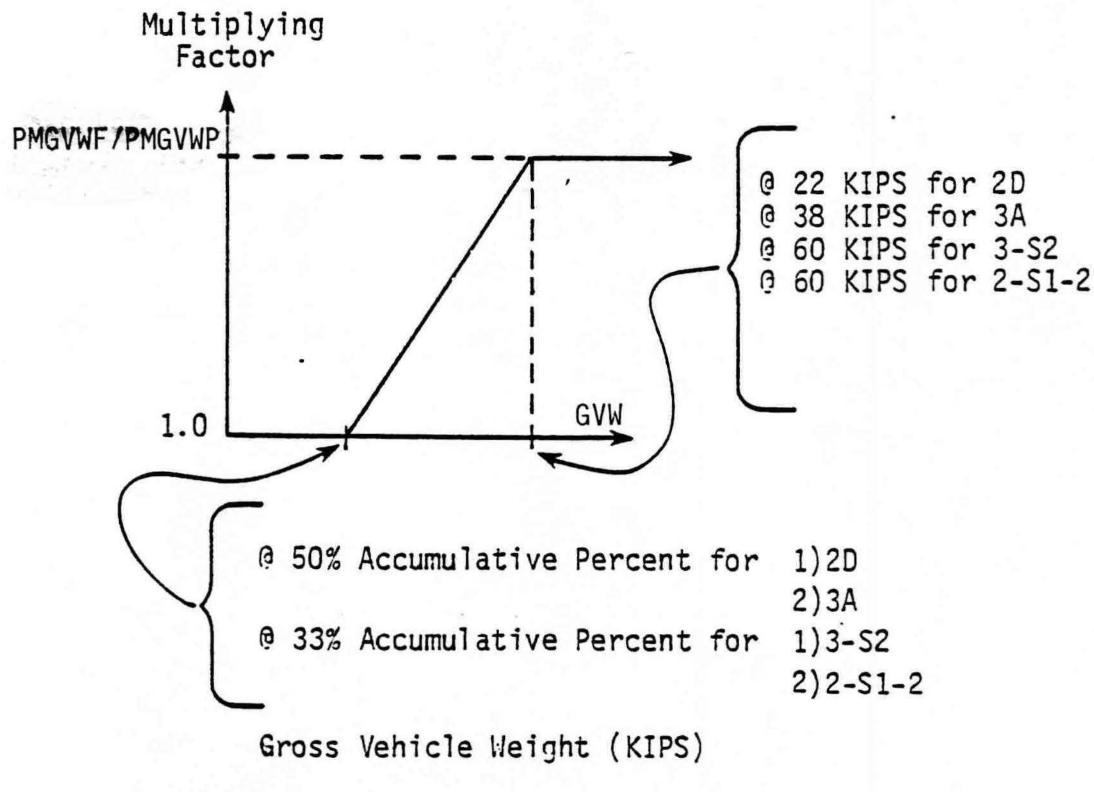


Figure 6-2. Multiplying Factor versus GVW Relationship for Modifying Data Generated under the Privious Law as Developed by Larkin,[14]

3. The axle weight data was also observed, showing change in axle weight and GVW according to the 1975 increase in limits, but no change in the distribution of steering axle weight.

4. The increase in GVW is mainly governed by the increase in axle weight [44].

As several tests have shown, it is not feasible to establish a definite percentage in which to begin the shifting for the four different truck types. As to the latest runs using 1979 data, truck types 2D and 3A experience a shifting in their GVW from 40% and 30% up to 100%, while truck types 3-S2 and 2-S1-2 experience shifting from 0% to 100% inclusive.

However, more data is needed in order to establish a definite percentage from which to begin the shifting so the user would rather input the percentage to use according to the most recent results available (Fig. 6-3).

Once the shifted GVW is obtained, the axle weight distribution is obtained manually for each truck type, according to previous results and to the existing weight limits. First, the front axle (FA) or steering axle weight is obtained, with the following equations.

Truck Type	Equation
2D	FA = 2.0 + 0.27GVW
3A	FA = 2.9 + 0.2GVW
3-S-2	FA = 6.0 + 0.05GVW
2-S1-2	FA = 7.5 + 0.03GVW

Afterwards, subtracting the FA, as each truck type has either single axles or tandem axles, the remaining weight is distributed evenly among the loading axles (Fig. 6-4).

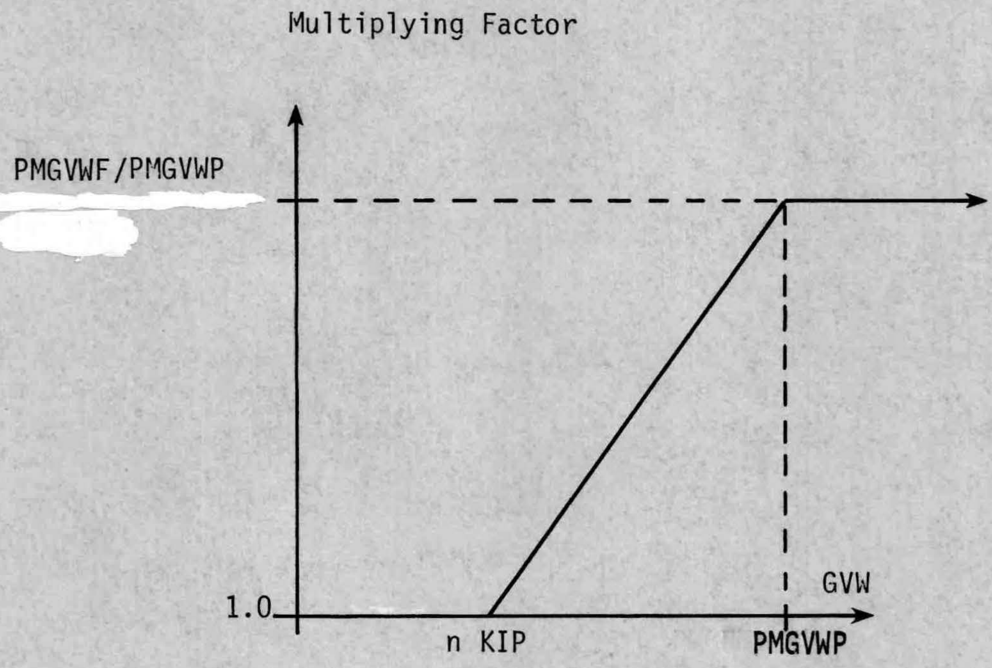
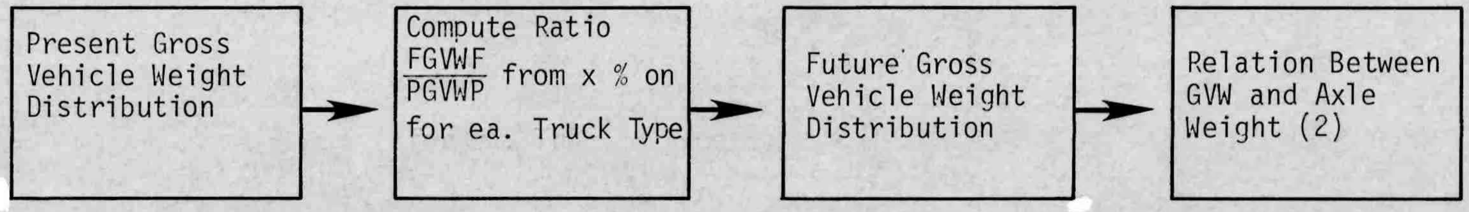


Figure 6-3. Modified SDHPT Shifting Procedure



(1) The user will decide on the %; the previous SSP computed the ratio for all trucks.

(2) The axle weight distribution is obtained manually for each type of truck.

Figure 6-4. Modified SDHPT Shifting Procedure

6.3 The Plotting Subroutine

As an auxiliary procedure that will enable the user to show the shifted results in a graphic form, the PLOTTING subroutine was added to the RENU Program. The plotting subroutine [46] permits comparison of two or more sets of data of which usually one is the unshifted cumulative frequency and the other is the shifted result. For the sake of clarity, it is advisable not to compare more than 4 sets of data, shifted and unshifted, at the same time.

The type of curve provided by the PLOTTING subroutine is of a simple form, with two coordinates, the X coordinate being the GVW (kips) or TAW (Tandem axle weight), providing up to 120 kips in the first case or 60 kips in the second case; the Y coordinate is the accumulated percent shifted. The usual graph is S-shaped, with an upper asymptote, as shown in Figure 6-5.

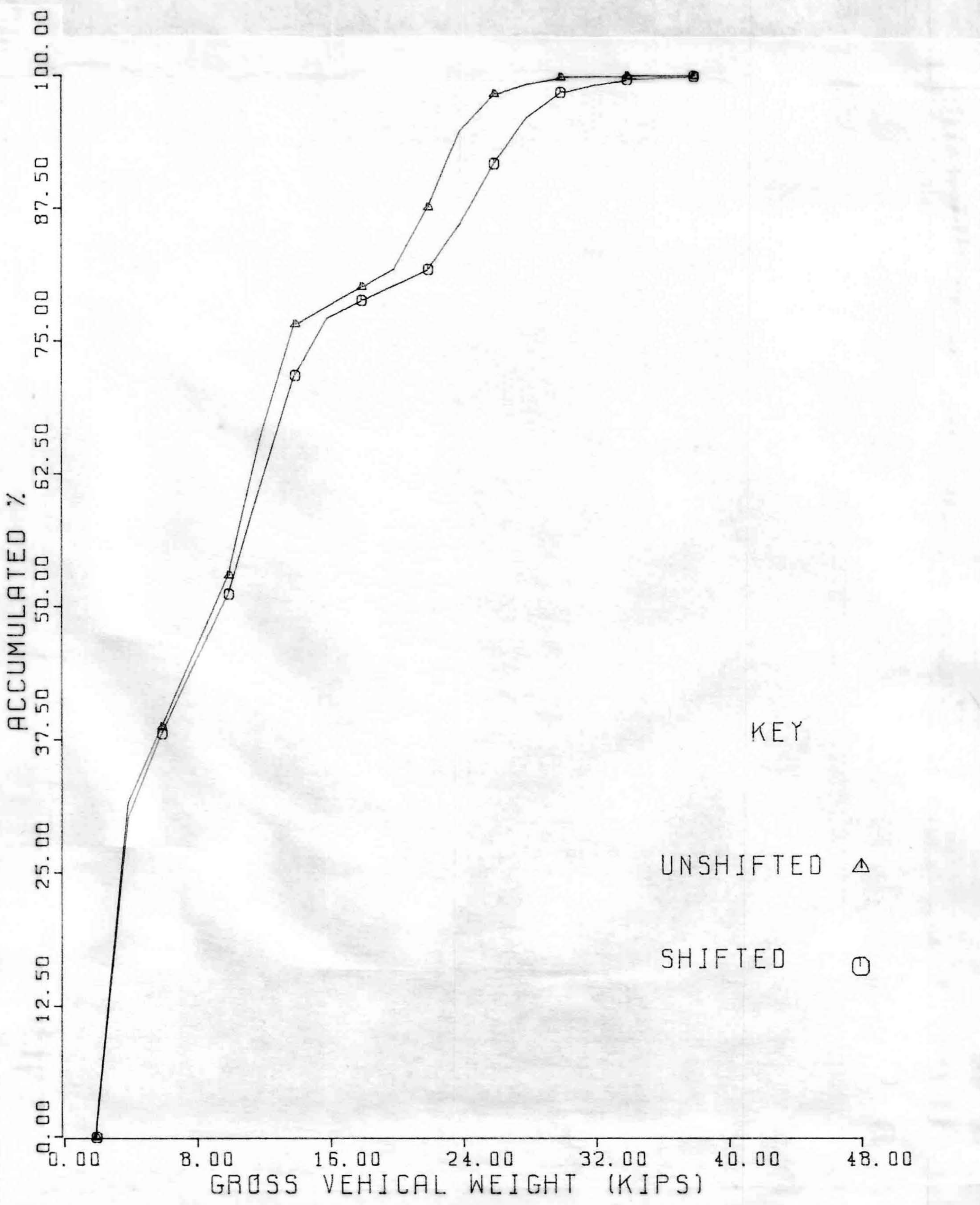


Figure 6-5. Shifting Procedure

Chapter 7

APPLICATIONS OF THE MODEL

7.1 Introduction

After developing RENU we have reached the stage at which we introduce a procedure designed to note specific strengths and general usefulness of the program. The purpose of this chapter is to identify a set of meaningful scenarios of the Texas highway system and produce rehabilitation and cost estimates by running RENU under conditions specified in each scenario.

The results from all the scenarios can be combined to assist decision making concerning the estimation of rehabilitation and maintenance funds needed in each period of a specified planning horizon. In Chapter 7 the results obtained for the scenarios will be used to assess the relative impact that factors such as the Highway Cost Index (HCI), change in the load limits and pavement performance have on funds needed.

Twelve scenarios were utilized to demonstrate RENU's response to changes in various input parameters. The flexible pavement network for Texas was the basis for the scenarios. The state was divided into two major geographical areas based upon main distress types prevailing in each area. Area 1 included District 1 and Districts 10 through 22, where pavements fail mainly because of alligator cracking. Area 2 includes Districts 2 through 9 and 23 through 25, where pavements fail mostly because of severe transversal cracking. Pavements were classified according to the following characteristics:

(a) Interstate, Farm to Market, U.S.-State

(b) rural or urban

(c) high or low traffic intensity

(d) hot mix, overlay or surface treated

The classification of Texas pavements was performed using the SDHPT Road Life and Road Inventory files.

The twelve scenarios were divided into two groups. The first group, consisting of eight scenarios, corresponds to different combinations of possible values for the HCI, load limits, and pavement performance. For each of these factors minimum and maximum levels were chosen to reflect realistic changes of interest. In these scenarios the rehabilitation needs are generated by the presence of several types of distress. In the second group, consisting of four additional scenarios, the rehabilitation needs are generated by significant worsening of riding conditions (PSI).

The following assumptions were made for the first eight scenarios:

(a) All pavements fail because of distress and thus receive a one inch overlay. Pavements in POTTS receive a 1½ inch overlay.

(b) The target value for pavements older than terminal serviceability (POTTS) is 10%.

(c) Maintenance and rehabilitation costs are the same statewide (based on costs obtained from District 17).

(d) The upper and lower values for the HCI are 12% and 2%.

(e) The upper and lower values for the legal load limits are:

Single axle : 26 - 22.4 kips

Tandem axle : 44 - 36 kips

Gross Weight : 120-80

(f) The performance factor is defined as the time between the first and second overlays (except for the lanemiles in POTTS). The upper and lower values for this factor were set to 12 and 5 years, respectively. For a planning horizon of 18 years (through the year 2000), the minimum value of the performance factor corresponds to two overlays for all pavements not in POTTS. On the other hand, the maximum value (12 years) corresponds to only a fraction of the pavement receiving two overlays.

For the pavements in the second group, it is assumed that all pavements fail because of serviceability. The performance factor is defined as a terminal serviceability index (P_t) between 2.25 and 2.75, with an asymptotic serviceability value (P_f) of 2.0. The serviceability performance models for flexible pavements contained within RENU were developed using $P_t = 2.5$ and $P_f = 2.0$. Wide variations from these values should not be considered to avoid possibly illogical results.

7.2 Description of Texas Flexible Pavement Scenarios

The twelve scenarios for the application of the RENU program covering meaningful conditions concerning the HCI, load limits and performance factors are described in Tables 7.1 and 7.2. Table 7.1 specifies the values of each factor in each scenario. Table 7.2 summarizes some of the most important input parameters common to all scenarios.

7.3 Results

The output from RENU corresponding to each highway scenario can be classified as follows:

(a) Undiscounted Maintenance Costs for the Proposed Load Limits - Summarized in Table 7.3.

(b) Undiscounted Rehabilitation Costs for the Proposed Load Limits - Table 7.4.

(c) An Economic Analysis - Table 7.5.

(d) Increase in Costs per Lane Mile Due to Increased Load Limits - Table 7.6.

TABLE 7-1. DESCRIPTION OF SCENARIOS

Factor \ Scenarios	1	2	3	4	5	6	7	8	9	10	11	12
Highway Cost Index (%)	2	12	2	12	2	12	2	12	2	2	12	12
Proposed Load Limits (kips)												
Single axle	22.4	22.4	26	26	22.4	22.4	26	26	22.4	22.4	26	26
Tandem axle	36	36	44	44	36	36	44	44	36	36	44	44
Gross weight	120	120	120	120	120	120	120	120	120	120	120	120
Performance												
Time between first and second overlay **(years)	12	12	12	12	5	5	5	5	NA	NA	NA	NA
Terminal serviceability	NA	NA	NA	NA	NA	NA	NA	NA	2.25	2.75	2.25	2.75
Minimum overlay thickness (inches)	1	1	1	1	1	1	1	1	1	1	1	1
Maximum overlay thickness (inches)	NA	NA	NA	NA	NA	NA	NA	NA	6,4,5*	6,4,5*	6,4,5*	6,4,5*

(**) for all lane miles except those in POTTS

(*) 6" Interstate

4" FM

5" US & State

TABLE 7-2. INPUT PARAMETERS COMMON TO ALL SCENARIOS

Input Parameter	Value
Analysis period	18 yrs
Annual Interest Rate	4% + HCI
Lane width	
Interstate	12 ft
FM	11 ft
US - State	12 ft
Percent paved shoulders	
Interstate	95%
FM	10%
US - State	10%
Cost of HMAC for overlay	$94.73/cy
Cost of turf material for shoulder	$.06/sy/in
Unit cost of bituminous patching	$3.04/sy
Unit cost of bituminous crack sealing	$.25/linear ft
Unit cost of bituminous base & surface repair	$59.10/cy
Maintenance cost per yr per lane mile for POTTS	
Interstate	$1800/lane mi/yr
FM	$ 750/lane mi/yr
US - State	$ 750/lane mi/yr
Present load limits (kips)	
Single axle	20 kips
Tandem axle	34 kips
Gross weight	80 kips
Annual growth rate for ESALS	2%
Total lane miles	150,615

TABLE 7-3. UNDISCOUNTED MAINTENANCE COSTS FOR PROPOSED LOAD LIMITS (ALL COSTS IN MILLIONS OF DOLLARS)

Scenarios

Year	1	2	3	4	5	6	7	8	9	10	11	12
1	82.656	90.759	96 830	106.323	84.389	92.662	94.267	103.508	88.752	88.316	99.828	101.130
2	83.543	100.727	93.832	113.133	84.267	101.600	91.968	110.885	79.599	79.286	90.005	89.179
3	77.899	103.130	78.857	104.399	84.449	111.801	84.002	111.210	73.737	72.928	84.459	82.747
4	70.260	102.137	48.356	70.294	76.406	111.070	63.768	92.699	59.974	61.444	67.908	66.213
5	58.918	94.046	33.158	52.927	65.789	105.012	38.019	60.686	51.746	51.885	47.135	39.859
6	44.774	78.475	26.633	46.680	43.471	76.191	21.251	37.246	35.535	36.006	37.903	33.676
7	38.179	73.477	21.158	40.719	30.046	57.825	12.758	24.553	26.040	28.174	35.625	33.915
8	32.638	68.971	16.053	33.923	20.426	43.165	10.990	23.223	22.185	23.560	43.500	43.468
9	33.409	77.522	18.528	42.991	17.614	40.871	13.719	31.834	24.085	24.154	61.115	62.581
10	32.124	81.848	25.123	64.012	19.117	48.707	20.481	52.182	30.159	30.010	91.335	94.450
11	34.558	96.683	33.525	93.791	23.071	64.545	30.435	85.148	38.871	38.247	131.519	136.637
12	36.585	112.387	39.972	122.792	28.678	88.097	40.252	123.654	46.868	45.941	171.103	177.525
13	36.927	124.561	42.552	143.533	34.097	115.015	48.093	162.226	51.931	50.930	197.631	203.145
14	36.115	133.763	42.500	157.414	39.270	145.449	52.705	195.212	53.636	52.791	204.570	206.157
15	34.312	139.548	41.626	169.292	43.020	174.959	53.351	217.000	52.005	41.545	193.655	189.552
16	31.853	142.244	41.323	184.534	45.415	202.811	50.394	225.043	47.233	47.346	173.469	164.524
17	30.120	147.695	40.780	199.966	45.178	221.531	44.867	220.005	40.515	41.236	153.385	142.463
18	29.738	160.118	38.416	206.843	42.973	231.376	38.302	206.227	33.707	34.824	139.194	129.346

TABLE 7-4. UNDISCOUNTED REHABILITATION COSTS FOR PROPOSED LOAD LIMITS (ALL COSTS IN MILLIONS OF DOLLAR)

	Scenarios											
Years	1	2	3	4	5	6	7	8	9	10	11	12
1	436.553	479.353	721.899	792.674	439.598	482.696	616.490	676.931	644.581	955.850	2775.638	1675.498
2	578.952	678.038	568.774	685.766	293.372	353.715	565.479	681.793	580.915	877.701	3077.114	1592.100
3	286.492	379.286	771.684	1021.629	470.263	622.579	623.548	825.512	1082.015	1063.966	3576.678	1592.283
4	278.537	404.907	405.136	588.943	325.712	473.484	831.786	1209.160	596.366	1476.087	3431.303	2244.425
5	578.757	828.036	309.871	494.618	904.260	1443.389	731.216	1167.173	862.500	1208.584	2221.088	1285.845
6	224.143	392.856	442.376	775.353	517.017	906.177	514.117	1006,258	872,832	946.559	1355.157	803.954
7	374.247	720.252	555.818	1069.691	568.115	1093.357	349.420	672.472	397.011	864,580	502,060	204,489
8	152.438	322.135	340.694	719.960	411.581	869.760	322.989	682.545	278.048	749.417	237.256	44.934
9	69.452	1089.314	257.988	584.711	355.920	825.875	182.002	422.317	152.922	217.612	44.312	8.076
10	315.454	803.742	180.204	459.140	344.997	879.013	167.108	425.773	25.199	102.889	27.667	5.017
11	284.874	796.986	116.493	325.909	185.721	519.588	122.173	341.802	10.443	50.345	6.164	1.481
12	364.438	1119.540	65.005	199.691	148.256	455.438	87.823	269.789	6.215	21.037	3.672	.512
13	169.438	571.539	—	—	150.001	505.975	67.504	227.702	1.083	7.883	1.203	.144
14	213.834	792.006	—	—	102.054	377.991	33.520	124.153	.513	2.372	.822	.116
15	103.619	421.415	—	—	94.446	384.110	37.493	152.484	.296	1.435	.219	—
16	79.737	356.088	—	—	73.745	329.323	14.363	64.142	.199	.670	.192	—
17	75.133	368.413	—	—	53.014	259.953	11.861	58.162	.050	.379	—	—
18	36.619	197.167	—	—	36.069	194.206	3.174	17.091	.025	.164	—	—

TABLE 7.5. ECONOMIC ANALYSIS (ALL COSTS IN MILLIONS OF DOLLARS)

Proposed less Present	Scenarios 1	2	3	4	5	6	7	8	9	10	11	12
Present Value	298.266	288.61	708.799	538.534	251.971	208.107	585.337	434.085	1081.056	524.703	4427.027	2633.939
Uniform Annual Equivalent	27.548	49.609	65.462	92.565	23.271	35.769	54.061	74.611	99.845	48.46	760.939	452.735
Total Undiscounted cost	223.486	-751.613	-54.673	-3749.117	51.108	-957.389	-99.007	-2858.902	1287.221	230.584	2628.425	2080.473

TABLE 7.6. INCREASE IN COST/LANEMILE DUE TO CHANGE IN LOAD LIMITS (COST IN DOLLARS)

Scenario 1	2	3	4	5	6	7	8	9	10	11	12
182.90	329.38	434.63	614.58	154.57	237.49	358.94	495.38	662.90	321.75	5052.21	300.59

Chapter 8

SENSITIVITY ANALYSIS

8.1 Sensitivity Analysis for Flexible Pavements

The purpose of this chapter is to present a sensitivity analysis that was performed utilizing the first eight scenarios for flexible pavements. By employing a statistically designed experiment a number of factors can be studied to gain insight of their simultaneous effects on the response under investigation.

The factors studied in this analysis were the Highway Cost Index, the proposed load limits and the pavement performance to ascertain their effects or influence on the following six response variables:

(a) The change in the uniform annual maintenance, rehabilitation and total costs, of the present and proposed load limits for an 18 year analysis period.

(b) The change in the uniform annual maintenance, rehabilitation and total costs of the present and proposed load limits for a 9 year analysis period.

These costs do not include salvage value computations.

To explore such situations completely we cannot vary one factor at a time, we must rather consider all combinations of the factors. This plan is called a factorial design. This approach allows for the determination of main and interactive effects. A main effect may be defined as the change in response, say cost, produced by a change in the level of the factor. An interaction between two factors denotes that a change in response between levels of one factor is not the same for all levels of the other factor.

For the three variables in this analysis, a 2^3 design (the eight scenarios) covers all possible combinations of the testing conditions. Thus, six factorial designs were utilized, one for each of the response variables.

TABLE 8-1. LEVELS FOR EACH FACTOR

Variables	Low Level	High Level
HCI	2%	12%
Proposed Load Limits	22.4-36-120 kips	26-44-120 kips
Performance	12 years	5 years

A computerized package available for IBM computers, the Statistical Analysis System (SAS), was used to perform the calculations of the analysis.

8.2 Sensitivity Analysis

To estimate main effects and interactions effects, the following two formulas were utilized:

For main effects

$$E[X_i] = \frac{1}{2^{n-1}} \sum_k C_{ik} Y_k \qquad (8\text{-}1)$$

where

C_{ik} = +1 or -1, and

$\sum_k C_{ik} = 0$ for all i

For interactions:

$$E[X_i X_j] = \frac{1}{2^{n-1}} \sum_k C_{ijk} Y_k \qquad (8\text{-}2)$$

where

C_{ijk} = +1 or -1, and

$\sum_k C_{ijk}$ = 0 for all i, j

Tables 8-3 through 8-8 produced by SAS show the significant factors and their corresponding effects for each of the response variables.

In these tables x_1, x_2, x_3, are HCI, load limits and performance, respectively. Table 8-2 shows the values of the response variables y_1, y_2, y_3, y_4, y_5, y_6 on page 78.

8.3 Discussion of Results

The effect of the load limits was the most predominant among all the response variables tested, proving significant in every test.

For the eighteen year planning horizon all of the factors proved significant including an interaction between x_1 and x_2 for the change in rehabilitation and total uniform annual costs. In the case of the shorter planning period 9 years only the proposed load limit proved significant.

Table 8-9 summarizes the significant factors for each response variable plus their effects.

8.4 Sensitivity Analysis for Rigid Pavements

A separate sensitivity study was made concerning the new rigid pavement features included in the RENU program. Three new variables were selected for this sensitivity analysis. They were:

(1) modulus of elasticity of concrete,

(2) terminal level of PSI, and

(3) number of failures per mile.

Two levels of each variable were chosen, and a 2^3, or 8, observation factorial was performed. The dependent variable being considered was the Net Present Worth Delta Cost. This variable represents the change in the total overall cost produced when changing from the present to proposed axle load limits.

Table 8.10 indicates the values selected for the two levels of each variable, and the results calculated for Delta Cost by RENU. Figure 8.1 shows an illustration of how the Delta Cost changed as a function of the levels of the three independent variables. Increasing the concrete modulus, terminal PSI, and number of failures all had the effect of reducing the Delta Cost. The variable with the most sensitivity of these three was the failure per mile with the terminal PSI being somewhat less sensitive. Very little effect was noticed by the change in concrete modulus. Since the slopes of the lines in Figure 8.1 seem to remain constant, there is no indication of any change in Delta Costs caused by the interactive effects of any two variables.

TABLE 8-2. VALUES OF RESPONSE VARIABLES FOR ANALYSIS OF VARIANCE

S T A T I S T I C A L   A N A L Y S I S   S Y S T E M 1

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OBS	X1	X2	X3	Y1	Y2	Y3	Y4	Y5	Y6
1	-1	-1	-1	0.441	26.042	26.483	0.748	48.265	49.013
2	1	-1	-1	0.782	48.506	49.288	1.037	72.514	73.551
3	-1	1	-1	-3.263	51.264	48.001	-7.571	176.993	169.422
4	1	1	-1	-5.709	92.366	86.657	-10.968	264.582	253.614
5	-1	-1	1	0.321	18.429	18.750	-0.591	59.274	58.678
6	1	-1	1	2.802	33.458	36.260	-4.005	88.577	84.572
7	-1	1	1	-1.798	40.587	38.789	-9.998	139.475	129.477
8	1	1	1	-1.144	72.363	71.219	-15.206	206.897	191.691

TABLE 8-3. RESULTS OF THE ANALYSIS OF VARIANCE FOR THE CHANGE IN THE UNIFORM ANNUAL MAINTENANCE COSTS OF PRESENT AND PROPOSED LOAD LIMITS FOR AN 18-YEAR ANALYSIS PERIOD.

S T A T I S T I C A L   A N A L Y S I S   S Y S T E M   2

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GENERAL LINEAR MODELS PROCEDURE

DEPENDENT VARIABLE: Y1

SOURCE	DF	SUM OF SQUARES	MEAN SQUARE	F VALUE	PR > F	R-SQUARE	C.V.
MODEL	6	49.26711200	8.21118533	71.28	0.0904	0.997667	35.8785
ERROR	1	0.11520000	0.11520000		STD DEV		Y1 MEAN
CORRECTED TOTAL	7	49.38231200			0.33941125		-0.94600000

SOURCE	DF	TYPE I SS	F VALUE	PR > F	DF	TYPE IV SS	F VALUE	PR > F
X1	1	0.13261250	1.15	0.4776	1	0.13261250	1.15	0.4776
X2	1	33.04845000	286.88	0.0375	1	33.04845000	286.88	0.0375
X3	1	7.86061250	68.23	0.0767	1	7.86061250	68.23	0.0767
X1*X2	1	2.66112450	23.10	0.1306	1	2.66112450	23.10	0.1306
X1*X3	1	3.43220000	29.79	0.1154	1	3.43220000	29.79	0.1154
X2*X3	1	2.13211250	18.51	0.1454	1	2.13211250	18.51	0.1454

PARAMETER	ESTIMATE	T FOR H0: PARAMETER=0	PR > |T|	STD ERROR OF ESTIMATE
INTERCEPT	-0.94600000	-7.88	0.0803	0.12000000
X1	0.12875000	1.07	0.4776	0.12000000
X2	-2.03250000	-16.94	0.0375	0.12000000
X3	0.99125000	8.26	0.0767	0.12000000
X1*X2	-0.57675000	-4.81	0.1306	0.12000000
X1*X3	0.65500000	5.46	0.1154	0.12000000
X2*X3	0.51625000	4.30	0.1454	0.12000000

TABLE 8-4. RESULTS OF THE ANALYSIS OF VARIANCE FOR THE CHANGE IN THE UNIFORM ANNUAL REHABILITATION COSTS OF PRESENT AND PROPOSED LOAD LIMITS FOR AN 18-YEAR ANALYSIS PERIOD.

S T A T I S T I C A L   A N A L Y S I S   S Y S T E M   3

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GENERAL LINEAR MODELS PROCEDURE

DEPENDENT VARIABLE: Y2

SOURCE	DF	SUM OF SQUARES	MEAN SQUARE	F VALUE	PR > F	R-SQUARE	C.V.
MODEL	6	4195.25933175	699.20988862	1564.28	0.0194	0.999893	1.3964
ERROR	1	0.44698513	0.44698513		STD DEV		Y2 MEAN
CORRECTED TOTAL	7	4195.70631688			0.66856946		47.87687500

SOURCE	DF	TYPE I SS	F VALUE	PR > F	DF	TYPE IV SS	F VALUE	PR > F
X1	1	1522.71970512	3406.65	0.0109	1	1522.71970512	3406.65	0.0109
X2	1	2117.21512813	4736.66	0.0092	1	2117.21512812	4736.66	0.0092
X3	1	355.65778512	795.68	0.0226	1	355.65778512	795.68	0.0226
X1*X2	1	156.51227813	350.15	0.0340	1	156.51227813	350.15	0.0340
X1*X3	1	35.11639012	78.56	0.0715	1	35.11639012	78.56	0.0715
X2*X3	1	8.03804513	17.98	0.1474	1	8.03804513	17.98	0.1474

PARAMETER	ESTIMATE	T FOR H0: PARAMETER=0	PR > |T|	STD ERROR OF ESTIMATE
INTERCEPT	47.87687500	202.55	0.0031	0.23637500
X1	13.79637500	58.37	0.0109	0.23637500
X2	16.26812500	68.82	0.0092	0.23637500
X3	-6.66762500	-28.21	0.0226	0.23637500
X1*X2	4.42312500	18.71	0.0340	0.23637500
X1*X3	-2.09512500	-8.86	0.0715	0.23637500
X2*X3	-1.00237500	-4.24	0.1474	0.23637500

TABLE 8-5. RESULTS OF THE ANALYSIS OF VARIANCE FOR THE CHANGE IN THE UNIFORM ANNUAL TOTAL COSTS OF PRESENT AND PROPOSED LOAD LIMITS FOR AN 18-YEAR ANALYSIS PERIOD.

GENERAL LINEAR MODELS PROCEDURE

DEPENDENT VARIABLE: Y3

SOURCE	DF	SUM OF SQUARES	MEAN SQUARE	F VALUE	PR > F	R-SQUARE	C.V.
MODEL	6	3567.10589375	594.51764896	5487.26	0.0103	0.999970	0.7014
ERROR	1	0.10834513	0.10834513		STD DEV		Y3 MEAN
CORRECTED TOTAL	7	3567.21423888			0.32915821		46.93087500

SOURCE	DF	TYPE I SS	F VALUE	PR > F	DF	TYPE IV SS	F VALUE	PR > F
X1	1	1551.27285012	14317.88	0.0053	1	1551.27285012	14317.88	0.0053
X2	1	1621.22415312	14963.52	0.0052	1	1621.22415312	14963.52	0.0052
X3	1	257.76986512	2379.16	0.0130	1	257.76986512	2379.16	0.0130
X1*X2	1	118.35680512	1092.41	0.0193	1	118.35680513	1092.41	0.0193
X1*X3	1	16.59168012	153.14	0.0513	1	16.59168012	153.14	0.0513
X2*X3	1	1.89054013	17.45	0.1496	1	1.89054013	17.45	0.1496

PARAMETER	ESTIMATE	T FOR H0: PARAMETER=0	PR > |T|	STD ERROR OF ESTIMATE
INTERCEPT	46.93087500	403.27	0.0016	0.11637500
X1	13.92512500	119.66	0.0053	0.11637500
X2	14.23562500	122.33	0.0052	0.11637500
X3	-5.67637500	-48.78	0.0130	0.11637500
X1*X2	3.84637500	33.05	0.0193	0.11637500
X1*X3	-1.44012500	-12.37	0.0513	0.11637500
X2*X3	-0.48612500	-4.18	0.1496	0.11637500

TABLE 8-6. RESULTS OF THE ANALYSIS OF VARIANCE FOR THE CNAHGE IN THE UNIFORM ANNUAL MAINTENANCE COSTS OF PRESENT AND PROPOSED LOAD LIMITS FOR A 9-YEAR ANLAYSIS PERIOD.

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GENERAL LINEAR MODELS PROCEDURE

DEPENDENT VARIABLE: Y4

SOURCE	DF	SUM OF SQUARES	MEAN SQUARE	F VALUE	PR > F	R-SQUARE	C.V.
MODEL	6	255.46686150	42.57781025	95.15	0.0783	0.998252	11.4950
ERROR	1	0.44745800	0.44745800		STD DEV		Y4 MEAN
CORRECTED TOTAL	7	255.91431950			0.66892302		-5.81925000

SOURCE	DF	TYPE I SS	F VALUE	PR > F	DF	TYPE IV SS	F VALUE	PR > F
X1	1	17.19911250	38.44	0.1018	1	17.19911250	38.44	0.1018
X2	1	209.42857800	468.04	0.0294	1	209.42857800	468.04	0.0294
X3	1	21.27476450	47.55	0.0917	1	21.27476450	47.55	0.0917
X1*X2	1	3.75380000	8.39	0.2116	1	3.75380000	8.39	0.2116
X1*X3	1	3.80052450	8.49	0.2104	1	3.80052450	8.49	0.2104
X2*X3	1	0.01008200	0.02	0.9051	1	0.01008200	0.02	0.9051

PARAMETER	ESTIMATE	T FOR H0: PARAMETER=0	PR > |T|	STD ERROR OF ESTIMATE
INTERCEPT	-5.81925000	-24.61	0.0259	0.23650000
X1	-1.46625000	-6.20	0.1018	0.23650000
X2	-5.11650000	-21.63	0.0294	0.23650000
X3	-1.63075000	-6.90	0.0917	0.23650000
X1*X2	-0.68500000	-2.90	0.2116	0.23650000
X1*X3	-0.68925000	-2.91	0.2104	0.23650000
X2*X3	-0.03550000	-0.15	0.9051	0.23650000

TABLE 8-7. RESULTS OF THE ANLAYSIS OF VARIANCE FOR THE CHANGE IN THE UNIFORM ANNUAL REHABILITATION COSTS OF THE PRESENT AND PROPOSED LOAD LIMITS FOR A 9-YEAR ANALYSIS PERIOD.

S T A T I S T I C A L   A N A L Y S I S   S Y S T E M   6

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GENERAL LINEAR MODELS PROCEDURE

DEPENDENT VARIABLE: Y5

SOURCE	DF	SUM OF SQUARES	MEAN SQUARE	F VALUE	PR > F	R-SQUARE	C.V.
MODEL	6	42913.00146175	7152.16691029	89.95	0.0805	0.998151	6.7516
ERROR	1	79.51235512	79.51235512		STD DEV		Y5 MEAN
CORRECTED TOTAL	7	42992.51381687			8.91697006		132.07212500

SOURCE	DF	TYPE I SS	F VALUE	PR > F	DF	TYPE IV SS	F VALUE	PR > F
X1	1	5437.31562112	68.38	0.0766	1	5437.31562112	58.38	0.0766
X2	1	33711.26831112	423.98	0.0309	1	33711.26831112	423.98	0.0309
X3	1	580.22914512	7.30	0.2257	1	580.22914512	7.30	0.2257
X1*X2	1	1286.74108512	16.18	0.1551	1	1286.74108512	16.18	0.1551
X1*X3	1	28.55034612	0.36	0.6563	1	28.55034612	0.36	0.6563
X2*X3	1	1868.89695312	23.50	0.1295	1	1868.89695312	23.50	0.1295

PARAMETER	ESTIMATE	T FOR H0: PARAMETER=0	PR > |T|	STD ERROR OF ESTIMATE
INTERCEPT	132.07212500	41.89	0.0152	3.15262500
X1	26.07037500	8.27	0.0766	3.15262500
X2	64.91462500	20.59	0.0309	3.15262500
X3	-8.51637500	-2.70	0.2257	3.15262500
X1*X2	12.68237500	4.02	0.1551	3.15262500
X1*X3	-1.88912500	-0.60	0.6563	3.15262500
X2*X3	-15.28437500	-4.85	0.1295	3.15262500

TABLE 8-8. RESULTS OF THE ANALYSIS OF VARIANCE FOR THE CHANGE IN THE UNIFORM ANNUAL TOTAL COSTS OF THE PRESENT AND PROPOSED LOAD LIMITS FOR A 9-YEAR ANALYSIS PERIOD.

S T A T I S T I C A L   A N A L Y S I S   S Y S T E M   7

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GENERAL LINEAR MODELS PROCEDURE

DEPENDENT VARIABLE: Y6

SOURCE	DF	SUM OF SQUARES	MEAN SQUARE	F VALUE	PR > F	R-SQUARE	C.V.
MODEL	6	37356.05824300	6226.00970717	91.48	0.0799	0.998181	6.5344
ERROR	1	68.05944450	68.05944450		STD DEV		Y6 MEAN
CORRECTED TOTAL	7	37424.11768750			8.24981482		125.2522[illegible]000

SOURCE	DF	TYPE I SS	F VALUE	PR > F	DF	TYPE IV SS	F VALUE	PR > F
X1	1	4843.14978050	71.16	0.0751	1	4843.14978050	71.16	0.0751
X2	1	28607.12401250	420.33	0.0310	1	28607.12401250	420.33	0.0310
X3	1	823.81464050	12.10	0.1782	1	823.81464050	12.10	0.1782
X1*X2	1	1151.37608450	16.92	0.1518	1	1151.37608450	16.92	0.1518
X1*X3	1	53.15836050	0.78	0.5392	1	53.15836050	0.78	0.5392
X2*X3	1	1877.43536450	27.59	0.1198	1	1877.43536450	27.59	0.1198

PARAMETER	ESTIMATE	T FOR H0: PARAMETER=0	PR > |T|	STD ERROR OF ESTIMATE
INTERCEPT	126.25225000	43.29	0.0147	2.91675000
X1	24.60475000	8.44	0.0751	2.91675000
X2	59.79875000	20.50	0.0310	2.91675000
X3	-10.14775000	-3.48	0.1782	2.91675000
X1*X2	11.99675000	4.11	0.1518	2.91675000
X1*X3	-2.57775000	-0.88	0.5392	2.91675000
X2*X3	-15.31925000	-5.25	0.1198	2.91675000

TABLE 8-9. SUMMARY OF SIGNIFICANT FACTORS FOR EACH RESPONSE VARIABLE

Response / Factor	y_1	y_2	y_3	y_4	y_5	y_6
X_1	—	27.59275	27.85025	—	—	—
X_2	-4.065	32.53625	28.47125	-10.233	129.82925	119.5975
X_3	—	-13.33525	-11.35275	—	—	—
X_1X_2	—	8.84625	7.69275	—	—	—

As an example of the interpretation of Table 8-9, for an 18 year analysis period the mean change or reduction in the maintenance annual uniform costs from the present to proposed load limits is 4.065 million dollars (response y_1). This can be rationalized by the effect of an increased overlay activity (y_2 or y_5) thus reducing the age of the existing pavements which signifies reduced maintenance costs.

In the cases of y_2, y_3 an interacting effect appears to exist between the HCI and the proposed load limit. A graphical illustration of interaction effects is given in Figures 8-1 and 8-2. Parallel or nearly parallel lines denote that there is not interaction present, while lines sloping away from each other signify a significant interaction effect, as seen for the interaction of X_1 and y_2 in the first set of graphs.

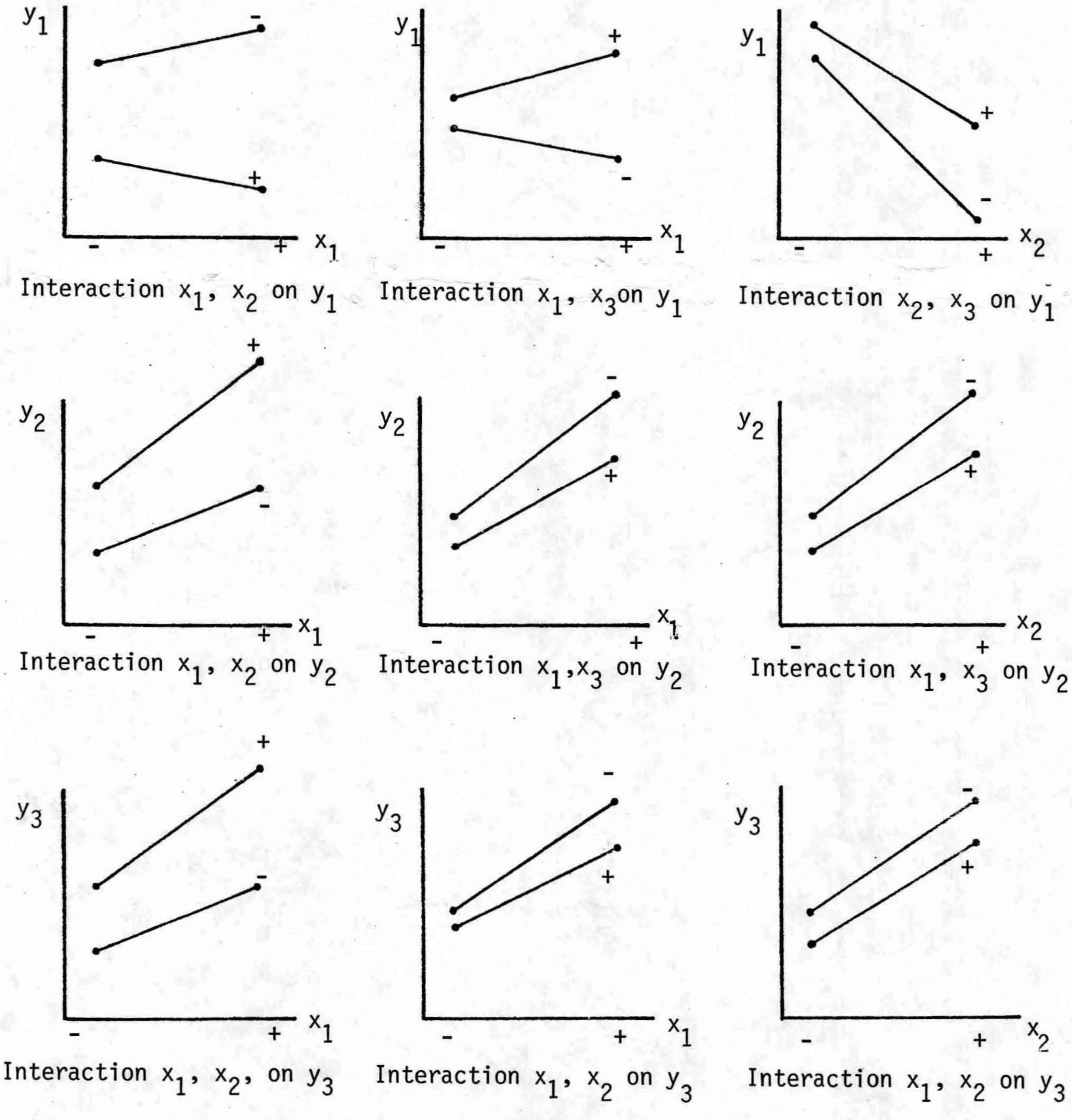


Figure 8-1. Two Factor Interaction in a Factorial Experiment for Responses y_1, y_2, and y_3

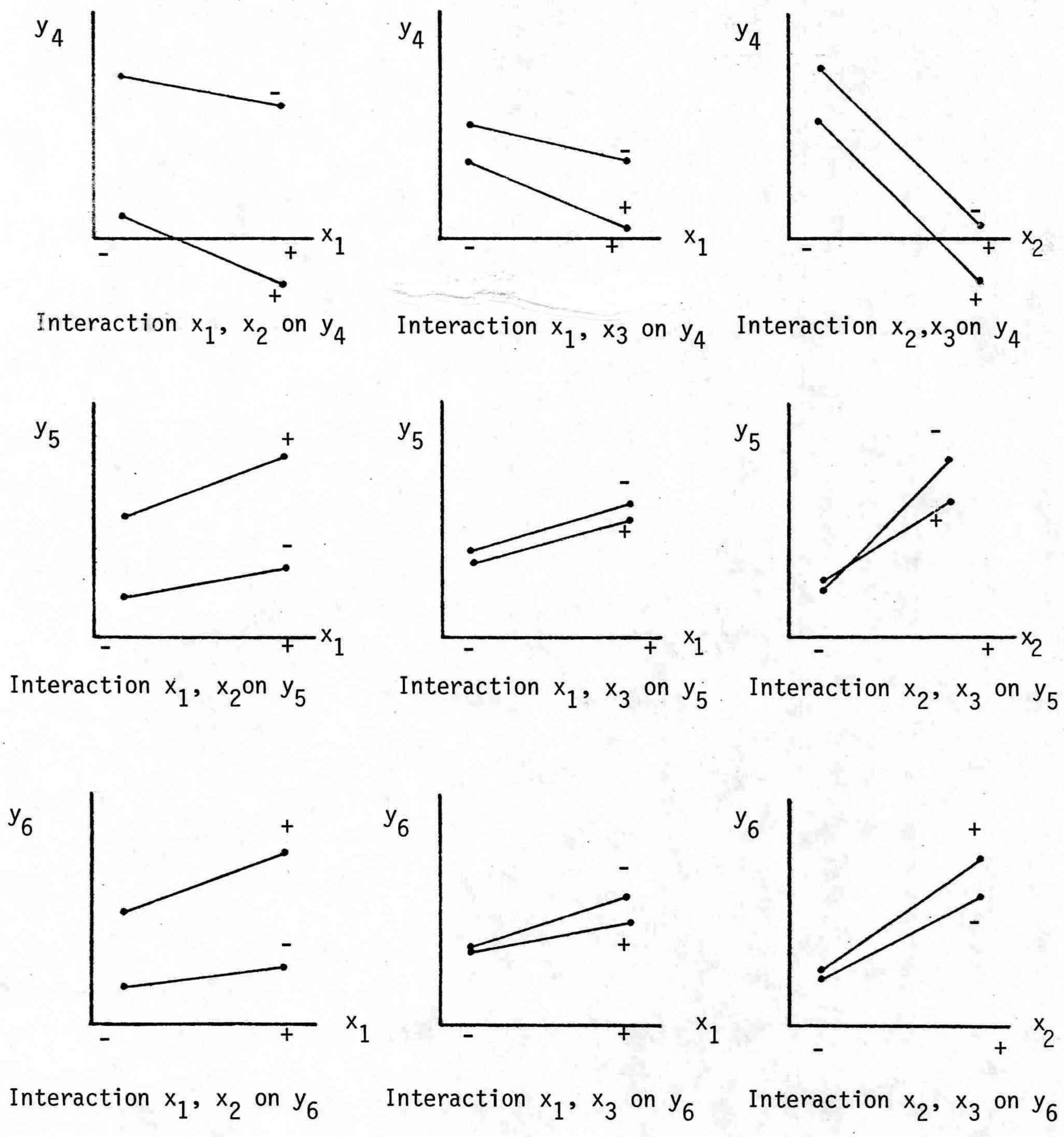


Figure 8-1. Two Factor Interaction in a Factorial Experiment for Responses y_1, y_2, and y_3

TABLE 8.10. VALUES ARE NEW VARIABLES AND RESULTS.

Case	Concrete Modulus (PSI)	Terminal PSI	Number of Failures	NPW Delta Cost (Millions of Dollars)
1	4.5×10^6	3.0	2.0	23.42
2	4.5×10^6	3.0	8.0	18.37
3	4.5×10^6	2.5	2.0	26.00
4	4.5×10^6	2.5	8.0	20.96
5	6.0×10^6	3.0	2.0	23.07
6	6.0×10^6	3.0	8.0	18.04
7	6.0×10^6	2.5	2.0	25.60
8	6.0×10^6	2.5	8.0	20.57

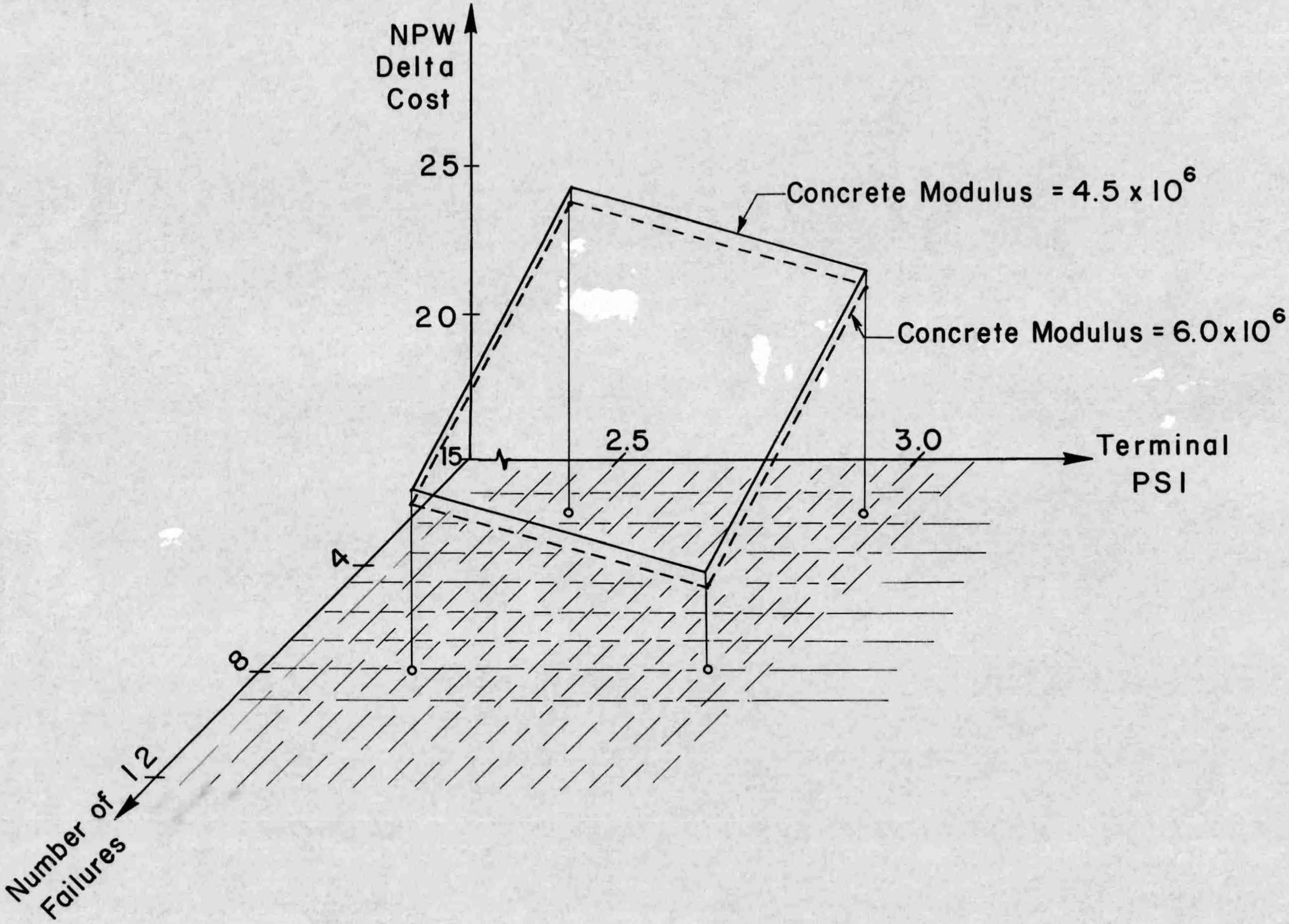


Figure 8.2: NPW delta cost as function of new variables.

Chapter 9

SUMMARY AND RECOMMENDATION

Briefly, the overall methodology can be synthetized presenting the basic changes made to the NULOAD program in order to obtain RENU: (a) a load distribution procedure has been modified to investigate the shift toward higher loads if new legal axle load limit is considered, (b) the Texas performance equations has been incorporated as an alternative to the AASHTO equations, (c) survivor curves has been generated and integrated to RENU, and (d) the capabilities of the model has been improved in the sense that the rehabilitation costs can be determined considering life cycles for both the current and new axle load legal limits.

The final recommendations of this research can be summarized as follows:

(a) Implementation of RENU in the SDHPT to forecast maintenance and rehabilitation costs considering appropriate levels of significant factors affecting the performance of Texas pavements.

(b) As future activities in other TTI projects such as studies 284 ("Flexible Pavement Data Base and Design") and 325 ("Estimating Remaining Service Life of Flexible Pavements"), research should be conducted to improve the equations to forecast pavement remaining service life and survivor mileage of pavements of a specific age, RENU should be properly modified to reflect such improvement. The current version can be modified to reflect such improvement. The current version of RENU will allow these modifications without major difficulties.

(c) Emphasis is placed on the importance of maintaining an updated data base which recognizes differences among districts as a result of changes in climate, soil, traffic, and other conditions. In this way, RENU will produce reliable results for each of the 25 districts of the Texas highway system.

REFERENCES

1. AASHTO "AASHTO Interim Guide for Design of Pavement Structure - 1972" Washington, D. C., 1972.

2. Austin Research Engineers, Inc., "Effects of Changes in Legal Load Limits on Pavement Costs," Vol. 1, FHWA Report No. FHWA-RD-79-73, July 1978.

3. Austin Research Engineers, Inc., "Effects of Changes in Legal Load Limits on Pavement Costs," Vol. 2, FHWA Report No. FHWA-RD-79-74, July 1978.

4. Butler, B. D., Jr., "Economic Analysis of Roadway Occupancy for Freeway Pavement Maintenance and Rehabilitation," Report Nos. FHWA-RD-76-14 and FHWA-RD-76-15, Vols. I, II, and III, 1974.

5. Carmichael, R. F., III, F. L. Roberts, P. R. Jordahl, H. J. Treybig, and F. N. Finn, "Effect of Changes in Legal Load Limits on Pavement Cost," Research Report No. FHWA-RD-78-98 ARE Inc., July 1978.

6. Carmichael, R. F., III, R. F. Roberts, P. R. Jordahl, H. J. Treybig, and F. N. Finn, "Effect of Chantes in Legal Load Limits on Pavement Cost, "Vol III, User's Manual from Program NULOAD, Report No. FHWA-RD-78-98 ARE, Inc. July 1978.

7. Cori, I. E., and B. G. Bullard, "A System for Estimating Present Surface Conditions and Remaining Service Life of Existing Pavements Sections" Public Roads, Vol. 36, No. 5, 1970, pp. 93-118.

8. De Weille, J., Quantification of Road User Savings, International Bank for Reconstructionand Development, 1966.

9. Garcia-Diaz, A., "A Bridge Replacement Model for Allocating Limited Funds in the Rehabilitation of Rural Transportation Networks," Ph.D. Thesis, University of Illinois, 1978.

10. Gutierrez de Velasco, Manuel, and B. F. McCullough, "Summary Report for 1978 CRCP Condition Survey in Texas," Research Report 177-20, Project 3-8-75-177, CTR, December 1979.

11. Highway Research Board, "The AASHTO Road Test: Report 1-History and Description of Project." Special Report 61 A, 1962.

12. Highway Research Board, "The AASHTO Road Test: Report 5-Pavement Research," Special Report 61 B, 1962.

13. Lang, A.S., and Wohl, M., "Evaluation of Highway Impact," in Some Evaluation of Highway Improvement Impacts, Highway Research Board, 1960.

14. Larkin, John G., "Modelling Future Truck Weight Patterns as Influenced by Alternative Vehicle Weight Legislation, "Master Thesis, The University of Texas at Austin, December 1978.

15. Long, Jim, and B. F. McCullough, "Condition Survey of CRCP in Walker and Fall-McLennan Counties," CTR, 1980.

16. Lytton, R. L. "Exponential Probability Density Function consistent with Deterioration Rate Concepts", Private Notes (unpublished). Texas A&M University, November 1980.

17. Lytton, R. L. "Relation of Distress and Structural Concept with Performance", Private notes (unpublished) Texas A&M University, November 1980.

18. McFarland, W. F., "Salvage Values in Pavement Analysis," unpublished report, undated.

19. McKinsey & Company, Inc., Guide to the Highway Rehabilitation Forecasting Model, 1976.

20. Manheim, M. L., "Highway Route Location as a Hierarchically-Structured Sequential Decision Process," Ph.D. Thesis, Department of the Environmental, TRRL Report 674, Crowthorne, 1975.

21. Mood, M. Alexander and F. A. Graybill, Introduction to the Theory of statistics, McGraw-Hill Book Co., New York 1963.

22. 1980 Heavy Construction Cost File A Construction Publishing Company Book, New York, 1980.

23. Noble, C. S., and B. F. Mccullough, "Distress Prediction Models for CRCP," Research Report 177-21, Project 3-8-75-177, CTR. December 1979.

24. Oglesby, C. H., and Altenhofen, M. J., "Economics of Design Standards for Low-Volume Rural Roads," National Cooperative Highway Research Program Report 63, 1969.

25. Robinson, R., Hide, H., Hodges, J. W., and Rolt, J., "A Road Transport Investment Model for Developing Countries," Department of the Environment, TRRL Report 674, Crowthorne, 1975.

26. Roussas, G. G., "First Course in Mathematical Statistics". University of Wisconsin, 1973.

27. Shippy, K. G., "A Look at Road User Costs," Proceedings of the Twenty-Eighth Annual Ohio Transportation Engineering Conference, 1974, pp. 176-185.

28. SHDPT, Interoffice Memorandum with Updated Documentation for REHAB, 1979.

29. SDHPT, Highway Cost Index Procedures, 1978.

30. Stark, P. A. "Introduction to Numerical Methods". Macmillan Publishing Co, 1970.

31. State Department of Highway and Public Transportation, Inc., "Evaluation of Computer Programs NULOAD and REHAB, Vol. 1, Research Report 298/312-1, September 1980.

32. State Department of Highway and Public Transportation, Inc., Evaluation of Computer Programs NULOAD and REHAB, Vol 2. Research Report 298/312.1, September 1980.

33. State Department of Highway and Public Transportation, Inc., Evaluation of Computer Programs NULOAD and REHAB, Vol 3. Research Report 298/312.1, September 1980.

34. State Department of Highway and Public Transportation, Inc., "Flexible Pavement Deflection Equation Using Elastic Moduli and Field Measurements", Research Report 207-7F, August 1979 (Russian Eqs.)

35. State Department of Highway and Public Transportation, "Forecast of the Highway Cost Index", 1980-1984" July 1980.

36. Strauss, J., B. F. McCullough, and W. R. Hudson, "CRCP Structural Performance and Design/Construction Variables," Research Report 177-7, Project 3-8-177, CTR, May 1977.

37. Texas Highway Department, "Texas SDHPT Temperature Constant, "Research Report No. 32.

38. Torres, Victor, and B. F. McCullough, "The Effect of the Aggregate Type on the CRCP Thickness," Technical Memorandum No. 249-30, CTR, December 1980.

39. Walton, C. M. et al., "Heavy Trucks on Texas Highways: An Economic Evaluation," Transportation Research Record, No. 725, 1979.

40. Whiteside, R. E.,T. Y. Chu, J. C. Cosby, R. L. Whitaker, and R. Winfrey, "Changes in Legal Vehicle Weights and Dimensions," NCHRP Report 141, Highway Research Board, 1973.

41. Winfrey, R., Economic Analysis for Highways, International Textbook Co., Scranton, PA., 1969.

42. Winfrey, R., "Statistical Analysis of Industrial Property Retirements," Bulletin 125, Iowa Engineering Experiment Station, Iowa State University, Ames, 1935, republished, 1967.

43. Yoder, E. J. and Witczak, Principles of Design, SEcond Edition, 1975.

44. Yu, Mark, and C. Michael Walton, "Estimating Vehicle Weight Distribution Shifts Resulting from Changes in Size and Weight Laws," Center for Transportation Research, The University of Texas at Austin, January 1981.

45. Yu, Mark and Ng Paul, "Truck Use on Texas Highways," Master's Thesis currently being developed. Center for Transportation Research, The University of Texas at Austin, 1981.

APPENDIX 1

REGRESSION ANALYSIS

The purpose of this appendix is to summarize the results concerning the performance of Texas flexible pavements. The appendix is divided into two parts. Part A corresponds to the serviceability methodology and Part B to the distress methodology. The following notation is used in the presentation of results:

TI = Thornthwaite Index
PR = Mean Presipitation
FTC = Freeze-thaw cycle
WFTC = Wet-thaw cycle
TM = Mean Annual Temperature
W = 18-Kip ESALs
ADT = Average Daily Traffic
DMD = Dynaflect
AS = Composite Stiffness
SCI = Subgrade Stiffness
TTC = Texas Triaxial Class
SLL = Liquid Limit
SPI = Plasticity Index
T = Years since reconstruction
SPP = % Subgrade Soil Passing Sieve 200

PART A SERVICEABILITY

1. Hot Mix Pavement, Rural, Low Traffic (P_i = 4.70)

$$K = 35,000 + 235. \; (SLL)^{-.2} \; (FTC)^{-.12} \; (TI)^{-.22} \quad \text{(A1-1)}$$

$$PF = 2.10 + 1236. \; (SLL)^{-0.8} \; (TM)^{-.3} \; (FTC)^{-.12} \; (WFTC)^{-.21} \; (TI)^{-.22} \; (AS)^{2.5} \quad \text{(A1-2)}$$

2. Hot Mix Pavement, Rural, High Traffic (Pi = 4.70)

$$K = 420,000 + 12,000 \; (T)^{.39} \; (AS)^{2.83} \; (TTC)^{0.12} \; (SCI)^{0.85} \quad \text{(A1-3)}$$

For PF use Eq.(A1-2).

3. Hot Mix Pavement, Urban, Low Traffic (Pi = 4.73)

$$K = 120,000 - 213 \times 10^{-12} \times (SLL)^{1.64} \; (DMD)^{-.46} \; (ALF)^{7.97} \; (AS)^{-1.43} \; (PR)^{-3.38} \; (W)^{-.25} \; (T)^{1.03} \quad \text{(A1-4)}$$

$$PF = 2.21 + 11.72 \; (SLL)^{-.08} \; (SCI)^{-.034} \; (ALF)^{-.167} \; (WFTC)^{-.08} \; (AS)^{.48} \; (T)^{-.059} \quad \text{(A1-5)}$$

4. Hot Mix Pavement, Urban, High Traffic (Pi = 4.73)

$$K = 1,330,000 - 2.33 \times 10^{-12} \times (SLL)^{1.64} \times (DMD)^{-.46} \times (ALF) \times (AS)^{-1.43} \times (PR)^{-3.38} \times (W)^{-.25} \times (T)^{1.03} \quad \text{(A1-6)}$$

For PF use Eq.(A1-5).

5. <u>Surface Treated Pavement, Rural</u> (P_i = 4.41)

$$K = 8,250 - 0.684 \ (DMO)^{.23} \ (TI)^{.38} \ X \ (WFTC)^{.18} \qquad (A1\text{-}7)$$

$$PF = 2.01 + 14.17 \ (SPI)^{0.018} \ (ALF)^{-.55} \ (FTC)^{-.24}$$
$$(TTC)^{-.17} \ (T)^{-.085} \ (W)^{-.55} \qquad (A1\text{-}8)$$

6. <u>Surface Treated Pavement, Urban</u> (Pi = 4.41)

$$K = 12,500 + 578 \ (DMO)^{.13} \ (TI)^{.33} \ (WFTC)^{.18} \ (W)^{.16} \ (T)^{1.48} \qquad (A1\text{-}9)$$

For PF use Eq. (A-8)

7. <u>Overlay, Rural, Low Traffic</u> (Pi = 4.81)

$$K = 58,300 + 1,253 \ X \ (SCI)^{-.32} \ X \ (DMD)^{1.4} \ X \ (TI)^{-.89} \ (T)^{.25}$$
$$(TTC)^{-1.74} \qquad (A1\text{-}10)$$

$$PF = 3.5 - .036 \ X \ (SCI)^{-.32} \ (DMD)^{1.4} \ (TI)^{.89} \ (T)^{.25} \ (TTC)^{.25} \qquad (A1\text{-}11)$$

8. <u>Overlay, Rural, High Traffic</u> (Pi = 4.81)

$$K = 620,000 - 12,320 \ X \ (SCI)^{-.53} \ X \ (DMD)^{1.5} \ (TI)^{.89} \ (T)^{.75} \qquad (A1\text{-}12)$$
$$(TTC)^{-1.74}$$

PF is calculated by Eq.(A1-11)

9. Overlay, Urban, Low Traffic (Pi = 4.81)

$$K = 183{,}000 - 231.6 \quad (T)^{1.76} \quad (SPP)^{.8} \quad (W)^{-.47} \qquad \text{(A1-13)}$$

$$PF = 2.00 + 1.31 \quad (SCI)^{-.15} \quad (T)^{-.021} \quad (PR)^{-.137} \qquad \text{(A1-14)}$$

10. Overlay, Urban, High Traffic (Pi = 4.81)

$$K = 1{,}833{,}000 = 2234 \quad (T)^{1.8} \quad (SPP)^{.6} \quad (W)^{-.27} \qquad \text{(A1-15)}$$

PF is calculated by Eq. (A1-14)

PART B: DISTRESS

1. TYPE OF PAVEMENT: HOT MIX

Rutting Severity

$$A1 = 10^{1.98} (SPI)^{-0.82} (ALF)^{0.47} (DMD)^{0.54} (W)^{-0.31} \quad (A1\text{-}16)$$

$$A2 = 10^{6.3} \quad (A1\text{-}17)$$

$$SF = 10^{9.42} (DMD)^{3.45} (W)^{-1.91} (SPI)^{-5.82} (ALF)^{2.80} \quad (A1\text{-}17)$$

Ravelling Severity

$$A1 = 10^{0.21} (ALF)^{-2.99} (DMD)^{0.80} (VOL)^{-0.88} (T)^{-1.17} (18\text{-}KIP)^{-0.33} (FTC)^{-0.89} \quad (A1\text{-}18)$$

$$A2 = 10^{6.961} \quad (A1\text{-}19)$$

$$SF = 10^{2.4} \quad (A1\text{-}20)$$

Flushing Severity

$$A1 = 10^{1.441} \quad (A1\text{-}21)$$

$$A2 = 10^{5.34} (AS)^{4.89} (ALF)^{-5.24} (SPI)^{-5.70} (WFTC)^{-1.72} (SLL)^{10.98} \quad (A1\text{-}22)$$

$$SF = 10^{0.27} \quad (A1\text{-}23)$$

Corrugations Severity

$$A1 = 10^{-1.77} (ALF)^{1.18} (FTC)^{0.51} (TTC)^{0.67} (T)^{0.91} (ADT)^{-0.86} (18\text{-}KIP)^{0.90} \quad (A1\text{-}24)$$

$$A2 = 0.00 \quad (A1\text{-}25)$$

$$SF = 10^{-5.96} (ALF)^{2.37} (FTC)^{1.03} (TTC)^{1.37} (T)^{1.91} (ADT)^{-1.74} (18\text{-}KIP)^{1.83} \quad (A1\text{-}26)$$

Alligator Cracking Severity

$$A1 = 10^{-.03} \quad (A1\text{-}27)$$

$$A2 = 10^{6.570} \quad (A1\text{-}28)$$

$$SF = 10^{-.5} (T)^{-5.84} (TTC)^{17.30} (SPI)^{09.82} (ADT)^{6.78} (18\text{-}KIP)^{-9.07} \quad (A1\text{-}29)$$

Longitudinal Cracking Severity

$$A1 = 10^{-.04} \quad (A1\text{-}30)$$

$$A2 = 10^{6.34} \quad (A1\text{-}31)$$

$$SF = 10^{-44.85} (TTC)^{14.61} (AS)^{-12.75} (TI)^{8.46} (FTC)^{1.71} (SLL)^{24.62} (SPI)^{-22.61} \quad (A1\text{-}32)$$

Transverse Cracking Severity

$$A1 = 10^{1.132} \quad \text{(A1-33)}$$

$$A2 = 10^{-14.64} (AS)^{5.74} (VOL)^{1.34} (SPP)^{17.44} (FTC)^{-0.25} (TIME\text{-}YRS)^{-2.35} \quad \text{(A1-34)}$$

$$SF = 10^{-0.754} \quad \text{(A1-35)}$$

Patching Severity

$$A1 = 10^{1.077} \quad \text{(A1-36)}$$

$$A2 = 10^{6.165} \quad \text{(A1-37)}$$

$$SF = 10^{7.90} (ADT)^{-0.62} (SCI)^{1.0} (PR)^{2.21} (SLL)^{-8.97} (SPI)^{6.34} \quad \text{(A1-38)}$$

Failures/Mile Severity

$$A1 = 10^{-1.37} (FTC)^{0.59} (TTC)^{2.13} (ALF)^{2.03} (ADT)^{-0.59} (SLL)^{-1.35} (18\text{-}KIP)^{0.60} \quad \text{(A1-39)}$$

$$A2 = 0.00 \quad \text{(A1-40)}$$

$$SF = 10^{-1.281} \quad \text{(A1-41)}$$

Rutting Area

$$AO = 10^{6.56} \quad \text{(A1-42)}$$

Ravelling Area

$$AO = 10^{6.96} \quad \text{(A1-43)}$$

Flushing Area

$$AO = 10^{6.82} \quad \text{(A1-44)}$$

Corrugations Area

$$AO = 0.000 \quad \text{(A1-45)}$$

Alligator Cracking Area

$$AO = 10^{6.81} \quad \text{(A1-46)}$$

Longitudinal Cracking Area

$$AO = 10^{5.5} \quad \text{(A1-47)}$$

Transverse Cracking Area

$$AO = 10^{5.49} \quad \text{(A1-48)}$$

Patching Area

$$AO = 10^{6.351} \quad \text{(A1-49)}$$

2. TYPE OF PAVEMENT: HOT MIX ON BLACK BASE

Rutting Severity

$$A1 = 10^{0.360} (TTC)^{-0.88} (VOL)^{0.36} (WFTC)^{0.23} (ADT)^{0.38} (18\text{-}KIP)^{-0.45} \quad (A1\text{-}50)$$

$$A2 = 10^{-7.35} (VOL)^{-1.34} (WFTC)^{1.81} (TTC)^{7.11} (ADT)^{-0.58} (ALF)^{11.23} (PR)^{-8.22} \quad (A1\text{-}51)$$

$$SF = 10^{-1.13} (VOL)^{2.44} (WFTC)^{0.90} (TTC)^{-5.25} (18\text{-}KIP)^{-2.32} (ADT)^{1.84} \quad (A1\text{-}52)$$

Ravelling Severity

$$A1 = 10^{0.07} \quad (A1\text{-}53)$$

$$A2 = 10^{3.74} (AS)^{3.73} (PR)^{-1.20} (SPI)^{1.93} (18\text{-}KIP)^{-1.41} (ADT)^{1.11} \quad (A1\text{-}54)$$

$$SF = 10^{-0.06} \quad (A1\text{-}55)$$

Flushing Severity

$$A1 = 10^{-9.57} (WFTC)^{0.37} (ADT)^{0.19} (SPP)^{6.17} (AS)^{4.56} (SPI)^{-1.83} (SLL)^{4.28} \quad (A1\text{-}56)$$

$$A2 = 10^{22.02} (AS)^{-3.15} (ALF)^{-7.40} (FTC)^{-2.90} (TTC)^{-3.54} (TIME\text{-}YRS)^{2.07} (ADT)^{-0.76} \quad (A1\text{-}57)$$

$$SF = 10^{-0.04} \quad (A1\text{-}58)$$

Corrugations Severity

$$A1 = 10^{-0.04} \quad (A1\text{-}59)$$

$$A2 = 10^{6.7} \quad (A1\text{-}60)$$

$$SF = 10^{-2.2} \quad (A1\text{-}61)$$

Alligator Cracking Severity

$$A1 = 10^{-0.03} (SCI)^{0.24} (ALF)^{-1.17} (TTC)^{1.25} (TI)^{-15.41} (TIME\text{-}YRS)^{1.24} \quad (A1\text{-}62)$$

$$A2 = 10^{6.88} \quad (A1\text{-}63)$$

$$SF = 10^{-1.07} (SCI)^{1.05} (ALF)^{-4.64} (SPI)^{1.97} (TIME\text{-}YRS)^{5.22} \quad (A1\text{-}64)$$

Longitudinal Cracking Severity

$$A1 = 10^{-0.02} (TI)^{-11.70} (TIME\text{-}YRS)^{0.54} (TTC)^{0.83} (SPI)^{-0.27} (18\text{-}KIP)^{-0.17} \quad (A1\text{-}65)$$

$$A2 = 10^{6.74} \quad (A1\text{-}66)$$

$$SF = 10^{-.26} (18\text{-}KIP)^{-1.35} (SPI)^{-1.29} (TIME\text{-}YRS)^{4.49} \quad (A1\text{-}67)$$

Transverse Cracking Severity

$$A1 = 10^{-0.473} (FTC)^{-0.26} (PR)^{-1.21} (18\text{-}KIP)^{-0.41} (SCI)^{-0.26} (TIME\text{-}YRS)^{2.12} \quad (A1\text{-}67)$$

$$A2 = 10^{-1.70} (TIME\text{-}YRS)^{-0.70} (PR)^{1.57} (FTC)^{0.83} (AS)^{-4.03} \quad (A1\text{-}68)$$

$$SF = 10^{11.79} (PR)^{-6.25} (18\text{-}KIP)^{-1.41} (FTC)^{-.269} (TIME\text{-}YRS)^{7.20} (AS)^{12.76} \quad (A1\text{-}69)$$

Patching Severity

$$A1 = 10^{-0.65} \quad (A1\text{-}70)$$

$$A2 = 10^{6.66} \quad (A1\text{-}71)$$

$$SF = 10^{-.2} \quad (A1\text{-}72)$$

Failure/Mile Severity

$$A1 = 10^{0.10} \quad (A1\text{-}73)$$

$$A2 = 0.00 \quad (A1\text{-}74)$$

$$SF = 10^{-.3} \quad (A1\text{-}75)$$

Rutting Area

$$AO = 10^{6.97} (SCI)^{0.0054} (SPI)^{0.0033} (FTC)^{-0.0029} (18\text{-}KIP)^{-.0098} (TIME\text{-}YRS)^{0.022} (ADT)^{-0.018} \quad (A1\text{-}77)$$

Ravelling Area

$$AO = 10^{5.20} (FTC)^{0.00076} (WFTC)^{-0.0011} (SPI)^{0.0012} (SPP)^{-0.010} (VOL)^{0.00040} (TIME\text{-}YRS)^{0.0017} \quad (A1\text{-}78)$$

Flushing Area

$$AO = 10^{4.98} (SPP)^{-0.013} (DMD)^{0.0034} (VOL)^{-0.0061} (18\text{-}KIP)^{-0.0012} (AS)^{-0.019} \quad (A1\text{-}79)$$

Corrugations Area

$$AO = 10^{6.2} \quad (A1\text{-}80)$$

Alligator Cracking Area

$$AO = 10^{7.01} \quad (A1\text{-}81)$$

Longitudinal Cracking Area

$$A0 = 10^{6.84} \tag{A1-82}$$

Transverse Cracking Area

$$A0 = 10^{6.13} \tag{A1-83}$$

Patching Area

$$A0 = 10^{6.78} \tag{A1-84}$$

3. TYPE OF PAVEMENT: SURFACE TREATED PAVEMENT

Rutting Severity

$$A1 = 10^{6.01} \quad (A1\text{-}85)$$

$$A2 = 10^{7.32} (ADT)^{-0.15} (TIME\text{-}YRS)^{-0.25} (SPI)^{-0.97} (PR)^{0.55} (SLL)^{1.83} (TTC)^{-1.75} \quad (A1\text{-}86)$$

$$SF = 10^{-0.2} \quad (A1\text{-}87)$$

Ravelling Severity

$$A1 = 10^{5.31} (VOL)^{-0.57} (AS)^{-2.42} (FTC)^{0.56} (PR)^{0.40} (WFTC)^{-0.39} (18\text{-}KIP)^{-0.064} \quad (A1\text{-}88)$$

$$A2 = 10^{6.05} (TI)^{0.67} (ALF)^{0.78} (VOL)^{0.23} (18\text{-}KIP)^{-0.24} (SPI)^{-1.46} (SLL)^{2.44} \quad (A1\text{-}89)$$

$$SF = 10^{-0.01} \quad (A1\text{-}90)$$

Flushing Severity

$$A1 = 10^{6.80} \quad (A1\text{-}91)$$

$$A2 = 10^{5.06} (WFTC)^{-0.15} (AS)^{-1.16} (SPI)^{0.38} (ADT)^{-0.30} (DMD)^{-0.36} \quad (A1\text{-}92)$$

$$SF = 10^{-.2} (ALF)^{-9.33} (TTC)^{14.63} (AS)^{19.30} \quad (A1\text{-}93)$$

Corrugations Severity

$$A1 = 10^{0.98} \qquad \text{(A1-94)}$$

$$A2 = 10^{6.18} \qquad \text{(A1-95)}$$

$$SF = 10^{-1.91} \qquad \text{(A1-96)}$$

Alligator Cracking Severity

$$A1 = 10^{1.49} \qquad \text{(A1-97)}$$

$$A2 = 10^{7.43} \qquad \text{(A1-98)}$$

$$SF = 10^{-0.25} \qquad \text{(A1-99)}$$

Longitudinal Cracking Severity

$$A1 = 10^{-.36} (SLL)^{0.33} (TI)^{0.39} (VOL)^{-0.076} (PR)^{-0.49} (TTC)^{1.28} \qquad \text{(A1-100)}$$

$$A2 = 10^{6.0} \qquad \text{(A1-101)}$$

$$SF = 10^{-11.07} (T)^{2.11} (PR)^{-5.10} (ALF)^{-6.78} (SPI)^{7.18} (TTC)^{14.39} \qquad \text{(A1-102)}$$

Transverse Cracking Severity

$$A1 = 10^{-.46} \quad (A1\text{-}103)$$

$$A2 = 10^{6.81} \quad (A1\text{-}104)$$

$$SF = 10^{-.07} \quad (A1\text{-}105)$$

Patching Severity

$$A1 = 10^{-1.60} \quad (A1\text{-}106)$$

$$A2 = 10^{6.86} \quad (A1\text{-}107)$$

$$SF = 10^{-.31} \quad (A1\text{-}108)$$

Failures/Mile Severity

$$A1 = 10^{-1.68} \quad (A1\text{-}109)$$

$$A2 = 10^{6.24} \quad (A1\text{-}110)$$

$$SF = 10^{-.06} \quad (A1\text{-}111)$$

Rutting Area

$$AO = 10^{7.05} \quad \text{(A1-112)}$$

Ravelling Area

$$AO = 10^{4.86} \ (PR)^{-0.31 \times 10^{-3}} \ (TI)^{0.52 \times 10^{-3}} \quad \text{(A1-113)}$$

Flushing Area

$$AO = 10^{4.96} \ (VOL)^{0.24 \times 10^{-3}} \ (TI)^{0.40 \times 10^{-3}} \ (W)^{-0.11 \times 10^{-3}} \quad \text{(A1-114)}$$

Corrugations Area

$$AO = 10^{6.23} \quad \text{(A1-115)}$$

Alligator Cracking Area

$$AO = 10^{7.47} \ (TI)^{-0.16 \times 10^{-3}} \ (DMD)^{-0.17 \times 10^{-3}} \quad \text{(A1-116)}$$

Longitudinal Cracking Area

$$AO = 10^{5.05} \ (AS)^{-0.55 \times 10^{-3}} \ (PR)^{0.26 \times 10^{-3}} \quad \text{(A1-117)}$$

Transverse Cracking Area

$$AO = 10^{6.84} \tag{A1-118}$$

Patching Area

$$AO = 10^{6.92} (DMD)^{0.14 \times 10^{-2}} (VOL)^{-0.20 \times 10^{-2}} (TI)^{-0.15 \times 10^{-2}} (SPP)^{-0.17 \times 10^{-2}} \tag{A1-119}$$

4. TYPE OF PAVEMENT: OVERLAYS

Rutting Severity

$$A1 = 10^{-.86} (TI)^{0.84} (PR)^{-0.69} (SPI)^{0.40} (ADT)^{0.25} (TIME\text{-}YRS)^{0.38} (18\text{-}KIP)^{-0.27} \quad (A1\text{-}120)$$

$$A2 = 10^{7.01}$$

$$SF = 10^{-12.95} (TI)^{3.55} (PR)^{-3.25} (SPI)^{1.85} (ADT)^{1.73} (18\text{-}KIP)^{-2.15} (TIME\text{-}YRS)^{3.27} \quad (A1\text{-}121)$$

Ravelling Severity

$$A1 = 10^{-.20} \quad (A1\text{-}122)$$

$$A2 = 10^{5.13} \quad (A1\text{-}123)$$

$$SF = 10^{-.25} \quad (A1\text{-}124)$$

Flushing Severity

$$A1 = 10^{1.33} \quad (A1\text{-}125)$$

$$A2 = 10^{5.03} \quad (A1\text{-}126)$$

$$SF = 10^{-0.71} \quad (A1\text{-}127)$$

Corrugations Severity

$$A1 = 10^{-4.95} (FTC)^{-0.063} (PR)^{-0.22} (SPP)^{4.58} \quad (A1\text{-}129)$$

$$A2 = 10^{6.191} \quad (A1\text{-}130)$$

$$SF = 10^{-17.11} (WFTC)^{-0.69} (W)^{0.11} (ALF)^{-0.98} (TTC)^{-2.34} (SPP)^{13.73} \quad (A1\text{-}131)$$

Alligator Cracking Severity

$$A1 = 10^{-0.48} \quad (A1\text{-}132)$$

$$A2 = 10^{6.74} \quad (A1\text{-}133)$$

$$SF = 10^{-0.03} \quad (A1\text{-}134)$$

Longitudinal Cracking Severity

$$A1 = 10^{-.41} \quad (A1\text{-}135)$$

$$A2 = 10^{4.21} (FTC)^{-0.17} (SCI)^{0.16} (TTC)^{-0.86} (ADT)^{0.18} (TI)^{-1.23} \quad (A1\text{-}134)$$

$$SF = 10^{-15.37} (SLL)^{-3.79} (ADT)^{-0.70} (TI)^{7.00} (FTC)^{1.88} (TTC)^{16.74} (T)^{-2.00} \quad (A1\text{-}135)$$

Transverse Cracking Severity

$$A1 = 10^{-.43} \quad \text{(A1-136)}$$

$$A2 = 10^{5.53} \quad \text{(A1-137)}$$

$$SF = 10^{-0.04} \quad \text{(A1-138)}$$

Patching Severity

$$A1 = 10^{-.27} \quad \text{(A1-139)}$$

$$A2 = 10^{6.78} \quad \text{(A1-140)}$$

$$SF = 10^{-.47} \quad \text{(A1-141)}$$

Failure/Mile Severity

$$A1 = 10^{-.30} \quad \text{(A1-142)}$$

$$A2 = 10^{.11} \quad \text{(A1-143)}$$

$$SF = 10^{-.50} \quad \text{(A1-144)}$$

Rutting Area

$$AO = 10^{7.17} (PR)^{0.011} (SPI)^{0.017} (SLL)^{-0.030} \quad (A1\text{-}145)$$

Ravelling Area

$$AO = 10^{5.246} \quad (A1\text{-}146)$$

Flushing Area

$$AO = 10^{5.14} \quad (A1\text{-}147)$$

Corrugations Area

$$AO = 10^{6.14} \quad (A1\text{-}148)$$

Alligator Cracking Area

$$AO = 10^{6.88} \quad (A1\text{-}149)$$

Longitudinal Cracking Area

$$AO = 10^{6.16} \quad (A1\text{-}150)$$

Transverse Cracking Area

$$AO = 10^{6.58} \qquad (A1\text{-}151)$$

Patching Area

$$AO = 10^{6.88} \qquad (A1\text{-}152)$$

5. TYPE OF PAVEMENT: THICK HOT MIX

Rutting Severity

$$A1 = 10^{-0.561} \quad \text{(A1-153)}$$

$$A2 = 10^{5.619} \quad \text{(A1-154)}$$

$$SF = 10^{-.852} \quad \text{(A1-155)}$$

Ravelling Severity

$$A1 = 10^{0.510} \quad \text{(A1-156)}$$

$$A2 = 10^{6.430} \quad \text{(A1-157)}$$

$$SF = 10^{-.50} \quad \text{(A1-158)}$$

Flushing Severity

$$A1 = 10^{-0.736} \quad \text{(A1-159)}$$

$$A2 = 10^{6.23} \quad \text{(A1-160)}$$

$$SF = 10^{-.048} \quad \text{(A1-161)}$$

Corrugations Severity

None

Alligator Cracking Severity

$$A1 = 10^{-0.82} \quad (A1\text{-}162)$$

$$A2 = 10^{5.88} \quad (A1\text{-}163)$$

$$SF = 10^{-3.52} \quad (A1\text{-}164)$$

Longitudinal Cracking Severity

$$A1 = 10^{-0.88} \quad (A1\text{-}165)$$

$$A2 = 10^{6.60} \quad (A1\text{-}166)$$

$$SF = 10^{-1.06} \quad (A1\text{-}167)$$

Transverse Cracking Severity

$$A1 = 10^{0.728} \quad (A1\text{-}168)$$

$$A2 = 10^{5.887} \quad (A1\text{-}169)$$

$$SF = 10^{-.294} \quad (A1\text{-}170)$$

Patching Severity

$$A1 = 10^{-0.92} \quad (A1-171)$$

$$A2 = 10^{5.33} \quad (A1-172)$$

$$SF = 10^{-0.89} \quad (A1-173)$$

Failures/Mile Severity

$$A1 = 10^{0.601} \quad (A1-174)$$

$$A2 = 10^{6.7} \quad (A1-175)$$

$$SF = 10^{-0.891} \quad (A1-176)$$

Rutting Area

$$A0 = 10^{6.95} \quad (A1-177)$$

Ravelling Area

$$A0 = 10^{4.58} \quad (A1-178)$$

Flushing Area

$$AO = 10^{4.408} \quad \text{(A1-179)}$$

Corrugations Area

$$AO = 10^{5.3} \quad \text{(A1-180)}$$

Alligator Cracking Area

$$AO = 10^{7.03} \quad \text{(A1-181)}$$

Longitudinal Cracking Area

$$AO = 10^{6.00} \quad \text{(A1-182)}$$

Transverse Cracking Area

$$AO = 10^{6.88} \quad \text{(A1-183)}$$

Patching Area

$$AO = 10^{6.65} \quad \text{(A1-184)}$$

APPENDIX 2

SURVIVOR CURVES

This Appendix contains the survivor curves computer program which was used to generate the set of survivor functions, for flexible pavements, actually used in the RENU program. The computer routine to generate survivor curves has not been incorporated to the RENU program because of the increase in computer time implied by the parameters estimation process, on the other hand, the survivor curves generation is a process which does not need to be repeted more than one time if the initial data is not changed, which is actual situation.

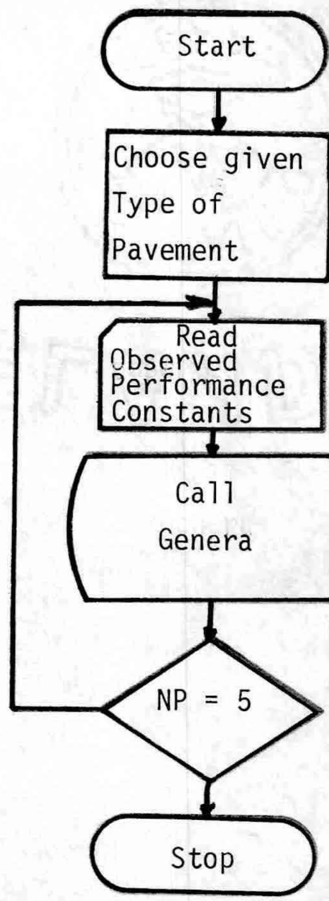
This Appendix has been divided in two parts: the first one contains the flow chart of the survivor curves generation process and the second part is a print out of the computer program. The computer program has the following structure:

(1) Subroutine GENERA which contains the procedure developed in Appendix 6 to generate a sample of values of 18-Kip ESALs corresponding with some critical value of the performance index.

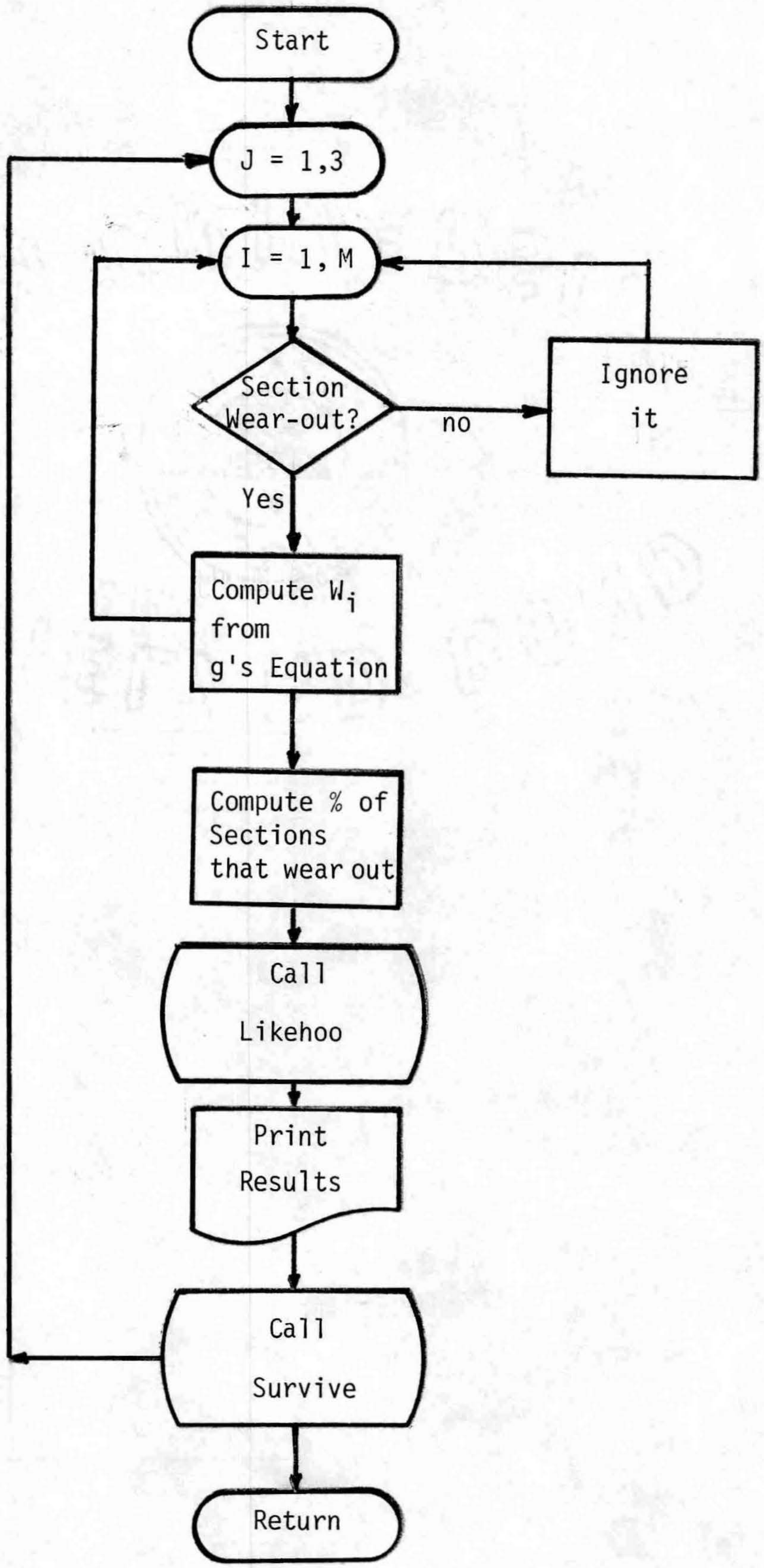
(2) Subroutine LIKEHO in which is solved the Gamma function and all the other statistical parameters needed are computed.

(3) BLOCK DATA where the information corresponding to P_f's and K's values is supplied.

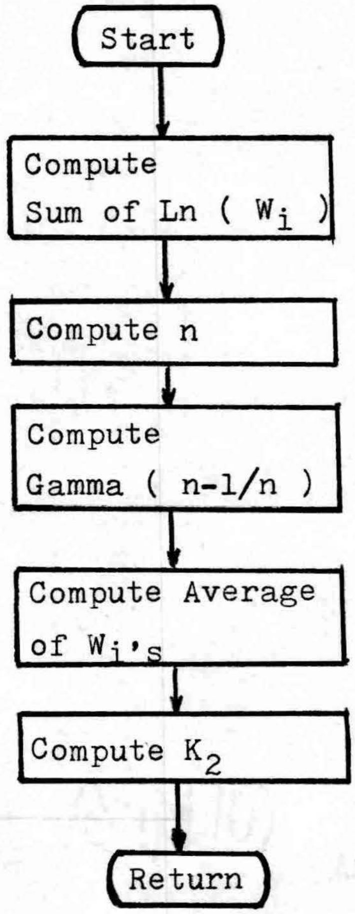
MAIN PROGRAM



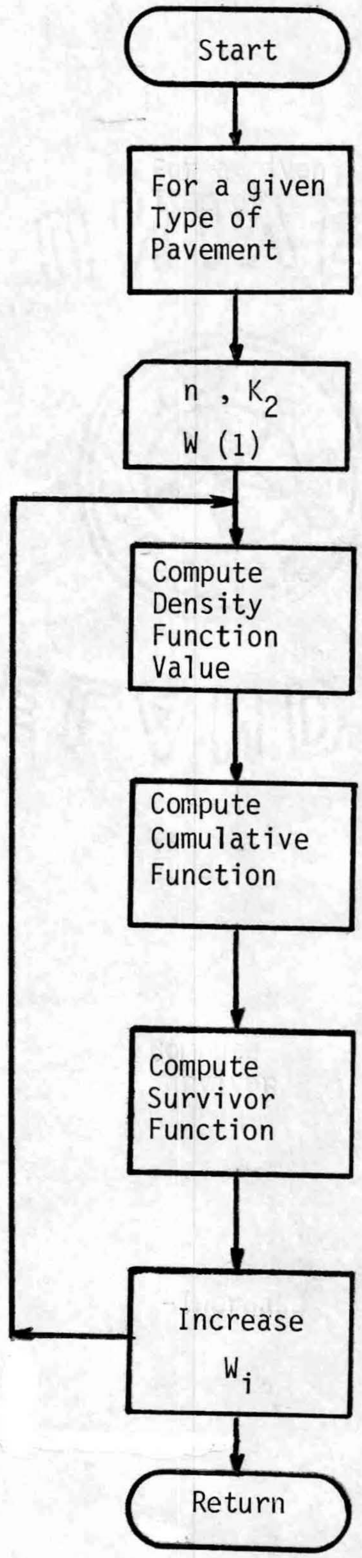
SUBROUTINE GENERA



SUBROUTINE LIKEHO



SUBROUTINE SURVIV



```
$OPTIONS
      DIMENSION TOTEA(100),LL(100),XNND(5000),W(200)
      COMMON PF1(31),XK1(31),PF2(48),XK2(48),PF3(48),XK3(48),PF4(48),
     *XK4(48),PF5(48),XK5(48),PI(5),P(9),PF(48),XK(48)
      DO 1 NP=1,5
      GO TO (10,20,30,40,50),NP
   10 CONTINUE
      WRITE(6,11)
   11 FORMAT(5X,'TYPE OF PAVEMENT HOT MIX PAVEMENT LOW'/)
      M = 31
      DO 111 K1=1,M
      PF(K1)=PF1(K1)
      XK(K1)=XK1(K1)* 238.48
  111 CONTINUE
      CALL GENERA(M,NP)
      GO TO 1
   20 CONTINUE
      WRITE(6,21)
   21 FORMAT(5X,'TYPE OF PAVEMENT SURFACE TREATED '/)
      M = 48
      DO 112 K2=1,M
      PF(K2)=PF2(K2)
      XK(K2)=XK2(K2) * 8.6
  112 CONTINUE
      CALL GENERA(M,NP)
      GO TO 1
   30 CONTINUE
      WRITE(6,31)
   31 FORMAT(5X,'TYPE OF PAVEMENT HOT MIX PAVEMENT HIGH'/)
      M = 48
      DO 113 K3=1,M
      PF(K3)=PF3(K3)
      XK(K3)=XK3(K3) * 2340.11
  113 CONTINUE
      CALL GENERA(M,NP)
      GO TO 1
   40 CONTINUE
      WRITE(6,41)
   41 FORMAT(5X,'TYPE OF PAVEMENT OVERLAY LOW'/)
      M = 48
      DO 114 K4=1,M
      PF(K4)=PF4(K4)
      XK(K4)=XK4(K4) * 74.89
  114 CONTINUE
      CALL GENERA(M,NP)
      GO TO 1
   50 CONTINUE
      WRITE(6,51)
   51 FORMAT(5X,'TYPE OF PAVEMENT OVERLAY HIGH'/)
      M = 48
      DO 115 K5=1,M
      PF(K5)=PF5(K5)
      XK(K5)=XK5(K5) * 753.3
  115 CONTINUE
      CALL GENERA(M,NP)
    1 CONTINUE
      STOP
      END
```

```
      SUBROUTINE GENERA(M,NP)
      DIMENSION TOTEA(100),LL(100),XNND(5000),W(200)
      COMMON PF1(31),XK1(31),PF2(48),XK2(48),PF3(48),XK3(48),PF4(48),
     *XK4(48),PF5(48),XK5(48),PI(5),P(9),PF(48),XK(48)
      WRITE(6,71)
   71 FORMAT(10X,'SERV. INDEX',10X,'% OF SECT. THAT WEAR OUT',5X,'LOG(K)
     *',10X,'N'//)
      DO 2 J=1,9
      L = 0
      DO 3 I=1,M
C*****CHECK OUT IF THE PAVEMENT WEAR OUT*********
      IF(PF(I).GE.P(J)) GO TO 63
      TOTEAL = (-XK(I)/ALOG((PI(NP) - P(J))/(PI(NP) - PF(I))))
      IF(NP.EQ.1.AND.TOTEAL.GT.500000.) TOTEAL=375000.
      IF(NP.EQ.2.AND.TOTEAL.GT.40000.) TOTEL=30000.
      IF(NP.EQ.3.AND.TOTEAL.GT.5000000.) TOTEAL=3750000.
      IF(NP.EQ.4.AND.TOTEAL.GT.450000.)TOTEAL=325000.
      IF(NP.EQ.5.AND.TOTEAL.GT.4000000.)TOTEAL=3250000.
      L = L + 1
      LL(L) = I
      TOTEA(L) = TOTEAL
      GO TO 3
   63 IF(I.LT.M) GO TO 3
      IF(L.EQ.0) GO TO 777
      GO TO 62
    3 CONTINUE
   62 XL = L
      WRITE(6,88) (TOTEA(I),I=1,L)
   88 FORMAT(5X,3F15.0)
      XM = M
      PWEAR = XL/XM
      PNWEAR = (XM-XL)/XM
      IF(L.LE.1) GO TO 777
      CALL LIKEHO(TOTEA,L,J,LL,XN,XKP,NP)
      GO TO 888
  777 CONTINUE
      PWEAR = 0.
      PNWEAR = 1.
      XN = 2222.
      XKP = 3333.
  888 WRITE(6,72) P(J),PWEAR,XKP,XN
   72 FORMAT(13X,F4.2,21X,F5.3,14X,F12.1,7X,F8.3/)
    2 CONTINUE
      RETURN
      END

      SUBROUTINE LIKEHO(TOTEA,L,J,LL,XN,XKP,NP)
      DIMENSION XNNE(1000)
      DIMENSION TOTEA(100),LL(100),XNND(5000),W(200)
      COMMON PF1(31),XK1(31),PF2(48),XK2(48),PF3(48),XK3(48),PF4(48),
     *XK4(48),PF5(48),XK5(48),PI(5),P(9),PF(48),XK(48)
      S1 = 0.
      DO 8 I=1,L
      Y = ALOG(TOTEA(I))
      S1 = S1 + Y
    8 CONTINUE
      XL = L
      STOT = 0.
```

```
      DO 10 J1=1,L
      STOT = STOT +TOTEA(J1)
 10   CONTINUE
      AVTOT = STOT / XL

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      S2 = 0.
      DO 91 J3=1,L
 91   S2 = S2 + (TOTEA(J3)-AVTCT)**2
      XLM = XL - 1.
      SM = S2/XLM
      SIGMA = SM**0.5
      CV = SIGMA / AVTCT

 TRIAL AND ERROR PRCCEDURE TO ESTIMATE N VALUES  USING CV VALUE

      XN = 2.5
      DO 200 KK=1,1000
      XNN1 = (XN - 1.) / XN
      XNND(1) = XNN1
      DO 133 I=1,10000
      IF(XNND(I).LT.40.) GC TC 214
      K1 = I + 1
      XNND(K1) = XNNC(I) - 1.
      IF(XNND(I+1).LT.40.) GC TC 214
133   CONTINUE
214   IF(I.EQ.1) GC TC 215
      ARGAMM = XNND(I+1)
      GO TO 216
215   ARGAMM = XNND(1)
      GO TO 217
216   CONTINUE
      PP1 = 1.
      L2 = I+1
      DO 134 J2=1,L2
      PP1 = PP1 * XNNC(J2)
134   CONTINUE
      GO TO 218
217   CONTINUE
      PP1 = 1.
218   GAMND = PP1 * GAMMA(ARGANM)
      XNN2 = (XN - 2.) / XN
      XNNE(1) = XNN2
      DO 136 JJ=1,10000
      IF(XNNE(JJ).LT.40.) GC TC 314
      K2 =JJ + 1
      XNNE(K2) = XNNE(JJ) - 1.
      IF(XNNE(JJ+1).LT.40.) GC TO 314
136   CONTINUE
314   IF(JJ.EQ.1) GC TC 315
      ARGAMM = XNNE(JJ+1)
      GO TO 316
315   ARGAMM = XNNE(1)
      GO TO 317
316   CONTINUE
      PP2 = 1.
      L3 =JJ + 1
      DO 137 J3=1,L3
      PP2 = PP2 * XNNE(J3)
```

```
 37 CONTINUE
    GO TO 318
317 CONTINUE
    PP2 = 1.
318 CONTINUE
    GAMNE = PP2 * GAMMA(ARGANN)
    YY =((GAMNE/GAMND) - 1.)**0.5
    ERROR = ABS(YY - CV)
    IF(ERROR.LT.0.01) GO TO 319
    XN = XN + 0.01
200 CONTINUE
319 IF(XN.LT.2.5) XN = 2.5
    XKP = XN * (ALOG(AVTOT) - ALOG(GAMND))
    RETURN
    END

    BLOCK DATA
    COMMON PF1(31),XK1(31),PF2(48),XK2(48),PF3(48),XK3(48),PF4(48),
   *XK4(48),PF5(48),XK5(48),PI(5),P(9),PF(48),XK(48)
    DATA PI/4.7,4.73,4.41,4.81,4.6/
    DATA P/2.2,2.3,2.4,2.5,2.6,2.7,2.8,2.9,2.95/
    DATA PF1/3.41,3.83,3.4,2.86,3.46,2.54,3.53,3.66,3.65,2.31,3.34,3.3
   *8,2.9,2.66,3.21,3.22,2.76,2.97,3.11,2.32,4.04,4.05,4.26,4.18,4.02,
   *3.88,3.75,3.36,2.65,2.9,4.09/
    DATA XK1/325.3,285.7,325.6,360.8,319.8,376.5,315.2,303.7,304.9,386
   *.6,330.3,326.9,358.7,371.,339.7,339.,365.7,354.3,345.9,386.2,258.6
   *,256.,217.6,234.6,260.4,279.7,294.1,329.2,371.4,358.2,250.7/
    DATA XK2/239.,354.2,307.44,391.1,317.5,253.5,317.7,277.83,223.5,22
   *1.6,292.2,209.7,229.6,264.9,233.9,223.2,240.9,317.2,252.1,310.,238
   *.9,374.4,305.2,231.9,411.2,249.4,233.,319.1,229.9,278.,240.2,258.9
   *,238.9,360.,336.3,342.6,320.,370.,303.1,260.1,305.6,302.3,329.4,31
   *7.9,290.9,349.6,243.2,274.7/
    DATA PF2/4.18,3.,3.65,2.23,3.53,4.10,3.53,3.93,4.26,4.27,3.8,4.32,
   *4.23,4.02,4.21,4.26,4.17,3.54,4.11,3.72,4.18,2.62,3.67,4.22,1.68,
   *4.12,4.22,3.52,4.23,3.92,4.18,4.07,4.19,2.90,3.29,3.19,3.51,2.71,3
   *.69,4.06,3.67,3.7,3.38,3.53,3.81,3.08,4.16,3.95/
    DATA XK3/279.2,284.,407.2,401.2,414.2,289.5,388.9,383.1,408.2,307.
   *6,402.6,407.6,328.6,372.1,386.,278.4,319.6,335.2,367.6,332.1,285.8
   *,291.5,333.1,304.3,240.6,341.4,394.3,389.5,402.8,318.5,298.3,255.4
   *,331.6,237.7,362.5,332.7,359.7,328.,349.1,372.2,414.3,387.7,404.6,
   *368.2,364.,323.8,376.,335.9/
    DATA PF3/3.59,3.55,1.48,1.65,1.26,3.51,1.97,2.11,1.45,2.,1.61,1.47
   *,3.07,2.34,2.03,3.6,3.19,2.98,2.44,3.03,3.54,3.49,3.01,3.36,3.85,
   *2.89,1.83,1.95,1.6,3.2,3.42,3.77,3.03,3.87,2.54,3.02,2.59,3.08,2.7
   *7,2.34,1.26,2.,1.55,2.45,2.51,3.14,2.26,2.97/
    DATA XK4/355.3,361.3,340.2,317.9,305.7,315.4,412.6,313.1,363.4,
   *384.2,336.8,234.1,233.2,287.7,300.1,247.2,406.,277.3,256.3,364.16,
   *364.,253.8,274.,264.,308.3,240.6,343.1,371.2,378.2,274.5,374.5,416
   *.4,262.,263.6,318.9,285.1,351.6,386.1,285.2,333.7,254.6,331.6,
   *368.,401.2,311.9,362.6,366.9,370.8/
    DATA PF4/3.07,2.96,3.31,3.61,3.75,3.64,1.71,3.67,2.92,2.48,3.36,4.
   *29,4.3,3.92,3.8,4.22,1.91,4.01,4.16,2.90,2.91,4.16,4.04,4.11,3.72,
   *4.26,3.26,2.76,2.61,4.03,2.69,1.6,4.12,4.11,4.46,3.6,3.95,3.13,2.4
   *3,3.94,3.4,4.17,3.43,2.83,2.05,3.68,2.93,2.85/
    DATA XK5/355.3,361.3,340.2,317.9,305.7,315.4,412.6,313.1,363.4,
   *384.2,336.8,234.1,233.2,287.7,300.1,247.2,406.,277.3,256.3,364.16,
   *364.,253.8,274.,264.,308.3,240.6,343.1,371.2,378.2,274.5,374.5,416
   *.4,262.,263.6,318.9,285.1,351.6,386.1,285.2,333.7,254.6,331.6,
   *368.,401.2,311.9,362.6,366.9,370.8/
    DATA PF5/3.07,2.96,3.31,3.61,3.75,3.64,1.71,3.67,2.92,2.48,3.36,4.

   *29,4.3,3.92,3.8,4.22,1.91,4.01,4.16,2.90,2.91,4.16,4.04,4.11,3.72,
   *4.26,3.26,2.76,2.61,4.03,2.69,1.6,4.12,4.11,4.46,3.6,3.95,3.13,2.4
   *3,3.94,3.4,4.17,3.43,2.83,2.05,3.68,2.93,2.85/
     END
```

APPENDIX 3

SURVIVOR CURVES FOR TEXAS PAVEMENTS

This appendix contains the graphical representation of the survivor functions corresponding to all the types of pavements considered for Texas.

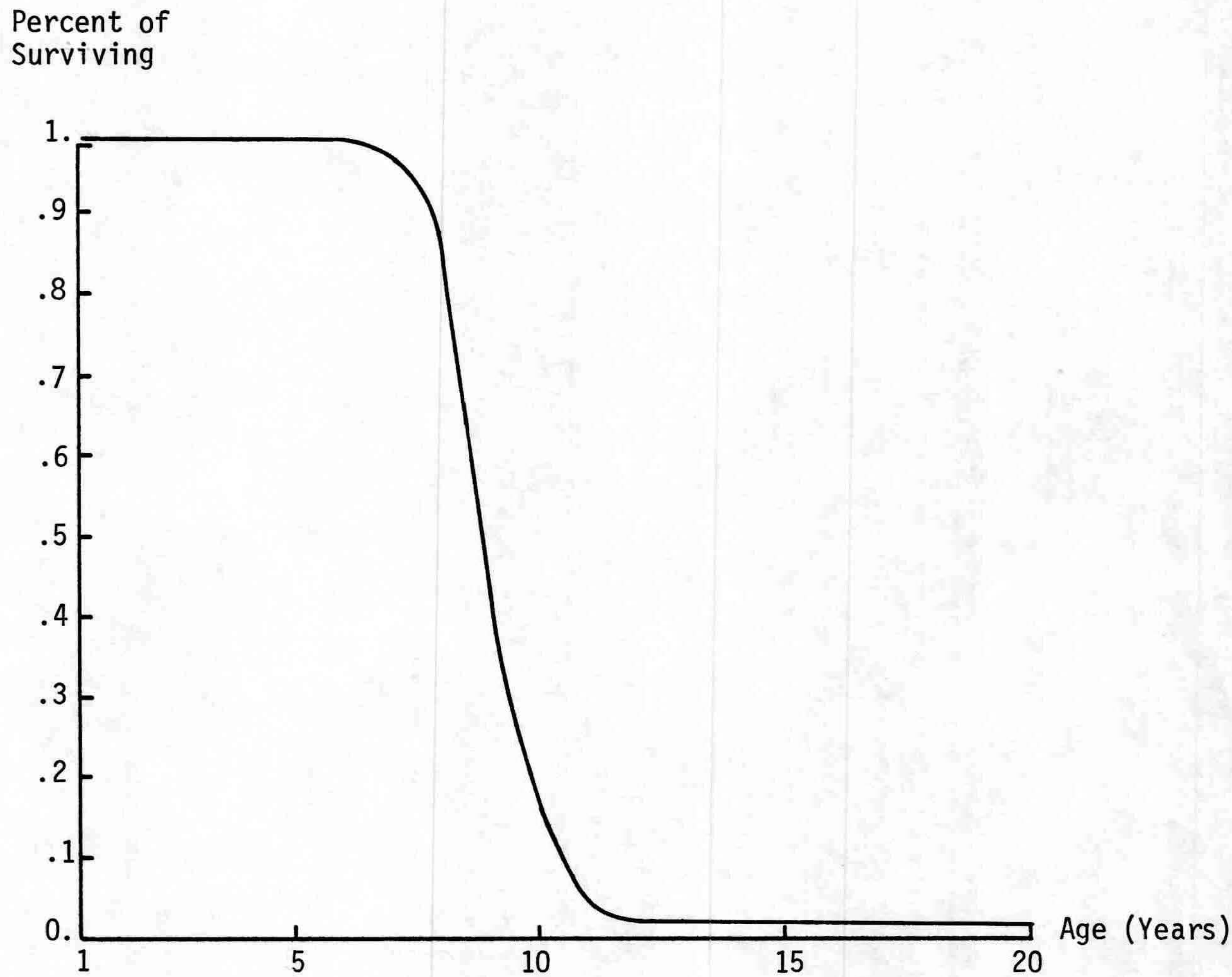

Figure 3A-1. Survivor Curve for Rural-Overlaid-Low Traffic Pavement Using Serviceability Criteria

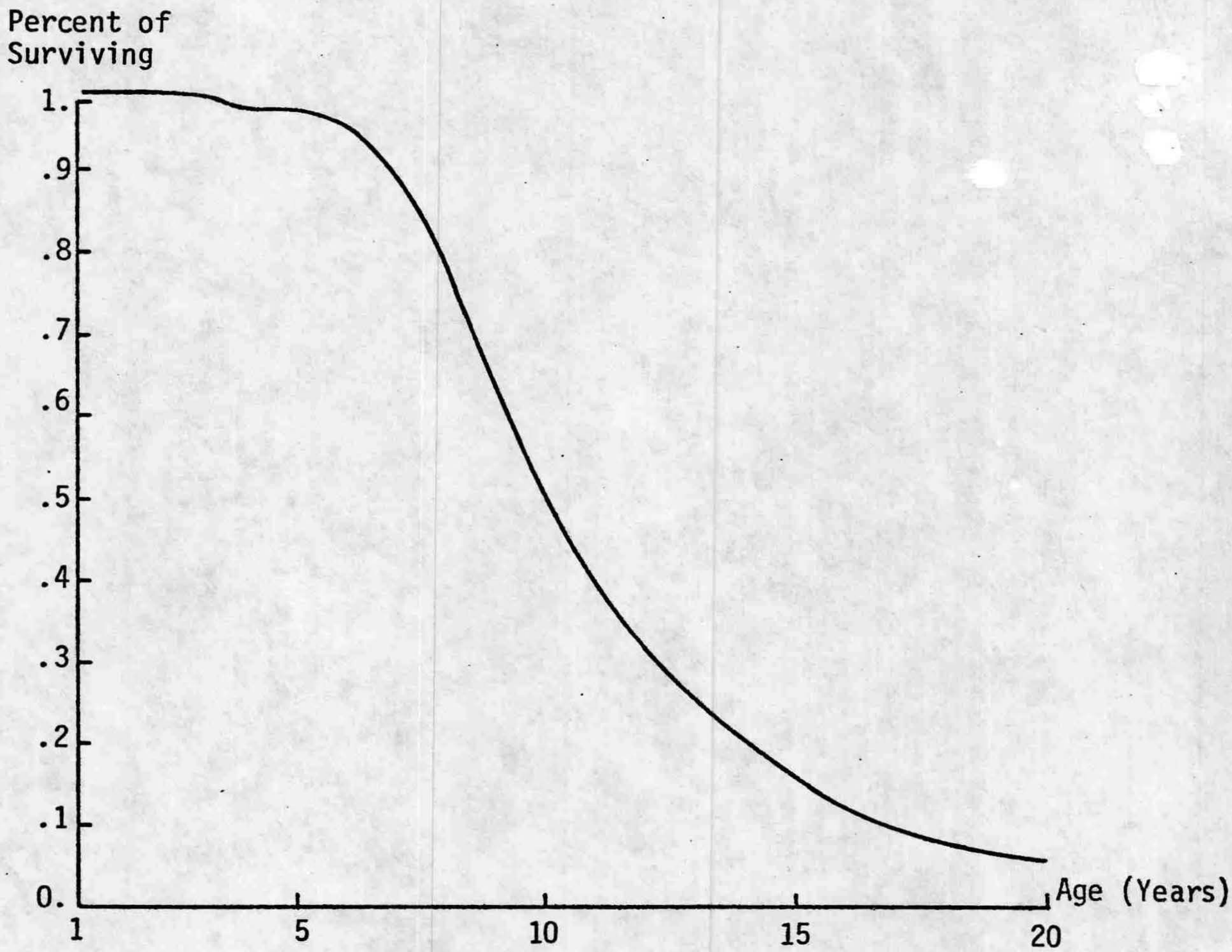

Figure 3A-2. Survivor Curve for Rural-Overlaid-Low Traffic Pavement Having Alligator Cracking (Area) Type of Distress

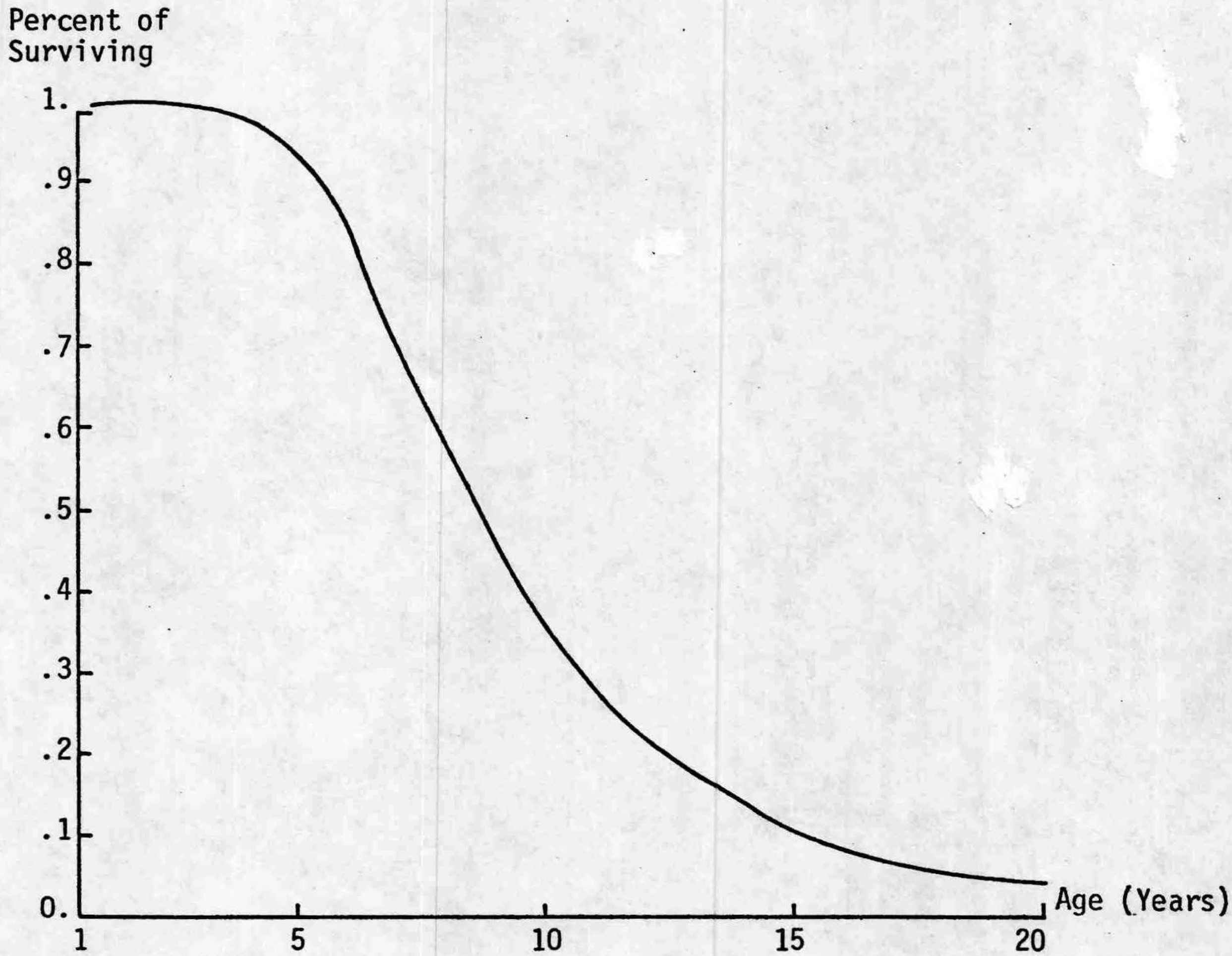

Figure 3A-3. Survivor Curve for Rural-Overlaid-Low Traffic Pavement Having Transversal Cracking (Severity) Type of Distress

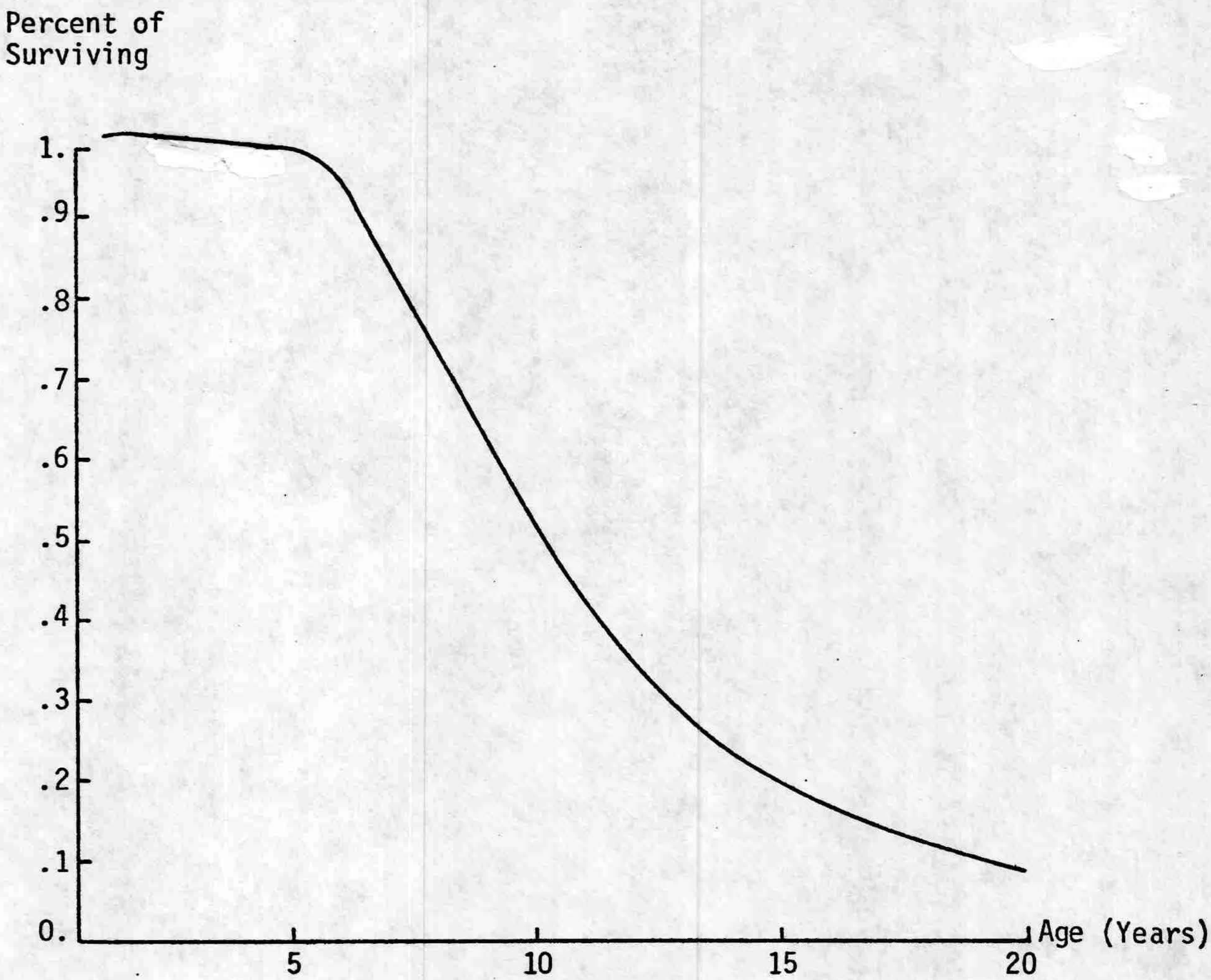

Figure 3A-4. Survivor Curve for Rural-Overlaid-High Traffic Pavement Having Alligator Cracking (Area) Type of Distress

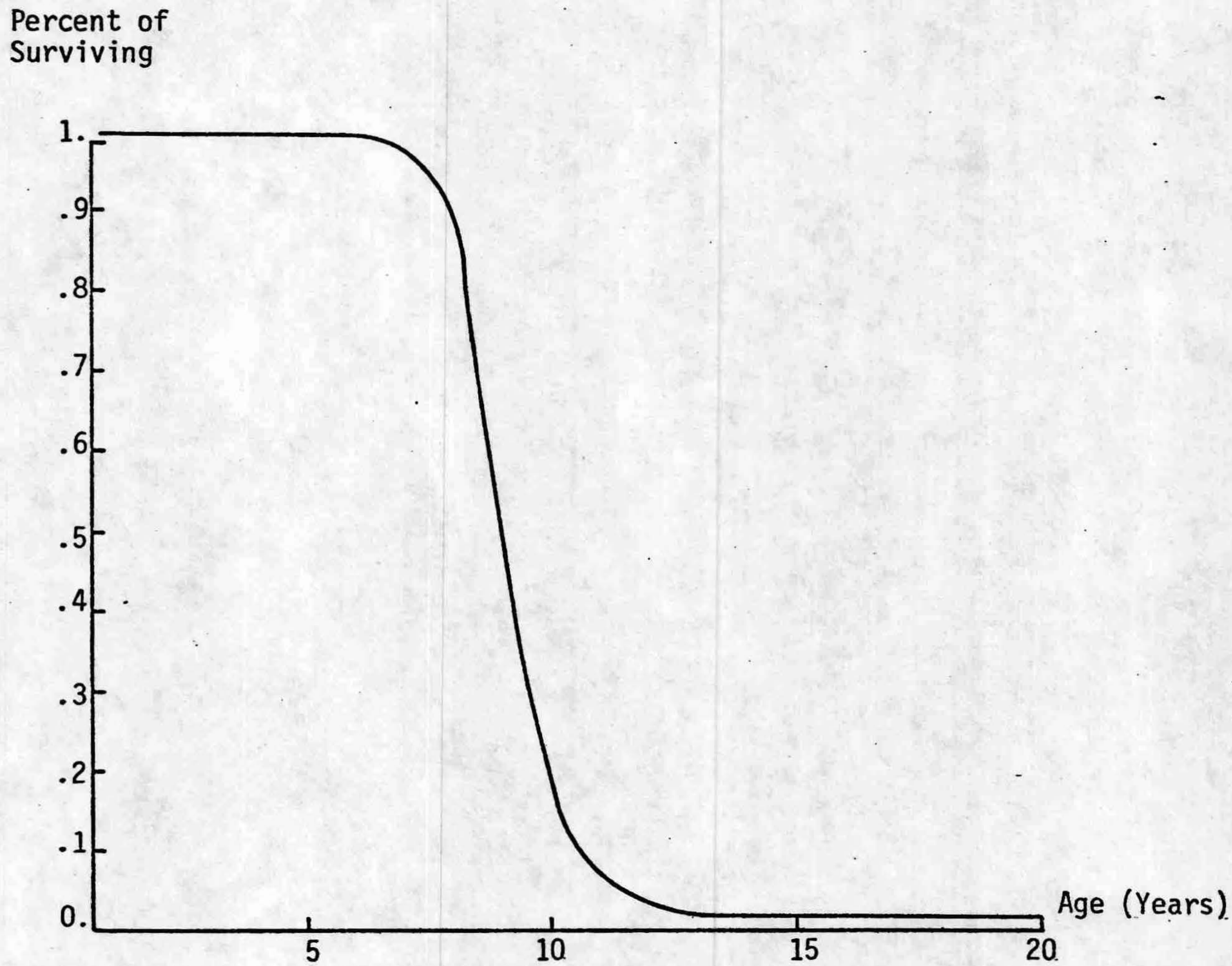

Figure 3A-5. Survivor Curve for Rural-Overlaid-High Traffic Pavement Using Serviceability Criteria

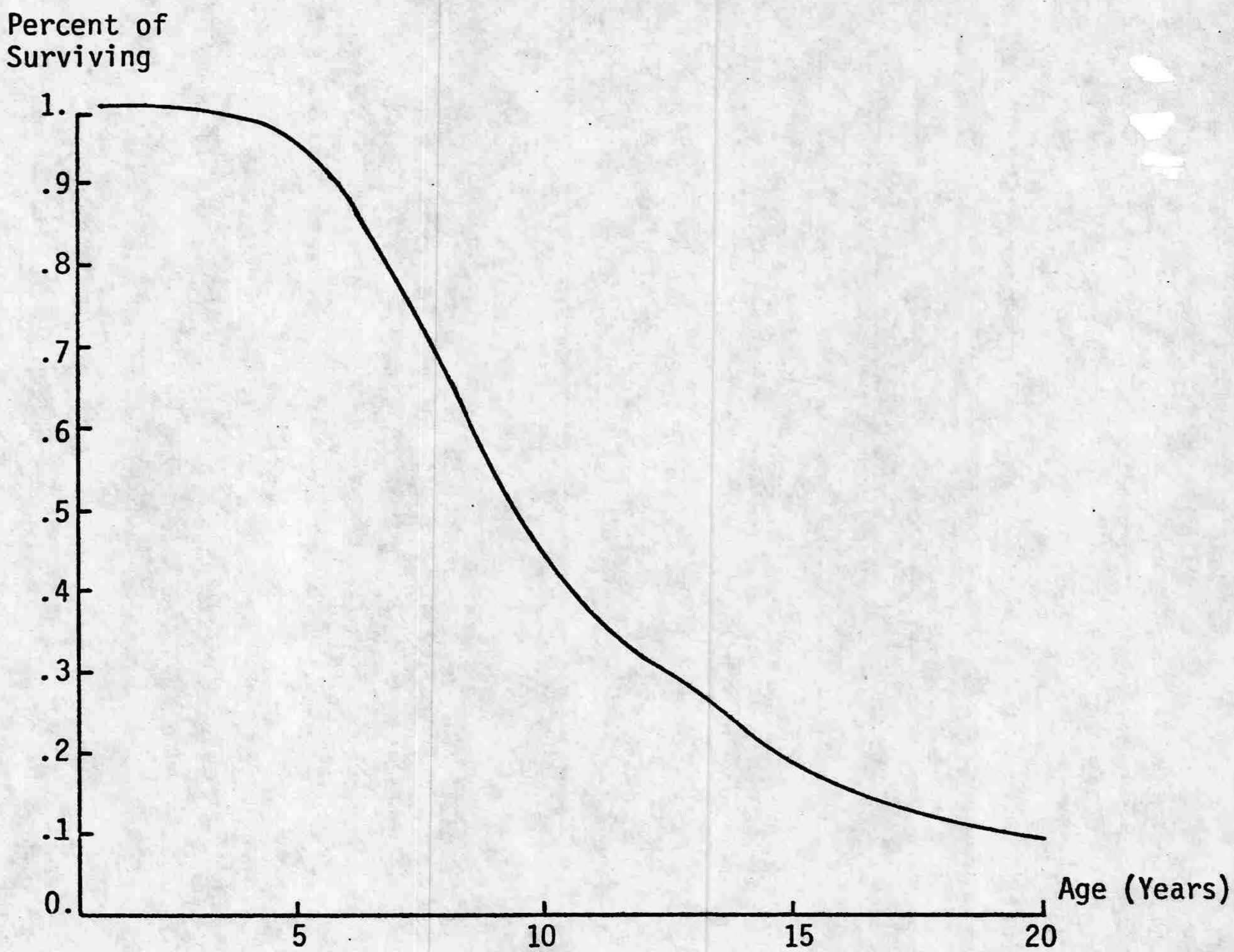

Figure 3A-6. Survivor Curve for Rural-Overlaid-High Traffic Pavement Having Transversal Cracking (Severity) Tyep of Distress

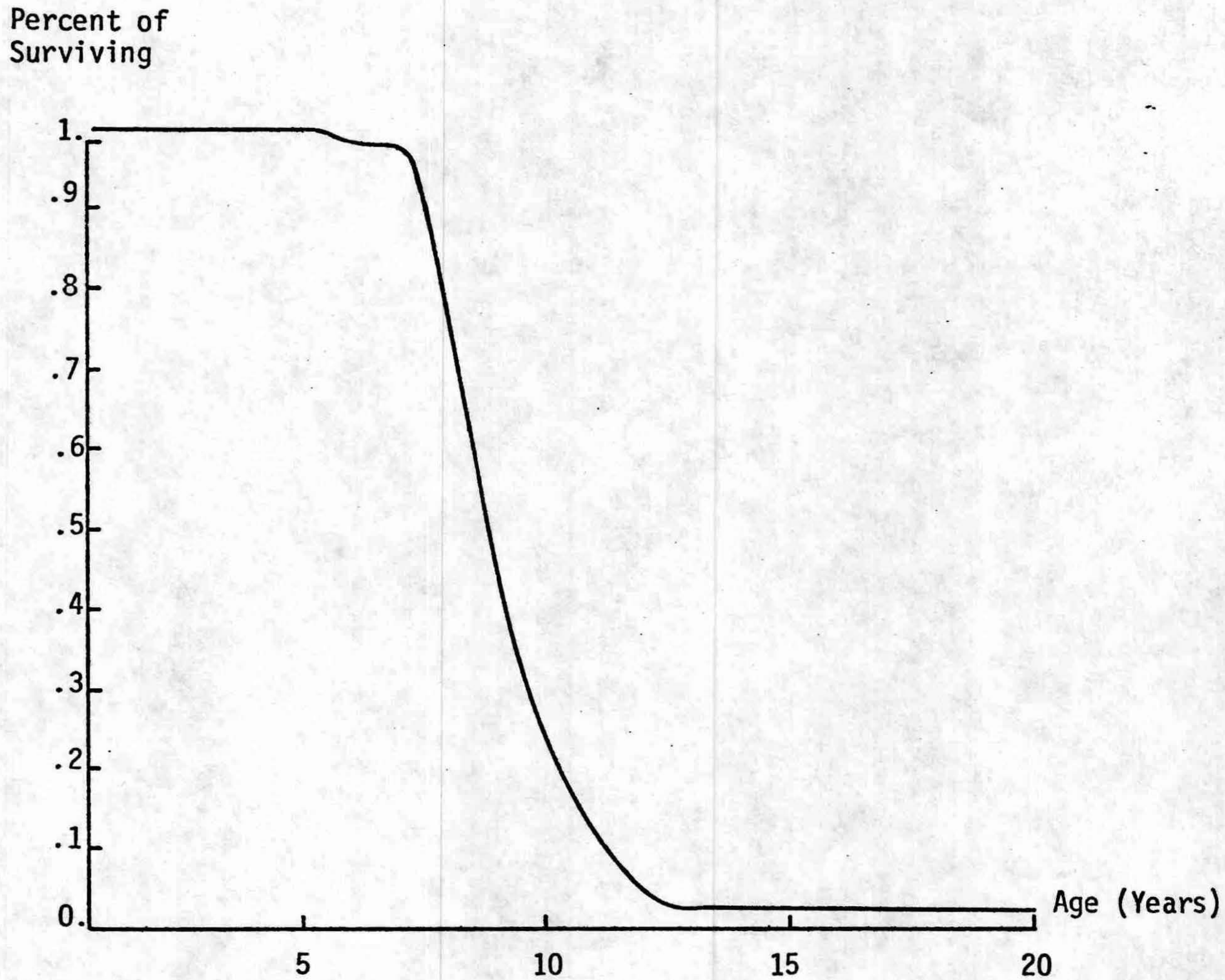

Figure 3A-7. Survivor Curve for Urban-Overlaid-Low Traffic Pavement Using Serviceability Criteria.

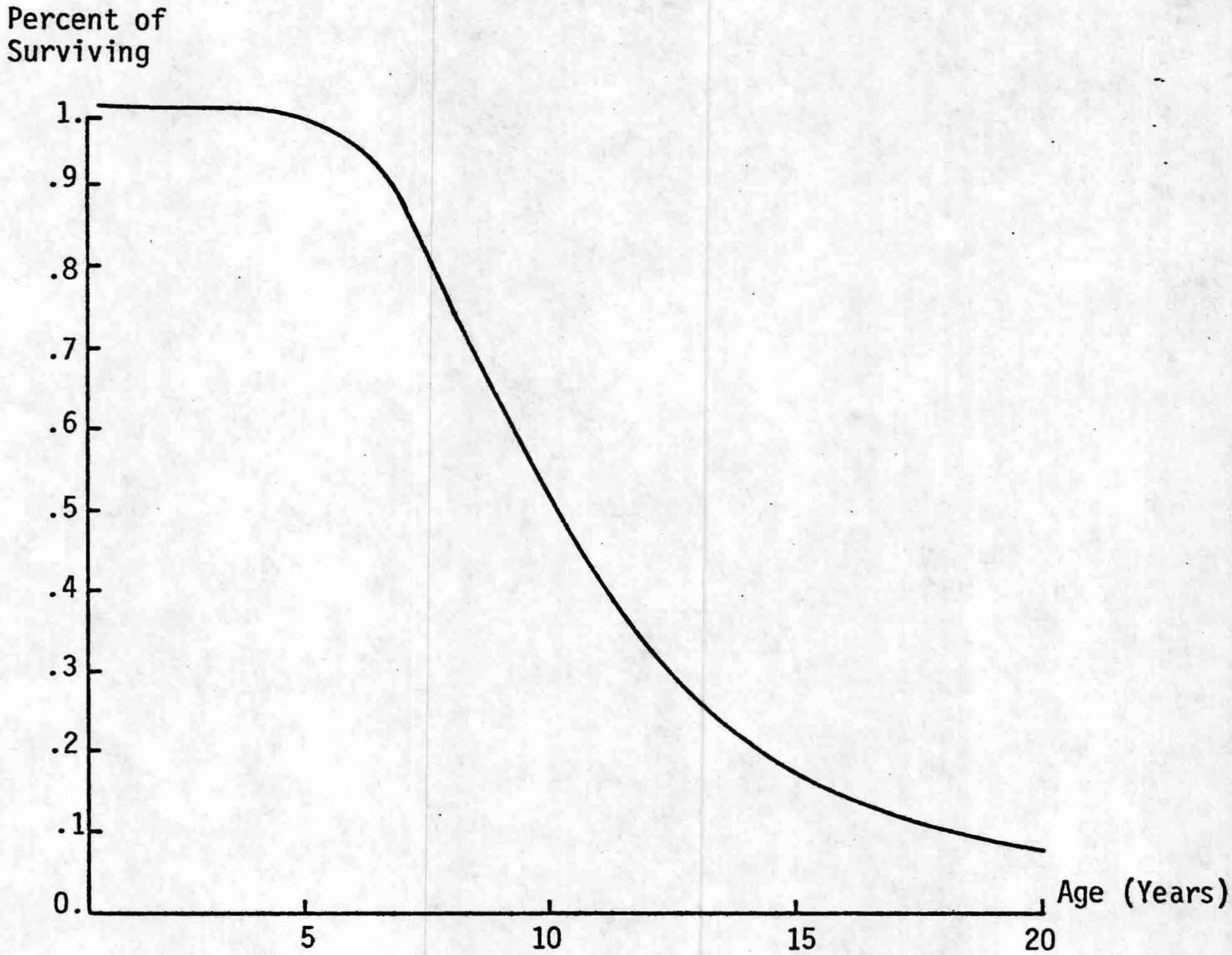

Figure 3A-8. Survivor Curve for Urban-Overlaid-Low Traffic Pavement Having Alligator Cracking (Area) Type of Distress

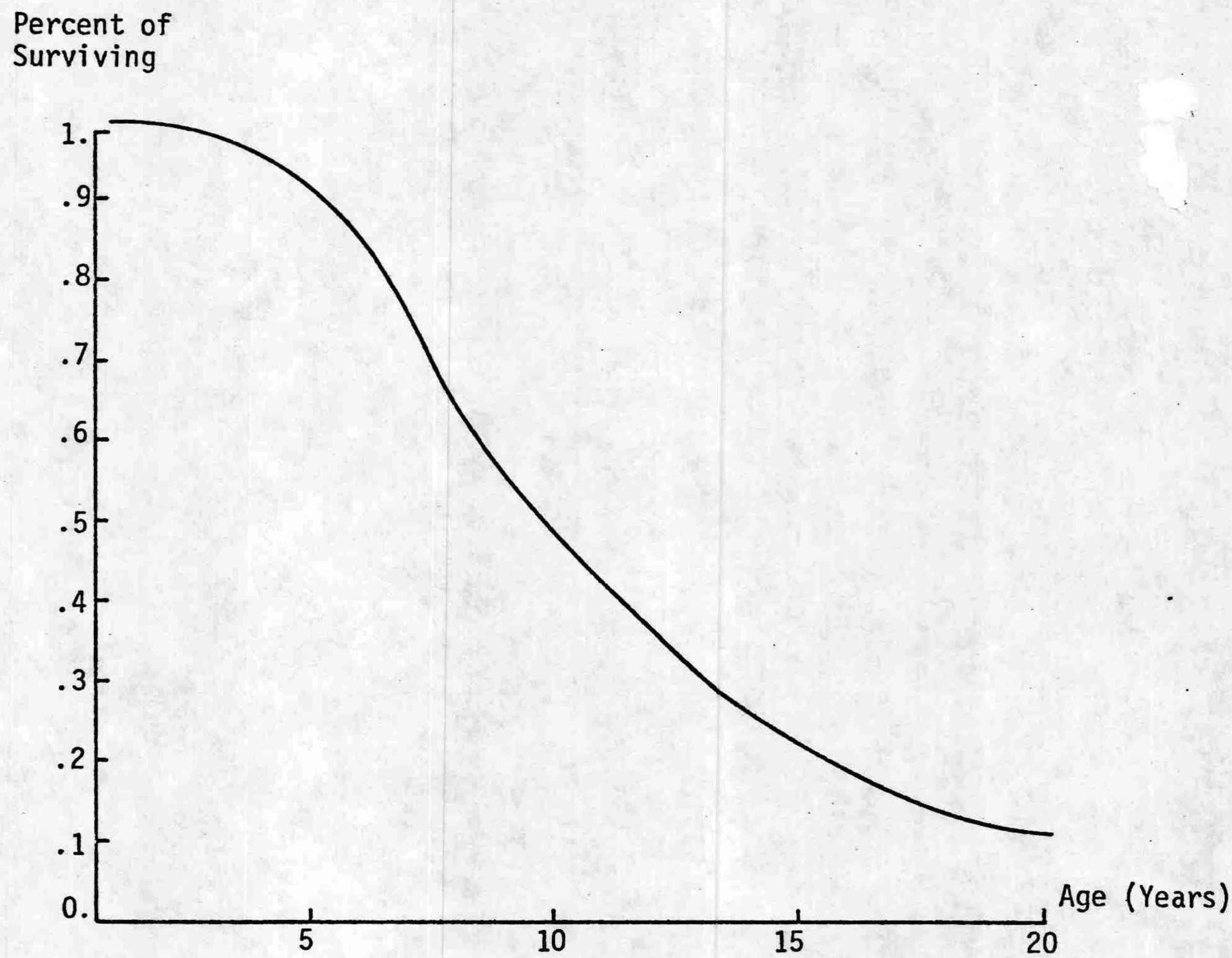

Figure 3A-9. Survivor Curve for Urgan-Overlaid-Low Traffic Pavement Having Transversal Cracking (Severity) Type of Distress

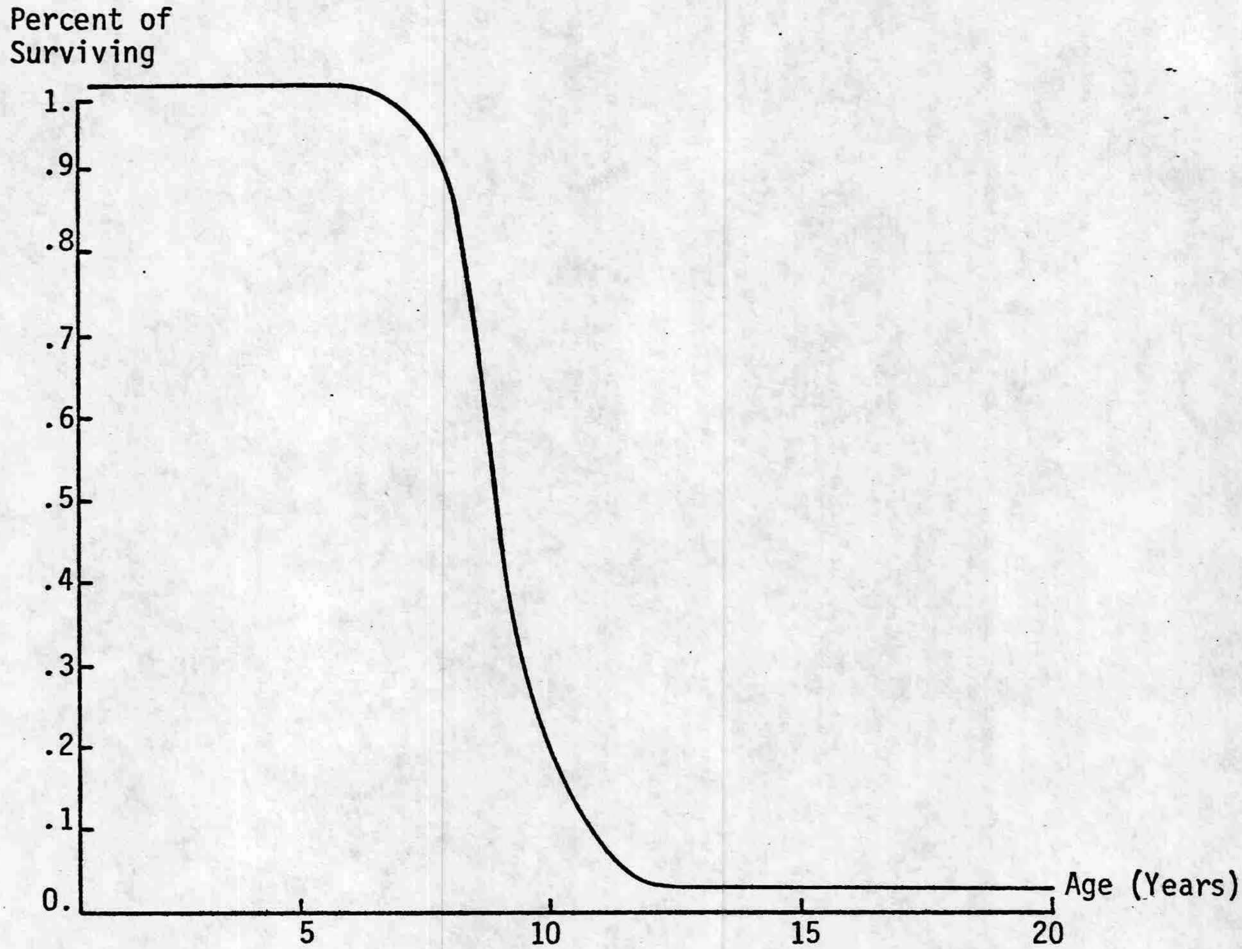

Figure 3A-10. Survivor Curve for Urban-Overlaid-High Traffic Pavement Using Serviceability Criteria

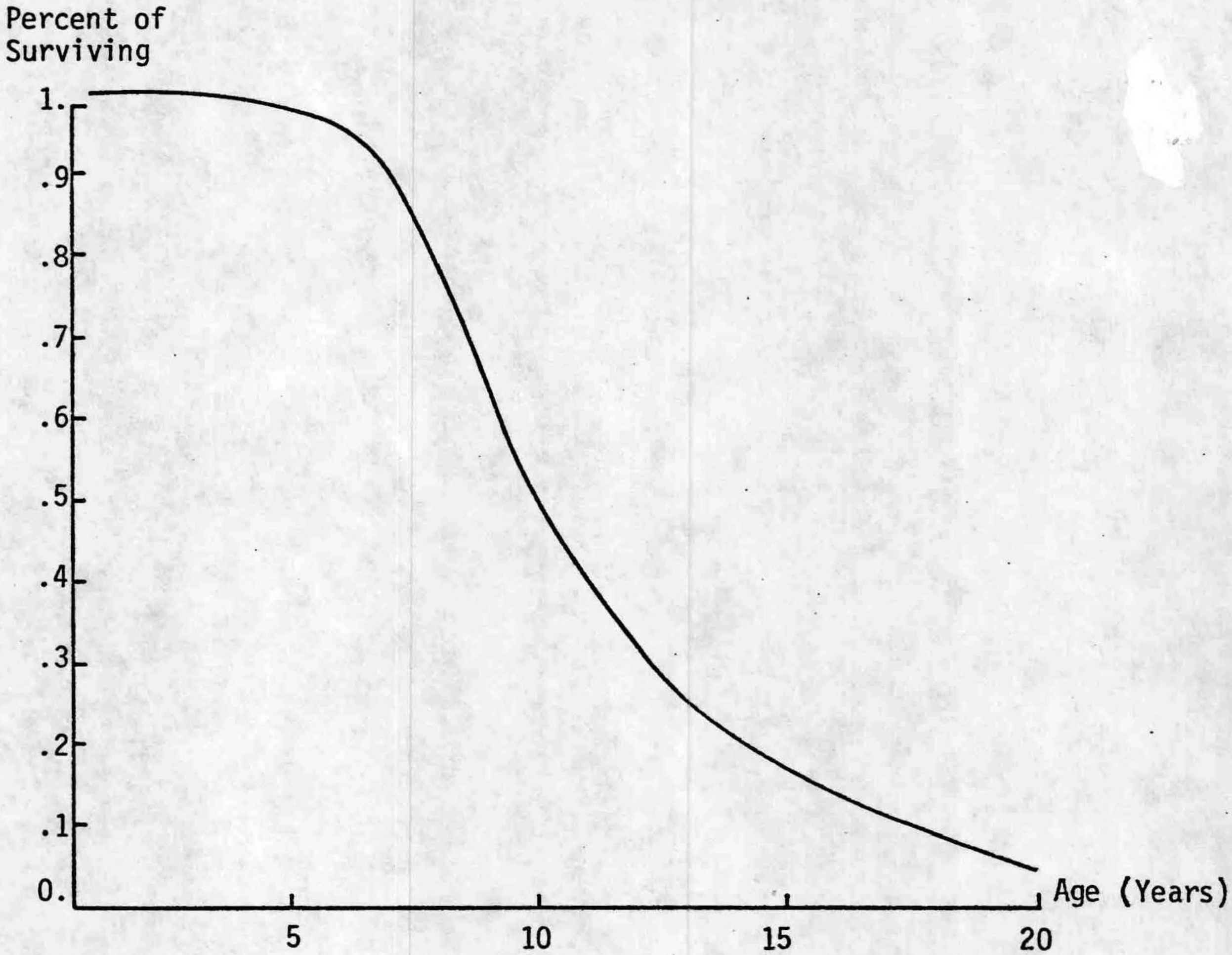

Figure 3A-11. Survivor Curve for Urban-Overlaid-High Traffic Pavement Having Alligator Cracking (Area) Type of Distress

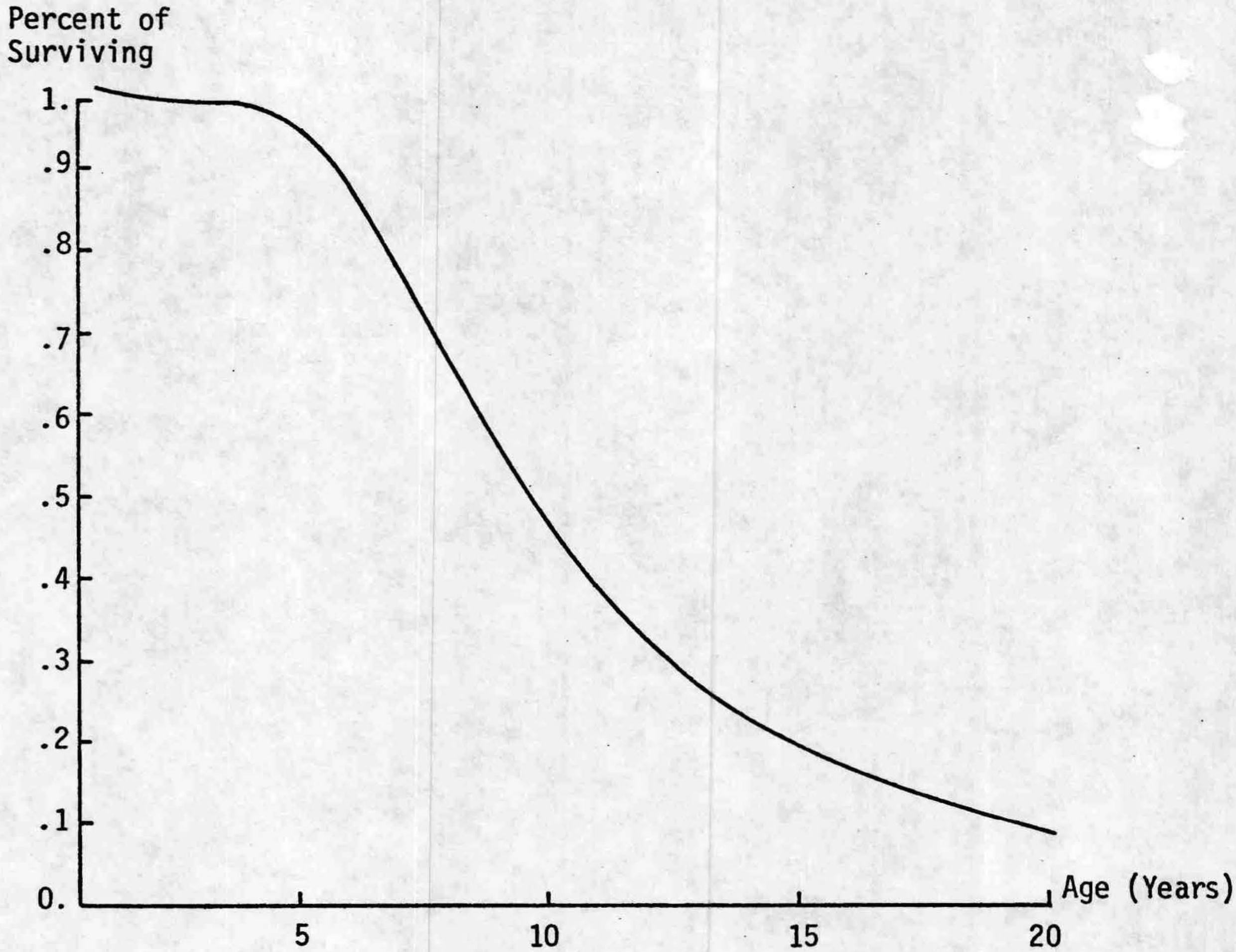

Figure 3A-12. Survivor Curve for Urban-Overlaid-High Traffic Pavement Having Transversal Cracking (Severity) Type of Distress

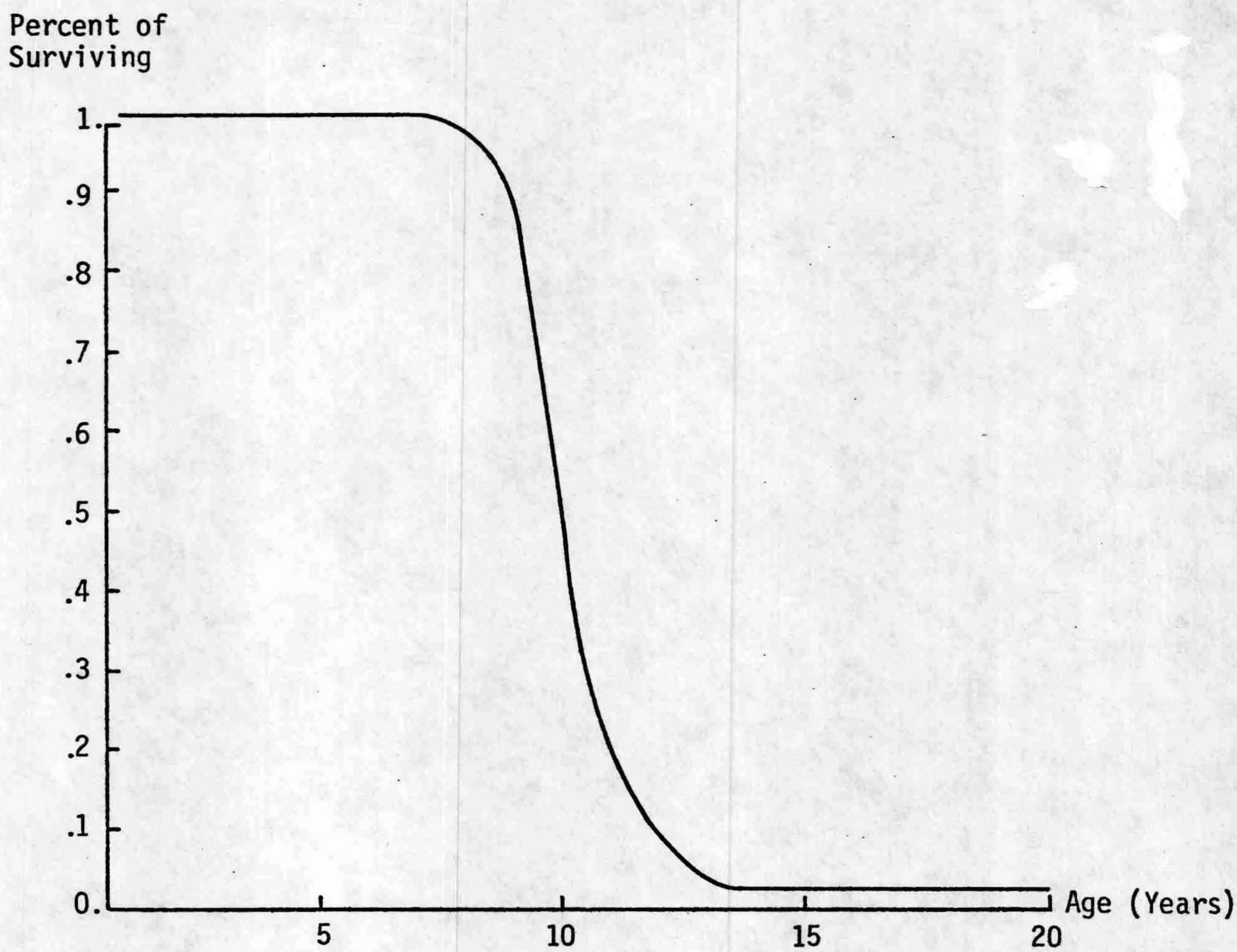

Figure 3A-13. Survivor Curve for Rural-Hot Mix-Low Traffic Pavement Using Serviceability Criteria.

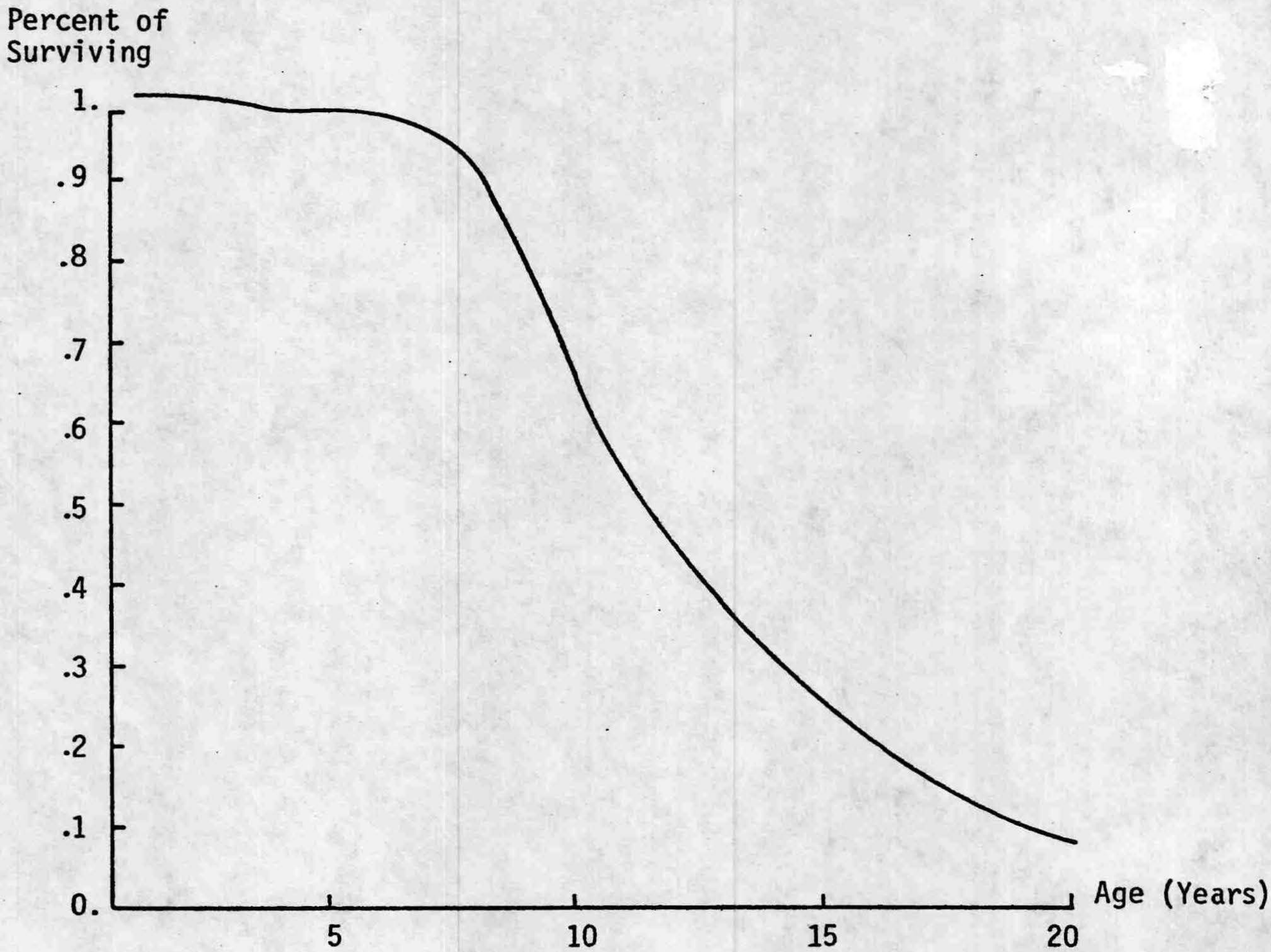

Figure 3A-14. Survivor Curve for Rural-Hot Mix-Low Traffic Pavement Having Alligator Cracking (Area) Type of Distress

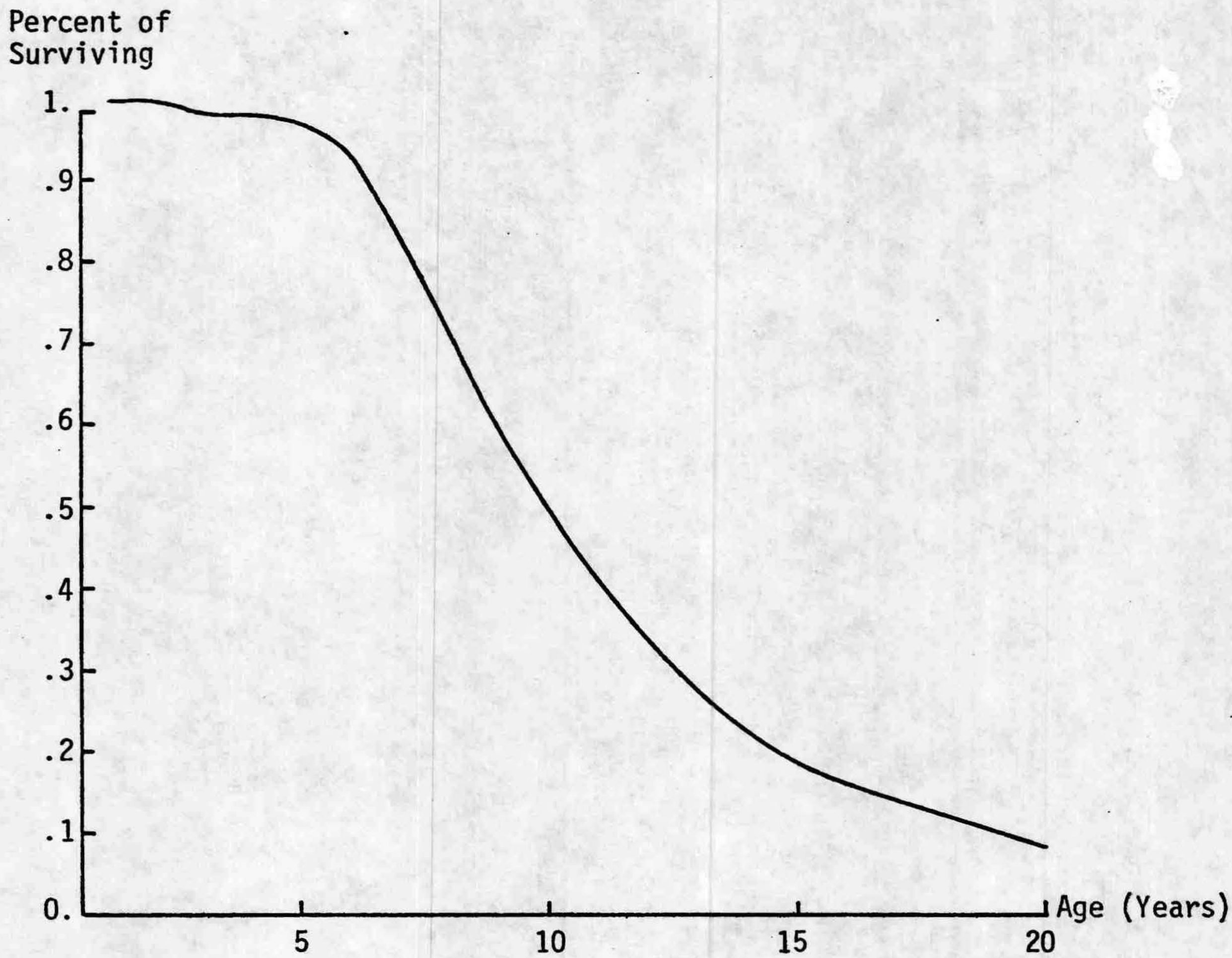

Figure 3A-15. Survivor Curve for Rural-Hot Mix-Low Traffic Pavement Having Transversal Cracking (Severity) Type of Distress

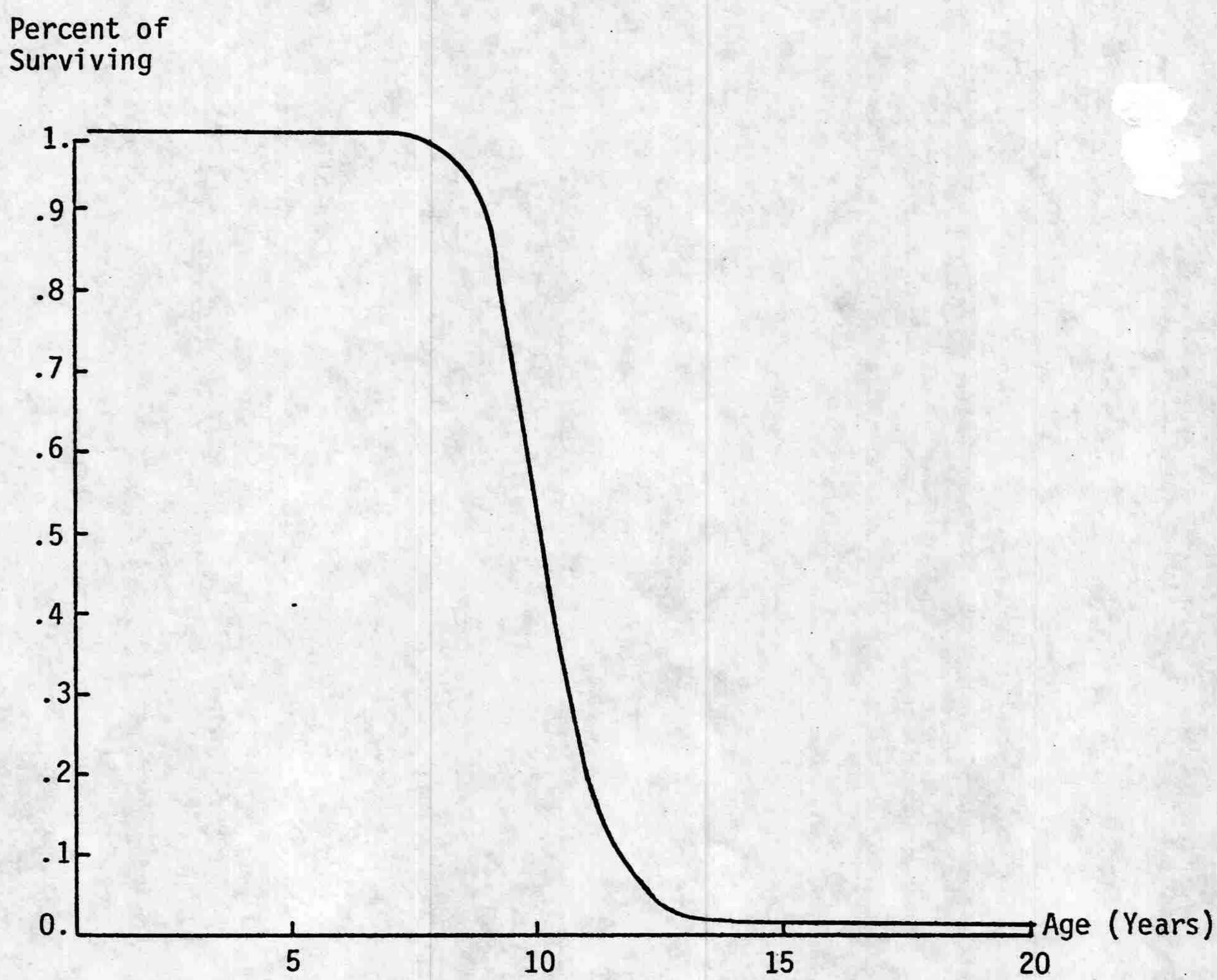

Figure 3A-16. Survivor Curve for Rural-Hot Mix-High Traffic Pavement Using Serviceability Criteria

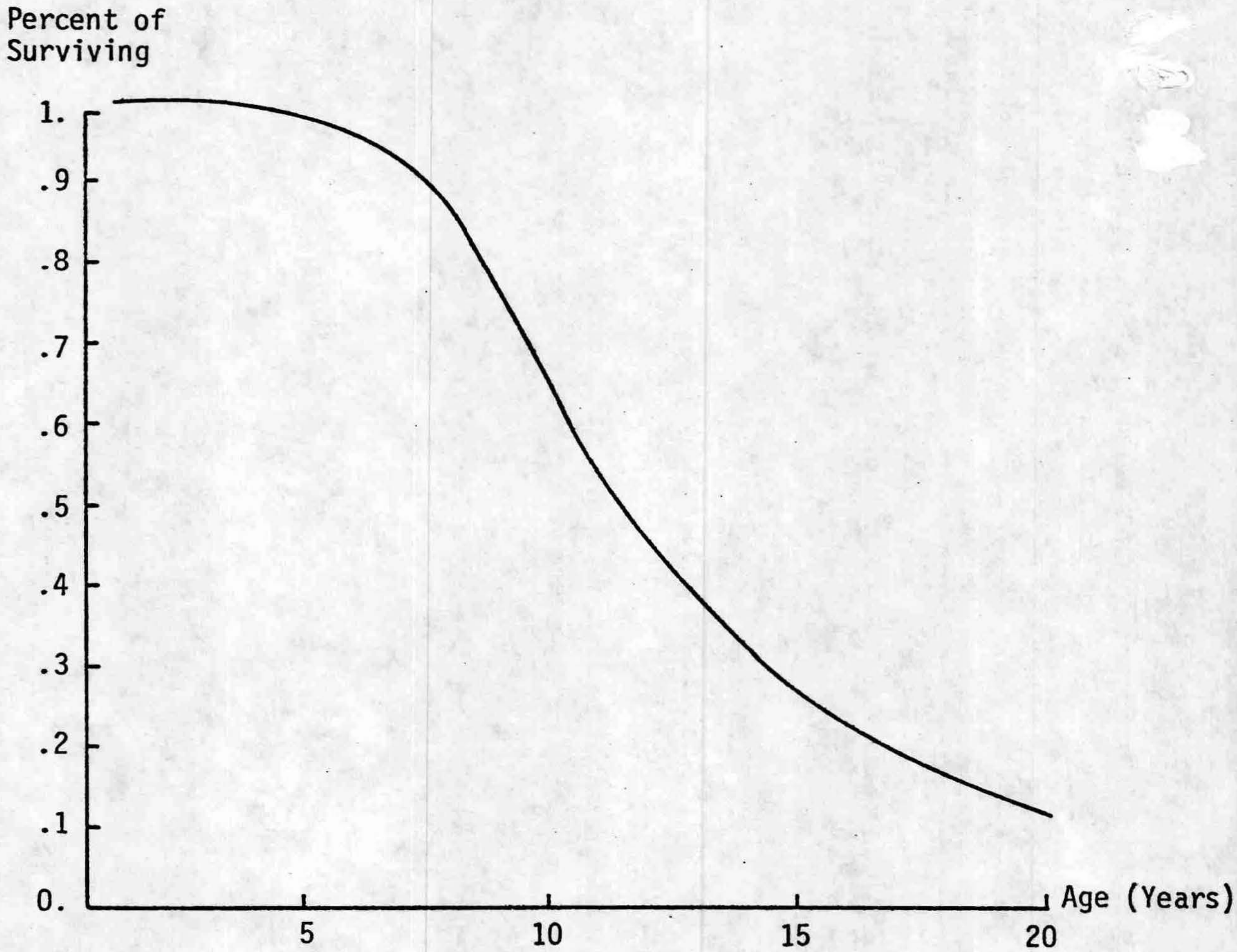

Figure 3A-17. Survivor Curve for Rural-Hot Mix-High Traffic Pavement Having Alligator Cracking (Area) Type of Distress

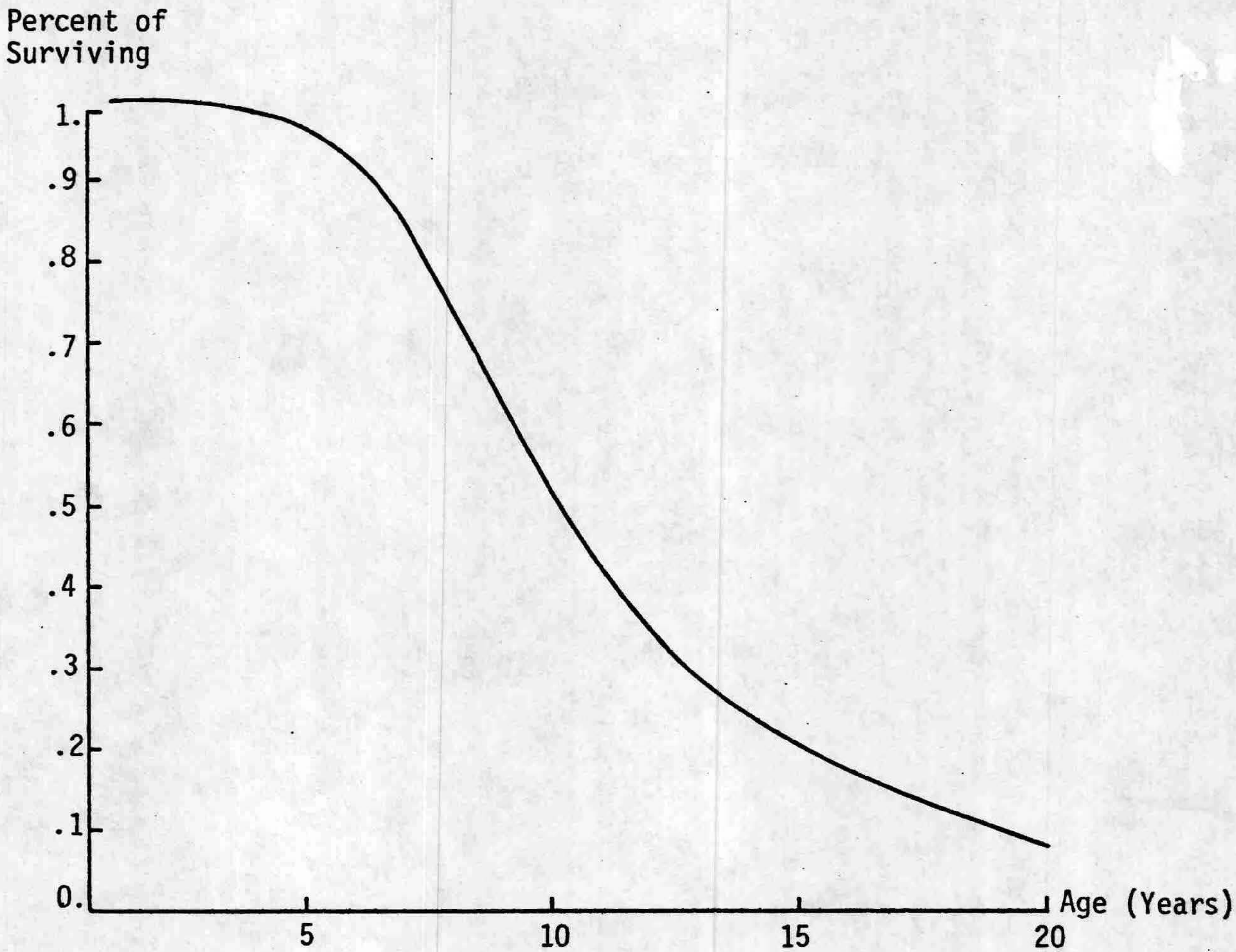

Figure 3A-18. Survivor Curve for Rural-Hot Mix-High Traffic Pavement Having Transversal Cracking (Severity) Type of Distress

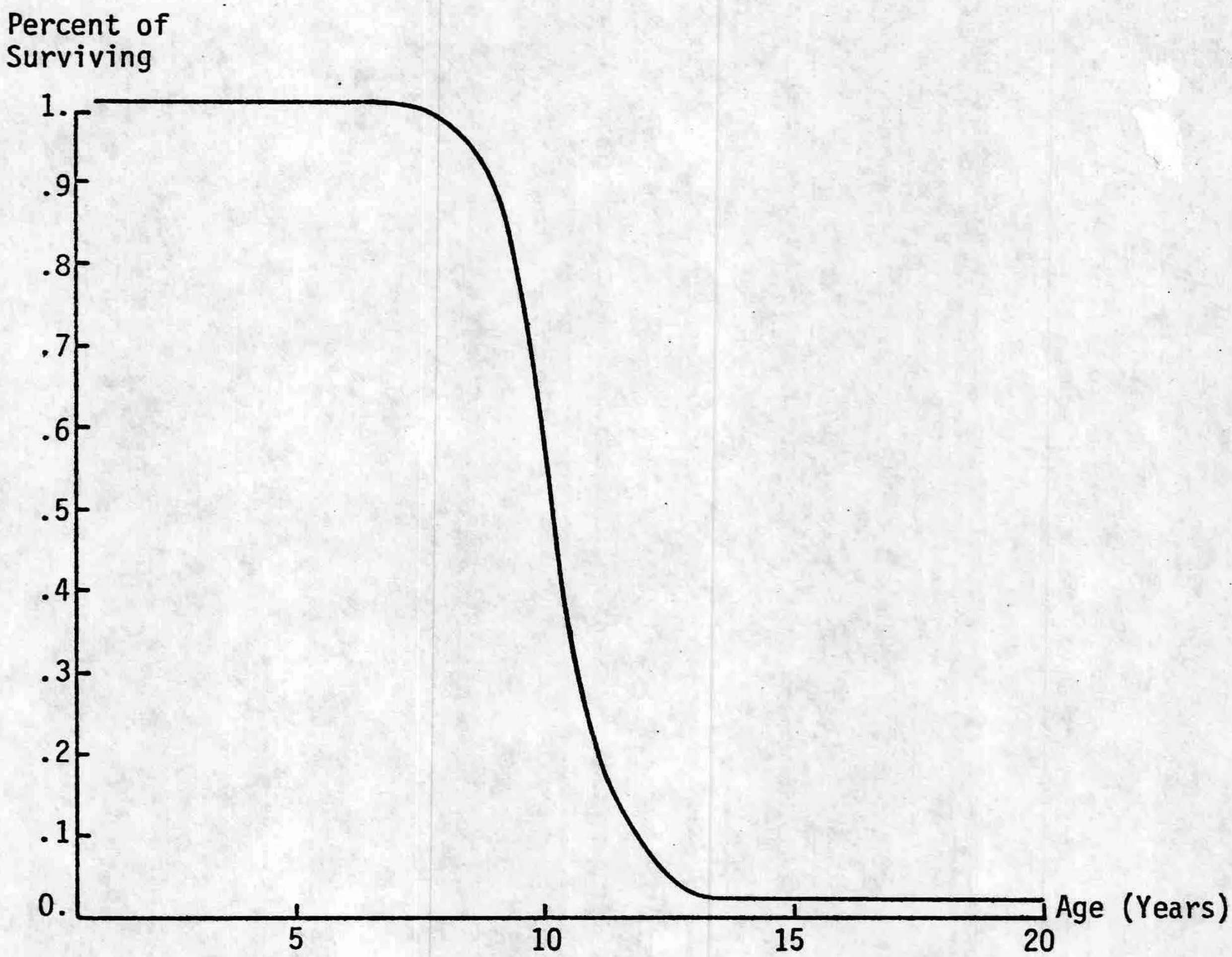

Figure 3A-19. Survivor Curve for Urban-Hot Mix Low Traffic Pavement Using Serviceability Criteria

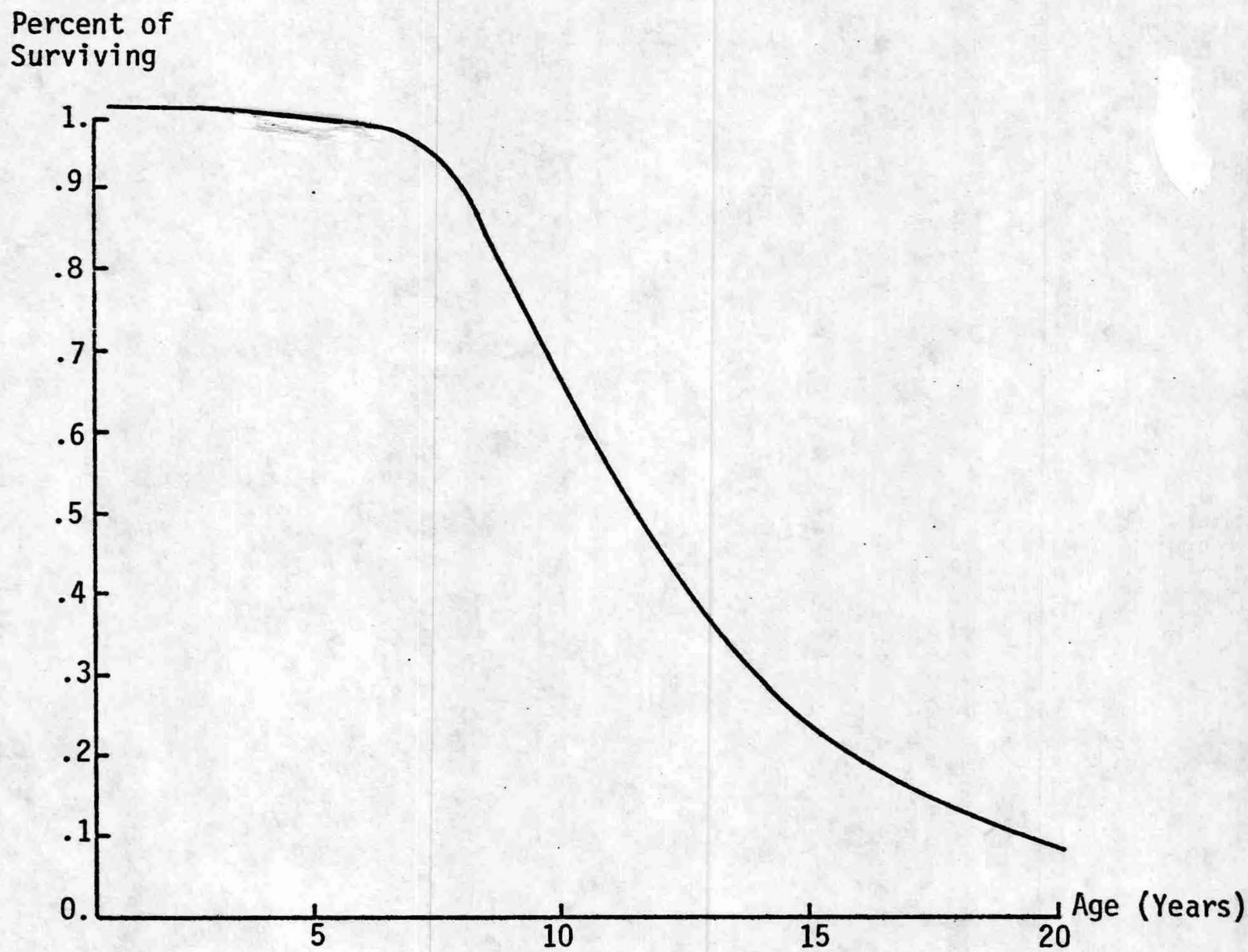

Figure 3A-20. Survivor Curve for Urban-Hot Mix-Low Traffic Pavement Having Alligator Cracking (Area) Type of Distress

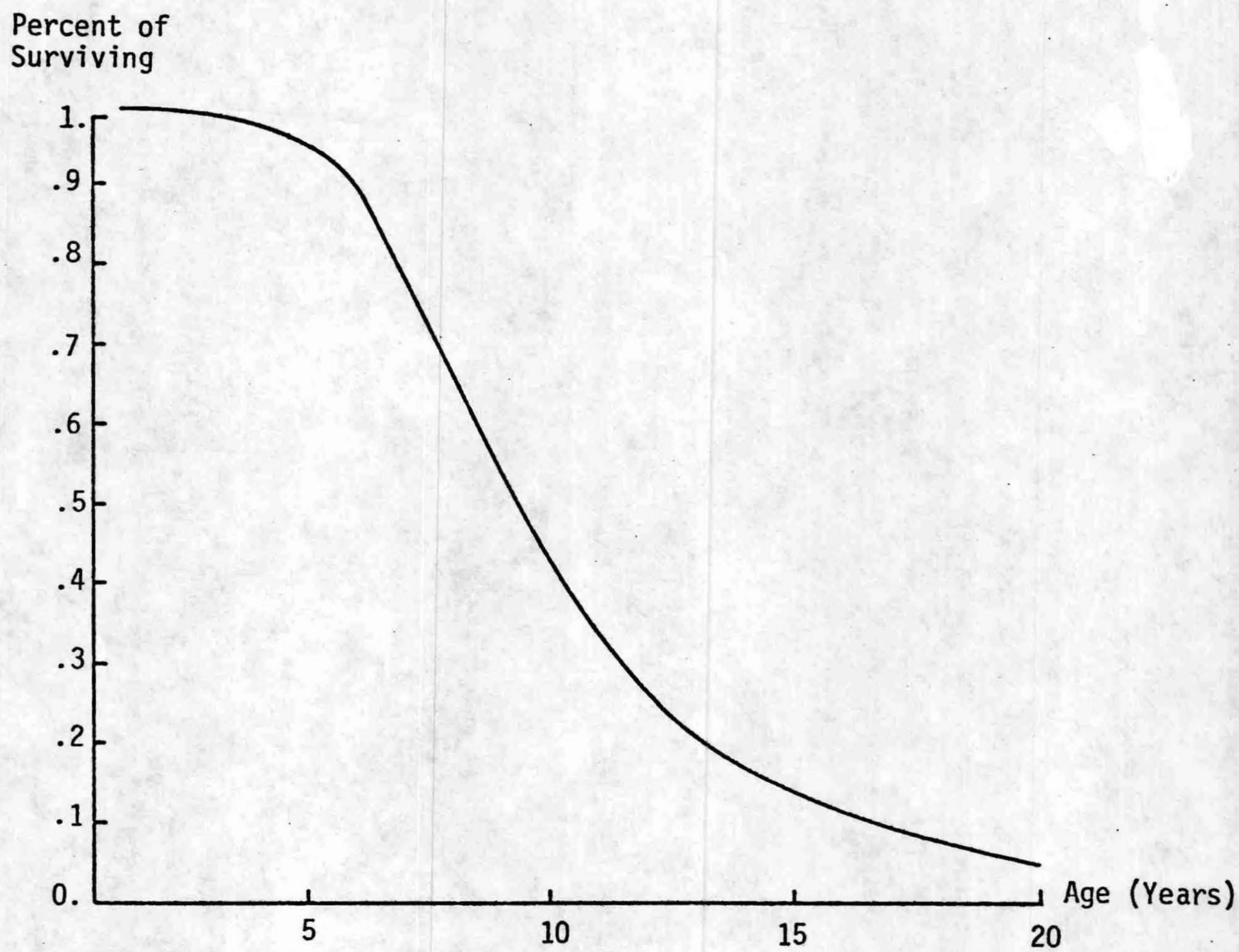

Figure 3A-21. Survivor Curve for Urban-Hot Mix Low Traffic Pavement Having Transversal Cracking (Severity) Type of Distress

Figure 3A-22. Survivor Curve for Urban-Hot Mix High Traffic Pavement Using Serviceability Criteria

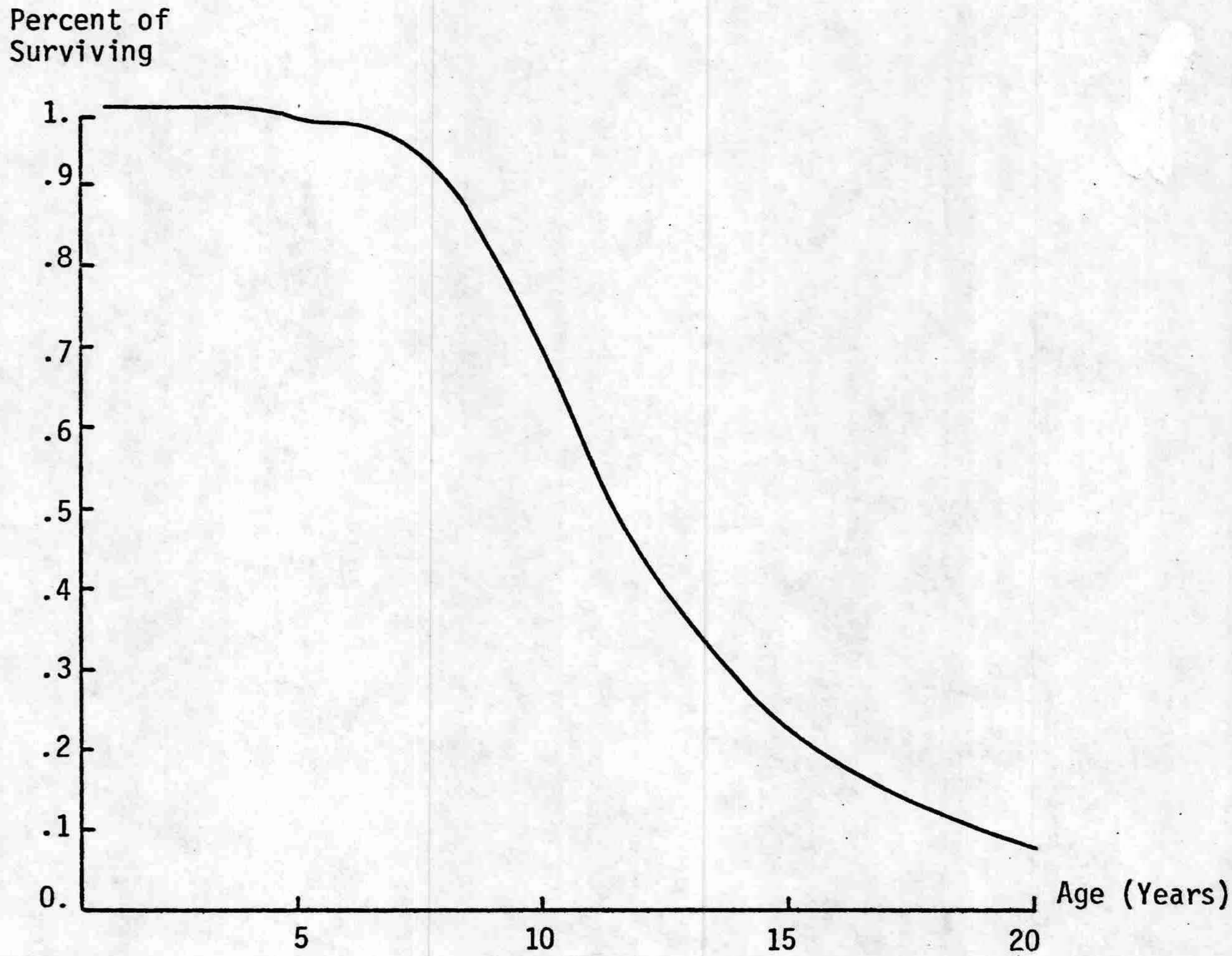

Figure 3A-23. Survivor Curve for Urban-Hot Mix-High Traffic Pavement Having Alligator Cracking (Area) Type of Distress

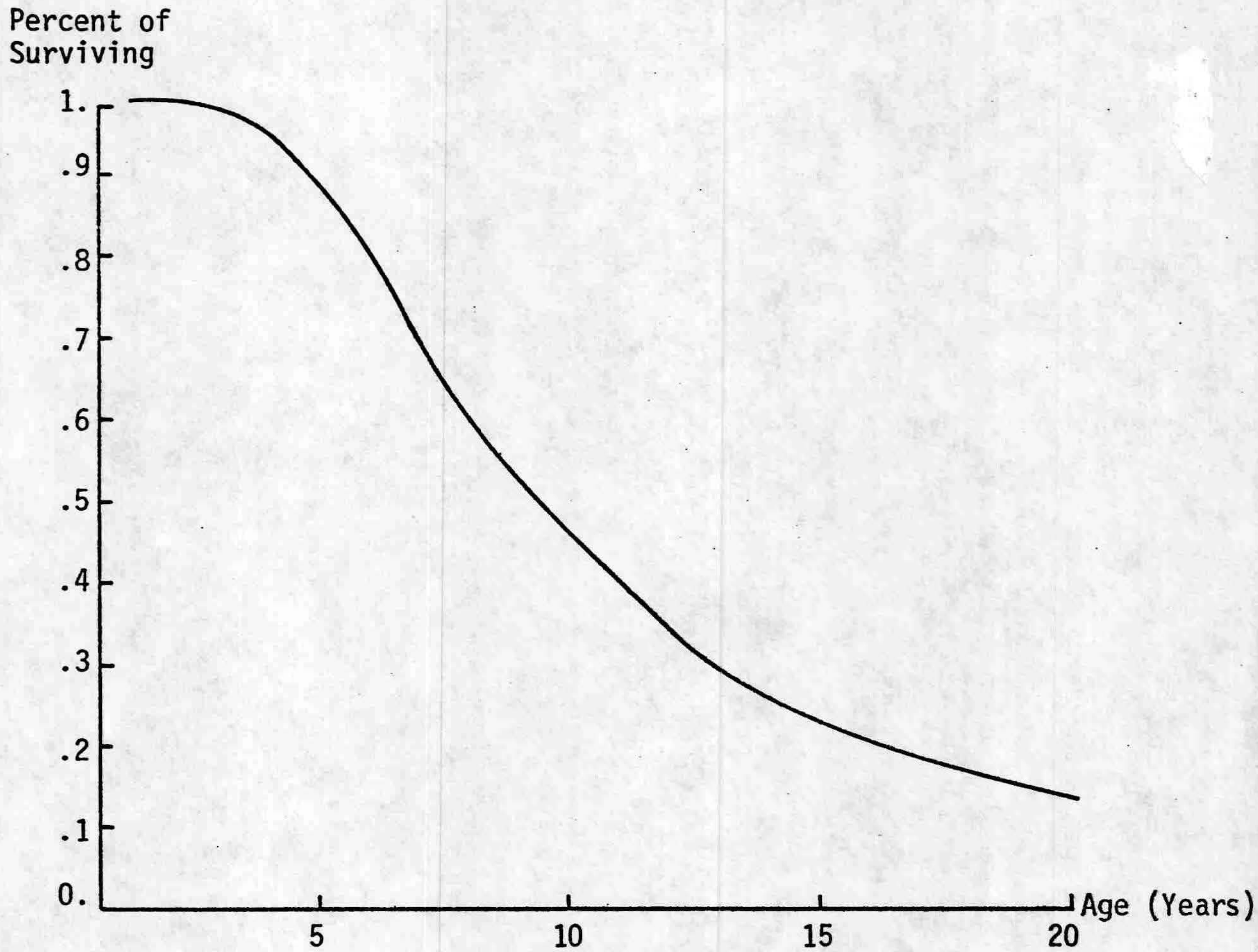

Figure 3A-24. Survivor Curve for Urgan-Hot Mix-High Traffic Pavement Having Transversal Cracking (Severity) Type of Distress

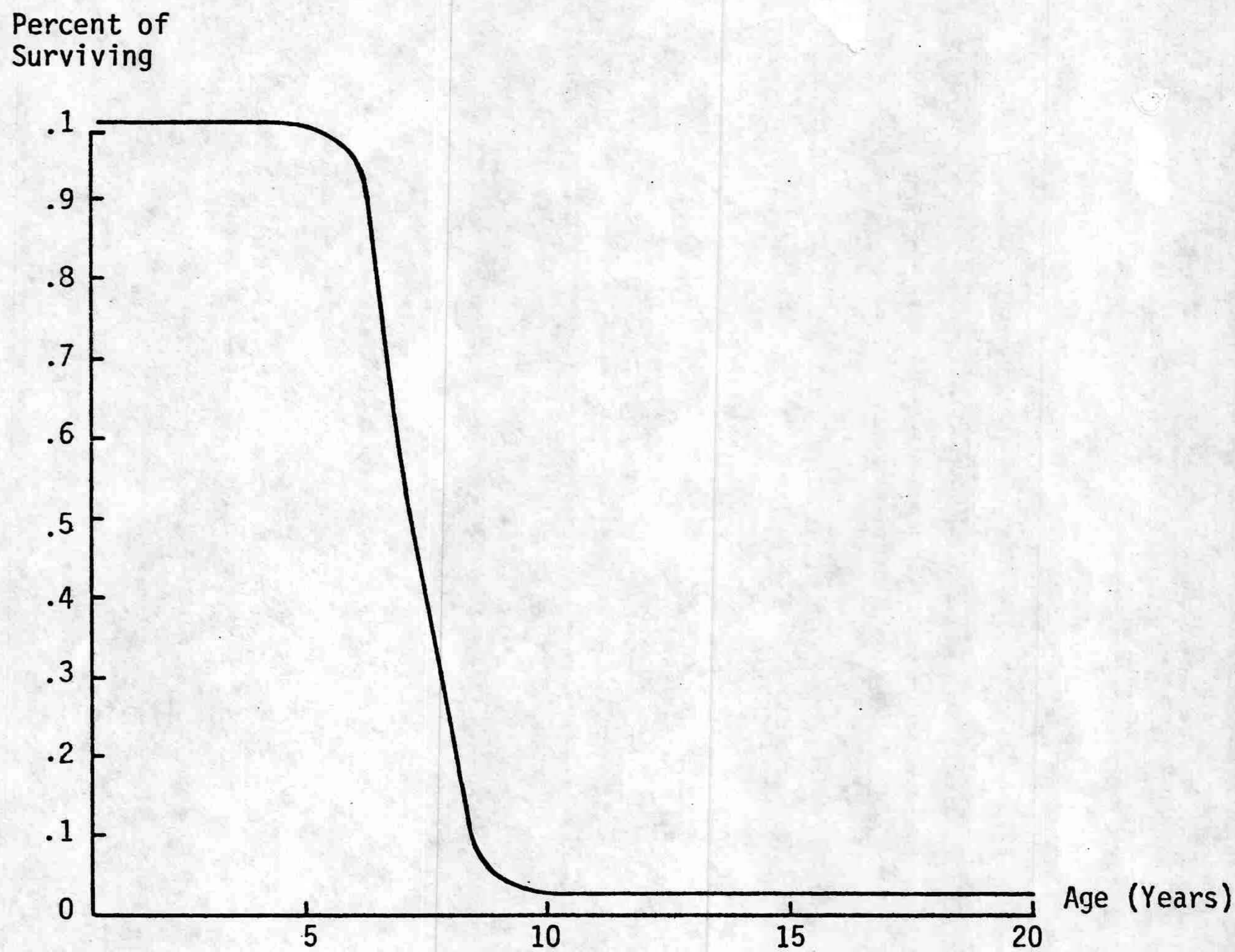

Figure 3A-25. Survivor curve for Rural-Surface Treated Pavement using Serviceability criteria

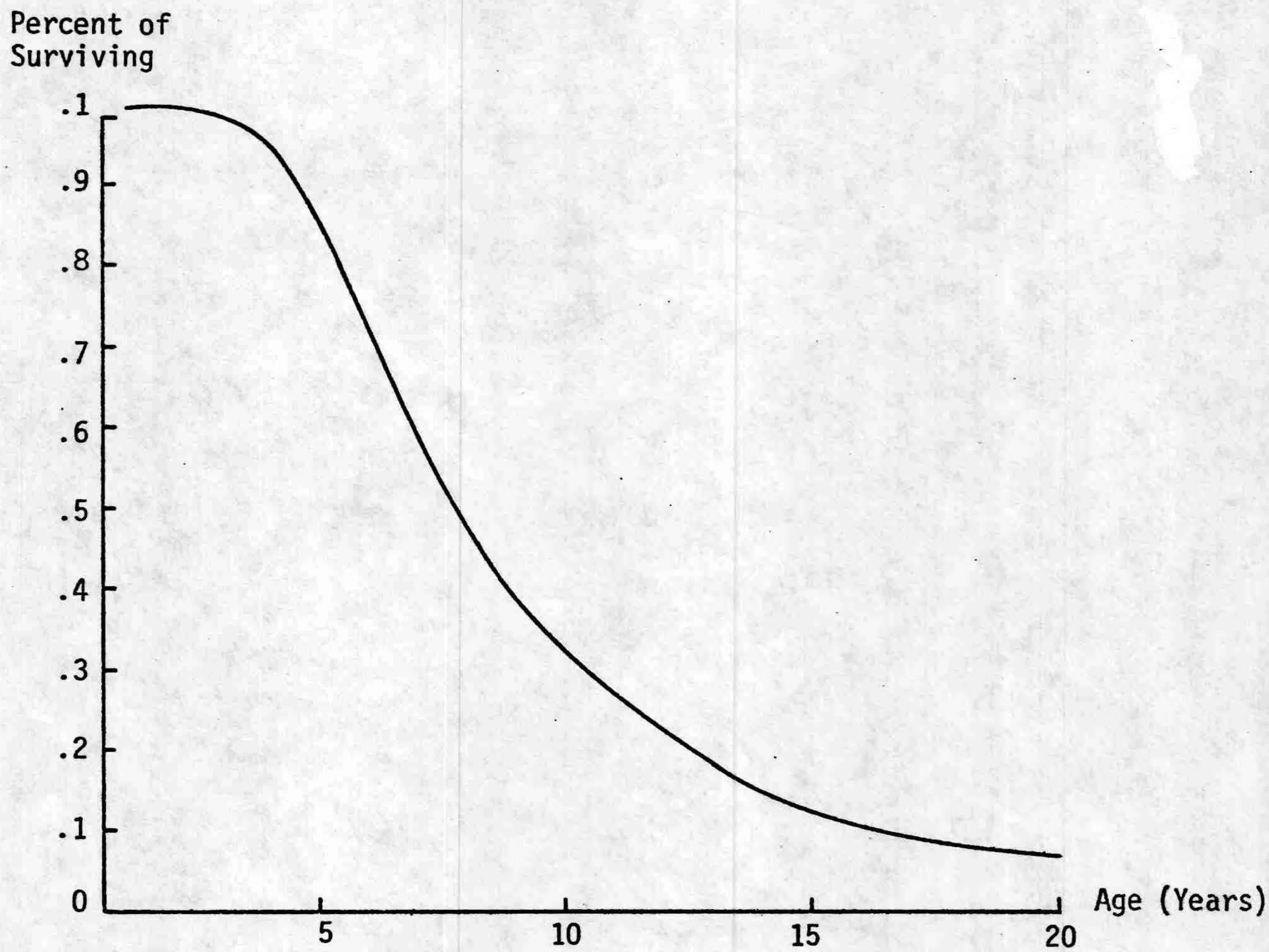

Figure 3A-26. Survivor curve for Rural-Surface Treated Pavement Having Alligator Cracking (Area) type of Distress

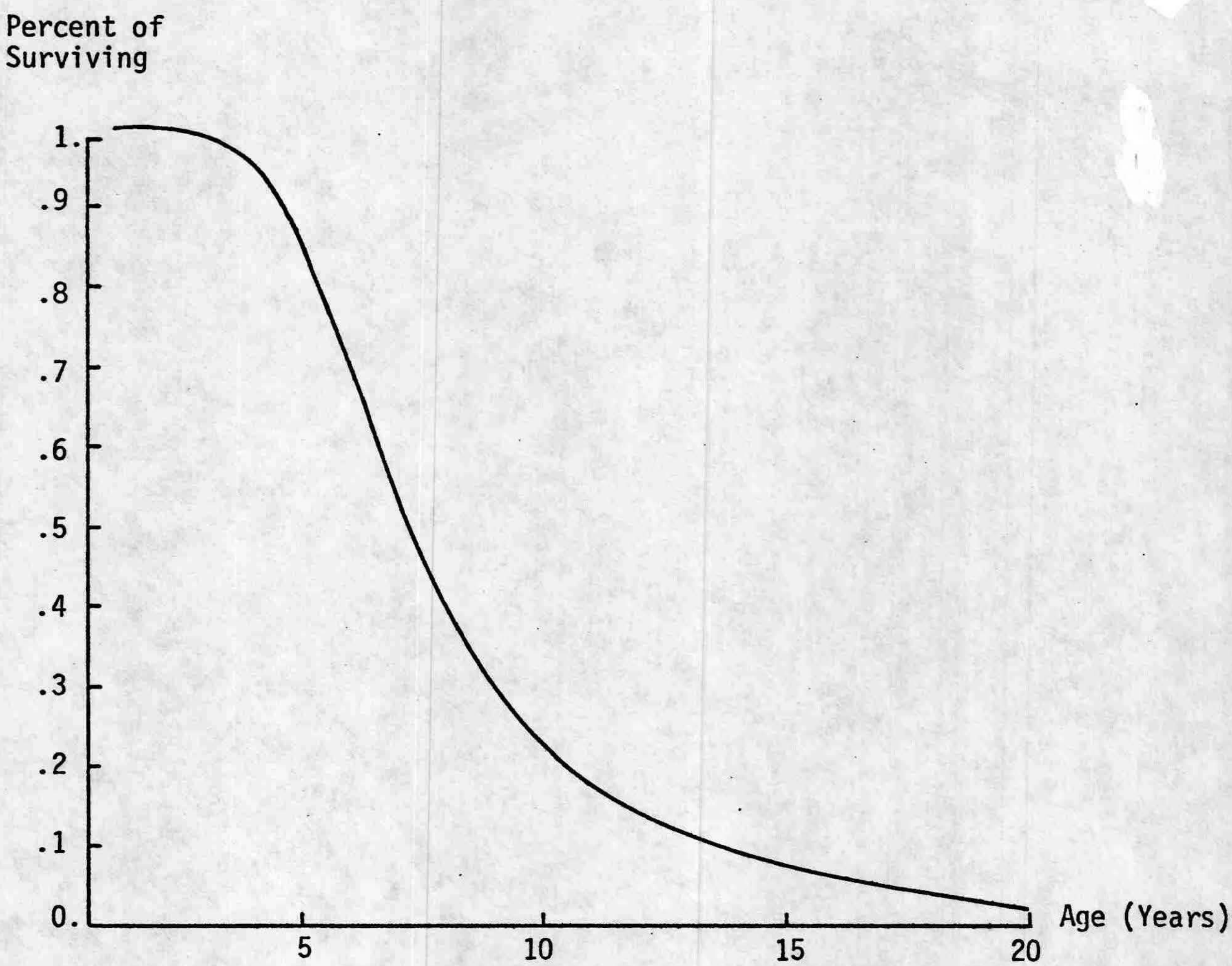

Figure 3A-27. Survivor Curve for Rural-Surface Treated Pavement Having Transversal Cracking (Severity) Type of Distress

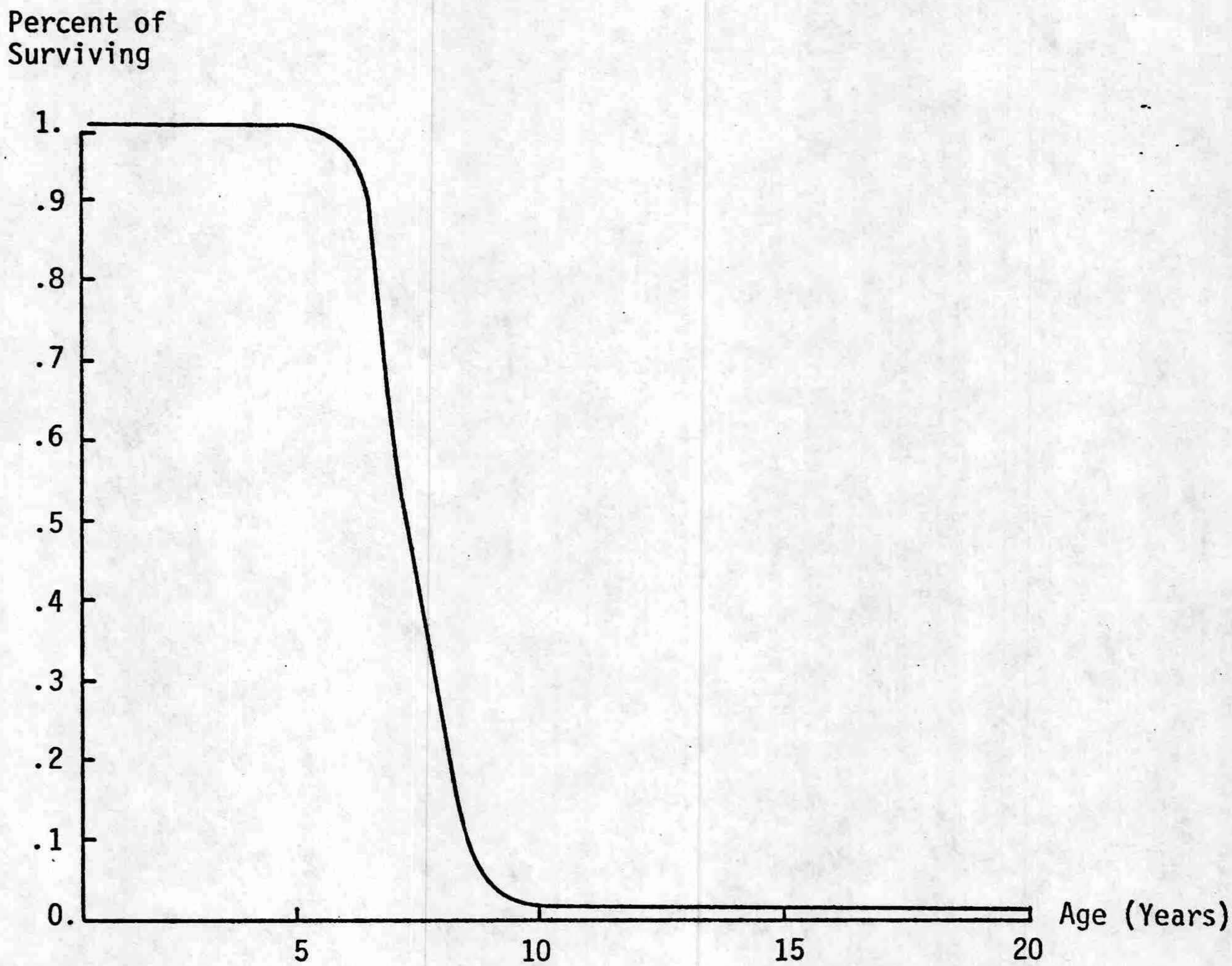

Figure 3A-28. Survivor Curve for Urban-Surface Treated Pavement using Serviceability Criteria

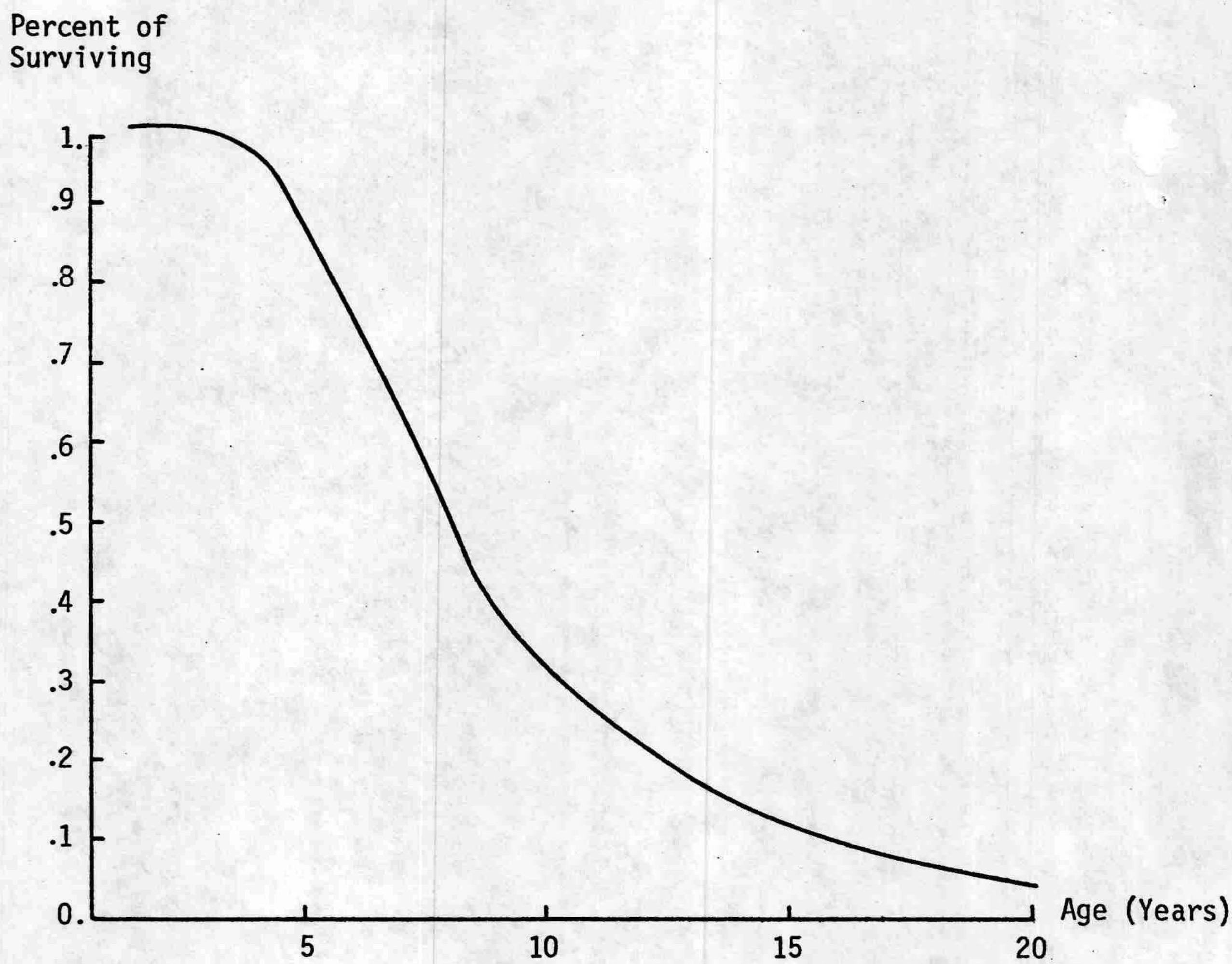

Figure 3A-29. Survivor Curve for Urban-Surface Treated Pavement Having Alligator Cracking (Area) Type of Distress

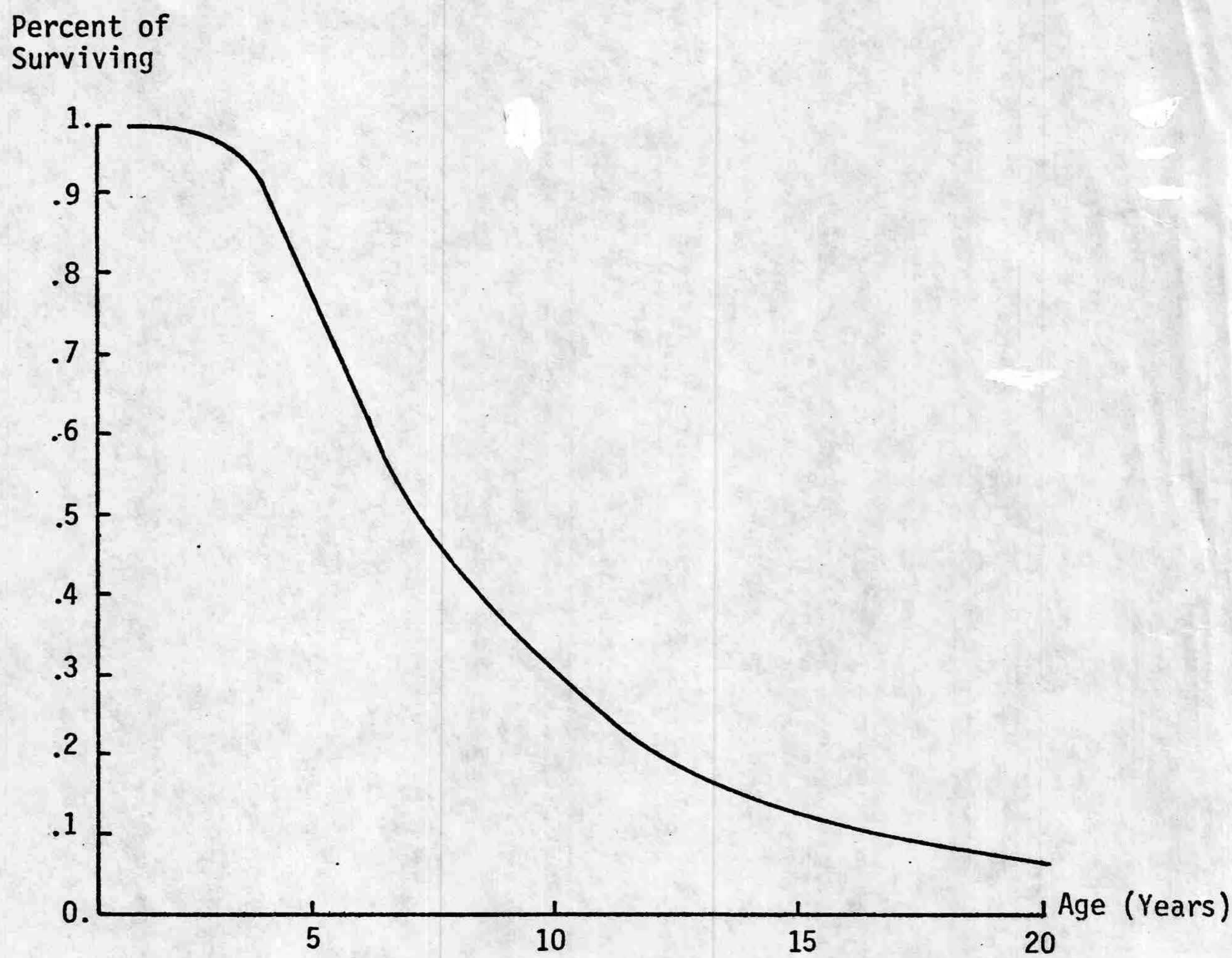

Figure 3A-30. Survivor Curve for Urban-Surface Teated Pavement Having Transversal Cracking (Severity) Type of Distress

APPENDIX 4

MAXIMUM LIKELIHOOD ESTIMATORS

Definition of Likelihood Function. The likelihood function of m random variables $W_1, W_2, \ldots, W_m$ is the joint density of the m random variables $g(W_1, W_2, \ldots, W_m; t, z)$ which is considered to be a function of t and z. In particular, if $W_1, W_2 \ldots, W_m$ is a random sample, the joint density function is:

$$g(W_1, \ldots, W_m; t, z) = f(W_1; t, z) \times \ldots \times f(W_m; t, z) \qquad \text{(A4-1)}$$

The likelihood $g(W_1, \ldots, W_m; t, z)$ gives the relative likelihood that the random variables assume a particular value $w_1, w_2, \ldots, w_m$.

A likelihood estimator can be defined as follows:

Let $L(t,z) = g(W_1, W_2, \ldots W_m; t, z)$ be

the likelihood function for the random variables $W_1, W_2, \ldots, W_m$. If $\hat{t}$ and $\hat{z}$ are the values of t and z in which maximizes $L(t,z)$, then $\hat{t}$ and $\hat{z}$ are the maximum likelihood estimators of t and z, respectively.

Many likelihood functions satisfy regularity conditions so that the maximum likelihood estimators are the solution of the simultaneous system of equations:

$$\frac{\partial L(t,z)}{\partial t} = 0 \qquad \text{(A4-2)}$$

$$\frac{\partial L(t,z)}{\partial z} = 0 \qquad \text{(A4-3)}$$

Also $L(t,z)$ and $L_n\left[L(t,z)\right]$ have their maximum at the same values of t and z, and it is sometimes easier to find the maximum of the logarithm of the likelihood function.

Given the cumulative density function

$$F(W; t, z) = e^{-z/W^t} \qquad \text{(A4-4)}$$

the corresponding density function of the random variable W is

$$f(W;t, z) = \frac{t\,z}{W^{t+1}} e^{-z/W^{t}} \tag{A4-5}$$

Defining the likelihood function as indicated in Eq. (A4-1) and using Eq. (A4-5) as density function of the random variable W , the solution of the system of Eqs. (A4-2), (A4-3) gives the following results in terms of the maximum likelihood estimators of t and z:

$$\sum_{i=1}^{m} Ln(W_i) - \frac{m}{t} = \frac{m}{\sum_{i=1}^{m} W_i^{-\hat{t}}} \left[\sum_{i=1}^{m} W_i^{-\hat{t}} Ln(W_i) \right] \tag{A4-6}$$

$$\hat{z} = \frac{m}{\sum_{i=1}^{m} W^{-\hat{t}}} \tag{A4-7}$$

An approximate solution to the system of equations defined by Eqs. (A4-6) and (A4-7) can be obtained by using a numerical method.

The previous result can be used to estimate the parameters of both the performance function and the survivor curve corresponding to a given type of flexible pavement. In the case of the performance function

$$g(W) = e^{-K/W}$$

t=1 and z=K. Therefore,

$$\hat{K} = \frac{m}{\sum_{i=1}^{m} W^{-1}} \tag{A4-8}$$

APPENDIX 5

MEAN AND COEFFICIENT OF VARIATION OF LOAD APPLICATIONS

The expected value of the random variable W presented in Appendix 4 can be obtained as follows:

$$E(W) = \int_{0}^{\infty} W\, f(W,t,z)\, dW \tag{A5-1}$$

in particular

$$E(W) = \int_{0}^{\infty} \frac{t\ z}{W^t}\, e^{-z/W^t}\, dW \tag{A5-2}$$

Integrating the above expression,

$$E(W) = Z^{1/t}\, \Gamma(\frac{t-1}{t}) \tag{A5-3}$$

where $\Gamma(\cdot)$ is the Gamma function.

Using the average of W_i's as estimator of E(W), Eq. (A5-3) can be written as:

$$\bar{W} = Z^{1/t}\, \Gamma(\frac{t-1}{t}) \tag{A5-4}$$

From Eq.(A5-4) the value of z can be derived as shown below

$$z = \left[\frac{\bar{W}}{\Gamma\, (\frac{t-1}{t})} \right]^t \tag{A5-5}$$

The variance of the random variable W can be obtained by the expression:

$$Var(W) = \int_0^{\infty} W^2 \frac{tz}{W^{t+1}} e^{-z/W^t} dW \qquad (A5\text{-}6)$$

Integrating, the resulting value for the variance is:

$$Var(W) = z^{2/t} [\Gamma(\frac{t-2}{t}) - \Gamma^2(\frac{t-1}{t})] \qquad (A5\text{-}7)$$

Therefore,the standard deviation is equal to:

$$\sigma(W) = z^{1/t} [\Gamma(\frac{t-2}{t}) - \Gamma^2(\frac{t-1}{t})]^{1/2} \qquad (A5\text{-}8)$$

The coefficient of variation is defined as:

$$CV = \frac{E(W)}{\sigma(W)} \simeq \frac{\bar{W}}{S_W} \qquad (A5\text{-}9)$$

Where $\bar{W}$ and S_W are the average and standard deviation of a random sample of W's. Using Eqs. (A5-3) and (A5-8), CV can be written as:

$$CV = \left\{ \frac{\Gamma(\frac{t-2}{t})}{\Gamma(\frac{t-1}{t})} - 1 \right\}^{1/2} \qquad (A5\text{-}10)$$

Eq.(A5-9) can be used to estimate the value of CV. Using this value, Eq.(A5-10) can be solved to obtain t.

The methodology presented in this appendix can be used as an alternative to the methodology presented in Appendix 4. A combination of both methodologies is also possible. For instance Eq. (A5-5) can be used to estimate z after using Eq.(A4-6) to estimate t.

APPENDIX 6

ESTIMATION OF FLEXIBLE PAVEMENT PARAMETERS

6A. PSI PERFORMANCE PARAMETERS

The estimation of the performance equations parameters can be accompanied by two methodologies:

(1) Statistical Approach: P_f and K can be obtained following the next steps:

Step 1. Fix n equal 1. It can be observed by experience that the value of n is around 1 in the case of performance equations.

Step 2. Observe a set of values of W (18-Kip ESALs) from historical data and for different representative sections of pavements.

Step 3. Use Eq. (A4-7) from Appendix 4 to comput K.

Step 4. Having K and a sample of values of Wi compute P_{fj}'s values by the expression:

$$P_{fj} = P_i - \frac{P_i - P_t}{e^{-K/W_j}} \qquad \text{(A6-1)}$$

Eq. (A6-1) was obtained from Eq. (3-15).

Step 5. Compute the average of P_{fj}'s values.

Step 6. Set P_f equal to $\bar{P}_f$ and adopt the K value from set (3).

(2) Mechanistic Approach: The technique presented through this approach is based upon a set of regression equations for K and P_f values, using as independent variables the mechanistic observations presented in Table 3-2. The methodology, which can be applied to any specific type of pavement, is as follows:

Step 1. Set n equal 1.

Step 2. Using the regression equation from Appendix 1A and the mechanistic variables contained in Table 3-2, compute R and P_f for different representative sections of the pavement type under consideration.

Step 3. Adopt the values of P_f equal to the $\bar{P}_f$, K equal to the $\bar{K}$.

6B. DISTRESS PERFORMANCE PARAMETERS

Similarly, for the distress case a statistical methodology and a mechanistic approach can be used to estimate the parameters of the performance equations.

(1) Statistical Approach: Five basic steps should be followed:

Step 1. Fix n equal 1, a_2 in a range between 0.20 and 0.30, and s_f in a range between 0.9 and 1.0.

Step 2. Observe different values of W_i (18-Kip ESALs) from historical data and for different representative sections of the type of pavement under consideration.

Step 3. Use Eq. (A4-7) from Appendix 4 to compute a_0 for the area equation.

Step 4. Use Eq. (A4-7) to compute a_2 for the severity equation using the values of a_1 and s_f assumed in Step 1.

Step 5. Compute the average values for a_0 and a_2.

(2) Mechanistic Approach: Three basic steps must be followed in that case:

Step 1. Set n equal 1.

Step 2. Using the regression equations contained in Appendix 1B, compute a_0, a_1, a_2, and s_f values for different representative pavement sections of the type of pavement under consideration.

Step 3. Adopt the average values of a_o, a_1, a_2. and s_f as representative magnitude for the constants.