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Performance of Continuously Reinforced Concrete Pavements, Volume IV—Resurfacings for CRC Pavements

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FOREWORD

This report is one volume of a seven-volume set presenting the results of a study to provide the state-of-the-art for the design, construction, maintenance and rehabilitation of Continuously Reinforced Concrete Pavements (CRCP). Through a through literature review of current and past research work in CRCP and extensive field and laboratory testing of 23 in-service CRC pavements, the effectiveness of various design and construction features were assessed; performance of CRCP was evaluated; and procedures for improving CRC pavement technology were recommended. The 23 test pavements were located in six states that participated in this national pooled fund study. In addition the data available for 83 CRCPs included in the General Pavement Study (GPS) number 5 of the Long Term Pavement Performance (LTPP) Program was presented and analyzed. A number of CRCP maintenance and rehabilitation techniques that have been used over the years, including joint and crack sealing, cathodic protection of reinforcing bars, full-depth patching, resurfacing, etc. were also evaluated. This report will be of interest to engineers and researchers concerned with the state-of-the-art design, construction, maintenance and rehabilitation of CRCP including predictive models. The study was made possible with the financial support of Arizona, Arkansas, Connecticut, Delaware, Illinois, Iowa, Louisiana, Oklahoma, Oregon, Pennsylvania, South Dakota, Texas and Wisconsin.

Sufficient copies of this report are being distributed to provide two copies to each FHWA regional office and three copies to each FHWA division office and each state highway agency. Direct distribution is being made to the division offices. Additional copies for the public are available from the National Technical Information Service (NTIS), United states Department of Commerce, 5285 Port Royal Road, Springfield, Virginia 22161.

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16. Abstract

This report is one of a series of reports prepared as part of a recent study sponsored by the Federal Highway Administration (FHWA) aimed at updating the state-of-the-art of the design, construction, maintenance, and rehabilitation of continuous reinforced concrete (CRC) pavements. The scope of work of the FHWA study included the following:

1. Conduct of a literature review and preparation of an annotated bibliography on CRC pavements and CRC overlays.
2. Conduct of a field investigation and laboratory testing related to 23 existing in-service pavement sections. This was done to evaluate the effect of various design features on CRC pavement performance, to identify any design or construction related problems, and to recommend procedures to improve CRC pavement technology.
3. Evaluation of the effectiveness of various maintenance and rehabilitation strategies for CRC pavements.
4. Preparation of a Summary Report on the current state of the practice for CRC pavements.

Each of the above four items is addressed in a separate report. The following reports have been prepared under this study:

Performance of CRC Pavements
- Volume I - Summary of Practice and Annotated Bibliography
- Volume II - Field Investigation of CRC Pavements
- Volume III - Analysis and Evaluation of Field Test Data
- Volume IV - Resurfacings for CRC Pavements
- Volume V - Maintenance and Repair of CRC Pavements
- Volume VI - CRC Pavement Design, Construction, and Performance
- Volume VII - Summary

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SI* (MODERN METRIC) CONVERSION FACTORS

APPROXIMATE CONVERSIONS TO SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
	LENGTH			
in	inches	25.4	millimeters	mm
ft	feet	0.305	meters	m
yd	yards	0.914	meters	m
mi	miles	1.61	kilometers	km
	AREA			
in^2	square inches	645.2	square millimeters	mm^2
ft^2	square feet	0.093	square meters	m^2
yd^2	square yards	0.836	square meters	m^2
ac	acres	0.405	hectares	ha
mi^2	square miles	2.59	square kilometers	km^2
	VOLUME			
fl oz	fluid ounces	29.57	milliliters	mL
gal	gallons	3.785	liters	L
ft^3	cubic feet	0.028	cubic meters	m^3
yd^3	cubic yards	0.765	cubic meters	m^3
NOTE: Volumes greater than 1000 l shall be shown in m^3.				
	MASS			
oz	ounces	28.35	grams	g
lb	pounds	0.454	kilograms	kg
T	short tons (2000 lb)	0.907	megagrams (or "metric ton")	Mg (or "t")
	TEMPERATURE (exact)			
°F	Fahrenheit temperature	5(F-32)/9 or (F-32)/1.8	Celcius temperature	°C
	ILLUMINATION			
fc	foot-candles	10.76	lux	lx
fl	foot-Lamberts	3.426	candela/m^2	cd/m^2
	FORCE and PRESSURE or STRESS			
lbf	poundforce	4.45	newtons	N
lbf/in^2	poundforce per square inch	6.89	kilopascals	kPa

APPROXIMATE CONVERSIONS FROM SI UNITS

Symbol	When You Know	Multiply By	To Find	Symbol
	LENGTH			
mm	millimeters	0.039	inches	in
m	meters	3.28	feet	ft
m	meters	1.09	yards	yd
km	kilometers	0.621	miles	mi
	AREA			
mm^2	square millimeters	0.0016	square inches	in^2
m^2	square meters	10.764	square feet	ft^2
m^2	square meters	1.195	square yards	yd^2
ha	hectares	2.47	acres	ac
km^2	square kilometers	0.386	square miles	mi^2
	VOLUME			
mL	milliliters	0.034	fluid ounces	fl oz
L	liters	0.264	gallons	gal
m^3	cubic meters	35.71	cubic feet	ft^3
m^3	cubic meters	1.307	cubic yards	yd^3
	MASS			
g	grams	0.035	ounces	oz
kg	kilograms	2.202	pounds	lb
Mg (or "t")	megagrams (or "metric ton")	1.103	short tons (2000 lb)	T
	TEMPERATURE (exact)			
°C	Celcius temperature	1.8C + 32	Fahrenheit temperature	°F
	ILLUMINATION			
lx	lux	0.0929	foot-candles	fc
cd/m^2	candela/m^2	0.2919	foot-Lamberts	fl
	FORCE and PRESSURE or STRESS			
N	newtons	0.225	poundforce	lbf
kPa	kilopascals	0.145	poundforce per square inch	lbf/in^2

* SI is the symbol for the International System of Units. Appropriate rounding should be made to comply with Section 4 of ASTM E380.

(Revised September 1993)

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CHAPTER 1 - BACKGROUND

Continuously reinforced concrete (CRC) pavement is portland cement concrete (PCC) pavement with continuously placed longitudinal steel reinforcement and no transverse expansion or construction joints. CRC pavements have been in use in the United States since 1938. In fact, several experimental pavements of this type had been constructed even earlier. Nominal thicknesses of 152 to 229 mm (6 to 9 in) were used, and the percentage of steel varied anywhere between 0.07 and 1.82 percent.[2] Very good performance has been observed in the pavements containing a larger percentage of steel. The initial idea behind this type of pavement was for it to be essentially a "zero-maintenance" type of pavement. Consequently, CRC pavements have been accepted as generally having a higher initial cost due to the cost of the steel reinforcement. Where design criteria are based on lower initial cost, CRC pavement design thicknesses have been reduced as much as 51 to 76 mm (2 to 3 in) in order to make them competitive as a design alternative. This under-design has led to several premature failures. Generally, CRC pavements that meet the design specifications perform well.

Inadequate design has been considered by many to be the cause of severely distressed CRC pavements. One of the most prevalent type of distresses in this type of pavement is punchout distress. This type of distress forms when two closely spaced transverse cracks join a longitudinal crack or joint. Joint spalling and faulting often accompany this type of distress. It has largely been believed that the primary cause of punchout distress in CRC pavements is related to insufficient pavement thickness, foundation support and accumulated traffic loadings. A study by Zollinger and Barenberg[2] conclusively related punchout to the loss of subbase support and the consequent loss of load transfer across the transverse joints. Hence, it has been recently pointed out that, for these types of pavements to perform well, it is necessary to provide adequate support underneath the pavement.

Good performance of CRC pavements is considered to be analogous to the provision of uniform foundation support. Previous CRC designs have not protected adequately against the loss of foundation support. Given this mode of failure, it is nearly impossible to protect against punchout distress on the basis of thickness design alone. This has led to the gradual failure of these pavement types across the country. Hence, the need for effective rehabilitation of CRC pavements has arisen.

History of Resurfacing

Rehabilitation of pavements is done mostly by using overlays. The objective of an overlay is to lengthen the life of a pavement and increase its rideability. The important role of the pavement network in the nation's economy has led to a continual search for ways to economically maintain the pavement system with minimal disruption to the traffic flow.

PCC and asphalt concrete (AC) are the two common types of overlays. Use of concrete overlays as a method of extending the life of the existing pavement has been a practice since 1913. Most state highways were originally constructed using about 102 to 152 mm (4 to 6 in) of concrete or a comparable thickness of flexible pavement. As the number of vehicles

increased, more concrete was added to increase the load carrying capacity of the original pavement. During the years prior to World War II, a few agencies experimented with 25.4 to 76 mm (1 to 3 in) of concrete bonded to the existing pavement as an overlay alternative.[9] During the war, pavement rehabilitation was minimal; because of increased traffic rates during that time, a tremendous back-log of vehicular pavement rehabilitation work developed. Thus, there was a need for a low-cost overlay whose construction caused the least disruption to traffic flow and provided a dramatic improvement in the rideability of the existing surface. AC overlay seemed to answer these criteria. During World War II, the country's economy and myriad other reasons forced engineers to use the aforementioned criteria to select the type of overlay. However, since then, the criteria for overlay type selection have been re-evaluated, with the result that the main objective of an overlay, to extend the life of a pavement, has been overlooked.[27] Although the times have changed, many engineers still think in terms of the war-time criteria. In addition, there is little understanding about the effectiveness of AC overlays on concrete pavements, or their effect on long-term performance or behavior of the concrete pavement.

Since traffic volumes for most primary systems have far exceeded original projections, pavements now require, or in the near future will require, rehabilitation much sooner than originally contemplated. Given the popular use of overlays as the most common form of rehabilitation measure used today, the design procedure adopted plays a very important role in the overlay type selection. The choice of the type of overlay must be made on the basis of life-cycle costs instead of on the initial costs.

Focus and Format

The primary focus of this report is to provide a framework for the design of CRC overlays or overlays on CRC pavements. This focus has been facilitated by provideing the results of a literature search of the latest field data and information regarding observed modes of failure.

Information on design and performance regarding different overlays for CRC pavements was collected. Specifically, documentation of the modes of failure of each was reviewed to identify causes/mechanisms related to each mode.

The following categories of overlays have been included in this study:

1. CRC overlays of flexible pavements
2. CRC overlays of jointed concrete pavements (unbonded overlays)
3. Unbonded CRC overlays of CRC pavements
4. Bonded CRC overlays of CRC pavements
5. AC overlays of CRC pavements

Different modes of failures have been identified for each type of CRC overlay mentioned above. The analysis of each provides a basis for future designs.

Synthesis of available performance data of CRC overlays on existing pavements is provided and discussed herein. Punchouts are the predominant form of distress in CRC pavements as well as in overlays. An evaluation of these overlays, taking into consideration the factors related to punchout distress formation and the relationship among these factors, can lead to updated guidelines for the design of CRC overlays.

The use of AC overlays on CRC pavements has been widespread. A study of the thermal insulative effects of such overlays on CRC pavements has been done. Rutting and reflective cracking were noticed to be a prevalent form of distress in these types of overlays. An analysis of the influence of rutting and reflective cracking on the life of an overlay, along with the causes related to delamination, can be used to predict the performance and life of overlays.

This report addresses the lack of guidelines for the design of CRC overlays on the basis of the effect of loss of support. Previous design procedures have considered the consequence of the lack of uniform support, but only based on the erodibility of the subbase material. Experiences with several premature failures in these types of overlays seem to indicate a need to consider the effect of loss of support directly in design. Any thickness design for an overlay must encompass the degree of slab support as the primary basis upon which to determine overlay thickness.

This study also considers the effect of climatic factors in the design and performance of CRC overlays or overlays on CRC pavements. Climate and fatigue effects have been identified in the literature as causes for the development of failures like rutting (predominantly in AC overlays of CRC pavements), reflective cracking, and delamination. However, there is little documentation of the effect of these failure modes on the performance or longevity of the overlay.

In terms of the organization of this report, Chapter 2 consists of a literature review of information available about the types of overlays described above. It also includes a review of the available design procedures for CRC overlays and a critique. Some construction projects are summarized and their failures identified.

Chapter 3 presents a framework for the design of CRC overlays that makes use of falling weight deflectometer (FWD) data to characterize the support underneath the overlay in terms of the "stiffness" of the pavement. In other words, field data have been synthesized to provide practical guidelines for the design of CRC overlays.

Chapter 4 examines the failure modes and effectiveness of AC overlays. Analytical/structural modeling of the failure modes predominant in AC overlays of CRC pavements and a study of the "effectiveness" of AC overlays on CRC pavements is presented. Since distresses like reflection cracking and rutting are common in these types of overlays, an analytical/modeling approach has been taken to examine the effectiveness of these overlays in extending the service life of the pavement. It should be noted that asphalt overlays on CRC pavements have been used extensively and appear to have extended the service life of CRC

pavements (as shown in Indiana). Some concluding recommendations regarding their use have been made.

Chapter 5 provides a life-cycle cost analysis of the various overlay alternatives involving the use of CRC pavements.

Chapter 6 summarizes the report. Some recommendations for future directions to be adopted in research has also been provided.

CHAPTER 2 - OVERLAY TYPES INVOLVING CRC PAVEMENTS

Overlays are a common method of rehabilitating existing pavements that have been severely distressed by roughness or have come to the end of their design life. Basically, two types of overlays have been routinely used: flexible and rigid overlays. There may, however, be several variations of these two basic types. Flexible overlays are those that have paving materials made from bituminous materials. Rigid overlays are those that are constructed from portland cement concrete (PCC).

The use of both these types of overlays as rehabilitation measures has been widespread. Asphalt overlays are commonly used, because of the low costs of initial construction and a dramatic improvement in the riding quality of the existing surface. Though the use of PCC pavement as a new pavement has been extensive, PCC has not been used extensively as a resurfacing material. A common perception among highway engineers is that concrete overlays are relatively new and unproven in their effectiveness. This perception holds true for continuously reinforced concrete (CRC) overlays as well. But findings from this study indicate the opposite, as will be explained in the following chapters. The resurgence in the consideration of concrete resurfacings is evidenced by the number of concrete resurfacing projects that have been constructed in the past 20 years. More than 1,000 m (1700 km) of CRC overlay has been built in United States since 1959 when the first CRC overlay was laid near Eddy, Texas. Some of the increased activity in the 1970s can be attributed to the emphasis placed on the Federal Highway Administration's (FHWA) 4-R program of Rehabilitation, Restoration, Resurfacing, and Reconstruction. The higher cost of bitumen in recent years has also led to renewed considerations based on a total-cost economic analysis. Other factors such as the use of CRC, fibrous concrete, and prestressed concrete as potential resurfacing materials have broadened the application of rigid overlays and attracted the attention of engineers.[9]

Types of Overlays Involving CRC Pavement

The failure of most highway pavements is related to excessive pavement distress and inadequate structural capacity for future needs, since traffic levels have been far higher than what was projected. Any rehabilitation alternative being considered must, hence, be able to meet the design requirements and not fail during its design life. Rigid overlays provide a feasible option in terms of strength and longevity.

For a nation whose network of pavements is now in need of rehabilitation, CRC pavements present themselves as a feasible and lucrative option, if life-cycle costs were the basis of design. If designed and constructed properly, CRC overlays can serve their lifetime with little or no maintenance. In general, thick AC overlays and unbonded PCC overlays are the most practical when the existing pavement is in poor condition and is more suited as a "support" for the overlay than as part of the pavement structural system. The types of overlays involving the use of CRC pavement that have been commonly used are:

1. CRC Overlays on Jointed Reinforced Concrete Pavements
2. AC Overlays on CRC Pavements

Each of these overlays is described in chapter 3 and chapter 4, respectively.

Overlays may be broadly classified as unbonded, partially bonded or bonded. Since the behavior of each class of overlays is different, the design and construction consideration of each must also be different.

Unbonded Overlays

Unbonded overlays (see figure 1) are overlays in which there is a minimum of bond between the existing pavement and the overlay. An asphalt layer is used as a separation layer to reduce bonding and serve as a stress relief course. This layer minimizes the reflection of cracks or joints into the surface layer. Because of the separation between the two layers, the maximum stress can occur in the overlay structure. Therefore, unbonded concrete overlays are normally thicker than bonded concrete pavements and sometimes as thick as or thicker than the existing pavement. An unbonded system is preferred to a bonded system when the existing pavement's remaining life is very short. The unbonded overlay can be constructed on the existing pavement with only minimal improvement of the existing pavement.

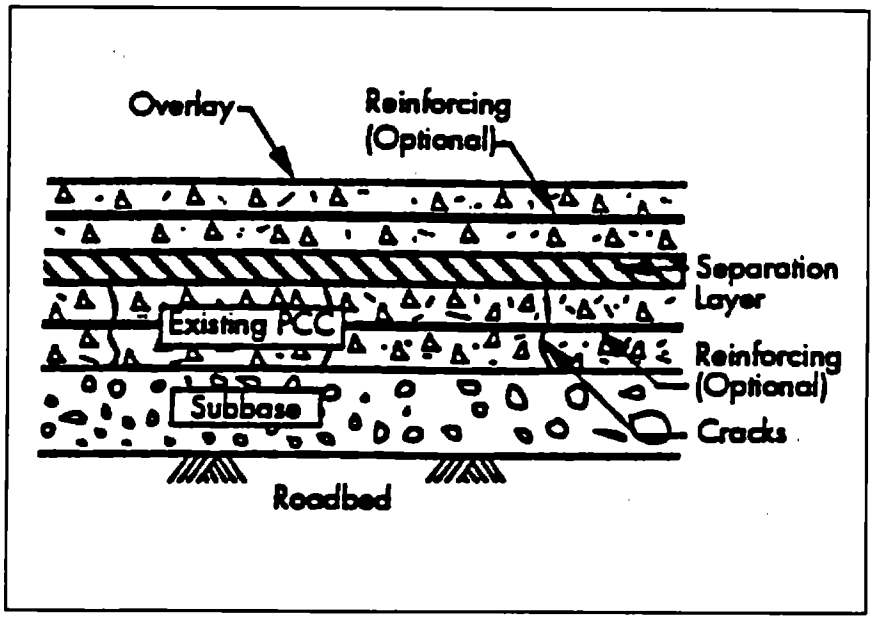


Figure 1. Unbonded concrete overlay.[11]

Partially Bonded Concrete Overlays

Partially bonded overlays are rarely used. The overlay is placed on the existing pavement without any specific preparation of the existing pavement surface. The pavement is partially bonded to the existing pavement, but the strength and uniformity of the bond are uncertain. The use of partially bonded CRC overlays precludes the necessity to match transverse joints, but matching of longitudinal joints is recommended.[35]

Bonded Concrete Overlays

Bonded concrete overlays (figure 2) are overlays in which the surface of the existing pavement is specially prepared by techniques such as cold milling or shot blasting. A bonding agent can also be used to ensure a good bond between the two layers. The improvement in rideability and increase in life span are affected by the composite structure formed by the existing pavement and the overlay.

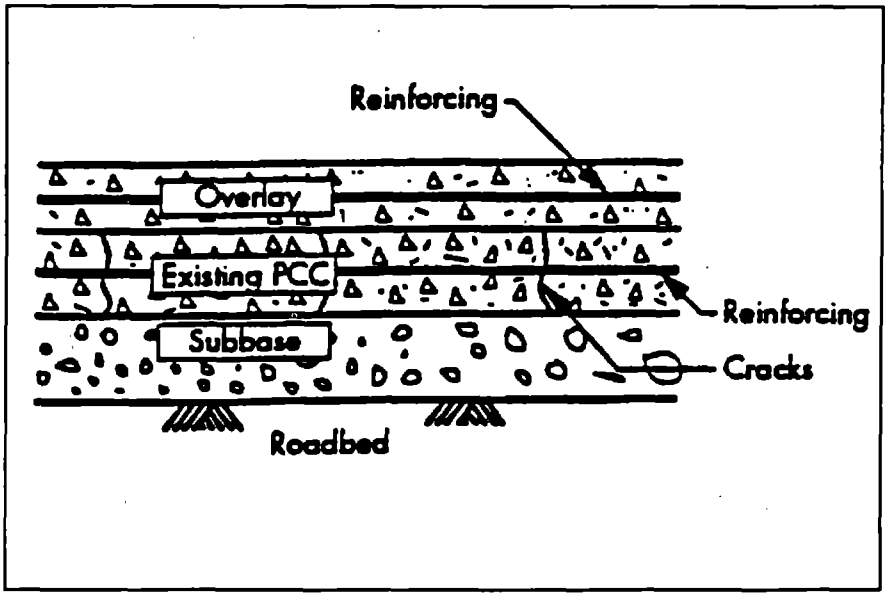


Figure 2. Bonded overlay.[11]

General Approaches to Overlay Design

In the United States, investigations of major highways and freeways for structural and/or functional failure indicate that the pavements built in the post-World War II era are nearing the end of their design lives. The structural and functional capability of these highways can be improved by either rehabilitation or reconstruction. Overlays are one of the most commonly used rehabilitation techniques and may be used to correct one or more of the following deficiencies in the existing pavement:

1. Excessive pavement distress,
2. Inadequate riding quality,
3. Reduced skid resistance,
4. Excessive maintenance requirements, and
5. Inadequate structural capacity for future needs.

The purpose of an overlay design must then be to satisfy any or all of the above intended functions. This means that the overlay must provide adequate performance for its intended purpose, over a given time. Several approaches to overlay design have been adopted in the past. Engineering Judgment, Empirical, Deflection-Based, and Mechanistic are among the basic overlay design approaches. Each will be discussed below.

Engineering Judgment

Most overlay designs in use today employ engineering judgment in one form or another. In its purest form, this method of design requires that the engineer determine the overlay thickness and material to be used based on a subjective analysis.[3] The experience of the engineer is relied upon to make decisions regarding the needed thickness as long as the pavement type, soil type, climate, traffic, and pavement condition are similar to those within the experience base. It is very difficult to transfer this experience and capability from older engineers to their less experienced replacements. When the engineer encounters new materials, new pavement types, or new conditions, valid design decisions cannot be made due to lack of experience with the new environment. In this lies the inherent weakness of this method of design.

Engineering judgment is still used in many agencies in certain circumstances. For instance, when a pavement is functionally deficient, i.e., lacking in skid resistance, or experiencing too much roughness and raveling, engineering judgment is a convenient method used to design an overlay to improve surface characteristics. The thickness of the overlay is often the practical minimum thickness required to level the surface and provide a safe ride, and is not being used to improve the structural adequacy of the pavement.

Very few design procedures address how to determine the thickness of asphalt overlays or concrete overlays for concrete pavements and fewer still address the thickness of overlay required when the primary goal is to reduce reflective cracks. As a result, many agencies use engineering judgment to design these overlays.

Empirical Procedures

Empirical procedures base overlay thickness design on known data such as age, traffic, construction, structural section, and environmental factors.[3] A relationship between the performance and overlay thickness, quantities mentioned above, etc., is developed from regression analysis of performance data. The required thickness is then determined using the regression equation. These procedures are valid only for the pavement population whose data was used to develop the equation. New materials, pavement types, or climatic conditions can invalidate the results.

Most current empirical design procedures employ the "thickness deficiency" concept. The basic idea of this approach is to first determine the thickness of a new pavement to be placed above the subgrade, given past and projected traffic levels. This thickness minus the thickness of the existing pavement adjusted for existing structural condition, will result in the required design thickness of the overlay. A typical thickness deficiency design equation is of the form:

$$T_{overlay} = T_{required} - C \cdot T_{present}$$

where

$T_{overlay}$ = Thickness of the overlay to be placed
$T_{required}$ = Total thickness required above the subgrade
$T_{present}$ = Thickness of the existing pavement
C = Condition factor to account for structural inadequacies in the existing pavement (typically ranges from 0.35 to 1.0).

One of the major difficulties that exists with this method is its use of the condition factor whose assignment is rather arbitrary and is based upon a "subjective" visual survey. It is generally acknowledged that a structural section that has been subjected to traffic loadings and environmental deterioration is not as structurally adequate as the same section when it was new. However, little agreement is reached concerning how to modify the existing thickness to account for the deterioration. If this method is used, a rational approach for adjusting the existing thickness or structural coefficients must be developed.

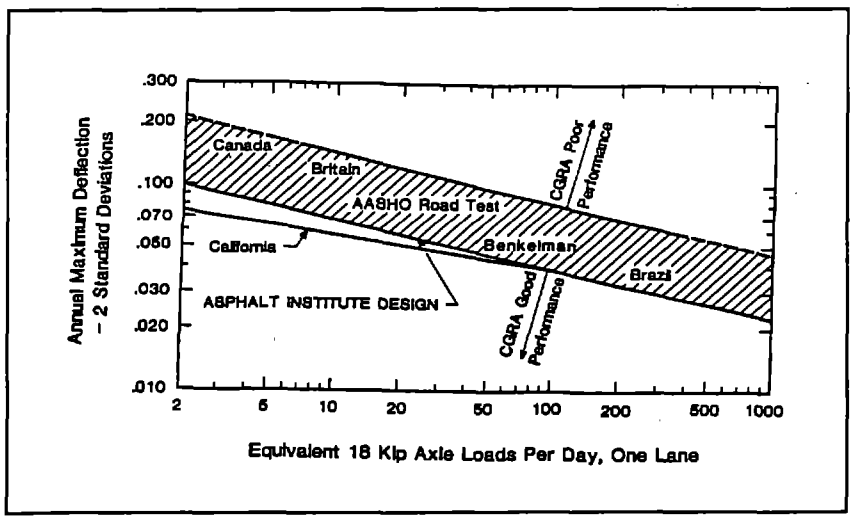


Figure 3. Compilation of beam deflection experience.[17]

Deflection-Based Methods

Empirically developed deflection-based overlay design procedures have gained wide acceptance and are used in several states including Utah, Texas, and Louisiana.

The basic concepts are that:

1. Similar pavements with higher deflections will fail more quickly than those with lower deflections under the same loading, as illustrated in figure 3 which was taken from the Asphalt Institute's deflection method for the design of asphalt concrete overlays of asphalt concrete pavements.

2. Increased asphalt thicknesses will decrease deflections, as illustrated in figure 4.

These concepts can then be combined to form a chart that gives the overlay thickness required for an existing deflection for different traffic levels, as illustrated in figure 5.

In 1973, after reviewing available methods for design of concrete resurfacings, Martin[22] concluded that they were purely subjective and should be reappraised. He proposed a method based on allowable deflection, which was one of the earliest of the deflection-based methods. Through a study of the American Association of State Highway

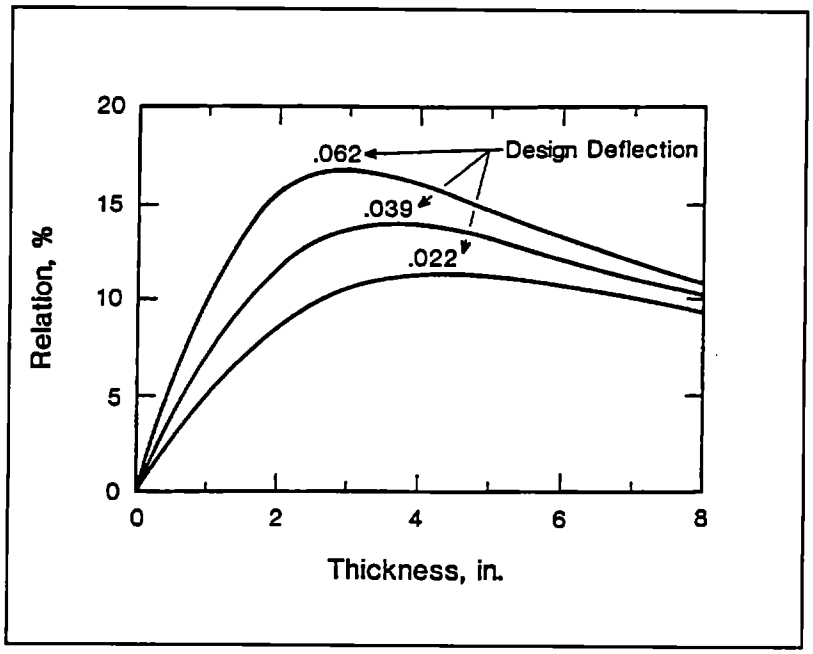


Figure 4. Percent reduction of deflection per inch of thickness.[17]

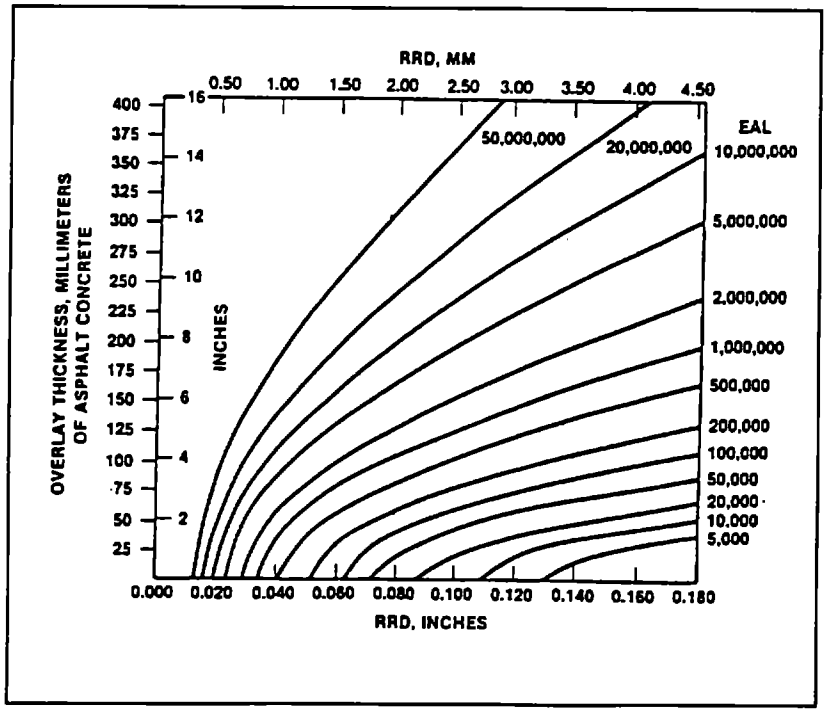


Figure 5. AC overlay thickness required to reduce pavement deflection value (rebound test).[18]

Officials (AASHTO) Road Test data, Martin selected 0.635 mm (0.025 in) as an allowable maximum calculated deflection for the design of concrete resurfacings. The influence of such variables as load, load location, traffic, load transfer across joints or cracks, effects of tied shoulders, slab support, joint type and spacing, and reinforcement design on slab deflection was discussed.[22] Using these variables, Martin developed the relationship given in table 1. In the development of the procedure, Martin used one-half of an axle-load as a static wheel load and calculated the slab thickness for deflection of 0.635 mm (0.025 in) for a range of foundation support, load transfer, and slab edge conditions. The use of this method is also purely judgmental since no clearly substantiated reasons for the use of 0.635 mm (0.025 in) has been provided other than that a study of the AASHTO Road Test seemed to indicate so. As will be shown later, after a thorough study of the deflection data obtained from non-destructive testing of several distressed pavement sections, the Indiana DOT concluded that deflections greater than 0.023 mm (0.9 mils) indicated a poor pavement. This observation seems to be contrary to Martin's selection of 0.635 mm (0.025 in) as maximum allowable deflection.

Table 1. Slab depth related to traffic, load transfer, shoulder type, and slab support for maximum calculated slab deflection of 0.635 mm (0.025 in).[45]

ADTST on Design Lane for 30 Years	Transverse Joint Load Transfer[a]	Shoulder Type[b]	Slab Depth[c] (in.)		
			k = 50	k = 150	k = 500
5,000	A	G or B	20.5	14.5	9.5
	A	C	14.5	10	7
	D or CR	G or B	14.5	10	7
	D or CR	C	11	8	5.5
2,000	A	G or B	17.5	12.5	8.5
	A	C	12.5	9	6
	D or CR	G or B	12.5	9	6
	D or CR	C	9.5	7	4.5
800	A	G or B	14.5	10	7
	A	C	10.5	7.5	5
	D or CR	G or B	10.5	7.5	5
	D or CR	C	8	6	4
200	A	G or B	11	8	5.5
	A	C	8	6	4
	D or CR	G or B	8	6	4
	D or CR	C	6	4.5	3

[a] A = aggregate interlock; D = dowels; and CR = continuous reinforcement.

[b] B = granular material; C = concrete; and B = bituminous material. Concrete shoulders are same depth as slab at pavement edge. Longitudinal joints are tied.

[c] Slab depth is that required at free edge or at longitudinal joint when adequately tied concrete shoulders are used. Slab depth must be sufficient to provide adequate cover for dowels or reinforcement.

Most deflection-based overlay design procedures are still empirical. Their applicability is limited to the performance data upon which they were developed. Measured deflections are influenced by the device and procedures used to measure them. The deflections used in the overlay design procedures are based on a specific device used to measure the deflection. In addition, the season and the time of the day in which testing is to be conducted for design purposes must be defined since the deflections vary not only with traffic, but also with temperature and other environmental factors. Otherwise, a procedure to convert deflections to some common basis must be provided. Methods to correct the deflections to a standard temperature must also be defined.[4]

Mechanistic Design Methods

To overcome the limitations of empirical methods, mechanistic theoretical methods were developed. Mechanistic design procedures use engineering mechanics that are based on a specific distress mechanism. This allows a theoretical model to be formulated to predict stresses, strains, and deflections for a specific pavement type that corresponds to the distress mechanism. The major advantage of these methods is that they are not bound to local conditions but can be applied anytime, anywhere. Many highway researchers are in agreement that, if these methods could be incorporated into a practical design procedure, more realistic prediction of pavement performance could be realized.

CRC Overlay Types and Performance

Different overlay types will result in different modes of failure. Therefore, each mode of failure must be identified in terms of its cause and mechanism. Only then can a model be developed to represent the mechanism. Overlay designs must therefore encompass the development of distress, within their framework.

CRC pavement has been used as an overlay type since 1959 when a 178-mm (7-in) CRC pavement was used to overlay a 152-mm (6-in) existing PC pavement with 89 mm (3.5 in) asphaltic concrete bondbreaker, on I-35 in Falls and McLennan Counties in Texas. The original pavement was constructed in 1934 and was overlaid with the purpose of strengthening the original pavement and improving its riding quality. Since then, several of these types of overlays have been used throughout the nation. However, there is little documentation of their performance in terms of failure modes. Discussed below is a review of the performance of different overlay pavements involving the use of CRC pavements. Identification and analysis of the failure modes is also provided.

Several projects across the nation and the State of Texas have been built that have used CRC pavement as an overlay alternative. The performance of these pavements has been mixed. Some of these overlay alternatives involving the use of CRC pavements are:

1. CRC overlays on flexible pavements
2. CRC overlays on jointed concrete pavement (unbonded)
3. CRC overlays on CRC pavements (unbonded)
4. CRC overlays on CRC pavements (bonded)
5. AC overlays on CRC pavements

CRC Overlays on Flexible Pavements

The first use of concrete to overlay an existing flexible pavement was in Terre Haute, Indiana, in 1918. The existing flexible pavement was overlaid with 76 mm (3 in) of reinforced concrete and was reported to be in a good condition after 8 years of service. During the 1940s and 1950s, several projects were initiated that involved overlaying existing flexible and brick pavements with reinforced or plain concrete, in order to upgrade the existing pavement and strengthen it.

Between 1967 and 1975, several existing flexible pavement highway sections were resurfaced using CRC pavement. These resurfacings ranged from 152 to 229 mm (6 to 9 in) in thickness with 0.5 to 0.6 percent of longitudinal steel. Generally, a minimum thickness of asphalt concrete was used on the existing flexible pavement as a leveling course before overlaying. Some of these CRC overlay projects were reported in 1975[25] to be in excellent condition after 2 to 6 years of service. Lokken[26] confirmed the excellent conditions of these CRC resurfacings in 1980. No data were available on the traffic levels.

In 1966, Westall[33] presented design and construction details for concrete overlays on flexible pavements on the basis of previous experience, and discussed the performance of several projects. Westall concluded that "concrete overlays built on asphalt pavements have demonstrated the feasibility of this type of construction when a change in pavement is planned and it is practicable to re-use an existing asphalt pavement."

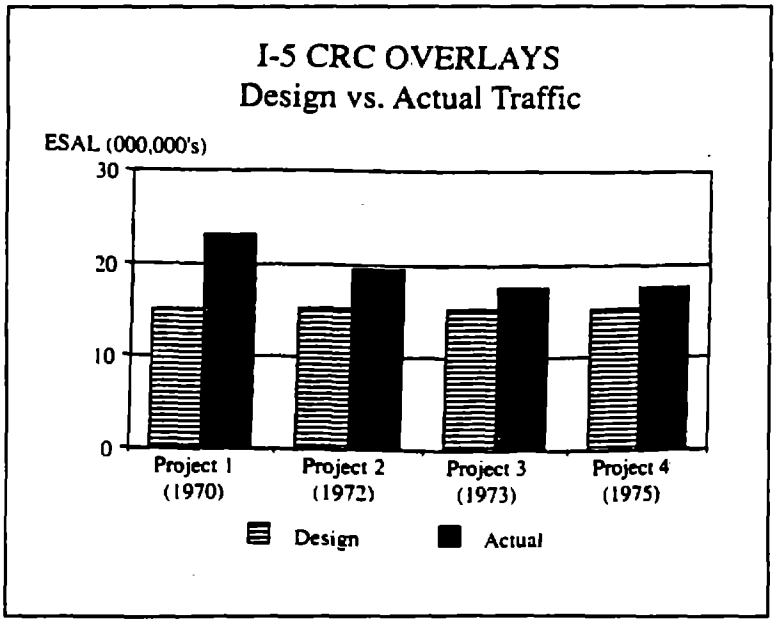


Figure 6. Design versus actual traffic.

Between 1970 and 1975, the Oregon DOT constructed four CRC overlays on I-5 between Portland and Salem. The rehabilitation was designed to upgrade an existing four-lane highway by overlaying the asphalt concrete pavement and widening it from four to six lanes. While these pavements are approaching 20 years in age, all four projects have carried traffic in excess of the initial design estimates (see figure 6). The structural condition and ride was reported to be exceptional. Surveys in 1988 showed that all projects were structurally sound, and ride ratings were also favorable (see Figure 7). The most significant distress was rutting in the wheel paths from studded tire wear. Isolated patching at some construction joints was also observed.[32] Outside of this and routine sealing of longitudinal joints, very little other maintenance has been required.

It can be concluded that experience has shown that CRC overlays are an economically feasible alternative. A CRC overlay of existing flexible pavements will essentially perform as a new pavement. The support may be assumed to be better, and chances of pumping are because of low underlying AC layer. Support-related problems, often reported in CRC pavements, may not be observed because of the high-strength base provided by the existing flexible pavement.

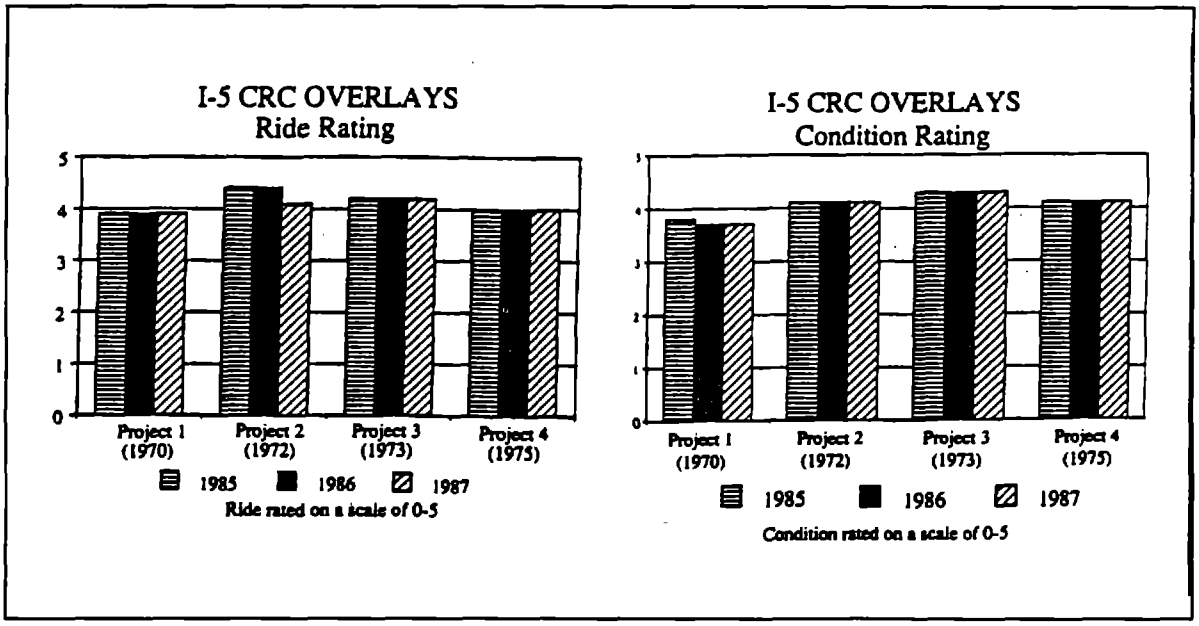


Figure 7. Ride rating and condition rating.[32]

CRC Overlays on Jointed Concrete Pavements (Unbonded)

During the 1960's and 1970's, many miles of unbonded concrete overlays were used for highway pavements. It was not widely used in airfields, where the purpose of overlays has generally been to upgrade existing structurally sound pavements and to carry increased design loadings. In this context, Mellinger[30] stated, "A nonbonded design requires a greater thickness of overlay and generally is used when the existing pavement is relatively thin in comparison with the thickness of the overlay or when the base pavement contains numerous cracks."

The use of CRC overlays with jointed concrete pavements have rarely been successful, (even in situations where low-level truck traffic is involved) primarily due to the lack of consideration of the requisite stiffness of the underlying support system. Overlaid jointed pavements may lack stiffness at certain transverse joints unless special measures are taken to stiffen worn or excessively wide joints. Asphalt overlays are not a substitute for properly designed and placed load transfer restoration devices that are used to improve load transfer and stiffness of the joint.

In 1971, I-69 in Indianapolis, Indiana, was overlaid using CRC pavement. The existing pavement consisted of a 229-mm (9-in) reinforced concrete pavement that had 12.2-m (40-ft) transverse joint spacings. Extensive spalling and raveling were observed at the transverse cracks and joints. This pavement was overlaid with 152 mm (6 in) of CRC using 0.15 mm (6 mils) of polyethythene sheeting as a bond-breaker. When the pavement was evaluated in 1975, it was noticed that the overlay had closely spaced transverse cracks, punchout failures, and several patches. The poor condition of the overlay was related to the unconventional bond-breaker and, consequently, the unrestrained movement of the overlay at the cracks. This might have been the cause of several reflection cracks, with the poor

condition of the underlying pavement contributing significantly to the condition of the overlay. Insufficient thickness of the overlay and the poor existing support conditions were termed as the primary causes of failure.

Recently, support-related problems have been noted with CRC overlays on existing concrete pavements[36, 37] in Pennsylvania and Wisconsin. A 178-mm (7-in) CRC overlay on 254-mm (10-in) jointed concrete pavement constructed between 1974 and 1976 in Pennsylvania had developed several punchouts, most (85 percent) of which occurred at the joints in the underlying pavement. The Pennsylvania CRC overlay originally contained 0.7 percent of steel reinforcement but the resulting punchout repairs were made with twice that amount of steel because analysis indicated that 0.7 percent steel reinforcement would not withstand the stresses induced by load applications and non-uniform support conditions at the joints in the original 254-mm (10-in) pavement.[36] A similar support-related problem, discussed later, was found in Wisconsin, as shown in figure 14. The patches that contained 1.4 percent steel reinforcement had close to a 90 percent success rate compared with the patches with 0.7 percent steel that had a 19 percent success rate.[37] The punchouts observed in this project had faulted 19 to 25.4 mm (3/4 to 1 in), and several of the bars were broken. The Pennsylvania investigators were also of the opinion that close crack spacing at the joint locations contributed to the distresses the pavement was experiencing. It was also noted that the dowels in the 254-mm (10-in) pavement were badly deteriorated and reduced in cross-sectional area. The load transfer efficiency was very poor on various sections of the project. Apparently, wide cracks had developed at several of the joint locations in the jointed concrete. These cracks occurred where there was no bond-breaker.

A project in Vicksburg, Mississippi, used a 152-mm (6-in) CRC to overlay an existing 203-mm (8-in) reinforced concrete pavement that was in a structurally good condition. A 2.54-mm (1-in) thick asphalt concrete bond-breaker was used. Ten years after the overlay was constructed, an inspection in August, 1981, revealed three distressed areas of punchouts, which were in sections where the CRC pavement was directly over the subgrade. Except for these areas, the unbonded CRC overlay was considered to be in good condition. No general conclusion can be made with regard to the effect the bond-breaker had on the pavement performance. The problem with punchout development appears to be more support-oriented.

A punchout study in Wisconsin[37] on a 203-mm (8-in) CRC overlay constructed in 1980 over a 254-mm (10-in) jointed concrete pavement with a nominal 38-mm (1.5-in) thick asphaltic interlayer noted several punchouts had developed over the original joint locations. Observations during patch repairs indicated the asphaltic concrete interlayer was undamaged except for areas directly below the punchout. In the punchout areas, the interlayer was either heavily damaged or completely missing. In many instances, the longitudinal steel was not ruptured but rather the concrete broke into small blocks and punched downward between the rebars. Deflection test results indicated that, during periods of low temperature, the deflections near the punchout were 110 percent larger than deflections taken between joints in the original pavement.[37] One conclusion of the study was that the loss of the interlayer was a secondary symptom that developed after the CRC pavement had been structurally damaged due to excessive deflection under traffic. The

Wisconsin study noted that the punchouts in the CRC overlay did not develop directly over the original joints but typically within a few feet downstream (with respect to the traffic flow) of them. In the aforementioned Pennsylvania punchout study, much of the cracking prior to punchout development occurred on the leave side of the original joint. The Wisconsin study indicated a mechanism that may explain the observed cracking pattern prior to punchout which is shown in figure 8. Figure 8 illustrates void formation under a CRC pavement resulting from no load transfer in the original jointed pavement. The report also noted that tensile cracks generally developed at the longitudinal steel, (figure 9), forming isolated blocks in the concrete pavement. Once load transfer (due to aggregate interlock) was lost because of slab "rocking" at the cracks, the isolated blocks would push between the steel without causing rupture. The interlayer material may also have disintegrated during the "rocking" process under load. The Wisconsin study concluded that adequate support is a necessary requisite for a CRC overlay on a jointed system to perform well. The load transfer in the existing pavement must also be given particular attention as must the presence of excessive crack widths.

CRC Overlays on CRC Pavements (Unbonded)

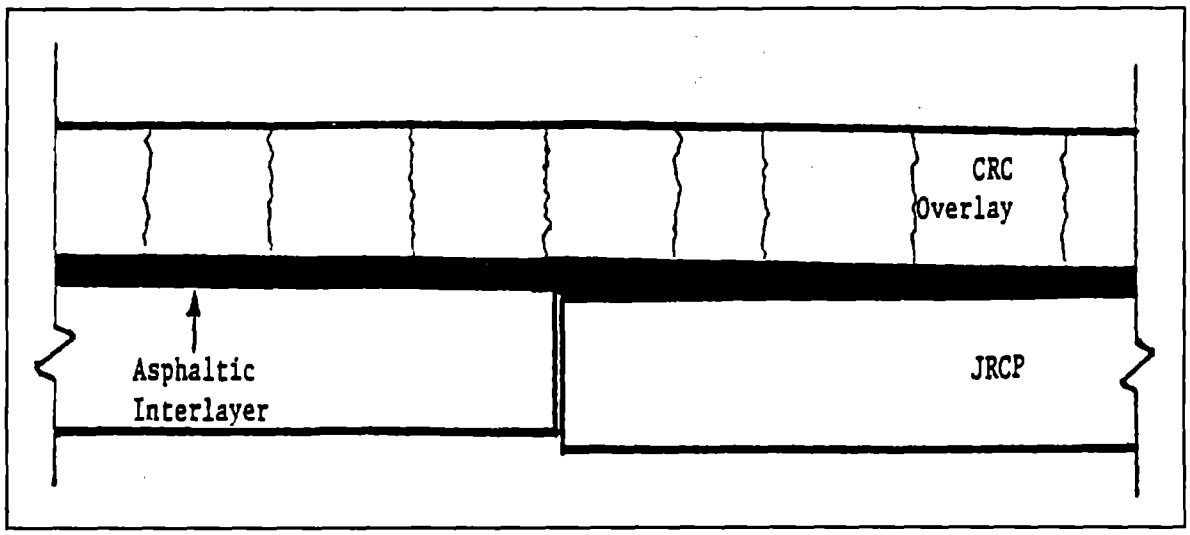


Figure 8. Longitudinal section of overall pavement structure at joint in JRCP illustrating cantilevering of CRC overlay because of differential vertical movement of JRCP slabs opposite joint.[37]

The first project of this type was constructed in 1981 in Mississippi. It was reported[28] in 1982 that the section began to show signs of longitudinal cracking in both lanes almost immediately. Spalling and extensive uncontrolled cracking followed.

Crawley[28] reported that the average maximum deflection of the original pavement was 0.017 mm (0.65 mils), with a standard deviation of 0.006 mm (0.24 mils). The ride quality, as determined by Mays Ride Meter, was 2.6. The average skid number for the pavement was 46. The average crack spacing was noted to be 2.1 m (7 ft). The cracks in

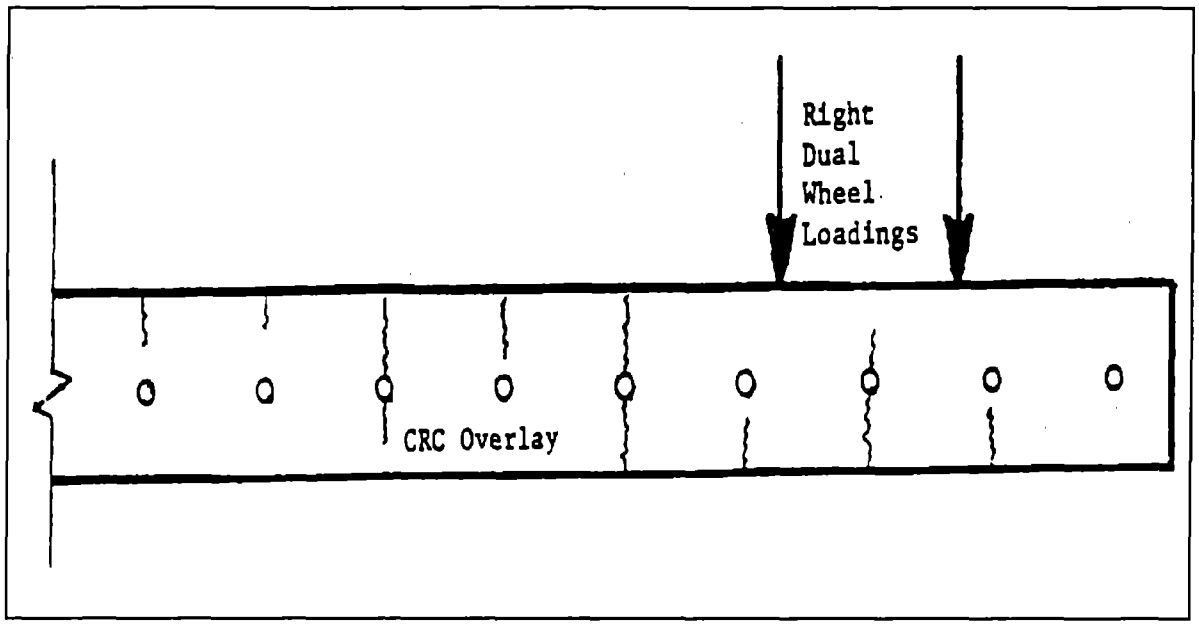


Figure 9. Transverse section of outside portion of driving lane of cantilevered CRC overlay illustrating formation of longitudinal cracks along planes of rebars in tension zones of the concrete.[37]

Figure 10. Extra wide crack in existing CRCP.

the existing pavement were very wide and there were many bituminous patches, as typified by figures 10 and 11, before the overlay was placed. Sixty-two distressed areas, as typified by figure 12, needed repair before the overlay was placed. The repair method chosen was to remove the concrete, steel, and asphaltic concrete currently in the distressed area and replace them with hot-plant-mix asphaltic concrete. It was thought that the resulting joints between the original CRC pavement and the new AC patch would result in an active joint because of the longitudinal movement of the CRC pavement. However, it seems as if these full-depth asphalt patches did not contribute to the load transfer in the pavement and may have contributed to its deterioration. Several reports have also cited instances of failure due to the presence of roller-compacted cement-treated base, which was highly susceptible to erosion, which would result in uneven support underneath the pavement. It may be assumed that the same

situation prevailed in Mississippi and that the support provided underneath the overlay was not sufficiently uniform.

Figure 11. Bituminous patch in existing CRCP.

Soon after construction, the overlay exhibited severe distresses in the form of spalling and punchouts. Inadequate design was thought to be the reason for the noted distresses. No transverse reinforcement was provided except where the shoulders were tied to the pavement using tie bars. Several punchouts had begun to develop soon after, and within 6 years, due to extensive maintenance problems, it was decided to overlay several segments of the CRC overlay with 114 mm (4.5 in) of AC.[38] Some sections of the CRC overlay are still being monitored. The primary reason for the failure of this type of overlay seems to be insufficient thickness and poor support and poor load transfer in the underlying CRC pavement.

Hence, it seems that consideration of the existing conditions of support underneath the overlay is extremely important in design. A lot of care must be taken to identify the distresses in the existing pavement system, analyze their cause and effect, and include the cause in design decisions. If this is accounted for in design by means of provision of sufficient thickness or uniform support, problems may not be encountered later. The role that underlying support plays in the performance of a CRC overlay is, therefore, very important and *must* be considered in analysis and design.

CRC Overlays on CRC Pavements (Bonded)

The use of a CRC bonded overlay has received a lot of attention in the past decade. Bonded overlays have been used to contribute to the structural capacity of the existing CRC pavement.[9] Bonded concrete has been used to resurface pavements that: (a) did not meet surface requirements during construction, (b) suffered surface damage during or

Figure 12. Close-up of crack spalling.

immediately following construction, and (c) required improvement in rideability or surface texture or both. After extensive field and laboratory test programs, Felt[39, 40] emphasized that the two main factors governing bond were the strength and integrity of the existing pavement and the cleanliness of its surface. It has been suggested that the good performance of a bonded overlay (when completely bonded) may be attributed to the fact that most of the tensile stresses would be taken by the existing pavement, unlike the unbonded resurfacing where the tensile stresses would have to be borne by the overlay.

A project in Road E-53, Greene County, Iowa, was completed as a study of concrete resurfacing for deteriorated highway pavement. The original pavement was 215 mm (8.5 in) of unjointed reinforced concrete constructed in 1921-22 and, although still serviceable, it contained extensive cracking, spalling, and raveling. Six sections were overlaid with varying thicknesses of bonded CRC in 1973. They were rated in 1978 on a scale of 0 to 100 on the basis of their condition and performance.[45] The 102-mm (4-in) CRC was rated highest at 84, and the 76-mm (3-in) CRC was rated at 54. The major defect noted at the time of the survey in the bonded CRC resurfacing was closely spaced transverse cracking accompanied by some spalling or raveling. Debonding was also observed.

Other projects[41] reported excellent performance after about 3 years of service, at traffic levels of about 5,000 AADT, with the only distress being observed as reflection cracking along the centerline where the longitudinal joint of the existing pavement was not matched and transverse cracking at some full depth patches in the existing pavement. The main problem with earlier projects seems to lie in an inadequate design and lack of proper guidelines regarding construction.

Five test sections were built in 1983 on the South Loop of Interstate Highway 610 in Houston, Texas. These sections consisted of three 51-mm (2-in) sections, and two 76-mm

(3-in) sections. The conditions and performance of these sections has been monitored regularly. These sections are performing well.

Following the reports of good performance on the IH 610 South Loop, 10 more sections were constructed on the outside lane of the IH 610 North Loop during December 1985 and January 1986. Delamination of the bonded concrete overlays was first noticed on the North Loop in March 1987. The Houston sections were surveyed, and a study[11] was made to see if delamination is a progressive problem. The study[11] concluded that delamination of bonded concrete overlays is an early-age problem that occurs within the first few weeks after construction. Delamination is not progressive over the period investigated, but the influence on the long-term performance of the pavement is uncertain. It was also concluded that, although the length of longitudinal cracking increased from 1987 to 1990, no evidence was found that it is related to traffic or time. Temperature difference was also found to be significant, but no conclusion was drawn since no data were available to indicate whether the pavement cracked before or after sawcutting.

Earlier, Voigt et al.[44] reported that delamination was found on two bonded CRC overlay projects over jointed concrete pavements. It was concluded that large temperature changes caused that delamination. The delamination occurred mostly at slab edges and transverse joints. However, later projects have shown that reflection cracking is predominant, and delamination of the overlay also seemed a good possibility, in which case, the overlay would have to take all the tensile strains induced, resulting in premature failure.

Criteria for Overlay Designs

The purpose of an overlay design procedure is to determine the overlay thickness required to provide a serviceable pavement over the design period. If the existing pavement is structurally inadequate for the expected traffic over the design period, the overlay thickness must be sufficient to increase the structural capacity of the pavement to carry the traffic for the design period. To provide these results, the following features should be included in the overlay design procedure:

1. The method must account for the many possible different conditions of the pavement prior to the time of the overlay.
2. The design procedure should account for subgrade support conditions as well as material properties in each of the pavement layers.
3. In the case of rigid pavements or overlays, representation of joints and cracks in both the existing pavement and the overlay must be possible.
4. Handling of multiple layers is required.
5. Accurate prediction of stresses, strains, and deflections is necessary if an analytical model is used.
6. The method should account for environmental influences on the performance of pavements.

CRC Pavement: Past and Existing Design Methodology

Early designs of CRC overlays were empirical and were based on the premise that CRC thicknesses did not have to be as great as jointed reinforced or jointed plain concrete pavement because of an assumed equivalency in structural adequacy.[15] One of the earliest forms of empirically developed overlay equations was based on the thickness deficiency concept, briefly described above and in detail later. The equations were initially developed for jointed plain concrete overlays of jointed plain concrete pavements, but have been modified since for applicability to all types of overlays. In case of CRC overlays, it has been proposed[20] that design thicknesses be reduced by a factor of 0.8 to make up for the costs and structural benefits provided by the steel reinforcement. Some of the earlier designs that were based on theoretical criteria are briefly discussed, followed by current methodology.

Early Theoretically Based Criteria for Overlay Design

Three theories[9] have been used for the determination of stresses and strains in rigid pavement systems: (a) finite-element plate (FEM) on an elastic-solid foundation, (b) plate on a Winkler foundation, and (c) elastic (visco-elastic) layer. The FEM elastic-solid theory is considered too complex and expensive to be used in a routine analysis. The plate on Winkler foundation may not be realistic since the theory is limited to two layers. The elastic layer theory, on the other hand, represented a simpler analysis procedure. It does assume, however, that the layers within a system are homogeneous and continuous in lateral extent. Thus, to make the theory applicable to rigid pavements that contain joints, cracks, and other discontinuities, certain functions must be handled by some other analysis methods such as the FEM-elastic-solid or engineering judgment. This, however, is a severe limitation to this theory.

Many authors have applied the elastic theory to the analysis of overlays, but few design procedures have been developed. In 1969, McCullough and Boedecker[21] made use of the linear-elastic layered theory for the design of CRC overlays. This paper was the basis for a method to design CRC overlays contained in a manual published by the United States Steel Corporation in 1970.[19] Four design charts were developed—three for highways and one for airfields. For the highway design charts (see Appendix A-I), the condition of the existing pavement is characterized as intact, broken, or shattered, which, like coefficient C in the previously discussed empirical form of equations, is based on the type and severity of cracking. The method recommends that the thickness of the overlay be greater than 76 mm (3 in) regardless of the value determined from the charts.

Treybig et al.[16] developed a similar method for the CRC overlays for airfield pavements in 1974. This method used nondestructive testing deflection measurements to characterize the existing pavement and, along with laboratory tests, to determine subgrade modulus values. The linear-elastic layer analysis was used to make a fatigue analysis for a range of overlay thicknesses and to determine a relationship between the overlay thickness and the level of reliability.

Corps of Engineers (COE) Procedure

The Corps of Engineers is one of the most widely used procedures. The Corps of Engineers[8] developed this procedure for the design of overlays for runways and taxiways. The U.S. Air Force and the Federal Aviation Administration (FAA) also use this procedure in their rehabilitation designs. The COE used accelerated test tracks to provide data to model the design of PCC overlays. Three models were developed for bonded, unbonded, and partially bonded overlays. The following equations were developed:

(1) Bonded Overlay: $h_o = h_d - h_b$

(2) Partially Bonded Overlay: $h_o^{1.4} = h_d^{1.4} - Ch_b^{1.4}$

(3) Unbonded Overlay: $h_o^2 = h_d^2 - Ch_b^2$

where

h_o = Concrete overlay thickness
h_d = Theoretical thickness if a new pavement were built
h_b = Existing pavement thickness
C = Coefficient depending on the structural value of the existing pavement and ranging between 0.35 and 1.00.

The equations for partially bonded and unbonded cases were subsequently changed to incorporate the variation in flexural strength between the existing pavement and the new pavement structure.[10] It is assumed in this method that fatigue is the cause for failure of the pavement.

Portland Cement Association (PCA) Procedure

The PCA procedure also used the COE equations to develop the design charts used to determine the required thickness of the overlay. The PCA procedure requires that a comprehensive evaluation of the existing pavement be made first. The evaluation program should consist of:

1. A visual pavement condition survey, to identify the type, quantity, and severity of all pavement distresses.
2. Nondestructive testing is done based on the results of the visual survey. If the condition survey indicates the existence of, or potential for, load-related distress, then load testing should be carried out to determine the severity of the problem. Load testing should be conducted at joints and cracks to determine the deflections. Results of load testing can be used to determine if loss of support exists and if load transfer across the joints and cracks is sufficient.
3. In-situ materials evaluation. The strength of the subsurface pavement layers is determined through back-calculation procedures. For the existing concrete

pavement, it is necessary to obtain representative values of the flexural strength and the modulus of elasticity. It is recommended that split tensile tests be performed on pavement cores. The design split tensile strength is determined using the following equation:

$$f_{te} = f_t - 1.65 * s_f$$

where

f_{te} = Effective split tensile strength, psi (1 psi = 6.89 kPa)
f_t = Average value of split tensile tests, psi
s_f = Standard deviation of the strength values, psi.

The design flexural strength is calculated as follows:

$$f_r = A * B * f_{te}$$

where

f_r = Design flexural strength, psi (1 psi = 6.89 kPa)
f_{te} = Effective split tensile strength, psi
A = Regression constant
B = Damage factor = 0.9.

The values of A range from 1.35 to 1.55. When available, a value of A based on local experience should be used. In the absence of local experience, a value of 1.45 should be used.[48] The constant B, equal to 0.9, is used to relate the strength of a concrete specimen obtained about 0.61 m (2 ft) away from the outside lane edge to that of concrete at the outside lane edge. It is assumed that concrete at the outside edge experiences higher stresses than concrete away from the edge. Thus, concrete at the outside lane edge will be more highly fatigued and will exhibit lower strength. The modulus of elasticity of the existing pavement may be determined by testing concrete cores in accordance with ASTM C469 or may be estimated from the following equation:

$$E_c = D * f_r$$

where

E_c = Design modulus of elasticity, psi (1 psi = 6.89 kPa)
D = Constant = 6000 to 7000
f_r = Design flexural strength, psi.

Stress data developed using JSLAB, a finite element program, were used to prepare design charts for the determination of unbonded overlay thickness.[48] These charts are applicable to existing concrete pavements having effective modulus of elasticity values ranging from 3,000,000 to 4,000,000 psi. Design charts are presented for three cases of existing pavement conditions. These cases are:

Case 1: Existing pavement exhibits a large amount of midslab and corner cracking and poor load transfer at the joints and cracks.

Case 2: Existing pavement exhibits a small amount of midslab and corner cracking. It exhibits reasonably good load transfer at the joints and cracks. Localized repairs are performed to correct distressed slabs.

Case 3: Existing pavement exhibits a small amount of midslab cracking and good load transfer at the cracks and joints. The loss of support is corrected by undersealing.

The design chart for case 1 was developed using data from the analysis of overlay sections containing a crack in the existing pavement directly under an edge load on the overlay. The design chart for case 3 was developed using data from the analysis of overlay sections with no cracking in the existing pavement. The design chart for case 2 was developed through interpolation between case 1 and case 3 conditions. The design charts are given in appendix A-II. The first step involved in the design process is the determination of the thickness of a full-depth concrete pavement for the overlay design traffic. The support condition used to determine this thickness was identified in the subgrade evaluation step. To determine this thickness, the following inputs are required:

1. Overlay Design Traffic
2. Pavement Support Condition
3. Concrete Flexural Strength

The full-depth thickness can be determined using the PCA or American Association of State and Highway Transportation Officials (AASHTO) method of new pavement design. The design charts are used to compute the thickness required for the overlay. Similar charts have also been prepared for bonded overlays.

FHWA/Texas Procedure

The FHWA/Texas design method (figure 13) formed the basis for subsequent improvements by Schnitter et al.,[12], Taute et al.,[23], and Seeds et al.,[34] for specific application in Texas. There are two major design concepts included in the procedure: (1) design against fatigue cracking due to repeated wheel loads, and (2) design to reduce or minimize reflection cracking. The procedure is automated[24] and four different computer programs are used as can be seen in table 2. The RPOD1 program is used to determine the required overlay thickness to prevent fatigue cracking and RFLCR1 is then used to check for reflection cracking. A flow chart of this procedure can be seen in figure 13.

Elastic-layer theory is the basic computational model for the fatigue analysis. It computes stresses and strains within the pavement system that are then input to a fatigue relation between stress or strain magnitude and repeated wheel load applications prior to

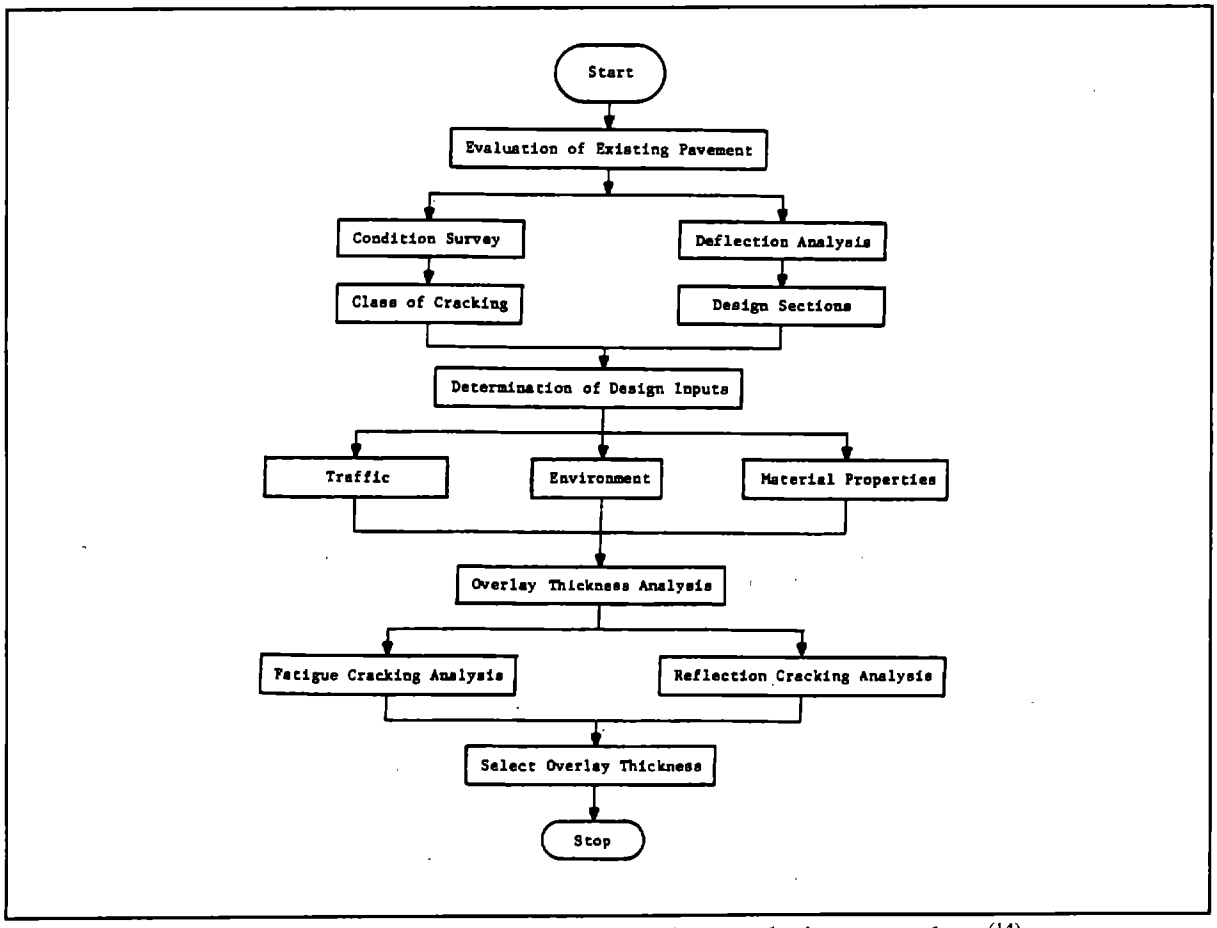


Figure 13. Flow chart of the FHWA/Texas design procedure.[14]

cracking. The pavement overlay thickness that reduces the design stress or strain to a level that will allow the desired pavement life is selected as the overlay design thickness.

Reflection cracking is analyzed through the use of a newly-developed mechanistic model,[24] which describes reflection cracking as a function of strength and creep properties of the existing materials, thicknesses of the pavement layers, volume changes due to temperature, friction forces between layers, and the width of stress relief layers. The overlay thickness as well as any other type of treatment necessary to reduce or minimize reflection cracking is selected on the basis of these factors.

AASHTO Procedure

The AASHTO method follows the COE method with a few variations. Serviceability-traffic concepts used in the general AASHTO design method are also used in the overlay design method. It uses life-cycle cost concepts to select the most cost-effective overlays. The guide also endorses nondestructive test methods for material characterization of in-situ pavement layer properties. The following equations are used for rigid concrete overlays on rigid pavements:

(1) Bonded overlay:
$D_{OL} = D_y - F_{RL}(D_{xeff})$

(2) Partially bonded overlay:
$D_{OL}^{1.4} = D_y^{1.4} - F_{RL}(D_{xeff})^{1.4}$

(3) Unbonded overlay:
$D_{OL}^2 = D_y^2 - F_{RL}(D_{xeff})^2$

where

D_{OL} = concrete overlay thickness
D_y = theoretical thickness if a new pavement were built
D_{xeff} = existing pavement effective thickness
F_{RL} = remaining life factor taking into consideration the remaining life of the existing pavement before overlay as well as the remaining life of the overlaid system after the overlaid traffic has been reached.

Table 2. Computer programs used in the FHWA Design Procedure.[24]

PROGRAM	FUNCTION
PLOT2:	Deflection Profile Plots profiles from measured deflections
TVAL2:	Statistical Analysis of Design Sections (1) Determines statistically whether selected design sections are significantly different (2) Determines means and standard deviations of deflection data (3) Determines design deflections
RPOD1:	Fatigue Cracking Analysis (1) Characterizes subgrade material using design deflection and laboratory data (2) Does remaining life analysis using Miner's linear damage hypothesis (3) Determines overlay thickness for specified design life, using fatigue principles
RFLCR1:	Reflection Cracking Analysis (1) Computes horizontal, thermally induced, tensile strains in AC overlay (2) Computes vertical, load associated, shear strains in AC overlay due to differential deflection at discontinuities in existing pavement

The terminal PSI value of 2.0 at the end of the overlay life is suggested by the AASHTO Design Guide. Methods are proposed in the Guide for determination of the existing pavement effective thickness as well as the remaining life of the existing pavement. Theoretical thickness, D_y, is the thickness calculated for a new concrete pavement that will last until the design overlay equivalent single axle load (ESAL) is obtained. The methods for determining most of the values are based on empirical tests.

Recently, a revision of the AASHTO overlay design procedure[53] was proposed. This procedure is based on the concept of structural deficiency, in which the required overlay thickness depends on the difference between the required structural capacity for future traffic and the effective structural capacity of the existing pavement. The required

structural capacity and existing structural capacity are determined from the AASHTO rigid pavement design equation.[1]

The AASHTO model was developed to predict the traffic that a PCC pavement could carry to failure, i.e., terminal serviceability. Thus, the failure criterion for the overlay thickness obtained using this model and the structural deficiency approach is also terminal serviceability. For an unbonded overlay design, this revised version recommends that the modulus of elasticity and modulus of rupture inputs used be the mean values for the overlay rather than for the existing PCC slab. A new PCC slab typically has an elastic modulus of 20,000 to 35,000 mPa (3 to 5 million psi) and a modulus of rupture of 4 to 5.5 mPa (600 to 800 psi). It is also recommended in reference 53 that the J load transfer factor should reflect the load transfer capability of the overlay, rather than that of the existing pavement, and that the design static k-value should be an appropriate value for the foundation beneath the existing pavement.

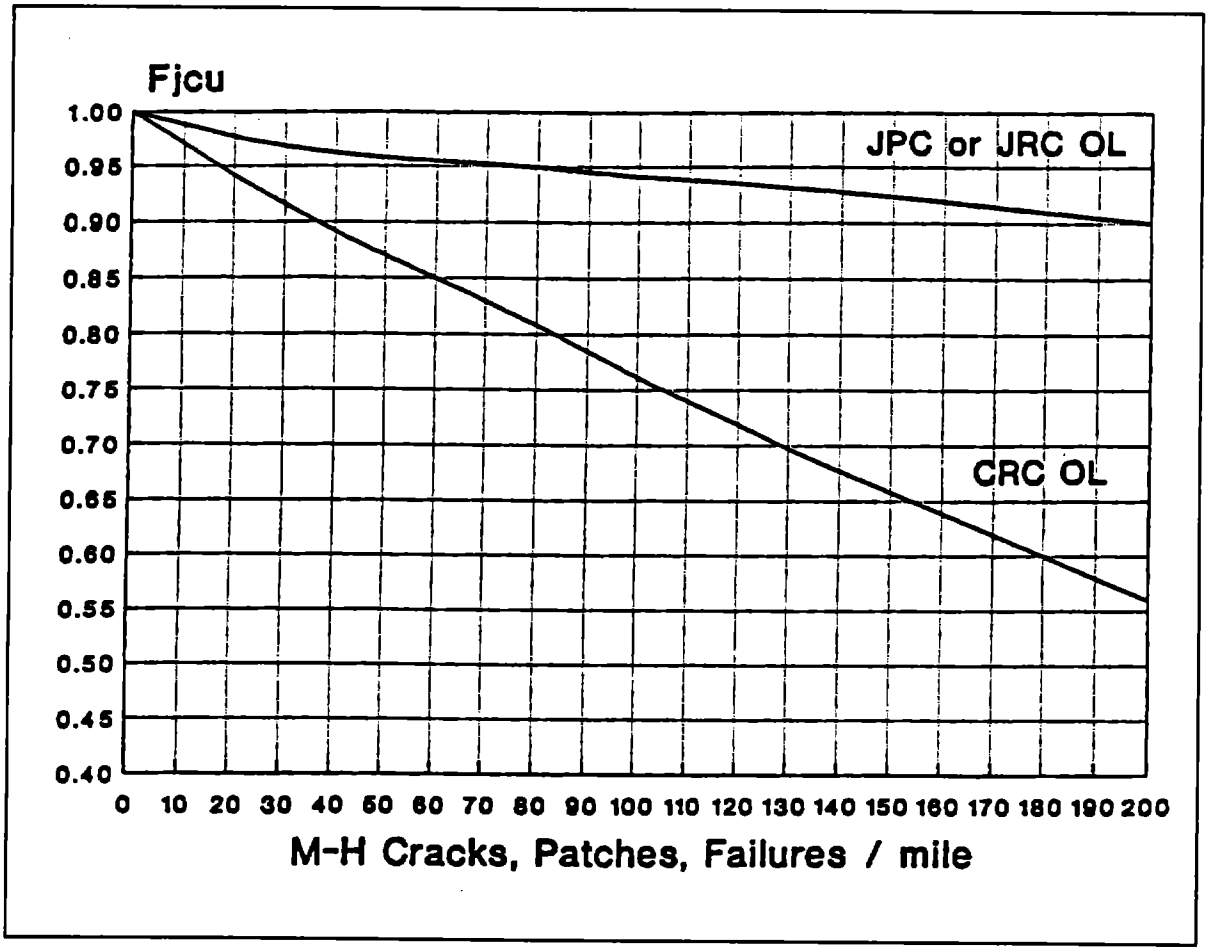
An F_{jc} factor was recommended to be used in order to account for the serviceability loss as result of the deterioration of cracks, patches, and localized failures. Unbonded CRC overlays have been shown to be more dependent on the underlying pavement's condition, particularly the uniformity of support and the degree of load transfer at the joints and cracks. The F_{jc} curves recommended in reference 53 are shown in figure 14.

Critique of Existing Design Procedures

Upon review of the available literature and field information of the various types of overlays involving the use of CRC, an appraisal of the available design methods in light of how appropriate they are for the thickness design of overlays is presented below.

It is apparent that the COE method is the most widely accepted, and has become the most popular method used today. However, one of its main drawbacks is that only the partially bonded and the unbonded cases consider the state of the existing pavement. The assignment of the condition factors that are used to account for the condition of the existing pavement is also empirical. One of the major drawbacks in this design method is that the COE defines failure as the application level, or the coverage, at which cracking or structural breakup first occurs; this, from a fatigue standpoint is an unacceptable criterion for highway overlays, especially for CRC pavement in which the pavement is expected to develop transverse cracks.

The PCA method assumes that a stress limit exists, and the limit is based on plain concrete tests. The PCA design procedure is also a type of thickness deficiency method. The PCA design is based on the results of the JSLAB program, which cannot model curling and warping stresses. The exclusion of these stresses can produce a non-conservative design. If these stresses are not included in design, stress ratios can be significantly underestimated, and fatigue life can be significantly reduced from its expected values. Hence, the use of this method to design any of the aforementioned overlays will not be appropriate.



1 mi = 1.7 km

Figure 14. Adjustment factor for the design of jointed and CRC unbonded overlays.[53]

The FHWA/Texas design method uses fatigue-based criteria. The consideration of the phenomenon of reflection cracking is also made, and the entire design is based on a fatigue concept to prevent class 3 and class 4 cracking (as defined in the AASHTO Design Guide). However, fatigue cracking is not as much a problem with CRC overlays as is the provision of support. No consideration of the existing support conditions in design is made except in the characterization of the subgrade or the subbase.

The 1986 AASHTO Design Guide was able to model a CRC slab analytically and calculate the structural response of the CRC pavement on the basis of the influence of factors such as crack spacing and other environmental conditions. How to predict or estimate the cracking or distress type, such as punchouts, correctly is a key problem for CRC pavement overlays, just as it is for CRC pavements. This problem has been shown to be closely related to pavement stiffness.[2] The AASHTO Design Guide[1] as well as the proposed revision both have their failure criterion defined as terminal serviceability. It is also based on a structural deficiency approach. One significant limitation to this approach is that the same thickness of the unbonded overlay will perform better on a thicker existing

pavement than on a thinner pavement. This design may only be applicable if sufficient support is provided to ensure that the pavement life is determined as a function of wear-out of the aggregate interlock at the transverse cracks. It is by no means well established that unbonded overlay performance is as sensitive to existing pavement thickness as the empirical equation suggests. A second significant limitation is that it is based on the assumption that the existing PCC slab and overlay slab are approximately equal in flexural strength, which is erroneous.

The existing design procedures for concrete overlays are based on several artificial factors, and the existing predictive models for faulting and cracking developed from the database may not provide a consistent evaluation for concrete overlay design. Because of all the deficiencies in the existing design methodology for the design of CRC overlays, a new framework of design is sought that will encompass the development of existing failure modes and the prevention of such as the primary basis for design. Recent developments in mechanistic analysis for concrete overlays should be used to consider both existing slab and overlay thickness, mechanistic data such as existing pavement stiffness, and overlay concrete properties.

CHAPTER 3 - DESIGN FRAMEWORK FOR OVERLAYS INVOLVING CRC PAVEMENT

A significant amount of literature was found to exist for CRC overlays and the maintenance and rehabilitation of CRC pavements. Several of the field experiences with CRC pavement and overlays, if synthesized properly, will provide guidelines upon which future designs may be based. This chapter will deal with the development of a design framework for overlays involving CRC pavement on the basis of the field performance observations and results of the literature survey.

Field, laboratory and analytical studies have shown that, of the different cracking and distress types that can develop in concrete pavements, shrinkage and debonding between the overlay and the underlying pavement tend to be the most prevalent distress in CRC pavement bonded overlays, and loss of support and spalling are more common in unbonded overlays. However, the analytical models for predicting the structural responses of a CRC overlay on the basis of these types of distress or failure mechanisms have only been partially developed for the design of concrete overlays.[2] Also, it is not clear from the literature whether a particular distress should be addressed in design or in construction. In other words, the information and knowledge available for developing new analytical and predictive models for CRC overlay design are limited.

Unbonded concrete overlays can be used to correct many deficiencies in concrete pavements. This technique may be cost-effective when the condition of the existing pavement is structurally deficient or has deteriorated past the point of economical restoration. The performance of unbonded concrete overlays is primarily dependent upon the bond-breaker material used and the thickness of required overlay. Poor performance of some unbonded overlays has been linked to the type of separation material used. When a bond-breaker material is incapable of remaining in place or isolating the movements of existing pavements from the new overlay, cracking in the overlay is accelerated. Several authors[2, 36, 37] have suggested an acute need to consider the support conditions underneath the existing pavement for the analysis and design of CRC overlays. Past research work has provided a preliminary basis for initiating the analysis and design for CRC pavement overlay systems. Some of the factors to be considered in the design framework for overlays involving CRC pavements are:

(a) Type of existing pavement:

Whether the existing pavement is rigid or flexible will make a considerable difference in the approach to be adopted in design. If a CRC overlay were to be placed on an existing flexible pavement, the design may be that of a new pavement since the asphalt layer will most likely not contribute significantly to the pavement system structurally. The asphalt pavement will act as a high strength base, reducing the potential of pumping. If an unbonded CRC overlay were to be placed on an existing rigid pavement, a two layer analysis must be performed to determine the required overlay thickness. A bonded overlay would require the consideration of a composite system for the pavement and the overlay.

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(b) Evaluation of the existing pavement:

Both the structural and functional condition of the existing pavement must be evaluated to provide input for the design of CRC overlays. Nondestructive testing (NDT) may be done to provide the needed information. Deflection measurements using the Falling Weight Deflectometer (FWD) also provide useful information regarding the basin area and the load transfer efficiency of the pavement. Other system properties like the k-value, the layer modulus, and the radius of relative stiffness may also be back-calculated from the measured deflections. The information may be synthesized to analyze the structural and functional condition of the existing pavement.

This chapter presents a framework for the design of CRC overlays based on the stiffness of the pavement system. Steel design is not addressed since its effect on the stiffness of the pavement system is negligible. The AASHTO design procedure provides a method to determine the design of steel depending on climatic factors.

CRC Pavement Performance Factors

The type of distress predominant in CRC overlays as well as CRC pavements is punchout distress and is of great concern to the highway engineer. It occurs most frequently in CRC pavements but the mechanism associated with it may occur in other rigid pavement types. Punchout distress is defined as a structural failure. It is typically associated with close transverse cracking bounded by a longitudinal crack on one side and a pavement edge on the other. Punchouts frequently develop at Y-cracks and may develop at an interior position, although not as frequently. A study in Texas[56] noted that most failures occurred in the outside lanes. Another study by La Coursiere et al.[57] of CRC pavements in Illinois indicated edge punchouts as a predominant structural distress. The causes of an edge punchout have been attributed to repeated traffic loadings and loss of support.[57] The study by La Coursiere pointed out that *loss of support leading to high pavement deflections under heavy loading* was the primary factor causing punchout distress. The mechanism associated with the development of punchouts has been previously noted and summarized[2] and has not been elaborated here. However, it has been recognized that the failure aspects pointed out above are entirely encompassed within the framework of the punchout mechanism and must be considered in design. The design of a CRC overlay or any overlay of CRC pavements must then take into consideration the development of pavement distress from two different but related failure aspects consisting of fatigue cracking and pavement support. Other distresses that have been associated with CRC overlays seem to be either an outcome of the punchout distress (like joint faulting and joint spalling) or the cause of it (like subbase erosion, pumping etc.).

Punchout development has largely been associated with poor foundation support and consequent loss of load transfer because of vehicular fatigue loading. Hence, good performance is analogous to the provision of uniform support. To minimize fatigue cracking, load transfer-thickness requirements should be adopted. Pavement support, uniformly provided and distributed, has been recognized for several years as the key to long-term performance of CRC pavements and, more recently, for CRC overlays as well.

However, the consequence of lack of uniform support seems to have been considered directly only in the design of CRC pavements based on the erodibility of the subbase material. Experiences in Pennsylvania[36] and Wisconsin[37] and more recently in Arkansas (subsequently reported) seem to indicate the need for considering a nonuniformly supported condition for CRC overlays, especially those on jointed concrete systems.

Characterization of support for the purpose of support analysis for design must encompass measured field data in order to predict the structural response of the pavement system. Nondestructive testing such as FWD provides useful information about the existing pavement like the plate deflection, load transfer efficiency and the deflected basin area. Based on the analysis of the field data collected from I-30W in Arkansas, described below, the following types of field information may be useful in the characterization of the existing pavement support.

Basin Area

Basin area is the deflected shape of the basin under a given load. In essence, it is a measure of the amount of deflection that the load will cause at specified distances from the load. The deflection decreases as the distance from the load increases. After a study of deflections along a CRC pavement section in Indiana, it has been suggested[50] that any deflection higher than 0.23 mm (9 mils) is representative of poor pavement support.

The amount of deflection is dependant on the thickness of the pavement and the base underneath and the layer moduli of each as well as the k-value of the subgrade. If the pavement system were considered as a slab-on-grade, the basin area would be a function of the k-value of the subgrade and the "stiffness" (which is described later) of the pavement system.

FWD gives the response of the pavement surface in terms of the deflection. Westergaard's solutions for slab-on-grade, interior position of the load deflections allow the back-calculation of a k-value underneath the pavement and atop the base. An effective value for the elastic modulus may also be derived in the same manner.

Permanent deformations of the subgrade or loss of support cause a discontinuity of support. This is considered one of the stress-inducing factors in the analysis of rigid pavements.[7] A concrete pavement slab deforms under load depending on the position, magnitude and area of contact of the load on the pavement surface. The resistance to deformation depends on the "stiffness" of the supporting medium, as well as the flexural stiffness of the slab. This parameter is called the radius of relative stiffness (ℓ) and depends on the properties of the slab and the foundation.

$$\ell = \left(\frac{E\, h^3}{12\,(1-\nu^2)k} \right)^{\frac{1}{4}}$$

where

E = Concrete modulus of elasticity (psi)
h = Thickness (in)
ν = Poisson's ratio
k = Foundation modulus (psi/in).

The term Eh^3 can be referred to as the stiffness of the pavement system. Given two pavements with the same slab modulus of elasticity, but different types of subgrades, the CRC pavement on the weaker foundations will deflect more, giving a decreased resistance to deformation or a lower ℓ, which corresponds to a lower Eh^3 term. By increasing foundation support, the deflection under the load decreases, thus giving a flatter basin area and improved resistance to deformation (figure 15). Similarly, two pavements with different stiffnesses (Eh^3 terms) will deform differently, the pavement with the lower stiffness deforming more than the other, corresponding to a basin shape indicating poor support (figure 16).

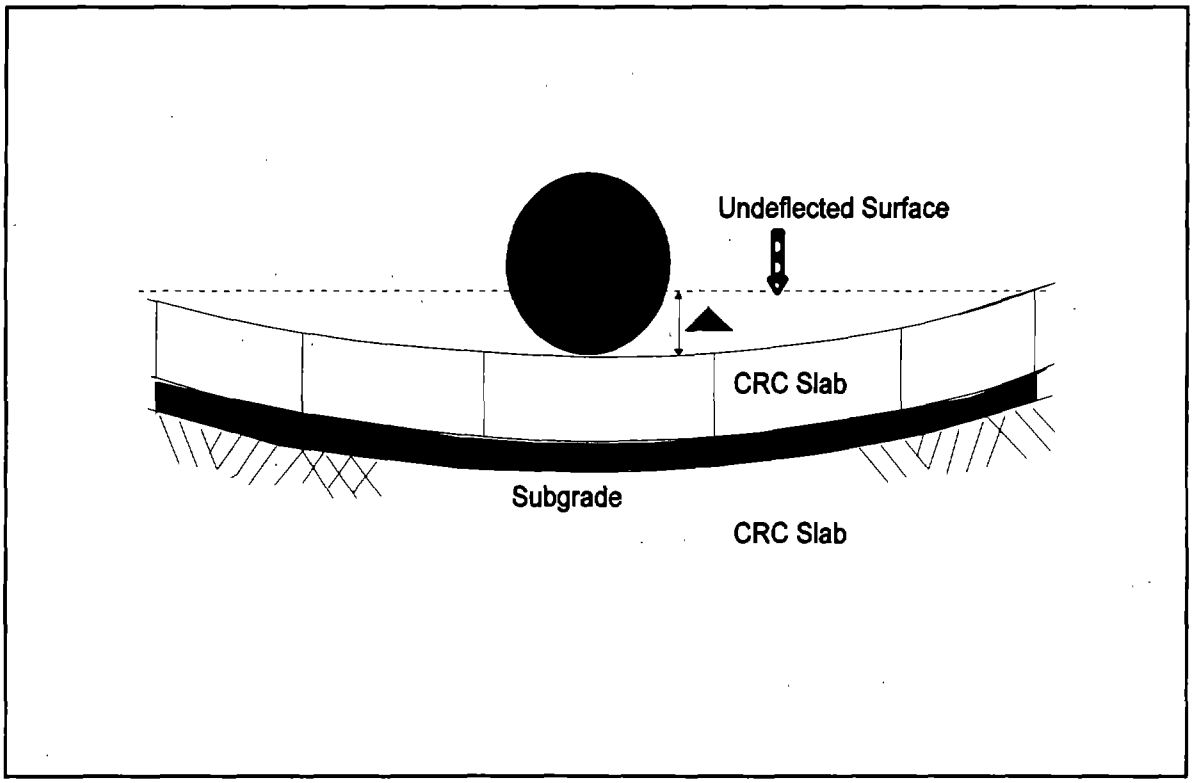


Figure 15. Shape of the deflection basin under a slab of high stiffness and weak subgrade support.

Load Transfer Efficiency

Joints have been recognized as the focal point for pavement distress in jointed concrete pavements and, consequently, are in many instances the sources of problems that develop in CRC overlays placed on jointed pavement systems. Data related to slab

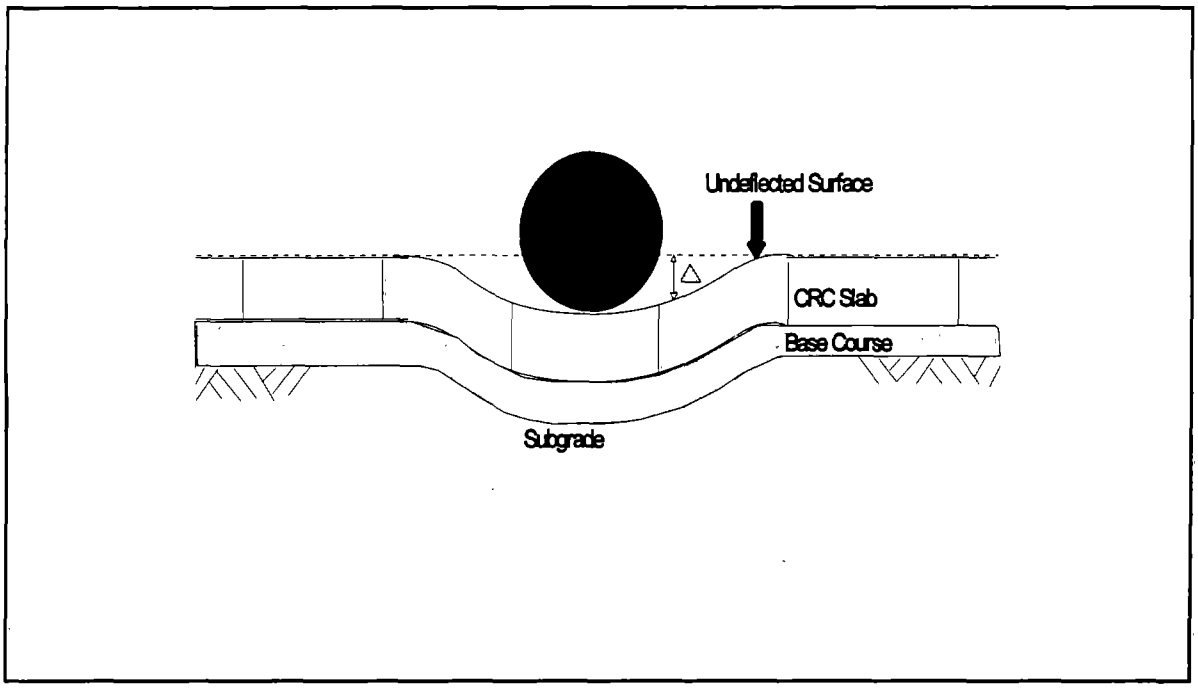


Figure 16. Shape of the deflection basin under a slab of low stiffness and strong subgrade support.

deflections and joint load transfer efficiency (LTE) are obtained during FWD testing and are a primary way of characterizing support conditions under the original pavement. The LTE may be defined as the deflection on the unloaded side of the joint or crack divided by the deflection on the loaded side of the joint (figure 17).

$$LTE = \Delta L/\Delta A \text{ X } 100\ \%$$

where

ΔL = Unloaded deflection
ΔA = Loaded deflection.

Experience with CRC overlays on JRCP in Pennsylvania[7] showed that most punchouts were on joints in the existing pavement. It was also noticed that, at places under the punchout, there was no bond-breaker present and, in places where there were no punchouts, the bond-breaker was intact. This seemed to indicate that particular attention must be paid to joints in the existing pavement, especially in their capacity to transfer load from one side of the joint to the other. As mentioned earlier, loss of support underneath the pavement will result in a loss of load transfer across the transverse cracks and, hence, distresses like joint faulting and joint spalling will be manifest. A CRC overlay placed on such a pavement will be highly susceptible to punching out as a result of problems in the existing slab.

Therefore, the LTE of joints in the underlying pavement have a tremendous effect on the stresses that develop in a CRC overlay and, consequently, on the degree of

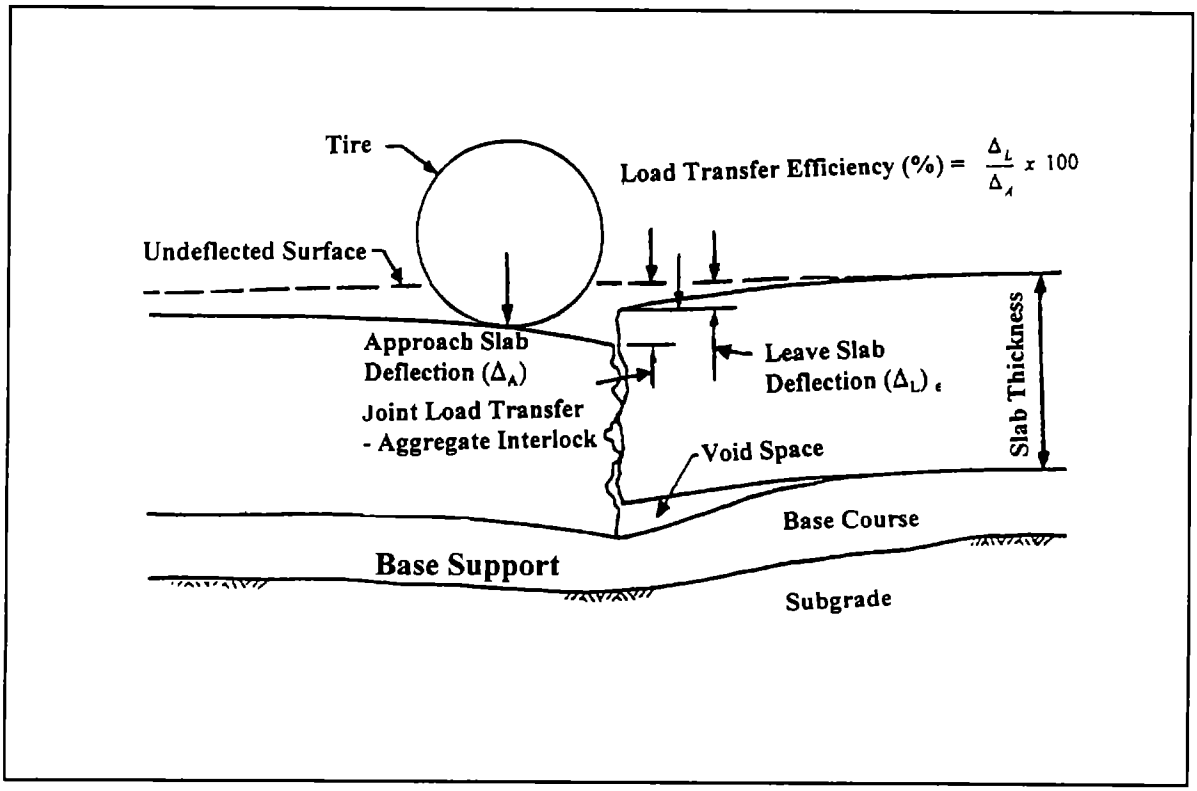


Figure 17. Load transfer efficiency.[51]

performance expected of the overlay under repetitive loading. A perfectly efficient system for transferring load from one side of the joint to the other can significantly reduce the deflection that would occur from a free edge condition.[51] The objective of a perfectly efficient system for transferring load is to minimize tensile stresses and, in the case of an overlay, to minimize deflections in the pavement that result when loads are applied at a joint in the slab. However, it is important to point out that a perfectly efficient system of load transfer under a CRC overlay as provided by an existing jointed system ensures complete deflection compatibility under load between overlay and the supporting slab.[13]

A lower LTE in the existing pavement will result in a lower "stiffness" of the slab and a lower degree of support for the overlay. To increase the "stiffness" of the pavement system, the overlay should have a greater thickness to make up for the lower stiffness of the existing pavement. In general, low values of LTE would require overlays with greater design thicknesses and vice-versa.

Load transfer is therefore noted to have a significant effect on the performance of CRC pavements. Colley and Humphrey[43] investigated load transfer across joints through aggregate interlock in concrete pavements in the field and laboratory. The laboratory tests consisted of repetitive loads applied to slab specimens of 178 to 229 mm (7 and 9 in). It was observed that, as the number of load cycles increased, the load transfer effectiveness (in percent) decreased for the same magnitude of joint opening. When applied to CRC pavements or overlays, these results imply loss of load transfer efficiency with increased load applications.

The test results further showed that increased pavement thickness improved load transfer efficiency dramatically, as did a stabilized base (in the case of an overlay, comparable to the provision of support afforded by the existing pavement). It should also be noted that a decrease in load magnitude significantly decreased the rate of loss of LTE. This implies a shear stress effect on LTE.

A study by Zollinger and Berenberg[2] modeled varying degrees of LTE with the ILLI SLAB program and noted that, as LTE decreased, the transverse bending stress (illustrated in figure 18) becomes critical and is a causative factor leading to the formation of a longitudinal crack approximately 7.62 m (30 in) from the edge. The formation of the longitudinal crack is a step in the punchout process. Hence, an item of most concern should be the quality of load transfer across the transverse cracks throughout the pavement life.

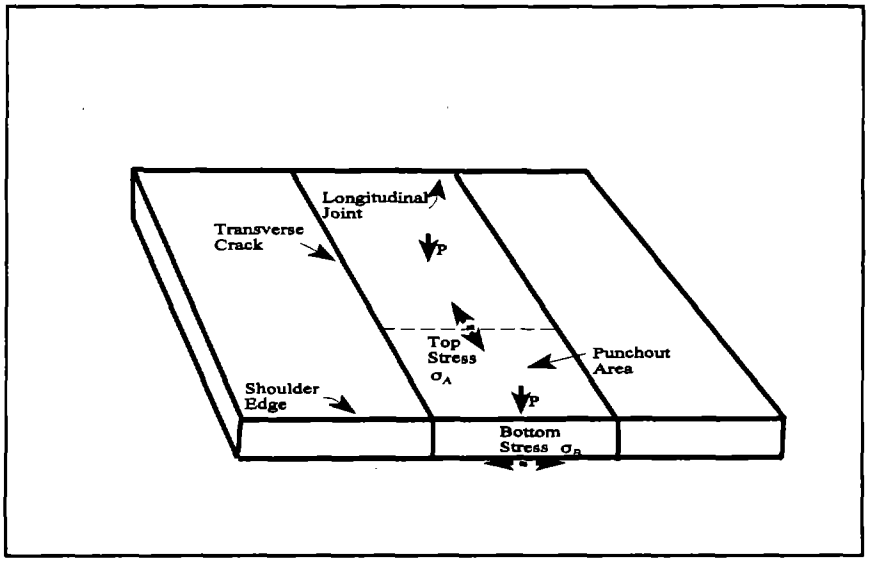


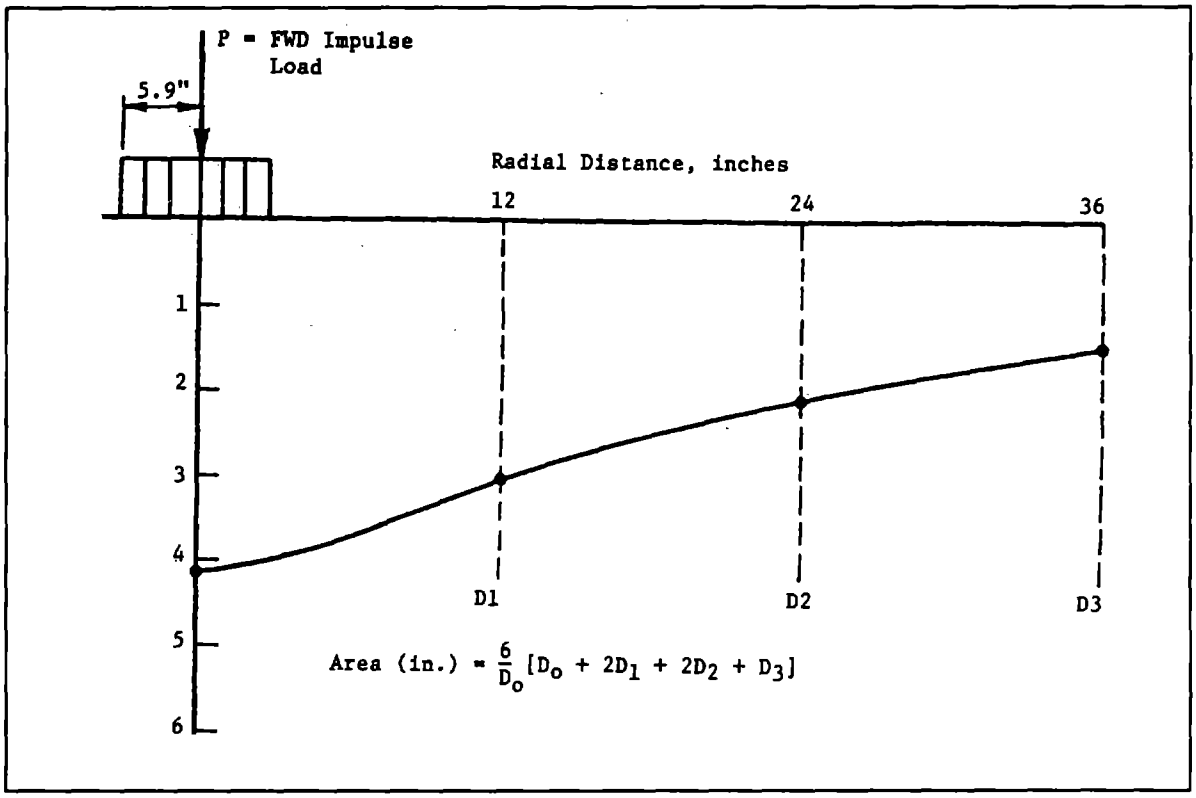
Figure 18. Critical wheel load stresses in a loaded CRC pavement system.[2]

Characterization of Existing Support Conditions

Localized stresses can be quite intensive, leading to functional failures in the pavement. These stresses result from unsupported conditions and dictate consideration of design or construction methods to minimize the effect of unsupported conditions. The characterization and analysis of the support under a CRC overlay is based on the "effective stiffness" of the pavement system as determined from NDT results.

The use of the FWD data seem to play a key role in developing an approach to CRC overlay thickness design. The deflections measured can be used to calculate the basin area and to provide useful information for the pavement support analysis. Other parameters of the existing pavement, like the foundation modulus or the k-value and the elastic modulus, may also be back-calculated where necessary.

The results of the FWD field measurements are described in terms of the plate deflection (D_0), the LTE, and the deflection basin area (figure 19). The basin area has been shown to be useful in interpreting measured deflection profiles by the FWD[47] and is calculated from the sensor deflections as:



1 in = 25.4 mm

Figure 19. The deflection basin area concept.[51]

$$\text{Basin area} = (\text{spacing}/2D_o)[D_o + 2\{D_1 + D_2 + \ldots + D_{n-1}\} + D_n]$$
$$= (6/D_o)(D_o + 2\{D_1 + D_2\} + D_3)$$

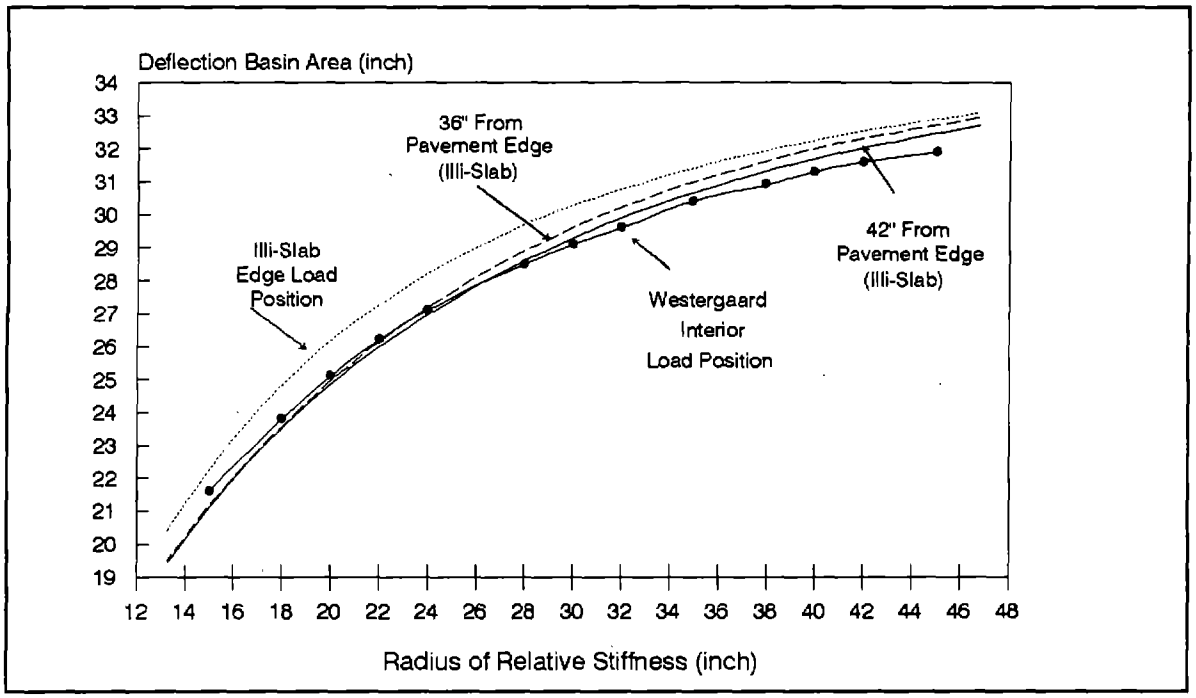
where

D_i = measured sensor deflection (i= 0,n)
n = number of sensors on one side of load plate minus one
spc = sensor spacing.

The area equation expressed for four sensors at 305 mm (12 in) centers, has units of length usually expressed in inches and the maximum value of area is equal to the sensor spacing times the term n.

The NDT results are also useful in back-calculating material properties of the pavement system such as the modulus of elasticity (E) and the modulus of subgrade reaction (k). Particular interest lies in back-calculated E values and the radius of relative stiffness (ℓ) at the transverse cracks, which is a function of E and k. A method of back-calculation of these parameters based on ILLI SLAB modeling of the deflection basin (iterative calculation with various values of concrete modulus (E) and the foundation modulus (k) to match the deflection basin measured by the FWD for a given load condition), has been discussed by others for jointed concrete pavements.[47] The back-calculation can also be

accomplished for a range of thicknesses using a deflection basin area-radius of relative stiffness relationship plotted in figure 20.



1 in = 25.4 mm

Figure 20. Variation of deflection basin area with ℓ.[2]

Unique E and k values can be determined from ℓ (radius of relative stiffness) values obtained from figure 20 and Westergaard solutions[49] for slab-on-grade deflections at the edge and interior load positions. The theoretical interior loading solution is applied to the wheel path load position in the actual pavement since the load behavior between the two positions was shown to be similar. Simplified forms of the Westergaard solutions rearranged to solve for k value (assuming μ is 0.15 for concrete) are:

Interior Load Position:

$$k = \frac{P}{8D_0\ell^2}\left(1-\left(\frac{a}{\ell}\right)^2(0.217-0.367\log\frac{a}{\ell})\right) \quad (1)$$

Edge Load Position:

$$k = \frac{P}{2.32D_0\ell^2}\left(1-0.820\left(\frac{a}{\ell}\right)\right)$$

where P is the applied load, a is the radius of the FWD load plate 150 mm (5.9055 in), E is found from the expression for ℓ (explained in a previous section), and k is determined from the above equations, leading to:

$$E = k(\ell)^4(11.73)/h^3 \quad (2)$$

where h is the pavement thickness.

The response in terms of the radius of relative stiffness is similar to the previously discussed parameters, LTE and basin area. This suggests that ℓ may also be useful in determining potential punchout areas in CRC pavement systems. Since there exists a relationship between the radius of relative stiffness and the basin area (figure 20), interest lies in the characterization of the support underneath the pavement system in terms of the deflection basin area. A poor basin area, therefore, is also representative of poor support conditions and, consequently, a potential area in which punchout distress can develop.

Pavement Structural Analysis

Application of theoretically sound mechanistic concepts to the structural evaluation of the existing pavement has been prompted by the development of commercially available devices for NDT like the FWD. This can be achieved by matching the theoretically predicted response of the system with the corresponding behavior observed "in situ" through the selection of appropriate system parameters such as layer thicknesses and moduli.[52]

Based on field studies, failure modes relating to punchout distress have been proposed as the fundamental design mechanism for CRC pavements as well as CRC overlays. The analysis of these failure modes is based *a priori* on the existence of uniform support conditions.[2] A good indicator of existing support conditions is in the measured deflection basin area. Hence, an analysis should incorporate a structural model that will allow the matching of the basin area as it is measured in the field.

The deflection profile for a pavement system as present in the field may be generated using the ILLI SLAB finite element program. The ILLI SLAB program enables the modeling of various conditions such as different levels of LTE, positions of the load, and varying subgrade k-value. The program may be used to determine the deflection basin under the load and pavement system and LTE as it exists in the field. The basin area as measured in the field from FWD is matched with the basin area generated by ILLI SLAB using suitable input values.

A closed-form solution was been suggested by Ioannides et al.[52] for back-calculation purposes and has been referred to subsequently. A slab with a joint/crack is characterized according to field conditions so as to simulate support conditions and joint LTE, which resembles that provided by the existing pavement. The purpose of this is to back-calculate either an "effective" layer modulus or a composite k-value as determined by collected

field data. An "effective" layer modulus or an "effective stiffness" may then be determined for every level of LTE measured across the joint/crack.

For the purpose of analysis, two categories are presented:

(1) Unbonded layers
(2) Bonded layers

In either case, it is most appropriate to consider the base or the subbase as a part of the pavement support.

Unbonded layers

Two layers are unbonded when they have an interface between them and act independently of each other. For modeling an unbonded condition, a two layer analysis may be used where the existing pavement is modeled atop a stabilized base (if one exists) and the subgrade. This approach may provide a back-calculated k-value for use in the overlay design (an effective layer modulus[52] may also be considered below, in keeping consistency with the above statement). If the existing slab has no stabilized base, two-layer analysis is appropriate for an overlay using as input a subgrade k-value or a back-calculated subgrade k-value determined using Westergaard's analysis and the position of the load.

The medium thick-plate theory provides that an "effective" plate exists for any two layers and its structural parameters can be defined in terms of the layers in the original pavement.[52] For any two unbonded layers, then,

$$E_e h_e^3 = E_1 h_1^3 + E_2 h_2^3 \qquad (3)$$

where

$E_e h_e^3$ = Effective or composite pavement stiffness
E_i = Flexural moduli of the layers in the original pavement
h_i = Thicknesses of the layers in the original pavement

The subscript e denotes the properties of the effective layer imagined to rest on the same foundation as the original composite pavement and the subscript i denotes the layer number.

Thus, a term for the "effective stiffness" for the pavement system may be derived from the layer thicknesses and moduli of the constitutive layers.

Bonded layers

Two layers may be termed "bonded" when there is no interface between them and they act like a single, composite layer. A bonded overlay may be modeled by treating the overlay and the existing slab as one layer with an effective stiffness by the use of the parallel axes theorem. Ioannides et al.[52] used this theorem, which involved determining the

neutral axis of the composite system in the calculation of an effective thickness. This composite layer can be modeled atop the base and the subgrade using a back-calculated k-value for the slab support. As noted above, the flexural stiffness of such a "composite" the original pavement may be determined using the parallel axes theorem.[52] Here, the composite layer consisting of the two bonded layers is assumed to act as one plate whose neutral axis is at "x" from the top of the plate:

$$\frac{E_c\, h_c^3}{12} = \frac{E_1\, h_1^3}{12} + E_1\, h_1 \left[x - \frac{h_1}{2} \right]^2 + E_2\, h_2 \left[h_1 - x + \frac{h_2}{2} \right]^2 \qquad (4)$$

The above equation involves the term x, which is the distance of the neutral axis of the composite system from the top of the pavement layer configuration. The depth to the neutral axis is determined as follows:[52]

$$x = \frac{E_1\, h_1\, \frac{h_1}{2} + E_2\, h_2\, (h_1 + \frac{h_2}{2})}{E_1\, h_1 + E_2\, h_2} \qquad (5)$$

The effective pavement stiffness ($E_e h_e^3$) may also be determined from the expression for the measured ℓ-value (ℓ_m) in the field and the above parameters as:

$$E_e h_e^3 = \ell_m^4\, 12\, (1-\nu^2)\, k \qquad (6)$$

and

$$\ell_m = a_1 + a_2 \cdot \text{Area} + a_3 \cdot \text{Area}^2 \qquad (7)$$

and $a_1 = 74.32$, $a_2 = -4.185$, and $a_3 = 0.003163$ are regression constants where $r^2 = 0.99498$ and a SEE of 0.75481 was found for the expression shown in equation 7.

Hence, from the FWD measurements made on the field, the ℓ_m may be obtained using a regression type equation. This value may then be used to determine the "effective stiffness" of the original pavement on the foundation. For every basin area measured in the field, there exists an effective stiffness that may be used to characterize the slab.

In general, for an unbonded system as shown above, the effective stiffness is given as in equation 3, and for a bonded system it is determined by equations 4 and 5. The development of this homogeneous "effective" plate or slab is useful in modeling the overlay conditions, as illustrated in figure 21, and representing it as a pavement system, as illustrated in figure 22. E_1 and h_1 are the elastic modulus and the thickness of the overlay and E_2 and h_2 are those of the original pavement beneath (figure 23) and are considered to be effective values since they represent the condition of the joints. Consequently, this pavement is represented as the pavement system shown in figure 22.

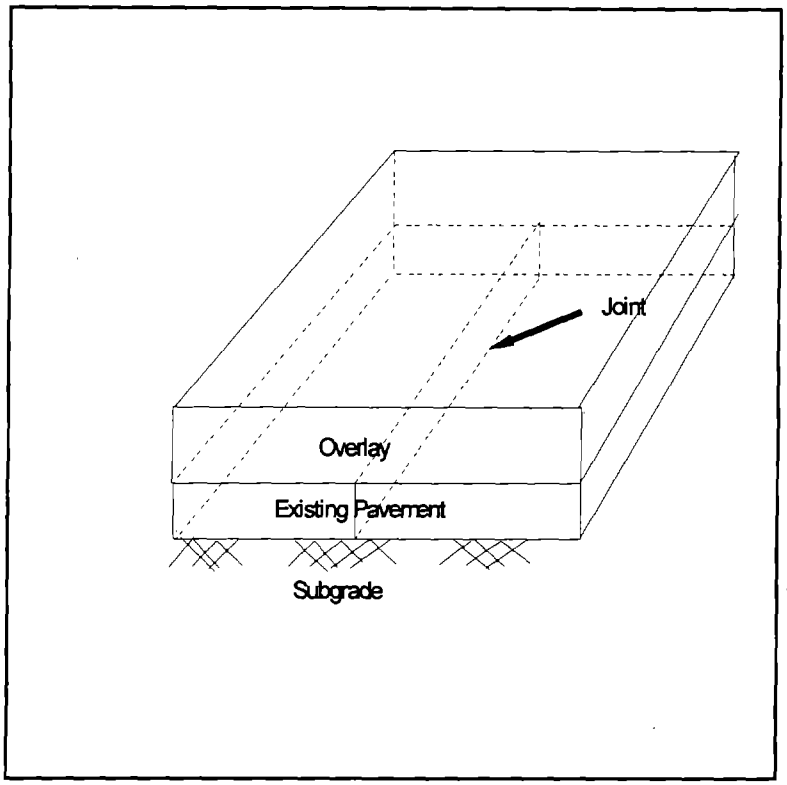


Figure 21. Section of the existing slab.

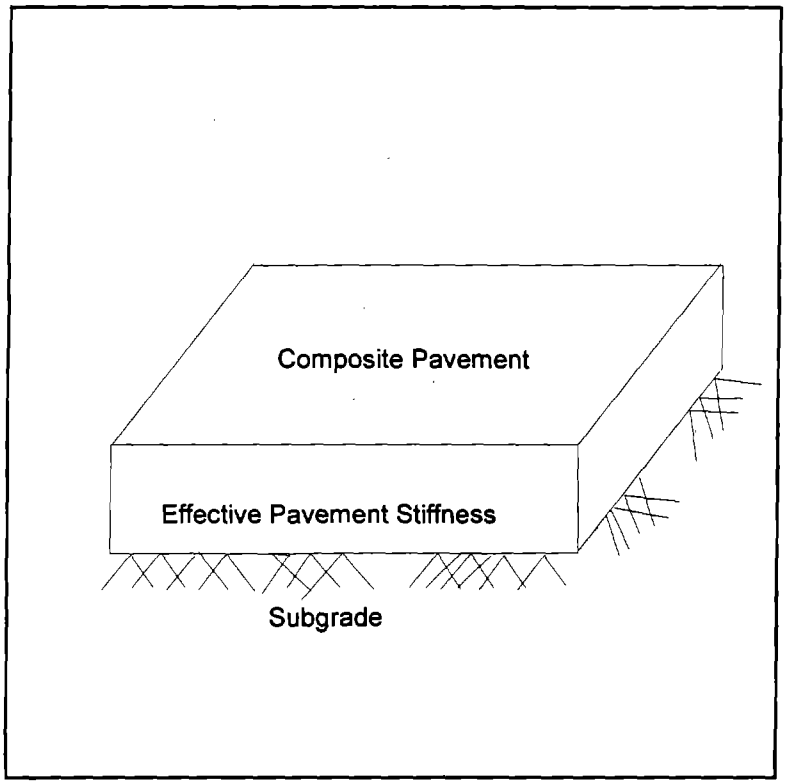


Figure 22. Section of the composite pavement.

Design Criteria

Early thickness designs for CRC pavements as well as CRC overlays were based on the premise that CRC thicknesses did not have to be as great as that of jointed pavements because of an assumed equivalence in structural adequacy.[2, 15] However, the under-design failed to provide adequate support and has resulted in several failures. Observations of distresses in these pavements seem to point at a deficiency in load transferring capabilities. It has been noted in a study by Zollinger and Barenberg[2] that a loss of support can lead to a loss of load transfer in the pavement slab. Hence, any design criteria for CRC pavement/overlay must be developed on the basis of a fully-supported condition under the pavement or overlay.

Loss of support may occur due to several reasons, such as loss of load transfer across the joint as caused by subbase erosion or other conditions at the joint location in the existing layer. This may lead to an unsupported condition because deflection continuity is not maintained, i.e., there are unequal deflection characteristics between adjacent slabs of the supporting pavement,

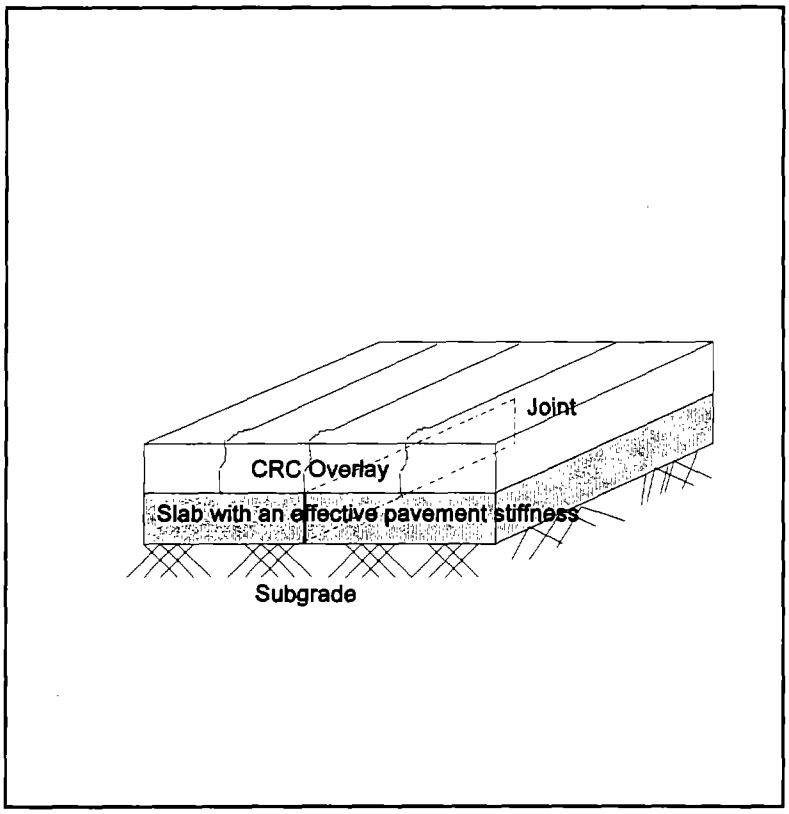


Figure 23. Section of the slab with the overlay.

which will cause the overlay and the underlying pavement to separate while under load.[2] Thus, the CRC overlay is forced to "bridge over" this separated area, causing high localized stresses. This is a significant behavioral characteristic, and the minimization of such is the basis for the design analysis for CRC overlay discussed herein.

To represent a fully supported condition of the existing slab, the CRC pavement overlay is modeled as a continuous slab without joints, which represents a CRC pavement with 100 percent or perfect load transfer across the transverse cracks (figure 22). Again in this case, two different situations might arise when modeling the fully supported condition of the slab under the overlay. The slab may either be bonded or unbonded to the existing slab. Since both the overlay and the existing slab are assumed to have the same elastic modulus for a fully supported condition, a homogeneous "effective" layer with an effective thickness may be used to develop the design criteria[52] in the case of a bonded overlay. The overlay and the original pavement are transformed into one layer with an equivalent thickness and layer modulus. In the case of an unbonded existing slab, the overlay and the existing slab are modeled using two-layer analysis, as described previously, where a back-calculated k-value is used.

Using the equations described above for unbonded as well as bonded conditions, a relationship between the "effective" pavement stiffness and the basin area for different thicknesses of the overlay can be established. This relationship serves as a basis in the design to maintain deflection continuity between different layers of the pavement system. For instance, in an unbonded system, the effective stiffness is found from equation 3. For a fully supported system, since the elastic modulus of both the layers is assumed to be the same, a value for $E_e h_e^3$ for different thicknesses of the overlay can be determined.

Characterization of Field Behavior

A similar approach is developed to account for conditions in the field where the load transfer between the existing layer and the overlay are different (figure 23). This difference

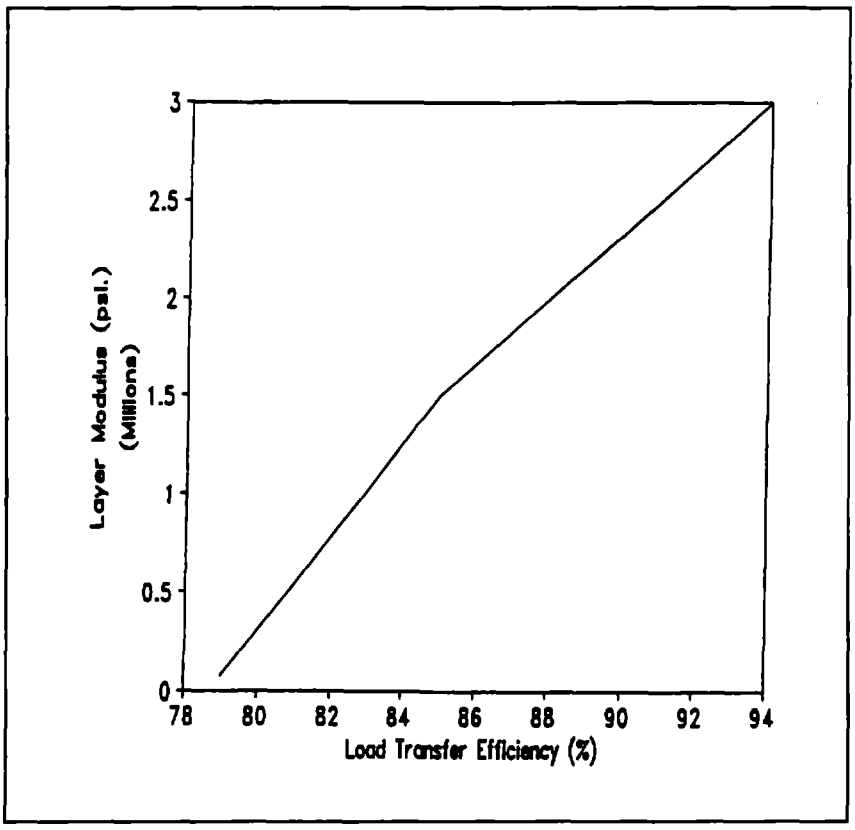


Figure 24. Variation of load transfer with layer modulus.

is based on the use of effective stiffness to represent different load transfer conditions in the existing slab. The load transfer in the existing pavement system is a function of slab and support conditions as reflected in the measured basin areas. The basin area is in turn a function of the pavement "stiffness" where the effective stiffness may be found for every level of load transfer measured across each joint/crack in the original pavement. The pavement response can be stiffened by improving the load transfer across the joints or increasing the pavement stiffness. Improvement of the existing load transfer is represented by improving the "effective" pavement stiffness in the pavement system and is used to determine a relation (shown in figure 24 as based on ILLI-SLAB results) between layer stiffness and load transfer for a common value of basin area. If factors such as the subgrade k-value, Poisson's ratio, and the layer thicknesses are held constant, the radius of relative stiffness (ℓ) is a function of the effective layer modulus, as described previously. It is noted that an "effective" ℓ (ℓ_e) value may be found to vary with the "effective" layer modulus where the effective modulus is substituted for the concrete modulus in the expression for ℓ. This is employed to develop curves for various levels of load transfer. The load transfer in the overlay is assumed to be perfect.

The thickness of the overlay at which the design $E_e h_e^3$ and the field $E_e h_e^3$ are equal may be considered the design thickness of the overlay to be placed upon the existing pavement to ensure adequate pavement behavior and performance, as shown in figure 25. Field measurements, hence, will form the basis upon which to take design decisions. Field curves may be developed for the measured basin areas and LTE and for improved levels of LTE (figure 26). The unbonded design overlay thickness is determined as:

$$h_{overlay} = \left(\frac{Design\ E_e\ h_e^3}{E_{overlay}} - \frac{h_2^3}{n} \right)^{\frac{1}{3}} \qquad (8)$$

where the notations are the same as used earlier and n is the modular ratio of the overlay and the effective flexural modulus in the original slab.

As pointed out previously, another phase of design analysis related to fatigue cracking can be considered at this point in the design process. This particular phase considers punchout development due to fatigue cracking caused by repetitive loading. Assuming that the requirements for the uniform support conditions can be met, the focus of a thickness design procedure should incorporate methods of crack width prediction. It has been pointed our previously[2] that load transfer is a function of crack width and is related to the pavement thickness. Therefore, a thickness design should be based on crack width and consequently, the load transfer selected for design.

The emphasis of a thickness design procedure for CRC pavements should be to maintain a high level of LTE and to limit fatigue cracking from developing into premature punchout distress under uniform support conditions. The inclusion of load transfer in the thickness design can be implemented by use of design curves shown in figure 27. This figure illustrates the effect of crack width on the required thickness for a given level of load transfer. This is derived from the PCA test results,[45] which indicated that pavement thickness design should be coordinated with the design crack width.

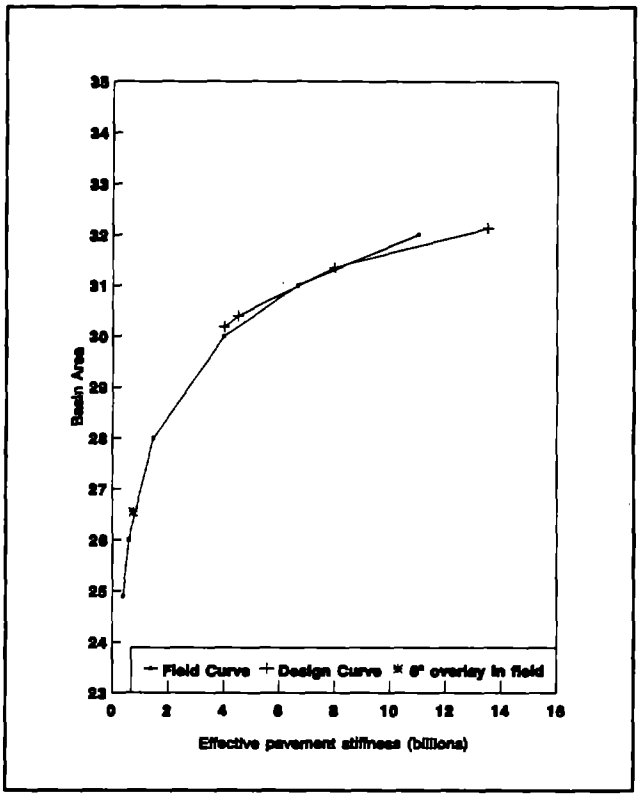


Figure 25. Design criteria based on effective pavement stiffness.

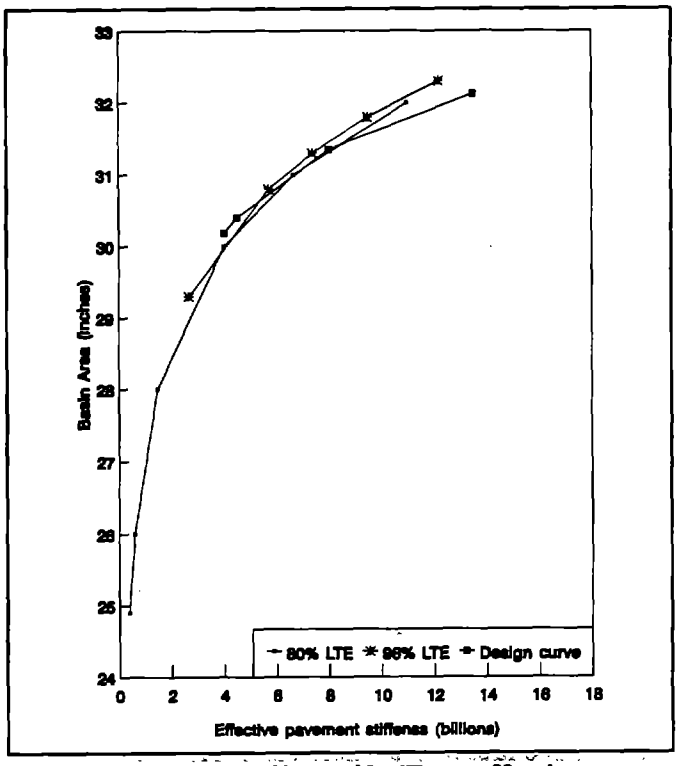
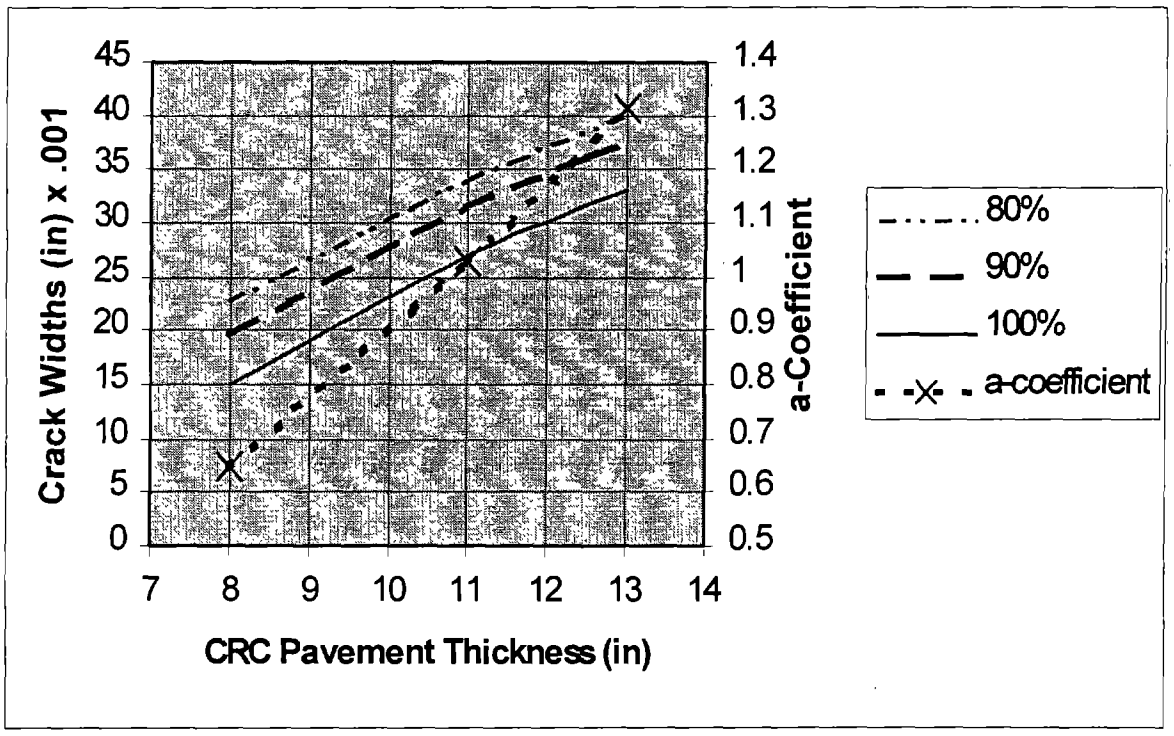


Figure 26. Effect of LTE on effective pavement stiffness.



1 in = 25.4 mm

Figure 27. Maximum crack width limits.[1]

Reliability Analysis

Reliability is a crucial factor in design analysis, since some assurance against pavement failure is desired, depending upon the level of use, against uncertainty and the variability of pavement support and material properties. Otherwise, it should be pointed out that any design method based on average conditions has only a 50 percent chance of fulfilling its required design life.[1]

Reliability in overlay design is sometimes regarded in the sense that the overlay will remain in service at least as long as it was intended to without requiring undue interruption.[1] Technically speaking, however, reliability is the probability of not failing or one minus probability of failing to meet established criteria[51] The factors that cause a broader difference in the degrees of reliability are the degree of variability in material properties and layer thicknesses.[51] Some measures may be taken to limit the variability of material properties and layer thicknesses, primarily through quality control in the construction operation. However, there is still a need to consider it in design.

Therefore, even with the application of quality control, there will be some amount of variability. The variability that is inherent in a pavement results in an uncertainty in the life of a pavement. In a mechanistic approach to design and analysis, the variability may be accounted for by several factors like the thickness of the layers and the material properties. The decisive factor in a thickness design approach is the thickness of the overlay whose

variability results in the variance of the basin area, the controlling aspect in this design approach.

An approach to quantifying the reliability analysis of the overlay design is derived on the basis of equation 2 rewritten as:

$$E_e\, h_e^3 = f = \ell_e^4\, 12\, (1-\nu^2)\, k \tag{9}$$

as previously described. It is of interest to determine an estimate of the variance of the function f using the method known as first order, second moment (FOSM) approximation:

$$Var[f] = \sum_i \left(\frac{\partial f}{\partial X_i}\right)^2 Var[X_i] + \sum_i \sum_j \frac{\partial f}{\partial X_i}\,\frac{\partial f}{\partial X_j}\, cov(X_i\, X_j)$$

$$Var[f] = \sum_i \left(\frac{\partial f}{\partial X_i}\right)^2 Var[X_i] + \sum_i \sum_j \frac{\partial f}{\partial X_i}\,\frac{\partial f}{\partial X_j}\, \zeta_{X_i X_j}\, \sigma_{X_i X_j}$$

where

X_i = Independent variables of function f
Cov = Covariance of X_i and X_j
$\zeta\, X_i\, X_j$ = Correlation coefficient of X_i and X_j

The ℓ value indicated in equation (9) is an effective value since it is based on basin areas (as determined from the relation shown in figure 20) representing less than fully supported conditions. It is therefore possible to include the variability in field measurements in the reliability determinations for design. To include the variability of field measurements:

$$Var[f] = \left[\frac{\partial f}{\partial \ell_e}\right]^2 Var[\ell_e] + \left[\frac{\partial f}{\partial \nu}\right]^2 Var[\nu] + \left[\frac{\partial f}{\partial k}\right]^2 Var[k] + \frac{\partial f}{\partial \ell_e}\,\frac{\partial f}{\partial k}\, \sigma_{\ell_e}\, \sigma_k\, \zeta_{\ell_e k}$$

where

$\partial f/\partial \ell_e = 4\cdot\ell_e^3\cdot 12\cdot(1-\nu^2)\cdot k$
$\partial f/\partial \nu = -\ell_e^4\cdot 12\cdot k\cdot 2\nu$
$\partial f/\partial k = \ell_e^4\cdot 12\cdot(1-\nu^2)$

The variance of ℓ_e is found from the equation 7 rewritten as:

$$\ell_e = g = a_1 + a_2\cdot \text{Area} + a_3\cdot \text{Area}^2$$

and a_1, a_2, and a_3 are regression constants defined previously.

$$\mathrm{Var}[\ell_e] = (\partial g/\partial \mathrm{Area})^2 \cdot \mathrm{Var}[\mathrm{Area}]$$

where

$$\partial g/\partial \mathrm{Area} = a_2 + 2a_3 \cdot \mathrm{Area}$$

and the variance of area is derived from the measured basin areas.

The variance of k may be determined as:

$$Var[k] = \left(\frac{\partial k}{\partial D_o}\right)^2 Var[D_o] + \left(\frac{\partial k}{\partial \ell_e}\right)^2 Var[\ell_e]$$

where

$$\frac{\partial k}{\partial D_o} = \frac{-P}{8\, D_o\, \ell_e}\left[1 - \left(\frac{a}{\ell_e}\right)^2\left(0.217 - 0.367\log\left(\frac{a}{\ell_e}\right)\right)\right]$$

$$\frac{\partial k}{\partial \ell_e} = \frac{-2P}{8\, D_o^2\, \ell_e^2}\left[1 - \left(\frac{a}{\ell_e}\right)^2\left(0.217 - 0.367\log\left(\frac{a}{\ell_e}\right)\right)\right]$$

$$+ \frac{-P}{8\, D_o\, \ell_e^2}\left[\left(\frac{-2a^2}{\ell_e^3}\right)\left(0.217 - 0.367\log\frac{a}{\ell_e}\right) + \left(\frac{a}{\ell_e}\right)^2\left(\frac{-0.367}{\ell_e}\right)\right]$$

and also depends on the $\mathrm{Var}[\ell_e]$ as defined previously.

It can also be shown that:

$$\mathrm{Var}[X] = \sigma_x^2 = (\mathrm{cv}[X])^2$$

where X is the mean parameter value.

It is noted that the variance of $E_e h_e^3$ determined, in part from field measurements, is also a function of the coefficient of variation of Poisson's ratio. The cv of Poisson's ratio may be assumed to range from 10 to 15 percent. Knowing the mean value of $E_e h_e^3$ and the standard deviation of $E_e h_e^3 (\sigma_s)$, the design $E_e h_e^3$ may be calculated for different levels of reliability (assuming a normal distribution) from:

$$\text{Design } E_e h_e^3 = E_e h_e^3 + Z_R \cdot \sigma_s$$

$$Design \quad E_e h_e^3 = \overline{E_e h_e^3} + Z_R \cdot cv[E_{eh_e}{}^3] \; \overline{E_e h_e^3}$$
$$= \overline{E_e h_e^3} \; (1 + Z_R \cdot cv \; [E_e h_e^3])$$

A multiplying factor (MF) may be derived as:

$$MF = \text{Design } E_e h_e^3 / E_e h_e^3 = 1 + Z_R \cdot cv[E_e h_e^3]$$

The multiplier Z_R is chosen from a table of constants which correspond to different levels of reliability. A set of the Z_R and MF values and corresponding levels of reliability and various effective stiffness coefficient of variations are given below.

Reliability	Z_R	Multiplying Factor		
		cv=15%	cv=20%	cv=25%
50	0.0	1.00	1.00	1.00
84	1.00	1.15	1.20	1.25
90	1.28	1.19	1.26	1.32
95	1.64	1.25	1.33	1.41
99	2.33	1.35	1.47	1.58
99.5	2.57	1.39	1.51	1.64
99.9	3.08	1.46	1.62	1.77

The unbonded design overlay thickness is determined as earlier, only including the reliability term this time as:

$$h_{overlay} = \left(\frac{Design \; E_e \; h_e^3}{E_{overlay}} - \frac{h_2^3}{n} \right)^{\frac{1}{3}} \tag{10}$$

where n is the modular ratio of the overlay and the effective flexural modulus in the original slab.

The advantage of using reliability design is that only those pavement thicknesses and quality control programs are used that will ensure a required level of reliability.[1] For any given level of variability of the design quantities, a higher level of reliability will require a thicker and more costly design (figure 28).

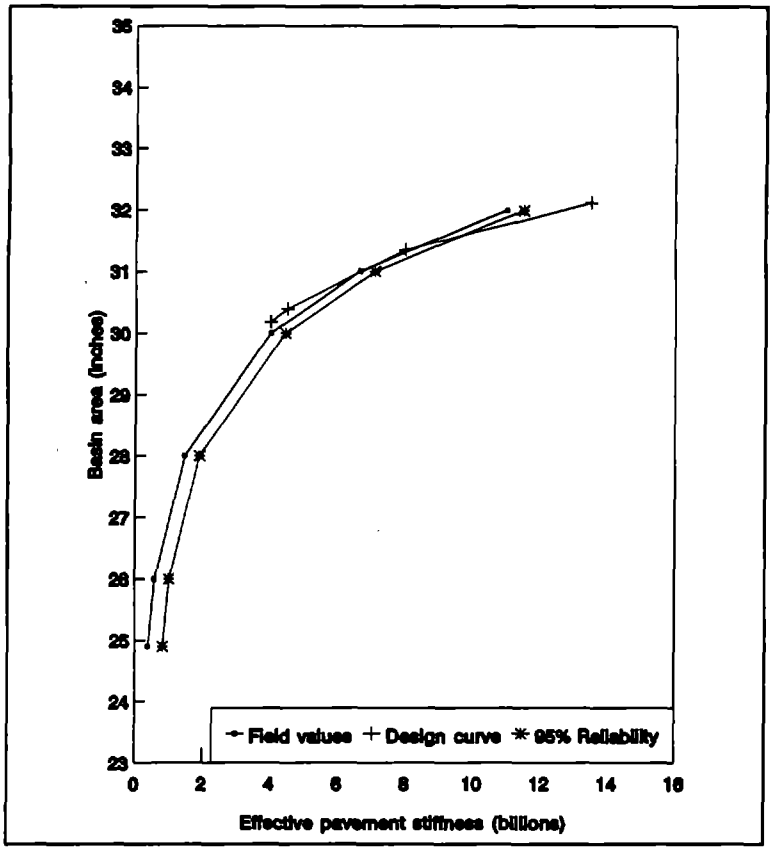


Figure 28. Effective pavement stiffness requirements at 95 percent reliability.

Case Study

Data were obtained from I-30W in Arkansas for a 254-mm (10-in) JRCP consisting of 4.57 x 3.66 m (15 x 12 ft) slabs (doweled at 13.7 m (45 ft) joints and hinged at 4.57 m (15 ft) joints). NDT was conducted on the test site to gain further insight into the CRC overlay behavior leading to distress. During the testing process, data relative to the slab deflection, LTE and pavement stiffness was obtained. The FWD used for the field testing was the Dynatest Model 8000 which is widely used in the United States. The testing equipment consist of trailer-mounted drop weight device weighing approximately 6.67 kN (1500 lbs). The impulse force, created by dropping masses from different heights, varied between different levels. The load measured by a transducer is transmitted to the pavement through a load plate having a radius of 150 mm (5.9055 in). Deflections are measured using velocity transducers mounted on a bar that is lowered simultaneously with the load plate to the pavement surface. The loading plate and the deflection sensors layout is on 305-mm (12-in) centers. Most of the testing was conducted in the outer wheel path with the center of the loaded plate varying between 0.91 to 1.07 mm (36 and 42 in) from the pavement edge.

The two portions of the I-30 from which deflection data was obtained are shown in figures 29 and 30. The data were categorized in two groups for the approach and leave side of the transverse crack. Data for the approach side were obtained with the load plate of the FWD located on the side of the transfer crack from which the traffic approaches the crack. The leave side of the crack is the opposite side. The deflections were taken in the morning as well as in the afternoon to provide more information regarding the effect of temperature on deflections and crack widths. From FWD measurements, the average basin area was found to be 632 mm (24.9 in) and the LTE was about 80 percent.

Appendix B contains detailed information on the data collected, and graphs of the various parameters including the basin area, radius of relative stiffness, and LTE of the original pavement. For the convenience of the reader, a sample graph of the basin area,

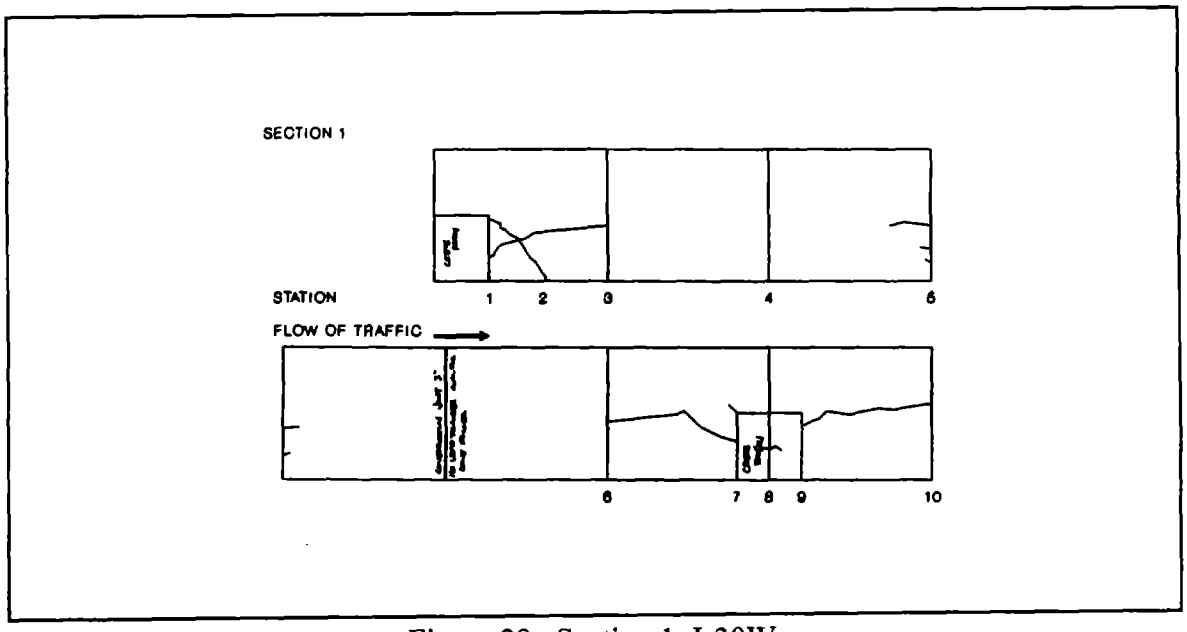


Figure 29. Section 1: I-30W.

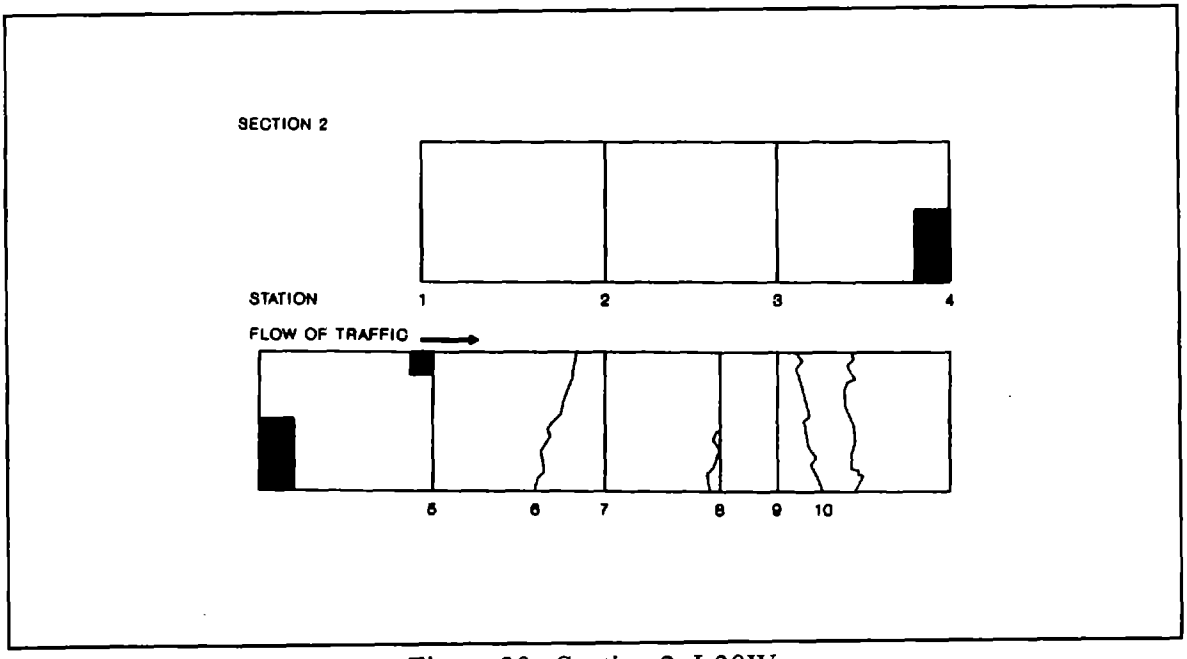


Figure 30. Section 2: I-30W.

radius of relative stiffness, and LTE measured along Section 1 in the mid-slab during the morning are shown in figures 31-33. Morning test results indicate that significant differences in LTE can exist between two groups of cracks (typified by Stations 6 and 9 on Section 1). However, the load transfer differences between the two groups are less in the afternoon. The closing of cracks because of an increase in the pavement temperature can cause a dramatic increase in LTE. ℓ values are determined and shown for the test results for the I-30W sections.

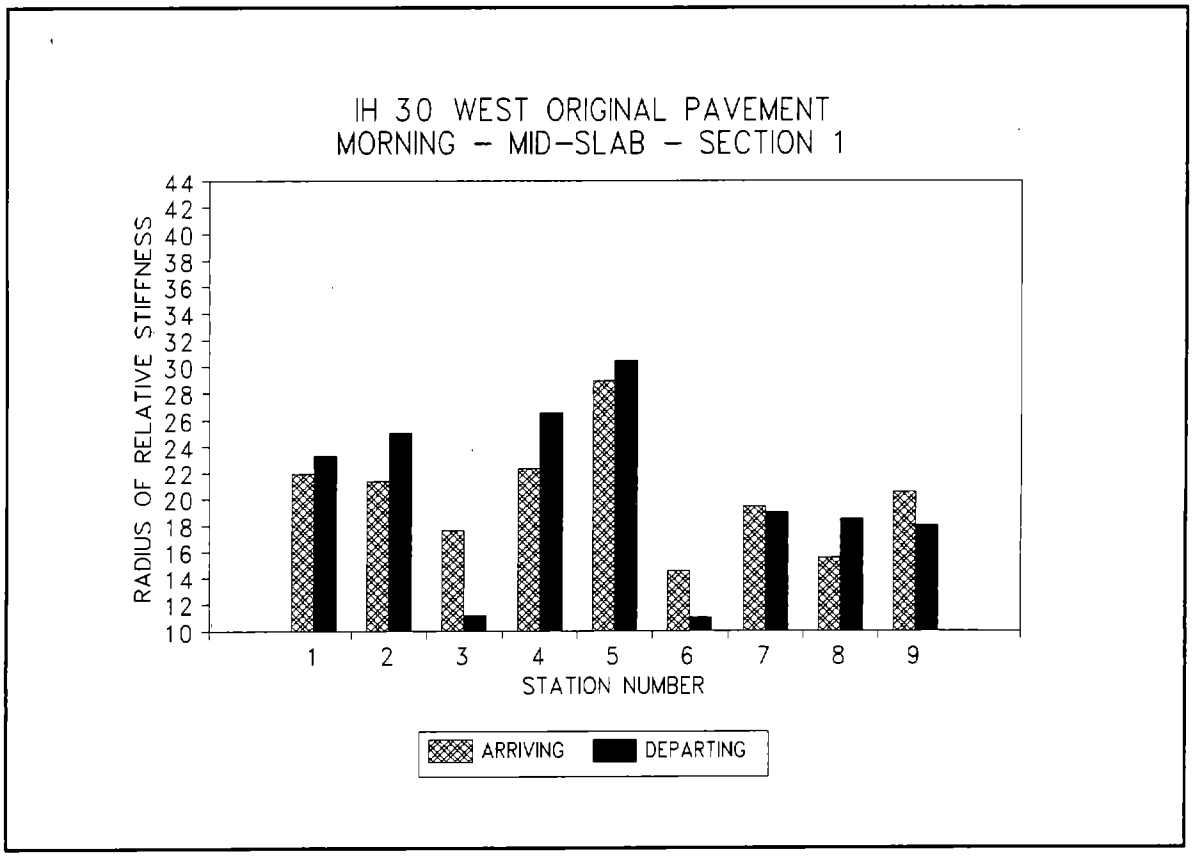


Figure 31. Radius of relative stiffness versus station (I-30W, morning, mid-slab, section 1).

Using ILLI SLAB, the basin area and the deflection profile was matched for a 66.7-kN (15,000-lb) load positioned at mid-slab. Based on the information available regarding the R-value of the soil (approximately 5), the subgrade modulus or the k-value was approximated at 20 MPa/m (75 psi/in).

The modulus (E value) of the existing JRCP was found to be about 27,600 MPa (4,000,000 psi) from the testing of the cores. The E value for the 6-in cement treated base (CTB) that lay below the JRCP was taken as 2,760 MPa (400,000 psi) (equation 1). A back-calculated composite k-value (247 psi/in) was determined from equation 1 to account for the 152 mm (6 in) CTB to facilitate two-layer analysis since the overlay is to be unbonded. It is also of interest to represent the 10-in original JRCP in the two-layer analysis as an equivalent layer with different composite layer modulus for different levels of load transfer as described previously. Another approach, employing two layer analysis, may be to use the subgrade k-value and determine a initial composite modulus of the CTB and the original 254-mm (10-in) JRCP. With either approach, the basin area measured in the field is matched to determine a composite k-value or an effective layer modulus. Different levels of load transfer were represented in terms of a corresponding effective modulus and are shown in figure 24.

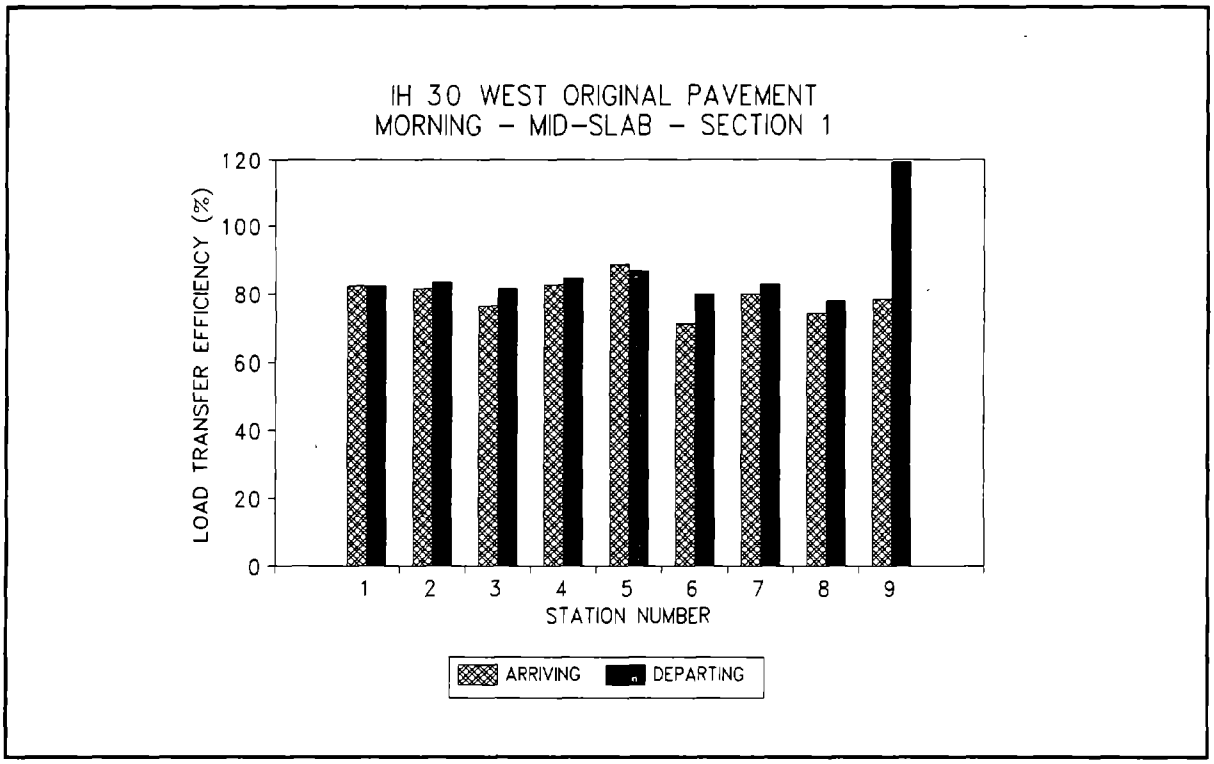


Figure 32. Load transfer efficiency versus station (I-30W, morning, mid-slab, section 1).

After the field-measured deflection basin was matched and the effective modulus of the existing pavement was found, the pavement overlay thickness was determined using equation 8. The effective layer stiffness was improved by improving the existing load transfer condition. Increase in the overlay thickness resulted in a different basin area for each thickness.

The relationship between the calculated basin area and the composite overlay stiffness is compared against the fully supported conditions as illustrated in figures 26 and 28. The curve developed by modeling field conditions was found to meet the design curve at a thickness of about 292 mm (11.5 in). This was taken as the design thickness at 50 percent reliability. It should be pointed out that a certain length of I-30 was overlaid with a 6-in layer of CRC that was tested with the FWD shortly after construction. FWD testing was done on sections of this overlay and the results are graphically illustrated in appendix C. The average basin area for the overlay is shown in figure 25 and suggests the trends are correct for this analysis. The 152-mm (6-in) overlay was not considered to be a successful design since it was reconstructed in 1996.[65]

As noted above, the design thickness of 292 mm (11.5 in) is considered to be that at 50 percent reliability. Reliability equations provided above may be used to develop curves at different levels of reliability, as illustrated in figure 34. It can be seen that, by improving the load transfer in the existing pavement, lower thicknesses can be expected at higher reliability levels.

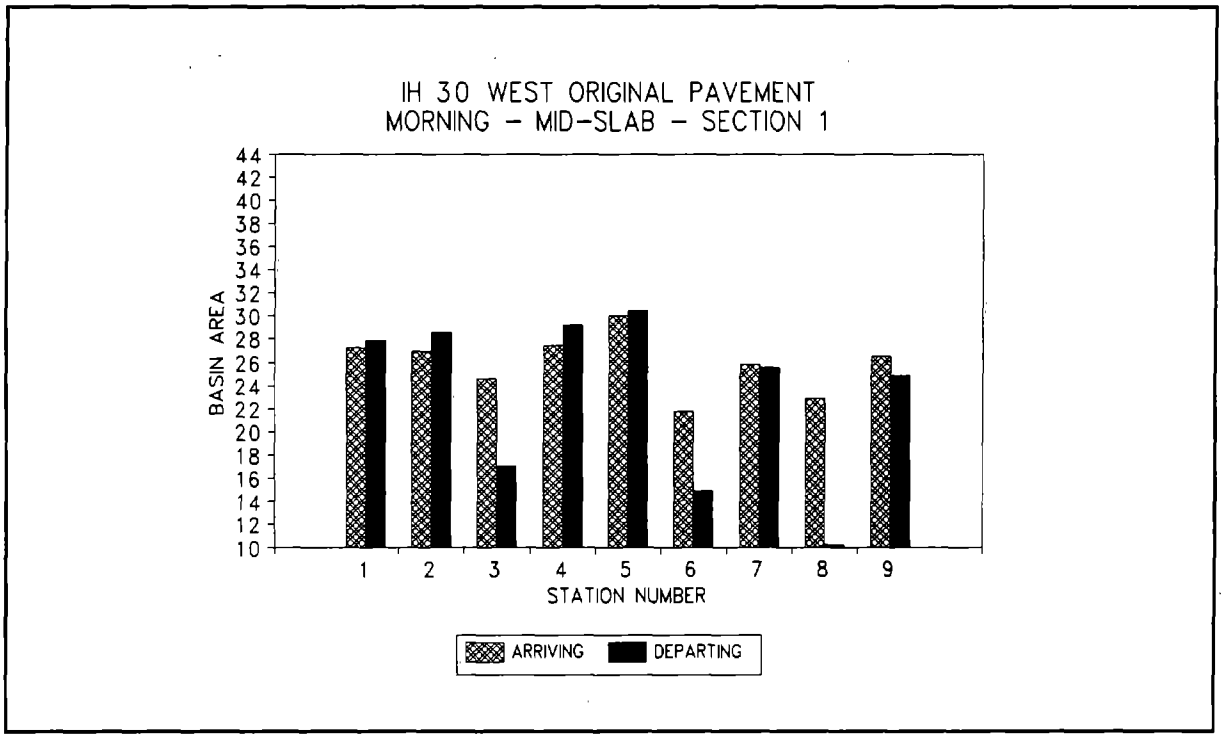


Figure 33. Basin area versus station (I-30W, morning, mid-slab, section 1).

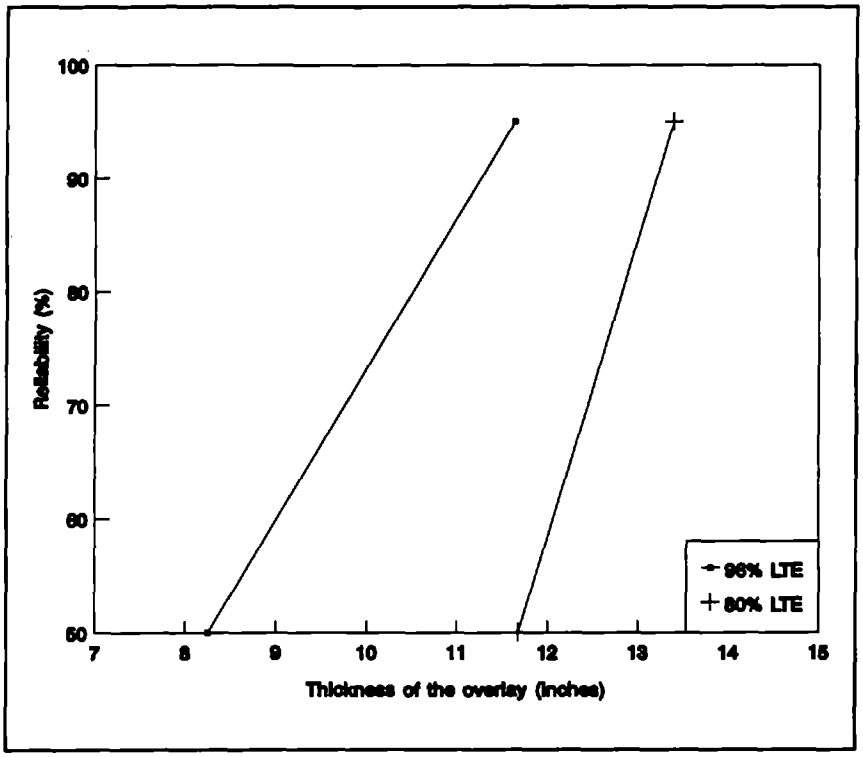


Figure 34. Reliability versus thickness of overlay.

The design of a continuously reinforced concrete (CRC) overlay consists of two phases one related to the pavement support conditions and the other related to fatigue damage due to repetitive loading. The first phase should be applicable to bituminous overlays on jointed or continuously reinforced concrete pavements. The experience in Arkansas, on a preliminary basis, indicated the trends in overlay thickness suggested by this

approach are appropriate and that different levels of reliability may be useful in determining the minimum acceptable pavement condition that can be rehabilitated with an overlay.

CHAPTER 4 - DESIGN CONSIDERATIONS FOR AC OVERLAY ON CRC PAVEMENTS

All highway pavements, whether rigid or flexible, require maintenance to keep them in a satisfactory condition. The type of maintenance will vary from pavement to pavement, depending on the extent and type of distresses the pavement is developing. Asphaltic concrete overlays are often the most economical form of maintenance available to overcome a wide variety of defects in both rigid and flexible pavements. Many states have overlaid substantial portions of their PCC highway pavement mileage with AC, and anticipate overlaying more in the near future. Since many more pavements are rehabilitated by resurfacing than by reconstruction, the mileage of bare PCC highway pavement in service in the United States is declining, while the mileage of AC-overlaid PCC highway pavement is growing. As a result, the design and performance evaluation of AC-overlaid PCC pavements is becoming increasingly important.

An asphaltic overlay consists of one or more layers of compacted AC placed over the existing pavement. A tack coat of asphalt cement is typically applied to the existing pavement surface prior to the construction of the overlay in order to seal the existing cracks and to promote adhesion between the overlay and the existing pavement surface.

AC-overlaid PCC pavements are perhaps the most difficult of all pavement types to evaluate while in service. One source of the difficulty is the combined action of layers, which exhibit very different behavior characteristics. Such layers pose a challenge in understanding and modeling the structural[5] and damage[6] response of the pavements to applied loads. There is little documentation available on the performance of these types of pavements in terms of distress development and repair.

This chapter provides a brief overview of the distresses that are common in AC overlays of CRC pavements. It also discusses an analytical modeling of the failures of these overlays to provide guidelines upon which to base future decisions regarding their use.

Prevalent AC Overlay Distresses

The evaluation of the performance of AC-overlaid PCC pavements is dependent on recognition of the distresses that typically develop in this type of pavement. Distresses commonly seen in these pavements typically include reflection cracks, rutting, weathering and raveling of the AC surface, and delamination of the AC overlay. Each of these is briefly discussed in the following section. Analytical approaches with respect to the modeling of some of the distresses is provided subsequently in this chapter.

Reflection Cracking

A distress commonly encountered in this method of rehabilitation is the "reflection cracking" of the overlay, where the cracking or joint pattern existing in the original pavement layer is extended into and through the new surface layer. These cracks deteriorate

over time due to spalling and, combined with fatigue, can lead to a deteriorated level of performance. The basic mechanism of reflection cracking in AC overlays of PCC pavements is the concentration of strain in the overlay at the vicinity of the joints or cracks in the existing pavement because of horizontal and vertical movements of the joints. This movement may cause either bending or shear stresses induced by wheel loads, or horizontal contraction induced by temperature changes. Load-induced movements are influenced by the thickness of the overlay and the thickness and stiffness of the existing pavement. Temperature-induced movements are influenced by daily and seasonal variations in temperature, the coefficient of thermal expansion of the existing slab, and the spacing of joints and cracks.[5] A reflection crack in AC-overlaid CRC pavement is shown in figure 35. In an AC overlay of CRC pavements, reflection cracks typically develop relatively soon after the overlay is placed. The rate at which they deteriorate depends on the factors listed above as well as the applied traffic level.

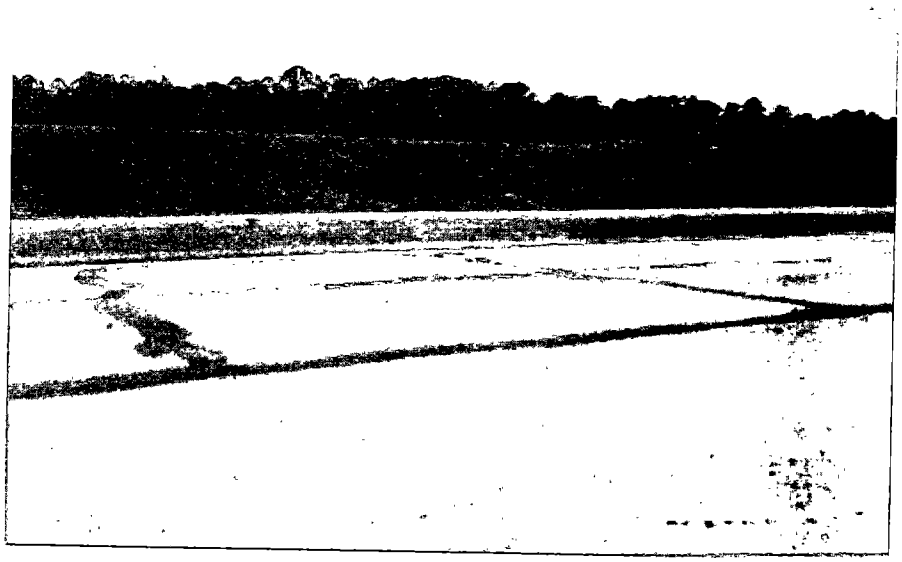


Figure 35. Reflection crack in AC-overlaid CRCP.

Rutting

Rutting is defined as "a permanent deformation in and of the pavement layers or subgrades caused by consolidation or lateral movement of the materials due to traffic loads."[54] Pavement rutting is the result of channelized traffic that causes differential surface deformation in the areas of intensive load applications.[46]

Obviously, roadway wheelpaths will be subjected to this type of load condition. Generally, the amount of pavement rutting depends on the following parameters:[46]

(1) The distribution of the traffic loads transversely across the paving lane;
(2) The stresses, which depend on the response characteristics of the layer materials, induced in the pavement system; and
(3) The permanent strains, which depend on the pavement deformation characteristics of layer materials, induced as a result of the stresses.

A typical rut in an AC overlay of a CRC pavement is shown in figure 36.

Raveling and Weathering

Figure 36. A typical rut in AC overlay of CRC pavement.

Raveling and weathering are progressive deterioration of an AC surface as a result of loss of aggregate particles (raveling) and asphalt binder (weathering) from the surface downward.[55] Raveling and weathering occur as a result of loss of bond between aggregates and the asphalt binder. This may be due to hardening of the asphalt cement, dust on the aggregate that interferes with asphalt adhesion, or localized areas of segregation in the AC where fine aggregate particles are lacking. Raveling and weathering, like rutting, may pose a safety hazard if deteriorated areas of the surface collect enough water to cause hydroplaning or wheel spray.

Delamination

In light of the above distress types observed in AC overlays of PCC pavements, it is also of interest to mention a less frequently noticed distress, delamination. It occurs with the intrusion of water into a crack opening, typically a reflection crack in the asphalt surface, and can result in loss of bond between the new surface and the existing pavement surface at the location of the reflection crack. Thus, it can be seen that the occurrence of reflection cracking prematurely shortens the useful life of the overlay. The crack widths of the asphalt overlay can play an important role in this type of distress. The tighter the crack, the lower the probability of water intrusion and subsequent delamination.

Field Survey

Several sections on I-45 in District 17 in Texas were surveyed for the existence of punchout repairs and other distresses in AC overlays of CRC pavements. It was of interest to study the structural failure aspects of the overlay. A total of eight sections were surveyed, seven on I-45, and one on I-35 in McLennan County. A brief summary of the pavement type, subbase, traffic, and the failures observed has been tabulated in table 3.

Table 3. Data related to distresses in AC overlays of CRCP.

Route	Thickness of Existing CRCP	Date of First Overlay (3"-6")	Date of Second Overlay	Average Traffic (AADT)	Sub-base Type	No. of Punchouts per Mile (Average)	Other Remarks
I-45N MP 100-112	8 (May, 1968)	1984	1989 (2"-4")	23500	Crushed Stone	4.2	Rutting Visible
I-45S	8" 1968	1985	1989 (1"-3")	23500	Crushed Stone	6.1	Spalling
I-45N MP 152.2-164.0	8" 1967	1985	1991 (1"-3")	16500	Asphalt Treated	2.3	-
I-45S	8" 1967	1985	-	14000	Asphalt Treated	3.2	Lots of Visible Patches
I-35N MP 325.2-334.0	8" 1965	1981	Reconstr ucted in 1991	54000	Crushed Stone	-	-
I-45N MP 165.0-181.0	8" 1969	1988	-	15500	Asphalt Treated	3.1	-
I-45S Madison County	8" 1965	1987	-	16200	Crushed Stone	>10	Rutting and Patches

1 in = 25.4 mm

It was noticed that a large number of the punchout repairs occurred in low-lying areas in the vicinity of a bridge structure. This may be because such areas are prone to trap run-off water that may later contribute to subbase erosion, and loss of support leading to the propagation of the punchouts. This is typified in figure 37, which is a photograph of a patch on an AC overlay of CRC pavement showing deterioration at the edges. Figure 38 is the photograph of a patch on a CRC pavement, prior to overlaying.

Several inner lane punchouts were also observed (figure 39). An edge punchout is delineated from an interior punchout by the presumption that the interior punchout is caused by poor concrete consolidation while the edge punchout is caused by repeated traffic loads and loss of pavement support,[57] but interior punchouts are most likely due to the same causes related to edge punchouts. On the Southbound lane of I-45 (MP 165.0-MP 172.2), a large portion had its edges patched, as shown in figure 40. It was believed that the

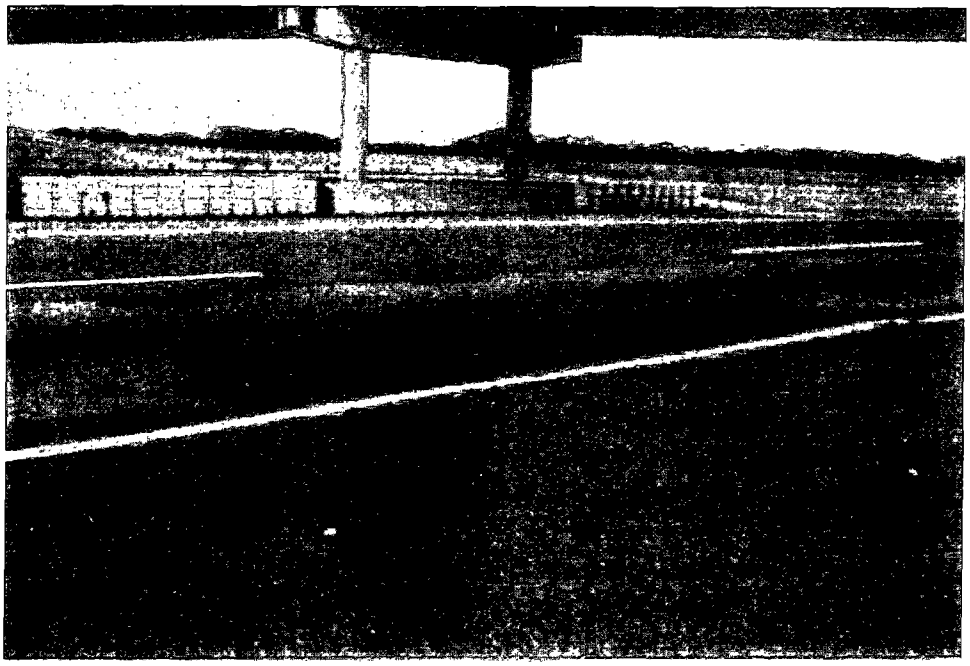


Figure 37. Patch in AC overlay of CRC on I-45N.

combined effect of the temperature-induced movements and traffic loadings caused the longitudinal cracks and several transverse cracks along the edge to reflect through. The patching was done in order to prevent the reflection cracks from deteriorating into punchouts.[58]

Figure 38. Patch in CRC pavement.

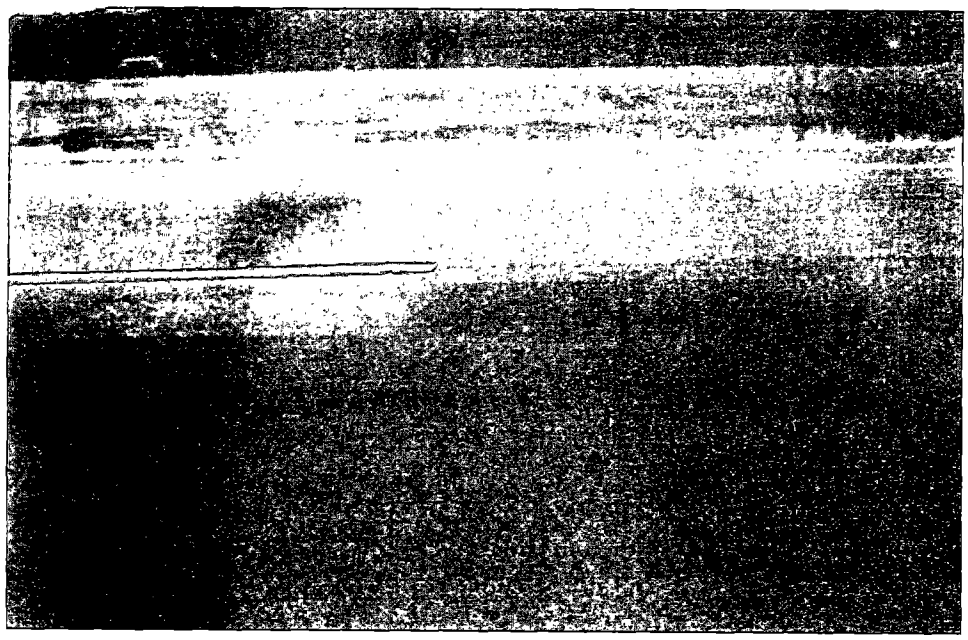


Figure 39. Inner lane failures on I-45S.

Wheelpath ruts were prominently visible (figure 41), and heavy patching had been done on those sections to prevent safety hazards. It was observed that sections that were CRC pavements with crushed stone bases that were badly distressed prior to overlay had more rutting than the other sections that were on a high-strength asphalt-treated base.

Most of the sections had at least one or two surface coats (approximately between 25.4 mm (1 in) and 63 mm (2.5 in) thick) since they were overlaid. It was reported[58] that the surface coats were used to repair rutting in those sections.

It was concluded that rutting and reflection cracking were the most common types of distresses in AC overlays of CRC pavements. An analytical/modeling study of these distresses is considered, in order to study the factors affecting the development of these distresses in greater detail.

Effect of Environmental Conditions on Crack Width

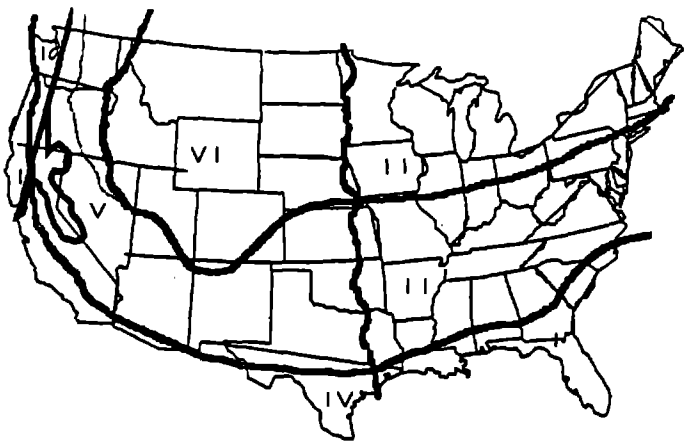
Since the field survey seemed to indicate that distresses such as reflection cracking and punchouts are prevalent in AC overlays of CRC pavements, an investigation of the factors affecting crack widths in CRC pavement was made. The performance of CRC pavements has been closely related to keeping the cracks as tightly closed as possible to ensure good aggregate interlock and load transfer.

The most important factors influencing the crack widths in CRC pavements are the drying shrinkage (a nonreversible effect) of concrete and the temperature drops. In an

Figure 40. Edge patching on AC overlays of CRCP (I-45S).

Figure 41. Wheelpath ruts in AC overlays of CRCP (I-45S).

overlaid CRC pavement, the crack widths in the overlay would be primarily influenced by the temperature variations at the surface of the AC. Both large seasonal temperature variations and daily temperature cycles will affect the movement at the existing joint or crack, thus affecting the crack widths. It is of interest to examine the insulative property afforded to the CRC pavement by the AC overlay. A study was made to simulate the climatic influences on pavement performance, especially the effects of temperature.



REGION	CHARACTERISTICS
I	Wet, no freeze
II	Wet, freeze-thaw cycling
III	Wet, hard-freeze, spring thaw
IV	Dry, no freeze
V	Dry, freeze-thaw cycling
VI	Dry, hard-freeze, spring thaw

Figure 42. The six climatic regions in the United States.

Low seasonal temperatures produce a horizontal opening because of the contraction of the original surface. A thermally induced tensile stress is also produced in the overlay from the contraction that the AC overlay experiences. The combination of these stresses will be most severe directly over the joint or crack.

Daily temperature cycles also produce thermal tensile stresses in the AC overlay. In the underlying CRC pavement, the temperature cycling produces temperature gradients in the slab that result in nonuniform vertical opening and closing of the cracks. When the temperature drops and the top of the slab is cooler than the bottom, the crack width is greater at the top. This causes an opening that, while less severe than a low temperature opening, occurs far more often.

The Integrated Model of the Climatic Effects on Pavements[59] was used to model a rigid pavement with an AC overlay. This program computes variations of parameters like the pore water pressure, temperature, and the elastic modulus with respect to time. This program predicts both changes in material properties and structural response on a seasonal basis. It has four components each of which are integrated to model the environmental effects on a pavement system. The Climatic-Materials-Structural Model (CMS) developed by Dempsey et al.[60] was used to analyze the effects of temperature on AC overlays of CRC pavements. Figure 43 shows the temperature drops at the top of the overlay (ambient drop in temperature) and at the bottom of the overlay for various thicknesses of the overlay and different climatic regions in the United States. (Different climatic regions have been

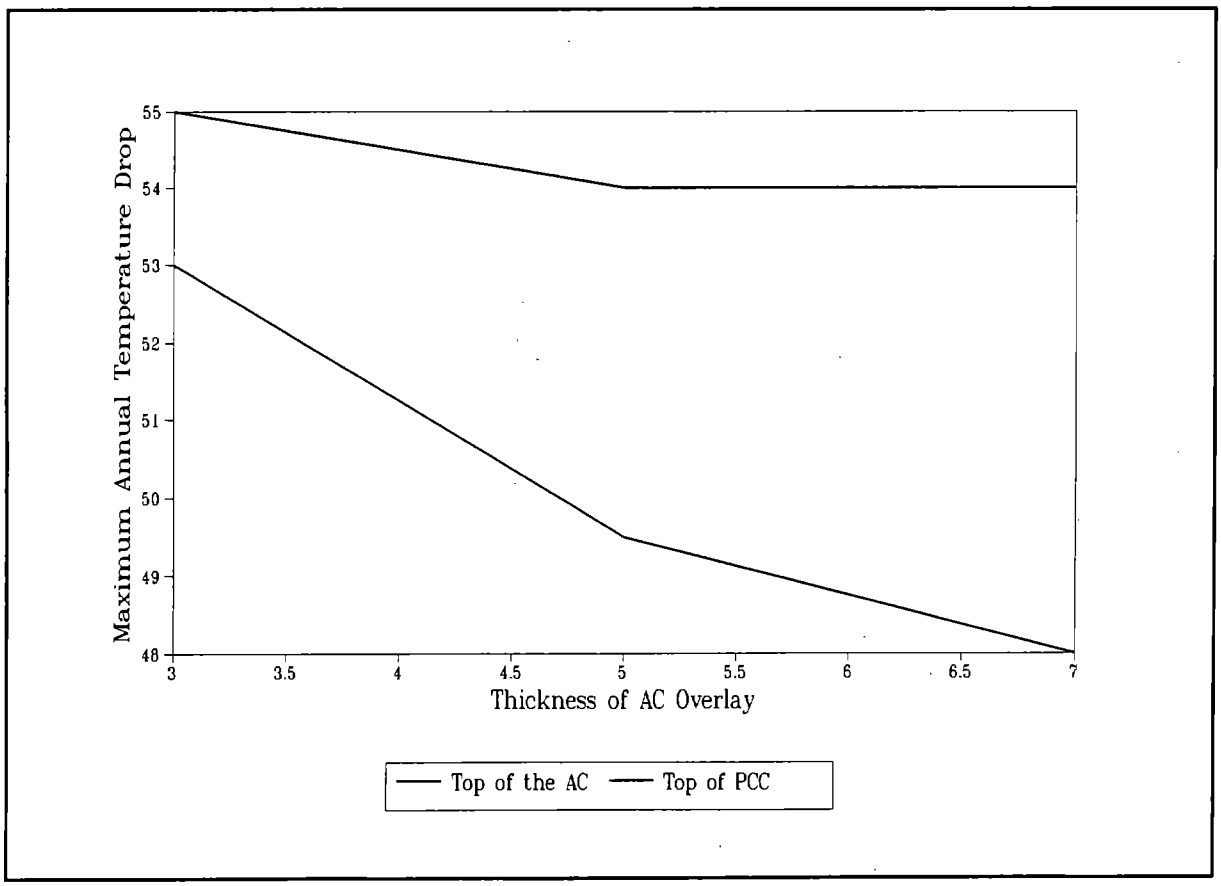


Figure 43. Temperature variations at the surface of AC overlay and PCC surface.

identified in the United States, as shown in figure 42). It can be seen that the insulative effect of a 76-mm (3-in) or a 127-mm (5-in) overlay is minimal and that the change in crack widths that would result from a drop in temperature at the surface of the underlying PCC pavement would quickly propagate a crack to the AC surface. Also, the movements in the PCC pavement resulting from the changes in the temperature could result in wide cracks in the AC surface, and the subsequent intrusion of water, leading to other performance problems. It has been recommended in reference 71 that the minimum thickness of AC overlay for ambient maximum temperature drops of about 10°C (50°F) should be about 102 to 127 mm (4 to 5 in) to prevent reflection crack propagation.

Effect of Reflection Cracking on Life of AC Overlays

In recent years, several mechanistic approaches have been developed to predict reflection cracking that have only considered the strain at the lower surface of the asphalt layer as a fatigue-related design criterion. This approach relies on the relationships developed between the calculated strain and fatigue damage due to traffic loadings. But the real need is to view the situation in a mechanistic way that accurately reflects fatigue damage caused by environmental loads since the stresses under wheel load applications tend to be compressive rather than tensile. This requires the use of fracture mechanics to appropriately consider the growth of a crack that reflects through the overlay from a pre-existing crack below the overlay. Reflection cracking is often the primary cause of

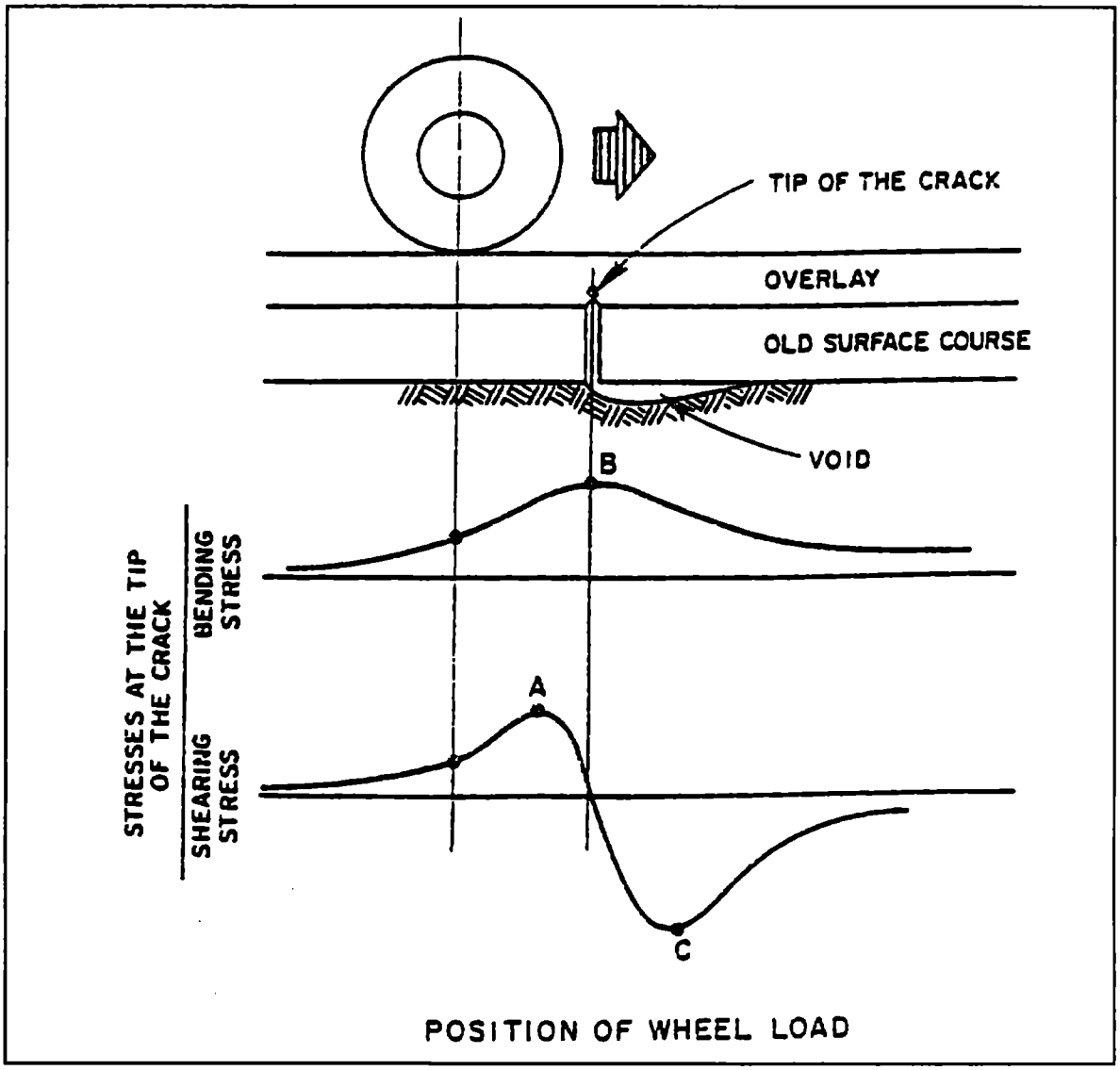


Figure 44. Stresses induced at the cracked section due to a moving wheel load.[62]

deterioration of bituminous concrete overlays. Also, raveling and spalling of the overlay often accompany these cracks.

The basic mechanisms generally assumed to lead to reflection cracking are the vertical and horizontal movement of the underlying pavement layer. These damaging movements may be caused by traffic loading, thermally induced contractions and expansions, or a combination of these mechanisms. Figure 44 shows the stresses induced within the overlay under the influence of a moving wheel load. It can be observed that three pulses of high stress concentrations occur at the tip of the crack as the wheel passes over it. As the wheel approaches the crack, the shear stress at the cracked section will momentarily reach a maximum at point A. This shear stress pulse will then be followed by a maximum

bending stress pulse at point B. Finally, there will be a third stress pulse at point C, which is similar to the first except that it acts in the opposite direction.

In addition to the influence of the traffic loads, contraction and expansion of the pavement and the underlying layers with changes in temperature can also contribute to the growth of reflection cracks. Contraction of the underlying pavement layers under low temperatures, and curling of the old pavement surface associated with it, produces high tensile stresses at the tip of the crack. The growth of cracks within the overlay under the influence of these cyclic loadings can be better explained using fracture mechanics principles.

Experimental investigations carried out at the Ohio State University[61, 62, 63] and the Texas A&M University[64] have verified the applicability of fracture mechanics principles in predicting fatigue life of AC mixtures. Schapery[29] indicated that the rate of crack propagation in AC can be predicted by using the Paris relation:

$$\frac{dc}{dN} = A(\Delta K)^n$$

where

ΔK	=	Stress intensity factor amplitude
A, n	=	Fracture parameters of the material
c	=	Crack length
N	=	Number of loading cycles.

Many investigations have adopted this approach for the analysis of fatigue crack growth and failure of AC overlays.

Reflection cracking can have a considerable influence on the life of an AC overlay of CRC pavements. The Overlay Design Equation is a computer program that was developed as part of a FHWA study on the development and design of "asphalt concrete overlay design equations and design procedures to address reflective cracking." Design equations were developed for six climatic zones (shown in figure 42) for flexible overlays of both flexible and rigid pavements for each climatic zone for which data were available.

The overlay design procedure is mechanistic-empirical in concept. Mechanistic equations were developed to represent the existing pavement as a beam on elastic foundation. In-situ deflection testing was used to determine the structural parameters needed to characterize the pavement. Basic asphalt properties are used with fracture mechanics concepts to calculate the rate at which cracks in the existing pavement will propagate through the overlay because of traffic and thermal action. These mechanistic equations were calibrated with in-service data for each of the climatic zones.

These equations were made by supplying information on the types and thicknesses of layers in the pavement (in our case, a fixed thickness of PCC pavement and variable

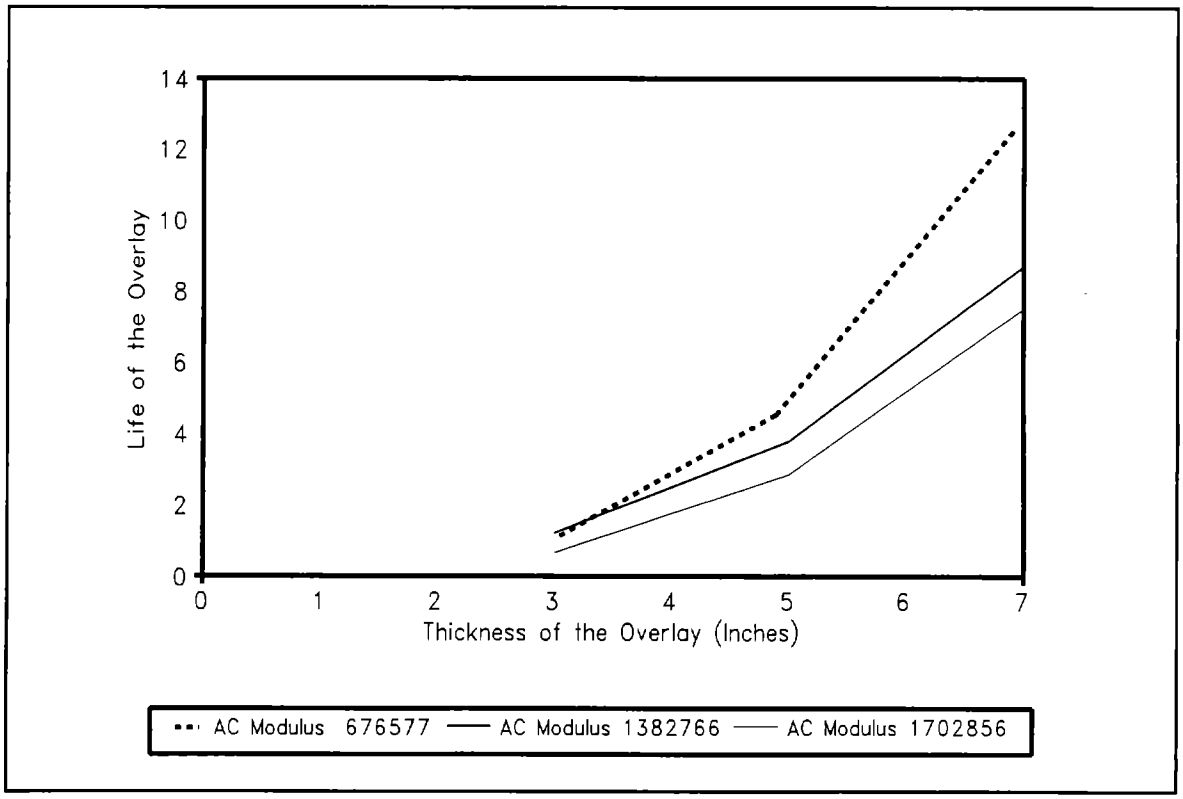


Figure 45. Effect of reflective cracking on life of AC overlay as a function of different moduli of AC.

thicknesses of AC overlay), the material properties of the AC being considered for use in the overlay, the environmental zone in which the pavement is located, and the traffic to which the overlay is expected to be subjected. The program then provides an estimate of the time and the number of traffic loads to failure by reflective cracking.

Deteriorated reflection cracks reduce a pavement's serviceability and may result from poor load transfer in the existing pavement. This condition of poor load transfer in the existing pavement can be given as an input parameter to the program for calculating the bending efficiency factor for the cracked pavement. The poor support conditions can also contribute to propagation of reflection cracking in the overlay. The program ODE was run for different values of k, moduli of AC, and drops in temperature. The results are shown graphically (see figures 45-47).

Effect of Rutting on Overlay Life

Rutting is another major distress in AC overlays of CRC pavements. Wheelpath ruts greater than one-third to one-half in (25.4 mm = 1 in) are considered to pose a safety hazard, because of the potential for hydroplaning, wheelspray, and vehicle handling difficulties.[5]

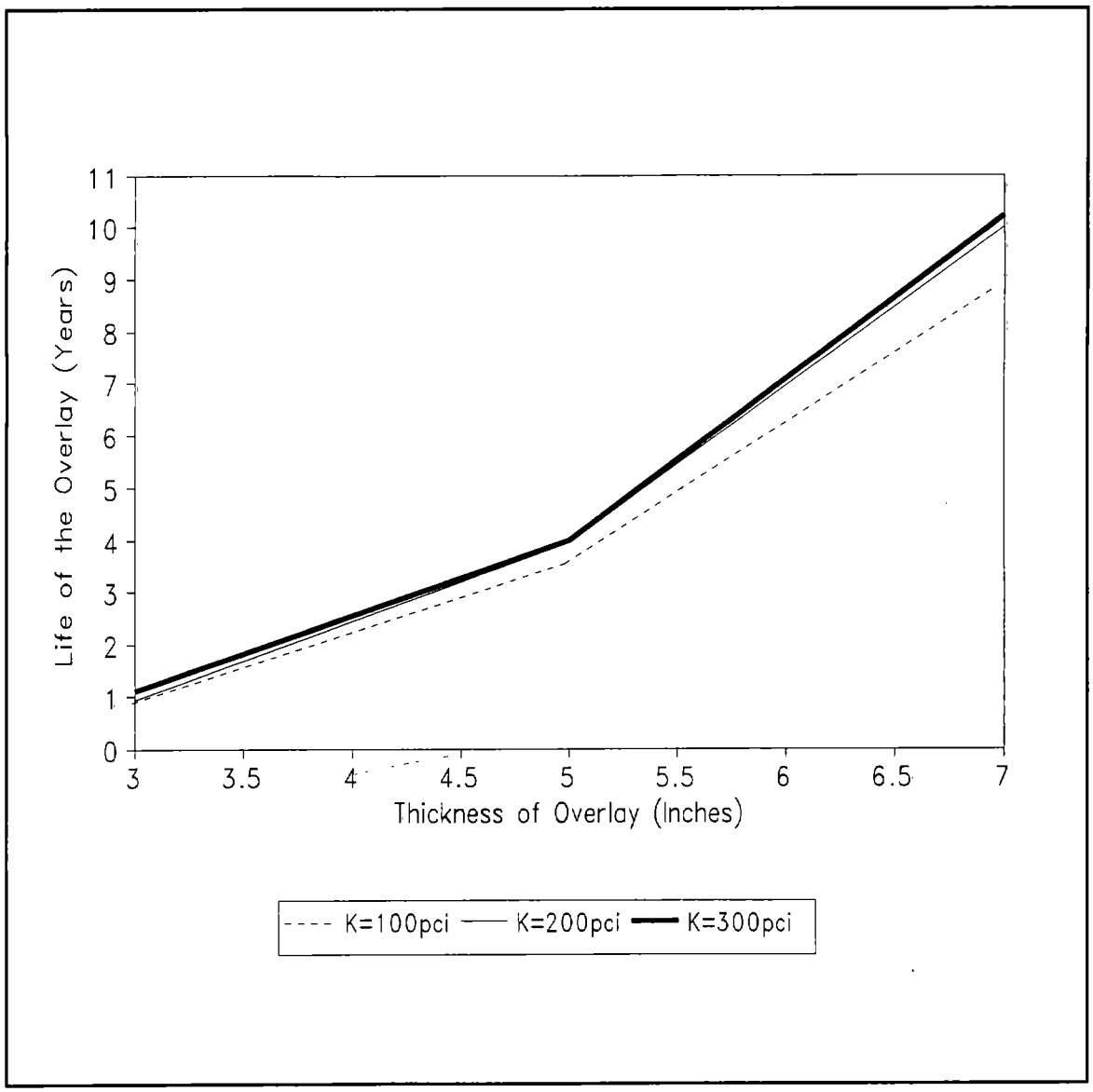


Figure 46. Effect of reflective cracking on life of the AC overlay as a function of different subgrade moduli.

A number of procedures are available to estimate the amount of rutting from repeated traffic loading. These are in general:[54] (1) the use of elastic theory to predict stresses coupled with permanent strains determined by repeated laboratory tests; (2) the use of linear viscoelastic theory together with creep and recovery tests; or (3) the use of statistical regression analysis.

Very few models have been developed that can predict rutting accurately. A computer program named FLEXPASS developed by Tseng[54] at Texas A&M University was built for this purpose. It is an accurate and efficient program and may be used for computing the responses of pavement structure and its distresses in terms of stress, strain, deflection, rutting, and slope variance. The loads applied may include single or tandem axles, and single or dual tires, which apply calculated horizontal and vertical tire contact

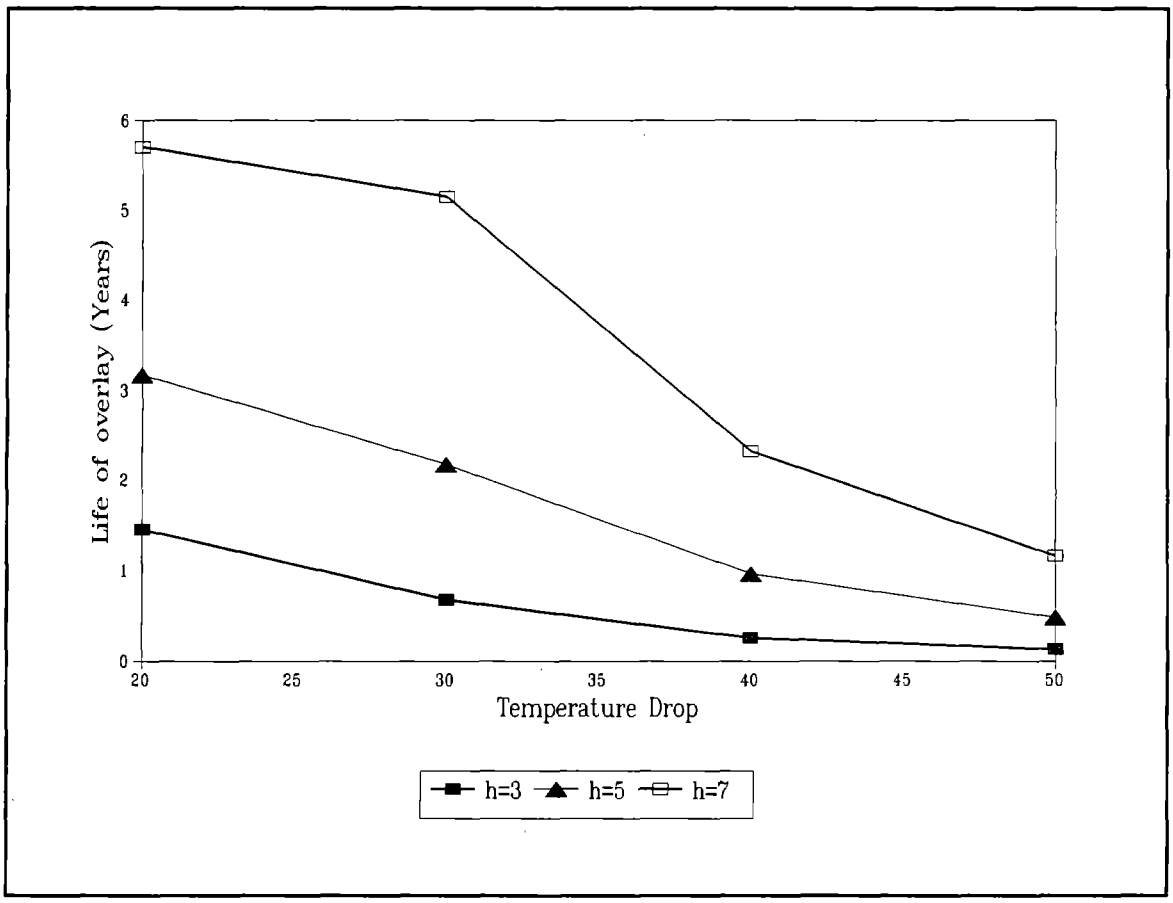


Figure 47. Effect of reflective cracking on life of the overlay as a function of different daily drops in temperature.

pressure distributions to the surface. The computer program can take into account realistic distributions of both vertical and horizontal tire contact pressures as opposed to the conventional assumption that tire contact pressure is a vertical uniform load. The effect of slip between the layers is also included to account for those conditions in which adhesion between the layers is imperfect. It is possible to model up to six different layers in this program.

This program was used to study the effects of different tire pressures and traffic on rutting in different thicknesses of AC overlays. Tire pressure was observed to have an important effect on rutting, more so because the underlying layer was a CRC pavement (stiff). Interestingly, the 127-mm (5-in) overlay rutted more than the 76-mm (3-in) overlay (figures 48 and 49). This may be because of the development of the tension zone at the bottom of the asphalt layer. An analysis of the rutting due to different tire pressures and the time before any given thickness of the overlay ruts to 7.6 mm (0.3 in) (considered a medium- to high- level-severity rut) is also presented (see figure 50).

On the whole, AC overlays seem to have performed well. The life of CRC pavements in particular have been extended after the application of an AC overlay. In

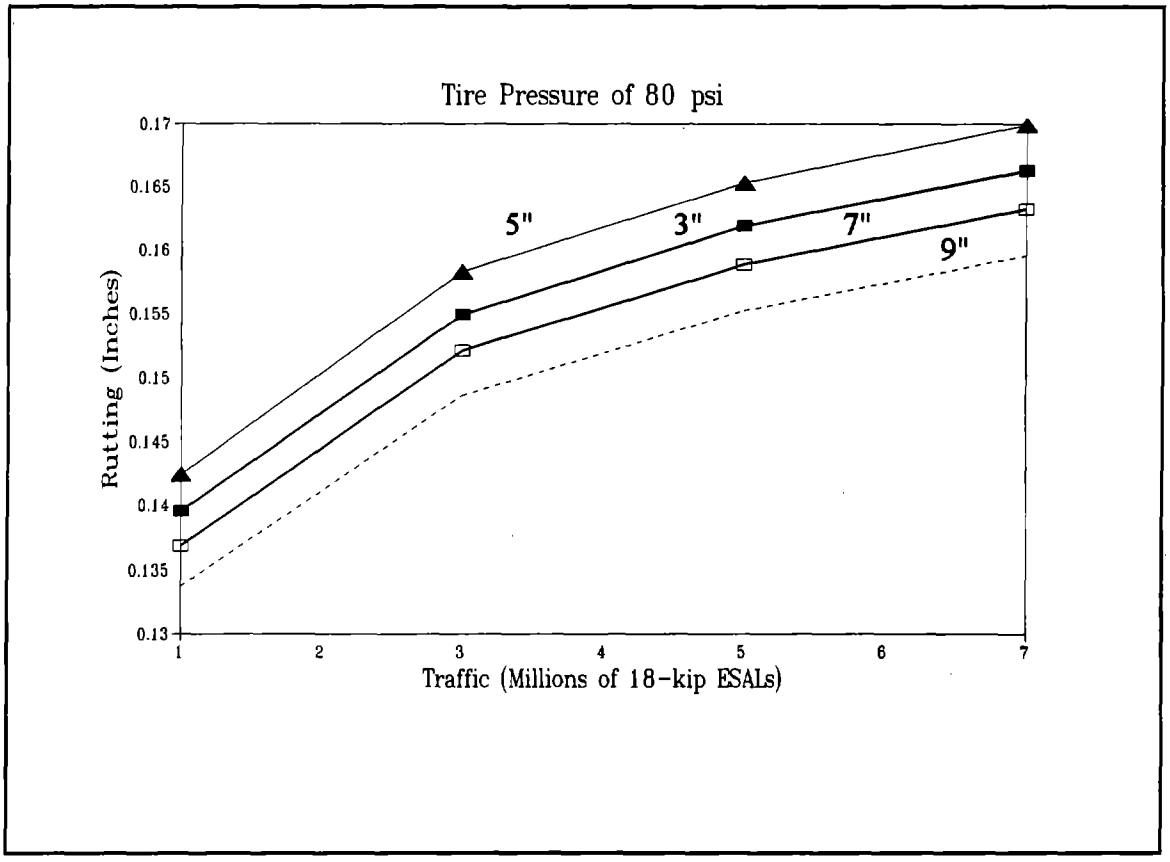


Figure 48. Rutting of AC overlay under a tire pressure of 80 psi.

Indiana, the AC overlays on the Statesville CRC pavement were reported to have increased the life of an overlay. However, it must be pointed out that the effect of temperature and other environmental factors play an important role in the development of distresses like reflection cracking. Large drops in temperature hasten the phenomenon of reflection cracking. In the presence of moisture or in a region where a wet, freeze-thaw type of climate exists, there is a good possibility that the asphalt layer will be delaminated from the underlying CRC pavement because of intrusion of water into the reflection cracks. This and several other local distresses experienced with AC overlays of CRC pavements are primarily due to the effects of climate.

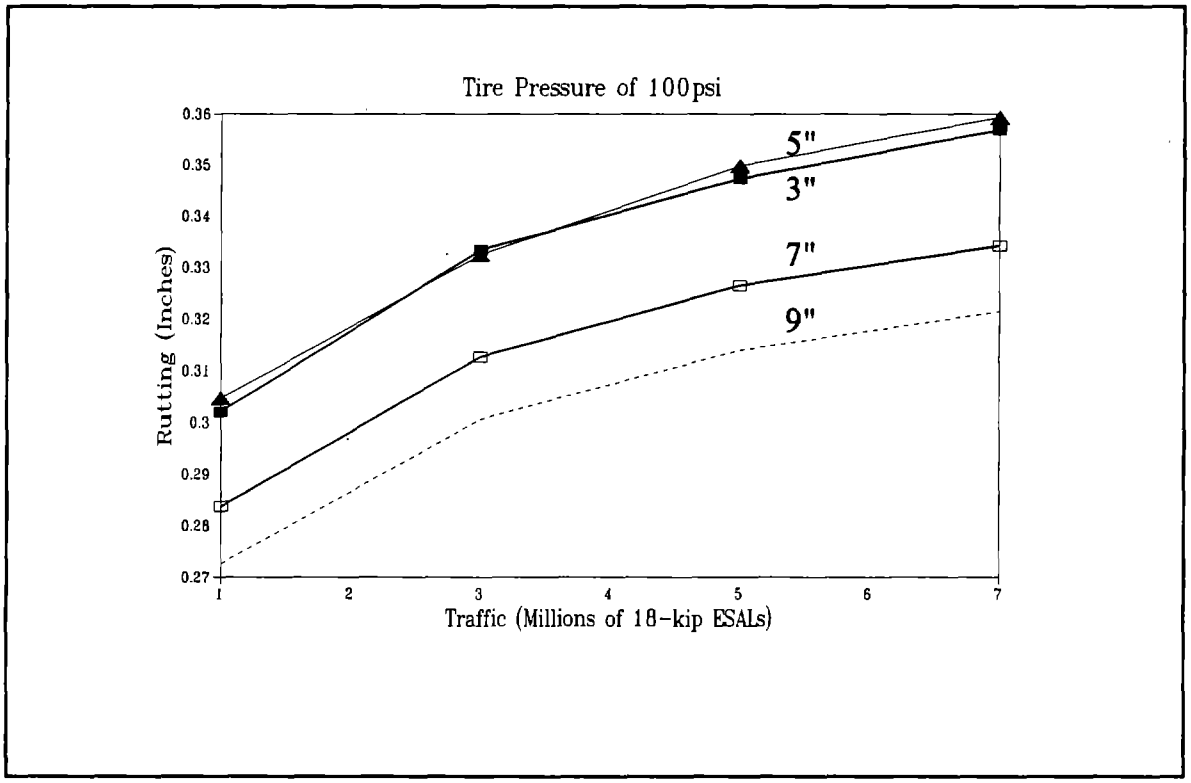


Figure 49. Rutting of AC overlay under a tire pressure of 100 psi.

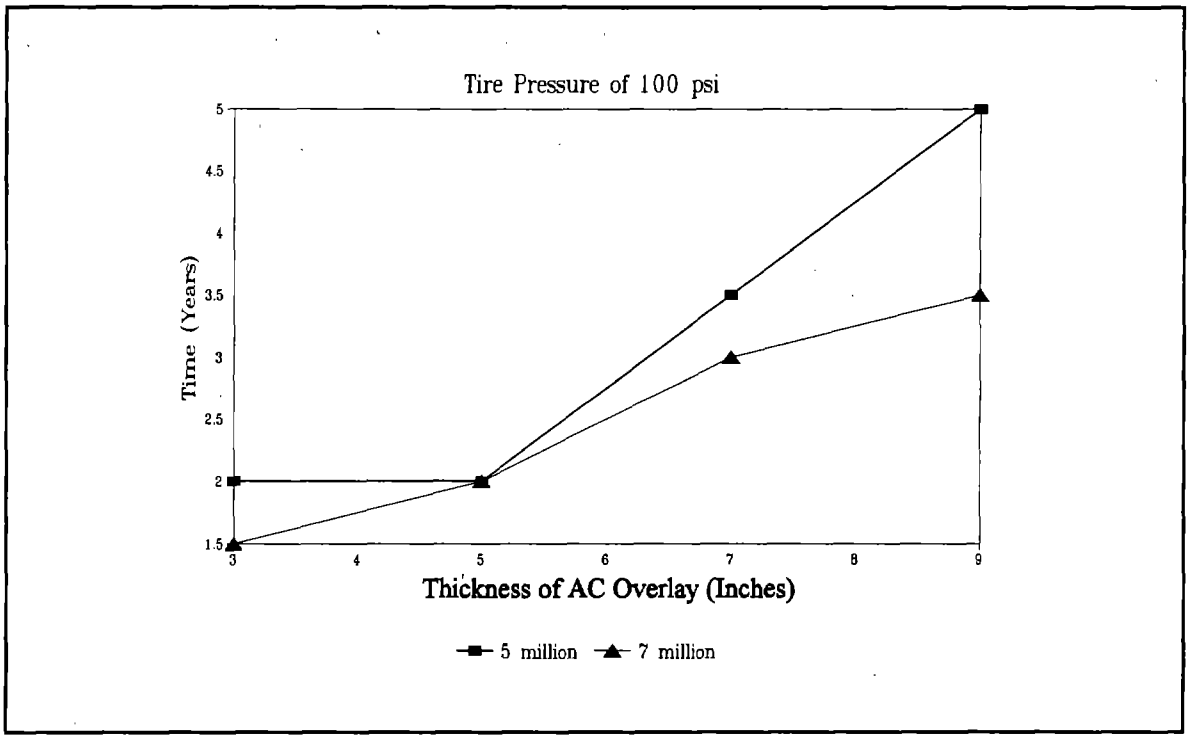


Figure 50. Time for AC overlay to rut 7.6 mm (0.3 in).

CHAPTER 5 - LIFE-CYCLE COST ANALYSIS OF OVERLAY ALTERNATIVES

There is always more than one alternative rehabilitation design. Each alternative has its own associated costs, constructability, maintainability, and other unique characteristics. It is desirable to select the preferred alternative, or one that meets all of the engineering criteria (e.g., project constraints such as traffic control and initial funding) and is cost-effective.

The complex process involved in selecting the preferred alternative is perhaps the most difficult and controversial part of the entire process of developing rehabilitation projects. The common practice of selecting an alternative only because it has the lowest first initial construction costs is poor engineering practice, and can lead to much higher future pavement costs and management problems for an agency. Therefore, it is of interest to examine the different rehabilitation alternatives with regard to their *life-cycle cost-effectiveness*. This chapter aims to provide a life-cycle cost analysis of three typical rehabilitation alternatives: overlay of CRCP with AC, bonded overlay of CRC with CRC, and overlay of JRCP with CRCP. Another alternative is also considered: the construction of an unbonded overlay after restoration of the existing pavement.

Life-Cycle Cost Analysis

A life-cycle cost analysis is strongly recommended before the rehabilitation alternative is chosen. This must be done within the constraints already programmed for the initial rehabilitation construction cost for the project. The associated cost of the rehabilitation alternative over the selected analysis period is normally the major consideration in selecting the preferred alternative.

Life-cycle costs can be expressed as a present worth or as an equivalent uniform annual cost. Using the present worth method, all future costs are adjusted to a present worth using a selected discount rate. The costs incurred at any time in the future can be combined with the initial construction costs to give a total present worth cost over the analysis period. The present worth for the various alternatives being considered are presented in the following sections.

AC Overlay of CRC Pavement

This method of rehabilitation is very common, and an analytical study of the distresses prevalent in this type of overlay was discussed in chapter 4. This section aims to view the costs associated with the construction and maintenance of such an overlay over a period of 20 years. The anticipated costs over the lifetime of an AC overlay of a CRC pavement may be categorized as follows:

- Initial construction
- Regular maintenance of thermal cracks and reflective cracks

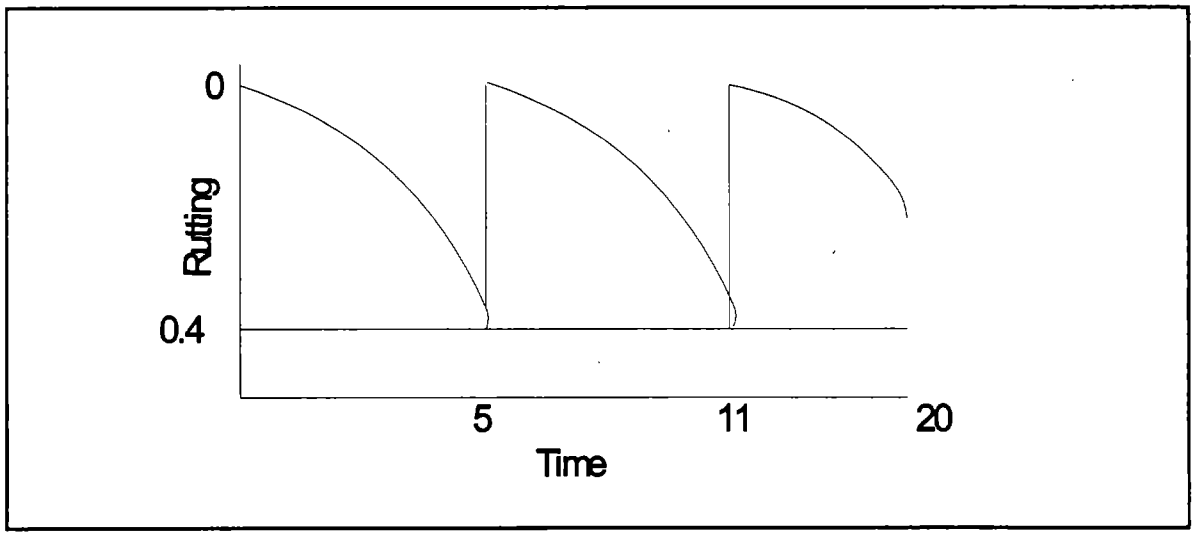


Figure 51. Rutting life of an AC overlay.

Table 4. AC overlay of CRC pavement (No joint restoration).

Year	Item	Unit Cost	Total Quantity	Total, $/mile	Present Worth
1	4" AC Surface	$15.00/SY	7016	105240	105240
2	Routine Maintenance (Thermal Crack Sealing)	$1.20/LF	845	1014	937.5
3	Thermal Crack Sealing	$1.20/LF	845	1014	901.4
5	Overlay Due to Excessive Rutting (1.5" AC Surface)	$15.00/SY	3500	52500	43150
8	Routine Maintenance	$1.20/LF	845	1014	740.92
11	Overlay Due to Excessive Rutting	$15.00/SY	3500	52500	34103
15	Patching	$76.00/ton	845	64220	35650
Total Present Worth, $/mile					220,722

1 LF = 0.305 M 1 SY = 0.836 M^2 1 CY = 0.760 M^3

Note: DW based on a discount rate of 4%.

- Patching of localized failures
- Laying a seal coat and/or milling
- Major rehabilitation activity like another overlay

The average costs of all of the items, expressed in present worth dollars, are shown in table 4, and a total life-cycle cost is also calculated.

The analytical modeling of AC overlays of PCC pavements discussed in chapter 4 showed that, because of rutting, the life of these types of overlays did not exceed 5 to 6 years. FLEXPASS was used to model the AC overlay, and an overlay was placed every time the rutting level reached 10 mm (0.4 in). This means that every 5 to 6 years, rehabilitation is required (figure 51). This may be in the form of another overlay. Micro-surfacing and recycling are other rehabilitation alternatives that have been used to correct the problem of rutting. Data obtained from I-45 in Texas seemed to substantiate the analysis fairly thoroughly.

CRC Overlay of JRCP

A cost analysis of the design presented in chapter 3 is presented. Two cases may be considered here: one design did not take into account the restoration of the joints in the original pavement, and the other was a thickness design of an overlay that was placed on a pavement whose load transfer had been restored. The design for an overlay that was placed on the existing pavement without restoration of the load transfer required a thickness of 292 mm (11.5 in) at 50 percent reliability. For a joint LTE of 96 percent in the original pavement, a thickness of 203 mm (8.0 in) was required at 50 percent reliability. Restoration of the concrete pavement is required when it reaches a certain level of distress (figure 52). The modeling of the CRC overlay was done using the EXPEAR program developed at the University of Illinois at Urbana-Champaign.[31] When the pavement develops a certain number of punchouts per mile (in our case, 10), some rehabilitation is required. The various activities involved in concrete pavement restoration are:

- Full-depth repair
- Partial-depth repair
- Load transfer restoration
- Slab stabilization by means of subsealing

Costs for each of the above activities are shown in table 5 as converted to present worth. The anticipated life-cycle cost analysis of both these designs is presented in tables 5 and 6, respectively.

Bonded Overlay of CRC with CRC

The thickness of such an overlay is much less than for unbonded ones, and not as much cost is involved in its construction. The major costs involved with this type of overlay are for surface preparation. It is already assumed that such an overlay can only be placed on an existing slab that is structurally sound. Hence, restoration of the original pavement will also not figure in the costs. Since the basic concept of a CRC pavement is for it to be ideally zero-

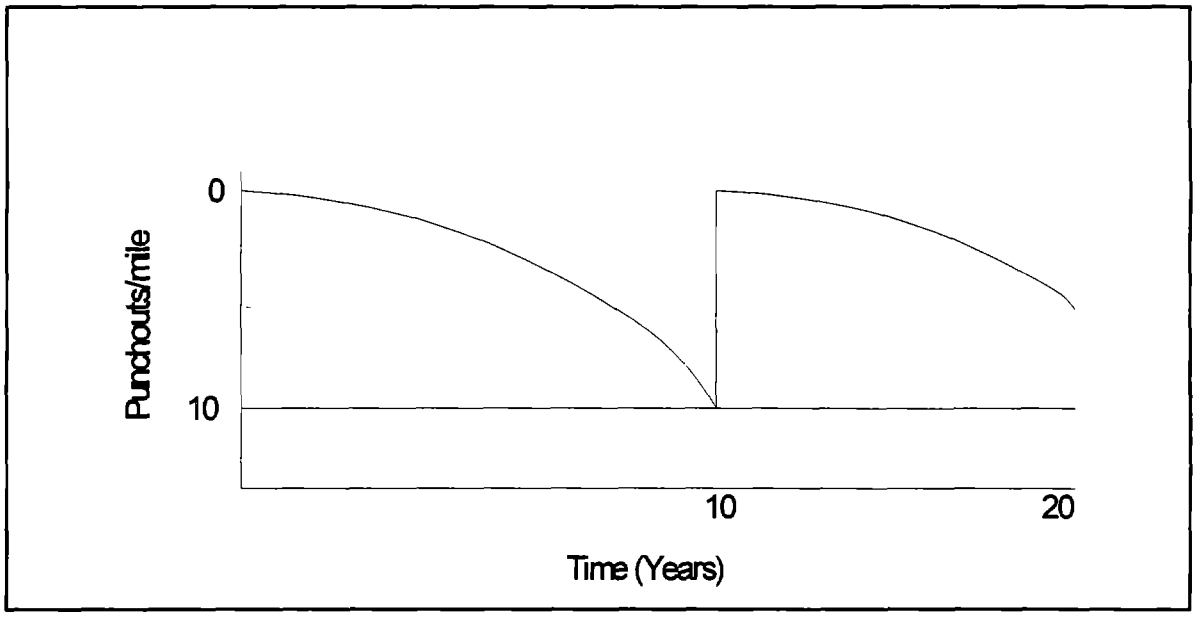


Figure 52. Rate of punchout development.

Table 5. 11" Unbonded CRC overlay.

Year	Item	Unit Cost	Total Quantity	Total $/mile	Present Worth
1	11" CRC Overlay, Placement	$35.00/CY	4290	150150	150150
10	CRC Longitudinal Joint Rout and Seal	$1.00/LF	5264	5264	3556
10	CRC Crack Rout and Seal	$0.80/LF	1908	1431	966
15	Slab Stabilization	$1.25/SY	3520	4400	2443
15	Full-depth Repairs	$60.00/SY	85	5088	2825
Total Present Worth, $/mile					159,940

1 LF = 0.305 M 1 SY = 0.836 M^2 1 CY = 0.760

Table 6. 8" Unbonded CRC overlay, after restoring joints in the existing pavement.

Year	Item	Unit Cost	Total Quantity	Total $/mile	Present Worth
1	Slab Stabilization	$1.25/SY	3520	4400	4400
1	Load Transfer Restoration	$65.00 Each	700	45500	45500
1	Full-depth Repairs	$65.00/SY	254	16510	16510
1	8" CRC Placement	$35.00/CY	3118.22	109137.	109137.
10	CRC Longitudinal Joint Rout and Seal	$1.00/LF	5264	5264	3556
10	CRC Crack Rout and Seal	$0.80/LF	1908	1431	966
Total Present Worth, $/mile					180,069

1 LF = 0.305 M 1 SY = 0.836 M^2 1CY = 0.760 M^3

maintenance, no major rehabilitation activity is foreseen. The thickness design for this overlay was done using the principles discussed in chapter 3. The major activities involved include the furnishing and placement of the bonded overlay, surface preparation, and routine joint and crack routing and sealing. The costs involved are shown in table 7.

Some of the conclusions that may be drawn from the cost-analysis study are:

1. The average life of an AC overlay due to rutting ranges from 5 to 6 years, and that another overlay is required at that time.
2. A bonded CRC overlay of a CRC pavement seems to be the most cost-effective rehabilitation alternative for a sound existing pavement. However, complete bonding must be achieved, notwithstanding which delamination may occur, leading to other problems. So, the most important aspect in a bonded overlay is its construction. If it is constructed properly, it can be a cost-effective alternative and can serve its life with little or no maintenance.
3. The cost-study also showed that an unbonded overlay of 279 mm (11 in) of CRC laid on an existing jointed pavement cost less than a 203-mm (8-in) CRC unbonded overlay of an existing jointed pavement whose joints had been restored.
4. The AC overlay of a CRC pavement was found to be the least cost-effective. This is because of the multiple overlays that have to be laid during its design life.

Table 7. CRC bonded overlay.

Year	Item	Unit Cost	Total Quantity	Total $/mile	Present Worth
1	4" CRC Bonded Overlay, Furnish	$35.00/CY	1558.35	54542.0	54542.0
1	4" CRC Bonded Overlay, Placement	$2.75/SY	14080	38720	38720
1	Surface Preparation	$1.25/SY	14080	17600	17600
10	CRC Longitudinal Joint Rout and Seal	$1.00/LF	5264	5264	3556
10	CRC Crack Rout and Seal	$0.80/LF	1908	1431	966
Total Present Worth of Option, $/mile					115,384

1 LF = 0.305 M 1 SY = 0.836 M^2 1 CY = 0.760 M^3

5. Though the cost of initial construction of an AC overlay is much lower than that of the other alternatives, in the long run, a rigid overlay constructed and designed properly provides a cost-effective solution.
6. The costs shown in the chapter are for 50 percent reliability. Higher reliability levels may yield different results.

CHAPTER 6 - SUMMARY, CONCLUSIONS, AND FURTHER DIRECTIONS IN RESEARCH

Though a number of experimental CRC overlays on existing concrete pavements have been constructed all over the United States, little documentation is available about their design and performance. This report reviewed some of the available literature on the performance of these overlays. A design framework is then presented that considers the development of punchout distress in overlays involving CRC pavements. Since the development of punchout distress has largely been associated with lack of adequate support and the loss of load transfer due to fatigue, a design framework has been suggested that characterizes the existing support conditions by making use of NDT data.

The effect of AC overlays on the long-term performance of CRCP was also studied. Their insulative effect as well as the life of the overlay in relation to both reflection cracking and rutting was studied. Several conclusions were drawn as a result of the analytical modeling of the distresses.

Conclusions

Some conclusions that were reached during the course of this study are as follows:

- Provision of adequate support is recognized as the focus of good performance of overlays involving CRC pavements.
- Available design procedures do not account for lack of support mechanistically in thickness design.
- Characterization of existing support conditions under the overlay may be done using field data like the deflections from the FWD.
- Load transfer in the existing pavement must be kept adequately high in order to maintain high stiffness.
- Fully supported conditions must be attained for an overlay to perform well. Fully supported conditions under the overlay may be achieved by providing sufficient stiffness to the pavement system.
- The design framework provided in chapter 4 may be applicable for the design of any type of overlay, whether rigid or flexible.
- The Integrated Climatic Effects Model was found to be a very effective tool in analyzing the effects of climate on pavements. It is a particularly useful method of simulating temperature conditions in pavements and may be used in design.
- On the basis of results of the ODE program, the following conclusions have been drawn:

    1. In pavements with good support conditions, the reflection cracks were delayed. This offers a good reason for repairing the pavement before overlaying. Permanent patching of punchouts and working cracks with tied or welded reinforced PCC full-depth repairs will delay for many years the occurrence and deterioration of reflection cracks in AC overlays of CRCP. Improving subdrainage conditions and

subsealing in areas where the slab has lost support will also discourage reflection crack occurrence and deterioration.

2. Regions with high number of freeze-thaw cycles are more prone to reflection crack occurrence and deterioration.
3. Condition of the existing PCC pavement plays a very important role in the development and deterioration of reflection cracks and, hence, particular attention must be paid to repair any working cracks before the overlay is laid.

- The analytical model (FLEXPASS) used to predict rutting was found to be very accurate, as verified by the information collected in the field.
- The use of AC overlays in climatic zones where daily and seasonal temperature drops are not high is not warranted.
- The additional thickness of the AC overlay does not contribute to the stiffness of the pavement.

Recommendations for Future Research

Many areas were identified through the course of this research as being in need of further development, as follows:

- Current design procedures for overlays involving the use of CRC pavement are empirical in nature, and mechanistic designs should replace them. The design procedures should take into consideration the development of distress from the pavement support standpoint as well as the fatigue standpoint.
- Load-transfer-required thickness relationships should be developed, similar to that developed by Zollinger and Barenberg[2] to take fatigue into consideration.
- Little information is available on the performance of various overlay types and their relationship to the development of distresses. Documentation of distresses and their modes of failure will be very useful in developing guidelines for future developments in thickness design of overlays involving CRC pavements.
- No method is available that will accurately predict the development and deterioration of reflection cracks of AC overlays on PCC pavements. This area deserves further study.
- A survival analysis of AC overlays on CRC pavements should be developed.

LIST OF REFERENCES

1. AASHTO Guide for Design of Pavement Structures, American Association of State Highway and Transportation Officials, Washington, D.C., 1986.

2. Zollinger, D.G., and Barenberg, E.J., "Continuously Reinforced Pavements: Punchouts and Other Distresses and Implications for Design," FHWA/IL/UI 227, University of Illinois, IHR-518, Illinois Cooperative Highway Research Program, March 1990.

3. Smith, R.E., Palmieri, R.P., Darter, M.I., Lytton, R.L., "Pavement Overlay Design Procedures and Assumptions, Volume I, Analysis of Existing Procedures," FHWA/RD-85/006, October 1984.

4. Smith, R.E., and R.L. Lytton, "Synthesis Study of Nondestructive Testing Devices for Use in Overlay Thickness Design of Flexible Pavements," FHWA/RD-83/097, Federal Highway Administration, U.S.D.O.T., Washington, D.C., 1983.

5. Hall, K.T., "Performance, Evaluation, and Rehabilitation of Asphalt-Overlaid Concrete Pavements," Ph.D. dissertation, University of Illinois at Urbana-Champaign, 1991.

6. Miner, M.A., "Cumulative Damage in Fatigue," Journal of Applied Mechanics, September 1945.

7. Yoder, E.J., and Witczak, E.M., "Principles of Pavement Design," John Wiley & Sons, Inc., New York.

8. Tayabji, S.D., and Okamoto, P.A., "Thickness Design of Concrete Resurfacing," Third International Conference on Concrete Pavement Design and Rehabilitation, Purdue University, West Lafayette, Indiana, 1985.

9. Hutchinson, R.L., "Resurfacing with Portland Cement Concrete," National Cooperative Highway Research Program-Synthesis of Highway Practice Special Report 99, 1982.

10. Department of the Army and the Air Force, "Rigid Pavements for Airfields," Technical Manual 5-825-3, Washington, D.C., August 1988.

11. van Metzinger, W.A., McCullough, F.B., Fowler, D.W., "An Empirical-Mechanistic Design Method Using Bonded Concrete Overlays for the Rehabilitation of Pavements," CTR Research Report 1205-1, January 1991.

12. Schnitter, O., Hudson, W.R., and McCullough, B.F., "A Rigid Pavement Overlay Design Procedure for Texas SHDPT," Research Report 177-13, Center for Highway Research, The University of Texas at Austin, May 1978.

13. Sriraman, S., Zollinger, D.G., "Application of Reliability to the Preliminary Analysis of Overlay Designs," Presented at The 72nd Annual Meeting, Transportation Research Board, Washington, D.C., January 1993.

14. Treybig, H.J., McCullough, B.F., Smith, P., and Von Quintus, H., "Overlay Design and Reflection Cracking Analysis for Rigid Pavements," Volume 2-Design Procedures, Research Report No. FHWA-RD-77-67, Federal Highway Administration, Washington, D.C., August 1977.

15. Gregory, J.M., "The Continuously Reinforced Concrete Overlay on Trunk Road A3 at Horndean," Transportation and Road Research Laboratory, Supplementary Report 742, October 1982.

16. Treybig, H.J., McCullough, B.F., Hudson, W.R., "Continuously Reinforced Concrete Airfield Pavement," Vol. II: "Design Manual for Continuously Reinforced Concrete Overlay Pavements," U.S.A.F. Weapons Laboratory Technical Report 73-229 (Federal Aviation Administration Report RD-73-33-2), 1974.

17. Kingham, R.I., "Development of the Asphalt Institute's Deflection Method for Designing Asphalt Concrete Overlays for Asphalt Pavements," Research Report 69-3, The Asphalt Institute, College Park, Maryland, 1969.

18. "Asphalt Overlays for Highway and Street Rehabilitation," Manual Series No. 17(MS-17), The Asphalt Institute, College Park, Maryland, 1983.

19. McCullough, B.F., "Design Manual for Continuously Reinforced Concrete Pavement," United States Steel Corporation, 1970.

20. "Design and Construction, Continuously Reinforced Concrete Overlays," Concrete Reinforcing Steel Institute, Continuously Reinforced Pavement Group, Chicago, 1968.

21. McCullough, B.F., and Boedecker, K.J., "Use of Linear-Elastic Layered Theory for the Design of CRCP Overlays," Highway Research Record 291, 1969.

22. Martin, R., "Design Considerations for Resurfacing Pavements with Concrete," Highway Research Record No. 435, 1973.

23. Taute, A., McCullough, B.F., and Hudson, W.R., "Improvements to the Materials Characterization and Fatigue Life Prediction Methods of the Texas Rigid Pavement Overlay Design Procedure," Research Report 249-1, Center for Transportation Research, The University of Texas at Austin, November 1981.

24. Treybig, H.J., McCullough, B.F., Smith, P., and Von Quintus, H., "Overlay Design and Reflection Cracking Analysis for Rigid Pavements, Vol. I, Development of New Design Criteria," Report No. FHWA-RD-77-66. Federal Highway Administration, U.S. Department of Transportation, Washington, D.C., 1981.

25. "Continuously Reinforced Concrete Overlays: 1975 Condition Survey," Publication SR 180.01 P, Ad Hod Committee for Concrete Overlays, Portland Cement Association, 1976.

26. Lokken, E.C., "Concrete Overlays for Concrete and Asphalt Pavements," Proceedings, 2nd International Conference on Concrete Pavement Design. Purdue University, April 14-16, 1981.

27. Schnoor, C.F., and E.J. Renier, "Portland Cement Concrete Overlays of Existing Asphalt Concrete Secondary Roads in Iowa," Transportation Research Record 702, 1979.

28. Crawley, A.B., "CRC Overlay of Existing CRCP," Report No. MSHD-RD-82-074-I, Mississippi State Highway Department and Federal Highway Administration, March 1982.

29. Schapery, R.A., "A Theory of Crack Growth in Viscoelastic Media," Report MM-2764-73-1, Mechanics and Materials Research Center, Texas A&M University, 1973.

30 Mellinger, F.M., "Structural Design of Concrete Overlays," Journal of the American Concrete Institute, Vol. 60, No. 2, February 1963.

31. Darter, M.I., and Hall, K.T., "Structural Overlay Strategies for Jointed Concrete Pavements—Volume VI, Guidelines for the Selection of Rehabilitation Alternatives," [For Continuously Reinforced Concrete Pavements], FHWA-RD-89-145, 1990.

32. "Performance of CRC Overlays: A Study of CRC Resurfacing Projects in 4 States," Concrete Reinforcing Steel Institute, 1988.

33. Westall, W.G., "Concrete Overlays on Asphalt Pavements," Highway Research News, No. 22, February 1966.

34. Seeds, S., McCullough, B.F., and Hudson, W.R., "A Design System for Rigid Pavement Rehabilitation," Research Report 249-2, Center for Transportation Research, The University of Texas at Austin, January 1982.

35. "Rigid Pavements for Airfields Other Than Army," Departments of the Army and the Air Force Technical Manual, Army TM 5-824-3, Air Force AFM 88-6, Ch. 3, 1979.

36. Turgeon, R., and Ishman, K.D., "Evaluation of Continuously Reinforced Concrete Overlay and Repairs on Interstate Route 900, Erie County, Pennsylvania," Research Project 79-01, Commonwealth of Pennsylvania, Department of Transportation, November 1985.

37. Parry, J.M., "Investigation of Punchouts in Continuously Reinforced Concrete Overlay on I-94 in Jackson and Moore Counties," Research Project 0624-42-41, Study No. 85-5, Division of Highways and Transportation Services, Wisconsin Department of Transportation.

38. Conversation with Richard Turner, Design Engineer, District 18, Mississippi State Highway Department of Transportation.

39. Felt, E.J., "Resurfacing and Patching Concrete Pavement with Bonded Concrete," Highway Research Board Proceedings, Vol. 35, Highway Research Board, National Research Council, Washington, D.C., 1956.

40. Felt, E.J., "Repair of Concrete Pavement," Journal of the American Concrete Institute, Vol. 57, No. 2, August 1960.

41. Darter, M.I., and Barenberg, E.J., "Bonded Concrete Overlays: Construction and Performance," Miscellaneous paper GL-80-11, Corps of Engineers Waterways Experiment Station, 1980.

42. Bagate, M, McCullough, F.B., Fowler, D.W., Muthu, M., "An Experimental Thin-Bonded Concrete Overlay Pavement," Research Report 357-2F, Center for Transportation Research, The University of Texas at Austin, November 1985.

43. Colley, B.E. and Humphrey, H.A., "Aggregate Interlock at Joints in Concrete Pavements," Highway Research Record, No. 189, Highway Research Board, Washington, D.C. 1967.

44. Voigt, G.F., Carpenter, S.H., and Darter M.I., "Rehabilitation of Concrete Pavements Volume II: Overlay Rehabilitation Techniques," Report No. FHWA-RD-88-072.

45. Betterton, R.H., Knutson, M.J., and Marks, V.J., "Fibrous P.C. Concrete Overlay Research in Greene County, Iowa." Research Project HR-165, Iowa Highway Research Board, 1978.

46. Page, G.C., "Rutting—Causes and Prevention," Prepared for Presentation at the Tri-Regional Pavement Rehabilitation Conference, May 1984.

47. Hoffman, M.S., and Thompson, M.R., "Mechanistic Interpretation of Nondestructive Pavement Testing Deflections," Civil Engineering Studies, Transportation Engineering Series No. 32, Illinois Cooperative Highway and Transportation Research Program Series No.190, University of Illinois at Urbana, Illinois.

48. Portland Cement Association, "Thickness Design for Concrete Highway and Street Pavements," EB109-01P, 1984.

49. Ioannides, A.M., Thompson M.R., and Barenberg, E.J., "The Westergaard Solutions Reconsidered," Department of Civil Engineering, University of Illinois at Urbana-Champaign, Paper presented at 1985 Annual Meeting of the Transportation Board, Washington, D.C., January 1985.

50. Yoder, E.J., "Maintenance Methods for Continuously Reinforced Concrete Pavements," FHWA/IN/JHRP-80/4, May 1980.

51. Foxworthy, P.T., "Concepts for the Development of a Nondestructive Testing and Evaluation System for Rigid Airfield Pavements," Ph.D. Thesis, Department of Civil Engineering, University of Illinois at Urbana-Champaign, 1985.

52. Ioannides, A.M., Khazanovich, L., and Becque, J.L., "Structural Evaluation of Base Layers in Concrete Pavement Systems," Transportation Research Board, 71st Annual Meeting, January 1992.

53. Darter, M.I., Elliott, R.P., and Hall, K.T., "Revision of AASHTO Overlay Design Procedures," Final Report, NCHRP Project 20-7/Task 39, 1991.

54. Tseng, K., "A Finite Element Method for the Performance Analysis of Flexible Pavements," Ph.D. Thesis, Department of Civil Engineering, Texas A&M University, August 1988.

55. Smith, K.D., Darter, M.I., Rauhut, J.B., and Hall, K.T.,"Distress Identification Manual for the Long-Term Pavement Performance (LTPP) Studies," Strategic Highway Research Program, 1988.

56. Brown, J.L., Buttler, L.J., and Ezzell, W.P.," Survey of Structural Failures in a Continuously Reinforced Concrete Pavement, "Report Number ss 1.1, Highway Design Division, Research Section, Texas Highway Department, October 1969.

57. La Coursiere, S.A., Darter, M.I., Smiley, S.A., "Performance of Continuously Reinforced Concrete Pavement in Illinois," Civil Engineering Studies, Transportation Engineering, Series No.10, University of Illinois, Urbana, Illinois, 1978.

58. Conversation with Guy Ward, District Engineer, Leon County, Texas Department of Transportation, November 1992.

59. Lytton, R.L., Pufahl, D.E., Michalak, C.H., Liang, H.S., and Dempsey, B.J.,"An Integrated Model of the Climatic Effects on Pavements", Report No. FHWA-RD-90-033, December 1989.

60. Dempsey, B.J., Herlach, W.A., and Patel, A.J., "The Climatic-Materials-Structural Pavement Analysis Program" Final Report, FHWA/RD-84/115, Volume 3, February 1985.

61. Majidzadeh, K., Buranarom, C., and Karakouzin, M., "Application of Fracture Mechanics for Improved Design of Bituminous Concrete," Report No. FHWA-RD-76-91, Vol. I, June 1976.

62. Majidzadeh, K., Ramsamooj, D.V., and Fletcher, T.A., "Analysis of Fatigue of Sand Asphalt Mixtures, "Proceedings, Association of Asphalt Paving Technologists, Vol. 30, 1969.

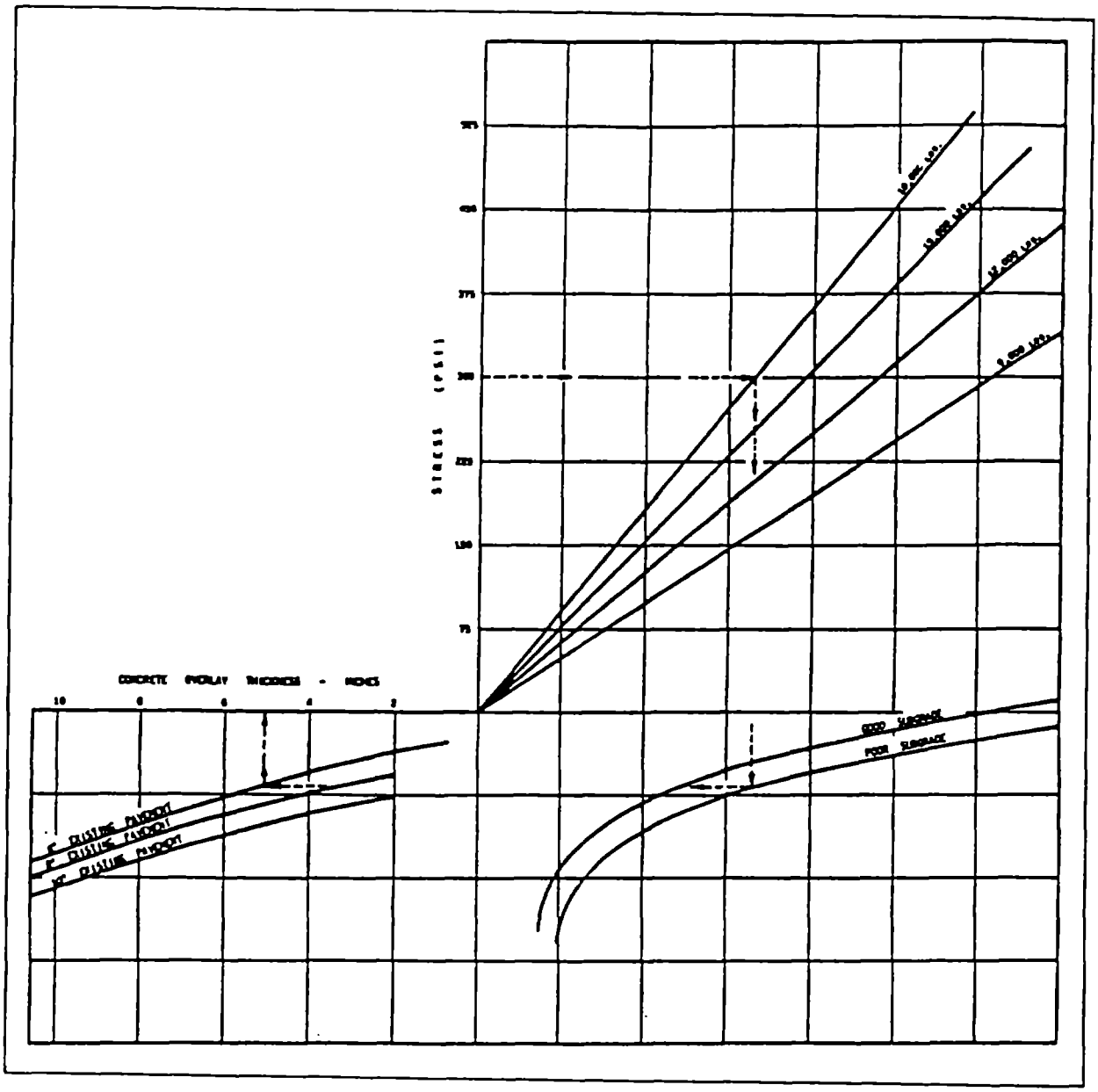
63. Majidzadeh, K., and Ramsamooj, D.V., "Development of Testing Procedures and a Method to Predict Fatigue Failures of Asphalt Concrete Pavement Systems," The Ohio State University, Research Foundation, Final Report, Project RF 2873, March 1973.

64. Jayawickrama, P.W., "Methodology for Predicting Asphalt Concrete Overlay Life Against Reflection Cracking," Ph.D. Thesis, Civil Engineering Department, Texas A&M University, May 1985.

65. McDermott, L.B., "Parts of Local I-30 Being Repaved, Rebuilt," *Texarkana Gazette*, Vol. 122, No. 111, Texarkana, Texas/Arkansas.

APPENDIX A-1

DESIGN CHARTS FOR CRC OVERLAYS USING LAYERED ELASTIC THEORY



1 in = 25.4 mm

Figure A-1. Design chart for CRC overlay (unbound, 0.5 to 1 in thick asphalt concrete unbonding medium).[18]

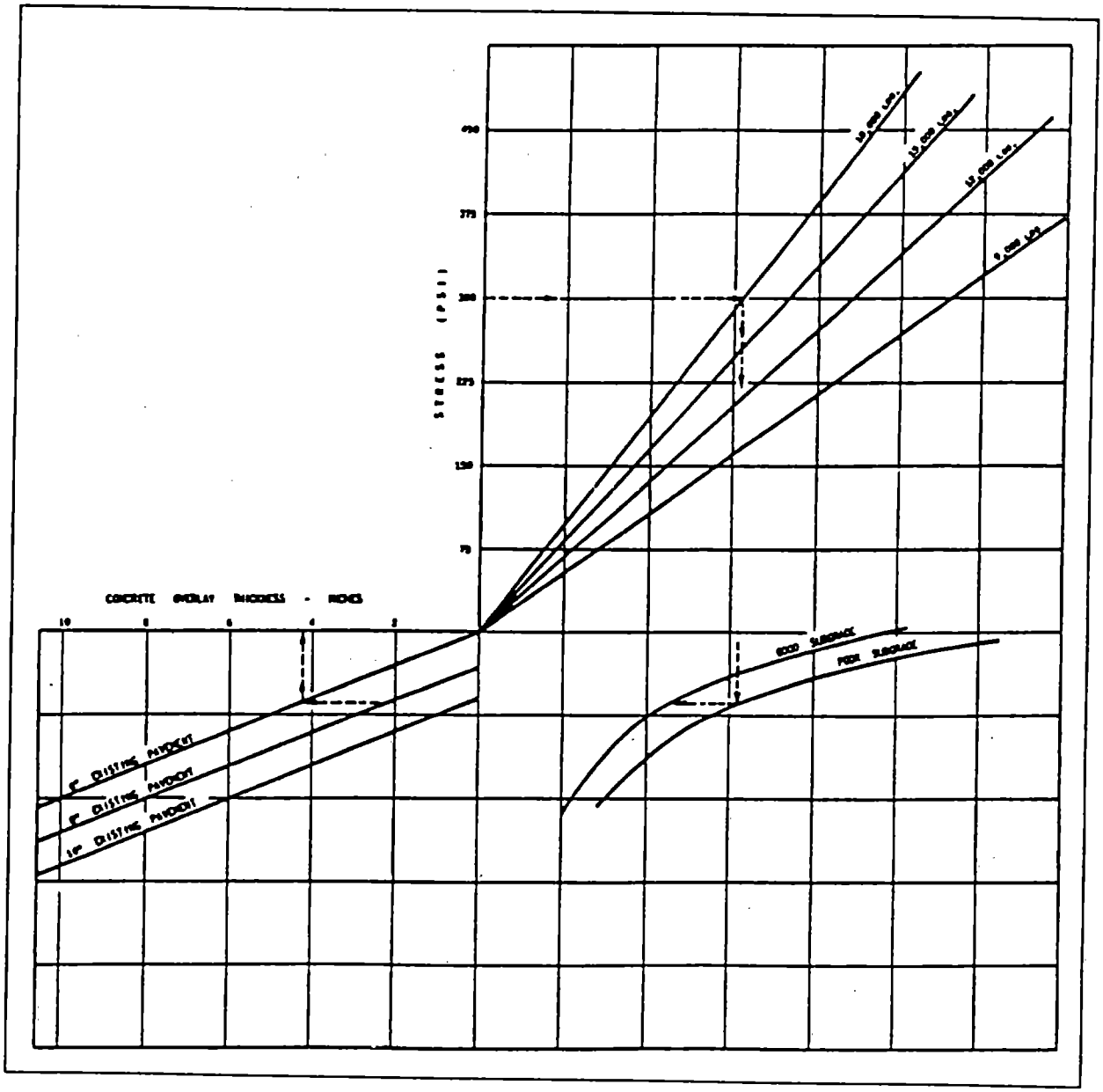


Figure A-2. Design chart for CRC overlay (partially bonded).[18]

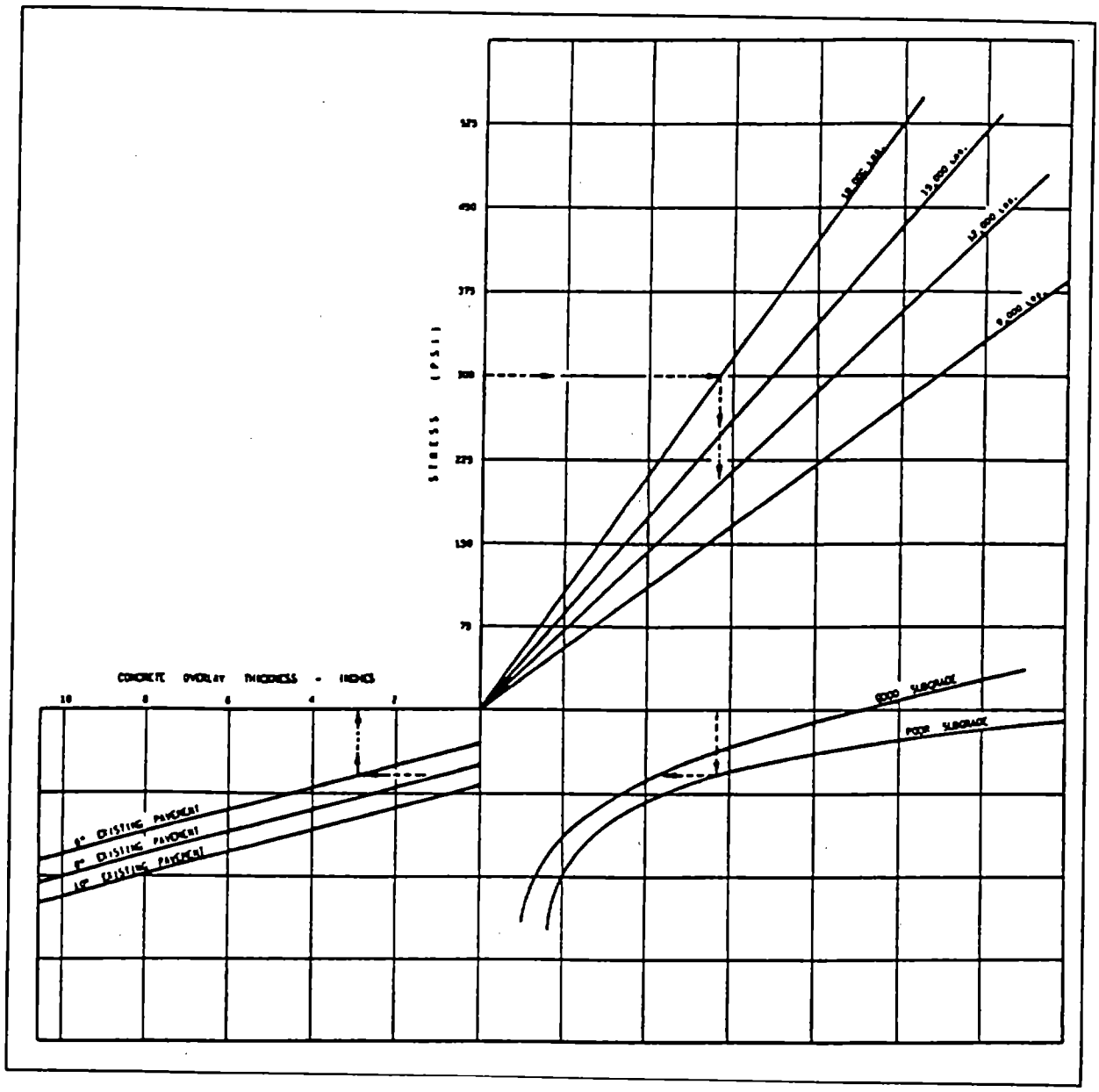


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Figure A-3. Design chart for CRC overlays (unbonded, 3-in thick asphalt concrete unbonding medium).[18]

APPENDIX A-2

PCA DESIGN CHARTS

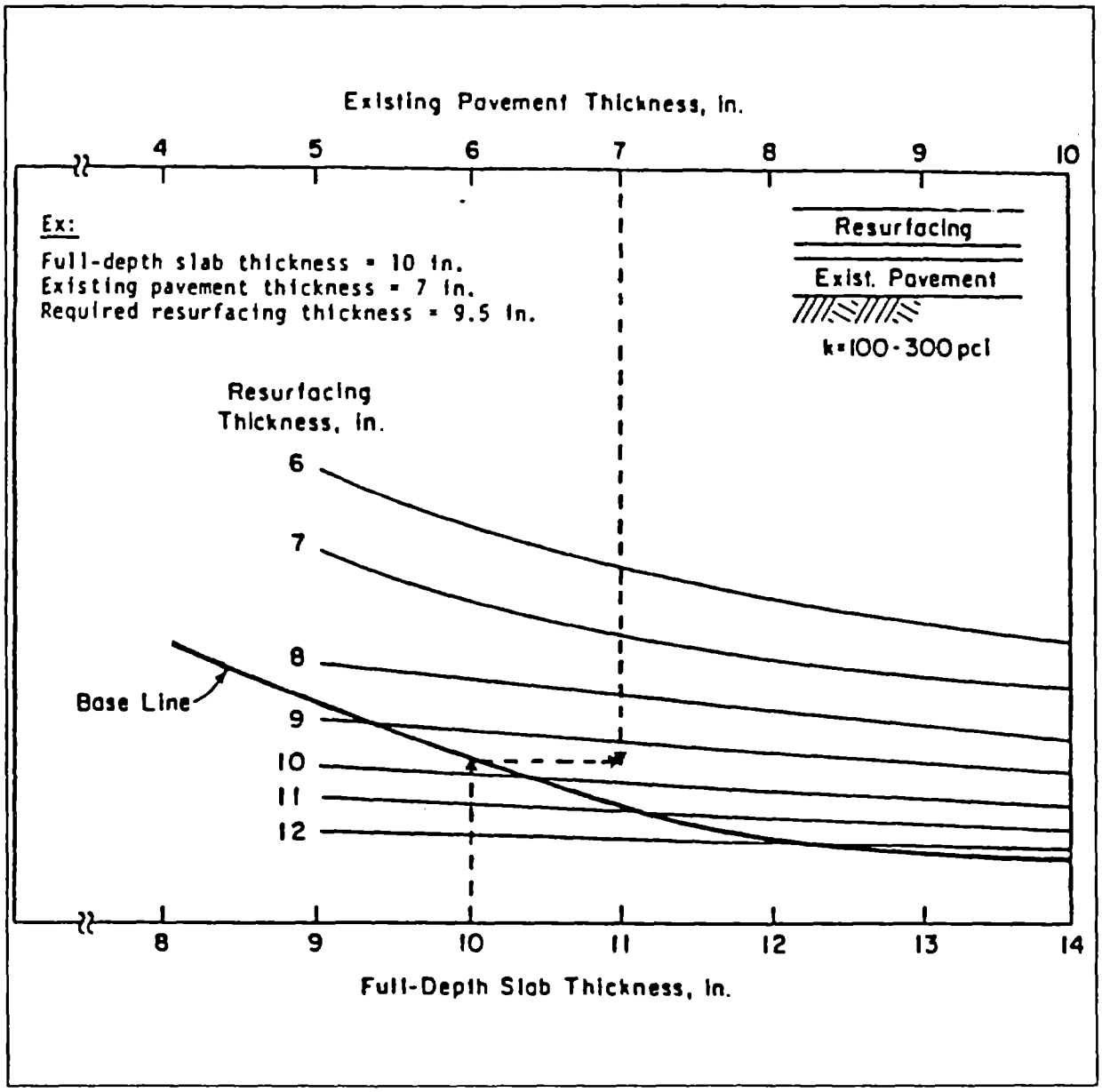

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Figure A-4. PCC overlay design chart for case 1 condition of existing pavement.[48]

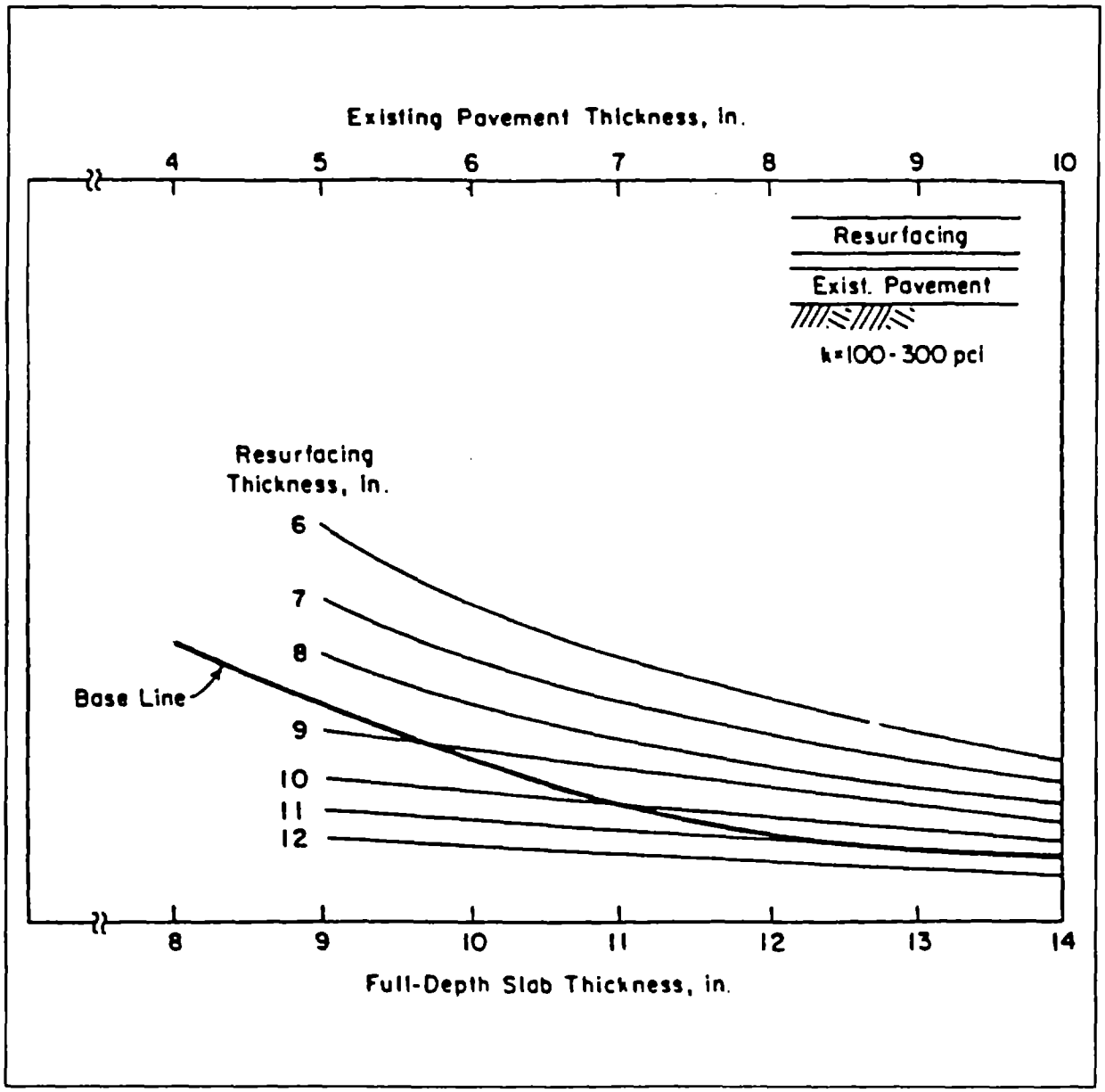

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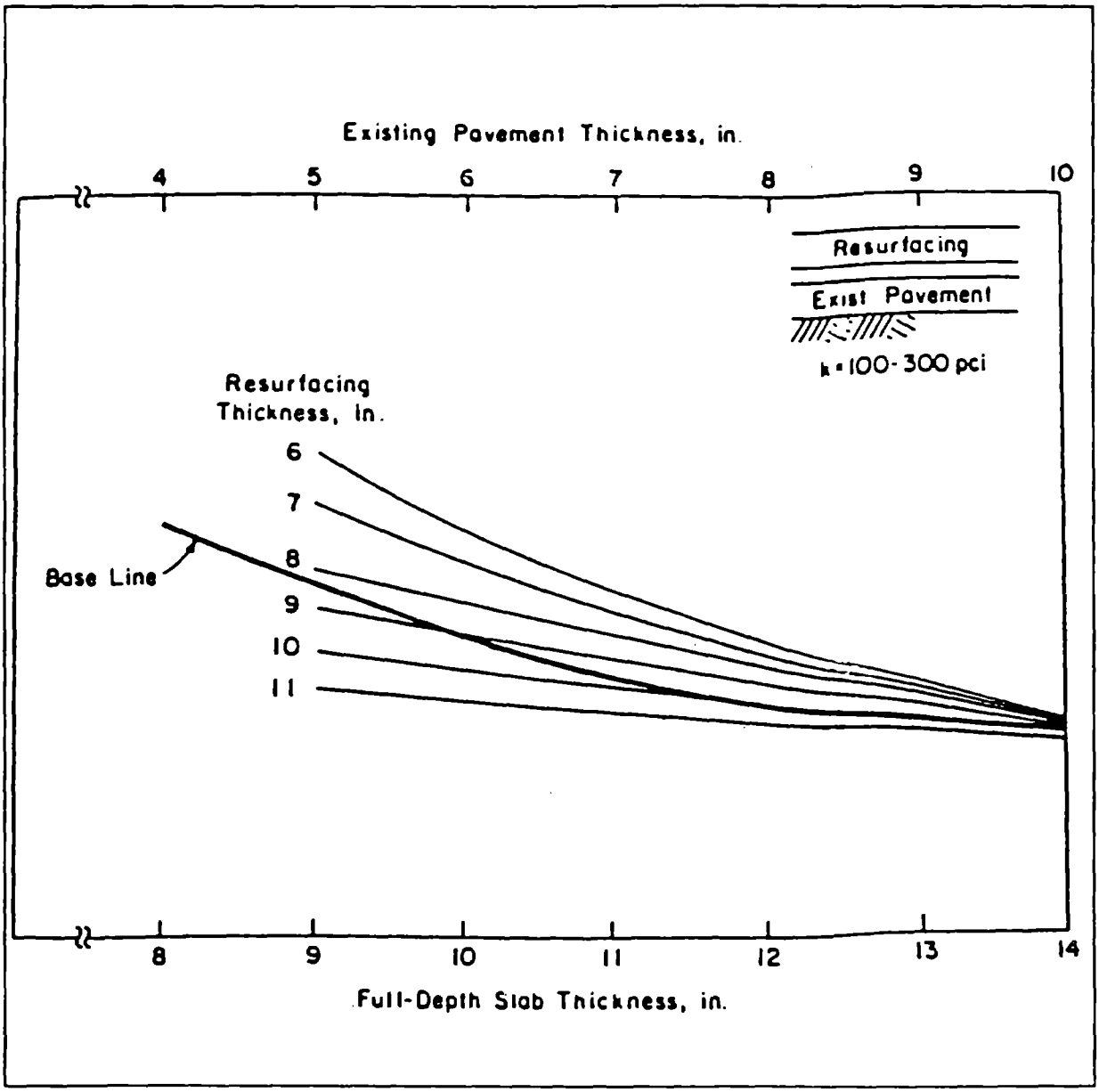
Figure A-5. PCC overlay design chart for case 2 condition of existing pavement.[48]



1 in = 25.4 mm

Figure A-6. PCC overlay design chart for case 3 condition of existing pavement.[48]

APPENDIX B

FWD DATA FOR THE ORIGINAL PAVEMENT: I-30W

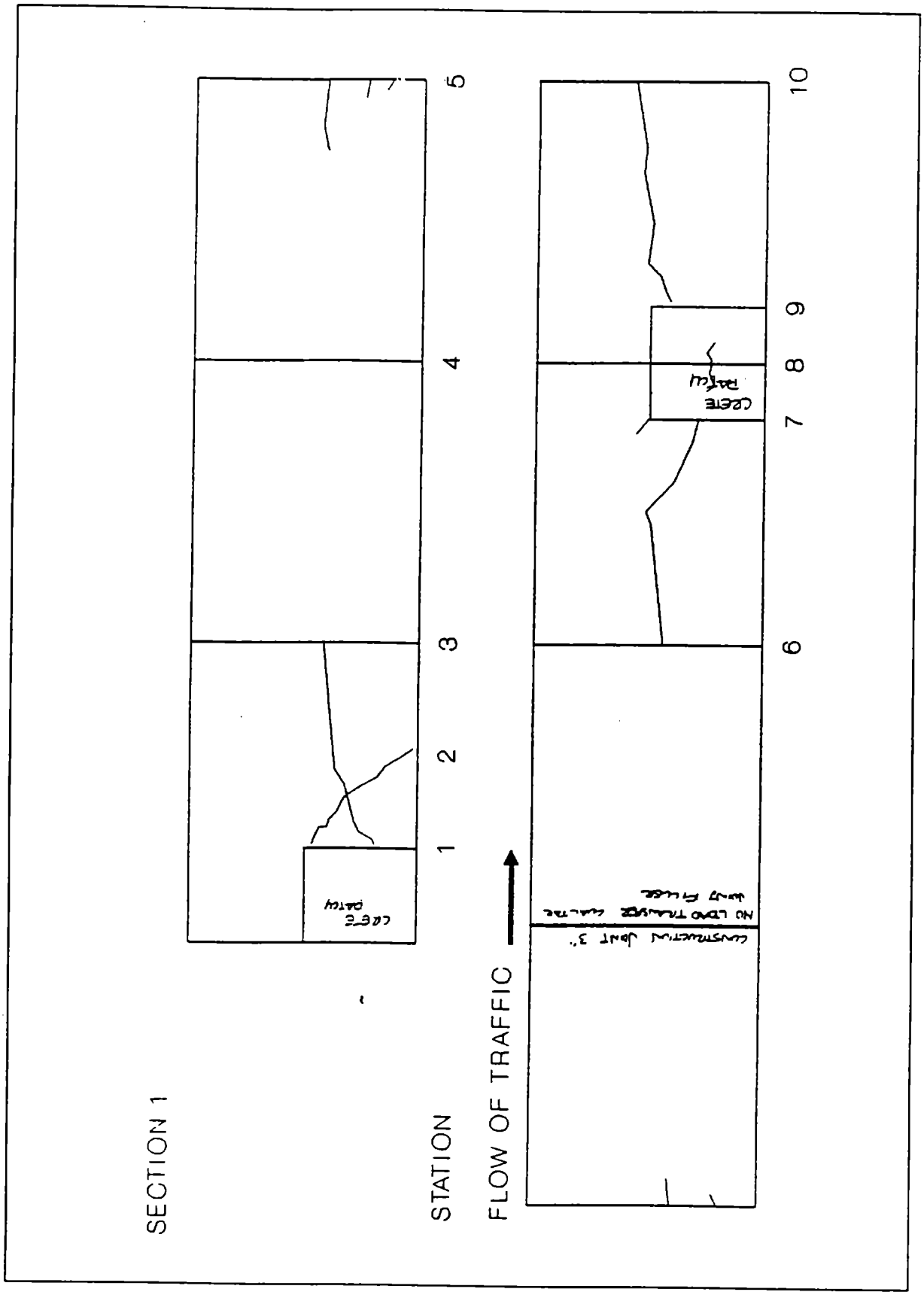


Figure B-1. Section 1: I-30W.

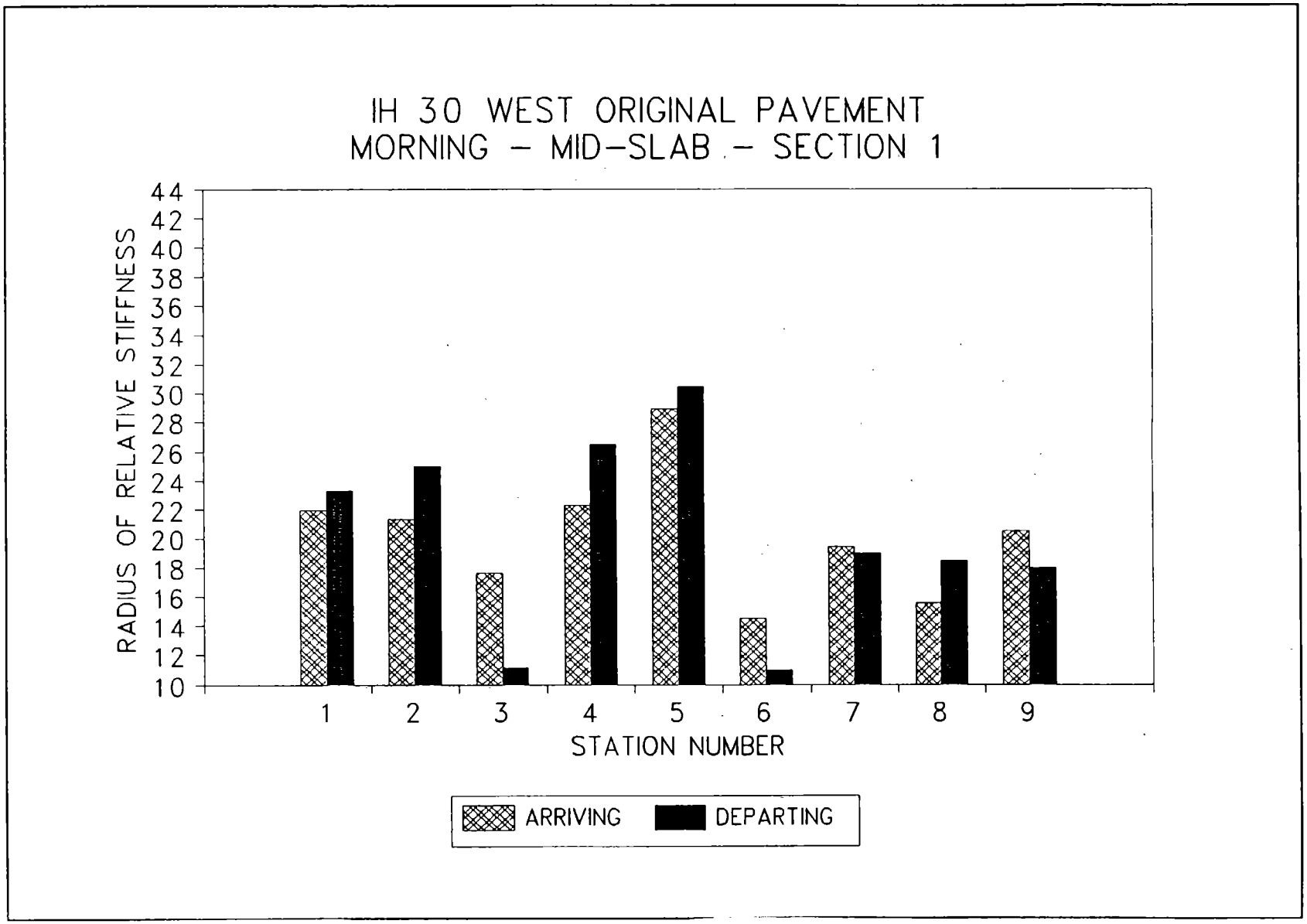


Figure B-2. Radius of relative stiffness versus station (I-30W, morning mid-slab section 1).

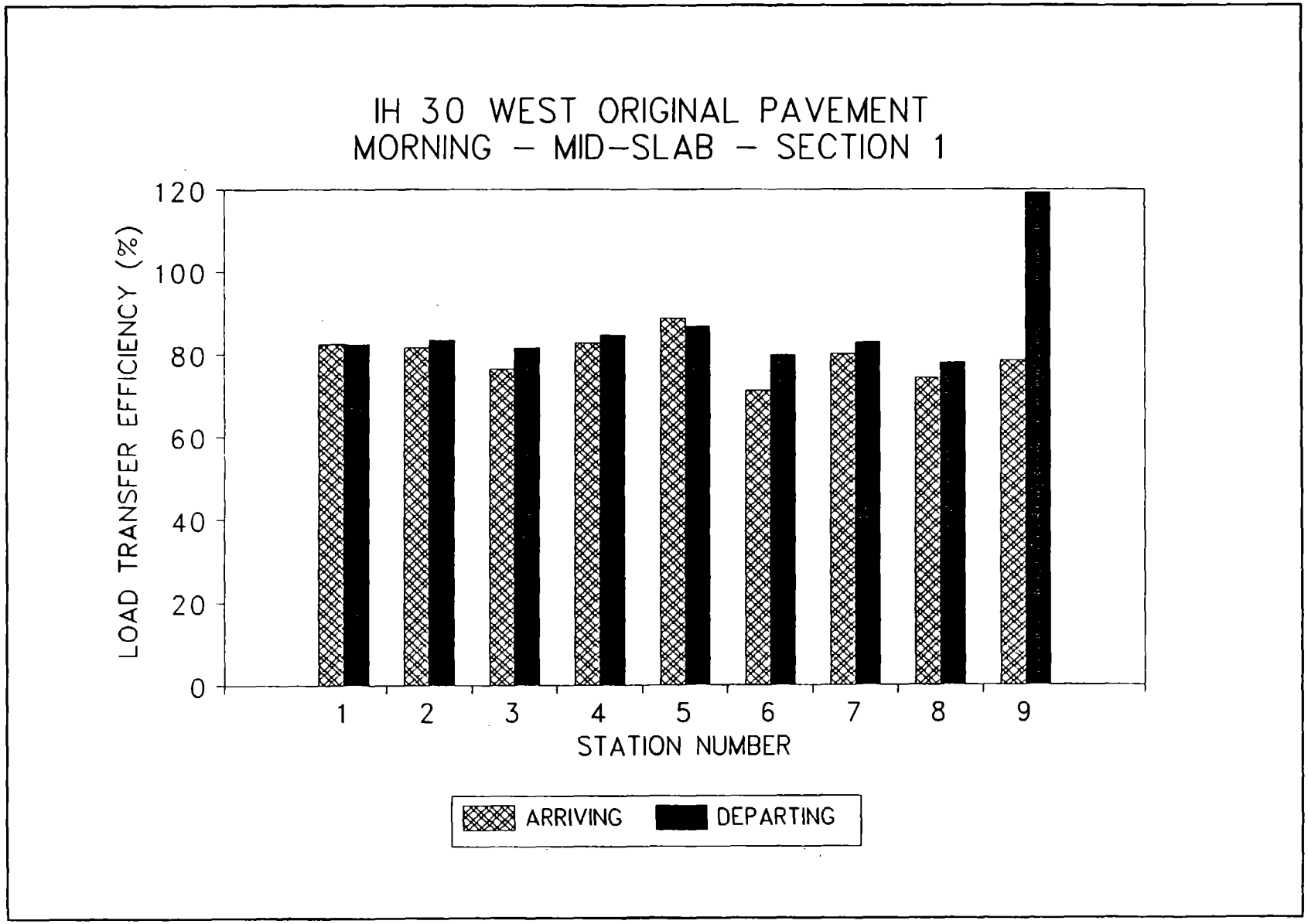


Figure B-3. Load transfer efficiency versus station (I-30W, morning mid-slab section 1).

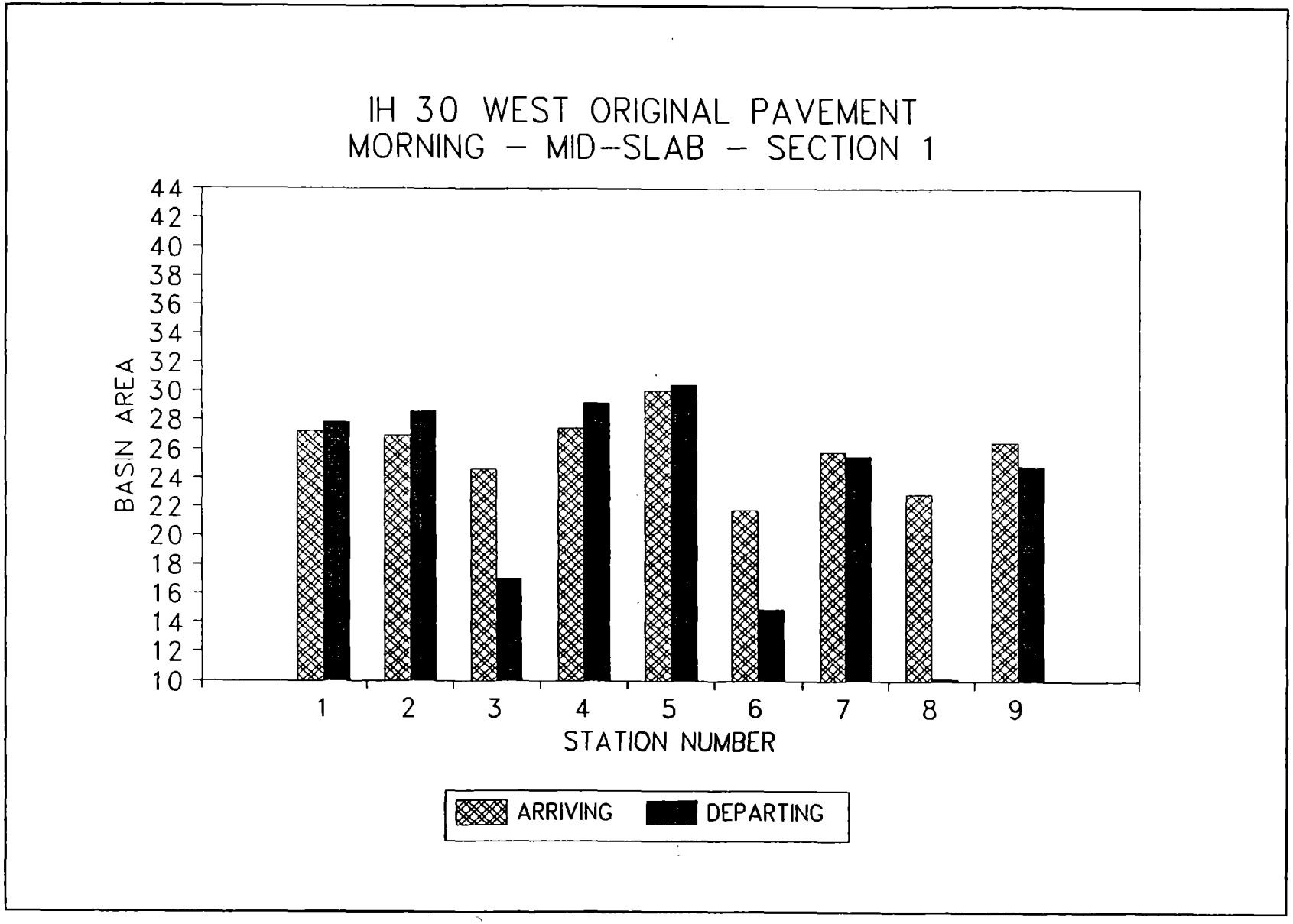


Figure B-4. Basin area versus station (I-30W, morning mid-slab section 1).

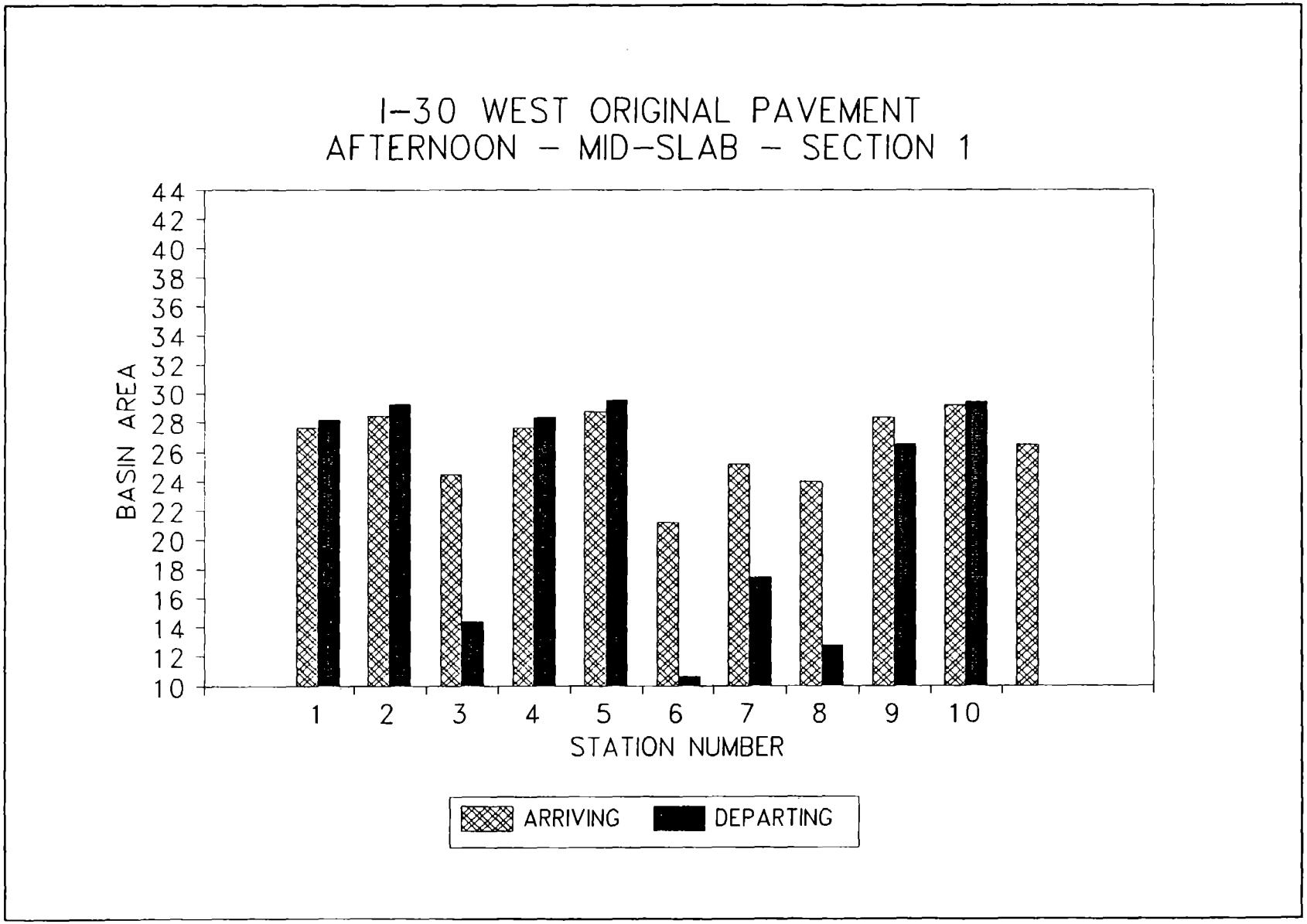


Figure B-5. Basin area versus station (I-30W, afternoon mid-slab section 1).

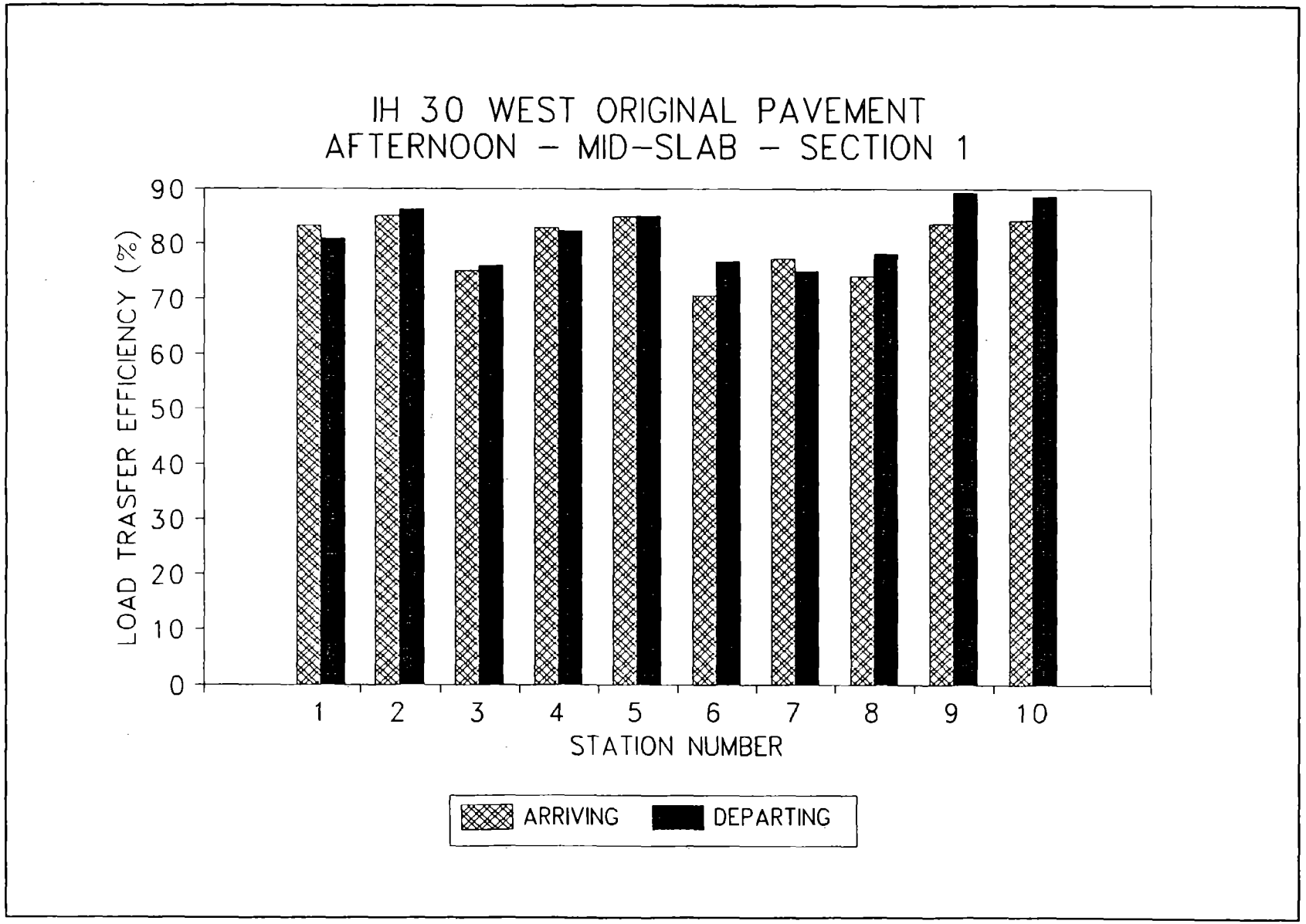


Figure B-6. Load transfer efficiency versus station (I-30W, afternoon mid-slab section 1).

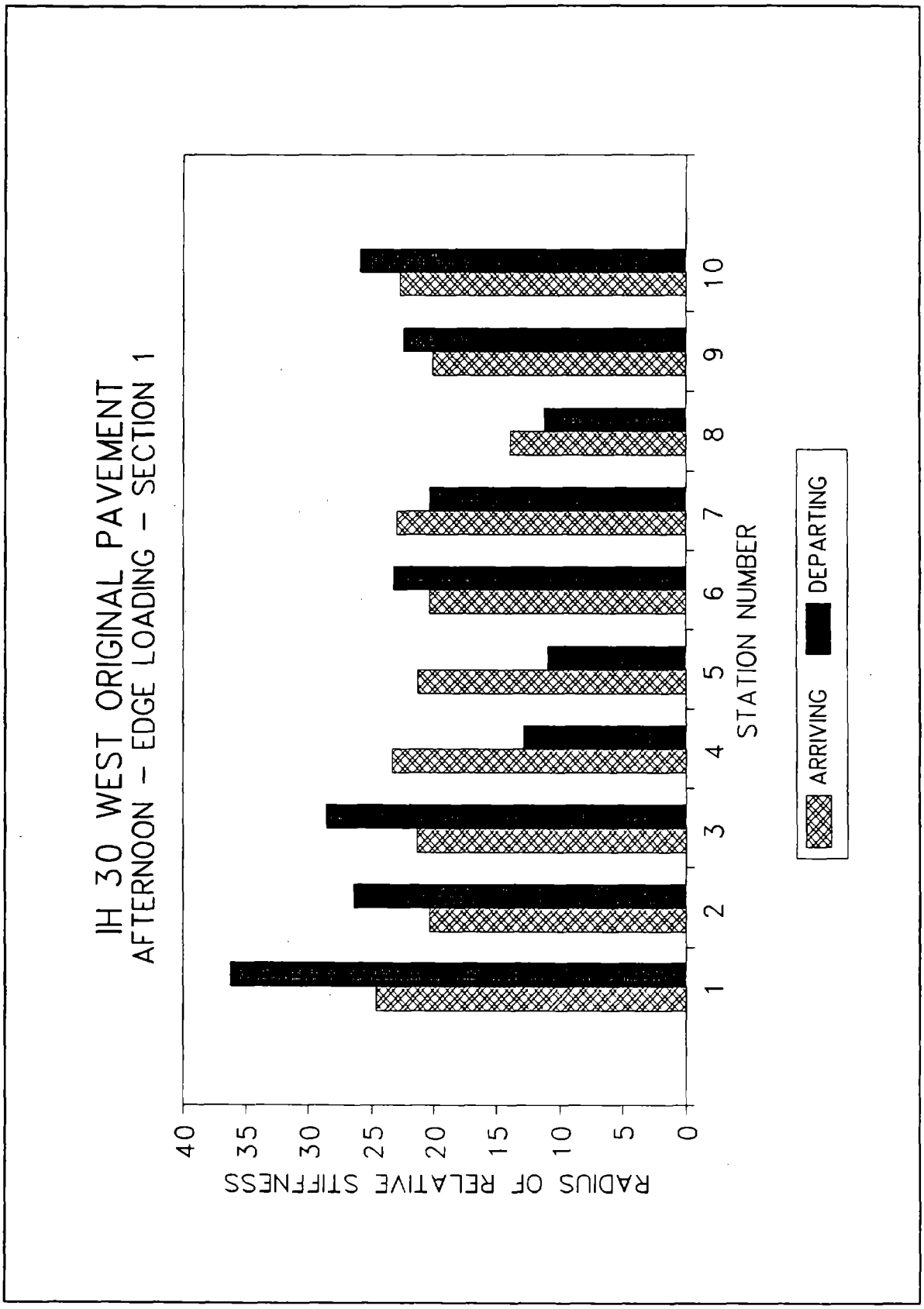


Figure B-7. ℓ versus station (I-30W, afternoon edge section 1).

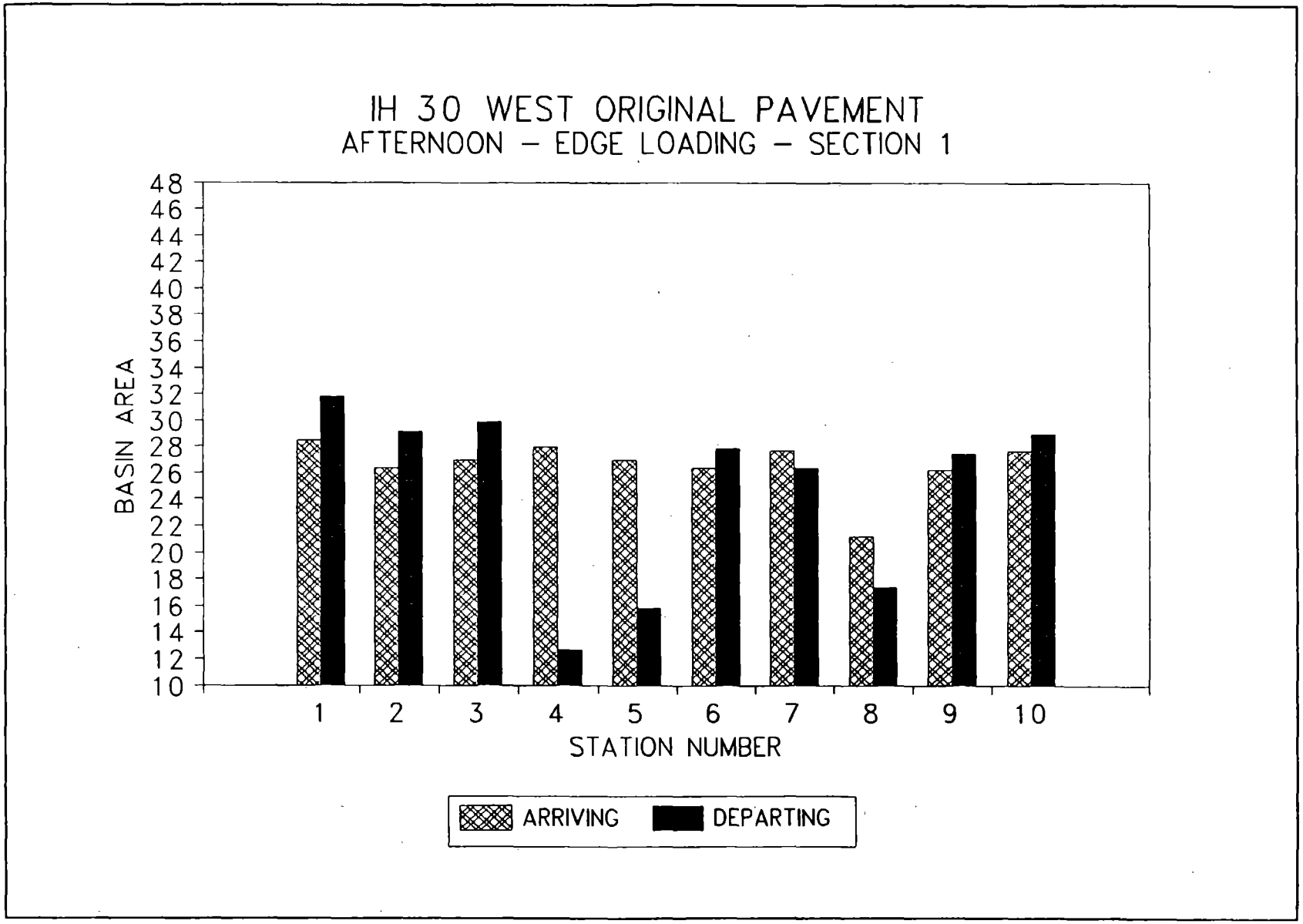


Figure B-8. Basin area versus station (I-30W, afternoon edge section1).

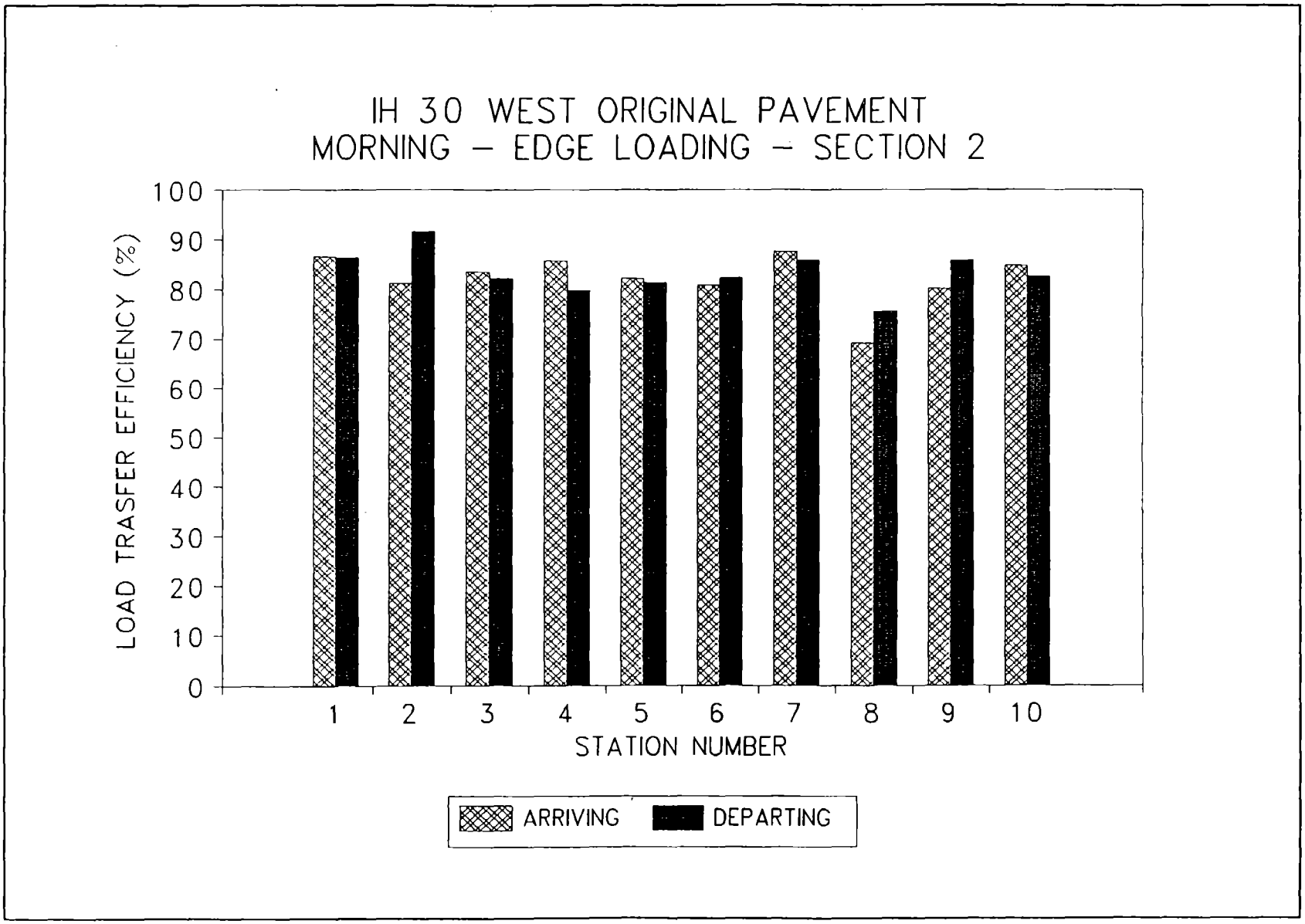


Figure B-9. LTE versus station (I-30W, morning, edge section 2).

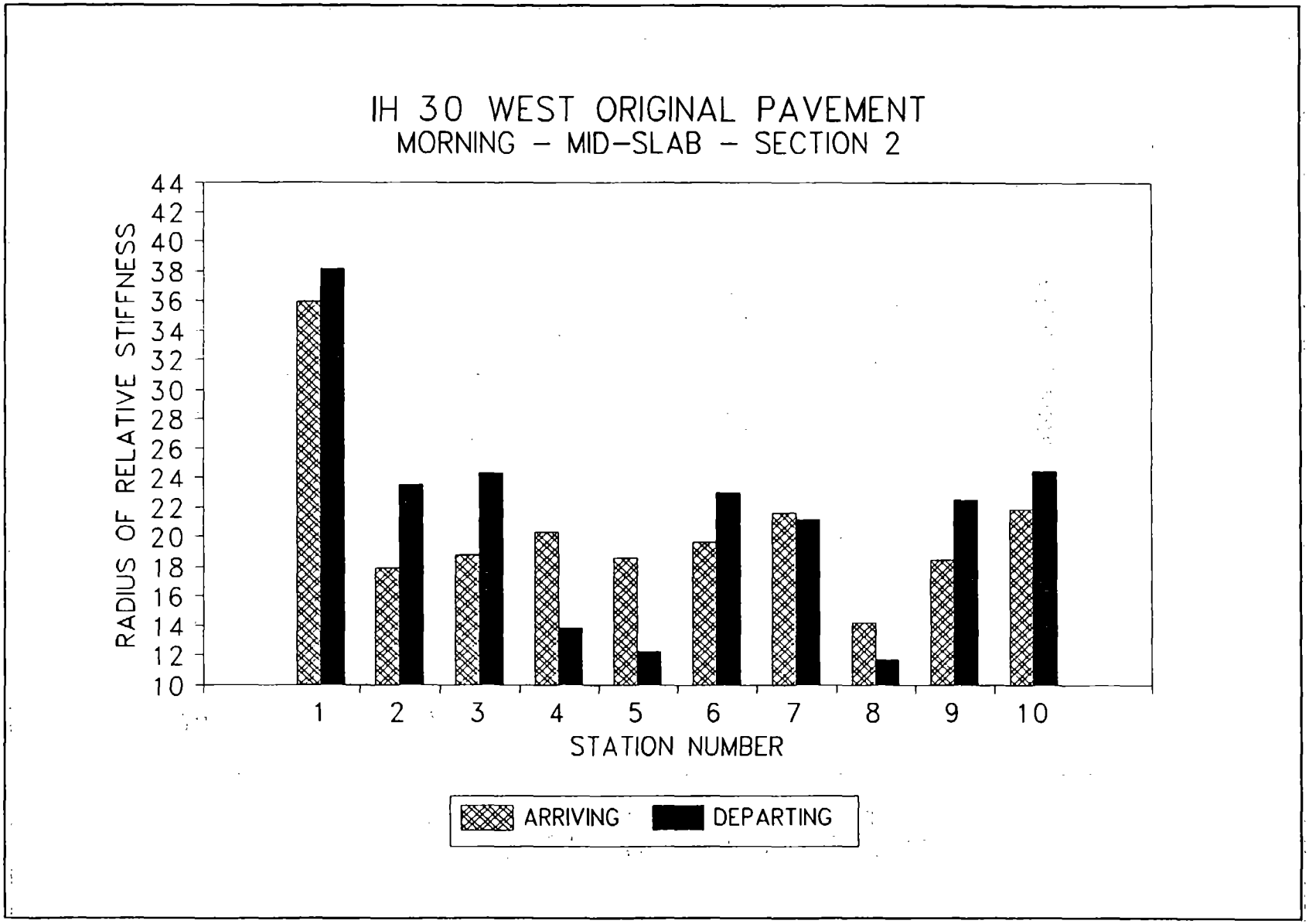


Figure B-10. ℓ versus station (I-30W, morning mid-slab section 2).

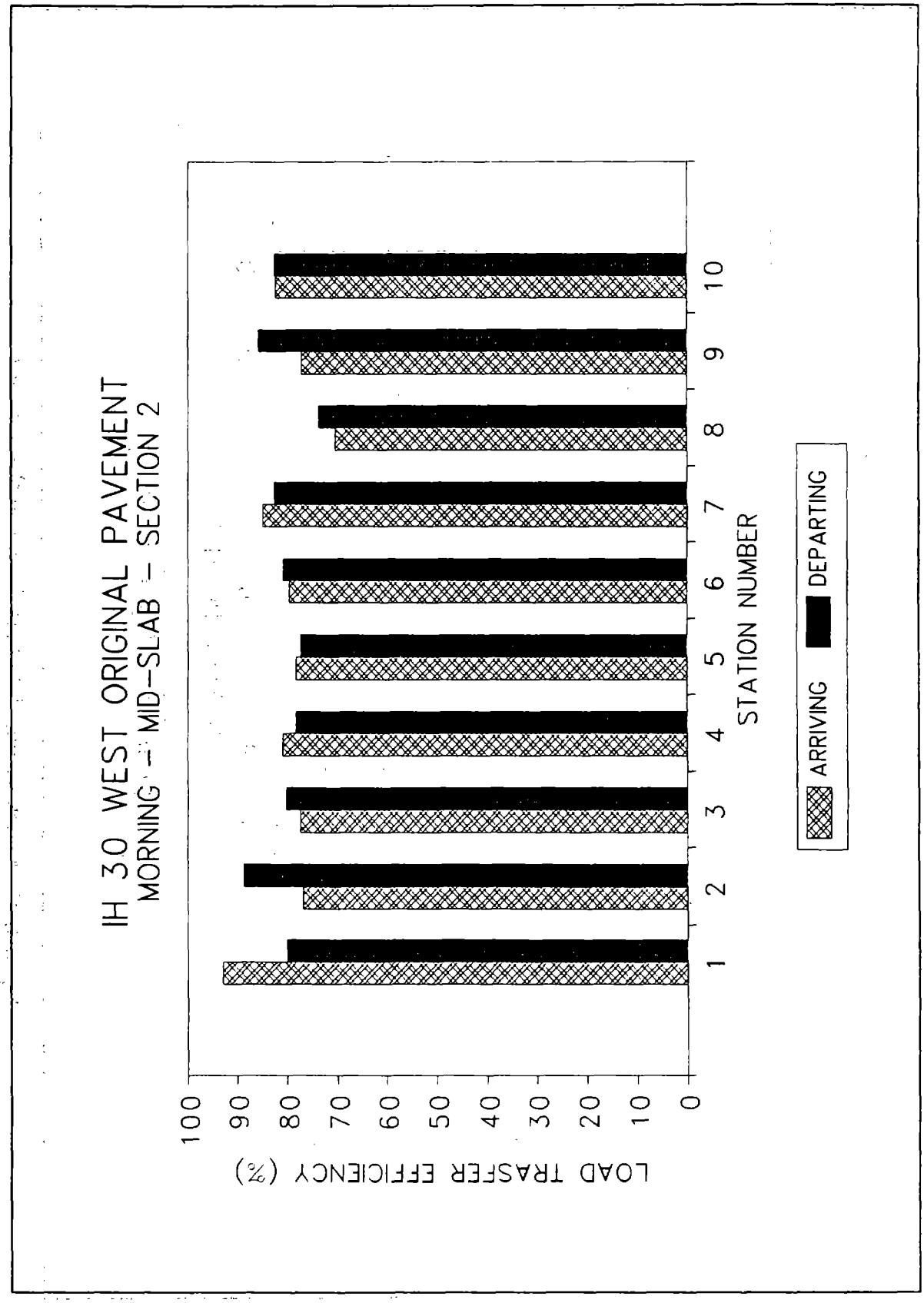


Figure B-11. LTE versus station (I-30W, morning mid-slab section 2).

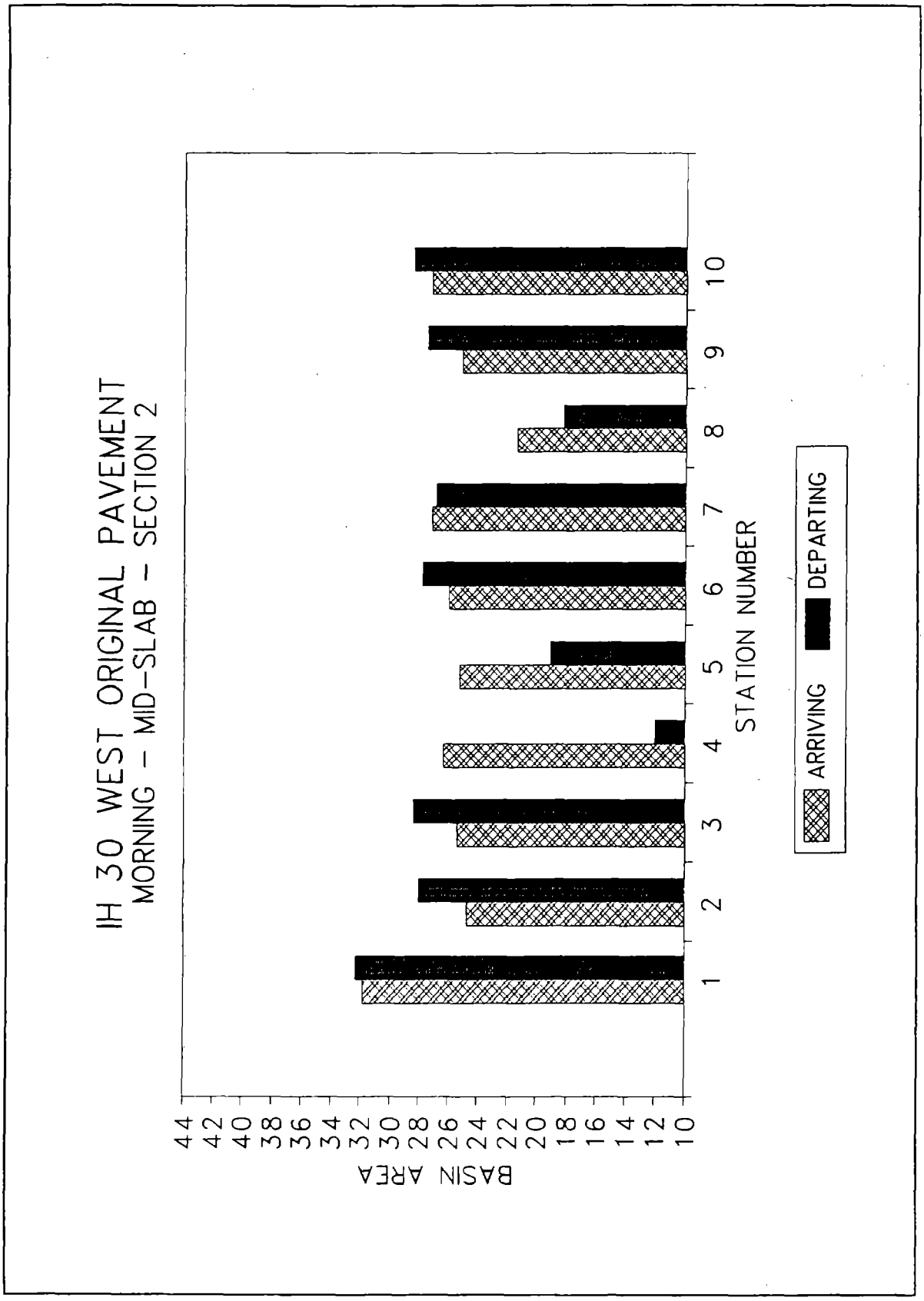


Figure B-12. Basin area versus station (I-30W, morning mid-slab section 2).

APPENDIX C

FWD DATA FOR THE CRC OVERLAY: I-30W

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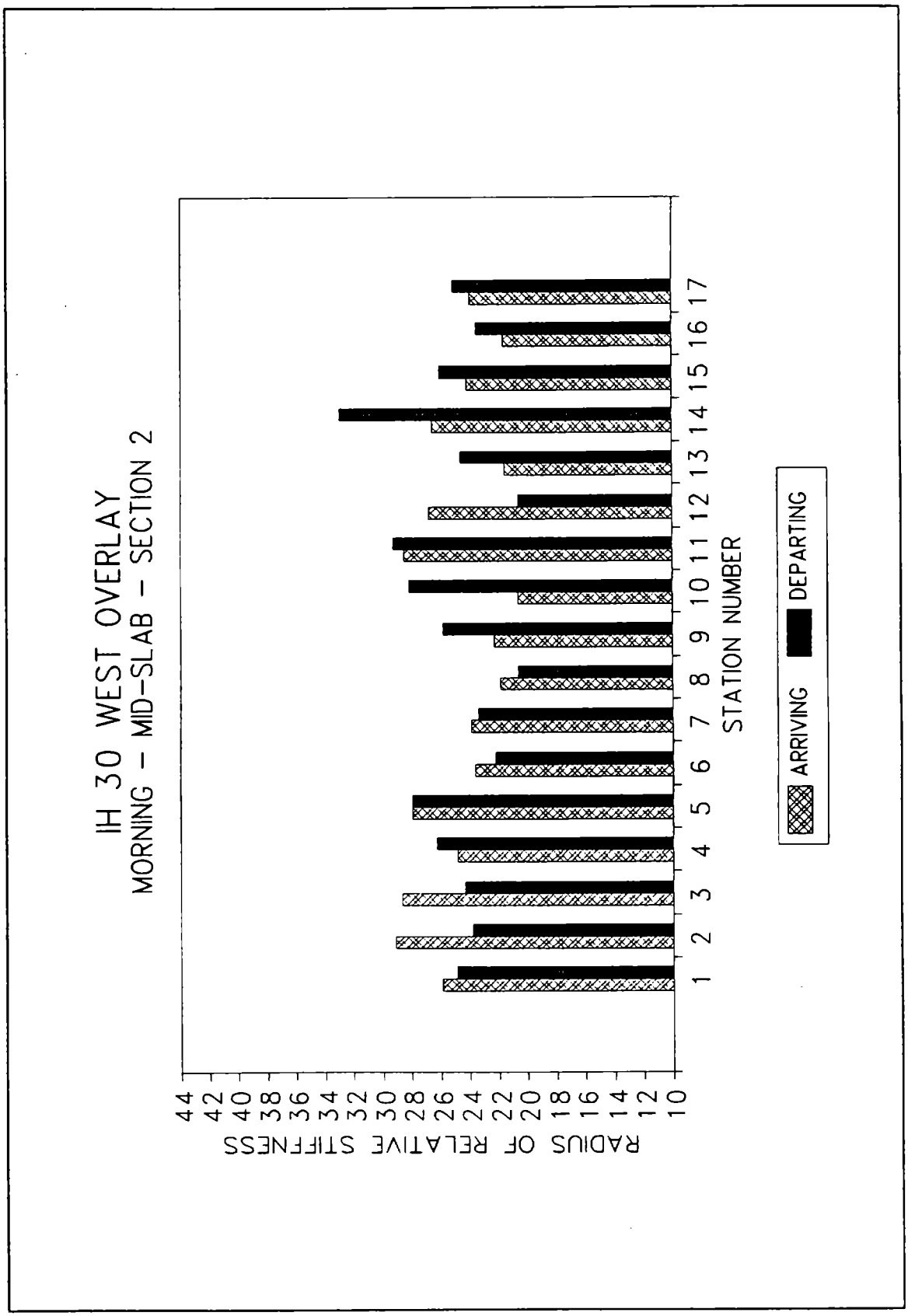

Figure C-1. ℓ versus station (I-30W, overlay, morning, mid-slab, section 2).

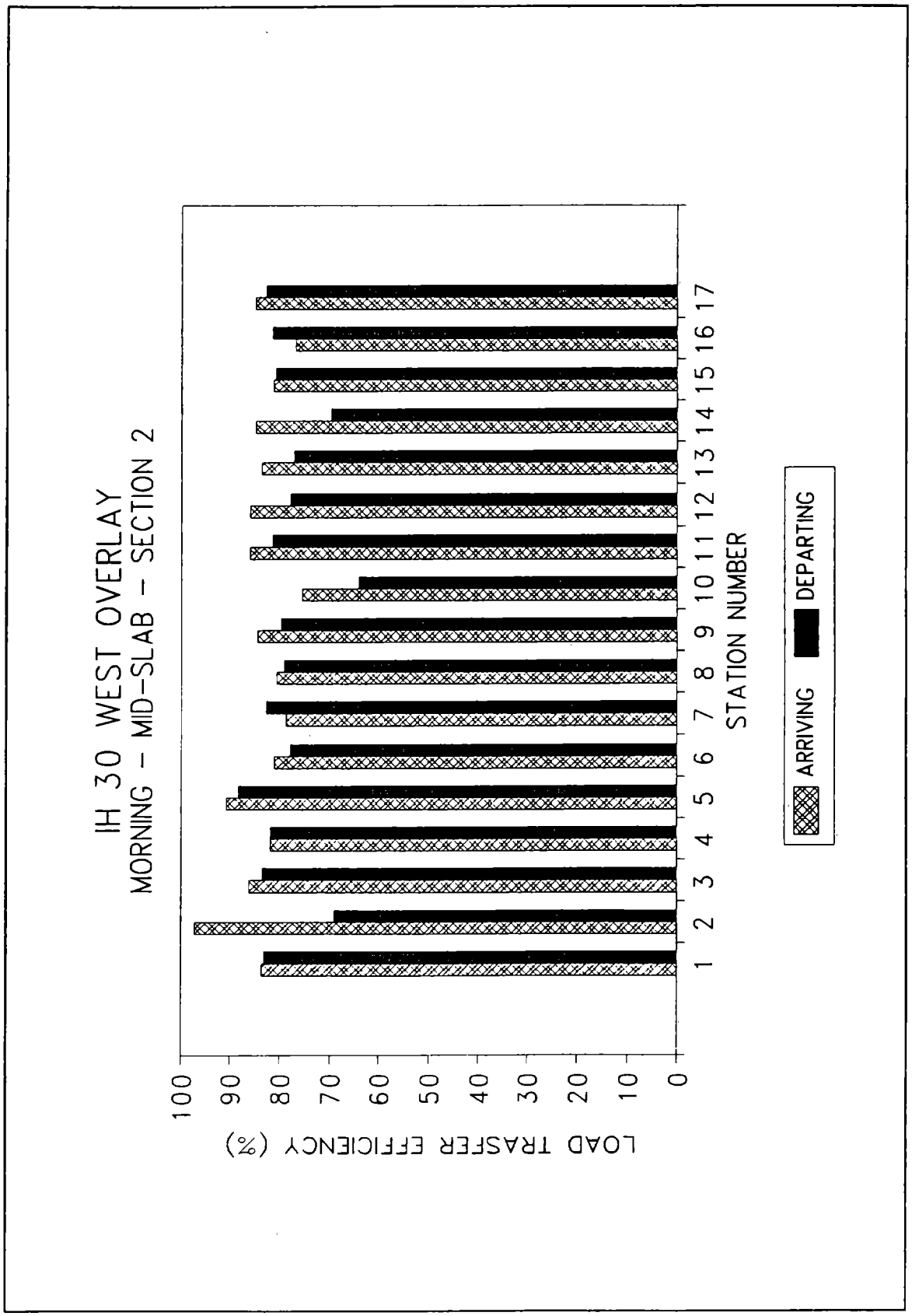

Figure C-2. LTE versus station (I-30W, overlay, morning, mid-slab, section 2).

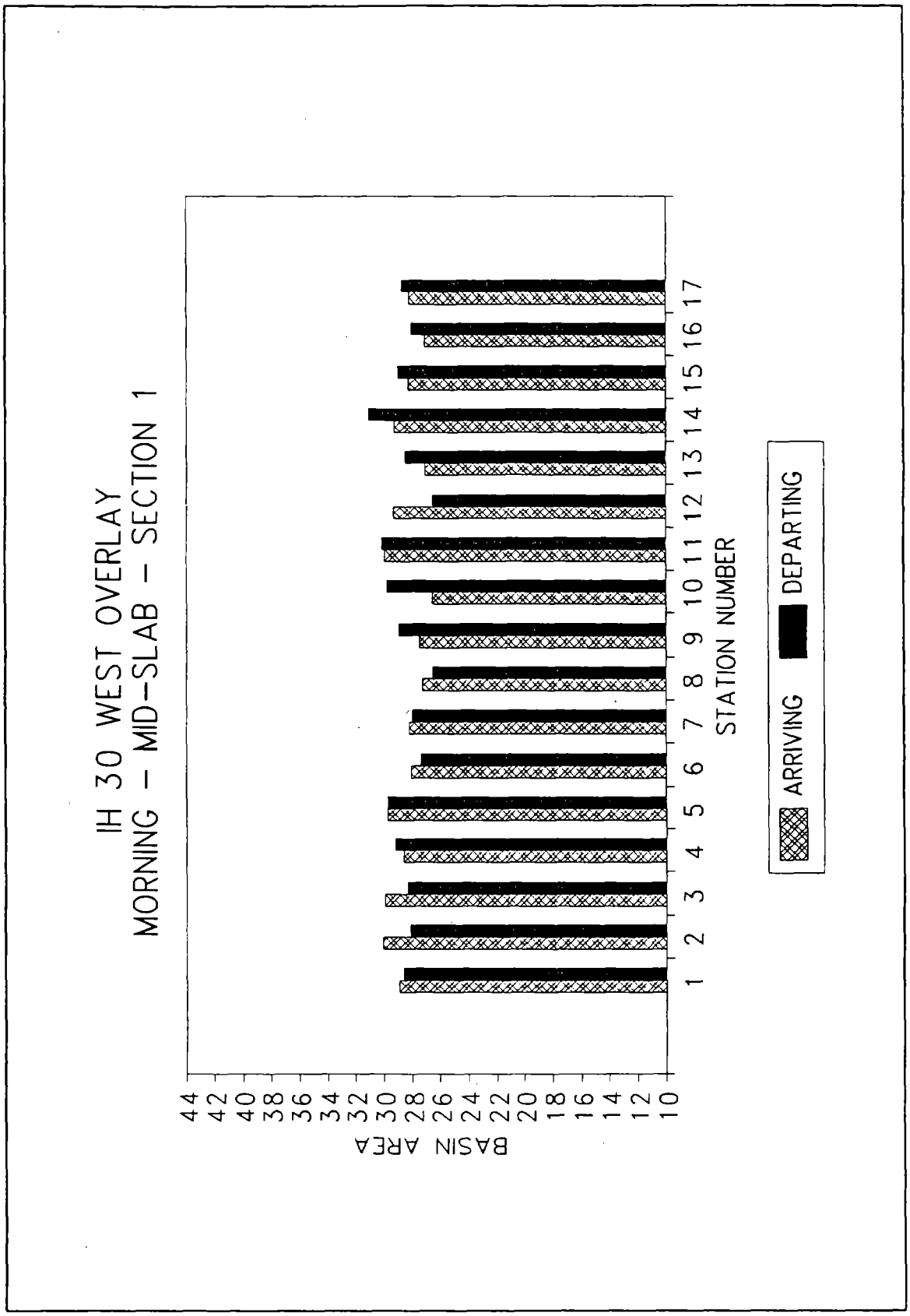

Figure C-3. Basin area versus station (I-30W, overlay, morning, mid-slab, section 2).

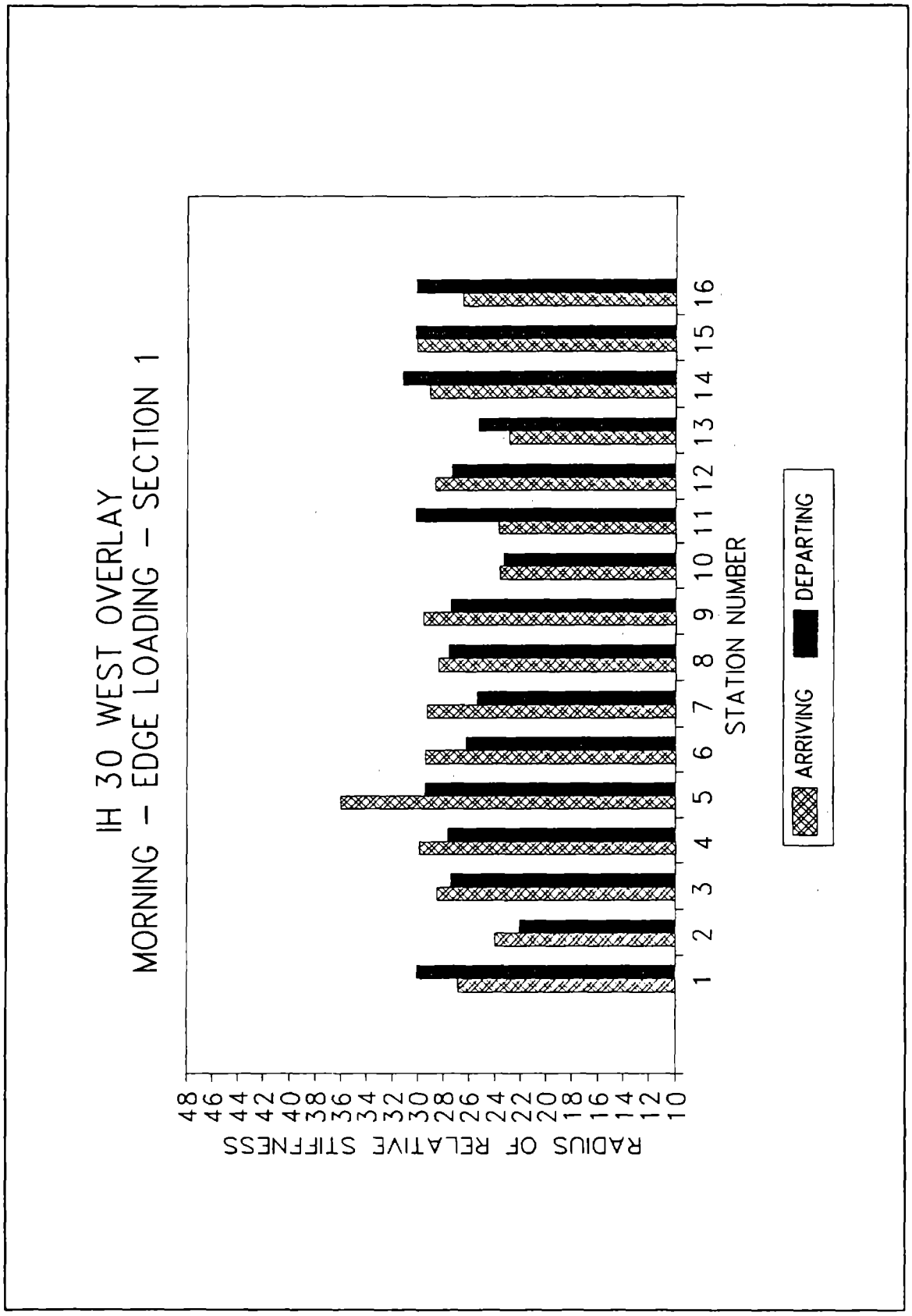

Figure C-4. ℓ versus station (I-30W, overlay, morning, edge, section 1).

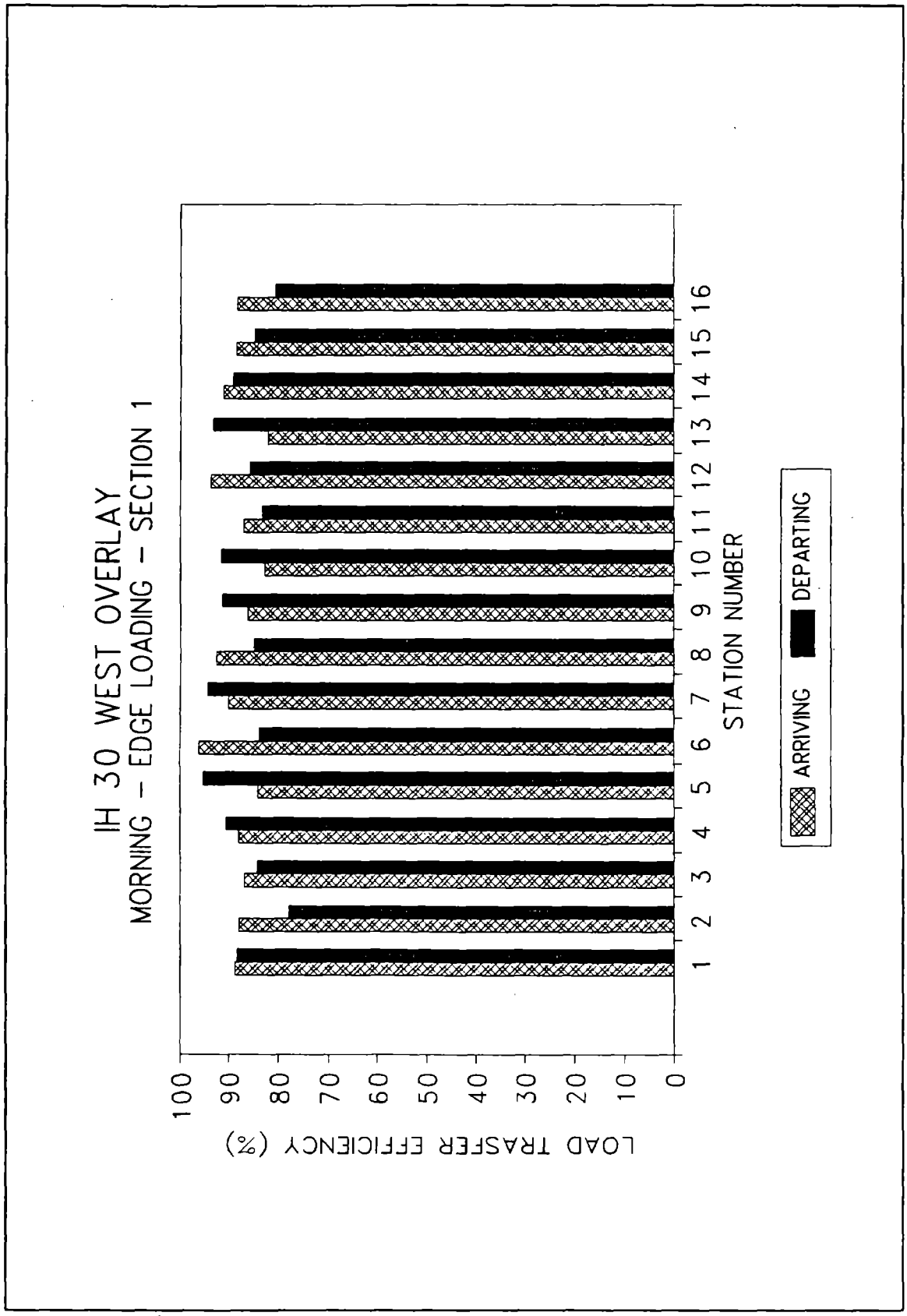

Figure C-5. LTE versus station (I-30W, overlay, morning, edge, section 1).

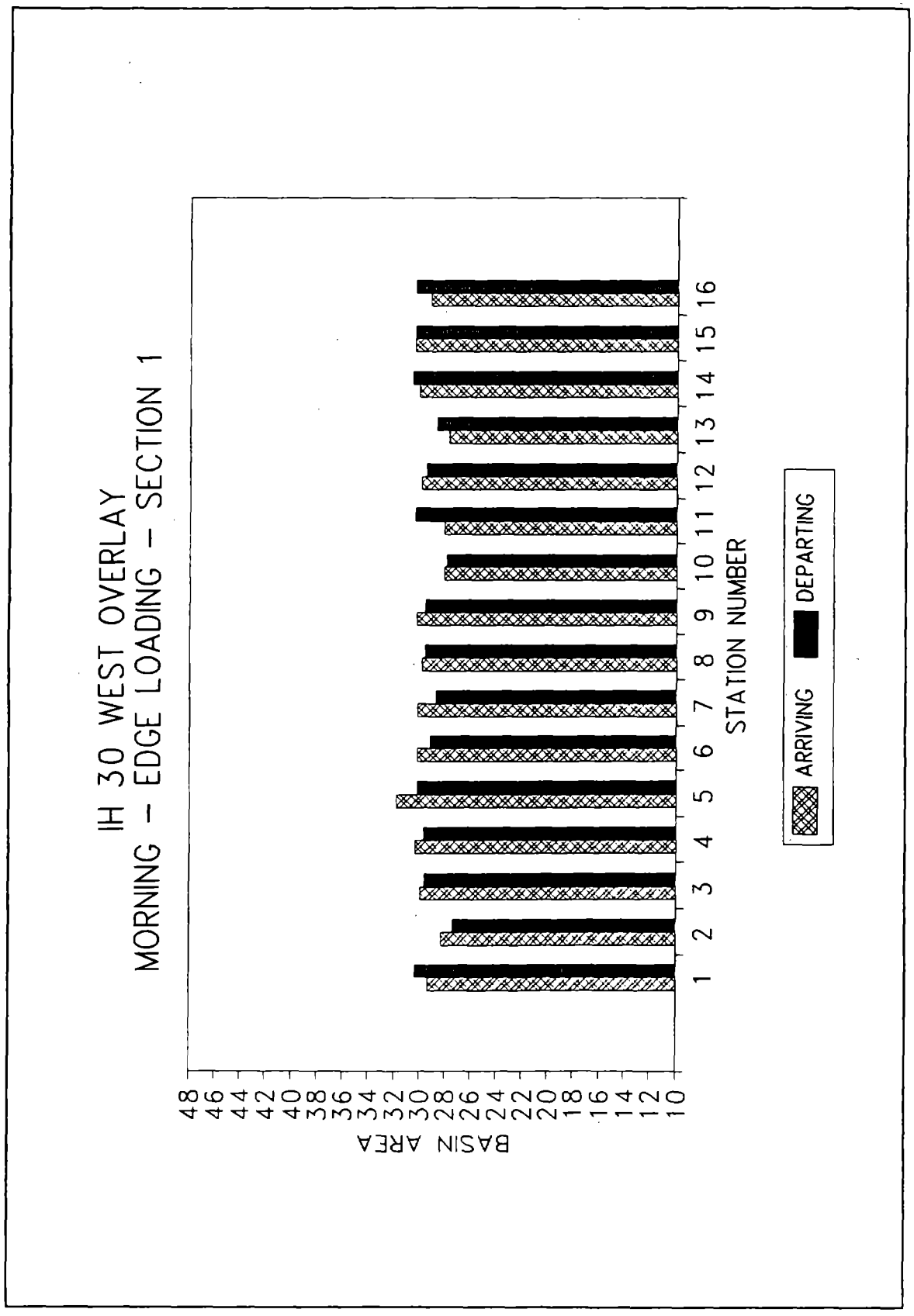

Figure C-6. Basin area versus station (I-30W, overlay, morning, edge, section 1).

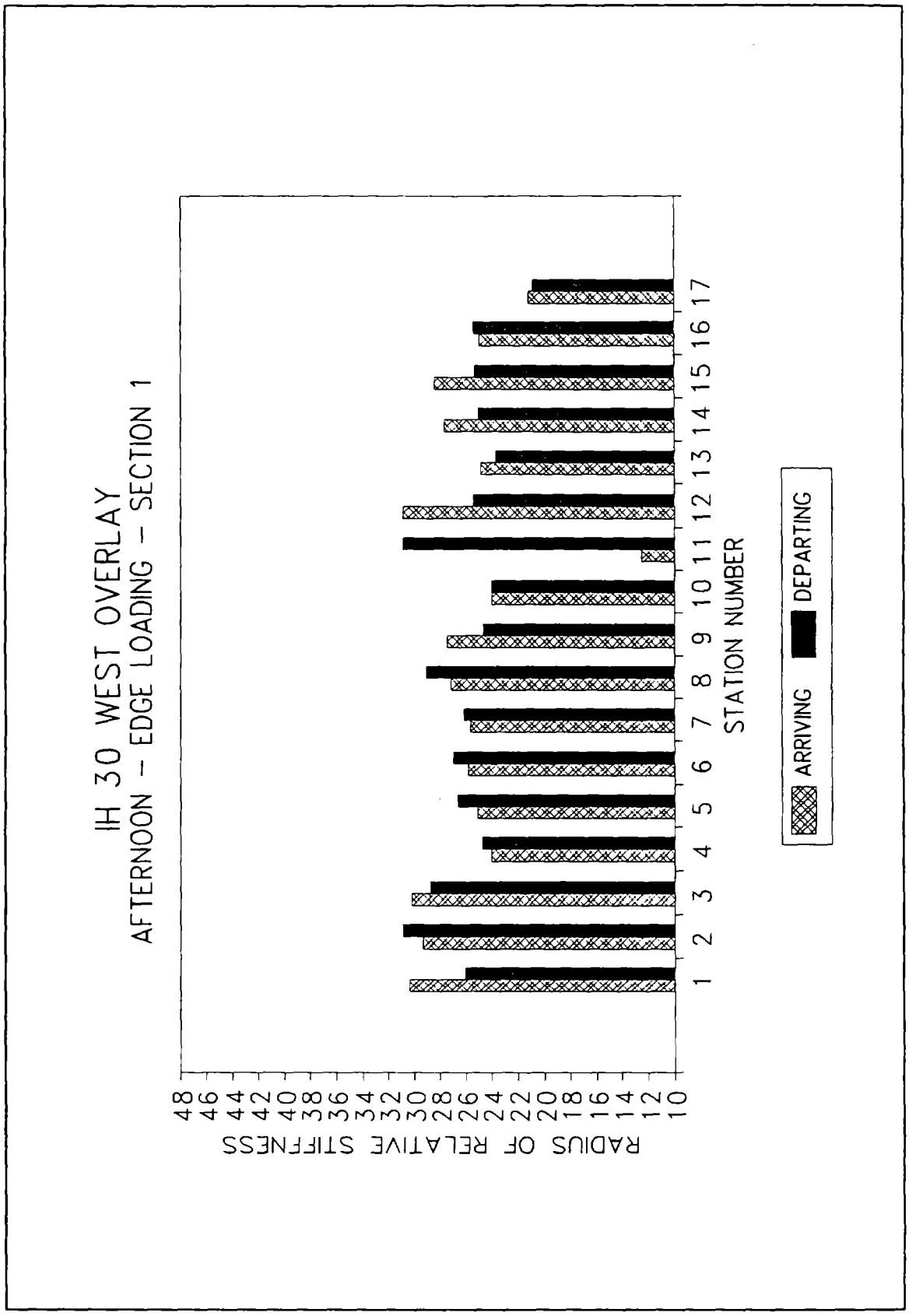

Figure C-7. ℓ versus station (I-30W, overlay, afternoon, edge, section 1).

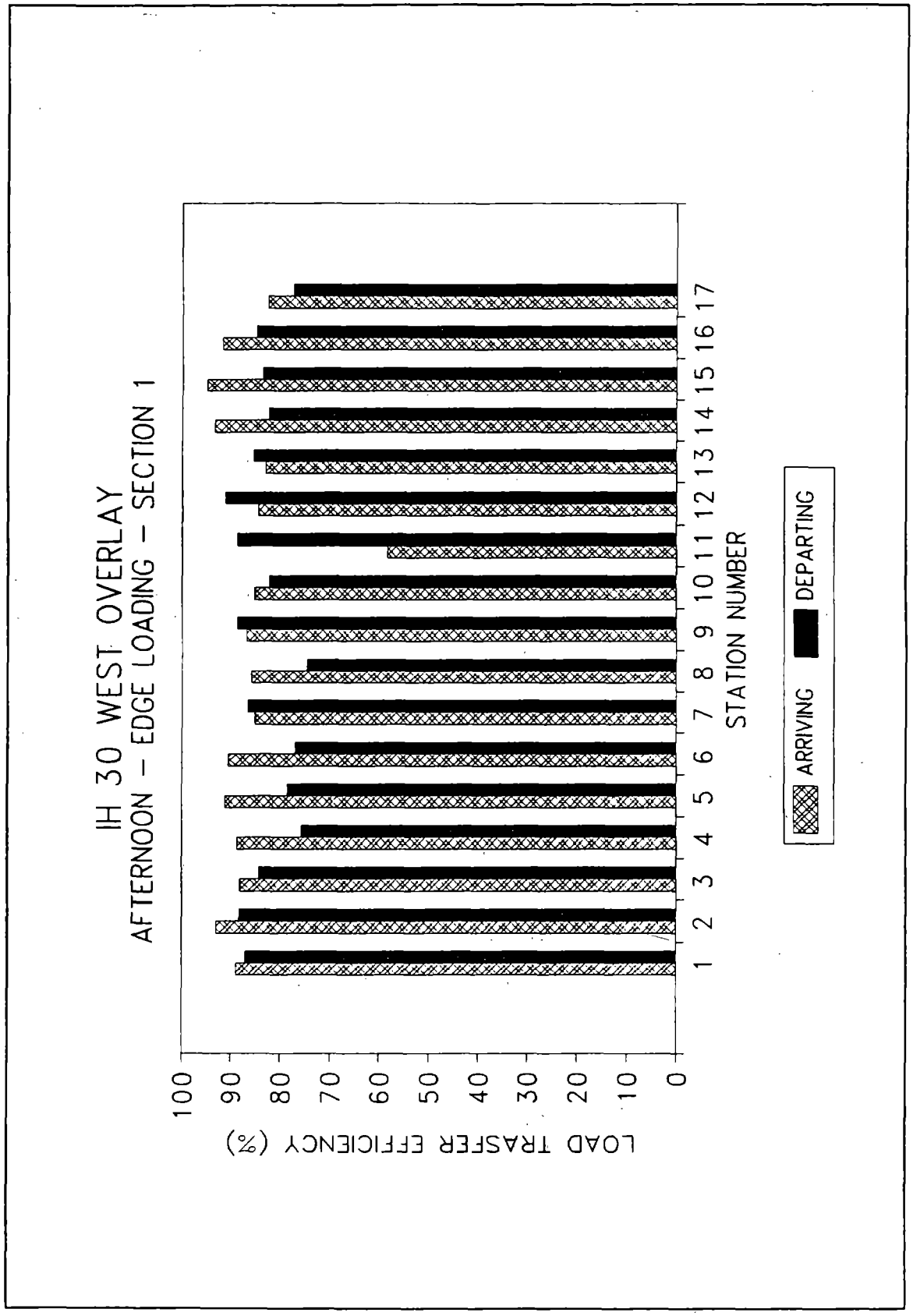

Figure C-8. LTE versus station (I-30W, overlay, afternoon, edge, section 1).

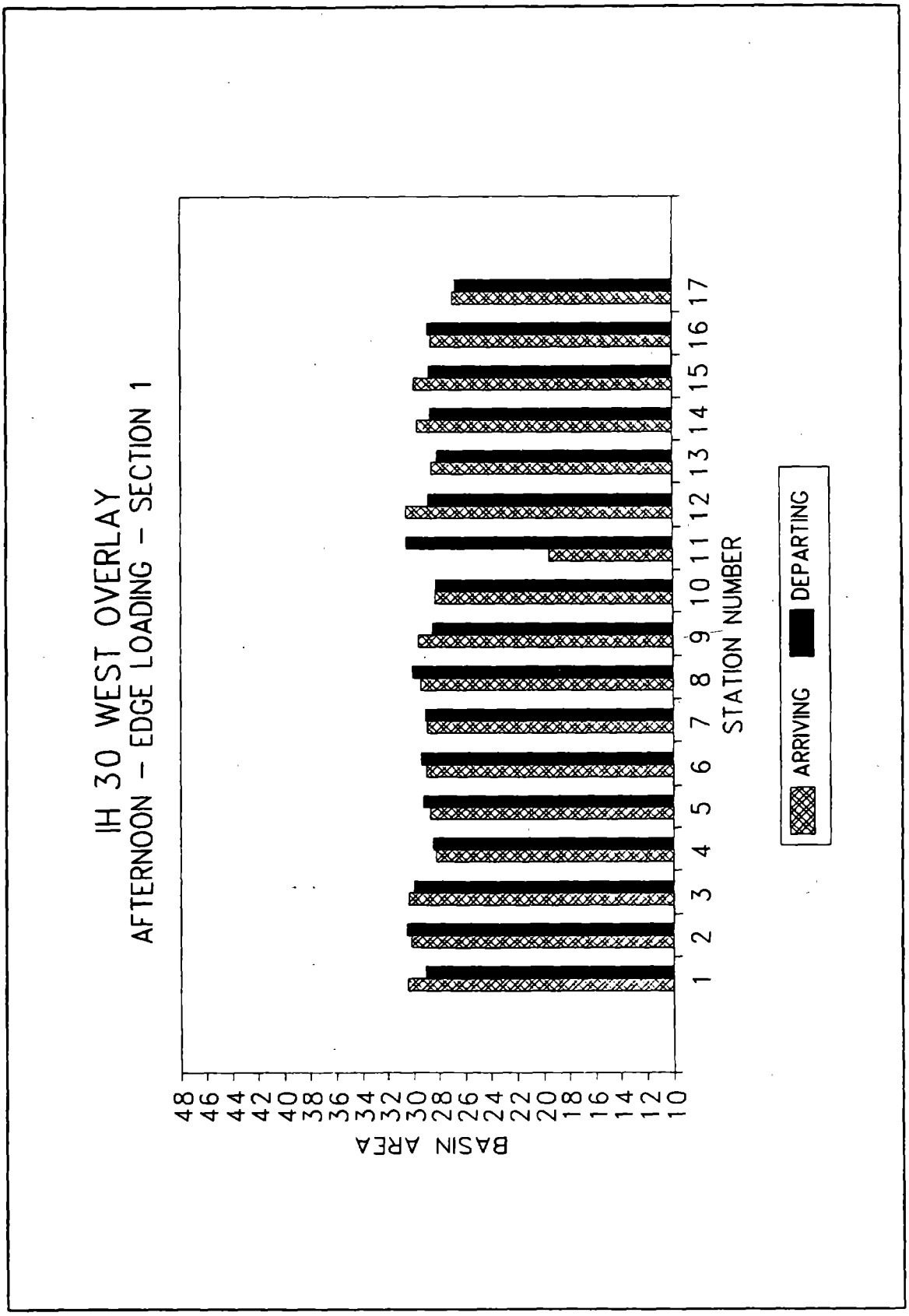

Figure C-9. Basin area versus station (I-30W, overlay, afternoon, edge, section 1).

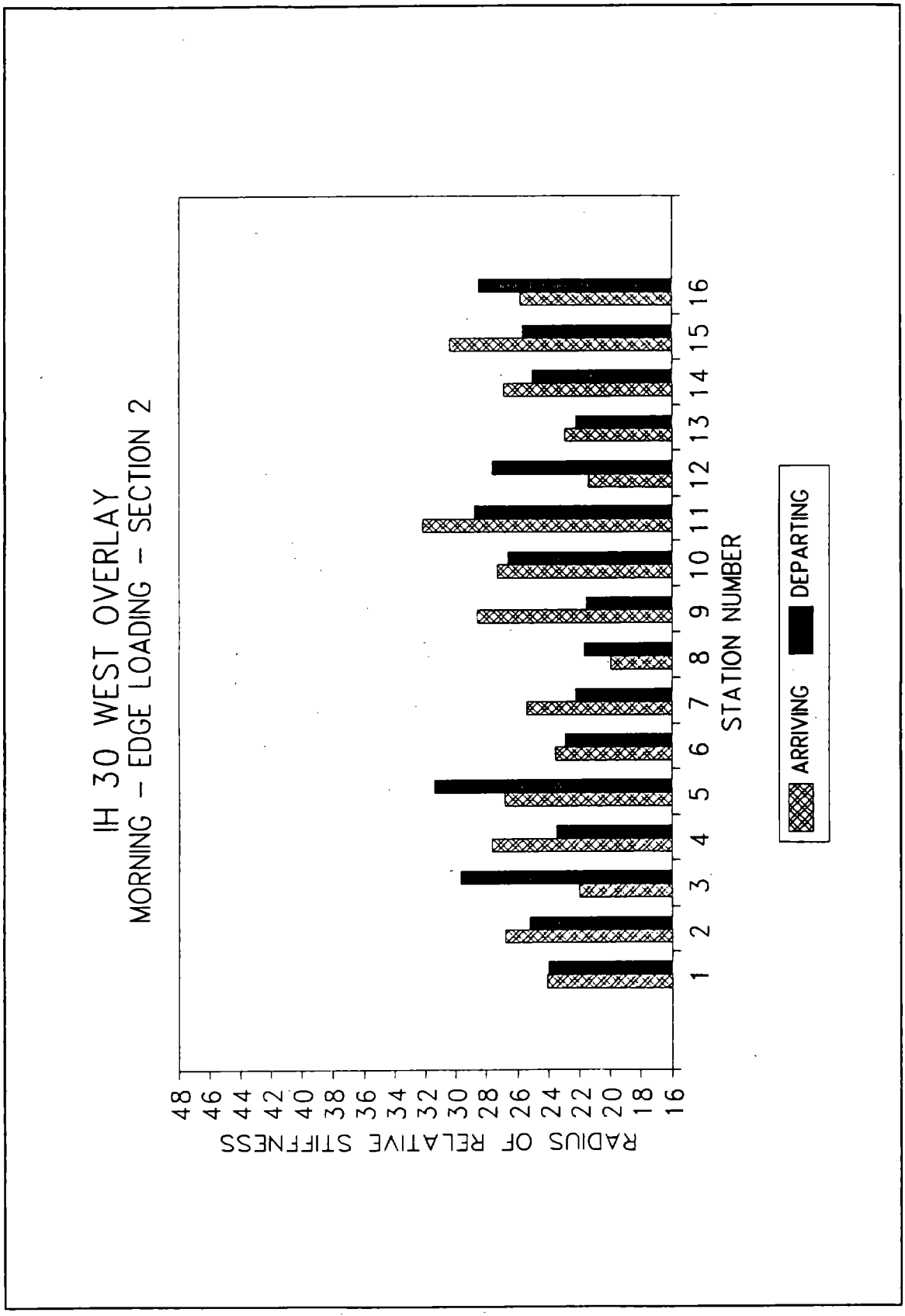

Figure C-10. ℓ versus station (I-30W, overlay, morning, edge, section 2).

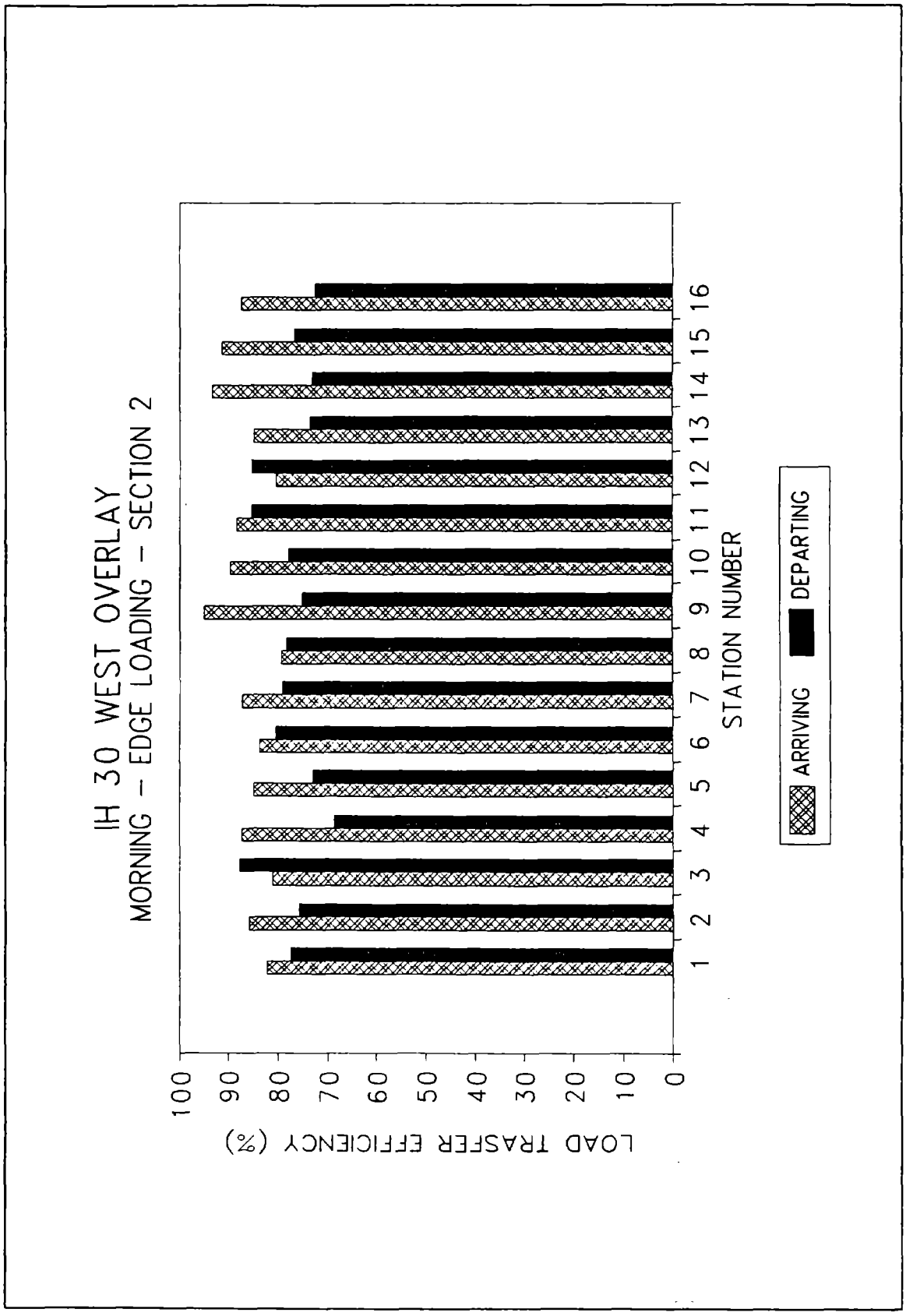

Figure C-11. LTE versus station (I-30W, overlay, morning, edge, section 2).

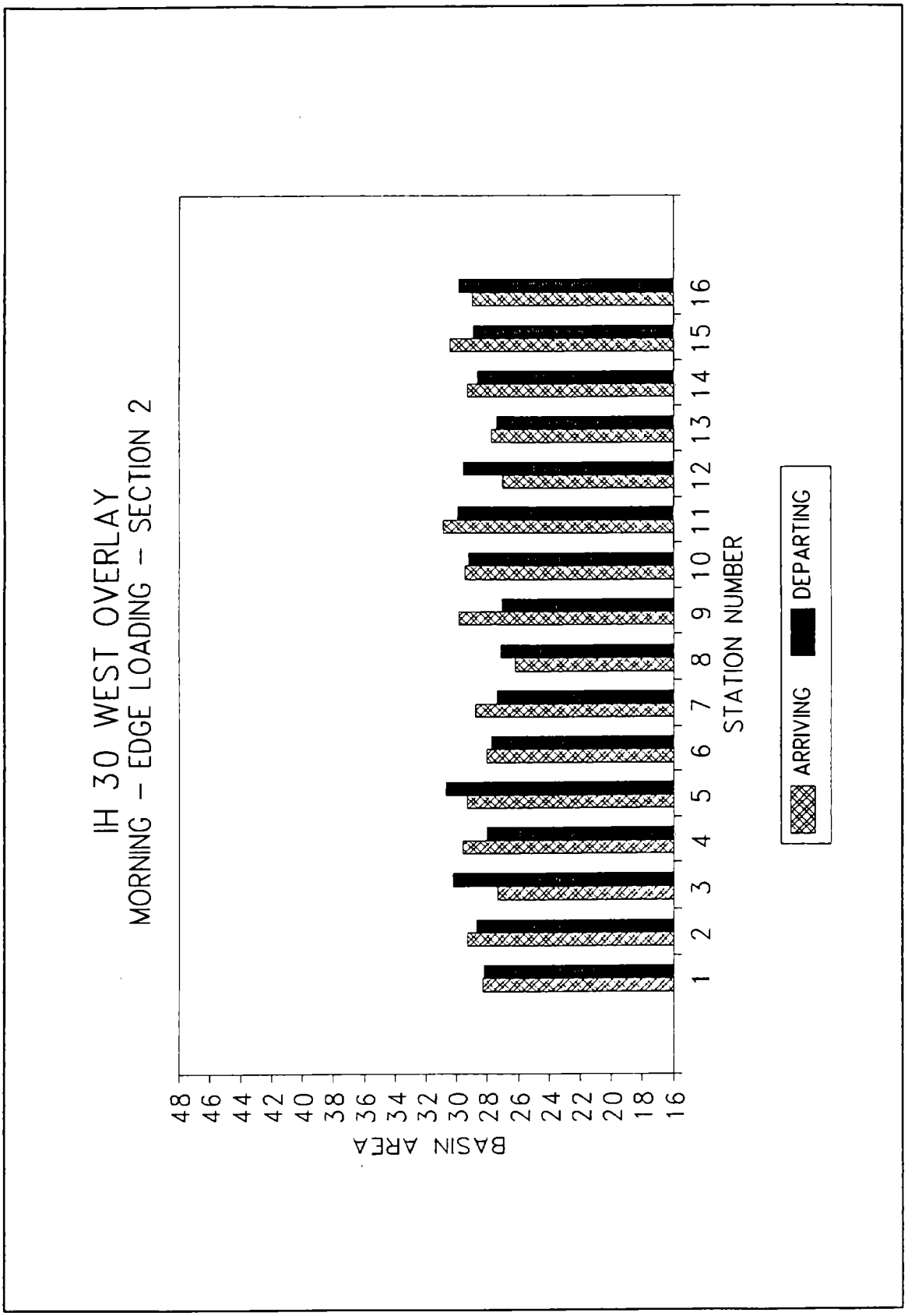

Figure C-12. Basin area versus station (I-30W, overlay, morning, edge, section 2).