Performance of Continuously Reinforced Concrete Pavements Volume V: Maintenance and Repair of 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.

Charles J. Nemmers, P.E.
Director, Office of Engineering
Research and Development

NOTICE

Technical Report Documentation Page

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16. Abstract

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

1. Conduct a literature review and preparation of an annotated bibliography on CRC pavements and CRC overlays.
2. Conduct a field investigation and laboratory testing related to 23 existing inservice 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. Evaluate the effectiveness of various maintenance and rehabilitation strategies for CRC pavements.
4. Prepare a Summary Report on the current state of the practice for CRC pavements.

The following reports have been prepared under this study:

Performance of Continuously Reinforced Concrete 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 Rehabilitation of CRC Pavements
- Volume VI - CRC Pavement Design, Construction, and Performance
- Volume VII - Summary

This report is volume V in the series and presents the results related to CRC pavement distress and procedures for repair of CRC pavements. Because the most troublesome problems with CRC pavements are punchout distresses and distresses associated with steel rupture, more emphasis is placed on repairs of these distresses.

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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 - INTRODUCTION

General

Continuously reinforced concrete (CRC) pavement is portland cement concrete (PCC) pavement with continuous longitudinal steel reinforcement and no intermediate transverse expansion or contraction joints. The continuous joint-free length of CRC pavements can extend to several thousand feet with breaks provided only at structures. Terminal anchorage is provided at the ends of the CRC pavement to restrain length changes due to temperature variations and drying shrinkage of concrete. The CRC pavements develop a random cracking pattern with cracks generally spaced at about 0.9 to 2.4 m (3 to 8 ft). The cracking pattern is governed by the environmental conditions at the time of construction, the amount of steel, concrete strength, and restraint due to friction between the slab and the base. The steel reinforcement restrains the opening of the cracks. Also, the larger amount of steel reinforcement, the more closely spaced the cracks. Most of the cracks form shortly after construction but additional cracking may develop over the next few years as a result of continued drying shrinkage of concrete, temperature variations, and traffic loading.

Although experimental CRC pavements were constructed in the late 1930's, the extensive use of CRC pavements began in the early 1960's during the early years of the U.S. Interstate System construction program. Currently, there are over 45 000 lane km (28,000 lane mi) of CRC pavements in the United States with pavements constructed in at least 35 States. CRC pavements are one of the few pavement types that are truly the ideal "zero-maintenance" pavement if they are designed and constructed properly. Many older CRC pavements are considered to have been under-designed, leading to shorter service lives when subjected to ever increasingly heavy truck traffic. Most have exceeded the original amount of traffic estimated for the 20 year design period before significant distress has developed.

Over the years, many State agencies have conducted research studies to develop better understanding of the effects of various design and construction features on the performance of CRC pavements. A large number of these studies have focused on pavement thickness, concrete aggregate type, amount of steel reinforcement, and base/subbase type. Studies have also been conducted to address the benefits of using epoxy-coated reinforcement and the effectiveness of permeable treated base layers under CRC pavements.

This report is one of a series of reports prepared as part of a recent National pooled funds study administered by the Federal Highway Administration (FHWA) aimed at updating the state of the art of the design, construction, maintenance, and rehabilitation of CRC pavements. The States of Arizona, Arkansas, Connecticut, Delaware, Illinois, Iowa, Louisiana, Oklahoma, Oregon, Pennsylvania, South Dakota, and Texas contributed funding to this effort and their representatives served on the Technical Advisory Board. The scope of work of the 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 at 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.

The following reports have been prepared under this study:

Volume I	-	Summary of Practice and Annotated Bibliography
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Volume IV	-	Resurfacings for CRC Pavements
Volume V	-	Maintenance and Rehabilitation of CRC Pavements
Volume VI	-	CRC Pavement Design, Construction, and Performance
Volume VII	-	Summary

This report is volume V in the series and presents the details on CRC pavement distresses and techniques for maintaining and repairing CRC pavements. Because the most troublesome problem with CRC pavements is punchout distress and distresses associated with steel rupture, more emphasis is placed on repairs of these distresses.

Pavement Distress Types

Distresses for CRC pavements can be grouped into the following categories:

1. Cracking
2. Surface Defects
3. Miscellaneous Distresses

Table 1 summarizes the various types of distress and unit of measurement. Some distresses also have defined severity levels. Distress types are discussed in detail in Chapter 2.

A major concern with CRC pavement is punchout distress. The definition of punchout distress is the area enclosed by two closely spaced (usually less than 0.6 m (2 ft)) transverse cracks, a short longitudinal crack, and the edge of the pavement or a longitudinal joint. The punchout distress is affected by crack spacing, pavement thickness, foundation support, and heavy truck loadings. The repair of punchout distress typically consists of full-depth patches. With time, as the number of full-depth patches increase, the pavement may be resurfaced with asphalt concrete or PCC or it may be reconstructed.

Table 1. Continuously reinforced concrete pavement distress types.

Distress Type	Unit of Measure	Defined Severity Levels?
Cracking		
Durability Cracking ("D" Cracking)	Number, Square Meters (ft^2)	Yes
Longitudinal Cracking	Meters (ft)	Yes
* Transverse Cracking (only medium or high severity)	Number, Meters (ft)	Yes
Surface Defects		
Map Cracking	Number, Square Meters (ft^2)	No
Scaling	Number, Square Meters (ft^2)	No
Polished Aggregate	Square Meters (ft^2)	No
Popouts	Number, Square Meters (ft^2)	No
Miscellaneous Distresses		
Blowups	Number	No
Transverse Construction Joint Deterioration	Number	Yes
Lane-to-Shoulder Dropoff	Millimeters (in)	No
Lane-to-Shoulder Separation	Millimeters (in)	No
Patch/Patch Deterioration	Number, Square Meters (ft^2)	Yes
* Punchouts	Number	Yes
Spalling of Longitudinal Joints	Meters (ft)	Yes
Water Bleeding and Pumping	Number, Meters (ft)	No
Longitudinal Joint Seal Damage	Number, Meters (ft)	No

Note: Severity levels are generally defined as low, medium, or high.
* These are considered the primary distress modes.

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CHAPTER 2 - DISTRESS TYPES

As with other pavement types, CRC pavements may exhibit various distresses after many years of service. If the CRC pavement is underdesigned or experiences construction related quality problems, premature failures may take place and many of the distress types may occur within 3 to 5 years of construction. Properly designed and constructed CRC pavements are capable of providing almost zero-maintenance service life of 20 or more years. The two most damaging distresses in CRC pavements are punchouts and steel rupture (wide transverse cracks). Steel ruptures develop many times into punchout failures.

This section discusses some of the most common distresses that affect CRC pavements. Some of these distresses are unique to CRC pavements, but many of them occur in all rigid pavements at a lesser or greater degree. Failures in CRC pavements are usually manifested as isolated areas of premature distress in different forms according to environment.

Punchouts

Edge punchout is major structural failure of CRC pavements. It is an area enclosed by two closely spaced transverse cracks, a short longitudinal crack, and the edge or a longitudinal joint of the pavement. As the cracks deteriorate under repeated loading, the adjacent pavement portion is broken into smaller pieces that are dislodged from the fractured area by traffic. Punchouts are a very serious hazard to motorists and are a maintenance and rehabilitation problem for the highway agency. Punchouts are considered by many as the most severe problem with CRC pavements. They may also occur as the result of Y cracks. Figures 1 and 2 illustrate CRC pavement edge punchouts, and figures 3 and 4 illustrate some Y punchouts. Close spacing of cracks does not necessarily result in punchouts. However, closely spaced cracks in the presence of weak support condition and heavy truck traffic will most likely result in punchouts.

Edge punchouts generally develop when there is a loss of support under the pavement near the cracks. This loss of support can be caused by either the weakening of the subbase or underlying material by moisture accumulation, localized disintegration of the subbase, or by the ejection of base or subgrade material by pumping along the edge. This loss of support causes higher deflections as the loads cross the pavement. As the pavement deflections become larger, the transverse cracks generally spall and fault. With time, the aggregate interlock deteriorates and the load transfer decreases. With the decrease in aggregate interlock, shear stresses are transferred to the steel reinforcing bars. When the aggregate interlock is lost or reduced considerably the section between deteriorated cracks begins to act as a cantilevered beam. The cantilevered section under repeated loading soon develops a longitudinal crack between the poorly performing transverse cracks. After the longitudinal crack develops, the pavement section bounded by the longitudinal and transverse crack begins to push or punch down into the subbase and subgrade material. The transverse cracks begin to spall more rapidly and become more severe. Under the repeated loading of the traffic and loss of aggregate interlock, the steel reinforcing bars bend and may eventually rupture.

In a study by Zollinger and Barenberg,[5] it was found that many steel reinforcing bars were broken at punchout areas. The breakage was due to the loss of load transfer along the transverse cracks due to poor base/subbase/subgrade support and/or yielding (rupturing) of the longitudinal steel. The breaks at the crack face result from the decreased cross section of the reinforcing bar usually caused by corrosion. The bar breakage away from the crack face is usually load induced and fatigue related. If crack spacing is short, the steel may not fail, which may lead to delamination and deep spalling of the pavement above the steel reinforcement. Figure 5 displays the morphology of the punchout distress.

The reinforcing bars in the area of the punchouts often exhibit a diamond shaped fracture or a void around the bar. The void is apparently the result of the diamond shaped fracture that can form 13 to 25 mm (0.5 to 1.0 in) away from the crack face.[5] Studies of bond stress in reinforcing bars have identified secondary cracks at the loaded face similar to the diamond shaped fractures found in Zollinger and Barenberg's study.[5] These diamond shaped fractures around the reinforcing bars are due to pull out forces. The initial secondary cracking (or cone pullout) forms because of bearing forces transferred to the concrete by the ribs of the reinforcement. Bearing stresses may crush the concrete in the immediate vicinity of the fractured area and lead to a void. The void apparently forms with the development of the punchout and may develop before permanent faulting. Voids around the reinforcing steel may also form when the load carrying capacity of the aggregate interlock at the transverse crack is exceeded (the base yield and allow the crack to open reducing aggregate interlock). The load on the reinforcement may then exceed the bearing capacity of the concrete. When the bearing capacity of the concrete is exceeded, the bars will begin to work loose and form a void under repeated loading. Zollinger and Barenberg[5] reported that they found that 78 percent of the cores taken in edge punchout locations exhibited either a diamond shaped fracture or a void around the reinforcing steel. Cores exhibiting either a void around the reinforcing bars or a pullout fracture are shown in figures 6 and 7.

A way to estimate where a punchout may occur by visual inspection is to look for pavement areas with closely spaced cracking (less than 0.9 m (3 ft)) that exhibit pumping and where the transverse cracks are beginning to spall. This area will eventually become a punchout. Strong, uniform base/subbase/subgrade support is needed to prevent this type of distress. However, stabilized bases (particularly cement treated bases, lean concrete bases, or cement stabilized permeable bases) may bond to the slab, increasing the effective slab thickness and result in an underdesigned slab.

Wide Transverse Cracks/Steel Rupture

Transverse cracking is any crack relatively perpendicular to the pavement center line.[8] Transverse cracking of CRC pavements is an expected occurrence and is not considered a distress. Tight transverse cracks that show little spalling are like transverse joints in jointed concrete pavements. Transverse cracks develop shortly after the slab is placed and begins to harden. Drying shrinkage occurs and is opposed by subgrade friction and the steel reinforcement, causing cracks to form. The reinforcing steel is then supposed to hold the pavement in contact across the cracks to create load transfer by aggregate interlock. If a transverse crack widens and load transfer is lost, the transverse crack becomes a potential distress. Only a medium and high severity transverse cracks are considered a distress.

Most often wide transverse cracks are associated with the rupture of the steel reinforcement or inadequate lap length of the reinforcing steel.[9] In the early years of constructing CRC pavements, the reinforcement was often not lapped over each other correctly and the steel separated or did not provide continuity. This quickly produced wide transverse cracks after construction. This is not a problem with the construction techniques used today. Premature wide transverse cracks result primarily because of use of low levels of steel reinforcement. As noted previously, the AASHTO design crack width of 1 mm (0.04 in) is not adequate. In Europe, the design crack width is 0.5 mm (0.02 in).

Exceptionally wide transverse cracks from 3 mm (⅛ in) to more than 25 mm (1 in) wide will develop when stresses in the steel exceed its tensile strength, resulting in steel rupture.[6] This phenomenon may occur when the steel is simply overstressed due to temperature and moisture changes. According to LaCoursiere et al.,[9] steel rupture mechanisms are believed to be similar to those of an edge punchout. As a transverse crack begins to deteriorate, it is believed that the characteristic of the adjacent pavement and the quality of the subbase support will determine if it will be a punchout or an individual wide crack. If there is pumping, high deflections, and another transverse crack is close, an edge punchout will probably occur. However, if these factors are not present or a large amount of corrosion has occurred, the steel bars at the outside edge may rupture. The rupture of the outside bars will place more stress on the remainder of the bars. The increased stresses will cause them to rupture and the crack will proceed from the outside across the lane. This condition is more prevalent in the northern states of the nation and has not been observed in the southern states.[6]

The wide cracks are believed to be caused by a combination of factors, including large crack spacing, high tensile stresses in the steel caused by cold temperatures and shrinkage, loss of support at the crack, high deflections, corrosion of the reinforcing bars, heavy repeated loads, and pumping of the subbase. The design of CRC pavements with jointed reinforced concrete pavement (JRCP) ramps can cause wide transverse cracks that have little to do with the environment. In Illinois, a wide transverse crack has been associated with the entrance and exit ramps at interchanges and rest areas.[9] The crack only occurred when a ramp was constructed as a jointed reinforced concrete pavement and was tied directly to the CRC pavement mainline. It appears there is sufficient movement at the contraction joint in the JRCP ramp to cause a wide crack in the CRC pavement. Similar problems have developed on I-29 in North Dakota where JPCP ramps were tied to the 178 mm (7 in) mainline CRC pavement. It was suggested that a normal shoulder be carried through the ramp area and the ramp-shoulder joint be placed on a strong base and not tied to minimize this problem on the mainline pavement.

Steel Corrosion

In states where deicing chemicals are used extensively, especially Minnesota and Wisconsin, there have been problems with premature deterioration of CRC pavements in the form of spalling, delaminations, and wide transverse cracks. In its initial form, corrosion is caused by stray current discharging from one area of the reinforcing steel, the anode, and being received by another area, the cathode. Oxidation occurs at the anode. In oxidation, the metal gives up electrons and the newly formed metal ion combines with oxygen and returns to the metal's natural state, iron oxide or rust. Hydrogen evolution or reduction occurs at the cathode.

Four elements are required to support corrosion: 1) an electrolyte, 2) a conductor, 3) an anode, and 4) a cathode. Corrosion occurs on the surface of the reinforcing steel. Initially the reinforcing steel will naturally produce a thin film of iron oxide (rust) that surrounds the steel bar. Concrete is normally highly alkaline and is usually well suited to protect the steel from corrosion since its pH remains high, about 12.5.[12] If the concrete becomes contaminated by an electrolyte such as chloride ions, the natural alkalinity of the concrete is lowered, providing the necessary electrolyte needed for a corrosion cell. The necessary alkalinity is about 350 parts per million (0.83 kg/m^3 (1.4 lb/yd^3) of concrete).[12] Concrete can normally provide the necessary moisture and oxygen needed for corrosion. The protective iron oxide film on the reinforcing steel is destabilized, and a galvanic corrosion cell is established and corrosion of the steel occurs. A corrosion cell is shown in figure 9. Chlorides in the form of deicing salts are considered the most damaging forms of electrolytes normally found in CRC pavements.

Corrosion affects CRC pavements in various ways. The corrosion may cause failure of the reinforcing steel through the reduction of cross-sectional area, thus reducing the tensile strength of the steel bars. With the tensile strength reduced, rupture failures often result. During the winter this crack may open because of the failure of steel, allowing incompressible materials to enter the crack. The following summer, when the pavement expands, compressive forces build up and a blowup or upward thrust of the pavement occurs at the crack. Other distresses that may occur due to corrosion are spalling, wide transverse cracks, punchouts, and delamination of the concrete.

A survey was conducted in Minnesota of all CRC pavements except Interstate I-94, and the results showed that all the sections tested had active corrosion and high chloride contents in the concrete. However, few of these sections had delaminations or spalling on more than 1 percent of its surface area.[12] A Wisconsin study concluded the following:[13]

1. There was no correlation between crack width and steel corrosion.
2. Once the corrosion rating reaches a moderate-heavy level, the pavement distress rate appears to increase sharply.
3. There was a correlation between the use of deicing salts and corrosion, but there was no correlation between the amount or frequency of use of deicing salts and corrosion.
4. Inadequate concrete cover of reinforcement increased corrosion.
5. The amount of copper in the manufacturing of the steel reinforcement affected the rate of corrosion.

Corrosion should be considered a major problem in states where extensive deicing salt usage is prevalent. Current AASHTO allowable crack width of 1 mm (0.04 in) is considered inadequate as this may contribute to early corrosion. An allowable crack width of 0.5 mm (0.02 in) is typically used in Europe.

Spalling

Spalling is the cracking, breaking, chipping, or fraying of slab edges within 51 mm (2 in) of a crack or joint. The spall does not extend vertically through the slab but intersects the crack at an angle. Figure 8 illustrates the spalling of a transverse crack. The spalling of CRC pavements cracks is primarily caused by high deflections resulting from heavy loads.[5] High stress concentrations, greater than the concrete's strength, can occur at the top of the pavement under heavy repeated loads. The result is a crumbling of the pavement's surface at the point of high stress concentrations. Spalling is not a significant problem with well designed CRC pavement and high quality concrete mix.

Slight spalling is a condition where the flaking of the concrete is confined to the mortar in the concrete matrix and may also be called raveling. Slight spalling is not a progressive form of deterioration, nor is it of structural consequences. The first spalling occurs at cracks, probably resulting from discontinuities that develop during the crack propagation process as the cracks seeks the path of least resistance. Spalling which deepens more rapidly than widening is primarily related to a structural weakness. Spalling which widens more rapidly than deepening is usually related to a weakness of the concrete surface.

In CRC pavements, spalling is sometimes related to the corrosion of reinforcing steel (usually with less than 75 mm concrete cover). As the corrosion product increases in size, it creates pressure that often becomes greater than the concrete strength, thus causing a crumbling of the concrete surface. This happens more frequently in the northern states where chemical deicers are used and where there is inadequate concrete cover over the steel on inadequate concrete density. Spalling is unsightly and, if it does become severe and causes roughness, spalling may require maintenance and repairs. Spalling often may be a precursor to more severe problems such as an edge punchouts.

Medium and High Severity Transverse Cracking

Because transverse cracking is expected to occur in CRC pavements, transverse cracking itself is not considered as a distress. However, in time, the transverse cracks may exhibit spalling, as discussed in the previous section. As such, a transverse crack may be designated as exhibiting low, medium, or high severity cracking. The Long Term Pavement Performance (LTPP) program defines severity of transverse cracking as follows:

1. **Low** - Crack widths $<$ 3 mm (0.125 in), no spalling, and no measurable faulting; or well-sealed and the width cannot be determined.
2. **Moderate** - Crack widths $\geq$ 3 mm (0.125 in) and $<$ 6 mm (0.25 in); or with spalling $<$ 75 mm (3 in); or faulting up to 6 mm (0.25 in).
3. **High** - Crack widths $\leq$ 6 mm (0.25 in); or with spalling $\geq$ 75 mm (3 in); or faulting $\geq$ 6 mm (0.25 in).

D-Cracking

D-cracking is associated with a characteristic pattern that appears at the surface as a series of closely spaced crescent-shaped hairline cracks adjacent and generally parallel to the transverse and longitudinal joints or cracks and free edges of the pavement.[8] The first indication of D-cracking is a discoloration of the pavement surface near the joints or cracks. This discoloration is caused by the release of lime (CaOH) from the minute cracks in the concrete. Lime, when it contacts the air, becomes carbonate and may appear as a grayish film on the pavement surface.[15] Evidence of D-cracking on the wearing surface first appears at the intersections of the transverse cracks with the longitudinal joint or crack and the free edges of the pavement. The cracking then progresses along the joints and cracks until it forms a nearly continuous network confined to the peripheral areas of the slab.

D-cracking usually originates in the lower portion of the pavement slab and progresses upward.[14] Because it usually starts at the bottom and progresses upwards, deterioration in the bottom of the slab can be quite extensive before signs of D-cracking are evident on the pavement surface. It is caused by the disintegration of critically saturated coarse aggregate particles from freezing and thawing. It is initiated when moisture penetrates open joints and cracks, and with moisture already present beneath the pavement, raises the degree of saturation of the coarse aggregate to a critical level. During freezing, pressure generated in the aggregate may exceed the internal strength of the aggregate and surrounding mortar, causing them to crack. With continued freezing and thawing, existing cracks may provide additional channels for the migration of moisture into the slab. These cracks may become additional sites for ice formation and the generation of excessive pressures operating within to widen the existing cracks. If allowed to continue, the entire pavement slab may completely turn to rubble in 8 to 15 years.[16] CRC pavements may become susceptible to D-cracking because of its close crack spacing. Extensive amounts of patching have been required on several CRC pavements projects in Illinois to repair D-cracking.[16]

The factors that have been linked to D-cracking include environmental conditions, coarse aggregate, fine aggregate, cement, pavement design, subsurface drainage, and traffic. The environmental factors that have the most effect are freezing and moisture. Continuous moisture availability is very important to D-cracking. The coarse aggregate in the portland cement concrete (PCC) is the primary factor influencing D-cracking. Aggregates that are susceptible have many characteristics in common.[14] The first is the composition. Most of the aggregates are sedimentary in origin, including both carbonate and silicate materials. The second is pore structure, which is the most important characteristic influencing the susceptibility. Aggregates with low permeability, high porosity, and small pore size are most vulnerable to D-cracking problems. The third is the absorption of the aggregate. The other factors, fine aggregate, cement, pavement design, subsurface drainage, and traffic, do not significantly influence the development of D-cracking but do affect the rate of deterioration of the concrete once the D-cracking has developed.

SHRP Study on D-Cracking

Considerable work on D-cracking has been performed recently as part of the Strategic Highway Research Program (SHRP) and other follow-up studies sponsored by FHWA. The SHRP studies and the post-SHRP studies have resulted in numerous guidelines on prevention and mitigation of D-cracking in concrete pavements. The reader is advised to refer to the most recent publications on this topic for further guidance.

Pumping

The mechanics of pumping under CRC pavements are identical to the factors that cause pumping under other rigid pavements. Pumping has been a problem in the United States since the 1940's, with the increase of wheel loads during World War II.[17] Pumping is the ejection of water and subgrade, subbase, and base material through the pavement's joints, cracks, and edges or the distribution of material underneath the slab.[17] The pumping or ejection of the water and material is caused by three factors: erodible subbase or subgrade, the presence of free water, and pavement deflections caused by heavy wheel loads.[17,18,19] Other factors have also been identified as affecting pumping:[10]

1. Permeability of the subgrade, subbase, and shoulder materials;
2. Subsurface drainage;
3. Crack spacing; and
4. Sealing of the lane-shoulder joint.

Because slabs naturally tend to lift up at the edges from warping and curling, there is usually some gap beneath the edges of the slab for water to accumulate either temporarily or permanently. When heavy loads deflect the pavement, the water is squeezed out from under the pavement under extremely high pressures. If the water is trapped at the interface between the pavement and subbase because of poor drainage conditions, the water may be forced from the interface through the longitudinal joint under the weight of an axle load. Under repeated axle loading, this process will tend to erode away the subbase material when enough moisture is trapped at the interface. The material and water are forced laterally and vertically along the pavement edge. The material is then either deposited under or on top of the shoulder. Pumping can be easily recognized by the presence of subbase material along the adjacent shoulder. The result is a void condition beneath the pavement. The voids change the slab support system from a uniformly supported condition to an unsupported condition at some points. Figure 11 demonstrates the results of pumping at a subsequent punchout failure. The use of widened truck lanes will substantially reduce edge pumping with or without the use of tied concrete shoulders. Also strong base/subbase/subgrade support is critical to prevent excessive deflection of CRC pavements which is relatively flexible.

Voids

Voids are areas under the pavement that do not support the pavement. The basic assumption in the design of CRC pavements is that there is full support throughout the length of the pavement. When a void develops beneath the pavement, this support condition is lost and the loads are not distributed as intended in design. The tensile stress in a pavement with a void increases significantly. Thus, the overall fatigue life of the pavement is significantly reduced when voids exist. The existence of voids causes punchouts, spalling, and other failures. A gap of only 1.27 mm (0.050 in) or greater between the slab and subbase will cause high deflections in the concrete pavement.[20] The use of 50 mm AC separator layer over an LCB has been very effective in Europe to provide a weak bond and uniform support with a high quality (6 percent AC) mix.

Voids develop in many ways, such as:[20]

1. Pumping of subbase material.
2. Deep soil movement such as swelling frost heaving and settlement action.
3. Mudjacking which causes the pavement to raise excessively, producing a high point with voids on each side.
4. Unconsolidated honey combed concrete.
5. Differential densification of the subbase.
6. Curling/warping of the slabs at the edge.

Birkhoff and McCullough[20] studied three different void detection systems: deflection-based methods, vibration, and visual inspection. Their findings were that the deflection-based method had a high probability of detecting the voids. The other two methods were found inefficient. Bukowski et al[21] reported that voids could be found by using infrared thermography. They were able to find the location of the voids 72 percent of the time. Another method that may also be used for void detection is ground penetrating radar (GPR). GPR has performed well in the field and during tests in locating the size and depth of the voids under both plain and reinforced concrete slabs.[22,23] However, it is still not reliable for detection of voids less than 3 mm (⅛ in) thick.[18]

Longitudinal Cracks

Longitudinal cracks are cracks that run parallel to the pavement center line. Longitudinal cracks, unlike transverse cracks, are not expected to occur in CRC pavements. There are generally two types of longitudinal cracks. The first type is the wandering uncontrolled cracking adjacent to the center line joint or lane joint. The crack may wander by as much as 0.9 m (3 ft) on either side of the center line. This type of crack is usually caused by late sawing, improper placement of the joint forming separator, or omission of the joint forming separator strip to produce (induce) the necessary longitudinal stress relief joint. This crack may be unsightly but it rarely advances to a problem stage unless other problems are present.

The second type of longitudinal crack, which is more serious, is one that is associated with the settlement of the subgrade or the deep foundation. The localized areas, where there is foundation settlement, produce an area of concentrated stresses. These concentrated stresses become greater than the flexural strength of the concrete, producing a crack. The crack begins to widen under repeated loading and allows water to enter, accelerating crack deterioration. In some cases (Michigan) this has been related to expansive slag aggregates used for the base material.

Blowups

Blowups are described as a localized upward movement of the pavement surface at transverse joints or cracks, often accompanied by the shattering of the concrete in that area.[8] For many years it was believed that blowups could not occur in CRC pavements because it has no transverse joints. However, blowups have been recorded in Mississippi, Illinois, and Maryland.[9] A pavement blowup often creates a dangerous situation for traffic because shattered concrete may be lying on the pavement requiring immediate attention and repair.

Blowups are caused by a buildup of expansive forces and most often occur at wide transverse cracks and expansion joints. These areas allow for incompressible materials to enter the crack or joint. When the slab begins to expand and there is no expansion for space, available longitudinal forces build up. As the temperature increases and the slab moisture is high, the slab expansion tends to be high. Often the concrete slab will rise above the subbase and the concrete will shatter 0.6 to 0.9 m (2 to 3 ft) away from the crack. Blowups have also occurred when there is a half-lane patch constructed with asphaltic concrete and during maintenance when one lane has been closed for patching.

The causes of the blowups are high ambient temperatures (and, consequently, high slab temperatures), high humidity, and a discontinuity caused by a construction joint or a wide transverse crack (steel rupture). Other factors that may influence CRC pavement blowups are, cluster cracking (closely spaced cracks) inadequate slab thickness, and restraint caused by tied ramps.

Lug Anchor Waves

The lug anchor system consists of heavily reinforced concrete lugs rigidly connected to CRC pavements. Most installations have three to four lugs at 6.1-m (20-ft) centers. Figure 12 represents a typical lug anchor design.

Lug anchor waves or lug rotation is the transverse undulation, or waves, in the pavement surface in lug anchor terminal systems. Lug anchors are a terminal treatment for the free ends of CRC pavement. If not restrained, CRC pavements will move outward by as much as 51 mm (2 in), and annual movements of 25 to 51 mm (1 to 2 in) are not uncommon. This free end or terminal end must either be restrained by an anchor system or accommodated by an expansion system.

Lug anchor waves produce undulations in the pavement that often causes roughness in the road. In severe cases these waves may create roughness so severe that repairs must be made, as shown in figure 13.

Wisconsin has reported that, of 372 lug anchor systems it had built, 38 percent of the systems 10 years or older either rode rough or have required rehabilitative measures.[13] Illinois has reported that its lug anchor problems were usually occurring after 5 years and with the design of four lugs at 12.2-m (40-ft) centers.[8] Their new design of three lugs at 6.1-m (20-ft) centers has not shown any rotation. Both reports recommended that the lug terminal systems be discontinued and be replaced with a wide flange beam joint systems. Wisconsin has also reported that it had installed 121 wide flange beam systems and that they were all performing satisfactorily. Even ones that were 12 years and older still had clearance for the expansion of the slab. Details of a wide flange terminal system can be found in references 2, 3, and 13. Maryland replaced wide flange beams with tied continuous steel at construction joints in 1995 and have encountered problems. A heavy duty doweled expansion joint on the sleeper slab is needed to provide for movement allowed with the wide flange terminal system.

Alkali-Silica Reactivity

Alkali-silica reactivity (ASR) is an expansive reaction between reactive forms of silica in aggregates and potassium or sodium alkalies. The potassium and sodium alkalies come mostly from cement but may also come from the aggregates, pozzolans, admixtures, and mixing water.

The ASR phenomenon has been recognized since the 1940's and there have been extensive studies in the affected central states of Colorado, Nebraska, and Kansas to minimize/eliminate the risks of ASR. Some of these states have reportedly not constructed ASR-affected pavements since the early 1960's. The recognition of the ASR problem was slower along the eastern coast states and the problem has only recently been recognized. Extensive ASR related problems have developed in Delaware, Pennsylvania, and Virginia.

In its simplest form, ASR can be seen as a two-step process:

1. Alkali + Silica = Gel Reaction Product
2. Gel Reaction Product + Moisture = Expansion

Actual expansion only occurs in the second step when the ASR gel reaction product swells as it absorbs the moisture. There are five factors that control the amount of ASR expansion:[3]

1. Nature of reactivity,
2. Amount of reactive silica,
3. Particle size of reactive material,
4. Amount of available alkali, and
5. Amount of available moisture.

Moisture availability is a major variable in the concrete and has a significant impact on the severity of distress and volume change due to ASR.

The expansion of the ASR gel induces pressures within the concrete and causes cracking of the aggregates and surrounding paste. The outward evidence of the expansion often results in "map-cracking" or "pattern-cracking." Cracking caused by ASR may be visible within a short period of time, 1 year, or it may be 15 to 20 years before signs of ASR are visible.[3] Restraint due to both abutting concrete and reinforcing steel influence the development of the extent of cracking. Cracking is not uniformly developed through the pavement structure, due in part to the restraint caused by the reinforcing steel.

Considerable work on ASR has been performed recently as part of SHRP and other follow-up studies. The SHRP and the post-SHRP studies have resulted in numerous guidelines on prevention and mitigation of ASR in concrete pavements. The reader is advised to refer to the most recent publications on this topic for further guidance.

The only indisputable evidence that ASR has developed in concrete is the presence of the ASR gel reaction product. In the early stages of ASR, or where there are only small quantities of the reactive gel, the ASR gel is virtually undetectable to the unaided eye. The gel is only noticeable under a microscope. Thus, ASR may go unrecognized in field structures for many years, before associated severe distresses manifest. The use of uranyl (uranium) acetate fluorescence method has been developed recently to identify ASR gel in the field. The ASR process can be arrested or retarded if moisture is kept from entering the ASR-affected concrete.

The ASR-affected concrete pavements are generally overlaid with hot-mix asphalt concrete to temporarily extend the service life of the pavements. The ASR-affected concrete pavements can also be overlaid with unbonded concrete overlays to ensure a longer term service life for the rehabilitation.

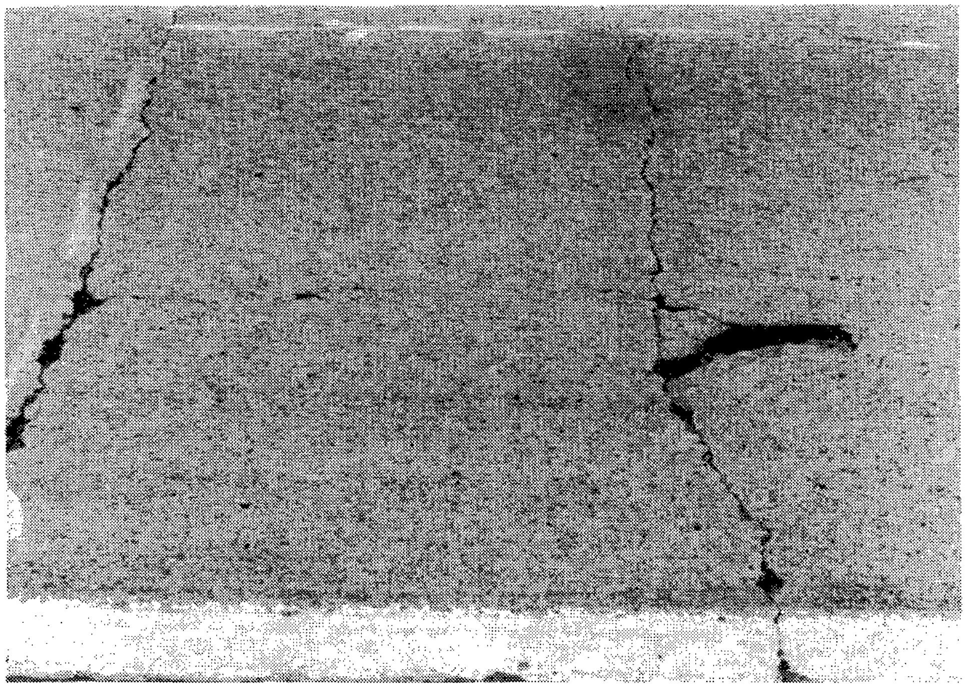


Figure 1. Low severity punchout distress.

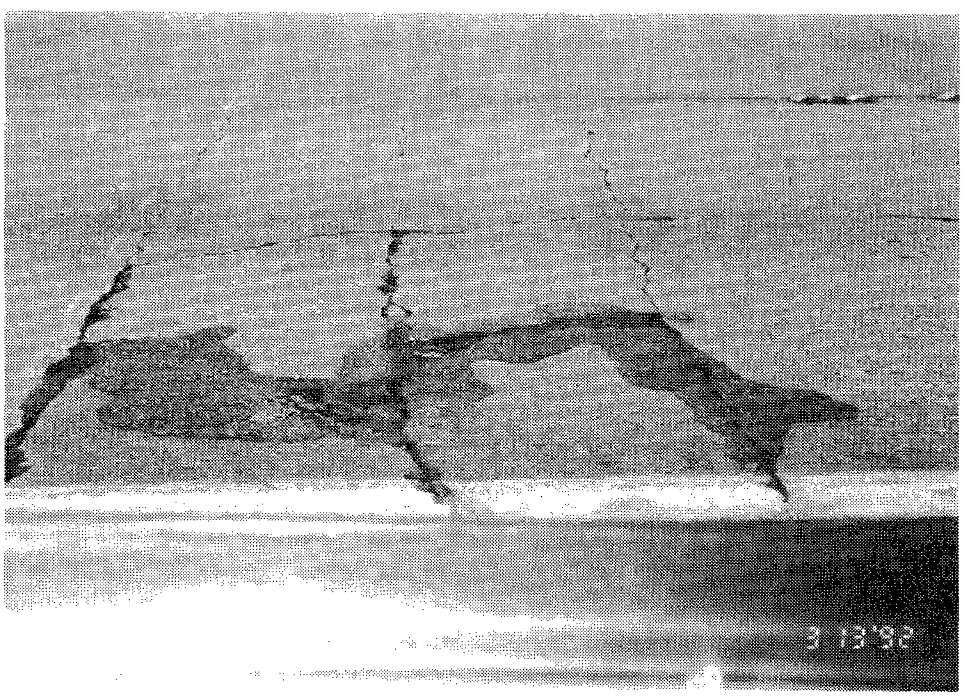


Figure 2. Moderate severity punchout distress.

Figure 3. Y crack.

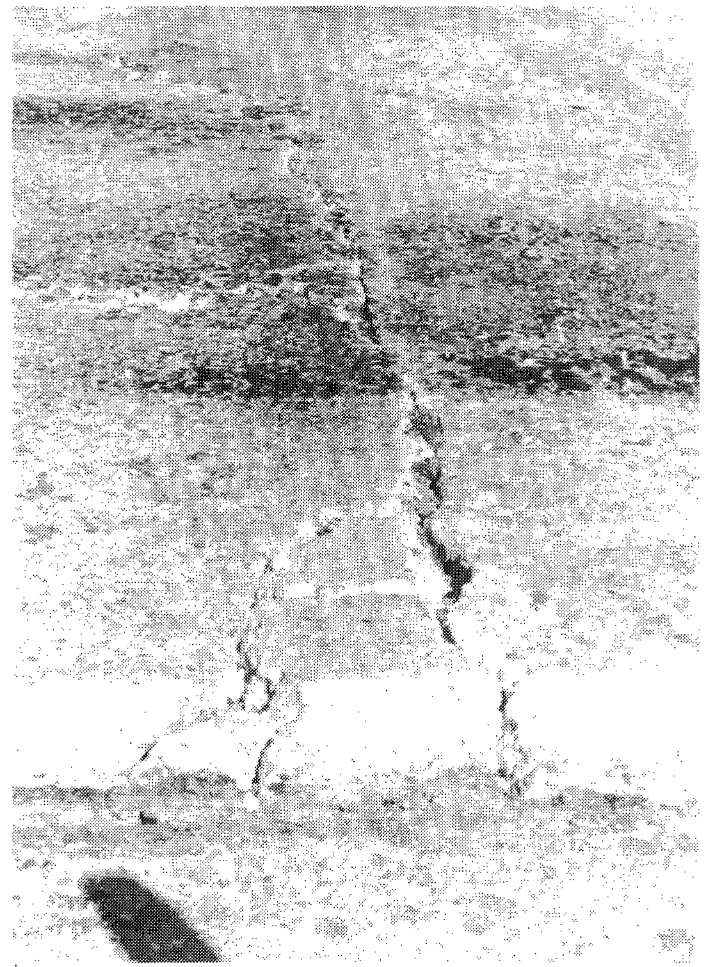


Figure 4. Y crack with punchout.

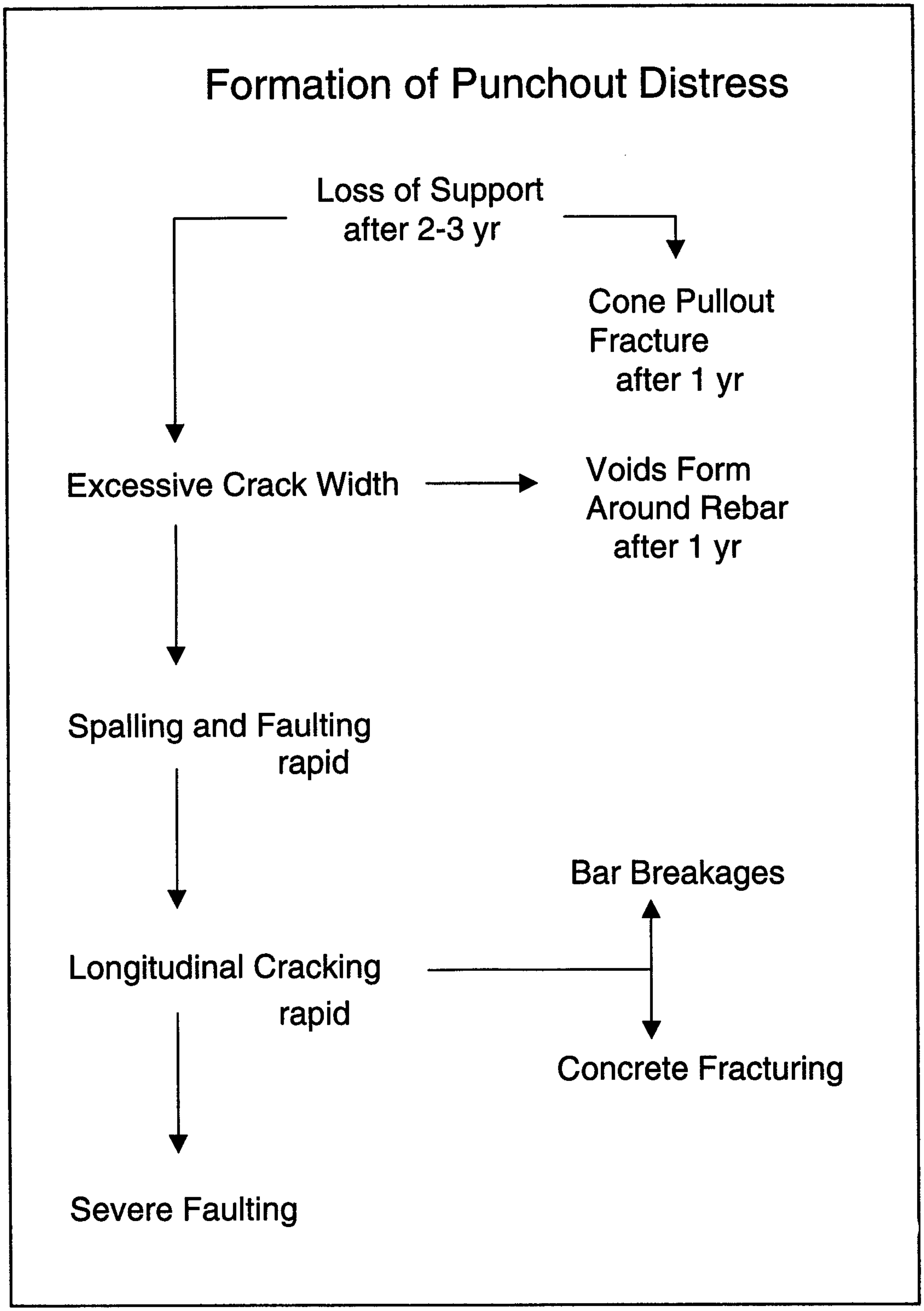


Figure 5. Morphology of a punchout.[5]

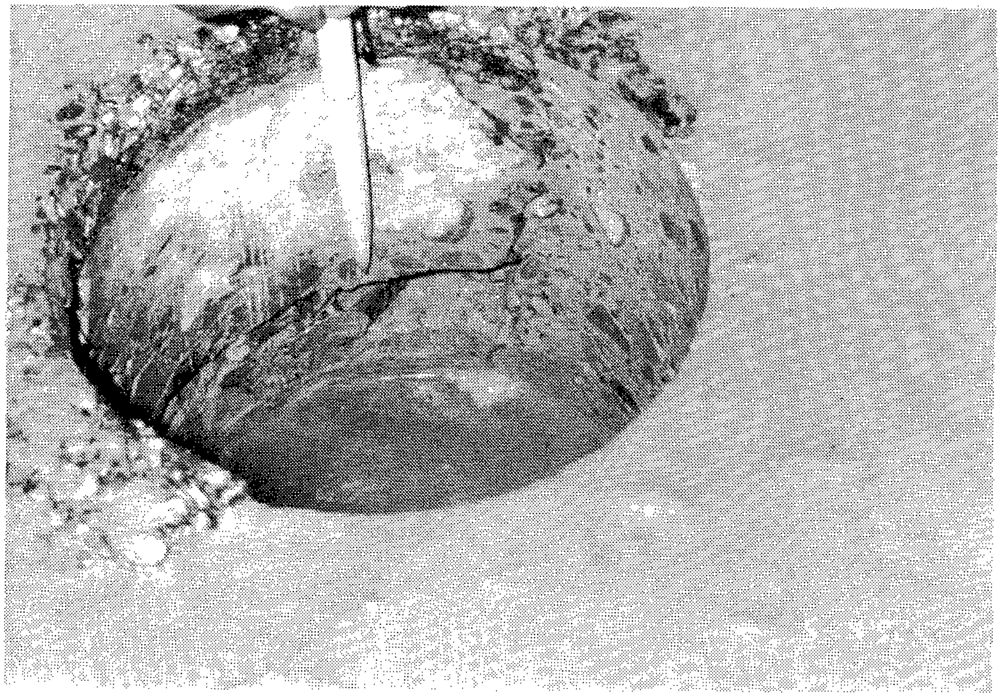


Figure 6. Void underneath reinforcing steel.

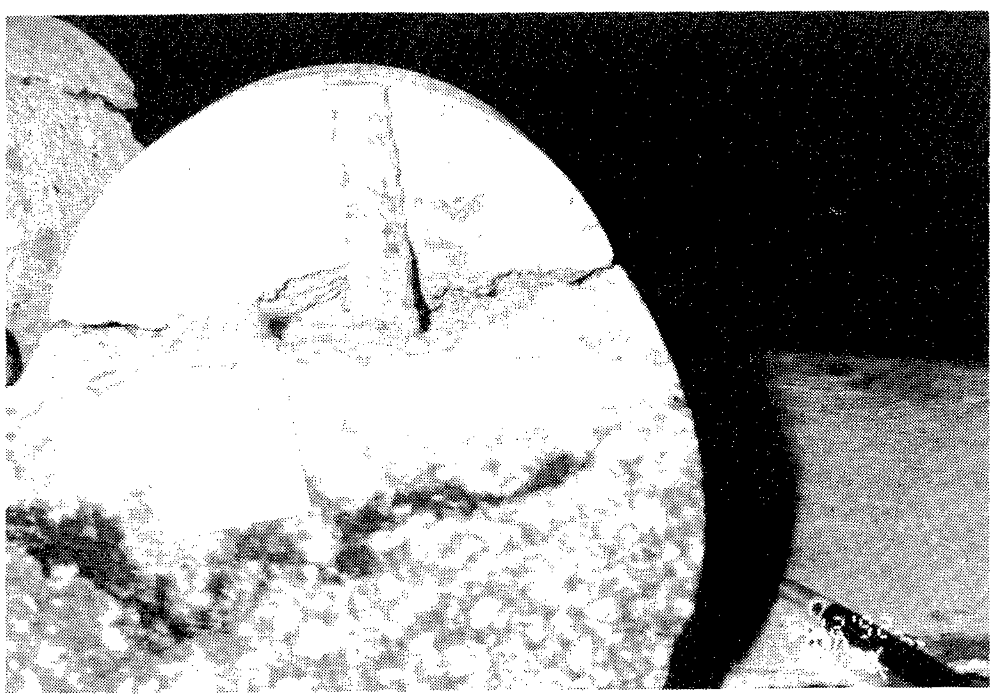


Figure 7. Pullout fracture.

Figure 8. Spalling of transverse crack.

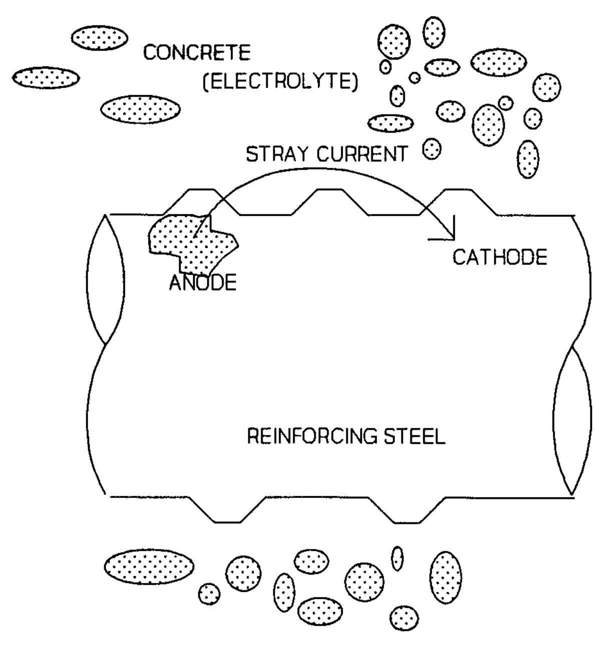
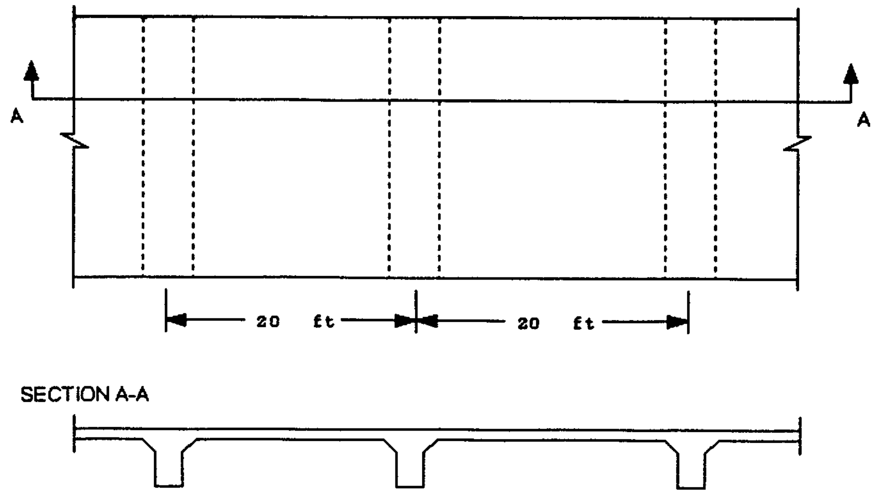


Figure 9. Corrosion cell of reinforcing steel.

Figure 10. Erosion of subbase under punchout distress.

Figure 11. Movement of material caused by pumping.



1 ft = 0.3 m

Figure 12. Lug anchor design.[3]

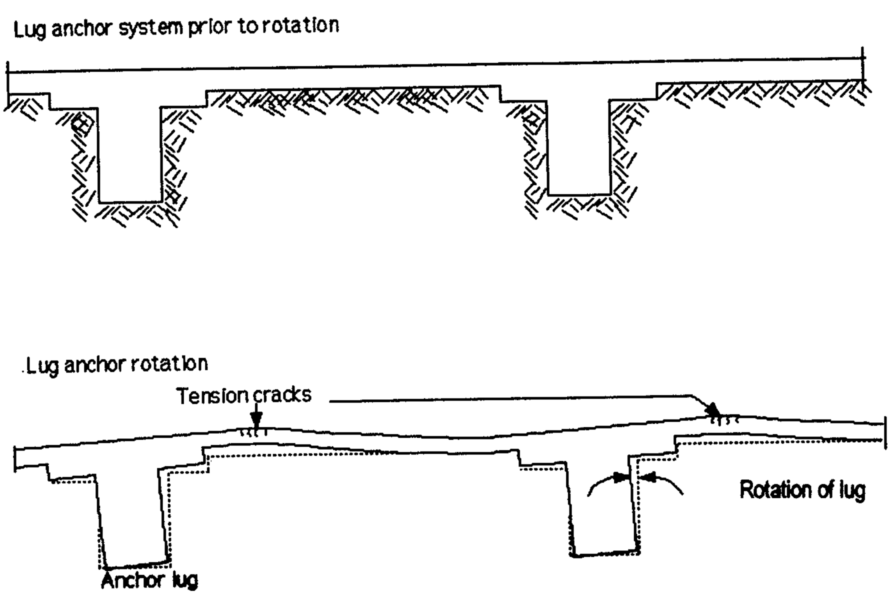


Figure 13. Lug anchor waves.[13]

CHAPTER 3 - PREVENTIVE MAINTENANCE METHODS

Preventive maintenance is applied to prevent or postpone more difficult and expensive rehabilitation to restore structural capacity and riding quality. In the previous section the mechanisms of CRC pavement distress development were discussed. Common elements among many of them were excessive moisture in the pavement structure, voids or loss of support, and steel corrosion. The following sections discuss preventive measures used to combat and correct these mechanisms.

Joint and Crack Sealing

Sealing cracks and joints reduces the chance of distresses caused by free water and deicing chemicals entering the pavement structure and from the infiltration of incompressible materials.

The most common sealing performed on CRC pavements is the joint between the shoulder and the mainline pavement. The other joint most commonly sealed in CRC pavement maintenance is the center line longitudinal joint. The normal transverse cracks in CRC pavements that are working satisfactorily are not normally sealed. However, wide transverse cracks that result due to longer crack spacings should be sealed. However, this is a temporary measure as wide transverse cracks should be repaired using retrofitted load transverse devices or using full-depth patches (see chapter 4).

The benefits of sealing or resealing have often been of short duration because of the rapid deterioration of the joint sealing compound. Typically, within 1 to 4 years the sealant has become ineffective. With the use of newer sealant materials and proper preparation of the joint or crack, the life of joint sealing can be extended. In general, the average frequency for sealing joints and cracks for CRC pavement is once every 5 years.

The types of sealants generally used fall into three categories:

1. Field-poured self-leveling sealants
    a. Hot poured
    b. Cold poured
2. Preformed compression seals
3. Field-poured non-leveling sealants

Factors that influence the performance of a sealant include the movement of the joint or crack, the sealant reservoir shape, the bonding between the sealant and side wall, and the properties of the sealant.[26]

Undersealing

Undersealing is the insertion by pressure of material beneath the slab and/or subbase to fill voids. It is intended to reduce deflection and resist pumping.[26] Undersealing may also be known as slab stabilization, pressure grouting, or slab sealing. Undersealing should not be confused with slab jacking. Slab jacking refers to the lifting of the slab at a depression to its original smooth profile. The purpose of slab jacking is to level out a depression and restore rideability, not to fill the voids.

Materials

The materials most often used in undersealing are cement grouts and very hard oxidized asphalt cement.[26] Cement grout will often contain fillers such as fly ash, lime stone dust, and fine sand.

The materials in the cement grout greatly affect the consistency, strength, and the durability of the mix. An important requirement is that the grout be flowable enough to fill the very small voids beneath the slab and/or subbase. Of the filler materials used in the grout, fly ash has performed the best due to its better flowability because of its spherical shaped particles. Any filler used in the grout should have rounded or spherical shaped particles, not a flat plate-like structure, to increase flowability. Various additives may be used to increase the strength and durability of the grout. These include water reducing agents, accelerators, fluidifires, and superplaticizers.[28] The grout should be the consistency of thick soup, which can be specified using the flow cone method according to the specifications of ASTM C-939. The time of efflux should be between 10 and 16 seconds for mixes using fly ash and 16 and 22 seconds for mixes using limestone dust. Grout with fly ash should meet the requirements of ASTM C-618. Limestone dust should meet the requirements of AASHTO M-17 for mineral fillers.[26]

A typical mix design for an undersealing grout is:[26]

1. One part by volume portland cement Type I or II (Type III may be used if there is a need for early strength).
2. Three parts by volume pozzolan (natural or artificial).
3. Water to achieve required fluidity.
4. An accelerator by approval when ambient temperatures are below 10°C (50°F).
5. Admixtures.

The asphalt cements used most often in undersealing have a penetration of 15 to 30 and a high softening point of 82.2 to 93.3°C (180 to 200°F).[28] The asphalt cement must have a viscosity suitable for pumping when heated to temperatures from 204.4 to 232.2°C (400 to 450°F). The Asphalt Institute recommends the use of asphalt cement as an underseal to meet the requirements of AASHTO M-238.

Equipment

The equipment needed for grout undersealing include:[27,28]

1. Air compressors to drive pneumatic hammers.

2. Pneumatic hammers equipped as drills or other drills that will cut 38- to 51-mm (1.5- to 2-in) holes through concrete and steel. Preferably, a pachometer should be used to avoid cutting steel. The drill should not be so heavy that it spalls the underside of the slab when it breaks through. ACPA recommends coring of drill holes to prevent clogging of voids.

3. A grout plant that can accurately measure and proportion by volume or weight and mix the various components of the grout. The plant must also contain a pump capable of applying 0.345 to 1.724 MPa (50 to 250 psi) of pressure at the end of the discharging pipe (depending on application). A colloidal mixing mill is required for fly ash grout.

4. Cylindrical wooden plugs or other approved plugs that can effectively plug holes until the grout has set.

5. Grout packers that can be inserted into the drilled holes and then seal the hole while the grout is being pumped.

The equipment needed with asphalt cement is similar to the equipment used with grout.[26] A pressure distributor, air compressor, air hammers with drills, asphalt and air hoses, and plugs. The insulated pressure distributor or tank truck must be capable of heating the asphalt to the required temperature and circulating it during the heating process. The equipment for pumping should be able to develop pressures of 0.552 MPa (80 psi).

Undersealing Procedure

Undersealing can be used in two ways. The first is blanket coverage, where a large area is undersealed. The second is localized undersealing, where localized distresses or patches are undersealed. Voids can be located by several means; deflection procedure, radar, visual inspection, and infrared thermography.[20,21,22,23] In Illinois, a visual inspection for pumping and deflection testing method has worked well.[19] In the deflection procedure, deflection tests are conducted every 3.0 m (10 ft) on the pavement near the edge of the mainline CRC pavements, and a mean deflection is found. Areas that have deflections greater than the mean are expected to have voids and are candidates for undersealing. Because many of the heavier loads travel in the outside lane, pumping and voids are found most often in this lane. It is probably most cost effective to do undersealing only in the outer lane. Any patches that show visual signs of pumping should be undersealed to lengthen their life. Blanket undersealing should only be done in those areas of the pavement where pumping and known voids are closely spaced. Generally localized undersealing is more cost effective.[28]

After the candidate areas have been located, undersealing can begin. Two decisions must be made before beginning undersealing. The first is the hole pattern and the second is the depth of the hole to drill. The hole pattern will vary depending on the results of the job as it goes along. Figure 14 shows a pattern that was used in Illinois for cement grout. The longitudinal spacing of 4.3 m (14 ft) can be shortened so better coverage can be achieved.[28] Figure 15 shows a hole pattern for asphalt cement that had satisfactory results in Illinois.[26]

The depth of the holes is dependent on the type of subbase used. If there is a granular subbase, the holes should be drilled or cored to the bottom of the slab. If the subbase is stabilized, the hole should be drilled or cored to the bottom of the stabilized subbase.

Georgia has found that if the grout hole will not take water it cannot be grouted. It is filled with patching material and skipped. This has resulted in a significant decrease in the use of grout material and resulted in more cost effective undersealing operations.

The process of pumping grouts varies greatly. When the asphalt cement is used some agencies require that the holes be blown out with air to remove free water. When free water contacts the very hot asphalt cement, steam is formed and the asphalt cement cools and hardens, preventing some of the voids from being filled.

Undersealing must be closely inspected by both the contractor and the inspector throughout the process. The purpose is to fill the voids and return full support to the slab, not to raise the slab above grade. Lifting the slab generally creates voids adjacent to where the undersealing is occurring. Project specifications should not allow the slab to be lifted more than a total of 3 mm (⅛ in).[26,28] Some monitoring devices that may be used are a cantilevered device or string line with a transit. Other factors that may determine when to cease pumping are the appearance of the undersealing material in adjacent holes, cracks, or joints, and the displacement of water from underneath the slab. The amount of time spent, or amount of material pumped, into one hole should be restricted.[28] There have been cases where the undersealing material was forced out of a shoulder pavement joint on the adjacent lane.[28]

Undersealing should never be done when ambient temperatures are under 4.4°C (40°F) for grouts and 1.7°C (35°F) for asphalt cement. Traffic should be kept off the pavement for at least 3 hours after grouting has occurred to allow adequate curing.[26]

Performance

In studies by Illinois the undersealed sections have performed well.[19,25] There have been some problems with a couple of asphalt undersealed sections. The asphalt undersealing, rather than decreasing the deflection measurements, has increased the deflections. The problem was believed to be that the sections had been undersealed with too much asphalt. In another report from Pennsylvania, they had the same problem with increased deflections after undersealing with asphalt. They concluded that too much asphalt was used and that there was a large amount of free water under the pavement that cooled the asphalt too quickly so not all the voids were filled.[29]

Undersealing is much more an art than a science. It has been performed for many years but its success depends highly on the experience of the contractor or the individual doing the work.[26,28]

Cathodic Protection

Cathodic protection is a technique that can halt corrosion in reinforcing steel. Cathodic protection has been used for many years to protect buried pipelines and to arrest corrosion in bridge decks.[12] Cathodic protection has rarely been used for CRC pavements.

Corrosion causes rust to accumulate because of a stray current being discharged from the anode and being received by the cathode. A driving potential of less than 500 milliamps is sufficient to cause current flow and thus to cause rust to form.[12] Rust accumulates only at the anode. It is this fact that makes cathodic protection work. The reinforcing steel is made into a receiving cathode. The stray current discharge stops and corrosion ceases.[12]

Design

The cathodic protection system combines the two methods that have already proven to be effective, cathodic protection of pipelines and bridge decks. Pipelines are cathodically protected by burying anodes in a conductive material placed near the in-place pipeline. A ground connection is made to the pipeline. Current flows from the anode through the soil to the pipeline, making it a current receiving cathode.[12]

Reinforcing steel in concrete bridge decks has also been protected by establishing stray current.[12] The most common design is to place special anodes on the deck surface. A conductive coke-modified asphalt overlay covered the anodes and deck to distribute current efficiently. An asphalt wearing course is placed over the conductive overlay to provide a driving surface.

The cathodic protection system design for CRC pavements, shown in figure 16, uses a combination of these two techniques. Anodes were placed parallel with the pavement in two different design types. The first design, figure 17, was to place the anodes in a continuous trench. The second design is to place the anodes in post holes, as shown in figure 18. The trenches and post holes are backfilled with conductive coke aggregate. The coke aggregate is used as a backfill because it increases the effective anode area and provides for efficient current distribution. Four ground connections attached to the reinforcing steel in the pavement complete the circuit. When energized, a potential field is created between the anodes and the reinforcing steel in the CRC pavements. Current is conductive to the reinforcing steel in a manner similar to the way the soil conducts a current to a pipeline.

The cathodic protection system design is based on a current requirement of 10.8 milliamps/m^2 (1.0 milliamps/ft^2) of steel surface.[12] The pavement was separated into 10 30.5-m (100-ft) sections so each section would require 1.3 amps for protection.

The anodes used for the trench system are a style commonly used for pipe protection,[12] Type "G" duriron anode, made of high silicon content chromium iron alloy. They are spaced at 15.2-m (50-ft) intervals to supply a maximum of 2.0 amps per 30.5-m (100-ft) sections. The Type "G" anode is also used in the post hole system. The anode is placed in a canister with coke aggregate. The canisters are placed at 15.2-m (50-ft) intervals. A rectifier is used to supply the constant current needed within the cathodic protection system.

Performance

In an experimental project, the cathodic protection system halted or at least slowed down the corrosion of the reinforcing steel.[12] The reinforcing steel in sections with cathodic protection exhibited less spalling than the steel in sections without the system. The test sections with cathodic protection also had areas that were delaminated less severely than the other sections north and south of the test section. Based on these observations, the cathodic protection system appeared to work.

Other findings were that both the post hole and trench system were equally effective. The trench system is more efficient than the post hole system because it requires less voltage to drive the same current to the reinforcement. The construction of the post hole system is quicker, simpler, and cheaper than the trench system. A system tested in Minnesota was found to be underpowered and was not able to apply the current necessary for full protection.[12]

Edge Drains

Edge drains are used to drain the pavement structure. The main purposes of underdrains are to reduce the time that free water is present at the slab-subbase-shoulder interface and to remove rainwater that has penetrated into the base of the pavement. Underdrains also control infiltration water, rising water tables, and internal flow of the water at the interface.[26,30]

The underdrains discussed are those that have been reported as being used in the maintenance of CRC pavements.[19,30] Drainage blankets, drainage layers, and filter layers will not be discussed because they are drainage systems that must be installed when the pavement is built.

Assessment of the need of subdrains begins with a distress survey; this is followed by an evaluation of the existing drainage, and ends with the design of the underdrain system, if needed.[26]

Sources of Water

For underdrain design, the sources from which the water comes must be known. Five different sources of water in the pavement structure have been identified:[26,30]

1. Seepage from higher ground - in cut sections where ditches are shallow and in areas with poorly drained ditches that hold water;
2. Rising ground water table - seasonal fluctuation of the water table;
3. Surface infiltration - water enters the pavement structure through cracks and joints;

4. Capillary movement of water from the water table - surface tension and capillary action transports water well above the water table and saturates the subgrade or subbase. Typical values of capillary rise are 1.2 to 2.4 m (4 to 8 ft) in sandy soils, 3.0 to 6.1 m (10 to 20 ft) for silty soils, and over 6.1 m (20 ft) for clayey soils; and
5. Vapor movement of water - temperature gradients cause water vapor present in the air voids to migrate and condense.

Generally, the only source addressed with edge drains is surface infiltration.

Methods of Controlling Water

There are three methods of controlling water in the pavement system: protection, desensitizing or rendering insensitive, and evacuation.[30] They can be used singularly or in combination. There is no one best solution to a drainage problem in a given area. What may work in one environment may not work in another.

Protection is the use of materials to protect the pavement or to prevent the infiltration of water into the pavement structure. This may be done with crack and joint sealing and the use of anti-capillary subbase courses. Desensitizing prevents the pavement from reacting to the influx of water. This can be done through compaction control, stabilization of the subbase, and undersealing. Evacuation removes the water before it can affect the pavement. This can be done with underdrains and drainage courses.

Design

The underdrains most commonly used in the maintenance of CRC pavements are longitudinal or edge drains and transverse drains.[19,30] In many states, edge drains and transverse drains are included in the initial construction of the pavement. The drains may be various types of tile or perforated plastic pipe.[26] More recently prefabricated geotextiles are also being used. The Iowa DOT reported that, with the use of a prefabricated edge drainage system, initial costs were cut by 50 percent compared with the use of a geotextile-wrapped aggregate trench drain.[31] More information about geotextiles and prefabricated systems can be obtained from their suppliers.

The sizes of the pipes that have been used in typical installation have ranged from 38 to 203 mm (1.5 to 8 in), with 102 mm (4 in) being the most common.[26] The larger sizes are preferred for cleaning and maintenance purposes. Underdrain systems should not have right angle bends because this will hamper the cleaning and maintenance of the drainage system.[30]

Transverse drains should be used when water is present in any full-depth repair of the pavement. The water that is present is probably trapped between the slab and subbase or subbase and subgrade. It probably will never be removed without the drain. By patching the area without a drain the water will remain and probably cause another distress. Patching without the drain is fixing the symptom not the cause.

Performance

There have been very few studies on underdrains, although it is known that drainage is a major factor in pavement design. An Illinois' study found that the use of edge drains may have stopped punchouts occurring in a test section.[25] The test was not conclusive because the edge drains were used with an asphalt overlay of the CRC pavements and undersealing. It was observed that water flowed from the drains soon after their installation.

An Indiana study[19] concluded just the opposite. The subdrained section had numerous failures occur though it was removing water as evidenced by a decrease in pumping and visual inspection of the drain outlet. It was also concluded the drains had been installed too deep and were draining some ground water away because, even during dry periods, water was still flowing out of the underdrains. It was recommended that a better means could be found to drain the pavement effectively. Several States have put a moratorium on the use of retro-fitted edge drains.

If a pavement is showing many distresses linked to excessive free water, underdrain systems should be installed. It is important that the drains and outlets be regularly maintained. When the drains are working they can be effective in reducing distress, but when they become clogged they are a source of free water and the cause of pavement distress.

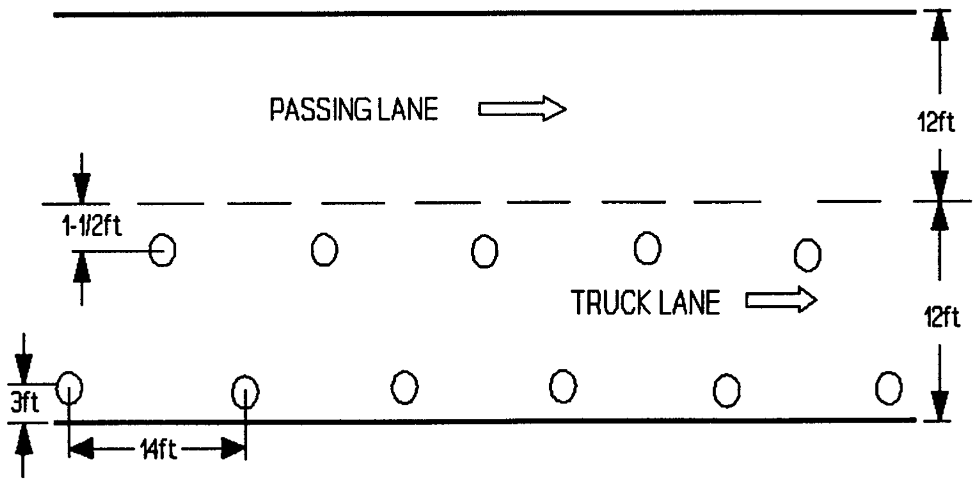


Figure 14. Typical hole pattern for grout undersealing.[24]

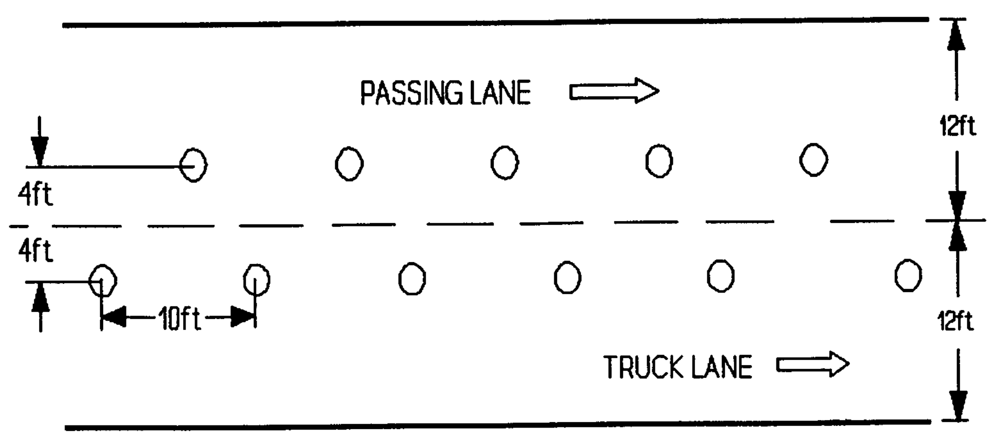
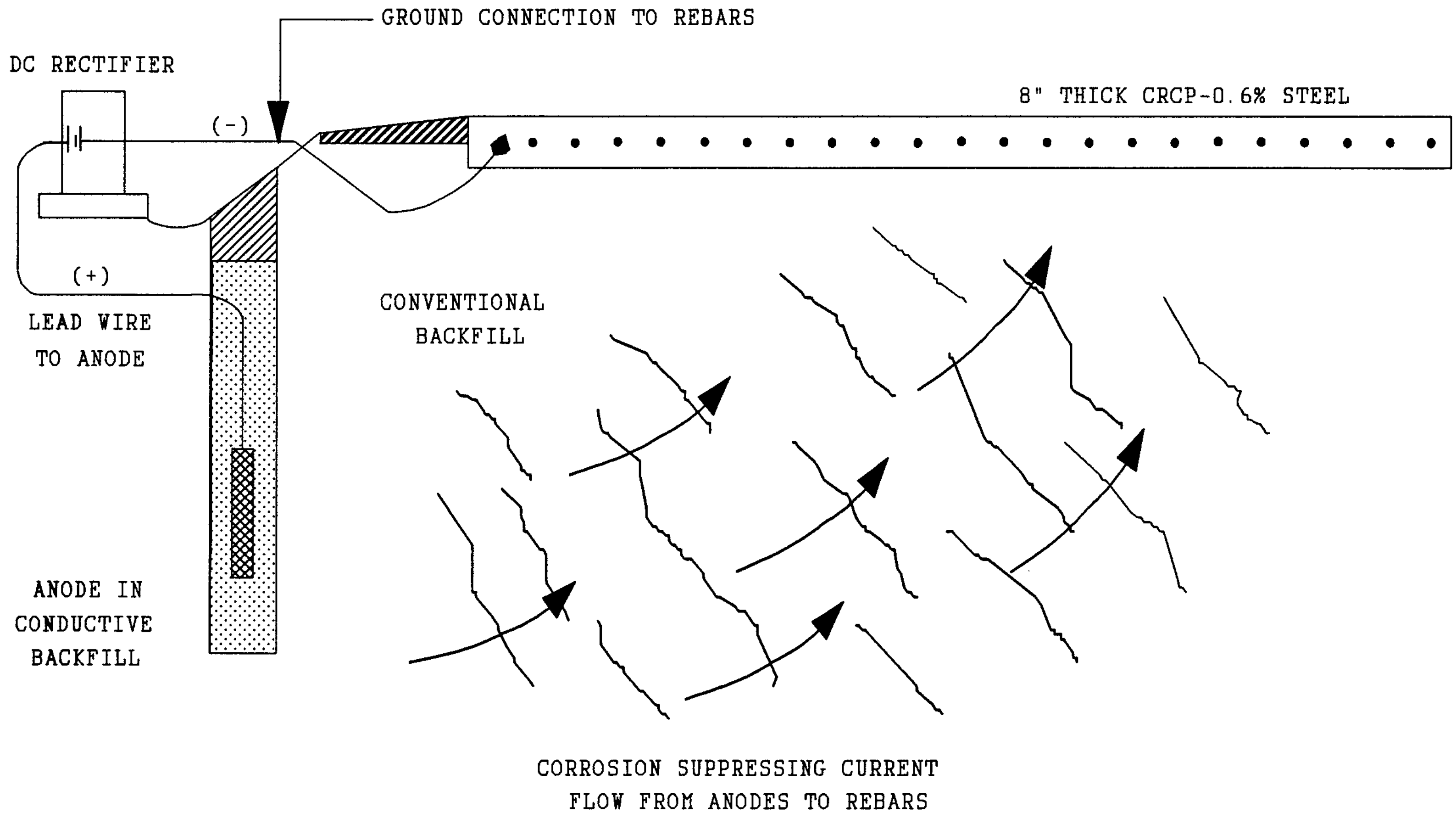
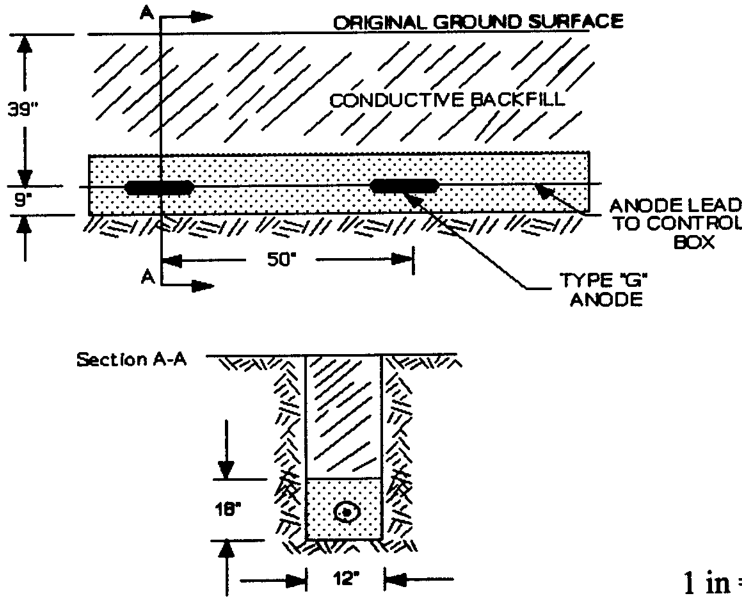


Figure 15. Typical hole pattern for asphalt undersealing.[24]



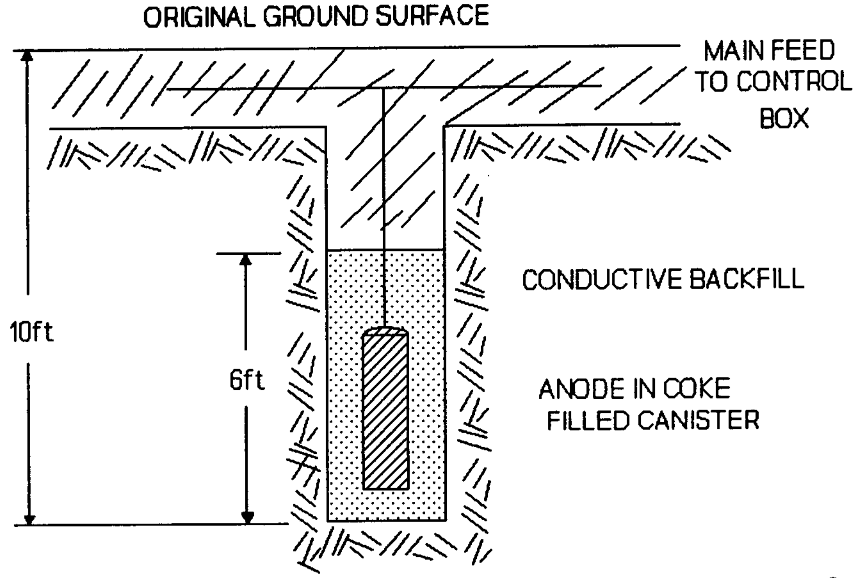
1 in = 25.4 mm

Figure 16. Cathodic protection design.[12]



1 in = 25.4 mm

Figure 17. Trench design.[13]



1 ft = 0.3 m

Figure 18. Post hole design.[13]

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CHAPTER 4 - REPAIR METHODS

As indicated previously, the primary repair activity for CRC pavements is full-depth patching. Full-depth patching is primarily used to correct punchouts and wide cracks. This restores the rideability of the pavement, prevents further deterioration of distressed areas, and prepares the pavement to receive an overlay. Full-depth patching of CRC pavements is different from full-depth patching of jointed concrete pavements. For CRC pavements, it is essential that steel continuity be maintained through the patch. Also, load transfer across the patch joint must be provided by roughening the patch joint faces. The CRC full-depth patching also requires exposing a certain length of the reinforcement in the existing concrete. Typically, full-depth patches for CRC pavements are a minimum of 1.8 m (6 ft) long and a full lane-width wide. Properly designed and constructed full-depth patches can provide good long-term performance. However, many full-depth patches exhibit premature failures because of poor installation conditions and inadequate construction quality.

Full-depth patching procedures are presented first for the conventional approach and then alternate procedures are presented.

Conventional Full-Depth Patching

Full-depth patching involves the following steps:

1. Patch delineation
2. Concrete breakout and removal
3. Base restoration
4. Reinforcement treatment
5. Concrete placement and curing

Details of each of the steps follow.

Patch Delineation

A critical step in performing full-depth repairs is identification of the specific distresses that require full-depth repair and selection of the repair boundaries. This is accomplished by an experienced crew performing a condition survey for the entire project in all lanes. The condition survey must be updated just prior to construction to account for any additional distress that may have occurred. The visual condition survey may be supplemented by nondestructive deflection testing using a falling weight deflectometer (FWD). The FWD can identify if suspect areas are potential sites for future punchouts. These areas exhibit higher deflections than the rest of the pavement areas.

The following minimum repair dimensions are recommended for CRC pavements:

- Length - Minimum of 1.8 m (6 ft) for repairs using tied steel, and 1.2 m (4 ft) for repairs using mechanically connected or welded steel.

- Width - Full-lane-width repairs are strongly recommended. However, a minimum width of 1.8 m (6 ft) may be used when all distress is contained within this width.

The above recommended minimum dimensions are necessary to provide adequate lap length and cleanout and minimize patch concrete rocking.

A full-depth patch is made up of three different sections, a center section and two end sections as shown in figure 19. The center section should include the concrete and subbase that have become deteriorated. The two end sections are the areas where the steel continuity will be established. The actual size of the patch is usually determined by the type of distress being repaired and the type of connection to be used for the steel reinforcement. The minimum length of the center section should be 0.6 m (2 ft). The length of the end sections is dependent on the type of connection used and the proximity to the adjacent transverse cracks. The patch face should not be closer than 450 mm (18 in) to a tight transverse crack.

Most often the patches used have been a full-lane-width wide. The performance of the half-lane-width patches has been mixed and use of such partial lane patches is not recommended. Some agencies have allowed just the distressed concrete to be removed and replaced, if the steel condition permits, and have noted no adverse effects.[28]

Most of the full-depth patches have been made at a right angle to the lane but some states (Iowa and Pennsylvania), have used skewed patches, usually 1:4. The patches were skewed to avoid simultaneous wheel crossing of the patch-pavement interface. These skewed patches tend to form unstable blocks as cracks become numerous.[33] The use of skewed patches is not recommended.

A maintenance project that was observed on I-45 in Texas delineated its patches in the following way. The patch area was determined by the Resident Engineer using his experience and knowledge. In general, the center section boundaries were determined by using the closest transverse cracks on either side from the punchout distress or the closest transverse cracks to where the longitudinal crack stopped. The outer section boundaries were determined by the specified distance according to splicing techniques: 610 mm (24 in) for tied splices and 152 mm (6 in) for either welded or mechanically coupled. Figures 20 and 21 show punchouts that were marked for removal.

The full-depth patch must be delineated properly to ensure that all of the fractured or deteriorated concrete and subbase materials are removed. There is no set of rules on how to properly delineate the patch but the size of the patch should depend on the extent of the distress.

Concrete Breakout and Removal

Two methods have been used to remove deteriorated concrete from the repair areas. These methods are:

1. Breakup and cleanout method - concrete to be removed is broken up by using jackhammers, drop hammers, or hydraulic ram and then removed using a backhoe or hand tools.

2. Lift-out method - concrete to be removed is lifted out in one or more pieces using chains and lift pins or other devices. This is the preferred method for concrete removal as less damage occurs in the base/subbase and it is less time consuming.

Full-depth saw cuts are made at a specified distance away from the distress, which has been marked out by an engineer. These full-depth saw cuts must cut through the reinforcing steel, if not the full slab thickness. The outer patch edges are usually established with a partial-depth saw cut usually 25 to 51 mm (1 to 2 in) deep. It is important not to cut the reinforcement; if it is nicked, the patch should be extended. The partial-depth saw cuts also should not cross any existing cracks. Figure 22 shows a patch with the saw cuts performed.

The partial-depth saw cuts are made at a specified distance away from the full-depth saw cuts according to how the reinforcement is to be connected, e.g., lap splice distance. For tied splicing laps, a minimum distance of 457 mm (18 in) is required. For mechanical connections or welded laps, a distance of 203 mm (8 in) is required. An alternate method that can be used without the full-depth saw cut is to break away the concrete at the specified distance down to the reinforcing steel. Once the steel is exposed it can be cut with either a torch or saw.

A Vermeer wheel saw should never be used to make the full-depth saw cut. It does not cut through the steel. It chews the steel on one side or the other of the wheel blade and badly bends the steel in the saw cut area, damaging the steel in the adjacent concrete. In one study in which the Vermeer wheel saw was used on a badly D-cracked section, it ripped the steel out of the pavement 2.4 m (8 ft) on each side of the cut.[33]

The removal of the center section can be accomplished by several different methods depending on the equipment available. These methods can be placed in two categories: break up and clean out, or lift out. Figure 23 shows the steps in the break up and clean out method. Figure 24 illustrates an alternative method. Those methods break the concrete in the center section with jack hammers or other equipment such as hydro-hammers, gravity drop hammers, and pneumatic air hammers attached to a back hoe, shown in figure 25. After the concrete has been broken up, it must be removed. It is recommended that a ball breaker never be used for the break up of the section because the large shock waves that it produces will generally damage adjacent concrete. Break up should begin at the center of the repair area and not at the saw cuts. The removal of the concrete can be facilitated with a backhoe equipped with a bucket to scoop the material out and load it into a truck. The teeth can be removed from the backhoe bucket to help prevent damage to the subbase.

The advantages of the break up and clean out method include:

1. A minimum amount of equipment is needed, especially if it is done by hand.
2. Pavement breakers effectively and efficiently break up the pavement.
3. A backhoe can rapidly clean out the patch area.

The disadvantages of this method include:

1. If it is done by hand, it is very time consuming and labor intensive, thus making it expensive.
2. If heavy equipment is used, damage to the subbase material can occur requiring the subbase to be replaced with new material or extra concrete.
3. It can cause potential damage to adjacent concrete.

In the lift out method, the center section is lifted out rather than broken up. The center section must be completely freed from the adjoining concrete and reinforcing steel. One method uses a front end loader or bulldozer to lift the slab up from one end. Chains are then connected to the exposed steel at the other end of the slab and then secured to the bucket or blade. The slab is then lifted out and either placed in a truck, or if the section is too big, a flat bed trailer. The section also may be laid off to the side of the road and broken up later. The other lift out method uses lift pins or other similar mechanisms, as shown in figure 26. Two or more holes are drilled into the concrete section and lift pins are inserted. A piece of heavy equipment or other mechanism is used to lift the section. Figures 27 and 28 show the use of a hydraulic system for the lift out procedure. The center section may also be cut into smaller pieces and lifted out, if the section is too heavy and large to handle.

The advantages of the lift out method include:

1. The method does not usually disturb the subbase and does not damage the adjacent concrete.
2. It is a more rapid removal procedure than the break up and clean out method.

The disadvantages of the lift out method include:

1. It may be difficult to dispose of large pieces of concrete.
2. Usually large equipment is needed.
3. The method does not work well if the pavement is badly D-cracked or has full-depth asphalt patches.

Removal of the concrete from the end sections is difficult and must be accomplished carefully. The concrete must be removed without damaging the reinforcement in the lap area. Spalling at the bottom of the slab beneath the reinforcing steel must be avoided. This task should be completed using only jackhammers, prying bars, picks, shovels, and other hand tools. It may be necessary to limit the size of the jackhammers being used for this operation to approximately 6.8 kg (15 lb), to avoid spalling the patch face. It is necessary to break up the concrete around the reinforcing steel without nicking, bending, or damaging the reinforcing steel in any way. The reinforcement <u>MUST NOT</u> be bent up to ease removal of the concrete, because the bars cannot be straightened properly afterward. Bent reinforcement will eventually cause spalling of the patch. The use of a drop hammer or hydro-hammer should never be used in the lap area. This equipment will typically damage the reinforcing steel and cause serious spalling of the slab beneath the steel. Figures 29, 30, 31, and 33 show the removal of concrete from the end sections.

Any steel that was accidentally bent during removal should be carefully straightened. The reinforcement in the end section should be carefully inspected for damage. If more than 10 percent of the bars are seriously damaged or corroded, or if three or more adjacent bars are broken, the ends of the repair should be extended by another required lap distance.[26,33]

Removing concrete from the end section is the most time consuming, labor intensive, and costly procedure in the full-depth patching procedure. If a less expensive way could be found to remove the material from the end section, the cost of patching CRC pavements would be dramatically reduced.

> *Multiple Lane Repairs* - If a distress such as a wide crack with ruptured steel occurs across all lanes, special considerations are necessary because of the high potential for blowups in the adjacent lane, for crushing of the new repair during the first few hours of curing by the expanding CRC pavement, and for cracking of the repair during the first night as the existing CRC pavement contracts. To minimize these problems, it is important that the concrete be placed in the afternoon to avoid being crushed by the expanding concrete and that the lane having the lowest truck traffic be repaired first.

Base Restoration

After the center and end sections have been removed, the subbase material should be inspected. Any loose material remaining from the breakup of the slab should be removed. If the subbase has been disturbed or is in poor condition, the material must be removed and replaced with either similar material, asphalt concrete, or PCC. Figure 33 shows a crew replacing the subbase. If excessive free moisture is present, it should be dried before placing the new material. If the subbase is saturated or the free water cannot be removed, it is recommended that a lateral underdrain be placed under the patch.

Developing adequate compaction of granular material while repairing the subbase is difficult in a confined repair area. Hand vibrators do not usually produce adequate compaction and lead to settlement of the repair. Replacing the deteriorated subbase or part of it with concrete is the best performance alternative and should not increase cost for repairs less than 3.0 m (10 ft) long. If the subbase is repaired using concrete, then a bondbreaker should be used between this replacement concrete and the overlying concrete patch.

Reinforcement Treatment

After the base restoration has been completed, new steel reinforcement should be installed. The type of reinforcing bar used should match the original grade, quality, and number. If wire mesh is the original reinforcement, it is difficult to match, so it maybe necessary to use deformed bars at the same percentage of steel as the wire mesh in the patch area. Illinois uses supplemental transverse steel in the patch as cheap insurance against longitudinal cracks or punchouts in the patched area.

There are three different methods used to connect the new steel to the original reinforcement: a tied splice, a welded splice, and mechanical connections. The required lap length for a tied reinforcement splice depends on the size and type of reinforcement. In general, the embedment length of 29 bar diameters is adequate. For No. 5 deformed bars the length needed would be 457 mm (18 in), and for No. 6 bars a length of 559 mm (22 in) is needed, as shown in figure 34. Mechanical connections can be used instead of tied splices and have a lap length of 102 mm (4 in). Minimum bar length is 8 in – stagger to prevent weak plane and poor consolidation. Figure 35 displays the dimensions of a mechanically coupled patch. Welded splices require a minimum lap length of 102 mm (4 in). A single lap of 6.4 mm (0.25 in) continuous weld should be made 102 mm (4 in) along both sides. This should provide adequate development of full bar strength.[25,26,28,34] Figure 36 illustrates a welded patch. The new reinforcing bars must be overlapped at the center and tied to avoid potential buckling when welding or mechanical couplers are used, otherwise the connections may be broken due to the movement of the CRC pavements, especially during hot days. For all three connections the new bars should not be closer than 51 mm (2 in) from the patch face.

Hunt reported on the cost effectiveness of mechanical couplers for the Pennsylvania Department of Transportation (DOT).[36] He found that the area needed to be removed by hand, the end sections, was reduced by 75 percent by using mechanical couplers. Pennsylvania DOT recommends that the tied end section be 610 and 152 mm (24 and 6 in) for mechanical couplers. Hunt's other findings were that the mechanical couplers produced a cost savings of 8.07 percent/m^2 (6.75 percent/yd^2); per patch, according the Pennsylvania specifications, there was a cost savings of 37.55 percent. These savings may also be correlated to the use of welded connections because the lap section is also only 152 mm (6 in). The cost of welding may be more expensive than mechanical couplers because of the need for a welding machine, but overall it should be cheaper than tied splices.

Another problem with the tied splices is shown in Figure 37. Some steel has been badly bent and the ties cannot possibly produce an effective embedment of 457 mm (18 in), resulting in the positive continuity of the steel reinforcement being lost. This is an example of poor workmanship and possibly the lack of adequate inspection and quality control on the patching project.

The performance of all three connection types has been satisfactory. There have been reports of poor performance of the welded connections but it has been determined to be caused by poor workmanship. There does not appear to be a difference in the performance among the three, if specifications have been followed correctly.

The reinforcing steel should be placed so a minimum of 64 mm (2.5 in) of concrete cover is provided. If the existing steel is less than 64 mm (2.5 in) below the surface, the splice bar should be placed underneath the existing bar. The reinforcing steel should be supported by chairs or other mechanisms so the reinforcement will not be permanently bent down during placement of the concrete. The reinforcing steel should also be inspected before concrete placement for patches of mud or other foreign material that may cause a loss of bond between the bar and the concrete.

Concrete Placement and Curing

Before placement of the concrete, the patch area should be cleaned, making sure any material that fell in during previous work is removed. Some agencies use high-pressure air hoses for cleanup, and good results have been reported. The air hoses generally will not remove heavier material that may be in the patch. If the shoulder has settled below the surface of the slab or is in poor condition, side forms should be placed to make it possible to strike off the concrete. The use of paved shoulders as a guide in surface finishing has produced poor riding quality of the patch.

It is important to develop a strong bond between the new patch and existing concrete. Some agencies have applied epoxies, cement grouts, and water to the patch face to achieve a good bond. There has been no significant difference in the performance of the three methods. The use of water is probably more cost effective than either epoxy or grout. Use of taper rather than vertical cut to increase patch support.

The overall cost of the patch increases with the time it takes to place and cure the concrete. The concrete placement and finishing techniques should follow the accepted standard procedures that are required by state specifications. **Once the concrete has been placed, it is very important that special attention be given to ensuring that the repair is well vibrated around the edges and beneath the reinforcement**. A value engineering report recommends the use of a mechanical vibrator, preferably hand held, be used to ensure that adequate consolidation has been achieved. The patch should be struck off two or three times to ensure the surface is smooth. The texture of the patch should be at least equal to the texture of the adjacent concrete. The patch should not be over-finished because it will decrease the strength of the surface. The addition of water to the concrete at the site should be avoided unless absolutely necessary because of the detrimental effects that it has on the strength development and increased shrinkage.

The concrete mixture selected depends on the time available for curing before the repair must be opened to traffic. When the patch can be cured for several days, similar to new construction, a regular concrete mix can be used. Often, the patch must be opened to traffic in one to three days. For these conditions early concrete strength is generally needed. Cements of Type I and Type III have been used in the mix design for concrete patches. High early strength of the concrete can be achieved by increasing the cement content, adding an accelerator or other admixtures, and minimizing mixing water. Many rapid setting materials that have been used to achieve early opening of repairs are readily available. The cost of many of these materials is much greater than conventional mixes, but the quicker the lane is opened, the lower overall cost of the patch. Some of these admixtures are summarized in references 33 and 38. Typical batch designs for repairs have used a seven to nine bag mix with an accelerator to permit openings in one to three days. Some of these batch designs may only take 3 hours to produce a strong enough patch, but 5 hours should be used as a minimum time before traffic is allowed on them to provide some factor of safety. Calcium chloride has been used as an accelerator, but the chloride may lead to reinforcing steel corrosion and has not been permitted by some state agencies. A report by Illinois concluded that increasing the cement factor above the standard seven bag mix is not considered cost and performance effective.

Many different materials have been used to help cure the concrete in the patch. Some of these materials are wet burlap, ponding of water, impervious paper, and curing membranes. The curing membrane, usually white pigmented, is considered the most cost effective and practical process used. On windy days with wind speeds greater than 16 km/h (10 mi/h), a clear polyethylene sheeting should be placed over the concrete to reduce the amount of moisture loss from the surface. To help accelerate the curing of the concrete (especially for fast track construction), a method has been developed to increase the internal temperature of the patch. This method uses a 102-mm- (4-in-) thick layer of insulation material over the patch. The insulation can raise the temperature in the center of the patch over ambient temperature as much as 16.7°C (30°F) by the age of 4 hours. This difference can be maintained over several hours. By maintaining a high temperature for curing, it can permit early opening of the roadway to traffic. The insulation should not be placed when the ambient temperature is greater than 32.2°C (90°F).

There are many factors that influence the time necessary for concrete to develop sufficient strength to safely resist traffic loads. The ambient temperature at placement has been found to be by far the most influential factor of strength development of the patches. Most State agencies do not allow a patch to be opened to traffic unless it has cured for a required number of days or unless cast specimens have a minimum modulus of rupture value of at least 2.07 MPa (300 psi).

Patching of CRC pavements should be done in the spring or fall when daily temperature extremes are at a minimum. <u>Patching before noon is not recommended</u>, especially during the summer months, as this tends to lead to the crushing of the weak concrete patch in the late afternoon due to the expansion of the CRC pavements ends. Local experience should be the controlling factor on the best time to patch the pavement.

Performance

Very few detailed studies have been conducted on the performance of full-depth concrete repairs. In general, full-depth patching of CRC pavements has performed satisfactorily. One of the few studies was conducted by the Illinois DOT. A survey of all the permanent patches Illinois had installed on its pavements that were 5 years and older was conducted. A total of 831 patches from ages of 1 year to 7 years were surveyed and rated. Sixty of the patches were partial-depth repairs. The patches were rated from excellent to poor using the following criteria:

Excellent - No visible cracks exist within the boundaries of the patch, the patch is smooth and flush with the adjacent pavement;

Good - One or more tight transverse cracks exist within the patch boundaries but no longitudinal or diagonal cracks are present;

Fair - Transverse cracks within patch boundaries and joint at the patch ends display considerable spalling and faulting, patch will probably be replaced in a year; and

Poor - The patch is seriously damaged and requires the removal and repatching of a major portion very soon.

The study showed 60 percent of the patches in excellent condition, 16 percent in good condition, 18 percent in fair condition, and 6 percent in poor condition.

Another report by Illinois reviewed the performance of tied and welded full-depth patches in a test section. Within the test sections studied, 30 welded full-depth repairs and 10 tied full-depth repairs were constructed. The performance of the patches was excellent. Only two of the welded splice patches and one of the tied splice patches had to be replaced over the 10-year performance study.

The types of distress that affected the patches in the Illinois survey were:

1. Irregular transverse cracks;
2. Edge punchouts;
3. Longitudinal joint failure;
4. Pumping; and
5. Spalling.

The distresses that affected the adjacent slab were:

1. Spalling;
2. Wide cracks; and
3. Edge punchouts.

The reasons for these distresses were listed in six categories, including:

1. Inadequate evaluation of pavement distress and incorrect location of patch boundaries;
2. Damage to the subbase, reinforcing steel, and adjacent slab during removal operations;
3. Lack of subbase/subgrade improvement;
4. Incorrect installation and splicing of reinforcing steel;
5. Expansion and contraction of the CRC pavement slab; and

6. Inadequate curing of the patch.

Alternative Patch Types

Because of the high cost of conventional full-depth patching, there have been some alternative patch types designed and tried. The alternatives are based on either doing the repair quicker by reducing the placement and curing time or reducing the amount of hand construction.

Texas DOT Procedure

Because of the time-consuming process of exposing the resurfacing steel at the patch ends, Texas DOT allows the use of grouted tie bars for full-depth patching of CRC pavements. After the deteriorated concrete is removed, holes are drilled into the existing concrete at mid-depth and tie bars are grouted using approved epoxy-based material. Tie bars are embedded at least 12 in into the existing concrete and are extended into the patch by 21 in for No. 5 bars or 25 in for No. 6 bars. The grouting of the tie bars must meet the requirements of the pull-out test. The pull-out strength of the grouted tie bars must be at least three-quarters of the yield strength of the tie bar, as determined using the procedure of ASTM E388. <u>The pull-out test is conducted within 18 hours after grouting of the tie bars into existing concrete</u>. Concrete is not placed in the patch area until the tie bar grout has attained sufficient strength to preclude displacement of the tie bars by the concrete.

Other than the specific procedures discussed above, the full-depth patch is constructed in the same manner as conventional full-depth patches.

Precast Slabs

The alternative that is most similar to the conventional method is the use of a precast slab. Figure 38 shows the design of a precast slab. The use of precast concrete slab is not a new idea. Precast repairs have been used on jointed pavements in Michigan, Florida, California, Virginia, and New York with satisfactory results. The difficulty with the use of the precast repair in CRC pavements is the necessity of maintaining reinforcing steel continuity. This continuity can be maintained by one of three methods. The first, positive connection, is the use of welds, commercial splicers, and couplers, similar to conventional CRC pavement repair. The second, passive connection, is when the steel is not connected. The steel reinforcing bars are anchored into the cast-in-place concrete by making sure the bars have proper embedment length to develop enough bond strength. The third is the use of both positive and passive connections.

The distressed area of the CRC pavement is removed in the same fashion as for conventional repair. The precast slab for the center section can be formed and then poured either on site or in a lay down yard. The steel configuration in the patch should match that of the CRC pavement slab. After the patch area has been cleaned out, a leveling course is placed on top of the subbase. The precast slab is then lifted and positioned into place, the steel reinforcement connected, and the end sections of the patch area are cast in place with a quick setting concrete or a polymer concrete.

The use of precast slabs for repair of CRC pavements appears to be structurally feasible but there is limited information on economic feasibility and performance. A study in Texas, in which two precast repairs were constructed, concluded that the repairs are cost effective when user costs are included. The lane closure time for these patches was only 6 hours because of the use of polymer concrete. No performance data have been recorded for these patches. Reference 40 provides more detailed information about the design of precast slabs used to patch CRC pavements.

Plain Concrete Patches

Plain concrete full-depth patches with active joints are not recommended for CRC pavements. Because of the large seasonal end movements (in excess of 50 mm (2 in)) typically associated with CRC pavements, the active joints do not perform well.

Full-Depth Asphalt Concrete Patches

Full-depth asphalt concrete patches with the steel removed have also been used. In all the studies reporting their use, none performed for more than 1 to 2 years. The asphalt concrete patches shoved and rutted. The adjacent CRC pavement often shows distresses. It is recommended that asphalt concrete patches only be used as temporary patches for less than 1 year until a permanent concrete patch can be constructed.

Repair of Wide Cracks

Wide cracks occurring prematurely in CRC pavements (within about 3 years) may be repaired using retrofit load transfer type procedures. Although no use of this technique has been reported, use of retrofit load transfer repair can mitigate deterioration of the concrete adjacent to the wide crack and possibly minimize occurrence of faulting and punchout at these areas.

The retrofit load transfer procedure involves the following steps:

1. Identify crack locations to be repaired - any crack where the steel has ruptured.
2. Determine the number of dowel bars to be used per lane width. Typically, a minimum of six dowel bars are used - three in each wheelpath. Dowel bars should typically be 37 mm (1½ in) in diameter and 18 in long.
3. Slot are made in the CRC pavement at the designated locations for the dowel bars.
4. Dowel bars, lightly greased, are positioned in the slot to ensure proper alignment. Expansion caps are used at each end of the dowel bar to ensure that the anticipated "free end" movement of the CRC pavement can be

accommodated. Repairs made on warm summer days will require less expansion space than repairs made during cooler days.

5. Fast setting and high-strength repair material is used to fill the slot. The repair material should be properly consolidated and cured as per the manufacturer's instructions before opening the repair area to traffic.

CRC pavements that have used relatively lower level of steel reinforcement typically exhibit large crack spacing (in excess of 3.3 m (10 ft)). These widely spaced cracks soon exhibit wide cracks as a result of yielding or rupture of the reinforcement. Lack of load transfer at these locations leads to spalling, faulting, and punchouts. Early repair of these wide cracks using the retrofit load transfer procedure can maintain the structural integrity across the wide cracks and extend the service life of the CRC pavement.

Summary

Properly designed and well-constructed full-depth patches can be expected to provide excellent long-term performance. Many State agencies routinely carry out large-scale full-depth patching projects with minimal impact on traffic operations by coordinating the repair process (generally nighttime or weekend work in high traffic areas) and by using accelerated paving (fast-track) procedures. However, attention to construction quality control cannot be over-emphasized because poor construction can lead to premature failure of the patches.

The actual construction of the full-depth repair must be carefully controlled through specifications and good site inspection. The damage to the surrounding concrete and to the subbase must be kept to a minimum, if it cannot be avoided. Restoring the continuity of reinforcement is critical to the performance of the full-depth patch. Also, patch joints must be rough-faced and vertical beneath the partial-depth saw cut to ensure adequate load transfer is maintained across the path joints. Consolidation of the concrete at the patch boundaries is critical.

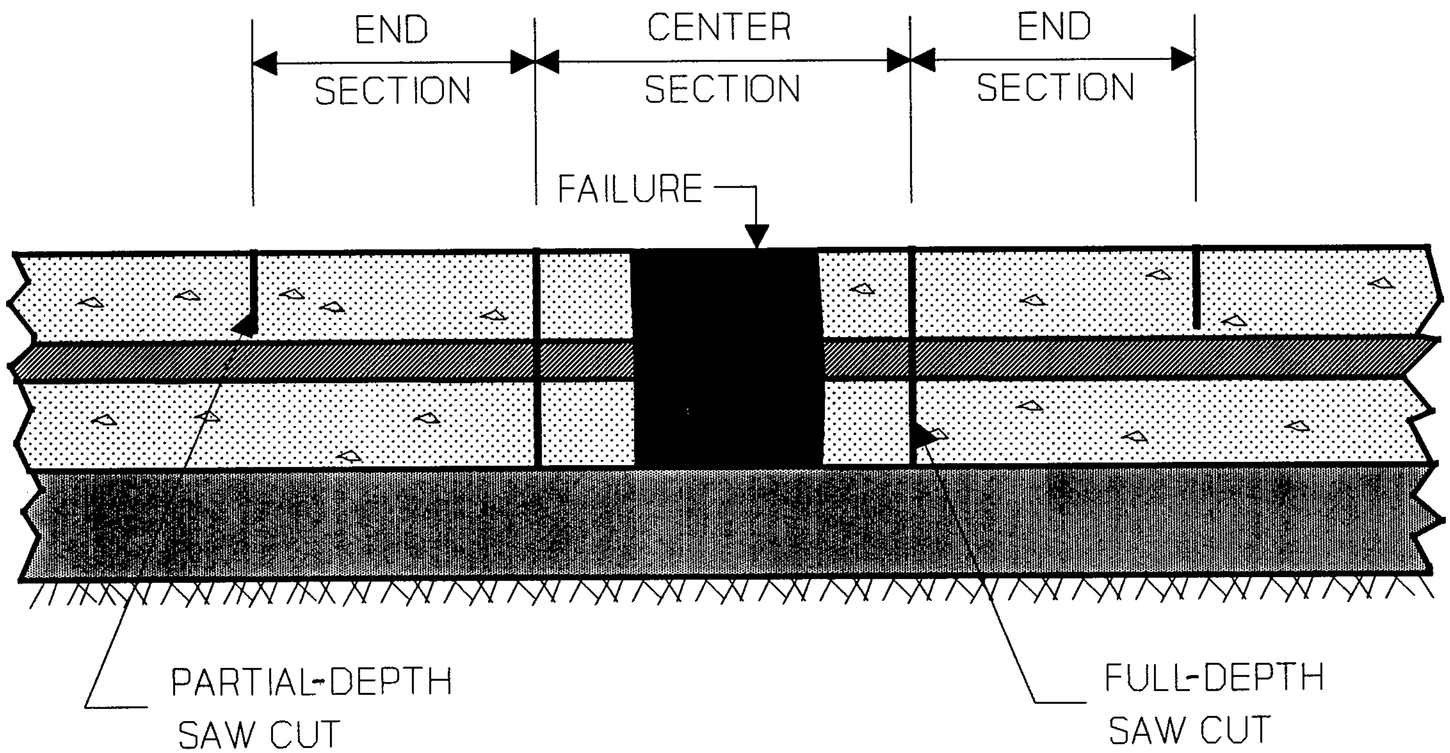


Figure 19. Full-depth patch section with saw cuts.

Figure 20. Marked out distress area.

Figure 21. Punchout marked out for removal.

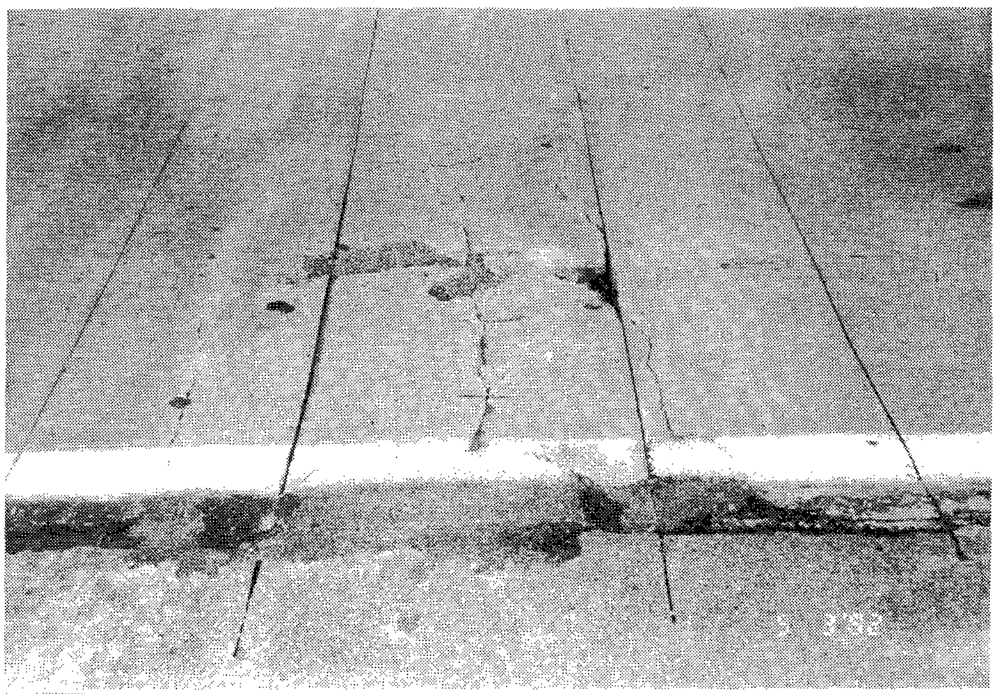
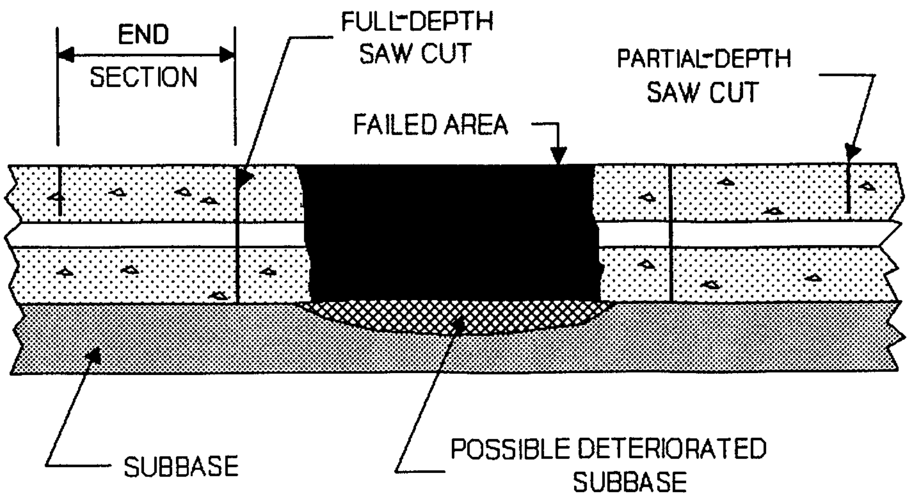
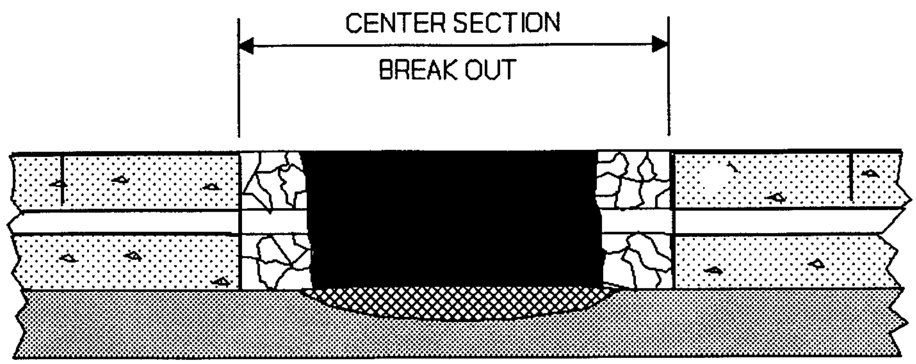


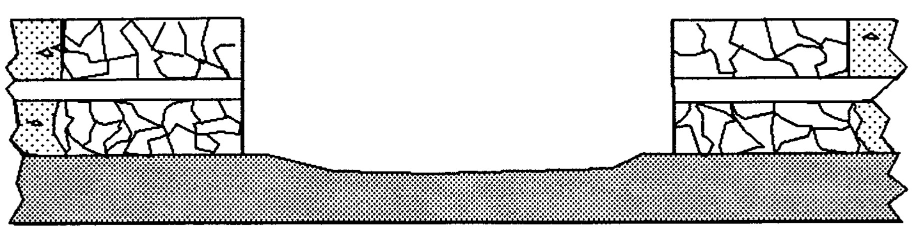
Figure 22. Distress area that has been saw cut.



A.) Partial-and Full-Depth Saw Cut



B.) Center Section Breakout



C.) End Section Breakout

Figure 23. Break up and clean out method.

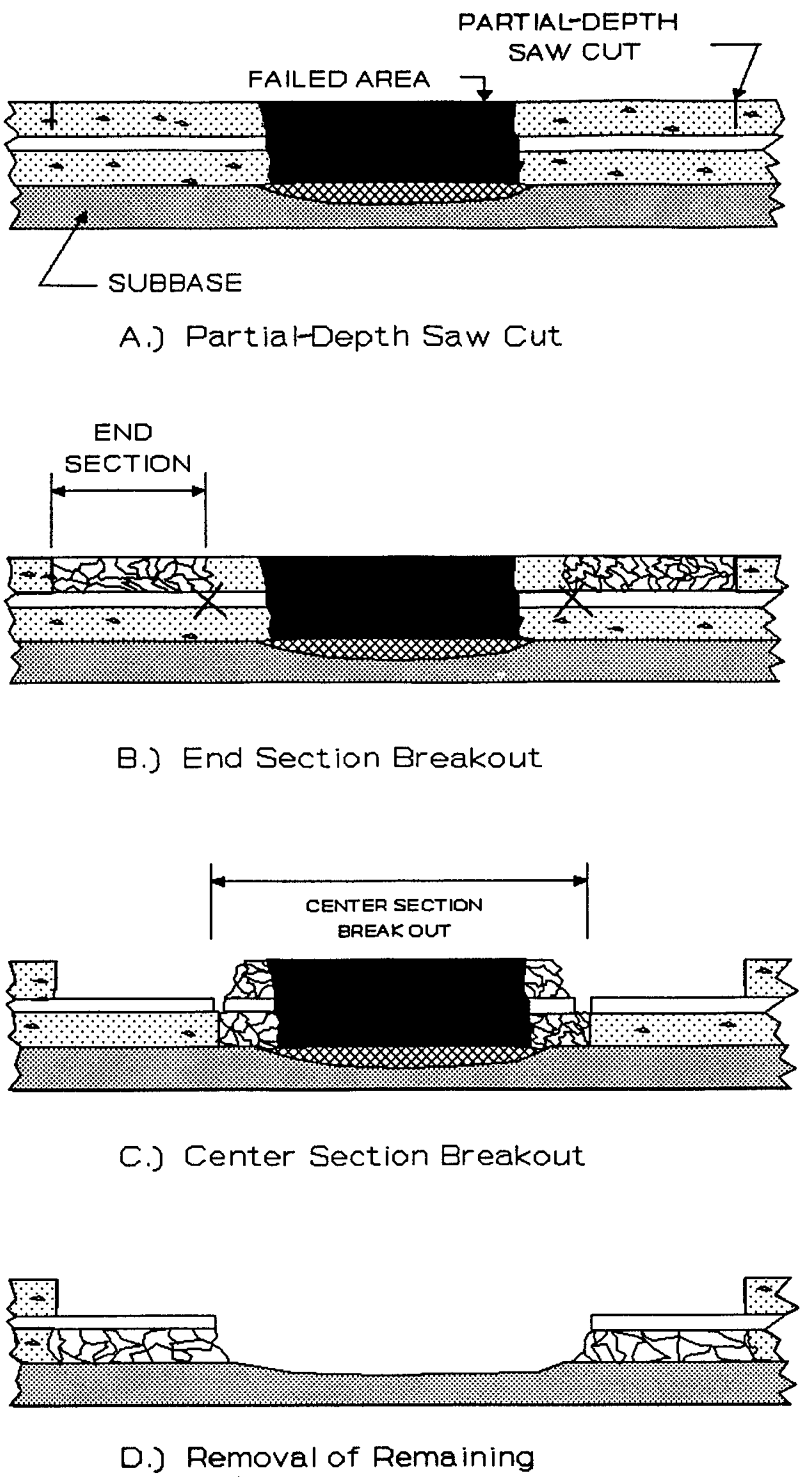


Figure 24. Alternative method.

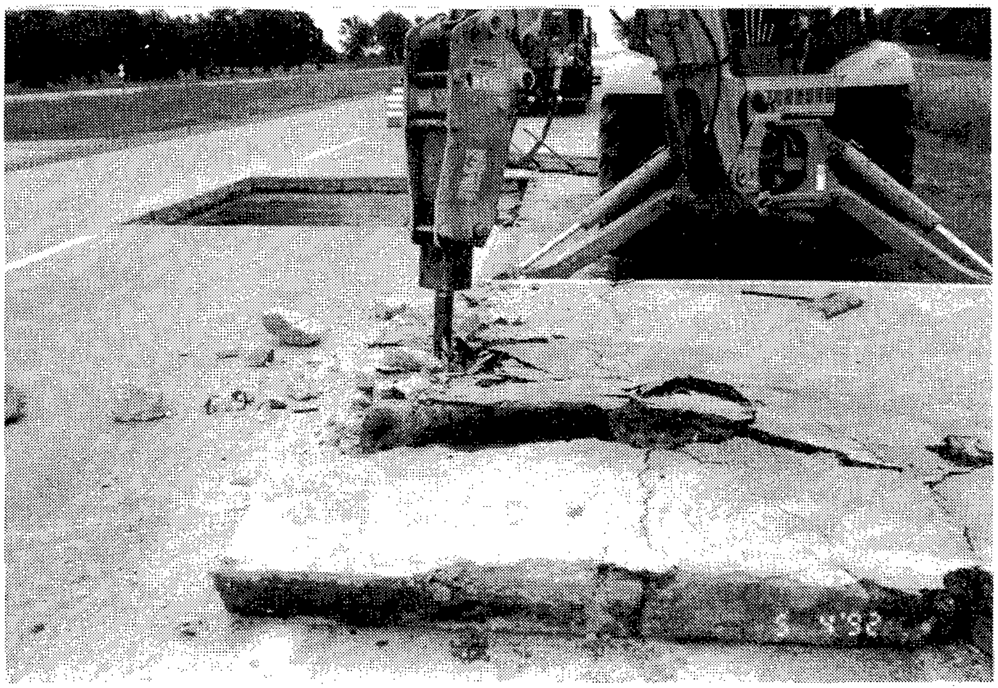


Figure 25. Hydraulic hammer attached to backhoe.

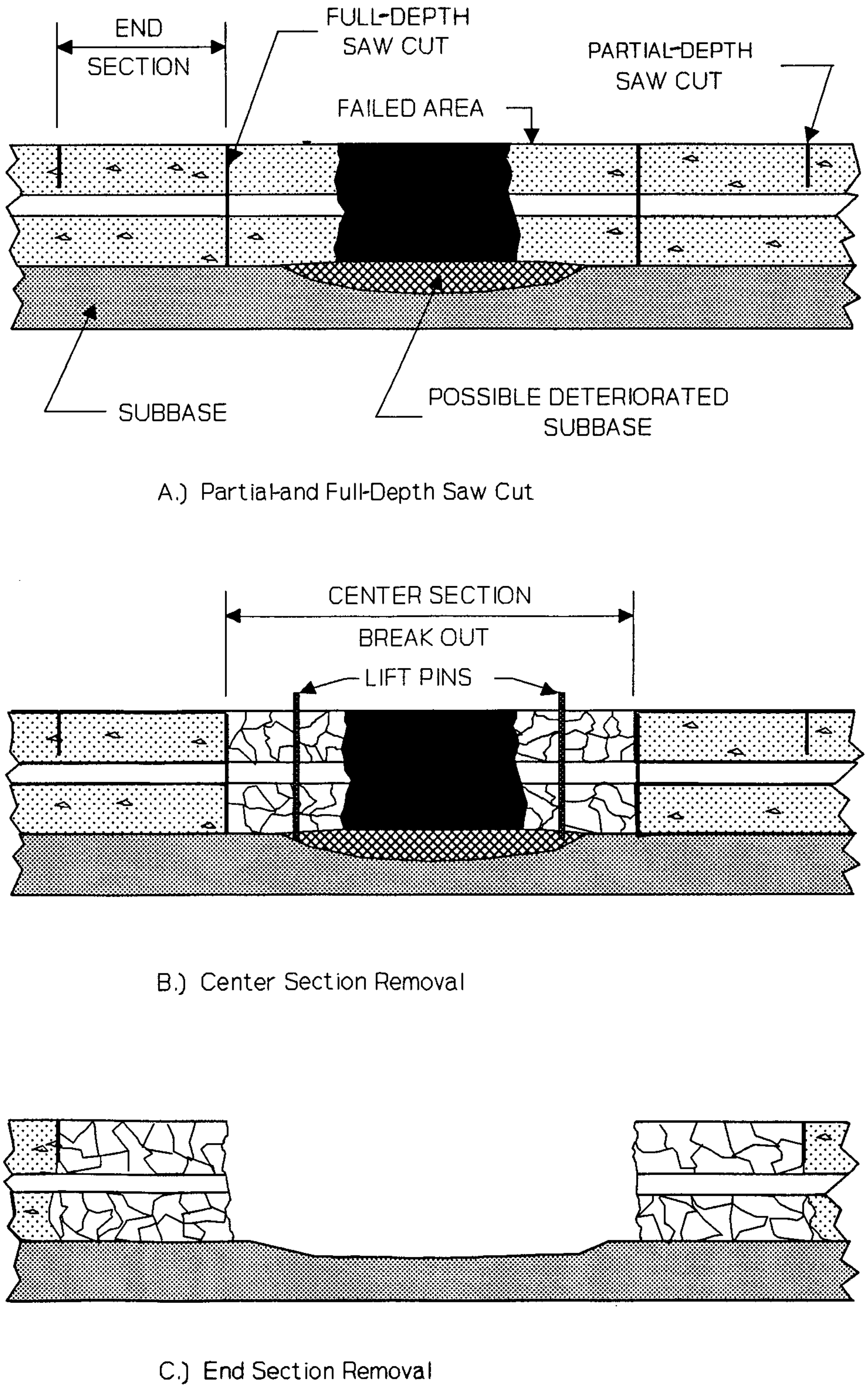


Figure 26. Lift out method.

Figure 27. Slab lifting device.

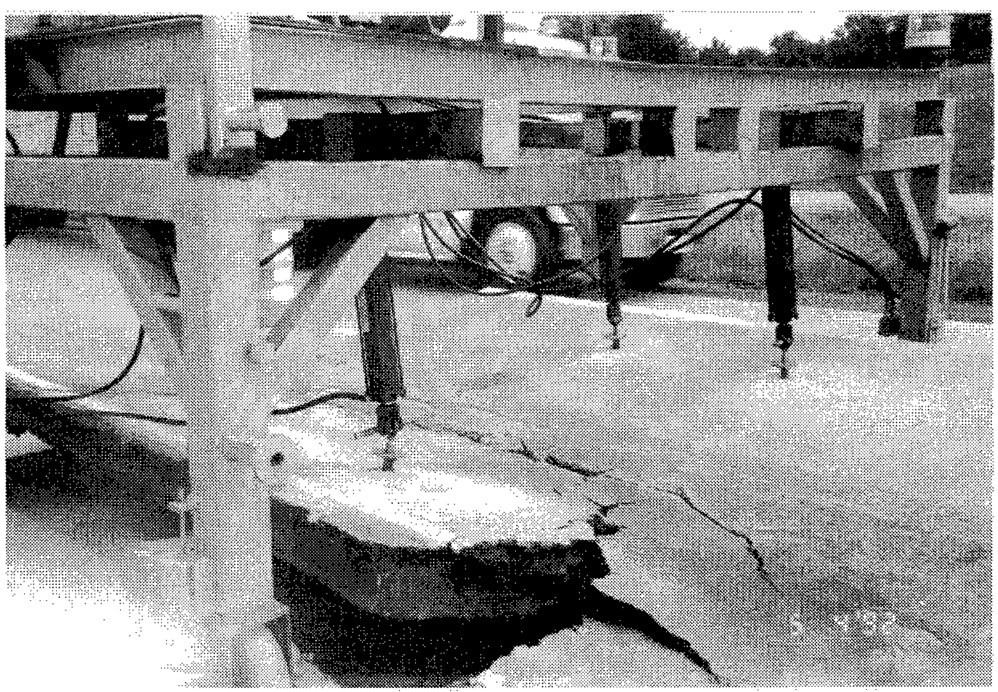


Figure 28. Slab lifting device – close-up view.

Figure 29. End section removal – using a jackhammer.

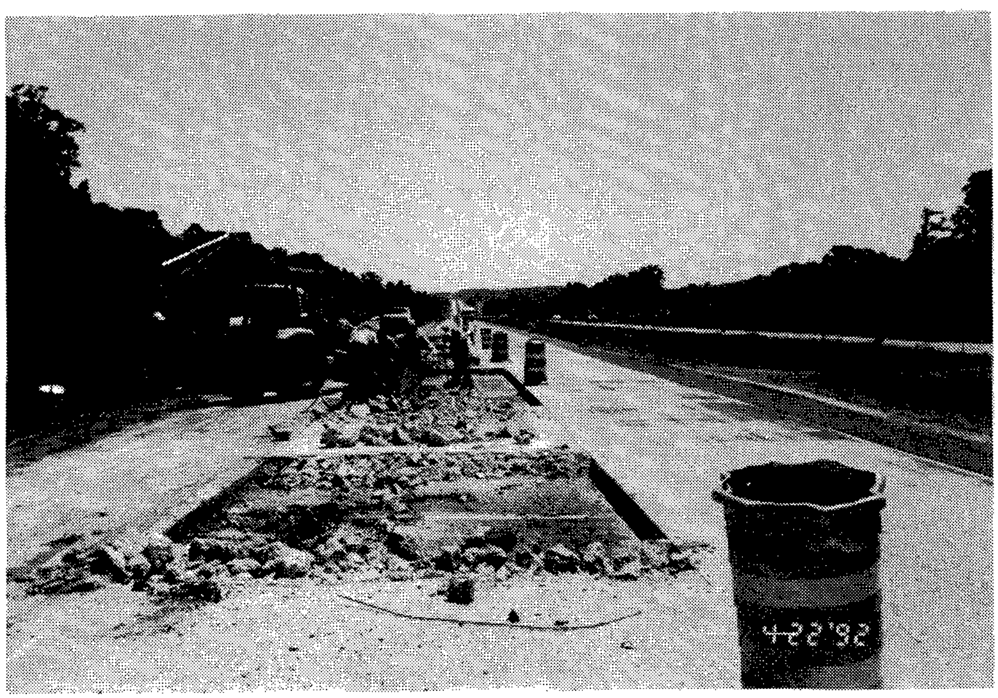
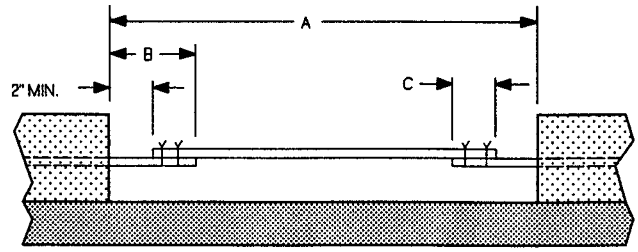


Figure 30. End section removal process.

Figure 31. End section removal – close-up view.

Figure 32. Cleaned out end section.

Figure 33. Replacing disturbed subbase with asphaltic concrete.

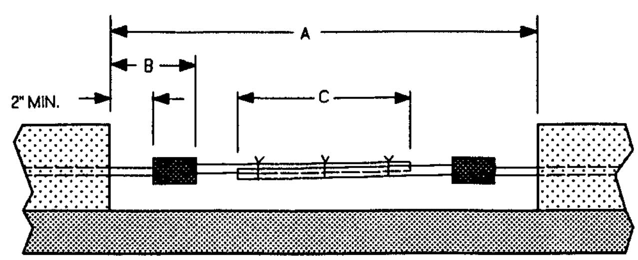


	A (Minimum)	B	C
No. 5 Bars	4ft 6in	18in	16in
No. 6 Bars	4ft 6in	22in	20in

1 ft = 0.3 m

Figure 34. Tied splice.

1 in = 25.4 mm

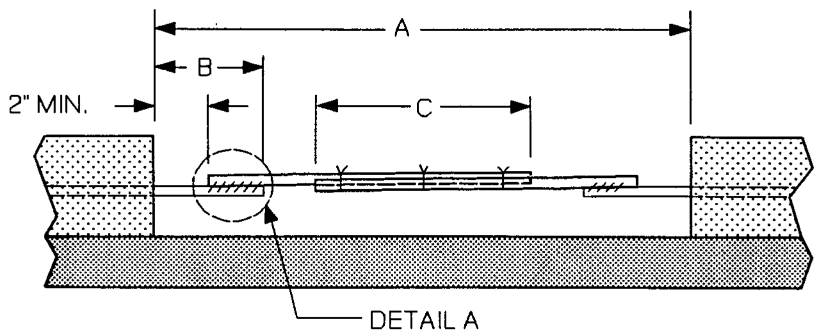


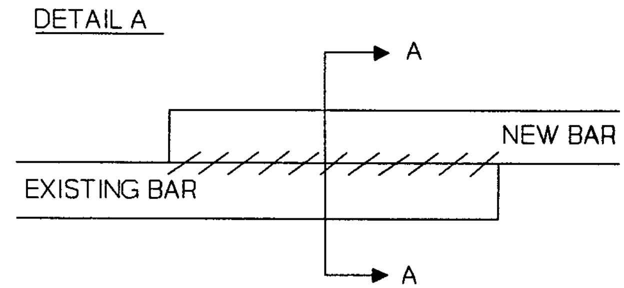
	A (Minimum)	B	C
No. 5 Bars	4ft 6in	6in	16in
No. 6 Bars	4ft 6in	6in	20in

1 ft = 0.3 m

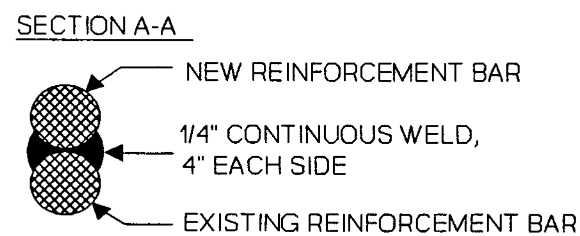
Figure 35. Mechanical coupler splice.

1 in = 25.4 mm





	A (Minimum)	B	C
No. 5 Bars	4ft 6in	18in	16in
No. 6 Bars	4ft 6in	22in	20in



1 ft = 0.3 m
1 in = 25.4 mm

Figure 36. Welded splice.

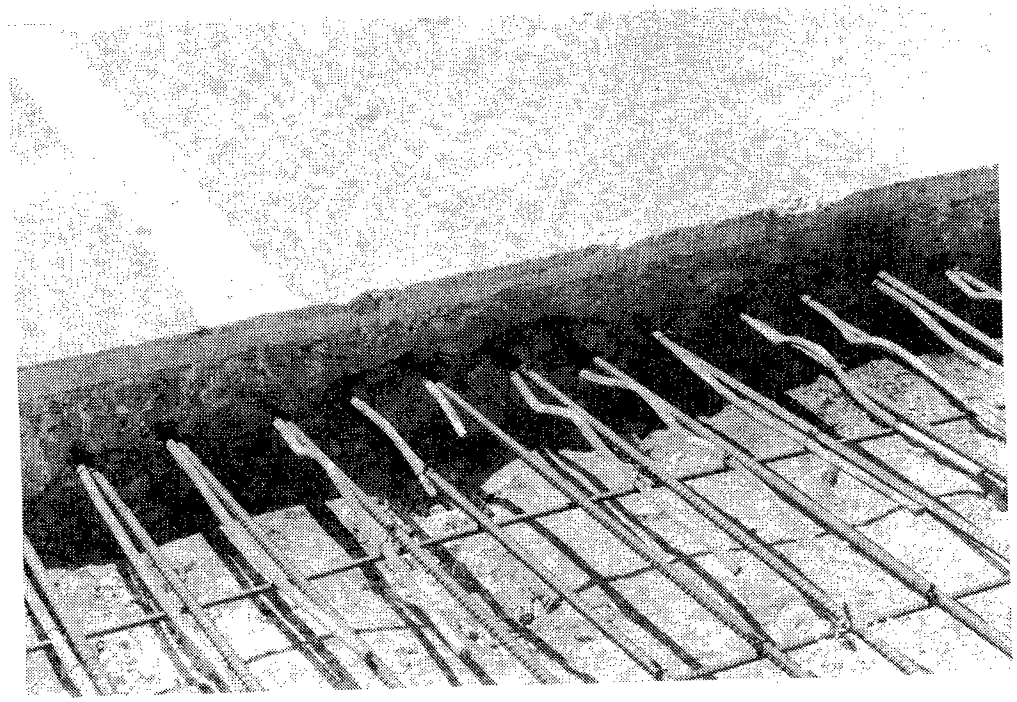


Figure 37. Poorly tied splices.

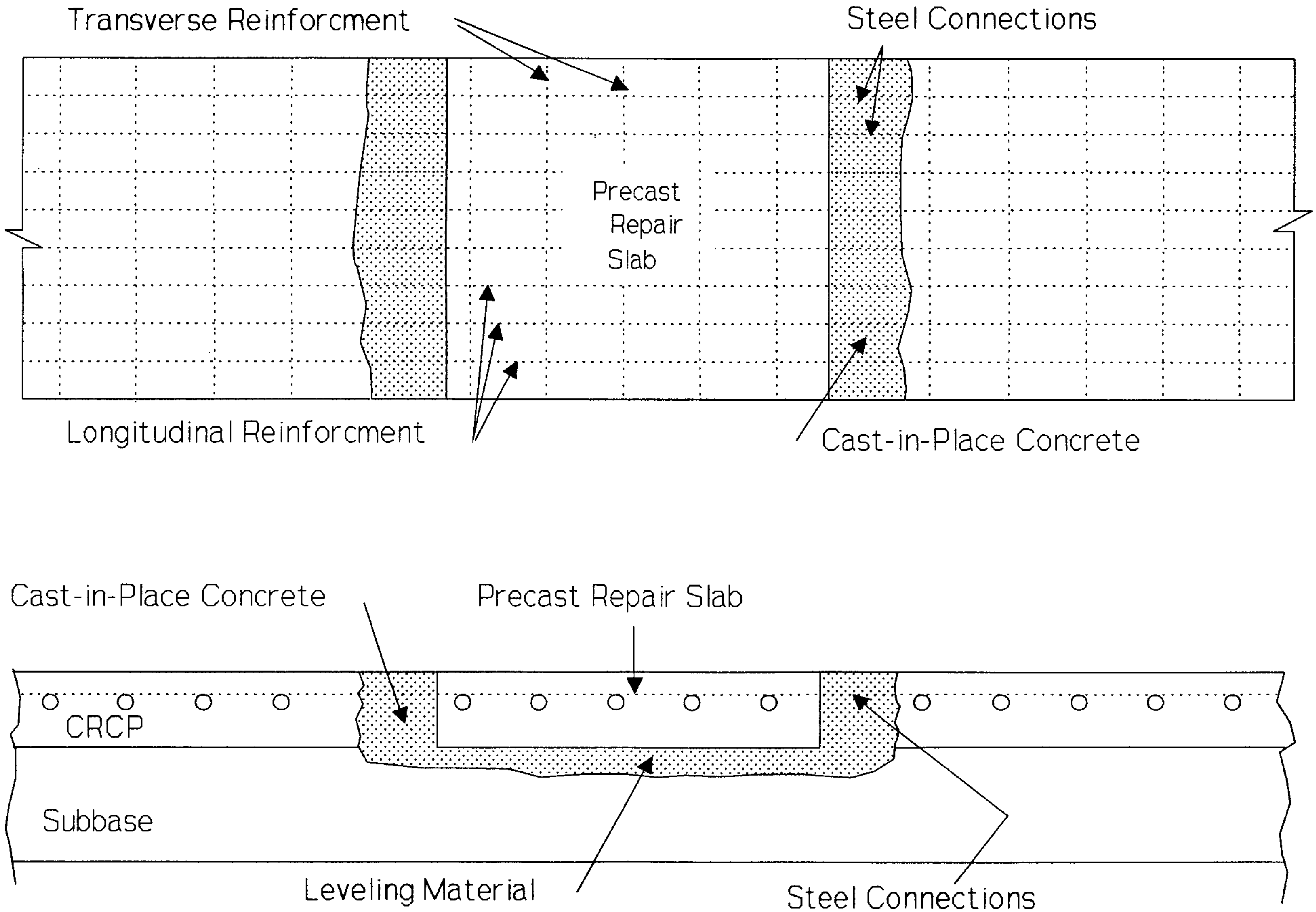


Figure 38. Precast slab repair.

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CHAPTER 5 - SUMMARY

The cost-effectiveness of rehabilitation depends on performing the appropriate and adequate rehabilitation at the appropriate time in the life of the pavement. The following questions must be answered to determine the cost-effectiveness.

1. How does repair plus an asphalt concrete overlay compare in cost effectiveness with repair only?

2. What quantity of pre-overlay repair is most cost effective?

3. What is the appropriate time to perform rehabilitation to maximize cost-effectiveness?

Over the last 20 years, Illinois and Texas DOTs have conducted studies to address the questions listed above. These studies indicate that, once punchout distress begins to show up, the rate of distress development increases with time/traffic application. Most agencies repair punchouts on an as-needed basis, generally a few locations at a time. Once a certain number of punchouts/km have been repaired, it is time to consider a more permanent rehabilitation of the pavement. Typically, the rehabilitation has involved use of an asphalt concrete overlay, although bonded concrete overlays and unbonded concrete overlays have also been used. Resurfacing the existing distressed CRC pavement at the right time can significantly extend the service life of the pavement at an overall lower cost. The generally recommended optimal time for resurfacing is when the number of failures exceed 10 per km (16 per mi) or the amount of repaired area exceeds about 2 percent of the total pavement area.

Highway agencies need to know when in the pavement's life rehabilitation will be most cost effective and, if adequate funding is not available at that time, the consequences of delaying rehabilitation. The answers to these questions will likely be different for each project. Delaying the overlay provides a diminishing return in savings. The saving achieved by deferring the overlay is mostly offset by the costs of keeping up with full-depth repairs, with the result that smaller savings are achieved in subsequent years. Performance data that illustrate the rate of deterioration with respect to traffic loading (ESALs) can be very valuable in determining the best time to perform major rehabilitation. Structural deterioration accelerates rapidly beyond the point at which the pavement's fatigue life is exceeded. Also, expenditures for full-depth repairs accelerate as the pavement continues to deteriorate. Delaying structural improvements significantly beyond the end of a pavement's structural life is very seldom cost-effective.

CRC pavement design is a very unforgiving pavement design. Any mistakes in the initial design, construction, or repair will, within the lifetime of the pavement, result in costly distresses. The patched area must be carefully demolished, removed, and replaced. The reinforcing steel must be reconnected to provide steel continuity. All these factors lead to the increased cost of repairing CRC pavements. The key points addressed in the report are summarized below:

1. Of the many distress types that affect CRC pavements, pumping, wide cracks, and punchouts are the most common and severe. The mechanisms of the distresses must be known and be recognizable. If the mechanisms causing the distresses are not addressed through the repair procedure, the repairs may also fail by the same mechanisms.

2. Preventive maintenance, often overlooked, is cost effective and efficient in reducing the deterioration of CRC pavements. Crack and joint sealing and underdrains will limit the amount of water in the pavement structure, thus reducing those distresses that are influenced by the presence of water such as pumping, punchouts, and voids. Where voids, pumping, and increased deflections exist, undersealing may be used to reestablish a fully supported condition to the pavement.

3. Continuity of the steel reinforcement must be maintained through the patch. Movement of the adjacent CRC pavement slabs will cause an untied patch to become distressed and deteriorate. The use of welded splices and mechanical couplers are viable alternatives to the use of tied splices.

4. Patches should be placed in the spring and fall when daily extreme temperatures are at a minimum and they should be placed after noon, to avoid possible crushing of the patch because of expansion of the CRC pavement.

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