Field Investigation of Punchout Distress in Continuously Reinforced Concrete Pavement in Illinois

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Most maintenance activities on continuously reinforced concrete (CRC) pavements are related in one way or another to punchout distress. Over several years of observation of CRC pavement, several symptoms related to the structural aspects of punchout distress have been noted. These symptoms include, but are not limited to, close crack spacing, surface widening and spalling of transverse cracks, development of longitudinal cracking, loss of load transfer, and subbase and subgrade pumping. Literature reviews have elaborated on punchout-related factors with respect to pavement performance. However, the punchout mechanism relating the various factors is not completely defined. Some questions exist concerning the sequence of events leading to the loss of load transfer across transverse cracks as a prerequisite to the development of a punchout. This sequence of events relates to the role reinforcement plays in the punchout process and whether the loss of aggregate interlock requires rupturing of the steel. If rupturing of the steel occurs, the question is whether rupturing occurs before or after the loss of aggregate interlock. Factors related to punchout distress as noted by this investigation and others are reviewed and a possible mechanism of punchout distress is addressed.

Punchout distress is regarded as the most severe performance-related problem that continuously reinforced concrete (CRC) pavements develop. Defined as a structural failure, punchout distress typically is associated with close transverse cracking and is bounded by a longitudinal crack on one side and a longitudinal edge joint on the other (Figure 1). Characteristically, the pavement pushes or punches downward under traffic loading, causing permanent deformation or faulting. The punchout can also, and frequently does, develop at Y-cracking, which is the development of closely spaced cracks that meet 2 to 3 ft from the pavement edge and form one crack. This type of cracking may also develop at an interior position in the pavement, but perhaps not as frequently.

Several investigators have noted problems associated with CRC pavement that have been the result of improper construction or easily identified design defects (*1*). In some instances failures have occurred at construction joints caused by the lack of consolidation. In other cases pavement failure has been attributed to improper lapping of the reinforcement (*2,3*). Problems such as these have given CRC pavement performance the reputation of being particularly sensitive to construction practice.

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Poor support conditions coupled with short cracking intervals have shown a strong correlation to a high frequency of punchout distress. Performance surveys have shown that the crack spacing distribution may include a wide range of cracking intervals from project to project. This range frequently includes crack intervals less than 2 ft. Factors related to environment and materials are known to influence crack spacing irregularities. However, the variabilities associated with these make difficult prediction and control of crack spacing within certain limits. Given the sometimes uncertain performance of CRC pavement in the past, it is necessary to improve the understanding of the failure mechanism associated with punchout distress, thus allowing CRC pavement design procedures to address the factors that influence the punchout process directly.

NATURE OF PUNCHOUT DISTRESS IN ILLINOIS CRC PAVEMENT

A field investigation was undertaken to further study a method of analysis on which to base a mechanistically oriented thickness design procedure for CRC pavements and to gain further knowledge of the mechanism related to the punchout distress as demonstrated by CRC pavements in Illinois. It is reasonable that design analysis should focus on the punchout mechanism, since punchout distress is recognized as the predominant distress type in CRC pavement (*4,5*). In this study, loss of support was found to be the primary cause of the punchout, which, incidentally, was recognized by previous investigators (*5,6*). This investigation examines factors associated with steel rupture, which is related to concrete fracture development around the reinforcement. Faulting occurred in most instances in which loss of support was indicated. Various loss-of-support mechanisms and failure modes are proposed based on the field observations.

CRC Pavement Survey

Several observation sections that represent the performance of CRC pavement within the state of Illinois were selected to study punchout development. The survey results from selected sections are presented in this paper. The sections selected for observation were chosen on the basis of performance, pavement thickness, subbase type, percent of reinforcement, and particularly susceptibility to D-cracking. It was important to

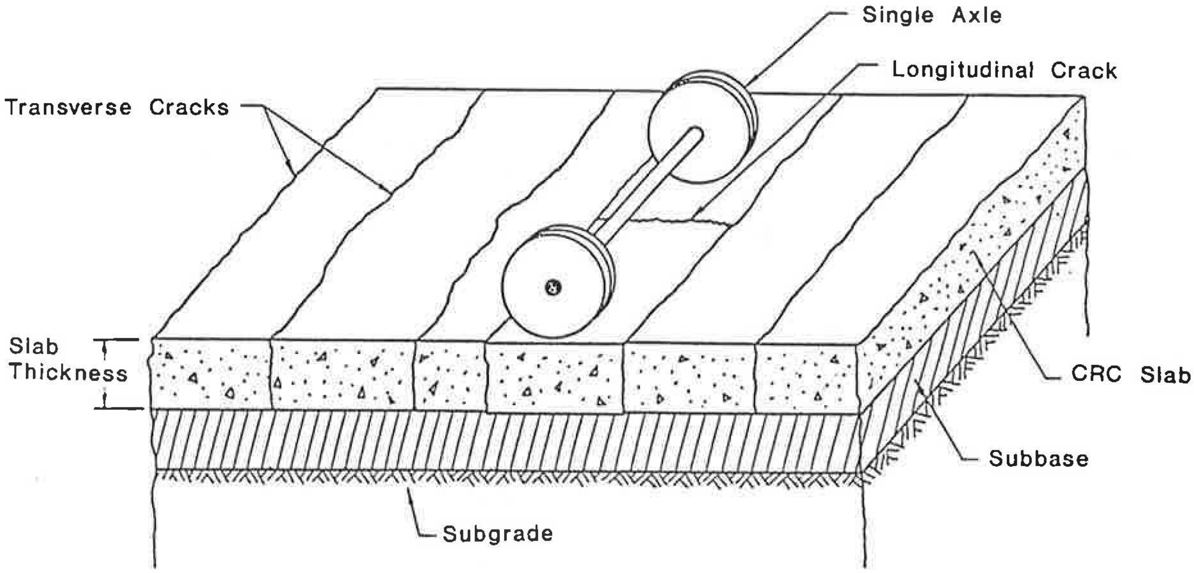

FIGURE 1 Formation of longitudinal crack between two transverse cracks; intermediate state of edge punchout (*6*).

TABLE 1 OBSERVATION AND SAMPLING SECTION DATA

Route	Thickness*	Reinforcement % Steel	Reinforcement Method	Subbase Type	Aggregate Source	Average** Traffic ESAL [1]	Average Crack Spacing	Punchouts (Per Mile)	Year Const.
I-72WB [4]									
MP41.9-48.5	8	0.59(#5)	Tube	CAM I	Thornton [2] Pit	2.97	4.8	11	1976
MP48.5-53.5	8	0.59(#5)	Tube	CAM I	Covington Pit	2.87	5.4	<1	1976
MP58.1-62.9	8	0.59(#5)	Tube	CAM II	Fairmount Quarry	2.87	6.0	9	1976
I-55NB [4]									
MP223.0-224.0	9	0.59(#5)	Tube	CAM I	Westowe Quarry	7.10	4.7	<1	Fall 1977
I-55SB [4]									
MP223.0-225.0	9	0.59(#5)	Tube	CAM I	Westowe Quarry	7.10	5.9	10	Spring 1978
I-39NB,SB [4]									
MP118.0-130.5	10	0.71(#6)	Tube	CAM II	Mulford [2] Quarry	4.7	7.4	NB:9 [3] SB:16	1981-82
MP130.5-139.0	10	0.71(#6)	Tube	CAM I	Mulford [2] Quarry	4.7	7.4	NB:2 [3] SB:7	1981-82
I-57NB [4]									
MP250.5-256.0	7	0.59(#5)	Tube	BAM	Fairmount Quarry	8.77	1.8	<1	1970
I-57SB [4]									
MP324.1-330.5	8	0.59(#5)	Chairs	BAM	2	11.1	4	27	1968
MP275.5-279.6	8	0.59(#5)	Tube	BAM	Lehigh Quarry	8.84	3.0	13	1970
FA 409 [4,5]									
STA3153-3199	7	0.70(#6)	Chair	CAM II	2	0.04	5.5	0	1986
STA3014-3153	8	0.73(#6)	Chair	CAM II	2	0.04	5.1	0	1986
STA2989-3014	9	0.72(#6)	Chair	CAM II	2	0.04	6.0	0	1986
US 50 [4]									
STA2767-2789	8	0.60(#6)	Tube	BAM	Columbia Quarry	0.04	6.0	0	1980

* Thickness in inches
** Average Crack Spacing in feet
[1] In the driving lane (in millions)
[2] Low D-cracked susceptible aggregate
[3] Total punchouts per mile
[4] 4 or 6 inch pipe edge drains
[5] Some sections with no edge drains

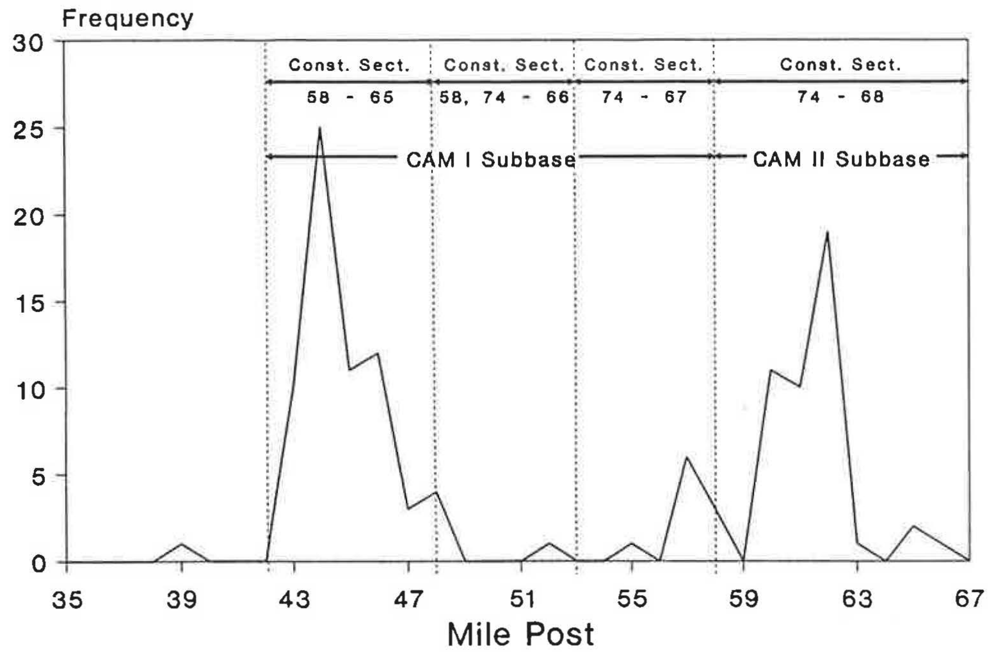

FIGURE 2 I-72WB CRC pavement survey of punchouts and patches in driving lane.

study the structural failure aspects related to punchout distress independent of D-cracking susceptible concrete. This type of study could not be done for some of the survey sections. Recent surveys have indicated that over 50 percent of the pavement mileage in Illinois has either a low, intermediate, or severe level of D-cracking (*7*). This percentage is even greater when the pavement mileage that contains D-cracking susceptible aggregate that is not currently showing visible signs of D-cracking is included. Table 1 summarizes the observation sections and pertinent data (including subbase data) relative to the punchout performance survey conducted in this study. The D-cracking information is based on core examinations and available quarry D-cracking evaluations. All the pavements included in this study have a bituminous shoulder except the FA-409 pavement sections.

The nature of punchout distress is then described by the factors affecting the punchout rate and performance from a few selected sites. Results of a survey of patches and punchouts in the truck lane from the I-72 westbound sites are shown in Figure 2. The approximate construction boundaries of each project are shown, delineating where different contractors and aggregate sources were used for construction of the pavement. Significantly different punchout rates are evidenced from construction section to construction section, which suggests that punchouts may be related to construction practice.

A similar pattern of punchout frequency was noted at another observation section on I-55. The punchout rate recorded for the southbound driving lane was as high as 20 punchouts per mile, as contrasted with the northbound lanes, which had a punchout rate near zero. Significant performance differences exist, similar to those in the sections on I-72 between the northbound and the southbound lanes, both of which have cement aggregate mixture (CAM) I type subbases. As shown by the data in Table 1, little difference is apparent in traffic level, aggregate source, and steel reinforcement between the northbound and southbound lanes, which, in this case, were constructed by the same contractor.

An observation section on I-39, with 4.7 18-kip equivalent single axle loads (in millions based on unpublished data from the Illinois Department of Transportation, 1983, traffic factor equations), is experiencing a relatively high punchout rate. The cracking pattern on I-39 is not uniform but divided between groups of wide and narrow crack intervals with a crack space distribution made up of a combination of the short and long crack spacings. Other factors leading to the formation of punchouts on I-39 will be discussed later, but the crack pattern in CRC pavement appears to be a factor in the punchout development. Nonuniform crack spacing distributions may lead to a greater tendency to punchout. In fact, the data from this study indicated that maintaining an average crack spacing within the limits designated in the literature is no guarantee that punchouts will not develop.

Close cracking intervals have been characteristic of good-performing CRC pavements in Illinois. The average cracking interval, which is now less than 2 ft, was noted on 0.7 percent and 1.0 percent reinforcement sections of the well-known Vandalia CRC pavement (constructed in 1947). It has been pointed out previously that the cracking intervals of the Vandalia pavement have shown some dependence on the percent of reinforcement (*8*). The percent of reinforcement also has had a profound effect on the punchout performance of the pavement, as indicated by the data shown in Figure 3. This effect may be indirectly related to the cracking intervals associated with the percent of reinforcement. Other pavement sections on the heavily traveled Stevenson (I-55) and the Dan Ryan (I-57) in the Chicago area have average crack spacing reported to be 3 ft or less (unpublished data, Illinois Department of Transportation, 1987).

A comparison between the northbound section on I-57 (Table 1) and the site on I-39 shows much better performance on I-57. This particular section is 3 in. less in thickness and has

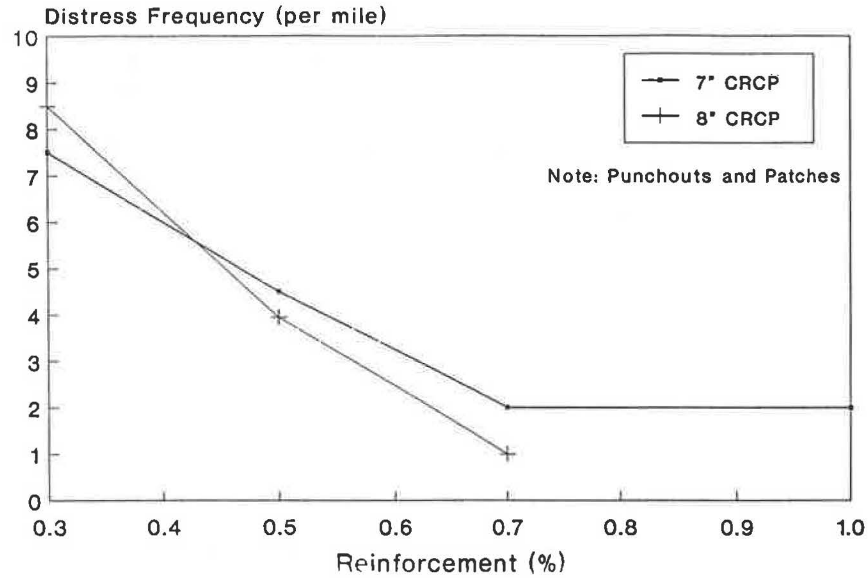

FIGURE 3 Effect of reinforcement on performance of Vandalia CRC pavement (US-40).

carried more than twice the traffic of I-39. A factor much more significant than pavement thickness is contributing to the markedly better performance of the I-57NB section over the I-39 section. The I-57 section contains a greater percentage of D-cracking susceptible aggregate, a bituminous aggregate mixture (BAM) subbase, and an average crack spacing of less than 2 ft with a uniform cracking pattern.

Although no significant distinctions could be made in the crack spacing distributions between the adjacent good- and the bad-performing sections of I-72WB (shown in Figure 2), the apparent differences lie in whether the coarse aggregate was D-cracking susceptible. The pavement thicknesses are equivalent and the average crack spacing is similar. The construction section containing the low percentage non–D-cracking susceptible aggregate source (Thornton Pit) is the one with the highest rate of distress.

The subbase types used on I-72WB are CAM I and CAM II. CAM I subbase is roller compacted and usually contains an 8 percent cement content, an aggregate blend of 75 percent coarse aggregate, and 25 percent natural sand (*9*). CAM II subbase type is equivalent to a lean concrete subbase. Two of the construction sections with high rates of distress had different subbase types, so the source of the distress may not be strictly attributable to the type of subbase. A standard procedure was followed on each construction section, making little difference in construction procedure between the sections. Especially with regard to the construction of the CAM I subbase, the standard procedure called for the trimming of the CAM I material with an autograder. This meant that the stabilized subbase was trimmed before placement of the 8-in. CRC pavement. The subbase material was not roller compacted again after the trimming operation was completed.

A limited survey of Y-cracking from selected pavements indicated that the pavement sections in which the reinforcement was placed using a tube device incur a higher incidence of Y-cracking (12 cracks per 100 ft) than pavements in which the reinforcement was placed using chairs (4 per 100 ft). The incidence of Y-cracking may be tied to the variability of the depth of reinforcement cover, which is largely a function of the placement method (*4*). Field data indicate that for a tube type placement method a trend of greater average rebar cover

FIGURE 4 Type 1 spalling, I-39SB.

and variation occurs as the thickness of the pavement increases. The standard deviation of rebar cover is lower for chair placement (2.2 in. versus 4.2 in. for tube placement).

Spall development was investigated on the I-57 sites, in which two types of spalls were noted. Spalling developing as a result of a spall mechanism suggested by McCullough et al. (*10*) is classified as a Type 1 spall (Figure 4). A Type 1 spall develops into an advanced stage when the pavement is broken out within the boundaries of the cracking. Although not in all cases, it was noted that Type 1 spalls tend to coincide with the longitudinal reinforcement in the pavement and can extend to a considerable depth into the pavement. Spalling of this nature that extends to the depth of the reinforcement severely reduces the aggregate interlock and may lead directly to punchout distress. It may also contribute to the Type 2 form of spalling shown in Figure 5 and can be classified as either minor or severe spalling as noted previously. These spalls tend to develop abruptly into a severe condition of spalling in a period of a few months. McCullough et al. (*11*) indicated that Type 2 spalling increases with crack spacing because of an increase in crack width. A low-severity form of Type 1 spall or concrete wearing also leads to widening of the crack width. The formation of longitudinal cracking is associated with spall development and concomitant loss of load transfer.

CRC Pavement Removal and Coring Program

As a part of this study, several pavement cores and slab sections containing punchouts in various stages of development were removed from the field and relocated to the Newmark Civil Engineering Laboratory for examination (*4*). A brief summary of the results of the laboratory examination is presented regarding the condition and performance of the steel in the pavement with respect to punchout distress. The condition of the reinforcement will be discussed later after factors related to the loss of subbase support are presented.

FIGURE 5 Type 2 spalling, I-39.

Loss of Subbase Support

In an effort to preserve the details of the punchout distress, short sections of CRC pavement were carefully lifted and removed from in-service pavements. On removal of the sections from pavements with CAM I subbase, severe erosion was noted in the surface of the subbase. The approximate area of surface erosion is shown in Figure 6. The most extensive erosion occurred along the shoulder where disintegrated subbase extended to a depth of 1 to 2 in. into the subbase. In the figure, the subbase surface was eroded approximately 24 in. along the pavement edge. The erosion extended another 12 in. in the vicinity of the punchout. Some of the erosion may have occurred while the punchout was developing because of slab rocking and other related motion under loads. However, the uniformly eroded 24-in. area shown in the diagram appears to have developed by other than load-associated means.

At other sample sections, punchout information was obtained during punchout repair operations in which erosion of the

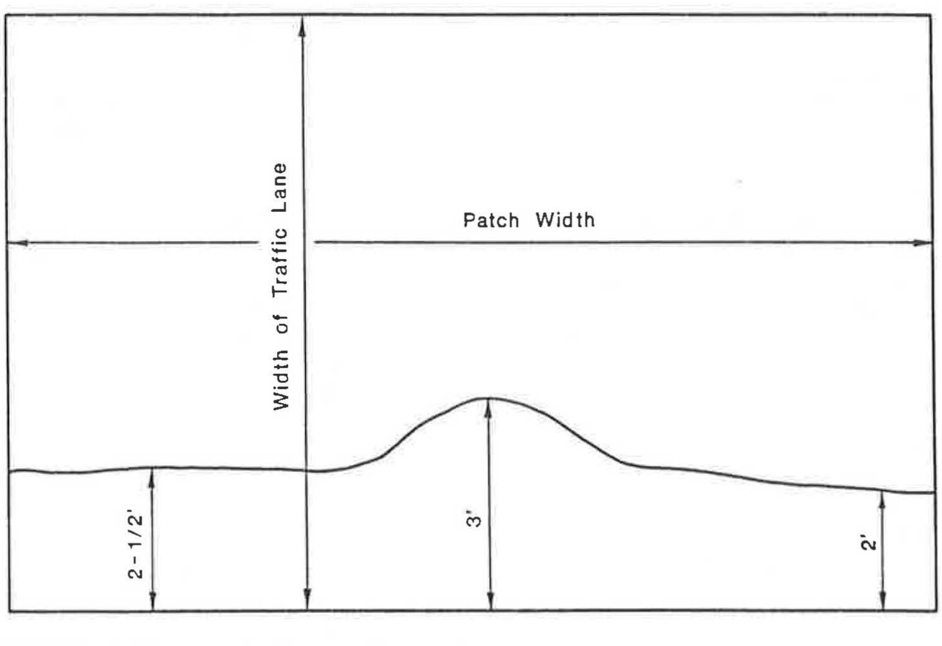

FIGURE 6 Diagram of subbase surface erosion at punchout repair site, I-72WB MP57.

TABLE 2 I-39SB: UNCONFINED COMPRESSIVE STRENGTH (CORRELATED TO DCP)

			Unconfined Compressive Strength		
Station	Subbase Type	Initial* Penetration (inches)	First Drop (psi)	Average of First Five Drops (psi)	Subbase$^{\Delta}$ Average (psi)
2209+00	CAM I	0.5	33	135	240
2141+50	"	"	33	84	200
2131+00	"	"	46	120	95
Patch #113	CAM II	0.4	20	53	90
1851+00+	"	0.2	33	118	155
1851+00	"	0.0	46	161	340
1841+50+	"	0.5	33	83	200
1841+50+	"	-	-	38	55
1841+50	"	0.0	-	110	290

* Penetration of the cone under the weight of the 8 kg hammer
\+ Measurements were made in the eroded area of the subbase
Δ over the full depth

same nature was noted in the CAM I subbase on I-39. While the punchouts on I-39 were being repaired, in situ strength tests were made on the subbase. The strength of the CAM I and CAM II subbases was found to be low. A summary of unconfined compressive strength determined from tests taken using the dynamic cone penetrometer (*12*) is shown in Table 2. Obviously, the measured subbase strengths are very low. The lowest-strength material is in the upper portion of the subbase. The eroded areas showed a lower strength than the noneroded areas. Some damage from slab rocking may have decreased the strength in the eroded areas. A stabilized, nonerodible subbase should have a compressive strength of 1,000 psi (*13*). It would be difficult to drive a dynamic cone penetrometer through a stabilized subbase of normal strength.

A coring program was instigated to obtain further information on the development of subbase erosion. Most cores obtained for the study were taken from transverse cracks. The descriptions of the various classification systems for transverse crack conditions and other core-related data are given in Table 3. A variety of sample sections, subbase types, crack conditions, and spacings were cored to obtain punchout information about the subbase performance in punched-out and non-punched-out areas.

A series of cores were obtained at the I-55 sections, which also had a CAM I subbase. All cored cracks on I-55 had some erosion at the pavement longitudinal edge joint. However, a significant difference exists between the erosion data from the northbound lanes and the erosion data from the southbound lanes. The extent of the erosion in the northbound lanes was much less than that in the southbound lanes. The punchout rate in the southbound driving lane was much higher than that in the northbound driving lane, as noted in Table 1. A slight amount of erosion under the northbound lanes was noted 22 in. from the pavement edge, but the support conditions were considerably better than those for the southbound lanes. The southbound lanes had wider longitudinal edge joints, which may be an indication of greater pumping susceptibility, compared with the longitudinal joint on the northbound lane, which was very narrow and tight. The crack classification was nearly the same on all cracks cored in this series and no faulting was noted. The cracking interval associated with the cored cracks ranged from 2 to 6 ft. Little or no shoulder staining was visible at the I-55 sections.

The information obtained from the field cores on CAM I subbase suggests that subbase erosion and loss of support must develop before the development of punchout distress. A series of cores obtained from a pavement with a BAM subbase taken from a sample section on I-57NB that exhibited a low punchout rate showed that subbase erosion was almost nonexistent. This conclusion emphasizes the need for nonerodible subbases under CRC pavements as a primary step toward improved pavement performance. The BAM type subbase may be a good candidate for a low-erodible, uniform supporting subbase, as long as drainage is provided.

A problem related to lean concrete subbases was noted in cores obtained from the pavement sample section on I-72WB, between MP 58.1 and MP 62.9. This section, which has a CAM II subbase, indicated a failure mode similar to that experienced in Wisconsin and Pennsylvania on CRC pavement overlays on jointed concrete pavement. This failure mode, illustrated in Figure 7, developed at a crack in the CAM II subbase. A 6-in.-diameter core was taken on a faulted crack in which the subbase portion included the crack in the subbase displaced approximately 3 in. from the crack in the pavement. On removal of the subbase core it was noted that the subbase crack face was spalled (the strength of this subbase was great

TABLE 3 CRC PAVEMENT CLASSIFICATION SYSTEMS

<u>CRCP Crack Classification (Modified AASHO Road Test-Report 5)</u>

C-1: Fine crack not visible under dry surface conditions at a distance of 15 feet. (Tight)

C-2: A crack that can be seen at 15 feet, but exhibits only minor spalling. The opening at the surface is 1/32" or less. (Open)

C-3: The crack is opened at the surface 1/32" or more for any portion of the crack length. The crack exhibits low to medium spalling. Amount of faulting is noted.

C-4: The crack is either very wide (>1/16") or sealed and exhibits medium to severe spalling. Amount of faulting is noted.

<u>IDOT Rebar Corrosion Rating</u>

1: Clear or free of rust

2: Slight rust with no appreciable reduction in cross-sectional area.

3: Moderate rust with no substantial reduction in cross-sectional area.

4: Heavy rust with a marked reduction in cross-sectional area.

<u>Diamond Void Size Classification</u>

Small: Radius of the void (from the surface of the rebar) is less than the diameter of the rebar.

Large: Radius of the void is greater than the diameter of the rebar.

<u>Number of D-Cracking Aggregate</u>

Number of aggregates on the surface of a 4 inch diameter core showing signs of D-Cracking. Other core diameters are listed along with the pavement thickness.

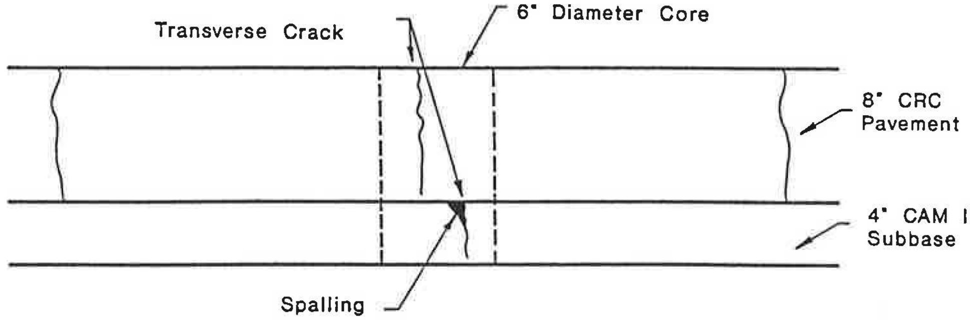

FIGURE 7 Spalling on crack face of CAM II subbase.

enough that a dynamic cone penetrometer could not be driven through it). A loss of load transfer in the subbase can lead to an unsupported condition in the CRC pavement, which can cause faulting and punchout distress.

Results from this investigation indicate that unsupported conditions are the main factor in the development of punchout distress and faulting of CRC pavements in Illinois. Loss of support conditions may result from soft and wet soil conditions in the case of BAM subbases, surface erosion caused by low strength in the case of CAM I and CAM II subbases, and unequal deflection characteristic between the pavement and the subbase in the case of high-strength CAM II subbases.

Subbase erosion has been identified as the primary cause of punchout-related distress in CRC pavements with CAM I subbases. It provides an explanation for the varying punchout rates evident in the data of Table 1, particularly for the various levels of performance between the I-39 and I-57NB sections. Apparently small amounts of subbase erosion can extend up to 24 in. from the pavement edge and still have adequate performing CRC pavement in Illinois. CRC pavement with CAM I and BAM subbases showing good performance show less erosion than those with a poor performance. The difference in the varying erosion rates between pavement sections noted in this paper under apparently identical traffic and climatic conditions was found to be dependent on whether the aggregate in the pavement was susceptible to D-cracking. D-cracking had an effect on the pavement behavior under load and is elaborated further elsewhere (*4*).

Condition of the Reinforcement

The factors related to the performance of the reinforcement in CRC pavement during punchout development are apparently secondary or consequential in nature. Details regarding the condition and behavior of the reinforcement in punchout distress are explained on the basis of the laboratory examination of slabs and cores removed from actual pavement. Cores taken from the slab specimens were split at the level of the reinforcement to allow examination of the rebar and the surrounding concrete. Examination of the cores showed that diamond-shaped fractures and voids were noted around the reinforcement in several of the cores. Voids around the reinforcement have been noted by other investigators (*6*) but were being attributed to expansive forces from corrosion. Figure 8 illustrates a diamond-shaped fracture pattern around the reinforcement found in several cores. A direct correlation between these fractures and corrosive forces is doubtful. However, it is apparent that the voids around the rebar are an end result of the diamond-shaped fracturing that can form from 0.5 to 1.0 in. from the crack face. The voids and fractures were found in the portion of the punched-out areas that had no faulting.

Bond studies of bars in tension (*14,15*) noted that secondary cracks similar to those in the fracturing shown in Figure 8 formed at the loaded face of the specimen at a threshold steel stress between 14 and 18 ksi. These fractures caused by pullout forces form the diamond-shaped configuration around the steel shown in Figure 8. Measured strains in the reinforcement at the crack face (*4*) predicted that steel stresses from climatic effects can exceed the threshold levels for a broad range of environmental conditions leading to the formation of the pullout cone. Once the fractures around the steel have formed, bearing stresses may crush the concrete in the immediate vicinity of the bar, which leads to a void, as shown in Figure 8. The cone pullout fractures will also lead to reduced bond stiffness as indicated by Mains (*16*).

Laboratory analysis of a slab section containing punchouts in the initial stages of development indicated void development around the steel before the crack developed faulting. The void apparently forms with the development of the punchout and may develop before permanent faulting. Consequently, some voids were present around the steel on nonfaulted cracks. The effect of these voids on load transfer is discussed later in this paper, but one would expect load transfer to decrease. As a result, participation of the reinforcement in the transfer of load across the transverse crack is minimized because of the pullout fracturing.

The field data indicate that 80 percent of the cores taken in the field study had either cone pullout fracture or a rebar void. Although one would expect a high correlation between rebar voids and pavement nonsupport, the data suggest that

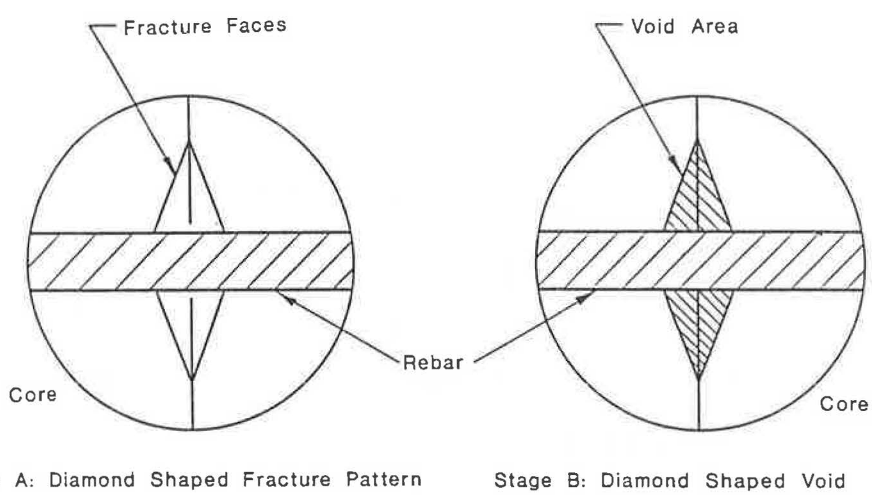

FIGURE 8 State development of void area around reinforcement.

unsupported conditions are coincidental with voids around the rebar only 55 percent of the time. A high correlation was found between the number of cores with either a rebar void or pullout fracture and subbase nonsupport. It is expected that cone pullout fracture may have some dependence on the length of crack spacing and climatic conditions at the time of construction, but strong correlations in this regard were not apparent in the data. At the I-39 site, where the cracking pattern tended to change from a long crack spacing to a short crack spacing, 100 percent of the cores that had rebar in them indicated pullout fracture. Examination of the cores taken at the I-57NB site revealed that 42 percent showed pullout fracture. This pavement had a closely spaced and uniformly distributed cracking pattern. Two sites on route US-50 were investigated for cone pullout fracture as a function of time and traffic. Although they are 6 years apart in age, these two sites were opened to traffic at the same time. Pullout fracture was noted in both of the cores obtained from the older site, whereas it was indicated in only two of the seven cores taken in the newer section. At the time of coring, the newer section had been through one winter since construction.

Sections on I-57NB near MP 320 containing punchouts at further stages of development were visited and several instances of broken reinforcement were found. It was noted that tensile type breaks in the reinforcement were located at a small distance from the crack face (ranges from 1 to 2 in.). The steel breaks were straight across the bar cross section and orthogonal to the axis of the bar. Other punchouts with broken steel were also noted to be displaced from the crack face. Large diamond-shaped voids were also found around the reinforcement along with faulting on the crack face. In one instance, breaks in the rebar were found at the crack face where transverse steel coincided with the transverse crack. Otherwise, no other breaks were found at the crack face.

The bar breakages were noted at two locations, where one break occurred at the crack face and another break occurred approximately 2 in. from the crack face. A closer look at the rebar revealed several fractures of partial depth on one side of the steel. These fractures were found to correlate with the tensile side of the rebar from the bending stress that the bar underwent during the process of fault development of the punchout. This process (which may develop under 3 months) is illustrated in Figure 9. Once the void forms around the reinforcement either by cone pullout fracture or bearing failure, vertical displacement or faulting in the pavement will not put the bar under immediate shear stress. Further faulting applies a bending strain in the rebar on the under and top sides as shown in Figure 9*b*. The resulting fracturing is caused by the fatigue strain of several repeated loadings, since rebar is commonly bent to radii much shorter than those that occur in the faulted case without tensile fracture. Once the fracture has progressed through the rebar, as seen in Figure 9*c*, further faulting can develop. Eventually, the voided area will begin to make repeated contact with the rebar, which will either fracture the concrete or cause a shear-type failure in the reinforcement. This leads to short segments of broken steel resulting from the punchout process.

CRC Pavement NDT Program

Nondestructive testing (NDT) was conducted on several test sites to gain further insight into CRC pavement behavior leading to punchout-related distress. During the testing process, data relative to the slab deflection, load transfer efficiency, pavement stiffness, and crack width were obtained. The falling weight deflectometer (FWD) used for the field testing was the Dynatest Model 8000, which is widely used in the United States. Most of the testing was conducted in the outer wheel path, with the center of the loaded plate varying between 36 and 42 in. from the pavement edge. The FWD, which was calibrated monthly, has been shown to yield repeatable data (*17*). The crack openings were measured, but results with respect to crack width measurements are reported elsewhere (*4*).

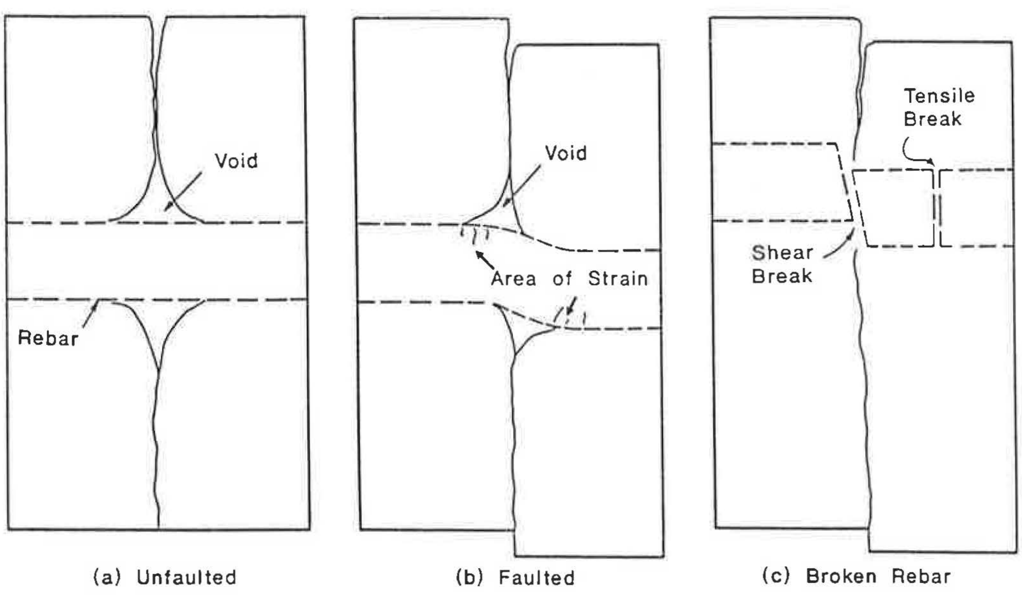

FIGURE 9 Broken reinforcement mechanism.

The results of the FWD field measurements are described in terms of the plate deflection (D_0), the load transfer efficiency (LTE), and the deflection basin area. The LTE is equal to the change in deflection on the unloaded side of the joint divided by the change in deflection of the loaded side of the joint. The basin area has been shown to be useful in interpreting measured deflection profiles by the FWD and is calculated from the sensor deflections (*18*).

FWD load transfer data were obtained from several transverse cracks for each group of crack classification (Table 3). The results indicate that significant differences in load transfer efficiency measured during the early hours of the day can exist between the C-1,2 and the C-3,4 classifications of cracks. However, these load transfer differences between the two groups are much fewer and tend to disappear if determined from results found during the afternoon. The closing of the cracks because of an increase in the pavement temperature can cause a dramatic increase in LTE.

The NDT results are also useful in backcalculating material properties of the pavement system, such as the modulus of elasticity (E) and the modulus of subgrade reaction (K). Particular interest lies in backcalculated E values and the radius of relative stiffness (l_k) at the transverse cracks, which is a function of E and K. [Note that $l_k = \{Eh^3/[12(1 - \mu)^2K]\}^{1/4}$ where h and μ are the pavement thickness and Poisson ratio, respectively.] A method of backcalculation of these parameters based on ILLI-SLAB modeling of the deflection basin [iterative calculation with various values of concrete modulus (E) and foundation modulus (K) to match the deflection basin measured by the FWD for a given load condition] has been discussed by others for jointed concrete pavements (*17–19*); however, little information is available about application of these methods to CRC pavement. A suggested procedure for CRC pavement, which is described elsewhere (*4*), is based on determining an effective radius of relative stiffness from modeling the measured deflection basin of the CRC pavement using a 100-ft continuous slab without joints.

This backcalculation procedure can be accomplished for a range of thicknesses and load positions. The calculations for l_k using ILLI-SLAB analysis at the wheel path locations were found to be similar to those of the closed form curve obtained from the theoretical solution (*19*) of a case of a circular load and a dense liquid foundation for an interior loading condition. Consequently, the theoretical curve could possibly be used in place of the ILLI-SLAB generated curves. As stated earlier, the FWD wheel path testing position varied anywhere from 36 to 42 in. from the pavement edge. Given the deflection basin area from the FWD test results, the radius of relative stiffness can be obtained from Figure 10 for either the edge load or wheel path load positions for any pavement thickness.

The determination of unique E and K values is discussed elsewhere (*19*) from l_k values obtained from Figure 10 and Westergaard solutions (*20*) for slab-on-grade deflections at the edge and interior load positions. The theoretical interior loading solution is applied to the wheel path load position in the actual pavement since the load behavior between the two positions was shown to be similar.

The l_k can be useful in determining potential punchout areas in CRC pavement. On the basis of the field data (*4*), if voids are present, the LTE and the l_k values are low (usually below 70 percent and 25.0, respectively) and if they are not, the l_k values are high. The l_k values at which rebar voids may be beginning to develop are not clearly defined. The data (*4*) indicated the ranges of l_k where problems may and may not be developing. This approach to CRC pavement evaluation can be useful as an indicator of potential punchout since subbase erosion and rebar voids tend to lead to lower l_k values. Comparisons between the l_k values from the NDT data (using Figure 10) and calculated l_k values (using the backcalculated K value, a range of concrete modulus, and the pavement thickness, h) may provide a basis for evaluation.

Calculated l_k values for a range of E values between 3 and 4 million psi are plotted as shaded-in evaluation limits to be

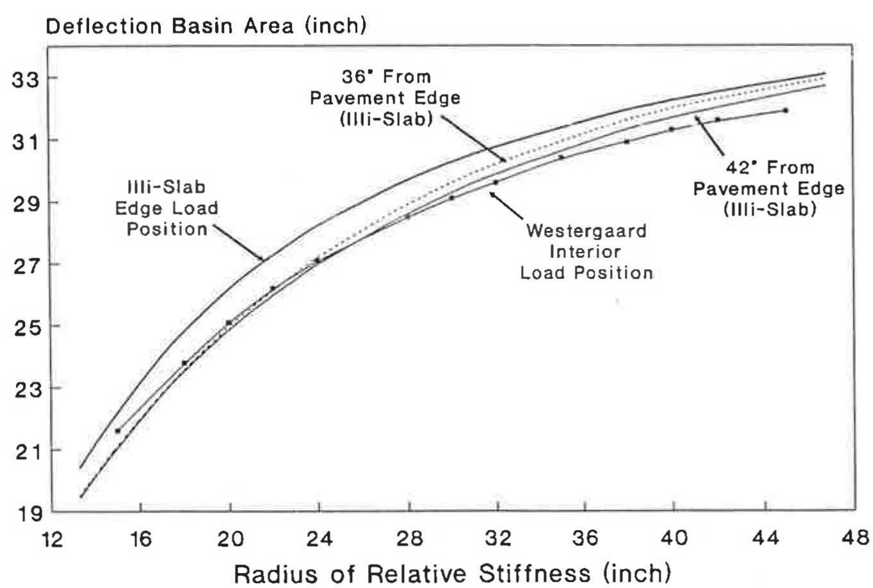

FIGURE 10 Variation of deflection basin area with *l*.

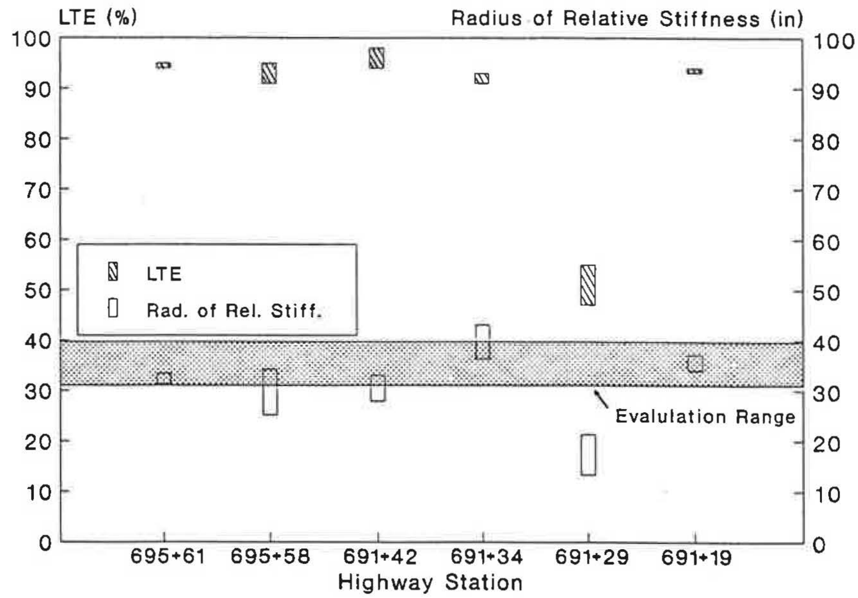

FIGURE 11 Comparison of LTE and l_k for I-55NB.

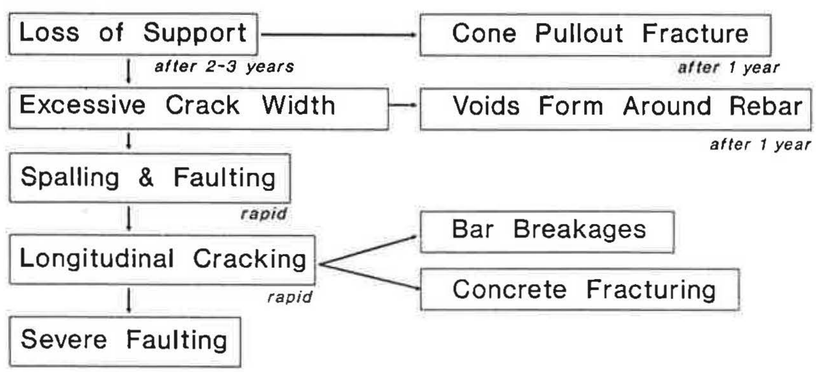

FIGURE 12 Morphology of punchout distress.

compared with l_k values from the NDT data illustrated in Figure 11. Figure 11 shows a comparison of LTEs and l_k values determined for the same test locations on I-55SB (approach and leave data are illustrated). At two test locations low l_k values were determined; these values had corresponding LTEs over 90 percent, suggesting that the radius of relative stiffness may be more effective in terms of CRC pavement evaluation than LTE. The shaded area is the limit range below which problems may exist. Evidently, potential punchout-related distress in CRC pavement may be evaluated from a combination of transverse crack observation, load transfer efficiency, and pavement radius of relative stiffness values.

Punchout Mechanism

Based on an appraisal of the previously noted factors, a punchout mechanism is postulated. A schematic diagram of the formation of punchout distress is shown in Figure 12. This diagram may apply to premature punchouts and those that occur at the end of the pavement life, depending on the cause of loss of load transfer. A noted prerequisite to punchout formation is the loss of subbase support. (This is particularly true in the case of premature punchout distress.) Support loss can develop in a number of ways. Support loss normally will develop if the subbase is susceptible to erosion when the necessary moisture conditions or discontinuities of load transfer or faulting in the subbase exist, all of which cause a nonuniform support condition in the pavement system. Stabilized subbases may also erode if they become or are weakened in the upper surface. Weakening may occur from poor compaction or poor strength gain. Wet subgrade conditions can also contribute to nonuniform support. Loss of support, depending on its nature, can develop as early as 2 to 3 years. The time for punchout development may be more dependent on the length of time associated with the development of unsupported conditions rather than the given pavement thickness (within the range of thicknesses, subbases and levels of traffic included in this study) due to the intensity of the stresses associated with it. Time to punchout from erosion of CAM I subbases ranged from 3 to 8 years in Illinois. CAM II subbases that are low in strength usually lead to punchouts by the third year after construction. The CAM II subbases of normal strength may lead to punchouts more gradually over time and more in the 5- to 8-year range.

Punchout distress may be related to the loss of bond such as that caused by cone pullout fracture around the reinforcement. These fractures appear to form from a poor combination of long and short cracking intervals and a large drop in pavement temperature. In these instances, this distress can begin to form within 1 year after construction and may be mitigated if a uniform crack pattern develops (where steel and bond stresses are low). The likelihood that this distress will develop within the first 24 hr after construction was not supported by the strain data collected under this study. The data indicate that either pullout fractures or loss of support leads to a reduction in pavement bending stiffness.

Excessive crack widths can also lead to reduced bending stiffness in the vicinity of the transverse crack. Crack widths tend to open under loading repetitions, particularly when fracturing associated with Type 1 spalling developed at the time of initial drying shrinkage. Widened cracks can also occur with poor combinations of short and long cracking intervals. A combination of a short segment of pavement between two long segments is particularly susceptible to punchout distress because the stresses that cause longitudinal cracking increase significantly as the crack spacing decreases after load transfer across the crack is lost. These stresses are discussed and elaborated on elsewhere (*4*). Because the crack width affects the bending stiffness and load transfer efficiency, wide cracks will also increase the pumping potential of the pavement.

The bending stiffness is also reduced when bar looseness increases or the concrete within the pullout fracture disintegrates, leaving a void around the reinforcement. Rebar voids are instigated by bearing stresses on the concrete and can form quickly, at least within 1 year after poor support conditions have formed. The looseness around the steel may also be affected by the crack width since the aggregate interlock decreases as the transverse cracks widen.

A reduction in bending stiffness at the transverse cracks can cause spalling and faulting, leading to a loss of load transfer. After load transfer is lost, longitudinal cracking can form quickly—possibly simultaneously. The formation of a longitudinal crack should not be considered as the cause of the punchout but only a consequence of it. Type 2 spalling leads to a significant reduction in load transfer across the transverse cracks, which, as pointed out above, dramatically increases the stresses that lead to longitudinal cracking. Rebar breakages and pavement delamination are consequences of the above processes and are the result of loss of load transfer. Rebar breakages can occur either at the crack face or displaced from it. The breaks at the crack face are usually a consequence of corrosion, which reduces the cross-sectional area of the bar. This corrosion weakens the bar and makes it susceptible to an abrupt tensile failure caused by temperature contraction or wheel load-induced shear stress. This process may begin on the pavement edge, leading to a widened crack there, and proceeding to the next bar until the entire lane is encompassed. Consequently, a wide and faulted crack develops. The bar breakages displaced from the crack face are load induced and fatigue related. Stresses forming this break are not great enough until most of the load transfer from aggregate interlock is lost. Once the bar has failed, the faulting can progress rapidly to a severe level since the reinforcement was transferring most of the load before failure. If the crack spacing is short, as in the case of a Y-crack, the rebar may not fail, which may lead to delamination and deep spalling of the concrete pavement above the reinforcement.

SUMMARY

The fundamental cause of punchout distress has been identified as loss of subbase support. The probability of punchout is enhanced when loss of support and nonuniformly distributed crack spacing combine under load to destroy the load transfer across the transverse crack. There are instances, reported in the literature and found in this study, where nonuniform support has precipitated localized short crack spacing, leading ultimately to punchout failure. Whether the punchout is at the pavement edge or in the wheel path, the cause is still the same. Pavement faulting, which is characteristic of the punchout distress, cannot develop unless the support underneath that portion of the pavement is lost. Therefore, definitions in the literature appraising various causes to edge and interior punchout are unnecessary.

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